STEEL CONSTRUCTION

MANUAL

An Online Resource

AMERICAN INSTITUTE
OF
STEEL CONSTRUCTION

FIFTEENTH EDITION

CONTENTS

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DEDICATION

This edition of the *AISC Steel Construction Manual* is dedicated to Robert O. Disque, a retired AISC staff member and long-time member of the AISC Committee on Manuals. Bob, or Mr. Steel, as his friends on the Committee call him, worked closely with the Committee on Manuals, developing the 1st Edition of the LRFD *Manual of Steel Construction* and the 9th Edition ASD *Manual of Steel Construction*. After retiring from AISC in 1991, Bob continued to be involved with the Committee as a member.

He joined AISC in 1959, after working as a structural designer for firms in Philadelphia and New York. His career at AISC began as a District Engineer in Pittsburgh, where he marketed to architects and engineers by providing them with the latest technical information on structural steel. After a brief period as Assistant Chief Engineer, he was promoted to Chief Engineer in 1963 at AISC headquarters, which, at that time, was in New York City. In this capacity, Bob supervised 32 engineers throughout the country. In 1964, he launched the first AISC lecture series on steel design, educating thousands of engineers across the country on various topics related to steel design and construction.

In 1979, Bob left AISC for a brief stint as an associate professor of Civil Engineering at the University of Maine, only to return to AISC a few years later as Assistant Director of Engineering in Chicago, where AISC made its home in the early 1980s. It was at this time that he worked on the development of the two aforementioned AISC *Manuals*. In 1991, Bob retired from AISC and joined the consulting firm of Gibble, Norden, Champion and Brown in Old Saybrook, Connecticut.

Bob invented many things that today are the norm. He created the “snug tight” concept for bolted joints, in conjunction with his contemporary and fellow Manual Committee member Ted Winneberger of W&W Steel Company of Oklahoma City. He coined the term “anchor rods” to highlight that bolts are not rods; the astute reader will also note that it incorporates his initials. He advanced the use of flexible moment connections, formerly known as “Type 2 with Wind Connections,” as a simplified and economical design approach based on the beneficial inelastic behavior of steel.

Bob shared his knowledge of structural steel by authoring numerous papers and the textbook, *Applied Plastic Design of Steel*. He also co-authored the textbook, *Load and Resistance Factor Design of Steel Structures*, with Louis F. Geschwindner and Reidar Bjorhovde. Of greatest importance to this Manual, however, Bob always emphasized that the Manual is not a textbook, but rather a handbook to provide design guidance and aids for practicing engineers.

For all that he has done to advance the practice of structural steel design, this Committee of friends and former colleagues is pleased to dedicate this 15th Edition *Manual* to Mr. Steel.

FOREWORD

The American Institute of Steel Construction, founded in 1921, is the nonprofit technical standards developer and trade organization for the fabricated structural steel industry in the United States. AISC is headquartered in Chicago and has a long tradition of service to the steel construction industry providing timely and reliable information.

The continuing financial support and active participation of Members in the engineering, research and development activities of the Institute make possible the publishing of this *Steel Construction Manual.* Those Members include the following: Full Members engaged in the fabrication, production and sale of structural steel; Associate Members, who include erectors, detailers, service consultants, software developers, and steel product manufacturers; Professional Members, who are structural or civil engineers and architects, including architectural and engineering educators; Affiliate Members, who include general contractors, building inspectors and code officials; and Student Members.

The Institute's objective is to make structural steel the material of choice, by being the leader in structural-steel-related technical and market-building activities, including specification and code development, research, education, technical assistance, quality certification, standardization and market development.

To accomplish this objective, the Institute publishes manuals, design guides and specifications. Best known and most widely used is the *Steel Construction Manual*, which holds a highly respected position in engineering literature. The Manual is based on the *Specification for Structural Steel Buildings* and the *Code of Standard Practice for Steel Buildings and Bridges*. Both standards are included in the Manual for easy reference.

The Institute also publishes technical information and timely articles in its *Engineering Journal*, Design Guide series, *Modern Steel Construction* magazine, and other design aids and research reports. Nearly all of the information AISC publishes is available for download from the AISC web site at **www.aisc.org**.

PREFACE

This Manual is the 15th Edition of the AISC *Steel Construction Manual*, which was first published in 1927. It replaces the 14th Edition *Manual* originally published in 2011.

The following specifications, codes and standards are printed in Part 16 of this Manual:

- 2016 AISC *Specification for Structural Steel Buildings*
- 2014 RCSC *Specification for Structural Joints Using High-Strength Bolts*
- 2016 AISC *Code of Standard Practice for Steel Buildings and Bridges*

The following resources supplement the Manual and are available on the AISC web site at **www.aisc.org:**

- AISC *Design Examples*, which illustrate the application of tables and specification provisions that are included in this Manual.
- AISC *Shapes Database V15.0 and V15.0H.*
- Background and supporting literature (references) for the AISC *Steel Construction Manual.*

The following major changes and improvements have been made in this revision:

- All tabular information and discussions are updated to comply with the 2016 *Specification for Structural Buildings*, and the standards and other documents referenced therein.
- Shape information is updated to ASTM A6/A6M-14 throughout this Manual. Larger pipe, HSS and angle sizes have also been incorporated into the dimensions and properties tables in Part 1.
- The available compressive strength tables are expanded to include 65- and 70-ksi steel for a limited number of shapes.
- In Part 6, a new design aid is included that provides the width-to-thickness slenderness limits for various steel strengths.
- In Part 6, a new design aid is included that provides the available flexural strength, available shear strength, available compressive strength, and available tensile strength for W-shapes in one table.
- In Part 9, a new interaction equation is provided for connection design based on a plastic strength approach.
- In Part 9, a new approach to designing coped beams is presented based on recent studies.

In addition, many other improvements have been made throughout this Manual.

By the AISC Committee on Manuals,

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The committee gratefully acknowledges the contributions made to this Manual by the AISC Committee on Specifications and the following individuals: W. Scott Goodrich, Heath Mitchell, William N. Scott, Marc L. Sorenson, and Sriramulu Vinnakota.

SCOPE

The specification requirements and other design recommendations and considerations summarized in this Manual apply in general to the design and construction of steel buildings and other structures.

The design of seismic force-resisting systems also must meet the requirements in the AISC *Seismic Provisions for Structural Steel Buildings*, except in the following cases for which use of the AISC *Seismic Provisions* is not required:

- Buildings and other structures in seismic design category (SDC) A
- Buildings and other structures in SDC B or C with R = 3 systems [steel systems not specifically detailed for seismic resistance per ASCE/SEI 7 Table 12.2-1 (ASCE, 2016)]
- Nonbuilding structures similar to buildings with $R = 1^1/_2$ braced-frame systems or R = 1 moment-frame systems (see ASCE/SEI 7 Table 15.4-1)
- Nonbuilding structures not similar to buildings (see ASCE/SEI 7 Table 15.4-2), which are designed to meet the requirements in other standards entirely

Conversely, use of the AISC *Seismic Provisions* is required in the following cases:

- Buildings and other structures in SDC B or C when one of the exemptions for steel seismic force-resisting systems above does not apply
- Buildings and other structures in SDC B or C that use composite seismic force-resisting systems (those containing composite steel-and-concrete members and those composed of steel members in combination with reinforced concrete members)
- Buildings in SDC D, E or F
- Nonbuilding structures in SDC D, E or F, when the exemption above does not apply

The AISC *Seismic Design Manual* provides guidance on the use of the AISC *Seismic Provisions*.

The Manual consists of seventeen parts addressing various topics related to steel building design and construction. Part 1 provides the dimensions and properties for structural products commonly used. For proper material specifications for these products, as well as general specification requirements and other design considerations, see Part 2. For the design of members, see Parts 3 through 6. For the design of connections, see Parts 7 through 15. For Specifications and Codes, see Part 16. For other miscellaneous information, see Part 17.

Tables in the Manual that present available strengths are developed using the geometric conditions indicated and the applicable limit states from the AISC *Specification for Structural Steel Buildings.* Given the nature of the tables, and the possible governing limit state for each table value, linear interpolation between tabulated values may or may not provide correct strengths.

REFERENCE

ASCE (2016), *Minimum Design Loads for Buildings and Other Structures*, including Supplement No. 1, ASCE/SEI 7-16, American Society of Civil Engineers, Reston, VA.

PART 1

DIMENSIONS AND PROPERTIES

SCOPE

The dimensions and properties for structural products commonly used in steel building design and construction are given in this Part. Although the dimensions and properties tabulated in Part 1 reflect “commonly” used structural products, some of the shapes listed are not commonly produced or stocked. These shapes are usually only produced to order, and will likely be subject to mill production schedules and minimum order quantities. For availability of shapes, go to **www.aisc.org**. For torsional and flexural-torsional properties of rolled shapes, see AISC Design Guide 9, *Torsional Analysis of Structural Steel Members* (Seaburg and Carter, 1997). For surface areas, box perimeters and areas, *W*/*D* ratios and *A*/*D* ratios, see AISC Design Guide 19, *Fire Resistance of Structural Steel Framing* (Ruddy et al., 2003).

STRUCTURAL PRODUCTS

W-, M-, S- and HP-Shapes

Four types of H-shaped (or I-shaped) members are covered in this Manual:

- W-shapes, which have essentially parallel inner and outer flange surfaces.
- M-shapes, which are H-shaped members that are not classified in ASTM A6 as W-, S- or HP-shapes. M-shapes may have a sloped inside flange face or other cross-section features that do not meet the criteria for W-, S- or HP-shapes.
- S-shapes (also known as American standard beams), which have a slope of approximately $16^2/_3$% (2 on 12) on the inner flange surfaces.
- HP-shapes (also known as bearing piles), which are similar to W-shapes except their webs and flanges are of equal thickness and the depth and flange width are nominally equal for a given designation.

These shapes are designated by the mark W, M, S or HP, nominal depth (in.) and nominal weight (lb/ft). For example, a W24×55 is a W-shape that is nominally 24 in. deep and weighs 55 lb/ft.

The following dimensional and property information is given in this Manual for the W-, M-, S- and HP-shapes covered in ASTM A6:

- Design dimensions, detailing dimensions, axial properties and flexural properties are given in Tables 1-1, 1-2, 1-3 and 1-4 for W-, M-, S- and HP-shapes, respectively.
- SI-equivalent designations are given in Table 17-1 for W-shapes and in Table 17-2 for M-, S- and HP-shapes.

Tabulated decimal values are appropriate for use in design calculations, whereas fractional values are appropriate for use in detailing. All decimal and fractional values are similar with one exception: Because of the variation in fillet sizes used in shape production, the decimal value, k_{des}, is conservatively presented based on the smallest fillet used in production, and the fractional value, k_{det}, is conservatively presented based on the largest fillet used in production. For the definitions of the tabulated variables, refer to the Nomenclature section at the back of this Manual.

When appropriate, this Manual presents tabulated values for the workable gage of a section. The term workable gage refers to the gage for fasteners in the flange that provides for entering and tightening clearances and edge distance and spacing requirements. When the listed value is footnoted, the actual size, combination, and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility. Other gages that provide for entering and tightening clearances and edge distance and spacing requirements can also be used. In Table 1-1, for the shapes W14×145 through W14×873, the Workable Gage column contains either 3-7½-3 or 3-8½-3, whereas for the remainder of the tabulated shapes only a single dimension is given. For these shapes, the three dimensions provide the workable dimension for a row of four holes across the flange. For example, a workable gage of 3-7½-3 indicates that the workable gage for the inner holes is 7½ in., and the workable gage between the inner and outer holes is 3 in.

Channels

Two types of channels are covered in this Manual:

- C-shapes (also known as American standard channels), which have a slope of approximately 16⅔% (2 on 12) on the inner flange surfaces.
- MC-shapes (also known as miscellaneous channels), which have a slope other than 16⅔% (2 on 12) on the inner flange surfaces.

These shapes are designated by the mark C or MC, nominal depth (in.) and nominal weight (lb/ft). For example, a C12×25 is a C-shape that is nominally 12 in. deep and weighs 25 lb/ft.

The following dimensional and property information is given in this Manual for the channels covered in ASTM A6:

- Design dimensions, detailing dimensions, and axial, flexural and torsional properties are given in Tables 1-5 and 1-6 for C- and MC-shapes, respectively.
- SI-equivalent designations are given in Table 17-3.

For the definitions of the tabulated variables, refer to the Nomenclature section at the back of this Manual.

Angles

Angles (also known as L-shapes) have legs of equal thickness and either equal or unequal leg sizes. Angles are designated by the mark L, leg sizes (in.) and thickness (in.). For example, an L4×3×½ is an angle with one 4 in. leg, one 3 in. leg, and ½ in. thickness.

The following dimensional and property information is given in this Manual for the angles covered in ASTM A6:

- Design dimensions, detailing dimensions, and axial, flexural and flexural-torsional properties are given in Table 1-7. The effects of leg-to-leg and toe fillet radii have been considered in the determination of these section properties. The S value that is given for the Z-Z axis in Table 1-7 is based on the largest perpendicular distance measured from the Z-Z axis to the center of the thickness at the tip of the angle toe(s) or heel. Additional properties of single angles are provided in the electronic shapes database available at **www.aisc.org/manualresources**. These properties are used for calculations involving Z-Z and W-W principal axes. For unequal leg angles, the database includes I, and values of S at the toe of the short leg, the heel, and the toe of the long leg for the Z-Z and W-W principal axes. For equal leg angles, the database includes I, and values of S at the toe of the leg and the heel for Z-Z and W-W principal axes.

- Workable gages in angle legs are tabulated in Table 1-7A.
- Width-to-thickness criteria for angles are tabulated in Table 1-7B.
- SI-equivalent designations are given in Table 17-4.

For the definitions of the tabulated variables, refer to the Nomenclature section at the back of this Manual.

Structural Tees (WT-, MT- and ST-Shapes)

Three types of structural tees are covered in this Manual:

- WT-shapes, which are made from W-shapes
- MT-shapes, which are made from M-shapes
- ST-shapes, which are made from S-shapes

These shapes are designated by the mark WT, MT or ST, nominal depth (in.) and nominal weight (lb/ft). WT-, MT- and ST-shapes are split (sheared or thermal-cut) from W-, M- and S-shapes, respectively, and have half the nominal depth and weight of that shape. For example, a WT12×27.5 is a structural tee split from a W-shape (W24×55), is nominally 12 in. deep and weighs 27.5 lb/ft. Although off-center splitting or splitting on two lines can be obtained by special order, the resulting nonstandard shape is not covered in this Manual.

The following dimensional and property information is given in this Manual for the structural tees cut from the W-, M- and S-shapes covered in ASTM A6:

- Design dimensions, detailing dimensions, and axial, flexural and torsional properties are given in Tables 1-8, 1-9 and 1-10 for WT-, MT- and ST-shapes, respectively.
- SI-equivalent designations are given in Table 17-5 for WT-shapes and in Table 17-6 for MT- and ST-shapes.

For the definitions of the tabulated variables, refer to the Nomenclature section at the back of this Manual.

Hollow Structural Sections (HSS)

Three types of HSS are covered in this Manual:

- Rectangular HSS, which have an essentially rectangular cross section, except for rounded corners, and uniform wall thickness, except at the weld seam(s)
- Square HSS, which have an essentially square cross section, except for rounded corners, and uniform wall thickness, except at the weld seam(s)
- Round HSS, which have an essentially round cross section and uniform wall thickness, except at the weld seam(s)

In each case, ASTM A500 covers only electric-resistance-welded (ERW) HSS with a maximum periphery of 64 in. The coverage of HSS in this Manual is similarly limited.

Rectangular HSS are designated by the mark HSS, overall outside dimensions (in.), and wall thickness (in.), with all dimensions expressed as fractional numbers. For example, an HSS10×10×1/2 is nominally 10 in. by 10 in. with a 1/2 in. wall thickness. Round HSS are designated by the term HSS, nominal outside diameter (in.), and wall thickness (in.) with both dimensions expressed to three decimal places. For example, an HSS10.000×0.500 is nominally 10 in. in diameter with a 1/2 in. nominal wall thickness.

Per AISC *Specification* Section B4.2, the wall thickness used in design, t_{des}, is taken as 0.93 times the nominal wall thickness for ASTM A500. The rationale for this requirement is explained in the corresponding *Specification* Commentary Section B4.2.

In calculating the b/t and h/t ratios in Tables 1-11 and 1-12, each corner radius is taken as $1.5t_{des}$ for rectangular and square HSS. This is in conformity with AISC *Specification* Section B4.1b(d), which states, "If the corner radius is not known, b and h shall be taken as the corresponding outside dimension minus three times the thickness. The thickness, t, shall be taken as the design wall thickness, per Section B4.2." In Table 1-11, b is the lesser value and h is the greater value of the outside dimensions. When using AISC *Specification* Table B4.1a, Case 6, with Table 1-11, b/t should be taken from the h/t column in the table. In other tabulated properties, each corner radius is taken as $2t_{des}$. In the tabulated workable flat dimensions of rectangular (and square) HSS, the outside corner radii are taken as $2.25t_{nom}$. The term workable flat refers to a reasonable flat width or depth of material for use in making connections to HSS. The workable flat dimension is provided as a reflection of current industry practice, although the tolerances of ASTM A500 allow a greater maximum corner radius of $3t_{nom}$.

The following dimensional and property information is given in this Manual for the HSS covered in ASTM A500, A501, A618 or A847:

- Design dimensions, detailing dimensions, and axial, strong-axis flexural, weak-axis flexural, torsional, and flexural-torsional properties are given in Tables 1-11 and 1-12 for rectangular and square HSS, respectively.
- Design dimensions, detailing dimensions, and axial, flexural and torsional properties are given in Table 1-13 for round HSS.
- SI-equivalent designations are given in Tables 17-7, 17-8 and 17-9 for rectangular, square and round HSS, respectively.
- Width-to-thickness criteria of rectangular and square HSS are given in Table 1-12A.

AISC *Specification* Chapter A references ASTM A1065 and ASTM A1085 for HSS. These specifications differ from the other current specifications in the controls on thickness and corner radii. Both specifications control wall thickness such that the geometrical properties can be determined based on the nominal wall thickness, t_{nom}. Dimensions and properties for ASTM A1085 are available at **www.aisc.org/manualresources**. Dimensions and properties for ASTM A1065 are available from the Steel Tube Institute (STI, 2015). ASTM A1065 retains the upper limit on the corner radius of $3t$, as required in ASTM A500. ASTM A1085 limits corner radius to a lower and upper limit depending on wall thickness as follows:

$$t \leq 0.400 \text{ in.} \quad R_{min} = 1.6t,\ R_{max} = 3t$$
$$t > 0.400 \text{ in.} \quad R_{min} = 1.8t,\ R_{max} = 3t$$

As was the case previously, due to production variations within specified limits for rectangular (and square) HSS, it is necessary to establish a basis for the calculation of properties affected by the corner radius dimension. The same radii that are used in the Part 1 tables are recommended for the properties of shapes produced to ASTM A1065 and ASTM A1085:

- b/t and h/t calculated using a corner radius of $1.5t_{nom}$ per AISC *Specification* Sections B4.1b(d) and B4.2 for HSS produced to ASTM A1065 and ASTM A1085
- Other tabulated properties are calculated using $2t_{nom}$
- Workable flat dimensions are calculated using $2.25t_{nom}$

For the definitions of the tabulated variables, refer to the Nomenclature section at the back of this Manual.

Pipes

Pipes have an essentially round cross section and uniform thickness, except at the weld seam(s) for welded pipe.

Pipes up to and including NPS 12 are designated by the term Pipe, nominal diameter (in.) and weight class (Std., x-Strong, xx-Strong). NPS stands for nominal pipe size. For example, Pipe 5 Std. denotes a pipe with a 5 in. nominal diameter and a 0.258 in. wall thickness, which corresponds to the standard weight series. Pipes with wall thicknesses that do not correspond to the foregoing weight classes are designated by the term Pipe, outside diameter (in.), and wall thickness (in.), with both expressed to three decimal places. For example, Pipe 14.000×0.375 and Pipe 5.563×0.500 are proper designations.

Per AISC *Specification* Section B4.2, the wall thickness used in design, t_{des}, is taken as 0.93 times the nominal wall thickness, t_{nom}. The rationale for this requirement is explained in the corresponding *Specification* Commentary Section B4.2.

The following dimensional and property information is given in this Manual for the pipes covered in ASTM A53:

- Design dimensions, detailing dimensions, and axial, flexural and torsional properties are given in Table 1-14.
- SI-equivalent designations are given in Table 17-9.

For the definitions of the tabulated variables, refer to the Nomenclature section at the back of this Manual.

Double Angles

Double angles (also known as 2L-shapes) are made with two angles that are interconnected through their back-to-back legs along the length of the member, either in contact for the full length or separated by spacers at the points of interconnection.

These shapes are designated by the mark 2L, the sizes and thickness of their legs (in.), and their orientation when the angle legs are not of equal size (LLBB or SLBB)[1]. For example, a 2L4×3×1/2 LLBB has two angles with one 4 in. leg and one 3 in. leg and the 4 in. legs are back-to-back; a 2L4×3×1/2 SLBB is similar, except the 3 in. legs are back-to-back. In both cases, the legs are 1/2 in. thick.

The following dimensional and property information is given in this Manual for the double angles built-up from the angles covered in ASTM A6:

- Design dimensions, detailing dimensions, and axial, strong-axis flexural, weak-axis flexural, torsional, and flexural-torsional properties are given in Table 1-15 for equal-leg, LLBB and SLBB angles. For angle legs 8 in. or less, angle separations of zero in., 3/8 in. and 3/4 in. are covered. For longer angle legs, angle separations of zero, 3/4 in. and 1 1/2 in. are covered. The effects of leg-to-leg and toe fillet radii have been considered in the determination of these section properties. For workable gages on legs of angles, see Table 1-7A.

[1] LLBB stands for long legs back-to-back. SLBB stands for short legs back-to-back. Alternatively, the orientations LLV and SLV, which stand for long legs vertical and short legs vertical, respectively, can be used.

For the definitions of the tabulated variables, refer to the Nomenclature section at the back of this Manual.

Double Channels

Double channels (also known as 2C- and 2MC-shapes) are made with two channels that are interconnected through their back-to-back webs along the length of the member, either in contact for the full length or separated by spacers at the points of interconnection.

These shapes are designated by the mark 2C or 2MC, nominal depth (in.), and nominal weight per channel (lb/ft). For example, a 2C12×25 is a double channel that consists of two channels that are each nominally 12 in. deep and each weigh 25 lb/ft.

The following dimensional and property information is given in this Manual for the double channels built-up from the channels covered in ASTM A6:

- Design dimensions, detailing dimensions, and axial, strong-axis flexural, and weak-axis flexural properties are given in Tables 1-16 and 1-17 for 2C- and 2MC-shapes, respectively. In each case, channel separations of zero, 3/8 in. and 3/4 in. are covered.

For the definitions of the tabulated variables, refer to the Nomenclature section at the back of this Manual.

W-Shapes and S-Shapes with Cap Channels

Common combined sections made with W- or S-shapes and channels (C- or MC-shapes) are tabulated in this Manual. In either case, the channel web is interconnected to the W-shape or S-shape top flange, respectively, with the flange toes down. The interconnection of the two elements must be designed for the horizontal shear, q, where

$$q = \frac{VQ}{I} \tag{1-1}$$

where

I = moment of inertia of the combined cross section, in.^{4}
Q = first moment of the channel area about the neutral axis of the combined cross section, in.^{3}
V = vertical shear, kips
q = horizontal shear, kip/in.

The effects of other forces, such as crane horizontal and lateral forces, may also require consideration, when applicable.

The following dimensional and property information is given in this Manual for combined sections built-up from the W-shapes, S-shapes and cap channels covered in ASTM A6:

- Design dimensions, detailing dimensions, and axial, strong-axis flexural, and weak-axis flexural properties of W-shapes with cap channels are given in Table 1-19.
- Design dimensions, detailing dimensions, and axial, strong-axis flexural, and weak-axis flexural properties of S-shapes with cap channels are given in Table 1-20.

For the definitions of the tabulated variables, refer to the Nomenclature section at the back of this Manual.

Plate and Bar Products

Plate products may be ordered as sheet, strip or bar material. Sheet and strip are distinguished from structural bars and plates by their dimensional characteristics, as outlined in Table 2-3 and Table 2-5.

The historical classification system for structural bars and plates suggests that there is only a physical difference between them based upon size and production procedure. In raw form, flat stock has historically been classified as a bar if it is less than or equal to 8 in. wide and as a plate if it is greater than 8 in. wide. Bars are rolled between horizontal and vertical rolls and trimmed to length by shearing or thermal cutting on the ends only. Plates are generally produced using one of two methods:

1. Sheared plates are rolled between horizontal rolls and trimmed to width and length by shearing or thermal cutting on the edges and ends; or
2. Stripped plates are sheared or thermal cut from wider sheared plates.

There is very little, if any, structural difference between plates and bars. Consequently, the term plate is becoming a universally applied term today and a PL$^1/_2$×4$^1/_2$×1 ft 3 in., for example, might be fabricated from plate or bar stock.

For structural plates, the preferred practice is to specify thickness in $^1/_{16}$ in. increments up to $^3/_8$ in. thickness, $^1/_8$ in. increments over $^3/_8$ in. to 1 in. thickness, and $^1/_4$ in. increments over 1 in. thickness. The current extreme width for sheared plates is 200 in. Because mill practice regarding plate widths vary, individual mills should be consulted to determine preferences.

For bars, the preferred practice is to specify width in $^1/_4$ in. increments, and thickness and diameter in $^1/_8$ in. increments.

Raised-Pattern Floor Plates

Weights of raised-pattern floor plates are given in Table 1-18. Raised-pattern floor plates are commonly available in widths up to 120 in. For larger plate widths, see literature available from floor plate producers.

Crane Rails

Although crane rails are not listed as structural steel in the AISC *Code of Standard Practice* Section 2.1, this information is provided because some fabricators may choose to provide crane rails. Crane rails are designated by unit weight in lb/yard. Dimensions and properties for the crane rails shown are given in Table 1-21. Crane rails can be either heat treated or end hardened to reduce wear. For additional information or for profiles and properties of crane rails not listed, manufacturer's catalogs should be consulted. For crane-rail connections, see Part 15.

Other Structural Products

The following other structural products are covered in this Manual as indicated:

- High-strength bolts, common bolts, washers, nuts and direct-tension-indicator washers are covered in Part 7.
- Welding filler metals and fluxes are covered in Part 8.

- Forged steel structural hardware items, such as clevises, turnbuckles, sleeve nuts, recessed-pin nuts, and cotter pins are covered in Part 15.
- Anchor rods and threaded rods are covered in Part 14.

STANDARD MILL PRACTICES

The production of structural products is subject to unavoidable variations relative to the theoretical dimensions and profiles, due to many factors, including roll wear, roll dressing practices and temperature effects. Such variations are limited by the dimensional and profile tolerances as summarized below.

Hot-Rolled Structural Shapes

Acceptable dimensional tolerances for hot-rolled structural shapes (W-, M-, S- and HP-shapes), channels (C- and MC-shapes), and angles are given in ASTM A6 Section 12 and summarized in Tables 1-22 through 1-26. Supplementary information, including permissible variations for sheet and strip and for other grades of steel, can also be found in literature from steel plate producers and the Association of Iron and Steel Technology.

Hollow Structural Sections

Acceptable dimensional tolerances for HSS are given in ASTM A500 Section 11, A501 Section 12, A618 Section 8, A847 Section 10, A1065 Section 8, or A1085 Section 12, as applicable, and summarized in Tables 1-27 and 1-28, for rectangular and square, and round HSS, respectively. Supplementary information can also be found in literature from HSS producers and the Steel Tube Institute.

Pipes

Acceptable dimensional tolerances for pipes are given in ASTM A53 Section 10 and summarized in Table 1-28. Supplementary information can also be found in literature from pipe producers.

Plate Products

Acceptable dimensional tolerances for plate products are given in ASTM A6 Section 12 and summarized in Table 1-29. Note that plate thickness can be specified in inches or by weight per square foot, and separate tolerances apply to each method. No decimal edge thickness can be assured for plate specified by the latter method. Supplementary information, including permissible variations for sheet and strip and for other grades of steel, can also be found in literature from steel plate producers and the Association of Iron and Steel Technology.

PART 1 REFERENCES

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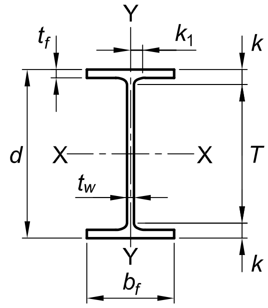


Table 1-1
W-Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		Web $\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k k_{des}	Distance k k_{det}	Distance k_1	Distance T	Distance Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W44×335[c]	98.5	44.0	44	1.03	1	1/2	15.9	16	1.77	1 3/4	2.56	3	1 3/4	38	5 1/2
×290[c]	85.4	43.6	43 5/8	0.865	7/8	7/16	15.8	15 7/8	1.58	1 9/16	2.36	2 13/16	1 5/8		
×262[c]	77.2	43.3	43 1/4	0.785	13/16	7/16	15.8	15 3/4	1.42	1 7/16	2.20	2 5/8	1 5/8		
×230[c,v]	67.8	42.9	42 7/8	0.710	11/16	3/8	15.8	15 3/4	1.22	1 1/4	2.01	2 7/16	1 9/16		
W40×655[h]	193	43.6	43 5/8	1.97	2	1	16.9	16 7/8	3.54	3 9/16	4.72	4 13/16	2 3/16	34	7 1/2
×593[h]	174	43.0	43	1.79	1 13/16	15/16	16.7	16 3/4	3.23	3 1/4	4.41	4 1/2	2 1/8		
×503[h]	148	42.1	42	1.54	1 9/16	13/16	16.4	16 3/8	2.76	2 3/4	3.94	4	2		
×431[h]	127	41.3	41 1/4	1.34	1 5/16	11/16	16.2	16 1/4	2.36	2 3/8	3.54	3 5/8	1 7/8		
×397[h]	117	41.0	41	1.22	1 1/4	5/8	16.1	16 1/8	2.20	2 3/16	3.38	3 1/2	1 13/16		
×372[h]	110	40.6	40 5/8	1.16	1 3/16	5/8	16.1	16 1/8	2.05	2 1/16	3.23	3 5/16	1 13/16		
×362[h]	106	40.6	40 1/2	1.12	1 1/8	9/16	16.0	16	2.01	2	3.19	3 1/4	1 3/4		
×324	95.3	40.2	40 1/8	1.00	1	1/2	15.9	15 7/8	1.81	1 13/16	2.99	3 1/16	1 11/16		
×297[c]	87.3	39.8	39 7/8	0.930	15/16	1/2	15.8	15 7/8	1.65	1 5/8	2.83	2 15/16	1 11/16		
×277[c]	81.5	39.7	39 3/4	0.830	13/16	7/16	15.8	15 7/8	1.58	1 9/16	2.76	2 7/8	1 5/8		
×249[c]	73.5	39.4	39 3/8	0.750	3/4	3/8	15.8	15 3/4	1.42	1 7/16	2.60	2 11/16	1 9/16		
×215[c]	63.5	39.0	39	0.650	5/8	5/16	15.8	15 3/4	1.22	1 1/4	2.40	2 1/2	1 9/16		
×199[c]	58.8	38.7	38 5/8	0.650	5/8	5/16	15.8	15 3/4	1.07	1 1/16	2.25	2 5/16	1 9/16		
W40×392[h]	116	41.6	41 5/8	1.42	1 7/16	3/4	12.4	12 3/8	2.52	2 1/2	3.70	3 13/16	1 15/16	34	7 1/2
×331[h]	97.7	40.8	40 3/4	1.22	1 1/4	5/8	12.2	12 1/8	2.13	2 1/8	3.31	3 3/8	1 13/16		
×327[h]	95.9	40.8	40 3/4	1.18	1 3/16	5/8	12.1	12 1/8	2.13	2 1/8	3.31	3 3/8	1 13/16		
×294	86.2	40.4	40 3/8	1.06	1 1/16	9/16	12.0	12	1.93	1 15/16	3.11	3 3/16	1 3/4		
×278	82.3	40.2	40 1/8	1.03	1	1/2	12.0	12	1.81	1 13/16	2.99	3 1/16	1 3/4		
×264	77.4	40.0	40	0.960	15/16	1/2	11.9	11 7/8	1.73	1 3/4	2.91	3	1 11/16		
×235[c]	69.1	39.7	39 3/4	0.830	13/16	7/16	11.9	11 7/8	1.58	1 9/16	2.76	2 7/8	1 5/8		
×211[c]	62.1	39.4	39 3/8	0.750	3/4	3/8	11.8	11 3/4	1.42	1 7/16	2.60	2 11/16	1 9/16		
×183[c]	53.3	39.0	39	0.650	5/8	5/16	11.8	11 3/4	1.20	1 3/16	2.38	2 1/2	1 9/16		
×167[c]	49.3	38.6	38 5/8	0.650	5/8	5/16	11.8	11 3/4	1.03	1	2.21	2 5/16	1 9/16		
×149[c,v]	43.8	38.2	38 1/4	0.630	5/8	5/16	11.8	11 3/4	0.830	13/16	2.01	2 1/8	1 1/2		

[c] Shape is slender for compression with $F_y = 50$ ksi.
[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi.

Table 1-1 (continued)
W-Shapes
Properties

W44–W40

Nom-inal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	*I*	*S*	*r*	*Z*	*I*	*S*	*r*	*Z*			*J*	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
335	4.50	38.0	31100	1410	17.8	1620	1200	150	3.49	236	4.24	42.2	74.7	535000
290	5.02	45.0	27000	1240	17.8	1410	1040	132	3.49	205	4.20	42.0	50.9	461000
262	5.57	49.6	24100	1110	17.7	1270	923	117	3.47	182	4.17	41.9	37.3	405000
230	6.45	54.8	20800	971	17.5	1100	796	101	3.43	157	4.13	41.7	24.9	346000
655	2.39	17.3	56500	2590	17.1	3080	2870	340	3.86	542	4.71	40.1	589	1150000
593	2.58	19.1	50400	2340	17.0	2760	2520	302	3.80	481	4.63	39.8	445	997000
503	2.98	22.3	41600	1980	16.8	2320	2040	249	3.72	394	4.50	39.3	277	789000
431	3.44	25.5	34800	1690	16.6	1960	1690	208	3.65	328	4.41	38.9	177	638000
397	3.66	28.0	32000	1560	16.6	1800	1540	191	3.64	300	4.38	38.8	142	579000
372	3.93	29.5	29600	1460	16.5	1680	1420	177	3.60	277	4.33	38.6	116	528000
362	3.99	30.5	28900	1420	16.5	1640	1380	173	3.60	270	4.33	38.6	109	513000
324	4.40	34.2	25600	1280	16.4	1460	1220	153	3.58	239	4.27	38.4	79.4	448000
297	4.80	36.8	23200	1170	16.3	1330	1090	138	3.54	215	4.22	38.2	61.2	399000
277	5.03	41.2	21900	1100	16.4	1250	1040	132	3.58	204	4.25	38.1	51.5	379000
249	5.55	45.6	19600	993	16.3	1120	926	118	3.55	182	4.21	38.0	38.1	334000
215	6.45	52.6	16700	859	16.2	964	803	101	3.54	156	4.19	37.8	24.8	284000
199	7.39	52.6	14900	770	16.0	869	695	88.2	3.45	137	4.12	37.6	18.3	246000
392	2.45	24.1	29900	1440	16.1	1710	803	130	2.64	212	3.30	39.1	172	306000
331	2.86	28.0	24700	1210	15.9	1430	644	106	2.57	172	3.21	38.7	105	241000
327	2.85	29.0	24500	1200	16.0	1410	640	105	2.58	170	3.21	38.7	103	239000
294	3.11	32.2	21900	1080	15.9	1270	562	93.5	2.55	150	3.16	38.5	76.6	208000
278	3.31	33.3	20500	1020	15.8	1190	521	87.1	2.52	140	3.13	38.4	65.0	192000
264	3.45	35.6	19400	971	15.8	1130	493	82.6	2.52	132	3.12	38.3	56.1	181000
235	3.77	41.2	17400	875	15.9	1010	444	74.6	2.54	118	3.11	38.1	41.3	161000
211	4.17	45.6	15500	786	15.8	906	390	66.1	2.51	105	3.07	38.0	30.4	141000
183	4.92	52.6	13200	675	15.7	774	331	56.0	2.49	88.3	3.04	37.8	19.3	118000
167	5.76	52.6	11600	600	15.3	693	283	47.9	2.40	76.0	2.98	37.6	14.0	99700
149	7.11	54.3	9800	513	15.0	598	229	38.8	2.29	62.2	2.89	37.4	9.36	80000

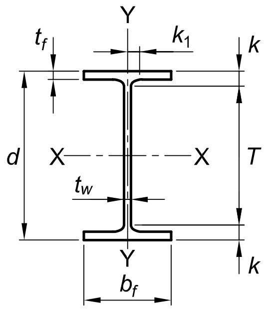


Table 1-1 (continued)
W-Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		Web $\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k k_{des}	Distance k k_{det}	k_1	T	Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W36×925[h]	272	43.1	43 1/8	3.02	3	1 1/2	18.6	18 5/8	4.53	4 1/2	5.28	5 3/8	2 5/16	32 3/8	7 1/2
×853[h]	251	43.1	43 1/8	2.52	2 1/2	1 1/4	18.2	18 1/4	4.53	4 1/2	5.28	5 3/8	2 1/16		
×802[h]	236	42.6	42 5/8	2.38	2 3/8	1 3/16	18.0	18	4.29	4 5/16	5.04	5 1/8	2		
×723[h]	213	41.8	41 3/4	2.17	2 3/16	1 1/8	17.8	17 3/4	3.90	3 7/8	4.65	4 11/16	1 7/8		
×652[h]	192	41.1	41	1.97	2	1	17.6	17 5/8	3.54	3 9/16	4.49	4 13/16	2 3/16	31 3/8	
×529[h]	156	39.8	39 3/4	1.61	1 5/8	13/16	17.2	17 1/4	2.91	2 15/16	3.86	4 3/16	2		
×487[h]	143	39.3	39 3/8	1.50	1 1/2	3/4	17.1	17 1/8	2.68	2 11/16	3.63	4	1 7/8		
×441[h]	130	38.9	38 7/8	1.36	1 3/8	11/16	17.0	17	2.44	2 7/16	3.39	3 3/4	1 7/8		
×395[h]	116	38.4	38 3/8	1.22	1 1/4	5/8	16.8	16 7/8	2.20	2 3/16	3.15	3 7/16	1 13/16		
×361[h]	106	38.0	38	1.12	1 1/8	9/16	16.7	16 3/4	2.01	2	2.96	3 5/16	1 3/4		
×330	96.9	37.7	37 5/8	1.02	1	1/2	16.6	16 5/8	1.85	1 7/8	2.80	3 1/8	1 3/4		
×302	89.0	37.3	37 3/8	0.945	15/16	1/2	16.7	16 5/8	1.68	1 11/16	2.63	3	1 11/16		
×282[c]	82.9	37.1	37 1/8	0.885	7/8	7/16	16.6	16 5/8	1.57	1 9/16	2.52	2 7/8	1 5/8		
×262[c]	77.2	36.9	36 7/8	0.840	13/16	7/16	16.6	16 1/2	1.44	1 7/16	2.39	2 3/4	1 5/8		
×247[c]	72.5	36.7	36 5/8	0.800	13/16	7/16	16.5	16 1/2	1.35	1 3/8	2.30	2 5/8	1 5/8		
×231[c]	68.2	36.5	36 1/2	0.760	3/4	3/8	16.5	16 1/2	1.26	1 1/4	2.21	2 9/16	1 9/16		
W36×256	75.3	37.4	37 3/8	0.960	15/16	1/2	12.2	12 1/4	1.73	1 3/4	2.48	2 15/16	1 11/16	31 1/2	5 1/2
×232[c]	68.0	37.1	37 1/8	0.870	7/8	7/16	12.1	12 1/8	1.57	1 9/16	2.32	2 13/16	1 5/8		
×210[c]	61.9	36.7	36 3/4	0.830	13/16	7/16	12.2	12 1/8	1.36	1 3/8	2.11	2 5/8	1 5/8		
×194[c]	57.0	36.5	36 1/2	0.765	3/4	3/8	12.1	12 1/8	1.26	1 1/4	2.01	2 1/2	1 9/16		
×182[c]	53.6	36.3	36 3/8	0.725	3/4	3/8	12.1	12 1/8	1.18	1 3/16	1.93	2 3/8	1 9/16		
×170[c]	50.0	36.2	36 1/8	0.680	11/16	3/8	12.0	12	1.10	1 1/8	1.85	2 3/8	1 9/16		
×160[c]	47.0	36.0	36	0.650	5/8	5/16	12.0	12	1.02	1	1.77	2 1/4	1 9/16		
×150[c]	44.3	35.9	35 7/8	0.625	5/8	5/16	12.0	12	0.940	15/16	1.69	2 3/16	1 1/2		
×135[c,v]	39.9	35.6	35 1/2	0.600	5/8	5/16	12.0	12	0.790	13/16	1.54	2 1/16	1 1/2		
W33×387[h]	114	36.0	36	1.26	1 1/4	5/8	16.2	16 1/4	2.28	2 1/4	3.07	3 9/16	1 13/16	28 7/8	5 1/2
×354[h]	104	35.6	35 1/2	1.16	1 3/16	5/8	16.1	16 1/8	2.09	2 1/16	2.88	3 3/8	1 13/16		
×318	93.7	35.2	35 1/8	1.04	1 1/16	9/16	16.0	16	1.89	1 7/8	2.68	3 3/16	1 3/4		
×291	85.6	34.8	34 7/8	0.960	15/16	1/2	15.9	15 7/8	1.73	1 3/4	2.52	2 15/16	1 11/16		
×263	77.4	34.5	34 1/2	0.870	7/8	7/16	15.8	15 3/4	1.57	1 9/16	2.36	2 13/16	1 5/8		
×241[c]	71.1	34.2	34 1/8	0.830	13/16	7/16	15.9	15 7/8	1.40	1 3/8	2.19	2 11/16	1 5/8		
×221[c]	65.3	33.9	33 7/8	0.775	3/4	3/8	15.8	15 3/4	1.28	1 1/4	2.06	2 1/2	1 5/8		
×201[c]	59.1	33.7	33 5/8	0.715	11/16	3/8	15.7	15 3/4	1.15	1 1/8	1.94	2 7/16	1 9/16		

[c] Shape is slender for compression with $F_y = 50$ ksi.
[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi.

Table 1-1 (continued)
W-Shapes
Properties

W36–W33

Nom-inal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z			J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
925	2.05	10.8	73000	3390	16.4	4130	4940	531	4.26	862	5.30	38.6	1430	1840000
853	2.01	12.9	70000	3250	16.7	3920	4600	505	4.28	805	5.22	38.6	1240	1710000
802	2.10	13.7	64800	3040	16.6	3660	4210	468	4.22	744	5.15	38.3	1050	1540000
723	2.28	15.0	57300	2740	16.4	3270	3700	416	4.17	658	5.06	37.9	785	1330000
652	2.48	16.3	50600	2460	16.2	2910	3230	367	4.10	581	4.96	37.6	593	1130000
529	2.96	19.9	39600	1990	16.0	2330	2490	289	4.00	454	4.80	36.9	327	846000
487	3.19	21.4	36000	1830	15.8	2130	2250	263	3.96	412	4.74	36.6	258	754000
441	3.48	23.6	32100	1650	15.7	1910	1990	235	3.92	368	4.69	36.5	194	661000
395	3.83	26.3	28500	1490	15.7	1710	1750	208	3.88	325	4.61	36.2	142	575000
361	4.16	28.6	25700	1350	15.6	1550	1570	188	3.85	293	4.58	36.0	109	509000
330	4.49	31.4	23300	1240	15.5	1410	1420	171	3.83	265	4.53	35.9	84.3	456000
302	4.96	33.9	21100	1130	15.4	1280	1300	156	3.82	241	4.53	35.6	64.3	412000
282	5.29	36.2	19600	1050	15.4	1190	1200	144	3.80	223	4.50	35.5	52.7	378000
262	5.75	38.2	17900	972	15.3	1100	1090	132	3.76	204	4.46	35.5	41.6	342000
247	6.11	40.1	16700	913	15.2	1030	1010	123	3.74	190	4.42	35.4	34.7	316000
231	6.54	42.2	15600	854	15.1	963	940	114	3.71	176	4.40	35.2	28.7	292000
256	3.53	33.8	16800	895	14.9	1040	528	86.5	2.65	137	3.24	35.7	52.9	168000
232	3.86	37.3	15000	809	14.8	936	468	77.2	2.62	122	3.21	35.5	39.6	148000
210	4.48	39.1	13200	719	14.6	833	411	67.5	2.58	107	3.18	35.3	28.0	128000
194	4.81	42.4	12100	664	14.6	767	375	61.9	2.56	97.7	3.15	35.2	22.2	116000
182	5.12	44.8	11300	623	14.5	718	347	57.6	2.55	90.7	3.13	35.1	18.5	107000
170	5.47	47.7	10500	581	14.5	668	320	53.2	2.53	83.8	3.11	35.1	15.1	98500
160	5.88	49.9	9760	542	14.4	624	295	49.1	2.50	77.3	3.09	35.0	12.4	90200
150	6.37	51.9	9040	504	14.3	581	270	45.1	2.47	70.9	3.06	35.0	10.1	82200
135	7.56	54.1	7800	439	14.0	509	225	37.7	2.38	59.7	2.99	34.8	7.00	68100
387	3.55	23.7	24300	1350	14.6	1560	1620	200	3.77	312	4.49	33.7	148	459000
354	3.85	25.7	22000	1240	14.5	1420	1460	181	3.74	282	4.44	33.5	115	408000
318	4.23	28.7	19500	1110	14.5	1270	1290	161	3.71	250	4.40	33.3	84.4	357000
291	4.60	31.0	17700	1020	14.4	1160	1160	146	3.68	226	4.34	33.1	65.1	319000
263	5.03	34.3	15900	919	14.3	1040	1040	131	3.66	202	4.31	32.9	48.7	281000
241	5.66	35.9	14200	831	14.1	940	933	118	3.62	182	4.29	32.8	36.2	251000
221	6.20	38.5	12900	759	14.1	857	840	106	3.59	164	4.25	32.6	27.8	224000
201	6.85	41.7	11600	686	14.0	773	749	95.2	3.56	147	4.21	32.6	20.8	198000

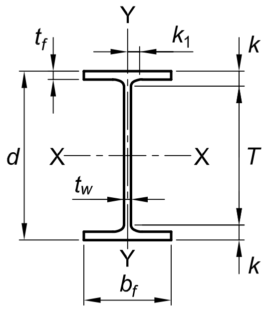


Table 1-1 (continued)
W-Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		Web $\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k k_{des}	Distance k k_{det}	Distance k_1	Distance T	Distance Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W33×169[c]	49.5	33.8	33 7/8	0.670	11/16	3/8	11.5	11 1/2	1.22	1 1/4	1.92	2 7/16	1 9/16	28 7/8	5 1/2
×152[c]	44.9	33.5	33 1/2	0.635	5/8	5/16	11.6	11 5/8	1.06	1 1/16	1.76	2 5/16	1 1/2		
×141[c]	41.5	33.3	33 1/4	0.605	5/8	5/16	11.5	11 1/2	0.960	15/16	1.66	2 3/16	1 1/2		
×130[c]	38.3	33.1	33 1/8	0.580	9/16	5/16	11.5	11 1/2	0.855	7/8	1.56	2 1/8	1 1/2		
×118[c,v]	34.7	32.9	32 7/8	0.550	9/16	5/16	11.5	11 1/2	0.740	3/4	1.44	2	1 1/2		
W30×391[h]	115	33.2	33 1/4	1.36	1 3/8	11/16	15.6	15 5/8	2.44	2 7/16	3.23	3 3/4	1 7/8	25 3/4	5 1/2
×357[h]	105	32.8	32 3/4	1.24	1 1/4	5/8	15.5	15 1/2	2.24	2 1/4	3.03	3 1/2	1 13/16		
×326[h]	95.9	32.4	32 3/8	1.14	1 1/8	9/16	15.4	15 3/8	2.05	2 1/16	2.84	3 5/16	1 3/4		
×292	86.0	32.0	32	1.02	1	1/2	15.3	15 1/4	1.85	1 7/8	2.64	3 1/8	1 3/4		
×261	77.0	31.6	31 5/8	0.930	15/16	1/2	15.2	15 1/8	1.65	1 5/8	2.44	2 15/16	1 11/16		
×235	69.3	31.3	31 1/4	0.830	13/16	7/16	15.1	15	1.50	1 1/2	2.29	2 3/4	1 5/8		
×211	62.3	30.9	31	0.775	3/4	3/8	15.1	15 1/8	1.32	1 5/16	2.10	2 9/16	1 5/8		
×191[c]	56.1	30.7	30 5/8	0.710	11/16	3/8	15.0	15	1.19	1 3/16	1.97	2 1/2	1 9/16		
×173[c]	50.9	30.4	30 1/2	0.655	5/8	5/16	15.0	15	1.07	1 1/16	1.85	2 5/16	1 9/16		
W30×148[c]	43.6	30.7	30 5/8	0.650	5/8	5/16	10.5	10 1/2	1.18	1 3/16	1.83	2 1/2	1 9/16	25 3/4	5 1/2
×132[c]	38.8	30.3	30 1/4	0.615	5/8	5/16	10.5	10 1/2	1.00	1	1.65	2 1/4	1 1/2		
×124[c]	36.5	30.2	30 1/8	0.585	9/16	5/16	10.5	10 1/2	0.930	15/16	1.58	2 1/4	1 1/2		
×116[c]	34.2	30.0	30	0.565	9/16	5/16	10.5	10 1/2	0.850	7/8	1.50	2 1/8	1 1/2		
×108[c]	31.7	29.8	29 7/8	0.545	9/16	5/16	10.5	10 1/2	0.760	3/4	1.41	2	1 1/2		
×99[c]	29.0	29.7	29 5/8	0.520	1/2	1/4	10.5	10 1/2	0.670	11/16	1.32	2	1 1/2		
×90[c,v]	26.3	29.5	29 1/2	0.470	1/2	1/4	10.4	10 3/8	0.610	5/8	1.26	1 7/8	1 7/16		
W27×539[h]	159	32.5	32 1/2	1.97	2	1	15.3	15 1/4	3.54	3 9/16	4.33	4 7/16	1 13/16	23	5 1/2[g]
×368[h]	109	30.4	30 3/8	1.38	1 3/8	11/16	14.7	14 5/8	2.48	2 1/2	3.27	3 11/16	1 7/8		5 1/2
×336[h]	99.2	30.0	30	1.26	1 1/4	5/8	14.6	14 1/2	2.28	2 1/4	3.07	3 1/2	1 13/16		
×307[h]	90.2	29.6	29 5/8	1.16	1 3/16	5/8	14.4	14 1/2	2.09	2 1/16	2.88	3 5/16	1 13/16		
×281	83.1	29.3	29 1/4	1.06	1 1/16	9/16	14.4	14 3/8	1.93	1 15/16	2.72	3 1/8	1 3/4		
×258	76.1	29.0	29	0.980	1	1/2	14.3	14 1/4	1.77	1 3/4	2.56	3	1 11/16		
×235	69.4	28.7	28 5/8	0.910	15/16	1/2	14.2	14 1/4	1.61	1 5/8	2.40	2 7/8	1 11/16		
×217	63.9	28.4	28 3/8	0.830	13/16	7/16	14.1	14 1/8	1.50	1 1/2	2.29	2 11/16	1 5/8		
×194	57.1	28.1	28 1/8	0.750	3/4	3/8	14.0	14	1.34	1 5/16	2.13	2 9/16	1 9/16		
×178	52.5	27.8	27 3/4	0.725	3/4	3/8	14.1	14 1/8	1.19	1 3/16	1.98	2 3/8	1 9/16		
×161[c]	47.6	27.6	27 5/8	0.660	11/16	3/8	14.0	14	1.08	1 1/16	1.87	2 5/16	1 9/16		
×146[c]	43.2	27.4	27 3/8	0.605	5/8	5/16	14.0	14	0.975	1	1.76	2 3/16	1 1/2		

[c] Shape is slender for compression with $F_y = 50$ ksi.
[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.
[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi.

Table 1-1 (continued)
W-Shapes
Properties

W33–W27

Nom-inal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z			J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
169	4.71	44.7	9290	549	13.7	629	310	53.9	2.50	84.4	3.03	32.6	17.7	82400
152	5.48	47.2	8160	487	13.5	559	273	47.2	2.47	73.9	3.01	32.4	12.4	71700
141	6.01	49.6	7450	448	13.4	514	246	42.7	2.43	66.9	2.98	32.3	9.70	64400
130	6.73	51.7	6710	406	13.2	467	218	37.9	2.39	59.5	2.94	32.2	7.37	56600
118	7.76	54.5	5900	359	13.0	415	187	32.6	2.32	51.3	2.89	32.2	5.30	48300
391	3.19	19.7	20700	1250	13.4	1450	1550	198	3.67	310	4.37	30.8	173	366000
357	3.45	21.6	18700	1140	13.3	1320	1390	179	3.64	279	4.31	30.6	134	324000
326	3.75	23.4	16800	1040	13.2	1190	1240	162	3.60	252	4.26	30.4	103	287000
292	4.12	26.2	14900	930	13.2	1060	1100	144	3.58	223	4.22	30.2	75.2	250000
261	4.59	28.7	13100	829	13.1	943	959	127	3.53	196	4.16	30.0	54.1	215000
235	5.02	32.2	11700	748	13.0	847	855	114	3.51	175	4.13	29.8	40.3	190000
211	5.74	34.5	10300	665	12.9	751	757	100	3.49	155	4.11	29.6	28.4	166000
191	6.35	37.7	9200	600	12.8	675	673	89.5	3.46	138	4.06	29.5	21.0	146000
173	7.04	40.8	8230	541	12.7	607	598	79.8	3.42	123	4.03	29.3	15.6	129000
148	4.44	41.6	6680	436	12.4	500	227	43.3	2.28	68.0	2.77	29.5	14.5	49400
132	5.27	43.9	5770	380	12.2	437	196	37.2	2.25	58.4	2.75	29.3	9.72	42100
124	5.65	46.2	5360	355	12.1	408	181	34.4	2.23	54.0	2.73	29.3	7.99	38600
116	6.17	47.8	4930	329	12.0	378	164	31.3	2.19	49.2	2.70	29.2	6.43	34900
108	6.89	49.6	4470	299	11.9	346	146	27.9	2.15	43.9	2.67	29.0	4.99	30900
99	7.80	51.9	3990	269	11.7	312	128	24.5	2.10	38.6	2.62	29.0	3.77	26800
90	8.52	57.5	3610	245	11.7	283	115	22.1	2.09	34.7	2.60	28.9	2.84	24000
539	2.15	12.1	25600	1570	12.7	1890	2110	277	3.65	437	4.41	29.0	496	443000
368	2.96	17.3	16200	1060	12.2	1240	1310	179	3.48	279	4.15	27.9	170	255000
336	3.19	18.9	14600	972	12.1	1130	1180	162	3.45	252	4.10	27.7	131	226000
307	3.46	20.6	13100	887	12.0	1030	1050	146	3.41	227	4.04	27.5	101	199000
281	3.72	22.5	11900	814	12.0	936	953	133	3.39	206	4.00	27.4	79.5	178000
258	4.03	24.4	10800	745	11.9	852	859	120	3.36	187	3.96	27.2	61.6	159000
235	4.41	26.2	9700	677	11.8	772	769	108	3.33	168	3.92	27.1	47.0	141000
217	4.71	28.7	8910	627	11.8	711	704	100	3.32	154	3.89	26.9	37.6	128000
194	5.24	31.8	7860	559	11.7	631	619	88.1	3.29	136	3.85	26.8	27.1	111000
178	5.92	32.9	7020	505	11.6	570	555	78.8	3.25	122	3.83	26.6	20.1	98400
161	6.49	36.1	6310	458	11.5	515	497	70.9	3.23	109	3.79	26.5	15.1	87300
146	7.16	39.4	5660	414	11.5	464	443	63.5	3.20	97.7	3.76	26.4	11.3	77200

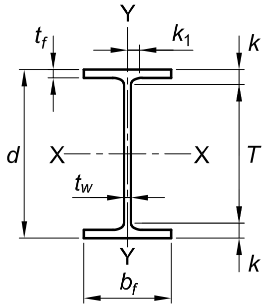


Table 1-1 (continued)
W-Shapes
Dimensions

Shape	Area, A	Depth, d		Web			Flange				Distance				
				Thickness, t_w		$\frac{t_w}{2}$	Width, b_f		Thickness, t_f		k		k_1	T	Workable Gage
											k_{des}	k_{det}			
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W27×129[c]	37.8	27.6	27 5/8	0.610	5/8	5/16	10.0	10	1.10	1 1/8	1.70	2 5/16	1 1/2	23	5 1/2
×114[c]	33.6	27.3	27 1/4	0.570	9/16	5/16	10.1	10 1/8	0.930	15/16	1.53	2 1/8	1 1/2		
×102[c]	30.0	27.1	27 1/8	0.515	1/2	1/4	10.0	10	0.830	13/16	1.43	2 1/16	1 7/16		
×94[c]	27.6	26.9	26 7/8	0.490	1/2	1/4	10.0	10	0.745	3/4	1.34	1 15/16	1 7/16		
×84[c]	24.7	26.7	26 3/4	0.460	7/16	1/4	10.0	10	0.640	5/8	1.24	1 7/8	1 7/16		
W24×370[h]	109	28.0	28	1.52	1 1/2	3/4	13.7	13 5/8	2.72	2 3/4	3.22	4	2	20	5 1/2
×335[h]	98.3	27.5	27 1/2	1.38	1 3/8	11/16	13.5	13 1/2	2.48	2 1/2	2.98	3 3/4	1 7/8		
×306[h]	89.7	27.1	27 1/8	1.26	1 1/4	5/8	13.4	13 3/8	2.28	2 1/4	2.78	3 9/16	1 13/16		
×279[h]	81.9	26.7	26 3/4	1.16	1 3/16	5/8	13.3	13 1/4	2.09	2 1/16	2.59	3 3/8	1 13/16		
×250	73.5	26.3	26 3/8	1.04	1 1/16	9/16	13.2	13 1/8	1.89	1 7/8	2.39	3 1/8	1 3/4		
×229	67.2	26.0	26	0.960	15/16	1/2	13.1	13 1/8	1.73	1 3/4	2.23	3	1 11/16		
×207	60.7	25.7	25 3/4	0.870	7/8	7/16	13.0	13	1.57	1 9/16	2.07	2 7/8	1 5/8		
×192	56.5	25.5	25 1/2	0.810	13/16	7/16	13.0	13	1.46	1 7/16	1.96	2 3/4	1 5/8		
×176	51.7	25.2	25 1/4	0.750	3/4	3/8	12.9	12 7/8	1.34	1 5/16	1.84	2 5/8	1 9/16		
×162	47.8	25.0	25	0.705	11/16	3/8	13.0	13	1.22	1 1/4	1.72	2 1/2	1 9/16		
×146	43.0	24.7	24 3/4	0.650	5/8	5/16	12.9	12 7/8	1.09	1 1/16	1.59	2 3/8	1 9/16		
×131	38.6	24.5	24 1/2	0.605	5/8	5/16	12.9	12 7/8	0.960	15/16	1.46	2 1/4	1 1/2		
×117[c]	34.4	24.3	24 1/4	0.550	9/16	5/16	12.8	12 3/4	0.850	7/8	1.35	2 1/8	1 1/2		
×104[c]	30.7	24.1	24	0.500	1/2	1/4	12.8	12 3/4	0.750	3/4	1.25	2 1/16	1 7/16		
W24×103[c]	30.3	24.5	24 1/2	0.550	9/16	5/16	9.00	9	0.980	1	1.48	2 1/4	1 1/2	20	5 1/2
×94[c]	27.7	24.3	24 1/4	0.515	1/2	1/4	9.07	9 1/8	0.875	7/8	1.38	2 1/8	1 7/16		
×84[c]	24.7	24.1	24 1/8	0.470	1/2	1/4	9.02	9	0.770	3/4	1.27	2 1/16	1 7/16		
×76[c]	22.4	23.9	23 7/8	0.440	7/16	1/4	8.99	9	0.680	11/16	1.18	1 15/16	1 7/16		
×68[c]	20.1	23.7	23 3/4	0.415	7/16	1/4	8.97	9	0.585	9/16	1.09	1 7/8	1 7/16		
W24×62[c]	18.2	23.7	23 3/4	0.430	7/16	1/4	7.04	7	0.590	9/16	1.09	1 1/2	1 1/16	20 3/4	3 1/2[g]
×55[c,v]	16.2	23.6	23 5/8	0.395	3/8	3/16	7.01	7	0.505	1/2	1.01	1 7/16	1	20 3/4	3 1/2[g]

[c] Shape is slender for compression with $F_y = 50$ ksi.
[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.
[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi.

Table 1-1 (continued)
W-Shapes
Properties

W27–W24

Nom-inal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z			J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
129	4.55	39.7	4760	345	11.2	395	184	36.8	2.21	57.6	2.66	26.5	11.1	32500
114	5.41	42.5	4080	299	11.0	343	159	31.5	2.18	49.3	2.65	26.4	7.33	27600
102	6.03	47.1	3620	267	11.0	305	139	27.8	2.15	43.4	2.62	26.3	5.28	24000
94	6.70	49.5	3270	243	10.9	278	124	24.8	2.12	38.8	2.59	26.2	4.03	21300
84	7.78	52.7	2850	213	10.7	244	106	21.2	2.07	33.2	2.54	26.1	2.81	17900
370	2.51	14.2	13400	957	11.1	1130	1160	170	3.27	267	3.92	25.3	201	186000
335	2.73	15.6	11900	864	11.0	1020	1030	152	3.23	238	3.86	25.0	152	161000
306	2.94	17.1	10700	789	10.9	922	919	137	3.20	214	3.81	24.8	117	142000
279	3.18	18.6	9600	718	10.8	835	823	124	3.17	193	3.76	24.6	90.5	125000
250	3.49	20.7	8490	644	10.7	744	724	110	3.14	171	3.71	24.4	66.6	108000
229	3.79	22.5	7650	588	10.7	675	651	99.4	3.11	154	3.67	24.3	51.3	96100
207	4.14	24.8	6820	531	10.6	606	578	88.8	3.08	137	3.62	24.1	38.3	84100
192	4.43	26.6	6260	491	10.5	559	530	81.8	3.07	126	3.60	24.0	30.8	76300
176	4.81	28.7	5680	450	10.5	511	479	74.3	3.04	115	3.57	23.9	23.9	68400
162	5.31	30.6	5170	414	10.4	468	443	68.4	3.05	105	3.57	23.8	18.5	62600
146	5.92	33.2	4580	371	10.3	418	391	60.5	3.01	93.2	3.53	23.6	13.4	54600
131	6.70	35.6	4020	329	10.2	370	340	53.0	2.97	81.5	3.49	23.5	9.50	47100
117	7.53	39.2	3540	291	10.1	327	297	46.5	2.94	71.4	3.46	23.5	6.72	40800
104	8.50	43.1	3100	258	10.1	289	259	40.7	2.91	62.4	3.42	23.4	4.72	35200
103	4.59	39.2	3000	245	10.0	280	119	26.5	1.99	41.5	2.40	23.5	7.07	16600
94	5.18	41.9	2700	222	9.87	254	109	24.0	1.98	37.5	2.40	23.4	5.26	15000
84	5.86	45.9	2370	196	9.79	224	94.4	20.9	1.95	32.6	2.37	23.3	3.70	12800
76	6.61	49.0	2100	176	9.69	200	82.5	18.4	1.92	28.6	2.33	23.2	2.68	11100
68	7.66	52.0	1830	154	9.55	177	70.4	15.7	1.87	24.5	2.30	23.1	1.87	9430
62	5.97	50.1	1550	131	9.23	153	34.5	9.80	1.38	15.7	1.75	23.1	1.71	4620
55	6.94	54.6	1350	114	9.11	134	29.1	8.30	1.34	13.3	1.72	23.1	1.18	3870

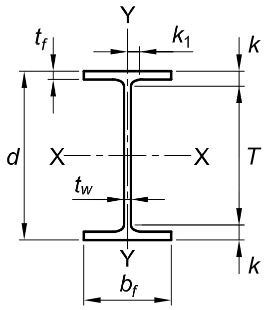


Table 1-1 (continued)
W-Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		Web $\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k k_{des}	Distance k k_{det}	Distance k_1	Distance T	Distance Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W21×275[h]	81.8	24.1	24 1/8	1.22	1 1/4	5/8	12.9	12 7/8	2.19	2 3/16	3.37	3 7/16	1 13/16	17 1/4	5 1/2
×248	73.8	23.7	23 3/4	1.10	1 1/8	9/16	12.8	12 3/4	1.99	2	3.17	3 1/4	1 3/4		
×223	66.5	23.4	23 3/8	1.00	1	1/2	12.7	12 5/8	1.79	1 13/16	2.97	3 1/16	1 11/16		
×201	59.3	23.0	23	0.910	15/16	1/2	12.6	12 5/8	1.63	1 5/8	2.13	2 7/8	1 11/16		
×182	53.6	22.7	22 3/4	0.830	13/16	7/16	12.5	12 1/2	1.48	1 1/2	1.98	2 3/4	1 5/8		
×166	48.8	22.5	22 1/2	0.750	3/4	3/8	12.4	12 3/8	1.36	1 3/8	1.86	2 5/8	1 9/16		
×147	43.2	22.1	22	0.720	3/4	3/8	12.5	12 1/2	1.15	1 1/8	1.65	2 7/16	1 9/16		
×132	38.8	21.8	21 7/8	0.650	5/8	5/16	12.4	12 1/2	1.04	1 1/16	1.54	2 1/4	1 9/16		
×122	35.9	21.7	21 5/8	0.600	5/8	5/16	12.4	12 3/8	0.960	15/16	1.46	2 1/4	1 1/2		
×111	32.6	21.5	21 1/2	0.550	9/16	5/16	12.3	12 3/8	0.875	7/8	1.38	2 1/8	1 1/2		
×101[c]	29.8	21.4	21 3/8	0.500	1/2	1/4	12.3	12 1/4	0.800	13/16	1.30	2 1/16	1 7/16		
W21×93	27.3	21.6	21 5/8	0.580	9/16	5/16	8.42	8 3/8	0.930	15/16	1.43	1 5/8	15/16	18 3/8	5 1/2
×83[c]	24.4	21.4	21 3/8	0.515	1/2	1/4	8.36	8 3/8	0.835	13/16	1.34	1 1/2	7/8		
×73[c]	21.5	21.2	21 1/4	0.455	7/16	1/4	8.30	8 1/4	0.740	3/4	1.24	1 7/16	7/8		
×68[c]	20.0	21.1	21 1/8	0.430	7/16	1/4	8.27	8 1/4	0.685	11/16	1.19	1 3/8	7/8		
×62[c]	18.3	21.0	21	0.400	3/8	3/16	8.24	8 1/4	0.615	5/8	1.12	1 5/16	13/16		
×55[c]	16.2	20.8	20 3/4	0.375	3/8	3/16	8.22	8 1/4	0.522	1/2	1.02	1 3/16	13/16		
×48[c,f]	14.1	20.6	20 5/8	0.350	3/8	3/16	8.14	8 1/8	0.430	7/16	0.930	1 1/8	13/16		
W21×57[c]	16.7	21.1	21	0.405	3/8	3/16	6.56	6 1/2	0.650	5/8	1.15	1 5/16	13/16	18 3/8	3 1/2
×50[c]	14.7	20.8	20 7/8	0.380	3/8	3/16	6.53	6 1/2	0.535	9/16	1.04	1 1/4	13/16		
×44[c]	13.0	20.7	20 5/8	0.350	3/8	3/16	6.50	6 1/2	0.450	7/16	0.950	1 1/8	13/16		

[c] Shape is slender for compression with $F_y = 50$ ksi.
[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.
[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Table 1-1 (continued)
W-Shapes
Properties

W21

Nom-inal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z			J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
275	2.95	14.2	7690	638	9.70	749	787	122	3.10	191	3.68	21.9	107	94400
248	3.22	15.8	6830	576	9.62	671	699	109	3.08	170	3.63	21.7	80.7	82400
223	3.55	17.5	6080	520	9.56	601	614	96.7	3.04	150	3.57	21.6	59.5	71700
201	3.86	20.6	5310	461	9.47	530	542	86.1	3.02	133	3.55	21.4	40.9	62000
182	4.22	22.6	4730	417	9.40	476	483	77.2	3.00	119	3.51	21.2	30.7	54400
166	4.57	25.0	4280	380	9.36	432	435	70.0	2.99	108	3.48	21.1	23.6	48500
147	5.44	26.1	3630	329	9.17	373	376	60.1	2.95	92.6	3.46	21.0	15.4	41100
132	6.01	28.9	3220	295	9.12	333	333	53.5	2.93	82.3	3.43	20.8	11.3	36000
122	6.45	31.3	2960	273	9.09	307	305	49.2	2.92	75.6	3.40	20.7	8.98	32700
111	7.05	34.1	2670	249	9.05	279	274	44.5	2.90	68.2	3.37	20.6	6.83	29200
101	7.68	37.5	2420	227	9.02	253	248	40.3	2.89	61.7	3.35	20.6	5.21	26200
93	4.53	32.3	2070	192	8.70	221	92.9	22.1	1.84	34.7	2.24	20.7	6.03	9940
83	5.00	36.4	1830	171	8.67	196	81.4	19.5	1.83	30.5	2.21	20.6	4.34	8630
73	5.60	41.2	1600	151	8.64	172	70.6	17.0	1.81	26.6	2.19	20.5	3.02	7410
68	6.04	43.6	1480	140	8.60	160	64.7	15.7	1.80	24.4	2.17	20.4	2.45	6760
62	6.70	46.9	1330	127	8.54	144	57.5	14.0	1.77	21.7	2.15	20.4	1.83	5960
55	7.87	50.0	1140	110	8.40	126	48.4	11.8	1.73	18.4	2.11	20.3	1.24	4980
48	9.47	53.6	959	93.0	8.24	107	38.7	9.52	1.66	14.9	2.05	20.2	0.803	3950
57	5.04	46.3	1170	111	8.36	129	30.6	9.35	1.35	14.8	1.68	20.5	1.77	3190
50	6.10	49.4	984	94.5	8.18	110	24.9	7.64	1.30	12.2	1.64	20.3	1.14	2570
44	7.22	53.6	843	81.6	8.06	95.4	20.7	6.37	1.26	10.2	1.60	20.3	0.770	2110

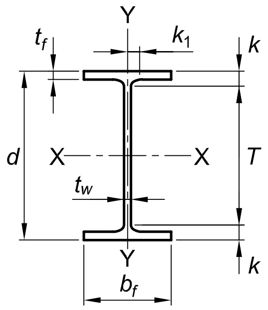


Table 1-1 (continued)
W-Shapes
Dimensions

Shape	Area, A	Depth, d		Web			Flange				Distance				
				Thickness, t_w		$\frac{t_w}{2}$	Width, b_f		Thickness, t_f		k		k_1	T	Workable Gage
											k_{des}	k_{det}			
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W18×311[h]	91.6	22.3	22 3/8	1.52	1 1/2	3/4	12.0	12	2.74	2 3/4	3.24	3 9/16	1 9/16	15 1/8	5 1/2
×283[h]	83.3	21.9	21 7/8	1.40	1 3/8	11/16	11.9	11 7/8	2.50	2 1/2	3.00	3 3/8	1 1/2		
×258[h]	76.0	21.5	21 1/2	1.28	1 1/4	5/8	11.8	11 3/4	2.30	2 5/16	2.70	3 3/16	1 7/16		
×234[h]	68.6	21.1	21	1.16	1 3/16	5/8	11.7	11 5/8	2.11	2 1/8	2.51	3	1 3/8		
×211	62.3	20.7	20 5/8	1.06	1 1/16	9/16	11.6	11 1/2	1.91	1 15/16	2.31	2 13/16	1 3/8		
×192	56.2	20.4	20 3/8	0.960	15/16	1/2	11.5	11 1/2	1.75	1 3/4	2.15	2 5/8	1 5/16		
×175	51.4	20.0	20	0.890	7/8	7/16	11.4	11 3/8	1.59	1 9/16	1.99	2 7/16	1 1/4		
×158	46.3	19.7	19 3/4	0.810	13/16	7/16	11.3	11 1/4	1.44	1 7/16	1.84	2 3/8	1 1/4		
×143	42.0	19.5	19 1/2	0.730	3/4	3/8	11.2	11 1/4	1.32	1 5/16	1.72	2 3/16	1 3/16		
×130	38.3	19.3	19 1/4	0.670	11/16	3/8	11.2	11 1/8	1.20	1 3/16	1.60	2 1/16	1 3/16		
×119	35.1	19.0	19	0.655	5/8	5/16	11.3	11 1/4	1.06	1 1/16	1.46	1 15/16	1 3/16		
×106	31.1	18.7	18 3/4	0.590	9/16	5/16	11.2	11 1/4	0.940	15/16	1.34	1 13/16	1 1/8		
×97	28.5	18.6	18 5/8	0.535	9/16	5/16	11.1	11 1/8	0.870	7/8	1.27	1 3/4	1 1/8		
×86	25.3	18.4	18 3/8	0.480	1/2	1/4	11.1	11 1/8	0.770	3/4	1.17	1 5/8	1 1/16		
×76[c]	22.3	18.2	18 1/4	0.425	7/16	1/4	11.0	11	0.680	11/16	1.08	1 9/16	1 1/16		
W18×71	20.9	18.5	18 1/2	0.495	1/2	1/4	7.64	7 5/8	0.810	13/16	1.21	1 1/2	7/8	15 1/2	3 1/2[g]
×65	19.1	18.4	18 3/8	0.450	7/16	1/4	7.59	7 5/8	0.750	3/4	1.15	1 7/16	7/8		
×60[c]	17.6	18.2	18 1/4	0.415	7/16	1/4	7.56	7 1/2	0.695	11/16	1.10	1 3/8	13/16		
×55[c]	16.2	18.1	18 1/8	0.390	3/8	3/16	7.53	7 1/2	0.630	5/8	1.03	1 5/16	13/16		
×50[c]	14.7	18.0	18	0.355	3/8	3/16	7.50	7 1/2	0.570	9/16	0.972	1 1/4	13/16		
W18×46[c]	13.5	18.1	18	0.360	3/8	3/16	6.06	6	0.605	5/8	1.01	1 1/4	13/16	15 1/2	3 1/2[g]
×40[c]	11.8	17.9	17 7/8	0.315	5/16	3/16	6.02	6	0.525	1/2	0.927	1 3/16	13/16		
×35[c]	10.3	17.7	17 3/4	0.300	5/16	3/16	6.00	6	0.425	7/16	0.827	1 1/8	3/4		
W16×100	29.4	17.0	17	0.585	9/16	5/16	10.4	10 3/8	0.985	1	1.39	1 7/8	1 1/8	13 1/4	5 1/2
×89	26.2	16.8	16 3/4	0.525	1/2	1/4	10.4	10 3/8	0.875	7/8	1.28	1 3/4	1 1/16		
×77	22.6	16.5	16 1/2	0.455	7/16	1/4	10.3	10 1/4	0.760	3/4	1.16	1 5/8	1 1/16		
×67[c]	19.6	16.3	16 3/8	0.395	3/8	3/16	10.2	10 1/4	0.665	11/16	1.07	1 9/16	1		
W16×57	16.8	16.4	16 3/8	0.430	7/16	1/4	7.12	7 1/8	0.715	11/16	1.12	1 3/8	7/8	13 5/8	3 1/2[g]
×50[c]	14.7	16.3	16 1/4	0.380	3/8	3/16	7.07	7 1/8	0.630	5/8	1.03	1 5/16	13/16		
×45[c]	13.3	16.1	16 1/8	0.345	3/8	3/16	7.04	7	0.565	9/16	0.967	1 1/4	13/16		
×40[c]	11.8	16.0	16	0.305	5/16	3/16	7.00	7	0.505	1/2	0.907	1 3/16	13/16		
×36[c]	10.6	15.9	15 7/8	0.295	5/16	3/16	6.99	7	0.430	7/16	0.832	1 1/8	3/4		
W16×31[c]	9.13	15.9	15 7/8	0.275	1/4	1/8	5.53	5 1/2	0.440	7/16	0.842	1 1/8	3/4	13 5/8	3 1/2
×26[c,v]	7.68	15.7	15 3/4	0.250	1/4	1/8	5.50	5 1/2	0.345	3/8	0.747	1 1/16	3/4	13 5/8	3 1/2

[c] Shape is slender for compression with $F_y = 50$ ksi.
[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.
[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi.

Table 1-1 (continued)
W-Shapes
Properties

W18–W16

Nominal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z			J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
311	2.19	10.4	6970	624	8.72	754	795	132	2.95	207	3.53	19.6	176	76200
283	2.38	11.3	6170	565	8.61	676	704	118	2.91	185	3.47	19.4	134	65900
258	2.56	12.5	5510	514	8.53	611	628	107	2.88	166	3.42	19.2	103	57600
234	2.76	13.8	4900	466	8.44	549	558	95.8	2.85	149	3.37	19.0	78.7	50100
211	3.02	15.1	4330	419	8.35	490	493	85.3	2.82	132	3.32	18.8	58.6	43400
192	3.27	16.7	3870	380	8.28	442	440	76.8	2.79	119	3.28	18.7	44.7	38000
175	3.58	18.0	3450	344	8.20	398	391	68.8	2.76	106	3.24	18.4	33.8	33300
158	3.92	19.8	3060	310	8.12	356	347	61.4	2.74	94.8	3.20	18.3	25.2	29000
143	4.25	22.0	2750	282	8.09	322	311	55.5	2.72	85.4	3.17	18.2	19.2	25700
130	4.65	23.9	2460	256	8.03	290	278	49.9	2.70	76.7	3.13	18.1	14.5	22700
119	5.31	24.5	2190	231	7.90	262	253	44.9	2.69	69.1	3.13	17.9	10.6	20300
106	5.96	27.2	1910	204	7.84	230	220	39.4	2.66	60.5	3.10	17.8	7.48	17400
97	6.41	30.0	1750	188	7.82	211	201	36.1	2.65	55.3	3.08	17.7	5.86	15800
86	7.20	33.4	1530	166	7.77	186	175	31.6	2.63	48.4	3.05	17.6	4.10	13600
76	8.11	37.8	1330	146	7.73	163	152	27.6	2.61	42.2	3.02	17.5	2.83	11700
71	4.71	32.4	1170	127	7.50	146	60.3	15.8	1.70	24.7	2.05	17.7	3.49	4700
65	5.06	35.7	1070	117	7.49	133	54.8	14.4	1.69	22.5	2.03	17.7	2.73	4240
60	5.44	38.7	984	108	7.47	123	50.1	13.3	1.68	20.6	2.02	17.5	2.17	3850
55	5.98	41.1	890	98.3	7.41	112	44.9	11.9	1.67	18.5	2.00	17.5	1.66	3430
50	6.57	45.2	800	88.9	7.38	101	40.1	10.7	1.65	16.6	1.98	17.4	1.24	3040
46	5.01	44.6	712	78.8	7.25	90.7	22.5	7.43	1.29	11.7	1.58	17.5	1.22	1720
40	5.73	50.9	612	68.4	7.21	78.4	19.1	6.35	1.27	10.0	1.56	17.4	0.810	1440
35	7.06	53.5	510	57.6	7.04	66.5	15.3	5.12	1.22	8.06	1.51	17.3	0.506	1140
100	5.29	24.3	1490	175	7.10	198	186	35.7	2.51	54.9	2.92	16.0	7.73	11900
89	5.92	27.0	1300	155	7.05	175	163	31.4	2.49	48.1	2.88	15.9	5.45	10200
77	6.77	31.2	1110	134	7.00	150	138	26.9	2.47	41.1	2.85	15.7	3.57	8590
67	7.70	35.9	954	117	6.96	130	119	23.2	2.46	35.5	2.82	15.6	2.39	7300
57	4.98	33.0	758	92.2	6.72	105	43.1	12.1	1.60	18.9	1.92	15.7	2.22	2660
50	5.61	37.4	659	81.0	6.68	92.0	37.2	10.5	1.59	16.3	1.89	15.7	1.52	2270
45	6.23	41.1	586	72.7	6.65	82.3	32.8	9.34	1.57	14.5	1.87	15.5	1.11	1990
40	6.93	46.5	518	64.7	6.63	73.0	28.9	8.25	1.57	12.7	1.86	15.5	0.794	1730
36	8.12	48.1	448	56.5	6.51	64.0	24.5	7.00	1.52	10.8	1.83	15.5	0.545	1460
31	6.28	51.6	375	47.2	6.41	54.0	12.4	4.49	1.17	7.03	1.42	15.5	0.461	739
26	7.97	56.8	301	38.4	6.26	44.2	9.59	3.49	1.12	5.48	1.38	15.4	0.262	565

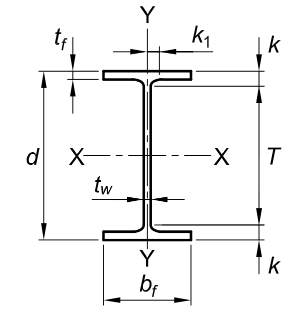


Table 1-1 (continued)
W-Shapes
Dimensions

Shape	Area, A	Depth, d		Web			Flange				Distance				
				Thickness, t_w		$\frac{t_w}{2}$	Width, b_f		Thickness, t_f		k		k_1	T	Workable Gage
											k_{des}	k_{det}			
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W14×873[h]	257	23.6	23 5/8	3.94	3 15/16	2	18.8	18 3/4	5.51	5 1/2	6.10	6 3/16	2 9/16	11 1/4	3-8 1/2-3[g]
×808[h]	238	22.8	22 3/4	3.74	3 3/4	1 7/8	18.6	18 5/8	5.12	5 1/8	5.71	5 3/4	2 1/2	11 1/4	3-8 1/2-3[g]
×730[h]	215	22.4	22 3/8	3.07	3 1/16	1 9/16	17.9	17 7/8	4.91	4 15/16	5.51	6 3/16	2 3/4	10	3-7 1/2-3[g]
×665[h]	196	21.6	21 5/8	2.83	2 13/16	1 7/16	17.7	17 5/8	4.52	4 1/2	5.12	5 13/16	2 5/8		3-7 1/2-3[g]
×605[h]	178	20.9	20 7/8	2.60	2 5/8	1 5/16	17.4	17 3/8	4.16	4 3/16	4.76	5 7/16	2 1/2		3-7 1/2-3
×550[h]	162	20.2	20 1/4	2.38	2 3/8	1 3/16	17.2	17 1/4	3.82	3 13/16	4.42	5 1/8	2 3/8		
×500[h]	147	19.6	19 5/8	2.19	2 3/16	1 1/8	17.0	17	3.50	3 1/2	4.10	4 13/16	2 5/16		
×455[h]	134	19.0	19	2.02	2	1	16.8	16 7/8	3.21	3 3/16	3.81	4 1/2	2 1/4		
×426[h]	125	18.7	18 5/8	1.88	1 7/8	15/16	16.7	16 3/4	3.04	3 1/16	3.63	4 5/16	2 1/8		
×398[h]	117	18.3	18 1/4	1.77	1 3/4	7/8	16.6	16 5/8	2.85	2 7/8	3.44	4 1/8	2 1/8		
×370[h]	109	17.9	17 7/8	1.66	1 11/16	13/16	16.5	16 1/2	2.66	2 11/16	3.26	3 15/16	2 1/16		
×342[h]	101	17.5	17 1/2	1.54	1 9/16	13/16	16.4	16 3/8	2.47	2 1/2	3.07	3 3/4	2		
×311[h]	91.4	17.1	17 1/8	1.41	1 7/16	3/4	16.2	16 1/4	2.26	2 1/4	2.86	3 9/16	1 15/16		
×283[h]	83.3	16.7	16 3/4	1.29	1 5/16	11/16	16.1	16 1/8	2.07	2 1/16	2.67	3 3/8	1 7/8		
×257	75.6	16.4	16 3/8	1.18	1 3/16	5/8	16.0	16	1.89	1 7/8	2.49	3 3/16	1 13/16		
×233	68.5	16.0	16	1.07	1 1/16	9/16	15.9	15 7/8	1.72	1 3/4	2.32	3	1 3/4		
×211	62.0	15.7	15 3/4	0.980	1	1/2	15.8	15 3/4	1.56	1 9/16	2.16	2 7/8	1 11/16		
×193	56.8	15.5	15 1/2	0.890	7/8	7/16	15.7	15 3/4	1.44	1 7/16	2.04	2 3/4	1 11/16		
×176	51.8	15.2	15 1/4	0.830	13/16	7/16	15.7	15 5/8	1.31	1 5/16	1.91	2 5/8	1 5/8		
×159	46.7	15.0	15	0.745	3/4	3/8	15.6	15 5/8	1.19	1 3/16	1.79	2 1/2	1 9/16		
×145	42.7	14.8	14 3/4	0.680	11/16	3/8	15.5	15 1/2	1.09	1 1/16	1.69	2 3/8	1 9/16		
W14×132	38.8	14.7	14 5/8	0.645	5/8	5/16	14.7	14 3/4	1.03	1	1.63	2 5/16	1 9/16	10	5 1/2
×120	35.3	14.5	14 1/2	0.590	9/16	5/16	14.7	14 5/8	0.940	15/16	1.54	2 1/4	1 1/2		
×109	32.0	14.3	14 3/8	0.525	1/2	1/4	14.6	14 5/8	0.860	7/8	1.46	2 3/16	1 1/2		
×99[f]	29.1	14.2	14 1/8	0.485	1/2	1/4	14.6	14 5/8	0.780	3/4	1.38	2 1/16	1 7/16		
×90[f]	26.5	14.0	14	0.440	7/16	1/4	14.5	14 1/2	0.710	11/16	1.31	2	1 7/16		
W14×82	24.0	14.3	14 1/4	0.510	1/2	1/4	10.1	10 1/8	0.855	7/8	1.45	1 11/16	1 1/16	10 7/8	5 1/2
×74	21.8	14.2	14 1/8	0.450	7/16	1/4	10.1	10 1/8	0.785	13/16	1.38	1 5/8	1 1/16		
×68	20.0	14.0	14	0.415	7/16	1/4	10.0	10	0.720	3/4	1.31	1 9/16	1 1/16		
×61	17.9	13.9	13 7/8	0.375	3/8	3/16	10.0	10	0.645	5/8	1.24	1 1/2	1		
W14×53	15.6	13.9	13 7/8	0.370	3/8	3/16	8.06	8	0.660	11/16	1.25	1 1/2	1	10 7/8	5 1/2
×48	14.1	13.8	13 3/4	0.340	5/16	3/16	8.03	8	0.595	5/8	1.19	1 7/16	1		
×43[c]	12.6	13.7	13 5/8	0.305	5/16	3/16	8.00	8	0.530	1/2	1.12	1 3/8	1		

[c] Shape is slender for compression with $F_y = 50$ ksi.
[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.
[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.
[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Table 1-1 (continued)
W-Shapes
Properties

W14

Nom-inal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	*I*	*S*	*r*	*Z*	*I*	*S*	*r*	*Z*			*J*	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
873	1.71	2.89	18100	1530	8.39	2030	6170	656	4.90	1020	6.04	18.1	2270	505000
808	1.82	3.04	15900	1390	8.17	1830	5550	597	4.83	930	5.94	17.7	1840	434000
730	1.82	3.71	14300	1280	8.17	1660	4720	527	4.69	816	5.68	17.5	1450	362000
665	1.95	4.03	12400	1150	7.98	1480	4170	472	4.62	730	5.57	17.1	1120	305000
605	2.09	4.39	10800	1040	7.80	1320	3680	423	4.55	652	5.44	16.7	869	258000
550	2.25	4.79	9430	931	7.63	1180	3250	378	4.49	583	5.35	16.4	669	219000
500	2.43	5.21	8210	838	7.48	1050	2880	339	4.43	522	5.26	16.1	514	187000
455	2.62	5.66	7190	756	7.33	936	2560	304	4.38	468	5.17	15.8	395	160000
426	2.75	6.08	6600	706	7.26	869	2360	283	4.34	434	5.11	15.7	331	144000
398	2.92	6.44	6000	656	7.16	801	2170	262	4.31	402	5.05	15.5	273	129000
370	3.10	6.89	5440	607	7.07	736	1990	241	4.27	370	5.00	15.2	222	116000
342	3.31	7.41	4900	558	6.98	672	1810	221	4.24	338	4.95	15.0	178	103000
311	3.59	8.09	4330	506	6.88	603	1610	199	4.20	304	4.87	14.8	136	89100
283	3.89	8.84	3840	459	6.79	542	1440	179	4.17	274	4.80	14.6	104	77700
257	4.23	9.71	3400	415	6.71	487	1290	161	4.13	246	4.75	14.5	79.1	67800
233	4.62	10.7	3010	375	6.63	436	1150	145	4.10	221	4.69	14.3	59.5	59000
211	5.06	11.6	2660	338	6.55	390	1030	130	4.07	198	4.64	14.1	44.6	51500
193	5.45	12.8	2400	310	6.50	355	931	119	4.05	180	4.59	14.1	34.8	45900
176	5.97	13.7	2140	281	6.43	320	838	107	4.02	163	4.55	13.9	26.5	40500
159	6.54	15.3	1900	254	6.38	287	748	96.2	4.00	146	4.51	13.8	19.7	35600
145	7.11	16.8	1710	232	6.33	260	677	87.3	3.98	133	4.47	13.7	15.2	31700
132	7.15	17.7	1530	209	6.28	234	548	74.5	3.76	113	4.23	13.7	12.3	25500
120	7.80	19.3	1380	190	6.24	212	495	67.5	3.74	102	4.20	13.6	9.37	22700
109	8.49	21.7	1240	173	6.22	192	447	61.2	3.73	92.7	4.17	13.4	7.12	20200
99	9.34	23.5	1110	157	6.17	173	402	55.2	3.71	83.6	4.14	13.4	5.37	18000
90	10.2	25.9	999	143	6.14	157	362	49.9	3.70	75.6	4.10	13.3	4.06	16000
82	5.92	22.4	881	123	6.05	139	148	29.3	2.48	44.8	2.85	13.4	5.07	6710
74	6.41	25.4	795	112	6.04	126	134	26.6	2.48	40.5	2.83	13.4	3.87	5990
68	6.97	27.5	722	103	6.01	115	121	24.2	2.46	36.9	2.80	13.3	3.01	5380
61	7.75	30.4	640	92.1	5.98	102	107	21.5	2.45	32.8	2.78	13.3	2.19	4710
53	6.11	30.9	541	77.8	5.89	87.1	57.7	14.3	1.92	22.0	2.22	13.2	1.94	2540
48	6.75	33.6	484	70.2	5.85	78.4	51.4	12.8	1.91	19.6	2.20	13.2	1.45	2240
43	7.54	37.4	428	62.6	5.82	69.6	45.2	11.3	1.89	17.3	2.18	13.2	1.05	1950

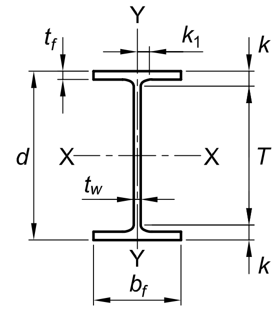


Table 1-1 (continued)
W-Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		Web $\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k k_{des}	Distance k k_{det}	Distance k_1	Distance T	Distance Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W14×38[c]	11.2	14.1	14 1/8	0.310	5/16	3/16	6.77	6 3/4	0.515	1/2	0.915	1 1/4	13/16	11 5/8	3 1/2[g]
×34[c]	10.0	14.0	14	0.285	5/16	3/16	6.75	6 3/4	0.455	7/16	0.855	1 3/16	3/4		3 1/2
×30[c]	8.85	13.8	13 7/8	0.270	1/4	1/8	6.73	6 3/4	0.385	3/8	0.785	1 1/8	3/4		3 1/2
W14×26[c]	7.69	13.9	13 7/8	0.255	1/4	1/8	5.03	5	0.420	7/16	0.820	1 1/8	3/4	11 5/8	2 3/4[g]
×22[c]	6.49	13.7	13 3/4	0.230	1/4	1/8	5.00	5	0.335	5/16	0.735	1 1/16	3/4	11 5/8	2 3/4[g]
W12×336[h]	98.9	16.8	16 7/8	1.78	1 3/4	7/8	13.4	13 3/8	2.96	2 15/16	3.55	3 7/8	1 11/16	9 1/8	5 1/2
×305[h]	89.5	16.3	16 3/8	1.63	1 5/8	13/16	13.2	13 1/4	2.71	2 11/16	3.30	3 5/8	1 5/8		
×279[h]	81.9	15.9	15 7/8	1.53	1 1/2	3/4	13.1	13 1/8	2.47	2 1/2	3.07	3 3/8	1 5/8		
×252[h]	74.1	15.4	15 3/8	1.40	1 3/8	11/16	13.0	13	2.25	2 1/4	2.85	3 1/8	1 1/2		
×230[h]	67.7	15.1	15	1.29	1 5/16	11/16	12.9	12 7/8	2.07	2 1/16	2.67	2 15/16	1 1/2		
×210	61.8	14.7	14 3/4	1.18	1 3/16	5/8	12.8	12 3/4	1.90	1 7/8	2.50	2 13/16	1 7/16		
×190	56.0	14.4	14 3/8	1.06	1 1/16	9/16	12.7	12 5/8	1.74	1 3/4	2.33	2 5/8	1 3/8		
×170	50.0	14.0	14	0.960	15/16	1/2	12.6	12 5/8	1.56	1 9/16	2.16	2 7/16	1 5/16		
×152	44.7	13.7	13 3/4	0.870	7/8	7/16	12.5	12 1/2	1.40	1 3/8	2.00	2 5/16	1 1/4		
×136	39.9	13.4	13 3/8	0.790	13/16	7/16	12.4	12 3/8	1.25	1 1/4	1.85	2 1/8	1 1/4		
×120	35.2	13.1	13 1/8	0.710	11/16	3/8	12.3	12 3/8	1.11	1 1/8	1.70	2	1 3/16		
×106	31.2	12.9	12 7/8	0.610	5/8	5/16	12.2	12 1/4	0.990	1	1.59	1 7/8	1 1/8		
×96	28.2	12.7	12 3/4	0.550	9/16	5/16	12.2	12 1/8	0.900	7/8	1.50	1 13/16	1 1/8		
×87	25.6	12.5	12 1/2	0.515	1/2	1/4	12.1	12 1/8	0.810	13/16	1.41	1 11/16	1 1/16		
×79	23.2	12.4	12 3/8	0.470	1/2	1/4	12.1	12 1/8	0.735	3/4	1.33	1 5/8	1 1/16		
×72	21.1	12.3	12 1/4	0.430	7/16	1/4	12.0	12	0.670	11/16	1.27	1 9/16	1 1/16		
×65[f]	19.1	12.1	12 1/8	0.390	3/8	3/16	12.0	12	0.605	5/8	1.20	1 1/2	1		
W12×58	17.0	12.2	12 1/4	0.360	3/8	3/16	10.0	10	0.640	5/8	1.24	1 1/2	15/16	9 1/4	5 1/2
×53	15.6	12.1	12	0.345	3/8	3/16	10.0	10	0.575	9/16	1.18	1 3/8	15/16	9 1/4	5 1/2
W12×50	14.6	12.2	12 1/4	0.370	3/8	3/16	8.08	8 1/8	0.640	5/8	1.14	1 1/2	15/16	9 1/4	5 1/2
×45	13.1	12.1	12	0.335	5/16	3/16	8.05	8	0.575	9/16	1.08	1 3/8	15/16		
×40	11.7	11.9	12	0.295	5/16	3/16	8.01	8	0.515	1/2	1.02	1 3/8	7/8		
W12×35[c]	10.3	12.5	12 1/2	0.300	5/16	3/16	6.56	6 1/2	0.520	1/2	0.820	1 3/16	3/4	10 1/8	3 1/2
×30[c]	8.79	12.3	12 3/8	0.260	1/4	1/8	6.52	6 1/2	0.440	7/16	0.740	1 1/8	3/4		
×26[c]	7.65	12.2	12 1/4	0.230	1/4	1/8	6.49	6 1/2	0.380	3/8	0.680	1 1/16	3/4		
W12×22[c]	6.48	12.3	12 1/4	0.260	1/4	1/8	4.03	4	0.425	7/16	0.725	15/16	5/8	10 3/8	2 1/4[g]
×19[c]	5.57	12.2	12 1/8	0.235	1/4	1/8	4.01	4	0.350	3/8	0.650	7/8	9/16		
×16[c]	4.71	12.0	12	0.220	1/4	1/8	3.99	4	0.265	1/4	0.565	13/16	9/16		
×14[c,v]	4.16	11.9	11 7/8	0.200	3/16	1/8	3.97	4	0.225	1/4	0.525	3/4	9/16		

[c] Shape is slender for compression with $F_y = 50$ ksi.
[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.
[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.
[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi.

Table 1-1 (continued)
W-Shapes
Properties

W14–W12

Nom-inal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z			J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
38	6.57	39.6	385	54.6	5.87	61.5	26.7	7.88	1.55	12.1	1.82	13.6	0.798	1230
34	7.41	43.1	340	48.6	5.83	54.6	23.3	6.91	1.53	10.6	1.80	13.5	0.569	1070
30	8.74	45.4	291	42.0	5.73	47.3	19.6	5.82	1.49	8.99	1.77	13.4	0.380	887
26	5.98	48.1	245	35.3	5.65	40.2	8.91	3.55	1.08	5.54	1.30	13.5	0.358	405
22	7.46	53.3	199	29.0	5.54	33.2	7.00	2.80	1.04	4.39	1.27	13.4	0.208	314
336	2.26	5.47	4060	483	6.41	603	1190	177	3.47	274	4.13	13.8	243	57000
305	2.45	5.98	3550	435	6.29	537	1050	159	3.42	244	4.05	13.6	185	48600
279	2.66	6.35	3110	393	6.16	481	937	143	3.38	220	4.00	13.4	143	42000
252	2.89	6.96	2720	353	6.06	428	828	127	3.34	196	3.93	13.2	108	35800
230	3.11	7.56	2420	321	5.97	386	742	115	3.31	177	3.87	13.0	83.8	31200
210	3.37	8.23	2140	292	5.89	348	664	104	3.28	159	3.81	12.8	64.7	27200
190	3.65	9.16	1890	263	5.82	311	589	93.0	3.25	143	3.77	12.7	48.8	23600
170	4.03	10.1	1650	235	5.74	275	517	82.3	3.22	126	3.70	12.4	35.6	20100
152	4.46	11.2	1430	209	5.66	243	454	72.8	3.19	111	3.66	12.3	25.8	17200
136	4.96	12.3	1240	186	5.58	214	398	64.2	3.16	98.0	3.61	12.2	18.5	14700
120	5.57	13.7	1070	163	5.51	186	345	56.0	3.13	85.4	3.56	12.0	12.9	12400
106	6.17	15.9	933	145	5.47	164	301	49.3	3.11	75.1	3.52	11.9	9.13	10700
96	6.76	17.7	833	131	5.44	147	270	44.4	3.09	67.5	3.49	11.8	6.85	9410
87	7.48	18.9	740	118	5.38	132	241	39.7	3.07	60.4	3.46	11.7	5.10	8270
79	8.22	20.7	662	107	5.34	119	216	35.8	3.05	54.3	3.43	11.7	3.84	7330
72	8.99	22.6	597	97.4	5.31	108	195	32.4	3.04	49.2	3.41	11.6	2.93	6540
65	9.92	24.9	533	87.9	5.28	96.8	174	29.1	3.02	44.1	3.38	11.5	2.18	5780
58	7.82	27.0	475	78.0	5.28	86.4	107	21.4	2.51	32.5	2.81	11.6	2.10	3570
53	8.69	28.1	425	70.6	5.23	77.9	95.8	19.2	2.48	29.1	2.79	11.5	1.58	3160
50	6.31	26.8	391	64.2	5.18	71.9	56.3	13.9	1.96	21.3	2.25	11.6	1.71	1880
45	7.00	29.6	348	57.7	5.15	64.2	50.0	12.4	1.95	19.0	2.23	11.5	1.26	1650
40	7.77	33.6	307	51.5	5.13	57.0	44.1	11.0	1.94	16.8	2.21	11.4	0.906	1440
35	6.31	36.2	285	45.6	5.25	51.2	24.5	7.47	1.54	11.5	1.79	12.0	0.741	879
30	7.41	41.8	238	38.6	5.21	43.1	20.3	6.24	1.52	9.56	1.77	11.9	0.457	720
26	8.54	47.2	204	33.4	5.17	37.2	17.3	5.34	1.51	8.17	1.75	11.8	0.300	607
22	4.74	41.8	156	25.4	4.91	29.3	4.66	2.31	0.848	3.66	1.04	11.9	0.293	164
19	5.72	46.2	130	21.3	4.82	24.7	3.76	1.88	0.822	2.98	1.02	11.9	0.180	131
16	7.53	49.4	103	17.1	4.67	20.1	2.82	1.41	0.773	2.26	0.983	11.7	0.103	96.9
14	8.82	54.3	88.6	14.9	4.62	17.4	2.36	1.19	0.753	1.90	0.961	11.7	0.0704	80.4

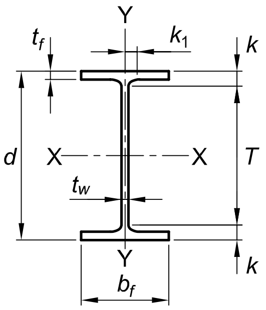


Table 1-1 (continued)
W-Shapes
Dimensions

Shape	Area, A	Depth, d		Web: Thickness, t_w		Web: $\frac{t_w}{2}$	Flange: Width, b_f		Flange: Thickness, t_f		Distance: k k_{des}	Distance: k k_{det}	Distance: k_1	Distance: T	Distance: Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W10×112	32.9	11.4	11 3/8	0.755	3/4	3/8	10.4	10 3/8	1.25	1 1/4	1.75	1 15/16	1	7 1/2	5 1/2
×100	29.3	11.1	11 1/8	0.680	11/16	3/8	10.3	10 3/8	1.12	1 1/8	1.62	1 13/16	1		
×88	26.0	10.8	10 7/8	0.605	5/8	5/16	10.3	10 1/4	0.990	1	1.49	1 11/16	15/16		
×77	22.7	10.6	10 5/8	0.530	1/2	1/4	10.2	10 1/4	0.870	7/8	1.37	1 9/16	7/8		
×68	19.9	10.4	10 3/8	0.470	1/2	1/4	10.1	10 1/8	0.770	3/4	1.27	1 7/16	7/8		
×60	17.7	10.2	10 1/4	0.420	7/16	1/4	10.1	10 1/8	0.680	11/16	1.18	1 3/8	13/16		
×54	15.8	10.1	10 1/8	0.370	3/8	3/16	10.0	10	0.615	5/8	1.12	1 5/16	13/16		
×49	14.4	10.0	10	0.340	5/16	3/16	10.0	10	0.560	9/16	1.06	1 1/4	13/16		
W10×45	13.3	10.1	10 1/8	0.350	3/8	3/16	8.02	8	0.620	5/8	1.12	1 5/16	13/16	7 1/2	5 1/2
×39	11.5	9.92	9 7/8	0.315	5/16	3/16	7.99	8	0.530	1/2	1.03	1 3/16	13/16		
×33	9.71	9.73	9 3/4	0.290	5/16	3/16	7.96	8	0.435	7/16	0.935	1 1/8	3/4		
W10×30	8.84	10.5	10 1/2	0.300	5/16	3/16	5.81	5 3/4	0.510	1/2	0.810	1 1/8	11/16	8 1/4	2 3/4[g]
×26	7.61	10.3	10 3/8	0.260	1/4	1/8	5.77	5 3/4	0.440	7/16	0.740	1 1/16	11/16		
×22[c]	6.49	10.2	10 1/8	0.240	1/4	1/8	5.75	5 3/4	0.360	3/8	0.660	15/16	5/8		
W10×19	5.62	10.2	10 1/4	0.250	1/4	1/8	4.02	4	0.395	3/8	0.695	15/16	5/8	8 3/8	2 1/4[g]
×17[c]	4.99	10.1	10 1/8	0.240	1/4	1/8	4.01	4	0.330	5/16	0.630	7/8	9/16		
×15[c]	4.41	9.99	10	0.230	1/4	1/8	4.00	4	0.270	1/4	0.570	13/16	9/16		
×12[c,f]	3.54	9.87	9 7/8	0.190	3/16	1/8	3.96	4	0.210	3/16	0.510	3/4	9/16		
W8×67	19.7	9.00	9	0.570	9/16	5/16	8.28	8 1/4	0.935	15/16	1.33	1 5/8	15/16	5 3/4	5 1/2
×58	17.1	8.75	8 3/4	0.510	1/2	1/4	8.22	8 1/4	0.810	13/16	1.20	1 1/2	7/8		
×48	14.1	8.50	8 1/2	0.400	3/8	3/16	8.11	8 1/8	0.685	11/16	1.08	1 3/8	13/16		
×40	11.7	8.25	8 1/4	0.360	3/8	3/16	8.07	8 1/8	0.560	9/16	0.954	1 1/4	13/16		
×35	10.3	8.12	8 1/8	0.310	5/16	3/16	8.02	8	0.495	1/2	0.889	1 3/16	13/16		
×31[f]	9.13	8.00	8	0.285	5/16	3/16	8.00	8	0.435	7/16	0.829	1 1/8	3/4		
W8×28	8.25	8.06	8	0.285	5/16	3/16	6.54	6 1/2	0.465	7/16	0.859	15/16	5/8	6 1/8	4
×24	7.08	7.93	7 7/8	0.245	1/4	1/8	6.50	6 1/2	0.400	3/8	0.794	7/8	9/16	6 1/8	4
W8×21	6.16	8.28	8 1/4	0.250	1/4	1/8	5.27	5 1/4	0.400	3/8	0.700	7/8	9/16	6 1/2	2 3/4[g]
×18	5.26	8.14	8 1/8	0.230	1/4	1/8	5.25	5 1/4	0.330	5/16	0.630	13/16	9/16	6 1/2	2 3/4[g]
W8×15	4.44	8.11	8 1/8	0.245	1/4	1/8	4.02	4	0.315	5/16	0.615	13/16	9/16	6 1/2	2 1/4[g]
×13	3.84	7.99	8	0.230	1/4	1/8	4.00	4	0.255	1/4	0.555	3/4	9/16		
×10[c,f]	2.96	7.89	7 7/8	0.170	3/16	1/8	3.94	4	0.205	3/16	0.505	11/16	1/2		

[c] Shape is slender for compression with $F_y = 50$ ksi.
[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.
[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.

Table 1-1 (continued)
W-Shapes
Properties

W10–W8

Nom-inal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z			J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
112	4.17	10.4	716	126	4.66	147	236	45.3	2.68	69.2	3.08	10.2	15.1	6020
100	4.62	11.6	623	112	4.60	130	207	40.0	2.65	61.0	3.04	10.0	10.9	5150
88	5.18	13.0	534	98.5	4.54	113	179	34.8	2.63	53.1	2.99	9.81	7.53	4330
77	5.86	14.8	455	85.9	4.49	97.6	154	30.1	2.60	45.9	2.95	9.73	5.11	3630
68	6.58	16.7	394	75.7	4.44	85.3	134	26.4	2.59	40.1	2.92	9.63	3.56	3100
60	7.41	18.7	341	66.7	4.39	74.6	116	23.0	2.57	35.0	2.88	9.52	2.48	2640
54	8.15	21.2	303	60.0	4.37	66.6	103	20.6	2.56	31.3	2.85	9.49	1.82	2320
49	8.93	23.1	272	54.6	4.35	60.4	93.4	18.7	2.54	28.3	2.84	9.44	1.39	2070
45	6.47	22.5	248	49.1	4.32	54.9	53.4	13.3	2.01	20.3	2.27	9.48	1.51	1200
39	7.53	25.0	209	42.1	4.27	46.8	45.0	11.3	1.98	17.2	2.24	9.39	0.976	992
33	9.15	27.1	171	35.0	4.19	38.8	36.6	9.20	1.94	14.0	2.20	9.30	0.583	791
30	5.70	29.5	170	32.4	4.38	36.6	16.7	5.75	1.37	8.84	1.60	9.99	0.622	414
26	6.56	34.0	144	27.9	4.35	31.3	14.1	4.89	1.36	7.50	1.58	9.86	0.402	345
22	7.99	36.9	118	23.2	4.27	26.0	11.4	3.97	1.33	6.10	1.55	9.84	0.239	275
19	5.09	35.4	96.3	18.8	4.14	21.6	4.29	2.14	0.874	3.35	1.06	9.81	0.233	104
17	6.08	36.9	81.9	16.2	4.05	18.7	3.56	1.78	0.845	2.80	1.04	9.77	0.156	85.1
15	7.41	38.5	68.9	13.8	3.95	16.0	2.89	1.45	0.810	2.30	1.01	9.72	0.104	68.3
12	9.43	46.6	53.8	10.9	3.90	12.6	2.18	1.10	0.785	1.74	0.983	9.66	0.0547	50.9
67	4.43	11.1	272	60.4	3.72	70.1	88.6	21.4	2.12	32.7	2.43	8.07	5.05	1440
58	5.07	12.4	228	52.0	3.65	59.8	75.1	18.3	2.10	27.9	2.39	7.94	3.33	1180
48	5.92	15.9	184	43.2	3.61	49.0	60.9	15.0	2.08	22.9	2.35	7.82	1.96	931
40	7.21	17.6	146	35.5	3.53	39.8	49.1	12.2	2.04	18.5	2.31	7.69	1.12	726
35	8.10	20.5	127	31.2	3.51	34.7	42.6	10.6	2.03	16.1	2.28	7.63	0.769	619
31	9.19	22.3	110	27.5	3.47	30.4	37.1	9.27	2.02	14.1	2.26	7.57	0.536	530
28	7.03	22.3	98.0	24.3	3.45	27.2	21.7	6.63	1.62	10.1	1.84	7.60	0.537	312
24	8.12	25.9	82.7	20.9	3.42	23.1	18.3	5.63	1.61	8.57	1.81	7.53	0.346	259
21	6.59	27.5	75.3	18.2	3.49	20.4	9.77	3.71	1.26	5.69	1.46	7.88	0.282	152
18	7.95	29.9	61.9	15.2	3.43	17.0	7.97	3.04	1.23	4.66	1.43	7.81	0.172	122
15	6.37	28.1	48.0	11.8	3.29	13.6	3.41	1.70	0.876	2.67	1.06	7.80	0.137	51.8
13	7.84	29.9	39.6	9.91	3.21	11.4	2.73	1.37	0.843	2.15	1.03	7.74	0.0871	40.8
10	9.61	40.5	30.8	7.81	3.22	8.87	2.09	1.06	0.841	1.66	1.01	7.69	0.0426	30.9

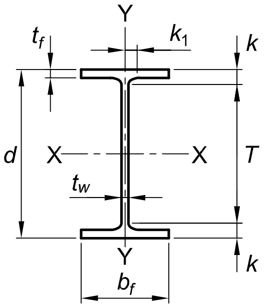


Table 1-1 (continued)
W-Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		Web $\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k k_{des}	Distance k k_{det}	Distance k_1	Distance T	Distance Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W6×25	7.34	6.38	6 3/8	0.320	5/16	3/16	6.08	6 1/8	0.455	7/16	0.705	15/16	9/16	4 1/2	3 1/2
×20	5.87	6.20	6 1/4	0.260	1/4	1/8	6.02	6	0.365	3/8	0.615	7/8	9/16		
×15[f]	4.43	5.99	6	0.230	1/4	1/8	5.99	6	0.260	1/4	0.510	3/4	9/16		
W6×16	4.74	6.28	6 1/4	0.260	1/4	1/8	4.03	4	0.405	3/8	0.655	7/8	9/16	4 1/2	2 1/4[g]
×12	3.55	6.03	6	0.230	1/4	1/8	4.00	4	0.280	1/4	0.530	3/4	9/16		
×9[f]	2.68	5.90	5 7/8	0.170	3/16	1/8	3.94	4	0.215	3/16	0.465	11/16	1/2		
×8.5[f]	2.52	5.83	5 7/8	0.170	3/16	1/8	3.94	4	0.195	3/16	0.445	11/16	1/2		
W5×19	5.56	5.15	5 1/8	0.270	1/4	1/8	5.03	5	0.430	7/16	0.730	13/16	7/16	3 1/2	2 3/4[g]
×16	4.71	5.01	5	0.240	1/4	1/8	5.00	5	0.360	3/8	0.660	3/4	7/16	3 1/2	2 3/4[g]
W4×13	3.83	4.16	4 1/8	0.280	1/4	1/8	4.06	4	0.345	3/8	0.595	3/4	1/2	2 5/8	2 1/4[g]

[f] Shape exceeds compact limit for flexure with F_y = 50 ksi.
[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.

Table 1-1 (continued)
W-Shapes
Properties

W6–W4

Nom-inal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z			J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
25	6.68	15.5	53.4	16.7	2.70	18.9	17.1	5.61	1.52	8.56	1.74	5.93	0.461	150
20	8.25	19.1	41.4	13.4	2.66	14.9	13.3	4.41	1.50	6.72	1.70	5.84	0.240	113
15	11.5	21.6	29.1	9.72	2.56	10.8	9.32	3.11	1.45	4.75	1.66	5.73	0.101	76.5
16	4.98	19.1	32.1	10.2	2.60	11.7	4.43	2.20	0.967	3.39	1.13	5.88	0.223	38.2
12	7.14	21.6	22.1	7.31	2.49	8.30	2.99	1.50	0.918	2.32	1.08	5.75	0.0903	24.7
9	9.16	29.2	16.4	5.56	2.47	6.23	2.20	1.11	0.905	1.72	1.06	5.69	0.0405	17.7
8.5	10.1	29.1	14.9	5.10	2.43	5.73	1.99	1.01	0.890	1.56	1.05	5.64	0.0333	15.8
19	5.85	13.7	26.3	10.2	2.17	11.6	9.13	3.63	1.28	5.53	1.45	4.72	0.316	50.9
16	6.94	15.4	21.4	8.55	2.13	9.63	7.51	3.00	1.26	4.58	1.43	4.65	0.192	40.6
13	5.88	10.6	11.3	5.46	1.72	6.28	3.86	1.90	1.00	2.92	1.16	3.82	0.151	14.0

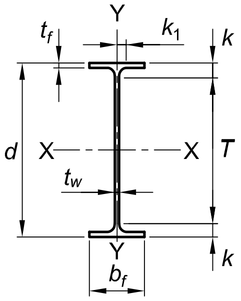


Table 1-2
M-Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		Web $\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k	Distance k_1	Distance T	Distance Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.
M12.5×12.4[c,v]	3.63	12.5	12 1/2	0.155	1/8	1/16	3.75	3 3/4	0.228	1/4	9/16	3/8	11 3/8	–
×11.6[c,v]	3.40	12.5	12 1/2	0.155	1/8	1/16	3.50	3 1/2	0.211	3/16	9/16	3/8	11 3/8	–
M12×11.8[c]	3.47	12.0	12	0.177	3/16	1/8	3.07	3 1/8	0.225	1/4	9/16	3/8	10 7/8	–
×10.8[c,v]	3.18	12.0	12	0.160	3/16	1/8	3.07	3 1/8	0.210	3/16	9/16	3/8	10 7/8	–
M12×10[c,v]	2.95	12.0	12	0.149	1/8	1/16	3.25	3 1/4	0.180	3/16	1/2	3/8	11	–
M10×9[c]	2.65	10.0	10	0.157	3/16	1/8	2.69	2 3/4	0.206	3/16	9/16	3/8	8 7/8	–
×8[c,v]	2.37	9.95	10	0.141	1/8	1/16	2.69	2 3/4	0.182	3/16	9/16	3/8	8 7/8	–
M10×7.5[c,v]	2.22	9.99	10	0.130	1/8	1/16	2.69	2 3/4	0.173	3/16	7/16	5/16	9 1/8	–
M8×6.5[c]	1.92	8.00	8	0.135	1/8	1/16	2.28	2 1/4	0.189	3/16	9/16	3/8	6 7/8	–
×6.2[c]	1.82	8.00	8	0.129	1/8	1/16	2.28	2 1/4	0.177	3/16	7/16	1/4	7 1/8	–
M6×4.4[c]	1.29	6.00	6	0.114	1/8	1/16	1.84	1 7/8	0.171	3/16	3/8	1/4	5 1/4	–
×3.7[c]	1.09	5.92	5 7/8	0.0980	1/8	1/16	2.00	2	0.129	1/8	5/16	1/4	5 1/4	–
M5×18.9[t]	5.56	5.00	5	0.316	5/16	3/16	5.00	5	0.416	7/16	13/16	1/2	3 3/8	2 3/4[g]
M4×6[f]	1.75	3.80	3 3/4	0.130	1/8	1/16	3.80	3 3/4	0.160	3/16	1/2	3/8	2 3/4	–
×4.08	1.27	4.00	4	0.115	1/8	1/16	2.25	2 1/4	0.170	3/16	9/16	3/8	2 7/8	–
×3.45	1.01	4.00	4	0.0920	1/16	1/16	2.25	2 1/4	0.130	1/8	1/2	3/8	3	–
×3.2	1.01	4.00	4	0.0920	1/16	1/16	2.25	2 1/4	0.130	1/8	1/2	3/8	3	–
M3×2.9	0.914	3.00	3	0.0900	1/16	1/16	2.25	2 1/4	0.130	1/8	1/2	3/8	2	–

[c] Shape is slender for compression with $F_y = 36$ ksi.
[f] Shape exceeds compact limit for flexure with $F_y = 36$ ksi.
[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.
[t] Shape has tapered flanges while other M-shapes have parallel flange surfaces.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(b)(1)(i) with $F_y = 36$ ksi.
– Indicates flange is too narrow to establish a workable gage.

Table 1-2 (continued)
M-Shapes
Properties

M-SHAPES

Nominal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	$\frac{J}{S_x h_o}$	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z				J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.		in.^{4}	in.^{6}
12.4	8.22	74.8	89.3	14.2	4.96	16.5	2.01	1.07	0.744	1.68	0.933	12.3	0.000283	0.0493	76.0
11.6	8.29	74.8	80.3	12.8	4.86	15.0	1.51	0.864	0.667	1.37	0.852	12.3	0.000263	0.0414	57.1
11.8	6.81	62.5	72.2	12.0	4.56	14.3	1.09	0.709	0.559	1.15	0.731	11.8	0.000355	0.0500	37.7
10.8	7.30	69.2	66.7	11.1	4.58	13.2	1.01	0.661	0.564	1.07	0.732	11.8	0.000300	0.0393	35.0
10	9.03	74.7	61.7	10.3	4.57	12.2	1.03	0.636	0.592	1.02	0.768	11.8	0.000240	0.0292	35.9
9	6.53	58.4	39.0	7.79	3.83	9.22	0.672	0.500	0.503	0.809	0.650	9.79	0.000411	0.0314	16.1
8	7.39	65.0	34.6	6.95	3.82	8.20	0.593	0.441	0.500	0.711	0.646	9.77	0.000328	0.0224	14.2
7.5	7.77	71.0	33.0	6.60	3.85	7.77	0.562	0.418	0.503	0.670	0.646	9.82	0.000289	0.0187	13.5
6.5	6.03	53.8	18.5	4.63	3.11	5.43	0.376	0.329	0.443	0.529	0.563	7.81	0.000509	0.0184	5.73
6.2	6.44	56.5	17.6	4.39	3.10	5.15	0.352	0.308	0.439	0.495	0.560	7.82	0.000455	0.0156	5.38
4.4	5.39	47.0	7.23	2.41	2.36	2.80	0.180	0.195	0.372	0.311	0.467	5.83	0.000707	0.00990	1.53
3.7	7.75	54.7	5.96	2.01	2.34	2.33	0.173	0.173	0.398	0.273	0.499	5.79	0.000459	0.00530	1.45
18.9	6.01	11.2	24.2	9.67	2.08	11.1	8.70	3.48	1.25	5.33	1.44	4.58	0.00709	0.313	45.7
6	11.9	22.0	4.72	2.48	1.64	2.74	1.47	0.771	0.915	1.18	1.04	3.64	0.00208	0.0184	4.87
4.08	6.62	26.4	3.53	1.77	1.67	2.00	0.325	0.289	0.506	0.453	0.593	3.83	0.00218	0.0147	1.19
3.45	8.65	33.9	2.86	1.43	1.68	1.60	0.248	0.221	0.496	0.346	0.580	3.87	0.00148	0.00820	0.930
3.2	8.65	33.9	2.86	1.43	1.68	1.60	0.248	0.221	0.496	0.346	0.580	3.87	0.00148	0.00820	0.930
2.9	8.65	23.6	1.50	1.00	1.28	1.12	0.248	0.221	0.521	0.344	0.597	2.87	0.00275	0.00790	0.511

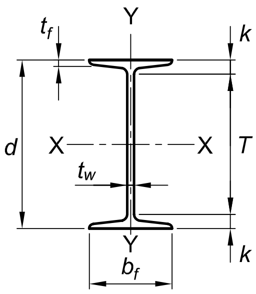
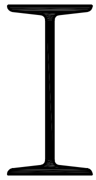


Table 1-3
S-Shapes
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		Web $\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k	Distance T	Distance Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.
S24×121	35.5	24.5	24 1/2	0.800	13/16	7/16	8.05	8	1.09	1 1/16	2	20 1/2	4
×106	31.1	24.5	24 1/2	0.620	5/8	5/16	7.87	7 7/8	1.09	1 1/16	2	20 1/2	4
S24×100	29.3	24.0	24	0.745	3/4	3/8	7.25	7 1/4	0.870	7/8	1 3/4	20 1/2	4
×90	26.5	24.0	24	0.625	5/8	5/16	7.13	7 1/8	0.870	7/8	1 3/4	20 1/2	↓
×80	23.5	24.0	24	0.500	1/2	1/4	7.00	7	0.870	7/8	1 3/4	20 1/2	
S20×96	28.2	20.3	20 1/4	0.800	13/16	7/16	7.20	7 1/4	0.920	15/16	1 3/4	16 3/4	4
×86	25.3	20.3	20 1/4	0.660	11/16	3/8	7.06	7	0.920	15/16	1 3/4	16 3/4	4
S20×75	22.0	20.0	20	0.635	5/8	5/16	6.39	6 3/8	0.795	13/16	1 5/8	16 3/4	3 1/2[g]
×66	19.4	20.0	20	0.505	1/2	1/4	6.26	6 1/4	0.795	13/16	1 5/8	16 3/4	3 1/2[g]
S18×70	20.5	18.0	18	0.711	11/16	3/8	6.25	6 1/4	0.691	11/16	1 1/2	15	3 1/2[g]
×54.7	16.0	18.0	18	0.461	7/16	1/4	6.00	6	0.691	11/16	1 1/2	15	3 1/2[g]
S15×50	14.7	15.0	15	0.550	9/16	5/16	5.64	5 5/8	0.622	5/8	1 3/8	12 1/4	3 1/2[g]
×42.9	12.6	15.0	15	0.411	7/16	1/4	5.50	5 1/2	0.622	5/8	1 3/8	12 1/4	3 1/2[g]
S12×50	14.7	12.0	12	0.687	11/16	3/8	5.48	5 1/2	0.659	11/16	1 7/16	9 1/8	3[g]
×40.8	11.9	12.0	12	0.462	7/16	1/4	5.25	5 1/4	0.659	11/16	1 7/16	9 1/8	3[g]
S12×35	10.2	12.0	12	0.428	7/16	1/4	5.08	5 1/8	0.544	9/16	1 3/16	9 5/8	3[g]
×31.8	9.31	12.0	12	0.350	3/8	3/16	5.00	5	0.544	9/16	1 3/16	9 5/8	3[g]
S10×35	10.3	10.0	10	0.594	5/8	5/16	4.94	5	0.491	1/2	1 1/8	7 3/4	2 3/4[g]
×25.4	7.45	10.0	10	0.311	5/16	3/16	4.66	4 5/8	0.491	1/2	1 1/8	7 3/4	2 3/4[g]
S8×23	6.76	8.00	8	0.441	7/16	1/4	4.17	4 1/8	0.425	7/16	1	6	2 1/4[g]
×18.4	5.40	8.00	8	0.271	1/4	1/8	4.00	4	0.425	7/16	1	6	2 1/4[g]
S6×17.25	5.05	6.00	6	0.465	7/16	1/4	3.57	3 5/8	0.359	3/8	13/16	4 3/8	–
×12.5	3.66	6.00	6	0.232	1/4	1/8	3.33	3 3/8	0.359	3/8	13/16	4 3/8	–
S5×10	2.93	5.00	5	0.214	3/16	1/8	3.00	3	0.326	5/16	3/4	3 1/2	–
S4×9.5	2.79	4.00	4	0.326	5/16	3/16	2.80	2 3/4	0.293	5/16	3/4	2 1/2	–
×7.7	2.26	4.00	4	0.193	3/16	1/8	2.66	2 5/8	0.293	5/16	3/4	2 1/2	–
S3×7.5	2.20	3.00	3	0.349	3/8	3/16	2.51	2 1/2	0.260	1/4	5/8	1 3/4	–
×5.7	1.66	3.00	3	0.170	3/16	1/8	2.33	2 3/8	0.260	1/4	5/8	1 3/4	–

[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.
– Indicates flange is too narrow to establish a workable gage.

Table 1-3 (continued)
S-Shapes
Properties



S-SHAPES

Nom-inal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z			J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
121	3.69	25.9	3160	258	9.43	306	83.0	20.6	1.53	36.3	1.94	23.4	12.8	11400
106	3.61	33.4	2940	240	9.71	279	76.8	19.5	1.57	33.4	1.93	23.4	10.1	10500
100	4.16	27.8	2380	199	9.01	239	47.4	13.1	1.27	24.0	1.66	23.1	7.59	6350
90	4.09	33.1	2250	187	9.21	222	44.7	12.5	1.30	22.4	1.66	23.1	6.05	5980
80	4.02	41.4	2100	175	9.47	204	42.0	12.0	1.34	20.8	1.67	23.1	4.89	5620
96	3.91	21.1	1670	165	7.71	198	49.9	13.9	1.33	24.9	1.71	19.4	8.40	4690
86	3.84	25.6	1570	155	7.89	183	46.6	13.2	1.36	23.1	1.71	19.4	6.65	4370
75	4.02	26.6	1280	128	7.62	152	29.5	9.25	1.16	16.7	1.49	19.2	4.59	2720
66	3.93	33.5	1190	119	7.83	139	27.5	8.78	1.19	15.4	1.49	19.2	3.58	2530
70	4.52	21.5	923	103	6.70	124	24.0	7.69	1.08	14.3	1.42	17.3	4.10	1800
54.7	4.34	33.2	801	89.0	7.07	104	20.7	6.91	1.14	12.1	1.42	17.3	2.33	1550
50	4.53	22.7	485	64.7	5.75	77.0	15.6	5.53	1.03	10.0	1.32	14.4	2.12	805
42.9	4.42	30.4	446	59.4	5.95	69.2	14.3	5.19	1.06	9.08	1.31	14.4	1.54	737
50	4.16	13.7	303	50.6	4.55	60.9	15.6	5.69	1.03	10.3	1.32	11.3	2.77	501
40.8	3.98	20.6	270	45.1	4.76	52.7	13.5	5.13	1.06	8.86	1.30	11.3	1.69	433
35	4.67	23.1	228	38.1	4.72	44.6	9.84	3.88	0.980	6.80	1.22	11.5	1.05	323
31.8	4.60	28.3	217	36.2	4.83	41.8	9.33	3.73	1.00	6.44	1.21	11.5	0.878	306
35	5.03	13.4	147	29.4	3.78	35.4	8.30	3.36	0.899	6.19	1.16	9.51	1.29	188
25.4	4.75	25.6	123	24.6	4.07	28.3	6.73	2.89	0.950	4.99	1.14	9.51	0.603	152
23	4.91	14.1	64.7	16.2	3.09	19.2	4.27	2.05	0.795	3.67	0.999	7.58	0.550	61.2
18.4	4.71	22.9	57.5	14.4	3.26	16.5	3.69	1.84	0.827	3.18	0.985	7.58	0.335	52.9
17.25	4.97	9.67	26.2	8.74	2.28	10.5	2.29	1.28	0.673	2.35	0.859	5.64	0.371	18.2
12.5	4.64	19.4	22.0	7.34	2.45	8.45	1.80	1.08	0.702	1.86	0.831	5.64	0.167	14.3
10	4.61	16.8	12.3	4.90	2.05	5.66	1.19	0.795	0.638	1.37	0.754	4.67	0.114	6.52
9.5	4.77	8.33	6.76	3.38	1.56	4.04	0.887	0.635	0.564	1.13	0.698	3.71	0.120	3.05
7.7	4.54	14.1	6.05	3.03	1.64	3.50	0.748	0.562	0.576	0.970	0.676	3.71	0.0732	2.57
7.5	4.83	5.38	2.91	1.94	1.15	2.35	0.578	0.461	0.513	0.821	0.638	2.74	0.0896	1.08
5.7	4.48	11.0	2.50	1.67	1.23	1.94	0.447	0.383	0.518	0.656	0.605	2.74	0.0433	0.838

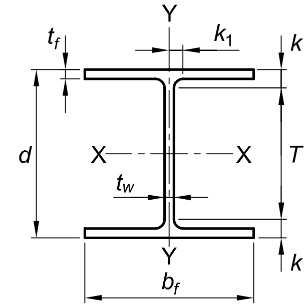


Table 1-4
HP-Shapes
Dimensions

Shape	Area, A	Depth, d		Web: Thickness, t_w		Web: $\frac{t_w}{2}$	Flange: Width, b_f		Flange: Thickness, t_f		Distance: k	Distance: k_1	Distance: T	Distance: Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.
HP18×204	60.2	18.3	18 1/4	1.13	1 1/8	9/16	18.1	18 1/8	1.13	1 1/8	2 5/16	1 3/4	13 1/2	7 1/2
×181	53.2	18.0	18	1.00	1	1/2	18.0	18	1.00	1	2 3/16	1 11/16		
×157[f]	46.2	17.7	17 3/4	0.870	7/8	7/16	17.9	17 7/8	0.870	7/8	2 1/16	1 5/8		
×135[f]	39.9	17.5	17 1/2	0.750	3/4	3/8	17.8	17 3/4	0.750	3/4	1 15/16	1 9/16		
HP16×183	54.1	16.5	16 1/2	1.13	1 1/8	9/16	16.3	16 1/2	1.13	1 1/8	2 5/16	1 3/4	11 3/4	5 1/2
×162	47.7	16.3	16 1/4	1.00	1	1/2	16.1	16 1/8	1.00	1	2 3/16	1 11/16		
×141	41.7	16.0	16	0.875	7/8	7/16	16.0	16	0.875	7/8	2 1/16	1 5/8		
×121[f]	35.8	15.8	15 3/4	0.750	3/4	3/8	15.9	15 7/8	0.750	3/4	1 15/16	1 9/16		
×101[f]	29.9	15.5	15 1/2	0.625	5/8	5/16	15.8	15 3/4	0.625	5/8	1 13/16	1 1/2		
×88[c,f]	25.8	15.3	15 3/8	0.540	9/16	5/16	15.7	15 11/16	0.540	9/16	1 3/4	1 7/16		
HP14×117[f]	34.4	14.2	14 1/4	0.805	13/16	7/16	14.9	14 7/8	0.805	13/16	2 1/16	1 5/8	11 1/4	5 1/2
×102[f]	30.1	14.0	14	0.705	11/16	3/8	14.8	14 3/4	0.705	11/16	1 15/16	1 9/16		
×89[f]	26.1	13.8	13 7/8	0.615	5/8	5/16	14.7	14 3/4	0.615	5/8	1 7/8	1 1/2		
×73[c,f]	21.4	13.6	13 5/8	0.505	1/2	1/4	14.6	14 5/8	0.505	1/2	1 3/4	1 7/16		
HP12×89	25.9	12.4	12 3/8	0.720	3/4	3/8	12.3	12 3/8	0.720	3/4	1 5/8	1 3/16	9 1/2	5 1/2
×84	24.6	12.3	12 1/4	0.685	11/16	3/8	12.3	12 1/4	0.685	11/16	1 9/16	1 3/16		
×74[f]	21.8	12.1	12 1/8	0.605	5/8	5/16	12.2	12 1/4	0.610	5/8	1 1/2	1 1/8		
×63[f]	18.4	11.9	12	0.515	1/2	1/4	12.1	12 1/8	0.515	1/2	1 7/16	1 1/16		
×53[c,f]	15.5	11.8	11 3/4	0.435	7/16	1/4	12.0	12	0.435	7/16	1 5/16	1 1/16		
HP10×57	16.7	9.99	10	0.565	9/16	5/16	10.2	10 1/4	0.565	9/16	1 1/4	15/16	7 1/2	5 1/2
×42[f]	12.4	9.70	9 3/4	0.415	7/16	1/4	10.1	10 1/8	0.420	7/16	1 1/8	13/16	7 1/2	5 1/2
HP8×36[f]	10.6	8.02	8	0.445	7/16	1/4	8.16	8 1/8	0.445	7/16	1 1/8	7/8	5 3/4	5 1/2

[c] Shape is slender for compression with $F_y = 50$ ksi.
[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.

Table 1-4 (continued)
HP-Shapes
Properties

HP-SHAPES

Nom-inal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	$\frac{J}{S_x h_o}$	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z				J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.		in.^{4}	in.^{6}
204	8.01	12.1	3480	380	7.60	433	1120	124	4.31	191	5.03	17.2	0.00451	29.5	82500
181	9.00	13.6	3020	336	7.53	379	974	108	4.28	167	4.96	17.0	0.00362	20.7	70400
157	10.3	15.6	2570	290	7.46	327	833	93.1	4.25	143	4.92	16.8	0.00285	13.9	59000
135	11.9	18.2	2200	251	7.43	281	706	79.3	4.21	122	4.85	16.8	0.00216	9.12	49500
183	7.21	10.5	2510	304	6.81	349	818	100	3.89	156	4.55	15.4	0.00576	26.9	48300
162	8.05	11.9	2190	269	6.78	306	697	86.6	3.82	134	4.45	15.3	0.00457	18.8	40800
141	9.14	13.6	1870	234	6.70	264	599	74.9	3.79	116	4.40	15.1	0.00365	12.9	34300
121	10.6	15.9	1590	201	6.66	226	504	63.4	3.75	97.6	4.34	15.1	0.00275	8.35	28500
101	12.6	19.0	1300	168	6.59	187	412	52.2	3.71	80.1	4.27	14.9	0.00203	5.07	22800
88	14.5	22.0	1110	145	6.56	161	349	44.5	3.68	68.2	4.21	14.8	0.00161	3.45	19000
117	9.25	14.2	1220	172	5.96	194	443	59.5	3.59	91.4	4.15	13.4	0.00348	8.02	19900
102	10.5	16.2	1050	150	5.92	169	380	51.4	3.56	78.8	4.10	13.3	0.00270	5.39	16800
89	11.9	18.5	904	131	5.88	146	326	44.3	3.53	67.7	4.05	13.2	0.00207	3.59	14200
73	14.4	22.6	729	107	5.84	118	261	35.8	3.49	54.6	4.00	13.1	0.00143	2.01	11200
89	8.54	13.6	693	112	5.17	127	224	36.4	2.94	56.0	3.42	11.7	0.00376	4.92	7640
84	8.97	14.2	650	106	5.14	120	213	34.6	2.94	53.2	3.41	11.6	0.00345	4.24	7140
74	10.0	16.1	569	93.8	5.11	105	186	30.4	2.92	46.6	3.38	11.5	0.00276	2.98	6160
63	11.8	18.9	472	79.1	5.06	88.3	153	25.3	2.88	38.7	3.33	11.4	0.00202	1.83	5000
53	13.8	22.3	393	66.7	5.03	74.0	127	21.1	2.86	32.2	3.29	11.4	0.00148	1.12	4080
57	9.03	13.9	294	58.8	4.18	66.5	101	19.7	2.45	30.3	2.84	9.43	0.00355	1.97	2240
42	12.0	18.9	210	43.4	4.13	48.3	71.7	14.2	2.41	21.8	2.77	9.28	0.00202	0.813	1540
36	9.16	14.2	119	29.8	3.36	33.6	40.3	9.88	1.95	15.2	2.26	7.58	0.00341	0.770	578

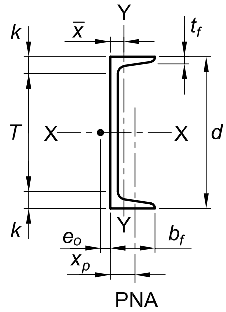


Table 1-5
C-Shapes
Dimensions

Shape	Area, A	Depth, d		Web: Thickness, t_w		Web: $\frac{t_w}{2}$	Flange: Width, b_f		Flange: Average Thickness, t_f		Distance: k	Distance: T	Distance: Workable Gage	r_{ts}	h_o
	in.²	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
C15×50	14.7	15.0	15	0.716	11/16	3/8	3.72	3 3/4	0.650	5/8	1 7/16	12 1/8	2 1/4	1.17	14.4
×40	11.8	15.0	15	0.520	1/2	1/4	3.52	3 1/2	0.650	5/8	1 7/16	↓	2	1.15	14.4
×33.9	10.0	15.0	15	0.400	3/8	3/16	3.40	3 3/8	0.650	5/8	1 7/16	↓	2	1.13	14.4
C12×30	8.81	12.0	12	0.510	1/2	1/4	3.17	3 1/8	0.501	1/2	1 1/8	9 3/4	1 3/4[g]	1.01	11.5
×25	7.34	12.0	12	0.387	3/8	3/16	3.05	3	0.501	1/2	1 1/8	↓	↓	1.00	11.5
×20.7	6.08	12.0	12	0.282	5/16	3/16	2.94	3	0.501	1/2	1 1/8	↓	↓	0.983	11.5
C10×30	8.81	10.0	10	0.673	11/16	3/8	3.03	3	0.436	7/16	1 1/16	8	1 3/4[g]	0.924	9.56
×25	7.35	10.0	10	0.526	1/2	1/4	2.89	2 7/8	0.436	7/16	1 1/16	↓	1 3/4[g]	0.911	9.56
×20	5.87	10.0	10	0.379	3/8	3/16	2.74	2 3/4	0.436	7/16	1 1/16	↓	1 1/2[g]	0.894	9.56
×15.3	4.48	10.0	10	0.240	1/4	1/8	2.60	2 5/8	0.436	7/16	1 1/16	↓	1 1/2[g]	0.868	9.56
C9×20	5.87	9.00	9	0.448	7/16	1/4	2.65	2 5/8	0.413	7/16	1	7	1 1/2[g]	0.850	8.59
×15	4.40	9.00	9	0.285	5/16	3/16	2.49	2 1/2	0.413	7/16	1	↓	1 3/8[g]	0.825	8.59
×13.4	3.94	9.00	9	0.233	1/4	1/8	2.43	2 3/8	0.413	7/16	1	↓	1 3/8[g]	0.814	8.59
C8×18.75	5.51	8.00	8	0.487	1/2	1/4	2.53	2 1/2	0.390	3/8	15/16	6 1/8	1 1/2[g]	0.800	7.61
×13.75	4.03	8.00	8	0.303	5/16	3/16	2.34	2 3/8	0.390	3/8	15/16	↓	1 3/8[g]	0.774	7.61
×11.5	3.37	8.00	8	0.220	1/4	1/8	2.26	2 1/4	0.390	3/8	15/16	↓	1 3/8[g]	0.756	7.61
C7×14.75	4.33	7.00	7	0.419	7/16	1/4	2.30	2 1/4	0.366	3/8	7/8	5 1/4	1 1/4[g]	0.738	6.63
×12.25	3.59	7.00	7	0.314	5/16	3/16	2.19	2 1/4	0.366	3/8	7/8	↓	↓	0.722	6.63
×9.8	2.87	7.00	7	0.210	3/16	1/8	2.09	2 1/8	0.366	3/8	7/8	↓	↓	0.698	6.63
C6×13	3.82	6.00	6	0.437	7/16	1/4	2.16	2 1/8	0.343	5/16	13/16	4 3/8	1 3/8[g]	0.689	5.66
×10.5	3.07	6.00	6	0.314	5/16	3/16	2.03	2	0.343	5/16	13/16	↓	1 1/8[g]	0.669	5.66
×8.2	2.39	6.00	6	0.200	3/16	1/8	1.92	1 7/8	0.343	5/16	13/16	↓	1 1/8[g]	0.643	5.66
C5×9	2.64	5.00	5	0.325	5/16	3/16	1.89	1 7/8	0.320	5/16	3/4	3 1/2	1 1/8[g]	0.616	4.68
×6.7	1.97	5.00	5	0.190	3/16	1/8	1.75	1 3/4	0.320	5/16	3/4	3 1/2	–	0.584	4.68
C4×7.25	2.13	4.00	4	0.321	5/16	3/16	1.72	1 3/4	0.296	5/16	3/4	2 1/2	1[g]	0.563	3.70
×6.25	1.84	4.00	4	0.247	1/4	1/8	1.65	1 5/8	0.296	5/16	3/4	↓	–	0.549	3.70
×5.4	1.58	4.00	4	0.184	3/16	1/8	1.58	1 5/8	0.296	5/16	3/4	↓	–	0.528	3.70
×4.5	1.34	4.00	4	0.125	1/8	1/16	1.52	1 1/2	0.296	5/16	3/4	↓	–	0.506	3.70
C3×6	1.76	3.00	3	0.356	3/8	3/16	1.60	1 5/8	0.273	1/4	11/16	1 5/8	–	0.519	2.73
×5	1.47	3.00	3	0.258	1/4	1/8	1.50	1 1/2	0.273	1/4	11/16	↓	–	0.496	2.73
×4.1	1.20	3.00	3	0.170	3/16	1/8	1.41	1 3/8	0.273	1/4	11/16	↓	–	0.469	2.73
×3.5	1.09	3.00	3	0.132	1/8	1/16	1.37	1 3/8	0.273	1/4	11/16	↓	–	0.456	2.73

[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.
– Indicates flange is too narrow to establish a workable gage.

Table 1-5 (continued)
C-Shapes
Properties

C-SHAPES

Nominal Wt.	Shear Ctr., e_o	Axis X-X				Axis Y-Y						Torsional Properties			
		I	S	r	Z	I	S	r	$\bar{x}$	Z	x_p	J	C_w	$\bar{r}_o$	H
lb/ft	in.	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{6}	in.	
50	0.583	404	53.8	5.24	68.5	11.0	3.77	0.865	0.799	8.14	0.490	2.65	492	5.49	0.937
40	0.767	348	46.5	5.43	57.5	9.17	3.34	0.883	0.778	6.84	0.392	1.45	410	5.71	0.927
33.9	0.896	315	42.0	5.61	50.8	8.07	3.09	0.901	0.788	6.19	0.332	1.01	358	5.94	0.920
30	0.618	162	27.0	4.29	33.8	5.12	2.05	0.762	0.674	4.32	0.367	0.861	151	4.54	0.919
25	0.746	144	24.0	4.43	29.4	4.45	1.87	0.779	0.674	3.82	0.306	0.538	130	4.72	0.909
20.7	0.870	129	21.5	4.61	25.6	3.86	1.72	0.797	0.698	3.47	0.253	0.369	112	4.93	0.899
30	0.368	103	20.7	3.43	26.7	3.93	1.65	0.668	0.649	3.78	0.441	1.22	79.5	3.63	0.921
25	0.494	91.1	18.2	3.52	23.1	3.34	1.47	0.675	0.617	3.18	0.367	0.687	68.3	3.76	0.912
20	0.636	78.9	15.8	3.67	19.4	2.80	1.31	0.690	0.606	2.70	0.294	0.368	56.9	3.93	0.900
15.3	0.796	67.3	13.5	3.88	15.9	2.27	1.15	0.711	0.634	2.34	0.224	0.209	45.5	4.19	0.884
20	0.515	60.9	13.5	3.22	16.9	2.41	1.17	0.640	0.583	2.46	0.326	0.427	39.4	3.46	0.899
15	0.681	51.0	11.3	3.40	13.6	1.91	1.01	0.659	0.586	2.04	0.245	0.208	31.0	3.69	0.882
13.4	0.742	47.8	10.6	3.48	12.6	1.75	0.954	0.666	0.601	1.94	0.219	0.168	28.2	3.79	0.875
18.75	0.431	43.9	11.0	2.82	13.9	1.97	1.01	0.598	0.565	2.17	0.344	0.434	25.1	3.05	0.894
13.75	0.604	36.1	9.02	2.99	11.0	1.52	0.848	0.613	0.554	1.73	0.252	0.186	19.2	3.26	0.874
11.5	0.697	32.5	8.14	3.11	9.63	1.31	0.775	0.623	0.572	1.57	0.211	0.130	16.5	3.41	0.862
14.75	0.441	27.2	7.78	2.51	9.75	1.37	0.772	0.561	0.532	1.63	0.309	0.267	13.1	2.75	0.875
12.25	0.538	24.2	6.92	2.59	8.46	1.16	0.696	0.568	0.525	1.42	0.257	0.161	11.2	2.86	0.862
9.8	0.647	21.2	6.07	2.72	7.19	0.957	0.617	0.578	0.541	1.26	0.205	0.0996	9.15	3.02	0.845
13	0.380	17.3	5.78	2.13	7.29	1.05	0.638	0.524	0.514	1.35	0.318	0.237	7.19	2.37	0.858
10.5	0.486	15.1	5.04	2.22	6.18	0.860	0.561	0.529	0.500	1.14	0.256	0.128	5.91	2.48	0.842
8.2	0.599	13.1	4.35	2.34	5.16	0.687	0.488	0.536	0.512	0.987	0.199	0.0736	4.70	2.65	0.824
9	0.427	8.89	3.56	1.84	4.39	0.624	0.444	0.486	0.478	0.913	0.264	0.109	2.93	2.10	0.815
6.7	0.552	7.48	2.99	1.95	3.55	0.470	0.372	0.489	0.484	0.757	0.215	0.0549	2.22	2.26	0.790
7.25	0.386	4.58	2.29	1.47	2.84	0.425	0.337	0.447	0.459	0.695	0.266	0.0817	1.24	1.75	0.767
6.25	0.447	4.19	2.10	1.51	2.55	0.374	0.312	0.451	0.453	0.623	0.233	0.0549	1.07	1.81	0.753
5.4	0.501	3.85	1.92	1.56	2.29	0.312	0.277	0.444	0.457	0.565	0.231	0.0399	0.921	1.88	0.742
4.5	0.556	3.53	1.77	1.62	2.05	0.265	0.253	0.445	0.473	0.495	0.305	0.0306	0.778	1.97	0.727
6	0.322	2.07	1.38	1.09	1.74	0.300	0.263	0.413	0.455	0.543	0.294	0.0725	0.462	1.40	0.690
5	0.392	1.85	1.23	1.12	1.52	0.241	0.228	0.405	0.439	0.464	0.245	0.0425	0.379	1.45	0.673
4.1	0.461	1.65	1.10	1.18	1.32	0.191	0.196	0.398	0.437	0.399	0.262	0.0269	0.307	1.53	0.655
3.5	0.493	1.57	1.04	1.20	1.24	0.169	0.182	0.394	0.443	0.364	0.296	0.0226	0.276	1.57	0.646

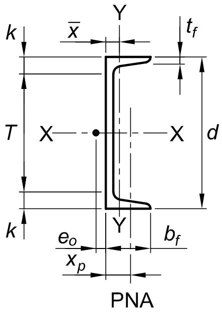
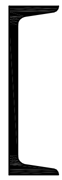


Table 1-6
MC-Shapes
Dimensions

Shape	Area, A	Depth, d		Web: Thickness, t_w		Web: $\frac{t_w}{2}$	Flange: Width, b_f		Flange: Average Thickness, t_f		Distance: k	Distance: T	Distance: Workable Gage	r_{ts}	h_o
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
MC18×58	17.1	18.0	18	0.700	11/16	3/8	4.20	4 1/4	0.625	5/8	1 7/16	15 1/8	2 1/2	1.35	17.4
×51.9	15.3	18.0	18	0.600	5/8	5/16	4.10	4 1/8	0.625	5/8	1 7/16			1.35	17.4
×45.8	13.5	18.0	18	0.500	1/2	1/4	4.00	4	0.625	5/8	1 7/16			1.34	17.4
×42.7	12.6	18.0	18	0.450	7/16	1/4	3.95	4	0.625	5/8	1 7/16			1.34	17.4
MC13×50	14.7	13.0	13	0.787	13/16	7/16	4.41	4 3/8	0.610	5/8	1 7/16	10 1/8	2 1/2	1.41	12.4
×40	11.7	13.0	13	0.560	9/16	5/16	4.19	4 1/8	0.610	5/8	1 7/16			1.38	12.4
×35	10.3	13.0	13	0.447	7/16	1/4	4.07	4 1/8	0.610	5/8	1 7/16			1.35	12.4
×31.8	9.35	13.0	13	0.375	3/8	3/16	4.00	4	0.610	5/8	1 7/16			1.34	12.4
MC12×50	14.7	12.0	12	0.835	13/16	7/16	4.14	4 1/8	0.700	11/16	1 5/16	9 3/8	2 1/2	1.37	11.3
×45	13.2	12.0	12	0.710	11/16	3/8	4.01	4	0.700	11/16	1 5/16			1.35	11.3
×40	11.8	12.0	12	0.590	9/16	5/16	3.89	3 7/8	0.700	11/16	1 5/16			1.33	11.3
×35	10.3	12.0	12	0.465	7/16	1/4	3.77	3 3/4	0.700	11/16	1 5/16			1.30	11.3
×31	9.12	12.0	12	0.370	3/8	3/16	3.67	3 5/8	0.700	11/16	1 5/16		2 1/4	1.28	11.3
MC12×14.3[c]	4.18	12.0	12	0.250	1/4	1/8	2.12	2 1/8	0.313	5/16	3/4	10 1/2	1 1/4[g]	0.672	11.7
MC12×10.6[c]	3.10	12.0	12	0.190	3/16	1/8	1.50	1 1/2	0.309	5/16	3/4	10 1/2	–	0.478	11.7
MC10×41.1	12.1	10.0	10	0.796	13/16	7/16	4.32	4 3/8	0.575	9/16	1 5/16	7 3/8	2 1/2[g]	1.44	9.43
×33.6	9.87	10.0	10	0.575	9/16	5/16	4.10	4 1/8	0.575	9/16	1 5/16			1.40	9.43
×28.5	8.37	10.0	10	0.425	7/16	1/4	3.95	4	0.575	9/16	1 5/16			1.36	9.43
MC10×25	7.34	10.0	10	0.380	3/8	3/16	3.41	3 3/8	0.575	9/16	1 5/16	7 3/8	2[g]	1.17	9.43
×22	6.45	10.0	10	0.290	5/16	3/16	3.32	3 3/8	0.575	9/16	1 5/16	7 3/8	2[g]	1.14	9.43
MC10×8.4[c]	2.46	10.0	10	0.170	3/16	1/8	1.50	1 1/2	0.280	1/4	3/4	8 1/2	–	0.486	9.72
×6.5[c]	1.95	10.0	10	0.152	1/8	1/16	1.17	1 1/8	0.202	3/16	9/16	8 7/8	–	0.363	9.80
MC9×25.4	7.47	9.00	9	0.450	7/16	1/4	3.50	3 1/2	0.550	9/16	1 1/4	6 1/2	2[g]	1.20	8.45
×23.9	7.02	9.00	9	0.400	3/8	3/16	3.45	3 1/2	0.550	9/16	1 1/4	6 1/2	2[g]	1.18	8.45
MC8×22.8	6.70	8.00	8	0.427	7/16	1/4	3.50	3 1/2	0.525	1/2	1 3/16	5 5/8	2[g]	1.20	7.48
×21.4	6.28	8.00	8	0.375	3/8	3/16	3.45	3 1/2	0.525	1/2	1 3/16	5 5/8	2[g]	1.18	7.48
MC8×20	5.87	8.00	8	0.400	3/8	3/16	3.03	3	0.500	1/2	1 1/8	5 3/4	2[g]	1.03	7.50
×18.7	5.50	8.00	8	0.353	3/8	3/16	2.98	3	0.500	1/2	1 1/8	5 3/4	2[g]	1.02	7.50
MC8×8.5	2.50	8.00	8	0.179	3/16	1/8	1.87	1 7/8	0.311	5/16	13/16	6 3/8	1 1/8[g]	0.624	7.69

[c] Shape is slender for compression with $F_y = 36$ ksi.
[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.
– Indicates flange is too narrow to establish a workable gage.

Table 1-6 (continued)
MC-Shapes
Properties



MC18–MC8

Nom-inal Wt.	Shear Ctr., e_o	Axis X-X				Axis Y-Y						Torsional Properties			
		I	S	r	Z	I	S	r	$\bar{x}$	Z	x_p	J	C_w	$\bar{r}_o$	H
lb/ft	in.	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{6}	in.	
58	0.695	675	75.0	6.29	95.4	17.6	5.28	1.02	0.862	10.7	0.474	2.81	1070	6.56	0.944
51.9	0.797	627	69.6	6.41	87.3	16.3	5.02	1.03	0.858	9.86	0.424	2.03	985	6.70	0.939
45.8	0.909	578	64.2	6.55	79.2	14.9	4.77	1.05	0.866	9.14	0.374	1.45	897	6.87	0.933
42.7	0.969	554	61.5	6.64	75.1	14.3	4.64	1.07	0.877	8.82	0.349	1.23	852	6.97	0.930
50	0.815	314	48.3	4.62	60.8	16.4	4.77	1.06	0.974	10.2	0.566	2.96	558	5.07	0.875
40	1.03	273	41.9	4.82	51.2	13.7	4.24	1.08	0.963	8.66	0.452	1.55	462	5.32	0.859
35	1.16	252	38.8	4.95	46.5	12.3	3.97	1.09	0.980	8.04	0.396	1.13	412	5.50	0.849
31.8	1.24	239	36.7	5.05	43.4	11.4	3.79	1.10	1.00	7.69	0.360	0.937	380	5.64	0.842
50	0.741	269	44.9	4.28	56.5	17.4	5.64	1.09	1.05	10.9	0.613	3.23	411	4.77	0.859
45	0.844	251	41.9	4.36	52.0	15.8	5.30	1.09	1.04	10.1	0.550	2.33	373	4.88	0.851
40	0.952	234	39.0	4.46	47.7	14.2	4.98	1.10	1.04	9.31	0.490	1.69	336	5.01	0.842
35	1.07	216	36.0	4.59	43.2	12.6	4.64	1.11	1.05	8.62	0.428	1.24	297	5.18	0.831
31	1.17	202	33.7	4.71	39.7	11.3	4.37	1.11	1.08	8.15	0.425	1.00	267	5.34	0.822
14.3	0.435	76.1	12.7	4.27	15.9	1.00	0.574	0.489	0.377	1.21	0.174	0.117	32.8	4.37	0.965
10.6	0.284	55.3	9.22	4.22	11.6	0.378	0.307	0.349	0.269	0.635	0.129	0.0596	11.7	4.27	0.983
41.1	0.864	157	31.5	3.61	39.3	15.7	4.85	1.14	1.09	9.49	0.604	2.26	269	4.26	0.790
33.6	1.06	139	27.8	3.75	33.7	13.1	4.35	1.15	1.09	8.28	0.494	1.20	224	4.47	0.770
28.5	1.21	126	25.3	3.89	30.0	11.3	3.99	1.16	1.12	7.59	0.419	0.791	193	4.68	0.752
25	1.03	110	22.0	3.87	26.2	7.25	2.96	0.993	0.953	5.65	0.367	0.638	124	4.46	0.803
22	1.12	102	20.5	3.99	23.9	6.40	2.75	0.997	0.990	5.29	0.467	0.510	110	4.62	0.791
8.4	0.332	31.9	6.39	3.61	7.92	0.326	0.268	0.364	0.284	0.548	0.123	0.0413	7.00	3.68	0.972
6.5	0.182	22.9	4.59	3.43	5.90	0.133	0.137	0.262	0.194	0.284	0.0975	0.0191	2.76	3.46	0.988
25.4	0.986	87.9	19.5	3.43	23.5	7.57	2.99	1.01	0.970	5.70	0.415	0.691	104	4.08	0.770
23.9	1.04	84.9	18.9	3.48	22.5	7.14	2.89	1.01	0.981	5.51	0.390	0.599	98.0	4.15	0.763
22.8	1.04	63.8	15.9	3.09	19.1	7.01	2.81	1.02	1.01	5.37	0.419	0.572	75.2	3.84	0.715
21.4	1.09	61.5	15.4	3.13	18.2	6.58	2.71	1.02	1.02	5.18	0.452	0.495	70.8	3.91	0.707
20	0.843	54.4	13.6	3.04	16.4	4.42	2.02	0.867	0.840	3.86	0.367	0.441	47.8	3.58	0.779
18.7	0.889	52.4	13.1	3.09	15.6	4.15	1.95	0.868	0.849	3.72	0.344	0.380	45.0	3.65	0.773
8.5	0.542	23.3	5.82	3.05	6.95	0.624	0.431	0.500	0.428	0.875	0.156	0.0587	8.21	3.24	0.910

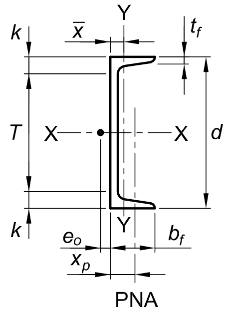


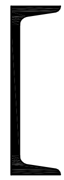
Table 1-6 (continued)
MC-Shapes
Dimensions

Shape	Area, A	Depth, d		Web			Flange				Distance			r_{ts}	h_o
				Thickness, t_w		$\frac{t_w}{2}$	Width, b_f		Average Thickness, t_f		k	T	Workable Gage		
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
MC7×22.7	6.67	7.00	7	0.503	1/2	1/4	3.60	3 5/8	0.500	1/2	1 1/8	4 3/4	2[g]	1.23	6.50
×19.1	5.61	7.00	7	0.352	3/8	3/16	3.45	3 1/2	0.500	1/2	1 1/8	4 3/4	2[g]	1.19	6.50
MC6×18	5.29	6.00	6	0.379	3/8	3/16	3.50	3 1/2	0.475	1/2	1 1/16	3 7/8	2[g]	1.20	5.53
×15.3	4.49	6.00	6	0.340	5/16	3/16	3.50	3 1/2	0.385	3/8	7/8	4 1/4	2[g]	1.20	5.62
MC6×16.3	4.79	6.00	6	0.375	3/8	3/16	3.00	3	0.475	1/2	1 1/16	3 7/8	1 3/4[g]	1.03	5.53
×15.1	4.44	6.00	6	0.316	5/16	3/16	2.94	3	0.475	1/2	1 1/16	3 7/8	1 3/4[g]	1.01	5.53
MC6×12	3.53	6.00	6	0.310	5/16	3/16	2.50	2 1/2	0.375	3/8	7/8	4 1/4	1 1/2[g]	0.856	5.63
MC6×7	2.09	6.00	6	0.179	3/16	1/8	1.88	1 7/8	0.291	5/16	3/4	4 1/2	–	0.638	5.71
×6.5	1.95	6.00	6	0.155	1/8	1/16	1.85	1 7/8	0.291	5/16	3/4	4 1/2	–	0.631	5.71
MC4×13.8	4.03	4.00	4	0.500	1/2	1/4	2.50	2 1/2	0.500	1/2	1	2	–	0.851	3.50
MC3×7.1	2.11	3.00	3	0.312	5/16	3/16	1.94	2	0.351	3/8	13/16	1 3/8	–	0.657	2.65

[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.

– Indicates flange is too narrow to establish a workable gage.

Table 1-6 (continued)
MC-Shapes
Properties



MC7–MC3

Nominal Wt.	Shear Ctr., e_o	Axis X-X				Axis Y-Y						Torsional Properties			
		I	S	r	Z	I	S	r	$\bar{x}$	Z	x_p	J	C_w	$\bar{r}_o$	H
lb/ft	in.	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{6}	in.	
22.7	1.01	47.4	13.5	2.67	16.4	7.24	2.83	1.04	1.04	5.38	0.477	0.625	58.3	3.53	0.659
19.1	1.15	43.1	12.3	2.77	14.5	6.06	2.55	1.04	1.08	4.85	0.579	0.407	49.3	3.70	0.638
18	1.17	29.7	9.89	2.37	11.7	5.88	2.47	1.05	1.12	4.68	0.644	0.379	34.6	3.46	0.563
15.3	1.16	25.3	8.44	2.38	9.91	4.91	2.01	1.05	1.05	3.85	0.511	0.223	30.0	3.41	0.579
16.3	0.930	26.0	8.66	2.33	10.4	3.77	1.82	0.887	0.927	3.47	0.465	0.336	22.1	3.11	0.643
15.1	0.982	24.9	8.30	2.37	9.83	3.46	1.73	0.883	0.940	3.30	0.543	0.285	20.5	3.18	0.634
12	0.725	18.7	6.24	2.30	7.47	1.85	1.03	0.724	0.704	1.97	0.294	0.155	11.3	2.80	0.740
7	0.583	11.4	3.81	2.34	4.50	0.603	0.439	0.537	0.501	0.865	0.174	0.0464	4.00	2.63	0.830
6.5	0.612	11.0	3.66	2.38	4.28	0.565	0.422	0.539	0.513	0.836	0.191	0.0412	3.75	2.68	0.824
13.8	0.643	8.85	4.43	1.48	5.53	2.13	1.29	0.727	0.849	2.40	0.508	0.373	4.84	2.23	0.550
7.1	0.574	2.72	1.81	1.14	2.24	0.666	0.518	0.562	0.653	0.998	0.414	0.0928	0.915	1.76	0.516

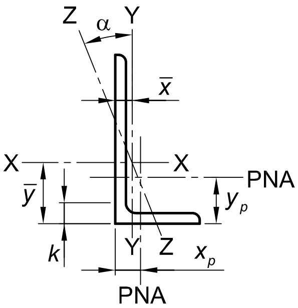


Table 1-7
Angles
Properties

Shape	k	Wt.	Area, A	Axis X-X						Flexural-Torsional Properties		
				I	S	r	$\bar{y}$	Z	y_p	J	C_w	$\bar{r}_o$
	in.	lb/ft	in.^{2}	in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{6}	in.
L12×12×$1\frac{3}{8}$	$2\frac{1}{16}$	105	31.1	413	48.6	3.64	3.50	88.1	1.30	19.9	211	6.51
×$1\frac{1}{4}$	$1\frac{15}{16}$	96.4	28.4	381	44.6	3.66	3.45	80.7	1.18	14.9	160	6.54
×$1\frac{1}{8}$	$1\frac{13}{16}$	87.2	25.8	350	40.7	3.68	3.41	73.7	1.08	11.1	120	6.58
×1	$1\frac{11}{16}$	77.8	23.0	315	36.5	3.70	3.36	65.9	0.958	7.80	84.5	6.61
L10×10×$1\frac{3}{8}$	$2\frac{3}{16}$	87.1	25.6	231	33.0	3.00	3.00	59.9	1.28	16.4	118	5.36
×$1\frac{1}{4}$	$2\frac{1}{16}$	79.9	23.4	213	30.2	3.02	2.95	54.9	1.17	12.3	89.4	5.39
×$1\frac{1}{8}$	$1\frac{15}{16}$	72.3	21.3	196	27.6	3.03	2.90	50.2	1.07	9.21	67.3	5.41
×1	$1\frac{13}{16}$	64.7	19.0	177	24.8	3.05	2.86	45.0	0.950	6.46	47.6	5.46
×$\frac{7}{8}$	$1\frac{11}{16}$	56.9	16.8	158	21.9	3.07	2.80	39.9	0.840	4.39	32.5	5.47
×$\frac{3}{4}$	$1\frac{9}{16}$	49.1	14.5	139	19.2	3.10	2.76	34.6	0.725	2.80	20.9	5.53
L8×8×$1\frac{1}{8}$	$1\frac{3}{4}$	56.9	16.8	98.1	17.5	2.41	2.40	31.6	1.05	7.13	32.5	4.29
×1	$1\frac{5}{8}$	51.0	15.1	89.1	15.8	2.43	2.36	28.5	0.944	5.08	23.4	4.32
×$\frac{7}{8}$	$1\frac{1}{2}$	45.0	13.3	79.7	14.0	2.45	2.31	25.3	0.831	3.46	16.1	4.36
×$\frac{3}{4}$	$1\frac{3}{8}$	38.9	11.5	69.9	12.2	2.46	2.26	22.0	0.719	2.21	10.4	4.39
×$\frac{5}{8}$	$1\frac{1}{4}$	32.7	9.69	59.6	10.3	2.48	2.21	18.6	0.606	1.30	6.16	4.42
×$\frac{9}{16}$	$1\frac{3}{16}$	29.6	8.77	54.2	9.33	2.49	2.19	16.8	0.548	0.961	4.55	4.43
×$\frac{1}{2}$	$1\frac{1}{8}$	26.4	7.84	48.8	8.36	2.49	2.17	15.1	0.490	0.683	3.23	4.45
L8×6×1	$1\frac{1}{2}$	44.2	13.1	80.9	15.1	2.49	2.65	27.3	1.45	4.34	16.3	3.88
×$\frac{7}{8}$	$1\frac{3}{8}$	39.1	11.5	72.4	13.4	2.50	2.60	24.3	1.43	2.96	11.3	3.92
×$\frac{3}{4}$	$1\frac{1}{4}$	33.8	9.99	63.5	11.7	2.52	2.55	21.1	1.34	1.90	7.28	3.95
×$\frac{5}{8}$	$1\frac{1}{8}$	28.5	8.41	54.2	9.86	2.54	2.50	17.9	1.27	1.12	4.33	3.98
×$\frac{9}{16}$	$1\frac{1}{16}$	25.7	7.61	49.4	8.94	2.55	2.48	16.2	1.24	0.823	3.20	3.99
×$\frac{1}{2}$	1	23.0	6.80	44.4	8.01	2.55	2.46	14.6	1.20	0.584	2.28	4.01
×$\frac{7}{16}$	$\frac{15}{16}$	20.2	5.99	39.3	7.06	2.56	2.43	12.9	1.15	0.396	1.55	4.02
L8×4×1	$1\frac{1}{2}$	37.4	11.1	69.7	14.0	2.51	3.03	24.3	2.45	3.68	12.9	3.75
×$\frac{7}{8}$	$1\frac{3}{8}$	33.1	9.79	62.6	12.5	2.53	2.99	21.7	2.41	2.51	8.89	3.78
×$\frac{3}{4}$	$1\frac{1}{4}$	28.7	8.49	55.0	10.9	2.55	2.94	18.9	2.34	1.61	5.75	3.80
×$\frac{5}{8}$	$1\frac{1}{8}$	24.2	7.16	47.0	9.20	2.56	2.89	16.1	2.27	0.955	3.42	3.83
×$\frac{9}{16}$	$1\frac{1}{16}$	21.9	6.49	42.9	8.34	2.57	2.86	14.6	2.23	0.704	2.53	3.84
×$\frac{1}{2}$	1	19.6	5.80	38.6	7.48	2.58	2.84	13.1	2.20	0.501	1.80	3.86
×$\frac{7}{16}$	$\frac{15}{16}$	17.2	5.11	34.2	6.59	2.59	2.81	11.6	2.16	0.340	1.22	3.87
L7×4×$\frac{3}{4}$	$1\frac{1}{4}$	26.2	7.74	37.8	8.39	2.21	2.50	14.8	1.84	1.47	3.97	3.31
×$\frac{5}{8}$	$1\frac{1}{8}$	22.1	6.50	32.4	7.12	2.23	2.45	12.5	1.80	0.868	2.37	3.34
×$\frac{1}{2}$	1	17.9	5.26	26.6	5.79	2.25	2.40	10.2	1.74	0.456	1.25	3.37
×$\frac{7}{16}$	$\frac{15}{16}$	15.7	4.63	23.6	5.11	2.26	2.38	9.03	1.71	0.310	0.851	3.38
×$\frac{3}{8}$	$\frac{7}{8}$	13.6	4.00	20.5	4.42	2.27	2.35	7.81	1.67	0.198	0.544	3.40

Note: For workable gages, refer to Table 1-7A. For width-to-thickness criteria, refer to Table 1-7B.

Table 1-7 (continued)
Angles
Properties

L12–L7

Shape	Axis Y-Y						Axis Z-Z			
	I	S	r	$\bar{x}$	Z	x_p	I	S	r	Tan α
	in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	
L12×12×1^3/8	413	48.6	3.64	3.50	88.1	1.30	165	33.3	2.30	1.00
×1^1/4	381	44.6	3.66	3.45	80.7	1.18	152	31.1	2.31	1.00
×1^1/8	350	40.7	3.68	3.41	73.7	1.08	140	29.0	2.33	1.00
×1	315	36.5	3.70	3.36	65.9	0.958	126	26.5	2.34	1.00
L10×10×1^3/8	231	33.0	3.00	3.00	59.8	1.28	93.3	22.0	1.91	1.00
×1^1/4	213	30.2	3.02	2.95	54.9	1.17	85.4	20.5	1.91	1.00
×1^1/8	196	27.6	3.03	2.90	50.2	1.07	78.2	19.1	1.92	1.00
×1	177	24.8	3.05	2.86	45.0	0.950	70.4	17.4	1.92	1.00
×^{7}/8	158	21.9	3.07	2.80	39.9	0.840	62.8	15.9	1.93	1.00
×^{3}/4	139	19.2	3.10	2.76	34.6	0.725	55.7	14.3	1.96	1.00
L8×8×1^1/8	98.1	17.5	2.41	2.40	31.6	1.05	40.7	12.0	1.56	1.00
×1	89.1	15.8	2.43	2.36	28.5	0.944	36.8	11.0	1.56	1.00
×^{7}/8	79.7	14.0	2.45	2.31	25.3	0.831	32.7	10.0	1.57	1.00
×^{3}/4	69.9	12.2	2.46	2.26	22.0	0.719	28.5	8.91	1.57	1.00
×^{5}/8	59.6	10.3	2.48	2.21	18.6	0.606	24.2	7.73	1.58	1.00
×^{9}/16	54.2	9.33	2.49	2.19	16.8	0.548	21.9	7.06	1.58	1.00
×^{1}/2	48.8	8.36	2.49	2.17	15.1	0.490	19.8	6.45	1.59	1.00
L8×6×1	38.8	8.92	1.72	1.65	16.2	0.819	21.3	7.61	1.28	0.542
×^{7}/8	34.9	7.94	1.74	1.60	14.4	0.719	18.9	6.70	1.28	0.546
×^{3}/4	30.8	6.92	1.75	1.56	12.5	0.624	16.6	5.85	1.29	0.550
×^{5}/8	26.4	5.88	1.77	1.51	10.5	0.526	14.1	4.91	1.29	0.554
×^{9}/16	24.1	5.34	1.78	1.49	9.52	0.476	12.8	4.46	1.30	0.556
×^{1}/2	21.7	4.79	1.79	1.46	8.52	0.425	11.5	3.98	1.30	0.557
×^{7}/16	19.3	4.23	1.80	1.44	7.50	0.374	10.2	3.52	1.31	0.559
L8×4×1	11.6	3.94	1.03	1.04	7.73	0.694	7.83	3.45	0.844	0.247
×^{7}/8	10.5	3.51	1.04	0.997	6.77	0.612	6.97	3.04	0.846	0.252
×^{3}/4	9.37	3.07	1.05	0.949	5.82	0.531	6.14	2.65	0.850	0.257
×^{5}/8	8.11	2.62	1.06	0.902	4.86	0.448	5.24	2.24	0.856	0.262
×^{9}/16	7.44	2.38	1.07	0.878	4.39	0.406	4.78	2.03	0.859	0.264
×^{1}/2	6.75	2.15	1.08	0.854	3.91	0.363	4.32	1.82	0.863	0.266
×^{7}/16	6.03	1.90	1.09	0.829	3.42	0.319	3.84	1.61	0.867	0.268
L7×4×^{3}/4	9.00	3.01	1.08	1.00	5.60	0.553	5.63	2.56	0.855	0.324
×^{5}/8	7.79	2.56	1.10	0.958	4.69	0.464	4.81	2.17	0.860	0.329
×^{1}/2	6.48	2.10	1.11	0.910	3.77	0.376	3.94	1.75	0.866	0.334
×^{7}/16	5.79	1.86	1.12	0.886	3.31	0.331	3.50	1.55	0.869	0.337
×^{3}/8	5.06	1.61	1.12	0.861	2.84	0.286	3.04	1.33	0.873	0.339

Note: For workable gages, refer to Table 1-7A. For width-to-thickness criteria, refer to Table 1-7B.

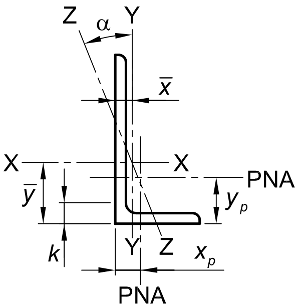


Table 1-7 (continued)
Angles
Properties

Shape	k	Wt.	Area, A	Axis X-X						Flexural-Torsional Properties		
				I	S	r	$\bar{y}$	Z	y_p	J	C_w	$\bar{r}_o$
	in.	lb/ft	in.^{2}	in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{6}	in.
L6×6×1	1 1/2	37.4	11.0	35.4	8.55	1.79	1.86	15.4	0.917	3.68	9.24	3.18
×7/8	1 3/8	33.1	9.75	31.9	7.61	1.81	1.81	13.7	0.813	2.51	6.41	3.21
×3/4	1 1/4	28.7	8.46	28.1	6.64	1.82	1.77	11.9	0.705	1.61	4.17	3.24
×5/8	1 1/8	24.2	7.13	24.1	5.64	1.84	1.72	10.1	0.594	0.955	2.50	3.28
×9/16	1 1/16	21.9	6.45	22.0	5.12	1.85	1.70	9.18	0.538	0.704	1.85	3.29
×1/2	1	19.6	5.77	19.9	4.59	1.86	1.67	8.22	0.481	0.501	1.32	3.31
×7/16	15/16	17.2	5.08	17.6	4.06	1.86	1.65	7.25	0.423	0.340	0.899	3.32
×3/8	7/8	14.9	4.38	15.4	3.51	1.87	1.62	6.27	0.365	0.218	0.575	3.34
×5/16	13/16	12.4	3.67	13.0	2.95	1.88	1.60	5.26	0.306	0.129	0.338	3.35
L6×4×7/8	1 3/8	27.2	8.00	27.7	7.13	1.86	2.12	12.7	1.43	2.03	4.04	2.82
×3/4	1 1/4	23.6	6.94	24.5	6.23	1.88	2.07	11.1	1.37	1.31	2.64	2.85
×5/8	1 1/8	20.0	5.86	21.0	5.29	1.89	2.03	9.44	1.31	0.775	1.59	2.88
×9/16	1 1/16	18.1	5.31	19.2	4.81	1.90	2.00	8.59	1.28	0.572	1.18	2.90
×1/2	1	16.2	4.75	17.3	4.31	1.91	1.98	7.71	1.25	0.407	0.843	2.91
×7/16	15/16	14.3	4.18	15.4	3.81	1.92	1.95	6.81	1.22	0.276	0.575	2.93
×3/8	7/8	12.3	3.61	13.4	3.30	1.93	1.93	5.89	1.19	0.177	0.369	2.94
×5/16	13/16	10.3	3.03	11.4	2.77	1.94	1.90	4.96	1.15	0.104	0.217	2.96
L6×3 1/2×1/2	1	15.3	4.50	16.6	4.23	1.92	2.07	7.49	1.50	0.386	0.779	2.88
×3/8	7/8	11.7	3.44	12.9	3.23	1.93	2.02	5.74	1.41	0.168	0.341	2.90
×5/16	13/16	9.80	2.89	10.9	2.72	1.94	2.00	4.84	1.38	0.0990	0.201	2.92
L5×5×7/8	1 3/8	27.2	8.00	17.8	5.16	1.49	1.56	9.31	0.800	2.07	3.53	2.64
×3/4	1 1/4	23.6	6.98	15.7	4.52	1.50	1.52	8.14	0.698	1.33	2.32	2.67
×5/8	1 1/8	20.0	5.90	13.6	3.85	1.52	1.47	6.93	0.590	0.792	1.40	2.70
×1/2	1	16.2	4.79	11.3	3.15	1.53	1.42	5.66	0.479	0.417	0.744	2.73
×7/16	15/16	14.3	4.22	10.0	2.78	1.54	1.40	5.00	0.422	0.284	0.508	2.74
×3/8	7/8	12.3	3.65	8.76	2.41	1.55	1.37	4.33	0.365	0.183	0.327	2.76
×5/16	13/16	10.3	3.07	7.44	2.04	1.56	1.35	3.65	0.307	0.108	0.193	2.77
L5×3 1/2×3/4	1 3/16	19.8	5.85	13.9	4.26	1.55	1.74	7.60	1.10	1.09	1.52	2.36
×5/8	1 1/16	16.8	4.93	12.0	3.63	1.56	1.69	6.50	1.06	0.651	0.918	2.39
×1/2	15/16	13.6	4.00	10.0	2.97	1.58	1.65	5.33	1.00	0.343	0.491	2.42
×3/8	13/16	10.4	3.05	7.75	2.28	1.59	1.60	4.09	0.933	0.150	0.217	2.45
×5/16	3/4	8.70	2.56	6.58	1.92	1.60	1.57	3.45	0.904	0.0883	0.128	2.47
×1/4	11/16	7.00	2.07	5.36	1.55	1.61	1.55	2.78	0.860	0.0464	0.0670	2.48

Note: For workable gages, refer to Table 1-7A. For width-to-thickness criteria, refer to Table 1-7B.

Table 1-7 (continued)
Angles
Properties

L6–L5

Shape	Axis Y-Y						Axis Z-Z			
	I	S	r	$\bar{x}$	Z	x_p	I	S	r	Tan α
	in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	
L6×6×1	35.4	8.55	1.79	1.86	15.4	0.917	14.9	5.67	1.17	1.00
×7/8	31.9	7.61	1.81	1.81	13.7	0.813	13.3	5.20	1.17	1.00
×3/4	28.1	6.64	1.82	1.77	11.9	0.705	11.6	4.64	1.17	1.00
×5/8	24.1	5.64	1.84	1.72	10.1	0.594	9.81	4.04	1.17	1.00
×9/16	22.0	5.12	1.85	1.70	9.18	0.538	8.90	3.71	1.18	1.00
×1/2	19.9	4.59	1.86	1.67	8.22	0.481	8.06	3.42	1.18	1.00
×7/16	17.6	4.06	1.86	1.65	7.25	0.423	7.05	3.03	1.18	1.00
×3/8	15.4	3.51	1.87	1.62	6.27	0.365	6.21	2.71	1.19	1.00
×5/16	13.0	2.95	1.88	1.60	5.26	0.306	5.20	2.30	1.19	1.00
L6×4×7/8	9.70	3.37	1.10	1.12	6.26	0.667	5.82	2.91	0.854	0.421
×3/4	8.63	2.95	1.12	1.07	5.42	0.578	5.08	2.50	0.856	0.428
×5/8	7.48	2.52	1.13	1.03	4.56	0.488	4.32	2.12	0.859	0.435
×9/16	6.86	2.29	1.14	1.00	4.13	0.443	3.93	1.91	0.861	0.438
×1/2	6.22	2.06	1.14	0.981	3.69	0.396	3.54	1.72	0.864	0.440
×7/16	5.56	1.83	1.15	0.957	3.24	0.348	3.14	1.51	0.867	0.443
×3/8	4.86	1.58	1.16	0.933	2.79	0.301	2.73	1.31	0.870	0.446
×5/16	4.13	1.34	1.17	0.908	2.33	0.253	2.31	1.09	0.874	0.449
L6×3 1/2×1/2	4.24	1.59	0.968	0.829	2.88	0.375	2.59	1.34	0.756	0.343
×3/8	3.33	1.22	0.984	0.781	2.18	0.287	2.01	1.02	0.763	0.349
×5/16	2.84	1.03	0.991	0.756	1.82	0.241	1.70	0.859	0.767	0.352
L5×5×7/8	17.8	5.16	1.49	1.56	9.31	0.800	7.60	3.44	0.971	1.00
×3/4	15.7	4.52	1.50	1.52	8.14	0.698	6.55	3.05	0.972	1.00
×5/8	13.6	3.85	1.52	1.47	6.93	0.590	5.62	2.70	0.975	1.00
×1/2	11.3	3.15	1.53	1.42	5.66	0.479	4.64	2.31	0.980	1.00
×7/16	10.0	2.78	1.54	1.40	5.00	0.422	4.04	2.04	0.983	1.00
×3/8	8.76	2.41	1.55	1.37	4.33	0.365	3.55	1.83	0.986	1.00
×5/16	7.44	2.04	1.56	1.35	3.65	0.307	3.00	1.57	0.990	1.00
L5×3 1/2×3/4	5.52	2.20	0.974	0.993	4.07	0.585	3.23	1.90	0.744	0.464
×5/8	4.80	1.88	0.987	0.947	3.43	0.493	2.74	1.59	0.746	0.472
×1/2	4.02	1.55	1.00	0.901	2.79	0.400	2.26	1.30	0.750	0.479
×3/8	3.15	1.19	1.02	0.854	2.12	0.305	1.73	0.983	0.755	0.485
×5/16	2.69	1.01	1.02	0.829	1.77	0.256	1.47	0.826	0.758	0.489
×1/4	2.20	0.816	1.03	0.804	1.42	0.207	1.19	0.665	0.761	0.491

Note: For workable gages, refer to Table 1-7A. For width-to-thickness criteria, refer to Table 1-7B.

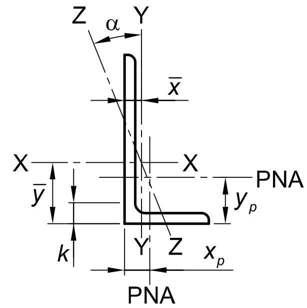


Table 1-7 (continued)
Angles
Properties

Shape	k	Wt.	Area, A	Axis X-X						Flexural-Torsional Properties		
				I	S	r	$\bar{y}$	Z	y_p	J	C_w	$\bar{r}_o$
	in.	lb/ft	in.^{2}	in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{6}	in.
L5×3×1/2	15/16	12.8	3.75	9.43	2.89	1.58	1.74	5.12	1.25	0.322	0.444	2.38
×7/16	7/8	11.3	3.31	8.41	2.56	1.59	1.72	4.53	1.22	0.220	0.304	2.39
×3/8	13/16	9.80	2.86	7.35	2.22	1.60	1.69	3.93	1.19	0.141	0.196	2.41
×5/16	3/4	8.20	2.41	6.24	1.87	1.61	1.67	3.32	1.14	0.0832	0.116	2.42
×1/4	11/16	6.60	1.94	5.09	1.51	1.62	1.64	2.68	1.12	0.0438	0.0606	2.43
L4×4×3/4	1 1/8	18.5	5.44	7.62	2.79	1.18	1.27	5.02	0.680	1.02	1.12	2.10
×5/8	1	15.7	4.61	6.62	2.38	1.20	1.22	4.28	0.576	0.610	0.680	2.13
×1/2	7/8	12.8	3.75	5.52	1.96	1.21	1.18	3.50	0.469	0.322	0.366	2.16
×7/16	13/16	11.3	3.30	4.93	1.73	1.22	1.15	3.10	0.413	0.220	0.252	2.18
×3/8	3/4	9.80	2.86	4.32	1.50	1.23	1.13	2.69	0.358	0.141	0.162	2.19
×5/16	11/16	8.20	2.40	3.67	1.27	1.24	1.11	2.26	0.300	0.0832	0.0963	2.21
×1/4	5/8	6.60	1.93	3.00	1.03	1.25	1.08	1.82	0.241	0.0438	0.0505	2.22
L4×3 1/2×1/2	7/8	11.9	3.50	5.30	1.92	1.23	1.24	3.46	0.500	0.301	0.302	2.03
×3/8	3/4	9.10	2.68	4.15	1.48	1.25	1.20	2.66	0.427	0.132	0.134	2.06
×5/16	11/16	7.70	2.25	3.53	1.25	1.25	1.17	2.24	0.400	0.0782	0.0798	2.08
×1/4	5/8	6.20	1.82	2.89	1.01	1.26	1.14	1.81	0.360	0.0412	0.0419	2.09
L4×3×5/8	1	13.6	3.99	6.01	2.28	1.23	1.37	4.08	0.808	0.529	0.472	1.91
×1/2	7/8	11.1	3.25	5.02	1.87	1.24	1.32	3.36	0.750	0.281	0.255	1.94
×3/8	3/4	8.50	2.49	3.94	1.44	1.26	1.27	2.60	0.680	0.123	0.114	1.97
×5/16	11/16	7.20	2.09	3.36	1.22	1.27	1.25	2.19	0.656	0.0731	0.0676	1.98
×1/4	5/8	5.80	1.69	2.75	0.988	1.27	1.22	1.77	0.620	0.0386	0.0356	1.99
L3 1/2×3 1/2×1/2	7/8	11.1	3.25	3.63	1.48	1.05	1.05	2.66	0.464	0.281	0.238	1.87
×7/16	13/16	9.80	2.89	3.25	1.32	1.06	1.03	2.36	0.413	0.192	0.164	1.89
×3/8	3/4	8.50	2.50	2.86	1.15	1.07	1.00	2.06	0.357	0.123	0.106	1.90
×5/16	11/16	7.20	2.10	2.44	0.969	1.08	0.979	1.74	0.300	0.0731	0.0634	1.92
×1/4	5/8	5.80	1.70	2.00	0.787	1.09	0.954	1.41	0.243	0.0386	0.0334	1.93
L3 1/2×3×1/2	7/8	10.2	3.02	3.45	1.45	1.07	1.12	2.61	0.480	0.260	0.191	1.75
×7/16	13/16	9.10	2.67	3.10	1.29	1.08	1.09	2.32	0.449	0.178	0.132	1.76
×3/8	3/4	7.90	2.32	2.73	1.12	1.09	1.07	2.03	0.407	0.114	0.0858	1.78
×5/16	11/16	6.60	1.95	2.33	0.951	1.09	1.05	1.72	0.380	0.0680	0.0512	1.79
×1/4	5/8	5.40	1.58	1.92	0.773	1.10	1.02	1.39	0.340	0.0360	0.0270	1.80
L3 1/2×2 1/2×1/2	7/8	9.40	2.77	3.24	1.41	1.08	1.20	2.52	0.730	0.234	0.159	1.66
×3/8	3/4	7.20	2.12	2.56	1.09	1.10	1.15	1.96	0.673	0.103	0.0714	1.69
×5/16	11/16	6.10	1.79	2.20	0.925	1.11	1.13	1.67	0.636	0.0611	0.0426	1.71
×1/4	5/8	4.90	1.45	1.81	0.753	1.12	1.10	1.36	0.600	0.0322	0.0225	1.72

Note: For workable gages, refer to Table 1-7A. For width-to-thickness criteria, refer to Table 1-7B.

Table 1-7 (continued)
Angles
Properties

L5–L3$^{1}/_{2}$

Shape	Axis Y-Y						Axis Z-Z			Tan α
	I	S	r	$\bar{x}$	Z	x_p	I	S	r	
	in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	
L5×3×1/2	2.55	1.13	0.824	0.746	2.08	0.375	1.55	0.957	0.642	0.357
×7/16	2.29	1.00	0.831	0.722	1.82	0.331	1.37	0.840	0.644	0.361
×3/8	2.01	0.874	0.838	0.698	1.57	0.286	1.20	0.727	0.646	0.364
×5/16	1.72	0.739	0.846	0.673	1.31	0.241	1.01	0.608	0.649	0.368
×1/4	1.41	0.600	0.853	0.648	1.05	0.194	0.825	0.491	0.652	0.371
L4×4×3/4	7.62	2.79	1.18	1.27	5.02	0.680	3.25	1.81	0.774	1.00
×5/8	6.62	2.38	1.20	1.22	4.28	0.576	2.76	1.60	0.774	1.00
×1/2	5.52	1.96	1.21	1.18	3.50	0.469	2.25	1.35	0.776	1.00
×7/16	4.93	1.73	1.22	1.15	3.10	0.413	1.99	1.22	0.777	1.00
×3/8	4.32	1.50	1.23	1.13	2.69	0.358	1.73	1.08	0.779	1.00
×5/16	3.67	1.27	1.24	1.11	2.26	0.300	1.46	0.930	0.781	1.00
×1/4	3.00	1.03	1.25	1.08	1.82	0.241	1.19	0.778	0.783	1.00
L4×3 1/2×1/2	3.76	1.50	1.04	0.994	2.69	0.438	1.79	1.16	0.716	0.750
×3/8	2.96	1.16	1.05	0.947	2.06	0.335	1.39	0.939	0.719	0.755
×5/16	2.52	0.980	1.06	0.923	1.74	0.281	1.16	0.806	0.721	0.757
×1/4	2.07	0.794	1.07	0.897	1.40	0.228	0.953	0.653	0.723	0.759
L4×3×5/8	2.85	1.34	0.845	0.867	2.45	0.499	1.59	1.13	0.631	0.534
×1/2	2.40	1.10	0.858	0.822	1.99	0.406	1.30	0.929	0.633	0.542
×3/8	1.89	0.851	0.873	0.775	1.52	0.311	1.00	0.699	0.636	0.551
×5/16	1.62	0.721	0.880	0.750	1.28	0.261	0.849	0.590	0.638	0.554
×1/4	1.33	0.585	0.887	0.725	1.03	0.211	0.692	0.474	0.639	0.558
L3 1/2×3 1/2×1/2	3.63	1.48	1.05	1.05	2.66	0.464	1.51	1.02	0.679	1.00
×7/16	3.25	1.32	1.06	1.03	2.36	0.413	1.33	0.911	0.681	1.00
×3/8	2.86	1.15	1.07	1.00	2.06	0.357	1.17	0.830	0.683	1.00
×5/16	2.44	0.969	1.08	0.979	1.74	0.300	0.984	0.713	0.685	1.00
×1/4	2.00	0.787	1.09	0.954	1.41	0.243	0.802	0.594	0.688	1.00
L3 1/2×3×1/2	2.32	1.09	0.877	0.869	1.97	0.431	1.15	0.846	0.618	0.713
×7/16	2.09	0.971	0.885	0.846	1.75	0.381	1.02	0.773	0.620	0.717
×3/8	1.84	0.847	0.892	0.823	1.52	0.331	0.894	0.693	0.622	0.720
×5/16	1.58	0.718	0.900	0.798	1.28	0.279	0.758	0.602	0.624	0.722
×1/4	1.30	0.585	0.908	0.773	1.04	0.226	0.622	0.486	0.628	0.725
L3 1/2×2 1/2×1/2	1.36	0.756	0.701	0.701	1.39	0.396	0.781	0.651	0.532	0.485
×3/8	1.09	0.589	0.716	0.655	1.07	0.303	0.609	0.499	0.535	0.495
×5/16	0.937	0.501	0.723	0.632	0.900	0.256	0.518	0.418	0.538	0.500
×1/4	0.775	0.410	0.731	0.607	0.728	0.207	0.426	0.341	0.541	0.504

Note: For workable gages, refer to Table 1-7A. For width-to-thickness criteria, refer to Table 1-7B.

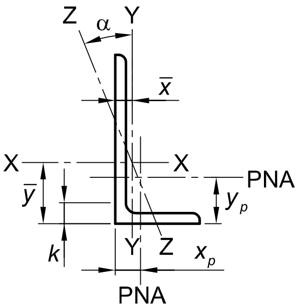


Table 1-7 (continued)
Angles
Properties

Shape	k	Wt.	Area, A	Axis X-X						Flexural-Torsional Properties		
				I	S	r	$\bar{y}$	Z	y_p	J	C_w	$\bar{r}_o$
	in.	lb/ft	in.^{2}	in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{6}	in.
L3×3×1/2	7/8	9.40	2.76	2.20	1.06	0.895	0.929	1.91	0.460	0.230	0.144	1.59
×7/16	13/16	8.30	2.43	1.98	0.946	0.903	0.907	1.70	0.405	0.157	0.100	1.60
×3/8	3/4	7.20	2.11	1.75	0.825	0.910	0.884	1.48	0.352	0.101	0.0652	1.62
×5/16	11/16	6.10	1.78	1.50	0.699	0.918	0.860	1.26	0.297	0.0597	0.0390	1.64
×1/4	5/8	4.90	1.44	1.23	0.569	0.926	0.836	1.02	0.240	0.0313	0.0206	1.65
×3/16	9/16	3.71	1.09	0.948	0.433	0.933	0.812	0.774	0.182	0.0136	0.00899	1.67
L3×2 1/2×1/2	7/8	8.50	2.50	2.07	1.03	0.910	0.995	1.86	0.500	0.213	0.112	1.46
×7/16	13/16	7.60	2.22	1.87	0.921	0.917	0.972	1.66	0.463	0.146	0.0777	1.48
×3/8	3/4	6.60	1.93	1.65	0.803	0.924	0.949	1.45	0.427	0.0943	0.0507	1.49
×5/16	11/16	5.60	1.63	1.41	0.681	0.932	0.925	1.23	0.392	0.0560	0.0304	1.51
×1/4	5/8	4.50	1.32	1.16	0.555	0.940	0.900	1.00	0.360	0.0296	0.0161	1.52
×3/16	9/16	3.39	1.00	0.899	0.423	0.947	0.874	0.761	0.333	0.0130	0.00705	1.54
L3×2×1/2	13/16	7.70	2.26	1.92	1.00	0.922	1.08	1.78	0.740	0.192	0.0908	1.39
×3/8	11/16	5.90	1.75	1.54	0.779	0.937	1.03	1.39	0.667	0.0855	0.0413	1.42
×5/16	5/8	5.00	1.48	1.32	0.662	0.945	1.01	1.19	0.632	0.0510	0.0248	1.43
×1/4	9/16	4.10	1.20	1.09	0.541	0.953	0.980	0.969	0.600	0.0270	0.0132	1.45
×3/16	1/2	3.07	0.917	0.847	0.414	0.961	0.952	0.743	0.555	0.0119	0.00576	1.46
L2 1/2×2 1/2×1/2	3/4	7.70	2.26	1.22	0.716	0.735	0.803	1.29	0.452	0.188	0.0791	1.30
×3/8	5/8	5.90	1.73	0.972	0.558	0.749	0.758	1.01	0.346	0.0833	0.0362	1.33
×5/16	9/16	5.00	1.46	0.837	0.474	0.756	0.735	0.853	0.292	0.0495	0.0218	1.35
×1/4	1/2	4.10	1.19	0.692	0.387	0.764	0.711	0.695	0.238	0.0261	0.0116	1.36
×3/16	7/16	3.07	0.901	0.535	0.295	0.771	0.687	0.529	0.180	0.0114	0.00510	1.38
L2 1/2×2×3/8	5/8	5.30	1.55	0.914	0.546	0.766	0.826	0.982	0.433	0.0746	0.0268	1.22
×5/16	9/16	4.50	1.32	0.790	0.465	0.774	0.803	0.839	0.388	0.0444	0.0162	1.23
×1/4	1/2	3.62	1.07	0.656	0.381	0.782	0.779	0.688	0.360	0.0235	0.00868	1.25
×3/16	7/16	2.75	0.818	0.511	0.293	0.790	0.754	0.529	0.319	0.0103	0.00382	1.26
L2 1/2×1 1/2×1/4	1/2	3.19	0.947	0.594	0.364	0.792	0.866	0.644	0.606	0.0209	0.00694	1.19
×3/16	7/16	2.44	0.724	0.464	0.280	0.801	0.839	0.497	0.569	0.00921	0.00306	1.20
L2×2×3/8	5/8	4.70	1.37	0.476	0.348	0.591	0.632	0.629	0.343	0.0658	0.0174	1.05
×5/16	9/16	3.92	1.16	0.414	0.298	0.598	0.609	0.537	0.290	0.0393	0.0106	1.06
×1/4	1/2	3.19	0.944	0.346	0.244	0.605	0.586	0.440	0.236	0.0209	0.00572	1.08
×3/16	7/16	2.44	0.722	0.271	0.188	0.612	0.561	0.338	0.181	0.00921	0.00254	1.09
×1/8	3/8	1.65	0.491	0.189	0.129	0.620	0.534	0.230	0.123	0.00293	0.000789	1.10

Note: For workable gages, refer to Table 1-7A. For width-to-thickness criteria, refer to Table 1-7B.

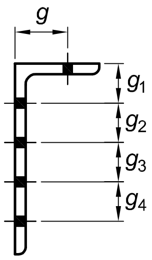
Table 1-7 (continued)
Angles
Properties

L3–L2

Shape	Axis Y-Y						Axis Z-Z			
	I	S	r	$\bar{x}$	Z	x_p	I	S	r	Tan α
	in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	
L3×3×1/2	2.20	1.06	0.895	0.929	1.91	0.460	0.922	0.704	0.580	1.00
×7/16	1.98	0.946	0.903	0.907	1.70	0.405	0.817	0.638	0.580	1.00
×3/8	1.75	0.825	0.910	0.884	1.48	0.352	0.716	0.573	0.581	1.00
×5/16	1.50	0.699	0.918	0.860	1.26	0.297	0.606	0.497	0.583	1.00
×1/4	1.23	0.569	0.926	0.836	1.02	0.240	0.490	0.415	0.585	1.00
×3/16	0.948	0.433	0.933	0.812	0.774	0.182	0.373	0.324	0.586	1.00
L3×2 1/2×1/2	1.29	0.736	0.718	0.746	1.34	0.417	0.665	0.568	0.516	0.666
×7/16	1.17	0.656	0.724	0.724	1.19	0.370	0.594	0.521	0.516	0.671
×3/8	1.03	0.573	0.731	0.701	1.03	0.322	0.514	0.463	0.517	0.675
×5/16	0.888	0.487	0.739	0.677	0.873	0.272	0.435	0.403	0.518	0.679
×1/4	0.734	0.397	0.746	0.653	0.707	0.220	0.355	0.329	0.520	0.683
×3/16	0.568	0.303	0.753	0.627	0.536	0.167	0.271	0.246	0.521	0.687
L3×2×1/2	0.667	0.470	0.543	0.580	0.887	0.377	0.409	0.411	0.425	0.413
×3/8	0.539	0.368	0.555	0.535	0.679	0.292	0.319	0.313	0.426	0.426
×5/16	0.467	0.314	0.562	0.511	0.572	0.247	0.271	0.263	0.428	0.432
×1/4	0.390	0.258	0.569	0.487	0.463	0.200	0.223	0.214	0.431	0.437
×3/16	0.305	0.198	0.577	0.462	0.351	0.153	0.173	0.163	0.435	0.442
L2 1/2×2 1/2×1/2	1.22	0.716	0.735	0.803	1.29	0.452	0.526	0.461	0.481	1.00
×3/8	0.972	0.558	0.749	0.758	1.01	0.346	0.400	0.374	0.481	1.00
×5/16	0.837	0.474	0.756	0.735	0.853	0.292	0.338	0.325	0.481	1.00
×1/4	0.692	0.387	0.764	0.711	0.695	0.238	0.276	0.273	0.482	1.00
×3/16	0.535	0.295	0.771	0.687	0.529	0.180	0.209	0.215	0.482	1.00
L2 1/2×2×3/8	0.513	0.361	0.574	0.578	0.657	0.310	0.273	0.295	0.419	0.612
×5/16	0.446	0.309	0.581	0.555	0.557	0.264	0.233	0.261	0.420	0.618
×1/4	0.372	0.253	0.589	0.532	0.454	0.214	0.192	0.213	0.423	0.624
×3/16	0.292	0.195	0.597	0.508	0.347	0.164	0.148	0.162	0.426	0.628
L2 1/2×1 1/2×1/4	0.160	0.142	0.411	0.372	0.261	0.189	0.0977	0.120	0.321	0.354
×3/16	0.126	0.110	0.418	0.347	0.198	0.145	0.0754	0.0906	0.324	0.360
L2×2×3/8	0.476	0.348	0.591	0.632	0.629	0.343	0.203	0.227	0.386	1.00
×5/16	0.414	0.298	0.598	0.609	0.537	0.290	0.172	0.200	0.386	1.00
×1/4	0.346	0.244	0.605	0.586	0.440	0.236	0.142	0.171	0.387	1.00
×3/16	0.271	0.188	0.612	0.561	0.338	0.181	0.109	0.137	0.389	1.00
×1/8	0.189	0.129	0.620	0.534	0.230	0.123	0.0756	0.100	0.391	1.00

Note: For workable gages, refer to Table 1-7A. For width-to-thickness criteria, refer to Table 1-7B.

Table 1-7A
Workable Gages in Angle Legs, in.



Leg	12	10	8	7	6	5	4	$3\frac{1}{2}$	3	$2\frac{1}{2}$	2	$1\frac{3}{4}$	$1\frac{1}{2}$	$1\frac{3}{8}$	$1\frac{1}{4}$	1
g	6	5	$4\frac{1}{2}$	4	$3\frac{1}{2}$	3	$2\frac{1}{2}$	2	$1\frac{3}{4}$	$1\frac{3}{8}$	$1\frac{1}{8}$	1	$\frac{7}{8}$	$\frac{7}{8}$	$\frac{3}{4}$	$\frac{5}{8}$
g_1	3	3	3	$2\frac{1}{2}$	$2\frac{1}{4}$	2	–	–	–	–	–	–	–	–	–	–
g_2	$2\frac{1}{2}$	$2\frac{1}{2}$	3	3	$2\frac{1}{2}$	2	–	–	–	–	–	–	–	–	–	–
g_3	$2\frac{1}{2}$	$2\frac{1}{2}$	–	–	–	–	–	–	–	–	–	–	–	–	–	–
g_4	$2\frac{1}{2}$	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–

Note: Other gages are permitted to suit specific requirements subject to clearances and edge distance limitations.

Table 1-7B
Width-to-Thickness Criteria for Angles

F_y = 36 ksi				F_y = 50 ksi			
t	Compression	Flexure		*t*	Compression	Flexure	
	Nonslender up to	Compact up to	Noncompact up to		Nonslender up to	Compact up to	Noncompact up to
	Width of angle leg, in.				Width of angle leg, in.		
1 3/8	12	12	–	1 3/8	12	12	–
1 1/4	↓	↓	–	1 1/4	↓	↓	–
1 1/8	↓	↓	–	1 1/8	↓	↓	–
1	↓	↓	–	1	10	↓	–
7/8	10	10	–	7/8	8	10	–
3/4	8	10	–	3/4	8	8	10
5/8	8	8	–	5/8	6	8	–
9/16	7	8	–	9/16	6	7	8
1/2	6	7	8	1/2	5	6	↓
7/16	5	6	↓	7/16	4	5	↓
3/8	4	5	↓	3/8	4	4	↓
5/16	4	4	6	5/16	3	4	6
1/4	3	3 1/2	5	1/4	2 1/2	3	5
3/16	2	2 1/2	3	3/16	2	2	3
1/8	1 1/2	1 1/2	2	1/8	1	1 1/2	2

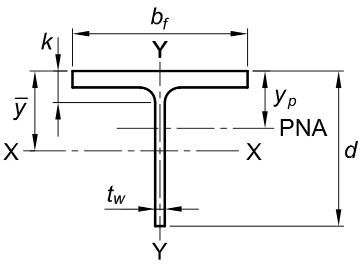


Table 1-8
WT-Shapes
Dimensions

Shape	Area, A	Depth, d		Stem: Thickness, t_w		Stem: $\frac{t_w}{2}$	Stem: Area	Flange: Width, b_f		Flange: Thickness, t_f		Distance: k: k_{des}	Distance: k: k_{det}	Distance: Workable Gage
	in.^{2}	in.		in.		in.	in.^{2}	in.		in.		in.	in.	in.
WT22×167.5[c]	49.2	22.0	22	1.03	1	$\frac{1}{2}$	22.6	15.9	16	1.77	$1\frac{3}{4}$	2.56	3	$5\frac{1}{2}$
×145[c]	42.6	21.8	$21\frac{3}{4}$	0.865	$\frac{7}{8}$	$\frac{7}{16}$	18.9	15.8	$15\frac{7}{8}$	1.58	$1\frac{9}{16}$	2.36	$2\frac{13}{16}$	
×131[c]	38.5	21.7	$21\frac{5}{8}$	0.785	$\frac{13}{16}$	$\frac{7}{16}$	17.0	15.8	$15\frac{3}{4}$	1.42	$1\frac{7}{16}$	2.20	$2\frac{5}{8}$	
×115[c,v]	33.9	21.5	$21\frac{1}{2}$	0.710	$\frac{11}{16}$	$\frac{3}{8}$	15.2	15.8	$15\frac{3}{4}$	1.22	$1\frac{1}{4}$	2.01	$2\frac{7}{16}$	
WT20×327.5[h]	96.4	21.8	$21\frac{3}{4}$	1.97	2	1	42.9	16.9	$16\frac{7}{8}$	3.54	$3\frac{9}{16}$	4.72	$4\frac{13}{16}$	$7\frac{1}{2}$
×296.5[h]	87.2	21.5	$21\frac{1}{2}$	1.79	$1\frac{13}{16}$	$\frac{15}{16}$	38.5	16.7	$16\frac{3}{4}$	3.23	$3\frac{1}{4}$	4.41	$4\frac{1}{2}$	
×251.5[h]	74.0	21.0	21	1.54	$1\frac{9}{16}$	$\frac{13}{16}$	32.3	16.4	$16\frac{3}{8}$	2.76	$2\frac{3}{4}$	3.94	4	
×215.5[h]	63.3	20.6	$20\frac{5}{8}$	1.34	$1\frac{5}{16}$	$\frac{11}{16}$	27.6	16.2	$16\frac{1}{4}$	2.36	$2\frac{3}{8}$	3.54	$3\frac{5}{8}$	
×198.5[h]	58.3	20.5	$20\frac{1}{2}$	1.22	$1\frac{1}{4}$	$\frac{5}{8}$	25.0	16.1	$16\frac{1}{8}$	2.20	$2\frac{3}{16}$	3.38	$3\frac{1}{2}$	
×186[h]	54.7	20.3	$20\frac{3}{8}$	1.16	$1\frac{3}{16}$	$\frac{5}{8}$	23.6	16.1	$16\frac{1}{8}$	2.05	$2\frac{1}{16}$	3.23	$3\frac{5}{16}$	
×181[c,h]	53.2	20.3	$20\frac{1}{4}$	1.12	$1\frac{1}{8}$	$\frac{9}{16}$	22.7	16.0	16	2.01	2	3.19	$3\frac{1}{4}$	
×162[c]	47.7	20.1	$20\frac{1}{8}$	1.00	1	$\frac{1}{2}$	20.1	15.9	$15\frac{7}{8}$	1.81	$1\frac{13}{16}$	2.99	$3\frac{1}{16}$	
×148.5[c]	43.6	19.9	$19\frac{7}{8}$	0.930	$\frac{15}{16}$	$\frac{1}{2}$	18.5	15.8	$15\frac{7}{8}$	1.65	$1\frac{5}{8}$	2.83	$2\frac{15}{16}$	
×138.5[c]	40.7	19.8	$19\frac{7}{8}$	0.830	$\frac{13}{16}$	$\frac{7}{16}$	16.5	15.8	$15\frac{7}{8}$	1.58	$1\frac{9}{16}$	2.76	$2\frac{7}{8}$	
×124.5[c]	36.7	19.7	$19\frac{3}{4}$	0.750	$\frac{3}{4}$	$\frac{3}{8}$	14.8	15.8	$15\frac{3}{4}$	1.42	$1\frac{7}{16}$	2.60	$2\frac{11}{16}$	
×107.5[c,v]	31.8	19.5	$19\frac{1}{2}$	0.650	$\frac{5}{8}$	$\frac{5}{16}$	12.7	15.8	$15\frac{3}{4}$	1.22	$1\frac{1}{4}$	2.40	$2\frac{1}{2}$	
×99.5[c,v]	29.2	19.3	$19\frac{3}{8}$	0.650	$\frac{5}{8}$	$\frac{5}{16}$	12.6	15.8	$15\frac{3}{4}$	1.07	$1\frac{1}{16}$	2.25	$2\frac{5}{16}$	
WT20×196[h]	57.8	20.8	$20\frac{3}{4}$	1.42	$1\frac{7}{16}$	$\frac{3}{4}$	29.4	12.4	$12\frac{3}{8}$	2.52	$2\frac{1}{2}$	3.70	$3\frac{13}{16}$	$7\frac{1}{2}$
×165.5[h]	48.8	20.4	$20\frac{3}{8}$	1.22	$1\frac{1}{4}$	$\frac{5}{8}$	24.9	12.2	$12\frac{1}{8}$	2.13	$2\frac{1}{8}$	3.31	$3\frac{3}{8}$	
×163.5[h]	47.9	20.4	$20\frac{3}{8}$	1.18	$1\frac{3}{16}$	$\frac{5}{8}$	24.1	12.1	$12\frac{1}{8}$	2.13	$2\frac{1}{8}$	3.31	$3\frac{3}{8}$	
×147[c]	43.1	20.2	$20\frac{1}{4}$	1.06	$1\frac{1}{16}$	$\frac{9}{16}$	21.4	12.0	12	1.93	$1\frac{15}{16}$	3.11	$3\frac{3}{16}$	
×139[c]	41.0	20.1	$20\frac{1}{8}$	1.03	1	$\frac{1}{2}$	20.6	12.0	12	1.81	$1\frac{13}{16}$	2.99	$3\frac{1}{16}$	
×132[c]	38.7	20.0	20	0.960	$\frac{15}{16}$	$\frac{1}{2}$	19.2	11.9	$11\frac{7}{8}$	1.73	$1\frac{3}{4}$	2.91	3	
×117.5[c]	34.6	19.8	$19\frac{7}{8}$	0.830	$\frac{13}{16}$	$\frac{7}{16}$	16.5	11.9	$11\frac{7}{8}$	1.58	$1\frac{9}{16}$	2.76	$2\frac{7}{8}$	
×105.5[c]	31.1	19.7	$19\frac{5}{8}$	0.750	$\frac{3}{4}$	$\frac{3}{8}$	14.8	11.8	$11\frac{3}{4}$	1.42	$1\frac{7}{16}$	2.60	$2\frac{11}{16}$	
×91.5[c,v]	26.7	19.5	$19\frac{1}{2}$	0.650	$\frac{5}{8}$	$\frac{5}{16}$	12.7	11.8	$11\frac{3}{4}$	1.20	$1\frac{3}{16}$	2.38	$2\frac{1}{2}$	
×83.5[c,v]	24.5	19.3	$19\frac{1}{4}$	0.650	$\frac{5}{8}$	$\frac{5}{16}$	12.5	11.8	$11\frac{3}{4}$	1.03	1	2.21	$2\frac{5}{16}$	
×74.5[c,v]	21.9	19.1	$19\frac{1}{8}$	0.630	$\frac{5}{8}$	$\frac{5}{16}$	12.0	11.8	$11\frac{3}{4}$	0.830	$\frac{13}{16}$	2.01	$2\frac{1}{8}$	

[c] Shape is slender for compression with $F_y = 50$ ksi.
[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
[v] Shear strength controlled by buckling effects ($C_{v2} < 1.0$) with $F_y = 50$ ksi.

Table 1-8 (continued)
WT-Shapes
Properties

WT22–WT20

Nom-inal Wt.	Compact Section Criteria		Axis X-X						Axis Y-Y				Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{d}{t_w}$	I	S	r	$\bar{y}$	Z	y_p	I	S	r	Z	J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{6}
167.5	4.50	21.4	2170	131	6.63	5.53	234	1.54	600	75.2	3.49	118	37.2	438
145	5.02	25.2	1830	111	6.54	5.26	196	1.35	521	65.9	3.49	102	25.4	275
131	5.57	27.6	1640	99.4	6.53	5.19	176	1.22	462	58.6	3.47	90.9	18.6	200
115	6.45	30.3	1440	88.6	6.53	5.17	157	1.07	398	50.5	3.43	78.3	12.4	139
327.5	2.39	11.1	3730	234	6.22	5.85	426	2.85	1440	170	3.86	271	293	3190
296.5	2.58	12.0	3310	209	6.16	5.66	379	2.61	1260	151	3.80	240	221	2340
251.5	2.98	13.6	2730	174	6.07	5.38	314	2.25	1020	124	3.72	197	138	1400
215.5	3.44	15.4	2290	148	6.01	5.18	266	1.95	843	104	3.65	164	88.2	881
198.5	3.66	16.8	2070	134	5.96	5.03	240	1.81	771	95.7	3.63	150	70.6	677
186	3.93	17.5	1930	126	5.95	4.98	225	1.70	709	88.3	3.60	138	57.7	558
181	3.99	18.1	1870	122	5.92	4.91	217	1.66	691	86.3	3.60	135	54.2	511
162	4.40	20.1	1650	108	5.88	4.77	192	1.50	609	76.6	3.57	119	39.6	362
148.5	4.80	21.4	1500	98.9	5.87	4.71	176	1.38	546	69.0	3.54	107	30.5	279
138.5	5.03	23.9	1360	88.6	5.78	4.50	157	1.29	522	65.9	3.58	102	25.7	218
124.5	5.55	26.3	1210	79.4	5.75	4.41	140	1.16	463	58.8	3.55	90.8	19.0	158
107.5	6.45	30.0	1030	68.0	5.71	4.28	120	1.01	398	50.5	3.54	77.8	12.4	101
99.5	7.39	29.7	988	66.5	5.81	4.47	117	0.929	347	44.1	3.45	68.2	9.12	83.5
196	2.45	14.6	2270	153	6.27	5.94	275	2.33	401	64.9	2.64	106	85.4	796
165.5	2.86	16.7	1880	128	6.21	5.74	231	2.00	322	52.9	2.57	85.7	52.5	484
163.5	2.85	17.3	1840	125	6.19	5.66	224	1.98	320	52.7	2.58	85.0	51.4	449
147	3.11	19.1	1630	111	6.14	5.51	199	1.80	281	46.7	2.55	75.0	38.2	322
139	3.31	19.5	1550	106	6.14	5.51	191	1.71	261	43.5	2.52	69.9	32.4	282
132	3.45	20.8	1450	99.2	6.11	5.41	178	1.63	246	41.3	2.52	66.0	27.9	233
117.5	3.77	23.9	1260	85.7	6.04	5.17	153	1.45	222	37.3	2.54	59.0	20.6	156
105.5	4.17	26.3	1120	76.7	6.01	5.08	137	1.31	195	33.0	2.51	52.1	15.2	113
91.5	4.92	30.0	955	65.7	5.98	4.97	117	1.13	165	28.0	2.49	44.0	9.65	71.2
83.5	5.76	29.7	899	63.7	6.05	5.19	115	1.10	141	23.9	2.40	37.8	6.99	62.9
74.5	7.11	30.3	815	59.7	6.10	5.45	108	1.72	114	19.4	2.29	30.9	4.66	51.9

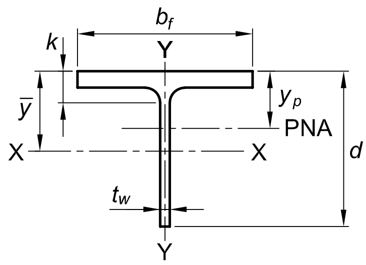


Table 1-8 (continued)
WT-Shapes
Dimensions

Shape	Area, A	Depth, d		Stem				Flange				Distance		
				Thickness, t_w		$\frac{t_w}{2}$	Area	Width, b_f		Thickness, t_f		k		Workable Gage
												k_{des}	k_{det}	
	in.^{2}	in.		in.		in.	in.^{2}	in.		in.		in.	in.	in.
WT18×462.5[h]	136	21.6	21 5/8	3.02	3	1 1/2	65.2	18.6	18 5/8	4.53	4 1/2	5.28	5 3/8	7 1/2
×426.5[h]	126	21.6	21 5/8	2.52	2 1/2	1 1/4	54.4	18.2	18 1/4	4.53	4 1/2	5.28	5 3/8	
×401[h]	118	21.3	21 1/4	2.38	2 3/8	1 3/16	50.7	18.0	18	4.29	4 5/16	5.04	5 1/8	
×361.5[h]	107	20.9	20 7/8	2.17	2 3/16	1 1/8	45.4	17.8	17 3/4	3.90	3 7/8	4.65	4 11/16	
×326[h]	96.2	20.5	20 1/2	1.97	2	1	40.4	17.6	17 5/8	3.54	3 9/16	4.49	4 13/16	
×264.5[h]	77.8	19.9	19 7/8	1.61	1 5/8	13/16	32.0	17.2	17 1/4	2.91	2 15/16	3.86	4 3/16	
×243.5[h]	71.7	19.7	19 5/8	1.50	1 1/2	3/4	29.5	17.1	17 1/8	2.68	2 11/16	3.63	4	
×220.5[h]	64.9	19.4	19 3/8	1.36	1 3/8	11/16	26.4	17.0	17	2.44	2 7/16	3.39	3 3/4	
×197.5[h]	58.1	19.2	19 1/4	1.22	1 1/4	5/8	23.4	16.8	16 7/8	2.20	2 3/16	3.15	3 7/16	
×180.5[h]	53.0	19.0	19	1.12	1 1/8	9/16	21.3	16.7	16 3/4	2.01	2	2.96	3 5/16	
×165[c]	48.4	18.8	18 7/8	1.02	1	1/2	19.2	16.6	16 5/8	1.85	1 7/8	2.80	3 1/8	
×151[c]	44.5	18.7	18 5/8	0.945	15/16	1/2	17.6	16.7	16 5/8	1.68	1 11/16	2.63	3	
×141[c]	41.5	18.6	18 1/2	0.885	7/8	7/16	16.4	16.6	16 5/8	1.57	1 9/16	2.52	2 7/8	
×131[c]	38.5	18.4	18 3/8	0.840	13/16	7/16	15.5	16.6	16 1/2	1.44	1 7/16	2.39	2 3/4	
×123.5[c]	36.3	18.3	18 3/8	0.800	13/16	7/16	14.7	16.5	16 1/2	1.35	1 3/8	2.30	2 5/8	
×115.5[c]	34.1	18.2	18 1/4	0.760	3/4	3/8	13.9	16.5	16 1/2	1.26	1 1/4	2.21	2 9/16	
WT18×128[c]	37.6	18.7	18 3/4	0.960	15/16	1/2	18.0	12.2	12 1/4	1.73	1 3/4	2.48	2 15/16	5 1/2
×116[c]	34.0	18.6	18 1/2	0.870	7/8	7/16	16.1	12.1	12 1/8	1.57	1 9/16	2.32	2 13/16	
×105[c]	30.9	18.3	18 3/8	0.830	13/16	7/16	15.2	12.2	12 1/8	1.36	1 3/8	2.11	2 5/8	
×97[c]	28.5	18.2	18 1/4	0.765	3/4	3/8	14.0	12.1	12 1/8	1.26	1 1/4	2.01	2 1/2	
×91[c]	26.8	18.2	18 1/8	0.725	3/4	3/8	13.2	12.1	12 1/8	1.18	1 3/16	1.93	2 3/8	
×85[c]	25.0	18.1	18 1/8	0.680	11/16	3/8	12.3	12.0	12	1.10	1 1/8	1.85	2 3/8	
×80[c]	23.5	18.0	18	0.650	5/8	5/16	11.7	12.0	12	1.02	1	1.77	2 1/4	
×75[c]	22.1	17.9	17 7/8	0.625	5/8	5/16	11.2	12.0	12	0.940	15/16	1.69	2 3/16	
×67.5[c,v]	19.9	17.8	17 3/4	0.600	5/8	5/16	10.7	12.0	12	0.790	13/16	1.54	2 1/16	
WT16.5×193.5[h]	57.0	18.0	18	1.26	1 1/4	5/8	22.6	16.2	16 1/4	2.28	2 1/4	3.07	3 9/16	5 1/2
×177[h]	52.1	17.8	17 3/4	1.16	1 3/16	5/8	20.6	16.1	16 1/8	2.09	2 1/16	2.88	3 3/8	
×159	46.8	17.6	17 5/8	1.04	1 1/16	9/16	18.3	16.0	16	1.89	1 7/8	2.68	3 3/16	
×145.5[c]	42.8	17.4	17 3/8	0.960	15/16	1/2	16.7	15.9	15 7/8	1.73	1 3/4	2.52	2 15/16	
×131.5[c]	38.7	17.3	17 1/4	0.870	7/8	7/16	15.0	15.8	15 3/4	1.57	1 9/16	2.36	2 13/16	
×120.5[c]	35.6	17.1	17 1/8	0.830	13/16	7/16	14.2	15.9	15 7/8	1.40	1 3/8	2.19	2 11/16	
×110.5[c]	32.6	17.0	17	0.775	3/4	3/8	13.1	15.8	15 3/4	1.28	1 1/4	2.06	2 1/2	
×100.5[c]	29.7	16.8	16 7/8	0.715	11/16	3/8	12.0	15.7	15 3/4	1.15	1 1/8	1.94	2 7/16	

[c] Shape is slender for compression with F_y = 50 ksi.
[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
[v] Shear strength controlled by buckling effects (C_{v2} < 1.0) with F_y = 50 ksi.

Table 1-8 (continued)
WT-Shapes
Properties

WT18–WT16.5

Nominal Wt.	Compact Section Criteria		Axis X-X						Axis Y-Y				Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{d}{t_w}$	I	S	r	$\bar{y}$	Z	y_p	I	S	r	Z	J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{6}
462.5	2.05	7.15	5130	337	6.14	6.36	617	3.66	2470	266	4.26	431	707	9680
426.5	2.01	8.57	4480	286	5.96	5.95	533	3.46	2300	253	4.27	403	615	7100
401	2.10	8.95	4110	265	5.90	5.80	491	3.28	2100	233	4.22	372	519	5830
361.5	2.28	9.63	3610	235	5.81	5.55	434	3.01	1850	208	4.16	329	390	4250
326	2.48	10.4	3160	208	5.74	5.35	383	2.73	1610	184	4.10	290	295	3070
264.5	2.96	12.4	2440	164	5.60	4.96	298	2.26	1240	145	4.00	227	163	1600
243.5	3.19	13.1	2220	150	5.57	4.84	272	2.10	1120	131	3.96	206	128	1250
220.5	3.48	14.3	1980	134	5.52	4.69	242	1.91	997	117	3.92	184	96.6	914
197.5	3.83	15.7	1740	119	5.47	4.53	213	1.73	877	104	3.88	162	70.7	652
180.5	4.16	17.0	1570	107	5.43	4.42	192	1.59	786	94.0	3.85	146	54.1	491
165	4.49	18.4	1410	97.0	5.39	4.30	173	1.46	711	85.5	3.83	132	42.0	372
151	4.96	19.8	1280	88.8	5.37	4.22	158	1.33	648	77.8	3.82	120	32.1	285
141	5.29	21.0	1190	82.6	5.36	4.16	146	1.25	599	72.2	3.80	112	26.3	231
131	5.75	21.9	1110	77.5	5.36	4.14	137	1.16	545	65.8	3.76	102	20.8	185
123.5	6.11	22.9	1040	73.3	5.36	4.12	129	1.10	507	61.4	3.74	94.8	17.3	155
115.5	6.54	23.9	978	69.1	5.36	4.10	122	1.03	470	57.0	3.71	88.0	14.3	129
128	3.53	19.5	1210	87.4	5.66	4.92	156	1.54	264	43.2	2.65	68.5	26.4	205
116	3.86	21.4	1080	78.5	5.63	4.82	140	1.40	234	38.6	2.62	60.9	19.7	151
105	4.48	22.0	985	73.1	5.65	4.87	131	1.27	206	33.8	2.58	53.4	13.9	119
97	4.81	23.8	901	67.0	5.62	4.80	120	1.18	187	30.9	2.56	48.8	11.1	92.7
91	5.12	25.1	845	63.1	5.62	4.77	113	1.11	174	28.8	2.55	45.3	9.20	77.6
85	5.47	26.6	786	58.9	5.61	4.73	105	1.04	160	26.6	2.53	41.8	7.51	63.2
80	5.88	27.7	740	55.8	5.61	4.74	100	0.980	147	24.6	2.50	38.6	6.17	53.6
75	6.37	28.6	698	53.1	5.62	4.78	95.5	0.923	135	22.5	2.47	35.4	5.04	46.0
67.5	7.56	29.7	637	49.7	5.66	4.96	90.1	1.23	113	18.9	2.38	29.8	3.48	37.3
193.5	3.55	14.3	1460	107	5.07	4.27	193	1.76	810	100	3.77	156	73.9	615
177	3.85	15.3	1320	96.8	5.03	4.15	174	1.62	729	90.6	3.74	141	57.1	468
159	4.23	16.9	1160	85.8	4.99	4.02	154	1.46	645	80.7	3.71	125	42.1	335
145.5	4.60	18.1	1060	78.3	4.96	3.93	140	1.35	581	73.1	3.68	113	32.5	256
131.5	5.03	19.9	943	70.2	4.93	3.83	125	1.23	517	65.5	3.65	101	24.3	188
120.5	5.66	20.6	872	65.8	4.96	3.84	116	1.12	466	58.8	3.62	90.8	18.0	146
110.5	6.20	21.9	799	60.8	4.95	3.81	107	1.03	420	53.2	3.59	82.1	13.9	113
100.5	6.85	23.5	725	55.5	4.95	3.77	97.8	0.940	375	47.6	3.56	73.3	10.4	84.9

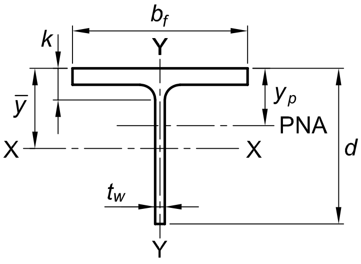


Table 1-8 (continued)
WT-Shapes
Dimensions

Shape	Area, A	Depth, d		Stem: Thickness, t_w		Stem: $\frac{t_w}{2}$	Stem: Area	Flange: Width, b_f		Flange: Thickness, t_f		Distance: k, k_{des}	Distance: k, k_{det}	Distance: Workable Gage
	in.^{2}	in.		in.		in.	in.^{2}	in.		in.		in.	in.	in.
WT16.5×84.5[c]	24.7	16.9	16 7/8	0.670	11/16	3/8	11.3	11.5	11 1/2	1.22	1 1/4	1.92	2 7/16	5 1/2
×76[c]	22.5	16.7	16 3/4	0.635	5/8	5/16	10.6	11.6	11 5/8	1.06	1 1/16	1.76	2 5/16	
×70.5[c]	20.7	16.7	16 5/8	0.605	5/8	5/16	10.1	11.5	11 1/2	0.960	15/16	1.66	2 3/16	
×65[c]	19.1	16.5	16 1/2	0.580	9/16	5/16	9.60	11.5	11 1/2	0.855	7/8	1.56	2 1/8	
×59[c,v]	17.4	16.4	16 3/8	0.550	9/16	5/16	9.04	11.5	11 1/2	0.740	3/4	1.44	2	
WT15×195.5[h]	57.6	16.6	16 5/8	1.36	1 3/8	11/16	22.6	15.6	15 5/8	2.44	2 7/16	3.23	3 3/4	5 1/2
×178.5[h]	52.5	16.4	16 3/8	1.24	1 1/4	5/8	20.3	15.5	15 1/2	2.24	2 1/4	3.03	3 1/2	
×163[h]	48.0	16.2	16 1/4	1.14	1 1/8	9/16	18.5	15.4	15 3/8	2.05	2 1/16	2.84	3 5/16	
×146	43.0	16.0	16	1.02	1	1/2	16.3	15.3	15 1/4	1.85	1 7/8	2.64	3 1/8	
×130.5	38.5	15.8	15 3/4	0.930	15/16	1/2	14.7	15.2	15 1/8	1.65	1 5/8	2.44	2 15/16	
×117.5[c]	34.7	15.7	15 5/8	0.830	13/16	7/16	13.0	15.1	15	1.50	1 1/2	2.29	2 3/4	
×105.5[c]	31.1	15.5	15 1/2	0.775	3/4	3/8	12.0	15.1	15 1/8	1.32	1 5/16	2.10	2 9/16	
×95.5[c]	28.0	15.3	15 3/8	0.710	11/16	3/8	10.9	15.0	15	1.19	1 3/16	1.97	2 1/2	
×86.5[c]	25.4	15.2	15 1/4	0.655	5/8	5/16	10.0	15.0	15	1.07	1 1/16	1.85	2 5/16	
WT15×74[c]	21.8	15.3	15 3/8	0.650	5/8	5/16	10.0	10.5	10 1/2	1.18	1 3/16	1.83	2 1/2	5 1/2
×66[c]	19.5	15.2	15 1/8	0.615	5/8	5/16	9.32	10.5	10 1/2	1.00	1	1.65	2 1/4	
×62[c]	18.2	15.1	15 1/8	0.585	9/16	5/16	8.82	10.5	10 1/2	0.930	15/16	1.58	2 1/4	
×58[c]	17.1	15.0	15	0.565	9/16	5/16	8.48	10.5	10 1/2	0.850	7/8	1.50	2 1/8	
×54[c]	15.9	14.9	14 7/8	0.545	9/16	5/16	8.13	10.5	10 1/2	0.760	3/4	1.41	2	
×49.5[c]	14.5	14.8	14 7/8	0.520	1/2	1/4	7.71	10.5	10 1/2	0.670	11/16	1.32	2	
×45[c,v]	13.2	14.8	14 3/4	0.470	1/2	1/4	6.94	10.4	10 3/8	0.610	5/8	1.26	1 7/8	
WT13.5×269.5[h]	79.3	16.3	16 1/4	1.97	2	1	32.0	15.3	15 1/4	3.54	3 9/16	4.33	4 7/16	5 1/2[g]
×184[h]	54.2	15.2	15 1/4	1.38	1 3/8	11/16	21.0	14.7	14 5/8	2.48	2 1/2	3.27	3 11/16	5 1/2
×168[h]	49.5	15.0	15	1.26	1 1/4	5/8	18.9	14.6	14 1/2	2.28	2 1/4	3.07	3 1/2	
×153.5[h]	45.2	14.8	14 3/4	1.16	1 3/16	5/8	17.2	14.4	14 1/2	2.09	2 1/16	2.88	3 5/16	
×140.5	41.5	14.6	14 5/8	1.06	1 1/16	9/16	15.5	14.4	14 3/8	1.93	1 15/16	2.72	3 1/8	
×129	38.1	14.5	14 1/2	0.980	1	1/2	14.2	14.3	14 1/4	1.77	1 3/4	2.56	3	
×117.5	34.7	14.3	14 3/8	0.910	15/16	1/2	13.0	14.2	14 1/4	1.61	1 5/8	2.40	2 7/8	
×108.5	32.0	14.2	14 1/4	0.830	13/16	7/16	11.8	14.1	14 1/8	1.50	1 1/2	2.29	2 11/16	
×97[c]	28.6	14.1	14	0.750	3/4	3/8	10.5	14.0	14	1.34	1 5/16	2.13	2 9/16	
×89[c]	26.3	13.9	13 7/8	0.725	3/4	3/8	10.1	14.1	14 1/8	1.19	1 3/16	1.98	2 3/8	
×80.5[c]	23.8	13.8	13 3/4	0.660	11/16	3/8	9.10	14.0	14	1.08	1 1/16	1.87	2 5/16	
×73[c]	21.6	13.7	13 3/4	0.605	5/8	5/16	8.28	14.0	14	0.975	1	1.76	2 3/16	

[c] Shape is slender for compression with $F_y = 50$ ksi.
[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.
[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
[v] Shear strength controlled by buckling effects ($C_{v2} < 1.0$) with $F_y = 50$ ksi.

Table 1-8 (continued)
WT-Shapes
Properties

WT16.5–WT13.5

Nominal Wt.	Compact Section Criteria		Axis X-X						Axis Y-Y				Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{d}{t_w}$	I	S	r	$\bar{y}$	Z	y_p	I	S	r	Z	J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{6}
84.5	4.71	25.2	649	51.1	5.12	4.21	90.8	1.08	155	27.0	2.50	42.1	8.81	55.4
76	5.48	26.3	592	47.4	5.14	4.26	84.5	0.967	136	23.6	2.47	36.9	6.16	43.0
70.5	6.01	27.6	552	44.7	5.15	4.29	79.8	0.901	123	21.3	2.43	33.4	4.84	35.4
65	6.73	28.4	513	42.1	5.18	4.36	75.6	0.832	109	18.9	2.38	29.7	3.67	29.3
59	7.76	29.8	469	39.2	5.20	4.47	70.8	0.862	93.5	16.3	2.32	25.6	2.64	23.4
195.5	3.19	12.2	1220	96.9	4.61	4.00	177	1.85	774	99.2	3.67	155	86.3	636
178.5	3.45	13.2	1090	87.2	4.56	3.87	159	1.70	693	89.6	3.64	140	66.6	478
163	3.75	14.2	981	78.8	4.52	3.76	143	1.56	622	81.0	3.60	126	51.2	361
146	4.12	15.7	861	69.6	4.48	3.62	125	1.41	549	71.9	3.58	111	37.5	257
130.5	4.59	17.0	765	62.4	4.46	3.54	112	1.27	480	63.3	3.53	97.9	26.9	184
117.5	5.02	18.9	674	55.1	4.41	3.41	98.2	1.15	427	56.8	3.51	87.5	20.1	133
105.5	5.74	20.0	610	50.5	4.43	3.39	89.5	1.03	378	50.1	3.49	77.2	14.1	96.4
95.5	6.35	21.5	549	45.7	4.42	3.34	80.8	0.935	336	44.7	3.46	68.9	10.5	71.2
86.5	7.04	23.2	497	41.7	4.42	3.31	73.5	0.851	299	39.9	3.42	61.4	7.78	53.0
74	4.44	23.5	466	40.6	4.63	3.84	72.2	1.04	114	21.7	2.28	33.9	7.24	37.6
66	5.27	24.7	421	37.4	4.66	3.90	66.8	0.921	98.0	18.6	2.25	29.2	4.85	28.5
62	5.65	25.8	396	35.3	4.66	3.90	63.1	0.867	90.4	17.2	2.23	27.0	3.98	23.9
58	6.17	26.5	373	33.7	4.67	3.94	60.4	0.815	82.1	15.6	2.19	24.6	3.21	20.5
54	6.89	27.3	349	32.0	4.69	4.01	57.7	0.757	73.0	13.9	2.15	21.9	2.49	17.3
49.5	7.80	28.5	322	30.0	4.71	4.09	54.4	0.912	63.9	12.2	2.10	19.3	1.88	14.3
45	8.52	31.5	290	27.1	4.69	4.04	49.0	0.835	57.3	11.0	2.09	17.3	1.41	10.5
269.5	2.15	8.30	1530	128	4.39	4.34	242	2.60	1060	138	3.65	218	247	1740
184	2.96	11.0	939	81.7	4.16	3.71	151	1.85	655	89.3	3.48	140	84.5	532
168	3.19	11.9	839	73.4	4.12	3.58	135	1.70	587	80.8	3.45	126	65.4	401
153.5	3.46	12.8	753	66.4	4.08	3.47	121	1.56	527	72.9	3.41	113	50.5	304
140.5	3.72	13.8	677	59.9	4.04	3.35	109	1.44	477	66.4	3.39	103	39.6	232
129	4.03	14.8	613	54.7	4.02	3.27	98.9	1.33	430	60.2	3.36	93.3	30.7	178
117.5	4.41	15.7	556	50.0	4.00	3.20	89.9	1.22	384	54.2	3.33	83.8	23.4	135
108.5	4.71	17.1	502	45.2	3.96	3.10	81.1	1.13	352	49.9	3.32	77.0	18.8	105
97	5.24	18.8	444	40.3	3.94	3.02	71.8	1.02	309	44.1	3.29	67.8	13.5	74.3
89	5.92	19.2	414	38.2	3.97	3.04	67.7	0.932	278	39.4	3.25	60.8	10.0	57.7
80.5	6.49	20.9	372	34.4	3.95	2.98	60.8	0.849	248	35.4	3.23	54.5	7.53	42.7
73	7.16	22.6	336	31.2	3.95	2.94	55.0	0.772	222	31.7	3.20	48.8	5.62	31.7

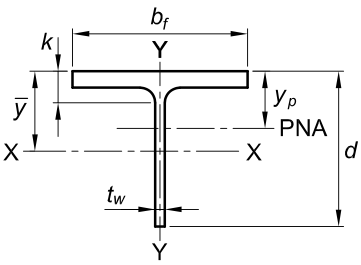
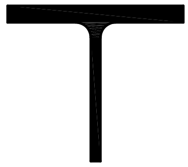


Table 1-8 (continued)
WT-Shapes
Dimensions

Shape	Area, A	Depth, d		Stem: Thickness, t_w		Stem: $\frac{t_w}{2}$	Stem: Area	Flange: Width, b_f		Flange: Thickness, t_f		Distance: k, k_{des}	Distance: k, k_{det}	Distance: Workable Gage
	in.^{2}	in.		in.		in.	in.^{2}	in.		in.		in.	in.	in.
WT13.5×64.5[c]	18.9	13.8	13 7/8	0.610	5/8	5/16	8.43	10.0	10	1.10	1 1/8	1.70	2 5/16	5 1/2
×57[c]	16.8	13.6	13 5/8	0.570	9/16	5/16	7.78	10.1	10 1/8	0.930	15/16	1.53	2 1/8	
×51[c]	15.0	13.5	13 1/2	0.515	1/2	1/4	6.98	10.0	10	0.830	13/16	1.43	2 1/16	
×47[c]	13.8	13.5	13 1/2	0.490	1/2	1/4	6.60	10.0	10	0.745	3/4	1.34	1 15/16	
×42[c]	12.4	13.4	13 3/8	0.460	7/16	1/4	6.14	10.0	10	0.640	5/8	1.24	1 7/8	
WT12×185[h]	54.5	14.0	14	1.52	1 1/2	3/4	21.3	13.7	13 5/8	2.72	2 3/4	3.22	4	5 1/2
×167.5[h]	49.1	13.8	13 3/4	1.38	1 3/8	11/16	19.0	13.5	13 1/2	2.48	2 1/2	2.98	3 3/4	
×153[h]	44.9	13.6	13 5/8	1.26	1 1/4	5/8	17.1	13.4	13 3/8	2.28	2 1/4	2.78	3 9/16	
×139.5[h]	41.0	13.4	13 3/8	1.16	1 3/16	5/8	15.5	13.3	13 1/4	2.09	2 1/16	2.59	3 3/8	
×125	36.8	13.2	13 1/8	1.04	1 1/16	9/16	13.7	13.2	13 1/8	1.89	1 7/8	2.39	3 1/8	
×114.5	33.6	13.0	13	0.960	15/16	1/2	12.5	13.1	13 1/8	1.73	1 3/4	2.23	3	
×103.5	30.3	12.9	12 7/8	0.870	7/8	7/16	11.2	13.0	13	1.57	1 9/16	2.07	2 7/8	
×96	28.2	12.7	12 3/4	0.810	13/16	7/16	10.3	13.0	13	1.46	1 7/16	1.96	2 3/4	
×88	25.8	12.6	12 5/8	0.750	3/4	3/8	9.47	12.9	12 7/8	1.34	1 5/16	1.84	2 5/8	
×81	23.9	12.5	12 1/2	0.705	11/16	3/8	8.81	13.0	13	1.22	1 1/4	1.72	2 1/2	
×73[c]	21.5	12.4	12 3/8	0.650	5/8	5/16	8.04	12.9	12 7/8	1.09	1 1/16	1.59	2 3/8	
×65.5[c]	19.3	12.2	12 1/4	0.605	5/8	5/16	7.41	12.9	12 7/8	0.960	15/16	1.46	2 1/4	
×58.5[c]	17.2	12.1	12 1/8	0.550	9/16	5/16	6.67	12.8	12 3/4	0.850	7/8	1.35	2 1/8	
×52[c]	15.3	12.0	12	0.500	1/2	1/4	6.02	12.8	12 3/4	0.750	3/4	1.25	2 1/16	
WT12×51.5[c]	15.1	12.3	12 1/4	0.550	9/16	5/16	6.75	9.00	9	0.980	1	1.48	2 1/4	5 1/2
×47[c]	13.8	12.2	12 1/8	0.515	1/2	1/4	6.26	9.07	9 1/8	0.875	7/8	1.38	2 1/8	
×42[c]	12.4	12.1	12	0.470	1/2	1/4	5.66	9.02	9	0.770	3/4	1.27	2 1/16	
×38[c]	11.2	12.0	12	0.440	7/16	1/4	5.26	8.99	9	0.680	11/16	1.18	1 15/16	5 1/2[g]
×34[c]	10.0	11.9	11 7/8	0.415	7/16	1/4	4.92	8.97	9	0.585	9/16	1.09	1 7/8	5 1/2[g]
WT12×31[c]	9.11	11.9	11 7/8	0.430	7/16	1/4	5.10	7.04	7	0.590	9/16	1.09	1 1/2	3 1/2
×27.5[c,v]	8.10	11.8	11 3/4	0.395	3/8	3/16	4.66	7.01	7	0.505	1/2	1.01	1 7/16	3 1/2

[c] Shape is slender for compression with $F_y = 50$ ksi.
[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.
[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
[v] Shear strength controlled by buckling effects ($C_{v2} < 1.0$) with $F_y = 50$ ksi.

Table 1-8 (continued)
WT-Shapes
Properties



WT13.5–WT12

Nom-inal Wt.	Compact Section Criteria		Axis X-X						Axis Y-Y				Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{d}{t_w}$	I	S	r	$\bar{y}$	Z	y_p	I	S	r	Z	J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{6}
64.5	4.55	22.6	323	31.0	4.13	3.39	55.1	0.945	92.2	18.4	2.21	28.8	5.55	24.0
57	5.41	23.9	289	28.3	4.15	3.42	50.4	0.832	79.3	15.8	2.18	24.6	3.65	17.5
51	6.03	26.2	258	25.3	4.14	3.37	45.0	0.750	69.6	13.9	2.15	21.7	2.63	12.6
47	6.70	27.6	239	23.8	4.16	3.41	42.4	0.692	62.0	12.4	2.12	19.4	2.01	10.2
42	7.78	29.1	216	21.9	4.18	3.48	39.2	0.621	52.8	10.6	2.07	16.6	1.40	7.79
185	2.51	9.20	779	74.7	3.78	3.57	140	1.99	581	85.1	3.27	133	100	553
167.5	2.73	10.0	686	66.3	3.73	3.42	123	1.82	513	75.9	3.23	119	75.6	405
153	2.94	10.8	611	59.4	3.69	3.29	110	1.67	460	68.6	3.20	107	58.4	305
139.5	3.18	11.6	546	53.6	3.65	3.18	98.8	1.54	412	61.9	3.17	96.3	45.1	230
125	3.49	12.7	478	47.2	3.61	3.05	86.5	1.39	362	54.9	3.14	85.2	33.2	165
114.5	3.79	13.5	431	42.9	3.58	2.96	78.1	1.28	326	49.7	3.11	77.0	25.5	125
103.5	4.14	14.8	382	38.3	3.55	2.87	69.3	1.17	289	44.4	3.08	68.6	19.1	91.3
96	4.43	15.7	350	35.2	3.53	2.80	63.5	1.09	265	40.9	3.07	63.1	15.3	72.5
88	4.81	16.8	319	32.2	3.51	2.74	57.8	1.00	240	37.2	3.04	57.3	11.9	55.8
81	5.31	17.7	293	29.9	3.50	2.70	53.3	0.921	221	34.2	3.05	52.6	9.22	43.8
73	5.92	19.1	264	27.2	3.50	2.66	48.2	0.833	195	30.3	3.01	46.6	6.70	31.9
65.5	6.70	20.2	238	24.8	3.52	2.65	43.9	0.750	170	26.5	2.97	40.7	4.74	23.1
58.5	7.53	22.0	212	22.3	3.51	2.62	39.2	0.672	149	23.2	2.94	35.7	3.35	16.4
52	8.50	24.0	189	20.0	3.51	2.59	35.1	0.600	130	20.3	2.91	31.2	2.35	11.6
51.5	4.59	22.4	204	22.0	3.67	3.01	39.2	0.841	59.7	13.3	1.99	20.7	3.53	12.3
47	5.18	23.7	186	20.3	3.67	2.99	36.1	0.764	54.5	12.0	1.98	18.7	2.62	9.57
42	5.86	25.7	166	18.3	3.67	2.97	32.5	0.685	47.2	10.5	1.95	16.3	1.84	6.90
38	6.61	27.3	151	16.9	3.68	3.00	30.1	0.622	41.3	9.18	1.92	14.3	1.34	5.30
34	7.66	28.7	137	15.6	3.70	3.06	27.9	0.560	35.2	7.85	1.87	12.3	0.932	4.08
31	5.97	27.7	131	15.6	3.79	3.46	28.4	1.28	17.2	4.90	1.38	7.85	0.850	3.92
27.5	6.94	29.9	117	14.1	3.80	3.50	25.6	1.53	14.5	4.15	1.34	6.65	0.588	2.93

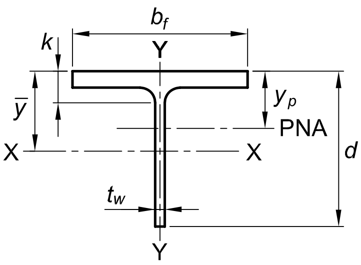


Table 1-8 (continued)
WT-Shapes
Dimensions

Shape	Area, A	Depth, d		Stem: Thickness, t_w		Stem: $\frac{t_w}{2}$	Stem: Area	Flange: Width, b_f		Flange: Thickness, t_f		Distance: k, k_{des}	Distance: k, k_{det}	Distance: Workable Gage
	in.²	in.		in.		in.	in.²	in.		in.		in.	in.	in.
WT10.5×137.5[h]	40.9	12.1	12 1/8	1.22	1 1/4	5/8	14.8	12.9	12 7/8	2.19	2 3/16	3.37	3 7/16	5 1/2
×124	37.0	11.9	11 7/8	1.10	1 1/8	9/16	13.1	12.8	12 3/4	1.99	2	3.17	3 1/4	
×111.5	33.2	11.7	11 3/4	1.00	1	1/2	11.7	12.7	12 3/4	1.79	1 13/16	2.97	3 1/16	
×100.5	29.6	11.5	11 1/2	0.910	15/16	1/2	10.5	12.6	12 5/8	1.63	1 5/8	2.13	2 7/8	
×91	26.8	11.4	11 3/8	0.830	13/16	7/16	9.43	12.5	12 1/2	1.48	1 1/2	1.98	2 3/4	
×83	24.4	11.2	11 1/4	0.750	3/4	3/8	8.43	12.4	12 3/8	1.36	1 3/8	1.86	2 5/8	
×73.5	21.6	11.0	11	0.720	3/4	3/8	7.94	12.5	12 1/2	1.15	1 1/8	1.65	2 7/16	
×66	19.4	10.9	10 7/8	0.650	5/8	5/16	7.09	12.4	12 1/2	1.04	1 1/16	1.54	2 1/4	
×61	17.9	10.8	10 7/8	0.600	5/8	5/16	6.50	12.4	12 3/8	0.960	15/16	1.46	2 1/4	
×55.5[c]	16.3	10.8	10 3/4	0.550	9/16	5/16	5.92	12.3	12 3/8	0.875	7/8	1.38	2 1/8	
×50.5[c]	14.9	10.7	10 5/8	0.500	1/2	1/4	5.34	12.3	12 1/4	0.800	13/16	1.30	2 1/16	
WT10.5×46.5[c]	13.7	10.8	10 3/4	0.580	9/16	5/16	6.27	8.42	8 3/8	0.930	15/16	1.43	1 5/8	5 1/2
×41.5[c]	12.2	10.7	10 3/4	0.515	1/2	1/4	5.52	8.36	8 3/8	0.835	13/16	1.34	1 1/2	
×36.5[c]	10.7	10.6	10 5/8	0.455	7/16	1/4	4.83	8.30	8 1/4	0.740	3/4	1.24	1 7/16	
×34[c]	10.0	10.6	10 5/8	0.430	7/16	1/4	4.54	8.27	8 1/4	0.685	11/16	1.19	1 3/8	
×31[c]	9.13	10.5	10 1/2	0.400	3/8	3/16	4.20	8.24	8 1/4	0.615	5/8	1.12	1 5/16	
×27.5[c]	8.10	10.4	10 3/8	0.375	3/8	3/16	3.90	8.22	8 1/4	0.522	1/2	1.02	1 3/16	
×24[c,f,v]	7.07	10.3	10 1/4	0.350	3/8	3/16	3.61	8.14	8 1/8	0.430	7/16	0.930	1 1/8	
WT10.5×28.5[c]	8.37	10.5	10 1/2	0.405	3/8	3/16	4.26	6.56	6 1/2	0.650	5/8	1.15	1 5/16	3 1/2
×25[c]	7.36	10.4	10 3/8	0.380	3/8	3/16	3.96	6.53	6 1/2	0.535	9/16	1.04	1 1/4	3 1/2[g]
×22[c,v]	6.49	10.3	10 3/8	0.350	3/8	3/16	3.62	6.50	6 1/2	0.450	7/16	0.950	1 1/8	3 1/2[g]

[c] Shape is slender for compression with $F_y = 50$ ksi.
[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.
[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.
[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
[v] Shear strength controlled by buckling effects ($C_{v2} < 1.0$) with $F_y = 50$ ksi.

Table 1-8 (continued)
WT-Shapes
Properties

WT10.5

Nom-inal Wt.	Compact Section Criteria		Axis X-X						Axis Y-Y				Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{d}{t_w}$	I	S	r	$\bar{y}$	Z	y_p	I	S	r	Z	J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{6}
137.5	2.95	9.92	420	45.7	3.20	2.90	86.3	1.59	394	61.1	3.10	95.1	53.5	224
124	3.22	10.8	368	40.3	3.15	2.77	75.7	1.45	349	54.5	3.07	84.8	40.2	163
111.5	3.55	11.7	324	35.9	3.12	2.66	66.7	1.31	307	48.3	3.04	74.9	29.6	117
100.5	3.86	12.6	285	31.9	3.10	2.57	58.6	1.18	271	43.1	3.02	66.5	20.4	85.4
91	4.22	13.7	253	28.5	3.07	2.48	52.1	1.07	241	38.6	3.00	59.5	15.3	63.0
83	4.57	14.9	226	25.5	3.04	2.39	46.3	0.983	217	35.0	2.99	53.9	11.8	47.3
73.5	5.44	15.3	204	23.7	3.08	2.39	42.4	0.864	188	30.0	2.95	46.3	7.69	32.5
66	6.01	16.8	181	21.1	3.06	2.33	37.6	0.780	166	26.7	2.93	41.1	5.62	23.4
61	6.45	18.0	166	19.3	3.04	2.28	34.3	0.724	152	24.6	2.91	37.8	4.47	18.4
55.5	7.05	19.6	150	17.5	3.03	2.23	31.0	0.662	137	22.2	2.90	34.1	3.40	13.8
50.5	7.68	21.4	135	15.8	3.01	2.18	27.9	0.605	124	20.2	2.89	30.8	2.60	10.4
46.5	4.53	18.6	144	17.9	3.25	2.74	31.8	0.812	46.4	11.0	1.84	17.3	3.01	9.33
41.5	5.00	20.8	127	15.7	3.22	2.66	28.0	0.728	40.7	9.74	1.83	15.2	2.16	6.50
36.5	5.60	23.3	110	13.8	3.21	2.60	24.4	0.647	35.3	8.51	1.81	13.3	1.51	4.42
34	6.04	24.7	103	12.9	3.20	2.59	22.9	0.606	32.4	7.83	1.80	12.2	1.22	3.62
31	6.70	26.3	93.8	11.9	3.21	2.58	21.1	0.554	28.7	6.97	1.77	10.9	0.913	2.78
27.5	7.87	27.7	84.4	10.9	3.23	2.64	19.4	0.493	24.2	5.89	1.73	9.18	0.617	2.08
24	9.47	29.4	74.9	9.90	3.26	2.74	17.8	0.459	19.4	4.76	1.66	7.44	0.400	1.52
28.5	5.04	25.9	90.4	11.8	3.29	2.85	21.2	0.638	15.3	4.67	1.35	7.40	0.884	2.50
25	6.10	27.4	80.3	10.7	3.30	2.93	19.4	0.771	12.5	3.82	1.30	6.08	0.570	1.89
22	7.22	29.4	71.1	9.68	3.31	2.98	17.6	1.06	10.3	3.18	1.26	5.07	0.383	1.40

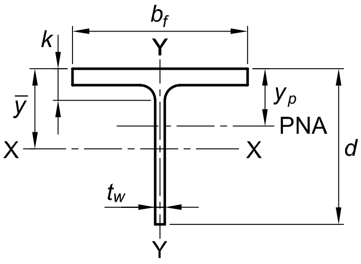
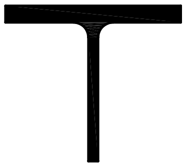


Table 1-8 (continued)
WT-Shapes
Dimensions

Shape	Area, A	Depth, d		Stem: Thickness, t_w		Stem: $\frac{t_w}{2}$	Stem: Area	Flange: Width, b_f		Flange: Thickness, t_f		Distance: k, k_{des}	Distance: k, k_{det}	Distance: Workable Gage
	in.^{2}	in.		in.		in.	in.^{2}	in.		in.		in.	in.	in.
WT9×155.5[h]	45.8	11.2	11 1/8	1.52	1 1/2	3/4	17.0	12.0	12	2.74	2 3/4	3.24	3 9/16	5 1/2
×141.5[h]	41.7	10.9	10 7/8	1.40	1 3/8	11/16	15.3	11.9	11 7/8	2.50	2 1/2	3.00	3 3/8	
×129[h]	38.0	10.7	10 3/4	1.28	1 1/4	5/8	13.7	11.8	11 3/4	2.30	2 5/16	2.70	3 3/16	
×117[h]	34.3	10.5	10 1/2	1.16	1 3/16	5/8	12.2	11.7	11 5/8	2.11	2 1/8	2.51	3	
×105.5	31.2	10.3	10 3/8	1.06	1 1/16	9/16	11.0	11.6	11 1/2	1.91	1 15/16	2.31	2 13/16	
×96	28.1	10.2	10 1/8	0.960	15/16	1/2	9.77	11.5	11 1/2	1.75	1 3/4	2.15	2 5/8	
×87.5	25.7	10.0	10	0.890	7/8	7/16	8.92	11.4	11 3/8	1.59	1 9/16	1.99	2 7/16	
×79	23.2	9.86	9 7/8	0.810	13/16	7/16	7.99	11.3	11 1/4	1.44	1 7/16	1.84	2 3/8	
×71.5	21.0	9.75	9 3/4	0.730	3/4	3/8	7.11	11.2	11 1/4	1.32	1 5/16	1.72	2 3/16	
×65	19.2	9.63	9 5/8	0.670	11/16	3/8	6.45	11.2	11 1/8	1.20	1 3/16	1.60	2 1/16	
×59.5	17.6	9.49	9 1/2	0.655	5/8	5/16	6.21	11.3	11 1/4	1.06	1 1/16	1.46	1 15/16	
×53	15.6	9.37	9 3/8	0.590	9/16	5/16	5.53	11.2	11 1/4	0.940	15/16	1.34	1 13/16	
×48.5	14.2	9.30	9 1/4	0.535	9/16	5/16	4.97	11.1	11 1/8	0.870	7/8	1.27	1 3/4	
×43[c]	12.7	9.20	9 1/4	0.480	1/2	1/4	4.41	11.1	11 1/8	0.770	3/4	1.17	1 5/8	
×38[c]	11.1	9.11	9 1/8	0.425	7/16	1/4	3.87	11.0	11	0.680	11/16	1.08	1 9/16	
WT9×35.5[c]	10.4	9.24	9 1/4	0.495	1/2	1/4	4.57	7.64	7 5/8	0.810	13/16	1.21	1 1/2	3 1/2[g]
×32.5[c]	9.55	9.18	9 1/8	0.450	7/16	1/4	4.13	7.59	7 5/8	0.750	3/4	1.15	1 7/16	
×30[c]	8.82	9.12	9 1/8	0.415	7/16	1/4	3.78	7.56	7 1/2	0.695	11/16	1.10	1 3/8	
×27.5[c]	8.10	9.06	9	0.390	3/8	3/16	3.53	7.53	7 1/2	0.630	5/8	1.03	1 5/16	
×25[c]	7.34	9.00	9	0.355	3/8	3/16	3.19	7.50	7 1/2	0.570	9/16	0.972	1 1/4	
WT9×23[c]	6.77	9.03	9	0.360	3/8	3/16	3.25	6.06	6	0.605	5/8	1.01	1 1/4	3 1/2[g]
×20[c]	5.88	8.95	9	0.315	5/16	3/16	2.82	6.02	6	0.525	1/2	0.927	1 3/16	
×17.5[c,v]	5.15	8.85	8 7/8	0.300	5/16	3/16	2.66	6.00	6	0.425	7/16	0.827	1 1/8	
WT8×50	14.7	8.49	8 1/2	0.585	9/16	5/16	4.96	10.4	10 3/8	0.985	1	1.39	1 7/8	5 1/2
×44.5	13.1	8.38	8 3/8	0.525	1/2	1/4	4.40	10.4	10 3/8	0.875	7/8	1.28	1 3/4	
×38.5[c]	11.3	8.26	8 1/4	0.455	7/16	1/4	3.76	10.3	10 1/4	0.760	3/4	1.16	1 5/8	
×33.5[c]	9.81	8.17	8 1/8	0.395	3/8	3/16	3.23	10.2	10 1/4	0.665	11/16	1.07	1 9/16	
WT8×28.5[c]	8.39	8.22	8 1/4	0.430	7/16	1/4	3.53	7.12	7 1/8	0.715	11/16	1.12	1 3/8	3 1/2[g]
×25[c]	7.37	8.13	8 1/8	0.380	3/8	3/16	3.09	7.07	7 1/8	0.630	5/8	1.03	1 5/16	
×22.5[c]	6.63	8.07	8 1/8	0.345	3/8	3/16	2.78	7.04	7	0.565	9/16	0.967	1 1/4	
×20[c]	5.89	8.01	8	0.305	5/16	3/16	2.44	7.00	7	0.505	1/2	0.907	1 3/16	3 1/2
×18[c]	5.29	7.93	7 7/8	0.295	5/16	3/16	2.34	6.99	7	0.430	7/16	0.832	1 1/8	3 1/2

[c] Shape is slender for compression with $F_y = 50$ ksi.
[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.
[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
[v] Shear strength controlled by buckling effects ($C_{v2} < 1.0$) with $F_y = 50$ ksi.

Table 1-8 (continued)
WT-Shapes
Properties



WT9–WT8

Nom-inal Wt.	Compact Section Criteria		Axis X-X						Axis Y-Y				Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{d}{t_w}$	I	S	r	$\bar{y}$	Z	y_p	I	S	r	Z	J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{6}
155.5	2.19	7.37	383	46.6	2.89	2.93	90.6	1.91	398	66.2	2.95	104	87.2	339
141.5	2.38	7.79	337	41.5	2.85	2.80	80.2	1.75	352	59.2	2.91	92.5	66.5	251
129	2.56	8.36	298	37.0	2.80	2.68	71.0	1.61	314	53.4	2.88	83.1	51.1	189
117	2.76	9.05	261	32.7	2.75	2.55	62.4	1.48	279	47.9	2.85	74.4	39.1	140
105.5	3.02	9.72	229	29.1	2.72	2.44	55.0	1.34	246	42.7	2.82	66.1	29.1	102
96	3.27	10.6	202	25.8	2.68	2.34	48.5	1.23	220	38.4	2.79	59.4	22.3	75.7
87.5	3.58	11.2	181	23.4	2.66	2.26	43.6	1.13	196	34.4	2.76	53.1	16.8	56.5
79	3.92	12.2	160	20.8	2.63	2.17	38.5	1.02	174	30.7	2.74	47.4	12.5	41.2
71.5	4.25	13.4	142	18.5	2.60	2.09	34.0	0.937	156	27.7	2.72	42.7	9.58	30.7
65	4.65	14.4	127	16.7	2.58	2.02	30.5	0.856	139	24.9	2.70	38.3	7.23	22.8
59.5	5.31	14.5	119	15.9	2.60	2.03	28.7	0.778	126	22.5	2.69	34.5	5.30	17.4
53	5.96	15.9	104	14.1	2.59	1.97	25.2	0.695	110	19.7	2.66	30.2	3.73	12.1
48.5	6.41	17.4	93.8	12.7	2.56	1.91	22.6	0.640	100	18.0	2.65	27.6	2.92	9.29
43	7.20	19.2	82.4	11.2	2.55	1.86	19.9	0.570	87.6	15.8	2.63	24.2	2.04	6.42
38	8.11	21.4	71.8	9.83	2.54	1.80	17.3	0.505	76.2	13.8	2.61	21.1	1.41	4.37
35.5	4.71	18.7	78.2	11.2	2.74	2.26	20.0	0.683	30.1	7.89	1.70	12.3	1.74	3.96
32.5	5.06	20.4	70.7	10.1	2.72	2.20	18.0	0.629	27.4	7.22	1.69	11.2	1.36	3.01
30	5.44	22.0	64.7	9.29	2.71	2.16	16.5	0.583	25.0	6.63	1.68	10.3	1.08	2.35
27.5	5.98	23.2	59.5	8.63	2.71	2.16	15.3	0.538	22.5	5.97	1.67	9.26	0.830	1.84
25	6.57	25.4	53.5	7.79	2.70	2.12	13.8	0.489	20.0	5.35	1.65	8.28	0.619	1.36
23	5.01	25.1	52.1	7.77	2.77	2.33	13.9	0.558	11.3	3.71	1.29	5.84	0.609	1.20
20	5.73	28.4	44.8	6.73	2.76	2.29	12.0	0.489	9.55	3.17	1.27	4.97	0.404	0.788
17.5	7.06	29.5	40.1	6.21	2.79	2.39	11.2	0.450	7.67	2.56	1.22	4.02	0.252	0.598
50	5.29	14.5	76.8	11.4	2.28	1.76	20.7	0.706	93.1	17.9	2.51	27.4	3.85	10.4
44.5	5.92	16.0	67.2	10.1	2.27	1.70	18.1	0.631	81.3	15.7	2.49	24.0	2.72	7.19
38.5	6.77	18.2	56.9	8.59	2.24	1.63	15.3	0.549	69.2	13.4	2.47	20.5	1.78	4.61
33.5	7.70	20.7	48.6	7.36	2.22	1.56	13.0	0.481	59.5	11.6	2.46	17.7	1.19	3.01
28.5	4.98	19.1	48.7	7.77	2.41	1.94	13.8	0.589	21.6	6.06	1.60	9.42	1.10	1.99
25	5.61	21.4	42.3	6.78	2.40	1.89	12.0	0.521	18.6	5.26	1.59	8.15	0.760	1.34
22.5	6.23	23.4	37.8	6.10	2.39	1.86	10.8	0.471	16.4	4.67	1.57	7.22	0.555	0.974
20	6.93	26.3	33.1	5.35	2.37	1.81	9.43	0.421	14.4	4.12	1.56	6.36	0.396	0.673
18	8.12	26.9	30.6	5.05	2.41	1.88	8.93	0.378	12.2	3.50	1.52	5.42	0.272	0.516

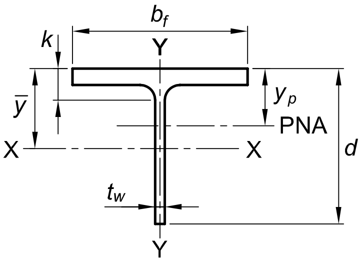
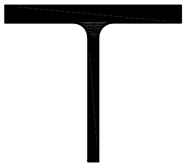


Table 1-8 (continued)
WT-Shapes
Dimensions

Shape	Area, A	Depth, d		Stem				Flange				Distance		
				Thickness, t_w		$\frac{t_w}{2}$	Area	Width, b_f		Thickness, t_f		k		Workable Gage
												k_{des}	k_{det}	
	in.^{2}	in.		in.		in.	in.^{2}	in.		in.		in.	in.	in.
WT8×15.5[c]	4.56	7.94	8	0.275	1/4	1/8	2.18	5.53	5 1/2	0.440	7/16	0.842	1 1/8	3 1/2
×13[c,v]	3.84	7.85	7 7/8	0.250	1/4	1/8	1.96	5.50	5 1/2	0.345	3/8	0.747	1 1/16	3 1/2
WT7×436.5[h]	129	11.8	11 3/4	3.94	3 15/16	2	46.5	18.8	18 3/4	5.51	5 1/2	6.10	6 3/16	8 1/2[g]
×404[h]	119	11.4	11 3/8	3.74	3 3/4	1 7/8	42.6	18.6	18 5/8	5.12	5 1/8	5.71	5 3/4	8 1/2
×365[h]	107	11.2	11 1/4	3.07	3 1/16	1 9/16	34.4	17.9	17 7/8	4.91	4 15/16	5.51	6 3/16	7 1/2[g]
×332.5[h]	97.8	10.8	10 7/8	2.83	2 13/16	1 7/16	30.6	17.7	17 5/8	4.52	4 1/2	5.12	5 13/16	7 1/2[g]
×302.5[h]	89.0	10.5	10 1/2	2.60	2 5/8	1 5/16	27.1	17.4	17 3/8	4.16	4 3/16	4.76	5 7/16	7 1/2
×275[h]	80.9	10.1	10 1/8	2.38	2 3/8	1 3/16	24.1	17.2	17 1/4	3.82	3 13/16	4.42	5 1/8	
×250[h]	73.5	9.80	9 3/4	2.19	2 3/16	1 1/8	21.5	17.0	17	3.50	3 1/2	4.10	4 13/16	
×227.5[h]	66.9	9.51	9 1/2	2.02	2	1	19.2	16.8	16 7/8	3.21	3 3/16	3.81	4 1/2	
×213[h]	62.7	9.34	9 3/8	1.88	1 7/8	15/16	17.5	16.7	16 3/4	3.04	3 1/16	3.63	4 5/16	
×199[h]	58.4	9.15	9 1/8	1.77	1 3/4	7/8	16.2	16.6	16 5/8	2.85	2 7/8	3.44	4 1/8	
×185[h]	54.4	8.96	9	1.66	1 11/16	13/16	14.8	16.5	16 1/2	2.66	2 11/16	3.26	3 15/16	
×171[h]	50.3	8.77	8 3/4	1.54	1 9/16	13/16	13.5	16.4	16 3/8	2.47	2 1/2	3.07	3 3/4	
×155.5[h]	45.7	8.56	8 1/2	1.41	1 7/16	3/4	12.1	16.2	16 1/4	2.26	2 1/4	2.86	3 9/16	
×141.5[h]	41.6	8.37	8 3/8	1.29	1 5/16	11/16	10.8	16.1	16 1/8	2.07	2 1/16	2.67	3 3/8	
×128.5	37.8	8.19	8 1/4	1.18	1 3/16	5/8	9.62	16.0	16	1.89	1 7/8	2.49	3 3/16	
×116.5	34.2	8.02	8	1.07	1 1/16	9/16	8.58	15.9	15 7/8	1.72	1 3/4	2.32	3	
×105.5	31.0	7.86	7 7/8	0.980	1	1/2	7.70	15.8	15 3/4	1.56	1 9/16	2.16	2 7/8	
×96.5	28.4	7.74	7 3/4	0.890	7/8	7/16	6.89	15.7	15 3/4	1.44	1 7/16	2.04	2 3/4	
×88	25.9	7.61	7 5/8	0.830	13/16	7/16	6.32	15.7	15 5/8	1.31	1 5/16	1.91	2 5/8	
×79.5	23.4	7.49	7 1/2	0.745	3/4	3/8	5.58	15.6	15 5/8	1.19	1 3/16	1.79	2 1/2	
×72.5	21.3	7.39	7 3/8	0.680	11/16	3/8	5.03	15.5	15 1/2	1.09	1 1/16	1.69	2 3/8	
WT7×66	19.4	7.33	7 3/8	0.645	5/8	5/16	4.73	14.7	14 3/4	1.03	1	1.63	2 5/16	5 1/2
×60	17.7	7.24	7 1/4	0.590	9/16	5/16	4.27	14.7	14 5/8	0.940	15/16	1.54	2 1/4	
×54.5	16.0	7.16	7 1/8	0.525	1/2	1/4	3.76	14.6	14 5/8	0.860	7/8	1.46	2 3/16	
×49.5[f]	14.6	7.08	7 1/8	0.485	1/2	1/4	3.43	14.6	14 5/8	0.780	3/4	1.38	2 1/16	
×45[f]	13.2	7.01	7	0.440	7/16	1/4	3.08	14.5	14 1/2	0.710	11/16	1.31	2	
WT7×41	12.0	7.16	7 1/8	0.510	1/2	1/4	3.65	10.1	10 1/8	0.855	7/8	1.45	1 11/16	5 1/2
×37	10.9	7.09	7 1/8	0.450	7/16	1/4	3.19	10.1	10 1/8	0.785	13/16	1.38	1 5/8	
×34	10.0	7.02	7	0.415	7/16	1/4	2.91	10.0	10	0.720	3/4	1.31	1 9/16	
×30.5[c]	8.96	6.95	7	0.375	3/8	3/16	2.60	10.0	10	0.645	5/8	1.24	1 1/2	

[c] Shape is slender for compression with $F_y = 50$ ksi.
[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.
[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.
[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
[v] Shear strength controlled by buckling effects ($C_{v2} < 1.0$) with $F_y = 50$ ksi.

Table 1-8 (continued)
WT-Shapes
Properties



WT8–WT7

Nom-inal Wt.	Compact Section Criteria		Axis X-X						Axis Y-Y				Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{d}{t_w}$	I	S	r	$\bar{y}$	Z	y_p	I	S	r	Z	J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{6}
15.5	6.28	28.9	27.5	4.64	2.45	2.02	8.27	0.413	6.20	2.24	1.17	3.51	0.230	0.366
13	7.97	31.4	23.5	4.09	2.47	2.09	7.36	0.372	4.79	1.74	1.12	2.73	0.130	0.243
436.5	1.71	2.99	1040	131	2.84	3.88	281	3.43	3080	328	4.89	511	1110	8980
404	1.82	3.05	898	116	2.75	3.69	249	3.20	2770	298	4.82	465	898	7000
365	1.82	3.65	739	95.4	2.62	3.47	211	3.00	2360	264	4.69	408	714	5250
332.5	1.95	3.82	622	82.1	2.52	3.25	182	2.77	2080	236	4.62	365	555	3920
302.5	2.09	4.04	524	70.6	2.43	3.05	157	2.55	1840	211	4.55	326	430	2930
275	2.25	4.24	442	60.9	2.34	2.85	136	2.35	1630	189	4.49	292	331	2180
250	2.43	4.47	375	52.7	2.26	2.67	117	2.16	1440	169	4.43	261	254	1620
227.5	2.62	4.71	321	45.9	2.19	2.51	102	1.99	1280	152	4.38	234	196	1210
213	2.75	4.97	287	41.4	2.14	2.40	91.7	1.88	1180	141	4.34	217	164	991
199	2.92	5.17	257	37.6	2.10	2.30	82.9	1.76	1090	131	4.31	201	135	801
185	3.10	5.40	229	33.9	2.05	2.19	74.4	1.65	994	121	4.27	185	110	640
171	3.31	5.69	203	30.4	2.01	2.09	66.2	1.54	903	110	4.24	169	88.3	502
155.5	3.59	6.07	176	26.7	1.96	1.97	57.7	1.41	807	99.4	4.20	152	67.5	375
141.5	3.89	6.49	153	23.5	1.92	1.86	50.4	1.29	722	89.7	4.17	137	51.8	281
128.5	4.23	6.94	133	20.7	1.88	1.75	43.9	1.18	645	80.7	4.13	123	39.3	209
116.5	4.62	7.50	116	18.2	1.84	1.65	38.2	1.08	576	72.5	4.10	110	29.6	154
105.5	5.06	8.02	102	16.2	1.81	1.57	33.4	0.980	513	65.0	4.07	98.9	22.2	113
96.5	5.45	8.70	89.8	14.4	1.78	1.49	29.4	0.903	466	59.3	4.05	90.1	17.3	87.2
88	5.97	9.17	80.5	13.0	1.76	1.43	26.3	0.827	419	53.5	4.02	81.3	13.2	65.2
79.5	6.54	10.1	70.2	11.4	1.73	1.35	22.8	0.751	374	48.1	4.00	73.0	9.84	47.9
72.5	7.11	10.9	62.5	10.2	1.71	1.29	20.2	0.688	338	43.7	3.98	66.2	7.56	36.3
66	7.15	11.4	57.8	9.57	1.73	1.29	18.6	0.658	274	37.2	3.76	56.5	6.13	26.6
60	7.80	12.3	51.7	8.61	1.71	1.24	16.5	0.602	247	33.7	3.74	51.2	4.67	20.0
54.5	8.49	13.6	45.3	7.56	1.68	1.17	14.4	0.548	223	30.6	3.73	46.3	3.55	15.0
49.5	9.34	14.6	40.9	6.88	1.67	1.14	12.9	0.500	201	27.6	3.71	41.8	2.68	11.1
45	10.2	15.9	36.5	6.16	1.66	1.09	11.5	0.456	181	25.0	3.70	37.8	2.03	8.31
41	5.92	14.0	41.2	7.14	1.85	1.39	13.2	0.593	74.1	14.6	2.48	22.4	2.53	5.63
37	6.41	15.8	36.0	6.25	1.82	1.32	11.5	0.541	66.9	13.3	2.48	20.2	1.93	4.19
34	6.97	16.9	32.6	5.69	1.81	1.29	10.4	0.498	60.7	12.1	2.46	18.4	1.50	3.21
30.5	7.75	18.5	28.9	5.07	1.80	1.25	9.15	0.448	53.7	10.7	2.45	16.4	1.09	2.29

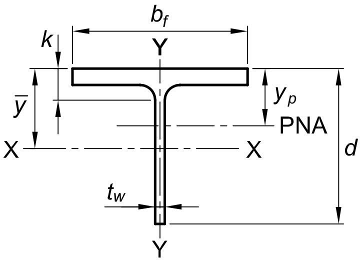


Table 1-8 (continued)
WT-Shapes
Dimensions

Shape	Area, A	Depth, d		Stem Thickness, t_w		Stem $\frac{t_w}{2}$	Stem Area	Flange Width, b_f		Flange Thickness, t_f		Distance k_{des}	Distance k_{det}	Distance Work-able Gage
	in.^{2}	in.		in.		in.	in.^{2}	in.		in.		in.	in.	in.
WT7×26.5[c]	7.80	6.96	7	0.370	3/8	3/16	2.58	8.06	8	0.660	11/16	1.25	1 1/2	5 1/2
×24[c]	7.07	6.90	6 7/8	0.340	5/16	3/16	2.34	8.03	8	0.595	5/8	1.19	1 7/16	
×21.5[c]	6.31	6.83	6 7/8	0.305	5/16	3/16	2.08	8.00	8	0.530	1/2	1.12	1 3/8	
WT7×19[c]	5.58	7.05	7	0.310	5/16	3/16	2.19	6.77	6 3/4	0.515	1/2	0.915	1 1/4	3 1/2[g]
×17[c]	5.00	6.99	7	0.285	5/16	3/16	1.99	6.75	6 3/4	0.455	7/16	0.855	1 3/16	3 1/2
×15[c]	4.42	6.92	6 7/8	0.270	1/4	1/8	1.87	6.73	6 3/4	0.385	3/8	0.785	1 1/8	3 1/2
WT7×13[c]	3.85	6.96	7	0.255	1/4	1/8	1.77	5.03	5	0.420	7/16	0.820	1 1/8	2 3/4[g]
×11[c,v]	3.25	6.87	6 7/8	0.230	1/4	1/8	1.58	5.00	5	0.335	5/16	0.735	1 1/16	2 3/4[g]
WT6×168[h]	49.5	8.41	8 3/8	1.78	1 3/4	7/8	14.9	13.4	13 3/8	2.96	2 15/16	3.55	3 7/8	5 1/2
×152.5[h]	44.7	8.16	8 1/8	1.63	1 5/8	13/16	13.3	13.2	13 1/4	2.71	2 11/16	3.30	3 5/8	
×139.5[h]	41.0	7.93	7 7/8	1.53	1 1/2	3/4	12.1	13.1	13 1/8	2.47	2 1/2	3.07	3 3/8	
×126[h]	37.1	7.71	7 3/4	1.40	1 3/8	11/16	10.7	13.0	13	2.25	2 1/4	2.85	3 1/8	
×115[h]	33.8	7.53	7 1/2	1.29	1 5/16	11/16	9.67	12.9	12 7/8	2.07	2 1/16	2.67	2 15/16	
×105	30.9	7.36	7 3/8	1.18	1 3/16	5/8	8.68	12.8	12 3/4	1.90	1 7/8	2.50	2 13/16	
×95	28.0	7.19	7 1/4	1.06	1 1/16	9/16	7.62	12.7	12 5/8	1.74	1 3/4	2.33	2 5/8	
×85	25.0	7.02	7	0.960	15/16	1/2	6.73	12.6	12 5/8	1.56	1 9/16	2.16	2 7/16	
×76	22.4	6.86	6 7/8	0.870	7/8	7/16	5.96	12.5	12 1/2	1.40	1 3/8	2.00	2 5/16	
×68	20.0	6.71	6 3/4	0.790	13/16	7/16	5.30	12.4	12 3/8	1.25	1 1/4	1.85	2 1/8	
×60	17.6	6.56	6 1/2	0.710	11/16	3/8	4.66	12.3	12 3/8	1.11	1 1/8	1.70	2	
×53	15.6	6.45	6 1/2	0.610	5/8	5/16	3.93	12.2	12 1/4	0.990	1	1.59	1 7/8	
×48	14.1	6.36	6 3/8	0.550	9/16	5/16	3.50	12.2	12 1/8	0.900	7/8	1.50	1 13/16	
×43.5	12.8	6.27	6 1/4	0.515	1/2	1/4	3.23	12.1	12 1/8	0.810	13/16	1.41	1 11/16	
×39.5	11.6	6.19	6 1/4	0.470	1/2	1/4	2.91	12.1	12 1/8	0.735	3/4	1.33	1 5/8	
×36	10.6	6.13	6 1/8	0.430	7/16	1/4	2.63	12.0	12	0.670	11/16	1.27	1 9/16	
×32.5[f]	9.54	6.06	6	0.390	3/8	3/16	2.36	12.0	12	0.605	5/8	1.20	1 1/2	
WT6×29	8.52	6.10	6 1/8	0.360	3/8	3/16	2.19	10.0	10	0.640	5/8	1.24	1 1/2	5 1/2
×26.5	7.78	6.03	6	0.345	3/8	3/16	2.08	10.0	10	0.575	9/16	1.18	1 3/8	5 1/2
WT6×25	7.30	6.10	6 1/8	0.370	3/8	3/16	2.26	8.08	8 1/8	0.640	5/8	1.14	1 1/2	5 1/2
×22.5	6.56	6.03	6	0.335	5/16	3/16	2.02	8.05	8	0.575	9/16	1.08	1 3/8	
×20[c]	5.84	5.97	6	0.295	5/16	3/16	1.76	8.01	8	0.515	1/2	1.02	1 3/8	

[c] Shape is slender for compression with $F_y = 50$ ksi.
[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.
[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.
[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
[v] Shear strength controlled by buckling effects ($C_{v2} < 1.0$) with $F_y = 50$ ksi.

Table 1-8 (continued)
WT-Shapes
Properties

WT7–WT6

Nominal Wt.	Compact Section Criteria		Axis X-X						Axis Y-Y				Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{d}{t_w}$	I	S	r	$\bar{y}$	Z	y_p	I	S	r	Z	J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{6}
26.5	6.11	18.8	27.6	4.94	1.88	1.38	8.87	0.484	28.8	7.15	1.92	11.0	0.967	1.46
24	6.75	20.3	24.9	4.49	1.88	1.35	8.00	0.440	25.7	6.40	1.91	9.80	0.723	1.07
21.5	7.54	22.4	21.9	3.98	1.86	1.31	7.05	0.395	22.6	5.65	1.89	8.64	0.522	0.751
19	6.57	22.7	23.3	4.22	2.04	1.54	7.45	0.412	13.3	3.94	1.55	6.07	0.398	0.554
17	7.41	24.5	20.9	3.83	2.04	1.53	6.74	0.371	11.6	3.45	1.53	5.32	0.284	0.400
15	8.74	25.6	19.0	3.55	2.07	1.58	6.25	0.329	9.79	2.91	1.49	4.49	0.190	0.287
13	5.98	27.3	17.3	3.31	2.12	1.72	5.89	0.383	4.45	1.77	1.08	2.76	0.179	0.207
11	7.46	29.9	14.8	2.91	2.14	1.76	5.20	0.325	3.50	1.40	1.04	2.19	0.104	0.134
168	2.26	4.72	190	31.2	1.96	2.31	68.4	1.84	593	88.6	3.47	137	120	481
152.5	2.45	5.01	162	27.0	1.90	2.16	59.1	1.69	525	79.3	3.42	122	92.0	356
139.5	2.66	5.18	141	24.1	1.86	2.05	51.9	1.56	469	71.3	3.38	110	70.9	267
126	2.89	5.51	121	20.9	1.81	1.92	44.8	1.42	414	63.6	3.34	97.9	53.5	195
115	3.11	5.84	106	18.5	1.77	1.82	39.4	1.31	371	57.5	3.31	88.4	41.6	148
105	3.37	6.24	92.1	16.4	1.73	1.72	34.5	1.21	332	51.9	3.28	79.7	32.1	112
95	3.65	6.78	79.0	14.2	1.68	1.62	29.8	1.10	295	46.5	3.25	71.2	24.3	82.1
85	4.03	7.31	67.8	12.3	1.65	1.52	25.6	0.994	259	41.2	3.22	62.9	17.7	58.3
76	4.46	7.89	58.5	10.8	1.62	1.43	22.0	0.896	227	36.4	3.19	55.6	12.8	41.3
68	4.96	8.49	50.6	9.46	1.59	1.35	19.0	0.805	199	32.1	3.16	48.9	9.21	28.9
60	5.57	9.24	43.4	8.22	1.57	1.28	16.2	0.716	172	28.0	3.13	42.7	6.42	19.7
53	6.17	10.6	36.3	6.92	1.53	1.19	13.6	0.637	151	24.7	3.11	37.5	4.55	13.6
48	6.76	11.6	32.0	6.12	1.51	1.13	11.9	0.580	135	22.2	3.09	33.7	3.42	10.1
43.5	7.48	12.2	28.9	5.60	1.50	1.10	10.7	0.527	120	19.9	3.07	30.2	2.54	7.34
39.5	8.22	13.2	25.8	5.03	1.49	1.06	9.49	0.480	108	17.9	3.05	27.1	1.91	5.43
36	8.99	14.3	23.2	4.54	1.48	1.02	8.48	0.439	97.5	16.2	3.04	24.6	1.46	4.07
32.5	9.92	15.5	20.6	4.06	1.47	0.985	7.50	0.398	87.2	14.5	3.02	22.0	1.09	2.97
29	7.82	16.9	19.1	3.76	1.50	1.03	6.97	0.426	53.5	10.7	2.51	16.2	1.05	2.08
26.5	8.69	17.5	17.7	3.54	1.51	1.02	6.46	0.389	47.9	9.58	2.48	14.5	0.788	1.53
25	6.31	16.5	18.7	3.79	1.60	1.17	6.88	0.452	28.2	6.97	1.96	10.6	0.855	1.23
22.5	7.00	18.0	16.6	3.39	1.59	1.13	6.10	0.408	25.0	6.21	1.95	9.47	0.627	0.885
20	7.77	20.2	14.4	2.95	1.57	1.09	5.28	0.365	22.0	5.50	1.94	8.38	0.452	0.620

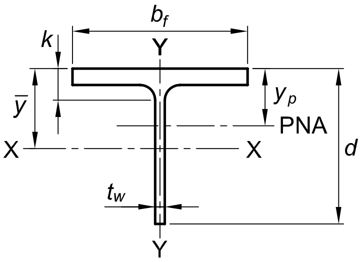


Table 1-8 (continued)
WT-Shapes
Dimensions

Shape	Area, A	Depth, d		Stem Thickness, t_w		Stem $\frac{t_w}{2}$	Stem Area	Flange Width, b_f		Flange Thickness, t_f		Distance k_{des}	Distance k_{det}	Distance Workable Gage
	in.^{2}	in.		in.		in.	in.^{2}	in.		in.		in.	in.	in.
WT6×17.5[c]	5.17	6.25	6 1/4	0.300	5/16	3/16	1.88	6.56	6 1/2	0.520	1/2	0.820	1 3/16	3 1/2
×15[c]	4.40	6.17	6 1/8	0.260	1/4	1/8	1.60	6.52	6 1/2	0.440	7/16	0.740	1 1/8	↓
×13[c]	3.82	6.11	6 1/8	0.230	1/4	1/8	1.41	6.49	6 1/2	0.380	3/8	0.680	1 1/16	↓
WT6×11[c]	3.24	6.16	6 1/8	0.260	1/4	1/8	1.60	4.03	4	0.425	7/16	0.725	15/16	2 1/4[g]
×9.5[c]	2.79	6.08	6 1/8	0.235	1/4	1/8	1.43	4.01	4	0.350	3/8	0.650	7/8	↓
×8[c]	2.36	6.00	6	0.220	1/4	1/8	1.32	3.99	4	0.265	1/4	0.565	13/16	↓
×7[c,v]	2.08	5.96	6	0.200	3/16	1/8	1.19	3.97	4	0.225	1/4	0.525	3/4	↓
WT5×56	16.5	5.68	5 5/8	0.755	3/4	3/8	4.29	10.4	10 3/8	1.25	1 1/4	1.75	1 15/16	5 1/2
×50	14.7	5.55	5 1/2	0.680	11/16	3/8	3.77	10.3	10 3/8	1.12	1 1/8	1.62	1 13/16	↓
×44	13.0	5.42	5 3/8	0.605	5/8	5/16	3.28	10.3	10 1/4	0.990	1	1.49	1 11/16	↓
×38.5	11.3	5.30	5 1/4	0.530	1/2	1/4	2.81	10.2	10 1/4	0.870	7/8	1.37	1 9/16	↓
×34	10.0	5.20	5 1/4	0.470	1/2	1/4	2.44	10.1	10 1/8	0.770	3/4	1.27	1 7/16	↓
×30	8.84	5.11	5 1/8	0.420	7/16	1/4	2.15	10.1	10 1/8	0.680	11/16	1.18	1 3/8	↓
×27	7.90	5.05	5	0.370	3/8	3/16	1.87	10.0	10	0.615	5/8	1.12	1 5/16	↓
×24.5	7.21	4.99	5	0.340	5/16	3/16	1.70	10.0	10	0.560	9/16	1.06	1 1/4	↓
WT5×22.5	6.63	5.05	5	0.350	3/8	3/16	1.77	8.02	8	0.620	5/8	1.12	1 5/16	↓
×19.5	5.73	4.96	5	0.315	5/16	3/16	1.56	7.99	8	0.530	1/2	1.03	1 3/16	↓
×16.5	4.85	4.87	4 7/8	0.290	5/16	3/16	1.41	7.96	8	0.435	7/16	0.935	1 1/8	↓
WT5×15	4.42	5.24	5 1/4	0.300	5/16	3/16	1.57	5.81	5 3/4	0.510	1/2	0.810	1 1/8	2 3/4[g]
×13[c]	3.81	5.17	5 1/8	0.260	1/4	1/8	1.34	5.77	5 3/4	0.440	7/16	0.740	1 1/16	↓
×11[c]	3.24	5.09	5 1/8	0.240	1/4	1/8	1.22	5.75	5 3/4	0.360	3/8	0.660	15/16	↓
WT5×9.5[c]	2.81	5.12	5 1/8	0.250	1/4	1/8	1.28	4.02	4	0.395	3/8	0.695	15/16	2 1/4[g]
×8.5[c]	2.50	5.06	5	0.240	1/4	1/8	1.21	4.01	4	0.330	5/16	0.630	7/8	↓
×7.5[c]	2.21	5.00	5	0.230	1/4	1/8	1.15	4.00	4	0.270	1/4	0.570	13/16	↓
×6[c,f]	1.77	4.94	4 7/8	0.190	3/16	1/8	0.938	3.96	4	0.210	3/16	0.510	3/4	↓
WT4×33.5	9.84	4.50	4 1/2	0.570	9/16	5/16	2.57	8.28	8 1/4	0.935	15/16	1.33	1 5/8	5 1/2
×29	8.54	4.38	4 3/8	0.510	1/2	1/4	2.23	8.22	8 1/4	0.810	13/16	1.20	1 1/2	↓
×24	7.05	4.25	4 1/4	0.400	3/8	3/16	1.70	8.11	8 1/8	0.685	11/16	1.08	1 3/8	↓
×20	5.87	4.13	4 1/8	0.360	3/8	3/16	1.49	8.07	8 1/8	0.560	9/16	0.954	1 1/4	↓
×17.5	5.14	4.06	4	0.310	5/16	3/16	1.26	8.02	8	0.495	1/2	0.889	1 3/16	↓
×15.5[f]	4.56	4.00	4	0.285	5/16	3/16	1.14	8.00	8	0.435	7/16	0.829	1 1/8	↓

[c] Shape is slender for compression with $F_y = 50$ ksi.
[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.
[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.
[v] Shear strength controlled by buckling effects ($C_{v2} < 1.0$) with $F_y = 50$ ksi.

Table 1-8 (continued)
WT-Shapes
Properties

WT6–WT4

Nom-inal Wt.	Compact Section Criteria		Axis X-X						Axis Y-Y				Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{d}{t_w}$	I	S	r	$\bar{y}$	Z	y_p	I	S	r	Z	J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{6}
17.5	6.31	20.8	16.0	3.23	1.76	1.30	5.71	0.394	12.2	3.73	1.54	5.73	0.369	0.437
15	7.41	23.7	13.5	2.75	1.75	1.27	4.83	0.337	10.2	3.12	1.52	4.78	0.228	0.267
13	8.54	26.6	11.7	2.40	1.75	1.25	4.20	0.295	8.66	2.67	1.51	4.08	0.150	0.174
11	4.74	23.7	11.7	2.59	1.90	1.63	4.63	0.402	2.33	1.15	0.847	1.83	0.146	0.137
9.5	5.72	25.9	10.1	2.28	1.90	1.65	4.11	0.348	1.88	0.939	0.821	1.49	0.0899	0.0934
8	7.53	27.3	8.70	2.04	1.92	1.74	3.72	0.639	1.41	0.706	0.773	1.13	0.0511	0.0678
7	8.82	29.8	7.67	1.83	1.92	1.76	3.32	0.760	1.18	0.593	0.753	0.947	0.0350	0.0493
56	4.17	7.52	28.6	6.40	1.32	1.21	13.4	0.791	118	22.6	2.67	34.6	7.50	16.9
50	4.62	8.16	24.5	5.56	1.29	1.13	11.4	0.711	103	20.0	2.65	30.5	5.41	11.9
44	5.18	8.96	20.8	4.77	1.27	1.06	9.65	0.631	89.3	17.4	2.63	26.5	3.75	8.02
38.5	5.86	10.0	17.4	4.05	1.24	0.990	8.06	0.555	76.8	15.1	2.60	22.9	2.55	5.31
34	6.58	11.1	14.9	3.49	1.22	0.932	6.85	0.493	66.7	13.2	2.58	20.0	1.78	3.62
30	7.41	12.2	12.9	3.04	1.21	0.884	5.87	0.438	58.1	11.5	2.57	17.5	1.23	2.46
27	8.15	13.6	11.1	2.64	1.19	0.836	5.05	0.395	51.7	10.3	2.56	15.6	0.909	1.78
24.5	8.93	14.7	10.0	2.39	1.18	0.807	4.52	0.361	46.7	9.34	2.54	14.1	0.693	1.33
22.5	6.47	14.4	10.2	2.47	1.24	0.907	4.65	0.413	26.7	6.65	2.01	10.1	0.753	0.981
19.5	7.53	15.7	8.84	2.16	1.24	0.876	3.99	0.359	22.5	5.64	1.98	8.57	0.487	0.616
16.5	9.15	16.8	7.71	1.93	1.26	0.869	3.48	0.305	18.3	4.60	1.94	7.00	0.291	0.356
15	5.70	17.5	9.28	2.24	1.45	1.10	4.01	0.380	8.35	2.87	1.37	4.41	0.310	0.273
13	6.56	19.9	7.86	1.91	1.44	1.06	3.39	0.330	7.05	2.44	1.36	3.75	0.201	0.173
11	7.99	21.2	6.88	1.72	1.46	1.07	3.02	0.282	5.71	1.99	1.33	3.05	0.119	0.107
9.5	5.09	20.5	6.68	1.74	1.54	1.28	3.10	0.349	2.15	1.07	0.874	1.67	0.116	0.0796
8.5	6.08	21.1	6.06	1.62	1.56	1.32	2.90	0.311	1.78	0.887	0.844	1.40	0.0776	0.0610
7.5	7.41	21.7	5.45	1.50	1.57	1.37	2.71	0.305	1.45	0.723	0.810	1.15	0.0518	0.0475
6	9.43	26.0	4.35	1.22	1.57	1.36	2.20	0.322	1.09	0.551	0.785	0.869	0.0272	0.0255
33.5	4.43	7.89	10.9	3.05	1.05	0.936	6.29	0.594	44.3	10.7	2.12	16.3	2.51	3.56
29	5.07	8.59	9.12	2.61	1.03	0.874	5.25	0.520	37.5	9.13	2.10	13.9	1.66	2.28
24	5.92	10.6	6.85	1.97	0.986	0.777	3.94	0.435	30.5	7.51	2.08	11.4	0.977	1.30
20	7.21	11.5	5.73	1.69	0.988	0.735	3.25	0.364	24.5	6.08	2.04	9.24	0.558	0.715
17.5	8.10	13.1	4.82	1.43	0.968	0.688	2.71	0.321	21.3	5.31	2.03	8.05	0.384	0.480
15.5	9.19	14.0	4.28	1.28	0.969	0.668	2.39	0.285	18.5	4.64	2.02	7.03	0.267	0.327

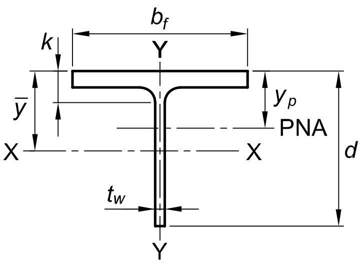


Table 1-8 (continued)
WT-Shapes
Dimensions

Shape	Area, A	Depth, d		Stem				Flange				Distance		
				Thickness, t_w		$\frac{t_w}{2}$	Area	Width, b_f		Thickness, t_f		k		Workable Gage
												k_{des}	k_{det}	
	in.^{2}	in.		in.		in.	in.^{2}	in.		in.		in.	in.	in.
WT4×14	4.12	4.03	4	0.285	5/16	3/16	1.15	6.54	6 1/2	0.465	7/16	0.859	15/16	4
×12	3.54	3.97	4	0.245	1/4	1/8	0.971	6.50	6 1/2	0.400	3/8	0.794	7/8	4
WT4×10.5	3.08	4.14	4 1/8	0.250	1/4	1/8	1.04	5.27	5 1/4	0.400	3/8	0.700	7/8	2 3/4[g]
×9	2.63	4.07	4 1/8	0.230	1/4	1/8	0.936	5.25	5 1/4	0.330	5/16	0.630	13/16	2 3/4[g]
WT4×7.5	2.22	4.06	4	0.245	1/4	1/8	0.993	4.02	4	0.315	5/16	0.615	13/16	2 1/4[g]
×6.5	1.92	4.00	4	0.230	1/4	1/8	0.919	4.00	4	0.255	1/4	0.555	3/4	
×5[c,f]	1.48	3.95	4	0.170	3/16	1/8	0.671	3.94	4	0.205	3/16	0.505	11/16	
WT3×12.5	3.67	3.19	3 1/4	0.320	5/16	3/16	1.02	6.08	6 1/8	0.455	7/16	0.705	15/16	3 1/2
×10	2.94	3.10	3 1/8	0.260	1/4	1/8	0.806	6.02	6	0.365	3/8	0.615	7/8	
×7.5[f]	2.21	3.00	3	0.230	1/4	1/8	0.689	5.99	6	0.260	1/4	0.510	3/4	
WT3×8	2.37	3.14	3 1/8	0.260	1/4	1/8	0.816	4.03	4	0.405	3/8	0.655	7/8	2 1/4[g]
×6	1.78	3.02	3	0.230	1/4	1/8	0.693	4.00	4	0.280	1/4	0.530	3/4	
×4.5[f]	1.34	2.95	3	0.170	3/16	1/8	0.502	3.94	4	0.215	3/16	0.465	11/16	
×4.25[f]	1.26	2.92	2 7/8	0.170	3/16	1/8	0.496	3.94	4	0.195	3/16	0.445	11/16	
WT2.5×9.5	2.78	2.58	2 5/8	0.270	1/4	1/8	0.695	5.03	5	0.430	7/16	0.730	13/16	2 3/4
×8	2.35	2.51	2 1/2	0.240	1/4	1/8	0.601	5.00	5	0.360	3/8	0.660	3/4	2 3/4
WT2×6.5	1.91	2.08	2 1/8	0.280	1/4	1/8	0.582	4.06	4	0.345	3/8	0.595	3/4	2 1/4

[c] Shape is slender for compression with $F_y = 50$ ksi.
[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.
[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.

Table 1-8 (continued)
WT-Shapes
Properties

WT4–WT2

Nom-inal Wt.	Compact Section Criteria		Axis X-X						Axis Y-Y				Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{d}{t_w}$	I	S	r	$\bar{y}$	Z	y_p	I	S	r	Z	J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{6}
14	7.03	14.1	4.23	1.28	1.01	0.734	2.38	0.315	10.8	3.31	1.62	5.04	0.268	0.230
12	8.12	16.2	3.53	1.08	0.999	0.695	1.98	0.272	9.14	2.81	1.61	4.28	0.173	0.144
10.5	6.59	16.6	3.90	1.18	1.12	0.831	2.11	0.292	4.88	1.85	1.26	2.84	0.141	0.0916
9	7.95	17.7	3.41	1.05	1.14	0.834	1.86	0.251	3.98	1.52	1.23	2.33	0.0855	0.0562
7.5	6.37	16.6	3.28	1.07	1.22	0.998	1.91	0.276	1.70	0.849	0.876	1.33	0.0679	0.0382
6.5	7.84	17.4	2.89	0.974	1.23	1.03	1.74	0.240	1.36	0.682	0.843	1.07	0.0433	0.0269
5	9.61	23.2	2.15	0.717	1.20	0.953	1.27	0.188	1.05	0.531	0.840	0.826	0.0212	0.0114
12.5	6.68	10.0	2.29	0.886	0.789	0.610	1.68	0.302	8.53	2.81	1.52	4.28	0.229	0.171
10	8.25	11.9	1.76	0.693	0.774	0.560	1.29	0.244	6.64	2.21	1.50	3.36	0.120	0.0858
7.5	11.5	13.0	1.41	0.577	0.797	0.558	1.03	0.185	4.66	1.56	1.45	2.37	0.0504	0.0342
8	4.98	12.1	1.69	0.685	0.844	0.676	1.25	0.294	2.21	1.10	0.966	1.69	0.111	0.0426
6	7.14	13.1	1.32	0.564	0.862	0.677	1.01	0.222	1.50	0.748	0.918	1.16	0.0449	0.0178
4.5	9.16	17.4	0.950	0.408	0.842	0.623	0.720	0.170	1.10	0.557	0.905	0.856	0.0202	0.00736
4.25	10.1	17.2	0.905	0.397	0.848	0.637	0.700	0.160	0.995	0.505	0.890	0.778	0.0166	0.00620
9.5	5.85	9.56	1.01	0.485	0.604	0.487	0.970	0.276	4.56	1.81	1.28	2.76	0.157	0.0775
8	6.94	10.5	0.845	0.413	0.599	0.458	0.801	0.235	3.75	1.50	1.26	2.28	0.0958	0.0453
6.5	5.88	7.43	0.526	0.321	0.524	0.440	0.616	0.236	1.93	0.950	1.00	1.46	0.0750	0.0233

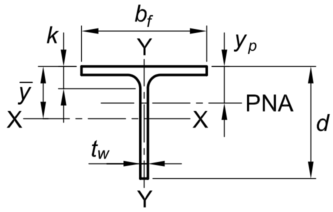


Table 1-9
MT-Shapes
Dimensions

Shape	Area, A	Depth, d		Stem				Flange				Distance	
				Thickness, t_w		$\frac{t_w}{2}$	Area	Width, b_f		Thickness, t_f		k	Workable Gage
	in.^{2}	in.		in.		in.	in.^{2}	in.		in.		in.	in.
MT6.25×6.2[c,v]	1.82	6.27	6 1/4	0.155	1/8	1/16	0.971	3.75	3 3/4	0.228	1/4	9/16	–
×5.8[c,v]	1.70	6.25	6 1/4	0.155	1/8	1/16	0.969	3.50	3 1/2	0.211	3/16	9/16	–
MT6×5.9[c]	1.74	6.00	6	0.177	3/16	1/8	1.06	3.07	3 1/8	0.225	1/4	9/16	–
×5.4[c,v]	1.59	5.99	6	0.160	3/16	1/8	0.958	3.07	3 1/8	0.210	3/16	9/16	–
×5[c,v]	1.48	5.99	6	0.149	1/8	1/16	0.892	3.25	3 1/4	0.180	3/16	1/2	–
MT5×4.5[c]	1.33	5.00	5	0.157	3/16	1/8	0.785	2.69	2 3/4	0.206	3/16	9/16	–
×4[c,v]	1.19	4.98	5	0.141	1/8	1/16	0.701	2.69	2 3/4	0.182	3/16	9/16	–
MT5×3.75[c,v]	1.11	5.00	5	0.130	1/8	1/16	0.649	2.69	2 3/4	0.173	3/16	7/16	–
MT4×3.25[c]	0.959	4.00	4	0.135	1/8	1/16	0.540	2.28	2 1/4	0.189	3/16	9/16	–
×3.1[c]	0.911	4.00	4	0.129	1/8	1/16	0.516	2.28	2 1/4	0.177	3/16	7/16	–
MT3×2.2[c]	0.647	3.00	3	0.114	1/8	1/16	0.342	1.84	1 7/8	0.171	3/16	3/8	–
×1.85[c]	0.545	2.96	3	0.0980	1/8	1/16	0.290	2.00	2	0.129	1/8	5/16	–
MT2.5×9.45[t]	2.78	2.50	2 1/2	0.316	5/16	3/16	0.790	5.00	5	0.416	7/16	13/16	2 3/4[g]
MT2×3[f]	0.875	1.90	1 7/8	0.130	1/8	1/16	0.247	3.80	3 3/4	0.160	3/16	1/2	–

[c] Shape is slender for compression with $F_y = 36$ ksi.
[f] Shape exceeds compact limit for flexure with $F_y = 36$ ksi.
[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.
[t] Shape has tapered flanges while all other MT-shapes have parallel flange surfaces.
[v] Shear strength controlled by buckling effects ($C_{v2} < 1.0$) with $F_y = 36$ ksi.
– Indicates flange is too narrow to establish a workable gage.

Table 1-9 (continued)
MT-Shapes
Properties

MT-SHAPES

Nominal Wt.	Compact Section Criteria		Axis X-X						Axis Y-Y				Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{d}{t_w}$	I	S	r	$\bar{y}$	Z	y_p	I	S	r	Z	J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{6}
6.2	8.22	40.4	7.29	1.61	2.01	1.74	2.92	0.372	1.00	0.536	0.746	0.839	0.0246	0.0284
5.8	8.29	40.3	6.94	1.57	2.03	1.84	2.86	0.808	0.756	0.432	0.669	0.684	0.0206	0.0268
5.9	6.82	33.9	6.61	1.61	1.96	1.89	2.89	1.13	0.543	0.354	0.561	0.575	0.0249	0.0337
5.4	7.31	37.4	6.03	1.46	1.95	1.86	2.63	1.05	0.506	0.330	0.566	0.532	0.0196	0.0250
5	9.03	40.2	5.62	1.36	1.96	1.86	2.45	1.08	0.517	0.318	0.594	0.509	0.0145	0.0202
4.5	6.53	31.8	3.47	1.00	1.62	1.54	1.81	0.808	0.336	0.250	0.505	0.403	0.0156	0.0138
4	7.39	35.3	3.08	0.894	1.62	1.52	1.61	0.809	0.296	0.220	0.502	0.354	0.0112	0.00989
3.75	7.77	38.4	2.91	0.836	1.63	1.51	1.51	0.759	0.281	0.209	0.505	0.334	0.00932	0.00792
3.25	6.03	29.6	1.57	0.558	1.29	1.18	1.01	0.472	0.188	0.165	0.444	0.264	0.00917	0.00463
3.1	6.44	31.0	1.50	0.533	1.29	1.18	0.967	0.497	0.176	0.154	0.441	0.247	0.00778	0.00403
2.2	5.38	26.3	0.579	0.268	0.949	0.841	0.483	0.190	0.0897	0.0973	0.374	0.155	0.00494	0.00124
1.85	7.75	30.2	0.483	0.226	0.945	0.827	0.409	0.174	0.0863	0.0863	0.400	0.136	0.00265	0.000754
9.45	6.01	7.91	1.05	0.528	0.617	0.512	1.03	0.276	4.35	1.74	1.26	2.66	0.156	0.0732
3	11.9	14.6	0.208	0.133	0.493	0.341	0.241	0.112	0.732	0.385	0.926	0.588	0.00919	0.00193

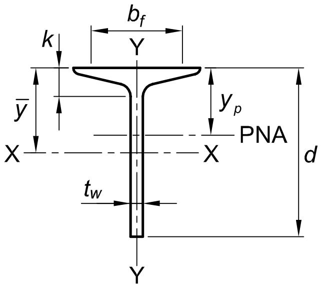


Table 1-10
ST-Shapes
Dimensions

Shape	Area, A	Depth, d		Stem: Thickness, t_w		Stem: $\frac{t_w}{2}$	Stem: Area	Flange: Width, b_f		Flange: Thickness, t_f		Distance: k	Distance: Workable Gage
	in.^{2}	in.		in.		in.	in.^{2}	in.		in.		in.	in.
ST12×60.5	17.8	12.3	12 1/4	0.800	13/16	7/16	9.80	8.05	8	1.09	1 1/16	2	4
×53	15.6	12.3	12 1/4	0.620	5/8	5/16	7.60	7.87	7 7/8	1.09	1 1/16	2	4
ST12×50	14.7	12.0	12	0.745	3/4	3/8	8.94	7.25	7 1/4	0.870	7/8	1 3/4	4
×45	13.2	12.0	12	0.625	5/8	5/16	7.50	7.13	7 1/8	0.870	7/8	1 3/4	4
×40[c]	11.7	12.0	12	0.500	1/2	1/4	6.00	7.00	7	0.870	7/8	1 3/4	4
ST10×48	14.1	10.2	10 1/8	0.800	13/16	7/16	8.12	7.20	7 1/4	0.920	15/16	1 3/4	4
×43	12.7	10.2	10 1/8	0.660	11/16	3/8	6.70	7.06	7	0.920	15/16	1 3/4	4
ST10×37.5	11.0	10.0	10	0.635	5/8	5/16	6.35	6.39	6 3/8	0.795	13/16	1 5/8	3 1/2[g]
×33	9.70	10.0	10	0.505	1/2	1/4	5.05	6.26	6 1/4	0.795	13/16	1 5/8	3 1/2[g]
ST9×35	10.3	9.00	9	0.711	11/16	3/8	6.40	6.25	6 1/4	0.691	11/16	1 1/2	3 1/2[g]
×27.35	8.02	9.00	9	0.461	7/16	1/4	4.15	6.00	6	0.691	11/16	1 1/2	3 1/2[g]
ST7.5×25	7.34	7.50	7 1/2	0.550	9/16	5/16	4.13	5.64	5 5/8	0.622	5/8	1 3/8	3 1/2[g]
×21.45	6.30	7.50	7 1/2	0.411	7/16	1/4	3.08	5.50	5 1/2	0.622	5/8	1 3/8	3 1/2[g]
ST6×25	7.33	6.00	6	0.687	11/16	3/8	4.12	5.48	5 1/2	0.659	11/16	1 7/16	3[g]
×20.4	5.96	6.00	6	0.462	7/16	1/4	2.77	5.25	5 1/4	0.659	11/16	1 7/16	3[g]
ST6×17.5	5.12	6.00	6	0.428	7/16	1/4	2.57	5.08	5 1/8	0.544	9/16	1 3/16	3[g]
×15.9	4.65	6.00	6	0.350	3/8	3/16	2.10	5.00	5	0.544	9/16	1 3/16	3[g]
ST5×17.5	5.14	5.00	5	0.594	5/8	5/16	2.97	4.94	5	0.491	1/2	1 1/8	2 3/4[g]
×12.7	3.72	5.00	5	0.311	5/16	3/16	1.56	4.66	4 5/8	0.491	1/2	1 1/8	2 3/4[g]
ST4×11.5	3.38	4.00	4	0.441	7/16	1/4	1.76	4.17	4 1/8	0.425	7/16	1	2 1/4[g]
×9.2	2.70	4.00	4	0.271	1/4	1/8	1.08	4.00	4	0.425	7/16	1	2 1/4[g]
ST3×8.6	2.53	3.00	3	0.465	7/16	1/4	1.40	3.57	3 5/8	0.359	3/8	13/16	–
×6.25	1.83	3.00	3	0.232	1/4	1/8	0.696	3.33	3 3/8	0.359	3/8	13/16	–
ST2.5×5	1.46	2.50	2 1/2	0.214	3/16	1/8	0.535	3.00	3	0.326	5/16	3/4	–
ST2×4.75	1.40	2.00	2	0.326	5/16	3/16	0.652	2.80	2 3/4	0.293	5/16	3/4	–
×3.85	1.13	2.00	2	0.193	3/16	1/8	0.386	2.66	2 5/8	0.293	5/16	3/4	–
ST1.5×3.75	1.10	1.50	1 1/2	0.349	3/8	3/16	0.524	2.51	2 1/2	0.260	1/4	5/8	–
×2.85	0.830	1.50	1 1/2	0.170	3/16	1/8	0.255	2.33	2 3/8	0.260	1/4	5/8	–

[c] Shape is slender for compression with F_y = 36 ksi.
[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross section to ensure compatibility.
– Indicates flange is too narrow to establish a workable gage.

Table 1-10 (continued)
ST-Shapes
Properties

ST-SHAPES

Nom-inal Wt.	Compact Section Criteria		Axis X-X						Axis Y-Y				Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{d}{t_w}$	I	S	r	$\bar{y}$	Z	y_p	I	S	r	Z	J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{6}
60.5	3.69	15.4	259	30.1	3.82	3.63	54.5	1.26	41.5	10.3	1.53	18.1	6.38	27.5
53	3.61	19.8	216	24.1	3.72	3.28	43.3	1.02	38.4	9.76	1.57	16.7	5.05	15.0
50	4.17	16.1	215	26.3	3.83	3.84	47.5	2.16	23.7	6.55	1.27	12.0	3.76	19.5
45	4.10	19.2	190	22.6	3.79	3.60	41.1	1.42	22.3	6.27	1.30	11.2	3.01	12.1
40	4.02	24.0	162	18.6	3.72	3.30	33.6	0.909	21.0	6.00	1.34	10.4	2.44	6.94
48	3.91	12.7	143	20.3	3.18	3.13	36.9	1.35	25.0	6.93	1.33	12.5	4.16	15.0
43	3.84	15.4	124	17.2	3.13	2.91	31.1	0.972	23.3	6.59	1.36	11.6	3.30	9.17
37.5	4.02	15.7	109	15.8	3.15	3.07	28.6	1.34	14.8	4.62	1.16	8.36	2.28	7.21
33	3.94	19.8	92.9	12.9	3.10	2.81	23.4	0.841	13.7	4.39	1.19	7.70	1.78	4.02
35	4.52	12.7	84.5	14.0	2.87	2.94	25.1	1.78	12.0	3.84	1.08	7.17	2.02	7.03
27.35	4.34	19.5	62.3	9.60	2.79	2.51	17.3	0.737	10.4	3.45	1.14	6.06	1.16	2.26
25	4.53	13.6	40.5	7.72	2.35	2.25	14.0	0.826	7.79	2.76	1.03	4.99	1.05	2.02
21.45	4.42	18.2	32.9	5.99	2.29	2.01	10.8	0.605	7.13	2.59	1.06	4.54	0.765	0.995
25	4.17	8.73	25.1	6.04	1.85	1.84	11.0	0.758	7.79	2.84	1.03	5.16	1.36	1.97
20.4	3.98	13.0	18.9	4.27	1.78	1.58	7.71	0.577	6.74	2.57	1.06	4.43	0.842	0.787
17.5	4.67	14.0	17.2	3.95	1.83	1.65	7.12	0.543	4.92	1.94	0.980	3.40	0.524	0.556
15.9	4.60	17.1	14.8	3.30	1.78	1.51	5.94	0.480	4.66	1.87	1.00	3.22	0.438	0.364
17.5	5.03	8.42	12.5	3.62	1.56	1.56	6.58	0.673	4.15	1.68	0.899	3.10	0.633	0.725
12.7	4.75	16.1	7.79	2.05	1.45	1.20	3.70	0.403	3.36	1.44	0.950	2.49	0.300	0.173
11.5	4.91	9.07	5.00	1.76	1.22	1.15	3.19	0.439	2.13	1.02	0.795	1.84	0.271	0.168
9.2	4.71	14.8	3.49	1.14	1.14	0.942	2.07	0.336	1.84	0.922	0.827	1.59	0.167	0.0642
8.6	4.97	6.45	2.12	1.02	0.915	0.915	1.85	0.394	1.14	0.642	0.673	1.17	0.181	0.0772
6.25	4.64	12.9	1.26	0.547	0.831	0.692	1.01	0.271	0.901	0.541	0.702	0.930	0.0830	0.0197
5	4.60	11.7	0.671	0.348	0.677	0.570	0.650	0.239	0.597	0.398	0.638	0.686	0.0568	0.0100
4.75	4.78	6.13	0.462	0.319	0.575	0.553	0.592	0.250	0.444	0.317	0.564	0.565	0.0590	0.00995
3.85	4.54	10.4	0.307	0.198	0.522	0.448	0.381	0.204	0.374	0.281	0.576	0.485	0.0364	0.00457
3.75	4.83	4.30	0.200	0.187	0.426	0.432	0.351	0.219	0.289	0.230	0.513	0.411	0.0432	0.00496
2.85	4.48	8.82	0.114	0.0970	0.370	0.329	0.196	0.171	0.223	0.192	0.518	0.328	0.0216	0.00189

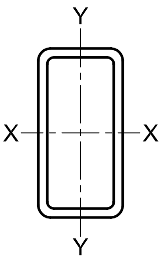


Table 1-11
Rectangular HSS
Dimensions and Properties

Shape	Design Wall Thick-ness, *t*	Nominal Wt.	Area, *A*	*b/t*	*h/t*	Axis X-X			
						I	*S*	*r*	*Z*
	in.	lb/ft	in.^{2}			in.^{4}	in.^{3}	in.	in.^{3}
HSS24×12×3/4	0.698	171.16	47.1	14.2	31.4	3440	287	8.55	359
×5/8	0.581	144.39	39.6	17.7	38.4	2940	245	8.62	304
×1/2	0.465	116.91	32.1	22.8	48.6	2420	202	8.68	248
HSS20×12×3/4	0.698	150.75	41.5	14.2	25.6	2190	219	7.26	270
×5/8	0.581	127.37	35.0	17.7	31.4	1880	188	7.33	230
×1/2	0.465	103.30	28.3	22.8	40.0	1550	155	7.39	188
×3/8	0.349	78.52	21.5	31.4	54.3	1200	120	7.45	144
×5/16	0.291	65.87	18.1	38.2	65.7	1010	101	7.48	122
HSS20×8×5/8	0.581	110.36	30.3	10.8	31.4	1440	144	6.89	185
×1/2	0.465	89.68	24.6	14.2	40.0	1190	119	6.96	152
×3/8	0.349	68.31	18.7	19.9	54.3	926	92.6	7.03	117
×5/16	0.291	57.36	15.7	24.5	65.7	786	78.6	7.07	98.6
HSS20×4×1/2	0.465	76.07	20.9	5.60	40.0	838	83.8	6.33	115
×3/8	0.349	58.10	16.0	8.46	54.3	657	65.7	6.42	89.3
×5/16	0.291	48.86	13.4	10.7	65.7	560	56.0	6.46	75.6
×1/4	0.233	39.43	10.8	14.2	82.8	458	45.8	6.50	61.5
HSS18×6×5/8	0.581	93.34	25.7	7.33	28.0	923	103	6.00	135
×1/2	0.465	76.07	20.9	9.90	35.7	770	85.6	6.07	112
×3/8	0.349	58.10	16.0	14.2	48.6	602	66.9	6.15	86.4
×5/16	0.291	48.86	13.4	17.6	58.9	513	57.0	6.18	73.1
×1/4	0.233	39.43	10.8	22.8	74.3	419	46.5	6.22	59.4
HSS16×12×3/4	0.698	130.33	35.9	14.2	19.9	1270	159	5.95	193
×5/8	0.581	110.36	30.3	17.7	24.5	1090	136	6.00	165
×1/2	0.465	89.68	24.6	22.8	31.4	904	113	6.06	135
×3/8	0.349	68.31	18.7	31.4	42.8	702	87.7	6.12	104
×5/16	0.291	57.36	15.7	38.2	52.0	595	74.4	6.15	87.7
HSS16×8×5/8	0.581	93.34	25.7	10.8	24.5	815	102	5.64	129
×1/2	0.465	76.07	20.9	14.2	31.4	679	84.9	5.70	106
×3/8	0.349	58.10	16.0	19.9	42.8	531	66.3	5.77	82.1
×5/16	0.291	48.86	13.4	24.5	52.0	451	56.4	5.80	69.4
×1/4	0.233	39.43	10.8	31.3	65.7	368	46.1	5.83	56.4

Note: For width-to-thickness criteria, refer to Table 1-12A.

Table 1-11 (continued)
Rectangular HSS
Dimensions and Properties

HSS24–HSS16

Shape	Axis Y-Y				Workable Flat		Torsion		Surface Area
	I	*S*	*r*	*Z*	Depth	Width	*J*	*C*	
	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{3}	ft^2/ft
HSS24×12×3/4	1170	195	4.98	221	20 5/8	8 5/8	2850	366	5.80
×5/8	1000	167	5.03	188	21 3/16	9 3/16	2430	310	5.83
×1/2	829	138	5.08	154	21 3/4	9 3/4	1980	252	5.87
HSS20×12×3/4	988	165	4.88	190	16 5/8	8 5/8	2220	303	5.13
×5/8	851	142	4.93	162	17 3/16	9 3/16	1890	257	5.17
×1/2	705	117	4.99	132	17 3/4	9 3/4	1540	209	5.20
×3/8	547	91.1	5.04	102	18 5/16	10 5/16	1180	160	5.23
×5/16	464	77.3	5.07	85.8	18 5/8	10 5/8	997	134	5.25
HSS20×8×5/8	338	84.6	3.34	96.4	17 3/16	5 3/16	916	167	4.50
×1/2	283	70.8	3.39	79.5	17 3/4	5 3/4	757	137	4.53
×3/8	222	55.6	3.44	61.5	18 5/16	6 5/16	586	105	4.57
×5/16	189	47.4	3.47	52.0	18 5/8	6 5/8	496	88.3	4.58
HSS20×4×1/2	58.7	29.3	1.68	34.0	17 3/4	–	195	63.8	3.87
×3/8	47.6	23.8	1.73	26.8	18 5/16	2 5/16	156	49.9	3.90
×5/16	41.2	20.6	1.75	22.9	18 5/8	2 5/8	134	42.4	3.92
×1/4	34.3	17.1	1.78	18.7	18 7/8	2 7/8	111	34.7	3.93
HSS18×6×5/8	158	52.7	2.48	61.0	15 3/16	3 3/16	462	109	3.83
×1/2	134	44.6	2.53	50.7	15 3/4	3 3/4	387	89.9	3.87
×3/8	106	35.5	2.58	39.5	16 5/16	4 5/16	302	69.5	3.90
×5/16	91.3	30.4	2.61	33.5	16 9/16	4 9/16	257	58.7	3.92
×1/4	75.1	25.0	2.63	27.3	16 7/8	4 7/8	210	47.7	3.93
HSS16x12×3/4	810	135	4.75	158	12 5/8	8 5/8	1610	240	4.47
×5/8	700	117	4.80	135	13 3/16	9 3/16	1370	204	4.50
×1/2	581	96.8	4.86	111	13 3/4	9 3/4	1120	166	4.53
×3/8	452	75.3	4.91	85.5	14 5/16	10 5/16	862	127	4.57
×5/16	384	64.0	4.94	72.2	14 5/8	10 5/8	727	107	4.58
HSS16×8×5/8	274	68.6	3.27	79.2	13 3/16	5 3/16	681	132	3.83
×1/2	230	57.6	3.32	65.5	13 3/4	5 3/4	563	108	3.87
×3/8	181	45.3	3.37	50.8	14 5/16	6 5/16	436	83.4	3.90
×5/16	155	38.7	3.40	43.0	14 5/8	6 5/8	369	70.4	3.92
×1/4	127	31.7	3.42	35.0	14 7/8	6 7/8	300	57.0	3.93

– Indicates flat depth or width is too small to establish a workable flat.

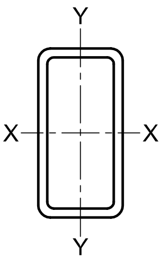


Table 1-11 (continued)
Rectangular HSS
Dimensions and Properties

Shape	Design Wall Thickness, *t*	Nominal Wt.	Area, *A*	*b/t*	*h/t*	Axis X-X			
						I	*S*	*r*	*Z*
	in.	lb/ft	in.^{2}			in.^{4}	in.^{3}	in.	in.^{3}
HSS16×4×5/8	0.581	76.33	21.0	3.88	24.5	539	67.3	5.06	92.9
×1/2	0.465	62.46	17.2	5.60	31.4	455	56.9	5.15	77.3
×3/8	0.349	47.90	13.2	8.46	42.8	360	45.0	5.23	60.2
×5/16	0.291	40.35	11.1	10.7	52.0	308	38.5	5.27	51.1
×1/4	0.233	32.63	8.96	14.2	65.7	253	31.6	5.31	41.7
×3/16	0.174	24.73	6.76	20.0	89.0	193	24.2	5.35	31.7
HSS14×10×5/8	0.581	93.34	25.7	14.2	21.1	687	98.2	5.17	120
×1/2	0.465	76.07	20.9	18.5	27.1	573	81.8	5.23	98.8
×3/8	0.349	58.10	16.0	25.7	37.1	447	63.9	5.29	76.3
×5/16	0.291	48.86	13.4	31.4	45.1	380	54.3	5.32	64.6
×1/4	0.233	39.43	10.8	39.9	57.1	310	44.3	5.35	52.4
HSS14×6×5/8	0.581	76.33	21.0	7.33	21.1	478	68.3	4.77	88.7
×1/2	0.465	62.46	17.2	9.90	27.1	402	57.4	4.84	73.6
×3/8	0.349	47.90	13.2	14.2	37.1	317	45.3	4.91	57.3
×5/16	0.291	40.35	11.1	17.6	45.1	271	38.7	4.94	48.6
×1/4	0.233	32.63	8.96	22.8	57.1	222	31.7	4.98	39.6
×3/16	0.174	24.73	6.76	31.5	77.5	170	24.3	5.01	30.1
HSS14×4×5/8	0.581	67.82	18.7	3.88	21.1	373	53.3	4.47	73.1
×1/2	0.465	55.66	15.3	5.60	27.1	317	45.3	4.55	61.0
×3/8	0.349	42.79	11.8	8.46	37.1	252	36.0	4.63	47.8
×5/16	0.291	36.10	9.92	10.7	45.1	216	30.9	4.67	40.6
×1/4	0.233	29.23	8.03	14.2	57.1	178	25.4	4.71	33.2
×3/16	0.174	22.18	6.06	20.0	77.5	137	19.5	4.74	25.3
HSS12×10×1/2	0.465	69.27	19.0	18.5	22.8	395	65.9	4.56	78.8
×3/8	0.349	53.00	14.6	25.7	31.4	310	51.6	4.61	61.1
×5/16	0.291	44.60	12.2	31.4	38.2	264	44.0	4.64	51.7
×1/4	0.233	36.03	9.90	39.9	48.5	216	36.0	4.67	42.1
HSS12×8×5/8	0.581	76.33	21.0	10.8	17.7	397	66.1	4.34	82.1
×1/2	0.465	62.46	17.2	14.2	22.8	333	55.6	4.41	68.1
×3/8	0.349	47.90	13.2	19.9	31.4	262	43.7	4.47	53.0
×5/16	0.291	40.35	11.1	24.5	38.2	224	37.4	4.50	44.9
×1/4	0.233	32.63	8.96	31.3	48.5	184	30.6	4.53	36.6
×3/16	0.174	24.73	6.76	43.0	66.0	140	23.4	4.56	27.8

Note: For width-to-thickness criteria, refer to Table 1-12A.

Table 1-11 (continued)
Rectangular HSS
Dimensions and Properties

HSS16-HSS12

Shape	Axis Y-Y				Workable Flat		Torsion		Surface Area
	I	S	r	Z	Depth	Width	J	C	
	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{3}	ft^2/ft
HSS16×4×$\frac{5}{8}$	54.1	27.0	1.60	32.5	$13\frac{3}{16}$	–	174	60.5	3.17
×$\frac{1}{2}$	47.0	23.5	1.65	27.4	$13\frac{3}{4}$	–	150	50.7	3.20
×$\frac{3}{8}$	38.3	19.1	1.71	21.7	$14\frac{5}{16}$	$2\frac{5}{16}$	120	39.7	3.23
×$\frac{5}{16}$	33.2	16.6	1.73	18.5	$14\frac{5}{8}$	$2\frac{5}{8}$	103	33.8	3.25
×$\frac{1}{4}$	27.7	13.8	1.76	15.2	$14\frac{7}{8}$	$2\frac{7}{8}$	85.2	27.6	3.27
×$\frac{3}{16}$	21.5	10.8	1.78	11.7	$15\frac{3}{16}$	$3\frac{3}{16}$	65.5	21.1	3.28
HSS14×10×$\frac{5}{8}$	407	81.5	3.98	95.1	$11\frac{3}{16}$	$7\frac{3}{16}$	832	146	3.83
×$\frac{1}{2}$	341	68.1	4.04	78.5	$11\frac{3}{4}$	$7\frac{3}{4}$	685	120	3.87
×$\frac{3}{8}$	267	53.4	4.09	60.7	$12\frac{5}{16}$	$8\frac{5}{16}$	528	91.8	3.90
×$\frac{5}{16}$	227	45.5	4.12	51.4	$12\frac{9}{16}$	$8\frac{9}{16}$	446	77.4	3.92
×$\frac{1}{4}$	186	37.2	4.14	41.8	$12\frac{7}{8}$	$8\frac{7}{8}$	362	62.6	3.93
HSS14×6×$\frac{5}{8}$	124	41.2	2.43	48.4	$11\frac{3}{16}$	$3\frac{3}{16}$	334	83.7	3.17
×$\frac{1}{2}$	105	35.1	2.48	40.4	$11\frac{3}{4}$	$3\frac{3}{4}$	279	69.3	3.20
×$\frac{3}{8}$	84.1	28.0	2.53	31.6	$12\frac{5}{16}$	$4\frac{5}{16}$	219	53.7	3.23
×$\frac{5}{16}$	72.3	24.1	2.55	26.9	$12\frac{9}{16}$	$4\frac{9}{16}$	186	45.5	3.25
×$\frac{1}{4}$	59.6	19.9	2.58	22.0	$12\frac{7}{8}$	$4\frac{7}{8}$	152	36.9	3.27
×$\frac{3}{16}$	45.9	15.3	2.61	16.7	$13\frac{3}{16}$	$5\frac{3}{16}$	116	28.0	3.28
HSS14×4×$\frac{5}{8}$	47.2	23.6	1.59	28.5	$11\frac{1}{4}$	–	148	52.6	2.83
×$\frac{1}{2}$	41.2	20.6	1.64	24.1	$11\frac{3}{4}$	–	127	44.1	2.87
×$\frac{3}{8}$	33.6	16.8	1.69	19.1	$12\frac{1}{4}$	$2\frac{1}{4}$	102	34.6	2.90
×$\frac{5}{16}$	29.2	14.6	1.72	16.4	$12\frac{5}{8}$	$2\frac{5}{8}$	87.7	29.5	2.92
×$\frac{1}{4}$	24.4	12.2	1.74	13.5	$12\frac{7}{8}$	$2\frac{7}{8}$	72.4	24.1	2.93
×$\frac{3}{16}$	19.0	9.48	1.77	10.3	$13\frac{1}{8}$	$3\frac{1}{8}$	55.8	18.4	2.95
HSS12×10×$\frac{1}{2}$	298	59.7	3.96	69.6	$9\frac{3}{4}$	$7\frac{3}{4}$	545	102	3.53
×$\frac{3}{8}$	234	46.9	4.01	54.0	$10\frac{5}{16}$	$8\frac{5}{16}$	421	78.3	3.57
×$\frac{5}{16}$	200	40.0	4.04	45.7	$10\frac{9}{16}$	$8\frac{9}{16}$	356	66.1	3.58
×$\frac{1}{4}$	164	32.7	4.07	37.2	$10\frac{7}{8}$	$8\frac{7}{8}$	289	53.5	3.60
HSS12×8×$\frac{5}{8}$	210	52.5	3.16	61.9	$9\frac{3}{16}$	$5\frac{3}{16}$	454	97.7	3.17
×$\frac{1}{2}$	178	44.4	3.21	51.5	$9\frac{3}{4}$	$5\frac{3}{4}$	377	80.4	3.20
×$\frac{3}{8}$	140	35.1	3.27	40.1	$10\frac{5}{16}$	$6\frac{5}{16}$	293	62.1	3.23
×$\frac{5}{16}$	120	30.1	3.29	34.1	$10\frac{9}{16}$	$6\frac{9}{16}$	248	52.4	3.25
×$\frac{1}{4}$	98.8	24.7	3.32	27.8	$10\frac{7}{8}$	$6\frac{7}{8}$	202	42.5	3.27
×$\frac{3}{16}$	75.7	18.9	3.35	21.1	$11\frac{1}{8}$	$7\frac{1}{8}$	153	32.2	3.28

– Indicates flat depth or width is too small to establish a workable flat.

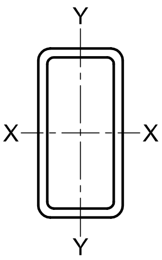
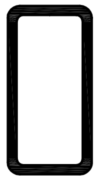


Table 1-11 (continued)
Rectangular HSS
Dimensions and Properties

Shape	Design Wall Thick-ness, *t*	Nominal Wt.	Area, *A*	*b/t*	*h/t*	Axis X-X			
						I	*S*	*r*	*Z*
	in.	lb/ft	$in.^2$			$in.^4$	$in.^3$	in.	$in.^3$
HSS12×6×5/8	0.581	67.82	18.7	7.33	17.7	321	53.4	4.14	68.8
×1/2	0.465	55.66	15.3	9.90	22.8	271	45.2	4.21	57.4
×3/8	0.349	42.79	11.8	14.2	31.4	215	35.9	4.28	44.8
×5/16	0.291	36.10	9.92	17.6	38.2	184	30.7	4.31	38.1
×1/4	0.233	29.23	8.03	22.8	48.5	151	25.2	4.34	31.1
×3/16	0.174	22.18	6.06	31.5	66.0	116	19.4	4.38	23.7
HSS12×4×5/8	0.581	59.32	16.4	3.88	17.7	245	40.8	3.87	55.5
×1/2	0.465	48.85	13.5	5.60	22.8	210	34.9	3.95	46.7
×3/8	0.349	37.69	10.4	8.46	31.4	168	28.0	4.02	36.7
×5/16	0.291	31.84	8.76	10.7	38.2	144	24.1	4.06	31.3
×1/4	0.233	25.82	7.10	14.2	48.5	119	19.9	4.10	25.6
×3/16	0.174	19.63	5.37	20.0	66.0	91.8	15.3	4.13	19.6
HSS12×3 1/2×3/8	0.349	36.41	10.0	7.03	31.4	156	26.0	3.94	34.7
×5/16	0.291	30.78	8.46	9.03	38.2	134	22.4	3.98	29.6
HSS12×3×5/16	0.291	29.72	8.17	7.31	38.2	124	20.7	3.90	27.9
×1/4	0.233	24.12	6.63	9.88	48.5	103	17.2	3.94	22.9
×3/16	0.174	18.35	5.02	14.2	66.0	79.6	13.3	3.98	17.5
HSS12×2×5/16	0.291	27.59	7.59	3.87	38.2	104	17.4	3.71	24.5
×1/4	0.233	22.42	6.17	5.58	48.5	86.9	14.5	3.75	20.1
×3/16	0.174	17.08	4.67	8.49	66.0	67.4	11.2	3.80	15.5
HSS10×8×5/8	0.581	67.82	18.7	10.8	14.2	253	50.5	3.68	62.2
×1/2	0.465	55.66	15.3	14.2	18.5	214	42.7	3.73	51.9
×3/8	0.349	42.79	11.8	19.9	25.7	169	33.9	3.79	40.5
×5/16	0.291	36.10	9.92	24.5	31.4	145	29.0	3.82	34.4
×1/4	0.233	29.23	8.03	31.3	39.9	119	23.8	3.85	28.1
×3/16	0.174	22.18	6.06	43.0	54.5	91.4	18.3	3.88	21.4
HSS10×6×5/8	0.581	59.32	16.4	7.33	14.2	201	40.2	3.50	51.3
×1/2	0.465	48.85	13.5	9.90	18.5	171	34.3	3.57	43.0
×3/8	0.349	37.69	10.4	14.2	25.7	137	27.4	3.63	33.8
×5/16	0.291	31.84	8.76	17.6	31.4	118	23.5	3.66	28.8
×1/4	0.233	25.82	7.10	22.8	39.9	96.9	19.4	3.69	23.6
×3/16	0.174	19.63	5.37	31.5	54.5	74.6	14.9	3.73	18.0

Note: For width-to-thickness criteria, refer to Table 1-12A.

Table 1-11 (continued)
Rectangular HSS
Dimensions and Properties



HSS12–HSS10

Shape	Axis Y-Y				Workable Flat		Torsion		Surface Area
	I	*S*	*r*	*Z*	Depth	Width	*J*	*C*	
	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{3}	ft^2/ft
HSS12×6×5/8	107	35.5	2.39	42.1	9 3/16	3 3/16	271	71.1	2.83
×1/2	91.1	30.4	2.44	35.2	9 3/4	3 3/4	227	59.0	2.87
×3/8	72.9	24.3	2.49	27.7	10 5/16	4 5/16	178	45.8	2.90
×5/16	62.8	20.9	2.52	23.6	10 9/16	4 9/16	152	38.8	2.92
×1/4	51.9	17.3	2.54	19.3	10 7/8	4 7/8	124	31.6	2.93
×3/16	40.0	13.3	2.57	14.7	11 3/16	5 3/16	94.6	24.0	2.95
HSS12×4×5/8	40.4	20.2	1.57	24.5	9 3/16	–	122	44.6	2.50
×1/2	35.3	17.7	1.62	20.9	9 3/4	–	105	37.5	2.53
×3/8	28.9	14.5	1.67	16.6	10 5/16	2 5/16	84.1	29.5	2.57
×5/16	25.2	12.6	1.70	14.2	10 5/8	2 5/8	72.4	25.2	2.58
×1/4	21.0	10.5	1.72	11.7	10 7/8	2 7/8	59.8	20.6	2.60
×3/16	16.4	8.20	1.75	9.00	11 3/16	3 3/16	46.1	15.7	2.62
HSS12×3 1/2×3/8	21.3	12.2	1.46	14.0	10 5/16	–	64.7	25.5	2.48
×5/16	18.6	10.6	1.48	12.1	10 5/8	–	56.0	21.8	2.50
HSS12×3×5/16	13.1	8.73	1.27	10.0	10 5/8	–	41.3	18.4	2.42
×1/4	11.1	7.38	1.29	8.28	10 7/8	–	34.5	15.1	2.43
×3/16	8.72	5.81	1.32	6.40	11 3/16	2 3/16	26.8	11.6	2.45
HSS12×2×5/16	5.10	5.10	0.820	6.05	10 5/8	–	17.6	11.6	2.25
×1/4	4.41	4.41	0.845	5.08	10 7/8	–	15.1	9.64	2.27
×3/16	3.55	3.55	0.872	3.97	11 3/16	–	12.0	7.49	2.28
HSS10×8×5/8	178	44.5	3.09	53.3	7 3/16	5 3/16	346	80.4	2.83
×1/2	151	37.8	3.14	44.5	7 3/4	5 3/4	288	66.4	2.87
×3/8	120	30.0	3.19	34.8	8 5/16	6 5/16	224	51.4	2.90
×5/16	103	25.7	3.22	29.6	8 5/8	6 5/8	190	43.5	2.92
×1/4	84.7	21.2	3.25	24.2	8 7/8	6 7/8	155	35.3	2.93
×3/16	65.1	16.3	3.28	18.4	9 3/16	7 3/16	118	26.7	2.95
HSS10×6×5/8	89.4	29.8	2.34	35.8	7 3/16	3 3/16	209	58.6	2.50
×1/2	76.8	25.6	2.39	30.1	7 3/4	3 3/4	176	48.7	2.53
×3/8	61.8	20.6	2.44	23.7	8 5/16	4 5/16	139	37.9	2.57
×5/16	53.3	17.8	2.47	20.2	8 5/8	4 5/8	118	32.2	2.58
×1/4	44.1	14.7	2.49	16.6	8 7/8	4 7/8	96.7	26.2	2.60
×3/16	34.1	11.4	2.52	12.7	9 3/16	5 3/16	73.8	19.9	2.62

– Indicates flat depth or width is too small to establish a workable flat.

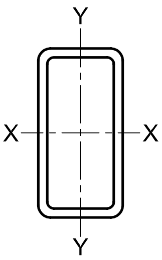


Table 1-11 (continued)
Rectangular HSS
Dimensions and Properties

Shape	Design Wall Thick-ness, *t*	Nominal Wt.	Area, *A*	*b/t*	*h/t*	Axis X-X			
						I	*S*	*r*	*Z*
	in.	lb/ft	in.^{2}			in.^{4}	in.^{3}	in.	in.^{3}
HSS10×5×3/8	0.349	35.13	9.67	11.3	25.7	120	24.1	3.53	30.4
×5/16	0.291	29.72	8.17	14.2	31.4	104	20.8	3.56	26.0
×1/4	0.233	24.12	6.63	18.5	39.9	85.8	17.2	3.60	21.3
×3/16	0.174	18.35	5.02	25.7	54.5	66.2	13.2	3.63	16.3
HSS10×4×5/8	0.581	50.81	14.0	3.88	14.2	149	29.9	3.26	40.3
×1/2	0.465	42.05	11.6	5.60	18.5	129	25.8	3.34	34.1
×3/8	0.349	32.58	8.97	8.46	25.7	104	20.8	3.41	27.0
×5/16	0.291	27.59	7.59	10.7	31.4	90.1	18.0	3.44	23.1
×1/4	0.233	22.42	6.17	14.2	39.9	74.7	14.9	3.48	19.0
×3/16	0.174	17.08	4.67	20.0	54.5	57.8	11.6	3.52	14.6
×1/8	0.116	11.56	3.16	31.5	83.2	39.8	7.97	3.55	10.0
HSS10×3 1/2×1/2	0.465	40.34	11.1	4.53	18.5	118	23.7	3.26	31.9
×3/8	0.349	31.31	8.62	7.03	25.7	96.1	19.2	3.34	25.3
×5/16	0.291	26.53	7.30	9.03	31.4	83.2	16.6	3.38	21.7
×1/4	0.233	21.57	5.93	12.0	39.9	69.1	13.8	3.41	17.9
×3/16	0.174	16.44	4.50	17.1	54.5	53.6	10.7	3.45	13.7
×1/8	0.116	11.13	3.04	27.2	83.2	37.0	7.40	3.49	9.37
HSS10×3×3/8	0.349	30.03	8.27	5.60	25.7	88.0	17.6	3.26	23.7
×5/16	0.291	25.46	7.01	7.31	31.4	76.3	15.3	3.30	20.3
×1/4	0.233	20.72	5.70	9.88	39.9	63.6	12.7	3.34	16.7
×3/16	0.174	15.80	4.32	14.2	54.5	49.4	9.87	3.38	12.8
×1/8	0.116	10.71	2.93	22.9	83.2	34.2	6.83	3.42	8.80
HSS10×2×3/8	0.349	27.48	7.58	2.73	25.7	71.7	14.3	3.08	20.3
×5/16	0.291	23.34	6.43	3.87	31.4	62.6	12.5	3.12	17.5
×1/4	0.233	19.02	5.24	5.58	39.9	52.5	10.5	3.17	14.4
×3/16	0.174	14.53	3.98	8.49	54.5	41.0	8.19	3.21	11.1
×1/8	0.116	9.86	2.70	14.2	83.2	28.5	5.70	3.25	7.65
HSS9×7×5/8	0.581	59.32	16.4	9.05	12.5	174	38.7	3.26	48.3
×1/2	0.465	48.85	13.5	12.1	16.4	149	33.0	3.32	40.5
×3/8	0.349	37.69	10.4	17.1	22.8	119	26.4	3.38	31.8
×5/16	0.291	31.84	8.76	21.1	27.9	102	22.6	3.41	27.1
×1/4	0.233	25.82	7.10	27.0	35.6	84.1	18.7	3.44	22.2
×3/16	0.174	19.63	5.37	37.2	48.7	64.7	14.4	3.47	16.9

Note: For width-to-thickness criteria, refer to Table 1-12A.

Table 1-11 (continued)
Rectangular HSS
Dimensions and Properties

HSS10–HSS9

Shape	Axis Y-Y				Workable Flat		Torsion		Surface Area
	I	S	r	Z	Depth	Width	J	C	
	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{3}	ft^2/ft
HSS10×5×$\tfrac{3}{8}$	40.6	16.2	2.05	18.7	$8\tfrac{5}{16}$	$3\tfrac{5}{16}$	100	31.2	2.40
×$\tfrac{5}{16}$	35.2	14.1	2.07	16.0	$8\tfrac{5}{8}$	$3\tfrac{5}{8}$	86.0	26.5	2.42
×$\tfrac{1}{4}$	29.3	11.7	2.10	13.2	$8\tfrac{7}{8}$	$3\tfrac{7}{8}$	70.7	21.6	2.43
×$\tfrac{3}{16}$	22.7	9.09	2.13	10.1	$9\tfrac{3}{16}$	$4\tfrac{3}{16}$	54.1	16.5	2.45
HSS10×4×$\tfrac{5}{8}$	33.5	16.8	1.54	20.6	$7\tfrac{3}{16}$	–	95.7	36.7	2.17
×$\tfrac{1}{2}$	29.5	14.7	1.59	17.6	$7\tfrac{3}{4}$	–	82.6	31.0	2.20
×$\tfrac{3}{8}$	24.3	12.1	1.64	14.0	$8\tfrac{5}{16}$	$2\tfrac{5}{16}$	66.5	24.4	2.23
×$\tfrac{5}{16}$	21.2	10.6	1.67	12.1	$8\tfrac{5}{8}$	$2\tfrac{5}{8}$	57.3	20.9	2.25
×$\tfrac{1}{4}$	17.7	8.87	1.70	10.0	$8\tfrac{7}{8}$	$2\tfrac{7}{8}$	47.4	17.1	2.27
×$\tfrac{3}{16}$	13.9	6.93	1.72	7.66	$9\tfrac{3}{16}$	$3\tfrac{3}{16}$	36.5	13.1	2.28
×$\tfrac{1}{8}$	9.65	4.83	1.75	5.26	$9\tfrac{7}{16}$	$3\tfrac{7}{16}$	25.1	8.90	2.30
HSS10×$3\tfrac{1}{2}$×$\tfrac{1}{2}$	21.4	12.2	1.39	14.7	$7\tfrac{3}{4}$	–	63.2	26.5	2.12
×$\tfrac{3}{8}$	17.8	10.2	1.44	11.8	$8\tfrac{5}{16}$	–	51.5	21.1	2.15
×$\tfrac{5}{16}$	15.6	8.92	1.46	10.2	$8\tfrac{5}{8}$	–	44.6	18.0	2.17
×$\tfrac{1}{4}$	13.1	7.51	1.49	8.45	$8\tfrac{7}{8}$	–	37.0	14.8	2.18
×$\tfrac{3}{16}$	10.3	5.89	1.51	6.52	$9\tfrac{3}{16}$	$2\tfrac{11}{16}$	28.6	11.4	2.20
×$\tfrac{1}{8}$	7.22	4.12	1.54	4.48	$9\tfrac{7}{16}$	$2\tfrac{15}{16}$	19.8	7.75	2.22
HSS10×3×$\tfrac{3}{8}$	12.4	8.28	1.22	9.73	$8\tfrac{5}{16}$	–	37.8	17.7	2.07
×$\tfrac{5}{16}$	11.0	7.30	1.25	8.42	$8\tfrac{5}{8}$	–	33.0	15.2	2.08
×$\tfrac{1}{4}$	9.28	6.19	1.28	6.99	$8\tfrac{7}{8}$	–	27.6	12.5	2.10
×$\tfrac{3}{16}$	7.33	4.89	1.30	5.41	$9\tfrac{3}{16}$	$2\tfrac{3}{16}$	21.5	9.64	2.12
×$\tfrac{1}{8}$	5.16	3.44	1.33	3.74	$9\tfrac{7}{16}$	$2\tfrac{7}{16}$	14.9	6.61	2.13
HSS10×2×$\tfrac{3}{8}$	4.70	4.70	0.787	5.76	$8\tfrac{5}{16}$	–	15.9	11.0	1.90
×$\tfrac{5}{16}$	4.24	4.24	0.812	5.06	$8\tfrac{5}{8}$	–	14.2	9.56	1.92
×$\tfrac{1}{4}$	3.67	3.67	0.838	4.26	$8\tfrac{7}{8}$	–	12.2	7.99	1.93
×$\tfrac{3}{16}$	2.97	2.97	0.864	3.34	$9\tfrac{3}{16}$	–	9.74	6.22	1.95
×$\tfrac{1}{8}$	2.14	2.14	0.890	2.33	$9\tfrac{7}{16}$	–	6.90	4.31	1.97
HSS9×7×$\tfrac{5}{8}$	117	33.5	2.68	40.5	$6\tfrac{3}{16}$	$4\tfrac{3}{16}$	235	62.0	2.50
×$\tfrac{1}{2}$	100	28.7	2.73	34.0	$6\tfrac{3}{4}$	$4\tfrac{3}{4}$	197	51.5	2.53
×$\tfrac{3}{8}$	80.4	23.0	2.78	26.7	$7\tfrac{5}{16}$	$5\tfrac{5}{16}$	154	40.0	2.57
×$\tfrac{5}{16}$	69.2	19.8	2.81	22.8	$7\tfrac{5}{8}$	$5\tfrac{5}{8}$	131	33.9	2.58
×$\tfrac{1}{4}$	57.2	16.3	2.84	18.7	$7\tfrac{7}{8}$	$5\tfrac{7}{8}$	107	27.6	2.60
×$\tfrac{3}{16}$	44.1	12.6	2.87	14.3	$8\tfrac{3}{16}$	$6\tfrac{3}{16}$	81.7	20.9	2.62

– Indicates flat depth or width is too small to establish a workable flat.

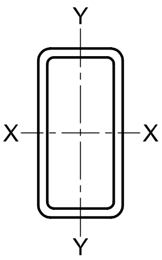


Table 1-11 (continued)
Rectangular HSS
Dimensions and Properties

Shape	Design Wall Thickness, *t*	Nominal Wt.	Area, *A*	*b/t*	*h/t*	Axis X-X			
						I	*S*	*r*	*Z*
	in.	lb/ft	in.^{2}			in.^{4}	in.^{3}	in.	in.^{3}
HSS9×5×5/8	0.581	50.81	14.0	5.61	12.5	133	29.6	3.08	38.5
×1/2	0.465	42.05	11.6	7.75	16.4	115	25.5	3.14	32.5
×3/8	0.349	32.58	8.97	11.3	22.8	92.5	20.5	3.21	25.7
×5/16	0.291	27.59	7.59	14.2	27.9	79.8	17.7	3.24	22.0
×1/4	0.233	22.42	6.17	18.5	35.6	66.1	14.7	3.27	18.1
×3/16	0.174	17.08	4.67	25.7	48.7	51.1	11.4	3.31	13.8
HSS9×3×1/2	0.465	35.24	9.74	3.45	16.4	80.8	18.0	2.88	24.6
×3/8	0.349	27.48	7.58	5.60	22.8	66.3	14.7	2.96	19.7
×5/16	0.291	23.34	6.43	7.31	27.9	57.7	12.8	3.00	16.9
×1/4	0.233	19.02	5.24	9.88	35.6	48.2	10.7	3.04	14.0
×3/16	0.174	14.53	3.98	14.2	48.7	37.6	8.35	3.07	10.8
HSS8×6×5/8	0.581	50.81	14.0	7.33	10.8	114	28.5	2.85	36.1
×1/2	0.465	42.05	11.6	9.90	14.2	98.2	24.6	2.91	30.5
×3/8	0.349	32.58	8.97	14.2	19.9	79.1	19.8	2.97	24.1
×5/16	0.291	27.59	7.59	17.6	24.5	68.3	17.1	3.00	20.6
×1/4	0.233	22.42	6.17	22.8	31.3	56.6	14.2	3.03	16.9
×3/16	0.174	17.08	4.67	31.5	43.0	43.7	10.9	3.06	13.0
HSS8×4×5/8	0.581	42.30	11.7	3.88	10.8	82.0	20.5	2.64	27.4
×1/2	0.465	35.24	9.74	5.60	14.2	71.8	17.9	2.71	23.5
×3/8	0.349	27.48	7.58	8.46	19.9	58.7	14.7	2.78	18.8
×5/16	0.291	23.34	6.43	10.7	24.5	51.0	12.8	2.82	16.1
×1/4	0.233	19.02	5.24	14.2	31.3	42.5	10.6	2.85	13.3
×3/16	0.174	14.53	3.98	20.0	43.0	33.1	8.27	2.88	10.2
×1/8	0.116	9.86	2.70	31.5	66.0	22.9	5.73	2.92	7.02
HSS8×3×1/2	0.465	31.84	8.81	3.45	14.2	58.6	14.6	2.58	20.0
×3/8	0.349	24.93	6.88	5.60	19.9	48.5	12.1	2.65	16.1
×5/16	0.291	21.21	5.85	7.31	24.5	42.4	10.6	2.69	13.9
×1/4	0.233	17.32	4.77	9.88	31.3	35.5	8.88	2.73	11.5
×3/16	0.174	13.25	3.63	14.2	43.0	27.8	6.94	2.77	8.87
×1/8	0.116	9.01	2.46	22.9	66.0	19.3	4.83	2.80	6.11

Note: For width-to-thickness criteria, refer to Table 1-12A.

Table 1-11 (continued)
Rectangular HSS
Dimensions and Properties

HSS9–HSS8

Shape	Axis Y-Y				Workable Flat		Torsion		Surface Area
	I	*S*	*r*	*Z*	Depth	Width	*J*	*C*	
	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{3}	ft^2/ft
HSS9×5×5/8	52.0	20.8	1.92	25.3	6 3/16	2 3/16	128	42.5	2.17
×1/2	45.2	18.1	1.97	21.5	6 3/4	2 3/4	109	35.6	2.20
×3/8	36.8	14.7	2.03	17.1	7 5/16	3 5/16	86.9	27.9	2.23
×5/16	32.0	12.8	2.05	14.6	7 5/8	3 5/8	74.4	23.8	2.25
×1/4	26.6	10.6	2.08	12.0	7 7/8	3 7/8	61.2	19.4	2.27
×3/16	20.7	8.28	2.10	9.25	8 3/16	4 3/16	46.9	14.8	2.28
HSS9×3×1/2	13.2	8.81	1.17	10.8	6 3/4	–	40.0	19.7	1.87
×3/8	11.2	7.45	1.21	8.80	7 5/16	–	33.1	15.8	1.90
×5/16	9.88	6.59	1.24	7.63	7 5/8	–	28.9	13.6	1.92
×1/4	8.38	5.59	1.27	6.35	7 7/8	–	24.2	11.3	1.93
×3/16	6.64	4.42	1.29	4.92	8 3/16	2 3/16	18.9	8.66	1.95
HSS8×6×5/8	72.3	24.1	2.27	29.5	5 3/16	3 3/16	150	46.0	2.17
×1/2	62.5	20.8	2.32	24.9	5 3/4	3 3/4	127	38.4	2.20
×3/8	50.6	16.9	2.38	19.8	6 5/16	4 5/16	100	30.0	2.23
×5/16	43.8	14.6	2.40	16.9	6 5/8	4 5/8	85.8	25.5	2.25
×1/4	36.4	12.1	2.43	13.9	6 7/8	4 7/8	70.3	20.8	2.27
×3/16	28.2	9.39	2.46	10.7	7 3/16	5 3/16	53.7	15.8	2.28
HSS8×4×5/8	26.6	13.3	1.51	16.6	5 3/16	–	70.3	28.7	1.83
×1/2	23.6	11.8	1.56	14.3	5 3/4	–	61.1	24.4	1.87
×3/8	19.6	9.80	1.61	11.5	6 5/16	2 5/16	49.3	19.3	1.90
×5/16	17.2	8.58	1.63	9.91	6 5/8	2 5/8	42.6	16.5	1.92
×1/4	14.4	7.21	1.66	8.20	6 7/8	2 7/8	35.3	13.6	1.93
×3/16	11.3	5.65	1.69	6.33	7 3/16	3 3/16	27.2	10.4	1.95
×1/8	7.90	3.95	1.71	4.36	7 7/16	3 7/16	18.7	7.10	1.97
HSS8×3×1/2	11.7	7.81	1.15	9.64	5 3/4	–	34.3	17.4	1.70
×3/8	10.0	6.63	1.20	7.88	6 5/16	–	28.5	14.0	1.73
×5/16	8.81	5.87	1.23	6.84	6 5/8	–	24.9	12.1	1.75
×1/4	7.49	4.99	1.25	5.70	6 7/8	–	20.8	10.0	1.77
×3/16	5.94	3.96	1.28	4.43	7 3/16	2 3/16	16.2	7.68	1.78
×1/8	4.20	2.80	1.31	3.07	7 7/16	2 7/16	11.3	5.27	1.80

– Indicates flat depth or width is too small to establish a workable flat.

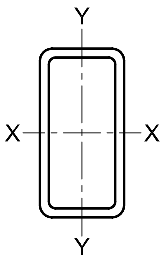
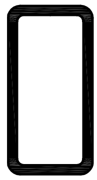


Table 1-11 (continued)
Rectangular HSS
Dimensions and Properties

Shape	Design Wall Thickness, *t*	Nominal Wt.	Area, *A*	*b/t*	*h/t*	Axis X-X *I*	Axis X-X *S*	Axis X-X *r*	Axis X-X *Z*
	in.	lb/ft	in.^{2}			in.^{4}	in.^{3}	in.	in.^{3}
HSS8×2×3/8	0.349	22.37	6.18	2.73	19.9	38.2	9.56	2.49	13.4
×5/16	0.291	19.08	5.26	3.87	24.5	33.7	8.43	2.53	11.6
×1/4	0.233	15.62	4.30	5.58	31.3	28.5	7.12	2.57	9.68
×3/16	0.174	11.97	3.28	8.49	43.0	22.4	5.61	2.61	7.51
×1/8	0.116	8.16	2.23	14.2	66.0	15.7	3.93	2.65	5.19
HSS7×5×1/2	0.465	35.24	9.74	7.75	12.1	60.6	17.3	2.50	21.9
×3/8	0.349	27.48	7.58	11.3	17.1	49.5	14.1	2.56	17.5
×5/16	0.291	23.34	6.43	14.2	21.1	43.0	12.3	2.59	15.0
×1/4	0.233	19.02	5.24	18.5	27.0	35.9	10.2	2.62	12.4
×3/16	0.174	14.53	3.98	25.7	37.2	27.9	7.96	2.65	9.52
×1/8	0.116	9.86	2.70	40.1	57.3	19.3	5.52	2.68	6.53
HSS7×4×1/2	0.465	31.84	8.81	5.60	12.1	50.7	14.5	2.40	18.8
×3/8	0.349	24.93	6.88	8.46	17.1	41.8	11.9	2.46	15.1
×5/16	0.291	21.21	5.85	10.7	21.1	36.5	10.4	2.50	13.1
×1/4	0.233	17.32	4.77	14.2	27.0	30.5	8.72	2.53	10.8
×3/16	0.174	13.25	3.63	20.0	37.2	23.8	6.81	2.56	8.33
×1/8	0.116	9.01	2.46	31.5	57.3	16.6	4.73	2.59	5.73
HSS7×3×1/2	0.465	28.43	7.88	3.45	12.1	40.7	11.6	2.27	15.8
×3/8	0.349	22.37	6.18	5.60	17.1	34.1	9.73	2.35	12.8
×5/16	0.291	19.08	5.26	7.31	21.1	29.9	8.54	2.38	11.1
×1/4	0.233	15.62	4.30	9.88	27.0	25.2	7.19	2.42	9.22
×3/16	0.174	11.97	3.28	14.2	37.2	19.8	5.65	2.45	7.14
×1/8	0.116	8.16	2.23	22.9	57.3	13.8	3.95	2.49	4.93
HSS7×2×1/4	0.233	13.91	3.84	5.58	27.0	19.8	5.67	2.27	7.64
×3/16	0.174	10.70	2.93	8.49	37.2	15.7	4.49	2.31	5.95
×1/8	0.116	7.31	2.00	14.2	57.3	11.1	3.16	2.35	4.13
HSS6×5×1/2	0.465	31.84	8.81	7.75	9.90	41.1	13.7	2.16	17.2
×3/8	0.349	24.93	6.88	11.3	14.2	33.9	11.3	2.22	13.8
×5/16	0.291	21.21	5.85	14.2	17.6	29.6	9.85	2.25	11.9
×1/4	0.233	17.32	4.77	18.5	22.8	24.7	8.25	2.28	9.87
×3/16	0.174	13.25	3.63	25.7	31.5	19.3	6.44	2.31	7.62
×1/8	0.116	9.01	2.46	40.1	48.7	13.4	4.48	2.34	5.24

Note: For width-to-thickness criteria, refer to Table 1-12A.

Table 1-11 (continued)
Rectangular HSS
Dimensions and Properties



HSS8–HSS6

Shape	Axis Y-Y				Workable Flat		Torsion		Surface Area
	I	*S*	*r*	*Z*	Depth	Width	*J*	*C*	
	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{3}	ft^2/ft
HSS8×2×3/8	3.73	3.73	0.777	4.61	6 5/16	–	12.1	8.65	1.57
×5/16	3.38	3.38	0.802	4.06	6 5/8	–	10.9	7.57	1.58
×1/4	2.94	2.94	0.827	3.43	6 7/8	–	9.36	6.35	1.60
×3/16	2.39	2.39	0.853	2.70	7 3/16	–	7.48	4.95	1.62
×1/8	1.72	1.72	0.879	1.90	7 7/16	–	5.30	3.44	1.63
HSS7×5×1/2	35.6	14.2	1.91	17.3	4 3/4	2 3/4	75.8	27.2	1.87
×3/8	29.3	11.7	1.97	13.8	5 5/16	3 5/16	60.6	21.4	1.90
×5/16	25.5	10.2	1.99	11.9	5 5/8	3 5/8	52.1	18.3	1.92
×1/4	21.3	8.53	2.02	9.83	5 7/8	3 7/8	42.9	15.0	1.93
×3/16	16.6	6.65	2.05	7.57	6 3/16	4 3/16	32.9	11.4	1.95
×1/8	11.6	4.63	2.07	5.20	6 7/16	4 7/16	22.5	7.79	1.97
HSS7×4×1/2	20.7	10.4	1.53	12.6	4 3/4	–	50.5	21.1	1.70
×3/8	17.3	8.63	1.58	10.2	5 5/16	2 5/16	41.0	16.8	1.73
×5/16	15.2	7.58	1.61	8.83	5 5/8	2 5/8	35.4	14.4	1.75
×1/4	12.8	6.38	1.64	7.33	5 7/8	2 7/8	29.3	11.8	1.77
×3/16	10.0	5.02	1.66	5.67	6 1/8	3 1/8	22.7	9.07	1.78
×1/8	7.03	3.51	1.69	3.91	6 7/16	3 7/16	15.6	6.20	1.80
HSS7×3×1/2	10.2	6.80	1.14	8.46	4 3/4	–	28.6	15.0	1.53
×3/8	8.71	5.81	1.19	6.95	5 5/16	–	23.9	12.1	1.57
×5/16	7.74	5.16	1.21	6.05	5 5/8	–	20.9	10.5	1.58
×1/4	6.60	4.40	1.24	5.06	5 7/8	–	17.5	8.68	1.60
×3/16	5.24	3.50	1.26	3.94	6 3/16	2 3/16	13.7	6.69	1.62
×1/8	3.71	2.48	1.29	2.73	6 7/16	2 7/16	9.48	4.60	1.63
HSS7×2×1/4	2.58	2.58	0.819	3.02	5 7/8	–	7.95	5.52	1.43
×3/16	2.10	2.10	0.845	2.39	6 3/16	–	6.35	4.32	1.45
×1/8	1.52	1.52	0.871	1.68	6 7/16	–	4.51	3.00	1.47
HSS6×5×1/2	30.8	12.3	1.87	15.2	3 3/4	2 3/4	59.8	23.0	1.70
×3/8	25.5	10.2	1.92	12.2	4 5/16	3 5/16	48.1	18.2	1.73
×5/16	22.3	8.91	1.95	10.5	4 5/8	3 5/8	41.4	15.6	1.75
×1/4	18.7	7.47	1.98	8.72	4 7/8	3 7/8	34.2	12.8	1.77
×3/16	14.6	5.84	2.01	6.73	5 3/16	4 3/16	26.3	9.76	1.78
×1/8	10.2	4.07	2.03	4.63	5 7/16	4 7/16	18.0	6.66	1.80

– Indicates flat depth or width is too small to establish a workable flat.

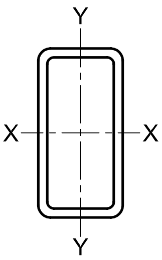


Table 1-11 (continued)
Rectangular HSS
Dimensions and Properties

Shape	Design Wall Thickness, *t*	Nominal Wt.	Area, *A*	*b/t*	*h/t*	Axis X-X			
						I	*S*	*r*	*Z*
	in.	lb/ft	in.^{2}			in.^{4}	in.^{3}	in.	in.^{3}
HSS6×4×1/2	0.465	28.43	7.88	5.60	9.90	34.0	11.3	2.08	14.6
×3/8	0.349	22.37	6.18	8.46	14.2	28.3	9.43	2.14	11.9
×5/16	0.291	19.08	5.26	10.7	17.6	24.8	8.27	2.17	10.3
×1/4	0.233	15.62	4.30	14.2	22.8	20.9	6.96	2.20	8.53
×3/16	0.174	11.97	3.28	20.0	31.5	16.4	5.46	2.23	6.60
×1/8	0.116	8.16	2.23	31.5	48.7	11.4	3.81	2.26	4.56
HSS6×3×1/2	0.465	25.03	6.95	3.45	9.90	26.8	8.95	1.97	12.1
×3/8	0.349	19.82	5.48	5.60	14.2	22.7	7.57	2.04	9.90
×5/16	0.291	16.96	4.68	7.31	17.6	20.1	6.69	2.07	8.61
×1/4	0.233	13.91	3.84	9.88	22.8	17.0	5.66	2.10	7.19
×3/16	0.174	10.70	2.93	14.2	31.5	13.4	4.47	2.14	5.59
×1/8	0.116	7.31	2.00	22.9	48.7	9.43	3.14	2.17	3.87
HSS6×2×3/8	0.349	17.27	4.78	2.73	14.2	17.1	5.71	1.89	7.93
×5/16	0.291	14.83	4.10	3.87	17.6	15.3	5.11	1.93	6.95
×1/4	0.233	12.21	3.37	5.58	22.8	13.1	4.37	1.97	5.84
×3/16	0.174	9.42	2.58	8.49	31.5	10.5	3.49	2.01	4.58
×1/8	0.116	6.46	1.77	14.2	48.7	7.42	2.47	2.05	3.19
HSS5×4×1/2	0.465	25.03	6.95	5.60	7.75	21.2	8.49	1.75	10.9
×3/8	0.349	19.82	5.48	8.46	11.3	17.9	7.17	1.81	8.96
×5/16	0.291	16.96	4.68	10.7	14.2	15.8	6.32	1.84	7.79
×1/4	0.233	13.91	3.84	14.2	18.5	13.4	5.35	1.87	6.49
×3/16	0.174	10.70	2.93	20.0	25.7	10.6	4.22	1.90	5.05
×1/8	0.116	7.31	2.00	31.5	40.1	7.42	2.97	1.93	3.50
HSS5×3×1/2	0.465	21.63	6.02	3.45	7.75	16.4	6.57	1.65	8.83
×3/8	0.349	17.27	4.78	5.60	11.3	14.1	5.65	1.72	7.34
×5/16	0.291	14.83	4.10	7.31	14.2	12.6	5.03	1.75	6.42
×1/4	0.233	12.21	3.37	9.88	18.5	10.7	4.29	1.78	5.38
×3/16	0.174	9.42	2.58	14.2	25.7	8.53	3.41	1.82	4.21
×1/8	0.116	6.46	1.77	22.9	40.1	6.03	2.41	1.85	2.93

Note: For width-to-thickness criteria, refer to Table 1-12A.

Table 1-11 (continued)
Rectangular HSS
Dimensions and Properties

HSS6–HSS5

Shape	Axis Y-Y				Workable Flat		Torsion		Surface Area
	I	*S*	*r*	*Z*	Depth	Width	*J*	*C*	
	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{3}	ft^2/ft
HSS6×4×1/2	17.8	8.89	1.50	11.0	3 3/4	–	40.3	17.8	1.53
×3/8	14.9	7.47	1.55	8.94	4 5/16	2 5/16	32.8	14.2	1.57
×5/16	13.2	6.58	1.58	7.75	4 5/8	2 5/8	28.4	12.2	1.58
×1/4	11.1	5.56	1.61	6.45	4 7/8	2 7/8	23.6	10.1	1.60
×3/16	8.76	4.38	1.63	5.00	5 3/16	3 3/16	18.2	7.74	1.62
×1/8	6.15	3.08	1.66	3.46	5 7/16	3 7/16	12.6	5.30	1.63
HSS6×3×1/2	8.69	5.79	1.12	7.28	3 3/4	–	23.1	12.7	1.37
×3/8	7.48	4.99	1.17	6.03	4 5/16	–	19.3	10.3	1.40
×5/16	6.67	4.45	1.19	5.27	4 5/8	–	16.9	8.91	1.42
×1/4	5.70	3.80	1.22	4.41	4 7/8	–	14.2	7.39	1.43
×3/16	4.55	3.03	1.25	3.45	5 3/16	2 3/16	11.1	5.71	1.45
×1/8	3.23	2.15	1.27	2.40	5 7/16	2 7/16	7.73	3.93	1.47
HSS6×2×3/8	2.77	2.77	0.760	3.46	4 5/16	–	8.42	6.35	1.23
×5/16	2.52	2.52	0.785	3.07	4 5/8	–	7.60	5.58	1.25
×1/4	2.21	2.21	0.810	2.61	4 7/8	–	6.55	4.70	1.27
×3/16	1.80	1.80	0.836	2.07	5 3/16	–	5.24	3.68	1.28
×1/8	1.31	1.31	0.861	1.46	5 7/16	–	3.72	2.57	1.30
HSS5×4×1/2	14.9	7.43	1.46	9.35	2 3/4	–	30.3	14.5	1.37
×3/8	12.6	6.30	1.52	7.67	3 5/16	2 5/16	24.9	11.7	1.40
×5/16	11.1	5.57	1.54	6.67	3 5/8	2 5/8	21.7	10.1	1.42
×1/4	9.46	4.73	1.57	5.57	3 7/8	2 7/8	18.0	8.32	1.43
×3/16	7.48	3.74	1.60	4.34	4 3/16	3 3/16	14.0	6.41	1.45
×1/8	5.27	2.64	1.62	3.01	4 7/16	3 7/16	9.66	4.39	1.47
HSS5×3×1/2	7.18	4.78	1.09	6.10	2 3/4	–	17.6	10.3	1.20
×3/8	6.25	4.16	1.14	5.10	3 5/16	–	14.9	8.44	1.23
×5/16	5.60	3.73	1.17	4.48	3 5/8	–	13.1	7.33	1.25
×1/4	4.81	3.21	1.19	3.77	3 7/8	–	11.0	6.10	1.27
×3/16	3.85	2.57	1.22	2.96	4 3/16	2 3/16	8.64	4.73	1.28
×1/8	2.75	1.83	1.25	2.07	4 7/16	2 7/16	6.02	3.26	1.30

– Indicates flat depth or width is too small to establish a workable flat.

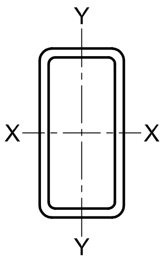


Table 1-11 (continued)
Rectangular HSS
Dimensions and Properties

Shape	Design Wall Thick-ness, *t*	Nominal Wt.	Area, *A*	*b/t*	*h/t*	Axis X-X			
						I	*S*	*r*	*Z*
	in.	lb/ft	in.^{2}			in.^{4}	in.^{3}	in.	in.^{3}
HSS5×2½×¼	0.233	11.36	3.14	7.73	18.5	9.40	3.76	1.73	4.83
×3/16	0.174	8.78	2.41	11.4	25.7	7.51	3.01	1.77	3.79
×⅛	0.116	6.03	1.65	18.6	40.1	5.34	2.14	1.80	2.65
HSS5×2×⅜	0.349	14.72	4.09	2.73	11.3	10.4	4.14	1.59	5.71
×5/16	0.291	12.70	3.52	3.87	14.2	9.35	3.74	1.63	5.05
×¼	0.233	10.51	2.91	5.58	18.5	8.08	3.23	1.67	4.27
×3/16	0.174	8.15	2.24	8.49	25.7	6.50	2.60	1.70	3.37
×⅛	0.116	5.61	1.54	14.2	40.1	4.65	1.86	1.74	2.37
HSS4×3×⅜	0.349	14.72	4.09	5.60	8.46	7.93	3.97	1.39	5.12
×5/16	0.291	12.70	3.52	7.31	10.7	7.14	3.57	1.42	4.51
×¼	0.233	10.51	2.91	9.88	14.2	6.15	3.07	1.45	3.81
×3/16	0.174	8.15	2.24	14.2	20.0	4.93	2.47	1.49	3.00
×⅛	0.116	5.61	1.54	22.9	31.5	3.52	1.76	1.52	2.11
HSS4×2½×⅜	0.349	13.44	3.74	4.16	8.46	6.77	3.38	1.35	4.48
×5/16	0.291	11.64	3.23	5.59	10.7	6.13	3.07	1.38	3.97
×¼	0.233	9.66	2.67	7.73	14.2	5.32	2.66	1.41	3.38
×3/16	0.174	7.51	2.06	11.4	20.0	4.30	2.15	1.44	2.67
×⅛	0.116	5.18	1.42	18.6	31.5	3.09	1.54	1.47	1.88
HSS4×2×⅜	0.349	12.17	3.39	2.73	8.46	5.60	2.80	1.29	3.84
×5/16	0.291	10.58	2.94	3.87	10.7	5.13	2.56	1.32	3.43
×¼	0.233	8.81	2.44	5.58	14.2	4.49	2.25	1.36	2.94
×3/16	0.174	6.87	1.89	8.49	20.0	3.66	1.83	1.39	2.34
×⅛	0.116	4.75	1.30	14.2	31.5	2.65	1.32	1.43	1.66
HSS3½×2½×⅜	0.349	12.17	3.39	4.16	7.03	4.75	2.72	1.18	3.59
×5/16	0.291	10.58	2.94	5.59	9.03	4.34	2.48	1.22	3.20
×¼	0.233	8.81	2.44	7.73	12.0	3.79	2.17	1.25	2.74
×3/16	0.174	6.87	1.89	11.4	17.1	3.09	1.76	1.28	2.18
×⅛	0.116	4.75	1.30	18.6	27.2	2.23	1.28	1.31	1.54
HSS3½×2×¼	0.233	7.96	2.21	5.58	12.0	3.17	1.81	1.20	2.36
×3/16	0.174	6.23	1.71	8.49	17.1	2.61	1.49	1.23	1.89
×⅛	0.116	4.33	1.19	14.2	27.2	1.90	1.09	1.27	1.34

Note: For width-to-thickness criteria, refer to Table 1-12A.

Table 1-11 (continued)
Rectangular HSS
Dimensions and Properties

HSS5–HSS3 1/2

Shape	Axis Y-Y				Workable Flat		Torsion		Surface Area
	I	*S*	*r*	*Z*	Depth	Width	*J*	*C*	
	in.⁴	in.³	in.	in.³	in.	in.	in.⁴	in.³	ft²/ft
HSS5×2 1/2×1/4	3.13	2.50	0.999	2.95	3 7/8	–	7.93	4.99	1.18
×3/16	2.53	2.03	1.02	2.33	4 3/16	–	6.26	3.89	1.20
×1/8	1.82	1.46	1.05	1.64	4 7/16	–	4.40	2.70	1.22
HSS5×2×3/8	2.28	2.28	0.748	2.88	3 5/16	–	6.61	5.20	1.07
×5/16	2.10	2.10	0.772	2.57	3 5/8	–	5.99	4.59	1.08
×1/4	1.84	1.84	0.797	2.20	3 7/8	–	5.17	3.88	1.10
×3/16	1.51	1.51	0.823	1.75	4 3/16	–	4.15	3.05	1.12
×1/8	1.10	1.10	0.848	1.24	4 7/16	–	2.95	2.13	1.13
HSS4×3×3/8	5.01	3.34	1.11	4.18	2 5/16	–	10.6	6.59	1.07
×5/16	4.52	3.02	1.13	3.69	2 5/8	–	9.41	5.75	1.08
×1/4	3.91	2.61	1.16	3.12	2 7/8	–	7.96	4.81	1.10
×3/16	3.16	2.10	1.19	2.46	3 3/16	–	6.26	3.74	1.12
×1/8	2.27	1.51	1.21	1.73	3 7/16	–	4.38	2.59	1.13
HSS4×2 1/2×3/8	3.17	2.54	0.922	3.20	2 5/16	–	7.57	5.32	0.983
×5/16	2.89	2.32	0.947	2.85	2 5/8	–	6.77	4.67	1.00
×1/4	2.53	2.02	0.973	2.43	2 7/8	–	5.78	3.93	1.02
×3/16	2.06	1.65	0.999	1.93	3 1/8	–	4.59	3.08	1.03
×1/8	1.49	1.19	1.03	1.36	3 7/16	–	3.23	2.14	1.05
HSS4×2×3/8	1.80	1.80	0.729	2.31	2 5/16	–	4.83	4.04	0.900
×5/16	1.67	1.67	0.754	2.08	2 5/8	–	4.40	3.59	0.917
×1/4	1.48	1.48	0.779	1.79	2 7/8	–	3.82	3.05	0.933
×3/16	1.22	1.22	0.804	1.43	3 3/16	–	3.08	2.41	0.950
×1/8	0.898	0.898	0.830	1.02	3 7/16	–	2.20	1.69	0.967
HSS3 1/2×2 1/2×3/8	2.77	2.21	0.904	2.82	–	–	6.16	4.57	0.900
×5/16	2.54	2.03	0.930	2.52	2 1/8	–	5.53	4.03	0.917
×1/4	2.23	1.78	0.956	2.16	2 3/8	–	4.75	3.40	0.933
×3/16	1.82	1.46	0.983	1.72	2 11/16	–	3.78	2.67	0.950
×1/8	1.33	1.06	1.01	1.22	2 15/16	–	2.67	1.87	0.967
HSS3 1/2×2×1/4	1.30	1.30	0.766	1.58	2 3/8	–	3.16	2.64	0.850
×3/16	1.08	1.08	0.792	1.27	2 11/16	–	2.55	2.09	0.867
×1/8	0.795	0.795	0.818	0.912	2 15/16	–	1.83	1.47	0.883

– Indicates flat depth or width is too small to establish a workable flat.

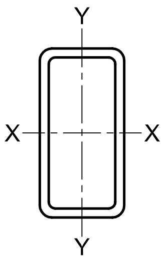


Table 1-11 (continued)
Rectangular HSS
Dimensions and Properties

Shape	Design Wall Thick-ness, *t*	Nominal Wt.	Area, *A*	*b/t*	*h/t*	Axis X-X *I*	Axis X-X *S*	Axis X-X *r*	Axis X-X *Z*
	in.	lb/ft	in.^{2}			in.^{4}	in.^{3}	in.	in.^{3}
HSS3$\frac{1}{2}$×1$\frac{1}{2}$×$\frac{1}{4}$	0.233	7.11	1.97	3.44	12.0	2.55	1.46	1.14	1.98
×$\frac{3}{16}$	0.174	5.59	1.54	5.62	17.1	2.12	1.21	1.17	1.60
×$\frac{1}{8}$	0.116	3.90	1.07	9.93	27.2	1.57	0.896	1.21	1.15
HSS3×2$\frac{1}{2}$×$\frac{5}{16}$	0.291	9.51	2.64	5.59	7.31	2.92	1.94	1.05	2.51
×$\frac{1}{4}$	0.233	7.96	2.21	7.73	9.88	2.57	1.72	1.08	2.16
×$\frac{3}{16}$	0.174	6.23	1.71	11.4	14.2	2.11	1.41	1.11	1.73
×$\frac{1}{8}$	0.116	4.33	1.19	18.6	22.9	1.54	1.03	1.14	1.23
HSS3×2×$\frac{5}{16}$	0.291	8.45	2.35	3.87	7.31	2.38	1.59	1.01	2.11
×$\frac{1}{4}$	0.233	7.11	1.97	5.58	9.88	2.13	1.42	1.04	1.83
×$\frac{3}{16}$	0.174	5.59	1.54	8.49	14.2	1.77	1.18	1.07	1.48
×$\frac{1}{8}$	0.116	3.90	1.07	14.2	22.9	1.30	0.867	1.10	1.06
HSS3×1$\frac{1}{2}$×$\frac{1}{4}$	0.233	6.26	1.74	3.44	9.88	1.68	1.12	0.982	1.51
×$\frac{3}{16}$	0.174	4.96	1.37	5.62	14.2	1.42	0.945	1.02	1.24
×$\frac{1}{8}$	0.116	3.48	0.956	9.93	22.9	1.06	0.706	1.05	0.895
HSS3×1×$\frac{3}{16}$	0.174	4.32	1.19	2.75	14.2	1.07	0.713	0.947	0.989
×$\frac{1}{8}$	0.116	3.05	0.840	5.62	22.9	0.817	0.545	0.987	0.728
HSS2$\frac{1}{2}$×2×$\frac{1}{4}$	0.233	6.26	1.74	5.58	7.73	1.33	1.06	0.874	1.37
×$\frac{3}{16}$	0.174	4.96	1.37	8.49	11.4	1.12	0.894	0.904	1.12
×$\frac{1}{8}$	0.116	3.48	0.956	14.2	18.6	0.833	0.667	0.934	0.809
HSS2$\frac{1}{2}$×1$\frac{1}{2}$×$\frac{1}{4}$	0.233	5.41	1.51	3.44	7.73	1.03	0.822	0.826	1.11
×$\frac{3}{16}$	0.174	4.32	1.19	5.62	11.4	0.882	0.705	0.860	0.915
×$\frac{1}{8}$	0.116	3.05	0.840	9.93	18.6	0.668	0.535	0.892	0.671
HSS2$\frac{1}{2}$×1×$\frac{3}{16}$	0.174	3.68	1.02	2.75	11.4	0.646	0.517	0.796	0.713
×$\frac{1}{8}$	0.116	2.63	0.724	5.62	18.6	0.503	0.403	0.834	0.532
HSS2$\frac{1}{4}$×2×$\frac{3}{16}$	0.174	4.64	1.28	8.49	9.93	0.859	0.764	0.819	0.952
×$\frac{1}{8}$	0.116	3.27	0.898	14.2	16.4	0.646	0.574	0.848	0.693
HSS2×1$\frac{1}{2}$×$\frac{3}{16}$	0.174	3.68	1.02	5.62	8.49	0.495	0.495	0.697	0.639
×$\frac{1}{8}$	0.116	2.63	0.724	9.93	14.2	0.383	0.383	0.728	0.475
HSS2×1×$\frac{3}{16}$	0.174	3.04	0.845	2.75	8.49	0.350	0.350	0.643	0.480
×$\frac{1}{8}$	0.116	2.20	0.608	5.62	14.2	0.280	0.280	0.679	0.366

Note: For width-to-thickness criteria, refer to Table 1-12A.

Table 1-11 (continued)
Rectangular HSS
Dimensions and Properties

HSS3 1/2–HSS2

Shape	Axis Y-Y				Workable Flat		Torsion		Surface Area
	I	S	r	Z	Depth	Width	J	C	
	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{3}	ft^2/ft
HSS3 1/2×1 1/2×1/4	0.638	0.851	0.569	1.06	2 3/8	–	1.79	1.88	0.767
×3/16	0.544	0.725	0.594	0.867	2 11/16	–	1.49	1.51	0.784
×1/8	0.411	0.548	0.619	0.630	2 15/16	–	1.09	1.08	0.800
HSS3×2 1/2×5/16	2.18	1.74	0.908	2.20	–	–	4.34	3.39	0.833
×1/4	1.93	1.54	0.935	1.90	–	–	3.74	2.87	0.850
×3/16	1.59	1.27	0.963	1.52	2 3/16	–	3.00	2.27	0.867
×1/8	1.16	0.931	0.990	1.09	2 7/16	–	2.13	1.59	0.883
HSS3×2×5/16	1.24	1.24	0.725	1.58	–	–	2.87	2.60	0.750
×1/4	1.11	1.11	0.751	1.38	–	–	2.52	2.23	0.767
×3/16	0.932	0.932	0.778	1.12	2 3/16	–	2.05	1.78	0.784
×1/8	0.692	0.692	0.804	0.803	2 7/16	–	1.47	1.25	0.800
HSS3×1 1/2×1/4	0.543	0.725	0.559	0.911	1 7/8	–	1.44	1.58	0.683
×3/16	0.467	0.622	0.584	0.752	2 3/16	–	1.21	1.28	0.700
×1/8	0.355	0.474	0.610	0.550	2 7/16	–	0.886	0.920	0.717
HSS3×1×3/16	0.173	0.345	0.380	0.432	2 3/16	–	0.526	0.792	0.617
×1/8	0.138	0.276	0.405	0.325	2 7/16	–	0.408	0.585	0.633
HSS2 1/2×2×1/4	0.930	0.930	0.731	1.17	–	–	1.90	1.82	0.683
×3/16	0.786	0.786	0.758	0.956	–	–	1.55	1.46	0.700
×1/8	0.589	0.589	0.785	0.694	–	–	1.12	1.04	0.717
HSS2 1/2×1 1/2×1/4	0.449	0.599	0.546	0.764	–	–	1.10	1.29	0.600
×3/16	0.390	0.520	0.572	0.636	–	–	0.929	1.05	0.617
×1/8	0.300	0.399	0.597	0.469	–	–	0.687	0.759	0.633
HSS2 1/2×1×3/16	0.143	0.285	0.374	0.360	–	–	0.412	0.648	0.534
×1/8	0.115	0.230	0.399	0.274	–	–	0.322	0.483	0.550
HSS2 1/4×2×3/16	0.713	0.713	0.747	0.877	–	–	1.32	1.30	0.659
×1/8	0.538	0.538	0.774	0.639	–	–	0.957	0.927	0.675
HSS2×1 1/2×3/16	0.313	0.417	0.554	0.521	–	–	0.664	0.822	0.534
×1/8	0.244	0.325	0.581	0.389	–	–	0.496	0.599	0.550
HSS2×1×3/16	0.112	0.225	0.365	0.288	–	–	0.301	0.505	0.450
×1/8	0.0922	0.184	0.390	0.223	–	–	0.238	0.380	0.467

– Indicates flat depth or width is too small to establish a workable flat.

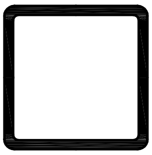
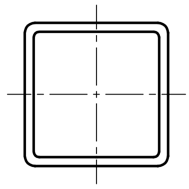


Table 1-12
Square HSS
Dimensions and Properties



HSS22–HSS12

Shape	Design Wall Thickness, t	Nominal Wt.	Area, A	b/t	h/t	I	S	r	Z	Workable Flat	Torsion J	Torsion C	Surface Area
	in.	lb/ft	in.^{2}			in.^{4}	in.^{3}	in.	in.^{3}	in.	in.^{4}	in.^{3}	ft^2/ft
HSS22×22×7/8	0.814	244.88	67.3	24.1	24.1	4970	452	8.59	530	18 1/16	7890	729	7.10
×3/4	0.698	212.00	58.2	28.5	28.5	4350	395	8.65	462	18 5/8	6860	632	7.13
HSS20×20×7/8	0.814	221.06	60.8	21.6	21.6	3670	367	7.77	433	16 1/16	5870	597	6.43
×3/4	0.698	191.58	52.6	25.6	25.6	3230	323	7.84	378	16 5/8	5110	519	6.47
×5/8	0.581	161.40	44.3	31.5	31.5	2750	275	7.88	320	17 3/16	4320	437	6.50
×1/2	0.465	130.52	35.8	40.0	40.0	2260	226	7.95	261	17 3/4	3510	355	6.53
HSS18×18×7/8	0.814	197.24	54.3	19.2	19.2	2630	292	6.96	346	14 1/16	4220	479	5.77
×3/4	0.698	171.16	47.1	22.8	22.8	2320	258	7.02	302	14 5/8	3690	417	5.80
×5/8	0.581	144.39	39.6	28.1	28.1	1980	220	7.07	257	15 3/16	3120	352	5.83
×1/2	0.465	116.91	32.1	35.7	35.7	1630	181	7.13	210	15 3/4	2540	286	5.87
HSS16×16×7/8	0.814	173.43	47.7	16.7	16.7	1800	225	6.14	268	12 1/16	2920	373	5.10
×3/4	0.698	150.75	41.5	19.9	19.9	1590	199	6.19	235	12 5/8	2560	326	5.13
×5/8	0.581	127.37	35.0	24.5	24.5	1370	171	6.25	200	13 3/16	2170	276	5.17
×1/2	0.465	103.30	28.3	31.4	31.4	1130	141	6.31	164	13 3/4	1770	224	5.20
×3/8	0.349	78.52	21.5	42.8	42.8	873	109	6.37	126	14 5/16	1350	171	5.23
×5/16	0.291	65.87	18.1	52.0	52.0	739	92.3	6.39	106	14 5/8	1140	144	5.25
HSS14×14×7/8	0.814	149.61	41.2	14.3	14.3	1170	167	5.33	201	10 1/16	1910	281	4.43
×3/4	0.698	130.33	35.9	17.0	17.0	1040	149	5.38	177	10 5/8	1680	246	4.47
×5/8	0.581	110.36	30.3	21.1	21.1	897	128	5.44	151	11 3/16	1430	208	4.50
×1/2	0.465	89.68	24.6	27.1	27.1	743	106	5.49	124	11 3/4	1170	170	4.53
×3/8	0.349	68.31	18.7	37.1	37.1	577	82.5	5.55	95.4	12 5/16	900	130	4.57
×5/16	0.291	57.36	15.7	45.1	45.1	490	69.9	5.58	80.5	12 5/8	759	109	4.58
HSS12×12×3/4	0.698	109.91	30.3	14.2	14.2	631	105	4.56	127	8 5/8	1030	177	3.80
×5/8	0.581	93.34	25.7	17.7	17.7	548	91.4	4.62	109	9 3/16	885	151	3.83
×1/2	0.465	76.07	20.9	22.8	22.8	457	76.2	4.68	89.6	9 3/4	728	123	3.87
×3/8	0.349	58.10	16.0	31.4	31.4	357	59.5	4.73	69.2	10 5/16	561	94.6	3.90
×5/16	0.291	48.86	13.4	38.2	38.2	304	50.7	4.76	58.6	10 5/8	474	79.7	3.92
×1/4	0.233	39.43	10.8	48.5	48.5	248	41.4	4.79	47.6	10 7/8	384	64.5	3.93
×3/16	0.174	29.84	8.15	66.0	66.0	189	31.5	4.82	36.0	11 3/16	290	48.6	3.95

Note: For width-to-thickness criteria, refer to Table 1-12A.

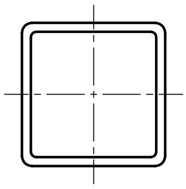
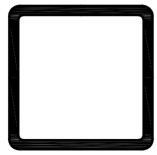


Table 1-12 (continued)
Square HSS
Dimensions and Properties



HSS10–HSS6

Shape	Design Wall Thickness, t	Nominal Wt.	Area, A	b/t	h/t	I	S	r	Z	Workable Flat	Torsion J	Torsion C	Surface Area
	in.	lb/ft	in.^{2}			in.^{4}	in.^{3}	in.	in.^{3}	in.	in.^{4}	in.^{3}	ft^2/ft
HSS10×10×3/4	0.698	89.50	24.7	11.3	11.3	347	69.4	3.75	84.7	6 5/8	578	119	3.13
×5/8	0.581	76.33	21.0	14.2	14.2	304	60.8	3.80	73.2	7 3/16	498	102	3.17
×1/2	0.465	62.46	17.2	18.5	18.5	256	51.2	3.86	60.7	7 3/4	412	84.2	3.20
×3/8	0.349	47.90	13.2	25.7	25.7	202	40.4	3.92	47.2	8 5/16	320	64.8	3.23
×5/16	0.291	40.35	11.1	31.4	31.4	172	34.5	3.94	40.1	8 5/8	271	54.8	3.25
×1/4	0.233	32.63	8.96	39.9	39.9	141	28.3	3.97	32.7	8 7/8	220	44.4	3.27
×3/16	0.174	24.73	6.76	54.5	54.5	108	21.6	4.00	24.8	9 3/16	167	33.6	3.28
HSS9×9×5/8	0.581	67.82	18.7	12.5	12.5	216	47.9	3.40	58.1	6 3/16	356	81.6	2.83
×1/2	0.465	55.66	15.3	16.4	16.4	183	40.6	3.45	48.4	6 3/4	296	67.4	2.87
×3/8	0.349	42.79	11.8	22.8	22.8	145	32.2	3.51	37.8	7 5/16	231	52.1	2.90
×5/16	0.291	36.10	9.92	27.9	27.9	124	27.6	3.54	32.1	7 5/8	196	44.0	2.92
×1/4	0.233	29.23	8.03	35.6	35.6	102	22.7	3.56	26.2	7 7/8	159	35.8	2.93
×3/16	0.174	22.18	6.06	48.7	48.7	78.2	17.4	3.59	20.0	8 3/16	121	27.1	2.95
×1/8	0.116	14.96	4.09	74.6	74.6	53.5	11.9	3.62	13.6	8 7/16	82.0	18.3	2.97
HSS8×8×5/8	0.581	59.32	16.4	10.8	10.8	146	36.5	2.99	44.7	5 3/16	244	63.2	2.50
×1/2	0.465	48.85	13.5	14.2	14.2	125	31.2	3.04	37.5	5 3/4	204	52.4	2.53
×3/8	0.349	37.69	10.4	19.9	19.9	100	24.9	3.10	29.4	6 5/16	160	40.7	2.57
×5/16	0.291	31.84	8.76	24.5	24.5	85.6	21.4	3.13	25.1	6 5/8	136	34.5	2.58
×1/4	0.233	25.82	7.10	31.3	31.3	70.7	17.7	3.15	20.5	6 7/8	111	28.1	2.60
×3/16	0.174	19.63	5.37	43.0	43.0	54.4	13.6	3.18	15.7	7 3/16	84.5	21.3	2.62
×1/8	0.116	13.26	3.62	66.0	66.0	37.4	9.34	3.21	10.7	7 7/16	57.3	14.4	2.63
HSS7×7×5/8	0.581	50.81	14.0	9.05	9.05	93.4	26.7	2.58	33.1	4 3/16	158	47.1	2.17
×1/2	0.465	42.05	11.6	12.1	12.1	80.5	23.0	2.63	27.9	4 3/4	133	39.3	2.20
×3/8	0.349	32.58	8.97	17.1	17.1	65.0	18.6	2.69	22.1	5 5/16	105	30.7	2.23
×5/16	0.291	27.59	7.59	21.1	21.1	56.1	16.0	2.72	18.9	5 5/8	89.7	26.1	2.25
×1/4	0.233	22.42	6.17	27.0	27.0	46.5	13.3	2.75	15.5	5 7/8	73.5	21.3	2.27
×3/16	0.174	17.08	4.67	37.2	37.2	36.0	10.3	2.77	11.9	6 3/16	56.1	16.2	2.28
×1/8	0.116	11.56	3.16	57.3	57.3	24.8	7.09	2.80	8.13	6 7/16	38.2	11.0	2.30
HSS6×6×5/8	0.581	42.30	11.7	7.33	7.33	55.2	18.4	2.17	23.2	3 3/16	94.9	33.4	1.83
×1/2	0.465	35.24	9.74	9.90	9.90	48.3	16.1	2.23	19.8	3 3/4	81.1	28.1	1.87
×3/8	0.349	27.48	7.58	14.2	14.2	39.5	13.2	2.28	15.8	4 5/16	64.6	22.1	1.90
×5/16	0.291	23.34	6.43	17.6	17.6	34.3	11.4	2.31	13.6	4 5/8	55.4	18.9	1.92
×1/4	0.233	19.02	5.24	22.8	22.8	28.6	9.54	2.34	11.2	4 7/8	45.6	15.4	1.93
×3/16	0.174	14.53	3.98	31.5	31.5	22.3	7.42	2.37	8.63	5 3/16	35.0	11.8	1.95
×1/8	0.116	9.86	2.70	48.7	48.7	15.5	5.15	2.39	5.92	5 7/16	23.9	8.03	1.97

Note: For width-to-thickness criteria, refer to Table 1-12A.

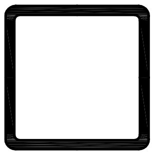
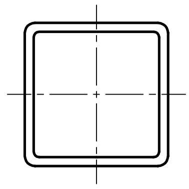


Table 1-12 (continued)
Square HSS
Dimensions and Properties



HSS5 1/2-HSS3

Shape	Design Wall Thickness, *t*	Nominal Wt.	Area, *A*	*b/t*	*h/t*	*I*	*S*	*r*	*Z*	Workable Flat	Torsion *J*	Torsion *C*	Surface Area
	in.	lb/ft	in.^{2}			in.^{4}	in.^{3}	in.	in.^{3}	in.	in.^{4}	in.^{3}	ft^2/ft
HSS5 1/2×5 1/2×3/8	0.349	24.93	6.88	12.8	12.8	29.7	10.8	2.08	13.1	3 13/16	49.0	18.4	1.73
×5/16	0.291	21.21	5.85	15.9	15.9	25.9	9.43	2.11	11.3	4 1/8	42.2	15.7	1.75
×1/4	0.233	17.32	4.77	20.6	20.6	21.7	7.90	2.13	9.32	4 3/8	34.8	12.9	1.77
×3/16	0.174	13.25	3.63	28.6	28.6	17.0	6.17	2.16	7.19	4 11/16	26.7	9.85	1.78
×1/8	0.116	9.01	2.46	44.4	44.4	11.8	4.30	2.19	4.95	4 15/16	18.3	6.72	1.80
HSS5×5×1/2	0.465	28.43	7.88	7.75	7.75	26.0	10.4	1.82	13.1	2 3/4	44.6	18.7	1.53
×3/8	0.349	22.37	6.18	11.3	11.3	21.7	8.68	1.87	10.6	3 5/16	36.1	14.9	1.57
×5/16	0.291	19.08	5.26	14.2	14.2	19.0	7.62	1.90	9.16	3 5/8	31.2	12.8	1.58
×1/4	0.233	15.62	4.30	18.5	18.5	16.0	6.41	1.93	7.61	3 7/8	25.8	10.5	1.60
×3/16	0.174	11.97	3.28	25.7	25.7	12.6	5.03	1.96	5.89	4 3/16	19.9	8.08	1.62
×1/8	0.116	8.16	2.23	40.1	40.1	8.80	3.52	1.99	4.07	4 7/16	13.7	5.53	1.63
HSS4 1/2×4 1/2×1/2	0.465	25.03	6.95	6.68	6.68	18.1	8.03	1.61	10.2	2 1/4	31.3	14.8	1.37
×3/8	0.349	19.82	5.48	9.89	9.89	15.3	6.79	1.67	8.36	2 13/16	25.7	11.9	1.40
×5/16	0.291	16.96	4.68	12.5	12.5	13.5	6.00	1.70	7.27	3 1/8	22.3	10.2	1.42
×1/4	0.233	13.91	3.84	16.3	16.3	11.4	5.08	1.73	6.06	3 3/8	18.5	8.44	1.43
×3/16	0.174	10.70	2.93	22.9	22.9	9.02	4.01	1.75	4.71	3 11/16	14.4	6.49	1.45
×1/8	0.116	7.31	2.00	35.8	35.8	6.35	2.82	1.78	3.27	3 15/16	9.92	4.45	1.47
HSS4×4×1/2	0.465	21.63	6.02	5.60	5.60	11.9	5.97	1.41	7.70	–	21.0	11.2	1.20
×3/8	0.349	17.27	4.78	8.46	8.46	10.3	5.13	1.47	6.39	2 5/16	17.5	9.14	1.23
×5/16	0.291	14.83	4.10	10.7	10.7	9.14	4.57	1.49	5.59	2 5/8	15.3	7.91	1.25
×1/4	0.233	12.21	3.37	14.2	14.2	7.80	3.90	1.52	4.69	2 7/8	12.8	6.56	1.27
×3/16	0.174	9.42	2.58	20.0	20.0	6.21	3.10	1.55	3.67	3 3/16	10.0	5.07	1.28
×1/8	0.116	6.46	1.77	31.5	31.5	4.40	2.20	1.58	2.56	3 7/16	6.91	3.49	1.30
HSS3 1/2×3 1/2×3/8	0.349	14.72	4.09	7.03	7.03	6.49	3.71	1.26	4.69	–	11.2	6.77	1.07
×5/16	0.291	12.70	3.52	9.03	9.03	5.84	3.34	1.29	4.14	2 1/8	9.89	5.90	1.08
×1/4	0.233	10.51	2.91	12.0	12.0	5.04	2.88	1.32	3.50	2 3/8	8.35	4.92	1.10
×3/16	0.174	8.15	2.24	17.1	17.1	4.05	2.31	1.35	2.76	2 11/16	6.56	3.83	1.12
×1/8	0.116	5.61	1.54	27.2	27.2	2.90	1.66	1.37	1.93	2 15/16	4.58	2.65	1.13
HSS3×3×3/8	0.349	12.17	3.39	5.60	5.60	3.78	2.52	1.06	3.25	–	6.64	4.74	0.900
×5/16	0.291	10.58	2.94	7.31	7.31	3.45	2.30	1.08	2.90	–	5.94	4.18	0.917
×1/4	0.233	8.81	2.44	9.88	9.88	3.02	2.01	1.11	2.48	–	5.08	3.52	0.933
×3/16	0.174	6.87	1.89	14.2	14.2	2.46	1.64	1.14	1.97	2 3/16	4.03	2.76	0.950
×1/8	0.116	4.75	1.30	22.9	22.9	1.78	1.19	1.17	1.40	2 7/16	2.84	1.92	0.967

Note: For width-to-thickness criteria, refer to Table 1-12A.
– Indicates flat depth or width is too small to establish a workable flat.

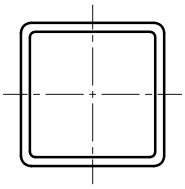
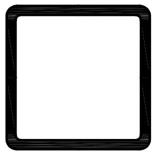


Table 1-12 (continued)
Square HSS
Dimensions and Properties



HSS2½–HSS2

Shape	Design Wall Thickness, *t*	Nominal Wt.	Area, *A*	*b*/*t*	*h*/*t*	*I*	*S*	*r*	*Z*	Workable Flat	Torsion *J*	Torsion *C*	Surface Area
	in.	lb/ft	in.^{2}			in.^{4}	in.^{3}	in.	in.^{3}	in.	in.^{4}	in.^{3}	ft^2/ft
HSS2½×2½×5/16	0.291	8.45	2.35	5.59	5.59	1.82	1.46	0.880	1.88	–	3.20	2.74	0.750
×¼	0.233	7.11	1.97	7.73	7.73	1.63	1.30	0.908	1.63	–	2.79	2.35	0.767
×3/16	0.174	5.59	1.54	11.4	11.4	1.35	1.08	0.937	1.32	–	2.25	1.86	0.784
×1/8	0.116	3.90	1.07	18.6	18.6	0.998	0.799	0.965	0.947	–	1.61	1.31	0.800
HSS2¼×2¼×¼	0.233	6.26	1.74	6.66	6.66	1.13	1.01	0.806	1.28	–	1.96	1.85	0.683
×3/16	0.174	4.96	1.37	9.93	9.93	0.953	0.847	0.835	1.04	–	1.60	1.48	0.700
×1/8	0.116	3.48	0.956	16.4	16.4	0.712	0.633	0.863	0.755	–	1.15	1.05	0.717
HSS2×2×¼	0.233	5.41	1.51	5.58	5.58	0.747	0.747	0.704	0.964	–	1.31	1.41	0.600
×3/16	0.174	4.32	1.19	8.49	8.49	0.641	0.641	0.733	0.797	–	1.09	1.14	0.617
×1/8	0.116	3.05	0.840	14.2	14.2	0.486	0.486	0.761	0.584	–	0.796	0.817	0.633

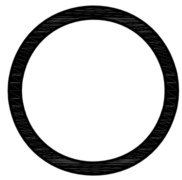
Note: For width-to-thickness criteria, refer to Table 1-12A.
– Indicates flat depth or width is too small to establish a workable flat.

Table 1-12A
Width-to-Thickness Criteria for Rectangular and Square HSS

Nominal Wall Thickness, in.	Width-to-Thickness Criteria for Rectangular and Square HSS			
	Compression	Flexure		Shear
	Nonslender up to	Compact up to	Compact up to	C_{v2} = 1.0 up to
	Flange Width, in.	Flange Width, in.	Web Height, in.	Web Height, in.
7/8	24	22	24	24
3/4	24	20		
5/8	20	16		
1/2	16	12		
3/8	12	10	20	20
5/16	10	8	18	18
1/4	8	6	14	14
3/16	6	5	10	10
1/8	4	3	7	7

Note: Width-to-thickness criteria given for F_y = 50 ksi.

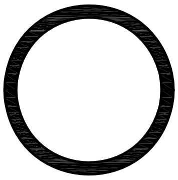
Table 1-13
Round HSS
Dimensions and Properties



HSS20.000–HSS10.000

Shape	Design Wall Thick-ness, t	Nom-inal Wt.	Area, A	D/t	I	S	r	Z	Torsion	
									J	C
	in.	lb/ft	in.^{2}		in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}
HSS20.000×0.500	0.465	104.00	28.5	43.0	1360	136	6.91	177	2720	272
×0.375[f]	0.349	78.67	21.5	57.3	1040	104	6.95	135	2080	208
HSS18.000×0.500	0.465	93.54	25.6	38.7	985	109	6.20	143	1970	219
×0.375[f]	0.349	70.66	19.4	51.6	754	83.8	6.24	109	1510	168
HSS16.000×0.625	0.581	103.00	28.1	27.5	838	105	5.46	138	1680	209
×0.500	0.465	82.85	22.7	34.4	685	85.7	5.49	112	1370	171
×0.438	0.407	72.87	19.9	39.3	606	75.8	5.51	99.0	1210	152
×0.375[f]	0.349	62.64	17.2	45.8	526	65.7	5.53	85.5	1050	131
×0.312[f]	0.291	52.32	14.4	55.0	443	55.4	5.55	71.8	886	111
×0.250[f]	0.233	42.09	11.5	68.7	359	44.8	5.58	57.9	717	89.7
HSS14.000×0.625	0.581	89.36	24.5	24.1	552	78.9	4.75	105	1100	158
×0.500	0.465	72.16	19.8	30.1	453	64.8	4.79	85.2	907	130
×0.375	0.349	54.62	15.0	40.1	349	49.8	4.83	65.1	698	100
×0.312[f]	0.291	45.65	12.5	48.1	295	42.1	4.85	54.7	589	84.2
×0.250[f]	0.233	36.75	10.1	60.1	239	34.1	4.87	44.2	478	68.2
HSS12.750×0.500	0.465	65.48	17.9	27.4	339	53.2	4.35	70.2	678	106
×0.375	0.349	49.61	13.6	36.5	262	41.0	4.39	53.7	523	82.1
×0.250[f]	0.233	33.41	9.16	54.7	180	28.2	4.43	36.5	359	56.3
HSS10.750×0.500	0.465	54.79	15.0	23.1	199	37.0	3.64	49.2	398	74.1
×0.375	0.349	41.59	11.4	30.8	154	28.7	3.68	37.8	309	57.4
×0.250[f]	0.233	28.06	7.70	46.1	106	19.8	3.72	25.8	213	39.6
HSS10.000×0.625	0.581	62.64	17.2	17.2	191	38.3	3.34	51.6	383	76.6
×0.500	0.465	50.78	13.9	21.5	159	31.7	3.38	42.3	317	63.5
×0.375	0.349	38.58	10.6	28.7	123	24.7	3.41	32.5	247	49.3
×0.312	0.291	32.31	8.88	34.4	105	20.9	3.43	27.4	209	41.9
×0.250	0.233	26.06	7.15	42.9	85.3	17.1	3.45	22.2	171	34.1
×0.188[f]	0.174	19.72	5.37	57.5	64.8	13.0	3.47	16.8	130	25.9

[f] Shape exceeds compact limit for flexure with $F_y = 46$ ksi.



HSS9.625–HSS6.875

Table 1-13 (continued)
Round HSS
Dimensions and Properties

Shape	Design Wall Thickness, t	Nominal Wt.	Area, A	D/t	I	S	r	Z	Torsion J	Torsion C
	in.	lb/ft	in.^{2}		in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}
HSS9.625×0.500	0.465	48.77	13.4	20.7	141	29.2	3.24	39.0	281	58.5
×0.375	0.349	37.08	10.2	27.6	110	22.8	3.28	30.0	219	45.5
×0.312	0.291	31.06	8.53	33.1	93.0	19.3	3.30	25.4	186	38.7
×0.250	0.233	25.06	6.87	41.3	75.9	15.8	3.32	20.6	152	31.5
×0.188^f	0.174	18.97	5.17	55.3	57.7	12.0	3.34	15.5	115	24.0
HSS8.625×0.625	0.581	53.45	14.7	14.8	119	27.7	2.85	37.7	239	55.4
×0.500	0.465	43.43	11.9	18.5	100	23.1	2.89	31.0	199	46.2
×0.375	0.349	33.07	9.07	24.7	77.8	18.0	2.93	23.9	156	36.1
×0.322	0.300	28.58	7.85	28.8	68.1	15.8	2.95	20.8	136	31.6
×0.250	0.233	22.38	6.14	37.0	54.1	12.5	2.97	16.4	108	25.1
×0.188^f	0.174	16.96	4.62	49.6	41.3	9.57	2.99	12.4	82.5	19.1
HSS7.625×0.375	0.349	29.06	7.98	21.8	52.9	13.9	2.58	18.5	106	27.8
×0.328	0.305	25.59	7.01	25.0	47.1	12.3	2.59	16.4	94.1	24.7
HSS7.500×0.500	0.465	37.42	10.3	16.1	63.9	17.0	2.49	23.0	128	34.1
×0.375	0.349	28.56	7.84	21.5	50.2	13.4	2.53	17.9	100	26.8
×0.312	0.291	23.97	6.59	25.8	42.9	11.4	2.55	15.1	85.8	22.9
×0.250	0.233	19.38	5.32	32.2	35.2	9.37	2.57	12.3	70.3	18.7
×0.188	0.174	14.70	4.00	43.1	26.9	7.17	2.59	9.34	53.8	14.3
HSS7.000×0.500	0.465	34.74	9.55	15.1	51.2	14.6	2.32	19.9	102	29.3
×0.375	0.349	26.56	7.29	20.1	40.4	11.6	2.35	15.5	80.9	23.1
×0.312	0.291	22.31	6.13	24.1	34.6	9.88	2.37	13.1	69.1	19.8
×0.250	0.233	18.04	4.95	30.0	28.4	8.11	2.39	10.7	56.8	16.2
×0.188	0.174	13.69	3.73	40.2	21.7	6.21	2.41	8.11	43.5	12.4
×0.125^f	0.116	9.19	2.51	60.3	14.9	4.25	2.43	5.50	29.7	8.49
HSS6.875×0.500	0.465	34.07	9.36	14.8	48.3	14.1	2.27	19.1	96.7	28.1
×0.375	0.349	26.06	7.16	19.7	38.2	11.1	2.31	14.9	76.4	22.2
×0.312	0.291	21.89	6.02	23.6	32.7	9.51	2.33	12.6	65.4	19.0
×0.250	0.233	17.71	4.86	29.5	26.8	7.81	2.35	10.3	53.7	15.6
×0.188	0.174	13.44	3.66	39.5	20.6	5.99	2.37	7.81	41.1	12.0

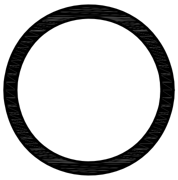
[f] Shape exceeds compact limit for flexure with F_y = 46 ksi.

Table 1-13 (continued)
Round HSS
Dimensions and Properties

HSS6.625–HSS5.000

Shape	Design Wall Thickness, t	Nominal Wt.	Area, A	D/t	I	S	r	Z	Torsion J	Torsion C
	in.	lb/ft	in.^{2}		in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}
HSS6.625×0.500	0.465	32.74	9.00	14.2	42.9	13.0	2.18	17.7	85.9	25.9
×0.432	0.402	28.60	7.86	16.5	38.2	11.5	2.20	15.6	76.4	23.1
×0.375	0.349	25.06	6.88	19.0	34.0	10.3	2.22	13.8	68.0	20.5
×0.312	0.291	21.06	5.79	22.8	29.1	8.79	2.24	11.7	58.2	17.6
×0.280	0.260	18.99	5.20	25.5	26.4	7.96	2.25	10.5	52.7	15.9
×0.250	0.233	17.04	4.68	28.4	23.9	7.22	2.26	9.52	47.9	14.4
×0.188	0.174	12.94	3.53	38.1	18.4	5.54	2.28	7.24	36.7	11.1
×0.125[f]	0.116	8.69	2.37	57.1	12.6	3.79	2.30	4.92	25.1	7.59
HSS6.000×0.500	0.465	29.40	8.09	12.9	31.2	10.4	1.96	14.3	62.4	20.8
×0.375	0.349	22.55	6.20	17.2	24.8	8.28	2.00	11.2	49.7	16.6
×0.312	0.291	18.97	5.22	20.6	21.3	7.11	2.02	9.49	42.6	14.2
×0.280	0.260	17.12	4.69	23.1	19.3	6.45	2.03	8.57	38.7	12.9
×0.250	0.233	15.37	4.22	25.8	17.6	5.86	2.04	7.75	35.2	11.7
×0.188	0.174	11.68	3.18	34.5	13.5	4.51	2.06	5.91	27.0	9.02
×0.125[f]	0.116	7.85	2.14	51.7	9.28	3.09	2.08	4.02	18.6	6.19
HSS5.563×0.500	0.465	27.06	7.45	12.0	24.4	8.77	1.81	12.1	48.8	17.5
×0.375	0.349	20.80	5.72	15.9	19.5	7.02	1.85	9.50	39.0	14.0
×0.258	0.240	14.63	4.01	23.2	14.2	5.12	1.88	6.80	28.5	10.2
×0.188	0.174	10.80	2.95	32.0	10.7	3.85	1.91	5.05	21.4	7.70
×0.134[f]	0.124	7.78	2.12	44.9	7.84	2.82	1.92	3.67	15.7	5.64
HSS5.500×0.500	0.465	26.73	7.36	11.8	23.5	8.55	1.79	11.8	47.0	17.1
×0.375	0.349	20.55	5.65	15.8	18.8	6.84	1.83	9.27	37.6	13.7
×0.258	0.240	14.46	3.97	22.9	13.7	5.00	1.86	6.64	27.5	10.0
HSS5.000×0.500	0.465	24.05	6.62	10.8	17.2	6.88	1.61	9.60	34.4	13.8
×0.375	0.349	18.54	5.10	14.3	13.9	5.55	1.65	7.56	27.7	11.1
×0.312	0.291	15.64	4.30	17.2	12.0	4.79	1.67	6.46	24.0	9.58
×0.258	0.240	13.08	3.59	20.8	10.2	4.08	1.69	5.44	20.4	8.15
×0.250	0.233	12.69	3.49	21.5	9.94	3.97	1.69	5.30	19.9	7.95
×0.188	0.174	9.67	2.64	28.7	7.69	3.08	1.71	4.05	15.4	6.15
×0.125	0.116	6.51	1.78	43.1	5.31	2.12	1.73	2.77	10.6	4.25

[f] Shape exceeds compact limit for flexure with F_y = 46 ksi.

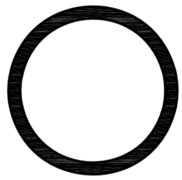


HSS4.500–
HSS2.500

Table 1-13 (continued)
Round HSS
Dimensions and Properties

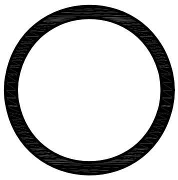
Shape	Design Wall Thick-ness, *t*	Nom-inal Wt.	Area, *A*	*D*/*t*	*I*	*S*	*r*	*Z*	Torsion	
									J	*C*
	in.	lb/ft	in.^{2}		in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}
HSS4.500×0.375	0.349	16.54	4.55	12.9	9.87	4.39	1.47	6.03	19.7	8.78
×0.337	0.313	15.00	4.12	14.4	9.07	4.03	1.48	5.50	18.1	8.06
×0.237	0.220	10.80	2.96	20.5	6.79	3.02	1.52	4.03	13.6	6.04
×0.188	0.174	8.67	2.36	25.9	5.54	2.46	1.53	3.26	11.1	4.93
×0.125	0.116	5.85	1.60	38.8	3.84	1.71	1.55	2.23	7.68	3.41
HSS4.000×0.313	0.291	12.34	3.39	13.7	5.87	2.93	1.32	4.01	11.7	5.87
×0.250	0.233	10.00	2.76	17.2	4.91	2.45	1.33	3.31	9.82	4.91
×0.237	0.220	9.53	2.61	18.2	4.68	2.34	1.34	3.15	9.36	4.68
×0.226	0.210	9.12	2.50	19.0	4.50	2.25	1.34	3.02	9.01	4.50
×0.220	0.205	8.89	2.44	19.5	4.41	2.21	1.34	2.96	8.83	4.41
×0.188	0.174	7.66	2.09	23.0	3.83	1.92	1.35	2.55	7.67	3.83
×0.125	0.116	5.18	1.42	34.5	2.67	1.34	1.37	1.75	5.34	2.67
HSS3.500×0.313	0.291	10.66	2.93	12.0	3.81	2.18	1.14	3.00	7.61	4.35
×0.300	0.279	10.26	2.82	12.5	3.69	2.11	1.14	2.90	7.38	4.22
×0.250	0.233	8.69	2.39	15.0	3.21	1.83	1.16	2.49	6.41	3.66
×0.216	0.201	7.58	2.08	17.4	2.84	1.63	1.17	2.19	5.69	3.25
×0.203	0.189	7.15	1.97	18.5	2.70	1.54	1.17	2.07	5.41	3.09
×0.188	0.174	6.66	1.82	20.1	2.52	1.44	1.18	1.93	5.04	2.88
×0.125	0.116	4.51	1.23	30.2	1.77	1.01	1.20	1.33	3.53	2.02
HSS3.000×0.250	0.233	7.35	2.03	12.9	1.95	1.30	0.982	1.79	3.90	2.60
×0.216	0.201	6.43	1.77	14.9	1.74	1.16	0.992	1.58	3.48	2.32
×0.203	0.189	6.07	1.67	15.9	1.66	1.10	0.996	1.50	3.31	2.21
×0.188	0.174	5.65	1.54	17.2	1.55	1.03	1.00	1.39	3.10	2.06
×0.152	0.141	4.63	1.27	21.3	1.30	0.865	1.01	1.15	2.59	1.73
×0.134	0.124	4.11	1.12	24.2	1.16	0.774	1.02	1.03	2.32	1.55
×0.125	0.116	3.84	1.05	25.9	1.09	0.730	1.02	0.965	2.19	1.46
HSS2.875×0.250	0.233	7.02	1.93	12.3	1.70	1.18	0.938	1.63	3.40	2.37
×0.203	0.189	5.80	1.59	15.2	1.45	1.01	0.952	1.37	2.89	2.01
×0.188	0.174	5.40	1.48	16.5	1.35	0.941	0.957	1.27	2.70	1.88
×0.125	0.116	3.67	1.01	24.8	0.958	0.667	0.976	0.884	1.92	1.33
HSS2.500×0.250	0.233	6.01	1.66	10.7	1.08	0.862	0.806	1.20	2.15	1.72
×0.188	0.174	4.65	1.27	14.4	0.865	0.692	0.825	0.943	1.73	1.38
×0.125	0.116	3.17	0.869	21.6	0.619	0.495	0.844	0.660	1.24	0.990

Table 1-13 (continued)
Round HSS
Dimensions and Properties



HSS2.375–HSS1.660

Shape	Design Wall Thick-ness, t	Nom-inal Wt.	Area, A	D/t	I	S	r	Z	Torsion	
									J	C
	in.	lb/ft	in.^{2}		in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}
HSS2.375×0.250	0.233	5.68	1.57	10.2	0.910	0.766	0.762	1.07	1.82	1.53
×0.218	0.203	5.03	1.39	11.7	0.824	0.694	0.771	0.960	1.65	1.39
×0.188	0.174	4.40	1.20	13.6	0.733	0.617	0.781	0.845	1.47	1.23
×0.154	0.143	3.66	1.00	16.6	0.627	0.528	0.791	0.713	1.25	1.06
×0.125	0.116	3.01	0.823	20.5	0.527	0.443	0.800	0.592	1.05	0.887
HSS1.900×0.188	0.174	3.44	0.943	10.9	0.355	0.374	0.613	0.520	0.710	0.747
×0.145	0.135	2.72	0.749	14.1	0.293	0.309	0.626	0.421	0.586	0.617
×0.120	0.111	2.28	0.624	17.1	0.251	0.264	0.634	0.356	0.501	0.527
HSS1.660×0.140	0.130	2.27	0.625	12.8	0.184	0.222	0.543	0.305	0.368	0.444

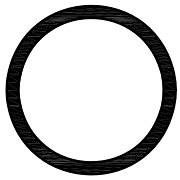


PIPE

Table 1-14
Pipes
Dimensions and Properties

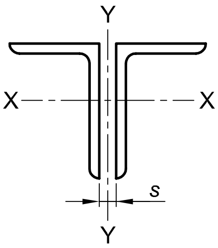
Shape	Nominal Wt.	Dimensions: Outside Diameter	Dimensions: Inside Diameter	Nominal Wall Thickness	Design Wall Thickness	Area	D/t	I	S	r	J	Z
	lb/ft	in.	in.	in.	in.	in.^{2}		in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}
Standard Weight (Std.)												
Pipe 26 Std.	103	26.000	25.3	0.375	0.349	28.2	74.5	2320	178	9.07	4640	230
Pipe 24 Std.	94.7	24.000	23.3	0.375	0.349	26.0	68.8	1820	152	8.36	3640	196
Pipe 20 Std.	78.7	20.000	19.3	0.375	0.349	21.6	57.3	1040	104	6.95	2090	135
Pipe 18 Std.	70.7	18.000	17.3	0.375	0.349	19.4	51.6	756	84.0	6.24	1510	109
Pipe 16 Std.	62.6	16.000	15.3	0.375	0.349	17.2	45.8	527	65.9	5.53	1050	85.7
Pipe 14 Std.	54.6	14.000	13.3	0.375	0.349	15.0	40.1	350	50.0	4.83	700	65.2
Pipe 12 Std.	49.6	12.750	12.0	0.375	0.349	13.7	36.5	262	41.0	4.39	523	53.7
Pipe 10 Std.	40.5	10.750	10.0	0.365	0.340	11.5	31.6	151	28.1	3.68	302	36.9
Pipe 8 Std.	28.6	8.625	7.98	0.322	0.300	7.85	28.8	68.1	15.8	2.95	136	20.8
Pipe 6 Std.	19.0	6.625	6.07	0.280	0.261	5.20	25.4	26.5	7.99	2.25	52.9	10.6
Pipe 5 Std.	14.6	5.563	5.05	0.258	0.241	4.01	23.1	14.3	5.14	1.88	28.6	6.83
Pipe 4 Std.	10.8	4.500	4.03	0.237	0.221	2.96	20.4	6.82	3.03	1.51	13.6	4.05
Pipe $3^1/_2$ Std.	9.12	4.000	3.55	0.226	0.211	2.50	19.0	4.52	2.26	1.34	9.04	3.03
Pipe 3 Std.	7.58	3.500	3.07	0.216	0.201	2.07	17.4	2.85	1.63	1.17	5.69	2.19
Pipe $2^1/_2$ Std.	5.80	2.875	2.47	0.203	0.189	1.61	15.2	1.45	1.01	0.952	2.89	1.37
Pipe 2 Std.	3.66	2.375	2.07	0.154	0.143	1.02	16.6	0.627	0.528	0.791	1.25	0.713
Pipe $1^1/_2$ Std.	2.72	1.900	1.61	0.145	0.135	0.749	14.1	0.293	0.309	0.626	0.586	0.421
Pipe $1^1/_4$ Std.	2.27	1.660	1.38	0.140	0.130	0.625	12.8	0.184	0.222	0.543	0.368	0.305
Pipe 1 Std.	1.68	1.315	1.05	0.133	0.124	0.469	10.6	0.0830	0.126	0.423	0.166	0.177
Pipe $^3/_4$ Std.	1.13	1.050	0.824	0.113	0.105	0.312	10.0	0.0350	0.0671	0.336	0.0700	0.0942
Pipe $^1/_2$ Std.	0.850	0.840	0.622	0.109	0.101	0.234	8.32	0.0160	0.0388	0.264	0.0320	0.0555
Extra Strong (x-Strong)												
Pipe 26 x-Strong	136	26.000	25.1	0.500	0.465	36.1	55.9	2950	227	9.03	5900	294
Pipe 24 x-Strong	126	24.000	23.1	0.500	0.465	33.3	51.6	2310	192	8.33	4620	250
Pipe 20 x-Strong	104	20.000	19.1	0.500	0.465	27.6	43.0	1320	132	6.91	2640	172
Pipe 18 x-Strong	93.5	18.000	17.1	0.500	0.465	24.8	38.7	956	106	6.21	1910	139
Pipe 16 x-Strong	82.9	16.000	15.1	0.500	0.465	22.0	34.4	665	83.1	5.50	1330	109
Pipe 14 x-Strong	72.2	14.000	13.1	0.500	0.465	19.2	30.1	440	62.9	4.79	880	82.7
Pipe 12 x-Strong	65.5	12.750	11.8	0.500	0.465	17.5	27.4	339	53.2	4.35	678	70.2
Pipe 10 x-Strong	54.8	10.750	9.75	0.500	0.465	15.1	23.1	199	37.0	3.64	398	49.2
Pipe 8 x-Strong	43.4	8.625	7.63	0.500	0.465	11.9	18.5	100	23.1	2.89	199	31.0
Pipe 6 x-Strong	28.6	6.625	5.76	0.432	0.403	7.83	16.4	38.3	11.6	2.20	76.6	15.6
Pipe 5 x-Strong	20.8	5.563	4.81	0.375	0.349	5.73	15.9	19.5	7.02	1.85	39.0	9.50
Pipe 4 x-Strong	15.0	4.500	3.83	0.337	0.315	4.14	14.3	9.12	4.05	1.48	18.2	5.53
Pipe $3^1/_2$ x-Strong	12.5	4.000	3.36	0.318	0.296	3.43	13.5	5.94	2.97	1.31	11.9	4.07
Pipe 3 x-Strong	10.3	3.500	2.90	0.300	0.280	2.83	12.5	3.70	2.11	1.14	7.40	2.91
Pipe $2^1/_2$ x-Strong	7.67	2.875	2.32	0.276	0.257	2.10	11.2	1.83	1.27	0.930	3.66	1.77
Pipe 2 x-Strong	5.03	2.375	1.94	0.218	0.204	1.40	11.7	0.827	0.696	0.771	1.65	0.964
Pipe $1^1/_2$ x-Strong	3.63	1.900	1.50	0.200	0.186	1.00	10.2	0.372	0.392	0.610	0.744	0.549
Pipe $1^1/_4$ x-Strong	3.00	1.660	1.28	0.191	0.178	0.837	9.33	0.231	0.278	0.528	0.462	0.393
Pipe 1 x-Strong	2.17	1.315	0.957	0.179	0.166	0.602	7.92	0.101	0.154	0.410	0.202	0.221
Pipe $^3/_4$ x-Strong	1.48	1.050	0.742	0.154	0.143	0.407	7.34	0.0430	0.0818	0.325	0.0860	0.119
Pipe $^1/_2$ x-Strong	1.09	0.840	0.546	0.147	0.137	0.303	6.13	0.0190	0.0462	0.253	0.0380	0.0686

Table 1-14 (continued)
Pipes
Dimensions and Properties



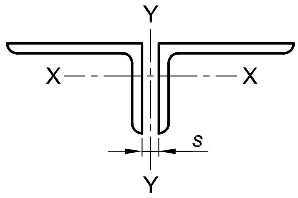
PIPE

Shape	Nominal Wt.	Dimensions		Nominal Wall Thickness	Design Wall Thickness	Area	D/t	*I*	*S*	*r*	*J*	*Z*
		Outside Diameter	Inside Diameter									
	lb/ft	in.	in.	in.	in.	$in.^2$		$in.^4$	$in.^3$	in.	$in.^4$	$in.^3$
Double-Extra Strong (xx-Strong)												
Pipe 12 xx-Strong	126	12.750	10.9	1.00	0.930	35.4	13.8	625	97.6	4.20	1250	134
Pipe 10 xx-Strong	104	10.750	8.94	1.00	0.930	28.8	11.6	354	65.6	3.51	709	90.9
Pipe 8 xx-Strong	72.5	8.625	6.88	0.875	0.816	20.0	10.6	154	35.8	2.78	308	49.9
Pipe 6 xx-Strong	53.2	6.625	4.90	0.864	0.805	14.7	8.23	63.5	19.2	2.08	127	27.4
Pipe 5 xx-Strong	38.6	5.563	4.06	0.750	0.699	10.7	7.96	32.2	11.6	1.74	64.4	16.7
Pipe 4 xx-Strong	27.6	4.500	3.15	0.674	0.628	7.66	7.17	14.7	6.53	1.39	29.4	9.50
Pipe 3 xx-Strong	18.6	3.500	2.30	0.600	0.559	5.17	6.26	5.79	3.31	1.06	11.6	4.89
Pipe 2$^1/_2$ xx-Strong	13.7	2.875	1.77	0.552	0.514	3.83	5.59	2.78	1.94	0.854	5.56	2.91
Pipe 2 xx-Strong	9.04	2.375	1.50	0.436	0.406	2.51	5.85	1.27	1.07	0.711	2.54	1.60



LLBB

Table 1-15
Double Angles
Properties

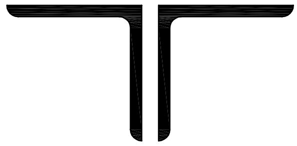


SLBB

Shape	Area, A	Radius of Gyration							
		LLBB				SLBB			
		r_y			r_x	r_y			r_x
		Separation, s, in.				Separation, s, in.			
		0	3/4	1 1/2		0	3/4	1 1/2	
	in.^{2}	in.	in.	in.	in.	in.	in.	in.	in.
2L12×12×1 3/8	62.2	5.06	5.32	5.60	3.64	5.06	5.32	5.60	3.64
×1 1/4	56.8	5.04	5.29	5.57	3.66	5.04	5.29	5.57	3.66
×1 1/8	51.6	5.02	5.28	5.55	3.68	5.02	5.28	5.55	3.68
×1	46.0	5.00	5.25	5.54	3.70	5.00	5.25	5.54	3.70
2L10×10×1 3/8	51.2	4.25	4.53	4.80	3.00	4.25	4.53	4.80	3.00
×1 1/4	46.8	4.22	4.49	4.78	3.02	4.22	4.49	4.78	3.02
×1 1/8	42.6	4.20	4.46	4.75	3.03	4.20	4.46	4.75	3.03
×1	38.0	4.18	4.45	4.73	3.05	4.18	4.45	4.73	3.05
×7/8	33.6	4.15	4.42	4.69	3.07	4.15	4.42	4.69	3.07
×3/4	29.0	4.15	4.41	4.68	3.10	4.15	4.41	4.68	3.10

Note: For width-to-thickness criteria, refer to Table 1-7B.

Table 1-15 (continued)
Double Angles
Properties



2L12–2L10

Shape	Flexural-Torsional Properties												Single Angle Properties	
	LLBB						SLBB							
	Separation, *s*, in.						Separation, *s*, in.						Area, *A*	r_z
	0		3/4		1 1/2		0		3/4		1 1/2			
	$\bar{r}_o$	*H*	$\bar{r}_o$	*H*	$\bar{r}_o$	*H*	$\bar{r}_o$	*H*	$\bar{r}_o$	*H*	$\bar{r}_o$	*H*		
	in.		in.		in.		in.		in.		in.		in.^{2}	in.
2L12×12×1 3/8	6.84	0.831	7.03	0.840	7.25	0.850	6.84	0.831	7.03	0.840	7.25	0.850	31.1	2.30
×1 1/4	6.84	0.829	7.03	0.839	7.24	0.848	6.84	0.829	7.03	0.839	7.24	0.848	28.4	2.31
×1 1/8	6.85	0.827	7.04	0.837	7.24	0.846	6.85	0.827	7.04	0.837	7.24	0.846	25.8	2.33
×1	6.85	0.826	7.03	0.834	7.25	0.844	6.85	0.826	7.03	0.840	7.25	0.844	23.0	2.34
2L10×10×1 3/8	5.69	0.835	5.90	0.847	6.12	0.858	5.69	0.835	5.90	0.847	6.12	0.858	25.6	1.91
×1 1/4	5.68	0.832	5.89	0.844	6.11	0.855	5.68	0.832	5.89	0.844	6.11	0.855	23.4	1.91
×1 1/8	5.68	0.831	5.88	0.842	6.10	0.853	5.68	0.831	5.88	0.842	6.10	0.853	21.3	1.92
×1	5.69	0.828	5.89	0.839	6.10	0.850	5.69	0.828	5.89	0.839	6.10	0.850	19.0	1.92
×7/8	5.68	0.827	5.87	0.838	6.08	0.849	5.68	0.827	5.87	0.838	6.08	0.849	16.8	1.93
×3/4	5.70	0.825	5.89	0.836	6.10	0.847	5.70	0.825	5.89	0.836	6.10	0.847	14.5	1.96

Note: For width-to-thickness criteria, refer to Table 1-7B.

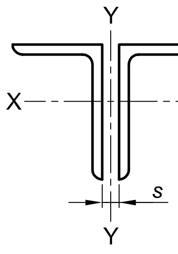
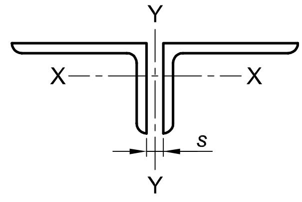


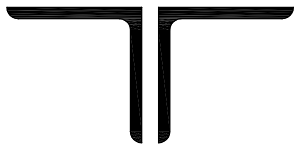
Table 1-15 (continued)
Double Angles
Properties



Shape	Area, A	Radius of Gyration							
		LLBB				SLBB			
		r_y			r_x	r_y			r_x
		Separation, s, in.				Separation, s, in.			
		0	3/8	3/4		0	3/8	3/4	
	in.^{2}	in.	in.	in.	in.	in.	in.	in.	in.
2L8×8×1 1/8	33.6	3.41	3.54	3.68	2.41	3.41	3.54	3.68	2.41
×1	30.2	3.39	3.52	3.66	2.43	3.39	3.52	3.66	2.43
×7/8	26.6	3.36	3.50	3.63	2.45	3.36	3.50	3.63	2.45
×3/4	23.0	3.34	3.47	3.61	2.46	3.34	3.47	3.61	2.46
×5/8	19.4	3.32	3.45	3.58	2.48	3.32	3.45	3.58	2.48
×9/16	17.5	3.31	3.44	3.57	2.49	3.31	3.44	3.57	2.49
×1/2	15.7	3.30	3.43	3.56	2.49	3.30	3.43	3.56	2.49
2L8×6×1	26.2	2.39	2.52	2.66	2.49	3.63	3.77	3.91	1.72
×7/8	23.0	2.37	2.50	2.63	2.50	3.61	3.75	3.89	1.74
×3/4	20.0	2.35	2.47	2.61	2.52	3.59	3.72	3.86	1.75
×5/8	16.8	2.33	2.45	2.59	2.54	3.57	3.70	3.84	1.77
×9/16	15.2	2.32	2.44	2.58	2.55	3.55	3.69	3.83	1.78
×1/2	13.6	2.31	2.43	2.56	2.55	3.54	3.68	3.81	1.79
×7/16	12.0	2.30	2.42	2.55	2.56	3.53	3.66	3.80	1.80
2L8×4×1	22.2	1.46	1.60	1.75	2.51	3.94	4.08	4.23	1.03
×7/8	19.6	1.44	1.57	1.72	2.53	3.91	4.06	4.21	1.04
×3/4	17.0	1.42	1.55	1.69	2.55	3.89	4.03	4.18	1.05
×5/8	14.3	1.39	1.52	1.66	2.56	3.86	4.00	4.15	1.06
×9/16	13.0	1.38	1.51	1.65	2.57	3.85	3.99	4.13	1.07
×1/2	11.6	1.38	1.50	1.63	2.58	3.83	3.97	4.12	1.08
×7/16	10.2	1.37	1.49	1.62	2.59	3.82	3.96	4.10	1.09
2L7×4×3/4	15.5	1.48	1.61	1.75	2.21	3.34	3.48	3.63	1.08
×5/8	13.0	1.45	1.58	1.73	2.23	3.31	3.46	3.60	1.10
×1/2	10.5	1.44	1.56	1.70	2.25	3.29	3.43	3.57	1.11
×7/16	9.26	1.43	1.55	1.68	2.26	3.28	3.42	3.56	1.12
×3/8	8.00	1.42	1.54	1.67	2.27	3.26	3.40	3.54	1.12
2L6×6×1	22.0	2.58	2.72	2.86	1.79	2.58	2.72	2.86	1.79
×7/8	19.5	2.56	2.70	2.84	1.81	2.56	2.70	2.84	1.81
×3/4	16.9	2.54	2.67	2.81	1.82	2.54	2.67	2.81	1.82
×5/8	14.3	2.52	2.65	2.79	1.84	2.52	2.65	2.79	1.84
×9/16	12.9	2.51	2.64	2.78	1.85	2.51	2.64	2.78	1.85
×1/2	11.5	2.50	2.63	2.76	1.86	2.50	2.63	2.76	1.86
×7/16	10.2	2.49	2.62	2.75	1.86	2.49	2.62	2.75	1.86
×3/8	8.76	2.48	2.60	2.74	1.87	2.48	2.60	2.74	1.87
×5/16	7.34	2.47	2.59	2.72	1.88	2.47	2.59	2.72	1.88

Note: For width-to-thickness criteria, refer to Table 1-7B.

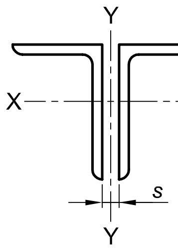
Table 1-15 (continued)
Double Angles
Properties



2L8–2L6

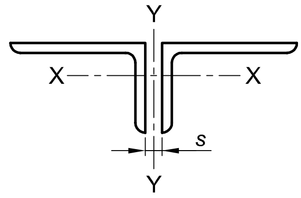
Shape	Flexural-Torsional Properties												Single Angle Properties	
	LLBB						SLBB							
	Separation, *s*, in.						Separation, *s*, in.						Area, *A*	r_z
	0		3/8		3/4		0		3/8		3/4			
	$\bar{r}_o$	*H*	$\bar{r}_o$	*H*	$\bar{r}_o$	*H*	$\bar{r}_o$	*H*	$\bar{r}_o$	*H*	$\bar{r}_o$	*H*		
	in.		in.		in.		in.		in.		in.		in.^{2}	in.
2L8×8×1 1/8	4.56	0.837	4.66	0.844	4.77	0.851	4.56	0.837	4.66	0.844	4.77	0.851	16.8	1.56
×1	4.56	0.834	4.66	0.841	4.77	0.848	4.56	0.834	4.66	0.841	4.77	0.848	15.1	1.56
×7/8	4.56	0.831	4.66	0.838	4.76	0.845	4.56	0.831	4.66	0.838	4.76	0.845	13.3	1.57
×3/4	4.56	0.829	4.66	0.836	4.76	0.843	4.56	0.829	4.66	0.836	4.76	0.843	11.5	1.57
×5/8	4.56	0.826	4.66	0.833	4.76	0.840	4.56	0.826	4.66	0.833	4.76	0.840	9.69	1.58
×9/16	4.56	0.825	4.65	0.832	4.75	0.839	4.56	0.825	4.65	0.832	4.75	0.839	8.77	1.58
×1/2	4.56	0.824	4.65	0.831	4.75	0.837	4.56	0.824	4.65	0.831	4.75	0.837	7.84	1.59
2L8×6×1	4.06	0.721	4.14	0.732	4.23	0.742	4.18	0.924	4.30	0.929	4.43	0.933	13.1	1.28
×7/8	4.07	0.718	4.14	0.728	4.23	0.739	4.17	0.922	4.29	0.926	4.42	0.930	11.5	1.28
×3/4	4.07	0.714	4.15	0.725	4.23	0.735	4.17	0.919	4.28	0.924	4.40	0.928	9.99	1.29
×5/8	4.08	0.712	4.16	0.722	4.24	0.732	4.16	0.917	4.27	0.921	4.39	0.926	8.41	1.29
×9/16	4.09	0.710	4.16	0.720	4.24	0.731	4.15	0.916	4.27	0.920	4.39	0.924	7.61	1.30
×1/2	4.09	0.709	4.16	0.719	4.24	0.729	4.15	0.915	4.26	0.919	4.38	0.923	6.80	1.30
×7/16	4.09	0.708	4.16	0.718	4.24	0.728	4.15	0.913	4.26	0.918	4.38	0.922	5.99	1.31
2L8×4×1	3.86	0.568	3.91	0.580	3.97	0.594	4.11	0.983	4.25	0.984	4.39	0.985	11.1	0.844
×7/8	3.87	0.566	3.92	0.577	3.98	0.590	4.09	0.981	4.22	0.982	4.37	0.984	9.79	0.846
×3/4	3.88	0.564	3.93	0.575	3.99	0.587	4.07	0.980	4.20	0.981	4.35	0.983	8.49	0.850
×5/8	3.89	0.562	3.94	0.573	3.99	0.585	4.05	0.979	4.18	0.980	4.32	0.981	7.16	0.856
×9/16	3.90	0.562	3.94	0.572	4.00	0.584	4.04	0.978	4.17	0.980	4.31	0.981	6.49	0.859
×1/2	3.90	0.561	3.95	0.571	4.00	0.583	4.03	0.978	4.16	0.979	4.30	0.980	5.80	0.863
×7/16	3.91	0.561	3.95	0.571	4.00	0.582	4.02	0.977	4.15	0.978	4.29	0.980	5.11	0.867
2L7×4×3/4	3.41	0.611	3.47	0.624	3.53	0.639	3.57	0.969	3.70	0.971	3.84	0.973	7.74	0.855
×5/8	3.42	0.608	3.47	0.621	3.54	0.635	3.55	0.967	3.68	0.969	3.82	0.971	6.50	0.860
×1/2	3.43	0.606	3.48	0.618	3.55	0.632	3.53	0.965	3.66	0.968	3.80	0.970	5.26	0.866
×7/16	3.43	0.605	3.49	0.617	3.55	0.630	3.53	0.964	3.66	0.967	3.79	0.969	4.63	0.869
×3/8	3.44	0.605	3.49	0.616	3.55	0.629	3.52	0.963	3.65	0.966	3.78	0.968	4.00	0.873
2L6×6×1	3.42	0.843	3.53	0.852	3.64	0.861	3.42	0.843	3.53	0.852	3.64	0.861	11.0	1.17
×7/8	3.42	0.839	3.53	0.848	3.63	0.857	3.42	0.839	3.53	0.848	3.63	0.857	9.75	1.17
×3/4	3.42	0.835	3.52	0.844	3.63	0.853	3.42	0.835	3.52	0.844	3.63	0.853	8.46	1.17
×5/8	3.42	0.831	3.52	0.840	3.62	0.849	3.42	0.831	3.52	0.840	3.62	0.849	7.13	1.17
×9/16	3.42	0.829	3.52	0.838	3.62	0.847	3.42	0.829	3.52	0.838	3.62	0.847	6.45	1.18
×1/2	3.42	0.827	3.52	0.836	3.62	0.846	3.42	0.827	3.52	0.836	3.62	0.846	5.77	1.18
×7/16	3.42	0.826	3.52	0.835	3.62	0.844	3.42	0.826	3.52	0.835	3.62	0.844	5.08	1.18
×3/8	3.42	0.824	3.51	0.833	3.61	0.842	3.42	0.824	3.51	0.833	3.61	0.842	4.38	1.19
×5/16	3.42	0.823	3.51	0.832	3.61	0.841	3.42	0.823	3.51	0.832	3.61	0.841	3.67	1.19

Note: For width-to-thickness criteria, refer to Table 1-7B.



LLBB

Table 1-15 (continued)
Double Angles
Properties

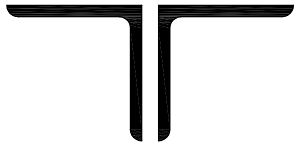


SLBB

Shape	Area, *A*	Radius of Gyration							
		LLBB				SLBB			
		r_y			r_x	r_y			r_x
		Separation, *s*, in.				Separation, *s*, in.			
		0	3/8	3/4		0	3/8	3/4	
	in.^{2}	in.	in.	in.	in.	in.	in.	in.	in.
2L6×4×7/8	16.0	1.57	1.71	1.86	1.86	2.82	2.96	3.11	1.10
×3/4	13.9	1.55	1.68	1.83	1.88	2.80	2.94	3.08	1.12
×5/8	11.7	1.53	1.66	1.80	1.89	2.77	2.91	3.06	1.13
×9/16	10.6	1.52	1.65	1.79	1.90	2.76	2.90	3.04	1.14
×1/2	9.50	1.51	1.64	1.77	1.91	2.75	2.89	3.03	1.14
×7/16	8.36	1.50	1.62	1.76	1.92	2.74	2.88	3.02	1.15
×3/8	7.22	1.49	1.61	1.75	1.93	2.73	2.86	3.00	1.16
×5/16	6.06	1.48	1.60	1.74	1.94	2.72	2.85	2.99	1.17
2L6×3 1/2×1/2	9.00	1.27	1.40	1.54	1.92	2.82	2.96	3.11	0.968
×3/8	6.88	1.26	1.38	1.52	1.93	2.80	2.94	3.08	0.984
×5/16	5.78	1.25	1.37	1.50	1.94	2.78	2.92	3.06	0.991
2L5×5×7/8	16.0	2.16	2.30	2.44	1.49	2.16	2.30	2.44	1.49
×3/4	14.0	2.13	2.27	2.41	1.50	2.13	2.27	2.41	1.50
×5/8	11.8	2.11	2.25	2.39	1.52	2.11	2.25	2.39	1.52
×1/2	9.58	2.09	2.22	2.36	1.53	2.09	2.22	2.36	1.53
×7/16	8.44	2.08	2.21	2.35	1.54	2.08	2.21	2.35	1.54
×3/8	7.30	2.07	2.20	2.34	1.55	2.07	2.20	2.34	1.55
×5/16	6.14	2.06	2.19	2.32	1.56	2.06	2.19	2.32	1.56
2L5×3 1/2×3/4	11.7	1.39	1.53	1.68	1.55	2.33	2.47	2.62	0.974
×5/8	9.86	1.37	1.50	1.65	1.56	2.30	2.45	2.59	0.987
×1/2	8.00	1.35	1.48	1.62	1.58	2.28	2.42	2.57	1.00
×3/8	6.10	1.33	1.46	1.59	1.59	2.26	2.39	2.54	1.02
×5/16	5.12	1.32	1.44	1.58	1.60	2.25	2.38	2.52	1.02
×1/4	4.14	1.31	1.43	1.57	1.61	2.23	2.37	2.51	1.03
2L5×3×1/2	7.50	1.11	1.24	1.39	1.58	2.35	2.50	2.64	0.824
×7/16	6.62	1.10	1.23	1.38	1.59	2.34	2.48	2.63	0.831
×3/8	5.72	1.09	1.22	1.36	1.60	2.33	2.47	2.62	0.838
×5/16	4.82	1.08	1.21	1.35	1.61	2.32	2.46	2.60	0.846
×1/4	3.88	1.07	1.19	1.33	1.62	2.30	2.44	2.58	0.853

Note: For width-to-thickness criteria, refer to Table 1-7B.

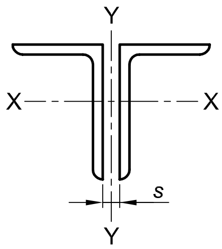
Table 1-15 (continued)
Double Angles
Properties



2L6–2L5

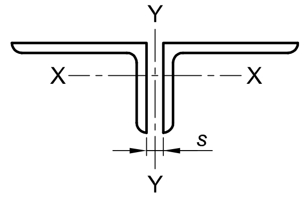
Shape	Flexural-Torsional Properties												Single Angle Properties	
	LLBB						SLBB							
	Separation, *s*, in.						Separation, *s*, in.						Area, *A*	r_z
	0		3/8		3/4		0		3/8		3/4			
	$\bar{r}_o$	*H*	$\bar{r}_o$	*H*	$\bar{r}_o$	*H*	$\bar{r}_o$	*H*	$\bar{r}_o$	*H*	$\bar{r}_o$	*H*		
	in.		in.		in.		in.		in.		in.		in.^{2}	in.
2L6×4×7/8	2.96	0.678	3.04	0.694	3.12	0.710	3.10	0.952	3.23	0.956	3.37	0.959	8.00	0.854
×3/4	2.97	0.673	3.04	0.688	3.12	0.705	3.09	0.949	3.22	0.953	3.35	0.957	6.94	0.856
×5/8	2.98	0.669	3.05	0.684	3.13	0.700	3.08	0.946	3.21	0.950	3.34	0.954	5.86	0.859
×9/16	2.98	0.667	3.05	0.682	3.13	0.697	3.07	0.945	3.20	0.949	3.33	0.953	5.31	0.861
×1/2	2.99	0.665	3.05	0.679	3.13	0.695	3.07	0.943	3.19	0.948	3.32	0.952	4.75	0.864
×7/16	2.99	0.663	3.06	0.678	3.13	0.693	3.06	0.942	3.19	0.946	3.31	0.950	4.18	0.867
×3/8	2.99	0.662	3.06	0.676	3.13	0.691	3.06	0.940	3.18	0.945	3.31	0.949	3.61	0.870
×5/16	3.00	0.661	3.06	0.674	3.13	0.689	3.05	0.939	3.17	0.944	3.30	0.948	3.03	0.874
2L6x3 1/2×1/2	2.94	0.615	2.99	0.630	3.06	0.646	3.04	0.964	3.17	0.967	3.31	0.969	4.50	0.756
×3/8	2.95	0.613	3.00	0.627	3.07	0.642	3.02	0.962	3.15	0.965	3.29	0.967	3.44	0.763
×5/16	2.95	0.612	3.00	0.625	3.07	0.641	3.02	0.960	3.14	0.964	3.28	0.966	2.89	0.767
2L5×5×7/8	2.85	0.845	2.96	0.856	3.07	0.866	2.85	0.845	2.96	0.856	3.07	0.866	8.00	0.971
×3/4	2.85	0.840	2.95	0.851	3.06	0.861	2.85	0.840	2.95	0.851	3.06	0.861	6.98	0.972
×5/8	2.85	0.835	2.95	0.846	3.06	0.857	2.85	0.835	2.95	0.846	3.06	0.857	5.90	0.975
×1/2	2.85	0.830	2.94	0.842	3.05	0.852	2.85	0.830	2.94	0.842	3.05	0.852	4.79	0.980
×7/16	2.85	0.828	2.94	0.839	3.05	0.850	2.85	0.828	2.94	0.839	3.05	0.850	4.22	0.983
×3/8	2.84	0.826	2.94	0.838	3.04	0.848	2.84	0.826	2.94	0.838	3.04	0.848	3.65	0.986
×5/16	2.84	0.825	2.94	0.836	3.04	0.847	2.84	0.825	2.94	0.836	3.04	0.847	3.07	0.990
2L5x3 1/2×3/4	2.49	0.699	2.57	0.717	2.66	0.736	2.60	0.943	2.73	0.949	2.86	0.953	5.85	0.744
×5/8	2.49	0.693	2.57	0.711	2.66	0.730	2.59	0.940	2.71	0.945	2.85	0.950	4.93	0.746
×1/2	2.50	0.688	2.58	0.705	2.66	0.724	2.58	0.936	2.70	0.942	2.83	0.947	4.00	0.750
×3/8	2.51	0.683	2.58	0.700	2.66	0.718	2.56	0.933	2.69	0.938	2.81	0.944	3.05	0.755
×5/16	2.51	0.682	2.58	0.698	2.66	0.716	2.56	0.931	2.68	0.937	2.81	0.942	2.56	0.758
×1/4	2.52	0.680	2.58	0.696	2.66	0.714	2.55	0.929	2.67	0.935	2.80	0.941	2.07	0.761
2L5×3×1/2	2.44	0.628	2.51	0.646	2.58	0.667	2.54	0.962	2.68	0.966	2.81	0.969	3.75	0.642
×7/16	2.45	0.626	2.51	0.644	2.58	0.664	2.54	0.961	2.67	0.964	2.80	0.968	3.31	0.644
×3/8	2.45	0.624	2.51	0.642	2.59	0.661	2.53	0.959	2.66	0.963	2.79	0.967	2.86	0.646
×5/16	2.46	0.623	2.52	0.640	2.59	0.659	2.52	0.958	2.65	0.962	2.78	0.965	2.41	0.649
×1/4	2.46	0.622	2.52	0.638	2.59	0.657	2.51	0.957	2.64	0.961	2.77	0.964	1.94	0.652

Note: For width-to-thickness criteria, refer to Table 1-7B.



LLBB

Table 1-15 (continued)
Double Angles
Properties

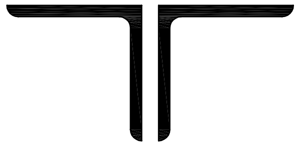


SLBB

Shape	Area, A	Radius of Gyration							
		LLBB				SLBB			
		r_y			r_x	r_y			r_x
		Separation, s, in.				Separation, s, in.			
		0	3/8	3/4		0	3/8	3/4	
	in.^{2}	in.	in.	in.	in.	in.	in.	in.	in.
2L4×4×3/4	10.9	1.73	1.88	2.03	1.18	1.73	1.88	2.03	1.18
×5/8	9.22	1.71	1.85	2.00	1.20	1.71	1.85	2.00	1.20
×1/2	7.50	1.69	1.83	1.97	1.21	1.69	1.83	1.97	1.21
×7/16	6.60	1.68	1.81	1.96	1.22	1.68	1.81	1.96	1.22
×3/8	5.72	1.67	1.80	1.94	1.23	1.67	1.80	1.94	1.23
×5/16	4.80	1.66	1.79	1.93	1.24	1.66	1.79	1.93	1.24
×1/4	3.86	1.65	1.78	1.91	1.25	1.65	1.78	1.91	1.25
2L4×3 1/2×1/2	7.00	1.44	1.57	1.72	1.23	1.75	1.89	2.03	1.04
×3/8	5.36	1.42	1.55	1.69	1.25	1.73	1.86	2.00	1.05
×5/16	4.50	1.40	1.53	1.68	1.25	1.72	1.85	1.99	1.06
×1/4	3.64	1.39	1.52	1.66	1.26	1.70	1.83	1.97	1.07
2L4×3×5/8	7.98	1.21	1.35	1.50	1.23	1.84	1.98	2.13	0.845
×1/2	6.50	1.19	1.32	1.47	1.24	1.81	1.95	2.10	0.858
×3/8	4.98	1.17	1.30	1.44	1.26	1.79	1.93	2.07	0.873
×5/16	4.18	1.16	1.29	1.43	1.27	1.78	1.91	2.06	0.880
×1/4	3.38	1.15	1.27	1.41	1.27	1.76	1.90	2.04	0.887
2L3 1/2×3 1/2×1/2	6.50	1.49	1.63	1.77	1.05	1.49	1.63	1.77	1.05
×7/16	5.78	1.48	1.61	1.76	1.06	1.48	1.61	1.76	1.06
×3/8	5.00	1.47	1.60	1.74	1.07	1.47	1.60	1.74	1.07
×5/16	4.20	1.46	1.59	1.73	1.08	1.46	1.59	1.73	1.08
×1/4	3.40	1.44	1.57	1.72	1.09	1.44	1.57	1.72	1.09
2L3 1/2×3×1/2	6.04	1.23	1.37	1.52	1.07	1.55	1.69	1.84	0.877
×7/16	5.34	1.22	1.36	1.51	1.08	1.54	1.67	1.82	0.885
×3/8	4.64	1.21	1.35	1.49	1.09	1.52	1.66	1.81	0.892
×5/16	3.90	1.20	1.33	1.48	1.09	1.51	1.65	1.79	0.900
×1/4	3.16	1.19	1.32	1.46	1.10	1.50	1.63	1.78	0.908
2L3 1/2×2 1/2×1/2	5.54	0.992	1.13	1.28	1.08	1.62	1.76	1.91	0.701
×3/8	4.24	0.970	1.11	1.25	1.10	1.59	1.73	1.88	0.716
×5/16	3.58	0.960	1.09	1.24	1.11	1.58	1.72	1.87	0.723
×1/4	2.90	0.950	1.08	1.22	1.12	1.57	1.70	1.85	0.731

Note: For width-to-thickness criteria, refer to Table 1-7B.

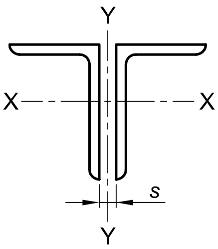
Table 1-15 (continued)
Double Angles
Properties



2L4–2L3½

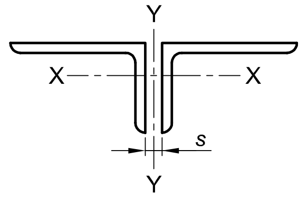
Shape	Flexural-Torsional Properties												Single Angle Properties	
	LLBB						SLBB							
	Separation, *s*, in.						Separation, *s*, in.						Area, *A*	r_z
	0		3/8		3/4		0		3/8		3/4			
	$\bar{r}_o$	*H*	$\bar{r}_o$	*H*	$\bar{r}_o$	*H*	$\bar{r}_o$	*H*	$\bar{r}_o$	*H*	$\bar{r}_o$	*H*		
	in.		in.		in.		in.		in.		in.		in.^{2}	in.
2L4×4×3/4	2.28	0.847	2.39	0.861	2.51	0.874	2.28	0.847	2.39	0.861	2.51	0.874	5.44	0.774
×5/8	2.28	0.841	2.39	0.854	2.50	0.868	2.28	0.841	2.39	0.854	2.50	0.868	4.61	0.774
×1/2	2.28	0.834	2.38	0.848	2.49	0.862	2.28	0.834	2.38	0.848	2.49	0.862	3.75	0.776
×7/16	2.28	0.832	2.38	0.846	2.49	0.859	2.28	0.832	2.38	0.846	2.49	0.859	3.30	0.777
×3/8	2.28	0.829	2.38	0.843	2.49	0.856	2.28	0.829	2.38	0.843	2.49	0.856	2.86	0.779
×5/16	2.28	0.826	2.37	0.840	2.48	0.854	2.28	0.826	2.37	0.840	2.48	0.854	2.40	0.781
×1/4	2.28	0.824	2.37	0.838	2.48	0.851	2.28	0.824	2.37	0.838	2.48	0.851	1.93	0.783
2L4×3½×1/2	2.14	0.784	2.23	0.802	2.33	0.819	2.16	0.882	2.28	0.893	2.40	0.904	3.50	0.716
×3/8	2.14	0.778	2.23	0.795	2.33	0.813	2.16	0.876	2.27	0.888	2.39	0.899	2.68	0.719
×5/16	2.14	0.775	2.23	0.792	2.33	0.810	2.16	0.874	2.26	0.885	2.38	0.896	2.25	0.721
×1/4	2.14	0.773	2.22	0.790	2.32	0.807	2.15	0.871	2.26	0.883	2.37	0.894	1.82	0.723
2L4×3×5/8	2.02	0.728	2.11	0.750	2.21	0.773	2.10	0.930	2.22	0.938	2.36	0.945	3.99	0.631
×1/2	2.02	0.721	2.11	0.743	2.20	0.765	2.09	0.925	2.21	0.933	2.34	0.940	3.25	0.633
×3/8	2.03	0.715	2.11	0.736	2.20	0.757	2.08	0.920	2.20	0.928	2.32	0.936	2.49	0.636
×5/16	2.03	0.712	2.11	0.733	2.20	0.754	2.07	0.918	2.19	0.926	2.32	0.934	2.09	0.638
×1/4	2.03	0.710	2.11	0.730	2.20	0.751	2.06	0.915	2.18	0.924	2.31	0.932	1.69	0.639
2L3½×3½×1/2	1.99	0.838	2.10	0.854	2.21	0.869	1.99	0.838	2.10	0.854	2.21	0.869	3.25	0.679
×7/16	1.99	0.835	2.09	0.851	2.21	0.866	1.99	0.835	2.09	0.851	2.21	0.866	2.89	0.681
×3/8	1.99	0.832	2.09	0.848	2.20	0.863	1.99	0.832	2.09	0.848	2.20	0.863	2.50	0.683
×5/16	1.99	0.829	2.09	0.845	2.20	0.860	1.99	0.829	2.09	0.845	2.20	0.860	2.10	0.685
×1/4	1.99	0.826	2.08	0.842	2.19	0.857	1.99	0.826	2.08	0.842	2.19	0.857	1.70	0.688
2L3½×3×1/2	1.85	0.780	1.94	0.801	2.05	0.822	1.88	0.892	2.00	0.904	2.13	0.915	3.02	0.618
×7/16	1.85	0.776	1.94	0.797	2.05	0.818	1.88	0.889	1.99	0.901	2.12	0.912	2.67	0.620
×3/8	1.85	0.773	1.94	0.794	2.05	0.814	1.88	0.885	1.99	0.898	2.11	0.910	2.32	0.622
×5/16	1.85	0.770	1.94	0.790	2.04	0.811	1.87	0.883	1.98	0.895	2.11	0.907	1.95	0.624
×1/4	1.85	0.767	1.94	0.787	2.04	0.807	1.87	0.880	1.98	0.893	2.10	0.905	1.58	0.628
2L3½×2½×1/2	1.75	0.706	1.83	0.732	1.93	0.759	1.82	0.938	1.95	0.946	2.08	0.953	2.77	0.532
×3/8	1.75	0.698	1.83	0.724	1.93	0.750	1.81	0.933	1.93	0.941	2.07	0.949	2.12	0.535
×5/16	1.76	0.695	1.83	0.720	1.92	0.746	1.80	0.930	1.92	0.939	2.06	0.947	1.79	0.538
×1/4	1.76	0.693	1.83	0.717	1.92	0.742	1.80	0.928	1.92	0.937	2.05	0.944	1.45	0.541

Note: For width-to-thickness criteria, refer to Table 1-7B.



LLBB

Table 1-15 (continued)
Double Angles
Properties

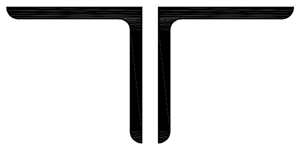


SLBB

Shape	Area, A	Radius of Gyration							
		LLBB				SLBB			
		r_y			r_x	r_y			r_x
		Separation, s, in.				Separation, s, in.			
		0	3/8	3/4		0	3/8	3/4	
	in.^{2}	in.	in.	in.	in.	in.	in.	in.	in.
2L3×3×1/2	5.52	1.29	1.43	1.58	0.895	1.29	1.43	1.58	0.895
×7/16	4.86	1.28	1.42	1.57	0.903	1.28	1.42	1.57	0.903
×3/8	4.22	1.27	1.41	1.55	0.910	1.27	1.41	1.55	0.910
×5/16	3.56	1.26	1.39	1.54	0.918	1.26	1.39	1.54	0.918
×1/4	2.88	1.25	1.38	1.52	0.926	1.25	1.38	1.52	0.926
×3/16	2.18	1.24	1.37	1.51	0.933	1.24	1.37	1.51	0.933
2L3×2 1/2×1/2	5.00	1.04	1.18	1.33	0.910	1.35	1.49	1.64	0.718
×7/16	4.44	1.02	1.16	1.32	0.917	1.34	1.48	1.63	0.724
×3/8	3.86	1.01	1.15	1.30	0.924	1.32	1.46	1.61	0.731
×5/16	3.26	1.00	1.14	1.29	0.932	1.31	1.45	1.60	0.739
×1/4	2.64	0.991	1.12	1.27	0.940	1.30	1.44	1.58	0.746
×3/16	2.00	0.980	1.11	1.25	0.947	1.29	1.42	1.57	0.753
2L3×2×1/2	4.52	0.795	0.940	1.10	0.922	1.42	1.56	1.72	0.543
×3/8	3.50	0.771	0.911	1.07	0.937	1.39	1.54	1.69	0.555
×5/16	2.96	0.760	0.897	1.05	0.945	1.38	1.52	1.67	0.562
×1/4	2.40	0.749	0.883	1.03	0.953	1.37	1.51	1.66	0.569
×3/16	1.83	0.739	0.869	1.02	0.961	1.35	1.49	1.64	0.577
2L2 1/2×2 1/2×1/2	4.52	1.09	1.23	1.39	0.735	1.09	1.23	1.39	0.735
×3/8	3.46	1.07	1.21	1.36	0.749	1.07	1.21	1.36	0.749
×5/16	2.92	1.05	1.19	1.34	0.756	1.05	1.19	1.34	0.756
×1/4	2.38	1.04	1.18	1.33	0.764	1.04	1.18	1.33	0.764
×3/16	1.80	1.03	1.17	1.31	0.771	1.03	1.17	1.31	0.771
2L2 1/2×2×3/8	3.10	0.815	0.957	1.11	0.766	1.13	1.27	1.42	0.574
×5/16	2.64	0.804	0.943	1.10	0.774	1.12	1.26	1.41	0.581
×1/4	2.14	0.794	0.930	1.08	0.782	1.10	1.24	1.39	0.589
×3/16	1.64	0.784	0.916	1.07	0.790	1.09	1.23	1.38	0.597
2L2 1/2×1 1/2×1/4	1.89	0.551	0.691	0.850	0.790	1.17	1.32	1.47	0.409
×3/16	1.45	0.541	0.677	0.833	0.800	1.16	1.30	1.46	0.416
2L2×2×3/8	2.74	0.865	1.01	1.17	0.591	0.865	1.01	1.17	0.591
×5/16	2.32	0.853	0.996	1.15	0.598	0.853	0.996	1.15	0.598
×1/4	1.89	0.842	0.982	1.14	0.605	0.842	0.982	1.14	0.605
×3/16	1.44	0.831	0.967	1.12	0.612	0.831	0.967	1.12	0.612
×1/8	0.982	0.818	0.951	1.10	0.620	0.818	0.951	1.10	0.620

Note: For width-to-thickness criteria, refer to Table 1-7B.

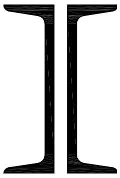
Table 1-15 (continued)
Double Angles
Properties



2L3–2L2

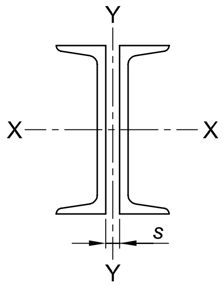
Shape	Flexural-Torsional Properties												Single Angle Properties	
	LLBB						SLBB							
	Separation, *s*, in.						Separation, *s*, in.						Area, *A*	r_z
	0		3/8		3/4		0		3/8		3/4			
	$\bar{r}_o$	*H*	$\bar{r}_o$	*H*	$\bar{r}_o$	*H*	$\bar{r}_o$	*H*	$\bar{r}_o$	*H*	$\bar{r}_o$	*H*		
	in.		in.		in.		in.		in.		in.		in.^{2}	in.
2L3×3×1/2	1.71	0.842	1.82	0.861	1.94	0.878	1.71	0.842	1.82	0.861	1.94	0.878	2.76	0.580
×7/16	1.71	0.838	1.82	0.857	1.94	0.874	1.71	0.838	1.82	0.857	1.94	0.874	2.43	0.580
×3/8	1.71	0.834	1.81	0.853	1.93	0.870	1.71	0.834	1.81	0.853	1.93	0.870	2.11	0.581
×5/16	1.71	0.830	1.81	0.849	1.93	0.866	1.71	0.830	1.81	0.849	1.93	0.866	1.78	0.583
×1/4	1.71	0.827	1.81	0.845	1.92	0.863	1.71	0.827	1.81	0.845	1.92	0.863	1.44	0.585
×3/16	1.71	0.823	1.80	0.842	1.91	0.859	1.71	0.823	1.80	0.842	1.91	0.859	1.09	0.586
2L3×2 1/2×1/2	1.57	0.774	1.66	0.800	1.78	0.824	1.61	0.905	1.73	0.918	1.86	0.929	2.50	0.516
×7/16	1.57	0.769	1.66	0.795	1.77	0.819	1.60	0.901	1.72	0.914	1.85	0.926	2.22	0.516
×3/8	1.57	0.764	1.66	0.790	1.77	0.815	1.60	0.897	1.72	0.911	1.85	0.923	1.93	0.517
×5/16	1.57	0.760	1.66	0.785	1.76	0.810	1.59	0.893	1.71	0.907	1.84	0.920	1.63	0.518
×1/4	1.57	0.756	1.66	0.781	1.76	0.806	1.59	0.890	1.70	0.904	1.83	0.917	1.32	0.520
×3/16	1.57	0.753	1.65	0.778	1.75	0.802	1.58	0.887	1.70	0.901	1.82	0.914	1.00	0.521
2L3×2×1/2	1.47	0.684	1.55	0.717	1.66	0.751	1.55	0.955	1.69	0.962	1.83	0.968	2.26	0.425
×3/8	1.48	0.675	1.55	0.707	1.65	0.739	1.54	0.949	1.67	0.957	1.81	0.963	1.75	0.426
×5/16	1.48	0.671	1.56	0.702	1.65	0.734	1.53	0.946	1.66	0.954	1.80	0.961	1.48	0.428
×1/4	1.48	0.668	1.56	0.698	1.65	0.730	1.52	0.944	1.65	0.952	1.79	0.959	1.20	0.431
×3/16	1.49	0.666	1.55	0.695	1.64	0.726	1.52	0.941	1.64	0.950	1.78	0.957	0.917	0.435
2L2 1/2×2 1/2×1/2	1.43	0.850	1.54	0.871	1.67	0.890	1.43	0.850	1.54	0.871	1.67	0.890	2.26	0.481
×3/8	1.42	0.839	1.53	0.861	1.65	0.881	1.42	0.839	1.53	0.861	1.65	0.881	1.73	0.481
×5/16	1.42	0.834	1.53	0.856	1.65	0.876	1.42	0.834	1.53	0.856	1.65	0.876	1.46	0.481
×1/4	1.42	0.829	1.52	0.852	1.64	0.872	1.42	0.829	1.52	0.852	1.64	0.872	1.19	0.482
×3/16	1.42	0.825	1.52	0.847	1.63	0.868	1.42	0.825	1.52	0.847	1.63	0.868	0.901	0.482
2L2 1/2×2×3/8	1.29	0.754	1.38	0.786	1.49	0.817	1.32	0.913	1.45	0.927	1.59	0.939	1.55	0.419
×5/16	1.29	0.748	1.38	0.781	1.49	0.812	1.32	0.909	1.44	0.923	1.58	0.936	1.32	0.420
×1/4	1.29	0.744	1.38	0.775	1.49	0.806	1.32	0.904	1.43	0.920	1.57	0.933	1.07	0.423
×3/16	1.29	0.740	1.38	0.771	1.48	0.801	1.31	0.901	1.43	0.916	1.56	0.929	0.818	0.426
2L2 1/2×1 1/2×1/4	1.21	0.629	1.28	0.668	1.38	0.711	1.26	0.962	1.40	0.969	1.55	0.975	0.947	0.321
×3/16	1.22	0.625	1.29	0.662	1.38	0.704	1.26	0.959	1.39	0.967	1.53	0.973	0.724	0.324
2L2×2×3/8	1.14	0.847	1.25	0.874	1.38	0.897	1.14	0.847	1.25	0.874	1.38	0.897	1.37	0.386
×5/16	1.14	0.841	1.25	0.868	1.37	0.891	1.14	0.841	1.25	0.868	1.37	0.891	1.16	0.386
×1/4	1.13	0.835	1.24	0.862	1.37	0.886	1.13	0.835	1.24	0.862	1.37	0.886	0.944	0.387
×3/16	1.13	0.830	1.24	0.857	1.36	0.882	1.13	0.830	1.24	0.857	1.36	0.882	0.722	0.389
×1/8	1.13	0.826	1.23	0.853	1.35	0.877	1.13	0.826	1.23	0.853	1.35	0.877	0.491	0.391

Note: For width-to-thickness criteria, refer to Table 1-7B.



2C-SHAPES

Table 1-16
2C-Shapes
Properties



Shape	Area, *A*	Axis Y-Y, Separation, *s*, in. = 0: *I*	0: *S*	0: *r*	0: *Z*	3/8: *I*	3/8: *S*	3/8: *r*	3/8: *Z*	3/4: *I*	3/4: *S*	3/4: *r*	3/4: *Z*	Axis X-X: r_x
	in.^{2}	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.
2C15×50	29.4	40.7	11.0	1.18	23.5	50.5	12.9	1.31	29.0	62.4	15.3	1.46	34.5	5.24
×40	23.6	32.6	9.25	1.18	18.4	40.2	10.9	1.31	22.8	49.6	12.7	1.45	27.2	5.43
×33.9	20.0	28.5	8.38	1.20	15.8	35.1	9.78	1.33	19.5	43.1	11.4	1.47	23.3	5.61
2C12×30	17.6	18.2	5.75	1.02	11.9	23.3	6.94	1.15	15.2	29.6	8.36	1.30	18.5	4.29
×25	14.7	15.6	5.11	1.03	9.89	19.8	6.12	1.16	12.6	25.0	7.32	1.31	15.4	4.43
×20.7	12.2	13.6	4.64	1.06	8.49	17.2	5.51	1.19	10.8	21.7	6.55	1.34	13.0	4.61
2C10×30	17.6	15.3	5.04	0.931	11.4	20.2	6.27	1.07	14.7	26.3	7.73	1.22	18.0	3.43
×25	14.7	12.3	4.25	0.914	9.06	16.2	5.27	1.05	11.8	21.1	6.48	1.20	14.6	3.52
×20	11.7	9.91	3.62	0.918	7.11	13.0	4.44	1.05	9.32	16.9	5.43	1.20	11.5	3.67
×15.3	8.96	8.14	3.13	0.953	5.68	10.6	3.80	1.09	7.36	13.7	4.59	1.23	9.04	3.88
2C9×20	11.7	8.80	3.32	0.866	6.84	11.8	4.15	1.00	9.05	15.6	5.15	1.15	11.2	3.22
×15	8.80	6.86	2.76	0.882	5.17	9.10	3.41	1.02	6.82	12.0	4.19	1.17	8.48	3.40
×13.4	7.88	6.34	2.61	0.897	4.74	8.39	3.20	1.03	6.21	11.0	3.92	1.18	7.69	3.48
2C8×18.75	11.0	7.46	2.95	0.823	6.23	10.2	3.75	0.962	8.29	13.7	4.71	1.11	10.4	2.82
×13.75	8.06	5.51	2.35	0.826	4.48	7.47	2.95	0.962	5.99	10.0	3.68	1.11	7.51	2.99
×11.5	6.74	4.82	2.13	0.846	3.86	6.50	2.66	0.982	5.12	8.66	3.29	1.13	6.38	3.11
2C7×14.75	8.66	5.18	2.25	0.773	4.61	7.21	2.90	0.912	6.23	9.85	3.68	1.07	7.85	2.51
×12.25	7.18	4.30	1.96	0.773	3.78	5.97	2.51	0.911	5.13	8.14	3.17	1.06	6.48	2.59
×9.8	5.74	3.59	1.72	0.791	3.11	4.95	2.17	0.929	4.18	6.72	2.73	1.08	5.26	2.72
2C6×13	7.64	4.11	1.91	0.734	3.92	5.85	2.50	0.876	5.35	8.13	3.21	1.03	6.77	2.13
×10.5	6.14	3.26	1.60	0.728	3.08	4.63	2.08	0.867	4.24	6.43	2.67	1.02	5.39	2.22
×8.2	4.78	2.63	1.37	0.741	2.45	3.72	1.76	0.881	3.34	5.14	2.24	1.04	4.24	2.34
2C5×9	5.28	2.45	1.30	0.682	2.52	3.59	1.73	0.824	3.51	5.09	2.25	0.982	4.50	1.84
×6.7	3.94	1.86	1.06	0.688	1.91	2.71	1.40	0.831	2.65	3.84	1.81	0.989	3.83	1.95
2C4×7.25	4.26	1.75	1.02	0.641	1.96	2.63	1.38	0.786	2.75	3.81	1.82	0.946	3.55	1.47
×6.25	3.54	1.36	0.824	0.620	1.54	2.06	1.12	0.763	2.20	3.01	1.49	0.922	2.87	1.50
×5.4	3.16	1.29	0.812	0.637	1.44	1.94	1.10	0.783	2.04	2.82	1.44	0.943	2.63	1.56
×4.5	2.76	1.25	0.789	0.673	1.36	1.86	1.05	0.820	1.88	2.66	1.36	0.981	2.40	1.63
2C3×6	3.52	1.33	0.833	0.614	1.60	2.06	1.15	0.764	2.26	3.03	1.54	0.927	2.92	1.09
×5	2.94	1.05	0.699	0.597	1.29	1.63	0.969	0.746	1.84	2.43	1.30	0.909	2.39	1.12
×4.1	2.40	0.842	0.597	0.591	1.05	1.32	0.827	0.741	1.50	1.97	1.10	0.905	1.95	1.18
×3.5	2.18	0.766	0.558	0.593	0.966	1.20	0.772	0.743	1.37	1.80	1.03	0.908	1.78	1.20

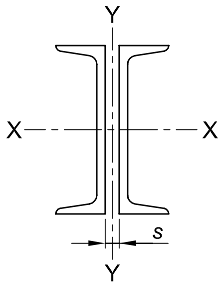
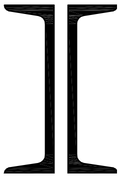


Table 1-17
2MC-Shapes
Properties

2MC18–2MC7

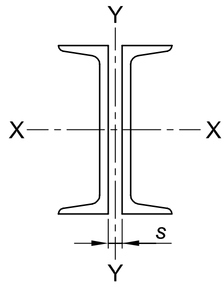
Shape	Area, A	Axis Y-Y												Axis X-X
		Separation, s, in.												
		0				3/8				3/4				r_x
		I	S	r	Z	I	S	r	Z	I	S	r	Z	
	in.^{2}	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.
2MC18×58	34.2	60.6	14.4	1.33	29.5	72.8	16.6	1.46	35.9	87.5	19.1	1.60	42.3	6.29
×51.9	30.6	55.0	13.4	1.34	26.3	65.9	15.4	1.47	32.0	79.0	17.6	1.61	37.7	6.41
×45.8	27.0	50.1	12.5	1.36	23.4	59.8	14.3	1.49	28.4	71.4	16.3	1.63	33.5	6.55
×42.7	25.2	47.8	12.1	1.38	22.1	57.0	13.8	1.51	26.8	67.9	15.7	1.64	31.6	6.64
2MC13×50	29.4	60.7	13.8	1.44	28.6	72.5	15.8	1.57	34.1	86.3	18.0	1.71	39.7	4.62
×40	23.4	49.1	11.7	1.45	22.7	58.4	13.4	1.58	27.2	69.4	15.2	1.72	31.6	4.82
×35	20.6	44.3	10.9	1.47	20.2	52.6	12.3	1.60	24.1	62.3	14.0	1.74	27.9	4.95
×31.8	18.7	41.5	10.4	1.49	18.7	49.2	11.7	1.62	22.2	58.2	13.3	1.76	25.7	5.05
2MC12×50	29.4	67.2	16.2	1.51	30.9	79.8	18.5	1.65	36.4	94.5	20.9	1.79	41.9	4.28
×45	26.4	59.9	14.9	1.51	27.5	71.1	16.9	1.64	32.4	84.1	19.2	1.79	37.4	4.36
×40	23.6	53.7	13.8	1.51	24.5	63.7	15.6	1.65	29.0	75.3	17.7	1.79	33.4	4.46
×35	20.6	48.0	12.7	1.53	21.6	56.8	14.4	1.66	25.5	67.1	16.2	1.81	29.4	4.59
×31	18.2	44.0	12.0	1.55	19.7	52.1	13.5	1.69	23.1	61.4	15.2	1.83	26.5	4.71
2MC12×14.3	8.36	3.19	1.50	0.618	3.15	4.66	2.02	0.747	4.72	6.73	2.70	0.897	6.29	4.27
2MC12×10.6[c]	6.20	1.21	0.804	0.441	1.67	2.05	1.21	0.575	2.83	3.33	1.78	0.733	3.99	4.22
2MC10×41.1	24.2	60.0	13.9	1.58	26.4	70.7	15.7	1.71	30.9	83.1	17.7	1.85	35.5	3.61
×33.6	19.7	49.5	12.1	1.58	21.5	58.2	13.6	1.72	25.2	68.3	15.3	1.86	28.9	3.75
×28.5	16.7	43.5	11.0	1.61	18.7	51.1	12.3	1.75	21.9	59.8	13.8	1.89	25.0	3.89
2MC10×25	14.7	27.8	8.18	1.38	14.0	33.6	9.36	1.51	16.8	40.4	10.7	1.66	19.5	3.87
×22	12.9	25.4	7.67	1.40	12.8	30.7	8.76	1.54	15.2	36.8	10.0	1.69	17.6	3.99
2MC10×8.4[c]	4.92	1.05	0.700	0.462	1.40	1.75	1.03	0.596	2.32	2.79	1.49	0.753	3.24	3.61
×6.5[c]	3.90	0.414	0.354	0.326	0.757	0.835	0.615	0.463	1.49	1.53	0.990	0.626	2.22	3.43
2MC9×25.4	14.9	29.2	8.34	1.40	14.5	35.2	9.53	1.53	17.3	42.2	10.9	1.68	20.1	3.43
×23.9	14.0	27.8	8.05	1.41	13.8	33.4	9.19	1.54	16.4	40.1	10.5	1.69	19.0	3.48
2MC8×22.8	13.4	27.7	7.91	1.44	13.5	33.2	9.01	1.58	16.0	39.7	10.2	1.72	18.6	3.09
×21.4	12.6	26.3	7.63	1.45	12.8	31.6	8.68	1.59	15.2	37.7	9.86	1.73	17.5	3.13
2MC8×20	11.7	17.1	5.66	1.21	9.88	21.2	6.61	1.34	12.1	26.2	7.70	1.49	14.3	3.04
×18.7	11.0	16.2	5.45	1.21	9.34	20.1	6.35	1.35	11.4	24.8	7.39	1.50	13.5	3.09
2MC8×8.5	5.00	2.16	1.15	0.658	2.14	3.14	1.52	0.793	3.08	4.47	1.99	0.946	4.02	3.05
2MC7×22.7	13.3	29.0	8.06	1.47	13.9	34.7	9.16	1.61	16.4	41.3	10.4	1.76	18.9	2.67
×19.1	11.2	25.1	7.27	1.50	12.1	30.0	8.25	1.64	14.2	35.7	9.34	1.78	16.3	2.77

[c] Shape is slender for compression with $F_y = 36$ ksi.



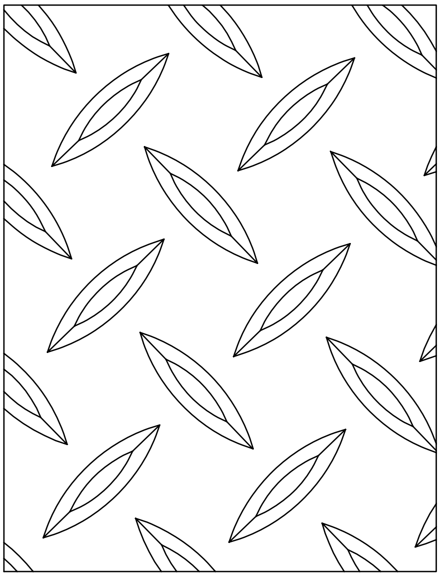
2MC6–2MC3

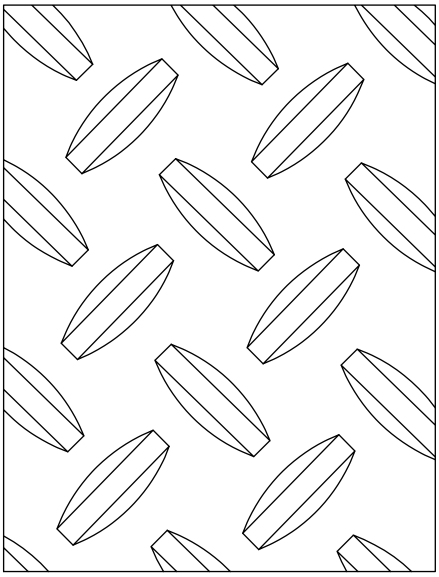
Table 1-17 (continued)
2MC-Shapes
Properties



Shape	Area, A	Axis Y-Y												Axis X-X
		Separation, s, in.												
		0				3/8				3/4				r_x
		I	S	r	Z	I	S	r	Z	I	S	r	Z	
	in.^{2}	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.
2MC6×18	10.6	25.0	7.13	1.54	11.8	29.8	8.07	1.68	13.8	35.3	9.11	1.83	15.8	2.37
×15.3	8.98	19.7	5.63	1.48	9.43	23.6	6.39	1.62	11.1	28.1	7.24	1.77	12.8	2.38
2MC6×16.3	9.58	15.8	5.26	1.28	8.88	19.4	6.10	1.42	10.7	23.8	7.05	1.58	12.5	2.33
×15.1	8.88	14.8	5.02	1.29	8.35	18.2	5.82	1.43	10.0	22.3	6.71	1.58	11.7	2.37
2MC6×12	7.06	7.21	2.89	1.01	4.97	9.32	3.47	1.15	6.29	11.9	4.15	1.30	7.62	2.30
2MC6×7	4.18	2.25	1.20	0.734	2.09	3.19	1.55	0.873	2.88	4.41	1.96	1.03	3.66	2.34
×6.5	3.90	2.15	1.16	0.744	2.00	3.04	1.49	0.883	2.73	4.20	1.89	1.04	3.46	2.38
2MC4×13.8	8.06	10.1	4.03	1.12	6.84	12.9	4.81	1.27	8.35	16.3	5.68	1.42	9.87	1.48
2MC3×7.1	4.22	3.13	1.62	0.862	2.76	4.31	2.03	1.01	3.55	5.79	2.50	1.17	4.34	1.14

Table 1-18
Weights of Raised-Pattern Floor Plates





Gauge No.	Wt., lb/ft^2	Nominal Thickness, in.	Wt., lb/ft^2	Nominal Thickness, in.	Wt., lb/ft^2
18	2.40	$^1/_8$	6.16	$^1/_2$	21.5
16	3.00	$^3/_{16}$	8.71	$^9/_{16}$	24.0
14	3.75	$^1/_4$	11.3	$^5/_8$	26.6
13	4.50	$^5/_{16}$	13.8	$^3/_4$	31.7
12	5.25	$^3/_8$	16.4	$^7/_8$	36.8
		$^7/_{16}$	18.9	1	41.9

Note: Thickness is measured near the edge of the plate, exclusive of raised pattern.

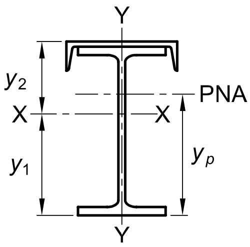
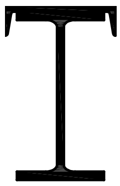


Table 1-19
W-Shapes with Cap Channels
Properties

W-Shape	Channel	Total Wt.	Total Area	Axis X-X			
				I	$S_1 = \frac{I}{y_1}$	$S_2 = \frac{I}{y_2}$	r
		lb/ft	in.^{2}	in.^{4}	in.^{3}	in.^{3}	in.
W36×150	MC18×42.7	193	56.8	12000	553	831	14.6
	C15×33.9	184	54.2	11500	546	764	14.6
W33×141	MC18×42.7	184	54.1	10000	490	750	13.6
	C15×33.9	175	51.5	9580	484	689	13.6
W33×118	MC18×42.7	161	47.2	8280	400	656	13.2
	C15×33.9	152	44.6	7900	395	596	13.3
W30×116	MC18×42.7	159	46.8	6900	365	598	12.1
	C15×33.9	150	44.1	6590	360	544	12.2
W30×99	MC18×42.7	142	41.6	5830	304	533	11.8
	C15×33.9	133	39.0	5550	300	481	11.9
W27×94	C15×33.9	128	37.6	4530	268	435	11.0
W27×84	C15×33.9	118	34.7	4050	237	403	10.8
W24×84	C15×33.9	118	34.7	3340	217	367	9.82
	C12×20.7	105	30.8	3030	211	302	9.92
W24×68	C15×33.9	102	30.0	2710	173	321	9.51
	C12×20.7	88.7	26.1	2440	168	258	9.67
W21×68	C15×33.9	102	30.0	2180	156	287	8.52
	C12×20.7	88.7	26.1	1970	152	232	8.67
W21×62	C15×33.9	95.9	28.2	2000	142	272	8.41
	C12×20.7	82.7	24.3	1800	138	218	8.59
W18×50	C15×33.9	83.9	24.6	1250	100	211	7.12
	C12×20.7	70.7	20.7	1120	97.3	166	7.35
W16×36	C15×33.9	69.9	20.5	748	64.5	160	6.04
	C12×20.7	56.7	16.6	670	62.8	123	6.34
W14×30	C12×20.7	50.7	14.9	447	46.7	98.1	5.47
	C10×15.3	45.3	13.3	420	46.0	84.5	5.61
W12×26	C12×20.7	46.7	13.7	318	36.8	82.1	4.81
	C10×15.3	41.3	12.1	299	36.3	70.5	4.96

Note: Width-to-thickness criteria not addressed in this table.

Table 1-19 (continued)
W-Shapes with Cap Channels
Properties



W-Shape	Channel	Axis X-X				Axis Y-Y			
		y_1	y_2	Z	y_p	I	S	r	Z
		in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}
W36×150	MC18×42.7	21.8	14.5	738	28.0	824	91.5	3.81	146
	C15×33.9	21.1	15.1	716	25.9	584	77.9	3.28	122
W33×141	MC18×42.7	20.4	13.3	652	27.0	800	88.9	3.85	142
	C15×33.9	19.8	13.9	635	24.9	561	74.8	3.30	118
W33×118	MC18×42.7	20.7	12.6	544	27.8	741	82.3	3.96	126
	C15×33.9	20.0	13.3	529	25.5	502	66.9	3.35	102
W30×116	MC18×42.7	18.9	11.5	492	26.1	718	79.8	3.92	124
	C15×33.9	18.3	12.1	480	23.8	479	63.8	3.29	100
W30×99	MC18×42.7	19.2	10.9	412	26.4	682	75.8	4.05	114
	C15×33.9	18.5	11.5	408	24.4	442	59.0	3.37	89.4
W27×94	C15×33.9	16.9	10.4	357	23.6	439	58.5	3.41	89.6
W27×84	C15×33.9	17.1	10.0	316	23.9	420	56.0	3.48	83.9
W24×84	C15×33.9	15.4	9.10	286	21.6	409	54.5	3.43	83.4
	C12×20.7	14.3	10.0	275	18.5	223	37.2	2.69	58.2
W24×68	C15×33.9	15.7	8.46	232	21.7	385	51.3	3.58	75.3
	C12×20.7	14.5	9.49	224	19.2	199	33.2	2.76	50.1
W21×68	C15×33.9	13.9	7.59	207	19.3	379	50.6	3.56	75.1
	C12×20.7	12.9	8.49	200	17.6	194	32.3	2.72	50.0
W21×62	C15×33.9	14.1	7.33	189	19.4	372	49.6	3.63	72.5
	C12×20.7	13.0	8.26	183	18.1	186	31.1	2.77	47.3
W18×50	C15×33.9	12.5	5.92	133	16.9	354	47.3	3.79	67.3
	C12×20.7	11.5	6.76	127	16.1	169	28.2	2.85	42.2
W16×36	C15×33.9	11.6	4.67	86.8	15.2	339	45.2	4.06	61.6
	C12×20.7	10.7	5.47	83.2	14.6	153	25.6	3.04	36.4
W14×30	C12×20.7	9.57	4.55	62.0	12.9	149	24.8	3.16	34.6
	C10×15.3	9.11	4.97	60.3	12.6	86.8	17.4	2.55	24.9
W12×26	C12×20.7	8.63	3.87	48.2	11.6	146	24.4	3.27	33.7
	C10×15.3	8.22	4.24	47.0	11.3	84.5	16.9	2.64	24.1

Note: Width-to-thickness criteria not addressed in this table.

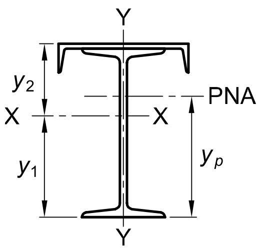
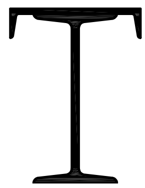


Table 1-20
S-Shapes with Cap Channels
Properties

S-Shape	Channel	Total Wt.	Total Area	Axis X-X			
				I	$S_1 = \frac{I}{y_1}$	$S_2 = \frac{I}{y_2}$	r
		lb/ft	in.^{2}	in.^{4}	in.^{3}	in.^{3}	in.
S24×80	C12×20.7	101	29.5	2750	191	278	9.66
	C10×15.3	95.3	27.9	2610	188	252	9.67
S20×66	C12×20.7	86.7	25.5	1620	132	202	7.97
	C10×15.3	81.3	23.9	1530	129	181	8.00
S15×42.9	C10×15.3	58.2	17.1	615	65.7	105	6.00
	C8×11.5	54.4	16.0	583	64.7	93.9	6.04
S12×31.8	C10×15.3	47.1	13.8	314	40.2	71.2	4.77
	C8×11.5	43.3	12.7	297	39.6	63.0	4.84
S10×25.4	C10×15.3	40.7	11.9	185	27.5	52.7	3.94
	C8×11.5	36.9	10.8	175	27.1	46.3	4.02

Note: Width-to-thickness criteria not addressed in this table.

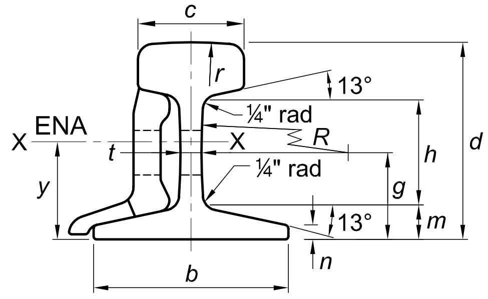
Table 1-20 (continued)
S-Shapes with Cap Channels
Properties



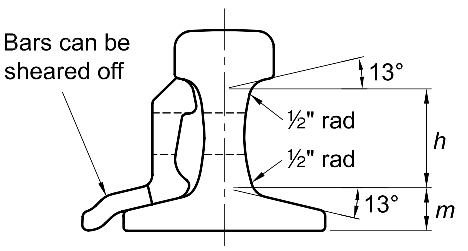
S-Shape	Channel	Axis X-X y_1	Axis X-X y_2	Axis X-X Z	Axis X-X y_p	Axis Y-Y I	Axis Y-Y S	Axis Y-Y r	Axis Y-Y Z
		in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}
S24×80	C12×20.7	14.4	9.90	256	18.1	171	28.5	2.41	46.4
	C10×15.3	13.9	10.4	246	16.5	109	21.8	1.98	36.8
S20×66	C12×20.7	12.3	7.99	180	16.0	156	26.1	2.48	41.0
	C10×15.3	11.8	8.44	173	14.4	94.7	18.9	1.99	31.3
S15×42.9	C10×15.3	9.37	5.87	87.6	12.8	81.5	16.3	2.18	25.0
	C8×11.5	9.01	6.21	86.5	11.6	46.8	11.7	1.71	18.7
S12×31.8	C10×15.3	7.82	4.42	54.0	10.6	76.5	15.3	2.36	22.3
	C8×11.5	7.50	4.72	52.4	10.3	41.8	10.5	1.82	16.1
S10×25.4	C10×15.3	6.73	3.51	37.2	9.03	73.9	14.8	2.49	20.9
	C8×11.5	6.45	3.77	36.1	8.82	39.2	9.81	1.90	14.6

Note: Width-to-thickness criteria not addressed in this table.

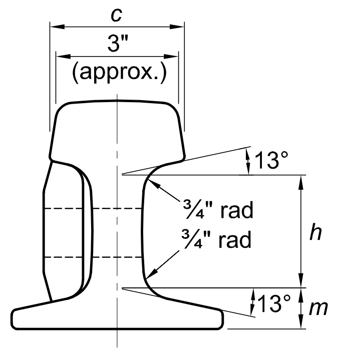
Table 1-21
Crane Rails
Dimensions and Properties



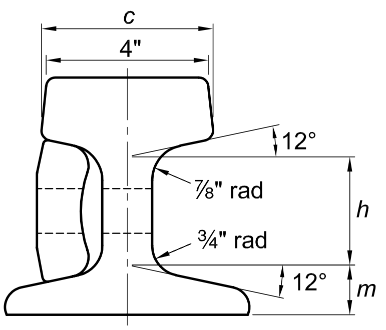
ASCE crane rails



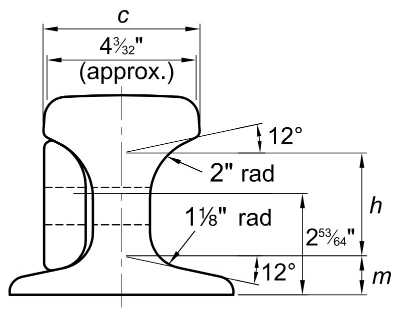
ASTM profile 104



ASTM profile 135



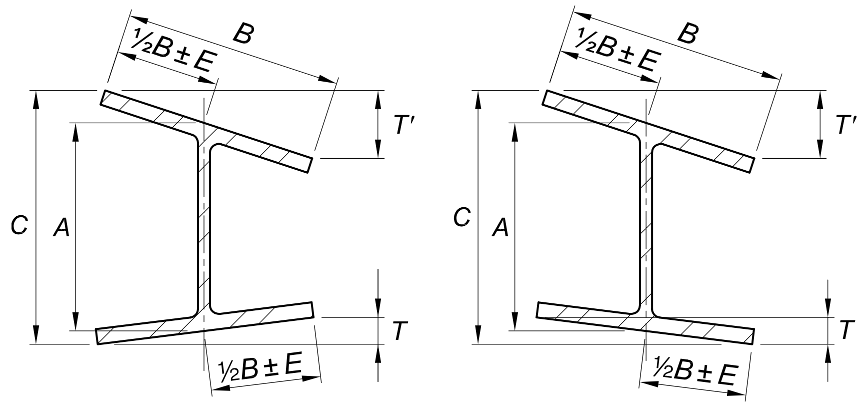
ASTM profile 171



ASTM profile 175

TYPE	Classification	Wt.	Depth, d	Gage, g	Base b	Base m	Base n	Head c	Head r	Web t	Web h	Web R	Axis X-X Area	Axis X-X I	Axis X-X S Head	Axis X-X S Base	Axis X-X y
		lb/yd	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.²	in.⁴	in.³	in.³	in.
ASCE	Light	30	3 1/8	1 25/64	3 1/8	17/32	11/64	1 11/16	12	21/64	1 23/32	12	3.00	4.10	2.55	–	–
		40	3 1/2	1 71/128	3 1/2	5/8	7/32	1 7/8	12	25/64	1 55/64	12	3.94	6.54	3.59	3.89	1.68
		50	3 7/8	1 23/32	3 7/8	11/16	1/4	2 1/8	12	7/16	2 1/16	12	4.90	10.1	5.10	–	1.88
		60	4 1/4	1 115/128	4 1/4	49/64	9/32	2 3/8	12	31/64	2 17/64	12	5.93	14.6	6.64	7.12	2.05
	–	70	4 5/8	2 3/64	4 5/8	13/16	9/32	2 7/16	12	33/64	2 15/32	12	6.81	19.7	8.19	8.87	2.22
		80	5	2 3/16	5	7/8	19/64	2 1/2	12	35/64	2 5/8	12	7.86	26.4	10.1	11.1	2.38
	Std.	85	5 3/16	2 17/64	5 3/16	57/64	19/64	2 9/16	12	9/16	2 3/4	12	8.33	30.1	11.1	12.2	2.47
		100	5 3/4	2 65/128	5 3/4	31/32	5/16	2 3/4	12	9/16	2 5/64	12	9.84	44.0	14.6	16.1	2.73
ASTM A759	Crane	104	5	2 7/16	5	1 1/16	1/2	2 1/2	12	1	2 7/16	3 1/2	10.3	29.8	10.7	13.5	2.21
		135	5 3/4	2 15/32	5 3/16	1 1/16	15/32	3 7/16	14	1 1/4	2 13/16	12	13.3	50.8	17.3	18.1	2.81
		171	6	2 5/8	6	1 1/4	5/8	4.3	Flat	1 1/4	2 3/4	Vert.	16.8	73.4	24.5	24.4	3.01
		175	6	2 21/32	6	1 9/64	1/2	4 1/4	18	1 1/2	3 7/64	Vert.	17.1	70.5	23.4	23.6	2.98

Table 1-22
ASTM A6 Tolerances for W-Shapes and HP-Shapes



Permissible Cross-Sectional Variations

Nominal Depth, in.	*A*, Depth at Web Centerline, in.		*B*, Flange Width, in.		*T* + *T′*, Flanges Out of Square, Max. in.	*E*[a], Web Off Center, in.	*C*, Max. Depth at any Cross Section over Theoretical Depth, in.
	Over	Under	Over	Under			
To 12, incl.	1/8	1/8	1/4	3/16	1/4	3/16	1/4
Over 12	1/8	1/8	1/4	3/16	5/16	3/16	1/4

Permissible Variations in Length

Nominal Depth[b]	Variations from Specified Length for Lengths Given, in.			
	30 ft and Under		Over 30 ft	
	Over	Under	Over	Under
Beams 24 in. and under	3/8	3/8	3/8 plus 1/16 for each additional 5 ft or fraction thereof	3/8
Beams over 24 in., All columns	1/2	1/2	1/2 plus 1/16 for each additional 5 ft or fraction thereof	1/2

Mill Straightness Tolerances[c]

Sizes	Length	Permissible Variation in Straightness, in.	
		Camber	Sweep
Flange width equal to or greater than 6 in.	All	$\frac{1}{8}\text{ in.} \times \frac{(\text{total length, ft})}{10}$	
Flange width less than 6 in.	All	$\frac{1}{8}\text{ in.} \times \frac{(\text{total length, ft})}{10}$	$\frac{1}{8}\text{ in.} \times \frac{(\text{total length, ft})}{5}$
Certain sections with a flange width approx. equal to depth & specified on order as columns[d]	45 ft and under	$\frac{1}{8}\text{ in.} \times \frac{(\text{total length, ft})}{10}$ with 3/8 in. max.	
	Over 45 ft	$\frac{3}{8}\text{ in.} + \left[\frac{1}{8}\text{ in.} \times \frac{(\text{total length, ft} - 45)}{10}\right]$	

Other Permissible Rolling Variations

Area and Weight	–2.5 to +3.0% from the theoretical cross-sectional area or the specified nominal weight[e]
Ends Out of Square	1/64 in., per in. of depth, or of flange width if it is greater than the depth

[a] Variation of 5/16 in. max. for sections over 426 lb/ft.
[b] For shapes specified in the order for use as bearing piles, the permitted variations are plus 5 in. and minus 0 in.
[c] The tolerances herein are taken from ASTM A6 and apply to the straightness of members received from the rolling mill, measured as illustrated in Figure 1-1.
[d] Applies only to W8×31and heavier, W10×49 and heavier, W12×65 and heavier, W14×90 and heavier, HP8×36, HP10×57, HP12×74 and heavier, and HP14×102 and heavier. If other sections are specified on the order as columns, the tolerance will be subject to negotiation with the manufacturer.
[e] For shapes with a nominal weight ≥ 100 lb/ft, the permitted variation is ±2.5% from the theoretical or specified amount.

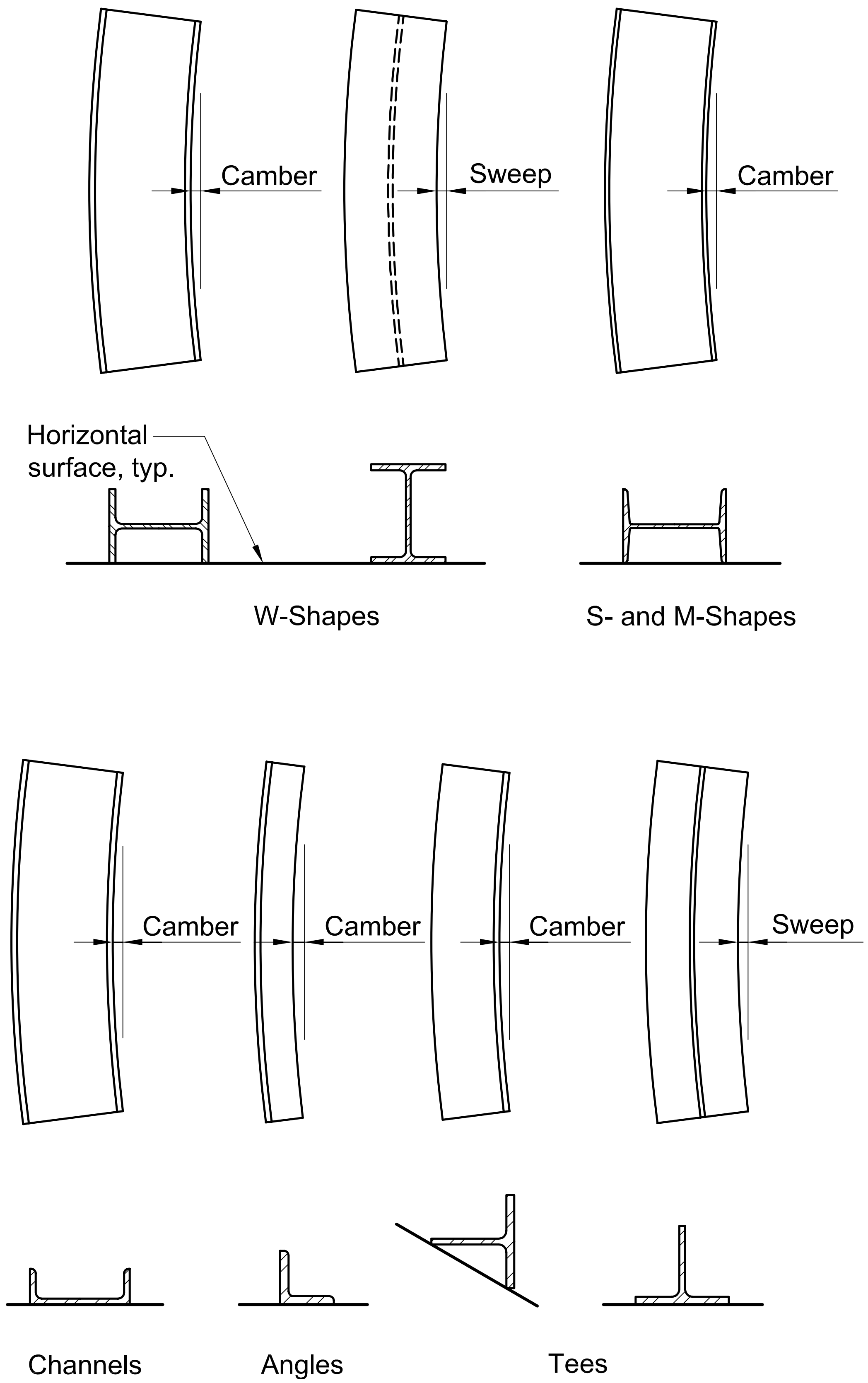
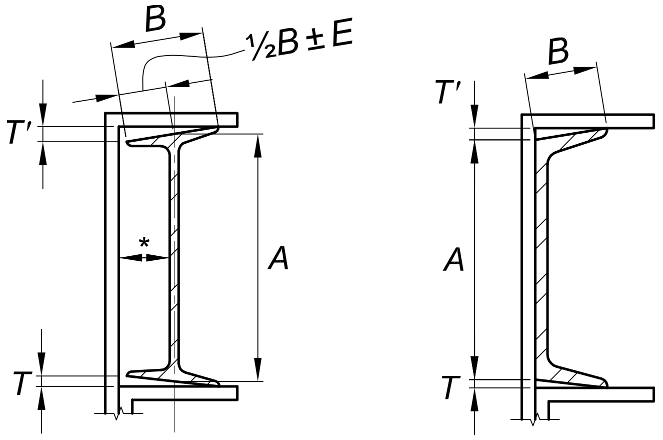


Fig. 1-1. Positions for measuring straightness.

Table 1-23
ASTM A6 Tolerances for S-Shapes, M-Shapes and Channels



*Back of square and centerline of web to be parallel when measuring "out-of-square".

Permissible Cross-Sectional Variations

Shape	Nominal Depth, in.	*A*[a], Depth, in.		*B*, Flange Width, in.		*T* + *T′*[b], Flanges Out of Square, per in. of *B*, in.	*E*, Web Off Center, in.
		Over	Under	Over	Under		
S shapes and M shapes	3 to 7, incl.	3/32	1/16	1/8	1/8	1/32	3/16
	Over 7 to 14, incl.	1/8	3/32	5/32	5/32		
	Over 14 to 24, incl.	3/16	1/8	3/16	3/16		
Channels	3 to 7, incl.	3/32	1/16	1/8	1/8	1/32	–
	Over 7 to 14, incl.	1/8	3/32	1/8	5/32		
	Over 14	3/16	1/8	1/8	3/16		

Permissible Variations in Length

Shape	Variations from Specified Length for Lengths Given[c], in.					
	5 to 10 ft, excl.	10 to 20 ft, excl.	20 to 30 ft, incl.	Over 30 to 40 ft, incl.	Over 40 to 65 ft, incl.	Over 65 ft
All	1	1 1/2	1 3/4	2 1/4	2 3/4	–

Mill Straightness Tolerances[d]

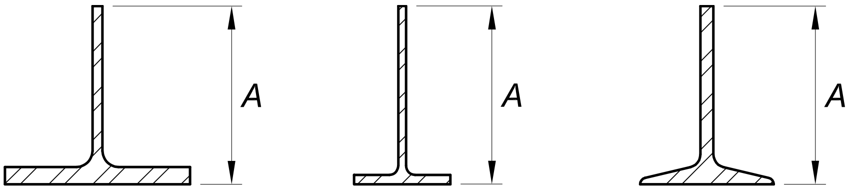
Camber	$\frac{1}{8}\text{ in.} \times \frac{(\text{total length, ft})}{5}$
Sweep	Due to the extreme variations in flexibility of these shapes, permitted variations for sweep are subject to negotiation between the manufacturer and purchaser for the individual sections involved.

Other Permissible Rolling Variations

Area and Weight	–2.5 to +3.0% from the theoretical cross-sectional area or the specified nominal weight[e]
Ends Out of Square	S-Shapes, M-Shapes and Channels: 1/64 in., per in. of depth

– Indicates that there is no requirement.
[a] *A* is measured at center line of web for S-shapes and M-shapes and at back of web for channels.
[b] *T* + *T′* applies when flanges of channels are toed in or out.
[c] The permitted variation under the specified length is 0 in. for all lengths. There are no requirements for lengths over 65 ft.
[d] The tolerances herein are taken from ASTM A6 and apply to the straightness of members received from the rolling mill, measured as illustrated in Figure 1-1.
[e] For shapes with a nominal weight ≥ 100 lb/ft, the permitted variation is ±2.5% from the theoretical or specified amount.

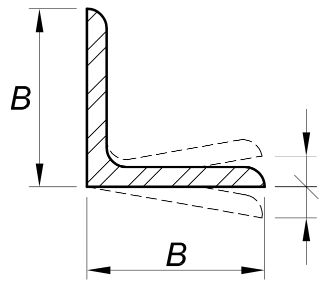
Table 1-24
ASTM A6 Tolerances for WT-, MT- and ST-Shapes



Permissible Variations in Depth	
Dimension A may be approximately one-half beam depth or any dimension resulting from off-center splitting or splitting on two lines, as specified in the order.	
Specified Depth, *A*, in.	**Variations in Depth *A*, Over and Under**
To 6, excl.	$\frac{1}{8}$
6 to 16, excl.	$\frac{3}{16}$
16 to 20, excl.	$\frac{1}{4}$
20 to 24, excl.	$\frac{5}{16}$
24 and over	$\frac{3}{8}$
The above variations in depths of tees include the permissible variations in depth for the beams before splitting	
Mill Straightness Tolerances[a]	
Camber and Sweep	$\frac{1}{8}$ in. $\times \frac{\text{(total length, ft)}}{5}$
Other Permissible Rolling Variations	
Other permissible variations in cross section as well as permissible variations in length, area, weight, ends out-of-square, and sweep for WTs will correspond to those of the beam before splitting.	

[a] The tolerances herein are taken from ASTM A6 and apply to the straightness of members received from the rolling mill, measured as illustrated in Figure 1-1. For tolerance on induced camber and sweep, see AISC *Code of Standard Practice* Section 6.4.4.

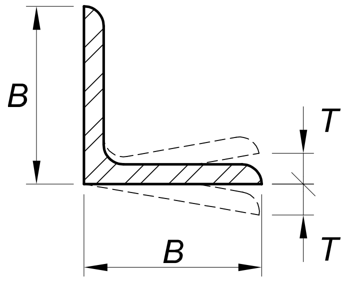
Table 1-25
ASTM A6 Tolerances for Angles, 3 in. and Larger



Permissible Cross-Sectional Variations				
Shape	Nominal Leg Size[a], in.	*B*, Leg Size, in. Over	*B*, Leg Size, in. Under	*T*, Out of Square per inch of *B*, in.
Angles	3 to 4, incl.	1/8	3/32	3/128[b]
	Over 4 to 6, incl.	1/8	1/8	
	Over 6 to 8, incl.	3/16	1/8	
	Over 8 to 10, incl.	1/4	1/4	
	Over 10	1/4	3/8	
Permissible Variations in Length				
Variations Over Specified Length for Lengths Given[c], in.				
5 to 10 ft, excl.	**10 to 20 ft, excl.**	**20 to 30 ft, incl.**	**Over 30 to 40 ft, incl.**	**Over 40 to 65 ft, incl.**
1	1 1/2	1 3/4	2 1/4	2 3/4
Mill Straightness Tolerances[d]				
Camber	$\frac{1}{8}\text{ in.} \times \frac{\text{(total length, ft)}}{5}$, applied to either leg			
Sweep	Due to the extreme variations in flexibility of these shapes, permitted variations for sweep are subject to negotiation between the manufacturer and purchaser for the individual sections involved.			
Other Permissible Rolling Variations				
Area and Weight	−2.5 to +3.0% from the theoretical cross-sectional area or the specified nominal weight			
Ends Out of Square	3/128 in. per in. of leg length, or 1 1/2°. Variations based on the longer leg of unequal angle.			

[a] For unequal leg angles, longer leg determines classification.
[b] 3/128 in. per in. = 1 1/2°
[c] The permitted variation under the specified length is 0 in. for all lengths. There are no requirements for lengths over 65 ft.
[d] The tolerances herein are taken from ASTM A6 and apply to the straightness of members received from the rolling mill, measured as illustrated in Figure 1-1.

Table 1-26
ASTM A6 Tolerances for Angles, < 3 in.



Permissible Cross-Sectional Variations

Nominal Leg Size[a], in.	Variations in Thickness for Thicknesses Given, Over and Under, in.			*B*, Leg Size, Over and Under, in.	*T*, Out of Square per Inch of *B*, in.
	$^{3}/_{16}$ and Under	Over $^{3}/_{16}$ to $^{3}/_{8}$ incl.	Over $^{3}/_{8}$		
1 and Under	0.008	0.010	–	$^{1}/_{32}$	$^{3}/_{128}$[b]
Over 1 to 2, incl.	0.010	0.010	0.012	$^{3}/_{64}$	
Over 2 to $2^{1}/_{2}$, incl.	0.012	0.015	0.015	$^{1}/_{16}$	
Over $2^{1}/_{2}$ to 3, excl.	–	–	–	$^{3}/_{32}$[e]	

Permissible Variations in Length

Section	Variations Over Specified Length for Lengths Given[c], in.				
	5 to 10 ft, excl.	10 to 20 ft, excl.	20 to 30 ft, incl.	Over 30 to 40 ft, incl.	40 to 65 ft, incl.
All bar-size angles	$^{5}/_{8}$	1	$1^{1}/_{2}$	2	$2^{1}/_{2}$

Mill Straightness Tolerances[d]

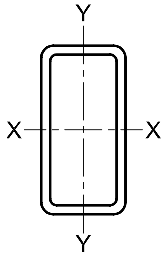
Camber	$^{1}/_{4}$ in. in any 5 ft, or $^{1}/_{4}$ in. $\times \dfrac{(\text{total length, ft})}{5}$, applied to either leg
Sweep	Due to the extreme variations in flexibility of these shapes, permitted variations for sweep are subject to negotiation between the manufacturer and purchaser for the individual sections involved.

Other Permissible Rolling Variations

Ends Out of Square	$^{3}/_{128}$ in. per in. of leg length, or $1^{1}/_{2}$°. Variations based on the longer leg of unequal angle.

– Indicates that there is no requirement.
[a] For unequal angles, longer leg determines classification.
[b] $^{3}/_{128}$ in. per in. = $1^{1}/_{2}$°
[c] The permitted variation under the specified length is 0 in. for all lengths. There are no requirements for lengths over 65 ft.
[d] The tolerances herein are taken from ASTM A6 and apply to the straightness of members received from the rolling mill, measured as illustrated in Figure 1-1.
[e] Leg size $^{1}/_{8}$ in. over permitted.

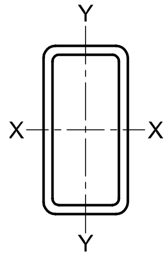
Table 1-27
ASTM Tolerances for Rectangular and Square HSS



<table>
<tr><th colspan="5">ASTM A500, ASTM A501, ASTM A618, ASTM A847 and ASTM A1085</th></tr>
<tr><td rowspan="3">Outside Dimensions</td><td colspan="4">The outside dimensions, measured across the flats at positions at least 2 in. from either end, shall not vary from the specified dimensions by more than the applicable amount given in the following table:</td></tr>
<tr><td colspan="2">Largest Outside Dimension Across Flats, in.</td><td colspan="2">Permissible Variation Over and Under Specified Dimensions[a,b], in.</td></tr>
<tr><td colspan="2">2 1/2 and under
Over 2 1/2 to 3 1/2, incl.
Over 3 1/2 to 5 1/2, incl.
Over 5 1/2</td><td colspan="2">0.020
0.025
0.030
1%[c]</td></tr>
<tr><td rowspan="5">Length</td><td colspan="4">HSS are commonly produced in random lengths, in multiple lengths, and in specific lengths. When specific lengths are ordered, the length tolerances shall be in accordance with the following table:</td></tr>
<tr><td colspan="4">Length tolerance for specific lengths, in.</td></tr>
<tr><td colspan="2">22 ft and under</td><td colspan="2">Over 22 ft[e]</td></tr>
<tr><td>Over</td><td>Under</td><td>Over</td><td>Under</td></tr>
<tr><td>1/2</td><td>1/4</td><td>3/4</td><td>1/4</td></tr>
<tr><td>Wall Thickness</td><td colspan="4">A500 and A847 only: The tolerance for wall thickness exclusive of the weld area shall be plus and minus 10% of the nominal wall thickness specified. The wall thickness is to be measured at the center of the flat.
A1085 only: The minimum wall thickness shall be 95% of the specified wall thickness. The maximum wall thickness, excluding the weld seam, shall not be more than 10% greater than the specified wall thickness. The wall thickness requirements shall apply only to the centers of the flats.</td></tr>
<tr><td>Weight</td><td colspan="4">A501 only: The weight of HSS, as specified in A501 Tables 3 and 4, shall not be less than the specified value by more than 3.5%.</td></tr>
<tr><td>Mass</td><td colspan="4">A618 only: The mass shall not be less than the specified value by more than 3.5%.
A1085 only: The mass shall not deviate from the specified value by more than –3.5% or +10%.</td></tr>
<tr><td>Straightness</td><td colspan="4">The permissible variation for straightness shall be 1/8 in. times the number of ft of total length divided by 5.</td></tr>
<tr><td>Squareness of Sides</td><td colspan="4">Adjacent sides may deviate from 90° by a tolerance of ± 2° maximum.</td></tr>
<tr><td>Radius of Corners</td><td colspan="4">The radius of any outside corner of the section shall not exceed 3 times the specified wall thickness[d, f].</td></tr>
</table>

[a] The respective outside dimension tolerances include the allowances for convexity and concavity. Measurement shall not include the weld reinforcement.
[b] A500, A847 and A1085 only: The tolerances given are for the large flat dimension only. For HSS having a ratio of outside large to small flat dimension less than 1.5, the tolerance on the small flat dimesion shall be identical to those given. For HSS having a ratio of outside large to small flat dimension in the range of 1.5 to 3.0 inclusive, the tolerance on the small flat dimesion shall be 1.5 times those given. For HSS having a ratio of outside large to small flat dimension greater than 3.0, the tolerance on the small flat dimension shall be 2.0 times those given.
[c] This value is 0.01 times the large flat dimension. A501 only: over 5 1/2 to 10 incl., this value is 0.01 times large flat dimension; over 10, this value is 0.02 times the large flat dimension.
[d] A501 only: The radius of any outside corner must not exceed 3 times the calculated nominal wall thickness.
[e] A501 and A618: The upper limit on specific length is 44 ft.
[f] A1085 only: Minimum radius is 1.6t when $t \le 0.400$ in. and 1.8t when $t > 0.400$ in.

Table 1-27 (continued)
ASTM Tolerances for Rectangular and Square HSS



ASTM A500, ASTM A501, ASTM A618, ASTM A847 and ASTM A1085

Twist

The tolerances for twist with respect to axial alignment of the section shall be as shown in the following table:

Specified Dimension of Longer Side, in.	Maximum Twist per 3 ft of length, in.
$1^1/_2$ and under	0.050
Over $1^1/_2$ to $2^1/_2$, incl.	0.062
Over $2^1/_2$ to 4, incl.	0.075
Over 4 to 6, incl.	0.087
Over 6 to 8, incl.	0.100
Over 8	0.112

Twist shall be determined by holding one end of the HSS down on a flat surface plate, measuring the height that each corner on the bottom side of the tubing extends above the surface plate near the opposite end of the HSS, and calculating the difference in the measured heights of such corners[g].

ASTM A1065

Outside Dimension

The outside dimensions, measured across the flats at portions at least 2 in. from either end, shall not vary from the specified dimensions by more than the applicable amount given in the following table:

Nominal Outside Large Flat Dimension	Permissible Variation Over and Under Nominal Outside Flat Dimentions[h]
Squares and rectangles with a large flat to small flat ratio less than 3.0	0.015 times each flat dimension
Rectangles with a large flat to small flat ratio equal to or greater than 3.0	0.020 times each flat dimension

Length	The permissible variation for length shall be +6 / –0 in.
Wall Thickness	The permissible variation in wall thickness shall be +0.03 / –0.01 in.
Straightness	The permissible variation for straightness shall be $^1/_8$ in. times the number of ft total length divided by 5.
Squareness of Sides	Adjacent sides may deviate from 90° by a tolerance of ±2° maximum.
Radius of Corners	Corners shall be bent with a bend radius three times the thickness, 3*t*, or greater.
Twist	The permissible twist shall not exceed $^1/_8$ in. per 3 ft of total length[i, j].
Weld Reinforcement	Weld reinforcement shall not exceed 0.125 in.

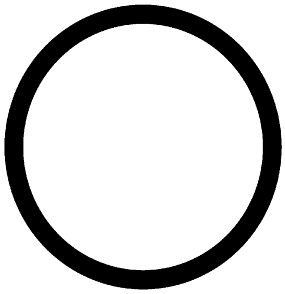
[g] A500, A501, A847 and A1085 only: For heavier sections it is permissible to use a suitable measuring device to determine twist. Twist measurements shall not be taken within 2 in. of the ends of the HSS.

[h] The respective outside dimension tolerances include the allowances for convexity and concavity. Measurement shall not include the weld reinforcement.

[i] Twist shall be determined by holding one end of the HSS down on a flat surface plate, measuring the height that each corner on the bottom side of the tubing extends above the surface plate near the opposite end of the HSS, and calculating the difference in heights of the corners.

[j] For heavier sections it is permissible to use a suitable measuring device to determine twist. Twist measurements shall not be taken within 2 in. of the ends of the HSS.

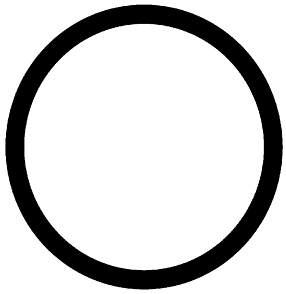
Table 1-28
ASTM Tolerances for Round HSS and Pipes



ASTM A53	
Weight	The weight as specified in A53 Table X2.2 and Table X2.3 or as calculated from the relevant equation in ASME B36.10M shall not vary by more than ±10%. Note that the weight tolerance is determined from the weights of the customary lifts of pipe as produced for shipment by the mill, divided by the number of ft of pipe in the lift. On pipe sizes over 4 in. where individual lengths may be weighed, the weight tolerance is applicable to the individual length.
Diameter	For pipe $1^{1}/_{2}$ in. and under, the outside diameter at any point shall not vary more than $\pm^{1}/_{64}$ in. from the specified outside diameter. For pipe 2 in. and over, the outside diameter shall not vary more than ±1% from the specified outside diameter.
Thickness	The minimum wall thickness at any point shall not be more than 12.5% under the specified wall thickness.
ASTM A500, ASTM A847 and ASTM A1085	
Outside Diameter[a]	For 1.900 in. and under in specified diameter, the outside diameter shall not vary more than ± 0.5%, rounded to the nearest 0.005 in., from the specified diameter. For 2.000 in. and over in specified diameter, the outside diameter shall not vary more than ± 0.75%, rounded to the nearest 0.005 in., from the specified diameter.
Thickness	A500 and A847 only: The wall thickness at any point, excluding the weld seam of welded tubing, shall not be more than 10% under or over the specified wall thickness. A1085 only: The minimum wall thickness shall be 95% of the specified wall thickness. The maximum wall thickness, excluding the weld seam, shall not be more than 10% greater than the specified wall thickness.
Mass (A1085 only)	The mass shall not deviate from the specified value by more than –3.5% or +10%.
ASTM A501 and ASTM A618	
Outside Diameter	For HSS $1^{1}/_{2}$ in. and under in nominal size, the outside diameter shall not vary more than $^{1}/_{64}$ in. over or more than $^{1}/_{32}$ in. under the specified diameter. For round hot-formed HSS 2 in. and over in nominal size, the outside diameter shall not vary more than ± 1% from the specified diameter.
Weight (A501 only)	The weight of HSS, as specified in A501 Table 5, shall not be less than the specified value by more than 3.5%.
Mass (A618 only)	The mass of HSS shall not be less than the specified value by more than 3.5%. The mass tolerance shall be determined from individual lengths or, for HSS $4^{1}/_{2}$ in. and under in outside diameter, shall be determined from masses of customary lifts produced by the mill.

[a] The outside diameter measurements shall be taken at least 2 in. from the end of the HSS.

Table 1-28 (continued)
ASTM Tolerances for Round HSS and Pipes



<table>
<tr><th colspan="5">ASTM A500, ASTM A501, ASTM A618, ASTM A847 and ASTM 1085</th></tr>
<tr><td rowspan="5">Length</td><td colspan="4">HSS are commonly produced in random mill lengths, in multiple lengths, and in specific lengths. When specific lengths are ordered, the length tolerances shall be in accordance with the following table:</td></tr>
<tr><td colspan="4">Length tolerance for specific cut lengths, in.</td></tr>
<tr><td colspan="2">22 ft and under</td><td colspan="2">Over 22 ft[b]</td></tr>
<tr><td>Over</td><td>Under</td><td>Over</td><td>Under</td></tr>
<tr><td>1/2</td><td>1/4</td><td>3/4</td><td>1/4</td></tr>
<tr><td>Straightness</td><td colspan="4">The permissible variation for straightness of HSS shall be 1/8 in. times the number of ft of total length divided by 5.</td></tr>
<tr><td colspan="5">[b] A501 and A618: The upper limit on specific length is 44 ft.</td></tr>
</table>

Table 1-29
Rectangular Plates

Permissible Variations from Flatness (Carbon Steel Only)

Specified Thickness, in.	Variations from Flatness for Specified Widths, in.										
	To 36, excl.	36 to 48, excl.	48 to 60, excl.	60 to 72, excl.	72 to 84, excl.	84 to 96, excl.	96 to 108, excl.	108 to 120, excl.	120 to 144, excl.	144 to 168, excl.	168 and over
To 1/4, excl.	9/16	3/4	15/16	1 1/4	1 3/8	1 1/2	1 5/8	1 3/4	1 7/8	–	–
1/4 to 3/8, excl.	1/2	5/8	3/4	15/16	1 1/8	1 1/4	1 3/8	1 1/2	1 5/8	–	–
3/8 to 1/2, excl.	1/2	9/16	5/8	5/8	3/4	7/8	1	1 1/8	1 1/4	1 7/8	2 1/8
1/2 to 3/4, excl.	7/16	1/2	9/16	5/8	5/8	3/4	1	1	1 1/8	1 1/2	2
3/4 to 1, excl.	7/16	1/2	9/16	5/8	5/8	5/8	3/4	7/8	1	1 3/8	1 3/4
1 to 2, excl.	3/8	1/2	1/2	9/16	9/16	5/8	5/8	5/8	1 1/16	1 1/8	1 1/2
2 to 4, excl.	5/16	3/8	7/16	1/2	1/2	1/2	1/2	9/16	5/8	7/8	1 1/8
4 to 6, excl.	3/8	7/16	1/2	1/2	9/16	9/16	5/8	3/4	7/8	7/8	1
6 to 8, excl.	7/16	1/2	1/2	5/8	11/16	3/4	7/8	7/8	1	1	1

Notes:
1. The longer dimension specified is considered the length, and permissible variations in flatness along the length shall not exceed the tabular amount for the specified width for plates up to 12 ft in length, or in any 12 ft for longer plates.
2. The flatness variations across the width shall not exceed the tabular amount for the specified width.
3. When the longer dimension is under 36 in., the permissible variation shall not exceed 1/4 in. When the longer dimension is from 36 to 72 in., inclusive, the permissible variation should not exceed 75% of the tabular amount for the specified width, but in no case less than 1/4 in.
4. These variations apply to plates which have a specified minimum tensile strength of not more than 60 ksi or comparable chemistry or hardness. The limits in the table are increased 50% for plates specified to a higher minimum tensile strength or comparable chemistry or hardness.
5. For plates 8 in. and over in thickness or 120 in. and over in width, see ASTM A6 Table 13.
6. Plates must be in a horizontal position on a flat surface when flatness is measured.

Permissible Variations in Camber[a] for Carbon Steel Sheared and Gas Cut Rectangular Plates

$$\text{Maximum permissible camber, in. (all thicknesses)} = \tfrac{1}{8}\text{ in.} \times \frac{(\text{total length, ft})}{5}$$

Permissible Variations in in Camber[a] for High-Strength Low-Alloy and Alloy Steel Sheared, Special-Cut, or Gas-Cut Rectangular Plates

Specified Dimension, in.		Permitted Camber, in.
Thickness	Width	
To 2, incl.	All	$\tfrac{1}{8}\text{ in.} \times \frac{(\text{total length, ft})}{5}$
Over 2 to 15, incl.	To 30, incl.	$\tfrac{3}{16}\text{ in.} \times \frac{(\text{total length, ft})}{5}$
	Over 30	$\tfrac{1}{4}\text{ in.} \times \frac{(\text{total length, ft})}{5}$

[a] Camber as it relates to plates is the horizontal edge curvature in the length, measured over the entire length of the plate in the flat position.

PART 2

GENERAL DESIGN CONSIDERATIONS

SCOPE

The specification requirements and other design considerations summarized in this Part apply in general to the design and construction of steel buildings. The specifications, codes and standards listed below are referenced throughout this Manual.

APPLICABLE SPECIFICATIONS, CODES AND STANDARDS

Specifications, Codes and Standards for Structural Steel Buildings

Subject to the requirements in the applicable building code and the contract documents, the design, fabrication and erection of structural steel buildings is governed as indicated in the AISC *Specification* Sections A1 and B2 as follows:

1. ASCE/SEI 7: *Minimum Design Loads and Associated Criteria for Buildings and Other Structures,* ASCE/SEI 7-16. Available from the American Society of Civil Engineers, ASCE/SEI 7 provides the general requirements for loads, load factors and load combinations (ASCE, 2016).
2. AISC *Specification*: The 2016 AISC *Specification for Structural Steel Buildings*, ANSI/AISC 360-16, included in Part 16 of this Manual and available at **www.aisc.org**, provides the general requirements for design and construction (AISC, 2016a).
3. AISC *Code of Standard Practice*: The 2016 AISC *Code of Standard Practice for Steel Buildings and Bridges*, ANSI/AISC 303-16, included in Part 16 of this Manual and available at **www.aisc.org**, provides the standard of custom and usage for the fabrication and erection of structural steel (AISC, 2016b).

Other referenced standards include:

1. RCSC *Specification*: The 2014 RCSC *Specification for Structural Joints Using High-Strength Bolts*, reprinted in Part 16 of this Manual with the permission of the Research Council on Structural Connections and available at **www.boltcouncil.org**, provides the additional requirements specific to bolted joints with high-strength bolts (RCSC, 2014).
2. AWS D1.1/D1.1M: *Structural Welding Code—Steel*, AWS D1.1/D1.1M:2015 (AWS, 2015). Available from the American Welding Society, AWS D1.1/D1.1M provides additional requirements specific to welded joints. Requirements for the proper specification of welds can be found in AWS A2.4: *Standard Symbols for Welding, Brazing, and Nondestructive Examination* (AWS, 2007). See also discussion of welding in Part 8.
3. ACI 318: *Building Code Requirements for Structural Concrete and Commentary,* ACI 318-14. Available from the American Concrete Institute, ACI 318 provides additional requirements for reinforced concrete, including composite design and the design of steel-to-concrete anchorage (ACI, 2014).

Various other specifications and standards from ACI, ASCE, ASME, ASNT, ASTM, AWS and SDI are also referenced in AISC *Specification* Section A2.

Additional Requirements for Seismic Applications

The 2016 AISC *Seismic Provisions for Structural Steel Buildings*, ANSI/AISC 341-16, apply as indicated in Section A1.1 of the 2016 AISC *Specification* and in the Scope provided at the front of this Manual. The AISC *Seismic Provisions* are available at **www.aisc.org** (AISC, 2016c).

Other AISC Reference Documents

The following other AISC publications may be of use in the design and construction of structural steel buildings:

1. AISC *Detailing for Steel Construction*, Third Edition, covers the standard practices and recommendations for steel detailing, including preparation of shop and erection drawings (AISC, 2009).
2. The AISC *Seismic Design Manual*, Second Edition, (AISC, 2012) provides guidance on steel design in seismic applications, in accordance with the 2010 AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2010).
3. The AISC *Design Examples* is an electronic companion to this Manual and can be found at **www.aisc.org/manualresources**. It includes design examples outlining the application of design aids and AISC *Specification* provisions developed in coordination with this Manual (AISC, 2017).

The following AISC Design Guides are available at **www.aisc.org** for in-depth coverage of specific topics in steel design:

1. *Base Plate and Anchor Rod Design*, Design Guide 1 (Fisher and Kloiber, 2006)
2. *Steel and Composite Beams with Web Openings*, Design Guide 2 (Darwin, 1990)
3. *Serviceability Design Considerations for Steel Buildings*, Design Guide 3 (West et al., 2003)
4. *Extended End-Plate Moment Connections—Seismic and Wind Applications*, Design Guide 4 (Murray and Sumner, 2003)
5. *Low- and Medium-Rise Steel Buildings*, Design Guide 5 (Allison, 1991)
6. *Load and Resistance Factor Design of W-Shapes Encased in Concrete*, Design Guide 6 (Griffis, 1992)
7. *Industrial Buildings—Roofs to Anchor Rods*, Design Guide 7 (Fisher, 2004)
8. *Partially Restrained Composite Connections*, Design Guide 8 (Leon et al., 1996)
9. *Torsional Analysis of Structural Steel Members*, Design Guide 9 (Seaburg and Carter, 1997)
10. *Erection Bracing of Low-Rise Structural Steel Buildings*, Design Guide 10 (Fisher and West, 1997)
11. *Vibrations of Steel-Framed Structural Systems Due to Human Activity*, Design Guide 11 (Murray et al., 2016)
12. *Modification of Existing Welded Steel Moment Frame Connections for Seismic Resistance*, Design Guide 12 (Gross et al., 1999)
13. *Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications*, Design Guide 13 (Carter, 1999)
14. *Staggered Truss Framing Systems*, Design Guide 14 (Wexler and Lin, 2002)
15. *Rehabilitation and Retrofit Guide—A Reference for Historic Shapes and Specifications*, Design Guide 15 (Brockenbrough and Schuster, 2017)
16. *Flush and Extended Multiple-Row Moment End-Plate Connections*, Design Guide 16 (Murray and Shoemaker, 2002)

17. *High Strength Bolts—A Primer for Structural Engineers,* Design Guide 17 (Kulak, 2002)
18. *Steel-Framed Open-Deck Parking Structures,* Design Guide 18 (Churches et al., 2003)
19. *Fire Resistance of Structural Steel Framing,* Design Guide 19 (Ruddy et al., 2003)
20. *Steel Plate Shear Walls,* Design Guide 20 (Sabelli and Bruneau, 2006)
21. *Welded Connections—A Primer for Engineers,* Design Guide 21 (Miller, 2017)
22. *Façade Attachments to Steel-Framed Buildings,* Design Guide 22 (Parker, 2008)
23. *Constructability of Structural Steel Buildings,* Design Guide 23 (Ruby, 2008)
24. *Hollow Structural Section Connections*, Design Guide 24 (Packer et al., 2010)
25. *Web-Tapered Frame Design*, Design Guide 25 (Kaehler et al., 2010)
26. *Design of Blast Resistant Structures*, Design Guide 26 (Gilsanz et al., 2013)
27. *Structural Stainless Steel*, Design Guide 27 (Baddoo, 2013)
28. *Stability Design of Steel Buildings*, Design Guide 28 (Griffis and White, 2013)
29. *Vertical Bracing Connections—Analysis and Design*, Design Guide 29 (Muir and Thornton, 2014)
30. *Sound Isolation and Noise Control in Steel Buildings*, Design Guide 30 (Markham and Ungar, 2015)
31. *Design of Castellated and Cellular Beams*, Design Guide 31 (Dinehart et al., 2016)
32. *Design of Steel-Plate Composite Walls*, Design Guide 32 (Varma and Bhardwaj, 2016)

The following Facts for Steel Buildings are available at **www.aisc.org** for practical guidance on specific topics in steel design:

1. *Fire*, Facts for Steel Buildings 1 (Gewain et al., 2003)
2. *Blast and Progressive Collapse,* Facts for Steel Buildings 2 (Marchand and Alfawakhiri, 2004)
3. *Earthquake and Seismic Design*, Facts for Steel Buildings 3 (Hamburger, 2009)
4. *Sound Isolation and Noise Control*, Facts for Steel Buildings 4 (Markham and Ungar, 2016)

OSHA REQUIREMENTS

OSHA *Safety and Health Standards for the Construction Industry*, *29 CFR 1926 Part R Safety Standards for Steel Erection* (OSHA, 2001) must be addressed in the design, detailing, fabrication and erection of steel structures. These regulations became effective on July 18, 2001.

Following is a brief summary of selected provisions and related recommendations. The full text of the regulations should be consulted and can be found at **www.osha.gov**. See also Barger and West (2001) for further information.

Columns and Column Base Plates

1. All column base plates must be designed and fabricated with a minimum of four anchor rods.
2. Posts (which weigh less than 300 lb) are distinguished from columns and excluded from the four-anchor-rod requirement.

3. Columns, column base plates, and their foundations must be designed to resist a minimum eccentric gravity load of 300 lb located 18 in. from the extreme outer face of the column in each direction at the top of the column shaft.
4. Column splices must be designed to meet the same load-resisting characteristics as columns.
5. Double connections through column webs or at beams that frame over the tops of columns must be designed to have at least one installed bolt remain in place to support the first beam while the second beam is being erected. Alternatively, the fabricator must supply a seat or equivalent device with a means of positive attachment to support the first beam while the second beam is being erected.

These features should be addressed in the construction documents. Items 1 through 4 are prescriptive, and alternative means such as guying are time consuming and costly. There are several methods to address the condition in item 5, as shown in Chapter 2 of AISC *Detailing for Steel Construction*.

Safety Cables

1. On multi-story structures, perimeter safety cables (two lines) are required at final interior and exterior perimeters of floors as soon as the deck is installed.
2. Perimeter columns must extend 48 in. above the finished floor (unless constructability does not allow) to allow the installation of perimeter safety cables.
3. Regulations prohibit field welding of attachments for installation of perimeter safety cables once the column has been erected.
4. Provision of some method of attaching the perimeter cable is required, but responsibility is not assigned either to the fabricator or to the erector. While this will be subject to normal business arrangements between the fabricator and the erector, holes for these cables are often punched or drilled in columns by the fabricator.

The primary consideration in the design of the frame based on these rules is that the position of the column splice is set with respect to the floor.

Beams and Bracing

1. Solid-web members (beams) must be connected with a minimum of two bolts or their equivalent before the crane load line is released.
2. Bracing members must be connected with a minimum of one bolt or its equivalent before the crane load line is released.

The OSHA regulations allow an alternative to these minimums, if an “equivalent as specified by the project structural engineer of record” is provided. If the project requirements do not permit the use of bolts as described in items 1 and 2, then the “equivalent” means should be provided in the construction documents. It is recommended that the “equivalent” means should utilize bolts and removable connection material, and should provide requirements for the final condition of the connection. Solutions that employ shoring or the need to hold the member on the crane should be avoided.

Cantilevers

1. The erector is responsible for the stability of cantilevers and their temporary supports until the final cantilever connection is completed. OSHA 1926.756(a)(2) requires that a competent person shall determine if more than two bolts are necessary to ensure the stability of cantilevered members. Cantilever connections must be evaluated for the loads imposed on them during erection and consideration must be made for the intermediate states of completion, including the connection of the backspan member opposing the cantilever.

Certain cantilever connections can facilitate the erector's work in this regard, such as shop attaching short cantilevers, one piece cantilever/backspan beams carried through or over the column at the cantilever and field bolted flange plates or end plate connections to the supporting member. To the extent allowed by the contract documents, the selection of details is up to the fabricator, subject to normal business relations between the fabricator and the erector.

Joists

1. Unless panelized, all joists 40 ft long and longer and their bearing members must have holes to allow for initial connections by bolting.
2. Establishment of bridging terminus points for joists is mandated according to OSHA and manufacturer guidelines.
3. A vertical stabilizer plate to receive the joist bottom chord must be provided at columns. Minimum sizes are given and the stabilizer plate must have a hole for the attachment of guying or plumbing cables.

These features should be addressed in the construction documents and shop drawings.

Walking/Working Surfaces

1. Framed metal deck openings must have structural members configured with projecting elements turned down to allow continuous decking, except where not allowed by design constraints or constructability. The openings in the metal deck are not to be cut until the hole is needed.
2. Steel headed stud anchors, threaded studs, reinforcing bars and deformed anchors that will project vertically from or horizontally across the top flange of the member are not to be attached to the top flanges of beams, joists or beam attachments until after the metal decking or other walking/working surface has been installed.

Framing at openings with down-turned elements and shop versus field attachment of anchors should be addressed in the construction documents and the shop drawings.

Controlling Contractor

1. The controlling contractor must provide adequate site access and adequate storage.
2. The controlling contractor must notify the erector of repairs or modifications to anchor rods in writing. Such modifications and repairs must be approved by the owner's designated representative for design.

3. The controlling contractor must give notice that the supporting foundations have achieved sufficient strength to allow safe steel erection.
4. The controlling contractor must either provide overhead protection or prohibit other trades from working under steel erection activities.

These provisions establish relationships among the erector, controlling contractor, and owner's representative for design that all parties need to be aware of.

USING THE 2016 AISC SPECIFICATION

The 2016 AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360-16) continues the format established in the 2005 edition of the *Specification* (AISC, 2005), ANSI/AISC 360-05, which unified the design provisions formerly presented in the 1989 *Specification for Structural Steel Buildings—Allowable Stress Design and Plastic Design* and the 1999 *Load and Resistance Factor Design Specification for Structural Steel Buildings*. The 2005 *Specification for Structural Steel Buildings* also integrated into a single document the information previously provided in the 1993 *Load and Resistance Factor Design Specification for Single-Angle Members* and the 1997 *Specification for the Design of Steel Hollow Structural Sections*. The 2016 AISC *Specification*, in combination with the 2016 *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341-16), brings together all of the provisions needed for the design of structural steel in buildings and other structures.

The 2016 AISC *Specification* continues to present two approaches for the design of structural steel members and connections. Chapter B establishes the general requirements for analysis and design. It states that "design for strength shall be performed according to the provisions for load and resistance factor design (LRFD) or to the provisions for allowable strength design (ASD)." These two approaches are equally valid for any structure for which the *Specification* is applicable. There is no preference stated or implied in the *Specification*.

The required strength of structural members and connections may be determined by elastic or inelastic analysis for the load combinations associated with LRFD and by elastic analysis for load combinations associated with ASD and as stipulated by the applicable building code. In all cases, the available strength must exceed the required strength. The AISC *Specification* gives provisions for determining the available strength as summarized below.

Load and Resistance Factor Design (LRFD)

The load combinations appropriate for LRFD are given in the applicable building code or, in its absence, ASCE/SEI 7 Section 2.3. For LRFD, the available strength is referred to as the design strength. All of the LRFD provisions are structured so that the design strength must equal or exceed the required strength. This is presented in AISC *Specification* Section B3.1 as

$$R_u \le \phi R_n \tag{2-1}$$

In this equation, R_u is the required strength determined by analysis for the LRFD load combinations, R_n is the nominal strength determined according to the AISC *Specification* provisions, and ϕ is the resistance factor given by the AISC *Specification* for a particular limit state. Throughout this Manual, tabulated values of ϕR_n, the design strength, are given for LRFD. These values are tabulated as blue numbers in columns with the heading LRFD.

If there is a desire to use the LRFD provisions in the form of stresses, the strength provisions can be transformed into stress provisions by factoring out the appropriate section property. In many cases, the provisions are already given directly in terms of stress.

Allowable Strength Design (ASD)

Allowable strength design is similar to what is known as allowable stress design in that they are both carried out at the same load level. Thus, the same load combinations are used. The difference is that for strength design, the primary provisions are given in terms of forces or moments rather than stresses. In every situation, these strength provisions can be transformed into stress provisions by factoring out the appropriate section property. In many cases, the provisions are already given directly in terms of stress.

The load combinations appropriate for ASD are given by the applicable building code or, in its absence, ASCE/SEI 7 Section 2.4. For ASD, the available strength is referred to as the allowable strength. All of the ASD provisions are structured so that the allowable strength must equal or exceed the required strength. This is presented in AISC *Specification* Section B3.2 as

$$R_a \leq \frac{R_n}{\Omega} \tag{2-2}$$

In this equation, R_a is the required strength determined by analysis for the ASD load combinations, R_n is the nominal strength determined according to the AISC *Specification* provisions, and Ω is the safety factor given by the *Specification* for a particular limit state. Throughout this Manual, tabulated values of R_n/Ω, the allowable strength, are given for ASD. These values are tabulated as black numbers on a green background in columns with the heading ASD.

DESIGN FUNDAMENTALS

It is commonly believed that ASD is an elastic design method based entirely on a stress format without limit states and LRFD is an inelastic design method based entirely on a strength format with limit states. Traditional ASD was based on limit-states principles too, but without the use of the term. Additionally, either method can be formulated in a stress or strength basis, and both take advantage of inelastic behavior. The AISC *Specification* highlights how similar LRFD and ASD are in its formulation, with identical provisions throughout for LRFD and ASD.

Design according to the AISC *Specification*, whether it is according to LRFD or ASD, is based on limit states design principles, which define the boundaries of structural usefulness. Strength limit states relate to load carrying capability and safety. Serviceability limit states relate to performance under normal service conditions. Structures must be proportioned so that no applicable strength or serviceability limit state is exceeded.

Normally, several limit states will apply in the determination of the nominal strength of a structural member or connection. The controlling limit state is normally the one that results in the least available strength. As an example, the controlling limit state for bending of a simple beam may be yielding, local buckling, or lateral-torsional buckling for strength, and deflection or vibration for serviceability. The tabulated values may either reflect a single limit state or a combination of several limit states. This will be clearly stated in the introduction to the particular tables.

Loads, Load Factors and Load Combinations

Based on AISC *Specification* Sections B3.1 and B3.2, the required strength (either P_u, M_u, V_u, etc., for LRFD or P_a, M_a, V_a, etc., for ASD) is determined for the appropriate load magnitudes, load factors and load combinations given in the applicable building code. These are usually based on ASCE/SEI 7, which may be used when there is no applicable building code.

Nominal Strengths, Resistance Factors, Safety Factors and Available Strengths

The AISC *Specification* requires that the available strength must be greater than or equal to the required strength for any element. The available strength is a function of the nominal strength given by the *Specification* and the corresponding resistance factor or safety factor. As discussed earlier, the required strength can be determined either with LRFD or ASD load combinations.

The available strength for LRFD is the design strength, which is calculated as the product of the resistance factor, ϕ, and the nominal strength (ϕP_n, ϕM_n, ϕV_n, etc.). The available strength for ASD is the allowable strength, which is calculated as the quotient of the nominal strength and the corresponding safety factor, Ω (P_n / Ω, M_n / Ω, V_n / Ω, etc.).

In LRFD, the margin of safety for the loads is contained in the load factors, and resistance factors, ϕ, to account for unavoidable variations in materials, design equations, fabrication and erection. In ASD, a single margin of safety for all of these effects is contained in the safety factor, Ω.

The resistance factors, ϕ, and safety factors, Ω, in the AISC *Specification* are based upon research, as discussed in the AISC *Specification* Commentary to Chapter B, and the experience and judgment of the AISC Committee on Specifications. In general, ϕ is less than unity and Ω is greater than unity. The higher the variability in the test data for a given nominal strength, the lower its ϕ factor and the higher its Ω factor will be. Some examples of ϕ and Ω factors for steel members are as follows:

$\phi = 0.90$ for limit states involving yielding
$\phi = 0.75$ for limit states involving rupture

$\Omega = 1.67$ for limit states involving yielding
$\Omega = 2.00$ for limit states involving rupture

The general relationship between the safety factor, Ω, and the resistance factor, ϕ, is

$$\Omega = \frac{1.5}{\phi} \tag{2-3}$$

Serviceability

Serviceability requirements of the AISC *Specification* are found in Section B3.8 and Chapter L. The serviceability limit states should be selected appropriately for the specific application as discussed in the *Specification* Commentary to Chapter L. Serviceability limit states and the appropriate load combinations for checking their conformance to serviceability requirements can be found in ASCE/SEI 7 Appendix C and its Commentary. It should be noted that the load combinations in ASCE/SEI 7 Section 2.3 for LRFD and

Section 2.4 for ASD are both for strength design, and are not necessarily appropriate for consideration of serviceability.

Guidance is also available in the Commentary to the AISC *Specification*, both in general and for specific criteria, including camber, deflection, drift, vibrations, wind-induced motion, expansion and contraction, and connection slip. Additionally, the applicable building code may provide some further guidance or establish requirements. See also the serviceability discussions in Parts 3 through 6, AISC Design Guide 3, *Serviceability Design Considerations for Steel Buildings* (West et al., 2003) and AISC Design Guide 11, *Vibrations of Steel-Framed Structural Systems Due to Human Activity* (Murray et al., 2016).

Structural Integrity

Structural integrity as addressed in building codes and AISC *Specification* Section B3.9, is a set of prescriptive requirements for connections that, when met, are intended to provide an unknown, but satisfactory, level of performance of the finished structure. The term structural integrity has often been used interchangeably with progressive collapse, but these two concepts have widely varying interpretations that can influence design in a variety of ways. Progressive collapse requirements generally are intended to prevent the collapse of a structure beyond a localized area of the structure where a structural element has been compromised. Progressive collapse requirements are often mandated for government facilities, or by owners for structures which have a high probability of being subject to terrorist attack.

Structural integrity has always been one of the goals for the structural engineer in engineering design, and for the committees writing design standards. However, it has only been since the collapse of the buildings at the World Trade Center that requirements with the stated purpose of addressing structural integrity have appeared in U.S. building codes. The first building code to incorporate specific structural integrity requirements was the 2008 New York City Building Code, which was quickly followed by requirements in the 2009 *International Building Code*. Although the requirements of these two building codes are both prescriptive in nature, there are some differences in requirements and their application. AISC *Specification* Section B3.9 addresses the requirements of the 2015 *International Building Code* (ICC, 2015).

The 2015 *International Building Code* stipulates minimum integrity provisions for buildings classified as high-rise and assigned to risk categories III or IV. High-rise buildings are defined as those having an occupied floor greater than 75 ft above fire department vehicle access. The structural integrity requirements state that column splices must resist a minimum tension force and beam end connections must resist a minimum axial tension force. The nominal axial tension strength of the beam end connection must equal or exceed either the required vertical shear strength for ASD or $^2/_3$ the required vertical shear strength for LRFD. These required strengths can be reduced by 50% if the beam supports a composite deck with the prescribed steel anchors (Geschwindner and Gustafson, 2010).

The *International Building Code* structural integrity requirements for the axial tension capacity of the beam end connections use a nominal strength basis reflecting the intent of the code to avoid brittle rupture failures of the connection components, rather than limiting deformations or yielding of those components. AISC *Specification* Section B3.9 is based on this difference in limit state requirements for resistance to the prescriptive structural integrity loads, as compared to those limit states required when designing for traditional load combinations.

Progressive Collapse

Progressive collapse is defined in ASCE/SEI 7-16 (ASCE, 2016) as "the spread of an initial local failure from element to element resulting, eventually, in the collapse of an entire structure or a disproportionately large part of it."

Progressive collapse requirements often involve assessment of the structure's ability to accommodate loss of a member that has been compromised through redistribution of forces throughout the remaining structure. Design for progressive collapse poses a particularly challenging problem since it is difficult to identify the load cases to be examined or the members that may be compromised. Two main sources of requirements for evaluation of structures for progressive collapse are the Department of Defense and the General Services Administration. For facilities covered by the Department of Defense, all new and existing buildings of three stories or more must be designed to avoid progressive collapse. The specific requirements are published in United Facilities Criteria 4-023-03, *Design of Buildings to Resist Progressive Collapse* (DOD, 2013).

For federal facilities under the jurisdiction of the General Services Administration, threat independent guidelines have been developed. The publication "Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects" (USGSA, 2003) provides an explicit process that any structural engineer could use to evaluate the progressive collapse potential of a multi-story facility.

Required Strength, Stability, Effective Length, and Second-Order Effects

As previously discussed, the AISC *Specification* requires that the required strength be less than or equal to the available strength in the design of every member and connection. Chapter C also requires that stability shall be provided for the structure as a whole and each of its elements. Any method that considers the influence of second-order effects, also known as P-delta effects, may be used. Thus, required strengths must be determined including second-order effects, as described in *Specification* Section C1. Note that *Specification* Section C2.1(b) permits an amplified first-order analysis as one method of second-order analysis, as provided in Appendix 8.

Second-order effects are the additional forces, moments and displacements resulting from the applied loads acting in their displaced positions as well as the changes from the undeformed to the deformed geometry of the structure. Second-order effects are obtained by considering equilibrium of the structure within its deformed geometry. There are numerous ways of accounting for these effects. The commentary to AISC *Specification* Chapter C provides some guidance on methods of second-order analysis and suggests several benchmark problems for checking the adequacy of analysis methods.

Since 1963, there have been provisions in the AISC Specifications to account for second-order effects. Initially, these provisions were embedded in the interaction equations. In past ASD Specifications, second-order effects were accounted for by the term

$$\frac{1}{1-\dfrac{f_a}{F'_e}}$$

found in the interaction equation. In past LRFD Specifications, the factors B_1 and B_2 from Chapter C of those specifications were used to amplify moments to account for second-order effects. B_1 was used to account for the second-order effects due to member curvature and B_2 was used to account for second-order effects due to sidesway. In both Specifications, more exact methods were permitted.

AISC *Specification* Section C1 and Appendix 7 provide three approaches that may be followed.

- The *direct analysis method* is provided in Chapter C. This is the most comprehensive and, as the name suggests, most direct approach to incorporating all necessary factors in the analysis. Through the use of notional loads, reduced stiffness, and a second-order analysis, the design can be carried out with the forces and moments from the analysis and an effective length equal to the member length, $K = 1.0$. Section C2 of the AISC *Specification* details the requirements for determination of required strengths using this method.
- The *effective length method* is given in AISC *Specification* Appendix 7, Section 7.2. In this method, all gravity-only load cases have a minimum lateral load equal to 0.2% of the story gravity load applied. A second-order analysis is carried out and the member strengths of columns and beam-columns are determined using effective lengths, determined by elastic buckling analysis, or more commonly, the alignment charts in the Commentary to the *Specification* when the associated assumptions are satisfied. The *Specification* permits $K = 1.0$ when the ratio of second-order drift to first-order drift is less than or equal to 1.1.
- The *first-order analysis method* is given in AISC *Specification* Appendix 7, Section 7.3. With this approach, second-order effects are captured through the application of an additional lateral load equal to at least 0.42% of the story gravity load applied in each load case. No further second-order analysis is necessary. The required strengths are taken as the forces and moments obtained from the analysis and the effective length factor is $K = 1.0$.

When a second-order analysis is called for in the above methods, AISC *Specification* Section C1 allows any method that properly considers P-delta effects. One such method is amplified first-order elastic analysis provided in *Specification* Appendix 8. This is a modified carryover of the B_1-B_2 approach used in previous LRFD Specifications, which was an extension of the simple approach taken in past ASD Specifications.

The AISC *Specification* fully integrates the provisions for stability with the specified methods of design. For all framing systems, when using the direct analysis method, AISC *Specification* Section C3 provides that the effective length for flexural buckling of all members shall be taken as the unbraced length unless a smaller value is justified by rational analysis. For the effective length method, AISC *Specification* Appendix 7, Section 7.2.3(a) provides that in braced frames, the effective length factor, K, may be taken as 1.0. For moment frames, Appendix 7, Section 7.2.3(b) requires that a critical buckling analysis to determine the critical buckling stress, F_e, be performed or effective length factors, K, be used. For the first-order analysis method, Appendix 7, Section 7.3.3 stipulates that the effective length for flexural buckling of all members shall be taken as the unbraced length unless a smaller value is justified by rational analysis. This is discussed in more detail in the Commentary to Appendix 7.

Simplified Determination of Required Strength

When a fast, conservative solution is desired, the following simplification of the effective length method can be used with the aid of Table 2-1. The features of each of the other methods of design for stability are summarized and compared in Table 2-2.

An approximate second-order analysis approach is provided in AISC *Specification* Appendix 8. Where the member amplification (P-δ) factor is small, that is, less than B_2, it is conservative to amplify the total moment and force by B_2. Thus, Equations A-8-1 and A-8-2 become

$$M_r = B_1 M_{nt} + B_2 M_{lt} = B_2 M_u \tag{2-4}$$

$$P_r = P_{nt} + B_2 P_{lt} = B_2 P_u \tag{2-5}$$

To use this simplified method, B_1 cannot exceed B_2. For members not subject to transverse loading between their ends, it is very unlikely that B_1 would be greater than 1.0. In addition, the simplified approach is not valid if the amplification factor, B_2, is greater than 1.5, because with the exception of taking $B_1 = B_2$, this simplified method meets the provisions of the effective length method in AISC *Specification* Appendix 7. It is up to the engineer to ensure that the frame is proportioned appropriately to use this simplified approach. In most designs it is not advisable to have a final structure where the second order amplification is greater than 1.5, although it is acceptable. In those cases, one should consider stiffening the structure.

Step 1: Perform a first-order elastic analysis. Gravity load cases must include a minimum lateral load at each story equal to 0.002 times the story gravity load where the story gravity load is the load introduced at that story, independent of any loads from above.

Step 2: Establish the design story drift limit and determine the lateral load that produces that drift. This is intended to be a measure of the lateral stiffness of the structure.

Step 3: Determine the ratio of the total story gravity load to the lateral load determined in Step 2. For an ASD design, this ratio must be multiplied by 1.6 before entering Table 2-1. This ratio is part of the determination of the calculation on the elastic critical buckling strength, $P_{e\,story}$, in AISC *Specification* Equation A-8-7, which includes the parameter R_M. R_M is a minimum of 0.85 for rigid frames and 1.0 for all other frames.

Step 4: Multiply all of the forces and moments from the first-order analysis by the value obtained from Table 2-1. Use the resulting forces and moments as the required strengths for the designs of all members and connections. Note that B_2 must be computed for each story and in each principal direction.

Step 5: For all cases where the multiplier is 1.1 or less, shown shaded in Table 2-1, the effective length may be taken as the member length, $K = 1.0$. For cases where the multiplier is greater than 1.1, but does not exceed 1.5, determine the effective length factor through analysis, such as with the alignment charts of the AISC *Specification* Commentary. For cases where no value is shown for the multiplier, the structure must be stiffened in order to use this simplified approach. Note that the multipliers are the same value for both $R_M = 0.85$ and 1.0 in most instances due to rounding. Where this is not the case, two values are given consistent with the two values of R_M, respectively.

Step 6: Ensure that the drift limit set in Step 2 is not exceeded and revise design as needed.

STABILITY BRACING

Per AISC *Specification* Section B3.4, at points of support, beams, girders and trusses shall be restrained against rotation about their longitudinal axis unless it can be shown that the restraint is not required (also a basic assumption stated in AISC *Specification* Section F1). Additionally, stability bracing with adequate strength and stiffness must be provided consistent with that assumed at braced points in the analysis for frames, columns and beams (see AISC *Specification* Appendix 6). Some guidance for special cases follows.

Simple-Span Beams

In general, adequate lateral bracing is provided to the compression flange of a simple-span beam by the connections of infill beams, joists, concrete slabs, metal deck, concrete slabs on metal deck, and similar framing elements.

Beam Ends Supported on Bearing Plates

The stability of a beam end supported on a bearing plate can be provided in one of several ways (see Figure 2-1):

1. The beam end can be built into solid concrete or masonry using anchorage devices.
2. The beam top flange can be stabilized through interconnection with a floor or roof system, provided that system itself is anchored to prevent its translation relative to the beam bearing.
3. A top-flange stability connection can be provided.
4. An end-plate or transverse stiffeners located over the bearing plate extending to near the top-flange *k*-distance can be provided. Such stiffeners must be welded to the top of the bottom flange and to the beam web, but need not extend to or be welded to the top flange.

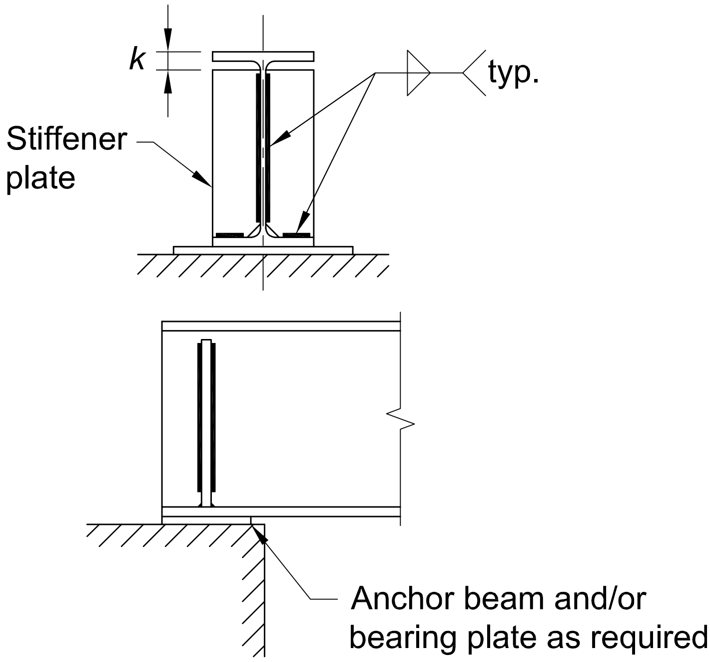
In each case, the beam and bearing plate must also be anchored to the support, as required. For the design of beam bearing plates, see Part 14.

In atypical framing situations, such as when very deep beams are used, the strength and stiffness requirements in AISC *Specification* Appendix 6 can be applied to ensure the stability of the assembly. It may also be possible to demonstrate in a limited number of cases, such as with beams with thick webs and relatively shallow depths, that the beam has been properly designed without providing the details described above. In this case, the beam and bearing plate must still be anchored to the support. In any case, it should be noted that the assembly must also meet the requirements in AISC *Specification* Section J10.

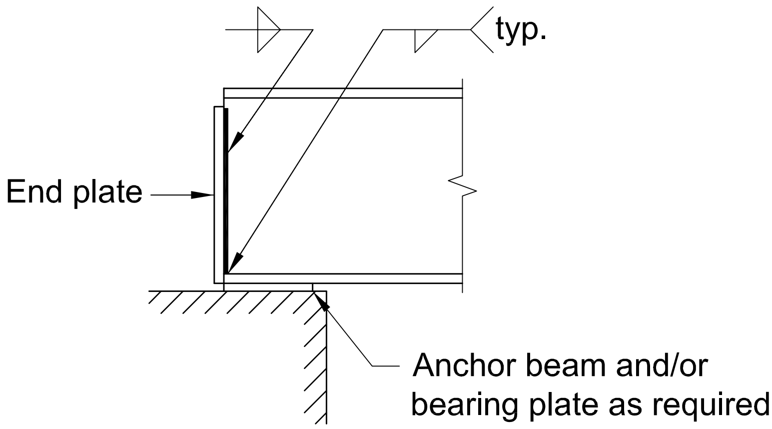
Beams and Girders Framing Continuously Over Columns

Roof framing is commonly configured with cantilevered beams that frame continuously over the tops of columns to support drop-in beams between the cantilevered segments (Rongoe, 1996; CISC, 1989). It is also commonly desirable to provide an assembly in which the intersection of the beam and column can be considered a braced point for the design of both the continuous cantilevering beam and the column top. The required stability can be provided in several ways (see Figures 2-2a through 2-2e):

1. When an infill beam frames into the continuous beam at the column top, the required stability normally can be provided by using connection element(s) for the infill beam that cover three-quarters or more of the T-dimension of the continuous beam. Alternatively, connection elements that cover less than three-quarters of the T-dimension of the continuous beam can be used in conjunction with partial-depth stiffeners in the beam web along with a moment connection between the column top and beam



(a) Stability provided with transverse stiffeners



(b) Stability provided with an end plate

Fig. 2-1. Beam end supported on bearing plate.

bottom to maintain alignment of the beam/column assembly. A cap plate of reasonable proportions and four bolts will normally suffice.

In either case, note that OSHA requires that, if two framing infill beams share common holes through a column web or the web of a beam that frames continuously over the top of a column,[1] the beam erected first must remain attached while connecting the second.

2. When joists frame into the continuous beam or girder, the required stability normally can be provided by using bottom chord extensions connected to the column top. The resulting continuity moments must be reported to the joist supplier for their use in the design

[1] This requirement applies only at the location of the column, not at locations away from the column.

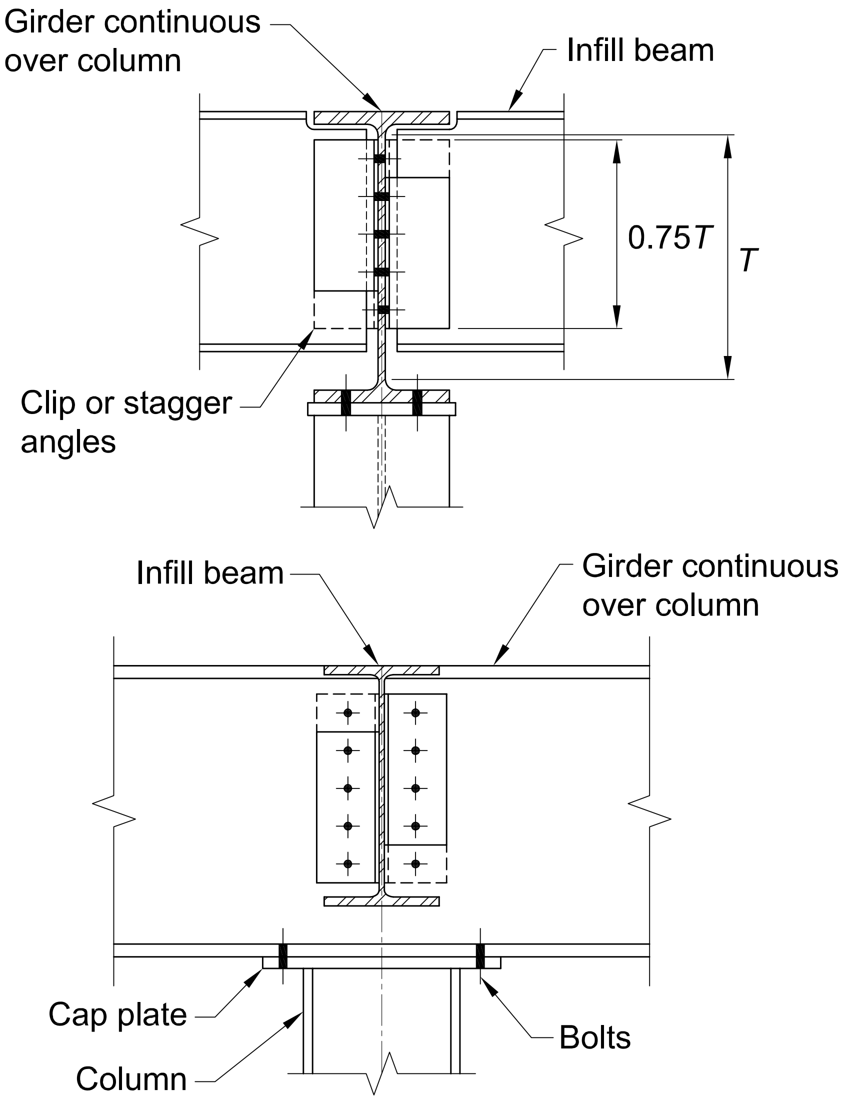


Fig. 2-2a. Beam framing continuously over column top, stability provided with connections of infill beams.

of the joists and bridging. Note that the continuous beam must still be checked for the concentrated force due to the column reaction per AISC *Specification* Section J10.

The position of the bottom chord extension relative to the column cap plate will affect the bottom chord connection detail. When the extension aligns with the cap plate, the load path and force transfer is direct. When the extension is below the column cap plate, the column must be designed to stabilize the beam bottom flange and the connection between the extension and the column must develop the continuity/brace force. When the extension is above the column top, the beam web must have the necessary strength and stiffness to adequately brace the beam bottom/column top.

3. If connection of the joist bottom chord extensions to the column must be avoided, the required stability can be provided with a diagonal brace that satisfies the strength and stiffness requirements in AISC *Specification* Appendix 6. Providing a relatively shallow angle with respect to the horizontal can minimize gravity-load effects in the diagonal brace.

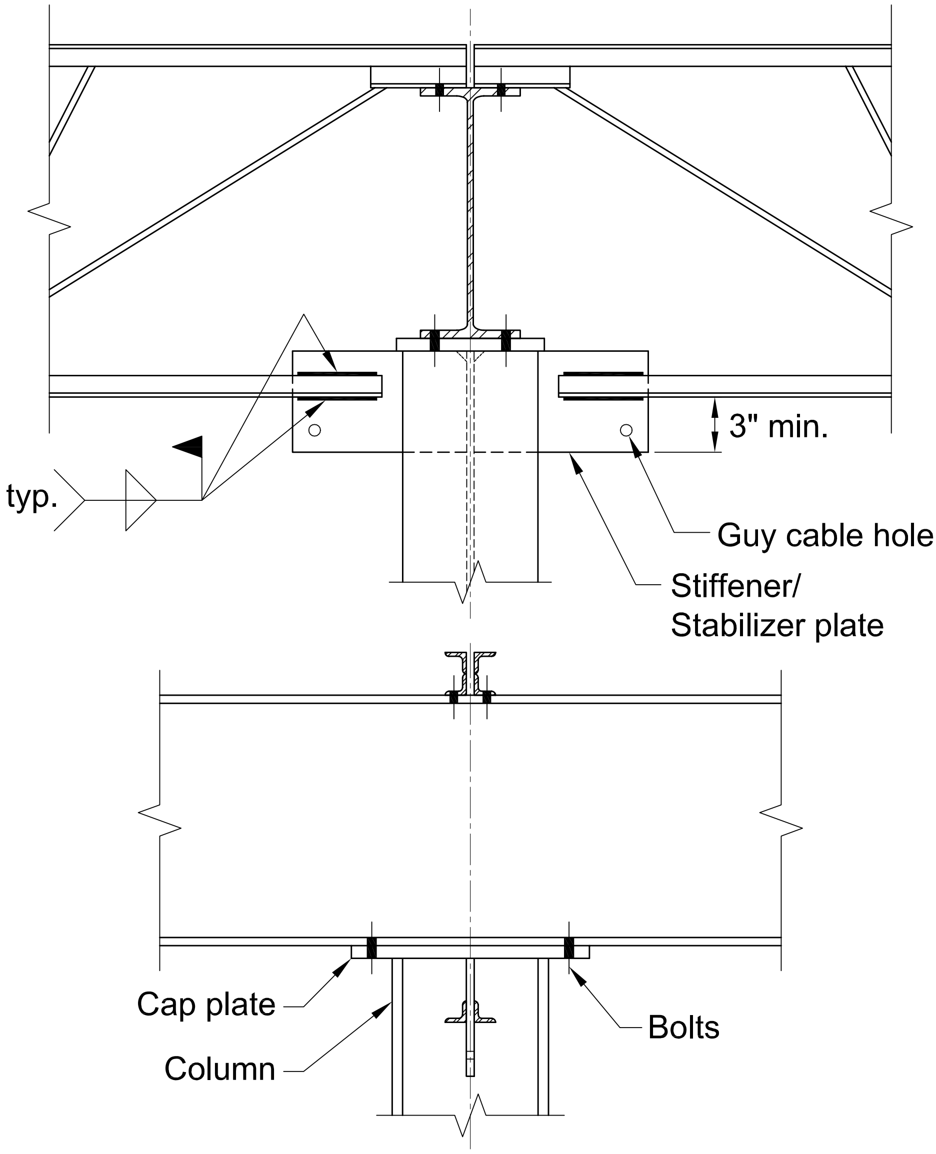


Fig. 2-2b. Beam framing continuously over column top, stability provided with welded joist-chord extensions at column top.

Alternatively, the required stability can be provided with stiffeners in the beam web along with a moment connection between the column top and beam bottom to maintain alignment of the beam/column assembly. A cap plate of reasonable proportions and four bolts will normally suffice.

In atypical framing situations, such as when very deep girders are used, the strength and stiffness requirements in AISC *Specification* Appendix 6 can be applied for both the beam and the column to ensure the stability of the assembly. It may also be possible to demonstrate in a limited number of cases, such as with continuous beams with thick webs and relatively shallow depths, that the column and beam have been properly designed without providing infill beam connections, connected joist extensions, stiffeners, or diagonal braces as described above. In this case, a properly designed moment connection is still required between the beam bottom flange and the column top. In any case, it should be noted that the assembly must also meet the requirements in AISC *Specification* Section J10.

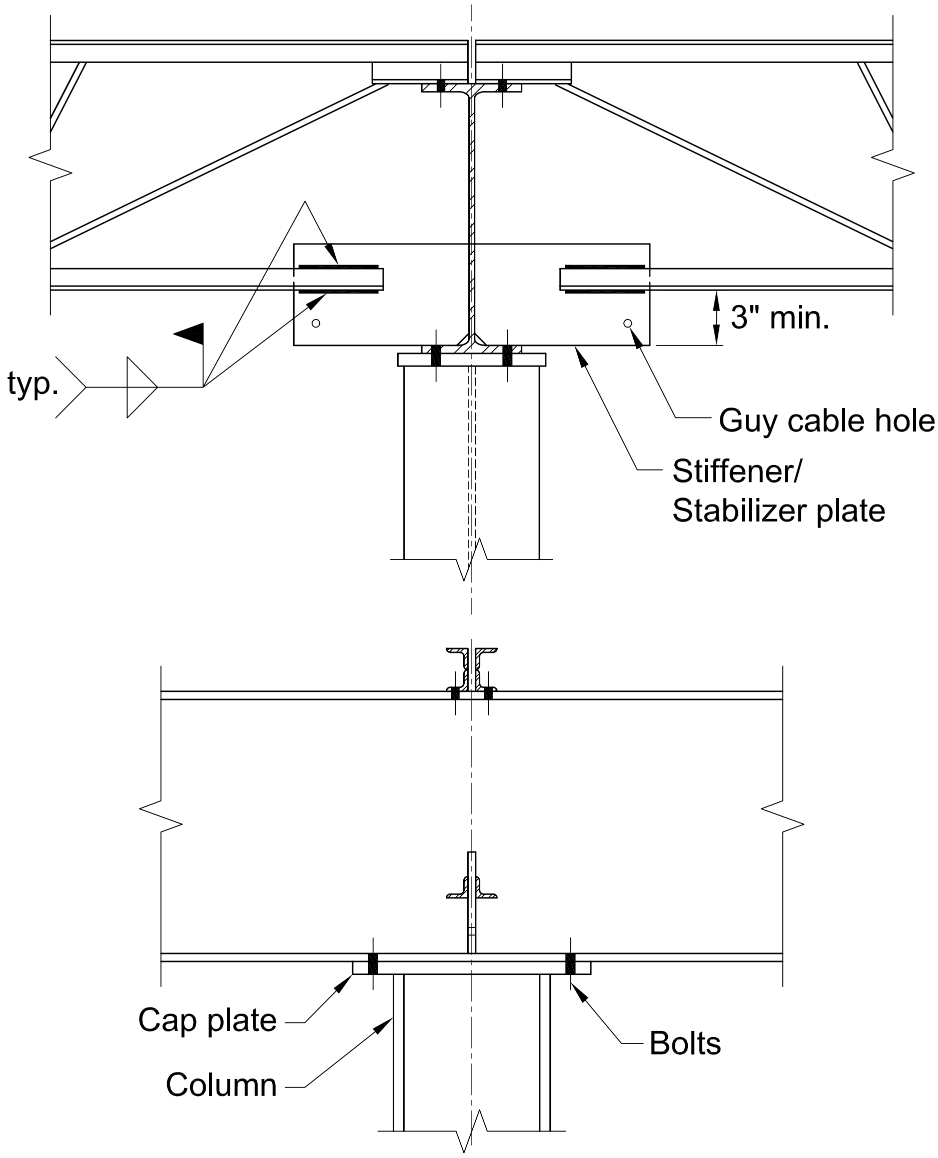


Fig. 2-2c. Beam framing continuously over column top, stability provided with welded joist-chord extensions above column top.

PROPERLY SPECIFYING MATERIALS

Availability

The general availability of structural shapes, HSS and pipe can be determined by checking the AISC database of available structural steel shapes, available at **www.aisc.org**. Generally, where many producers are listed, it is an indication that the particular shape is commonly available. However, except for the larger shapes, when only one or two producers are listed, it is prudent to consider contacting a steel fabricator to determine availability.

Material Specifications

Applicable material specifications are as shown in the following tables:

- Structural shapes in Table 2-4

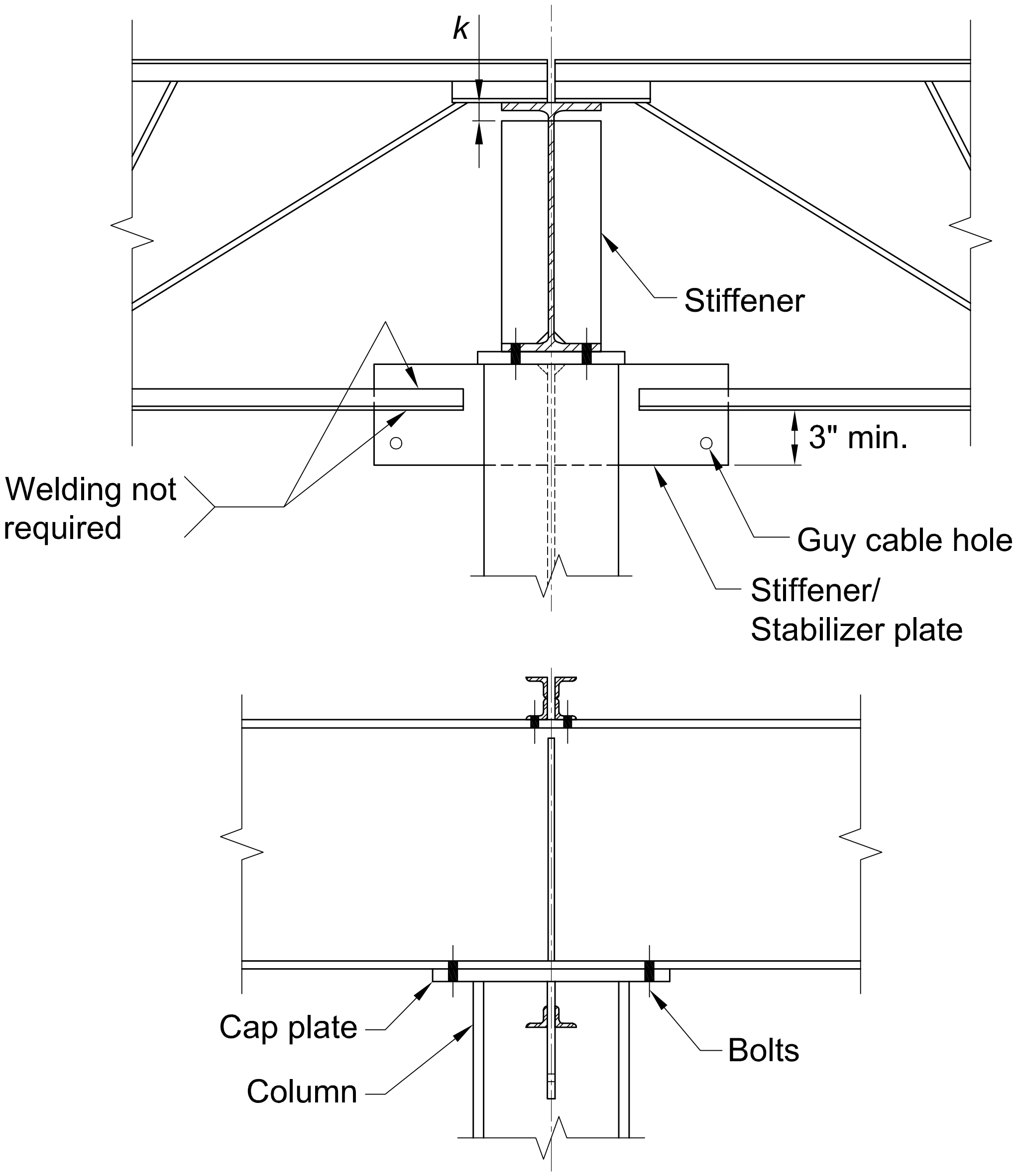


Fig. 2-2d. Beam framing continuously over column top, stability provided with transverse stiffeners, joist chord extensions located at column top not welded.

- Plate and bar products in Table 2-5
- Fastening products in Table 2-6

Preferred material specifications are indicated in black shading. The designation of preferred material specifications is based on consultations with fabricators to identify materials that are commonly used in steel construction, and reflects such factors as ready availablity, ease of ordering and delivery, and pricing. AISC recommends the use of preferred materials in structural steel designs, but the final decision is up to the designer based on project conditions. Other applicable material specifications are as shown in grey shading. The availability of grades other than the preferred material specification should be confirmed prior to their specification.

Cross-sectional dimensions and production tolerances are addressed as indicated under “Standard Mill Practices” in Part 1.

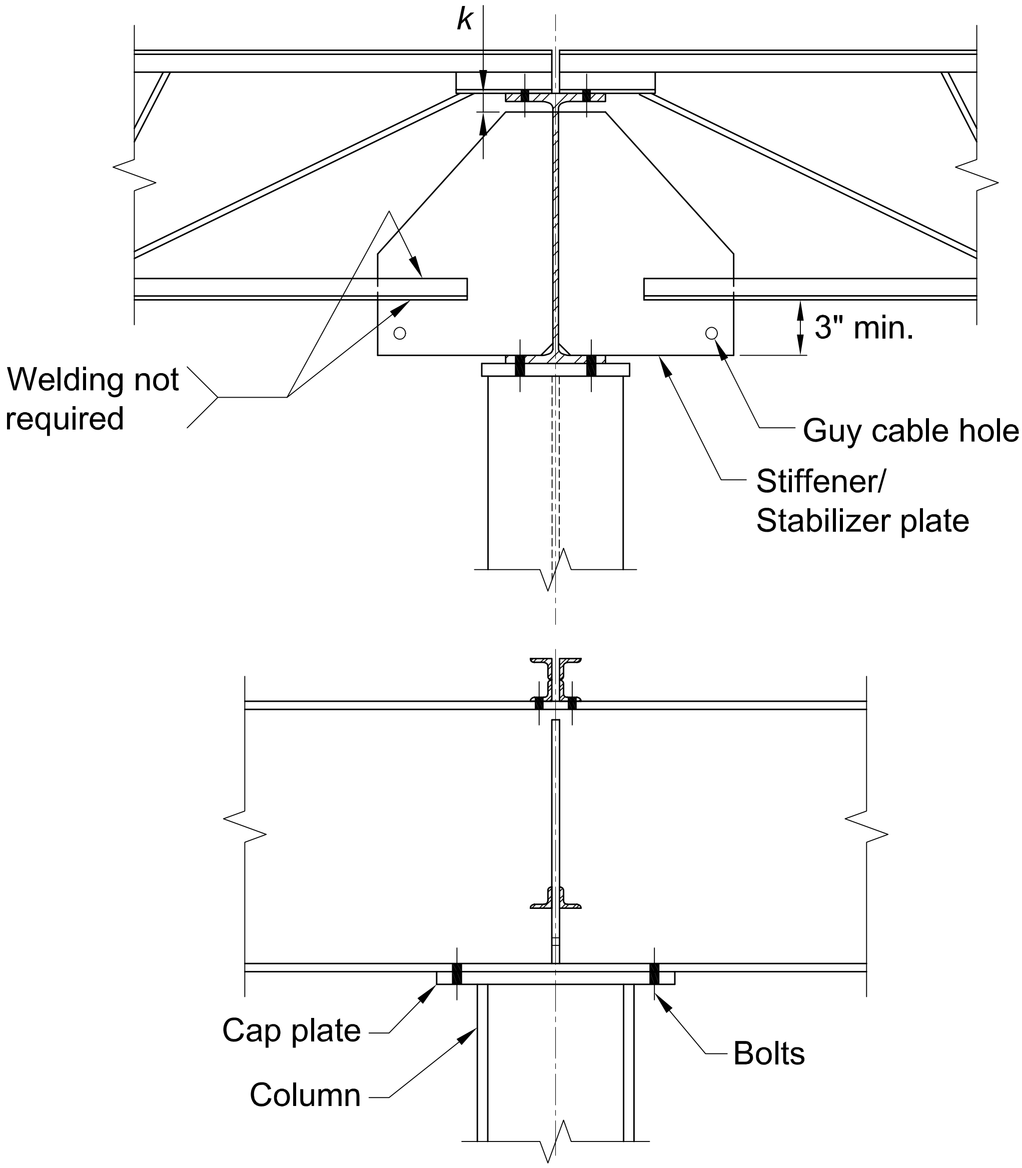


Fig. 2-2e. Beam framing continuously over column top, stability provided with stiffener plates, joist-chord extensions located above column top not welded.

Other Products

Anchor Rods

Although the AISC *Specification* permits other materials for use as anchor rods, ASTM F1554 is the preferred specification, since all anchor rod production requirements are together in a single specification. ASTM F1554 provides three grades, namely 36 ksi, 55 ksi and 105 ksi. All Grade 36 rods are weldable. Grade 55 rods are weldable only when they are made per Supplementary Requirement S1. The project specifications must indicate if the material is to conform to Supplementary Requirement S1. As a heat-treated material, Grade 105 rods cannot be welded. Grade 105 should be used only for limited applications that require its high strength. For more information, refer to AISC Design Guide 1, *Base Plate and Anchor Rod Design* (Fisher and Kloiber, 2006).

Raised-Pattern Floor Plates

ASTM A786 is the standard specification for rolled steel floor plates. As floor-plate design is seldom controlled by strength considerations, ASTM A786 "commercial grade" is commonly specified. If so, per ASTM A786-15, Section 5.1.3, "the product will be supplied 0.33% maximum carbon by heat analysis, and without specified mechanical properties." Alternatively, if a defined strength level is desired, ASTM A786 raised-pattern floor plate can be ordered to a defined plate specification, such as ASTM A36, A572 or A588; see ASTM A786 Sections 5.1.3, 7.1 and 8.

Sheet and Strip

Sheet and strip products, which are generally thinner than structural plate and bar products are produced to such ASTM specifications as A606 (see Table 2-3).

Filler Metal

The appropriate filler metal for structural steel is as summarized in AWS D1.1/D1.1M:2015 Table 3.1 for the various combinations of base metal specification and grade and electrode specification. Weld strengths in this Manual are based upon a tensile strength level of 70 ksi.

Steel Headed Stud Anchors

As specified in AWS D1.1/D1.1M:2015, Type B shear stud connectors (referred to in the AISC *Specification* as steel headed stud anchors) are used for the interconnection of steel and concrete elements in composite construction (F_u = 65 ksi).

Open-Web Steel Joists

The AISC *Code of Standard Practice* does not include steel joists in its definition of structural steel. Steel joists are designed and fabricated per the requirements of specifications published by the Steel Joist Institute (SJI). Refer to SJI literature for further information.

Castellated Beams

Castellated beams and cellular beams are members constructed by cutting along a staggered pattern down the web of a wide-flange member, offsetting the resulting pieces such that the deepest points of the cut are in contact, and welding the two pieces together, thereby creating a deeper member with openings along its web. For more information, refer to AISC Design Guide 31, *Design of Castellated and Cellular Beams* (Dinehart et al., 2016).

Steel Castings and Forgings

Steel castings are specified as ASTM A27 Grade 65-35 or ASTM A216 Grade 80-35. Steel forgings are specified as ASTM A668.

Forged Steel Structural Hardware

Forged steel structural hardware products, such as clevises, turnbuckles, eye nuts and sleeve nuts, are occasionally used in building design and construction. These products are generally forged according to ASTM A668 Class A requirements. ASTM A29 Grade 1035 material is commonly used in the manufacture of clevises and turnbuckles. ASTM A29

Grade 1030 material is commonly used in the manufacture of steel eye nuts and steel eye bolts. ASTM A29 Grade 1018 material is commonly used in the manufacture of sleeve nuts. Other products, such as steel rod ends, steel yoke ends and pins, cotter pins, and coupling nuts are commonly provided generically as “carbon steel.”

The dimensional and strength characteristics of these devices are fully described in the literature provided by their manufacturer. Note that manufacturers usually provide strength characteristics in terms of a “safe working load” with a safety factor as high as 5, assuming that the product will be used in rigging or similar applications subject to dynamic loading. The manufacturer’s safe working load may be overly conservative for permanent installations and similar applications subject to static loading only.

If desired, the published safe working load can be converted into an available strength with reliability consistent with that of other statically loaded structural materials. In this case, the nominal strength, R_n, is determined as:

$$R_n = (\textit{safe working load}) \times (\textit{manufacturer's safety factor}) \tag{2-6}$$

and the available strength, ϕR_n or R_n/Ω, is determined using

$$\phi = 0.50 \text{ (LRFD)} \qquad \Omega = 3.00 \text{ (ASD)}$$

Crane Rails

Crane rails are furnished to ASTM A759, ASTM A1, and/or manufacturer’s specifications and tolerances.

Most manufacturers chamfer the top and sides of the crane-rail head at the ends unless specified otherwise to reduce chipping of the running surfaces. Often, crane rails are ordered as end-hardened, which improves the resistance of the crane-rail ends to impact that occurs as the moving wheel contacts it during crane operation. Alternatively, the entire rail can be ordered as heat-treated. When maximum wheel loading or controlled cooling is needed, refer to manufacturers’ catalogs. Purchase orders for crane rails should be noted “for crane service.”

Light 40-lb rails are available in 30-ft lengths, 60-lb rails in 30-, 33- or 39-ft lengths, standard rails in 33- or 39-ft lengths and crane rails up to 80 ft. Consult manufacturer for availability of other lengths. Rails should be arranged so that joints on opposite sides of the crane runway will be staggered with respect to each other and with due consideration to the wheelbase of the crane. Rail joints should not occur at crane girder splices. Odd lengths that must be included to complete a run or obtain the necessary stagger should be not less than 10 ft long. Rails are furnished with standard drilling in both standard and odd lengths unless stipulated otherwise on the order.

CONTRACT DOCUMENT INFORMATION

Design Drawings, Specifications, and Other Contract Documents

CASE Document 962D, *A Guideline Addressing Coordination and Completeness of Structural Construction Documents* (CASE, 2013), provides comprehensive guidance on the preparation of structural design drawings.

Most provisions in the AISC *Specification*, RCSC *Specification*, AWS D1.1/D1.1M, and the AISC *Code of Standard Practice* are written in mandatory language. Some provisions require the communication of information in the contract documents, some provisions are invoked only when specified in the contract documents, and some provisions require the approval of the owner's designated representative for design if they are to be used. Following is a summary of these provisions in the AISC *Specification*, RCSC *Specification*, AISC *Code of Standard Practice* and AWS D1.1/D1.1M.

Required Information

The following communication of information is required in the contract documents:

1. Required drawing information, per AISC *Code of Standard Practice* Sections 3.1 and 3.1.1 through 3.1.6. and RCSC *Specification* Section 1.6 (bolting products and joint type)
2. Drawing numbers and revision numbers, per AISC *Code of Standard Practice* Sections 3.1 and 3.5
3. Structural system description, per AISC *Code of Standard Practice* Section 7.10.1
4. Installation schedule for nonstructural steel elements in the structural system, per AISC *Code of Standard Practice* Section 7.10.2
5. Project schedule, per AISC *Code of Standard Practice* Section 9.5.1
6. Complete information regarding base metal specification designation and the location, type, size and extent of all welds, per AWS D1.1/D1.1M clauses 1.4.1 and 2.3.4

Depending on the option(s) selected for connections (see AISC *Code of Standard Practice* Section 3.1.1), the information identified as required by AWS may not be fully available until this information is established as part of the connection work delegated to the fabricator.

Information Required Only When Specified

The following provisions are invoked only when specified in the contract documents:

1. Special material notch-toughness requirements, per AISC *Specification* Section A3.1c and Section A3.1d
2. Special connections requiring pretensioned or slip-critical bolted connections, per AISC *Specification* Section J3.1
3. Bolted joint requirements, per AISC *Specification* Section J3.1 and RCSC *Specification* Section 1.6
4. Special cambering considerations, per AISC *Code of Standard Practice* Sections 3.1 and 3.1.5
5. Special contours and finishing requirements for thermal cutting, per AISC *Specification* Sections M2.2 and M2.3, respectively
6. Corrosion protection requirements, if any, per AISC *Specification* Section M3 and AISC *Code of Standard Practice* Sections 6.5, 6.5.2 and 6.5.3
7. Responsibility for field touch-up painting, if painting is specified, per AISC *Specification* Section M4.6 and AISC *Code of Standard Practice* Section 6.5.4
8. Special quality control and inspection requirements, per AISC *Specification* Chapter N and AISC *Code of Standard Practice* Sections 8.1.3, 8.2 and 8.3
9. Evaluation procedures, per AISC *Specification* Section B7

10. Fatigue requirements, if any, per AISC *Specification* Section B3.11
11. Tolerance requirements other than those specified in the AISC *Code of Standard Practice*, per *Code of Standard Practice* Section 1.10
12. Designation of each connection as Option 1, 2 or 3, and identification of requirements for substantiating connection information, if any, per AISC *Code of Standard Practice* Section 3.1.1
13. Specific instructions to address items differently, if any, from requirements in the AISC *Code of Standard Practice*, per *Code of Standard Practice* Section 1.1
14. Submittal schedule for shop and erection drawings, per AISC *Code of Standard Practice* Section 4.2.3
15. Mill order timing, special mill testing, and special mill tolerances, per AISC *Code of Standard Practice* Sections 5.1, 5.1.1 and 5.1.4, respectively
16. Removal of backing bars and runoff tabs, per AISC *Code of Standard Practice* Section 6.3.2
17. Special erection mark requirements, per AISC *Code of Standard Practice* Section 6.6.1
18. Special delivery and erection sequences, per AISC *Code of Standard Practice* Sections 6.7.1 and 7.1, respectively
19. Special field splice requirements, per AISC *Code of Standard Practice* Section 6.7.4
20. Specials loads to be considered during erection, per AISC *Code of Standard Practice* Section 7.10.3
21. Special safety protection treatments, per AISC *Code of Standard Practice* Section 7.11.1
22. Identification of adjustable items, per AISC *Code of Standard Practice* Section 7.13.1.3
23. Cuts, alterations and holes for other trades, per AISC *Code of Standard Practice* Section 7.15
24. Revisions to the contract, per AISC *Code of Standard Practice* Section 9.3
25. Special terms of payment, per AISC *Code of Standard Practice* Section 9.6
26. Identification of architecturally exposed structural steel, per AISC *Code of Standard Practice* Section 10
27. Welding code (AWS D1.1/D1.1M) requirements that are applicable only when specified, per AWS D1.1/D1.1M clause 1.4.1
28. All additional nondestructive testing that is not specifically addressed in the welding code, per AWS D1.1/D1.1M clause 1.4.1
29. Requirements for inspection including verification inspection (see also AISC *Specification* Chapter N and AISC *Code of Standard Practice* Section 8), per AWS D1.1/D1.1M clauses 1.4.1 and 2.3.5.6
30. Weld acceptance criteria other than that specified in AWS D1.1/D1.1M clause 6, per AWS D1.1/D1.1.M clause 1.4.1
31. Charpy V-notch toughness criteria for weld metal, base metal, and/or heat affected zones (see also AISC *Specification* Sections A3.1 and J2.6), per AWS D1.1/D1.1M clauses 1.4.1 and 2.3.2
32. For "nontubular" applications, whether the structure is statically or cyclically loaded, per AWS D1.1/D1.1M clause 1.4.1
33. All additional requirements that are not specifically addressed in the welding code, per AWS D1.1/D1.1M clause 1.4.1

34. For original equipment manufacturer applications (see AWS D1.1/D1.1M clause 1.3.4), the responsibilities of the parties involved, per AWS D1.1/D1.1M clause 1.4.1
35. Designation of any welds that are required to be performed in the field, per AWS D1.1/D1.1M clause 2.3.1
36. Designation of joints where a specific assembly order, welding sequence, welding technique or other special precautions are required, per AWS D1.1/D1.1M clause 2.3.3
37. Details for special groove details, per AWS D1.1/D1.1M clause 2.3.5.5

Note: AWS D1.1/D1.1M also provides shop drawing requirements in clause 2.3.5.

Approvals Required

The following provisions require the approval of the owner's designated representative for design if they are to be used:

1. Bolted-joint-related approvals per RCSC *Specification* Commentary Section 1.6
2. Use of electronic or other copies of the design drawings by the fabricator, per AISC *Code of Standard Practice* Section 4.3
3. Use of stock materials not conforming to a specified ASTM specification, per AISC *Code of Standard Practice* Section 5.2.3
4. Correction of errors, per AISC *Code of Standard Practice* Section 7.14
5. Inspector-recommended deviations from contract documents, per AISC *Code of Standard Practice* Section 8.5.6
6. Contract price adjustment, per AISC *Code of Standard Practice* Section 9.4.2

Establishing Criteria for Connections

AISC *Code of Standard Practice* Section 3.1.1 provides the following three methods for the establishment of connection requirements.

In the first method, the complete design of all connections is shown in the structural design drawings. In this case, AISC *Code of Standard Practice* Commentary Section 3.1.1 provides a summary of the information that must be included in the structural design drawings. This method has the advantage that there is no need to provide connection loads, since the connections are completely designed in the structural design drawings. Additionally, it favors greater accuracy in the bidding process, since the connections are fully described in the contract documents.

In the second method, the fabricator is allowed to select or complete the connections while preparing the shop and erection drawings, using the information provided by the owner's designated representative for design per AISC *Code of Standard Practice* Section 3.1.1. In this case, AISC *Code of Standard Practice* Commentary Section 3.1.1 clarifies the intention that connections that can be selected or completed by the fabricator include those for which tables appear in the contract documents or this Manual. Other connections should be shown in detail in the structural design drawings.

In the third method, connections are designated in the contract documents to be designed by a licensed professional engineer working for the fabricator. The AISC *Code of Standard Practice* sets forth detailed provisions that, in the absence of contract provisions to the contrary, serve as the basis of the relationships among the parties. One feature of these provisions is that the fabricator is required to provide representative examples of connection design documentation early in the process, and the owner's designated representative for design is obliged is to review these submittals for conformity with the requirements of the contract documents. These early submittals are required in an attempt to avoid additional costs and/or delays as the approval process proceeds through subsequent shop drawings with connections developed from the original representative samples.

Methods one and two have the advantage that the fabricator's standard connections normally can be used, which often leads to project economy. However, the loads or other connection design criteria must be provided in the structural design drawings. Design loads and required strengths for connections should be provided in the structural design drawings and the design method used in the design of the frame (ASD or LRFD) must be indicated on the drawings.

In all three methods, the resulting shop and erection drawings must be submitted to the owner's designated representative for design for review and approval. As stated in the AISC *Code of Standard Practice* Section 4.4.1, the approval of shop and erection drawings constitutes "confirmation that the fabricator has correctly interpreted the contract documents" and that the reviewer has "reviewed and approved the connection details shown in the approval documents." Following is additional guidance for the communication of connection criteria to the connection designer.

Simple Shear Connections

The full force envelope should be given for each simple shear connection. Because of the potential for overestimation and underestimation inherent in approximate methods (Thornton, 1995), actual beam end reactions should be indicated on the design drawings. The most effective method to communicate this information is to place a numeric value at each end of each span in the framing plans.

In the past, beam end reactions were sometimes specified as a percentage of the uniform load tabulated in Part 3. This practice can result in either over- or under-specification of connection reactions and should not be used. The inappropriateness of this practice is illustrated in the following examples.

Overestimation:

1. When beams are selected for serviceability considerations or for shape repetition, the uniform load tables will often result in heavier connections than would be required by the actual design loads.
2. When beams have relatively short spans, the uniform load tables will often result in heavier connections than would be required by the actual design loads. If not addressed with the accurate load, many times the heavier connections will require extension of the connection below the bottom flange of the supported member, requiring that the flange on one or both sides of the web to be cut and chipped, a costly process.

Underestimation:

1. When beams support other framing beams or other concentrated loads occur on girders supporting beams, the end reactions can be higher than 50% of the total uniform load.
2. For composite beams, the end reactions can be higher than 50% of the total uniform load. The percentage requirement can be increased for this condition, but the resulting approach is still subject to the above considerations.

Moment Connections

The full force envelope should be given for each moment connection. If the owner's designated representative for design can select the governing load combination, its effect alone should be provided. Otherwise, the effects of all appropriate load combinations should be indicated. Additionally, the maximum moment imbalance should also be given for use in the check of panel-zone web shear.

Because of the potential for overestimation—and underestimation—inherent in approximate methods, it is recommended that the actual beam end reactions (moment, shear and other reactions, if any) be indicated in the structural design drawings. The most effective method to do so may be by tabulation for each joint and load combination.

Although not recommended, beam end reactions are sometimes specified by more general criteria, such as by function of the beam strength. It should be noted, however, that there are several situations in which this approach is not appropriate. For example:

1. When beams are selected for serviceability considerations or for shape repetition, this approach will often result in heavier connections than would be required by the actual design loads.
2. When the column(s) or other members that frame at the joint could not resist the forces and moments determined from the criteria so specified, this approach will often result in heavier connections than would be required by the actual design loads.

In some cases, the structural analysis may require that the actual connections be configured to match the assumptions used in the model. For example, it may be appropriate to release weak-axis moments in a beam-column joint where only strong-axis beam moment strength is required. Such requirements should be indicated in the structural design drawings.

Horizontal and Vertical Bracing Connections

The full force envelope should be given for each bracing-member end connection. If the owner's designated representative for design can select the governing load combination for the connection, its effect alone should be provided. Otherwise, the effects of all appropriate load combinations should be indicated in tabular form. This approach will allow a clear understanding of all of the forces on any given joint.

Because of the potential for overestimation—and underestimation—inherent in approximate methods, it is recommended that the actual reactions at the bracing member end (axial force and other reactions, if any) be indicated in the structural design drawings. It is also recommended that transfer forces, if any, be so indicated. The most effective method to do so may be by tabulation for each bracing member end and load combination.

Although not recommended, bracing member end reactions can be specified by more general criteria, such as by maximum member forces (tension or compression) or as a function of the member strength. It should be noted, however, that there are several situations in which such approaches are not appropriate. For example:

1. The specification of maximum member forces does not permit a check of the member forces at a joint if there are different load combinations governing the member designs at that joint. Nor does it reflect the possibility of load reversal as it may influence the design.
2. The specification of a percentage of member strength may not properly account for the interaction of forces at a joint or the transfer force through the joint. Additionally, it may not allow for a cross check of all forces at a joint.

In either case, this approach will often result in heavier connections than would be required by the actual design loads.

Bracing connections may involve the interaction of gravity and lateral loads on the frame. In some cases, such as V- and inverted V-bracing (also known as Chevron bracing), gravity loads alone may govern design of the braces and their connections. Thus, clarity in the specification of loads and reactions is critical to properly consider the potential interaction of gravity and lateral loads at floors and roofs.

Strut and Tie Connections

Floor and roof members in braced bays and adjacent bays may function as struts or ties in addition to carrying gravity loads. Therefore the recommendations for simple shear connections and bracing connections above apply in combination.

Truss Connections

The recommendations for horizontal and vertical bracing connections above also apply in general to bracing connections with the following additional comments.

Note that it is not necessary to specify a minimum connection strength as a percent of the member strength as a default. However, when trusses are shop assembled or field assembled on the ground for subsequent erection, consideration should be given to the loads that will be induced during handling, shipping and erection.

Column Splices

Column splices may resist moments, shears and tensions in addition to gravity forces. Typical column splices are discussed in Part 14. As in the case of the other connections discussed above, unless the column splices are fully designed in the construction documents, forces and moments for the splice designs should be provided in the construction documents. Since column splices are located away from the girder/column joint and moments vary in the height of the column, an accurate assessment of the forces and moments at the column splices will usually significantly reduce their cost and complexity.

CONSTRUCTABILITY

Constructability is a relatively new word for a well established idea. The design, detailing, fabrication and erection of structural steel is a process which in the end needs to result in a safe and economical steel frame. Building codes and the AISC *Specification* address strength and structural integrity. Constructability addresses the need for global economy in the fabricated and erected steel frame. Constructability must be “designed in,” influencing decision-making at all steps of the design process, from framing system selection, though member design, to connection selection and design. Constructability demands attention to detail and requires the designer to think ahead to the fabrication and erection of the steel frame. The goal is to design a steel frame that is relatively easy to detail, fabricate and erect. AISC provides guidance to the design community through its many publications and presentations, including Design Guide 23, *Constructability of Structural Steel Buildings* (Ruby, 2008).

Constructability focuses on such issues as framing layout, the number of pieces in an area of framing, three-dimensional connection geometry, swinging-in clearances, access to bolts, and access to welds. It involves the acknowledgement that numerous, seemingly small decisions can have an effect on the overall economy of the final erected steel frame. Fabricators and erectors have the knowledge that can assist in the design of constructible steel frames. Designers should seek their counsel.

TOLERANCES

The effects of mill, fabrication and erection tolerances all require consideration in the design and construction of structural steel buildings. However, the accumulation of the mill tolerances and fabrication tolerances shall not cause the erection tolerances to be exceeded, per AISC *Code of Standard Practice* Section 7.12.

Mill Tolerances

Mill tolerances are those variations that could be present in the product as-delivered from the rolling mill. These tolerances are given as follows:

1. For structural shapes and plates, see ASTM A6.
2. For HSS, see ASTM A500 (or other applicable ASTM specification for HSS).
3. For pipe, see ASTM A53.

A summary of standard mill practices is also given in Part 1.

Fabrication Tolerances

Fabrication tolerances are generally provided in AISC *Specification* Section M2 and AISC *Code of Standard Practice* Section 6.4. Additional requirements that govern fabrication are as follows:

1. Compression joint fit-up, per AISC *Specification* Section M4.4
2. Roughness limits for finished surfaces, per AISC *Code of Standard Practice* Section 6.2.2
3. Straightness of projecting elements of connection materials, per AISC *Code of Standard Practice* Section 6.3.1
4. Finishing requirements at locations of removal of run-off tabs and similar devices, per AISC *Code of Standard Practice* Section 6.3.2

Erection Tolerances

Erection tolerances are generally provided in AISC *Specification* Section M4 and AISC *Code of Standard Practice* Section 7.13. Note that the tolerances specified therein are predicated upon the proper installation of the following items by the owner's designated representative for construction:

1. Building lines and benchmarks, per AISC *Code of Standard Practice* Section 7.4
2. Anchorage devices, per AISC *Code of Standard Practice* Section 7.5
3. Bearing devices, per AISC *Code of Standard Practice* Section 7.6
4. Grout, per AISC *Code of Standard Practice* Section 7.7

Building Façade Tolerances

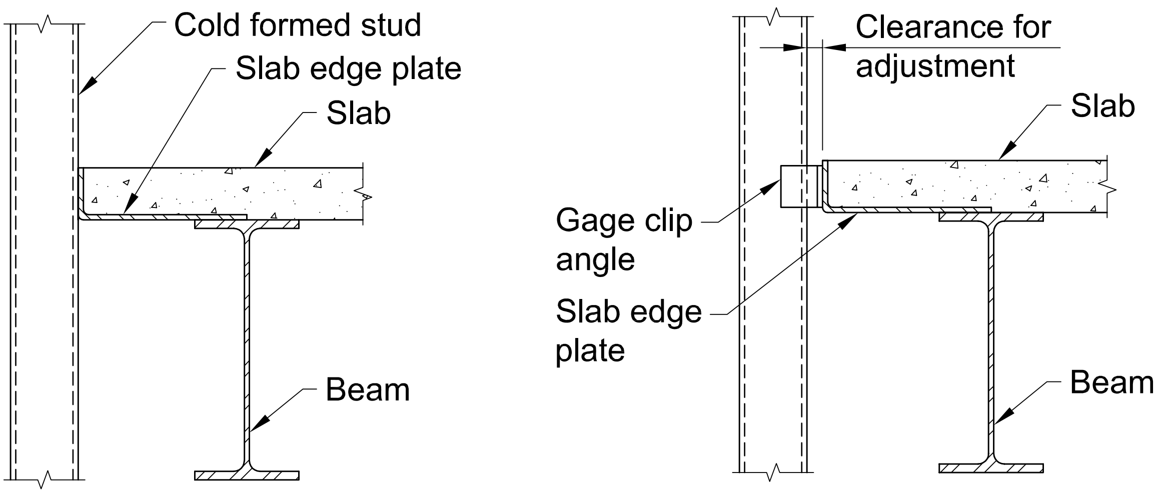
The preceding mill, fabrication and erection tolerances can be maintained with standard equipment and workmanship. However, the accumulated tolerances for the structural steel and the building façade must be accounted for in the design so that the two systems can be properly mated in the field. In the steel frame, this is normally accomplished by specifying adjustable connections in the contract documents, per AISC *Code of Standard Practice* Section 7.13.1.3. This section has three subsections. Subsection (a) addresses the vertical position of the adjustable items, subsection (b) addresses the horizontal position of the adjustable items, and subsection (c) addresses alignment of adjustable items at abutting ends.

The required adjustability normally can be determined from the range of adjustment in the building façade anchor connections, tolerances for the erection of the building façade, and the accumulation of mill, fabrication and erection tolerances at the mid-span point of the spandrel beam. The actual locations of the column bases, the actual slope of the columns, and the actual sweep of the spandrel beam all affect the accumulation of tolerances in the structural steel at this critical location. These conditions must be reflected in details that will allow successful erection of the steel frame and the façade, if each of these systems is properly constructed within its permitted tolerance envelope.

Figures 2-3(a), 2-4(a) and 2-5(a) illustrate details that are not recommended because they do not provide for adjustment. Figures 2-3(b), 2-4(b) and 2-5(b) illustrate recommended alternative details that do provide for adjustability. Note that diagonal structural and stability bracing elements have been omitted in these details to improve the clarity of presentation regarding adjustability. Also, note that all elements beyond the slab edge are normally not structural steel, per AISC *Code of Standard Practice* Section 2.2, and are shown for the purposes of illustration only.

The bolted details in Figures 2-4(b) and 2-5(b) can be used to provide field adjustability with slotted holes as shown. Further adjustability can be provided in these details, if necessary, by removing the bolts and clamping the connection elements for field welding. Alternatively, when the slab edge angle or plate in Figure 2-4(b) is shown as field welded and identified as adjustable in the contract documents, it can be provided to within a horizontal tolerance of $\pm 3/8$ in., per AISC *Code of Standard Practice* Section 7.13.1.3. However, if the item was not shown as field welded and identified as adjustable in the contract documents, it would likely be attached in the shop or attached in the field to facilitate the concrete pour and not be suitable to provide for the necessary adjustment. The details in Figures 2-3(b) and 2-4(b) do not readily permit vertical adjustment of the adjustable material. However, the vertical position tolerance of $\pm 3/8$ in. is less than the tolerance for the position of the spandrel member itself, see AISC *Code of Standard Practice* Section 7.13.1.2(b). The manufacturing tolerance for camber in the spandrel member is set by ASTM A6, as summarized in Table 1-22. The ASTM A6 limit for camber is $1/8$ in. per 10 ft of length, thus, in most situations the vertical position tolerance in AISC *Code of Standard Practice* Section 7.13.1.3(b) should be achieved indirectly. In general, spandrel members should not be cambered. Deflection of spandrel members should be controlled by member stiffness. Figure 2-5(b) shows a detail in which both horizontal and vertical adjustment can be achieved.

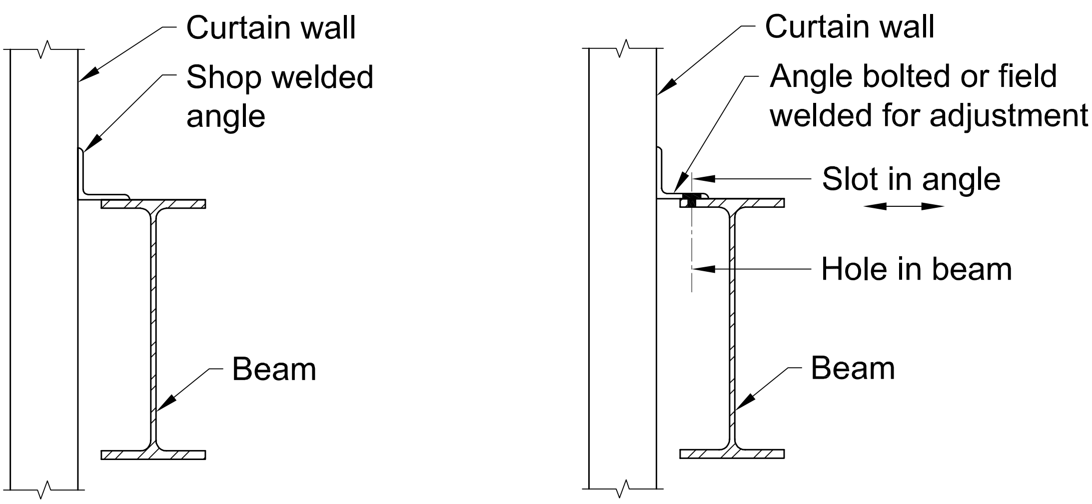
With adjustable connections specified in design and provided in fabrication, actions taken on the job site will allow for a successful façade installation. Per the AISC *Code of Standard Practice* definition of established column line (see *Code of Standard Practice* Glossary), proper placement of this line by the owner's designated representative for construction based upon the actual column-center locations will assure that all subcontractors are working from the same information. When sufficient adjustment cannot be accommodated within the adjustable connections provided, a common solution is to allow the building façade to deviate (or drift) from the theoretical location to follow the as-built locations of the structural steel framing and concrete floor slabs. A survey of the as-built locations of these elements can be used to adjust the placement of the building façade accordingly. In this case, the adjustable connections can serve to ensure that no abrupt changes occur in the façade. Building façade tolerances and other related issues are presented in detail in AISC Design Guide 22, *Façade Attachments to Steel-Framed Buildings* (Parker, 2008).



(a) Without adjustment (not recommended)

(b) With adjustment (recommended)

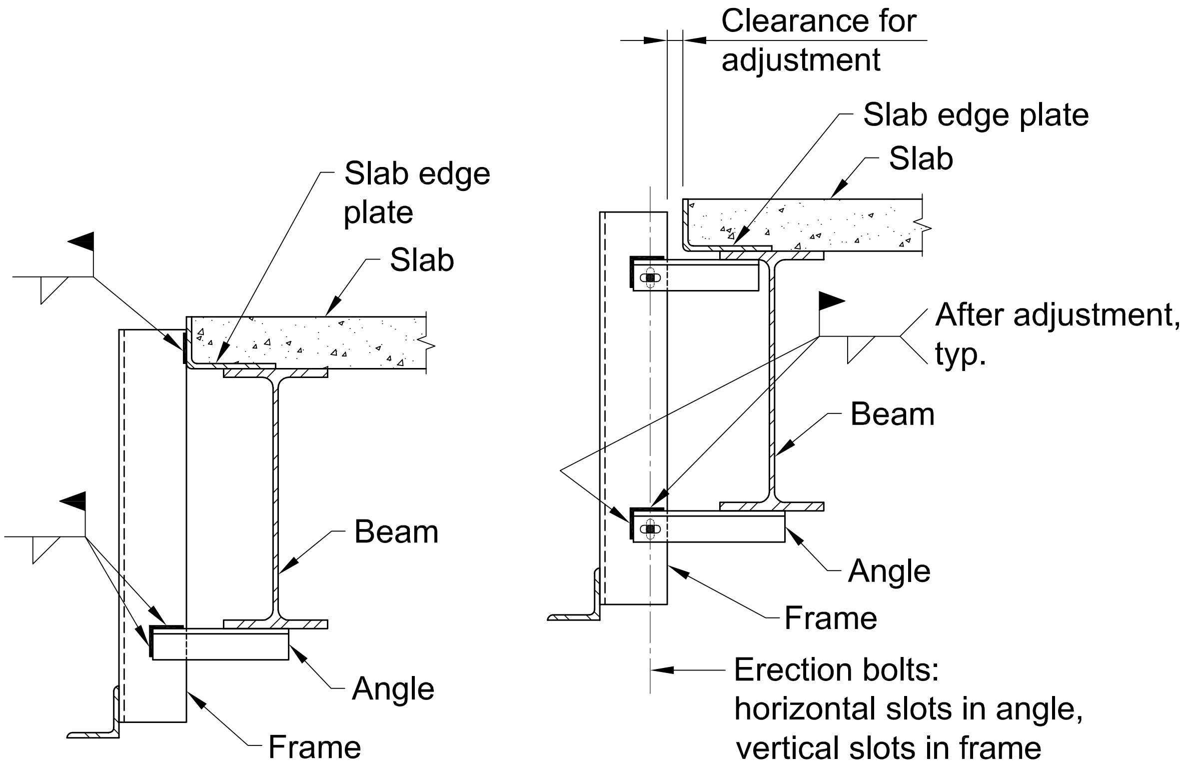
Fig. 2-3. Attaching cold-formed steel façade systems to structural steel framing.



(a) Without adjustment (not recommended)

(b) With adjustment (recommended)

Fig. 2-4. Attaching curtain wall façade systems to structural steel framing.



(a) Without adjustment (not recommended)

(b) With adjustment (recommended)

Fig. 2-5. Attaching masonry façade systems to structural steel framing.

QUALITY CONTROL AND QUALITY ASSURANCE

AISC *Specification* Chapter N addresses quality control and assurance. This chapter distinguishes between quality control, which is the responsibility of the fabricator and erector, and quality assurance, which is the responsibility of the owner, usually through third party inspectors. The new provisions bring together requirements from diverse sources of quality control (QC) and quality assurance (QA), so that plans for QC and QA can be established on a project-specific basis. Chapter N provides tabulated lists of inspection tasks for both QC and QA. As in the case of the AISC *Seismic Provisions*, these tasks are characterized as either "observe" or "perform." Tasks identified as "observe" are general and random. Tasks identified as "perform" are specific to the final acceptance of an item in the work. The characterization of tasks as observe and perform is a substitute for the distinction between periodic and continuous inspection used in other codes and standards, such as the *International Building Code*.

CAMBERING, CURVING AND STRAIGHTENING

Beam Camber and Sweep

Camber denotes a curve in the vertical plane. Sweep denotes a curve in the horizontal plane. Camber and sweep occur naturally in members as received from the mill. The deviation of the member from straight must be within the mill tolerances specified in ASTM A6/A6M.

When required by the contract documents, cambering and curving to a specified amount can be provided by the fabricator per AISC *Code of Standard Practice* Sections 6.4.2 and 6.4.4, either by cold bending or by hot bending.

Cambering and curving induce residual stresses similar to those that develop in rolled structural shapes as elements of the shape cool from the rolling temperature at different rates. These residual stresses do not affect the available strength of structural members, because the effect of residual stresses is considered in the provisions of the AISC *Specification*.

Cold Bending

The inelastic deformations required in common cold bending operations, such as for beam cambering, normally fall well short of the strain-hardening range. Specific limitations on cold-bending capabilities should be obtained from those that provide the service and from *Cold Bending of Wide-Flange Shapes for Construction* (Bjorhovde, 2006). However, the following general guidelines may be useful in the absence of other information:

1. The minimum radius for camber induced by cold bending in members up to a nominal depth of 30 in. is between 10 and 14 times the depth of the member. Deeper members may require a larger minimum radius.
2. A minimum length of 25 ft is commonly practical due to manufacturing/fabrication equipment.

When curvatures and the resulting inelastic deformations are significant and corrective measures are required, the effects of cold work on the strength and ductility of the structural steels largely can be eliminated by thermal stress relief or annealing.

Hot Bending

The controlled application of heat can be used in the shop and field to provide camber or curvature. The member is rapidly heated in selected areas that tend to expand, but are restrained by the adjacent cooler areas, causing inelastic deformations in the heated areas and a change in the shape of the cooled member.

The mechanical properties of steels are largely unaffected by such heating operations, provided the maximum temperature does not exceed the temperature limitations given in AISC *Specification* Section M2.1. Temperature-indicating crayons or other suitable means should be used during the heating process to ensure proper regulation of the temperature.

Heat curving induces residual stresses that are similar to those that develop in hot-rolled structural shapes as they cool from the rolling temperature because all parts of the shape do not cool at the same rate.

Truss Camber

Camber is provided in trusses, when required, by the fabricator per AISC *Code of Standard Practice* Section 6.4.5, by geometric relocation of panel points and adjustment of member lengths based upon the camber requirements as specified in the contract documents.

Straightening

All structural shapes are straightened at the mill after rolling, either by rotary or gag straightening, to meet the aforementioned mill tolerances. Similar processes and/or the controlled application of heat can be used in the shop or field to straighten a curved or distorted member. These processes are normally applied in a manner similar to those used to induce camber and curvature and described above.

FIRE PROTECTION AND ENGINEERING

Provisions for structural design for fire conditions are found in AISC *Specification* Appendix 4. Complete coverage of fire protection and engineering for steel structures is included in AISC Design Guide 19, *Fire Resistance of Structural Steel Framing* (Ruddy et al., 2003).

CORROSION PROTECTION

In building structures, corrosion protection is not required for steel that will be enclosed by building finish, coated with a contact-type fireproofing, or in contact with concrete. When enclosed, the steel is trapped in a controlled environment and the products required for corrosion are quickly exhausted, as indicated in AISC *Specification* Commentary Section M3. A similar situation exists when steel is fireproofed or in contact with concrete. Accordingly, shop primer or paint is not required unless specified in the contract documents, per AISC *Specification* Section M3.1. Per AISC *Code of Standard Practice* Section 6.5, steel that is to remain unpainted need only be cleaned of heavy deposits of oil and grease by appropriate means after fabrication.

Corrosion protection is required, however, in exterior exposed applications. Likewise, steel must be protected from corrosion in aggressively corrosive applications, such as a paper processing plant, a structure with oceanfront exposure, or when temperature changes can cause condensation. Corrosion should also be considered when connecting steel to dissimilar metals.

When surface preparation other than the cleaning described above is required, an appropriate grade of cleaning should be specified in the contract documents according to the Society for Protective Coatings (SSPC). A summary of the SSPC surface preparation standards (SSPC, 2014) is provided in Table 2-7. SSPC-SP 2 is the normal grade of cleaning when cleaning is required.

For further information, refer to the publications of SSPC, the American Galvanizers Association (AGA), and the National Association of Corrosion Engineers International (NACE). For corrosion protection of fasteners, see Part 7.

RENOVATION AND RETROFIT OF EXISTING STRUCTURES

The provisions in AISC *Specification* Section B7 govern the evaluation of existing structures. Historical data on available steel grades and hot-rolled structural shapes, including dimensions and properties, is available in AISC Design Guide 15, *Rehabilitation and Retrofit Guide—A Reference for Historic Shapes and Specifications* (Brockenbrough and Schuster, 2017), and the companion database of historic shape properties from 1873 to 1999 available at **www.aisc.org/manualresources**. See also Ricker (1988) and Tide (1990).

THERMAL EFFECTS

Expansion and Contraction

The average coefficient of expansion, ε, for structural steel between 70°F and 100°F is 0.0000065 for each °F (Camp et al., 1951). This value is a reasonable approximation of the coefficient of thermal expansion for temperatures less than 70°F. For temperatures from 100 to 1,200°F, the change in length per unit length per °F, ε, is:

$$\varepsilon = (6.1 + 0.0019t)10^{-6} \tag{2-7}$$

where t is the initial temperature in °F. The coefficients of expansion for other building materials can be found in Table 17-11.

Although buildings are typically constructed of flexible materials, expansion joints are often required in roofs and the supporting structure when horizontal dimensions are large. The maximum distance between expansion joints is dependent upon many variables, including ambient temperature during construction and the expected temperature range during the lifetime of the building.

Figure 2-6 (Federal Construction Council, 1974) provides guidance based on design temperature change for maximum spacing of structural expansion joints in beam-and-column-framed buildings with pinned column bases and heated interiors. The report includes data for numerous cities and gives five modification factors to be applied as appropriate:

1. If the building will be heated only and will have pinned column bases, use the maximum spacing as specified.
2. If the building will be air conditioned as well as heated, increase the maximum spacing by 15% provided the environmental control system will run continuously.
3. If the building will be unheated, decrease the maximum spacing by 33%.

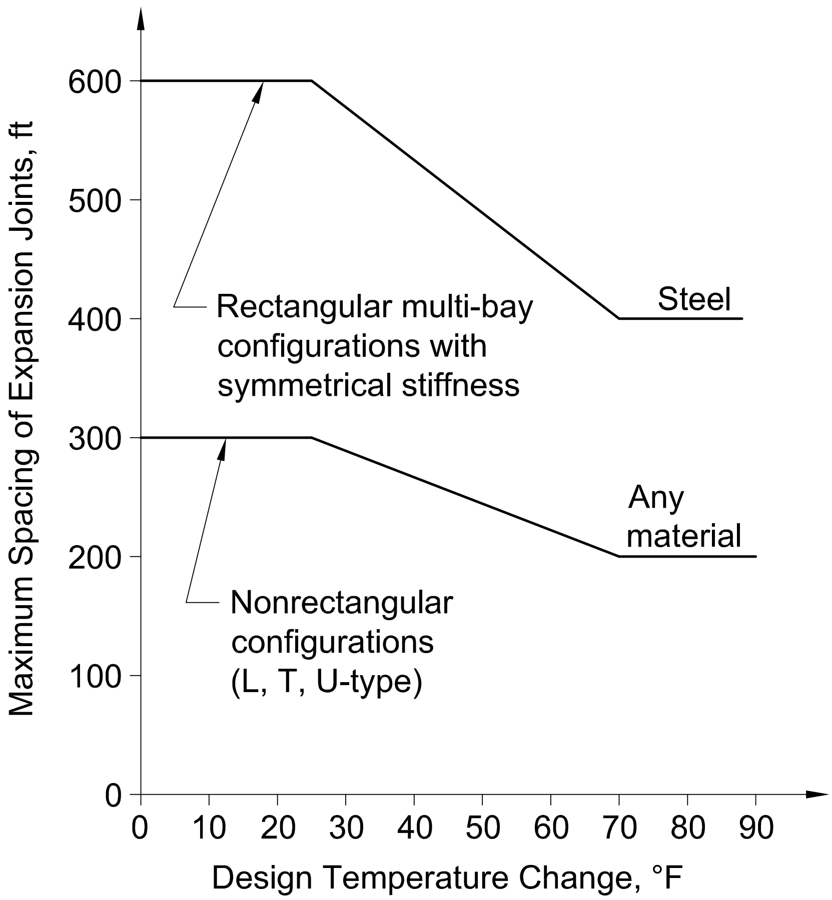


Fig. 2-6. Recommended maximum expansion joint spacing.

4. If the building will have fixed column bases, decrease the maximum spacing by 15%.
5. If the building will have substantially greater stiffness against lateral displacement in one of the plan dimensions, decrease the maximum spacing by 25%.

When more than one of these design conditions prevail in a building, the percentile factor to be applied is the algebraic sum of the adjustment factors of all the various applicable conditions. Most building codes include restrictions on location and maximum spacing of fire walls, which often become default locations for expansion joints.

The most effective expansion joint is a double line of columns that provides a complete and positive separation. Alternatively, low-friction sliding elements can be used. Such systems, however, are seldom totally friction-free and will induce some level of inherent restraint to movement.

Elevated-Temperature Service

For applications involving short-duration loading at elevated temperature, the variations in yield strength, tensile strength, and modulus of elasticity are given in AISC Design Guide 19, *Fire Resistance of Structural Steel Framing* (Ruddy et al., 2003). For applications involving long-duration loading at elevated temperatures, the effects of creep must also be considered. For further information, see Brockenbrough and Merritt (2011).

FATIGUE AND FRACTURE CONTROL

Avoiding Brittle Fracture

By definition, brittle fracture occurs by cleavage at a stress level below the yield strength. Generally, a brittle fracture can occur when there is a sufficiently adverse combination of tensile stress, temperature, strain rate and geometrical discontinuity (notch). The exact combination of these conditions and other factors that will cause brittle fracture cannot be readily calculated. Consequently, the best guide in selecting steel material that is appropriate for a given application is experience.

The steels listed in AISC *Specification* Section A3.1a, have been successfully used in a great number of applications, including buildings, bridges, transmission towers and transportation equipment, even at the lowest atmospheric temperatures encountered in the United States. Nonetheless, it is desirable to minimize the conditions that tend to cause brittle fracture: triaxial state-of-stress, increased strain rate, strain aging, stress risers, welding residual stresses, areas of reduced notch toughness, and low-temperature service.

1. Triaxial state-of-stress: While shear stresses are always present in a uniaxial or biaxial state-of-stress, the maximum shear stress approaches zero as the principal stresses approach a common value in a triaxial state-of-stress. A triaxial state-of-stress can also result from uniaxial loading when notches or geometrical discontinuities are present. A triaxial state-of-stress will cause the yield stress of the material to increase above its nominal value, resulting in brittle fracture by cleavage, rather than ductile shear deformations. As a result, in the absence of critical-size notches, the maximum stress is limited by the yield stress of the nearby unaffected material. Triaxial stress conditions should be avoided, when possible.
2. Increased strain rate: Gravity loads, wind loads and seismic loads have essentially similar strain rates. Impact loads, such as those associated with heavy cranes, and blast

loads normally have increased strain rates, which tend to increase the possibility of brittle fracture. Note, however, that a rapid strain rate or impact load is not a required condition for the occurrence of brittle fracture.

3. Strain aging: Cold working of steel and the strain aging that normally results generally increases the likelihood of brittle fracture, usually due to a reduction in ductility and notch toughness. The effects of cold work and strain aging can be minimized by selecting a generous forming radius to eliminate or minimize strain hardening.
4. Stress risers: Fabrication operations, such as flame cutting and welding, may induce geometric conditions or discontinuities that are crack-like in nature, creating stress risers. Intersecting welds from multiple directions should be avoided with properly sized weld access holes to minimize the interaction of these various stress fields. Such conditions should be avoided, when possible, or removed or repaired when they occur.
5. Welding residual stresses: In the as-welded condition, residual stresses near the yield strength of the material will be present in any weldment. Residual stresses and the possible accompanying distortions can be minimized through controlled welding procedures and fabrication methods, including the proper positioning of the components of the joint prior to welding, the selection of welding sequences that will minimize distortions, the use of preheat as appropriate, the deposition of a minimum volume of weld metal with a minimum number of passes for the design condition, and proper control of interpass temperatures and cooling rates. In fracture-sensitive applications, notch-toughness should be specified for both the base metal and the filler metal.
6. Areas of reduced notch toughness: Such areas can be found in the core areas of heavy shapes and plates and the *k*-area of rotary-straightened W-shapes. Accordingly, AISC *Specification* Sections A3.1c and Section A3.1d include special requirements for material notch toughness.
7. Low-temperature service: While steel yield strength, tensile strength, modulus of elasticity, and fatigue strength increase as temperature decreases, ductility and toughness decrease. Furthermore, there is a temperature below which steel subjected to tensile stress may fracture by cleavage, with little or no plastic deformation, rather than by shear, which is usually preceded by considerable inelastic deformation. Note that cleavage and shear are used in the metallurgical sense to denote different fracture mechanisms.

When notch-toughness is important, Charpy V-notch testing can be specified to ensure a certain level of energy absorption at a given temperature, such as 15 ft-lb at 70°F. Note that the appropriate test temperature may be higher than the lowest operating temperature depending upon the rate of loading. Although it is primarily intended for bridge-related applications, the information in ASTM A709 Section 10 (including Tables 9 and 10) may be useful in determining the proper level of notch toughness that should be specified.

In many cases, weld metal notch toughness exceeds that of the base metal. Filler metals can be selected to meet a desired minimum notch-toughness value. For each welding process, electrodes exist that have no specified notch toughness requirements. Such electrodes should not be assumed to possess any minimum notch-toughness value. When notch toughness is necessary for a given application, the desired value or an appropriate electrode should be specified in the contract documents.

For further information, refer to Fisher et al. (1998), Barsom and Rolfe (1999), and Rolfe (1977).

Avoiding Lamellar Tearing

Although lamellar tearing is less common today, the restraint against solidified weld deposit contraction inherent in some joint configurations can impose a tensile strain high enough to cause separation or tearing on planes parallel to the rolled surface of the element being joined. The incidence of this phenomenon can be reduced or eliminated through greater understanding by designers, detailers and fabricators of the inherent directionality of rolled steel, the importance of strains associated with solidified weld deposit contraction in the presence of high restraint (rather than externally applied design forces), and the need to adopt appropriate joint and welding details and procedures with proper weld metal for through-thickness connections.

Dexter and Melendrez (2000) demonstrate that W-shapes are not susceptible to lamellar tearing or other through-thickness failures when welded tee joints are made to the flanges at locations away from member ends. When needed for other conditions, special production practices can be specified for steel plates to assist in reducing the incidence of lamellar tearing by enhancing through-thickness ductility. For further information, refer to ASTM A770. However, it must be recognized that it is more important and effective to properly design, detail and fabricate to avoid highly restrained joints. AISC (1973) provides guidelines that minimize potential problems.

WIND AND SEISMIC DESIGN

In general, nearly all building design and construction can be classified into one of two categories: wind and low-seismic applications, and high-seismic applications. For additional discussion regarding seismic design and the applicability of the AISC *Seismic Provisions*, see the Scope statement at the front of this manual.

Wind and Low-Seismic Applications

Wind and low-seismic applications are those in which the AISC *Seismic Provisions* are not applicable. Such buildings are designed to meet the provisions in the AISC *Specification* based upon the code-specified forces distributed throughout the framing assuming a nominally elastic structural response. The resulting systems have normal levels of ductility. It is important to note that the applicable building code includes seismic design requirements even if the AISC *Seismic Provisions* are not applicable. See the AISC *Seismic Design Manual* for additional discussion.

High-Seismic Applications

High-seismic applications are those in which the building is designed to meet the provisions in both the AISC *Seismic Provisions* and the AISC *Specification*. Note that it does not matter if wind or earthquake controls in this case. High-seismic design and construction will generally cost more than wind and low-seismic design and construction, as the resulting systems are designed to have high levels of ductility.

High-seismic lateral framing systems are configured to be capable of withstanding strong ground motions as they undergo controlled ductile deformations to dissipate energy. Consider the following three examples:

1. Special Concentrically Braced Frames (SCBF)—SCBF are generally configured so that any inelasticity will occur by tension yielding and/or compression buckling in the braces. The connections of the braces to the columns and beams and between the columns and beams themselves must then be proportioned to remain nominally elastic as they undergo these deformations.
2. Eccentrically Braced Frames (EBF)—EBF are generally configured so that any inelasticity will occur by shear yielding and/or flexural yielding in the link. The beam outside the link, connections, braces and columns must then be proportioned to remain nominally elastic as they undergo these deformations.
3. Special Moment Frames (SMF)—SMF are generally configured so that any inelasticity will occur by flexural yielding in the girders near, but away from, the connection of the girders to the columns. The connections of the girders to the columns and the columns themselves must then be proportioned to remain nominally elastic as they undergo these deformations. Intermediate moment frames (IMF) and ordinary moment frames (OMF) are also configured to provide improved seismic performance, although successively lower than that for SMF.

The code-specified base accelerations used to calculate the seismic forces are not necessarily maximums, but rather, they represent the intensity of ground motions that have been selected by the code-writing authorities as reasonable for design purposes. Accordingly, the requirements in both the AISC *Seismic Provisions* and the AISC *Specification* must be met so that the resulting frames can then undergo controlled deformations in a ductile, well-distributed manner.

The design provisions for high-seismic systems are also intended to result in distributed deformations throughout the frame, rather than the formation of story mechanisms, so as to increase the level of available energy dissipation and corresponding level of ground motion that can be withstood.

The member sizes in high-seismic frames will be larger than those in wind and low-seismic frames. The connections will also be much more robust so they can transmit the member-strength-driven force demands. Net sections will often require special attention so as to avoid having fracture limit states control. Special material requirements, design considerations and construction practices must be followed. For further information on the design and construction of high-seismic systems, see the AISC *Seismic Provisions*.

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Table 2-1
Multipliers for Use With the Simplified Method

Design Story Drift Limit	Load Ratio from Step 3 (times 1.6 for ASD, 1.0 for LRFD)										
	0	5	10	20	30	40	50	60	80	100	120
H/100	1	1.1	1.1	1.3	**1.5**/1.4	When ratio exceeds 1.5, simplified method requires a stiffer structure.					
H/200	1	1	1.1	1.1	1.2	1.3	**1.4**/1.3	**1.5**/1.4			
H/300	1	1	1	1.1	1.1	1.2	1.2	1.3	**1.5**/1.4		
H/400	1	1	1	1.1	1.1	1.1	1.2	1.2	1.3	**1.4**/1.3	1.5
H/500	1	1	1	1	1.1	1.1	1.1	1.2	1.2	1.3	1.4

Note: Where two values are provided, the value in bold is the value associated with $R_M = 0.85$.
Interpolation between values in this table may produce an incorrect result.

Table 2-2
Summary Comparison of Methods for Stability Analysis and Design

	Direct Analysis Method	Effective Length Method	First-Order Analysis Method
Limitations on Use[a]	None	$\Delta_{2nd}/\Delta_{1st} \le 1.5$	$\Delta_{2nd}/\Delta_{1st} \le 1.5$ $\alpha P_r/P_y \le 0.5$
Analysis Type	Second-order elastic[b]		First-order elastic
Geometry of Structure	All three methods use the undeformed geometry in the analysis.		
Minimum or Additional Lateral Loads Required in the Analysis	Minimum;[c] 0.2% of the story gravity load	Minimum; 0.2% of the story gravity load	Additive; at least 0.42% of the story gravity load
Member Stiffnesses Used in the Analysis	Reduced *EA* and *EI*	Nominal *EA* and *EI*	
Design of Columns	$K = 1$ for all frames	$K = 1$ for braced frames. For moment frames, determine K from sidesway buckling analysis.[d]	$K = 1$ for all frames[e]
Specification Reference for Method	Chapter C	Appendix 7, Section 7.2	Appendix 7, Section 7.3

[a] $\Delta_{2nd}/\Delta_{1st}$ is the ratio of second-order drift to first-order drift, which can be taken to be equal to B_2 calculated per Appendix 8. $\Delta_{2nd}/\Delta_{1st}$ is determined using LRFD load combinations or a multiple of 1.6 times ASD load combinations.
[b] Either a general second-order analysis method or second-order analysis by amplified first-order analysis (the "B_1-B_2 method" described in Appendix 8) can be used.
[c] This notional load is additive if $\Delta_{2nd}/\Delta_{1st} > 1.5$.
[d] $K = 1$ is permitted for moment frames when $\Delta_{2nd}/\Delta_{1st} \le 1.1$.
[e] An additional amplification for member curvature effects is required for columns in moment frames.

Table 2-3
AISI Standard Nomenclature for Flat-Rolled Carbon Steel

Thickness, in.	Width, in.					
	To 3½ incl.	Over 3½ To 6	Over 6 To 8	Over 8 To 12	Over 12 To 48	Over 48
0.2300 & thicker	Bar	Bar	Bar	Plate	Plate	Plate
0.2299 to 0.2031	Bar	Bar	Strip	Strip	Sheet	Plate
0.2030 to 0.1800	Strip	Strip	Strip	Strip	Sheet	Plate
0.1799 to 0.0449	Strip	Strip	Strip	Strip	Sheet	Sheet
0.0448 to 0.0344	Strip	Strip				
0.0343 to 0.0255	Strip		Hot-rolled sheet and strip not generally produced in these widths and thicknesses			
0.0254 & thinner						

Table 2-4
Applicable ASTM Specifications for Various Structural Shapes

Steel Type	ASTM Designation		F_y Yield Stress[a] (ksi)	F_u Tensile Stress[a] (ksi)	Applicable Shape Series: W	M	S	HP	C	MC	L	HSS Rect.	HSS Round	Pipe
Carbon	A36		36	58–80[b]	▒	■	■	▒	■	■	■			
	A53 Gr. B		35	60										■
	A500	Gr. B	42	58									▒	
			46	58								▒		
		Gr. C	46	62									■	
			50	62								■		
	A501	Gr. A	36	58								▒	▒	
		Gr. B	50	70								▒	▒	
	A529[c]	Gr. 50	50	65–100	▒	▒	▒	▒	▒	▒	▒			
		Gr. 55	55	70–100	▒	▒	▒	▒	▒	▒	▒			
	A709	36	36	58–80[b]	▒	▒	▒	▒	▒	▒	▒			
	A1043[d,k]	36	36–52	58	▒									
		50	50–65	65	▒									
	A1085	Gr. A	50	65								▒	▒	
High-Strength Low-Alloy	A572	Gr. 42	42	60	▒	▒	▒	▒	▒	▒	▒			
		Gr. 50	50	65	▒	▒	▒	■	▒	▒	▒			
		Gr. 55	55	70	▒	▒	▒	▒	▒	▒	▒			
		Gr. 60[e]	60	75	▒	▒	▒	▒	▒	▒	▒			
		Gr. 65[e]	65	80	▒	▒	▒	▒	▒	▒	▒			
	A618[f]	Gr. Ia[k], Ib & II	50[g]	70[g]								▒	▒	
		Gr. III	50	65								▒	▒	
	A709	50	50	65	▒	▒	▒	▒	▒	▒	▒			
		50S	50–65	65	▒	▒	▒	▒	▒	▒	▒			
		50W	50	70	▒	▒	▒	▒	▒	▒	▒			
	A913	50	50[h]	65[h]	▒	▒	▒	▒	▒	▒	▒			
		60	60	75	▒	▒	▒	▒	▒	▒	▒			
		65	65	80	▒	▒	▒	▒	▒	▒	▒			
		70	70	90	▒	▒	▒	▒	▒	▒	▒			
	A992		50[i]	65[i]	■	▒	▒	▒	▒	▒	▒			
	A1065[k]	Gr. 50[j]	50	60								▒		

■ = Preferred material specification.
▒ = Other applicable material specification, the availability of which should be confirmed prior to specification.
□ = Material specification does not apply.

Footnotes on facing page.

Table 2-4 (continued)
Applicable ASTM Specifications for Various Structural Shapes

Steel Type	ASTM Designation		F_y Yield Stress[a] (ksi)	F_u Tensile Stress[a] (ksi)	Applicable Shape Series									
					W	M	S	HP	C	MC	L	HSS Rect.	HSS Round	Pipe
Corrosion Resistant High-Strength Low-Alloy	A588		50	70										
	A847[k]		50	70										
	A1065[k]	Gr. 50W[j]	50	70										

■ = Preferred material specification.
▢ (grey) = Other applicable material specification, the availability of which should be confirmed prior to specification.
□ = Material specification does not apply.

[a] Minimum, unless a range is shown.
[b] For wide-flange shapes with flange thicknesses over 3 in., only the minimum of 58 ksi applies.
[c] For shapes with a flange or leg thickness less than or equal to 1½ in. only. To improve weldability, a maximum carbon equivalent can be specified (per ASTM A529 Supplementary Requirement S78). If desired, maximum tensile stress of 90 ksi can be specified (per ASTM A529 Supplementary Requirement S79).
[d] For shape profiles with a flange width of 6 in. or greater.
[e] For shapes with a flange thickness less than or equal to 2 in. only.
[f] ASTM A618 can also be specified as corrosion-resistant; see ASTM A618.
[g] Minimum applies for walls nominally ¾ in. thick and under. For wall thickness over ¾ in., F_y = 46 ksi and F_u = 67 ksi.
[h] If desired, maximum yield stress of 65 ksi and maximum yield-to-tensile strength ratio of 0.85 can be specified (per ASTM A913 Supplementary Requirement S75).
[i] A maximum yield-to-tensile strength ratio of 0.85 and carbon equivalent formula are included as mandatory, and some variation is allowed, including for shapes tested with coupons cut from the web; see ASTM A992. If desired, maximum tensile stress of 90 ksi can be specified (per ASTM A992 Supplementary Requirement S79).
[j] The grades of ASTM A1065 may not be interchanged without approval of the purchaser.
[k] This specification is not a prequalified base metal per AWS D1.1/D1.1M:2015.

Table 2-5
Applicable ASTM Specifications for Plates and Bars

Steel Type	ASTM Designation		F_y Yield Stress[a] (ksi)	F_u Tensile Stress[a] (ksi)	Plates and Bars, in.									
					to 0.75 incl.	over 0.75 to 1.25 incl.	over 1.25 to 1.5 incl.	over 1.5 to 2 incl.	over 2 to 2.5 incl.	over 2.5 to 4 incl.	over 4 to 5 incl.	over 5 to 6 incl.	over 6 to 8 incl.	over 8
Carbon	A36		32	58–80										■
			36	58–80	■	■	■	■	■	■	■	■	■	
	A283[e]	Gr. C	30	55–75	▒	▒	▒	▒	d					
		Gr. D	33	60–80	▒	▒	▒	▒	d					
	A529	Gr. 50	50	65–100	▒	▒ b	▒ b	▒ b	▒ b	▒ b				
		Gr. 55	55	70–100	▒	▒ c	▒ c	▒ c	▒ c	▒ c				
	A709	Gr. 36	36	58–80	▒	▒	▒	▒	▒	▒				
High-Strength Low-Alloy	A572	Gr. 42	42	60	▒	▒	▒	▒	▒	▒	▒	▒		
		Gr. 50	50	65	■	■	■	■	■	■				
		Gr. 55	55	70	▒	▒	▒	▒						
		Gr. 60	60	75	▒	▒								
		Gr. 65	65	80	▒	▒								
	A709	Gr. 50	50	65	▒	▒	▒	▒	▒	▒				
	A1043[e]	Gr. 36	36–52	58	▒	▒	▒	▒	▒	▒	▒			
		Gr. 50	50–65	65	▒	▒	▒	▒	▒	▒	▒			
	A1066[e]	Gr. 50	50	65	▒	▒	▒	▒	▒	▒				
		Gr. 60	60	75	▒	▒	▒	▒	▒	▒				
		Gr. 65	65	80	▒	▒	▒	▒	▒	▒ f				
		Gr. 70	70	85	▒	▒	▒	▒						
		Gr. 80	80	90	▒	▒ g								
Corrosion Resistant High-Strength Low-Alloy	A242[e]		42	63				▒	▒	▒				
			46	67		▒	▒							
			50	70	▒									
	A588		42[e]	63								▒	▒	
			46[e]	67							▒			
			50	70	▒	▒	▒	▒	▒	▒				

■ = Preferred material specification.
▒ = Other applicable material specification, the availability of which should be confirmed prior to specification.
□ = Material specification does not apply.

Footnotes on facing page.

Table 2-5 (continued)
Applicable ASTM Specifications for Plates and Bars

Steel Type	ASTM Designation		F_y Yield Stress[a] (ksi)	F_u Tensile Stress[a] (ksi)	Plates and Bars, in. to 0.75 incl.	over 0.75 to 1.25 incl.	over 1.25 to 1.5 incl.	over 1.5 to 2 incl.	over 2 to 2.5 incl.	over 2.5 to 4 incl.	over 4 to 5 incl.	over 5 to 6 incl.	over 6 to 8 incl.	over 8
Quenched and Tempered Alloy	A514[e]		90	100–130	□	□	□	□	□	▒	▒	▒	□	□
			100	110–130	▒	▒	▒	▒	▒	□	□	□	□	□
Corrosion Resistant Quenched and Tempered Low-Alloy	A709	Gr. 50W	50	70	▒	▒	▒	▒	▒	▒	□	□	□	□
		Gr. HPS 50W	50	70	▒	▒	▒	▒	▒	▒	□	□	□	□
		Gr. HPS 70W	70	85–110	▒	▒	▒	▒	▒	▒	□	□	□	□
		Gr. HPS 100W[e]	90	100–130	□	□	□	□	□	▒	□	□	□	□
			100	110–130	▒	▒	▒	▒	▒	□	□	□	□	□

■ = Preferred material specification.
▒ = Other applicable material specification, the availability of which should be confirmed prior to specification.
□ = Material specification does not apply.

[a] Minimum, unless a range is shown.
[b] Applicable for plates to 1 in. thickness and bars to 3½ in. thickness.
[c] Applicable for plates to 1 in. thickness and bars to 3 in. thickness.
[d] Thickness is not limited to 2 in. in ASTM A283 and thicker plates may be obtained but availability should be confirmed.
[e] This specification is not a prequalified base metal per AWS D1.1/D1.1M:2015.
[f] Applicable for plates to 3 in. thickness.
[g] Applicable for plates to 1 in. thickness.

Table 2-6
Applicable ASTM Specifications for Various Types of Structural Fasteners

ASTM Designation		F_y Min. Yield Stress (ksi)	F_u Tensile Stress[a] (ksi)	Diameter Range (in.)	Bolts: High-Strength: Conventional	Bolts: High-Strength: Twist-Off-Type Tension-Control	Bolts: Common Bolts	Nuts	Washers: Hardened	Washers: Plain	Washers: Direct-Tension Indicator	Threaded Rods	Anchor Rods: Hooked	Anchor Rods: Headed	Anchor Rods: Threaded & Nutted
F3125	Gr. A325[d]	–	120	0.5 to 1.5	■										
F3125	Gr. F1852[d]	–	120	0.5 to 1.25		■									
F3125	Gr. A490[d]	–	150	0.5 to 1.5	■										
F3125	Gr. F2280[d]	–	150	0.5 to 1.25		■									
F3111		–	200	1 to 1.25 incl.	▒										
F3043		–	200	1 to 1.25 incl.		▒									
A194 Gr. 2H		–	–	0.25 to 4				▒							
A563		–	–	0.25 to 4				■							
F436		–	–	0.25 to 4[b]					■						
F844		–	–	any						■					
F959		–	–	0.5 to 1.5							■				
A36		36	58–80	to 10								■	▒	▒	▒
A193 Gr. B7		105	125	2.5 and under								▒	▒	▒	▒
A193 Gr. B7		95	115	over 2.5 to 4								▒	▒	▒	▒
A193 Gr. B7		75	100	over 4 to 7								▒	▒	▒	▒
A307 Gr. A		–	60	0.25 to 4			■					▒	▒	▒	▒
A354	Gr. BC	109	125	0.25 to 2.5 incl.	▒ e							▒ e	▒	▒	▒
A354	Gr. BC	99	115	over 2.5 to 4 incl.	▒ e							▒ e	▒	▒	▒
A354	Gr. BD	130	150	0.25 to 2.5 incl.	▒ e							▒ e	▒	▒	▒
A354	Gr. BD	115	140	2.5 to 4 incl.	▒ e							▒ e	▒	▒	▒
A449[d]		92	120	0.25 to 1 incl.	▒ e							▒ e	▒	▒	▒
A449[d]		81	105	over 1 to 1.5 incl.	▒ e							▒ e	▒	▒	▒
A449[d]		58	90	over 1.5 to 3 incl.	▒ e							▒ e	▒	▒	▒
A572	Gr. 42	42	60	to 6								▒	▒		▒
A572	Gr. 50	50	65	to 4[c]								▒	▒		▒
A572	Gr. 55	55	70	to 2								▒	▒		▒
A572	Gr. 60	60	75	to 3.5								▒	▒		▒
A572	Gr. 65	65	80	to 1.25								▒	▒		▒
A588		50	70	4 and under								▒	▒		▒
A588		46	67	over 4 to 5 incl.								▒	▒		▒
A588		42	63	over 5 to 8 incl.								▒	▒		▒
F1554	Gr. 36	36	58–80	0.25 to 4								▒	■	■	■
F1554	Gr. 55	55	75–95	0.25 to 4								▒	■	■	■
F1554	Gr. 105	105	125–150	0.25 to 3								▒	▒	▒	▒

■ = Preferred material specification.
▒ = Other applicable material specification, the availability of which should be confirmed prior to specification.
□ = Material specification does not apply.

– Indicates that a value is not specified in the material specification.
[a] Minimum, unless a range is shown.
[b] Diameter range is 2 in. to 12 in. for beveled and extra thick washers.
[c] ASTM A572 permits rod diameters up to 11 in., but practicality of threading should be confirmed before specification.
[d] When atmospheric corrosion resistance is desired, Type 3 can be specified.
[e] See AISC *Specification* Section J3.1 for limitations on use of ASTM A449, A354 Gr. BC and A354 Gr. BD.

Table 2-7
Summary of Surface Preparation Standards

SSPC Standard No.	Title	Description
SSPC-SP 1	Solvent Cleaning	Removal of all visible oil, grease, dirt, soil, salts and contaminants by cleaning with solvent, vapor, alkali, emulsion or steam.
SSPC-SP 2	Hand-Tool Cleaning	Removal of loose rust, loose mill scale, and loose paint by hand chipping, scraping, sanding and wire brushing.
SSPC-SP 3	Power-Tool Cleaning	Removal of all loose rust, loose mill scale, and loose paint by power tool chipping, descaling, sanding, wire brushing, and grinding.
SSPC-SP 5/ NACE No. 1*	White Metal Blast Cleaning	Removal of all visible rust, mill scale, paint and foreign matter by blast cleaning by wheel or nozzle (dry or wet) using sand, grit or shot; for very corrosive atmospheres where high cost of cleaning is warranted.
SSPC-SP 6/ NACE No. 3*	Commercial Blast Cleaning	Removal of all visible rust, mill scale, paint and foreign matter by blast cleaning; staining is permitted on no more than 33% of each 9 in.^{2} area of the cleaned surface; for conditions where a thoroughly cleaned surface is required.
SSPC-SP 7/ NACE No. 4*	Brush-Off Blast Cleaning	Blast cleaning of all except tightly adhering residues of mill scale, rust and coatings while uniformly roughening the surface.
SSPC-SP 8	Pickling	Complete removal of rust and mill scale by acid pickling, duplex pickling or electrolytic pickling.
SSPC-SP 10/ NACE No. 2*	Near-White Blast Cleaning	Removal of all visible rust, mill scale, paint and foreign matter by blast cleaning; staining is permitted on no more than 5% of each 9 in.^{2} area of the cleaned surface; for high humidity, chemical atmosphere, marine, or other corrosive environments.
SSPC-SP 11	Power-Tool Cleaning to Bare Metal	Complete removal of all visible oil, grease, coatings, rust, corrosion products, mill scale, and other foreign matter by power tools, with resultant minimum surface profile of 1 mil; trace amounts of coating and corrosion products may remain in the bottom of pits if the substrate was pitted prior to cleaning.
SSPC-SP 14/ NACE No. 8*	Industrial Blast Cleaning	Between SP 7 (brush-off) and SP 6 (commercial); the intent is to remove as much coating as possible; tightly adhering contaminants can remain on no more than 10% of each 9 in.^{2} area of the cleaned surface.
SSPC-SP 15	Commercial-Grade Power-Tool Cleaning	Between SP 3 and SP 11; complete removal of all visible oil, grease, dirt, rust, coating, mill scale, corrosion products, and other foreign matter by power tools with resultant minimum surface profile of 1 mil; random staining is limited to no more than 33% of each 9 in.^{2} of surface; trace amounts of coating and corrosion products may remain in the bottom of pits if the substrate was pitted prior to cleaning.
SSPC-SP 16	Brush-Off Blast Cleaning of Coated and Uncoated Galvanized Steel, Stainless Steel, and Non-Ferrous Metals	Requirements for removing loose contaminants and coating from coated and uncoated galvanized steel, stainless steels, and non-ferrous metals; cleaned surface is free of all visible oil, grease, dirt, dust, metal oxides (corrosion products), and other foreign matter; requires a minimum 19 μm (0.75 mil) profile on bare metal substrate.

* Standards are issued as joint standards by SSPC and NACE International.

Table 2-7 (continued)
Summary of Surface Preparation Standards

SSPC Standard No.	Title	Description
SSPC-SP WJ-1/ NACE WJ-1*	Waterjet Cleaning of Metals—Clean to Bare Substrate	When viewed without magnification, the metal surface shall have a matte (dull, mottled) finish and shall be free of all visible oil, grease, dirt, rust, and other corrosion products, previous coatings, mill scale, and foreign matter.
SSPC-SP WJ-2/ NACE WJ-2*	Waterjet Cleaning of Metals—Very Thorough Cleaning	When viewed without magnification, the metal surface shall have a matte (dull, mottled) finish and shall be free of all visible oil, grease, dirt, rust, and other corrosion products, except for randomly dispersed stains of rust and other corrosion products, tightly adherent thin coatings, and other tightly adherent foreign matter. The staining or tightly adherent matter shall be limited to no more than 5% of each 9 in.^{2} area of the cleaned surface.
SSPC-SP WJ-3/ NACE WJ-3*	Waterjet Cleaning of Metals—Thorough Cleaning	When viewed without magnification, the metal surface shall have a matte (dull, mottled) finish and shall be free of all visible oil, grease, dirt, rust, and other corrosion products, except for randomly dispersed stains of rust and other corrosion products, tightly adherent thin coatings, and other tightly adherent foreign matter. The staining or tightly adherent matter shall be limited to no more than 5% of each 9 in.^{2} area of the cleaned surface.
SSPC-SP WJ-4/ NACE WJ-4*	Waterjet Cleaning of Metals—Light Cleaning	When viewed without magnification, the metal surface shall be free of all visible oil, grease, dirt, dust, loose mill scale, loose rust and other corrosion products, and loose coating. Any residual material shall be tightly adhered to the metal substrate and may consist of randomly dispersed stains of rust and other corrosion products or previously applied coating, tightly adherent thin coatings, and other tightly adherent foreign matter.
SSPC-PA17	Conformance to Profile/Surface Roughness/Peak Count Requirements	A procedure suitable for shop or field use for determining compliance with specified profile ranges on a steel substrate.

* Standards are issued as joint standards by SSPC and NACE International.

PART 3

DESIGN OF FLEXURAL MEMBERS

SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of flexural members subject to uniaxial flexure without axial forces or torsion. For the design of members subject to biaxial flexure and/or flexure in combination with axial tension or compression and/or torsion, see Part 6.

SECTION PROPERTIES AND AREAS

For Flexure

Flexural design properties are based upon the full cross section with no reduction for bolt holes.

For Shear

For shear, the area is determined per AISC *Specification* Chapter G.

FLEXURAL STRENGTH

The nominal flexural strength of W-shapes is illustrated as a function of the unbraced length, L_b, in Figure 3-1. The available strength is determined as ϕM_n or M_n/Ω, which must equal or exceed the required strength (bending moment), M_u or M_a, respectively. The available flexural strength, ϕM_n or M_n/Ω, is determined per AISC *Specification* Chapter F. Table User Note F1.1 outlines the sections of Chapter F and the corresponding limit states applicable to each member type.

Braced, Compact Flexural Members

When flexural members are braced ($L_b \le L_p$) and compact ($\lambda \le \lambda_p$), yielding must be considered in the nominal moment strength of the member, in accordance with the requirements of AISC *Specification* Chapter F.

Unbraced Flexural Members

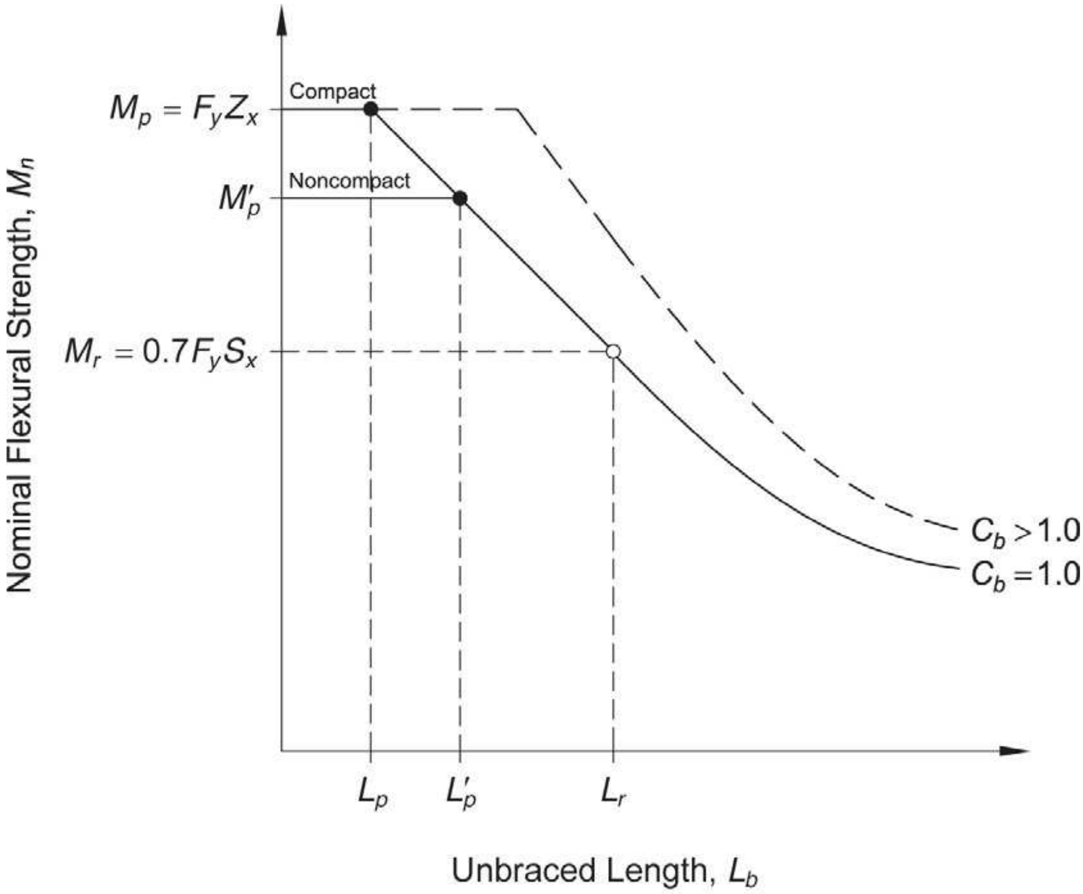
When flexural members are unbraced ($L_b > L_p$), have flange width-to-thickness ratios such that $\lambda > \lambda_p$, or have web width-to-thickness ratios such that $\lambda > \lambda_p$, lateral-torsional buckling and elastic buckling effects must be considered in the calculation of the nominal moment strength of the member.

Noncompact or Slender Cross Sections

For flexural members that have width-to-thickness ratios such that $\lambda > \lambda_p$, local buckling must be considered in the calculation of the nominal moment strength of the member.

Available Flexural Strength for Minor Axis Bending

The design of flexural members subject to minor axis bending is similar to that for major axis bending, except that lateral-torsional buckling and web local buckling do not apply. See AISC *Specification* Section F6.



$$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} \quad \text{(}\textit{Spec.}\text{ Eq. F2-5)}$$

$$L_r = 1.95 r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{S_x h_o} + \sqrt{\left(\frac{Jc}{S_x h_o}\right)^2 + 6.76\left(\frac{0.7F_y}{E}\right)^2}} \quad \text{(}\textit{Spec.}\text{ Eq. F2-6)}$$

$$M_r = 0.7F_y S_x \quad (3\text{-}1)$$

For cross sections with noncompact flanges:

$$M'_p = M_n = M_p - \left(M_p - 0.7F_y S_x\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \quad \text{(from }\textit{Spec.}\text{ Eq. F3-1)}$$

$$L'_p = L_p + \left(L_r - L_p\right)\frac{\left(M_p - M'_p\right)}{\left(M_p - M_r\right)} \quad (3\text{-}2)$$

Fig. 3-1. General available flexural strength of beams.

Use of Table 6-2 for Flexural Design of Beams

Table 6-2 may be used for flexural design of beams bent about either the major or minor axis. This table includes all W-shapes, not just those most commonly used as beams. Compact and noncompact section criteria from AISC *Specification* Chapter B have been incorporated in the development of the table. Therefore, no check of the width-to-thickness ratio of the compression elements of the cross section is necessary.

Available strengths from Table 6-2 may be used for flexural design of beams bent about their major axis over a range of unbraced lengths including $L_b > L_r$. The table already accounts for comparison of the unbraced lengths relative to L_p and L_r for the shapes listed in the table. The table also lists available strengths for bending about the minor axis. See the discussion in Part 6 for more information on use of Table 6-2 for design for flexure.

LOCAL BUCKLING

Determining the Width-to-Thickness Ratios of the Cross Section

Flexural members are classified for flexure on the basis of the width-to-thickness ratios of the various elements of the cross section. The width-to-thickness ratio, λ, is determined for each element of the cross section per AISC *Specification* Section B4.1. Limiting width-to-thickness ratios for various values of F_y may be found in Table 6-1b.

Classification of Cross Sections

Cross sections are classified as follows:

- Flexural members are compact (the plastic moment can be reached without local buckling) when λ is equal to or less than λ_p and the flange(s) are continuously connected to the web(s).
- Flexural members are noncompact (local buckling will occur, but only after initial yielding) when λ exceeds λ_p but is equal to or less than λ_r.
- Flexural members are slender-element cross sections (local buckling will occur prior to yielding) when λ exceeds λ_r.

The values of λ_p and λ_r are determined per AISC *Specification* Section B4.1.

LATERAL-TORSIONAL BUCKLING

Classification of Spans for Flexure

Flexural members bent about their major axis are classified on the basis of the length between braced points, L_b. Braced points are points at which support resistance against lateral-torsional buckling is provided per AISC *Specification* Appendix 6, Section 6.3. Classifications are determined as follows:

- If $L_b \le L_p$, flexural member is not subject to lateral-torsional buckling.
- If $L_p < L_b \le L_r$, flexural member is subject to inelastic lateral-torsional buckling.
- If $L_b > L_r$, flexural member is subject to elastic lateral-torsional buckling.

The values of L_p and L_r are determined per AISC *Specification* Chapter F. These values are presented in Tables 3-2, 3-6, 3-7, 3-8, 3-9, 3-10, 3-11 and 6-2. Note that for cross sections with noncompact flanges, the value given for L_p in these tables is L'_p as given in Equation 3-2 of Figure 3-1. In Tables 3-10 and 3-11, L_p is defined by • and L_r by ∘.

Lateral-torsional buckling does not apply to flexural members bent about their minor axis or round HSS bent about any axis, per AISC *Specification* Sections F6, F7 and F8.

Consideration of Moment Gradient

When $L_b > L_p$, the moment gradient between braced points can be considered in the determination of the available strength using the lateral-torsional buckling modification factor, C_b, herein referred to as the LTB modification factor. In the case of a uniform moment between braced points causing single-curvature of the member, $C_b = 1.0$. This represents the worst case and C_b can be conservatively taken equal to 1.0 for use with the maximum moment between braced points in most designs. See AISC *Specification* Commentary Section F1 for further discussion. A nonuniform moment gradient between braced points can be considered using C_b calculated as given in AISC *Specification* Equation F1-1. Exceptions are provided as follows:

1. As an alternative, when the moment diagram between braced points is a straight line, C_b can be calculated as given in AISC *Specification* Commentary Equation C-F1-1.
2. For cantilevers or overhangs where warping is prevented at the support and where the free end is unbraced, $C_b = 1.0$ per AISC *Specification* Section F1.
3. For tees with the stem in compression, $C_b = 1.0$ as recommended in AISC *Specification* Commentary Section F9.

AVAILABLE SHEAR STRENGTH

For flexural members, the available shear strength, ϕV_n or V_n/Ω, which must equal or exceed the required strength, V_u or V_a, respectively, is determined in accordance with AISC *Specification* Chapter G. Values of ϕV_n and V_n/Ω can be found in Tables 3-2, 3-6, 3-7, 3-8, 3-9, 3-16, 3-17 and 6-2.

STEEL W-SHAPE COMPOSITE BEAMS

The following pertains to W-shapes that act compositely with concrete slabs in regions of positive moment. For composite flexural members in regions of negative moment, see AISC *Specification* Chapter I. For further information on composite design and construction, see Viest et al. (1997).

Concrete Slab Effective Width

The effective width of a concrete slab acting compositely with a steel beam is determined per AISC *Specification* Section I3.1a.

Steel Anchors

Material, placement and spacing requirements for steel anchors are given in AISC *Specification* Chapter I. The nominal shear strength, Q_n, of one steel headed stud anchor is determined per AISC *Specification* Section I8.2a and is tabulated for common design conditions in Table 3-21. The horizontal shear strength, V', at the steel-concrete interface will be the least of the concrete crushing strength, steel section tensile yield strength, or the shear strength of the steel anchors. Table 3-21 considers only the limit state of shear strength of a steel headed stud anchor.

Available Flexural Strength for Positive Moment

The available flexural strength of a composite beam subject to positive moment is determined per AISC *Specification* Section I3.2a assuming a uniform compressive stress of $0.85f_c'$ and zero tensile strength in the concrete, and a uniform stress of F_y in the tension area (and compression area, if any) of the steel section. The position of the plastic neutral axis (PNA) can then be determined by static equilibrium.

Per AISC *Specification* Section I3.2d, enough steel anchors must be provided between a point of maximum moment and the nearest point of zero moment to transfer the total horizontal shear force, V', between the steel beam and concrete slab, where V' is determined per AISC *Specification* Section I3.2d.1.

Shored and Unshored Construction

The available flexural strength is identical for both shored and unshored construction. In unshored construction, issues such as lateral support during construction and construction-load deflection may require consideration.

Available Shear Strength

Per AISC *Specification* Section I4, the available shear strength for composite beams is determined in accordance with Chapter G.

OTHER SPECIFICATION REQUIREMENTS AND DESIGN CONSIDERATIONS

The following other specification requirements and design considerations apply to the design of flexural members.

Special Requirements for Heavy Shapes and Plates

For beams with complete-joint-penetration groove welded joints and made from heavy shapes with a flange thickness exceeding 2 in., see AISC *Specification* Section A3.1c.

For built-up sections consisting of plates with a thickness exceeding 2 in., see AISC *Specification* Section A3.1d.

Serviceability

Serviceability requirements, per AISC *Specification* Chapter L, should be appropriate for the application. This includes an appropriate limit on the deflection of the flexural member and the vibration characteristics of the system of which the flexural member is a part. See also AISC Design Guide 3, *Serviceability Design Considerations for Steel Buildings* (West et al., 2003), AISC Design Guide 5, *Low- and Medium-Rise Steel Buildings* (Allison, 1991), and AISC Design Guide 11, *Vibrations of Steel-Framed Structural Systems Due to Human Activity* (Murray et al., 2016).

The maximum vertical deflection, Δ, can be calculated using the equations given in Tables 3-22 and 3-23. Alternatively, for common cases of simple-span beams and I-shaped members and channels, the following equation can be used:

$$\Delta = ML^2/(C_1 I_x) \tag{3-3}$$

where

C_1 = loading constant (see Figure 3-2), which includes the numerical constants appropriate for the given loading pattern, E (29,000 ksi), and a ft-to-in. conversion factor of 1,728 in.^{3}/ft^3

I_x = moment of inertia, in.^{4}

L = span length, ft

M = maximum service-load moment, kip-ft

DESIGN TABLE DISCUSSION

Flexural Design Tables

Tabulated values account for element slenderness effects.

Table 3-1. Values of C_b for Simply Supported Beams

Values of the LTB modification factor, C_b, are given for various loading conditions on simply supported beams in Table 3-1.

W-Shape Selection Tables

Table 3-2. W-Shapes—Selection by Z_x

W-shapes are sorted in descending order by major axis flexural strength and then grouped in ascending order by weight with the lightest W-shape in each range in bold. Major axis

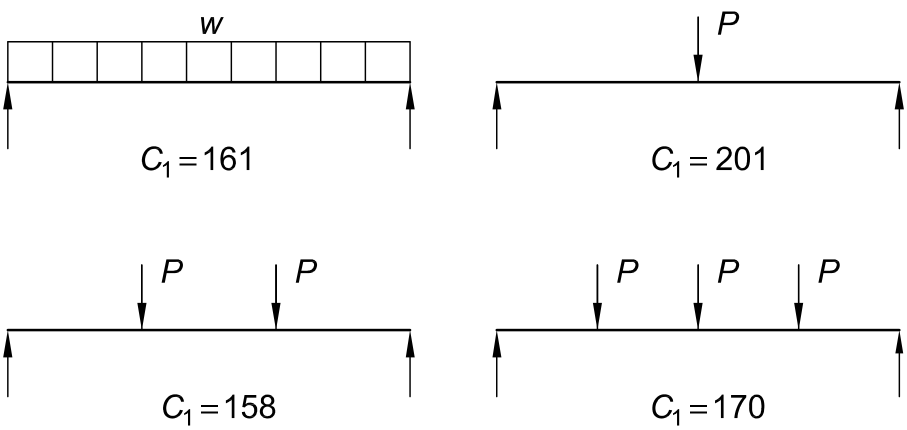


Fig. 3-2. Loading constants for use in determining simple beam deflections.

available strengths in flexure and shear are given for W-shapes with F_y = 50 ksi (ASTM A992). C_b is taken as unity.

For compact W-shapes, when $L_b \le L_p$, the major axis available flexural strength, $\phi_b M_{px}$ or M_{px}/Ω_b, can be determined using the tabulated strength values. When $L_p < L_b \le L_r$, linearly interpolate between the available strength at L_p and the available strength at L_r as follows:

LRFD	ASD
$\phi_b M_n = C_b\,[\phi_b M_{px} - \phi_b BF(L_b - L_p)] \le \phi_b M_{px}$ (3-4a)	$\dfrac{M_n}{\Omega_b} = C_b \left[\dfrac{M_{px}}{\Omega_b} - \dfrac{BF}{\Omega_b}(L_b - L_p)\right] \le \dfrac{M_{px}}{\Omega_b}$ (3-4b)

where

$$BF = \frac{\left(M_{px} - M_{rx}\right)}{\left(L_r - L_p\right)} \tag{3-5}$$

L_p = for compact sections, see Figure 3-1, AISC *Specification* Equation F2-5
= for noncompact sections, $L_p = L'_p$, see Figure 3-1, Equation 3-2

L_r = see Figure 3-1, AISC *Specification* Equation F2-6

$M_{px} = F_y Z_x$ for compact sections (*Spec*. Eq. F2-1)
= M'_p as given in Figure 3-1, from AISC *Specification* Equation F3-1, for noncompact sections

$M_{rx} = M_r = 0.7F_yS_x$ (3-1)

ϕ_b = 0.90

Ω_b = 1.67

When $L_b > L_r$, see Table 3-10.

The major axis available shear strength, $\phi_v V_{nx}$ or V_{nx}/Ω_v, can be determined using the tabulated value.

Table 3-3. W-Shapes—Selection by I_x

W-shapes are sorted in descending order by major axis moment of inertia, I_x, and then grouped in ascending order by weight with the lightest W-shape in each range in bold.

Table 3-4. W-Shapes—Selection by Z_y

W-shapes are sorted in descending order by minor axis flexural strength and then grouped in ascending order by weight with the lightest W-shape in each range in bold. Minor axis available strengths in flexure are given for W-shapes with F_y = 50 ksi (ASTM A992).

The minor axis available shear strength must be checked independently.

Table 3-5. W-Shapes—Selection by I_y

W-shapes are sorted in descending order by minor axis moment of inertia, I_y, and then grouped in ascending order by weight with the lightest W-shape in each range in bold.

Maximum Total Uniform Load Tables

Table 3-6. W-Shapes—Maximum Total Uniform Load

Maximum total uniform loads on braced ($L_b \leq L_p$) simple-span beams bent about the major axis are given for W-shapes with F_y = 50 ksi (ASTM A992). These tables include W-shapes that are most commonly used in flexure. The uniform load constant, $\phi_b W_c$ or W_c/Ω_b (kip-ft), divided by the span length, L (ft), provides the maximum total uniform load (kips) for a braced simple-span beam bent about the major axis. This is based on the available flexural strength as discussed for Table 3-2. Values are provided up to an arbitrary span-to-depth ratio of 30.

The major axis available shear strength, $\phi_v V_n$ or V_n/Ω_v, can be determined using the tabulated value. Above the heavy horizontal line in the tables, the maximum total uniform load is limited by the major axis available shear strength.

The tabulated values can also be used for braced simple-span beams with equal concentrated loads spaced as shown in Table 3-22a if the concentrated loads are first converted to an equivalent uniform load.

Table 3-7. S-Shapes—Maximum Total Uniform Load

Table 3-7 is similar to Table 3-6, except it covers S-shapes with F_y = 36 ksi (ASTM A36).

Table 3-8. C-Shapes—Maximum Total Uniform Load

Table 3-8 is similar to Table 3-6, except it covers C-shapes with F_y = 36 ksi (ASTM A36).

Table 3-9. MC-Shapes—Maximum Total Uniform Load

Table 3-9 is similar to Table 3-6, except it covers MC-shapes with F_y = 36 ksi (ASTM A36).

Plots of Available Flexural Strength vs. Unbraced Length

Table 3-10. W-Shapes—Plots of Available Moment vs. Unbraced Length

The major axis available flexural strength, $\phi_b M_n$ or M_n/Ω_b, is plotted as a function of the unbraced length, L_b, for W-shapes with F_y = 50 ksi (ASTM A992). The plots show the available strength for an unbraced length, L_b. The moment demand due to all applicable load combinations on that segment may not exceed the strength shown for L_b. C_b is taken as unity.

When the plotted curve is solid, the W-shape for that curve is the lightest cross section for a given combination of available flexural strength and unbraced length. When the plotted curve is dashed, a lighter W-shape than that for the plotted curve exists. The plotted curves are arbitrarily terminated at a span-to-depth ratio of 30 in most cases.

L_p is indicated in each curve by a solid dot (•). L_r is indicated in each curve by an open dot (◦).

Table 3-11. C- and MC-Shapes—Plots of Available Moment vs. Unbraced Length

Table 3-11 is similar to Table 3-10, except it covers C- and MC-shapes with F_y = 36 ksi (ASTM A36).

Available Flexural Strength of HSS

Table 3-12. Rectangular HSS—Available Flexural Strength

The available flexural strength is tabulated for rectangular HSS with F_y = 50 ksi (ASTM A500 Grade C) as determined by AISC *Specification* Section F7.

Table 3-13. Square HSS—Available Flexural Strength

Table 3-13 is similar to Table 3-12, except it covers square HSS with F_y = 50 ksi (ASTM A500 Grade C).

Table 3-14. Round HSS—Available Flexural Strength

Table 3-14 is similar to Table 3-12, except it covers round HSS with F_y = 46 ksi (ASTM A500 Grade C) and the available flexural strength is determined from AISC *Specification* Section F8.

Table 3-15. Pipe—Available Flexural Strength

Table 3-15 is similar to Table 3-14, except it covers HSS produced to a Pipe specification with F_y = 35 ksi (ASTM A53 Grade B).

Strength of Other Flexural Members

Tables 3-16 and 3-17. Available Shear Stress

The available shear stress for plate girders is plotted as a function of a/h and h/t_w in Tables 3-16 (for F_y = 36 ksi) and 3-17 (for F_y = 50 ksi). In Table 3-16a and Table 3-17a, tension field action is not included. In parts b and c of each table, tension field action is considered. Available strength obtained from Tables 3-16b, 3-16c, 3-17b or 3-17c (tension field action included) may be less than the available strength obtained from Table 3-16a or 3-17a (tension field action not included). In such cases, the larger strength may be used.

Table 3-18. Floor Plates

The recommended maximum uniformly distributed loads are given in Table 3-18 based upon simple-span bending between supports. Table 3-18a is for deflection-controlled applications and should be used with the appropriate serviceability load combinations. The tabulated values correspond to a maximum deflection of $L/100$. Table 3-18b is for flexural-strength-controlled applications and should be used with LRFD or ASD load combinations. The tabulated values correspond to a maximum bending stress of 24 ksi in LRFD and 16 ksi in ASD.

Composite Beam Selection Tables

Table 3-19. Composite W-Shapes

The available flexural strength is tabulated for W-shapes with F_y = 50 ksi (ASTM A992). The values tabulated are independent of the specific concrete flange properties allowing the designer to select an appropriate combination of concrete strength and slab geometry.

The location of the plastic neutral axis (PNA) is uniquely determined by the horizontal shear force, ΣQ_n, at the interface between the steel section and the concrete slab. With the knowledge of the location of the PNA and the distance to the centroid of the concrete flange force, ΣQ_n, the available flexural strength can be computed.

Available flexural strengths are tabulated for PNA locations at the seven locations shown. Five of these PNA locations are in the beam flange. The seventh PNA location is computed at the point where ΣQ_n equals $0.25F_yA_s$, and the sixth PNA location is halfway between the location of ΣQ_n at point five and point seven. A minimum degree of composite action of 25% has traditionally been used in the design of composite beams. This traditional minimum value alone may not provide enough ductility (slip capacity) at the beam/concrete interface. AISC *Specification* Commentary Section I3.2d provides guidance for consideration of ductility.

Table 3-19 can be used to design a composite beam by entering with a required flexural strength and determining the corresponding required ΣQ_n. Alternatively, Table 3-19 can be used to check the flexural strength of a composite beam by selecting a valid value of ΣQ_n, using Table 3-21. With the effective width of the concrete flange, b, determined per AISC *Specification* Section I3.1a, the appropriate value of the distance from concrete flange force to beam top flange, $Y2$, can be determined as

$$Y2 = Y_{con} - \frac{a}{2} \tag{3-6}$$

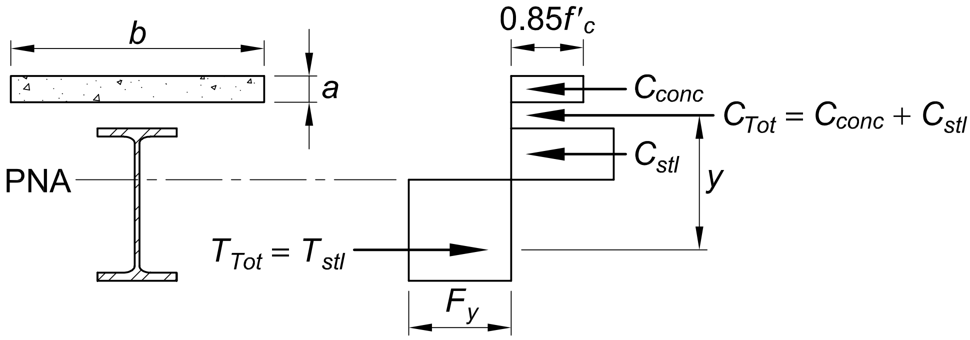
where

Y_{con} = distance from top of steel beam to top of concrete, in.

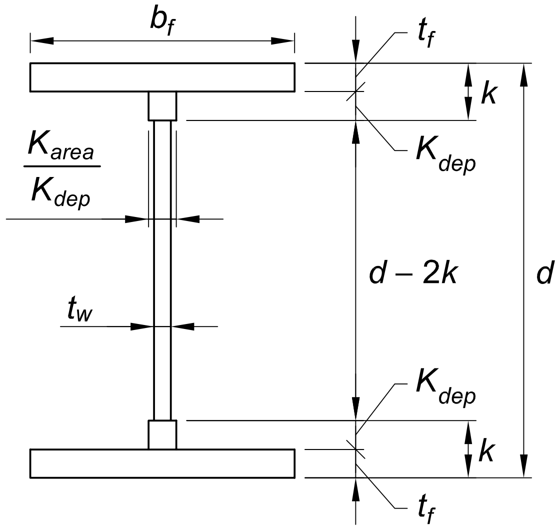
$$a = \frac{\Sigma Q_n}{0.85 f_c' b} \tag{3-7}$$

and the available flexural strength, $\phi_b M_n$ or M_n/Ω_b, can then be determined from Table 3-19. Values for the distance from the PNA to the beam top flange, $Y1$, are also tabulated for convenience. The parameters $Y1$ and $Y2$ are illustrated in Figure 3-3. Note that the model of the steel beam used in the calculation of the available strength assumes that

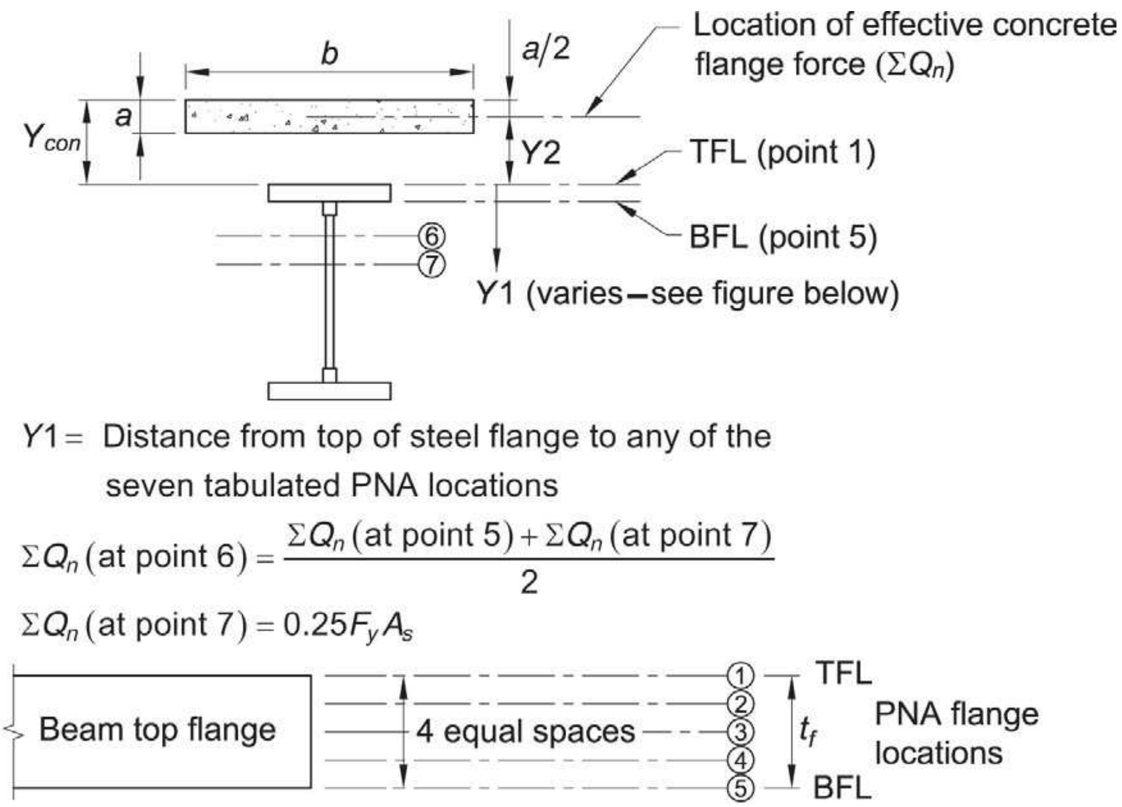
A_s = cross-sectional area of the steel section, in.^{2}
A_f = flange area, in.^{2} = $b_f t_f$
A_w = web area, in.^{2} = $(d - 2k)t_w$
K_{area} = $(A_s - 2A_f - A_w)/2$, in.^{2}
K_{dep} = $k - t_f$, in.



(a)



(b)



(c)

Fig. 3-3. Strength design models for composite beams.

Table 3-20. Lower-Bound Elastic Moment of Inertia

The lower-bound elastic moment of inertia of a composite beam can be used to calculate deflection. If calculated deflections using the lower-bound moment of inertia are acceptable, a more complete elastic analysis of the composite section can be avoided. The lower-bound elastic moment of inertia is based upon the area of the beam and an equivalent concrete area equal to $\Sigma Q_n / F_y$ as illustrated in Figure 3-4, where F_y = 50 ksi. The analysis includes only the horizontal shear force transferred by the steel anchors supplied. Thus, only the portion of the concrete flange used to balance ΣQ_n is included in the determination of the lower-bound moment of inertia.

The lower bound moment of inertia, therefore, is the moment of inertia of the cross section at the required strength level. This is smaller than the corresponding moment of inertia at the service load where deflection is calculated. The value for the lower bound moment of inertia can be calculated as illustrated in AISC *Specification* Commentary Section I3.2.

Table 3-21. Nominal Horizontal Shear Strength for One Steel Headed Stud Anchor, Q_n

The nominal shear strength of steel headed stud anchors is given in Table 3-21, in accordance with AISC *Specification* Chapter I. Nominal horizontal shear strength values are presented based upon the position of the steel anchor, profile of the deck, and orientation of the deck relative to the steel anchor. See AISC *Specification* Commentary Figure C-I8.1.

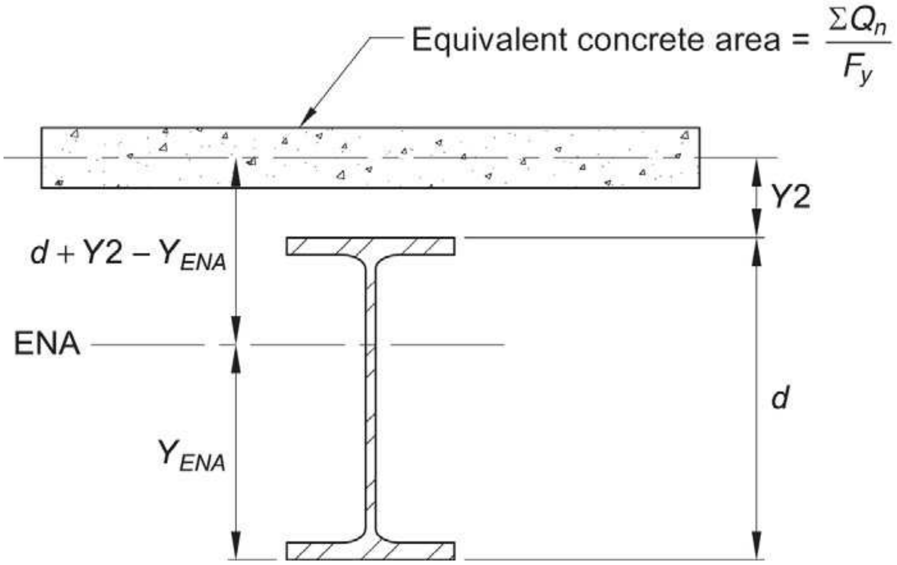


Fig. 3-4. Deflection design model for composite beams.

Beam Diagrams and Formulas

Table 3-22a. Concentrated Load Equivalents

Concentrated load equivalents are given in Table 3-22a for beams with various support conditions and loading characteristics.

Table 3-22b. Cantilevered Beams

Coefficients are provided in Table 3-22b for cantilevered beams with various support conditions and loading characteristics.

Table 3-22c. Continuous Beams

Coefficients are provided in Table 3-22c for continuous beams with various support conditions and loading characteristics.

Table 3-23. Shears, Moments and Deflections

Shears, moments and deflections are given in Table 3-23 for beams with various support conditions and loading characteristics.

PART 3 REFERENCES

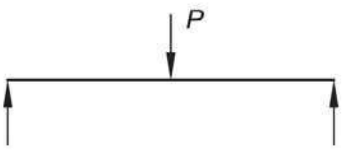
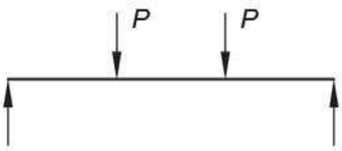
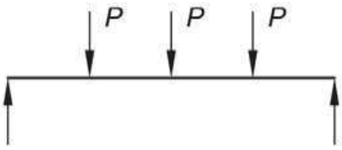
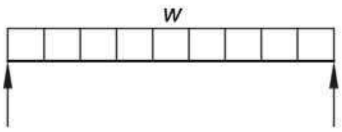
Allison, H.R. (1991), *Low- and Medium-Rise Steel Buildings*, Design Guide 5, AISC, Chicago, IL.

Murray, T.M., Allen, D.E., Ungar, E.E. and Davis, D.B. (2016), *Vibrations of Steel-Framed Structural Systems Due to Human Activity*, Design Guide 11, 2nd Ed., AISC, Chicago, IL.

Viest, I.M., Colaco, J.P., Furlong, R.W., Griffis, L.G., Leon, R.T. and Wyllie, L.A., Jr. (1997), *Composite Construction: Design for Buildings*, McGraw-Hill, New York, NY.

West, M.A., Fisher, J.M. and Griffis, L.G. (2003), *Serviceability Design Considerations for Steel Buildings*, Design Guide 3, 2nd Ed., AISC, Chicago, IL.

Table 3-1
Values of C_b for Simply Supported Beams

Load	Lateral Bracing Along Span	C_b
	None Load at midpoint	1.32
	At load point	1.67 1.67
	None Loads at third points	1.14
	At load points Loads symmetrically placed	1.67 1.00 1.67
	None Loads at quarter points	1.14
	At load points Loads at quarter points	1.67 1.11 1.11 1.67
	None	1.14
	At midpoint	1.30 1.30
	At third points	1.45 1.01 1.45
	At quarter points	1.52 1.06 1.06 1.52
	At fifth points	1.56 1.12 1.00 1.12 1.56

Note: Lateral bracing must always be provided at points of support per AISC *Specification* Chapter F.

F_y = 50 ksi

Table 3-2
W-Shapes
Selection by Z_x

Z_x

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W36×925^h	**4130**	**10300**	**15500**	**5920**	**8900**	**47.6**	**71.7**	**15.0**	**107**	**73000**	**2600**	**3900**
W36×853^h	**3920**	**9780**	**14700**	**5680**	**8530**	**48.3**	**72.7**	**15.1**	**100**	**70000**	**2170**	**3260**
W36×802^h	**3660**	**9130**	**13700**	**5310**	**7980**	**48.0**	**71.9**	**14.9**	**94.5**	**64800**	**2030**	**3040**
W36×723^h	**3270**	**8160**	**12300**	**4790**	**7190**	**47.6**	**72.2**	**14.7**	**85.5**	**57300**	**1810**	**2720**
W40×655^h	**3080**	**7680**	**11600**	**4520**	**6800**	**56.1**	**85.3**	**13.6**	**69.9**	**56500**	**1720**	**2580**
W36×652^h	**2910**	**7260**	**10900**	**4300**	**6460**	**46.8**	**70.3**	**14.5**	**77.7**	**50600**	**1620**	**2430**
W40×593^h	**2760**	**6890**	**10400**	**4090**	**6140**	**55.4**	**84.4**	**13.4**	**63.9**	**50400**	**1540**	**2310**
W36×529^h	**2330**	**5810**	**8740**	**3480**	**5220**	**46.4**	**70.1**	**14.1**	**64.3**	**39600**	**1280**	**1920**
W40×503^h	**2320**	**5790**	**8700**	**3460**	**5200**	**55.3**	**83.1**	**13.1**	**55.2**	**41600**	**1300**	**1950**
W36×487^h	**2130**	**5310**	**7990**	**3200**	**4800**	**46.0**	**69.5**	**14.0**	**59.9**	**36000**	**1180**	**1770**
W14×873^h	2030	5060	7610	2670	4020	7.67	11.5	17.3	329	18100	1860	2790
W40×431^h	**1960**	**4890**	**7350**	**2950**	**4440**	**53.6**	**80.4**	**12.9**	**49.1**	**34800**	**1110**	**1660**
W36×441^h	1910	4770	7160	2880	4330	45.3	67.9	13.8	55.5	32100	1060	1590
W27×539^h	1890	4720	7090	2740	4120	26.2	39.3	12.9	88.5	25600	1280	1920
W14×808^h	1830	4570	6860	2430	3650	7.33	11.0	17.1	309	15900	1710	2560
W40×397^h	**1800**	**4490**	**6750**	**2720**	**4100**	**52.4**	**78.4**	**12.9**	**46.7**	**32000**	**1000**	**1500**
W40×392^h	**1710**	**4270**	**6410**	**2510**	**3780**	**60.8**	**90.8**	**9.33**	**38.3**	**29900**	**1180**	**1770**
W36×395^h	1710	4270	6410	2600	3910	44.9	67.2	13.7	50.9	28500	937	1410
W40×372^h	**1680**	**4190**	**6300**	**2550**	**3830**	**51.7**	**77.9**	**12.7**	**44.4**	**29600**	**942**	**1410**
W14×730^h	1660	4140	6230	2240	3360	7.35	11.1	16.6	275	14300	1380	2060
W40×362^h	**1640**	**4090**	**6150**	**2480**	**3730**	**51.4**	**77.3**	**12.7**	**44.0**	**28900**	**909**	**1360**
W44×335	**1620**	**4040**	**6080**	**2460**	**3700**	**59.4**	**89.5**	**12.3**	**38.9**	**31100**	**906**	**1360**
W33×387^h	1560	3890	5850	2360	3540	38.3	57.8	13.3	53.3	24300	907	1360
W36×361^h	1550	3870	5810	2360	3540	43.6	65.6	13.6	48.2	25700	851	1280
W14×665^h	1480	3690	5550	2010	3020	7.10	10.7	16.3	253	12400	1220	1830
W40×324	**1460**	**3640**	**5480**	**2240**	**3360**	**49.0**	**74.1**	**12.6**	**41.2**	**25600**	**804**	**1210**
W30×391^h	1450	3620	5440	2180	3280	31.4	47.2	13.0	58.8	20700	903	1350
W40×331^h	1430	3570	5360	2110	3180	59.1	88.2	9.08	33.8	24700	996	1490
W33×354^h	1420	3540	5330	2170	3260	37.4	56.6	13.2	49.8	22000	826	1240

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Z_x

Table 3-2 (continued)
W-Shapes
Selection by Z_x

F_y = 50 ksi

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips	ft	ft		kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD			in.^{4}	ASD	LRFD
W44×290	**1410**	**3520**	**5290**	**2170**	**3260**	**54.9**	**82.5**	**12.3**	**36.9**	**27000**	**754**	**1130**
W40×327^h	1410	3520	5290	2100	3150	58.0	87.4	9.11	33.6	24500	963	1440
W36×330	1410	3520	5290	2170	3260	42.2	63.4	13.5	45.5	23300	769	1150
W40×297	1330	3320	4990	2040	3070	47.8	71.6	12.5	39.3	23200	740	1110
W30×357^h	1320	3290	4950	1990	2990	31.3	47.2	12.9	54.4	18700	813	1220
W14×605^h	1320	3290	4950	1820	2730	6.81	10.3	16.1	232	10800	1090	1630
W36×302	1280	3190	4800	1970	2970	40.5	60.8	13.5	43.6	21100	705	1060
W44×262	**1270**	**3170**	**4760**	**1940**	**2910**	**52.6**	**79.1**	**12.3**	**35.7**	**24100**	**680**	**1020**
W40×294	1270	3170	4760	1890	2840	56.9	85.4	9.01	31.5	21900	856	1280
W33×318	1270	3170	4760	1940	2910	36.8	55.4	13.1	46.5	19500	732	1100
W40×277	1250	3120	4690	1920	2890	45.8	68.7	12.6	38.8	21900	659	989
W27×368^h	1240	3090	4650	1850	2780	24.9	37.6	12.3	62.0	16200	839	1260
W40×278	1190	2970	4460	1780	2680	55.3	82.8	8.90	30.4	20500	828	1240
W36×282	1190	2970	4460	1830	2760	39.6	59.0	13.4	42.2	19600	657	985
W30×326^h	1190	2970	4460	1820	2730	30.3	45.6	12.7	50.6	16800	739	1110
W14×550^h	1180	2940	4430	1630	2440	6.65	10.1	15.9	213	9430	962	1440
W33×291	1160	2890	4350	1780	2680	36.0	54.2	13.0	43.8	17700	668	1000
W40×264	1130	2820	4240	1700	2550	53.8	81.3	8.90	29.7	19400	768	1150
W27×336^h	1130	2820	4240	1700	2550	25.0	37.7	12.2	57.0	14600	756	1130
W24×370^h	1130	2820	4240	1670	2510	20.0	30.0	11.6	69.2	13400	851	1280
W40×249	**1120**	**2790**	**4200**	**1730**	**2610**	**42.9**	**64.4**	**12.5**	**37.2**	**19600**	**591**	**887**
W44×230^v	**1100**	**2740**	**4130**	**1700**	**2550**	**46.8**	**71.2**	**12.1**	**34.3**	**20800**	**547**	**822**
W36×262	1100	2740	4130	1700	2550	38.1	57.9	13.3	40.6	17900	620	930
W30×292	1060	2640	3980	1620	2440	29.7	44.9	12.6	46.9	14900	653	979
W14×500^h	1050	2620	3940	1460	2200	6.43	9.65	15.6	196	8210	858	1290
W36×256	1040	2590	3900	1560	2350	46.5	70.0	9.36	31.5	16800	718	1080
W33×263	1040	2590	3900	1610	2410	34.1	51.9	12.9	41.6	15900	600	900
W36×247	1030	2570	3860	1590	2400	37.4	55.7	13.2	39.4	16700	587	881
W27×307^h	1030	2570	3860	1550	2330	25.1	37.7	12.0	52.6	13100	687	1030
W24×335^h	1020	2540	3830	1510	2270	19.9	30.2	11.4	63.1	11900	759	1140
W40×235	1010	2520	3790	1530	2300	51.0	76.7	8.97	28.4	17400	659	989

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 50 ksi; therefore, ϕ_v = 0.90 and Ω_v = 1.67.

F_y = 50 ksi

Table 3-2 (continued)
W-Shapes
Selection by Z_x

Z_x

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W40×215	**964**	**2410**	**3620**	**1500**	**2250**	**39.4**	**59.3**	**12.5**	**35.6**	**16700**	**507**	**761**
W36×231	963	2400	3610	1490	2240	35.7	53.7	13.1	38.6	15600	555	832
W30×261	943	2350	3540	1450	2180	29.1	44.0	12.5	43.4	13100	588	882
W33×241	940	2350	3530	1450	2180	33.5	50.2	12.8	39.7	14200	568	852
W36×232	936	2340	3510	1410	2120	44.8	67.0	9.25	30.0	15000	646	968
W27×281	936	2340	3510	1420	2140	24.8	36.9	12.0	49.1	11900	621	932
W14×455[h]	936	2340	3510	1320	1980	6.24	9.36	15.5	179	7190	768	1150
W24×306[h]	922	2300	3460	1380	2070	19.7	29.8	11.3	57.9	10700	683	1020
W40×211	**906**	**2260**	**3400**	**1370**	**2060**	**48.6**	**73.1**	**8.87**	**27.2**	**15500**	**591**	**887**
W40×199	**869**	**2170**	**3260**	**1340**	**2020**	**37.6**	**56.1**	**12.2**	**34.3**	**14900**	**503**	**755**
W14×426[h]	869	2170	3260	1230	1850	6.16	9.23	15.3	168	6600	703	1050
W33×221	857	2140	3210	1330	1990	31.8	47.8	12.7	38.2	12900	525	788
W27×258	852	2130	3200	1300	1960	24.4	36.5	11.9	45.9	10800	568	853
W30×235	847	2110	3180	1310	1960	28.0	42.7	12.4	41.0	11700	520	779
W24×279[h]	835	2080	3130	1250	1880	19.7	29.6	11.2	53.4	9600	619	929
W36×210	833	2080	3120	1260	1890	42.3	63.4	9.11	28.5	13200	609	914
W14×398[h]	801	2000	3000	1150	1720	5.95	8.96	15.2	158	6000	648	972
W40×183	**774**	**1930**	**2900**	**1180**	**1770**	**44.1**	**66.5**	**8.80**	**25.8**	**13200**	**507**	**761**
W33×201	773	1930	2900	1200	1800	30.3	45.6	12.6	36.7	11600	482	723
W27×235	772	1930	2900	1180	1780	24.1	36.0	11.8	42.9	9700	522	784
W36×194	767	1910	2880	1160	1740	40.4	61.4	9.04	27.6	12100	558	838
W18×311[h]	754	1880	2830	1090	1640	11.2	16.8	10.4	81.1	6970	678	1020
W30×211	751	1870	2820	1160	1750	26.9	40.5	12.3	38.7	10300	479	718
W21×275[h]	749	1870	2810	1110	1670	14.7	22.1	10.9	62.5	7690	588	882
W24×250	744	1860	2790	1120	1690	19.7	29.3	11.1	48.7	8490	547	821
W14×370[h]	736	1840	2760	1060	1590	5.87	8.80	15.1	148	5440	594	891
W36×182	**718**	**1790**	**2690**	**1090**	**1640**	**38.9**	**58.4**	**9.01**	**27.0**	**11300**	**526**	**790**
W27×217	711	1770	2670	1100	1650	23.0	35.1	11.7	40.8	8910	471	707

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Z_x

Table 3-2 (continued)
W-Shapes
Selection by Z_x

F_y = 50 ksi

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W40×167	**693**	**1730**	**2600**	**1050**	**1580**	**41.7**	**62.5**	**8.48**	**24.8**	**11600**	**502**	**753**
W18×283[h]	676	1690	2540	987	1480	11.1	16.7	10.3	73.6	6170	613	920
W30×191	675	1680	2530	1050	1580	25.6	38.6	12.2	36.8	9200	436	654
W24×229	675	1680	2530	1030	1540	19.0	28.9	11.0	45.2	7650	499	749
W14×342[h]	672	1680	2520	975	1460	5.73	8.62	15.0	138	4900	539	809
W21×248	671	1670	2520	1010	1510	14.3	21.9	10.9	57.1	6830	521	782
W36×170	668	1670	2510	1010	1530	37.8	56.1	8.94	26.4	10500	492	738
W27×194	631	1570	2370	976	1470	22.3	33.8	11.6	38.2	7860	422	632
W33×169	629	1570	2360	959	1440	34.2	51.5	8.83	26.7	9290	453	679
W36×160	**624**	**1560**	**2340**	**947**	**1420**	**36.1**	**54.2**	**8.83**	**25.8**	**9760**	**468**	**702**
W18×258[h]	611	1520	2290	898	1350	10.9	16.5	10.2	67.3	5510	550	826
W30×173	607	1510	2280	945	1420	24.1	36.8	12.1	35.5	8230	398	597
W24×207	606	1510	2270	927	1390	18.9	28.6	10.9	41.7	6820	447	671
W14×311[h]	603	1500	2260	884	1330	5.59	8.44	14.8	125	4330	482	723
W12×336[h]	603	1500	2260	844	1270	4.76	7.19	12.3	150	4060	598	897
W21×223	601	1500	2250	908	1370	14.5	21.6	10.7	51.4	6080	468	702
W40×149[v]	**598**	**1490**	**2240**	**896**	**1350**	**38.3**	**57.4**	**8.09**	**23.6**	**9800**	**432**	**650**
W36×150	581	1450	2180	880	1320	34.4	51.9	8.72	25.3	9040	449	673
W27×178	570	1420	2140	882	1330	21.6	32.5	11.5	36.4	7020	403	605
W33×152	559	1390	2100	851	1280	31.7	48.3	8.72	25.7	8160	425	638
W24×192	559	1390	2100	858	1290	18.4	28.0	10.8	39.7	6260	413	620
W18×234[h]	549	1370	2060	814	1220	10.8	16.4	10.1	61.4	4900	490	734
W14×283[h]	542	1350	2030	802	1200	5.52	8.36	14.7	114	3840	431	646
W12×305[h]	537	1340	2010	760	1140	4.64	6.97	12.1	137	3550	531	797
W21×201	530	1320	1990	805	1210	14.5	22.0	10.7	46.2	5310	419	628
W27×161	515	1280	1930	800	1200	20.6	31.3	11.4	34.7	6310	364	546
W33×141	**514**	**1280**	**1930**	**782**	**1180**	**30.3**	**45.7**	**8.58**	**25.0**	**7450**	**403**	**604**
W24×176	511	1270	1920	786	1180	18.1	27.7	10.7	37.4	5680	378	567
W36×135[v]	**509**	**1270**	**1910**	**767**	**1150**	**31.7**	**47.8**	**8.41**	**24.3**	**7800**	**384**	**577**
W30×148	500	1250	1880	761	1140	29.0	43.9	8.05	24.9	6680	399	599
W18×211	490	1220	1840	732	1100	10.7	16.2	9.96	55.7	4330	439	658
W14×257	487	1220	1830	725	1090	5.54	8.28	14.6	104	3400	387	581
W12×279[h]	481	1200	1800	686	1030	4.50	6.75	11.9	126	3110	487	730
W21×182	476	1190	1790	728	1090	14.4	21.8	10.6	42.7	4730	377	565
W24×162	468	1170	1760	723	1090	17.9	26.8	10.8	35.8	5170	353	529

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.

F_y = 50 ksi

Table 3-2 (continued)
W-Shapes
Selection by Z_x

Z_x

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.³	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.⁴	ASD	LRFD
W33×130	**467**	**1170**	**1750**	**709**	**1070**	**29.3**	**43.1**	**8.44**	**24.2**	**6710**	**384**	**576**
W27×146	464	1160	1740	723	1090	19.9	29.5	11.3	33.3	5660	332	497
W18×192	442	1100	1660	664	998	10.6	16.1	9.85	51.0	3870	392	588
W30×132	437	1090	1640	664	998	26.9	40.5	7.95	23.8	5770	373	559
W14×233	436	1090	1640	655	984	5.40	8.15	14.5	95.0	3010	342	514
W21×166	432	1080	1620	664	998	14.2	21.2	10.6	39.9	4280	338	506
W12×252[h]	428	1070	1610	617	927	4.43	6.68	11.8	114	2720	431	647
W24×146	418	1040	1570	648	974	17.0	25.8	10.6	33.7	4580	321	482
W33×118[v]	**415**	**1040**	**1560**	**627**	**942**	**27.2**	**40.6**	**8.19**	**23.4**	**5900**	**325**	**489**
W30×124	408	1020	1530	620	932	26.1	39.0	7.88	23.2	5360	353	530
W18×175	398	993	1490	601	903	10.6	15.8	9.75	46.9	3450	356	534
W27×129	395	986	1480	603	906	23.4	35.0	7.81	24.2	4760	337	505
W14×211	390	973	1460	590	887	5.30	7.94	14.4	86.6	2660	308	462
W12×230[h]	386	963	1450	561	843	4.31	6.51	11.7	105	2420	390	584
W30×116	**378**	**943**	**1420**	**575**	**864**	**24.8**	**37.4**	**7.74**	**22.6**	**4930**	**339**	**509**
W21×147	373	931	1400	575	864	13.7	20.7	10.4	36.3	3630	318	477
W24×131	370	923	1390	575	864	16.3	24.6	10.5	31.9	4020	296	445
W18×158	356	888	1340	541	814	10.5	15.9	9.68	42.8	3060	319	479
W14×193	355	886	1330	541	814	5.30	7.93	14.3	79.4	2400	276	414
W12×210	348	868	1310	510	767	4.25	6.45	11.6	95.8	2140	347	520
W30×108	**346**	**863**	**1300**	**522**	**785**	**23.5**	**35.5**	**7.59**	**22.1**	**4470**	**325**	**487**
W27×114	343	856	1290	522	785	21.7	32.8	7.70	23.1	4080	311	467
W21×132	333	831	1250	515	774	13.2	19.9	10.3	34.2	3220	283	425
W24×117	327	816	1230	508	764	15.4	23.3	10.4	30.4	3540	267	401
W18×143	322	803	1210	493	740	10.3	15.7	9.61	39.6	2750	285	427
W14×176	320	798	1200	491	738	5.20	7.83	14.2	73.2	2140	252	378
W30×99	**312**	**778**	**1170**	**470**	**706**	**22.2**	**33.4**	**7.42**	**21.3**	**3990**	**309**	**463**
W12×190	311	776	1170	459	690	4.18	6.33	11.5	87.3	1890	305	458
W21×122	307	766	1150	477	717	12.9	19.3	10.3	32.7	2960	260	391
W27×102	305	761	1140	466	701	20.1	29.8	7.59	22.3	3620	279	419
W18×130	290	724	1090	447	672	10.2	15.4	9.54	36.6	2460	259	388
W24×104	289	721	1080	451	677	14.3	21.3	10.3	29.2	3100	241	362
W14×159	287	716	1080	444	667	5.17	7.85	14.1	66.7	1900	224	335

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.

Z_x

Table 3-2 (continued)
W-Shapes
Selection by Z_x

F_y = 50 ksi

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.³	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.⁴	ASD	LRFD
W30×90[v]	**283**	**706**	**1060**	**428**	**643**	**20.6**	**30.8**	**7.38**	**20.9**	**3610**	**249**	**374**
W24×103	280	699	1050	428	643	18.2	27.4	7.03	21.9	3000	270	404
W21×111	279	696	1050	435	654	12.4	18.9	10.2	31.2	2670	237	355
W27×94	278	694	1040	424	638	19.1	28.5	7.49	21.6	3270	264	395
W12×170	275	686	1030	410	617	4.11	6.15	11.4	78.5	1650	269	403
W18×119	262	654	983	403	606	10.1	15.2	9.50	34.3	2190	249	373
W14×145	260	649	975	405	609	5.13	7.69	14.1	61.7	1710	201	302
W24×94	254	634	953	388	583	17.3	26.0	6.99	21.2	2700	250	375
W21×101	253	631	949	396	596	11.8	17.7	10.2	30.1	2420	214	321
W27×84	**244**	**609**	**915**	**372**	**559**	**17.6**	**26.4**	**7.31**	**20.8**	**2850**	**246**	**368**
W12×152	243	606	911	365	549	4.06	6.10	11.3	70.6	1430	238	358
W14×132	234	584	878	365	549	5.15	7.74	13.3	55.8	1530	190	284
W18×106	230	574	863	356	536	9.73	14.6	9.40	31.8	1910	221	331
W24×84	**224**	**559**	**840**	**342**	**515**	**16.2**	**24.2**	**6.89**	**20.3**	**2370**	**227**	**340**
W21×93	221	551	829	335	504	14.6	22.0	6.50	21.3	2070	251	376
W12×136	214	534	803	325	488	4.02	6.06	11.2	63.2	1240	212	318
W14×120	212	529	795	332	499	5.09	7.65	13.2	51.9	1380	171	257
W18×97	211	526	791	328	494	9.41	14.1	9.36	30.4	1750	199	299
W24×76	**200**	**499**	**750**	**307**	**462**	**15.1**	**22.6**	**6.78**	**19.5**	**2100**	**210**	**315**
W16×100	198	494	743	306	459	7.86	11.9	8.87	32.8	1490	199	298
W21×83	196	489	735	299	449	13.8	20.8	6.46	20.2	1830	220	331
W14×109	192	479	720	302	454	5.01	7.54	13.2	48.5	1240	150	225
W18×86	186	464	698	290	436	9.01	13.6	9.29	28.6	1530	177	265
W12×120	186	464	698	285	428	3.94	5.95	11.1	56.5	1070	186	279
W24×68	**177**	**442**	**664**	**269**	**404**	**14.1**	**21.2**	**6.61**	**18.9**	**1830**	**197**	**295**
W16×89	175	437	656	271	407	7.76	11.6	8.80	30.2	1300	176	265
W14×99[f]	173	430	646	274	412	4.91	7.36	13.5	45.3	1110	138	207
W21×73	172	429	645	264	396	12.9	19.4	6.39	19.2	1600	193	289
W12×106	164	409	615	253	381	3.93	5.89	11.0	50.7	933	157	236
W18×76	163	407	611	255	383	8.50	12.8	9.22	27.1	1330	155	232
W21×68	**160**	**399**	**600**	**245**	**368**	**12.5**	**18.8**	**6.36**	**18.7**	**1480**	**181**	**272**
W14×90[f]	157	382	574	250	375	4.82	7.26	15.1	42.5	999	123	185

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[f] Shape exceeds compact limit for flexure with F_y = 50 ksi; tabulated values have been adjusted accordingly.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 50 ksi; therefore, ϕ_v = 0.90 and Ω_v = 1.67.

F_y = 50 ksi

Table 3-2 (continued)
W-Shapes
Selection by Z_x

Z_x

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W24x62	**153**	**382**	**574**	**229**	**344**	**16.1**	**24.1**	**4.87**	**14.4**	**1550**	**204**	**306**
W16×77	150	374	563	234	352	7.34	11.1	8.72	27.8	1110	150	225
W12×96	147	367	551	229	344	3.85	5.78	10.9	46.7	833	140	210
W10×112	147	367	551	220	331	2.69	4.03	9.47	64.1	716	172	258
W18×71	146	364	548	222	333	10.4	15.8	6.00	19.6	1170	183	275
W21×62	**144**	**359**	**540**	**222**	**333**	**11.6**	**17.5**	**6.25**	**18.1**	**1330**	**168**	**252**
W14×82	139	347	521	215	323	5.40	8.10	8.76	33.2	881	146	219
W24×55[v]	**134**	**334**	**503**	**199**	**299**	**14.7**	**22.2**	**4.73**	**13.9**	**1350**	**167**	**252**
W18×65	133	332	499	204	307	9.98	15.0	5.97	18.8	1070	166	248
W12×87	132	329	495	206	310	3.81	5.73	10.8	43.1	740	129	193
W16×67	130	324	488	204	307	6.89	10.4	8.69	26.1	954	129	193
W10×100	130	324	488	196	294	2.64	4.00	9.36	57.9	623	151	226
W21×57	129	322	484	194	291	13.4	20.3	4.77	14.3	1170	171	256
W21×55	**126**	**314**	**473**	**192**	**289**	**10.8**	**16.3**	**6.11**	**17.4**	**1140**	**156**	**234**
W14×74	126	314	473	196	294	5.31	8.05	8.76	31.0	795	128	192
W18×60	123	307	461	189	284	9.62	14.4	5.93	18.2	984	151	227
W12×79	119	297	446	187	281	3.78	5.67	10.8	39.9	662	117	175
W14×68	115	287	431	180	270	5.19	7.81	8.69	29.3	722	116	174
W10×88	113	282	424	172	259	2.62	3.94	9.29	51.2	534	131	196
W18×55	**112**	**279**	**420**	**172**	**258**	**9.15**	**13.8**	**5.90**	**17.6**	**890**	**141**	**212**
W21×50	**110**	**274**	**413**	**165**	**248**	**12.1**	**18.3**	**4.59**	**13.6**	**984**	**158**	**237**
W12×72	108	269	405	170	256	3.69	5.56	10.7	37.5	597	106	159
W21×48[f]	**107**	**265**	**398**	**162**	**244**	**9.89**	**14.8**	**6.09**	**16.5**	**959**	**144**	**216**
W16×57	105	262	394	161	242	7.98	12.0	5.65	18.3	758	141	212
W14×61	102	254	383	161	242	4.93	7.48	8.65	27.5	640	104	156
W18×50	101	252	379	155	233	8.76	13.2	5.83	16.9	800	128	192
W10×77	97.6	244	366	150	225	2.60	3.90	9.18	45.3	455	112	169
W12×65[f]	96.8	237	356	154	231	3.58	5.39	11.9	35.1	533	94.4	142

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[f] Shape exceeds compact limit for flexure with F_y = 50 ksi; tabulated values have been adjusted accordingly.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 50 ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.

Z_x

Table 3-2 (continued)
W-Shapes
Selection by Z_x

F_y = 50 ksi

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W21×44	**95.4**	**238**	**358**	**143**	**214**	**11.1**	**16.8**	**4.45**	**13.0**	**843**	**145**	**217**
W16×50	92.0	230	345	141	213	7.69	11.4	5.62	17.2	659	124	186
W18×46	90.7	226	340	138	207	9.63	14.6	4.56	13.7	712	130	195
W14×53	87.1	217	327	136	204	5.22	7.93	6.78	22.3	541	103	154
W12×58	86.4	216	324	136	205	3.82	5.69	8.87	29.8	475	87.8	132
W10×68	85.3	213	320	132	199	2.58	3.85	9.15	40.6	394	97.8	147
W16×45	82.3	205	309	127	191	7.12	10.8	5.55	16.5	586	111	167
W18×40	**78.4**	**196**	**294**	**119**	**180**	**8.94**	**13.2**	**4.49**	**13.1**	**612**	**113**	**169**
W14×48	78.4	196	294	123	184	5.09	7.67	6.75	21.1	484	93.8	141
W12×53	77.9	194	292	123	185	3.65	5.50	8.76	28.2	425	83.5	125
W10×60	74.6	186	280	116	175	2.54	3.82	9.08	36.6	341	85.7	129
W16×40	**73.0**	**182**	**274**	**113**	**170**	**6.67**	**10.0**	**5.55**	**15.9**	**518**	**97.6**	**146**
W12×50	71.9	179	270	112	169	3.97	5.98	6.92	23.8	391	90.3	135
W8×67	70.1	175	263	105	159	1.75	2.59	7.49	47.6	272	103	154
W14×43	69.6	174	261	109	164	4.88	7.28	6.68	20.0	428	83.6	125
W10×54	66.6	166	250	105	158	2.48	3.75	9.04	33.6	303	74.7	112
W18×35	**66.5**	**166**	**249**	**101**	**151**	**8.14**	**12.3**	**4.31**	**12.3**	**510**	**106**	**159**
W12×45	64.2	160	241	101	151	3.80	5.80	6.89	22.4	348	81.1	122
W16×36	64.0	160	240	98.7	148	6.24	9.36	5.37	15.2	448	93.8	141
W14×38	61.5	153	231	95.4	143	5.37	8.20	5.47	16.2	385	87.4	131
W10×49	60.4	151	227	95.4	143	2.46	3.71	8.97	31.6	272	68.0	102
W8×58	59.8	149	224	90.8	137	1.70	2.55	7.42	41.6	228	89.3	134
W12×40	57.0	142	214	89.9	135	3.66	5.54	6.85	21.1	307	70.2	105
W10×45	54.9	137	206	85.8	129	2.59	3.89	7.10	26.9	248	70.7	106
W14×34	**54.6**	**136**	**205**	**84.9**	**128**	**5.01**	**7.55**	**5.40**	**15.6**	**340**	**79.8**	**120**
W16×31	**54.0**	**135**	**203**	**82.4**	**124**	**6.86**	**10.3**	**4.13**	**11.8**	**375**	**87.5**	**131**
W12×35	51.2	128	192	79.6	120	4.34	6.45	5.44	16.6	285	75.0	113
W8×48	49.0	122	184	75.4	113	1.67	2.55	7.35	35.2	184	68.0	102
W14×30	**47.3**	**118**	**177**	**73.4**	**110**	**4.63**	**6.95**	**5.26**	**14.9**	**291**	**74.5**	**112**
W10×39	46.8	117	176	73.5	111	2.53	3.78	6.99	24.2	209	62.5	93.7
W16×26[v]	**44.2**	**110**	**166**	**67.1**	**101**	**5.93**	**8.98**	**3.96**	**11.2**	**301**	**70.5**	**106**
W12×30	43.1	108	162	67.4	101	3.97	5.96	5.37	15.6	238	64.0	95.9

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.

F_y = 50 ksi

Table 3-2 (continued)
W-Shapes
Selection by Z_x

Z_x

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF/Ω_b	$\phi_b BF$	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W14×26	**40.2**	**100**	**151**	**61.7**	**92.7**	**5.33**	**8.11**	**3.81**	**11.0**	**245**	**70.9**	**106**
W8×40	39.8	99.3	149	62.0	93.2	1.64	2.46	7.21	29.9	146	59.4	89.1
W10×33	38.8	96.8	146	61.1	91.9	2.39	3.62	6.85	21.8	171	56.4	84.7
W12×26	**37.2**	**92.8**	**140**	**58.3**	**87.7**	**3.61**	**5.46**	**5.33**	**14.9**	**204**	**56.1**	**84.2**
W10×30	36.6	91.3	137	56.6	85.1	3.08	4.61	4.84	16.1	170	63.0	94.5
W8×35	34.7	86.6	130	54.5	81.9	1.62	2.43	7.17	27.0	127	50.3	75.5
W14×22	**33.2**	**82.8**	**125**	**50.6**	**76.1**	**4.78**	**7.27**	**3.67**	**10.4**	**199**	**63.0**	**94.5**
W10×26	31.3	78.1	117	48.7	73.2	2.91	4.34	4.80	14.9	144	53.6	80.3
W8×31[f]	30.4	75.8	114	48.0	72.2	1.58	2.37	7.18	24.8	110	45.6	68.4
W12×22	**29.3**	**73.1**	**110**	**44.4**	**66.7**	**4.68**	**7.06**	**3.00**	**9.13**	**156**	**64.0**	**95.9**
W8×28	27.2	67.9	102	42.4	63.8	1.67	2.50	5.72	21.0	98.0	45.9	68.9
W10×22	**26.0**	**64.9**	**97.5**	**40.5**	**60.9**	**2.68**	**4.02**	**4.70**	**13.8**	**118**	**49.0**	**73.4**
W12×19	**24.7**	**61.6**	**92.6**	**37.2**	**55.9**	**4.27**	**6.43**	**2.90**	**8.61**	**130**	**57.3**	**86.0**
W8×24	23.1	57.6	86.6	36.5	54.9	1.60	2.40	5.69	18.9	82.7	38.9	58.3
W10×19	**21.6**	**53.9**	**81.0**	**32.8**	**49.4**	**3.18**	**4.76**	**3.09**	**9.73**	**96.3**	**51.0**	**76.5**
W8×21	20.4	50.9	76.5	31.8	47.8	1.85	2.77	4.45	14.8	75.3	41.4	62.1
W12×16	**20.1**	**50.1**	**75.4**	**29.9**	**44.9**	**3.80**	**5.73**	**2.73**	**8.05**	**103**	**52.8**	**79.2**
W10×17	18.7	46.7	70.1	28.3	42.5	2.98	4.47	2.98	9.16	81.9	48.5	72.7
W12×14[v]	**17.4**	**43.4**	**65.3**	**26.0**	**39.1**	**3.43**	**5.17**	**2.66**	**7.73**	**88.6**	**42.8**	**64.3**
W8×18	17.0	42.4	63.8	26.5	39.9	1.74	2.61	4.34	13.5	61.9	37.4	56.2
W10×15	16.0	39.9	60.0	24.1	36.2	2.75	4.14	2.86	8.61	68.9	46.0	68.9
W8×15	13.6	33.9	51.0	20.6	31.0	1.90	2.85	3.09	10.1	48.0	39.7	59.6
W10×12[f]	**12.6**	**31.2**	**46.9**	**19.0**	**28.6**	**2.36**	**3.53**	**2.87**	**8.05**	**53.8**	**37.5**	**56.3**
W8×13	11.4	28.4	42.8	17.3	26.0	1.76	2.67	2.98	9.27	39.6	36.8	55.1
W8×10[f]	**8.87**	**21.9**	**32.9**	**13.6**	**20.5**	**1.54**	**2.30**	**3.14**	**8.52**	**30.8**	**26.8**	**40.2**

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.

I_x

Table 3-3
W-Shapes
Selection by I_x

Shape	I_x in.^{4}	Shape	I_x in.^{4}	Shape	I_x in.^{4}	Shape	I_x in.^{4}
W36×925[h]	**73000**	**W44×230**	**20800**	**W40×167**	**11600**	**W33×118**	**5900**
W36×853[h]	**70000**	W30×391[h]	20700	W33×201	11600	W30×132	5770
W36×802[h]	**64800**	W40×278	20500	W36×182	11300	W24×176	5680
W36×723[h]	**57300**	W40×249	19600	W27×258	10800	W27×146	5660
W40×655[h]	**56500**	W36×282	19600	W14×605[h]	10800	W18×258[h]	5510
W36×652[h]	**50600**	W33×318	19500	W24×306[h]	10700	W14×370[h]	5440
W40×593[h]	**50400**	W40×264	19400	W36×170	10500	W30×124	5360
W40×503[h]	**41600**	W30×357[h]	18700	W30×211	10300	W21×201	5310
W36×529[h]	39600	W14×873[h]	18100	**W40×149**	**9800**	W24×162	5170
W36×487[h]	**36000**	W36×262	17900	W36×160	9760	**W30×116**	**4930**
W40×431[h]	**34800**	W33×291	17700	W27×235	9700	W18×234[h]	4900
W36×441[h]	32100	W40×235	17400	W24×279[h]	9600	W14×342[h]	4900
W40×397[h]	**32000**	W36×256	16800	W14×550[h]	9430	W27×129	4760
W44×335	**31100**	W30×326[h]	16800	W33×169	9290	W21×182	4730
W40×392[h]	29900	**W40×215**	**16700**	W30×191	9200	W24×146	4580
W40×372[h]	29600	W36×247	16700	W36×150	9040	**W30×108**	**4470**
W40×362[h]	28900	W27×368[h]	16200	W27×217	8910	W18×211	4330
W36×395[h]	28500	W33×263	15900	W24×250	8490	W14×311[h]	4330
W44×290	**27000**	W14×808[h]	15900	W30×173	8230	W21×166	4280
W36×361[h]	25700	W36×231	15600	W14×500[h]	8210	W27×114	4080
W40×324	25600	**W40×211**	**15500**	W33×152	8160	W12×336[h]	4060
W27×539[h]	25600	W36×232	15000	W27×194	7860	W24×131	4020
W40×331[h]	24700	**W40×199**	**14900**	**W36×135**	**7800**	**W30×99**	**3990**
W40×327[h]	24500	W30×292	14900	W21×275[h]	7690	W18×192	3870
W33×387[h]	24300	W27×336[h]	14600	W24×229	7650	W14×283[h]	3840
W44×262	**24100**	W14×730[h]	14300	W33×141	7450	W21×147	3630
W36×330	23300	W33×241	14200	W14×455[h]	7190	W27×102	3620
W40×297	23200	W24×370[h]	13400	W27×178	7020	**W30×90**	**3610**
W33×354[h]	22000	**W40×183**	**13200**	W18×311[h]	6970	W12×305[h]	3550
W40×277	21900	W36×210	13200	W21×248	6830	W24×117	3540
W40×294	21900	W30×261	13100	W24×207	6820	W18×175	3450
W36×302	21100	W27×307[h]	13100	**W33×130**	**6710**	W14×257	3400
		W33×221	12900	W30×148	6680	W27×94	3270
		W14×665[h]	12400	W14×426[h]	6600	W21×132	3220
		W36×194	12100	W27×161	6310	W12×279[h]	3110
		W27×281	11900	W24×192	6260	W24×104	3100
		W24×335[h]	11900	W18×283[h]	6170	W18×158	3060
		W30×235	11700	W21×223	6080	W14×233	3010
				W14×398[h]	6000	W24×103	3000
						W21×122	2960

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Table 3-3 (continued)
W-Shapes
Selection by I_x

I_x

Shape	I_x in.^{4}
W27×84	**2850**
W18×143	2750
W12×252[h]	2720
W24×94	2700
W21×111	2670
W14×211	2660
W18×130	2460
W21×101	2420
W12×230[h]	2420
W14×193	2400
W24×84	**2370**
W18×119	2190
W14×176	2140
W12×210	2140
W24×76	**2100**
W21×93	2070
W18×106	1910
W14×159	1900
W12×190	1890
W24×68	**1830**
W21×83	1830
W18×97	1750
W14×145	1710
W12×170	1650
W21×73	1600
W24×62	**1550**
W18×86	1530
W14×132	1530
W16×100	1490
W21×68	1480
W12×152	1430
W14×120	1380
W24×55	**1350**
W21×62	1330
W18×76	1330
W16×89	1300
W14×109	1240
W12×136	1240
W21×57	1170
W18×71	1170

Shape	I_x in.^{4}
W21×55	**1140**
W16×77	1110
W14×99	1110
W18×65	1070
W12×120	1070
W14×90	999
W21×50	**984**
W18×60	984
W21×48	**959**
W16×67	954
W12×106	933
W18×55	890
W14×82	881
W21×44	**843**
W12×96	833
W18×50	800
W14×74	795
W16×57	758
W12×87	740
W14×68	722
W10×112	716
W18×46	712
W12×79	662
W16×50	659
W14×61	640
W10×100	623
W18×40	**612**
W12×72	597
W16×45	586
W14×53	541
W10×88	534
W12×65	533
W16×40	**518**

Shape	I_x in.^{4}
W18×35	**510**
W14×48	484
W12×58	475
W10×77	455
W16×36	448
W14×43	428
W12×53	425
W10×68	394
W12×50	391
W14×38	385
W16×31	**375**
W12×45	348
W10×60	341
W14×34	340
W12×40	307
W10×54	303
W16×26	**301**
W14×30	291
W12×35	285
W10×49	272
W8×67	272
W10×45	248
W14×26	**245**
W12×30	238
W8×58	228
W10×39	209
W12×26	**204**
W14×22	**199**
W8×48	184
W10×33	171
W10×30	170
W12×22	**156**
W8×40	146
W10×26	144
W12×19	**130**
W8×35	127
W10×22	118
W8×31	110

Shape	I_x in.^{4}
W12×16	**103**
W8×28	98.0
W10×19	96.3
W12×14	**88.6**
W8×24	82.7
W10×17	81.9
W8×21	75.3
W10×15	68.9
W8×18	61.9
W10×12	**53.8**
W8×15	48.0
W8×13	39.6
W8×10	**30.8**

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Z_y

Table 3-4
W-Shapes
Selection by Z_y

F_y = 50 ksi

Shape	Z_y	M_{ny}/Ω_b	$\phi_b M_{ny}$
		kip-ft	kip-ft
	in.^{3}	ASD	LRFD
W14×873[h]	**1020**	**2540**	**3830**
W14×808[h]	**930**	**2320**	**3490**
W36×925[h]	862	2120	3190
W14×730[h]	**816**	**2040**	**3060**
W36×853[h]	805	2010	3020
W36×802[h]	744	1860	2790
W14×665[h]	**730**	**1820**	**2740**
W36×723[h]	658	1640	2470
W14×605[h]	**652**	**1630**	**2450**
W14×550[h]	**583**	**1450**	**2190**
W36×652[h]	581	1450	2180
W40×655[h]	542	1350	2030
W14×500[h]	**522**	**1300**	**1960**
W40×593[h]	481	1200	1800
W14×455[h]	**468**	**1170**	**1760**
W36×529[h]	454	1130	1700
W27×539[h]	437	1090	1640
W14×426[h]	**434**	**1080**	**1630**
W36×487[h]	412	1030	1550
W14×398[h]	**402**	**1000**	**1510**
W40×503[h]	394	983	1480
W14×370[h]	**370**	**923**	**1390**
W36×441[h]	368	918	1380
W14×342[h]	**338**	**843**	**1270**
W40×431[h]	328	818	1230
W36×395[h]	325	811	1220
W33×387[h]	312	778	1170
W30×391[h]	310	773	1160

Shape	Z_y	M_{ny}/Ω_b	$\phi_b M_{ny}$
		kip-ft	kip-ft
	in.^{3}	ASD	LRFD
W14×311[h]	**304**	**758**	**1140**
W40×397[h]	300	749	1130
W36×361[h]	293	731	1100
W33×354[h]	282	704	1060
W30×357[h]	279	696	1050
W27×368[h]	279	696	1050
W40×372[h]	277	691	1040
W14×283[h]	**274**	**684**	**1030**
W12×336[h]	274	684	1030
W40×362[h]	270	674	1010
W24×370[h]	267	666	1000
W36×330	265	661	994
W30×326[h]	252	629	945
W27×336[h]	252	629	945
W33×318	250	624	938
W14×257	**246**	**614**	**923**
W12×305[h]	244	609	915
W36×302	241	601	904
W40×324	239	596	896
W24×335[h]	238	594	893
W44×335	236	589	885
W27×307[h]	227	566	851
W33×291	226	564	848
W36×282	223	556	836
W30×292	223	556	836
W14×233	**221**	**551**	**829**
W12×279[h]	220	549	825
W40×297	215	536	806
W24×306[h]	214	534	803
W40×392[h]	212	519	780
W18×311[h]	207	516	776
W27×281	206	514	773
W44×290	205	511	769
W40×277	204	509	765
W36×262	204	509	765
W33×263	202	504	758

Shape	Z_y	M_{ny}/Ω_b	$\phi_b M_{ny}$
		kip-ft	kip-ft
	in.^{3}	ASD	LRFD
W14×211	**198**	**494**	**743**
W30×261	196	489	735
W12×252[h]	196	489	735
W24×279[h]	193	482	724
W21×275[h]	191	477	716
W36×247	190	474	713
W27×258	187	467	701
W18×283[h]	185	462	694
W44×262	182	454	683
W40×249	182	454	683
W33×241	182	454	683
W14×193	**180**	**449**	**675**
W12×230[h]	177	442	664
W36×231	176	439	660
W30×235	175	437	656
W40×331[h]	172	423	636
W24×250	171	427	641
W40×327[h]	170	419	630
W21×248	170	424	638
W27×235	168	419	630
W18×258[h]	166	414	623
W33×221	164	409	615
W14×176	**163**	**407**	**611**
W12×210	159	397	596
W44×230[f]	157	392	589
W40×215	156	389	585
W30×211	155	387	581
W27×217	154	384	578
W24×229	154	384	578
W40×294	150	373	561
W21×223	150	374	563
W18×234[h]	149	372	559
W33×201	147	367	551

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

F_y = 50 ksi

Table 3-4 (continued)
W-Shapes
Selection by Z_y

Z_y

Shape	Z_y	M_{ny}/Ω_b	$\phi_b M_{ny}$
		kip-ft	kip-ft
	in.^{3}	ASD	LRFD
W14×159	**146**	**364**	**548**
W12×190	143	357	536
W40×278	140	348	523
W30×191	138	344	518
W40×199	137	342	514
W36×256	137	342	514
W24×207	137	342	514
W27×194	136	339	510
W21×201	133	332	499
W14×145	**133**	**332**	**499**
W40×264	132	329	495
W18×211	132	329	495
W24×192	126	314	473
W12×170	126	314	473
W30×173	123	307	461
W36×232	122	304	458
W27×178	122	304	458
W21×182	119	297	446
W18×192	119	297	446
W40×235	118	294	443
W24×176	115	287	431
W14×132	**113**	**282**	**424**
W12×152	111	277	416
W27×161	109	272	409
W21×166	108	269	405
W36×210	107	267	401
W18×175	106	264	398
W40×211	105	262	394
W24×162	105	262	394
W14×120	**102**	**254**	**383**
W12×136	98.0	245	368
W36×194	97.7	244	366
W27×146	97.7	244	366
W18×158	94.8	237	356
W24×146	93.2	233	350

Shape	Z_y	M_{ny}/Ω_b	$\phi_b M_{ny}$
		kip-ft	kip-ft
	in.^{3}	ASD	LRFD
W14×109	**92.7**	**231**	**348**
W21×147	92.6	231	347
W36×182	90.7	226	340
W40×183	88.3	220	331
W18×143	85.4	213	320
W12×120	85.4	213	320
W33×169	84.4	211	317
W36×170	83.8	209	314
W14×99[f]	**83.6**	**207**	**311**
W21×132	82.3	205	309
W24×131	81.5	203	306
W36×160	77.3	193	290
W18×130	76.7	191	288
W40×167	76.0	190	285
W21×122	75.6	189	283
W14×90[f]	**75.6**	**181**	**273**
W12×106	75.1	187	282
W33×152	73.9	184	277
W24×117	71.4	178	268
W36×150	70.9	177	266
W10×112	69.2	173	260
W18×119	69.1	172	259
W21×111	68.2	170	256
W30×148	68.0	170	255
W12×96	67.5	168	253
W33×141	66.9	167	251
W24×104	62.4	156	234
W40×149	62.2	155	233
W21×101	61.7	154	231
W10×100	61.0	152	229
W18×106	60.5	151	227

Shape	Z_y	M_{ny}/Ω_b	$\phi_b M_{ny}$
		kip-ft	kip-ft
	in.^{3}	ASD	LRFD
W12×87	**60.4**	**151**	**227**
W36×135	59.7	149	224
W33×130	59.5	148	223
W30×132	58.4	146	219
W27×129	57.6	144	216
W18×97	55.3	138	207
W16×100	54.9	137	206
W12×79	**54.3**	**135**	**204**
W30×124	54.0	135	203
W10×88	53.1	132	199
W33×118	51.3	128	192
W27×114	49.3	123	185
W30×116	49.2	123	185
W12×72	**49.2**	**123**	**185**
W18×86	48.4	121	182
W16×89	48.1	120	180
W10×77	45.9	115	172
W14×82	44.8	112	168
W12×65[f]	**44.1**	**107**	**161**
W30×108	43.9	110	165
W27×102	43.4	108	163
W18×76	42.2	105	158
W24×103	41.5	104	156
W16×77	41.1	103	154
W14×74	40.5	101	152
W10×68	40.1	100	150
W27×94	38.8	96.8	146
W30×99	38.6	96.3	145
W24×94	37.5	93.6	141
W14×68	36.9	92.1	138
W16×67	35.5	88.6	133

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[f] Shape exceeds compact limit for flexure with F_y = 50 ksi; tabulated values have been adjusted accordingly.

Z_y

Table 3-4 (continued)
W-Shapes
Selection by Z_y

F_y = 50 ksi

Shape	Z_y in.^{3}	M_{ny}/Ω_b kip-ft ASD	$\phi_b M_{ny}$ kip-ft LRFD
W10×60	**35.0**	**87.3**	**131**
W30×90	34.7	86.6	130
W21×93	34.7	86.6	130
W27×84	33.2	82.8	125
W14×61	32.8	81.8	123
W8×67	32.7	81.6	123
W24×84	32.6	81.3	122
W12×58	**32.5**	**81.1**	**122**
W10×54	**31.3**	**78.1**	**117**
W21×83	30.5	76.1	114
W12×53	**29.1**	**72.6**	**109**
W24×76	28.6	71.4	107
W10×49	**28.3**	**70.6**	**106**
W8×58	27.9	69.6	105
W21×73	26.6	66.4	99.8
W18×71	24.7	61.6	92.6
W24×68	24.5	61.1	91.9
W21×68	24.4	60.9	91.5
W8×48	**22.9**	**57.1**	**85.9**
W18×65	22.5	56.1	84.4
W14×53	22.0	54.9	82.5
W21×62	21.7	54.1	81.4
W12×50	21.3	53.1	79.9
W18×60	20.6	51.4	77.3
W10×45	**20.3**	**50.6**	**76.1**
W14×48	19.6	48.9	73.5
W12×45	**19.0**	**47.4**	**71.3**
W16×57	18.9	47.2	70.9
W18×55	18.5	46.2	69.4

Shape	Z_y in.^{3}	M_{ny}/Ω_b kip-ft ASD	$\phi_b M_{ny}$ kip-ft LRFD
W8×40	**18.5**	**46.2**	**69.4**
W21×55	18.4	45.9	69.0
W14×43	17.3	43.2	64.9
W10×39	**17.2**	**42.9**	**64.5**
W12×40	16.8	41.9	63.0
W18×50	16.6	41.4	62.3
W16×50	16.3	40.7	61.1
W8×35	**16.1**	**40.2**	**60.4**
W24×62	15.7	39.1	58.8
W21×48[f]	14.9	36.7	55.2
W21×57	14.8	36.9	55.5
W16×45	14.5	36.2	54.4
W8×31[f]	**14.1**	**35.1**	**52.8**
W10×33	14.0	34.9	52.5
W24×55	13.3	33.1	49.8
W16×40	12.7	31.7	47.6
W21×50	12.2	30.4	45.8
W14×38	12.1	30.2	45.4
W18×46	11.7	29.2	43.9
W12×35	11.5	28.7	43.1
W16×36	10.8	26.9	40.5
W14×34	10.6	26.4	39.8
W21×44	10.2	25.4	38.2
W8×28	**10.1**	**25.2**	**37.9**
W18×40	10.0	25.0	37.5
W12×30	9.56	23.9	35.9
W14×30	8.99	22.4	33.7
W10×30	8.84	22.1	33.2

Shape	Z_y in.^{3}	M_{ny}/Ω_b kip-ft ASD	$\phi_b M_{ny}$ kip-ft LRFD
W8×24	**8.57**	**21.4**	**32.1**
W12×26	8.17	20.4	30.6
W18×35	8.06	20.1	30.2
W10×26	7.50	18.7	28.1
W16×31	7.03	17.5	26.4
W10×22	**6.10**	**15.2**	**22.9**
W8×21	**5.69**	**14.2**	**21.3**
W14×26	5.54	13.8	20.8
W16×26	5.48	13.7	20.6
W8×18	**4.66**	**11.6**	**17.5**
W14×22	4.39	11.0	16.5
W12×22	3.66	9.13	13.7
W10×19	3.35	8.36	12.6
W12×19	2.98	7.44	11.2
W10×17	**2.80**	**6.99**	**10.5**
W8×15	**2.67**	**6.66**	**10.0**
W10×15	**2.30**	**5.74**	**8.63**
W12×16	2.26	5.63	8.46
W8×13	**2.15**	**5.36**	**8.06**
W12×14	1.90	4.74	7.13
W10×12[f]	**1.74**	**4.30**	**6.46**
W8×10[f]	**1.66**	**4.07**	**6.12**

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[f] Shape exceeds compact limit for flexure with F_y = 50 ksi; tabulated values have been adjusted accordingly.

Table 3-5
W-Shapes
Selection by I_y

I_y

Shape	I_y in.[4]	Shape	I_y in.[4]	Shape	I_y in.[4]	Shape	I_y in.[4]
W14×873[h]	**6170**	**W14×283[h]**	**1440**	**W14×193**	**931**	**W14×132**	**548**
W14×808[h]	**5550**	W40×372[h]	1420	W40×249	926	W21×201	542
W36×925[h]	4940	W36×330	1420	W44×262	923	W24×192	530
W14×730[h]	**4720**	W30×357[h]	1390	W24×306[h]	919	W36×256	528
W36×853[h]	4600	W40×362[h]	1380	W27×258	859	W40×278	521
W36×802[h]	4210	W27×368[h]	1310	W30×235	855	W12×170	517
W14×665[h]	**4170**	W36×302	1300	W33×221	840	W27×161	497
W36×723[h]	3700	W33×318	1290	**W14×176**	**838**	**W14×120**	**495**
W14×605[h]	**3680**	**W14×257**	**1290**	W12×252[h]	828	W40×264	493
W14×550[h]	**3250**	W30×326[h]	1240	W24×279[h]	823	W18×211	493
W36×652[h]	3230	W40×324	1220	W40×392[h]	803	W21×182	483
W14×500[h]	**2880**	W44×335	1200	W44×230	796	W24×176	479
W40×655[h]	2870	W36×282	1200	W40×215	803	W36×232	468
W14×455[h]	**2560**	W12×336[h]	1190	W18×311[h]	795	W12×152	454
W40×593[h]	2520	W27×336[h]	1180	W21×275[h]	787	**W14×109**	**447**
W36×529[h]	2490	W33×291	1160	W27×235	769	W40×235	444
W14×426[h]	**2360**	W24×370[h]	1160	W30×211	757	W27×146	443
W36×487[h]	2250	**W14×233**	**1150**	W33×201	749	W24×162	443
W14×398[h]	**2170**	W30×292	1100	**W14×159**	**748**	W18×192	440
W27×539[h]	2110	W40×297	1090	W12×230[h]	742	W21×166	435
W40×503[h]	2040	W36×262	1090	W24×250	724	W36×210	411
W36×441[h]	1990	W27×307[h]	1050	W27×217	704	**W14×99**	**402**
W14×370[h]	**1990**	W12×305[h]	1050	W18×283[h]	704	W12×136	398
W14×342[h]	**1810**	W44×290	1040	W21×248	699	W24×146	391
W36×395[h]	1750	W40×277	1040	W40×199	695	W18×175	391
W40×431[h]	1690	W33×263	1040	**W14×145**	**677**	W40×211	390
W33×387[h]	1620	W24×335[h]	1030	W30×191	673	W21×147	376
W14×311[h]	**1610**	**W14×211**	**1030**	W12×210	664	W36×194	375
W36×361[h]	1570	W36×247	1010	W24×229	651		
W30×391[h]	1550	W30×261	959	W40×331[h]	644		
W40×397[h]	1540	W27×281	953	W40×327[h]	640		
W33×354[h]	1460	W36×231	940	W18×258[h]	628		
		W12×279[h]	937	W27×194	619		
		W33×241	933	W21×223	614		
				W30×173	598		
				W12×190	589		
				W24×207	578		
				W40×294	562		
				W18×234[h]	558		
				W27×178	555		

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

I_y

Table 3-5 (continued)
W-Shapes
Selection by I_y

Shape	I_y, in.^{4}	Shape	I_y, in.^{4}	Shape	I_y, in.^{4}	Shape	I_y, in.^{4}
W14×90	**362**	**W12×65**	**174**	**W8×48**	**60.9**	**W8×28**	**21.7**
W36×182	347	W30×116	164	W18×71	60.3	W21×44	20.7
W18×158	347	W16×89	163	W14×53	57.7	W12×30	20.3
W12×120	345	W27×114	159	W21×62	57.5	W14×30	19.6
W24×131	340	W10×77	154	W12×50	56.3	W18×40	19.1
W21×132	333	W18×76	152	W18×65	54.8	**W8×24**	**18.3**
W40×183	331	W14×82	148	**W10×45**	**53.4**	W12×26	17.3
W36×170	320	W30×108	146	W14×48	51.4	W10×30	16.7
W18×143	311	W27×102	139	W18×60	50.1	W18×35	15.3
W33×169	310	W16×77	138	**W12×45**	**50.0**	W10×26	14.1
W21×122	305	W14×74	134	**W8×40**	**49.1**	W16×31	12.4
W12×106	301	W10×68	134	W21×55	48.4	**W10×22**	**11.4**
W24×117	297	W30×99	128	W14×43	45.2	**W8×21**	**9.77**
W36×160	295	W27×94	124	**W10×39**	**45.0**	W16×26	9.59
W40×167	283	W14×68	121	W18×55	44.9	W14×26	8.91
W18×130	278	W24×103	119	W12×40	44.1	**W8×18**	**7.97**
W21×111	274	W16×67	119	W16×57	43.1	W14×22	7.00
W33×152	273	**W10×60**	**116**	**W8×35**	**42.6**	W12×22	4.66
W36×150	270	W30×90	115	W18×50	40.1	W10×19	4.29
W12×96	270	W24×94	109	W21×48	38.7	W12×19	3.76
W24×104	259	W14×61	107	W16×50	37.2	**W10×17**	**3.56**
W18×119	253	**W12×58**	**107**	**W8×31**	**37.1**	**W8×15**	**3.41**
W21×101	248	W27×84	106	W10×33	36.6	**W10×15**	**2.89**
W33×141	246	**W10×54**	**103**	W24×62	34.5	W12×16	2.82
W12×87	**241**	**W12×53**	**95.8**	W16×45	32.8	**W8×13**	**2.73**
W10×112	236	W24×84	94.4	W21×57	30.6	W12×14	2.36
W40×149	229	**W10×49**	**93.4**	W24×55	29.1	**W10×12**	**2.18**
W30×148	227	W21×93	92.9	W16×40	28.9	**W8×10**	**2.09**
W36×135	225	W8×67	88.6	W14×38	26.7		
W18×106	220	W24×76	82.5	W21×50	24.9		
W33×130	218	W21×83	81.4	W16×36	24.5		
W12×79	**216**	W8×58	75.1	W12×35	24.5		
W10×100	207	W21×73	70.6	W14×34	23.3		
W18×97	201	W24×68	70.4	W18×46	22.5		
W30×132	196	W21×68	64.7				
W12×72	**195**						
W33×118	187						
W16×100	186						
W27×129	184						
W30×124	181						
W10×88	179						
W18×86	175						

F_y = 50 ksi

Table 3-6
Maximum Total Uniform Load, kips
W-Shapes

W44

Shape		W44× 335		W44× 290		W44× 262		W44× 230[v]	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	17	1810	2720						
	18	1800	2700	1510	2260	1360	2040		
	19	1700	2560	1480	2230	1330	2010		
	20	1620	2430	1410	2120	1270	1910	1090	1640
	21	1540	2310	1340	2010	1210	1810	1050	1570
	22	1470	2210	1280	1920	1150	1730	998	1500
	23	1410	2110	1220	1840	1100	1660	955	1430
	24	1350	2030	1170	1760	1060	1590	915	1380
	25	1290	1940	1130	1690	1010	1520	878	1320
	26	1240	1870	1080	1630	975	1470	844	1270
	27	1200	1800	1040	1570	939	1410	813	1220
	28	1150	1740	1010	1510	905	1360	784	1180
	29	1120	1680	970	1460	874	1310	757	1140
	30	1080	1620	938	1410	845	1270	732	1100
	32	1010	1520	879	1320	792	1190	686	1030
	34	951	1430	828	1240	746	1120	646	971
	36	898	1350	782	1180	704	1060	610	917
	38	851	1280	741	1110	667	1000	578	868
	40	808	1220	704	1060	634	953	549	825
	42	770	1160	670	1010	604	907	523	786
	44	735	1100	640	961	576	866	499	750
	46	703	1060	612	920	551	828	477	717
	48	674	1010	586	881	528	794	457	688
	50	647	972	563	846	507	762	439	660
	52	622	935	541	813	487	733	422	635
	54	599	900	521	783	469	706	407	611
	56	577	868	503	755	453	680	392	589
	58	558	838	485	729	437	657	379	569
	60	539	810	469	705	422	635	366	550
	62	522	784	454	682	409	615	354	532
	64	505	759	440	661	396	595	343	516
	66	490	736	426	641	384	577	333	500
	68	476	715	414	622	373	560	323	485
	70	462	694	402	604	362	544	314	471
	72	449	675	391	588	352	529	305	458

Beam Properties

ASD	LRFD	335 ASD	335 LRFD	290 ASD	290 LRFD	262 ASD	262 LRFD	230 ASD	230 LRFD
W_c/Ω_b	$\phi_b W_c$, kip-ft	32300	48600	28100	42300	25300	38100	22000	33000
M_p/Ω_b	$\phi_b M_p$, kip-ft	4040	6080	3520	5290	3170	4760	2740	4130
M_r/Ω_b	$\phi_b M_r$, kip-ft	2460	3700	2170	3260	1940	2910	1700	2550
BF/Ω_b	$\phi_b BF$, kips	59.4	89.5	54.9	82.5	52.6	79.1	46.8	71.2
V_n/Ω_v	$\phi_v V_n$, kips	906	1360	754	1130	680	1020	547	822

	335	290	262	230
Z_x, in.^{3}	1620	1410	1270	1100
L_p, ft	12.3	12.3	12.3	12.1
L_r, ft	38.9	36.9	35.7	34.3

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.

Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

W40

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

F_y = 50 ksi

Shape	W40× 655[h]		593[h]		503[h]		431[h]		397[h]		392[h]	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 14											2360	3540
15											2280	3420
16											2130	3210
17	3440	5150	3080	4620	2590	3890	2210	3320			2010	3020
18	3420	5130	3060	4600	2570	3870	2170	3270	2000	3000	1900	2850
19	3240	4860	2900	4360	2440	3660	2060	3090	1890	2840	1800	2700
20	3070	4620	2750	4140	2320	3480	1960	2940	1800	2700	1710	2570
21	2930	4400	2620	3940	2210	3310	1860	2800	1710	2570	1630	2440
22	2790	4200	2500	3760	2100	3160	1780	2670	1630	2450	1550	2330
23	2670	4020	2400	3600	2010	3030	1700	2560	1560	2350	1480	2230
24	2560	3850	2300	3450	1930	2900	1630	2450	1500	2250	1420	2140
25	2460	3700	2200	3310	1850	2780	1560	2350	1440	2160	1370	2050
26	2360	3550	2120	3180	1780	2680	1500	2260	1380	2080	1310	1970
27	2280	3420	2040	3070	1720	2580	1450	2180	1330	2000	1260	1900
28	2200	3300	1970	2960	1650	2490	1400	2100	1280	1930	1220	1830
29	2120	3190	1900	2860	1600	2400	1350	2030	1240	1860	1180	1770
30	2050	3080	1840	2760	1540	2320	1300	1960	1200	1800	1140	1710
32	1920	2890	1720	2590	1450	2180	1220	1840	1120	1690	1070	1600
34	1810	2720	1620	2440	1360	2050	1150	1730	1060	1590	1000	1510
36	1710	2570	1530	2300	1290	1930	1090	1630	998	1500	948	1430
38	1620	2430	1450	2180	1220	1830	1030	1550	945	1420	898	1350
40	1540	2310	1380	2070	1160	1740	978	1470	898	1350	853	1280
42	1460	2200	1310	1970	1100	1660	931	1400	855	1290	813	1220
44	1400	2100	1250	1880	1050	1580	889	1340	817	1230	776	1170
46	1340	2010	1200	1800	1010	1510	850	1280	781	1170	742	1120
48	1280	1930	1150	1730	965	1450	815	1230	749	1130	711	1070
50	1230	1850	1100	1660	926	1390	782	1180	719	1080	683	1030
52	1180	1780	1060	1590	891	1340	752	1130	691	1040	656	987
54	1140	1710	1020	1530	858	1290	724	1090	665	1000	632	950
56	1100	1650	984	1480	827	1240	699	1050	642	964	609	916
58	1060	1590	950	1430	798	1200	675	1010	619	931	588	884
60	1020	1540	918	1380	772	1160	652	980	599	900	569	855
62	992	1490	889	1340	747	1120	631	948	579	871	551	827
64	961	1440	861	1290	724	1090	611	919	561	844	533	802
66	931	1400	835	1250	702	1050	593	891	544	818	517	777
68	904	1360	810	1220	681	1020	575	865	528	794	502	754
70	878	1320	787	1180	662	994	559	840	513	771	488	733
72	854	1280	765	1150	643	967	543	817	499	750	474	713

Beam Properties

ASD	LRFD	655[h] ASD	LRFD	593[h] ASD	LRFD	503[h] ASD	LRFD	431[h] ASD	LRFD	397[h] ASD	LRFD	392[h] ASD	LRFD
W_c/Ω_b	$\phi_b W_c$, kip-ft	61500	92400	55100	82800	46300	69600	39100	58800	35900	54000	34100	51300
M_p/Ω_b	$\phi_b M_p$, kip-ft	7680	11600	6890	10400	5790	8700	4890	7350	4490	6750	4270	6410
M_r/Ω_b	$\phi_b M_r$, kip-ft	4520	6800	4090	6140	3460	5200	2950	4440	2720	4100	2510	3780
BF/Ω_b	$\phi_b BF$, kips	56.1	85.3	55.4	84.4	55.3	83.1	53.6	80.4	52.4	78.4	60.8	90.8
V_n/Ω_v	$\phi_v V_n$, kips	1720	2580	1540	2310	1300	1950	1110	1660	1000	1500	1180	1770

	655[h]	593[h]	503[h]	431[h]	397[h]	392[h]
Z_x, in.^{3}	3080	2760	2320	1960	1800	1710
L_p, ft	13.6	13.4	13.1	12.9	12.9	9.33
L_r, ft	69.9	63.9	55.2	49.1	46.7	38.3

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

F_y = 50 ksi

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

W40

Shape	W40× 372[h] ASD	372[h] LRFD	362[h] ASD	362[h] LRFD	331[h] ASD	331[h] LRFD	327[h] ASD	327[h] LRFD	324 ASD	324 LRFD	297 ASD	297 LRFD
Span, ft 14					1990	2990	1930	2890				
15					1900	2860	1880	2820				
16					1780	2680	1760	2640				
17	1880	2830			1680	2520	1660	2490			1480	2220
18	1860	2800	1820	2730	1590	2380	1560	2350	1610	2410	1470	2220
19	1760	2650	1720	2590	1500	2260	1480	2230	1530	2310	1400	2100
20	1680	2520	1640	2460	1430	2150	1410	2120	1460	2190	1330	2000
21	1600	2400	1560	2340	1360	2040	1340	2010	1390	2090	1260	1900
22	1520	2290	1490	2240	1300	1950	1280	1920	1320	1990	1210	1810
23	1460	2190	1420	2140	1240	1870	1220	1840	1270	1900	1150	1730
24	1400	2100	1360	2050	1190	1790	1170	1760	1210	1830	1110	1660
25	1340	2020	1310	1970	1140	1720	1130	1690	1170	1750	1060	1600
26	1290	1940	1260	1890	1100	1650	1080	1630	1120	1680	1020	1530
27	1240	1870	1210	1820	1060	1590	1040	1570	1080	1620	983	1480
28	1200	1800	1170	1760	1020	1530	1010	1510	1040	1560	948	1430
29	1160	1740	1130	1700	984	1480	970	1460	1000	1510	915	1380
30	1120	1680	1090	1640	951	1430	938	1410	971	1460	885	1330
32	1050	1580	1020	1540	892	1340	879	1320	911	1370	830	1250
34	986	1480	963	1450	839	1260	828	1240	857	1290	781	1170
36	931	1400	909	1370	793	1190	782	1180	809	1220	737	1110
38	882	1330	861	1290	751	1130	741	1110	767	1150	699	1050
40	838	1260	818	1230	714	1070	704	1060	729	1100	664	998
42	798	1200	779	1170	680	1020	670	1010	694	1040	632	950
44	762	1150	744	1120	649	975	640	961	662	995	603	907
46	729	1100	712	1070	620	933	612	920	634	952	577	867
48	699	1050	682	1030	595	894	586	881	607	913	553	831
50	671	1010	655	984	571	858	563	846	583	876	531	798
52	645	969	630	946	549	825	541	813	560	842	511	767
54	621	933	606	911	529	794	521	783	540	811	492	739
56	599	900	585	879	510	766	503	755	520	782	474	713
58	578	869	564	848	492	740	485	729	502	755	458	688
60	559	840	546	820	476	715	469	705	486	730	442	665
62	541	813	528	794	460	692	454	682	470	706	428	644
64	524	788	511	769	446	670	440	661	455	684	415	623
66	508	764	496	745	432	650	426	641	442	664	402	605
68	493	741	481	724	420	631	414	622	429	644	390	587
70	479	720	468	703	408	613	402	604	416	626	379	570
72	466	700	455	683	396	596	391	588	405	608	369	554
Beam Properties												
W_c/Ω_b / $\phi_b W_c$, kip-ft	33500	50400	32700	49200	28500	42900	28100	42300	29100	43800	26500	39900
M_p/Ω_b / $\phi_b M_p$, kip-ft	4190	6300	4090	6150	3570	5360	3520	5290	3640	5480	3320	4990
M_r/Ω_b / $\phi_b M_r$, kip-ft	2550	3830	2480	3730	2110	3180	2100	3150	2240	3360	2040	3070
BF/Ω_b / $\phi_b BF$, kips	51.7	77.9	51.4	77.3	59.1	88.2	58.0	87.4	49.0	74.1	47.8	71.6
V_n/Ω_v / $\phi_v V_n$, kips	942	1410	909	1360	996	1490	963	1440	804	1210	740	1110
Z_x, in.^{3}	1680		1640		1430		1410		1460		1330	
L_p, ft	12.7		12.7		9.08		9.11		12.6		12.5	
L_r, ft	44.4		44.0		33.8		33.6		41.2		39.3	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

W40

F_y = 50 ksi

Shape	W40×											
	294		278		277		264		249		235	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 14	1710	2570	1660	2480			1540	2300				
15	1690	2540	1580	2380			1500	2260			1320	1980
16	1580	2380	1480	2230			1410	2120			1260	1890
17	1490	2240	1400	2100			1330	1990			1190	1780
18	1410	2120	1320	1980	1320	1980	1250	1880			1120	1680
19	1330	2010	1250	1880	1310	1970	1190	1780	1180	1770	1060	1590
20	1270	1910	1190	1790	1250	1880	1130	1700	1120	1680	1010	1520
21	1210	1810	1130	1700	1190	1790	1070	1610	1060	1600	960	1440
22	1150	1730	1080	1620	1130	1700	1030	1540	1020	1530	916	1380
23	1100	1660	1030	1550	1080	1630	981	1470	972	1460	877	1320
24	1060	1590	990	1490	1040	1560	940	1410	931	1400	840	1260
25	1010	1520	950	1430	998	1500	902	1360	894	1340	806	1210
26	975	1470	914	1370	960	1440	867	1300	860	1290	775	1170
27	939	1410	880	1320	924	1390	835	1260	828	1240	747	1120
28	905	1360	848	1280	891	1340	806	1210	798	1200	720	1080
29	874	1310	819	1230	860	1290	778	1170	771	1160	695	1040
30	845	1270	792	1190	832	1250	752	1130	745	1120	672	1010
32	792	1190	742	1120	780	1170	705	1060	699	1050	630	947
34	746	1120	699	1050	734	1100	663	997	658	988	593	891
36	704	1060	660	992	693	1040	627	942	621	933	560	842
38	667	1000	625	939	657	987	594	892	588	884	531	797
40	634	953	594	893	624	938	564	848	559	840	504	758
42	604	907	566	850	594	893	537	807	532	800	480	721
44	576	866	540	811	567	852	513	770	508	764	458	689
46	551	828	516	776	542	815	490	737	486	730	438	659
48	528	794	495	744	520	781	470	706	466	700	420	631
50	507	762	475	714	499	750	451	678	447	672	403	606
52	487	733	457	687	480	721	434	652	430	646	388	583
54	469	706	440	661	462	694	418	628	414	622	373	561
56	453	680	424	638	446	670	403	605	399	600	360	541
58	437	657	410	616	430	647	389	584	385	579	348	522
60	422	635	396	595	416	625	376	565	373	560	336	505
62	409	615	383	576	402	605	364	547	361	542	325	489
64	396	595	371	558	390	586	352	530	349	525	315	473
66	384	577	360	541	378	568	342	514	339	509	305	459
68	373	560	349	525	367	551	332	499	329	494	296	446
70	362	544	339	510	356	536	322	484	319	480	288	433
72	352	529	330	496	347	521	313	471	310	467	280	421
Beam Properties												
W_c/Ω_b / $\phi_b W_c$, kip-ft	25300	38100	23800	35700	25000	37500	22600	33900	22400	33600	20200	30300
M_p/Ω_b / $\phi_b M_p$, kip-ft	3170	4760	2970	4460	3120	4690	2820	4240	2790	4200	2520	3790
M_r/Ω_b / $\phi_b M_r$, kip-ft	1890	2840	1780	2680	1920	2890	1700	2550	1730	2610	1530	2300
BF/Ω_b / $\phi_b BF$, kips	56.9	85.4	55.3	82.8	45.8	68.7	53.8	81.3	42.9	64.4	51.0	76.7
V_n/Ω_v / $\phi_v V_n$, kips	856	1280	828	1240	659	989	768	1150	591	887	659	989
Z_x, in.^{3}	1270		1190		1250		1130		1120		1010	
L_p, ft	9.01		8.90		12.6		8.90		12.5		8.97	
L_r, ft	31.5		30.4		38.8		29.7		37.2		28.4	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

F_y = 50 ksi

W40

Shape	W40× 215		211		199		183		167		149[v]	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 13									1000	1510	865	1300
14									988	1490	853	1280
15			1180	1770			1010	1520	922	1390	796	1200
16			1130	1700			966	1450	865	1300	746	1120
17			1060	1600	1010	1510	909	1370	814	1220	702	1060
18			1000	1510	964	1450	858	1290	768	1160	663	997
19	1010	1520	952	1430	913	1370	813	1220	728	1090	628	944
20	962	1450	904	1360	867	1300	772	1160	692	1040	597	897
21	916	1380	861	1290	826	1240	736	1110	659	990	568	854
22	875	1310	822	1240	788	1190	702	1060	629	945	543	815
23	837	1260	786	1180	754	1130	672	1010	601	904	519	780
24	802	1210	753	1130	723	1090	644	968	576	866	497	748
25	770	1160	723	1090	694	1040	618	929	553	832	477	718
26	740	1110	696	1050	667	1000	594	893	532	800	459	690
27	713	1070	670	1010	642	966	572	860	512	770	442	664
28	687	1030	646	971	619	931	552	829	494	743	426	641
29	664	997	624	937	598	899	533	801	477	717	412	619
30	641	964	603	906	578	869	515	774	461	693	398	598
32	601	904	565	849	542	815	483	726	432	650	373	561
34	566	851	532	799	510	767	454	683	407	611	351	528
36	534	803	502	755	482	724	429	645	384	578	332	498
38	506	761	476	715	456	686	407	611	364	547	314	472
40	481	723	452	680	434	652	386	581	346	520	298	449
42	458	689	431	647	413	621	368	553	329	495	284	427
44	437	657	411	618	394	593	351	528	314	473	271	408
46	418	629	393	591	377	567	336	505	301	452	259	390
48	401	603	377	566	361	543	322	484	288	433	249	374
50	385	578	362	544	347	521	309	464	277	416	239	359
52	370	556	348	523	334	501	297	447	266	400	230	345
54	356	536	335	503	321	483	286	430	256	385	221	332
56	344	516	323	485	310	466	276	415	247	371	213	320
58	332	499	312	469	299	449	266	400	238	358	206	309
60	321	482	301	453	289	435	257	387	231	347	199	299
62	310	466	292	438	280	420	249	375	223	335	193	289
64	301	452	283	425	271	407	241	363	216	325	187	280
66	292	438	274	412	263	395	234	352	210	315	181	272
68	283	425	266	400	255	383	227	341	203	306	176	264
70	275	413	258	388	248	372	221	332	198	297	171	256
72	267	402	251	378	241	362	215	323	192	289	166	249

Beam Properties

ASD	LRFD	215 ASD	215 LRFD	211 ASD	211 LRFD	199 ASD	199 LRFD	183 ASD	183 LRFD	167 ASD	167 LRFD	149[v] ASD	149[v] LRFD
W_c/Ω_b	$\phi_b W_c$, kip-ft	19200	28900	18100	27200	17300	26100	15400	23200	13800	20800	11900	17900
M_p/Ω_b	$\phi_b M_p$, kip-ft	2410	3620	2260	3400	2170	3260	1930	2900	1730	2600	1490	2240
M_r/Ω_b	$\phi_b M_r$, kip-ft	1500	2250	1370	2060	1340	2020	1180	1770	1050	1580	896	1350
BF/Ω_b	$\phi_b BF$, kips	39.4	59.3	48.6	73.1	37.6	56.1	44.1	66.5	41.7	62.5	38.3	57.4
V_n/Ω_v	$\phi_v V_n$, kips	507	761	591	887	503	755	507	761	502	753	432	650

Property	215	211	199	183	167	149[v]
Z_x, in.^{3}	964	906	869	774	693	598
L_p, ft	12.5	8.87	12.2	8.80	8.48	8.09
L_r, ft	35.6	27.2	34.3	25.8	24.8	23.6

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.

Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

W36

Table 3-6 (continued) Maximum Total Uniform Load, kips W-Shapes

F_y = 50 ksi

Shape	W36×							
	925[h]		853[h]		802[h]		723[h]	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 15	5210	7810						
16	5150	7740						
17	4850	7290						
18	4580	6880	4340	6520	4060	6080	3630	5440
19	4340	6520	4120	6190	3840	5780	3440	5160
20	4120	6200	3910	5880	3650	5490	3260	4910
21	3930	5900	3730	5600	3480	5230	3110	4670
22	3750	5630	3560	5350	3320	4990	2970	4460
23	3580	5390	3400	5110	3180	4770	2840	4270
24	3430	5160	3260	4900	3040	4580	2720	4090
25	3300	4960	3130	4700	2920	4390	2610	3920
26	3170	4770	3010	4520	2810	4220	2510	3770
27	3050	4590	2900	4360	2710	4070	2420	3630
28	2940	4430	2790	4200	2610	3920	2330	3500
29	2840	4270	2700	4060	2520	3790	2250	3380
30	2750	4130	2610	3920	2440	3660	2180	3270
32	2580	3870	2450	3680	2280	3430	2040	3070
34	2420	3640	2300	3460	2150	3230	1920	2890
36	2290	3440	2170	3270	2030	3050	1810	2730
38	2170	3260	2060	3090	1920	2890	1720	2580
40	2060	3100	1960	2940	1830	2750	1630	2450
42	1960	2950	1860	2800	1740	2610	1550	2340
44	1870	2820	1780	2670	1660	2500	1480	2230
46	1790	2690	1700	2560	1590	2390	1420	2130
48	1720	2580	1630	2450	1520	2290	1360	2040
50	1650	2480	1560	2350	1460	2200	1310	1960
52	1590	2380	1500	2260	1400	2110	1260	1890
54	1530	2290	1450	2180	1350	2030	1210	1820
56	1470	2210	1400	2100	1300	1960	1170	1750
58	1420	2140	1350	2030	1260	1890	1130	1690
60	1370	2070	1300	1960	1220	1830	1090	1640
62	1330	2000	1260	1900	1180	1770	1050	1580
64	1290	1940	1220	1840	1140	1720	1020	1530
66	1250	1880	1190	1780	1110	1660	989	1490
68	1210	1820	1150	1730	1070	1610	960	1440
70	1180	1770	1120	1680	1040	1570	932	1400
72	1140	1720	1090	1630	1010	1530	907	1360

Beam Properties

ASD	LRFD	925 ASD	925 LRFD	853 ASD	853 LRFD	802 ASD	802 LRFD	723 ASD	723 LRFD
W_c/Ω_b	$\phi_b W_c$, kip-ft	82400	124000	78200	118000	73100	110000	65300	98100
M_p/Ω_b	$\phi_b M_p$, kip-ft	10300	15500	9780	14700	9130	13700	8160	12300
M_r/Ω_b	$\phi_b M_r$, kip-ft	5920	8900	5680	8530	5310	7980	4790	7190
BF/Ω_b	$\phi_b BF$, kips	47.6	71.7	48.3	72.7	48.0	71.9	47.6	72.2
V_n/Ω_v	$\phi_v V_n$, kips	2600	3900	2170	3260	2030	3040	1810	2720
Z_x, in.^{3}		4130		3920		3660		3270	
L_p, ft		15.0		15.1		14.9		14.7	
L_r, ft		107		100		94.5		85.5	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

F_y = 50 ksi

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

W36

Shape	W36× 652[h] ASD	652[h] LRFD	529[h] ASD	529[h] LRFD	487[h] ASD	487[h] LRFD	441[h] ASD	441[h] LRFD	395[h] ASD	395[h] LRFD	361[h] ASD	361[h] LRFD
Span, ft 17	3240	4860										
18	3230	4850	2560	3840	2360	3540	2110	3170	1870	2810	1700	2550
19	3060	4590	2450	3680	2240	3360	2010	3020	1800	2700	1630	2450
20	2900	4370	2330	3500	2130	3200	1910	2870	1710	2570	1550	2330
21	2770	4160	2210	3330	2020	3040	1820	2730	1630	2440	1470	2210
22	2640	3970	2110	3180	1930	2900	1730	2600	1550	2330	1410	2110
23	2530	3800	2020	3040	1850	2780	1660	2490	1480	2230	1350	2020
24	2420	3640	1940	2910	1770	2660	1590	2390	1420	2140	1290	1940
25	2320	3490	1860	2800	1700	2560	1520	2290	1370	2050	1240	1860
26	2230	3360	1790	2690	1640	2460	1470	2200	1310	1970	1190	1790
27	2150	3230	1720	2590	1570	2370	1410	2120	1260	1900	1150	1720
28	2070	3120	1660	2500	1520	2280	1360	2050	1220	1830	1100	1660
29	2000	3010	1600	2410	1470	2200	1310	1980	1180	1770	1070	1600
30	1940	2910	1550	2330	1420	2130	1270	1910	1140	1710	1030	1550
32	1820	2730	1450	2180	1330	2000	1190	1790	1070	1600	967	1450
34	1710	2570	1370	2060	1250	1880	1120	1690	1000	1510	910	1370
36	1610	2430	1290	1940	1180	1780	1060	1590	948	1430	859	1290
38	1530	2300	1220	1840	1120	1680	1000	1510	898	1350	814	1220
40	1450	2180	1160	1750	1060	1600	953	1430	853	1280	773	1160
42	1380	2080	1110	1660	1010	1520	908	1360	813	1220	737	1110
44	1320	1980	1060	1590	966	1450	866	1300	776	1170	703	1060
46	1260	1900	1010	1520	924	1390	829	1250	742	1120	673	1010
48	1210	1820	969	1460	886	1330	794	1190	711	1070	645	969
50	1160	1750	930	1400	850	1280	762	1150	683	1030	619	930
52	1120	1680	894	1340	818	1230	733	1100	656	987	595	894
54	1080	1620	861	1290	787	1180	706	1060	632	950	573	861
56	1040	1560	830	1250	759	1140	681	1020	609	916	552	830
58	1000	1510	802	1210	733	1100	657	988	588	884	533	802
60	968	1460	775	1170	709	1070	635	955	569	855	516	775
62	937	1410	750	1130	686	1030	615	924	551	827	499	750
64	908	1360	727	1090	664	998	596	895	533	802	483	727
66	880	1320	705	1060	644	968	578	868	517	777	469	705
68	854	1280	684	1030	625	940	561	843	502	754	455	684
70	830	1250	664	999	607	913	545	819	488	733	442	664
72	807	1210	646	971	590	888	529	796	474	713	430	646
Beam Properties												
W_c/Ω_b / $\phi_b W_c$, kip-ft	58100	87300	46500	69900	42500	63900	38100	57300	34100	51300	30900	46500
M_p/Ω_b / $\phi_b M_p$, kip-ft	7260	10900	5810	8740	5310	7990	4770	7160	4270	6410	3870	5810
M_r/Ω_b / $\phi_b M_r$, kip-ft	4300	6460	3480	5220	3200	4800	2880	4330	2600	3910	2360	3540
BF/Ω_b / $\phi_b BF$, kips	46.8	70.3	46.4	70.1	46.0	69.5	45.3	67.9	44.9	67.2	43.6	65.6
V_n/Ω_v / $\phi_v V_n$, kips	1620	2430	1280	1920	1180	1770	1060	1590	937	1410	851	1280
Z_x, in.^{3}	2910		2330		2130		1910		1710		1550	
L_p, ft	14.5		14.1		14.0		13.8		13.7		13.6	
L_r, ft	77.7		64.3		59.9		55.5		50.9		48.2	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

W36

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

F_y = 50 ksi

Shape	W36×											
	330		302		282		262		247		231	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 17							1240	1860	1170	1760	1110	1660
18	1540	2310	1410	2110	1310	1970	1220	1830	1140	1720	1070	1610
19	1480	2230	1340	2020	1250	1880	1160	1740	1080	1630	1010	1520
20	1410	2120	1280	1920	1190	1790	1100	1650	1030	1550	961	1440
21	1340	2010	1220	1830	1130	1700	1050	1570	979	1470	915	1380
22	1280	1920	1160	1750	1080	1620	998	1500	934	1400	874	1310
23	1220	1840	1110	1670	1030	1550	955	1430	894	1340	836	1260
24	1170	1760	1060	1600	990	1490	915	1380	857	1290	801	1200
25	1130	1690	1020	1540	950	1430	878	1320	822	1240	769	1160
26	1080	1630	983	1480	914	1370	844	1270	791	1190	739	1110
27	1040	1570	946	1420	880	1320	813	1220	761	1140	712	1070
28	1010	1510	912	1370	848	1280	784	1180	734	1100	686	1030
29	970	1460	881	1320	819	1230	757	1140	709	1070	663	996
30	938	1410	852	1280	792	1190	732	1100	685	1030	641	963
32	879	1320	798	1200	742	1120	686	1030	642	966	601	903
34	828	1240	751	1130	699	1050	646	971	605	909	565	850
36	782	1180	710	1070	660	992	610	917	571	858	534	803
38	741	1110	672	1010	625	939	578	868	541	813	506	760
40	704	1060	639	960	594	893	549	825	514	773	481	722
42	670	1010	608	914	566	850	523	786	489	736	458	688
44	640	961	581	873	540	811	499	750	467	702	437	657
46	612	920	555	835	516	776	477	717	447	672	418	628
48	586	881	532	800	495	744	457	688	428	644	400	602
50	563	846	511	768	475	714	439	660	411	618	384	578
52	541	813	491	738	457	687	422	635	395	594	370	556
54	521	783	473	711	440	661	407	611	381	572	356	535
56	503	755	456	686	424	638	392	589	367	552	343	516
58	485	729	440	662	410	616	379	569	354	533	331	498
60	469	705	426	640	396	595	366	550	343	515	320	482
62	454	682	412	619	383	576	354	532	332	498	310	466
64	440	661	399	600	371	558	343	516	321	483	300	451
66	426	641	387	582	360	541	333	500	311	468	291	438
68	414	622	376	565	349	525	323	485	302	454	283	425
70	402	604	365	549	339	510	314	471	294	441	275	413
72	391	588	355	533	330	496	305	458	286	429	267	401
Beam Properties												
W_c/Ω_b / $\phi_b W_c$, kip-ft	28100	42300	25500	38400	23800	35700	22000	33000	20600	30900	19200	28900
M_p/Ω_b / $\phi_b M_p$, kip-ft	3520	5290	3190	4800	2970	4460	2740	4130	2570	3860	2400	3610
M_r/Ω_b / $\phi_b M_r$, kip-ft	2170	3260	1970	2970	1830	2760	1700	2550	1590	2400	1490	2240
BF/Ω_b / $\phi_b BF$, kips	42.2	63.4	40.5	60.8	39.6	59.0	38.1	57.9	37.4	55.7	35.7	53.7
V_n/Ω_v / $\phi_v V_n$, kips	769	1150	705	1060	657	985	620	930	587	881	555	832
Z_x, in.^{3}	1410		1280		1190		1100		1030		963	
L_p, ft	13.5		13.5		13.4		13.3		13.2		13.1	
L_r, ft	45.5		43.6		42.2		40.6		39.4		38.6	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

Table 3-6 (continued) Maximum Total Uniform Load, kips

W-Shapes

F_y = 50 ksi

W36

Shape	W36× 256		232		210		194		182		170	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 13					1220	1830	1120	1680	1050	1580	985	1480
14	1440	2150	1290	1940	1190	1790	1090	1640	1020	1540	952	1430
15	1380	2080	1250	1870	1110	1670	1020	1530	955	1440	889	1340
16	1300	1950	1170	1760	1040	1560	957	1440	896	1350	833	1250
17	1220	1840	1100	1650	978	1470	901	1350	843	1270	784	1180
18	1150	1730	1040	1560	924	1390	851	1280	796	1200	741	1110
19	1090	1640	983	1480	875	1320	806	1210	754	1130	702	1050
20	1040	1560	934	1400	831	1250	765	1150	717	1080	667	1000
21	988	1490	890	1340	792	1190	729	1100	682	1030	635	954
22	944	1420	849	1280	756	1140	696	1050	651	979	606	911
23	903	1360	812	1220	723	1090	666	1000	623	937	580	871
24	865	1300	778	1170	693	1040	638	959	597	898	556	835
25	830	1250	747	1120	665	1000	612	920	573	862	533	802
26	798	1200	719	1080	639	961	589	885	551	828	513	771
27	769	1160	692	1040	616	926	567	852	531	798	494	742
28	741	1110	667	1000	594	893	547	822	512	769	476	716
29	716	1080	644	968	573	862	528	793	494	743	460	691
30	692	1040	623	936	554	833	510	767	478	718	444	668
32	649	975	584	878	520	781	478	719	448	673	417	626
34	611	918	549	826	489	735	450	677	422	634	392	589
36	577	867	519	780	462	694	425	639	398	598	370	557
38	546	821	492	739	438	658	403	606	377	567	351	527
40	519	780	467	702	416	625	383	575	358	539	333	501
42	494	743	445	669	396	595	365	548	341	513	317	477
44	472	709	425	638	378	568	348	523	326	490	303	455
46	451	678	406	610	361	543	333	500	312	468	290	436
48	432	650	389	585	346	521	319	479	299	449	278	418
50	415	624	374	562	333	500	306	460	287	431	267	401
52	399	600	359	540	320	481	294	443	276	414	256	385
54	384	578	346	520	308	463	284	426	265	399	247	371
56	371	557	334	501	297	446	273	411	256	385	238	358
58	358	538	322	484	287	431	264	397	247	371	230	346
60	346	520	311	468	277	417	255	384	239	359	222	334
62	335	503	301	453	268	403	247	371	231	347	215	323
64	324	488	292	439	260	390	239	360	224	337	208	313
66	315	473	283	425	252	379	232	349	217	326	202	304
68	305	459	275	413	245	368	225	338	211	317	196	295
70	297	446	267	401	238	357	219	329	205	308	190	286
72	288	433	259	390	231	347	213	320	199	299	185	278
Beam Properties												
W_c/Ω_b / $\phi_b W_c$, kip-ft	20800	31200	18700	28100	16600	25000	15300	23000	14300	21500	13300	20000
M_p/Ω_b / $\phi_b M_p$, kip-ft	2590	3900	2340	3510	2080	3120	1910	2880	1790	2690	1670	2510
M_r/Ω_b / $\phi_b M_r$, kip-ft	1560	2350	1410	2120	1260	1890	1160	1740	1090	1640	1010	1530
BF/Ω_b / $\phi_b BF$, kips	46.5	70.0	44.8	67.0	42.3	63.4	40.4	61.4	38.9	58.4	37.8	56.1
V_n/Ω_v / $\phi_v V_n$, kips	718	1080	646	968	609	914	558	838	526	790	492	738
Z_x, in.^{3}	1040		936		833		767		718		668	
L_p, ft	9.36		9.25		9.11		9.04		9.01		8.94	
L_r, ft	31.5		30.0		28.5		27.6		27.0		26.4	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

W36–W33

F_y = 50 ksi

Shape		W36×						W33×					
		160		150		135[v]		387[h]		354[h]		318	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	12			898	1350								
	13	936	1400	892	1340	767	1150						
	14	890	1340	828	1250	726	1090						
	15	830	1250	773	1160	677	1020						
	16	778	1170	725	1090	635	954						
	17	733	1100	682	1030	598	898	1810	2720	1650	2480	1460	2200
	18	692	1040	644	968	564	848	1730	2600	1570	2370	1410	2120
	19	656	985	610	917	535	804	1640	2460	1490	2240	1330	2010
	20	623	936	580	872	508	764	1560	2340	1420	2130	1270	1910
	21	593	891	552	830	484	727	1480	2230	1350	2030	1210	1810
	22	566	851	527	792	462	694	1420	2130	1290	1940	1150	1730
	23	542	814	504	758	442	664	1350	2030	1230	1850	1100	1660
	24	519	780	483	726	423	636	1300	1950	1180	1780	1060	1590
	25	498	749	464	697	406	611	1250	1870	1130	1700	1010	1520
	26	479	720	446	670	391	587	1200	1800	1090	1640	975	1470
	27	461	693	430	646	376	566	1150	1730	1050	1580	939	1410
	28	445	669	414	623	363	545	1110	1670	1010	1520	905	1360
	29	429	646	400	601	350	527	1070	1610	977	1470	874	1310
	30	415	624	387	581	339	509	1040	1560	945	1420	845	1270
	32	389	585	362	545	317	477	973	1460	886	1330	792	1190
	34	366	551	341	513	299	449	916	1380	834	1250	746	1120
	36	346	520	322	484	282	424	865	1300	787	1180	704	1060
	38	328	493	305	459	267	402	819	1230	746	1120	667	1000
	40	311	468	290	436	254	382	778	1170	709	1070	634	953
	42	297	446	276	415	242	364	741	1110	675	1010	604	907
	44	283	425	264	396	231	347	708	1060	644	968	576	866
	46	271	407	252	379	221	332	677	1020	616	926	551	828
	48	259	390	242	363	212	318	649	975	590	888	528	794
	50	249	374	232	349	203	305	623	936	567	852	507	762
	52	240	360	223	335	195	294	599	900	545	819	487	733
	54	231	347	215	323	188	283	577	867	525	789	469	706
	56	222	334	207	311	181	273	556	836	506	761	453	680
	58	215	323	200	301	175	263	537	807	489	734	437	657
	60	208	312	193	291	169	255	519	780	472	710	422	635
	62	201	302	187	281	164	246	502	755	457	687	409	615
	64	195	293	181	272	159	239	487	731	443	666	396	595
	66	189	284	176	264	154	231	472	709	429	645	384	577
	68	183	275	171	256	149	225	458	688	417	626	373	560
	70	178	267	166	249	145	218	445	669	405	609	362	544
	72	173	260	161	242	141	212	432	650	394	592	352	529

Beam Properties

ASD	LRFD	W36×160 ASD	LRFD	W36×150 ASD	LRFD	W36×135 ASD	LRFD	W33×387 ASD	LRFD	W33×354 ASD	LRFD	W33×318 ASD	LRFD
W_c/Ω_b	$\phi_b W_c$, kip-ft	12500	18700	11600	17400	10200	15300	31100	46800	28300	42600	25300	38100
M_p/Ω_b	$\phi_b M_p$, kip-ft	1560	2340	1450	2180	1270	1910	3890	5850	3540	5330	3170	4760
M_r/Ω_b	$\phi_b M_r$, kip-ft	947	1420	880	1320	767	1150	2360	3540	2170	3260	1940	2910
BF/Ω_b	$\phi_b BF$, kips	36.1	54.2	34.4	51.9	31.7	47.8	38.3	57.8	37.4	56.6	36.8	55.4
V_n/Ω_v	$\phi_v V_n$, kips	468	702	449	673	384	577	907	1360	826	1240	732	1100

Property	W36×160	W36×150	W36×135	W33×387	W33×354	W33×318
Z_x, in.^{3}	624	581	509	1560	1420	1270
L_p, ft	8.83	8.72	8.41	13.3	13.2	13.1
L_r, ft	25.8	25.3	24.3	53.3	49.8	46.5

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.
Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

F_y = 50 ksi

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

W33

Shape	W33×											
	291		263		241		221		201		169	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 13											906	1360
14											897	1350
15											837	1260
16					1140	1700	1050	1580	964	1450	785	1180
17	1340	2000	1200	1800	1100	1660	1010	1510	908	1360	739	1110
18	1290	1930	1150	1730	1040	1570	950	1430	857	1290	697	1050
19	1220	1830	1090	1640	987	1480	900	1350	812	1220	661	993
20	1160	1740	1040	1560	938	1410	855	1290	771	1160	628	944
21	1100	1660	988	1490	893	1340	815	1220	735	1100	598	899
22	1050	1580	944	1420	853	1280	778	1170	701	1050	571	858
23	1010	1510	903	1360	816	1230	744	1120	671	1010	546	820
24	965	1450	865	1300	782	1180	713	1070	643	966	523	786
25	926	1390	830	1250	750	1130	684	1030	617	928	502	755
26	891	1340	798	1200	722	1080	658	989	593	892	483	726
27	858	1290	769	1160	695	1040	634	952	571	859	465	699
28	827	1240	741	1110	670	1010	611	918	551	828	448	674
29	798	1200	716	1080	647	972	590	887	532	800	433	651
30	772	1160	692	1040	625	940	570	857	514	773	418	629
32	724	1090	649	975	586	881	535	803	482	725	392	590
34	681	1020	611	918	552	829	503	756	454	682	369	555
36	643	967	577	867	521	783	475	714	429	644	349	524
38	609	916	546	821	494	742	450	677	406	610	330	497
40	579	870	519	780	469	705	428	643	386	580	314	472
42	551	829	494	743	447	671	407	612	367	552	299	449
44	526	791	472	709	426	641	389	584	351	527	285	429
46	503	757	451	678	408	613	372	559	335	504	273	410
48	482	725	432	650	391	588	356	536	321	483	262	393
50	463	696	415	624	375	564	342	514	309	464	251	377
52	445	669	399	600	361	542	329	494	297	446	241	363
54	429	644	384	578	347	522	317	476	286	429	232	349
56	413	621	371	557	335	504	305	459	276	414	224	337
58	399	600	358	538	323	486	295	443	266	400	216	325
60	386	580	346	520	313	470	285	429	257	387	209	315
62	373	561	335	503	303	455	276	415	249	374	202	304
64	362	544	324	488	293	441	267	402	241	362	196	295
66	351	527	315	473	284	427	259	390	234	351	190	286
68	340	512	305	459	276	415	252	378	227	341	185	278
70	331	497	297	446	268	403	244	367	220	331	179	270
72	322	483	288	433	261	392	238	357	214	322	174	262

Beam Properties

ASD	LRFD	291 ASD	291 LRFD	263 ASD	263 LRFD	241 ASD	241 LRFD	221 ASD	221 LRFD	201 ASD	201 LRFD	169 ASD	169 LRFD
W_c/Ω_b	$\phi_b W_c$, kip-ft	23200	34800	20800	31200	18800	28200	17100	25700	15400	23200	12600	18900
M_p/Ω_b	$\phi_b M_p$, kip-ft	2890	4350	2590	3900	2350	3530	2140	3210	1930	2900	1570	2360
M_r/Ω_b	$\phi_b M_r$, kip-ft	1780	2680	1610	2410	1450	2180	1330	1990	1200	1800	959	1440
BF/Ω_b	$\phi_b BF$, kips	36.0	54.2	34.1	51.9	33.2	50.2	31.8	47.8	30.3	45.6	34.2	51.5
V_n/Ω_v	$\phi_v V_n$, kips	668	1000	600	900	568	852	525	788	482	723	453	679

	291	263	241	221	201	169
Z_x, in.^{3}	1160	1040	940	857	773	629
L_p, ft	13.0	12.9	12.8	12.7	12.6	8.83
L_r, ft	43.8	41.6	39.7	38.2	36.7	26.7

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

W33–W30

F_y = 50 ksi

Shape	W33×								W30×			
	152		141		130		118[v]		391[h]		357[h]	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft												
12			806	1210	768	1150	650	977				
13	851	1280	789	1190	717	1080	637	958				
14	797	1200	733	1100	666	1000	592	889				
15	744	1120	684	1030	621	934	552	830				
16	697	1050	641	964	583	876	518	778	1810	2710	1630	2440
17	656	986	603	907	548	824	487	732	1700	2560	1550	2330
18	620	932	570	857	518	778	460	692	1610	2420	1460	2200
19	587	883	540	812	491	737	436	655	1520	2290	1390	2080
20	558	839	513	771	466	701	414	623	1450	2180	1320	1980
21	531	799	489	734	444	667	394	593	1380	2070	1250	1890
22	507	762	466	701	424	637	377	566	1320	1980	1200	1800
23	485	729	446	670	405	609	360	541	1260	1890	1150	1720
24	465	699	427	643	388	584	345	519	1210	1810	1100	1650
25	446	671	410	617	373	560	331	498	1160	1740	1050	1580
26	429	645	395	593	359	539	319	479	1110	1670	1010	1520
27	413	621	380	571	345	519	307	461	1070	1610	976	1470
28	398	599	366	551	333	500	296	445	1030	1550	941	1410
29	385	578	354	532	321	483	286	429	998	1500	909	1370
30	372	559	342	514	311	467	276	415	965	1450	878	1320
32	349	524	321	482	291	438	259	389	904	1360	823	1240
34	328	493	302	454	274	412	244	366	851	1280	775	1160
36	310	466	285	428	259	389	230	346	804	1210	732	1100
38	294	441	270	406	245	369	218	328	762	1140	693	1040
40	279	419	256	386	233	350	207	311	724	1090	659	990
42	266	399	244	367	222	334	197	296	689	1040	627	943
44	254	381	233	350	212	318	188	283	658	989	599	900
46	243	365	223	335	203	305	180	271	629	946	573	861
48	232	349	214	321	194	292	173	259	603	906	549	825
50	223	335	205	308	186	280	166	249	579	870	527	792
52	215	323	197	297	179	269	159	239	557	837	507	762
54	207	311	190	286	173	259	153	231	536	806	488	733
56	199	299	183	275	166	250	148	222	517	777	470	707
58	192	289	177	266	161	242	143	215	499	750	454	683
60	186	280	171	257	155	234	138	208	482	725	439	660
62	180	270	165	249	150	226	134	201	467	702	425	639
64	174	262	160	241	146	219	129	195	452	680	412	619
66	169	254	155	234	141	212	126	189	439	659	399	600
68	164	247	151	227	137	206	122	183	426	640	387	582
70	159	240	147	220	133	200	118	178	413	621	376	566
72	155	233	142	214	129	195	115	173	402	604	366	550
Beam Properties												
W_c/Ω_b / $\phi_b W_c$, kip-ft	11200	16800	10300	15400	9320	14000	8280	12500	28900	43500	26300	39600
M_p/Ω_b / $\phi_b M_p$, kip-ft	1390	2100	1280	1930	1170	1750	1040	1560	3620	5440	3290	4950
M_r/Ω_b / $\phi_b M_r$, kip-ft	851	1280	782	1180	709	1070	627	942	2180	3280	1990	2990
BF/Ω_b / $\phi_b BF$, kips	31.7	48.3	30.3	45.7	29.3	43.1	27.2	40.6	31.4	47.2	31.3	47.2
V_n/Ω_v / $\phi_v V_n$, kips	425	638	403	604	384	576	325	489	903	1350	813	1220
Z_x, in.^{3}	559		514		467		415		1450		1320	
L_p, ft	8.72		8.58		8.44		8.19		13.0		12.9	
L_r, ft	25.7		25.0		24.2		23.4		58.8		54.4	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.
Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

F_y = 50 ksi

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

W30

Shape	W30×											
	326[h]		292		261		235		211		191	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 15									958	1440	872	1310
16	1480	2220	1310	1960	1180	1760	1040	1560	937	1410	842	1270
17	1400	2100	1240	1870	1110	1660	994	1490	882	1330	793	1190
18	1320	1980	1180	1770	1050	1570	939	1410	833	1250	749	1130
19	1250	1880	1110	1670	991	1490	890	1340	789	1190	709	1070
20	1190	1790	1060	1590	941	1410	845	1270	750	1130	674	1010
21	1130	1700	1010	1510	896	1350	805	1210	714	1070	642	964
22	1080	1620	962	1450	856	1290	768	1160	681	1020	612	920
23	1030	1550	920	1380	818	1230	735	1100	652	980	586	880
24	990	1490	882	1330	784	1180	704	1060	625	939	561	844
25	950	1430	846	1270	753	1130	676	1020	600	901	539	810
26	914	1370	814	1220	724	1090	650	977	577	867	518	779
27	880	1320	784	1180	697	1050	626	941	555	834	499	750
28	848	1280	756	1140	672	1010	604	908	535	805	481	723
29	819	1230	730	1100	649	976	583	876	517	777	465	698
30	792	1190	705	1060	627	943	564	847	500	751	449	675
32	742	1120	661	994	588	884	528	794	468	704	421	633
34	699	1050	622	935	554	832	497	747	441	663	396	596
36	660	992	588	883	523	786	470	706	416	626	374	563
38	625	939	557	837	495	744	445	669	394	593	355	533
40	594	893	529	795	471	707	423	635	375	563	337	506
42	566	850	504	757	448	674	403	605	357	536	321	482
44	540	811	481	723	428	643	384	578	341	512	306	460
46	516	776	460	691	409	615	368	552	326	490	293	440
48	495	744	441	663	392	589	352	529	312	469	281	422
50	475	714	423	636	376	566	338	508	300	451	269	405
52	457	687	407	612	362	544	325	489	288	433	259	389
54	440	661	392	589	349	524	313	471	278	417	250	375
56	424	638	378	568	336	505	302	454	268	402	241	362
58	410	616	365	548	325	488	291	438	258	388	232	349
60	396	595	353	530	314	472	282	424	250	376	225	338
62	383	576	341	513	304	456	273	410	242	363	217	327
64	371	558	331	497	294	442	264	397	234	352	211	316
66	360	541	321	482	285	429	256	385	227	341	204	307
68	349	525	311	468	277	416	249	374	220	331	198	298
70	339	510	302	454	269	404	242	363	214	322	192	289
72	330	496	294	442	261	393	235	353	208	313	187	281
Beam Properties												
W_c/Ω_b / $\phi_b W_c$, kip-ft	23800	35700	21200	31800	18800	28300	16900	25400	15000	22500	13500	20300
M_p/Ω_b / $\phi_b M_p$, kip-ft	2970	4460	2640	3980	2350	3540	2110	3180	1870	2820	1680	2530
M_r/Ω_b / $\phi_b M_r$, kip-ft	1820	2730	1620	2440	1450	2180	1310	1960	1160	1750	1050	1580
BF/Ω_b / $\phi_b BF$, kips	30.3	45.6	29.7	44.9	29.1	44.0	28.0	42.7	26.9	40.5	25.6	38.6
V_n/Ω_v / $\phi_v V_n$, kips	739	1110	653	979	588	882	520	779	479	718	436	654
Z_x, in.^{3}	1190		1060		943		847		751		675	
L_p, ft	12.7		12.6		12.5		12.4		12.3		12.2	
L_r, ft	50.6		46.9		43.4		41.0		38.7		36.8	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

W30

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

F_y = 50 ksi

Shape	W30× 173 ASD	173 LRFD	148 ASD	148 LRFD	132 ASD	132 LRFD	124 ASD	124 LRFD	116 ASD	116 LRFD	108 ASD	108 LRFD
Span, ft												
10											650	974
11					745	1120	707	1060	678	1020	628	944
12			798	1200	727	1090	679	1020	629	945	576	865
13			768	1150	671	1010	626	942	580	872	531	798
14			713	1070	623	936	582	874	539	810	493	741
15	796	1190	665	1000	582	874	543	816	503	756	460	692
16	757	1140	624	938	545	819	509	765	472	709	432	649
17	713	1070	587	882	513	771	479	720	444	667	406	611
18	673	1010	554	833	485	728	452	680	419	630	384	577
19	638	958	525	789	459	690	429	644	397	597	363	546
20	606	911	499	750	436	656	407	612	377	567	345	519
21	577	867	475	714	415	624	388	583	359	540	329	494
22	551	828	454	682	396	596	370	556	343	515	314	472
23	527	792	434	652	379	570	354	532	328	493	300	451
24	505	759	416	625	363	546	339	510	314	473	288	433
25	485	728	399	600	349	524	326	490	302	454	276	415
26	466	700	384	577	335	504	313	471	290	436	266	399
27	449	674	370	556	323	486	302	453	279	420	256	384
28	433	650	356	536	312	468	291	437	269	405	247	371
29	418	628	344	517	301	452	281	422	260	391	238	358
30	404	607	333	500	291	437	271	408	251	378	230	346
32	379	569	312	469	273	410	254	383	236	354	216	324
34	356	536	294	441	257	386	240	360	222	334	203	305
36	337	506	277	417	242	364	226	340	210	315	192	288
38	319	479	263	395	230	345	214	322	199	298	182	273
40	303	455	250	375	218	328	204	306	189	284	173	260
42	288	434	238	357	208	312	194	291	180	270	164	247
44	275	414	227	341	198	298	185	278	171	258	157	236
46	263	396	217	326	190	285	177	266	164	247	150	226
48	252	379	208	313	182	273	170	255	157	236	144	216
50	242	364	200	300	174	262	163	245	151	227	138	208
52	233	350	192	288	168	252	157	235	145	218	133	200
54	224	337	185	278	162	243	151	227	140	210	128	192
56	216	325	178	268	156	234	145	219	135	203	123	185
58	209	314	172	259	150	226	140	211	130	196	119	179
60	202	304	166	250	145	219	136	204	126	189	115	173
62	195	294	161	242	141	211	131	197	122	183	111	167
64	189	285	156	234	136	205	127	191	118	177	108	162
66	184	276	151	227	132	199	123	185	114	172	105	157
68	178	268	147	221	128	193	120	180	111	167	102	153
70	173	260	143	214	125	187	116	175	108	162	98.7	148
72	168	253	139	208	121	182	113	170	105	158	95.9	144
Beam Properties												
W_c/Ω_b / $\phi_b W_c$, kip-ft	12100	18200	9980	15000	8720	13100	8140	12200	7540	11300	6910	10400
M_p/Ω_b / $\phi_b M_p$, kip-ft	1510	2280	1250	1880	1090	1640	1020	1530	943	1420	863	1300
M_r/Ω_b / $\phi_b M_r$, kip-ft	945	1420	761	1140	664	998	620	932	575	864	522	785
BF/Ω_b / $\phi_b BF$, kips	24.1	36.8	29.0	43.9	26.9	40.5	26.1	39.0	24.8	37.4	23.5	35.5
V_n/Ω_v / $\phi_v V_n$, kips	398	597	399	599	373	559	353	530	339	509	325	487
Z_x, in.^{3}	607		500		437		408		378		346	
L_p, ft	12.1		8.05		7.95		7.88		7.74		7.59	
L_r, ft	35.5		24.9		23.8		23.2		22.6		22.1	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

F_y = 50 ksi

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

W30–W27

Shape		W30×				W27×							
		99		90[v]		539[h]		368[h]		336[h]		307[h]	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	10	618	927										
	11	566	851	498	749								
	12	519	780	471	708								
	13	479	720	435	653								
	14	445	669	403	606	2560	3840	1680	2520	1510	2270		
	15	415	624	377	566	2510	3780	1650	2480	1500	2260	1370	2060
	16	389	585	353	531	2360	3540	1550	2330	1410	2120	1280	1930
	17	366	551	332	499	2220	3340	1460	2190	1330	1990	1210	1820
	18	346	520	314	472	2100	3150	1380	2070	1250	1880	1140	1720
	19	328	493	297	447	1990	2980	1300	1960	1190	1780	1080	1630
	20	311	468	282	425	1890	2840	1240	1860	1130	1700	1030	1550
	21	297	446	269	404	1800	2700	1180	1770	1070	1610	979	1470
	22	283	425	257	386	1710	2580	1130	1690	1030	1540	934	1400
	23	271	407	246	369	1640	2470	1080	1620	981	1470	894	1340
	24	259	390	235	354	1570	2360	1030	1550	940	1410	857	1290
	25	249	374	226	340	1510	2270	990	1490	902	1360	822	1240
	26	240	360	217	327	1450	2180	952	1430	867	1300	791	1190
	27	231	347	209	314	1400	2100	917	1380	835	1260	761	1140
	28	222	334	202	303	1350	2030	884	1330	806	1210	734	1100
	29	215	323	195	293	1300	1960	853	1280	778	1170	709	1070
	30	208	312	188	283	1260	1890	825	1240	752	1130	685	1030
	32	195	293	177	265	1180	1770	773	1160	705	1060	642	966
	34	183	275	166	250	1110	1670	728	1090	663	997	605	909
	36	173	260	157	236	1050	1580	688	1030	627	942	571	858
	38	164	246	149	223	993	1490	651	979	594	892	541	813
	40	156	234	141	212	943	1420	619	930	564	848	514	773
	42	148	223	134	202	898	1350	589	886	537	807	489	736
	44	142	213	128	193	857	1290	563	845	513	770	467	702
	46	135	203	123	185	820	1230	538	809	490	737	447	672
	48	130	195	118	177	786	1180	516	775	470	706	428	644
	50	125	187	113	170	754	1130	495	744	451	678	411	618
	52	120	180	109	163	725	1090	476	715	434	652	395	594
	54	115	173	105	157	699	1050	458	689	418	628	381	572
	56	111	167	101	152	674	1010	442	664	403	605	367	552
	58	107	161	97.4	146	650	978	427	641	389	584	354	533
	60	104	156	94.1	142	629	945	413	620	376	565	343	515
	62	100	151	91.1	137	608	915	399	600	364	547	332	498
	64	97.3	146	88.3	133	589	886	387	581	352	530	321	483
	66	94.4	142	85.6	129	572	859	375	564	342	514	311	468
	68	91.6	138	83.1	125	555	834	364	547	332	499	302	454
	70	89.0	134	80.7	121	539	810	354	531	322	484	294	441
	72	86.5	130	78.5	118	524	788	344	517	313	471	286	429
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	6230	9360	5650	8490	37700	56700	24800	37200	22600	33900	20600	30900
M_p/Ω_b	$\phi_b M_p$, kip-ft	778	1170	706	1060	4720	7090	3090	4650	2820	4240	2570	3860
M_r/Ω_b	$\phi_b M_r$, kip-ft	470	706	428	643	2740	4120	1850	2780	1700	2550	1550	2330
BF/Ω_b	$\phi_b BF$, kips	22.2	33.4	20.6	30.8	26.2	39.3	24.9	37.6	25.0	37.7	25.1	37.7
V_n/Ω_v	$\phi_v V_n$, kips	309	463	249	374	1280	1920	839	1260	756	1130	687	1030
Z_x, in.^{3}		312		283		1890		1240		1130		1030	
L_p, ft		7.42		7.38		12.9		12.3		12.2		12.0	
L_r, ft		21.3		20.9		88.5		62.0		57.0		52.6	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 50 ksi; therefore, ϕ_v = 0.90 and Ω_v = 1.67.
Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

W27

F_y = 50 ksi

Shape		W27×											
		281		258		235		217		194		178	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	14			1140	1710	1040	1570			843	1260	806	1210
	15	1240	1860	1130	1700	1030	1540	943	1410	840	1260	758	1140
	16	1170	1760	1060	1600	963	1450	887	1330	787	1180	711	1070
	17	1100	1650	1000	1500	906	1360	835	1250	741	1110	669	1010
	18	1040	1560	945	1420	856	1290	788	1190	700	1050	632	950
	19	983	1480	895	1350	811	1220	747	1120	663	996	599	900
	20	934	1400	850	1280	770	1160	710	1070	630	947	569	855
	21	890	1340	810	1220	734	1100	676	1020	600	901	542	814
	22	849	1280	773	1160	700	1050	645	970	572	860	517	777
	23	812	1220	739	1110	670	1010	617	927	548	823	495	743
	24	778	1170	709	1070	642	965	591	889	525	789	474	713
	25	747	1120	680	1020	616	926	568	853	504	757	455	684
	26	719	1080	654	983	593	891	546	820	484	728	438	658
	27	692	1040	630	947	571	858	526	790	466	701	421	633
	28	667	1000	607	913	550	827	507	762	450	676	406	611
	29	644	968	586	881	531	799	489	736	434	653	392	590
	30	623	936	567	852	514	772	473	711	420	631	379	570
	32	584	878	531	799	482	724	443	667	394	592	356	534
	34	549	826	500	752	453	681	417	627	370	557	335	503
	36	519	780	472	710	428	643	394	593	350	526	316	475
	38	492	739	448	673	406	609	373	561	331	498	299	450
	40	467	702	425	639	385	579	355	533	315	473	284	428
	42	445	669	405	609	367	551	338	508	300	451	271	407
	44	425	638	386	581	350	526	323	485	286	430	259	389
	46	406	610	370	556	335	503	309	464	274	412	247	372
	48	389	585	354	533	321	483	296	444	262	394	237	356
	50	374	562	340	511	308	463	284	427	252	379	228	342
	52	359	540	327	492	296	445	273	410	242	364	219	329
	54	346	520	315	473	285	429	263	395	233	351	211	317
	56	334	501	304	456	275	414	253	381	225	338	203	305
	58	322	484	293	441	266	399	245	368	217	326	196	295
	60	311	468	283	426	257	386	237	356	210	316	190	285
	62	301	453	274	412	249	374	229	344	203	305	184	276
	64	292	439	266	399	241	362	222	333	197	296	178	267
	66	283	425	258	387	233	351	215	323	191	287	172	259
	68	275	413	250	376	227	341	209	314	185	278	167	251
	70	267	401	243	365	220	331	203	305	180	270		
	72	259	390	236	355								
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	18700	28100	17000	25600	15400	23200	14200	21300	12600	18900	11400	17100
M_p/Ω_b	$\phi_b M_p$, kip-ft	2340	3510	2130	3200	1930	2900	1770	2670	1570	2370	1420	2140
M_r/Ω_b	$\phi_b M_r$, kip-ft	1420	2140	1300	1960	1180	1780	1100	1650	976	1470	882	1330
BF/Ω_b	$\phi_b BF$, kips	24.8	36.9	24.4	36.5	24.1	36.0	23.0	35.1	22.3	33.8	21.6	32.5
V_n/Ω_v	$\phi_v V_n$, kips	621	932	568	853	522	784	471	707	422	632	403	605
Z_x, in.^{3}		936		852		772		711		631		570	
L_p, ft		12.0		11.9		11.8		11.7		11.6		11.5	
L_r, ft		49.1		45.9		42.9		40.8		38.2		36.4	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

F_y = 50 ksi

W27

Shape	W27×											
	161		146		129		114		102		94	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 10									558	837	527	791
11					673	1010	622	934	553	832	504	758
12					657	988	571	858	507	763	462	695
13			663	995	606	912	527	792	468	704	427	642
14	729	1090	662	994	563	846	489	735	435	654	396	596
15	685	1030	617	928	526	790	456	686	406	610	370	556
16	642	966	579	870	493	741	428	643	380	572	347	521
17	605	909	545	819	464	697	403	605	358	538	326	491
18	571	858	515	773	438	658	380	572	338	508	308	463
19	541	813	487	733	415	624	360	542	320	482	292	439
20	514	773	463	696	394	593	342	515	304	458	277	417
21	489	736	441	663	375	564	326	490	290	436	264	397
22	467	702	421	633	358	539	311	468	277	416	252	379
23	447	672	403	605	343	515	298	447	265	398	241	363
24	428	644	386	580	329	494	285	429	254	381	231	348
25	411	618	370	557	315	474	274	412	244	366	222	334
26	395	594	356	535	303	456	263	396	234	352	213	321
27	381	572	343	516	292	439	254	381	225	339	206	309
28	367	552	331	497	282	423	245	368	217	327	198	298
29	354	533	319	480	272	409	236	355	210	316	191	288
30	343	515	309	464	263	395	228	343	203	305	185	278
32	321	483	289	435	246	370	214	322	190	286	173	261
34	302	454	272	409	232	349	201	303	179	269	163	245
36	286	429	257	387	219	329	190	286	169	254	154	232
38	271	407	244	366	207	312	180	271	160	241	146	219
40	257	386	232	348	197	296	171	257	152	229	139	209
42	245	368	221	331	188	282	163	245	145	218	132	199
44	234	351	210	316	179	269	156	234	138	208	126	190
46	223	336	201	303	171	258	149	224	132	199	121	181
48	214	322	193	290	164	247	143	214	127	191	116	174
50	206	309	185	278	158	237	137	206	122	183	111	167
52	198	297	178	268	152	228	132	198	117	176	107	160
54	190	286	172	258	146	219	127	191	113	169	103	154
56	184	276	165	249	141	212	122	184	109	163	99.1	149
58	177	266	160	240	136	204	118	177	105	158	95.7	144
60	171	258	154	232	131	198	114	172	101	153	92.5	139
62	166	249	149	225	127	191	110	166	98.2	148	89.5	135
64	161	241	145	218	123	185	107	161	95.1	143	86.7	130
66	156	234	140	211	119	180	104	156	92.2	139	84.1	126
68	151	227	136	205	116	174	101	151				
Beam Properties												
W_c/Ω_b / $\phi_b W_c$, kip-ft	10300	15500	9260	13900	7880	11900	6850	10300	6090	9150	5550	8340
M_p/Ω_b / $\phi_b M_p$, kip-ft	1280	1930	1160	1740	986	1480	856	1290	761	1140	694	1040
M_r/Ω_b / $\phi_b M_r$, kip-ft	800	1200	723	1090	603	906	522	785	466	701	424	638
BF/Ω_b / $\phi_b BF$, kips	20.6	31.3	19.9	29.5	23.4	35.0	21.7	32.8	20.1	29.8	19.1	28.5
V_n/Ω_v / $\phi_v V_n$, kips	364	546	332	497	337	505	311	467	279	419	264	395
Z_x, in.^{3}	515		464		395		343		305		278	
L_p, ft	11.4		11.3		7.81		7.70		7.59		7.49	
L_r, ft	34.7		33.3		24.2		23.1		22.3		21.6	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

W27–W24

F_y = 50 ksi

Shape		W27×		W24×									
		84		370[h]		335[h]		306[h]		279[h]		250	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	9	491	737										
	10	487	732										
	11	443	665										
	12	406	610										
	13	375	563	1700	2550	1520	2280	1370	2050	1240	1860	1090	1640
	14	348	523	1610	2420	1450	2190	1310	1980	1190	1790	1060	1590
	15	325	488	1500	2260	1360	2040	1230	1840	1110	1670	990	1490
	16	304	458	1410	2120	1270	1910	1150	1730	1040	1570	928	1400
	17	286	431	1330	1990	1200	1800	1080	1630	980	1470	874	1310
	18	271	407	1250	1880	1130	1700	1020	1540	926	1390	825	1240
	19	256	385	1190	1780	1070	1610	969	1460	877	1320	782	1170
	20	244	366	1130	1700	1020	1530	920	1380	833	1250	743	1120
	21	232	349	1070	1610	969	1460	876	1320	794	1190	707	1060
	22	221	333	1030	1540	925	1390	837	1260	758	1140	675	1010
	23	212	318	981	1470	885	1330	800	1200	725	1090	646	970
	24	203	305	940	1410	848	1280	767	1150	694	1040	619	930
	25	195	293	902	1360	814	1220	736	1110	667	1000	594	893
	26	187	282	867	1300	783	1180	708	1060	641	963	571	858
	27	180	271	835	1260	754	1130	682	1020	617	928	550	827
	28	174	261	806	1210	727	1090	657	988	595	895	530	797
	29	168	252	778	1170	702	1060	635	954	575	864	512	770
	30	162	244	752	1130	679	1020	613	922	556	835	495	744
	32	152	229	705	1060	636	956	575	864	521	783	464	698
	34	143	215	663	997	599	900	541	814	490	737	437	656
	36	135	203	627	942	566	850	511	768	463	696	413	620
	38	128	193	594	892	536	805	484	728	439	659	391	587
	40	122	183	564	848	509	765	460	692	417	626	371	558
	42	116	174	537	807	485	729	438	659	397	596	354	531
	44	111	166	513	770	463	695	418	629	379	569	338	507
	46	106	159	490	737	443	665	400	601	362	545	323	485
	48	101	153	470	706	424	638	383	576	347	522	309	465
	50	97.4	146	451	678	407	612	368	553	333	501	297	446
	52	93.7	141	434	652	392	588	354	532	321	482	286	429
	54	90.2	136	418	628	377	567	341	512	309	464	275	413
	56	87.0	131	403	605	364	546	329	494	298	447	265	399
	58	84.0	126	389	584	351	528	317	477	287	432	256	385
	60	81.2	122	376	565	339	510	307	461	278	418	248	372
	62	78.6	118	364	547	328	494	297	446	269	404	240	360
	64	76.1	114	352	530	318	478	288	432	260	391	232	349
	66	73.8	111	342	514	308	464	279	419	253	380		
	68			332	499	299	450						
	70			322	484								
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	4870	7320	22600	33900	20400	30600	18400	27700	16700	25100	14900	22300
M_p/Ω_b	$\phi_b M_p$, kip-ft	609	915	2820	4240	2540	3830	2300	3460	2080	3130	1860	2790
M_r/Ω_b	$\phi_b M_r$, kip-ft	372	559	1670	2510	1510	2270	1380	2070	1250	1880	1120	1690
BF/Ω_b	$\phi_b BF$, kips	17.6	26.4	20.0	30.0	19.9	30.2	19.7	29.8	19.7	29.6	19.7	29.3
V_n/Ω_v	$\phi_v V_n$, kips	246	368	851	1280	759	1140	683	1020	619	929	547	821
Z_x, in.^{3}		244		1130		1020		922		835		744	
L_p, ft		7.31		11.6		11.4		11.3		11.2		11.1	
L_r, ft		20.8		69.2		63.1		57.9		53.4		48.7	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

F_y = 50 ksi

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

W24

Shape	W24×											
	229		207		192		176		162		146	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 13	998	1500	894	1340	826	1240	756	1130	705	1060	642	963
14	962	1450	864	1300	797	1200	729	1100	667	1000	596	896
15	898	1350	806	1210	744	1120	680	1020	623	936	556	836
16	842	1270	756	1140	697	1050	637	958	584	878	521	784
17	793	1190	712	1070	656	986	600	902	549	826	491	738
18	749	1130	672	1010	620	932	567	852	519	780	464	697
19	709	1070	637	957	587	883	537	807	492	739	439	660
20	674	1010	605	909	558	839	510	767	467	702	417	627
21	642	964	576	866	531	799	486	730	445	669	397	597
22	612	920	550	826	507	762	464	697	425	638	379	570
23	586	880	526	790	485	729	443	667	406	610	363	545
24	561	844	504	758	465	699	425	639	389	585	348	523
25	539	810	484	727	446	671	408	613	374	562	334	502
26	518	779	465	699	429	645	392	590	359	540	321	482
27	499	750	448	673	413	621	378	568	346	520	309	464
28	481	723	432	649	398	599	364	548	334	501	298	448
29	465	698	417	627	385	578	352	529	322	484	288	432
30	449	675	403	606	372	559	340	511	311	468	278	418
32	421	633	378	568	349	524	319	479	292	439	261	392
34	396	596	356	535	328	493	300	451	275	413	245	369
36	374	563	336	505	310	466	283	426	259	390	232	348
38	355	533	318	478	294	441	268	403	246	369	220	330
40	337	506	302	455	279	419	255	383	234	351	209	314
42	321	482	288	433	266	399	243	365	222	334	199	299
44	306	460	275	413	254	381	232	348	212	319	190	285
46	293	440	263	395	243	365	222	333	203	305	181	273
48	281	422	252	379	232	349	212	319	195	293	174	261
50	269	405	242	364	223	335	204	307	187	281	167	251
52	259	389	233	350	215	323	196	295	180	270	160	241
54	250	375	224	337	207	311	189	284	173	260	155	232
56	241	362	216	325	199	299	182	274	167	251	149	224
58	232	349	209	313	192	289	176	264	161	242	144	216
60	225	338	202	303	186	280	170	256	156	234	139	209
62	217	327	195	293	180	270	165	247	151	226		
64	211	316	189	284								
Beam Properties												
W_c/Ω_b / $\phi_b W_c$, kip-ft	13500	20300	12100	18200	11200	16800	10200	15300	9340	14000	8340	12500
M_p/Ω_b / $\phi_b M_p$, kip-ft	1680	2530	1510	2270	1390	2100	1270	1920	1170	1760	1040	1570
M_r/Ω_b / $\phi_b M_r$, kip-ft	1030	1540	927	1390	858	1290	786	1180	723	1090	648	974
BF/Ω_b / $\phi_b BF$, kips	19.0	28.9	18.9	28.6	18.4	28.0	18.1	27.7	17.9	26.8	17.0	25.8
V_n/Ω_v / $\phi_v V_n$, kips	499	749	447	671	413	620	378	567	353	529	321	482
Z_x, in.^{3}	675		606		559		511		468		418	
L_p, ft	11.0		10.9		10.8		10.7		10.8		10.6	
L_r, ft	45.2		41.7		39.7		37.4		35.8		33.7	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

W24

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

F_y = 50 ksi

Shape		W24×											
		131		117		104		103		94		84	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	9											453	680
	10							539	809	501	751	447	672
	11					482	723	508	764	461	693	406	611
	12	593	889	535	802	481	723	466	700	422	635	373	560
	13	568	854	502	755	444	667	430	646	390	586	344	517
	14	528	793	466	701	412	619	399	600	362	544	319	480
	15	492	740	435	654	385	578	373	560	338	508	298	448
	16	462	694	408	613	361	542	349	525	317	476	279	420
	17	434	653	384	577	339	510	329	494	298	448	263	395
	18	410	617	363	545	320	482	310	467	282	423	248	373
	19	389	584	344	516	304	456	294	442	267	401	235	354
	20	369	555	326	491	288	434	279	420	253	381	224	336
	21	352	529	311	467	275	413	266	400	241	363	213	320
	22	336	505	297	446	262	394	254	382	230	346	203	305
	23	321	483	284	427	251	377	243	365	220	331	194	292
	24	308	463	272	409	240	361	233	350	211	318	186	280
	25	295	444	261	392	231	347	224	336	203	305	179	269
	26	284	427	251	377	222	333	215	323	195	293	172	258
	27	274	411	242	363	214	321	207	311	188	282	166	249
	28	264	396	233	350	206	310	200	300	181	272	160	240
	29	255	383	225	338	199	299	193	290	175	263	154	232
	30	246	370	218	327	192	289	186	280	169	254	149	224
	32	231	347	204	307	180	271	175	263	158	238	140	210
	34	217	326	192	289	170	255	164	247	149	224	132	198
	36	205	308	181	273	160	241	155	233	141	212	124	187
	38	194	292	172	258	152	228	147	221	133	201	118	177
	40	185	278	163	245	144	217	140	210	127	191	112	168
	42	176	264	155	234	137	206	133	200	121	181	106	160
	44	168	252	148	223	131	197	127	191	115	173	102	153
	46	161	241	142	213	125	188	121	183	110	166	97.2	146
	48	154	231	136	204	120	181	116	175	106	159	93.1	140
	50	148	222	131	196	115	173	112	168	101	152	89.4	134
	52	142	213	126	189	111	167	107	162	97.5	147	86.0	129
	54	137	206	121	182	107	161	103	156	93.9	141	82.8	124
	56	132	198	117	175	103	155	99.8	150	90.5	136	79.8	120
	58	127	191	113	169	99.5	149	96.4	145	87.4	131	77.1	116
	60	123	185	109	164	96.1	145	93.1	140	84.5	127	74.5	112
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	7390	11100	6530	9810	5770	8670	5590	8400	5070	7620	4470	6720
M_p/Ω_b	$\phi_b M_p$, kip-ft	923	1390	816	1230	721	1080	699	1050	634	953	559	840
M_r/Ω_b	$\phi_b M_r$, kip-ft	575	864	508	764	451	677	428	643	388	583	342	515
BF/Ω_b	$\phi_b BF$, kips	16.3	24.6	15.4	23.3	14.3	21.3	18.2	27.4	17.3	26.0	16.2	24.2
V_n/Ω_v	$\phi_v V_n$, kips	296	445	267	401	241	362	270	404	250	375	227	340
Z_x, in.^{3}		370		327		289		280		254		224	
L_p, ft		10.5		10.4		10.3		7.03		6.99		6.89	
L_r, ft		31.9		30.4		29.2		21.9		21.2		20.3	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

F_y = 50 ksi

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

W24

Shape	W24× 76		W24× 68		W24× 62		W24× 55[v]	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 7					408	611	335	503
8					382	574	334	503
9	421	631	393	590	339	510	297	447
10	399	600	353	531	305	459	267	402
11	363	545	321	483	278	417	243	365
12	333	500	294	443	254	383	223	335
13	307	462	272	408	235	353	206	309
14	285	429	252	379	218	328	191	287
15	266	400	236	354	204	306	178	268
16	250	375	221	332	191	287	167	251
17	235	353	208	312	180	270	157	236
18	222	333	196	295	170	255	149	223
19	210	316	186	279	161	242	141	212
20	200	300	177	266	153	230	134	201
21	190	286	168	253	145	219	127	191
22	181	273	161	241	139	209	122	183
23	174	261	154	231	133	200	116	175
24	166	250	147	221	127	191	111	168
25	160	240	141	212	122	184	107	161
26	154	231	136	204	117	177	103	155
27	148	222	131	197	113	170	99.1	149
28	143	214	126	190	109	164	95.5	144
29	138	207	122	183	105	158	92.2	139
30	133	200	118	177	102	153	89.2	134
32	125	188	110	166	95.4	143	83.6	126
34	117	176	104	156	89.8	135	78.7	118
36	111	167	98.1	148	84.8	128	74.3	112
38	105	158	93.0	140	80.4	121	70.4	106
40	99.8	150	88.3	133	76.3	115	66.9	101
42	95.0	143	84.1	126	72.7	109	63.7	95.7
44	90.7	136	80.3	121	69.4	104	60.8	91.4
46	86.8	130	76.8	115	66.4	99.8	58.1	87.4
48	83.2	125	73.6	111	63.6	95.6	55.7	83.8
50	79.8	120	70.7	106	61.1	91.8	53.5	80.4
52	76.8	115	67.9	102	58.7	88.3	51.4	77.3
54	73.9	111	65.4	98.3	56.6	85.0	49.5	74.4
56	71.3	107	63.1	94.8	54.5	82.0	47.8	71.8
58	68.8	103	60.9	91.6	52.7	79.1	46.1	69.3

Beam Properties

ASD	LRFD	76 ASD	76 LRFD	68 ASD	68 LRFD	62 ASD	62 LRFD	55[v] ASD	55[v] LRFD
W_c/Ω_b	$\phi_b W_c$, kip-ft	3990	6000	3530	5310	3050	4590	2670	4020
M_p/Ω_b	$\phi_b M_p$, kip-ft	499	750	442	664	382	574	334	503
M_r/Ω_b	$\phi_b M_r$, kip-ft	307	462	269	404	229	344	199	299
BF/Ω_b	$\phi_b BF$, kips	15.1	22.6	14.1	21.2	16.1	24.1	14.7	22.2
V_n/Ω_v	$\phi_v V_n$, kips	210	315	197	295	204	306	167	252

Property	76	68	62	55[v]
Z_x, in.^{3}	200	177	153	134
L_p, ft	6.78	6.61	4.87	4.73
L_r, ft	19.5	18.9	14.4	13.9

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.

Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

W21

F_y = 50 ksi

Shape	W21× 275[h]		248		223		201		182	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 12	1180	1760	1040	1560	936	1400	837	1260	754	1130
13	1150	1730	1030	1550	923	1390	814	1220	731	1100
14	1070	1610	957	1440	857	1290	756	1140	679	1020
15	997	1500	893	1340	800	1200	705	1060	633	952
16	934	1400	837	1260	750	1130	661	994	594	893
17	879	1320	788	1180	706	1060	622	935	559	840
18	831	1250	744	1120	666	1000	588	883	528	793
19	787	1180	705	1060	631	949	557	837	500	752
20	748	1120	670	1010	600	902	529	795	475	714
21	712	1070	638	959	571	859	504	757	452	680
22	680	1020	609	915	545	820	481	723	432	649
23	650	977	582	875	522	784	460	691	413	621
24	623	936	558	839	500	751	441	663	396	595
25	598	899	536	805	480	721	423	636	380	571
26	575	864	515	774	461	693	407	612	365	549
27	554	832	496	746	444	668	392	589	352	529
28	534	803	478	719	428	644	378	568	339	510
29	516	775	462	694	414	622	365	548	328	492
30	498	749	446	671	400	601	353	530	317	476
32	467	702	419	629	375	563	331	497	297	446
34	440	661	394	592	353	530	311	468	279	420
36	415	624	372	559	333	501	294	442	264	397
38	393	591	352	530	316	474	278	418	250	376
40	374	562	335	503	300	451	264	398	238	357
42	356	535	319	479	286	429	252	379	226	340
44	340	511	304	458	273	410	240	361	216	325
46	325	488	291	438	261	392	230	346	207	310
48	311	468	279	419	250	376	220	331	198	298
50	299	449	268	403	240	361	212	318	190	286
52	288	432	258	387	231	347	203	306	183	275
54	277	416	248	373	222	334	196	294	176	264
56	267	401	239	359	214	322	189	284	170	255
58	258	387	231	347	207	311				
60	249	375								
Beam Properties										
W_c/Ω_b / $\phi_b W_c$, kip-ft	15000	22500	13400	20100	12000	18000	10600	15900	9500	14300
M_p/Ω_b / $\phi_b M_p$, kip-ft	1870	2810	1670	2520	1500	2250	1320	1990	1190	1790
M_r/Ω_b / $\phi_b M_r$, kip-ft	1110	1670	1010	1510	908	1370	805	1210	728	1090
BF/Ω_b / $\phi_b BF$, kips	14.7	22.1	14.3	21.9	14.5	21.6	14.5	22.0	14.4	21.8
V_n/Ω_v / $\phi_v V_n$, kips	588	882	521	782	468	702	419	628	377	565
Z_x, in.^{3}	749		671		601		530		476	
L_p, ft	10.9		10.9		10.7		10.7		10.6	
L_r, ft	62.5		57.1		51.4		46.2		42.7	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

F_y = 50 ksi

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

W21

Shape	W21× 166		147		132		122		111		101	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 11			636	955	567	850	521	781	473	710	428	642
12	675	1010	620	933	554	833	511	768	464	698	421	633
13	663	997	573	861	511	768	471	708	428	644	388	584
14	616	926	532	799	475	714	438	658	398	598	361	542
15	575	864	496	746	443	666	409	614	371	558	337	506
16	539	810	465	699	415	624	383	576	348	523	316	474
17	507	762	438	658	391	588	360	542	328	492	297	446
18	479	720	414	622	369	555	340	512	309	465	281	422
19	454	682	392	589	350	526	323	485	293	441	266	399
20	431	648	372	560	332	500	306	461	278	419	252	380
21	411	617	355	533	317	476	292	439	265	399	240	361
22	392	589	338	509	302	454	279	419	253	380	230	345
23	375	563	324	487	289	434	266	400	242	364	220	330
24	359	540	310	466	277	416	255	384	232	349	210	316
25	345	518	298	448	266	400	245	368	223	335	202	304
26	332	498	286	430	256	384	236	354	214	322	194	292
27	319	480	276	414	246	370	227	341	206	310	187	281
28	308	463	266	400	237	357	219	329	199	299	180	271
29	297	447	257	386	229	344	211	318	192	289	174	262
30	287	432	248	373	222	333	204	307	186	279	168	253
32	269	405	233	350	208	312	191	288	174	262	158	237
34	254	381	219	329	195	294	180	271	164	246	149	223
36	240	360	207	311	185	278	170	256	155	233	140	211
38	227	341	196	294	175	263	161	242	147	220	133	200
40	216	324	186	280	166	250	153	230	139	209	126	190
42	205	309	177	266	158	238	146	219	133	199	120	181
44	196	295	169	254	151	227	139	209	127	190	115	173
46	187	282	162	243	144	217	133	200	121	182	110	165
48	180	270	155	233	138	208	128	192	116	174	105	158
50	172	259	149	224	133	200	123	184	111	167	101	152
52	166	249	143	215	128	192	118	177	107	161	97.1	146
54	160	240	138	207	123	185	113	171				
56	154	231										
Beam Properties												
W_c/Ω_b / $\phi_b W_c$, kip-ft	8620	13000	7450	11200	6650	9990	6130	9210	5570	8370	5050	7590
M_p/Ω_b / $\phi_b M_p$, kip-ft	1080	1620	931	1400	831	1250	766	1150	696	1050	631	949
M_r/Ω_b / $\phi_b M_r$, kip-ft	664	998	575	864	515	774	477	717	435	654	396	596
BF/Ω_b / $\phi_b BF$, kips	14.2	21.2	13.7	20.7	13.2	19.9	12.9	19.3	12.4	18.9	11.8	17.7
V_n/Ω_v / $\phi_v V_n$, kips	338	506	318	477	283	425	260	391	237	355	214	321
Z_x, in.^{3}	432		373		333		307		279		253	
L_p, ft	10.6		10.4		10.3		10.3		10.2		10.2	
L_r, ft	39.9		36.3		34.2		32.7		31.2		30.1	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

W21

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

F_y = 50 ksi

Shape	W21×									
	93		83		73		68		62	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 8	501	752	441	661	386	579	363	544	336	504
9	490	737	435	653	381	573	355	533	319	480
10	441	663	391	588	343	516	319	480	287	432
11	401	603	356	535	312	469	290	436	261	393
12	368	553	326	490	286	430	266	400	240	360
13	339	510	301	452	264	397	246	369	221	332
14	315	474	279	420	245	369	228	343	205	309
15	294	442	261	392	229	344	213	320	192	288
16	276	414	245	368	215	323	200	300	180	270
17	259	390	230	346	202	304	188	282	169	254
18	245	368	217	327	191	287	177	267	160	240
19	232	349	206	309	181	272	168	253	151	227
20	221	332	196	294	172	258	160	240	144	216
21	210	316	186	280	163	246	152	229	137	206
22	201	301	178	267	156	235	145	218	131	196
23	192	288	170	256	149	224	139	209	125	188
24	184	276	163	245	143	215	133	200	120	180
25	176	265	156	235	137	206	128	192	115	173
26	170	255	150	226	132	198	123	185	111	166
27	163	246	145	218	127	191	118	178	106	160
28	158	237	140	210	123	184	114	171	103	154
29	152	229	135	203	118	178	110	166	99.1	149
30	147	221	130	196	114	172	106	160	95.8	144
32	138	207	122	184	107	161	99.8	150	89.8	135
34	130	195	115	173	101	152	93.9	141	84.5	127
36	123	184	109	163	95.4	143	88.7	133	79.8	120
38	116	174	103	155	90.3	136	84.0	126	75.6	114
40	110	166	97.8	147	85.8	129	79.8	120	71.9	108
42	105	158	93.1	140	81.7	123	76.0	114	68.4	103
44	100	151	88.9	134	78.0	117	72.6	109	65.3	98.2
46	95.9	144	85.0	128	74.6	112	69.4	104	62.5	93.9
48	91.9	138	81.5	122	71.5	108	66.5	100	59.9	90.0
50	88.2	133	78.2	118	68.7	103	63.9	96.0	57.5	86.4
52	84.8	128	75.2	113	66.0	99.2	61.4	92.3	55.3	83.1
54	81.7	123								
Beam Properties										
W_c/Ω_b / $\phi_b W_c$, kip-ft	4410	6630	3910	5880	3430	5160	3190	4800	2870	4320
M_p/Ω_b / $\phi_b M_p$, kip-ft	551	829	489	735	429	645	399	600	359	540
M_r/Ω_b / $\phi_b M_r$, kip-ft	335	504	299	449	264	396	245	368	222	333
BF/Ω_b / $\phi_b BF$, kips	14.6	22.0	13.8	20.8	12.9	19.4	12.5	18.8	11.6	17.5
V_n/Ω_v / $\phi_v V_n$, kips	251	376	220	331	193	289	181	272	168	252
Z_x, in.^{3}	221		196		172		160		144	
L_p, ft	6.50		6.46		6.39		6.36		6.25	
L_r, ft	21.3		20.2		19.2		18.7		18.1	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

F_y = 50 ksi

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

W21

Shape	W21×									
	57		55		50		48[f]		44	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 6					316	474			290	435
7	342	513			314	471	288	433	272	409
8	322	484	312	468	274	413	265	398	238	358
9	286	430	279	420	244	367	235	354	212	318
10	257	387	251	378	220	330	212	318	190	286
11	234	352	229	344	200	300	193	289	173	260
12	215	323	210	315	183	275	176	265	159	239
13	198	298	193	291	169	254	163	245	146	220
14	184	276	180	270	157	236	151	227	136	204
15	172	258	168	252	146	220	141	212	127	191
16	161	242	157	236	137	206	132	199	119	179
17	151	228	148	222	129	194	125	187	112	168
18	143	215	140	210	122	183	118	177	106	159
19	136	204	132	199	116	174	111	168	100	151
20	129	194	126	189	110	165	106	159	95.2	143
21	123	184	120	180	105	157	101	152	90.7	136
22	117	176	114	172	99.8	150	96.3	145	86.6	130
23	112	168	109	164	95.5	143	92.1	138	82.8	124
24	107	161	105	158	91.5	138	88.2	133	79.3	119
25	103	155	101	151	87.8	132	84.7	127	76.2	114
26	99.0	149	96.7	145	84.4	127	81.5	122	73.2	110
27	95.4	143	93.1	140	81.3	122	78.4	118	70.5	106
28	92.0	138	89.8	135	78.4	118	75.6	114	68.0	102
29	88.8	133	86.7	130	75.7	114	73.0	110	65.7	98.7
30	85.8	129	83.8	126	73.2	110	70.6	106	63.5	95.4
32	80.5	121	78.6	118	68.6	103	66.2	99.5	59.5	89.4
34	75.7	114	74.0	111	64.6	97.1	62.3	93.6	56.0	84.2
36	71.5	108	69.9	105	61.0	91.7	58.8	88.4	52.9	79.5
38	67.8	102	66.2	99.5	57.8	86.8	55.7	83.8	50.1	75.3
40	64.4	96.8	62.9	94.5	54.9	82.5	52.9	79.6	47.6	71.6
42	61.3	92.1	59.9	90.0	52.3	78.6	50.4	75.8	45.3	68.1
44	58.5	88.0	57.2	85.9	49.9	75.0	48.1	72.3	43.3	65.0
46	56.0	84.1	54.7	82.2	47.7	71.7	46.0	69.2	41.4	62.2
48	53.6	80.6	52.4	78.8	45.7	68.8	44.1	66.3	39.7	59.6
50	51.5	77.4	50.3	75.6	43.9	66.0	42.4	63.7	38.1	57.2
52	49.5	74.4	48.4	72.7	42.2	63.5				

Beam Properties

ASD	LRFD	57 ASD	57 LRFD	55 ASD	55 LRFD	50 ASD	50 LRFD	48[f] ASD	48[f] LRFD	44 ASD	44 LRFD
W_c/Ω_b	$\phi_b W_c$, kip-ft	2570	3870	2510	3780	2200	3300	2120	3180	1900	2860
M_p/Ω_b	$\phi_b M_p$, kip-ft	322	484	314	473	274	413	265	398	238	358
M_r/Ω_b	$\phi_b M_r$, kip-ft	194	291	192	289	165	248	162	244	143	214
BF/Ω_b	$\phi_b BF$, kips	13.4	20.3	10.8	16.3	12.1	18.3	9.89	14.8	11.1	16.8
V_n/Ω_v	$\phi_v V_n$, kips	171	256	156	234	158	237	144	216	145	217

Property	57	55	50	48[f]	44
Z_x, in.^{3}	129	126	110	107	95.4
L_p, ft	4.77	6.11	4.59	6.09	4.45
L_r, ft	14.3	17.4	13.6	16.5	13.0

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[f] Shape does not meet compact limit for flexure with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

W18

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

F_y = 50 ksi

Shape		W18×											
		311[h]		283[h]		258[h]		234[h]		211		192	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	11	1360	2030	1230	1840	1100	1650	979	1470	878	1320	783	1180
	12	1250	1890	1120	1690	1020	1530	913	1370	815	1230	735	1110
	13	1160	1740	1040	1560	938	1410	843	1270	752	1130	679	1020
	14	1070	1620	964	1450	871	1310	783	1180	699	1050	630	947
	15	1000	1510	900	1350	813	1220	731	1100	652	980	588	884
	16	941	1410	843	1270	762	1150	685	1030	611	919	551	829
	17	885	1330	794	1190	717	1080	645	969	575	865	519	780
	18	836	1260	750	1130	678	1020	609	915	543	817	490	737
	19	792	1190	710	1070	642	965	577	867	515	774	464	698
	20	752	1130	675	1010	610	917	548	824	489	735	441	663
	21	717	1080	643	966	581	873	522	784	466	700	420	631
	22	684	1030	613	922	554	833	498	749	445	668	401	603
	23	654	983	587	882	530	797	476	716	425	639	384	577
	24	627	943	562	845	508	764	457	686	408	613	368	553
	25	602	905	540	811	488	733	438	659	391	588	353	530
	26	579	870	519	780	469	705	421	633	376	565	339	510
	27	557	838	500	751	452	679	406	610	362	544	327	491
	28	537	808	482	724	436	655	391	588	349	525	315	474
	29	519	780	465	699	421	632	378	568	337	507	304	457
	30	502	754	450	676	407	611	365	549	326	490	294	442
	31	485	730	435	654	393	591	353	531	315	474	285	428
	32	470	707	422	634	381	573	342	515	306	459	276	414
	33	456	685	409	615	370	555	332	499	296	445	267	402
	34	443	665	397	596	359	539	322	484	288	432	259	390
	35	430	646	386	579	348	524	313	471	279	420	252	379
	36	418	628	375	563	339	509	304	458	272	408	245	368
	37	407	611	365	548	330	495	296	445	264	397	238	358
	38	396	595	355	534	321	482	288	433	257	387	232	349
	39	386	580	346	520	313	470	281	422	251	377	226	340
	40	376	566	337	507	305	458	274	412	245	368	221	332
	42	358	539	321	483	290	436	261	392	233	350	210	316
	44	342	514	307	461	277	417	249	374	222	334	201	301
	46	327	492	293	441	265	398	238	358	213	320	192	288
	48	314	471	281	423	254	382	228	343	204	306	184	276
	50	301	452	270	406	244	367	219	329	196	294	176	265
	52	289	435	259	390	235	353	211	317				
	54	279	419	250	376								
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	15000	22600	13500	20300	12200	18300	11000	16500	9780	14700	8820	13300
M_p/Ω_b	$\phi_b M_p$, kip-ft	1880	2830	1690	2540	1520	2290	1370	2060	1220	1840	1100	1660
M_r/Ω_b	$\phi_b M_r$, kip-ft	1090	1640	987	1480	898	1350	814	1220	732	1100	664	998
BF/Ω_b	$\phi_b BF$, kips	11.2	16.8	11.1	16.7	10.9	16.5	10.8	16.4	10.7	16.2	10.6	16.1
V_n/Ω_v	$\phi_v V_n$, kips	678	1020	613	920	550	826	490	734	439	658	392	588
Z_x, in.^{3}		754		676		611		549		490		442	
L_p, ft		10.4		10.3		10.2		10.1		9.96		9.85	
L_r, ft		81.1		73.6		67.3		61.4		55.7		51.0	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Notes: For beams laterally unsupported, see Table 3-10.

Available strength tabulated above heavy line is limited by available shear strength.

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

F_y = 50 ksi

W18

Shape	W18×											
	175		158		143		130		119		106	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 10									498	747	441	662
11	712	1070	638	957	569	854	517	776	475	715	417	627
12	662	995	592	890	536	805	482	725	436	655	383	575
13	611	918	547	822	494	743	445	669	402	605	353	531
14	567	853	508	763	459	690	413	621	374	561	328	493
15	530	796	474	712	428	644	386	580	349	524	306	460
16	497	746	444	668	402	604	362	544	327	491	287	431
17	467	702	418	628	378	568	340	512	308	462	270	406
18	441	663	395	593	357	537	322	483	291	437	255	383
19	418	628	374	562	338	508	305	458	275	414	242	363
20	397	597	355	534	321	483	289	435	261	393	230	345
21	378	569	338	509	306	460	276	414	249	374	219	329
22	361	543	323	485	292	439	263	395	238	357	209	314
23	345	519	309	464	279	420	252	378	227	342	200	300
24	331	498	296	445	268	403	241	363	218	328	191	288
25	318	478	284	427	257	386	232	348	209	314	184	276
26	306	459	273	411	247	372	223	335	201	302	177	265
27	294	442	263	396	238	358	214	322	194	291	170	256
28	284	426	254	381	230	345	207	311	187	281	164	246
29	274	412	245	368	222	333	200	300	180	271	158	238
30	265	398	237	356	214	322	193	290	174	262	153	230
31	256	385	229	345	207	312	187	281	169	254	148	223
32	248	373	222	334	201	302	181	272	163	246	143	216
33	241	362	215	324	195	293	175	264	158	238	139	209
34	234	351	209	314	189	284	170	256	154	231	135	203
35	227	341	203	305	184	276	165	249	149	225	131	197
36	221	332	197	297	179	268	161	242	145	218	128	192
37	215	323	192	289	174	261	156	235	141	212	124	186
38	209	314	187	281	169	254	152	229	138	207	121	182
39	204	306	182	274	165	248	148	223	134	202	118	177
40	199	299	178	267	161	242	145	218	131	197	115	173
42	189	284	169	254	153	230	138	207	125	187	109	164
44	181	271	161	243	146	220	132	198	119	179	104	157
46	173	260	154	232	140	210	126	189	114	171	99.8	150
48	166	249	148	223	134	201	121	181				
50	159	239										
Beam Properties												
W_c/Ω_b / $\phi_b W_c$, kip-ft	7940	11900	7110	10700	6430	9660	5790	8700	5230	7860	4590	6900
M_p/Ω_b / $\phi_b M_p$, kip-ft	993	1490	888	1340	803	1210	724	1090	654	983	574	863
M_r/Ω_b / $\phi_b M_r$, kip-ft	601	903	541	814	493	740	447	672	403	606	356	536
BF/Ω_b / $\phi_b BF$, kips	10.6	15.8	10.5	15.9	10.3	15.7	10.2	15.4	10.1	15.2	9.73	14.6
V_n/Ω_v / $\phi_v V_n$, kips	356	534	319	479	285	427	259	388	249	373	221	331
Z_x, in.^{3}	398		356		322		290		262		230	
L_p, ft	9.75		9.68		9.61		9.54		9.50		9.40	
L_r, ft	46.9		42.8		39.6		36.6		34.3		31.8	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

W18

F_y = 50 ksi

Shape	W18×											
	97		86		76		71		65		60	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft												
7							366	549				
8							364	548	331	497	302	453
9							324	487	295	443	273	410
10	398	597	353	530	309	464	291	438	265	399	246	369
11	383	575	338	507	296	445	265	398	241	363	223	335
12	351	528	309	465	271	408	243	365	221	333	205	308
13	324	487	286	429	250	376	224	337	204	307	189	284
14	301	452	265	399	232	349	208	313	190	285	175	264
15	281	422	248	372	217	326	194	292	177	266	164	246
16	263	396	232	349	203	306	182	274	166	249	153	231
17	248	372	218	328	191	288	171	258	156	235	144	217
18	234	352	206	310	181	272	162	243	147	222	136	205
19	222	333	195	294	171	257	153	231	140	210	129	194
20	211	317	186	279	163	245	146	219	133	200	123	185
21	201	301	177	266	155	233	139	209	126	190	117	176
22	191	288	169	254	148	222	132	199	121	181	112	168
23	183	275	161	243	141	213	127	190	115	173	107	160
24	175	264	155	233	136	204	121	183	111	166	102	154
25	168	253	149	223	130	196	117	175	106	160	98.2	148
26	162	243	143	215	125	188	112	168	102	153	94.4	142
27	156	234	138	207	120	181	108	162	98.3	148	90.9	137
28	150	226	133	199	116	175	104	156	94.8	143	87.7	132
29	145	218	128	192	112	169	100	151	91.5	138	84.7	127
30	140	211	124	186	108	163	97.1	146	88.5	133	81.8	123
31	136	204	120	180	105	158	94.0	141	85.6	129	79.2	119
32	132	198	116	174	102	153	91.1	137	83.0	125	76.7	115
33	128	192	113	169	98.6	148	88.3	133	80.4	121	74.4	112
34	124	186	109	164	95.7	144	85.7	129	78.1	117	72.2	109
35	120	181	106	159	93.0	140	83.3	125	75.8	114	70.1	105
36	117	176	103	155	90.4	136	80.9	122	73.7	111	68.2	103
37	114	171	100	151	87.9	132	78.8	118	71.7	108	66.4	99.7
38	111	167	97.7	147	85.6	129	76.7	115	69.9	105	64.6	97.1
39	108	162	95.2	143	83.4	125	74.7	112	68.1	102	63.0	94.6
40	105	158	92.8	140	81.3	122	72.9	110	66.4	99.8	61.4	92.3
42	100	151	88.4	133	77.5	116	69.4	104	63.2	95.0	58.5	87.9
44	95.7	144	84.4	127	73.9	111	66.2	99.5	60.3	90.7	55.8	83.9
46	91.6	138	80.7	121			63.4	95.2	57.7	86.7		
Beam Properties												
W_c/Ω_b / $\phi_b W_c$, kip-ft	4210	6330	3710	5580	3250	4890	2910	4380	2650	3990	2460	3690
M_p/Ω_b / $\phi_b M_p$, kip-ft	526	791	464	698	407	611	364	548	332	499	307	461
M_r/Ω_b / $\phi_b M_r$, kip-ft	328	494	290	436	255	383	222	333	204	307	189	284
BF/Ω_b / $\phi_b BF$, kips	9.41	14.1	9.01	13.6	8.50	12.8	10.4	15.8	9.98	15.0	9.62	14.4
V_n/Ω_v / $\phi_v V_n$, kips	199	299	177	265	155	232	183	275	166	248	151	227
Z_x, in.^{3}	211		186		163		146		133		123	
L_p, ft	9.36		9.29		9.22		6.00		5.97		5.93	
L_r, ft	30.4		28.6		27.1		19.6		18.8		18.2	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

F_y = 50 ksi

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

W18–W16

Shape		W18×										W16×	
		55		50		46		40		35		100	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	6					261	391	226	338	212	319		
	7	282	424	256	383	259	389	224	336	190	285		
	8	279	420	252	379	226	340	196	294	166	249		
	9	248	373	224	337	201	302	174	261	147	222	398	597
	10	224	336	202	303	181	272	156	235	133	200	395	594
	11	203	305	183	275	165	247	142	214	121	181	359	540
	12	186	280	168	253	151	227	130	196	111	166	329	495
	13	172	258	155	233	139	209	120	181	102	153	304	457
	14	160	240	144	216	129	194	112	168	94.8	143	282	424
	15	149	224	134	202	121	181	104	157	88.5	133	263	396
	16	140	210	126	189	113	170	97.8	147	83.0	125	247	371
	17	132	198	119	178	106	160	92.1	138	78.1	117	232	349
	18	124	187	112	168	101	151	86.9	131	73.7	111	220	330
	19	118	177	106	159	95.3	143	82.4	124	69.9	105	208	313
	20	112	168	101	152	90.5	136	78.2	118	66.4	99.8	198	297
	21	106	160	96.0	144	86.2	130	74.5	112	63.2	95.0	188	283
	22	102	153	91.6	138	82.3	124	71.1	107	60.3	90.7	180	270
	23	97.2	146	87.7	132	78.7	118	68.0	102	57.7	86.7	172	258
	24	93.1	140	84.0	126	75.4	113	65.2	98.0	55.3	83.1	165	248
	25	89.4	134	80.6	121	72.4	109	62.6	94.1	53.1	79.8	158	238
	26	86.0	129	77.5	117	69.6	105	60.2	90.5	51.1	76.7	152	228
	27	82.8	124	74.7	112	67.1	101	58.0	87.1	49.2	73.9	146	220
	28	79.8	120	72.0	108	64.7	97.2	55.9	84.0	47.4	71.3	141	212
	29	77.1	116	69.5	104	62.4	93.8	54.0	81.1	45.8	68.8	136	205
	30	74.5	112	67.2	101	60.3	90.7	52.2	78.4	44.2	66.5	132	198
	31	72.1	108	65.0	97.7	58.4	87.8	50.5	75.9	42.8	64.4	127	192
	32	69.9	105	63.0	94.7	56.6	85.0	48.9	73.5	41.5	62.3	124	186
	33	67.7	102	61.1	91.8	54.9	82.5	47.4	71.3	40.2	60.5	120	180
	34	65.8	98.8	59.3	89.1	53.2	80.0	46.0	69.2	39.0	58.7	116	175
	35	63.9	96.0	57.6	86.6	51.7	77.7	44.7	67.2	37.9	57.0	113	170
	36	62.1	93.3	56.0	84.2	50.3	75.6	43.5	65.3	36.9	55.4	110	165
	37	60.4	90.8	54.5	81.9	48.9	73.5	42.3	63.6	35.9	53.9	107	161
	38	58.8	88.4	53.1	79.7	47.6	71.6	41.2	61.9	34.9	52.5	104	156
	39	57.3	86.2	51.7	77.7	46.4	69.8	40.1	60.3	34.0	51.2	101	152
	40	55.9	84.0	50.4	75.8	45.3	68.0	39.1	58.8	33.2	49.9	98.8	149
	42	53.2	80.0	48.0	72.1	43.1	64.8	37.3	56.0	31.6	47.5	94.1	141
	44	50.8	76.4	45.8	68.9	41.1	61.8	35.6	53.5	30.2	45.3		
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	2240	3360	2020	3030	1810	2720	1560	2350	1330	2000	3950	5940
M_p/Ω_b	$\phi_b M_p$, kip-ft	279	420	252	379	226	340	196	294	166	249	494	743
M_r/Ω_b	$\phi_b M_r$, kip-ft	172	258	155	233	138	207	119	180	101	151	306	459
BF/Ω_b	$\phi_b BF$, kips	9.15	13.8	8.76	13.2	9.63	14.6	8.94	13.2	8.14	12.3	7.86	11.9
V_n/Ω_v	$\phi_v V_n$, kips	141	212	128	192	130	195	113	169	106	159	199	298
Z_x, in.^{3}		112		101		90.7		78.4		66.5		198	
L_p, ft		5.90		5.83		4.56		4.49		4.31		8.87	
L_r, ft		17.6		16.9		13.7		13.1		12.3		32.8	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

W16

F_y = 50 ksi

Shape	W16×									
	89		77		67		57		50	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 7							282	423	248	372
8							262	394	230	345
9	353	529	300	450			233	350	204	307
10	349	525	299	450	258	386	210	315	184	276
11	318	477	272	409	236	355	191	286	167	251
12	291	438	250	375	216	325	175	263	153	230
13	269	404	230	346	200	300	161	242	141	212
14	250	375	214	321	185	279	150	225	131	197
15	233	350	200	300	173	260	140	210	122	184
16	218	328	187	281	162	244	131	197	115	173
17	205	309	176	265	153	229	123	185	108	162
18	194	292	166	250	144	217	116	175	102	153
19	184	276	158	237	137	205	110	166	96.6	145
20	175	263	150	225	130	195	105	158	91.8	138
21	166	250	143	214	124	186	99.8	150	87.4	131
22	159	239	136	205	118	177	95.3	143	83.5	125
23	152	228	130	196	113	170	91.1	137	79.8	120
24	146	219	125	188	108	163	87.3	131	76.5	115
25	140	210	120	180	104	156	83.8	126	73.5	110
26	134	202	115	173	99.8	150	80.6	121	70.6	106
27	129	194	111	167	96.1	144	77.6	117	68.0	102
28	125	188	107	161	92.7	139	74.9	113	65.6	98.6
29	120	181	103	155	89.5	134	72.3	109	63.3	95.2
30	116	175	99.8	150	86.5	130	69.9	105	61.2	92.0
31	113	169	96.6	145	83.7	126	67.6	102	59.2	89.0
32	109	164	93.6	141	81.1	122	65.5	98.4	57.4	86.3
33	106	159	90.7	136	78.6	118	63.5	95.5	55.6	83.6
34	103	154	88.1	132	76.3	115	61.6	92.6	54.0	81.2
35	99.8	150	85.5	129	74.1	111	59.9	90.0	52.5	78.9
36	97.0	146	83.2	125	72.1	108	58.2	87.5	51.0	76.7
37	94.4	142	80.9	122	70.1	105	56.6	85.1	49.6	74.6
38	91.9	138	78.8	118	68.3	103	55.2	82.9	48.3	72.6
39	89.6	135	76.8	115	66.5	100	53.7	80.8	47.1	70.8
40	87.3	131	74.9	113	64.9	97.5	52.4	78.8	45.9	69.0
42	83.2	125								
Beam Properties										
W_c/Ω_b / $\phi_b W_c$, kip-ft	3490	5250	2990	4500	2590	3900	2100	3150	1840	2760
M_p/Ω_b / $\phi_b M_p$, kip-ft	437	656	374	563	324	488	262	394	230	345
M_r/Ω_b / $\phi_b M_r$, kip-ft	271	407	234	352	204	307	161	242	141	213
BF/Ω_b / $\phi_b BF$, kips	7.76	11.6	7.34	11.1	6.89	10.4	7.98	12.0	7.69	11.4
V_n/Ω_v / $\phi_v V_n$, kips	176	265	150	225	129	193	141	212	124	186
Z_x, in.^{3}	175		150		130		105		92.0	
L_p, ft	8.80		8.72		8.69		5.65		5.62	
L_r, ft	30.2		27.8		26.1		18.3		17.2	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

F_y = 50 ksi

W16

Shape	W16×									
	45		40		36		31		26[v]	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 6					188	281	175	262	141	212
7	222	333	195	293	182	274	154	231	126	189
8	205	309	182	274	160	240	135	203	110	166
9	183	274	162	243	142	213	120	180	98.0	147
10	164	247	146	219	128	192	108	162	88.2	133
11	149	224	132	199	116	175	98.0	147	80.2	121
12	137	206	121	183	106	160	89.8	135	73.5	111
13	126	190	112	168	98.3	148	82.9	125	67.9	102
14	117	176	104	156	91.2	137	77.0	116	63.0	94.7
15	110	165	97.1	146	85.2	128	71.9	108	58.8	88.4
16	103	154	91.1	137	79.8	120	67.4	101	55.1	82.9
17	96.6	145	85.7	129	75.1	113	63.4	95.3	51.9	78.0
18	91.3	137	80.9	122	71.0	107	59.9	90.0	49.0	73.7
19	86.5	130	76.7	115	67.2	101	56.7	85.3	46.4	69.8
20	82.1	123	72.9	110	63.9	96.0	53.9	81.0	44.1	66.3
21	78.2	118	69.4	104	60.8	91.4	51.3	77.1	42.0	63.1
22	74.7	112	66.2	99.5	58.1	87.3	49.0	73.6	40.1	60.3
23	71.4	107	63.4	95.2	55.5	83.5	46.9	70.4	38.4	57.7
24	68.4	103	60.7	91.3	53.2	80.0	44.9	67.5	36.8	55.3
25	65.7	98.8	58.3	87.6	51.1	76.8	43.1	64.8	35.3	53.0
26	63.2	95.0	56.0	84.2	49.1	73.8	41.5	62.3	33.9	51.0
27	60.8	91.4	54.0	81.1	47.3	71.1	39.9	60.0	32.7	49.1
28	58.7	88.2	52.0	78.2	45.6	68.6	38.5	57.9	31.5	47.4
29	56.6	85.1	50.2	75.5	44.0	66.2	37.2	55.9	30.4	45.7
30	54.8	82.3	48.6	73.0	42.6	64.0	35.9	54.0	29.4	44.2
31	53.0	79.6	47.0	70.6	41.2	61.9	34.8	52.3	28.5	42.8
32	51.3	77.2	45.5	68.4	39.9	60.0	33.7	50.6	27.6	41.4
33	49.8	74.8	44.2	66.4	38.7	58.2	32.7	49.1	26.7	40.2
34	48.3	72.6	42.9	64.4	37.6	56.5	31.7	47.6	25.9	39.0
35	46.9	70.5	41.6	62.6	36.5	54.9	30.8	46.3	25.2	37.9
36	45.6	68.6	40.5	60.8	35.5	53.3	29.9	45.0	24.5	36.8
37	44.4	66.7	39.4	59.2	34.5	51.9	29.1	43.8	23.8	35.8
38	43.2	65.0	38.3	57.6	33.6	50.5	28.4	42.6	23.2	34.9
39	42.1	63.3	37.4	56.2	32.8	49.2	27.6	41.5	22.6	34.0
40	41.1	61.7	36.4	54.8						

Beam Properties

ASD	LRFD	45 ASD	45 LRFD	40 ASD	40 LRFD	36 ASD	36 LRFD	31 ASD	31 LRFD	26 ASD	26 LRFD
W_c/Ω_b	$\phi_b W_c$, kip-ft	1640	2470	1460	2190	1280	1920	1080	1620	882	1330
M_p/Ω_b	$\phi_b M_p$, kip-ft	205	309	182	274	160	240	135	203	110	166
M_r/Ω_b	$\phi_b M_r$, kip-ft	127	191	113	170	98.7	148	82.4	124	67.1	101
BF/Ω_b	$\phi_b BF$, kips	7.12	10.8	6.67	10.0	6.24	9.36	6.86	10.3	5.93	8.98
V_n/Ω_v	$\phi_v V_n$, kips	111	167	97.6	146	93.8	141	87.5	131	70.5	106

Property	45	40	36	31	26
Z_x, in.^{3}	82.3	73.0	64.0	54.0	44.2
L_p, ft	5.55	5.55	5.37	4.13	3.96
L_r, ft	16.5	15.9	15.2	11.8	11.2

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.

Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

W14

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

F_y = 50 ksi

Shape		W14× 82		74		68		61		53		48	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	8									206	309	188	282
	9	292	438	256	383	232	349	209	313	193	290	174	261
	10	277	417	251	378	230	345	204	306	174	261	156	235
	11	252	379	229	344	209	314	185	278	158	238	142	214
	12	231	348	210	315	191	288	170	255	145	218	130	196
	13	213	321	193	291	177	265	157	235	134	201	120	181
	14	198	298	180	270	164	246	145	219	124	187	112	168
	15	185	278	168	252	153	230	136	204	116	174	104	157
	16	173	261	157	236	143	216	127	191	109	163	97.8	147
	17	163	245	148	222	135	203	120	180	102	154	92.1	138
	18	154	232	140	210	128	192	113	170	96.6	145	86.9	131
	19	146	219	132	199	121	182	107	161	91.5	138	82.4	124
	20	139	209	126	189	115	173	102	153	86.9	131	78.2	118
	21	132	199	120	180	109	164	96.9	146	82.8	124	74.5	112
	22	126	190	114	172	104	157	92.5	139	79.0	119	71.1	107
	23	121	181	109	164	99.8	150	88.5	133	75.6	114	68.0	102
	24	116	174	105	158	95.6	144	84.8	128	72.4	109	65.2	98.0
	25	111	167	101	151	91.8	138	81.4	122	69.5	105	62.6	94.1
	26	107	160	96.7	145	88.3	133	78.3	118	66.9	101	60.2	90.5
	27	103	154	93.1	140	85.0	128	75.4	113	64.4	96.8	58.0	87.1
	28	99.1	149	89.8	135	82.0	123	72.7	109	62.1	93.3	55.9	84.0
	29	95.7	144	86.7	130	79.2	119	70.2	106	59.9	90.1	54.0	81.1
	30	92.5	139	83.8	126	76.5	115	67.9	102	58.0	87.1	52.2	78.4
	31	89.5	135	81.1	122	74.0	111	65.7	98.7	56.1	84.3	50.5	75.9
	32	86.7	130	78.6	118	71.7	108	63.6	95.6	54.3	81.7	48.9	73.5
	33	84.1	126	76.2	115	69.6	105	61.7	92.7	52.7	79.2	47.4	71.3
	34	81.6	123	74.0	111	67.5	101	59.9	90.0	51.1	76.9	46.0	69.2
	35	79.3	119	71.9	108	65.6	98.6						
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	2770	4170	2510	3780	2300	3450	2040	3060	1740	2610	1560	2350
M_p/Ω_b	$\phi_b M_p$, kip-ft	347	521	314	473	287	431	254	383	217	327	196	294
M_r/Ω_b	$\phi_b M_r$, kip-ft	215	323	196	294	180	270	161	242	136	204	123	184
BF/Ω_b	$\phi_b BF$, kips	5.40	8.10	5.31	8.05	5.19	7.81	4.93	7.48	5.22	7.93	5.09	7.67
V_n/Ω_v	$\phi_v V_n$, kips	146	219	128	192	116	174	104	156	103	154	93.8	141
Z_x, in.^{3}		139		126		115		102		87.1		78.4	
L_p, ft		8.76		8.76		8.69		8.65		6.78		6.75	
L_r, ft		33.2		31.0		29.3		27.5		22.3		21.1	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

F_y = 50 ksi

W14

Shape	W14× 43		38		34		30		26		22	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 5									142	213	126	189
6					160	239	149	224	134	201	110	166
7			175	262	156	234	135	203	115	172	94.7	142
8	167	251	153	231	136	205	118	177	100	151	82.8	125
9	154	232	136	205	121	182	105	158	89.2	134	73.6	111
10	139	209	123	185	109	164	94.4	142	80.2	121	66.3	99.6
11	126	190	112	168	99.1	149	85.8	129	72.9	110	60.2	90.5
12	116	174	102	154	90.8	137	78.7	118	66.9	101	55.2	83.0
13	107	161	94.4	142	83.8	126	72.6	109	61.7	92.8	51.0	76.6
14	99.2	149	87.7	132	77.8	117	67.4	101	57.3	86.1	47.3	71.1
15	92.6	139	81.8	123	72.7	109	62.9	94.6	53.5	80.4	44.2	66.4
16	86.8	131	76.7	115	68.1	102	59.0	88.7	50.1	75.4	41.4	62.3
17	81.7	123	72.2	109	64.1	96.4	55.5	83.5	47.2	70.9	39.0	58.6
18	77.2	116	68.2	103	60.5	91.0	52.5	78.8	44.6	67.0	36.8	55.3
19	73.1	110	64.6	97.1	57.4	86.2	49.7	74.7	42.2	63.5	34.9	52.4
20	69.5	104	61.4	92.3	54.5	81.9	47.2	71.0	40.1	60.3	33.1	49.8
21	66.2	99.4	58.5	87.9	51.9	78.0	45.0	67.6	38.2	57.4	31.6	47.4
22	63.1	94.9	55.8	83.9	49.5	74.5	42.9	64.5	36.5	54.8	30.1	45.3
23	60.4	90.8	53.4	80.2	47.4	71.2	41.0	61.7	34.9	52.4	28.8	43.3
24	57.9	87.0	51.1	76.9	45.4	68.3	39.3	59.1	33.4	50.3	27.6	41.5
25	55.6	83.5	49.1	73.8	43.6	65.5	37.8	56.8	32.1	48.2	26.5	39.8
26	53.4	80.3	47.2	71.0	41.9	63.0	36.3	54.6	30.9	46.4	25.5	38.3
27	51.5	77.3	45.5	68.3	40.4	60.7	35.0	52.6	29.7	44.7	24.5	36.9
28	49.6	74.6	43.8	65.9	38.9	58.5	33.7	50.7	28.7	43.1	23.7	35.6
29	47.9	72.0	42.3	63.6	37.6	56.5	32.6	48.9	27.7	41.6	22.9	34.3
30	46.3	69.6	40.9	61.5	36.3	54.6	31.5	47.3	26.7	40.2	22.1	33.2
31	44.8	67.4	39.6	59.5	35.2	52.8	30.5	45.8	25.9	38.9	21.4	32.1
32	43.4	65.3	38.4	57.7	34.1	51.2	29.5	44.3	25.1	37.7	20.7	31.1
33	42.1	63.3	37.2	55.9	33.0	49.6	28.6	43.0	24.3	36.5	20.1	30.2
34	40.9	61.4	36.1	54.3	32.1	48.2	27.8	41.7	23.6	35.5	19.5	29.3
35			35.1	52.7	31.1	46.8						

Beam Properties

ASD	LRFD	43 ASD	43 LRFD	38 ASD	38 LRFD	34 ASD	34 LRFD	30 ASD	30 LRFD	26 ASD	26 LRFD	22 ASD	22 LRFD
W_c/Ω_b	$\phi_b W_c$, kip-ft	1390	2090	1230	1850	1090	1640	944	1420	802	1210	663	996
M_p/Ω_b	$\phi_b M_p$, kip-ft	174	261	153	231	136	205	118	177	100	151	82.8	125
M_r/Ω_b	$\phi_b M_r$, kip-ft	109	164	95.4	143	84.9	128	73.4	110	61.7	92.7	50.6	76.1
BF/Ω_b	$\phi_b BF$, kips	4.88	7.28	5.37	8.20	5.01	7.55	4.63	6.95	5.33	8.11	4.78	7.27
V_n/Ω_v	$\phi_v V_n$, kips	83.6	125	87.4	131	79.8	120	74.5	112	70.9	106	63.0	94.5

	43	38	34	30	26	22
Z_x, in.^{3}	69.6	61.5	54.6	47.3	40.2	33.2
L_p, ft	6.68	5.47	5.40	5.26	3.81	3.67
L_r, ft	20.0	16.2	15.6	14.9	11.0	10.4

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

W12

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

F_y = 50 ksi

Shape	W12× 58		53		50		45		40		35	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 6											150	225
7					181	271	162	243			146	219
8					179	270	160	241	140	211	128	192
9	176	264	167	250	159	240	142	214	126	190	114	171
10	172	259	155	234	144	216	128	193	114	171	102	154
11	157	236	141	212	130	196	116	175	103	155	92.9	140
12	144	216	130	195	120	180	107	161	94.8	143	85.2	128
13	133	199	120	180	110	166	98.6	148	87.5	132	78.6	118
14	123	185	111	167	103	154	91.5	138	81.3	122	73.0	110
15	115	173	104	156	95.7	144	85.4	128	75.8	114	68.1	102
16	108	162	97.2	146	89.7	135	80.1	120	71.1	107	63.9	96.0
17	101	152	91.5	137	84.4	127	75.4	113	66.9	101	60.1	90.4
18	95.8	144	86.4	130	79.7	120	71.2	107	63.2	95.0	56.8	85.3
19	90.8	136	81.8	123	75.5	114	67.4	101	59.9	90.0	53.8	80.8
20	86.2	130	77.7	117	71.8	108	64.1	96.3	56.9	85.5	51.1	76.8
21	82.1	123	74.0	111	68.3	103	61.0	91.7	54.2	81.4	48.7	73.1
22	78.4	118	70.7	106	65.2	98.0	58.2	87.5	51.7	77.7	46.5	69.8
23	75.0	113	67.6	102	62.4	93.8	55.7	83.7	49.5	74.3	44.4	66.8
24	71.9	108	64.8	97.4	59.8	89.9	53.4	80.3	47.4	71.3	42.6	64.0
25	69.0	104	62.2	93.5	57.4	86.3	51.3	77.0	45.5	68.4	40.9	61.4
26	66.3	99.7	59.8	89.9	55.2	83.0	49.3	74.1	43.8	65.8	39.3	59.1
27	63.9	96.0	57.6	86.6	53.2	79.9	47.5	71.3	42.1	63.3	37.9	56.9
28	61.6	92.6	55.5	83.5	51.3	77.0	45.8	68.8	40.6	61.1	36.5	54.9
29	59.5	89.4	53.6	80.6	49.5	74.4	44.2	66.4	39.2	59.0	35.2	53.0
30	57.5	86.4	51.8	77.9	47.8	71.9	42.7	64.2			34.1	51.2
31											33.0	49.5

Beam Properties

ASD	LRFD	58 ASD	58 LRFD	53 ASD	53 LRFD	50 ASD	50 LRFD	45 ASD	45 LRFD	40 ASD	40 LRFD	35 ASD	35 LRFD
W_c/Ω_b	$\phi_b W_c$, kip-ft	1720	2590	1550	2340	1440	2160	1280	1930	1140	1710	1020	1540
M_p/Ω_b	$\phi_b M_p$, kip-ft	216	324	194	292	179	270	160	241	142	214	128	192
M_r/Ω_b	$\phi_b M_r$, kip-ft	136	205	123	185	112	169	101	151	89.9	135	79.6	120
BF/Ω_b	$\phi_b BF$, kips	3.82	5.69	3.65	5.50	3.97	5.98	3.80	5.80	3.66	5.54	4.34	6.45
V_n/Ω_v	$\phi_v V_n$, kips	87.8	132	83.5	125	90.3	135	81.1	122	70.2	105	75.0	113

	58	53	50	45	40	35
Z_x, in.^{3}	86.4	77.9	71.9	64.2	57.0	51.2
L_p, ft	8.87	8.76	6.92	6.89	6.85	5.44
L_r, ft	29.8	28.2	23.8	22.4	21.1	16.6

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

F_y = 50 ksi

W12

Shape	W12× 30		26		22		19		16		14[v]	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 3									106	158		
4					128	192	115	172	100	151	85.5	129
5					117	176	98.6	148	80.2	121	69.5	104
6	128	192	112	168	97.5	147	82.2	124	66.9	101	57.9	87.0
7	123	185	106	159	83.5	126	70.4	106	57.3	86.1	49.6	74.6
8	108	162	92.8	140	73.1	110	61.6	92.6	50.1	75.4	43.4	65.3
9	95.6	144	82.5	124	65.0	97.7	54.8	82.3	44.6	67.0	38.6	58.0
10	86.0	129	74.3	112	58.5	87.9	49.3	74.1	40.1	60.3	34.7	52.2
11	78.2	118	67.5	101	53.2	79.9	44.8	67.4	36.5	54.8	31.6	47.5
12	71.7	108	61.9	93.0	48.7	73.3	41.1	61.8	33.4	50.3	28.9	43.5
13	66.2	99.5	57.1	85.8	45.0	67.6	37.9	57.0	30.9	46.4	26.7	40.2
14	61.4	92.4	53.0	79.7	41.8	62.8	35.2	52.9	28.7	43.1	24.8	37.3
15	57.4	86.2	49.5	74.4	39.0	58.6	32.9	49.4	26.7	40.2	23.2	34.8
16	53.8	80.8	46.4	69.8	36.6	54.9	30.8	46.3	25.1	37.7	21.7	32.6
17	50.6	76.1	43.7	65.6	34.4	51.7	29.0	43.6	23.6	35.5	20.4	30.7
18	47.8	71.8	41.3	62.0	32.5	48.8	27.4	41.2	22.3	33.5	19.3	29.0
19	45.3	68.1	39.1	58.7	30.8	46.3	25.9	39.0	21.1	31.7	18.3	27.5
20	43.0	64.7	37.1	55.8	29.2	44.0	24.7	37.1	20.1	30.2	17.4	26.1
21	41.0	61.6	35.4	53.1	27.8	41.9	23.5	35.3	19.1	28.7	16.5	24.9
22	39.1	58.8	33.8	50.7	26.6	40.0	22.4	33.7	18.2	27.4	15.8	23.7
23	37.4	56.2	32.3	48.5	25.4	38.2	21.4	32.2	17.4	26.2	15.1	22.7
24	35.8	53.9	30.9	46.5	24.4	36.6	20.5	30.9	16.7	25.1	14.5	21.8
25	34.4	51.7	29.7	44.6	23.4	35.2	19.7	29.6	16.0	24.1	13.9	20.9
26	33.1	49.7	28.6	42.9	22.5	33.8	19.0	28.5	15.4	23.2	13.4	20.1
27	31.9	47.9	27.5	41.3	21.7	32.6	18.3	27.4	14.9	22.3	12.9	19.3
28	30.7	46.2	26.5	39.9	20.9	31.4	17.6	26.5	14.3	21.5	12.4	18.6
29	29.7	44.6	25.6	38.5	20.2	30.3	17.0	25.6	13.8	20.8	12.0	18.0
30	28.7	43.1	24.8	37.2	19.5	29.3	16.4	24.7	13.4	20.1		

Beam Properties

ASD	LRFD	30 ASD	30 LRFD	26 ASD	26 LRFD	22 ASD	22 LRFD	19 ASD	19 LRFD	16 ASD	16 LRFD	14 ASD	14 LRFD
W_c/Ω_b	$\phi_b W_c$, kip-ft	860	1290	743	1120	585	879	493	741	401	603	347	522
M_p/Ω_b	$\phi_b M_p$, kip-ft	108	162	92.8	140	73.1	110	61.6	92.6	50.1	75.4	43.4	65.3
M_r/Ω_b	$\phi_b M_r$, kip-ft	67.4	101	58.3	87.7	44.4	66.7	37.2	55.9	29.9	44.9	26.0	39.1
BF/Ω_b	$\phi_b BF$, kips	3.97	5.96	3.61	5.46	4.68	7.06	4.27	6.43	3.80	5.73	3.43	5.17
V_n/Ω_v	$\phi_v V_n$, kips	64.0	95.9	56.1	84.2	64.0	95.9	57.3	86.0	52.8	79.2	42.8	64.3

Property	30	26	22	19	16	14
Z_x, in.^{3}	43.1	37.2	29.3	24.7	20.1	17.4
L_p, ft	5.37	5.33	3.00	2.90	2.73	2.66
L_r, ft	15.6	14.9	9.13	8.61	8.05	7.73

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.

Notes: For beams laterally unsupported, see Table 3-10.

Available strength tabulated above heavy line is limited by available shear strength.

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

W10

F_y = 50 ksi

Shape	W10× 45		39		33		30		26	
Design	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Span, ft										
5							126	189	107	161
6					113	169	122	183	104	157
7	141	212	125	187	111	166	104	157	89.3	134
8	137	206	117	176	96.8	146	91.3	137	78.1	117
9	122	183	104	156	86.1	129	81.2	122	69.4	104
10	110	165	93.4	140	77.4	116	73.1	110	62.5	93.9
11	99.6	150	84.9	128	70.4	106	66.4	99.8	56.8	85.4
12	91.3	137	77.8	117	64.5	97.0	60.9	91.5	52.1	78.3
13	84.3	127	71.9	108	59.6	89.5	56.2	84.5	48.1	72.2
14	78.3	118	66.7	100	55.3	83.1	52.2	78.4	44.6	67.1
15	73.1	110	62.3	93.6	51.6	77.6	48.7	73.2	41.7	62.6
16	68.5	103	58.4	87.8	48.4	72.8	45.7	68.6	39.0	58.7
17	64.5	96.9	54.9	82.6	45.6	68.5	43.0	64.6	36.8	55.2
18	60.9	91.5	51.9	78.0	43.0	64.7	40.6	61.0	34.7	52.2
19	57.7	86.7	49.2	73.9	40.8	61.3	38.4	57.8	32.9	49.4
20	54.8	82.4	46.7	70.2	38.7	58.2	36.5	54.9	31.2	47.0
21	52.2	78.4	44.5	66.9	36.9	55.4	34.8	52.3	29.8	44.7
22	49.8	74.9	42.5	63.8	35.2	52.9	33.2	49.9	28.4	42.7
23	47.6	71.6	40.6	61.0	33.7	50.6	31.8	47.7	27.2	40.8
24	45.7	68.6	38.9	58.5	32.3	48.5	30.4	45.8	26.0	39.1
25	43.8	65.9					29.2	43.9	25.0	37.6
26							28.1	42.2		
Beam Properties										
W_c/Ω_b / $\phi_b W_c$, kip-ft	1100	1650	934	1400	774	1160	731	1100	625	939
M_p/Ω_b / $\phi_b M_p$, kip-ft	137	206	117	176	96.8	146	91.3	137	78.1	117
M_r/Ω_b / $\phi_b M_r$, kip-ft	85.8	129	73.5	111	61.1	91.9	56.6	85.1	48.7	73.2
BF/Ω_b / $\phi_b BF$, kips	2.59	3.89	2.53	3.78	2.39	3.62	3.08	4.61	2.91	4.34
V_n/Ω_v / $\phi_v V_n$, kips	70.7	106	62.5	93.7	56.4	84.7	63.0	94.5	53.6	80.3
Z_x, in.^{3}	54.9		46.8		38.8		36.6		31.3	
L_p, ft	7.10		6.99		6.85		4.84		4.80	
L_r, ft	26.9		24.2		21.8		16.1		14.9	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

F_y = 50 ksi

W10–W8

Shape	W10× 22 ASD	W10× 22 LRFD	W10× 19 ASD	W10× 19 LRFD	W10× 17 ASD	W10× 17 LRFD	W10× 15 ASD	W10× 15 LRFD	W10× 12[f] ASD	W10× 12[f] LRFD	W8× 67 ASD	W8× 67 LRFD
Span, ft 3					97.0	145	91.9	138	75.0	113		
4			102	153	93.3	140	79.8	120	62.4	93.8		
5	97.9	147	86.2	130	74.7	112	63.9	96.0	49.9	75.0		
6	86.5	130	71.9	108	62.2	93.5	53.2	80.0	41.6	62.5	205	308
7	74.1	111	61.6	92.6	53.3	80.1	45.6	68.6	35.7	53.6	200	300
8	64.9	97.5	53.9	81.0	46.7	70.1	39.9	60.0	31.2	46.9	175	263
9	57.7	86.7	47.9	72.0	41.5	62.3	35.5	53.3	27.7	41.7	155	234
10	51.9	78.0	43.1	64.8	37.3	56.1	31.9	48.0	25.0	37.5	140	210
11	47.2	70.9	39.2	58.9	33.9	51.0	29.0	43.6	22.7	34.1	127	191
12	43.2	65.0	35.9	54.0	31.1	46.8	26.6	40.0	20.8	31.3	117	175
13	39.9	60.0	33.2	49.8	28.7	43.2	24.6	36.9	19.2	28.9	108	162
14	37.1	55.7	30.8	46.3	26.7	40.1	22.8	34.3	17.8	26.8	99.9	150
15	34.6	52.0	28.7	43.2	24.9	37.4	21.3	32.0	16.6	25.0	93.3	140
16	32.4	48.8	26.9	40.5	23.3	35.1	20.0	30.0	15.6	23.5	87.5	131
17	30.5	45.9	25.4	38.1	22.0	33.0	18.8	28.2	14.7	22.1	82.3	124
18	28.8	43.3	24.0	36.0	20.7	31.2	17.7	26.7	13.9	20.8	77.7	117
19	27.3	41.1	22.7	34.1	19.6	29.5	16.8	25.3	13.1	19.7	73.6	111
20	25.9	39.0	21.6	32.4	18.7	28.1	16.0	24.0	12.5	18.8	70.0	105
21	24.7	37.1	20.5	30.9	17.8	26.7	15.2	22.9	11.9	17.9	66.6	100
22	23.6	35.5	19.6	29.5	17.0	25.5	14.5	21.8	11.3	17.1	63.6	95.6
23	22.6	33.9	18.7	28.2	16.2	24.4	13.9	20.9	10.9	16.3		
24	21.6	32.5	18.0	27.0	15.6	23.4	13.3	20.0	10.4	15.6		
25	20.8	31.2	17.2	25.9	14.9	22.4						

Beam Properties

ASD	LRFD	W10×22 ASD	W10×22 LRFD	W10×19 ASD	W10×19 LRFD	W10×17 ASD	W10×17 LRFD	W10×15 ASD	W10×15 LRFD	W10×12 ASD	W10×12 LRFD	W8×67 ASD	W8×67 LRFD
W_c/Ω_b	$\phi_b W_c$, kip-ft	519	780	431	648	373	561	319	480	250	375	1400	2100
M_p/Ω_b	$\phi_b M_p$, kip-ft	64.9	97.5	53.9	81.0	46.7	70.1	39.9	60.0	31.2	46.9	175	263
M_r/Ω_b	$\phi_b M_r$, kip-ft	40.5	60.9	32.8	49.4	28.3	42.5	24.1	36.2	19.0	28.6	105	159
BF/Ω_b	$\phi_b BF$, kips	2.68	4.02	3.18	4.76	2.98	4.47	2.75	4.14	2.36	3.53	1.75	2.59
V_n/Ω_v	$\phi_v V_n$, kips	49.0	73.4	51.0	76.5	48.5	72.7	46.0	68.9	37.5	56.3	103	154

Property	W10×22	W10×19	W10×17	W10×15	W10×12	W8×67
Z_x, in.^{3}	26.0	21.6	18.7	16.0	12.6	70.1
L_p, ft	4.70	3.09	2.98	2.86	2.87	7.49
L_r, ft	13.8	9.73	9.16	8.61	8.05	47.6

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[f] Shape does not meet compact limit for flexure with F_y = 50 ksi; tabulated values have been adjusted accordingly.

Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

W8

F_y = 50 ksi

Shape	W8× 58 ASD	58 LRFD	48 ASD	48 LRFD	40 ASD	40 LRFD	35 ASD	35 LRFD	31[f] ASD	31[f] LRFD	28 ASD	28 LRFD
Span, ft												
5											91.9	138
6	179	268			119	178	101	151	91.2	137	90.5	136
7	171	256	136	204	113	171	98.9	149	86.6	130	77.6	117
8	149	224	122	184	99.3	149	86.6	130	75.8	114	67.9	102
9	133	199	109	163	88.3	133	77.0	116	67.4	101	60.3	90.7
10	119	179	97.8	147	79.4	119	69.3	104	60.6	91.1	54.3	81.6
11	109	163	88.9	134	72.2	109	63.0	94.6	55.1	82.8	49.4	74.2
12	99.5	150	81.5	123	66.2	99.5	57.7	86.8	50.5	75.9	45.2	68.0
13	91.8	138	75.2	113	61.1	91.8	53.3	80.1	46.6	70.1	41.8	62.8
14	85.3	128	69.9	105	56.7	85.3	49.5	74.4	43.3	65.1	38.8	58.3
15	79.6	120	65.2	98.0	53.0	79.6	46.2	69.4	40.4	60.7	36.2	54.4
16	74.6	112	61.1	91.9	49.7	74.6	43.3	65.1	37.9	56.9	33.9	51.0
17	70.2	106	57.5	86.5	46.7	70.2	40.7	61.2	35.7	53.6	31.9	48.0
18	66.3	99.7	54.3	81.7	44.1	66.3	38.5	57.8	33.7	50.6	30.2	45.3
19	62.8	94.4	51.5	77.4	41.8	62.8	36.5	54.8	31.9	48.0	28.6	42.9
20	59.7	89.7	48.9	73.5	39.7	59.7	34.6	52.1	30.3	45.6	27.1	40.8
21	56.8	85.4	46.6	70.0								
Beam Properties												
W_c/Ω_b / $\phi_b W_c$, kip-ft	1190	1790	978	1470	794	1190	693	1040	606	911	543	816
M_p/Ω_b / $\phi_b M_p$, kip-ft	149	224	122	184	99.3	149	86.6	130	75.8	114	67.9	102
M_r/Ω_b / $\phi_b M_r$, kip-ft	90.8	137	75.4	113	62.0	93.2	54.5	81.9	48.0	72.2	42.4	63.8
BF/Ω_b / $\phi_b BF$, kips	1.70	2.55	1.67	2.55	1.64	2.46	1.62	2.43	1.58	2.37	1.67	2.50
V_n/Ω_v / $\phi_v V_n$, kips	89.3	134	68.0	102	59.4	89.1	50.3	75.5	45.6	68.4	45.9	68.9
Z_x, in.^{3}	59.8		49.0		39.8		34.7		30.4		27.2	
L_p, ft	7.42		7.35		7.21		7.17		7.18		5.72	
L_r, ft	41.6		35.2		29.9		27.0		24.8		21.0	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[f] Shape does not meet compact limit for flexure with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

F_y = 50 ksi

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

W8

Shape	W8× 24		21		18		15		13		10[f]	
Design / Span, ft	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
3							79.5	119	73.5	110	53.7	80.5
4			82.8	124	74.9	112	67.9	102	56.9	85.5	43.7	65.7
5	77.7	117	81.4	122	67.9	102	54.3	81.6	45.5	68.4	35.0	52.6
6	76.8	115	67.9	102	56.6	85.0	45.2	68.0	37.9	57.0	29.2	43.8
7	65.9	99.0	58.2	87.4	48.5	72.9	38.8	58.3	32.5	48.9	25.0	37.6
8	57.6	86.6	50.9	76.5	42.4	63.8	33.9	51.0	28.4	42.8	21.9	32.9
9	51.2	77.0	45.2	68.0	37.7	56.7	30.2	45.3	25.3	38.0	19.4	29.2
10	46.1	69.3	40.7	61.2	33.9	51.0	27.1	40.8	22.8	34.2	17.5	26.3
11	41.9	63.0	37.0	55.6	30.8	46.4	24.7	37.1	20.7	31.1	15.9	23.9
12	38.4	57.8	33.9	51.0	28.3	42.5	22.6	34.0	19.0	28.5	14.6	21.9
13	35.5	53.3	31.3	47.1	26.1	39.2	20.9	31.4	17.5	26.3	13.5	20.2
14	32.9	49.5	29.1	43.7	24.2	36.4	19.4	29.1	16.3	24.4	12.5	18.8
15	30.7	46.2	27.1	40.8	22.6	34.0	18.1	27.2	15.2	22.8	11.7	17.5
16	28.8	43.3	25.4	38.3	21.2	31.9	17.0	25.5	14.2	21.4	10.9	16.4
17	27.1	40.8	24.0	36.0	20.0	30.0	16.0	24.0	13.4	20.1	10.3	15.5
18	25.6	38.5	22.6	34.0	18.9	28.3	15.1	22.7	12.6	19.0	9.72	14.6
19	24.3	36.5	21.4	32.2	17.9	26.8	14.3	21.5	12.0	18.0	9.21	13.8
20			20.4	30.6	17.0	25.5	13.6	20.4				

Beam Properties

ASD	LRFD	24 ASD	24 LRFD	21 ASD	21 LRFD	18 ASD	18 LRFD	15 ASD	15 LRFD	13 ASD	13 LRFD	10[f] ASD	10[f] LRFD
W_c/Ω_b	$\phi_b W_c$, kip-ft	461	693	407	612	339	510	271	408	228	342	175	263
M_p/Ω_b	$\phi_b M_p$, kip-ft	57.6	86.6	50.9	76.5	42.4	63.8	33.9	51.0	28.4	42.8	21.9	32.9
M_r/Ω_b	$\phi_b M_r$, kip-ft	36.5	54.9	31.8	47.8	26.5	39.9	20.6	31.0	17.3	26.0	13.6	20.5
BF/Ω_b	$\phi_b BF$, kips	1.60	2.40	1.85	2.77	1.74	2.61	1.90	2.85	1.76	2.67	1.54	2.30
V_n/Ω_v	$\phi_v V_n$, kips	38.9	58.3	41.4	62.1	37.4	56.2	39.7	59.6	36.8	55.1	26.8	40.2

Property	24	21	18	15	13	10[f]
Z_x, in.^{3}	23.1	20.4	17.0	13.6	11.4	8.87
L_p, ft	5.69	4.45	4.34	3.09	2.98	3.14
L_r, ft	18.9	14.8	13.5	10.1	9.27	8.52

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[f] Shape does not meet compact limit for flexure with F_y = 50 ksi; tabulated values have been adjusted accordingly.

Notes: For beams laterally unsupported, see Table 3-10.
Available strength tabulated above heavy line is limited by available shear strength.

Table 3-7
Maximum Total Uniform Load, kips
S-Shapes

S24–S20

F_y = 36 ksi

Shape	S24×										S20×	
	121		106		100		90		80		96	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 6					515	772					468	702
7	564	847			491	737	432	648			407	611
8	550	826			429	645	399	599	346	518	356	535
9	489	734	437	656	382	574	354	533	326	490	316	475
10	440	661	401	603	343	516	319	480	293	441	285	428
11	400	601	365	548	312	469	290	436	267	401	259	389
12	366	551	334	502	286	430	266	400	244	367	237	356
13	338	508	308	464	264	397	245	369	226	339	219	329
14	314	472	286	430	245	369	228	343	209	315	203	305
15	293	441	267	402	229	344	213	320	195	294	190	285
16	275	413	251	377	215	323	199	300	183	275	178	267
17	259	389	236	354	202	304	188	282	172	259	167	252
18	244	367	223	335	191	287	177	266	163	245	158	238
19	231	348	211	317	181	272	168	252	154	232	150	225
20	220	330	200	301	172	258	160	240	147	220	142	214
21	209	315	191	287	164	246	152	228	140	210	136	204
22	200	300	182	274	156	235	145	218	133	200	129	194
23	191	287	174	262	149	224	139	208	127	192	124	186
24	183	275	167	251	143	215	133	200	122	184	119	178
25	176	264	160	241	137	206	128	192	117	176	114	171
26	169	254	154	232	132	199	123	184	113	169	109	164
27	163	245	149	223	127	191	118	178	109	163	105	158
28	157	236	143	215	123	184	114	171	105	157	102	153
29	152	228	138	208	118	178	110	165	101	152	98.1	147
30	147	220	134	201	114	172	106	160	97.7	147	94.9	143
32	137	207	125	188	107	161	99.7	150	91.6	138	88.9	134
34	129	194	118	177	101	152	93.8	141	86.2	130	83.7	126
36	122	184	111	167	95.4	143	88.6	133	81.4	122	79.0	119
38	116	174	106	159	90.4	136	84.0	126	77.2	116	74.9	113
40	110	165	100	151	85.9	129	79.8	120	73.3	110	71.1	107
42	105	157	95.5	143	81.8	123	76.0	114	69.8	105	67.8	102
44	99.9	150	91.1	137	78.1	117	72.5	109	66.6	100	64.7	97.2
46	95.6	144	87.2	131	74.7	112	69.4	104	63.7	95.8	61.9	93.0
48	91.6	138	83.5	126	71.6	108	66.5	99.9	61.1	91.8	59.3	89.1
50	88.0	132	80.2	121	68.7	103	63.8	95.9	58.6	88.1	56.9	85.5
52	84.6	127	77.1	116	66.1	99.3	61.4	92.2	56.4	84.7		
54	81.4	122	74.3	112	63.6	95.6	59.1	88.8	54.3	81.6		
56	78.5	118	71.6	108	61.3	92.2	57.0	85.6	52.4	78.7		
58	75.8	114	69.1	104	59.2	89.0	55.0	82.7	50.5	76.0		
60	73.3	110	66.8	100	57.2	86.0	53.2	79.9	48.9	73.4		
Beam Properties												
W_c/Ω_b / $\phi_b W_c$, kip-ft	4400	6610	4010	6030	3430	5160	3190	4800	2930	4410	2850	4280
M_p/Ω_b / $\phi_b M_p$, kip-ft	550	826	501	753	429	645	399	599	366	551	356	535
M_r/Ω_b / $\phi_b M_r$, kip-ft	324	488	302	454	250	376	235	353	220	331	207	312
BF/Ω_b / $\phi_b BF$, kips	11.4	17.1	11.0	16.5	11.6	17.5	11.4	17.1	10.8	16.2	7.63	11.5
V_n/Ω_v / $\phi_v V_n$, kips	282	423	219	328	257	386	216	324	173	259	234	351
Z_x, in.^{3}	306		279		239		222		204		198	
L_p, ft	6.37		6.54		5.29		5.41		5.58		5.54	
L_r, ft	26.2		24.7		20.7		19.8		19.2		24.9	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Notes: Beams must be laterally supported if Table 3-7 is used.
Available strength tabulated above heavy line is limited by available shear strength.

Table 3-7 (continued)
Maximum Total Uniform Load, kips
S-Shapes

F_y = 36 ksi

S20–S15

Shape	S20×						S18×				S15×	
	86		75		66		70		54.7		50	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 4							369	553			238	356
5			366	549			356	536			221	333
6	386	579	364	547	291	436	297	446	239	358	184	277
7	376	565	312	469	285	429	255	383	214	321	158	238
8	329	494	273	410	250	375	223	335	187	281	138	208
9	292	439	243	365	222	334	198	298	166	250	123	185
10	263	395	218	328	200	300	178	268	149	225	111	166
11	239	359	199	298	182	273	162	243	136	204	101	151
12	219	329	182	274	166	250	149	223	125	187	92.2	139
13	202	304	168	253	154	231	137	206	115	173	85.1	128
14	188	282	156	235	143	214	127	191	107	160	79.0	119
15	175	264	146	219	133	200	119	179	99.6	150	73.8	111
16	164	247	137	205	125	188	111	167	93.4	140	69.2	104
17	155	233	128	193	118	177	105	158	87.9	132	65.1	97.8
18	146	220	121	182	111	167	99.0	149	83.0	125	61.5	92.4
19	138	208	115	173	105	158	93.8	141	78.7	118	58.2	87.5
20	131	198	109	164	99.9	150	89.1	134	74.7	112	55.3	83.2
21	125	188	104	156	95.1	143	84.9	128	71.2	107	52.7	79.2
22	120	180	99.3	149	90.8	136	81.0	122	67.9	102	50.3	75.6
23	114	172	95.0	143	86.9	131	77.5	116	65.0	97.7	48.1	72.3
24	110	165	91.0	137	83.2	125	74.3	112	62.3	93.6	46.1	69.3
25	105	158	87.4	131	79.9	120	71.3	107	59.8	89.9	44.3	66.5
26	101	152	84.0	126	76.8	115	68.5	103	57.5	86.4	42.6	64.0
27	97.4	146	80.9	122	74.0	111	66.0	99.2	55.4	83.2	41.0	61.6
28	93.9	141	78.0	117	71.3	107	63.6	95.7	53.4	80.2	39.5	59.4
29	90.7	136	75.3	113	68.9	104	61.4	92.4	51.5	77.5	38.2	57.4
30	87.7	132	72.8	109	66.6	100	59.4	89.3	49.8	74.9	36.9	55.4
32	82.2	124	68.3	103	62.4	93.8	55.7	83.7	46.7	70.2	34.6	52.0
34	77.4	116	64.2	96.6	58.8	88.3	52.4	78.8	44.0	66.1	32.5	48.9
36	73.1	110	60.7	91.2	55.5	83.4	49.5	74.4	41.5	62.4	30.7	46.2
38	69.2	104	57.5	86.4	52.6	79.0	46.9	70.5	39.3	59.1		
40	65.7	98.8	54.6	82.1	49.9	75.1	44.6	67.0	37.4	56.2		
42	62.6	94.1	52.0	78.2	47.6	71.5	42.4	63.8	35.6	53.5		
44	59.8	89.8	49.6	74.6	45.4	68.2	40.5	60.9	34.0	51.1		
46	57.2	85.9	47.5	71.4	43.4	65.3						
48	54.8	82.4	45.5	68.4	41.6	62.6						
50	52.6	79.1	43.7	65.7	40.0	60.0						
Beam Properties												
W_c/Ω_b / $\phi_b W_c$, kip-ft	2630	3950	2180	3280	2000	3000	1780	2680	1490	2250	1110	1660
M_p/Ω_b / $\phi_b M_p$, kip-ft	329	494	273	410	250	375	223	335	187	281	138	208
M_r/Ω_b / $\phi_b M_r$, kip-ft	195	293	161	242	150	225	130	195	112	168	81.4	122
BF/Ω_b / $\phi_b BF$, kips	7.53	11.3	7.74	11.6	7.49	11.3	6.12	9.19	5.98	8.99	4.07	6.12
V_n/Ω_v / $\phi_v V_n$, kips	193	289	183	274	145	218	184	276	119	179	119	178
Z_x, in.^{3}	183		152		139		124		104		77.0	
L_p, ft	5.66		4.83		4.95		4.50		4.75		4.29	
L_r, ft	23.4		19.3		18.3		19.7		17.3		18.3	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Notes: Beams must be laterally supported if Table 3-7 is used.
Available strength tabulated above heavy line is limited by available shear strength.

Table 3-7 (continued)
Maximum Total Uniform Load, kips
S-Shapes

S15–S10

F_y = 36 ksi

Shape	S15×		S12×								S10×	
	42.9		50		40.8		35		31.8		35	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 2											171	257
3			237	356							170	255
4			219	329	160	240	148	222	121	181	127	191
5	178	266	175	263	151	228	128	193	120	181	102	153
6	166	249	146	219	126	190	107	161	100	150	84.8	127
7	142	214	125	188	108	163	91.6	138	85.8	129	72.7	109
8	124	187	109	164	94.7	142	80.1	120	75.1	113	63.6	95.6
9	110	166	97.2	146	84.2	126	71.2	107	66.7	100	56.5	85.0
10	99.4	149	87.5	132	75.7	114	64.1	96.3	60.1	90.3	50.9	76.5
11	90.4	136	79.6	120	68.9	103	58.3	87.6	54.6	82.1	46.2	69.5
12	82.9	125	72.9	110	63.1	94.9	53.4	80.3	50.1	75.2	42.4	63.7
13	76.5	115	67.3	101	58.3	87.6	49.3	74.1	46.2	69.5	39.1	58.8
14	71.0	107	62.5	94.0	54.1	81.3	45.8	68.8	42.9	64.5	36.3	54.6
15	66.3	99.6	58.3	87.7	50.5	75.9	42.7	64.2	40.0	60.2	33.9	51.0
16	62.2	93.4	54.7	82.2	47.3	71.1	40.1	60.2	37.5	56.4	31.8	47.8
17	58.5	87.9	51.5	77.4	44.6	67.0	37.7	56.7	35.3	53.1	29.9	45.0
18	55.2	83.0	48.6	73.1	42.1	63.2	35.6	53.5	33.4	50.2	28.3	42.5
19	52.3	78.7	46.1	69.2	39.9	59.9	33.7	50.7	31.6	47.5	26.8	40.2
20	49.7	74.7	43.8	65.8	37.9	56.9	32.0	48.2	30.0	45.1	25.4	38.2
21	47.4	71.2	41.7	62.6	36.1	54.2	30.5	45.9	28.6	43.0	24.2	36.4
22	45.2	67.9	39.8	59.8	34.4	51.7	29.1	43.8	27.3	41.0	23.1	34.8
23	43.2	65.0	38.1	57.2	32.9	49.5	27.9	41.9	26.1	39.3	22.1	33.2
24	41.4	62.3	36.5	54.8	31.6	47.4	26.7	40.1	25.0	37.6	21.2	31.9
25	39.8	59.8	35.0	52.6	30.3	45.5	25.6	38.5	24.0	36.1	20.3	30.6
26	38.2	57.5	33.7	50.6	29.1	43.8	24.7	37.1	23.1	34.7		
27	36.8	55.4	32.4	48.7	28.1	42.2	23.7	35.7	22.2	33.4		
28	35.5	53.4	31.3	47.0	27.0	40.7	22.9	34.4	21.5	32.2		
29	34.3	51.5	30.2	45.4	26.1	39.3	22.1	33.2	20.7	31.1		
30	33.1	49.8	29.2	43.8	25.2	37.9	21.4	32.1	20.0	30.1		
32	31.1	46.7										
34	29.2	44.0										
36	27.6	41.5										
Beam Properties												
W_c/Ω_b / $\phi_b W_c$, kip-ft	994	1490	875	1320	757	1140	641	963	601	903	509	765
M_p/Ω_b / $\phi_b M_p$, kip-ft	124	187	109	164	94.7	142	80.1	120	75.1	113	63.6	95.6
M_r/Ω_b / $\phi_b M_r$, kip-ft	74.7	112	63.6	95.6	56.7	85.2	47.9	72.0	45.5	68.4	37.0	55.6
BF/Ω_b / $\phi_b BF$, kips	4.01	6.03	2.22	3.33	2.31	3.48	2.45	3.69	2.43	3.66	1.51	2.26
V_n/Ω_v / $\phi_v V_n$, kips	88.8	133	119	178	79.8	120	74.0	111	60.5	90.7	85.5	128
Z_x, in.^{3}	69.2		60.9		52.7		44.6		41.8		35.4	
L_p, ft	4.41		4.29		4.41		4.08		4.16		3.74	
L_r, ft	16.8		24.9		20.8		17.2		16.3		21.4	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Notes: Beams must be laterally supported if Table 3-7 is used.
Available strength tabulated above heavy line is limited by available shear strength.

F_y = 36 ksi

Table 3-7 (continued)
Maximum Total Uniform Load, kips
S-Shapes

S10–S5

Shape	S10× 25.4		S8× 23		S8× 18.4		S6× 17.25		S6× 12.5		S5× 10	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 2			102	152			75.4	113			30.8	46.2
3			92.0	138	62.4	93.7	50.3	75.6	40.1	60.1	27.1	40.8
4	89.6	134	69.0	104	59.3	89.1	37.7	56.7	30.4	45.6	20.3	30.6
5	81.3	122	55.2	82.9	47.4	71.3	30.2	45.4	24.3	36.5	16.3	24.5
6	67.8	102	46.0	69.1	39.5	59.4	25.1	37.8	20.2	30.4	13.6	20.4
7	58.1	87.3	39.4	59.2	33.9	50.9	21.6	32.4	17.3	26.1	11.6	17.5
8	50.8	76.4	34.5	51.8	29.6	44.6	18.9	28.4	15.2	22.8	10.2	15.3
9	45.2	67.9	30.7	46.1	26.3	39.6	16.8	25.2	13.5	20.3	9.04	13.6
10	40.7	61.1	27.6	41.5	23.7	35.6	15.1	22.7	12.1	18.3	8.13	12.2
11	37.0	55.6	25.1	37.7	21.6	32.4	13.7	20.6	11.0	16.6	7.39	11.1
12	33.9	50.9	23.0	34.6	19.8	29.7	12.6	18.9	10.1	15.2	6.78	10.2
13	31.3	47.0	21.2	31.9	18.2	27.4	11.6	17.4	9.34	14.0		
14	29.1	43.7	19.7	29.6	16.9	25.5	10.8	16.2	8.67	13.0		
15	27.1	40.8	18.4	27.6	15.8	23.8	10.1	15.1	8.10	12.2		
16	25.4	38.2	17.2	25.9	14.8	22.3						
17	23.9	36.0	16.2	24.4	13.9	21.0						
18	22.6	34.0	15.3	23.0	13.2	19.8						
19	21.4	32.2	14.5	21.8	12.5	18.8						
20	20.3	30.6	13.8	20.7	11.9	17.8						
21	19.4	29.1										
22	18.5	27.8										
23	17.7	26.6										
24	16.9	25.5										
25	16.3	24.5										

Beam Properties

ASD / LRFD	S10×25.4 ASD	LRFD	S8×23 ASD	LRFD	S8×18.4 ASD	LRFD	S6×17.25 ASD	LRFD	S6×12.5 ASD	LRFD	S5×10 ASD	LRFD
W_c/Ω_b / $\phi_b W_c$, kip-ft	407	611	276	415	237	356	151	227	121	183	81.3	122
M_p/Ω_b / $\phi_b M_p$, kip-ft	50.8	76.4	34.5	51.8	29.6	44.6	18.9	28.4	15.2	22.8	10.2	15.3
M_r/Ω_b / $\phi_b M_r$, kip-ft	30.9	46.5	20.4	30.6	18.1	27.2	11.0	16.5	9.23	13.9	6.16	9.26
BF/Ω_b / $\phi_b BF$, kips	1.58	2.38	0.948	1.42	0.974	1.46	0.460	0.691	0.516	0.775	0.341	0.512
V_n/Ω_v / $\phi_v V_n$, kips	44.8	67.2	50.8	76.2	31.2	46.8	40.2	60.3	20.0	30.1	15.4	23.1
Z_x, in.^{3}	28.3		19.2		16.5		10.5		8.45		5.66	
L_p, ft	3.95		3.31		3.44		2.80		2.92		2.66	
L_r, ft	16.5		18.2		15.3		19.9		14.5		14.4	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Notes: Beams must be laterally supported if Table 3-7 is used.
Available strength tabulated above heavy line is limited by available shear strength.

S4–S3

Table 3-7 (continued)
Maximum Total Uniform Load, kips
S-Shapes

F_y = 36 ksi

Shape	S4×				S3×			
	9.5		7.7		7.5		5.7	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 2	29.0	43.6	22.2	33.4	16.9	25.4	13.9	21.0
3	19.4	29.1	16.8	25.2	11.3	16.9	9.29	14.0
4	14.5	21.8	12.6	18.9	8.44	12.7	6.97	10.5
5	11.6	17.5	10.1	15.1	6.75	10.2	5.58	8.38
6	9.68	14.5	8.38	12.6	5.63	8.46	4.65	6.98
7	8.29	12.5	7.19	10.8	4.82	7.25	3.98	5.99
8	7.26	10.9	6.29	9.45				
9	6.45	9.70	5.59	8.40				
10	5.81	8.73	5.03	7.56				
Beam Properties								
W_c/Ω_b / $\phi_b W_c$, kip-ft	58.1	87.3	50.3	75.6	33.8	50.8	27.9	41.9
M_p/Ω_b / $\phi_b M_p$, kip-ft	7.26	10.9	6.29	9.45	4.22	6.35	3.49	5.24
M_r/Ω_b / $\phi_b M_r$, kip-ft	4.25	6.39	3.81	5.73	2.44	3.67	2.10	3.16
BF/Ω_b / $\phi_b BF$, kips	0.190	0.285	0.202	0.304	0.0899	0.135	0.102	0.154
V_n/Ω_v / $\phi_v V_n$, kips	18.8	28.2	11.1	16.7	15.1	22.6	7.34	11.0
Z_x, in.^{3}	4.04		3.50		2.35		1.94	
L_p, ft	2.35		2.40		2.14		2.16	
L_r, ft	18.2		14.6		22.0		15.7	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Notes: Beams must be laterally supported if Table 3-7 is used.
Available strength tabulated above heavy line is limited by available shear strength.

F_y = 36 ksi

Table 3-8
Maximum Total Uniform Load, kips
C-Shapes

C15–C12

Shape	C15×						C12×					
	50		40		33.9		30		25		20.7	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 3	278	418					158	238	120	181		
4	246	370	202	303	155	233	121	183	106	159	87.5	132
5	197	296	165	248	146	219	97.1	146	84.5	127	73.6	111
6	164	247	138	207	122	183	81.0	122	70.4	106	61.3	92.2
7	141	211	118	177	104	157	69.4	104	60.4	90.7	52.6	79.0
8	123	185	103	155	91.3	137	60.7	91.3	52.8	79.4	46.0	69.1
9	109	164	91.8	138	81.1	122	54.0	81.1	46.9	70.6	40.9	61.4
10	98.4	148	82.6	124	73.0	110	48.6	73.0	42.3	63.5	36.8	55.3
11	89.5	135	75.1	113	66.4	99.8	44.2	66.4	38.4	57.7	33.4	50.3
12	82.0	123	68.9	104	60.8	91.4	40.5	60.8	35.2	52.9	30.7	46.1
13	75.7	114	63.6	95.5	56.2	84.4	37.4	56.2	32.5	48.8	28.3	42.5
14	70.3	106	59.0	88.7	52.1	78.4	34.7	52.1	30.2	45.4	26.3	39.5
15	65.6	98.6	55.1	82.8	48.7	73.2	32.4	48.7	28.2	42.3	24.5	36.9
16	61.5	92.5	51.6	77.6	45.6	68.6	30.4	45.6	26.4	39.7	23.0	34.6
17	57.9	87.0	48.6	73.1	42.9	64.5	28.6	42.9	24.9	37.4	21.6	32.5
18	54.7	82.2	45.9	69.0	40.6	61.0	27.0	40.6	23.5	35.3	20.4	30.7
19	51.8	77.9	43.5	65.4	38.4	57.8	25.6	38.4	22.2	33.4	19.4	29.1
20	49.2	74.0	41.3	62.1	36.5	54.9	24.3	36.5	21.1	31.8	18.4	27.6
21	46.9	70.5	39.3	59.1	34.8	52.3	23.1	34.8	20.1	30.2	17.5	26.3
22	44.7	67.3	37.6	56.5	33.2	49.9	22.1	33.2	19.2	28.9	16.7	25.1
23	42.8	64.3	35.9	54.0	31.7	47.7	21.1	31.7	18.4	27.6	16.0	24.0
24	41.0	61.7	34.4	51.8	30.4	45.7	20.2	30.4	17.6	26.5	15.3	23.0
25	39.4	59.2	33.1	49.7	29.2	43.9	19.4	29.2	16.9	25.4	14.7	22.1
26	37.9	56.9	31.8	47.8	28.1	42.2	18.7	28.1	16.3	24.4	14.2	21.3
27	36.5	54.8	30.6	46.0	27.0	40.6	18.0	27.0	15.6	23.5	13.6	20.5
28	35.2	52.8	29.5	44.4	26.1	39.2	17.3	26.1	15.1	22.7	13.1	19.7
29	33.9	51.0	28.5	42.8	25.2	37.8	16.7	25.2	14.6	21.9	12.7	19.1
30	32.8	49.3	27.5	41.4	24.3	36.6	16.2	24.3	14.1	21.2	12.3	18.4
31	31.8	47.7	26.7	40.1	23.6	35.4						
32	30.8	46.2	25.8	38.8	22.8	34.3						
33	29.8	44.8	25.0	37.6	22.1	33.3						
34	29.0	43.5	24.3	36.5	21.5	32.3						
35	28.1	42.3	23.6	35.5	20.9	31.4						
36	27.3	41.1	23.0	34.5	20.3	30.5						
37	26.6	40.0	22.3	33.6	19.7	29.7						
Beam Properties												
W_c/Ω_b / $\phi_b W_c$, kip-ft	984	1480	826	1240	730	1100	486	730	423	635	368	553
M_p/Ω_b / $\phi_b M_p$, kip-ft	123	185	103	155	91.3	137	60.7	91.3	52.8	79.4	46.0	69.1
M_r/Ω_b / $\phi_b M_r$, kip-ft	67.7	102	58.5	87.9	52.8	79.4	34.0	51.0	30.2	45.4	27.0	40.6
BF/Ω_b / $\phi_b BF$, kips	3.46	5.19	3.58	5.40	3.58	5.36	2.18	3.30	2.22	3.35	2.16	3.25
V_n/Ω_v / $\phi_v V_n$, kips	139	209	101	152	77.6	117	79.2	119	60.1	90.3	43.8	65.8
Z_x, in.^{3}	68.5		57.5		50.8		33.8		29.4		25.6	
L_p, ft	3.60		3.68		3.75		3.17		3.24		3.32	
L_r, ft	19.6		16.1		14.5		15.4		13.4		12.1	

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.67$	$\phi_b = 0.90$ $\phi_v = 0.90$

Notes: For beams laterally unsupported, see Table 3-11.
Available strength tabulated above heavy line is limited by available shear strength.

C10–C9

Table 3-8 (continued)
Maximum Total Uniform Load, kips
C-Shapes

F_y = 36 ksi

Shape		C10× 30		C10× 25		C10× 20		C10× 15.3		C9× 20	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	2	174	262	136	205	98.0	147			104	157
	3	128	192	111	166	92.9	140	62.1	93.3	81.0	122
	4	95.9	144	83.0	125	69.7	105	57.1	85.9	60.7	91.3
	5	76.7	115	66.4	99.8	55.8	83.8	45.7	68.7	48.6	73.0
	6	64.0	96.1	55.3	83.2	46.5	69.8	38.1	57.2	40.5	60.8
	7	54.8	82.4	47.4	71.3	39.8	59.9	32.6	49.1	34.7	52.1
	8	48.0	72.1	41.5	62.4	34.9	52.4	28.6	42.9	30.4	45.6
	9	42.6	64.1	36.9	55.4	31.0	46.6	25.4	38.2	27.0	40.6
	10	38.4	57.7	33.2	49.9	27.9	41.9	22.9	34.3	24.3	36.5
	11	34.9	52.4	30.2	45.4	25.3	38.1	20.8	31.2	22.1	33.2
	12	32.0	48.1	27.7	41.6	23.2	34.9	19.0	28.6	20.2	30.4
	13	29.5	44.4	25.5	38.4	21.4	32.2	17.6	26.4	18.7	28.1
	14	27.4	41.2	23.7	35.6	19.9	29.9	16.3	24.5	17.3	26.1
	15	25.6	38.4	22.1	33.3	18.6	27.9	15.2	22.9	16.2	24.3
	16	24.0	36.0	20.7	31.2	17.4	26.2	14.3	21.5	15.2	22.8
	17	22.6	33.9	19.5	29.4	16.4	24.6	13.4	20.2	14.3	21.5
	18	21.3	32.0	18.4	27.7	15.5	23.3	12.7	19.1	13.5	20.3
	19	20.2	30.4	17.5	26.3	14.7	22.1	12.0	18.1	12.8	19.2
	20	19.2	28.8	16.6	24.9	13.9	21.0	11.4	17.2	12.1	18.3
	21	18.3	27.5	15.8	23.8	13.3	20.0	10.9	16.4	11.6	17.4
	22	17.4	26.2	15.1	22.7	12.7	19.0	10.4	15.6	11.0	16.6
	23	16.7	25.1	14.4	21.7	12.1	18.2	9.93	14.9		
	24	16.0	24.0	13.8	20.8	11.6	17.5	9.52	14.3		
	25	15.3	23.1	13.3	20.0	11.2	16.8	9.14	13.7		
Beam Properties											
W_c/Ω_b	$\phi_b W_c$, kip-ft	384	577	332	499	279	419	229	343	243	365
M_p/Ω_b	$\phi_b M_p$, kip-ft	48.0	72.1	41.5	62.4	34.9	52.4	28.6	42.9	30.4	45.6
M_r/Ω_b	$\phi_b M_r$, kip-ft	26.0	39.1	22.9	34.4	19.9	29.9	17.0	25.5	17.0	25.5
BF/Ω_b	$\phi_b BF$, kips	1.27	1.91	1.40	2.11	1.48	2.22	1.44	2.16	1.12	1.68
V_n/Ω_v	$\phi_v V_n$, kips	87.0	131	68.0	102	49.0	73.7	31.0	46.7	52.2	78.4
Z_x, in.^{3}		26.7		23.1		19.4		15.9		16.9	
L_p, ft		2.78		2.81		2.87		2.96		2.66	
L_r, ft		20.1		16.1		13.0		11.0		14.6	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.67$	$\phi_v = 0.90$

Notes: For beams laterally unsupported, see Table 3-11.
Available strength tabulated above heavy line is limited by available shear strength.

F_y = 36 ksi

Table 3-8 (continued)
Maximum Total Uniform Load, kips
C-Shapes

C9–C8

Shape	C9×				C8×					
	15		13.4		18.75		13.7		11.5	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 2	66.4	99.7			99.9	150	62.7	94.2		
3	65.1	97.9	54.2	81.5	66.6	100	52.7	79.2	45.5	68.4
4	48.9	73.4	45.3	68.0	49.9	75.1	39.5	59.4	34.6	52.0
5	39.1	58.8	36.2	54.4	40.0	60.0	31.6	47.5	27.7	41.6
6	32.6	49.0	30.2	45.4	33.3	50.0	26.3	39.6	23.1	34.7
7	27.9	42.0	25.9	38.9	28.5	42.9	22.6	33.9	19.8	29.7
8	24.4	36.7	22.6	34.0	25.0	37.5	19.8	29.7	17.3	26.0
9	21.7	32.6	20.1	30.2	22.2	33.4	17.6	26.4	15.4	23.1
10	19.5	29.4	18.1	27.2	20.0	30.0	15.8	23.8	13.8	20.8
11	17.8	26.7	16.5	24.7	18.2	27.3	14.4	21.6	12.6	18.9
12	16.3	24.5	15.1	22.7	16.6	25.0	13.2	19.8	11.5	17.3
13	15.0	22.6	13.9	20.9	15.4	23.1	12.2	18.3	10.6	16.0
14	14.0	21.0	12.9	19.4	14.3	21.4	11.3	17.0	9.89	14.9
15	13.0	19.6	12.1	18.1	13.3	20.0	10.5	15.8	9.23	13.9
16	12.2	18.4	11.3	17.0	12.5	18.8	9.88	14.9	8.65	13.0
17	11.5	17.3	10.7	16.0	11.8	17.7	9.30	14.0	8.14	12.2
18	10.9	16.3	10.1	15.1	11.1	16.7	8.78	13.2	7.69	11.6
19	10.3	15.5	9.53	14.3	10.5	15.8	8.32	12.5	7.28	10.9
20	9.77	14.7	9.05	13.6	9.99	15.0	7.90	11.9	6.92	10.4
21	9.31	14.0	8.62	13.0						
22	8.88	13.4	8.23	12.4						
Beam Properties										
W_c/Ω_b | $\phi_b W_c$, kip-ft	195	294	181	272	200	300	158	238	138	208
M_p/Ω_b | $\phi_b M_p$, kip-ft	24.4	36.7	22.6	34.0	25.0	37.5	19.8	29.7	17.3	26.0
M_r/Ω_b | $\phi_b M_r$, kip-ft	14.2	21.4	13.3	20.0	13.8	20.8	11.3	17.0	10.2	15.4
BF/Ω_b | $\phi_b BF$, kips	1.18	1.77	1.17	1.77	0.829	1.24	0.929	1.39	0.909	1.36
V_n/Ω_v | $\phi_v V_n$, kips	33.2	49.9	27.1	40.8	50.4	75.7	31.4	47.1	22.8	34.2
Z_x, in.^{3}	13.6		12.6		13.9		11.0		9.63	
L_p, ft	2.74		2.77		2.49		2.55		2.59	
L_r, ft	11.4		10.7		16.0		11.7		10.4	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.67$	$\phi_v = 0.90$

Notes: For beams laterally unsupported, see Table 3-11.
Available strength tabulated above heavy line is limited by available shear strength.

C7–C6

Table 3-8 (continued)
Maximum Total Uniform Load, kips
C-Shapes

F_y = 36 ksi

Shape		C7× 14.75		C7× 12.25		C7× 9.8		C6× 13		C6× 10.5	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	2	70.1	105	56.9	85.5	38.0	57.2	52.4	78.7	44.4	66.7
	3	46.7	70.2	40.5	60.9	34.4	51.8	34.9	52.5	29.6	44.5
	4	35.0	52.7	30.4	45.7	25.8	38.8	26.2	39.4	22.2	33.4
	5	28.0	42.1	24.3	36.5	20.7	31.1	21.0	31.5	17.8	26.7
	6	23.4	35.1	20.3	30.5	17.2	25.9	17.5	26.2	14.8	22.2
	7	20.0	30.1	17.4	26.1	14.8	22.2	15.0	22.5	12.7	19.1
	8	17.5	26.3	15.2	22.8	12.9	19.4	13.1	19.7	11.1	16.7
	9	15.6	23.4	13.5	20.3	11.5	17.3	11.6	17.5	9.87	14.8
	10	14.0	21.1	12.2	18.3	10.3	15.5	10.5	15.7	8.88	13.3
	11	12.7	19.1	11.1	16.6	9.39	14.1	9.52	14.3	8.07	12.1
	12	11.7	17.6	10.1	15.2	8.61	12.9	8.73	13.1	7.40	11.1
	13	10.8	16.2	9.35	14.1	7.95	11.9	8.06	12.1	6.83	10.3
	14	10.0	15.0	8.68	13.1	7.38	11.1	7.48	11.2	6.34	9.53
	15	9.34	14.0	8.11	12.2	6.89	10.4	6.98	10.5	5.92	8.90
	16	8.76	13.2	7.60	11.4	6.46	9.72				
	17	8.24	12.4	7.15	10.7	6.08	9.14				
Beam Properties											
W_c/Ω_b	$\phi_b W_c$, kip-ft	140	211	122	183	103	155	105	157	88.8	133
M_p/Ω_b	$\phi_b M_p$, kip-ft	17.5	26.3	15.2	22.8	12.9	19.4	13.1	19.7	11.1	16.7
M_r/Ω_b	$\phi_b M_r$, kip-ft	9.78	14.7	8.70	13.1	7.63	11.5	7.27	10.9	6.34	9.53
BF/Ω_b	$\phi_b BF$, kips	0.620	0.931	0.661	0.986	0.677	1.01	0.413	0.623	0.458	0.689
V_n/Ω_v	$\phi_v V_n$, kips	37.9	57.0	28.4	42.7	19.0	28.6	33.9	51.0	24.4	36.6
Z_x, in.^{3}		9.75		8.46		7.19		7.29		6.18	
L_p, ft		2.34		2.36		2.41		2.18		2.20	
L_r, ft		14.8		12.2		10.2		16.3		12.6	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.67$	$\phi_v = 0.90$

Notes: For beams laterally unsupported, see Table 3-11.
Available strength tabulated above heavy line is limited by available shear strength.

F_y = 36 ksi

Table 3-8 (continued)
Maximum Total Uniform Load, kips
C-Shapes

C6–C4

Shape		C6×		C5×				C4×					
		8.2		9		6.7		7.25		6.25		5.4	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	2	31.0	46.7	31.5	47.4	24.6	36.9	20.4	30.7	18.3	27.5	16.5	24.7
	3	24.7	37.2	21.0	31.6	17.0	25.6	13.6	20.4	12.2	18.4	11.0	16.5
	4	18.5	27.9	15.8	23.7	12.8	19.2	10.2	15.3	9.16	13.8	8.23	12.4
	5	14.8	22.3	12.6	19.0	10.2	15.3	8.16	12.3	7.33	11.0	6.58	9.89
	6	12.4	18.6	10.5	15.8	8.50	12.8	6.80	10.2	6.11	9.18	5.49	8.24
	7	10.6	15.9	9.01	13.5	7.29	11.0	5.83	8.76	5.24	7.87	4.70	7.07
	8	9.27	13.9	7.89	11.9	6.38	9.59	5.10	7.67	4.58	6.89	4.11	6.18
	9	8.24	12.4	7.01	10.5	5.67	8.52	4.53	6.82	4.07	6.12	3.66	5.50
	10	7.42	11.1	6.31	9.48	5.10	7.67	4.08	6.13	3.66	5.51	3.29	4.95
	11	6.74	10.1	5.74	8.62	4.64	6.97						
	12	6.18	9.29	5.26	7.90	4.25	6.39						
	13	5.70	8.57										
	14	5.30	7.96										
	15	4.94	7.43										
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	74.2	111	63.1	94.8	51.0	76.7	40.8	61.3	36.6	55.1	32.9	49.5
M_p/Ω_b	$\phi_b M_p$, kip-ft	9.27	13.9	7.89	11.9	6.38	9.59	5.10	7.67	4.58	6.89	4.11	6.18
M_r/Ω_b	$\phi_b M_r$, kip-ft	5.47	8.22	4.48	6.73	3.76	5.65	2.88	4.33	2.64	3.97	2.41	3.63
BF/Ω_b	$\phi_b BF$, kips	0.477	0.713	0.287	0.435	0.313	0.471	0.165	0.249	0.176	0.265	0.186	0.279
V_n/Ω_v	$\phi_v V_n$, kips	15.5	23.3	21.0	31.6	12.3	18.5	16.6	25.0	12.8	19.2	9.52	14.3
Z_x, in.^{3}		5.16		4.39		3.55		2.84		2.55		2.29	
L_p, ft		2.23		2.02		2.04		1.86		1.88		1.85	
L_r, ft		10.2		13.9		10.4		15.3		12.9		11.0	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.67$	$\phi_v = 0.90$

Notes: For beams laterally unsupported, see Table 3-11.
Available strength tabulated above heavy line is limited by available shear strength.

C4–C3

Table 3-8 (continued)
Maximum Total Uniform Load, kips
C-Shapes

F_y = 36 ksi

Shape		C4×		C3×							
		4.5		6		5		4.1		3.5	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	2	12.9	19.4	12.5	18.8	10.9	16.4	9.49	14.3	8.91	13.4
	3	9.82	14.8	8.34	12.5	7.28	10.9	6.32	9.50	5.94	8.93
	4	7.37	11.1	6.25	9.40	5.46	8.21	4.74	7.13	4.46	6.70
	5	5.89	8.86	5.00	7.52	4.37	6.57	3.79	5.70	3.56	5.36
	6	4.91	7.38	4.17	6.26	3.64	5.47	3.16	4.75	2.97	4.46
	7	4.21	6.33	3.57	5.37	3.12	4.69	2.71	4.07	2.55	3.83
	8	3.68	5.54								
	9	3.27	4.92								
	10	2.95	4.43								
Beam Properties											
W_c/Ω_b	$\phi_b W_c$, kip-ft	29.4	44.3	25.0	37.6	21.8	32.8	19.0	28.5	17.8	26.8
M_p/Ω_b	$\phi_b M_p$, kip-ft	3.68	5.54	3.13	4.70	2.73	4.10	2.37	3.56	2.23	3.35
M_r/Ω_b	$\phi_b M_r$, kip-ft	2.23	3.35	1.74	2.61	1.55	2.32	1.38	2.08	1.31	1.97
BF/Ω_b	$\phi_b BF$, kips	0.184	0.278	0.0760	0.114	0.0861	0.130	0.0930	0.139	0.0962	0.144
V_n/Ω_v	$\phi_v V_n$, kips	6.47	9.72	13.8	20.8	10.0	15.0	6.60	9.91	5.12	7.70
Z_x, in.^{3}		2.05		1.74		1.52		1.32		1.24	
L_p, ft		1.85		1.72		1.69		1.66		1.64	
L_r, ft		9.73		20.0		15.4		12.3		11.2	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.67$	$\phi_v = 0.90$

Notes: For beams laterally unsupported, see Table 3-11.
Available strength tabulated above heavy line is limited by available shear strength.

F_y = 36 ksi

Table 3-9
Maximum Total Uniform Load, kips
MC-Shapes

MC18–MC13

Shape	MC18×								MC13×			
	58		51.9		45.8		42.7		50		40	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 3									265	398	188	283
4	326	490	279	420	233	350			218	328	184	276
5	274	412	251	377	228	342	210	315	175	263	147	221
6	229	343	209	314	190	285	180	270	146	219	123	184
7	196	294	179	269	163	244	154	232	125	188	105	158
8	171	258	157	236	142	214	135	203	109	164	92.0	138
9	152	229	139	210	126	190	120	180	97.1	146	81.8	123
10	137	206	125	189	114	171	108	162	87.4	131	73.6	111
11	125	187	114	171	103	156	98.1	147	79.4	119	66.9	101
12	114	172	105	157	94.9	143	89.9	135	72.8	109	61.3	92.2
13	105	159	96.5	145	87.6	132	83.0	125	67.2	101	56.6	85.1
14	97.9	147	89.6	135	81.3	122	77.1	116	62.4	93.8	52.6	79.0
15	91.4	137	83.6	126	75.9	114	72.0	108	58.3	87.6	49.1	73.7
16	85.7	129	78.4	118	71.1	107	67.5	101	54.6	82.1	46.0	69.1
17	80.6	121	73.8	111	67.0	101	63.5	95.4	51.4	77.3	43.3	65.1
18	76.2	114	69.7	105	63.2	95.0	60.0	90.1	48.5	73.0	40.9	61.4
19	72.2	108	66.0	99.2	59.9	90.0	56.8	85.4	46.0	69.1	38.7	58.2
20	68.6	103	62.7	94.3	56.9	85.5	54.0	81.1	43.7	65.7	36.8	55.3
21	65.3	98.1	59.7	89.8	54.2	81.5	51.4	77.2	41.6	62.5	35.0	52.7
22	62.3	93.7	57.0	85.7	51.7	77.8	49.1	73.7	39.7	59.7	33.4	50.3
23	59.6	89.6	54.5	82.0	49.5	74.4	46.9	70.5	38.0	57.1	32.0	48.1
24	57.1	85.9	52.3	78.6	47.4	71.3	45.0	67.6	36.4	54.7	30.7	46.1
25	54.8	82.4	50.2	75.4	45.5	68.4	43.2	64.9	35.0	52.5	29.4	44.2
26	52.7	79.3	48.3	72.5	43.8	65.8	41.5	62.4	33.6	50.5	28.3	42.5
27	50.8	76.3	46.5	69.8	42.2	63.4	40.0	60.1	32.4	48.6	27.3	41.0
28	49.0	73.6	44.8	67.3	40.7	61.1	38.5	57.9	31.2	46.9	26.3	39.5
29	47.3	71.1	43.3	65.0	39.2	59.0	37.2	55.9	30.1	45.3	25.4	38.1
30	45.7	68.7	41.8	62.9	37.9	57.0	36.0	54.1	29.1	43.8	24.5	36.9
32	42.8	64.4	39.2	58.9	35.6	53.5	33.7	50.7	27.3	41.0	23.0	34.6
34	40.3	60.6	36.9	55.5	33.5	50.3	31.7	47.7				
36	38.1	57.2	34.9	52.4	31.6	47.5	30.0	45.1				
38	36.1	54.2	33.0	49.6	30.0	45.0	28.4	42.7				
40	34.3	51.5	31.4	47.1	28.5	42.8	27.0	40.6				
42	32.6	49.1	29.9	44.9	27.1	40.7	25.7	38.6				
44	31.2	46.8	28.5	42.9	25.9	38.9	24.5	36.9				
Beam Properties												
W_c/Ω_b | $\phi_b W_c$, kip-ft	1370	2060	1250	1890	1140	1710	1080	1620	874	1310	736	1110
M_p/Ω_b | $\phi_b M_p$, kip-ft	171	258	157	236	142	214	135	203	109	164	92.0	138
M_r/Ω_b | $\phi_b M_r$, kip-ft	94.3	142	87.5	132	80.7	121	77.3	116	60.7	91.3	52.7	79.2
BF/Ω_b | $\phi_b BF$, kips	5.16	7.81	5.26	7.87	5.23	7.93	5.17	7.80	2.08	3.13	2.28	3.42
V_n/Ω_v | $\phi_v V_n$, kips	163	245	140	210	116	175	105	157	132	199	94.2	142
Z_x, in.^{3}	95.4		87.3		79.2		75.1		60.8		51.2	
L_p, ft	4.25		4.29		4.37		4.45		4.41		4.50	
L_r, ft	19.1		17.5		16.1		15.6		27.6		21.7	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.67$	$\phi_v = 0.90$

Notes: For beams laterally unsupported, see Table 3-11.
Available strength tabulated above heavy line is limited by available shear strength.

Table 3-9 (continued)
Maximum Total Uniform Load, kips
MC-Shapes

MC13–MC12

F_y = 36 ksi

Shape		MC13×				MC12×							
		35		31.8		50		45		40		35	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	3					259	390	220	331	183	275		
	4	150	226	126	190	203	305	187	281	171	258	144	217
	5	134	201	125	187	162	244	149	225	137	206	124	187
	6	111	167	104	156	135	203	125	187	114	172	103	156
	7	95.5	143	89.1	134	116	174	107	160	97.9	147	88.7	133
	8	83.5	126	78.0	117	101	153	93.4	140	85.7	129	77.6	117
	9	74.3	112	69.3	104	90.2	136	83.0	125	76.2	114	69.0	104
	10	66.8	100	62.4	93.7	81.2	122	74.7	112	68.6	103	62.1	93.3
	11	60.8	91.3	56.7	85.2	73.8	111	67.9	102	62.3	93.7	56.4	84.8
	12	55.7	83.7	52.0	78.1	67.7	102	62.3	93.6	57.1	85.9	51.7	77.8
	13	51.4	77.3	48.0	72.1	62.5	93.9	57.5	86.4	52.7	79.3	47.8	71.8
	14	47.7	71.7	44.6	67.0	58.0	87.2	53.4	80.2	49.0	73.6	44.3	66.7
	15	44.6	67.0	41.6	62.5	54.1	81.4	49.8	74.9	45.7	68.7	41.4	62.2
	16	41.8	62.8	39.0	58.6	50.7	76.3	46.7	70.2	42.8	64.4	38.8	58.3
	17	39.3	59.1	36.7	55.1	47.8	71.8	44.0	66.1	40.3	60.6	36.5	54.9
	18	37.1	55.8	34.7	52.1	45.1	67.8	41.5	62.4	38.1	57.2	34.5	51.8
	19	35.2	52.9	32.8	49.3	42.7	64.2	39.3	59.1	36.1	54.2	32.7	49.1
	20	33.4	50.2	31.2	46.9	40.6	61.0	37.4	56.2	34.3	51.5	31.0	46.7
	21	31.8	47.8	29.7	44.6	38.7	58.1	35.6	53.5	32.6	49.1	29.6	44.4
	22	30.4	45.7	28.4	42.6	36.9	55.5	34.0	51.1	31.2	46.8	28.2	42.4
	23	29.1	43.7	27.1	40.8	35.3	53.1	32.5	48.8	29.8	44.8	27.0	40.6
	24	27.8	41.9	26.0	39.1	33.8	50.9	31.1	46.8	28.6	42.9	25.9	38.9
	25	26.7	40.2	24.9	37.5	32.5	48.8	29.9	44.9	27.4	41.2	24.8	37.3
	26	25.7	38.6	24.0	36.1	31.2	46.9	28.7	43.2	26.4	39.6	23.9	35.9
	27	24.8	37.2	23.1	34.7	30.1	45.2	27.7	41.6	25.4	38.2	23.0	34.6
	28	23.9	35.9	22.3	33.5	29.0	43.6	26.7	40.1	24.5	36.8	22.2	33.3
	29	23.0	34.6	21.5	32.3	28.0	42.1	25.8	38.7	23.6	35.5	21.4	32.2
	30	22.3	33.5	20.8	31.2	27.1	40.7	24.9	37.4	22.9	34.3	20.7	31.1
	32	20.9	31.4	19.5	29.3								

Beam Properties

ASD	LRFD	MC13×35 ASD	MC13×35 LRFD	MC13×31.8 ASD	MC13×31.8 LRFD	MC12×50 ASD	MC12×50 LRFD	MC12×45 ASD	MC12×45 LRFD	MC12×40 ASD	MC12×40 LRFD	MC12×35 ASD	MC12×35 LRFD
W_c/Ω_b	$\phi_b W_c$, kip-ft	668	1000	624	937	812	1220	747	1120	686	1030	621	933
M_p/Ω_b	$\phi_b M_p$, kip-ft	83.5	126	78.0	117	101	153	93.4	140	85.7	129	77.6	117
M_r/Ω_b	$\phi_b M_r$, kip-ft	48.8	73.3	46.1	69.4	56.5	84.9	52.7	79.2	49.0	73.7	45.3	68.0
BF/Ω_b	$\phi_b BF$, kips	2.34	3.55	2.31	3.44	1.65	2.53	1.77	2.65	1.87	2.82	1.92	2.92
V_n/Ω_v	$\phi_v V_n$, kips	75.2	113	63.1	94.8	130	195	110	166	91.6	138	72.2	108

Property	MC13×35	MC13×31.8	MC12×50	MC12×45	MC12×40	MC12×35
Z_x, in.^{3}	46.5	43.4	56.5	52.0	47.7	43.2
L_p, ft	4.54	4.58	4.54	4.54	4.58	4.62
L_r, ft	19.4	18.4	31.5	27.5	24.2	21.4

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.67$	$\phi_v = 0.90$

Notes: For beams laterally unsupported, see Table 3-11.
Available strength tabulated above heavy line is limited by available shear strength.

F_y = 36 ksi

Table 3-9 (continued)
Maximum Total Uniform Load, kips
MC-Shapes

MC12–MC10

Shape		MC12× 31 ASD	MC12× 31 LRFD	MC12× 14.3 ASD	MC12× 14.3 LRFD	MC12× 10.6 ASD	MC12× 10.6 LRFD	MC10× 41.1 ASD	MC10× 41.1 LRFD	MC10× 33.6 ASD	MC10× 33.6 LRFD	MC10× 28.5 ASD	MC10× 28.5 LRFD
Span, ft	2			77.6	117	59.0	88.6	206	309				
	3			76.2	114	55.6	83.5	188	283	149	224	110	165
	4	115	173	57.1	85.9	41.7	62.6	141	212	121	182	108	162
	5	114	172	45.7	68.7	33.3	50.1	113	170	96.9	146	86.2	130
	6	95.1	143	38.1	57.2	27.8	41.8	94.1	141	80.7	121	71.9	108
	7	81.5	123	32.6	49.1	23.8	35.8	80.7	121	69.2	104	61.6	92.6
	8	71.3	107	28.6	42.9	20.8	31.3	70.6	106	60.5	91.0	53.9	81.0
	9	63.4	95.3	25.4	38.2	18.5	27.8	62.8	94.3	53.8	80.9	47.9	72.0
	10	57.1	85.8	22.9	34.3	16.7	25.1	56.5	84.9	48.4	72.8	43.1	64.8
	11	51.9	78.0	20.8	31.2	15.2	22.8	51.3	77.2	44.0	66.2	39.2	58.9
	12	47.5	71.5	19.0	28.6	13.9	20.9	47.1	70.7	40.4	60.7	35.9	54.0
	13	43.9	66.0	17.6	26.4	12.8	19.3	43.4	65.3	37.3	56.0	33.2	49.8
	14	40.8	61.3	16.3	24.5	11.9	17.9	40.3	60.6	34.6	52.0	30.8	46.3
	15	38.0	57.2	15.2	22.9	11.1	16.7	37.7	56.6	32.3	48.5	28.7	43.2
	16	35.7	53.6	14.3	21.5	10.4	15.7	35.3	53.1	30.3	45.5	26.9	40.5
	17	33.6	50.4	13.4	20.2	9.81	14.7	33.2	49.9	28.5	42.8	25.4	38.1
	18	31.7	47.6	12.7	19.1	9.26	13.9	31.4	47.2	26.9	40.4	24.0	36.0
	19	30.0	45.1	12.0	18.1	8.77	13.2	29.7	44.7	25.5	38.3	22.7	34.1
	20	28.5	42.9	11.4	17.2	8.34	12.5	28.2	42.4	24.2	36.4	21.6	32.4
	21	27.2	40.8	10.9	16.4	7.94	11.9	26.9	40.4	23.1	34.7	20.5	30.9
	22	25.9	39.0	10.4	15.6	7.58	11.4	25.7	38.6	22.0	33.1	19.6	29.5
	23	24.8	37.3	9.93	14.9	7.25	10.9	24.6	36.9	21.1	31.6	18.7	28.2
	24	23.8	35.7	9.52	14.3	6.95	10.4	23.5	35.4	20.2	30.3	18.0	27.0
	25	22.8	34.3	9.14	13.7	6.67	10.0	22.6	34.0	19.4	29.1	17.2	25.9
	26	21.9	33.0	8.79	13.2	6.41	9.64						
	27	21.1	31.8	8.46	12.7	6.17	9.28						
	28	20.4	30.6	8.16	12.3	5.95	8.95						
	29	19.7	29.6	7.88	11.8	5.75	8.64						
	30	19.0	28.6	7.62	11.4	5.56	8.35						
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	571	858	229	343	167	251	565	849	484	728	431	648
M_p/Ω_b	$\phi_b M_p$, kip-ft	71.3	107	28.6	42.9	20.8	31.3	70.6	106	60.5	91.0	53.9	81.0
M_r/Ω_b	$\phi_b M_r$, kip-ft	42.4	63.7	16.0	24.0	11.6	17.4	39.6	59.5	35.0	52.5	31.8	47.8
BF/Ω_b	$\phi_b BF$, kips	1.90	2.85	2.49	3.73	2.72	4.11	1.00	1.50	1.13	1.71	1.22	1.83
V_n/Ω_v	$\phi_v V_n$, kips	57.4	86.3	38.8	58.3	29.5	44.3	103	155	74.4	112	55.0	82.6
Z_x, in.^{3}		39.7		15.9		11.6		39.3		33.7		30.0	
L_p, ft		4.62		2.04		1.45		4.75		4.79		4.83	
L_r, ft		19.8		7.11		4.83		35.7		27.3		23.0	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.67$	$\phi_v = 0.90$

Notes: For beams laterally unsupported, see Table 3-11.
Available strength tabulated above heavy line is limited by available shear strength.

Table 3-9 (continued)
Maximum Total Uniform Load, kips
MC-Shapes

MC10–MC9

F_y = 36 ksi

Shape	MC10× 25		MC10× 22		MC10× 8.4		MC10× 6.5		MC9× 25.4		MC9× 23.9	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 2					44.0	66.1	39.3	59.1				
3	98.3	148			37.9	57.0	28.3	42.5	105	157	93.1	140
4	94.1	141	75.0	113	28.5	42.8	21.2	31.9	84.4	127	80.8	121
5	75.3	113	68.7	103	22.8	34.2	17.0	25.5	67.5	102	64.7	97.2
6	62.8	94.3	57.2	86.0	19.0	28.5	14.1	21.2	56.3	84.6	53.9	81.0
7	53.8	80.8	49.1	73.7	16.3	24.4	12.1	18.2	48.2	72.5	46.2	69.4
8	47.1	70.7	42.9	64.5	14.2	21.4	10.6	15.9	42.2	63.5	40.4	60.8
9	41.8	62.9	38.2	57.4	12.6	19.0	9.42	14.2	37.5	56.4	35.9	54.0
10	37.7	56.6	34.3	51.6	11.4	17.1	8.48	12.7	33.8	50.8	32.3	48.6
11	34.2	51.4	31.2	46.9	10.3	15.6	7.71	11.6	30.7	46.1	29.4	44.2
12	31.4	47.2	28.6	43.0	9.49	14.3	7.07	10.6	28.1	42.3	26.9	40.5
13	29.0	43.5	26.4	39.7	8.76	13.2	6.52	9.80	26.0	39.0	24.9	37.4
14	26.9	40.4	24.5	36.9	8.13	12.2	6.06	9.10	24.1	36.3	23.1	34.7
15	25.1	37.7	22.9	34.4	7.59	11.4	5.65	8.50	22.5	33.8	21.6	32.4
16	23.5	35.4	21.5	32.3	7.11	10.7	5.30	7.97	21.1	31.7	20.2	30.4
17	22.1	33.3	20.2	30.4	6.70	10.1	4.99	7.50	19.9	29.9	19.0	28.6
18	20.9	31.4	19.1	28.7	6.32	9.50	4.71	7.08	18.8	28.2	18.0	27.0
19	19.8	29.8	18.1	27.2	5.99	9.00	4.46	6.71	17.8	26.7	17.0	25.6
20	18.8	28.3	17.2	25.8	5.69	8.55	4.24	6.37	16.9	25.4	16.2	24.3
21	17.9	26.9	16.4	24.6	5.42	8.15	4.04	6.07	16.1	24.2	15.4	23.1
22	17.1	25.7	15.6	23.5	5.17	7.78	3.85	5.79	15.4	23.1	14.7	22.1
23	16.4	24.6	14.9	22.4	4.95	7.44	3.69	5.54				
24	15.7	23.6	14.3	21.5	4.74	7.13	3.53	5.31				
25	15.1	22.6	13.7	20.6	4.55	6.84	3.39	5.10				
Beam Properties												
W_c/Ω_b / $\phi_b W_c$, kip-ft	377	566	344	516	114	171	84.8	127	338	508	323	486
M_p/Ω_b / $\phi_b M_p$, kip-ft	47.1	70.7	42.9	64.5	14.2	21.4	10.6	15.9	42.2	63.5	40.4	60.8
M_r/Ω_b / $\phi_b M_r$, kip-ft	27.7	41.6	25.8	38.7	8.04	12.1	5.77	8.68	24.5	36.9	23.8	35.7
BF/Ω_b / $\phi_b BF$, kips	1.29	1.93	1.28	1.93	1.75	2.65	1.95	2.91	0.967	1.45	0.982	1.49
V_n/Ω_v / $\phi_v V_n$, kips	49.1	73.9	37.5	56.4	22.0	33.0	19.7	29.5	52.4	78.7	46.6	70.0
Z_x, in.^{3}	26.2		23.9		7.92		5.90		23.5		22.5	
L_p, ft	4.13		4.15		1.52		1.09		4.20		4.20	
L_r, ft	19.2		17.5		5.03		3.57		22.5		21.1	

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90
Ω_v = 1.67	ϕ_v = 0.90

Notes: For beams laterally unsupported, see Table 3-11.
Available strength tabulated above heavy line is limited by available shear strength.

F_y = 36 ksi

Table 3-9 (continued)
Maximum Total Uniform Load, kips
MC-Shapes

MC8–MC7

Shape	MC8×										MC7×	
	22.8		21.4		20		18.7		8.5		22.7	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 2					82.8	124			37.0	55.7	91.1	137
3	88.4	133	77.6	117	78.6	118	73.1	110	33.3	50.0	78.6	118
4	68.6	103	65.4	98.3	58.9	88.6	56.0	84.2	25.0	37.5	58.9	88.6
5	54.9	82.5	52.3	78.6	47.1	70.8	44.8	67.4	20.0	30.0	47.1	70.8
6	45.7	68.8	43.6	65.5	39.3	59.0	37.4	56.2	16.6	25.0	39.3	59.0
7	39.2	58.9	37.4	56.2	33.7	50.6	32.0	48.1	14.3	21.4	33.7	50.6
8	34.3	51.6	32.7	49.1	29.5	44.3	28.0	42.1	12.5	18.8	29.5	44.3
9	30.5	45.8	29.1	43.7	26.2	39.4	24.9	37.4	11.1	16.7	26.2	39.4
10	27.4	41.3	26.2	39.3	23.6	35.4	22.4	33.7	9.99	15.0	23.6	35.4
11	25.0	37.5	23.8	35.7	21.4	32.2	20.4	30.6	9.08	13.6	21.4	32.2
12	22.9	34.4	21.8	32.8	19.6	29.5	18.7	28.1	8.32	12.5	19.6	29.5
13	21.1	31.7	20.1	30.2	18.1	27.2	17.2	25.9	7.68	11.5	18.1	27.2
14	19.6	29.5	18.7	28.1	16.8	25.3	16.0	24.1	7.13	10.7	16.8	25.3
15	18.3	27.5	17.4	26.2	15.7	23.6	14.9	22.5	6.66	10.0	15.7	23.6
16	17.2	25.8	16.3	24.6	14.7	22.1	14.0	21.1	6.24	9.38	14.7	22.1
17	16.1	24.3	15.4	23.1	13.9	20.8	13.2	19.8	5.88	8.83	13.9	20.8
18	15.2	22.9	14.5	21.8	13.1	19.7	12.5	18.7	5.55	8.34		
19	14.4	21.7	13.8	20.7	12.4	18.6	11.8	17.7	5.26	7.90		
20	13.7	20.6	13.1	19.7	11.8	17.7	11.2	16.8	4.99	7.51		

Beam Properties

ASD	LRFD	22.8 ASD	22.8 LRFD	21.4 ASD	21.4 LRFD	20 ASD	20 LRFD	18.7 ASD	18.7 LRFD	8.5 ASD	8.5 LRFD	22.7 ASD	22.7 LRFD
W_c/Ω_b	$\phi_b W_c$, kip-ft	274	413	262	393	236	354	224	337	99.9	150	236	354
M_p/Ω_b	$\phi_b M_p$, kip-ft	34.3	51.6	32.7	49.1	29.5	44.3	28.0	42.1	12.5	18.8	29.5	44.3
M_r/Ω_b	$\phi_b M_r$, kip-ft	20.0	30.1	19.4	29.1	17.1	25.7	16.5	24.8	7.32	11.0	17.0	25.5
BF/Ω_b	$\phi_b BF$, kips	0.724	1.09	0.733	1.10	0.775	1.16	0.778	1.17	0.970	1.46	0.493	0.741
V_n/Ω_v	$\phi_v V_n$, kips	44.2	66.4	38.8	58.3	41.4	62.2	36.5	54.9	18.5	27.8	45.5	68.4

	22.8	21.4	20	18.7	8.5	22.7
Z_x, in.^{3}	19.1	18.2	16.4	15.6	6.95	16.4
L_p, ft	4.25	4.25	3.61	3.61	2.08	4.33
L_r, ft	24.0	22.4	19.6	18.4	7.42	29.7

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.67$	$\phi_v = 0.90$

Notes: For beams laterally unsupported, see Table 3-11.
Available strength tabulated above heavy line is limited by available shear strength.

Table 3-9 (continued)
Maximum Total Uniform Load, kips
MC-Shapes

MC7–MC6

F_y = 36 ksi

Shape		MC7×		MC6×							
		19.1		18		15.3		16.3		15.1	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	2			58.8	88.4	52.8	79.3	58.2	87.5	49.0	73.7
	3	63.7	95.8	56.0	84.2	47.5	71.4	49.8	74.9	47.1	70.8
	4	52.1	78.3	42.0	63.2	35.6	53.5	37.4	56.2	35.3	53.1
	5	41.7	62.6	33.6	50.5	28.5	42.8	29.9	44.9	28.3	42.5
	6	34.7	52.2	28.0	42.1	23.7	35.7	24.9	37.4	23.5	35.4
	7	29.8	44.7	24.0	36.1	20.3	30.6	21.4	32.1	20.2	30.3
	8	26.0	39.2	21.0	31.6	17.8	26.8	18.7	28.1	17.7	26.5
	9	23.2	34.8	18.7	28.1	15.8	23.8	16.6	25.0	15.7	23.6
	10	20.8	31.3	16.8	25.3	14.2	21.4	14.9	22.5	14.1	21.2
	11	18.9	28.5	15.3	23.0	12.9	19.5	13.6	20.4	12.8	19.3
	12	17.4	26.1	14.0	21.1	11.9	17.8	12.5	18.7	11.8	17.7
	13	16.0	24.1	12.9	19.4	11.0	16.5	11.5	17.3	10.9	16.3
	14	14.9	22.4	12.0	18.1	10.2	15.3	10.7	16.0	10.1	15.2
	15	13.9	20.9	11.2	16.8	9.49	14.3	9.96	15.0	9.42	14.2
	16	13.0	19.6								
	17	12.3	18.4								
Beam Properties											
W_c/Ω_b	$\phi_b W_c$, kip-ft	208	313	168	253	142	214	149	225	141	212
M_p/Ω_b	$\phi_b M_p$, kip-ft	26.0	39.2	21.0	31.6	17.8	26.8	18.7	28.1	17.7	26.5
M_r/Ω_b	$\phi_b M_r$, kip-ft	15.5	23.2	12.4	18.7	10.6	16.0	10.9	16.4	10.4	15.7
BF/Ω_b	$\phi_b BF$, kips	0.523	0.797	0.356	0.535	0.372	0.559	0.373	0.560	0.384	0.568
V_n/Ω_v	$\phi_v V_n$, kips	31.9	47.9	29.4	44.2	26.4	39.7	29.1	43.7	24.5	36.9
Z_x, in.^{3}		14.5		11.7		9.91		10.4		9.83	
L_p, ft		4.33		4.37		4.37		3.69		3.68	
L_r, ft		24.4		28.5		23.7		24.6		22.7	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.67$	$\phi_v = 0.90$

Notes: For beams laterally unsupported, see Table 3-11.
Available strength tabulated above heavy line is limited by available shear strength.

F_y = 36 ksi

Table 3-9 (continued)
Maximum Total Uniform Load, kips
MC-Shapes

MC6–MC3

Shape		MC6×						MC4×		MC3×	
		12		7		6.5		13.8		7.1	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	2	48.1	72.3	27.8	41.8	24.1	36.2	39.7	59.7	16.1	24.2
	3	35.8	53.8	21.6	32.4	20.5	30.8	26.5	39.8	10.7	16.1
	4	26.8	40.3	16.2	24.3	15.4	23.1	19.9	29.9	8.05	12.1
	5	21.5	32.3	12.9	19.4	12.3	18.5	15.9	23.9	6.44	9.68
	6	17.9	26.9	10.8	16.2	10.3	15.4	13.2	19.9	5.37	8.06
	7	15.3	23.1	9.24	13.9	8.79	13.2	11.4	17.1	4.60	6.91
	8	13.4	20.2	8.08	12.2	7.69	11.6	9.93	14.9		
	9	11.9	17.9	7.19	10.8	6.83	10.3	8.83	13.3		
	10	10.7	16.1	6.47	9.72	6.15	9.24	7.95	11.9		
	11	9.76	14.7	5.88	8.84	5.59	8.40				
	12	8.95	13.4	5.39	8.10	5.13	7.70				
	13	8.26	12.4	4.97	7.48	4.73	7.11				
	14	7.67	11.5	4.62	6.94	4.39	6.60				
	15	7.16	10.8	4.31	6.48	4.10	6.16				

Beam Properties

ASD	LRFD	MC6×12 ASD	MC6×12 LRFD	MC6×7 ASD	MC6×7 LRFD	MC6×6.5 ASD	MC6×6.5 LRFD	MC4×13.8 ASD	MC4×13.8 LRFD	MC3×7.1 ASD	MC3×7.1 LRFD
W_c/Ω_b	$\phi_b W_c$, kip-ft	107	161	64.7	97.2	61.5	92.4	79.5	119	32.2	48.4
M_p/Ω_b	$\phi_b M_p$, kip-ft	13.4	20.2	8.08	12.2	7.69	11.6	9.93	14.9	4.02	6.05
M_r/Ω_b	$\phi_b M_r$, kip-ft	7.85	11.8	4.79	7.20	4.60	6.92	5.57	8.37	2.28	3.42
BF/Ω_b	$\phi_b BF$, kips	0.414	0.627	0.490	0.744	0.485	0.735	0.126	0.189	0.0745	0.113
V_n/Ω_v	$\phi_v V_n$, kips	24.1	36.2	13.9	20.9	12.0	18.1	25.9	38.9	12.1	18.2

	MC6×12	MC6×7	MC6×6.5	MC4×13.8	MC3×7.1
Z_x, in.^{3}	7.47	4.50	4.28	5.53	2.24
L_p, ft	3.01	2.24	2.24	3.03	2.34
L_r, ft	16.4	8.96	8.61	37.6	25.7

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.67$	$\phi_v = 0.90$

Notes: For beams laterally unsupported, see Table 3-11.
Available strength tabulated above heavy line is limited by available shear strength.

Table 3-10
W-Shapes
Available Moment vs. Unbraced Length

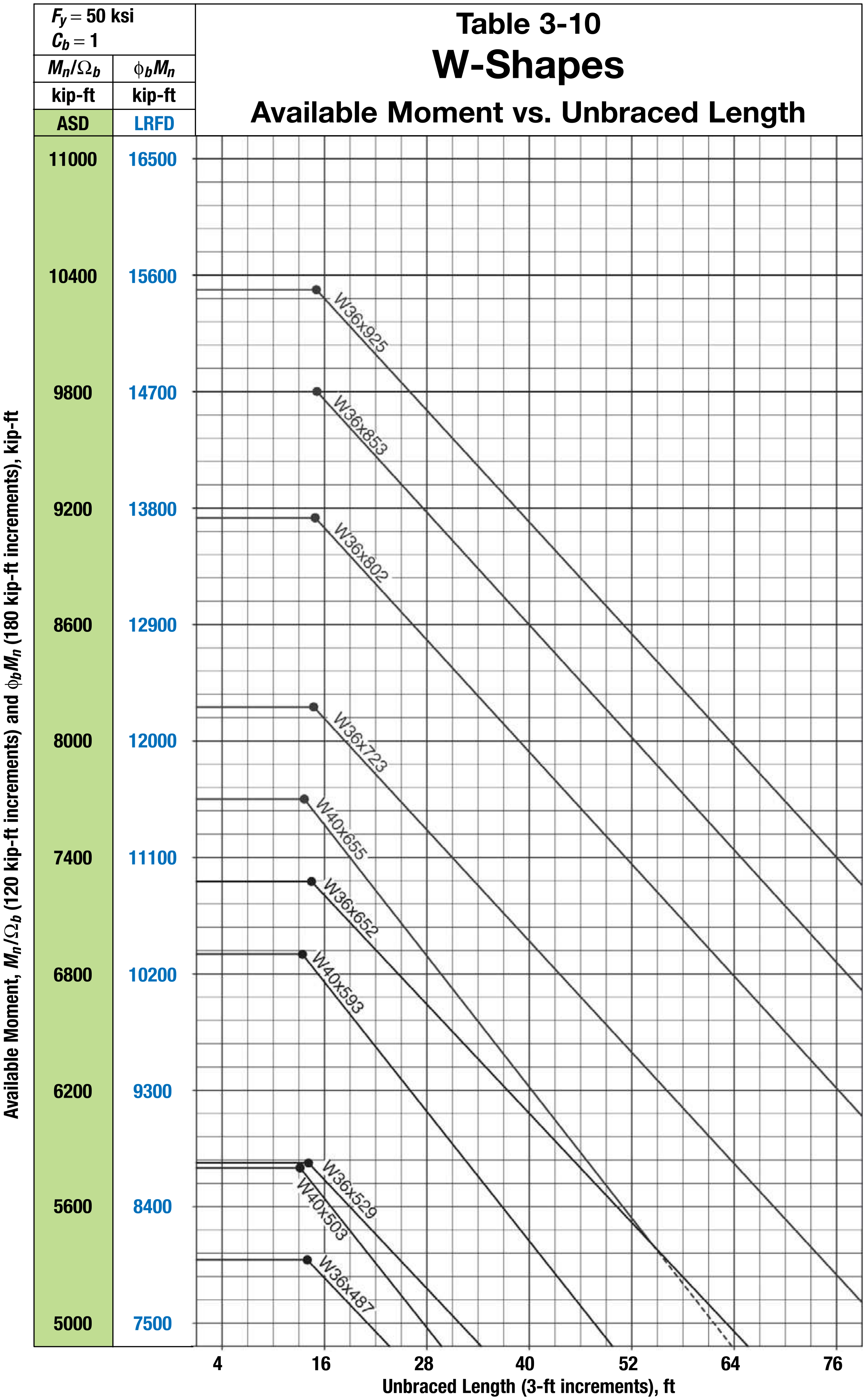
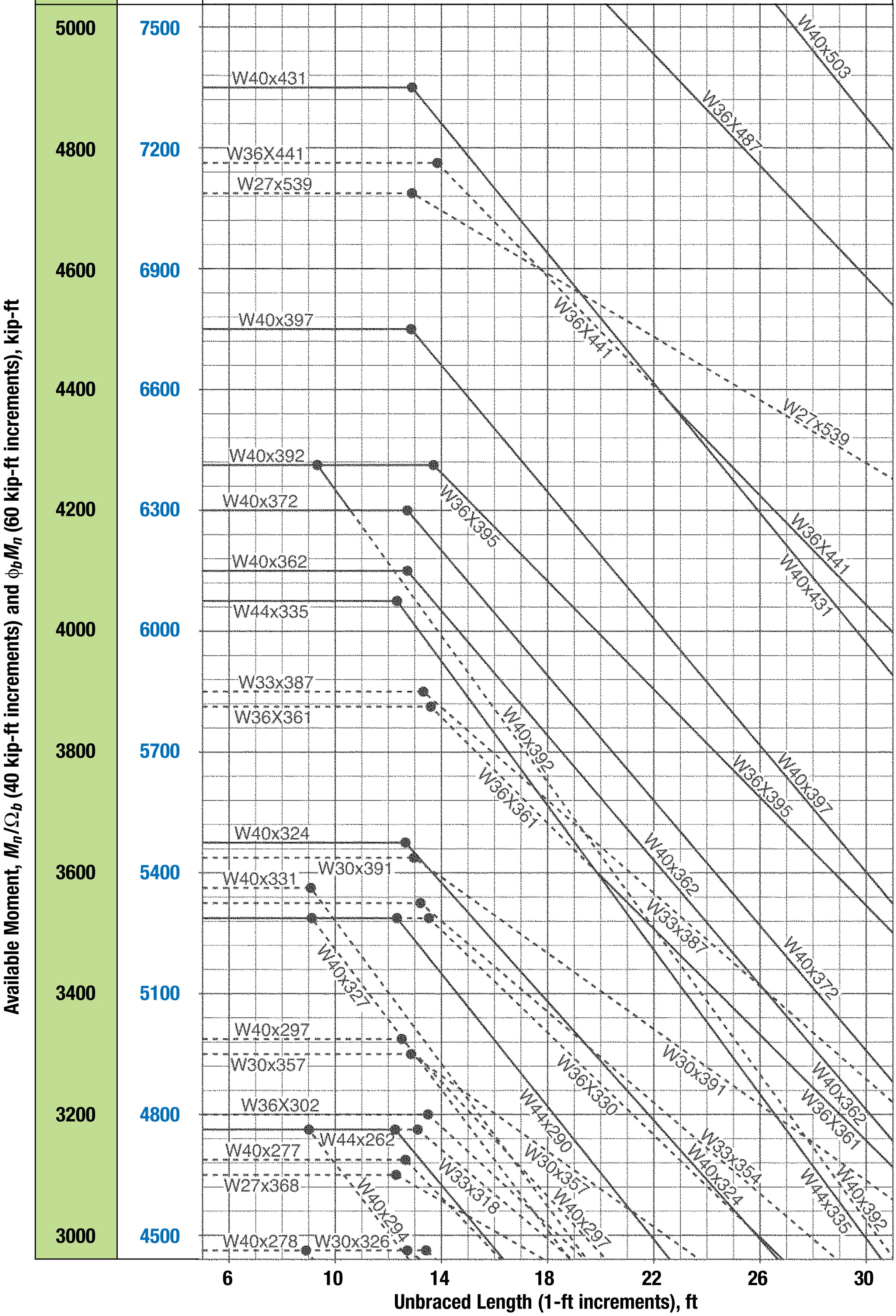
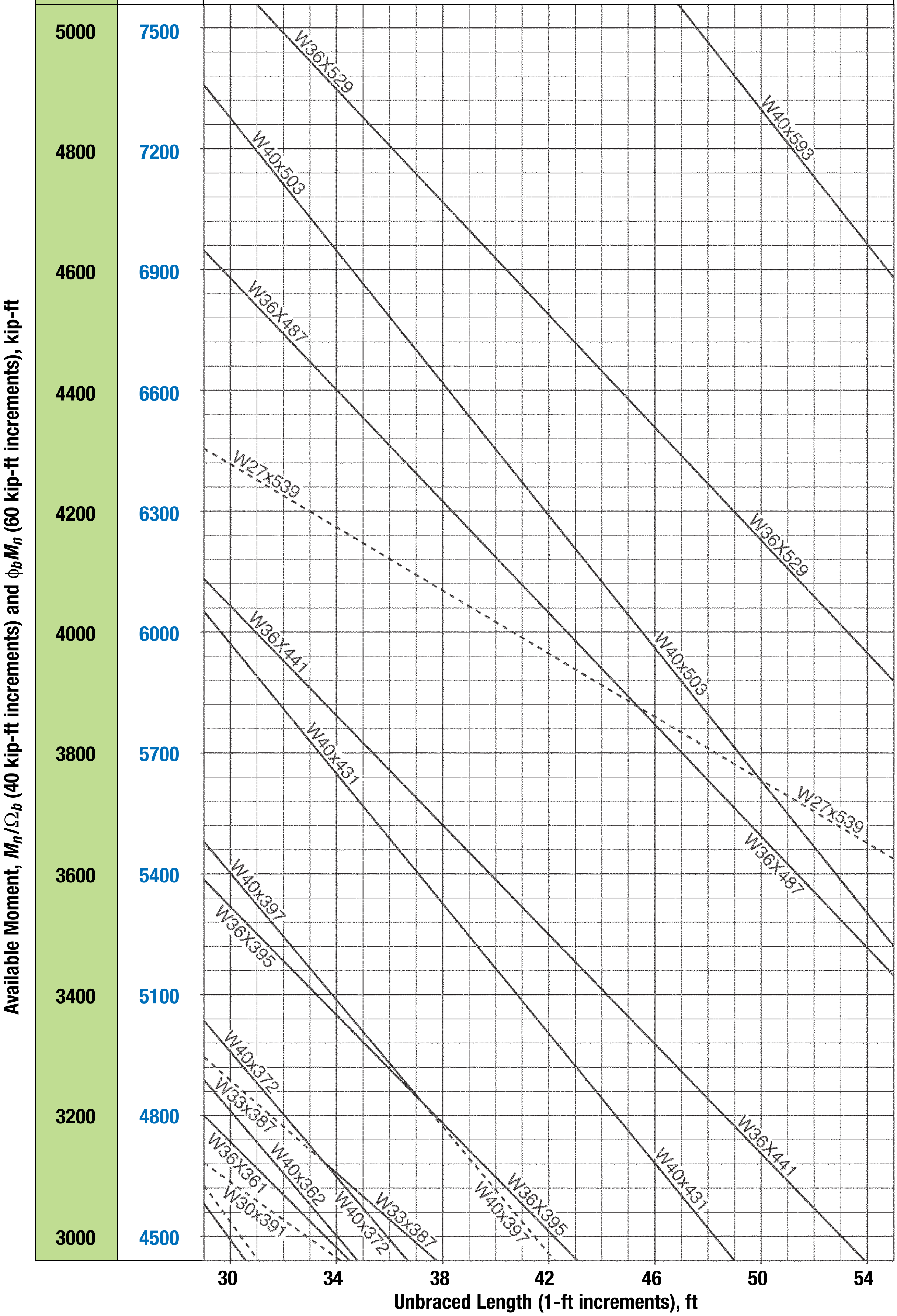


Table 3-10 (continued)
W-Shapes
Available Moment vs. Unbraced Length



F_y = 50 ksi C_b = 1		**Table 3-10 (continued)** **W-Shapes** **Available Moment vs. Unbraced Length**
M_n/Ω_b	$\phi_b M_n$	
kip-ft	kip-ft	
ASD	LRFD	



F_y = 50 ksi
C_b = 1

Table 3-10 (continued)
W-Shapes
Available Moment vs. Unbraced Length

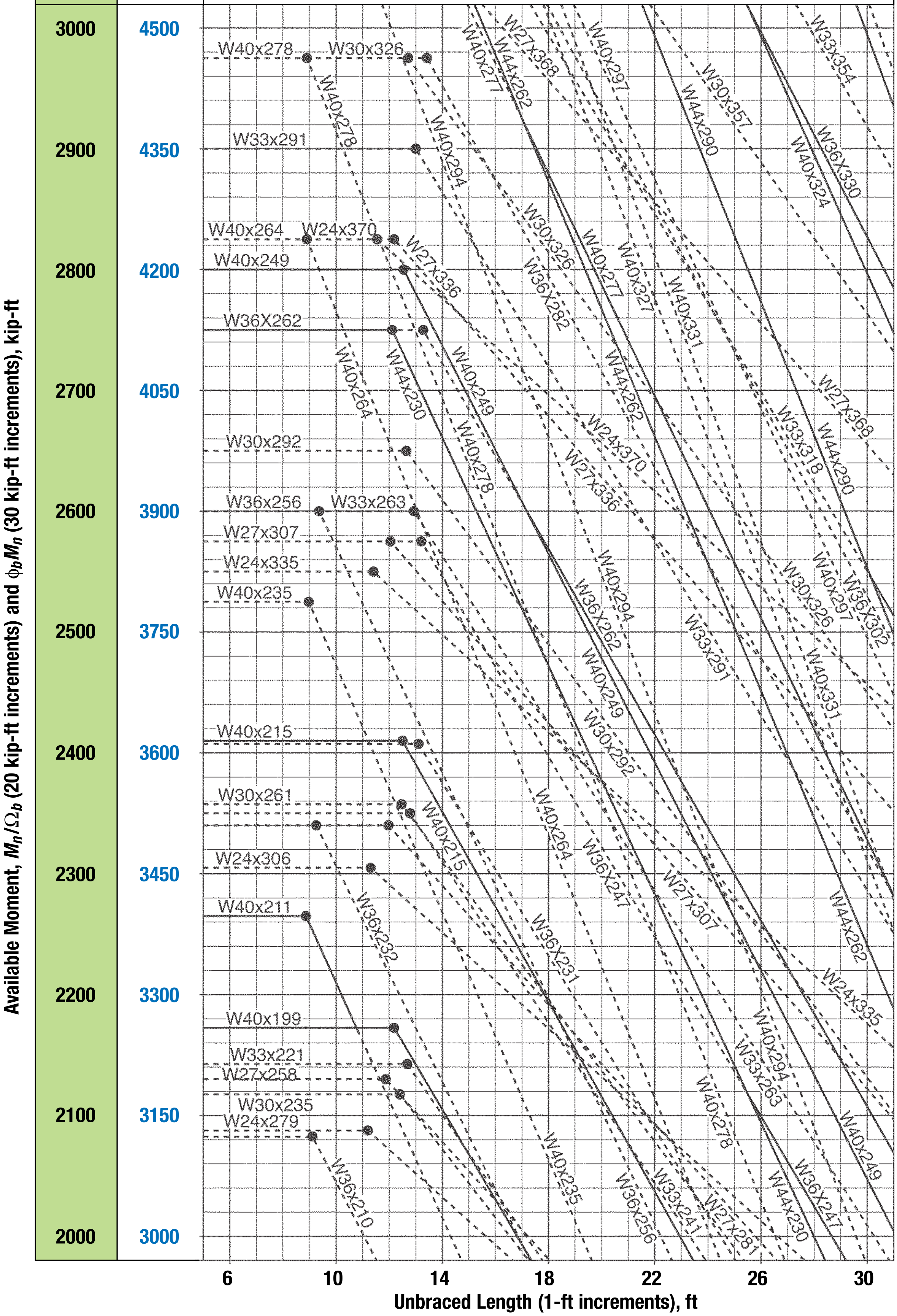
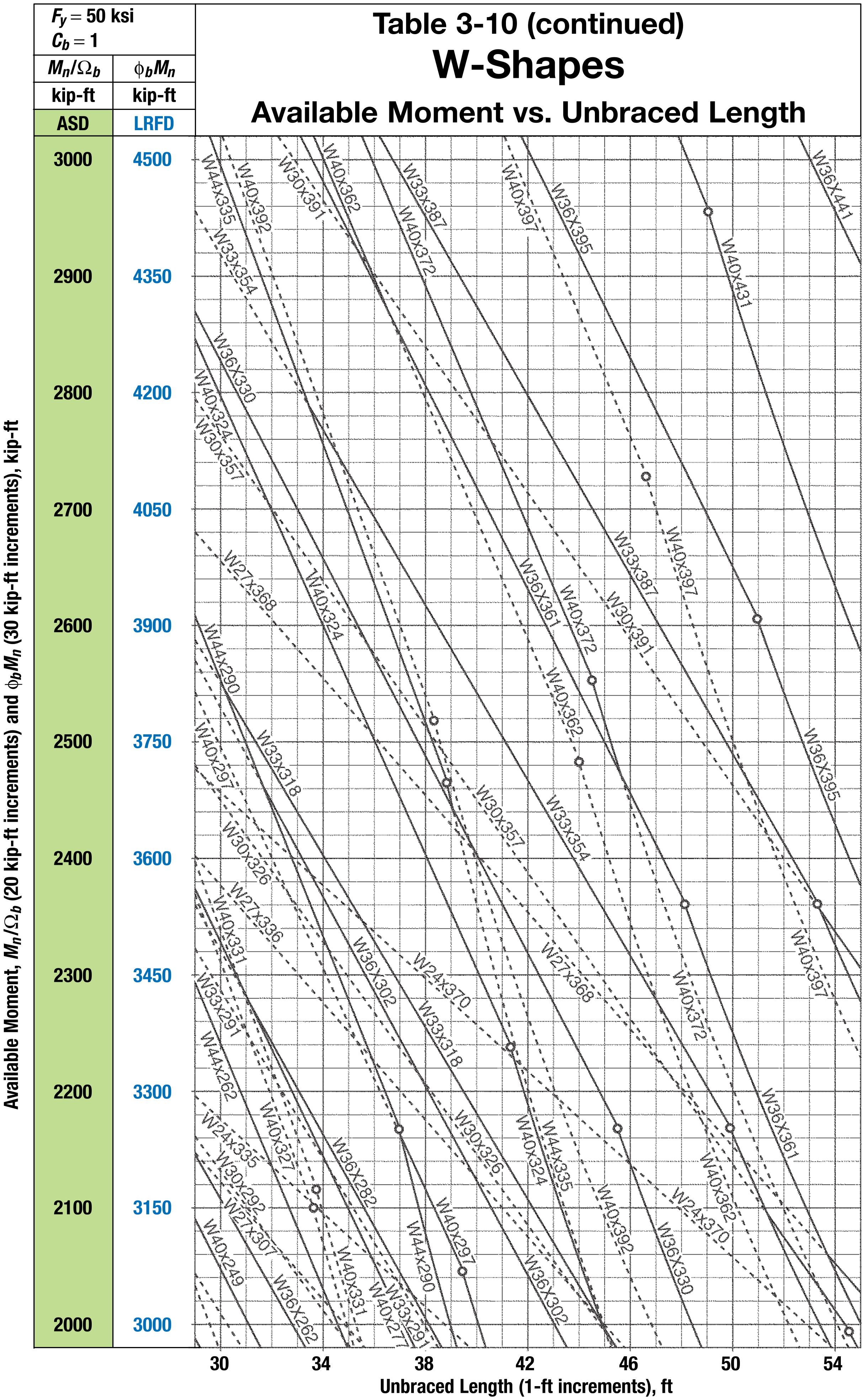


Table 3-10 (continued)
W-Shapes
Available Moment vs. Unbraced Length



F_y = 50 ksi
C_b = 1

M_n/Ω_b	$\phi_b M_n$
kip-ft	kip-ft
ASD	LRFD

Table 3-10 (continued)
W-Shapes
Available Moment vs. Unbraced Length

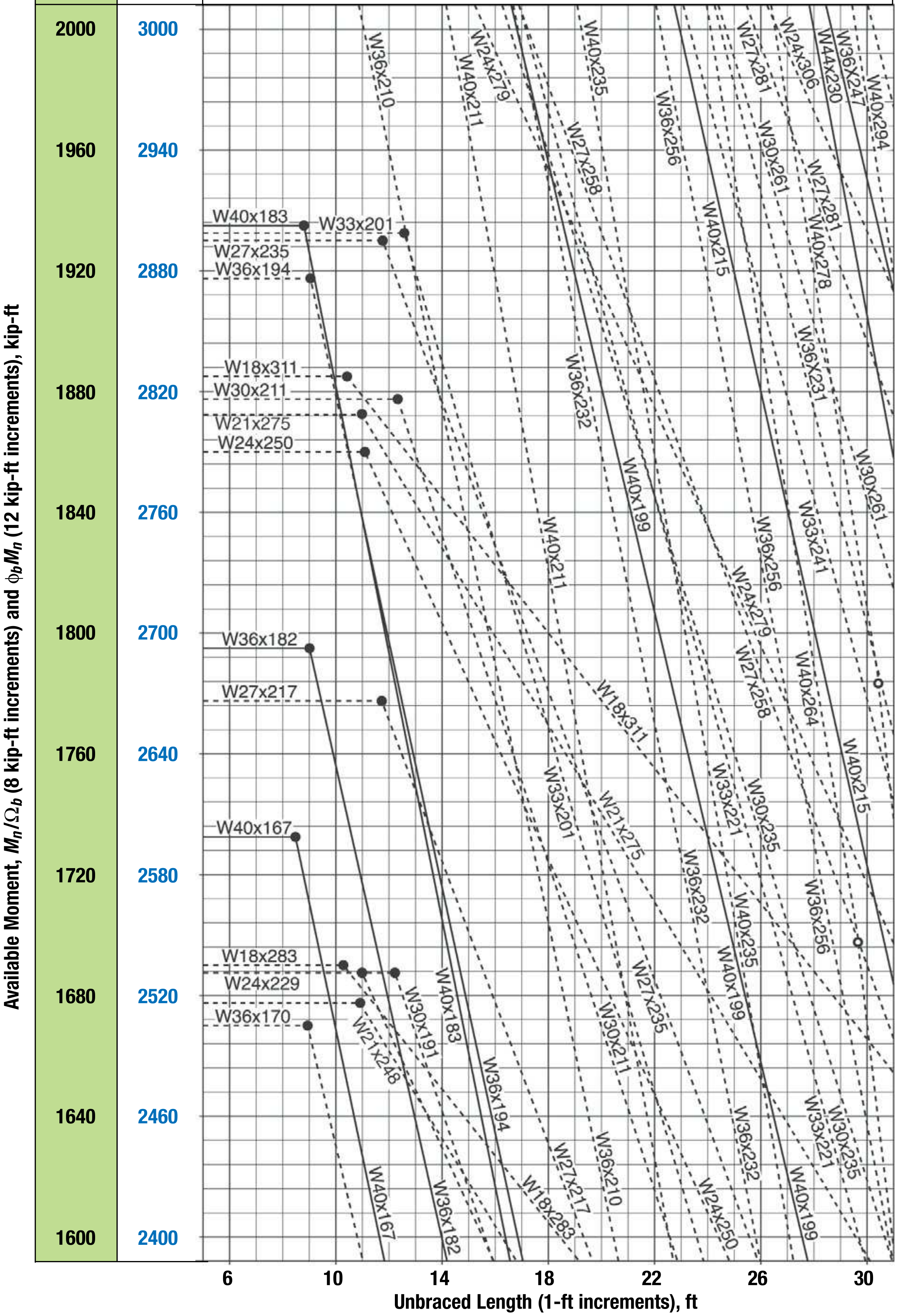
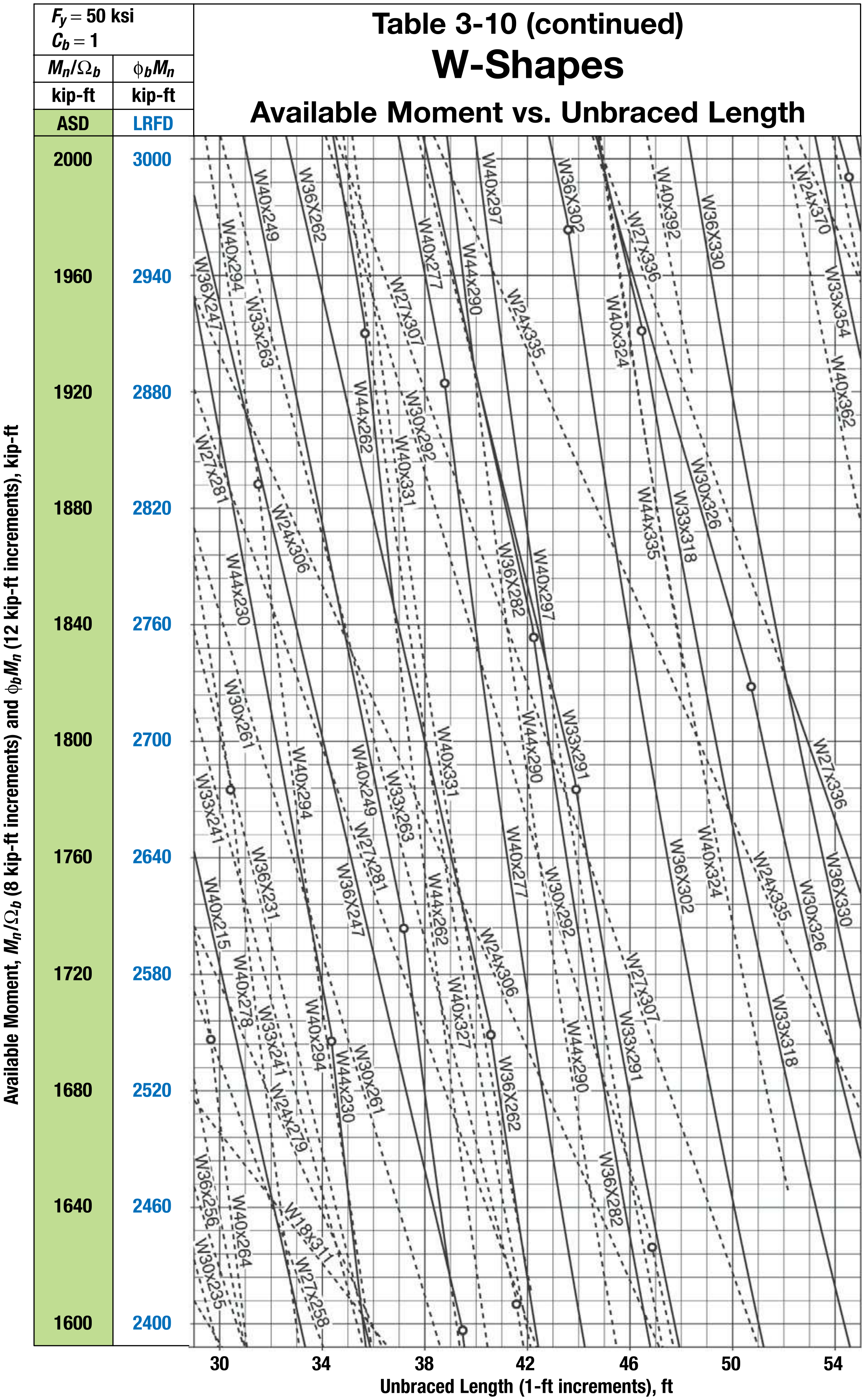


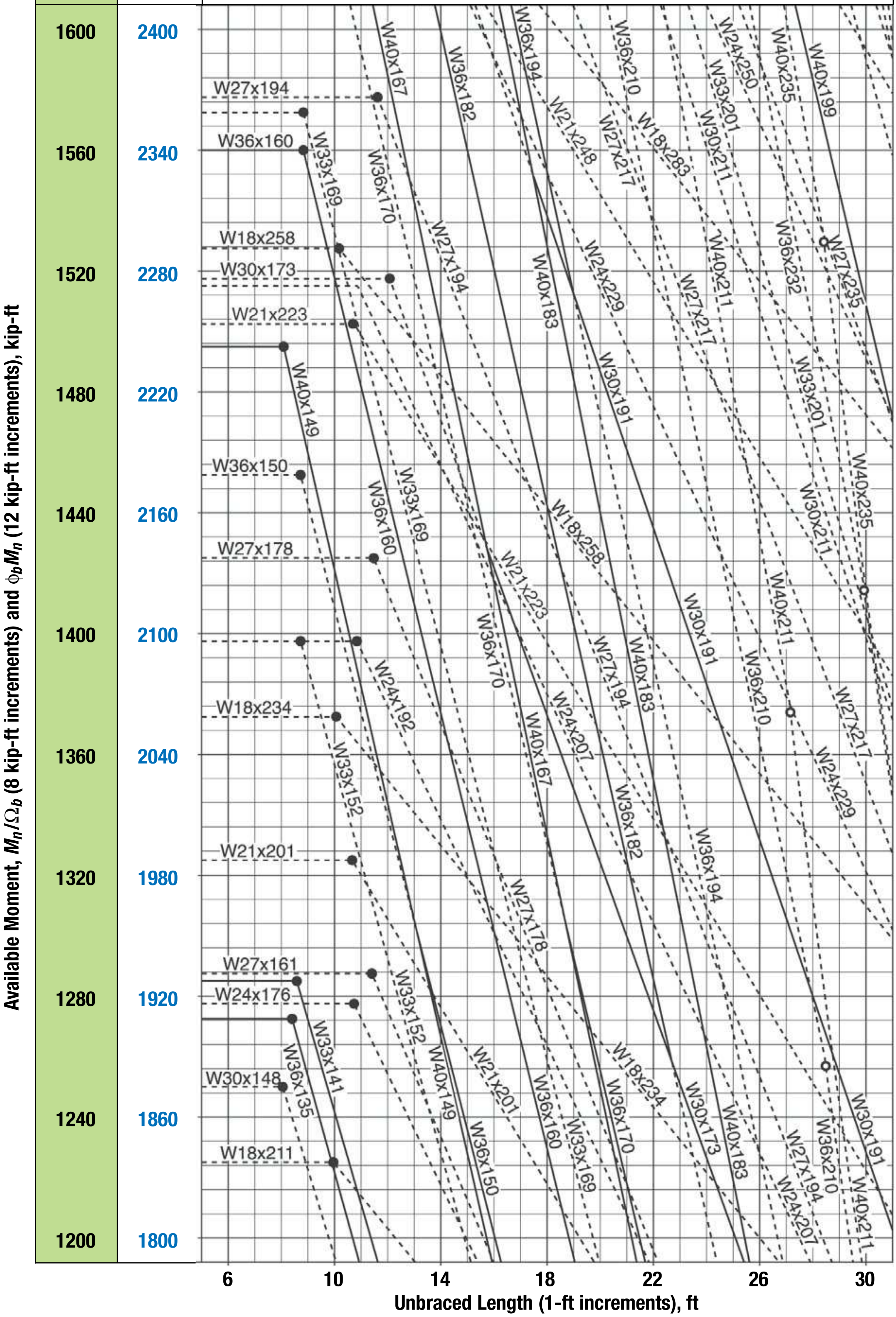
Table 3-10 (continued)
W-Shapes
Available Moment vs. Unbraced Length



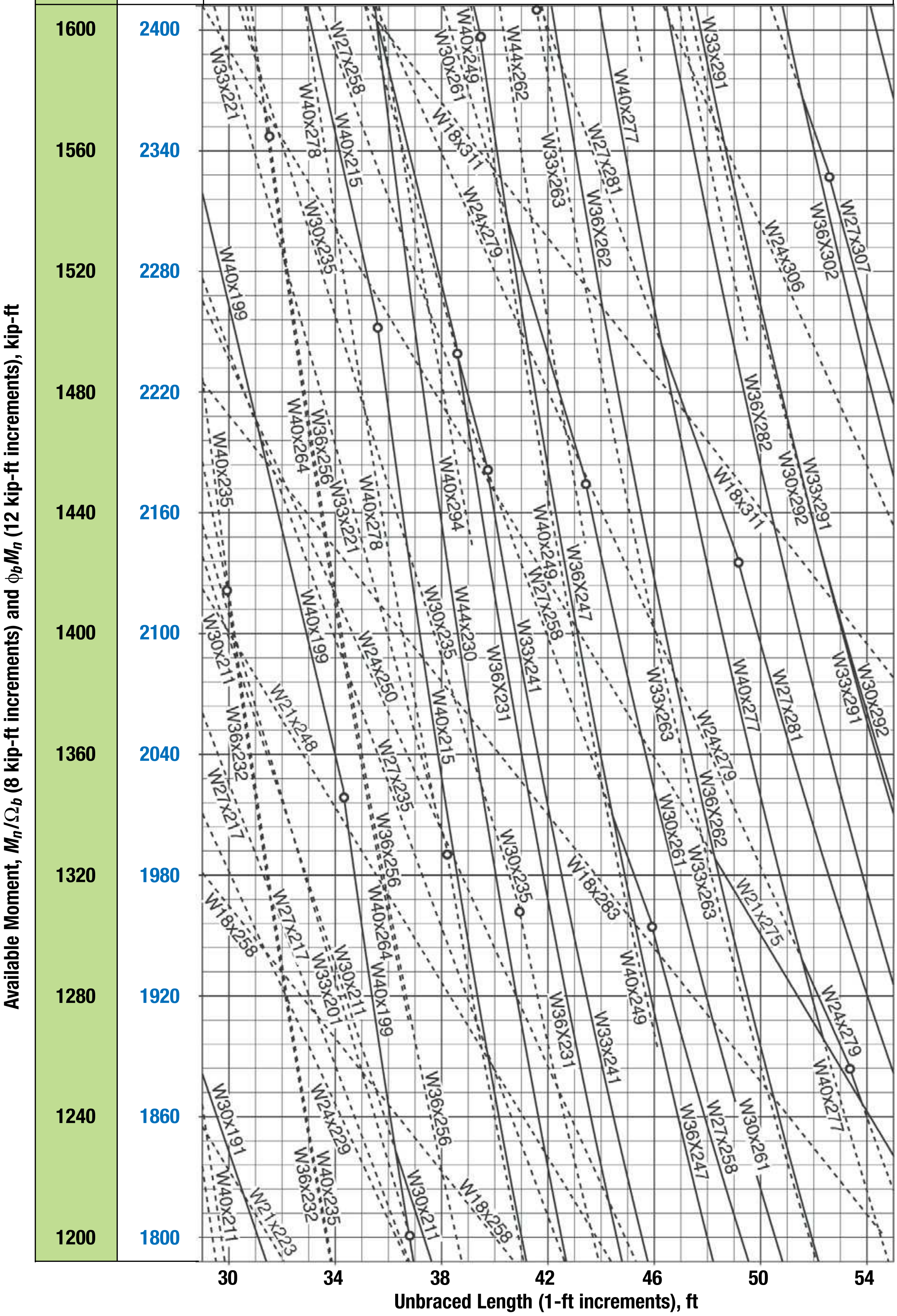
$F_y = 50$ ksi
$C_b = 1$

M_n/Ω_b	$\phi_b M_n$
kip-ft	kip-ft
ASD	LRFD

Table 3-10 (continued)
W-Shapes
Available Moment vs. Unbraced Length

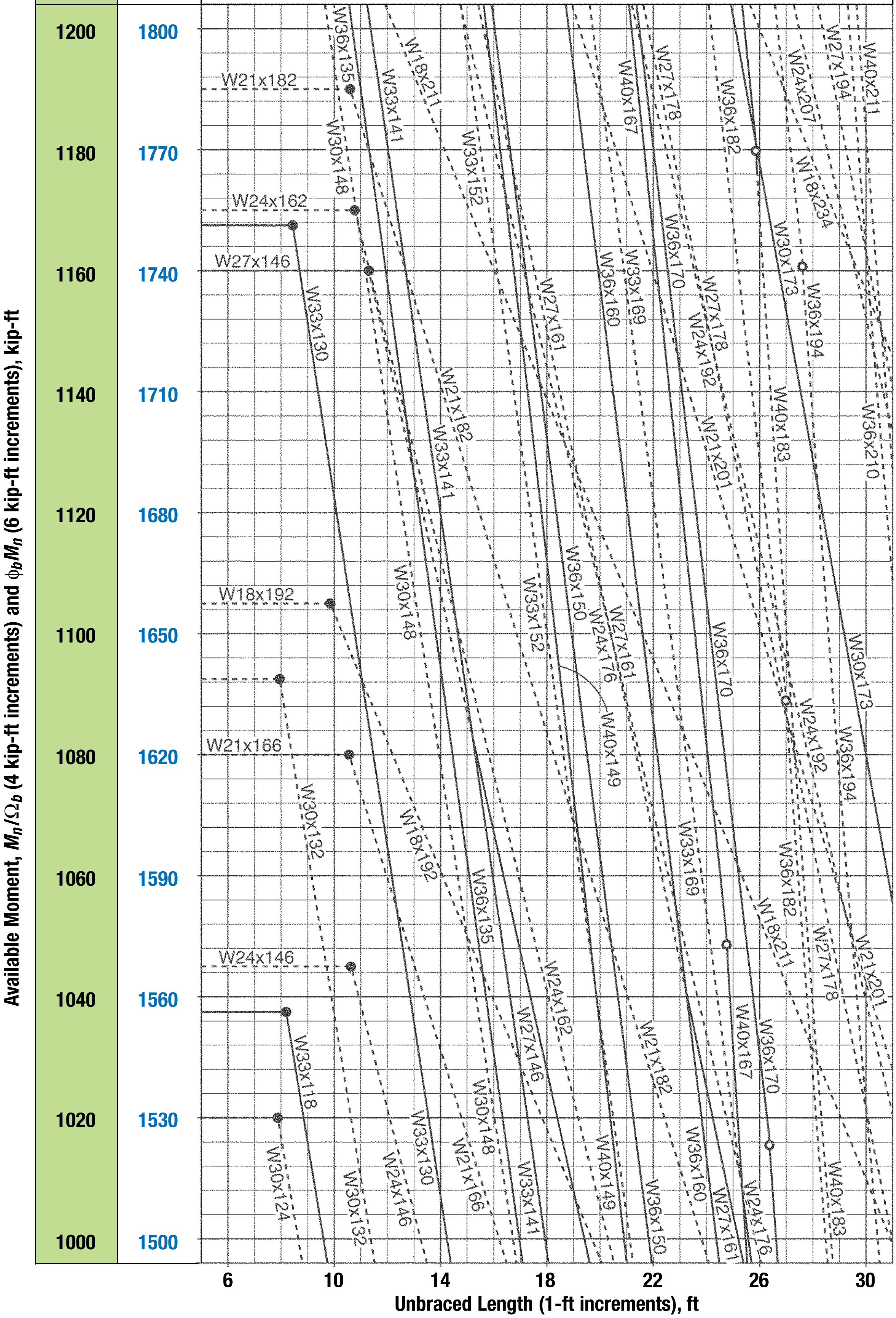


$F_y = 50$ ksi $C_b = 1$		**Table 3-10 (continued)** **W-Shapes** **Available Moment vs. Unbraced Length**
M_n/Ω_b	$\phi_b M_n$	
kip-ft	kip-ft	
ASD	LRFD	



F_y = 50 ksi, C_b = 1	
M_n/Ω_b	$\phi_b M_n$
kip-ft	kip-ft
ASD	LRFD

Table 3-10 (continued)
W-Shapes
Available Moment vs. Unbraced Length



$F_y = 50$ ksi $C_b = 1$		Table 3-10 (continued)
M_n/Ω_b	$\phi_b M_n$	**W-Shapes**
kip-ft	kip-ft	**Available Moment vs. Unbraced Length**
ASD	LRFD	

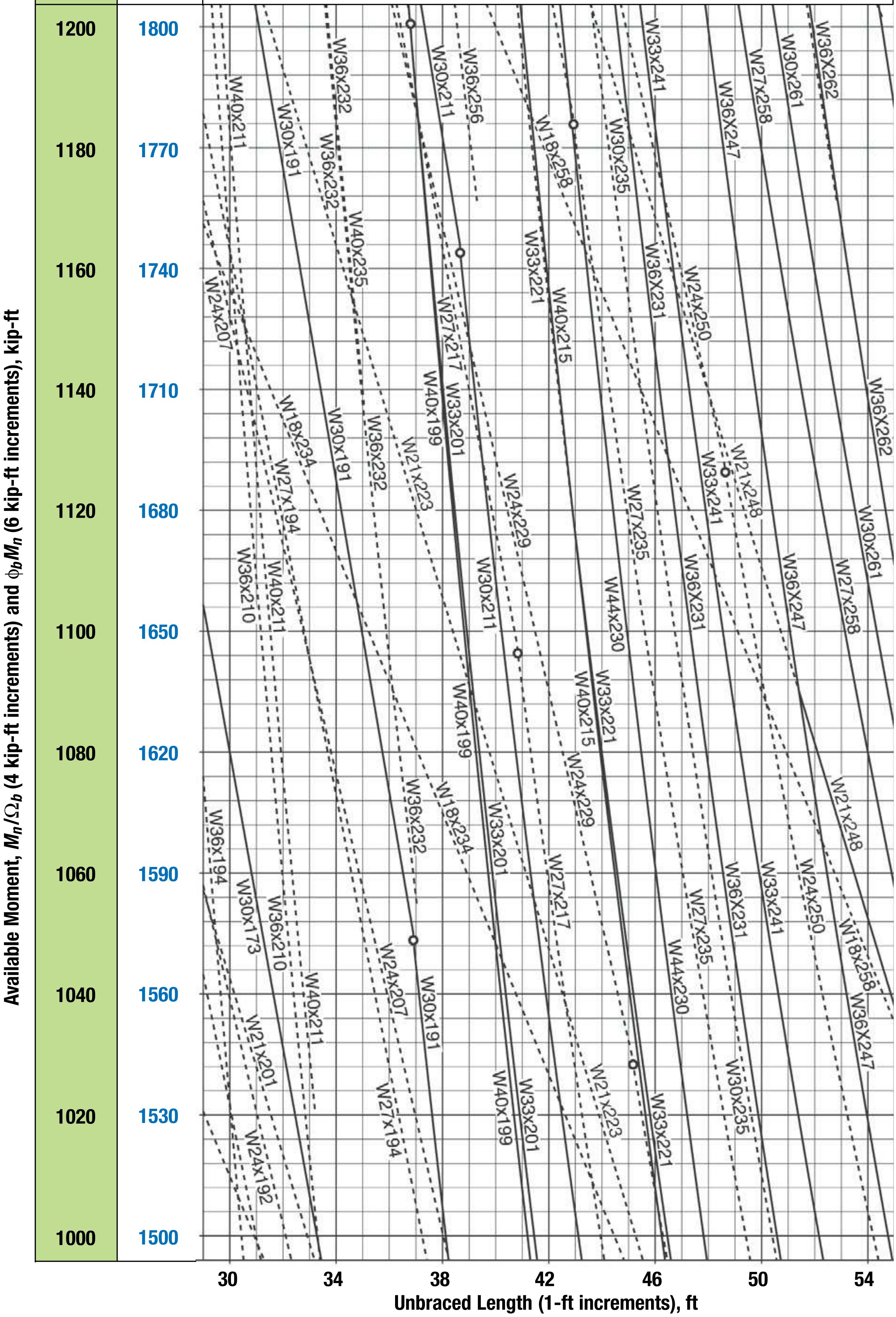
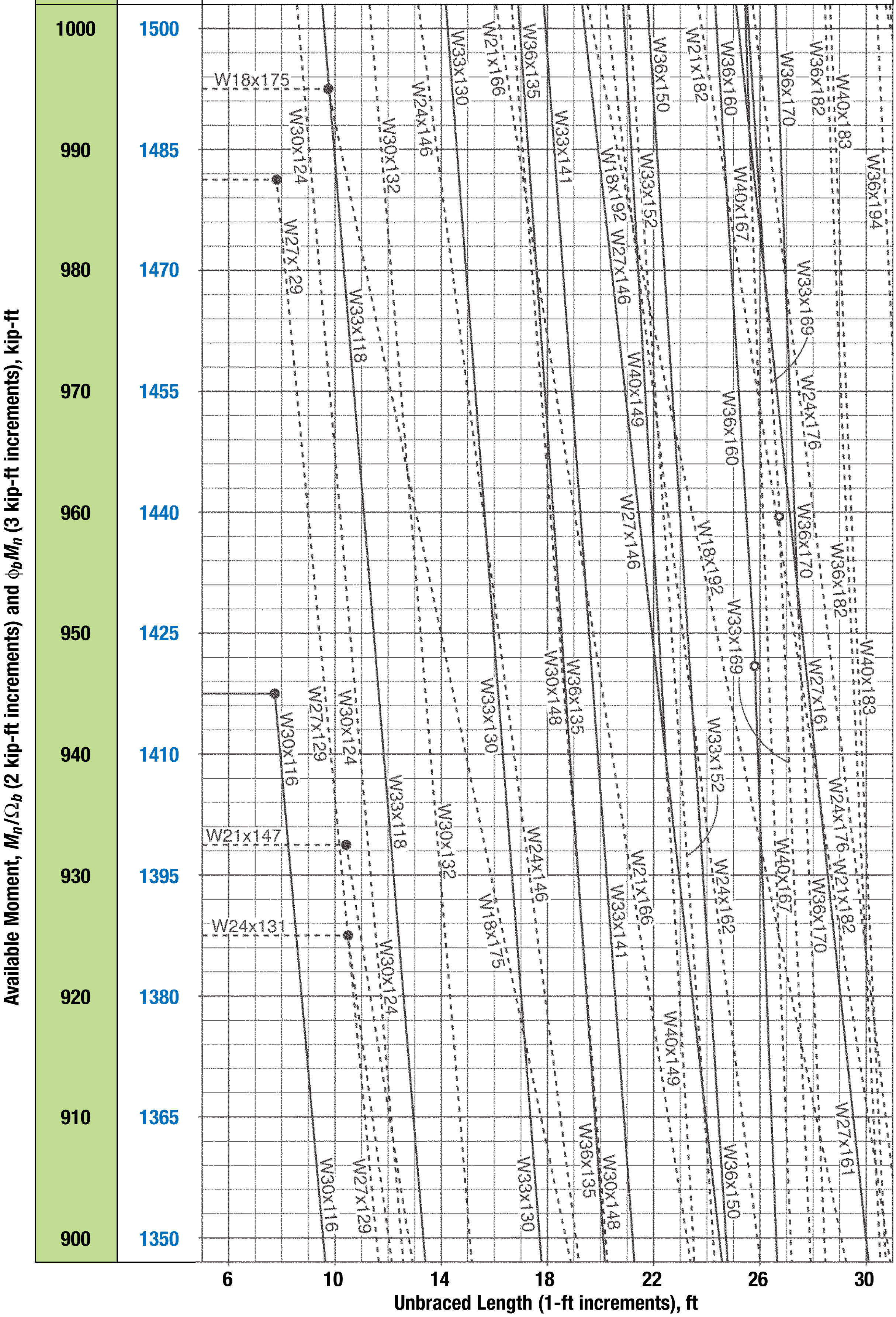


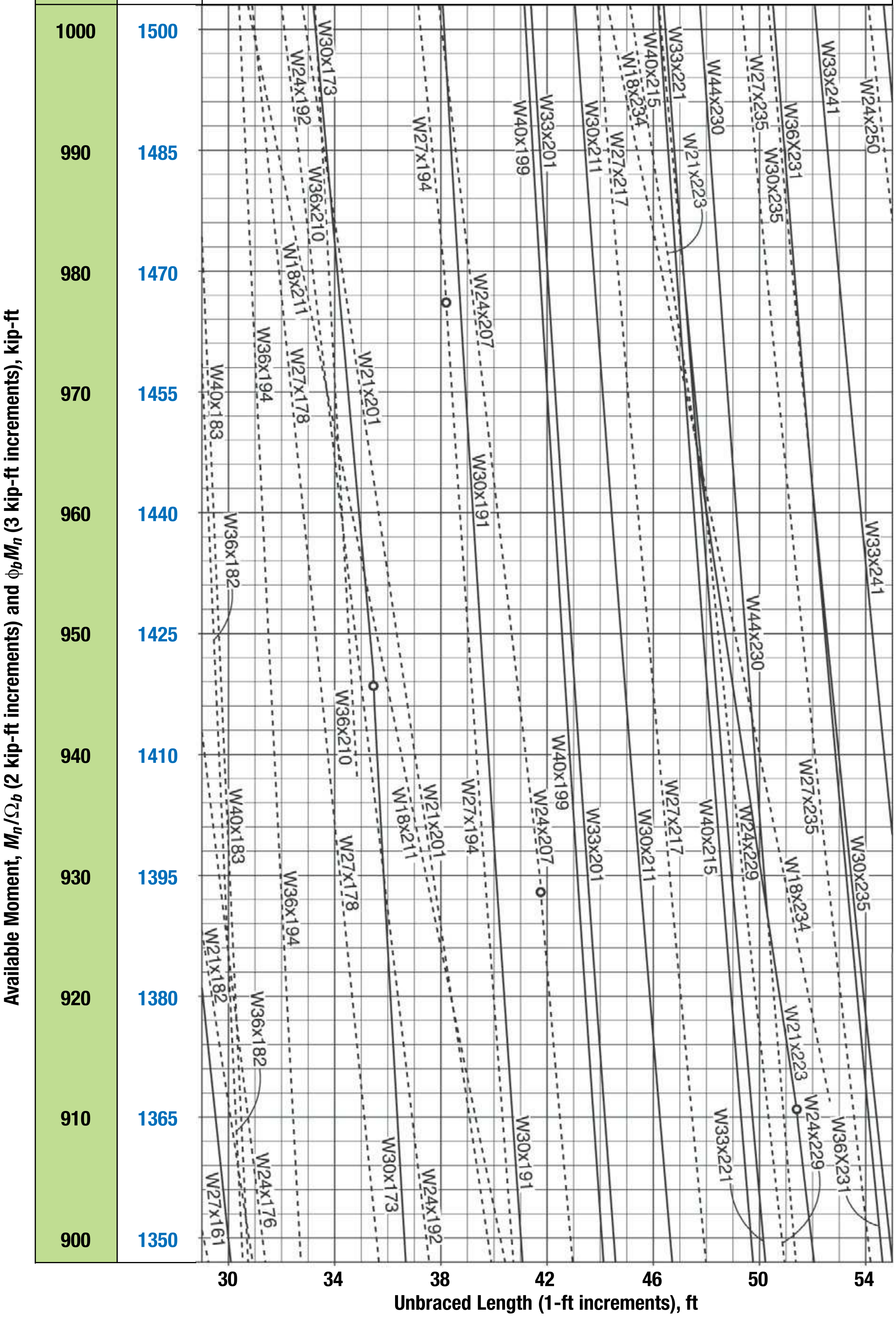
Table 3-10 (continued)
W-Shapes
Available Moment vs. Unbraced Length

$F_y = 50$ ksi
$C_b = 1$

M_n/Ω_b	$\phi_b M_n$
kip-ft	kip-ft
ASD	LRFD



F_y = 50 ksi C_b = 1		Table 3-10 (continued)
M_n/Ω_b kip-ft	$\phi_b M_n$ kip-ft	**W-Shapes**
ASD	LRFD	**Available Moment vs. Unbraced Length**



$F_y = 50$ ksi
$C_b = 1$

M_n/Ω_b	$\phi_b M_n$
kip-ft	kip-ft
ASD	LRFD

Table 3-10 (continued)
W-Shapes
Available Moment vs. Unbraced Length

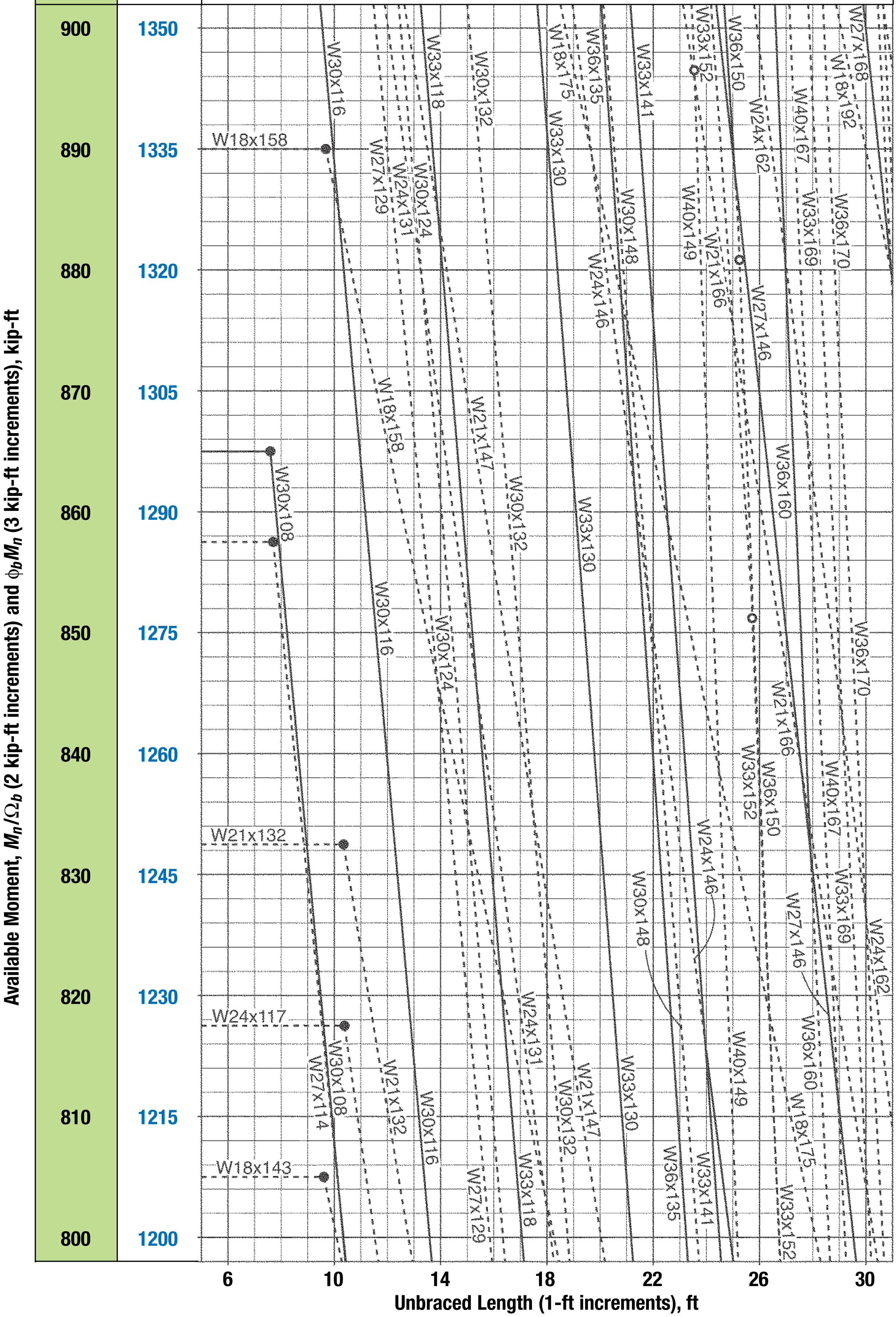
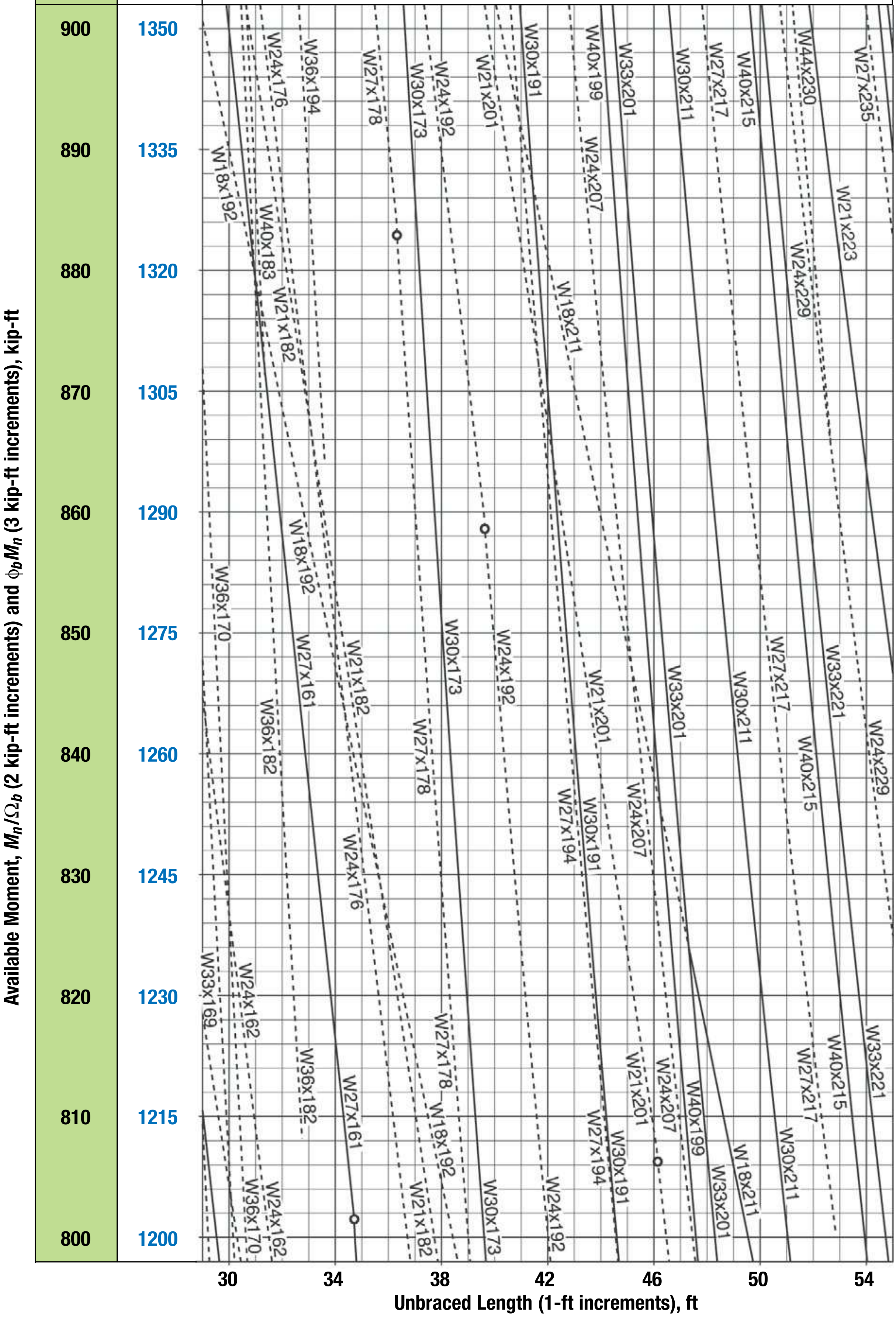


Table 3-10 (continued)
W-Shapes
Available Moment vs. Unbraced Length

F_y = 50 ksi
C_b = 1



F_y = 50 ksi C_b = 1		**Table 3-10 (continued)**
M_n/Ω_b	$\phi_b M_n$	**W-Shapes**
kip-ft	kip-ft	
ASD	LRFD	**Available Moment vs. Unbraced Length**

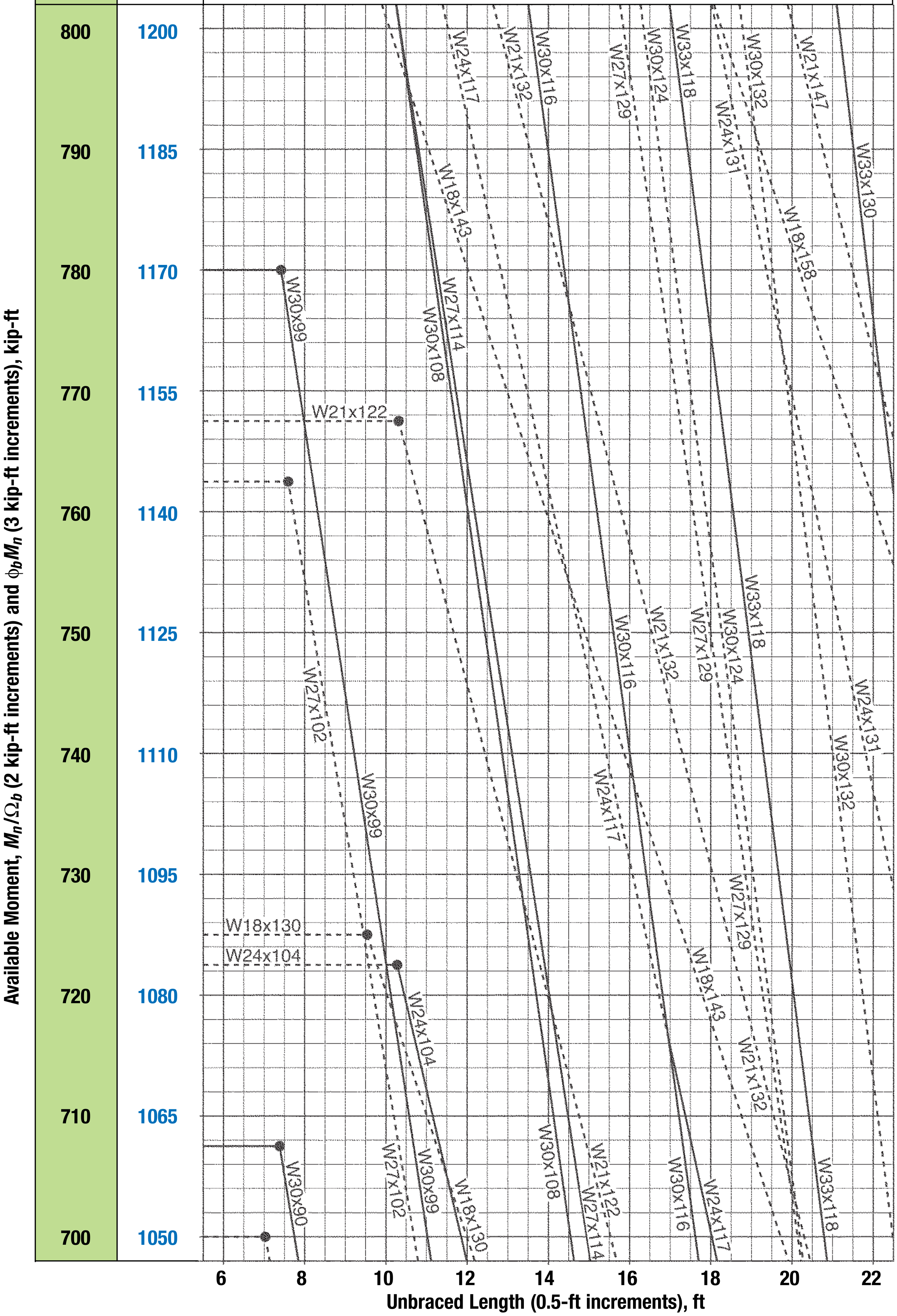
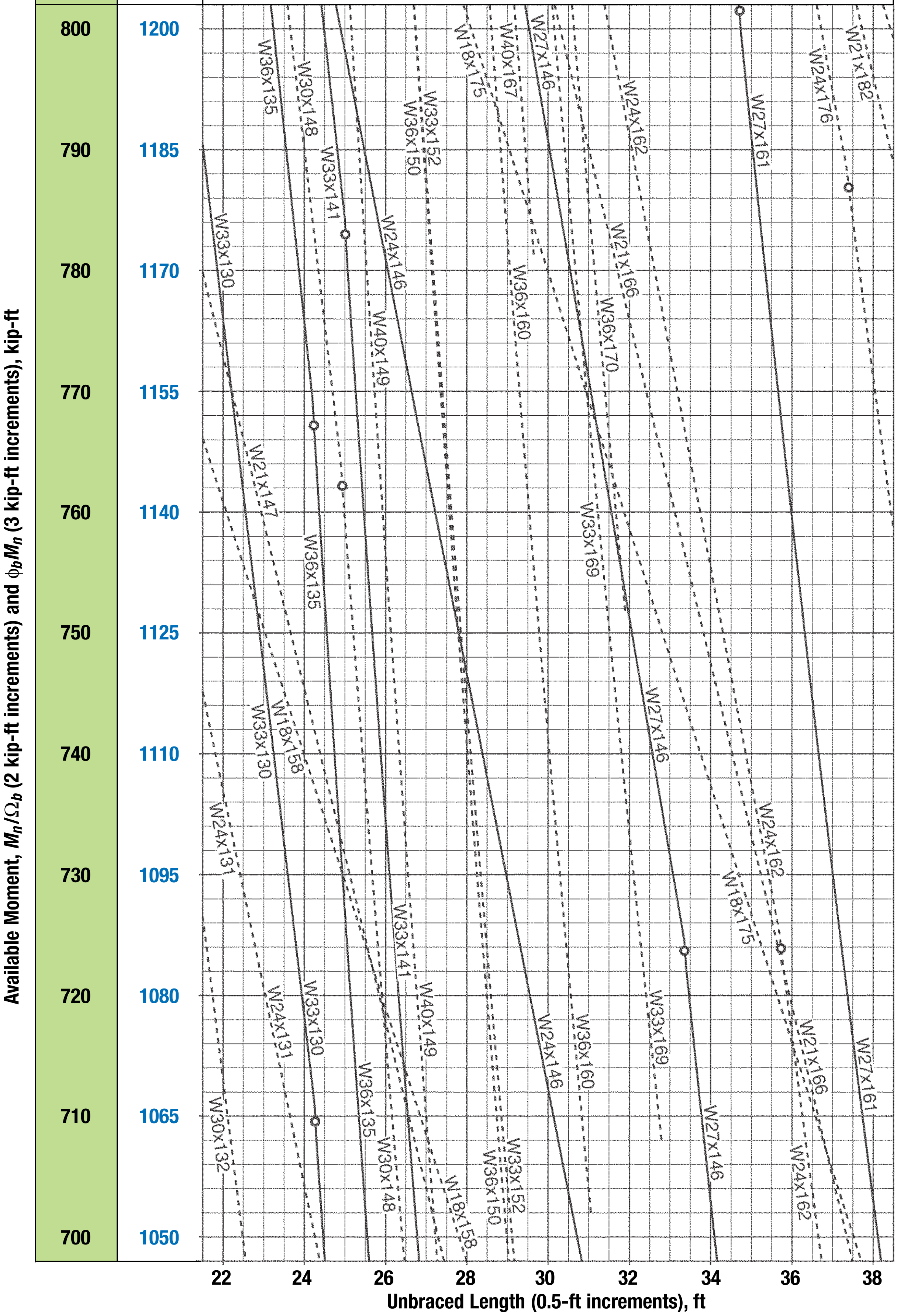


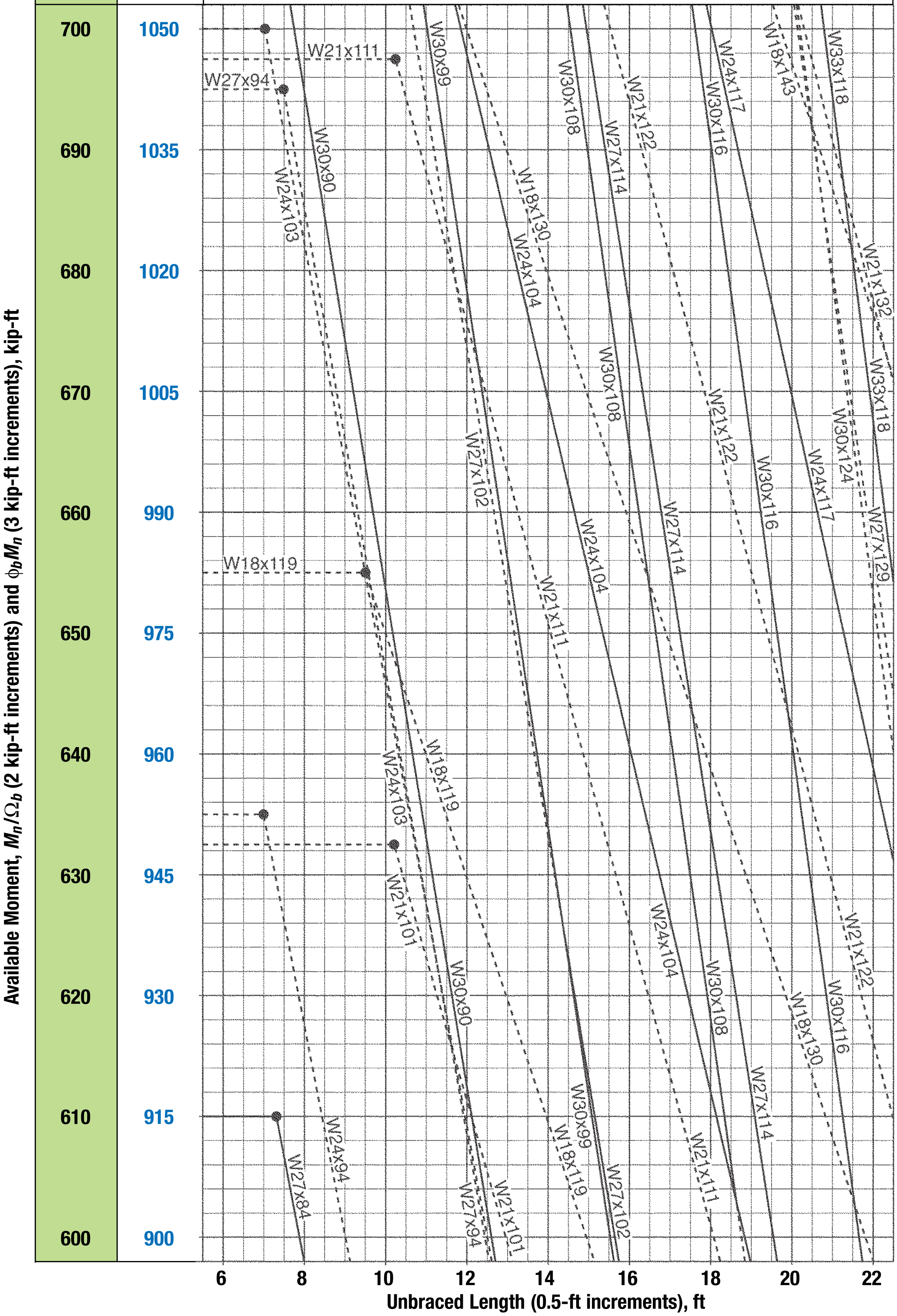
Table 3-10 (continued)
W-Shapes
Available Moment vs. Unbraced Length

F_y = 50 ksi
C_b = 1



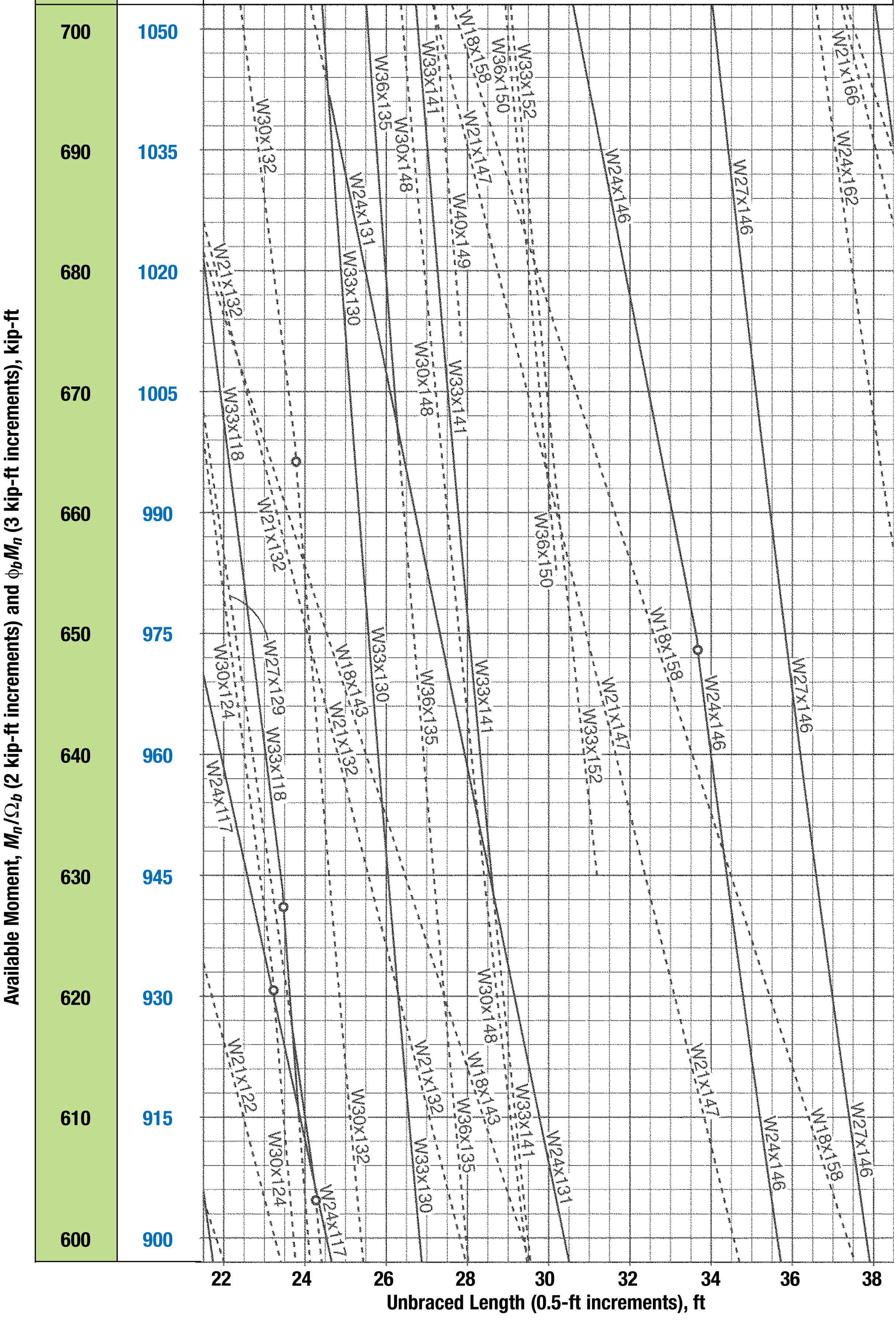
$F_y = 50$ ksi $C_b = 1$	
M_n/Ω_b	$\phi_b M_n$
kip-ft	kip-ft
ASD	LRFD

Table 3-10 (continued)
W-Shapes
Available Moment vs. Unbraced Length



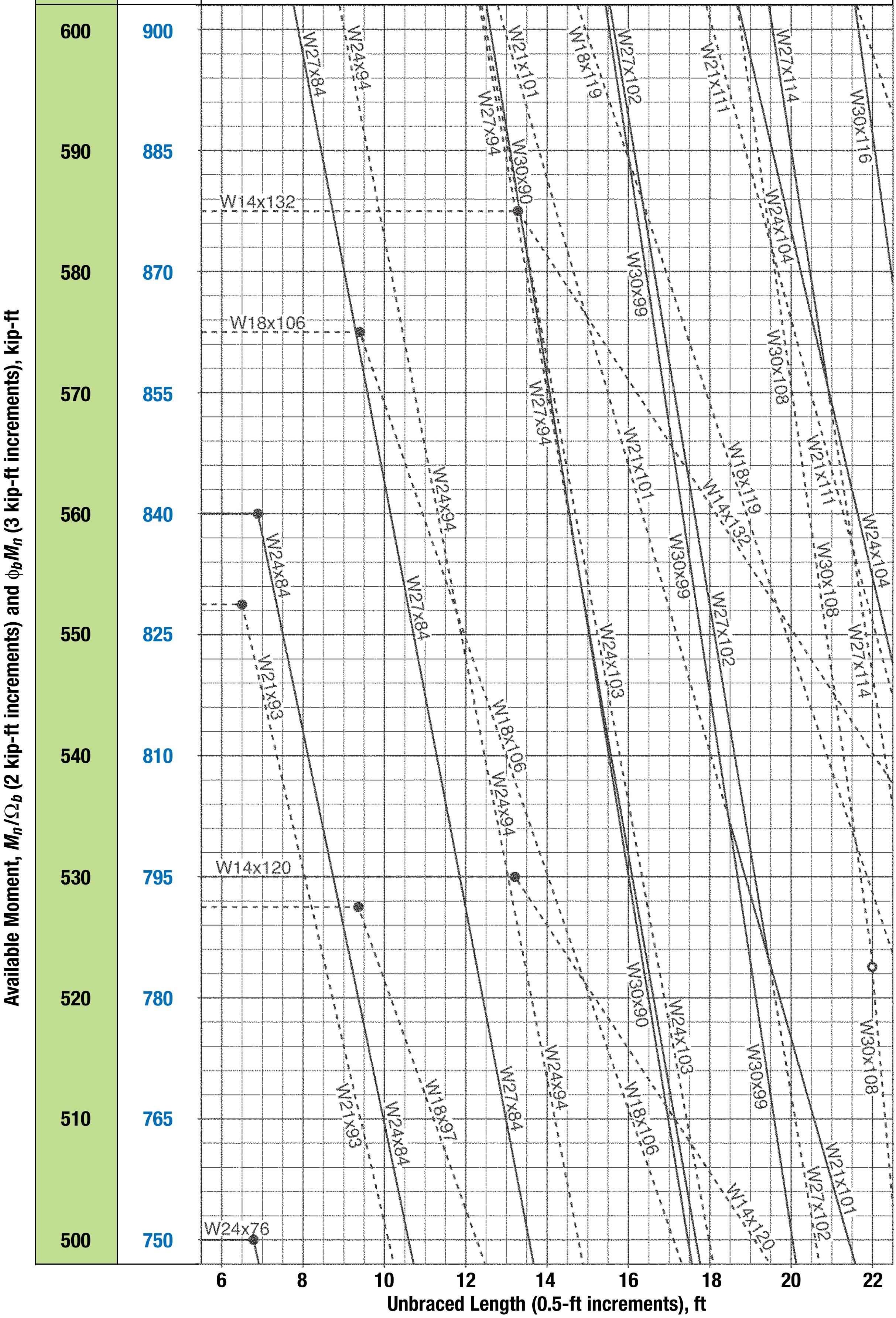
F_y = 50 ksi C_b = 1	
M_n/Ω_b	$\phi_b M_n$
kip-ft	kip-ft
ASD	LRFD

Table 3-10 (continued)
W-Shapes
Available Moment vs. Unbraced Length

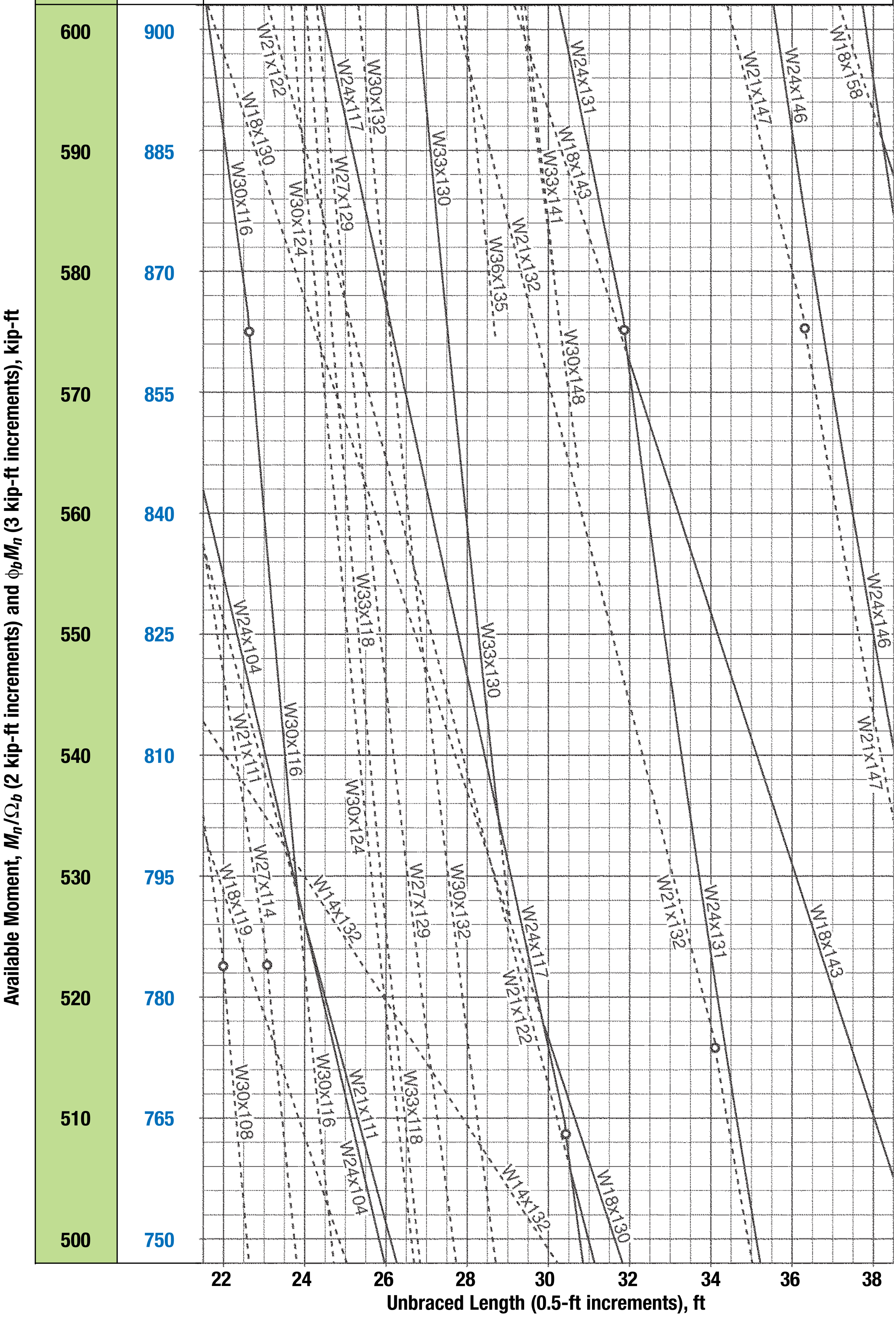


F_y = 50 ksi	
C_b = 1	
M_n/Ω_b	$\phi_b M_n$
kip-ft	kip-ft
ASD	LRFD

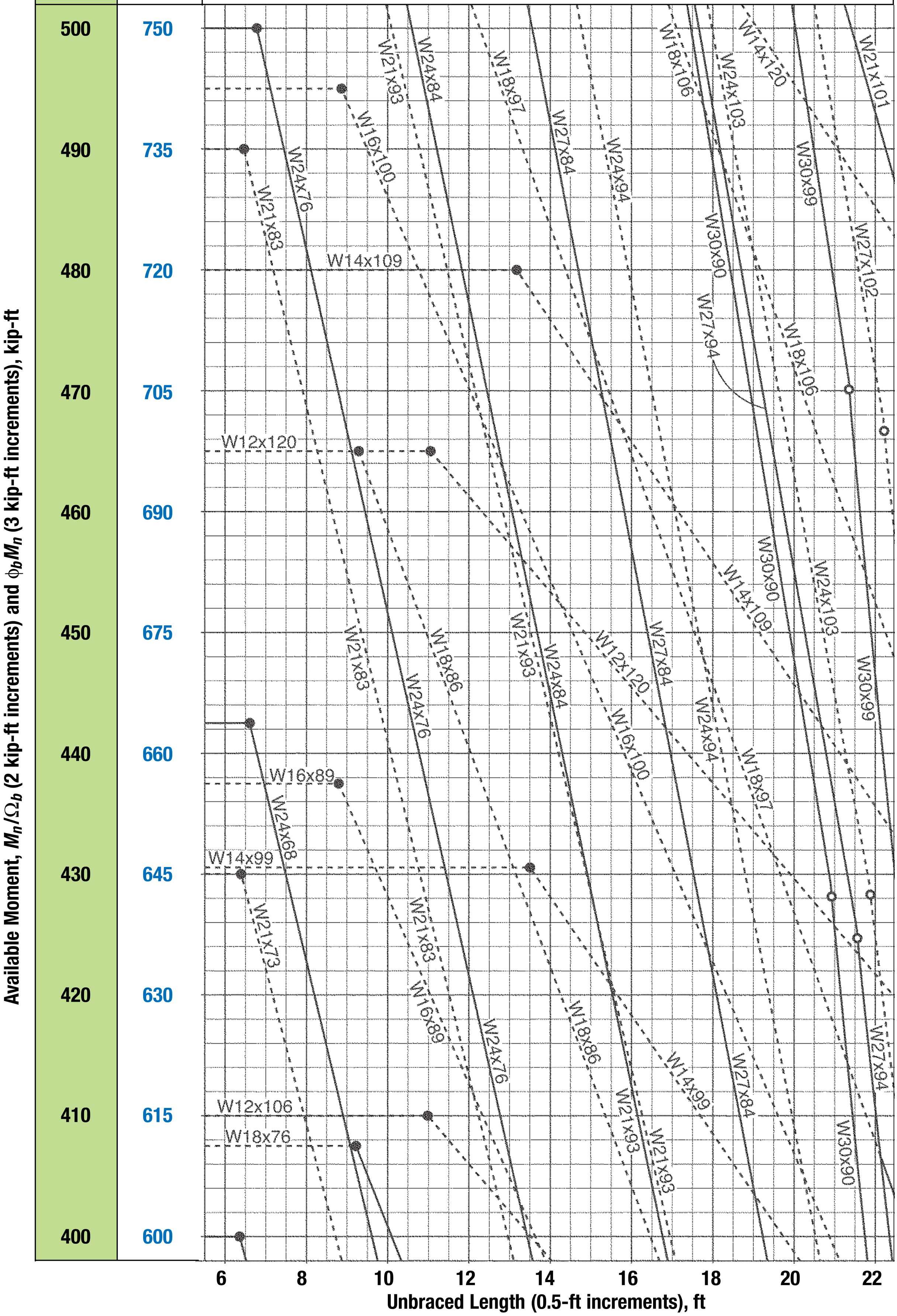
Table 3-10 (continued)
W-Shapes
Available Moment vs. Unbraced Length



$F_y = 50$ ksi $C_b = 1$		**Table 3-10 (continued)**
M_n/Ω_b	$\phi_b M_n$	**W-Shapes**
kip-ft	kip-ft	
ASD	LRFD	**Available Moment vs. Unbraced Length**



$F_y = 50$ ksi $C_b = 1$		**Table 3-10 (continued)** **W-Shapes** **Available Moment vs. Unbraced Length**
M_n/Ω_b	$\phi_b M_n$	
kip-ft	kip-ft	
ASD	LRFD	



F_y = 50 ksi C_b = 1	
M_n/Ω_b	$\phi_b M_n$
kip-ft	kip-ft
ASD	LRFD

Table 3-10 (continued)
W-Shapes
Available Moment vs. Unbraced Length

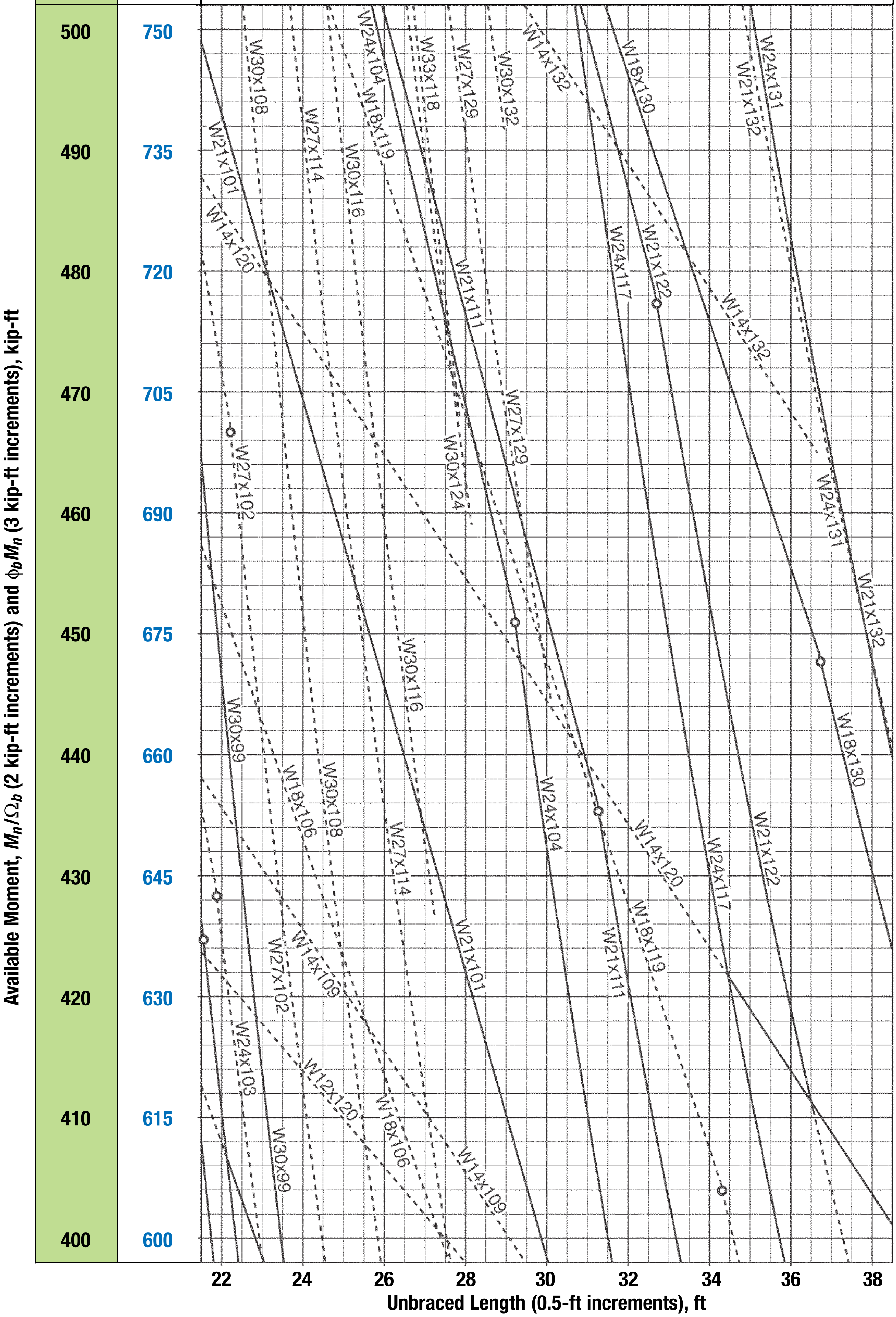
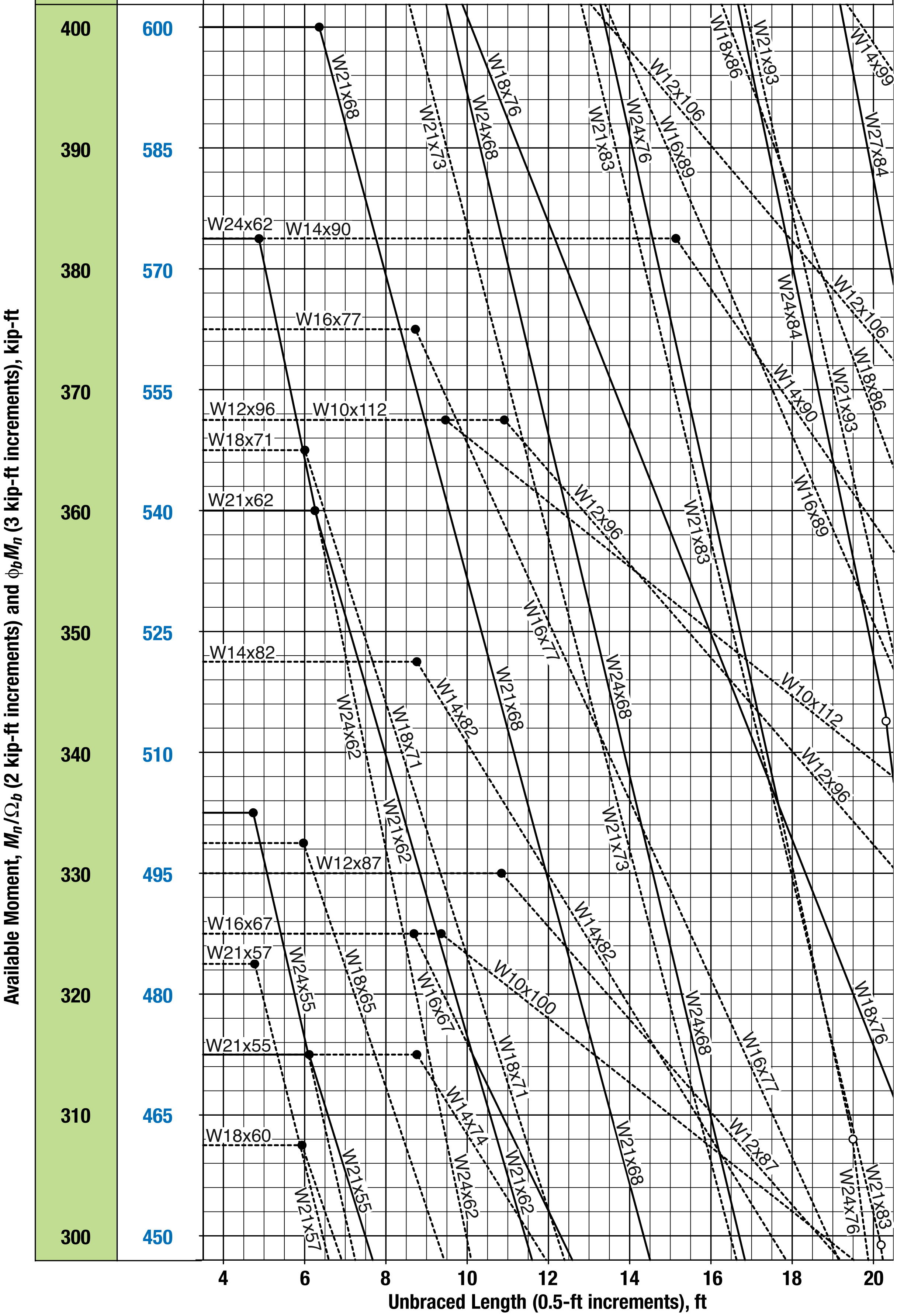


Table 3-10 (continued)
W-Shapes
Available Moment vs. Unbraced Length

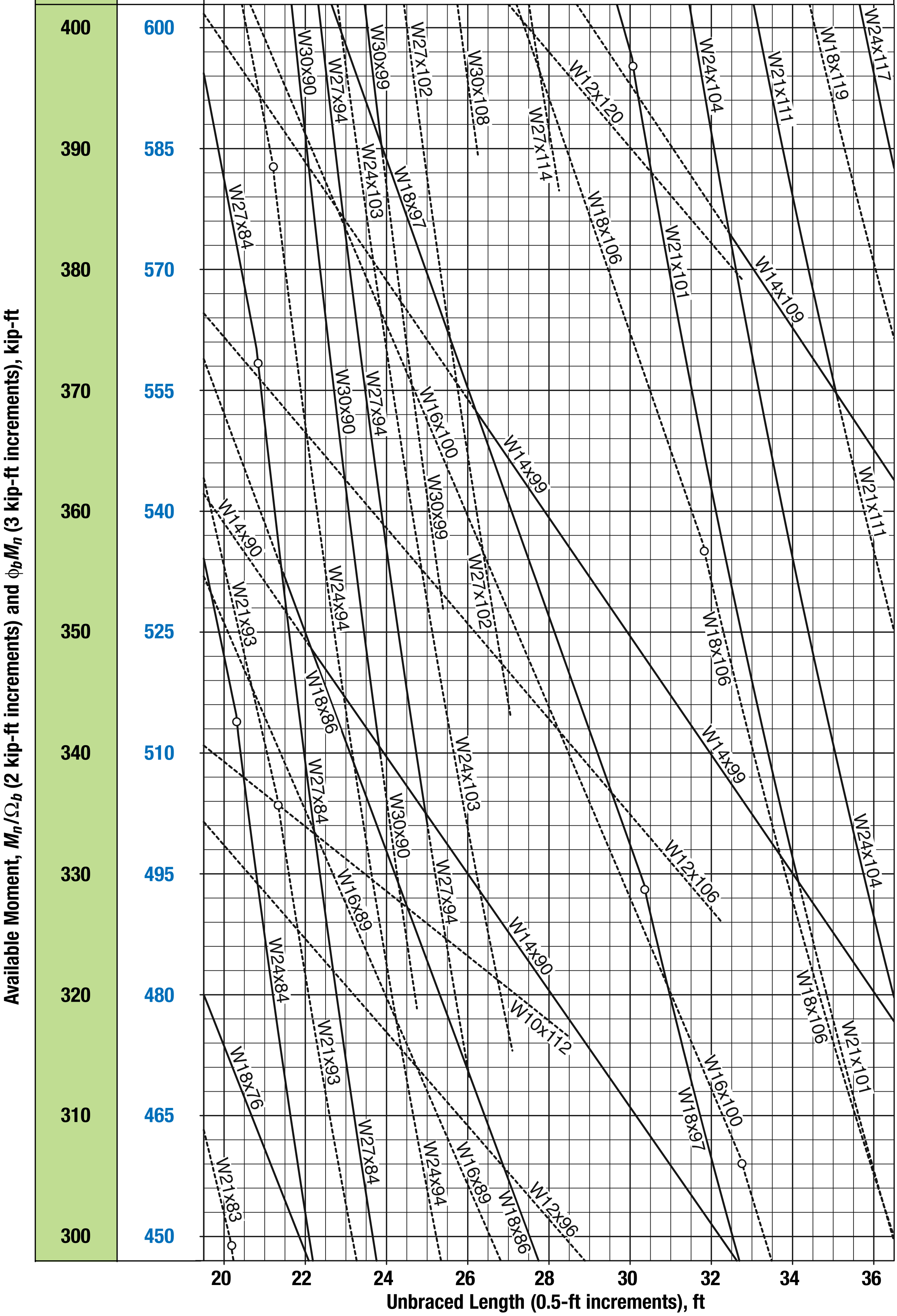
F_y = 50 ksi
C_b = 1

M_n/Ω_b	$\phi_b M_n$
kip-ft	kip-ft
ASD	LRFD



$F_y = 50$ ksi
$C_b = 1$

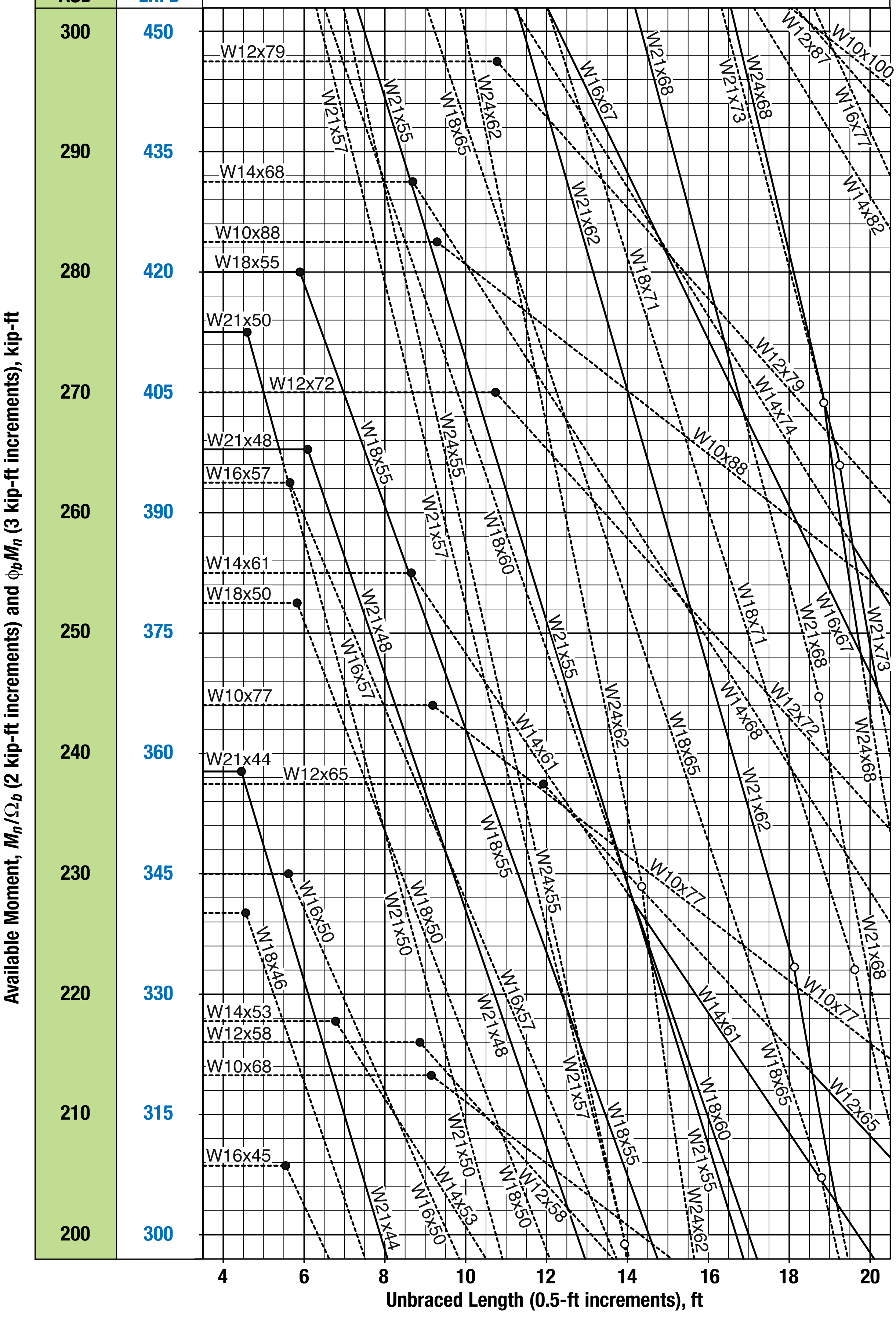
Table 3-10 (continued)
W-Shapes
Available Moment vs. Unbraced Length



$F_y = 50$ ksi
$C_b = 1$

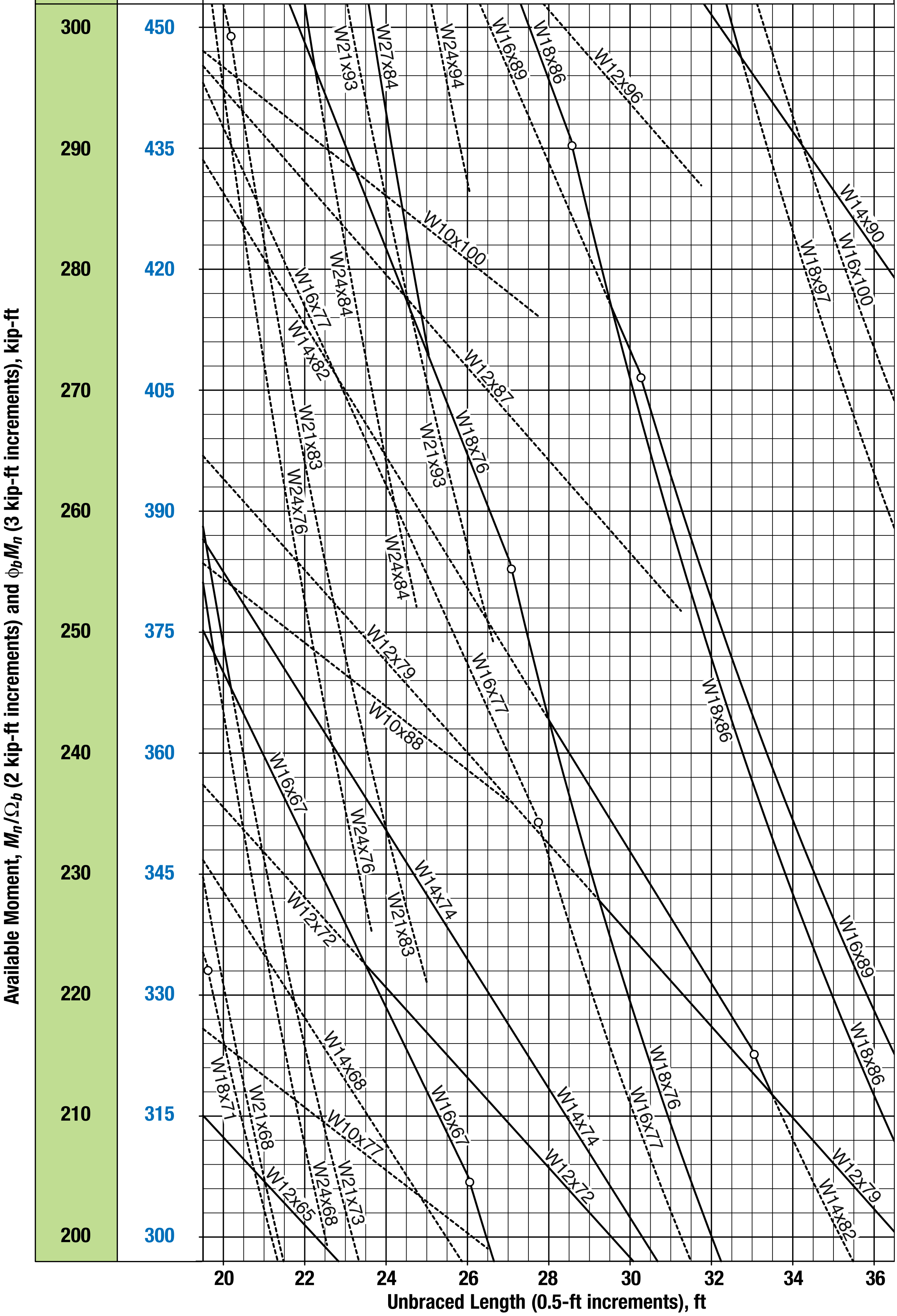
M_n/Ω_b	$\phi_b M_n$
kip-ft	kip-ft
ASD	LRFD

Table 3-10 (continued)
W-Shapes
Available Moment vs. Unbraced Length



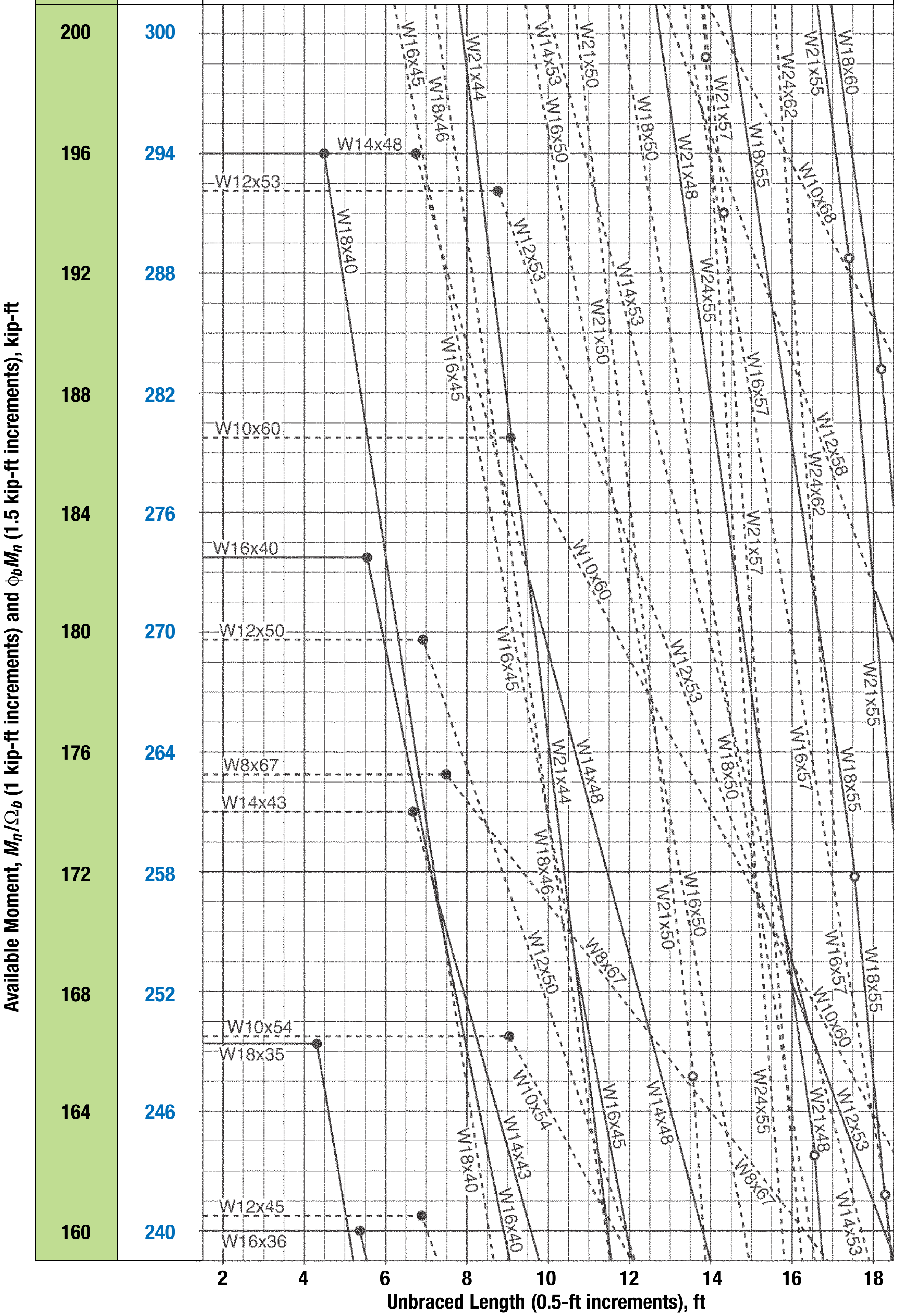
$F_y = 50$ ksi, $C_b = 1$	
M_n/Ω_b	$\phi_b M_n$
kip-ft	kip-ft
ASD	LRFD

Table 3-10 (continued)
W-Shapes
Available Moment vs. Unbraced Length



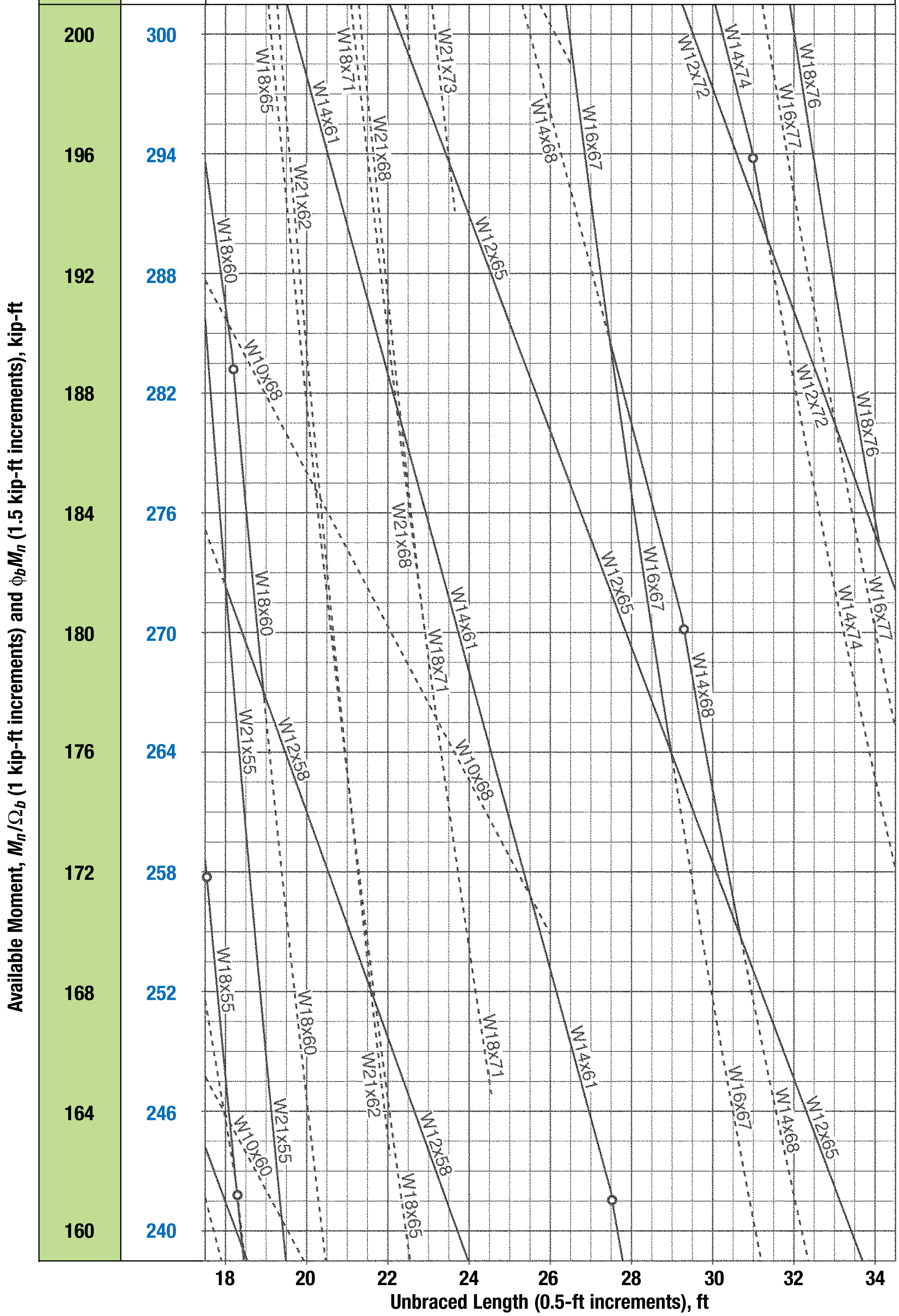
F_y = 50 ksi C_b = 1	
M_n/Ω_b	$\phi_b M_n$
kip-ft	kip-ft
ASD	LRFD

Table 3-10 (continued)
W-Shapes
Available Moment vs. Unbraced Length



F_y = 50 ksi C_b = 1	
M_n/Ω_b	$\phi_b M_n$
kip-ft	kip-ft
ASD	LRFD

Table 3-10 (continued)
W-Shapes
Available Moment vs. Unbraced Length



F_y = 50 ksi C_b = 1	
M_n/Ω_b	$\phi_b M_n$
kip-ft	kip-ft
ASD	LRFD

Table 3-10 (continued)
W-Shapes
Available Moment vs. Unbraced Length

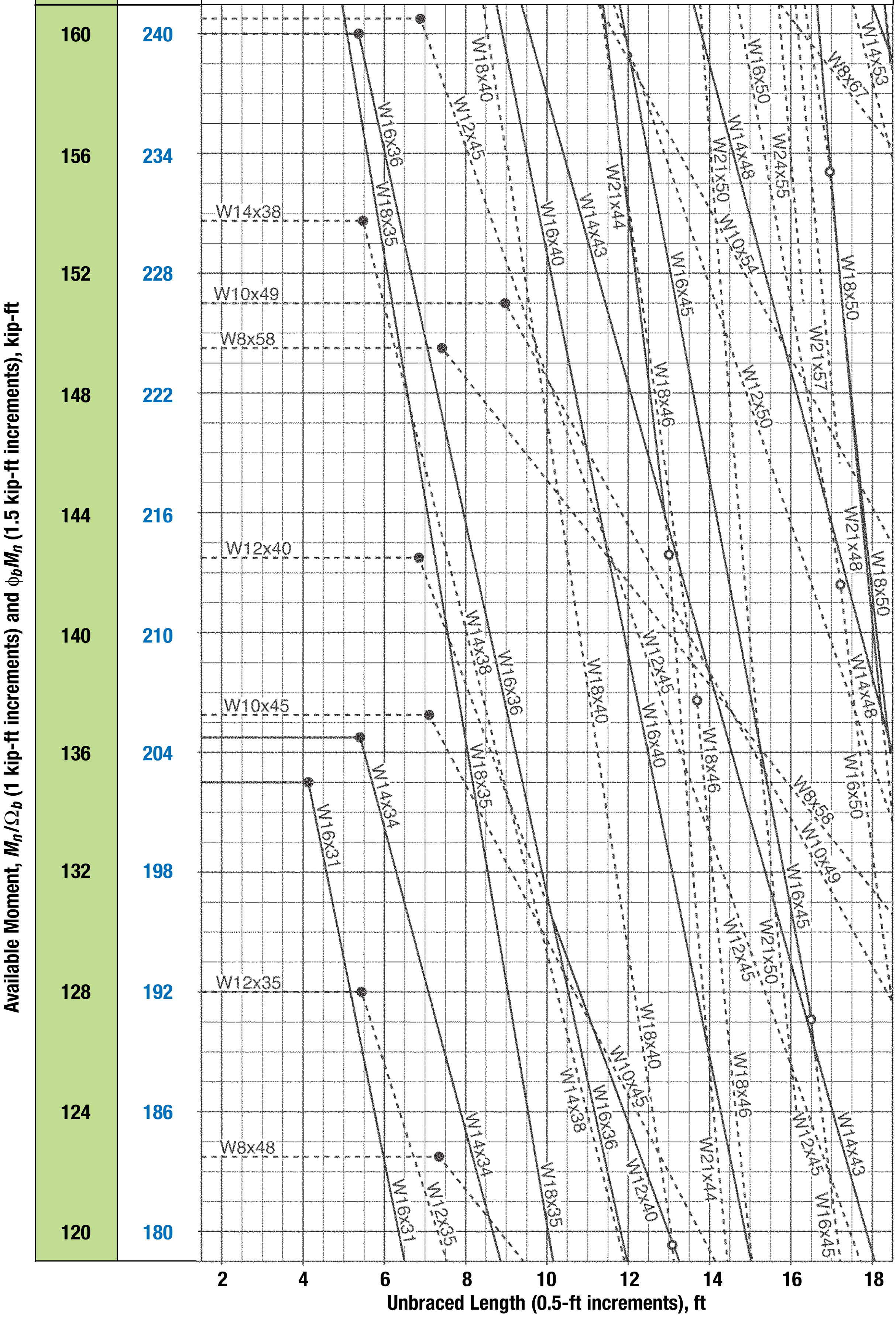
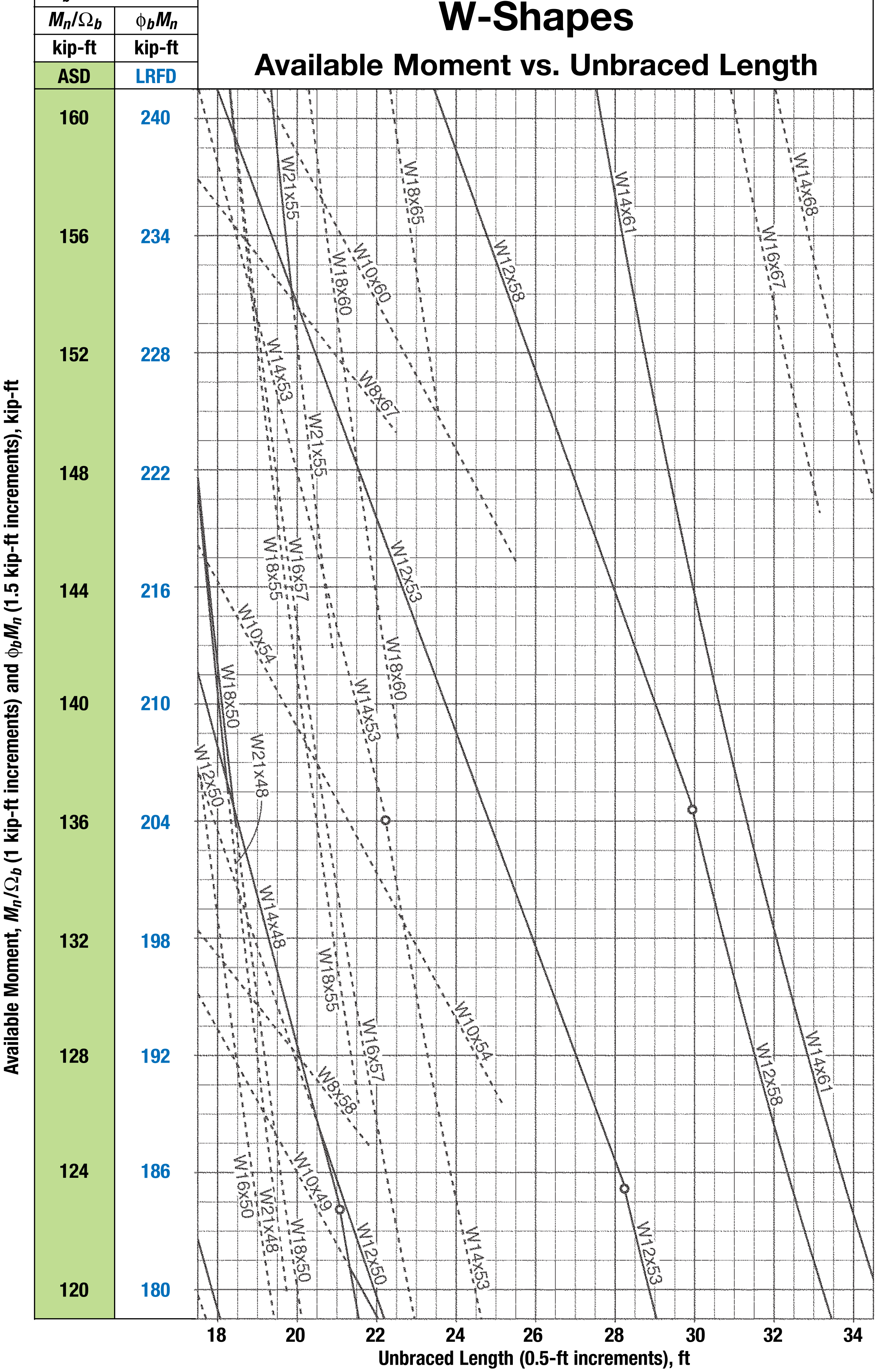


Table 3-10 (continued)
W-Shapes
Available Moment vs. Unbraced Length

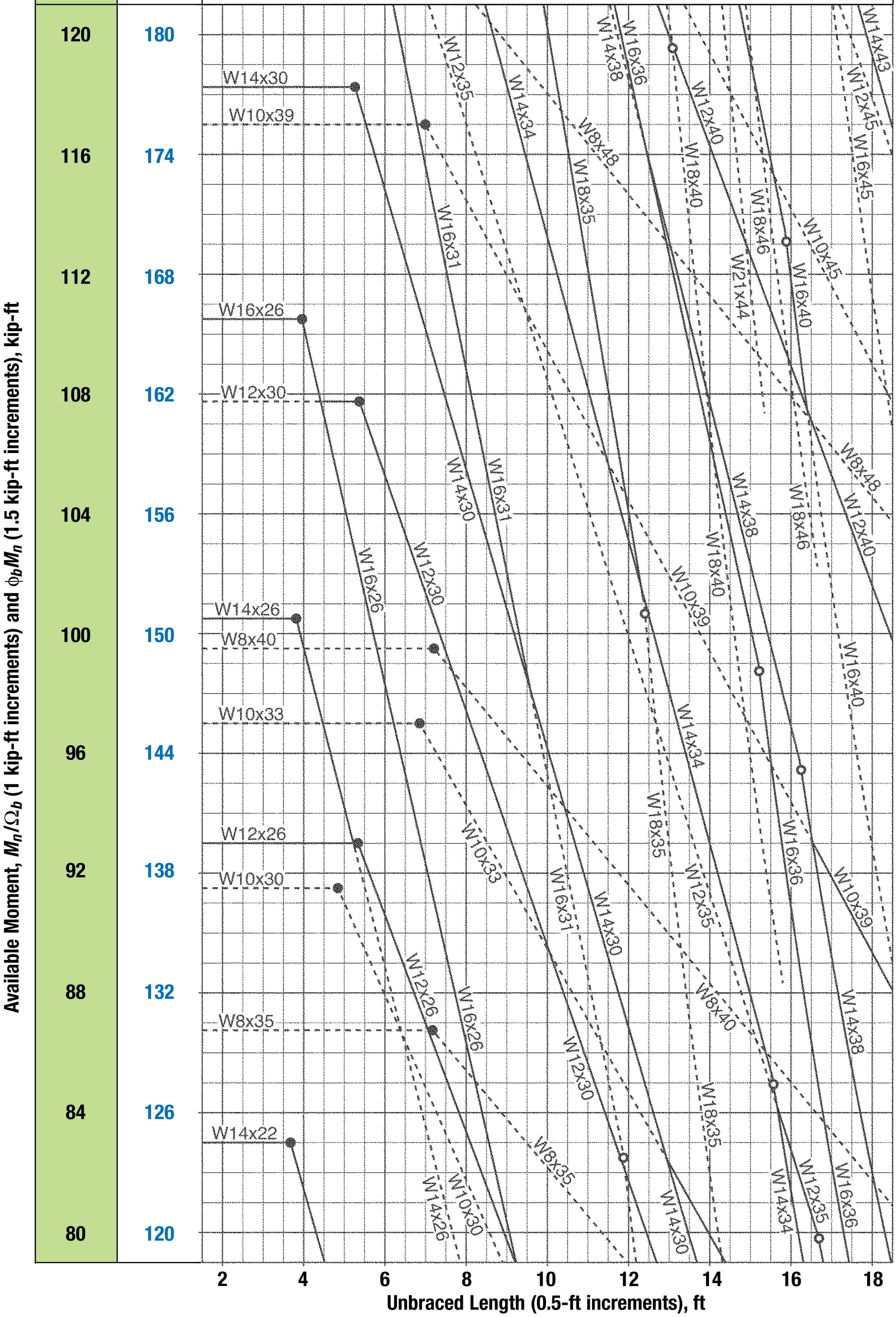
$F_y = 50$ ksi
$C_b = 1$



F_y = 50 ksi
C_b = 1

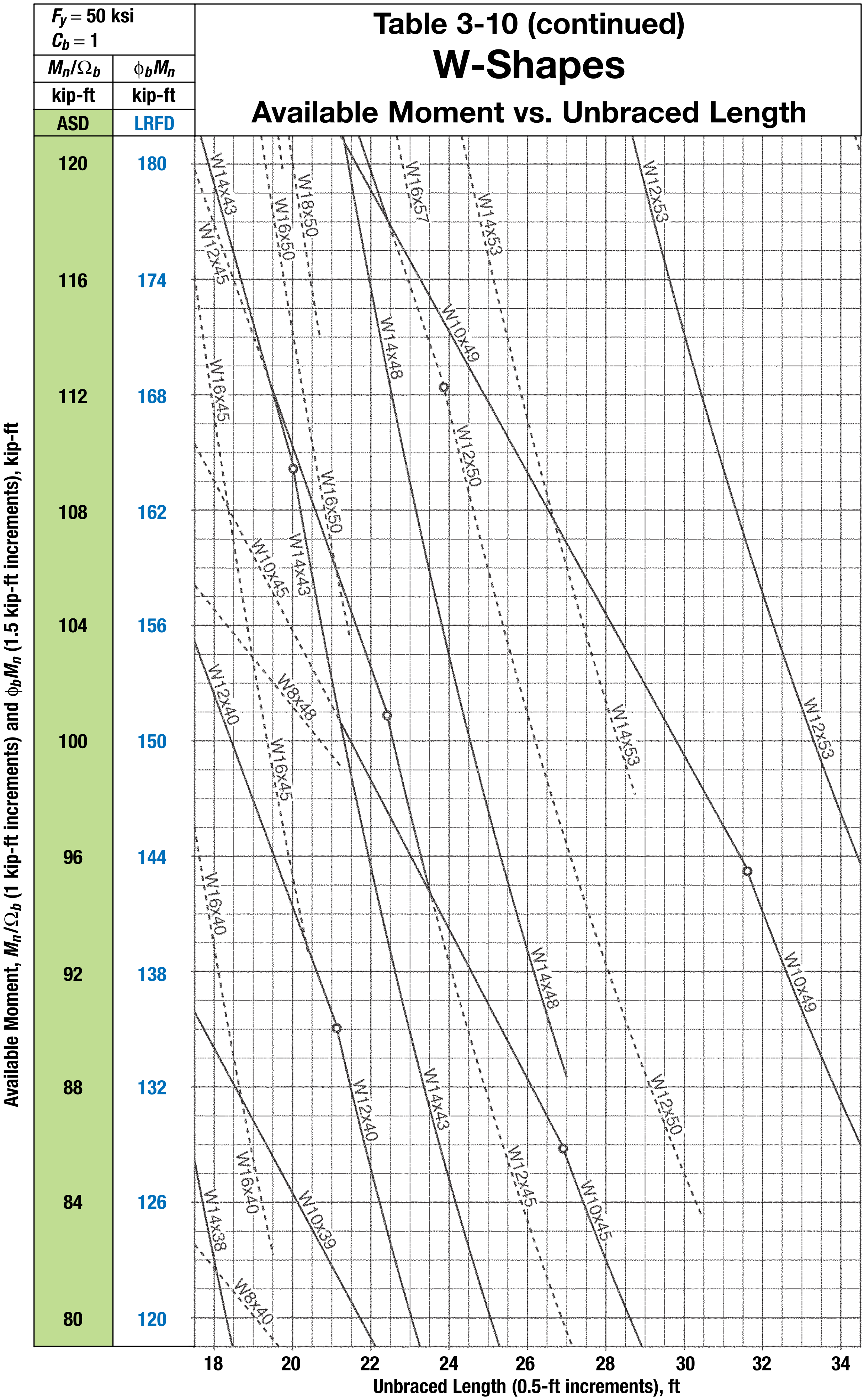
M_n/Ω_b	$\phi_b M_n$
kip-ft	kip-ft
ASD	LRFD

Table 3-10 (continued)
W-Shapes
Available Moment vs. Unbraced Length

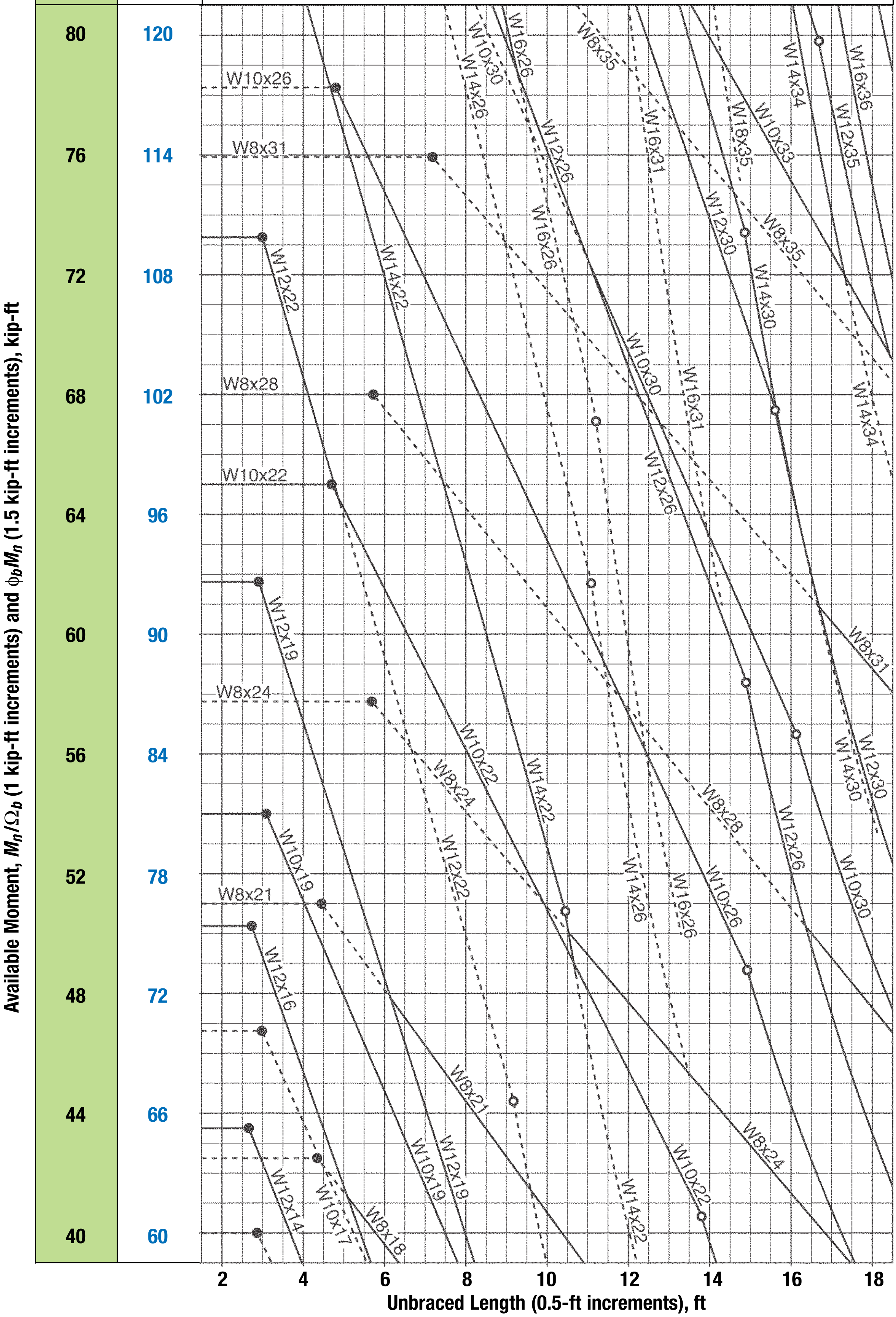


$F_y = 50$ ksi
$C_b = 1$

Table 3-10 (continued)
W-Shapes
Available Moment vs. Unbraced Length

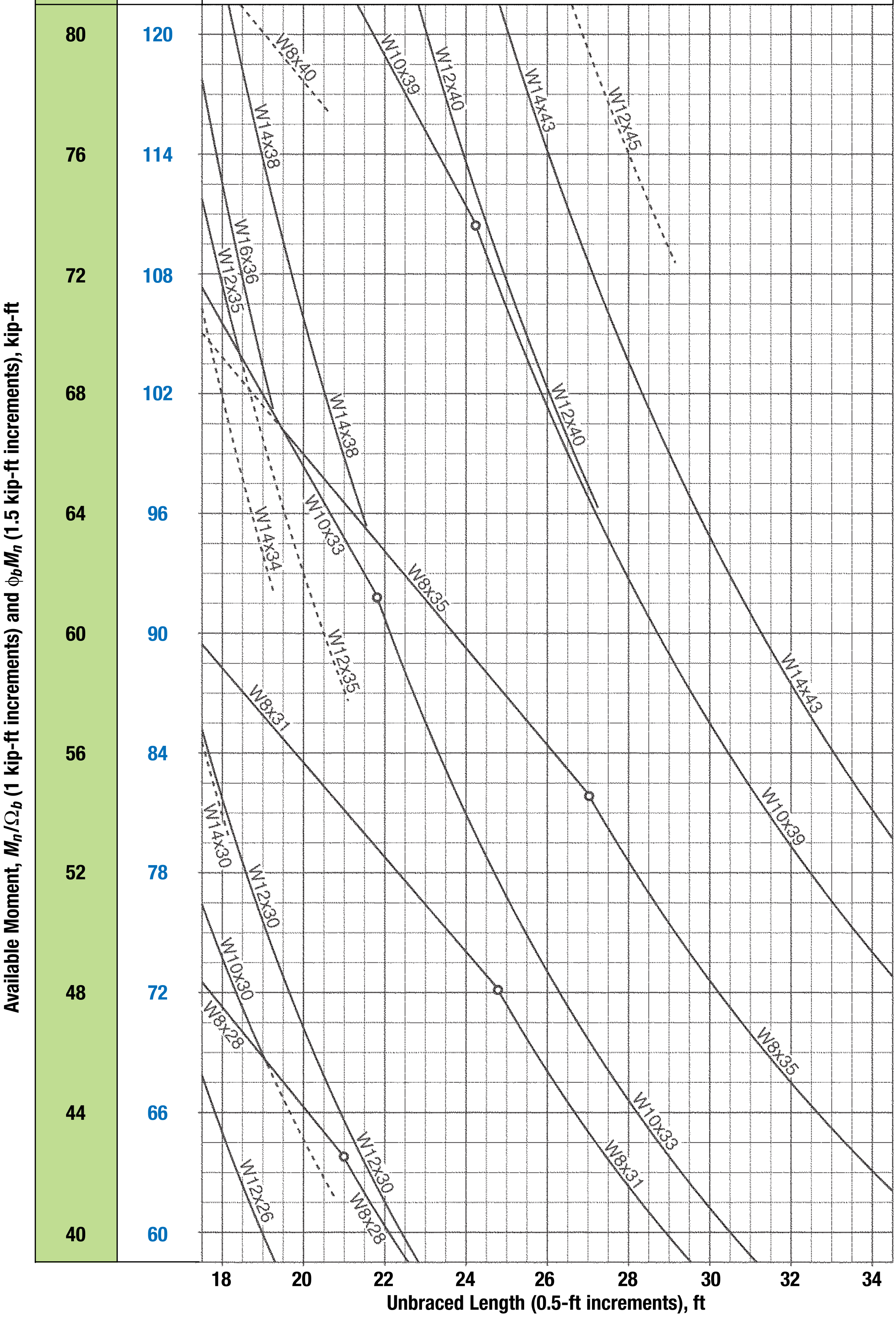


F_y = 50 ksi C_b = 1		Table 3-10 (continued) **W-Shapes** Available Moment vs. Unbraced Length
M_n/Ω_b	$\phi_b M_n$	
kip-ft	kip-ft	
ASD	LRFD	



F_y = 50 ksi C_b = 1	
M_n/Ω_b	$\phi_b M_n$
kip-ft	kip-ft
ASD	LRFD

Table 3-10 (continued)
W-Shapes
Available Moment vs. Unbraced Length



F_y = 50 ksi
C_b = 1

Table 3-10 (continued)
W-Shapes
Available Moment vs. Unbraced Length

M_n/Ω_b	$\phi_b M_n$
kip-ft	kip-ft
ASD	LRFD

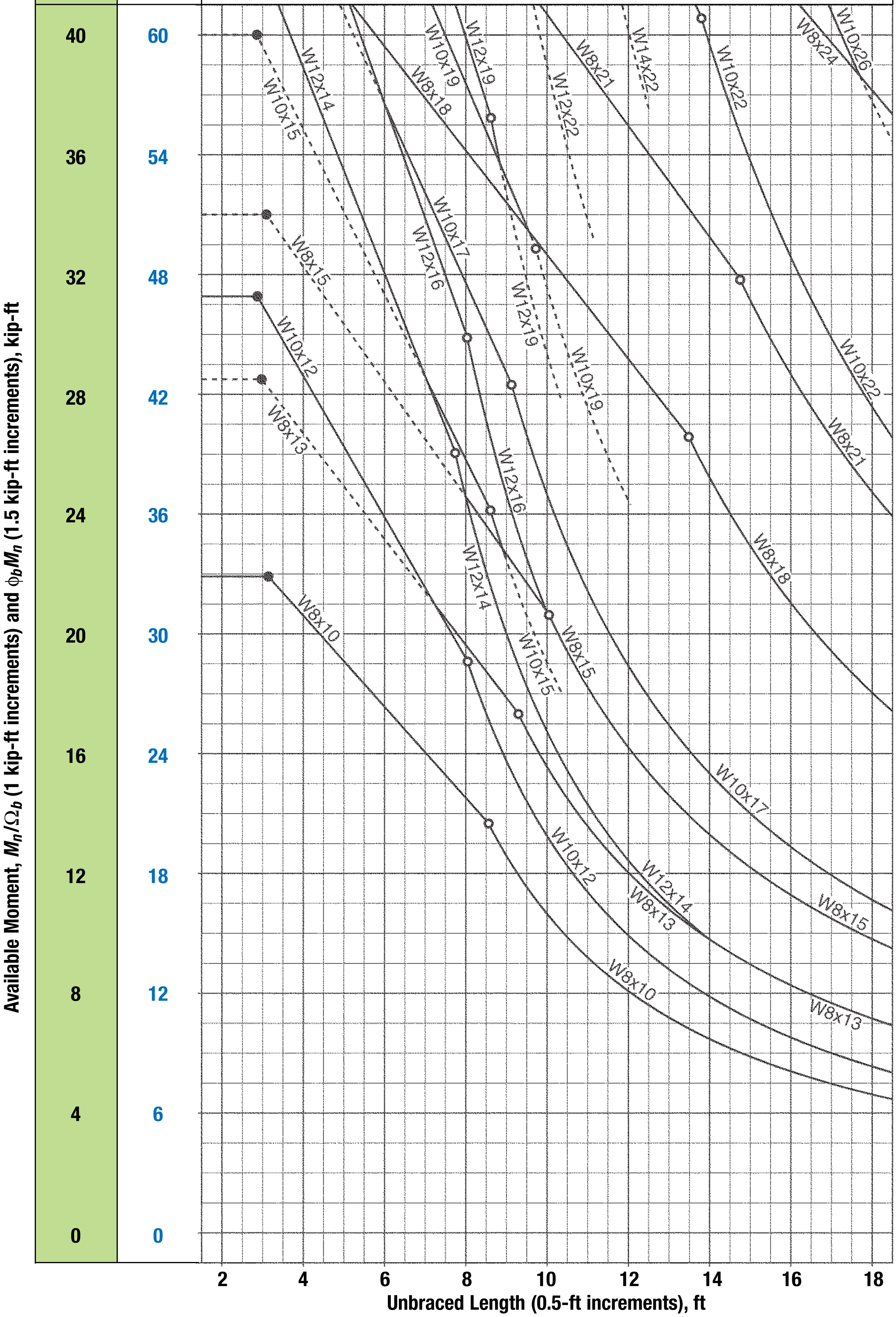
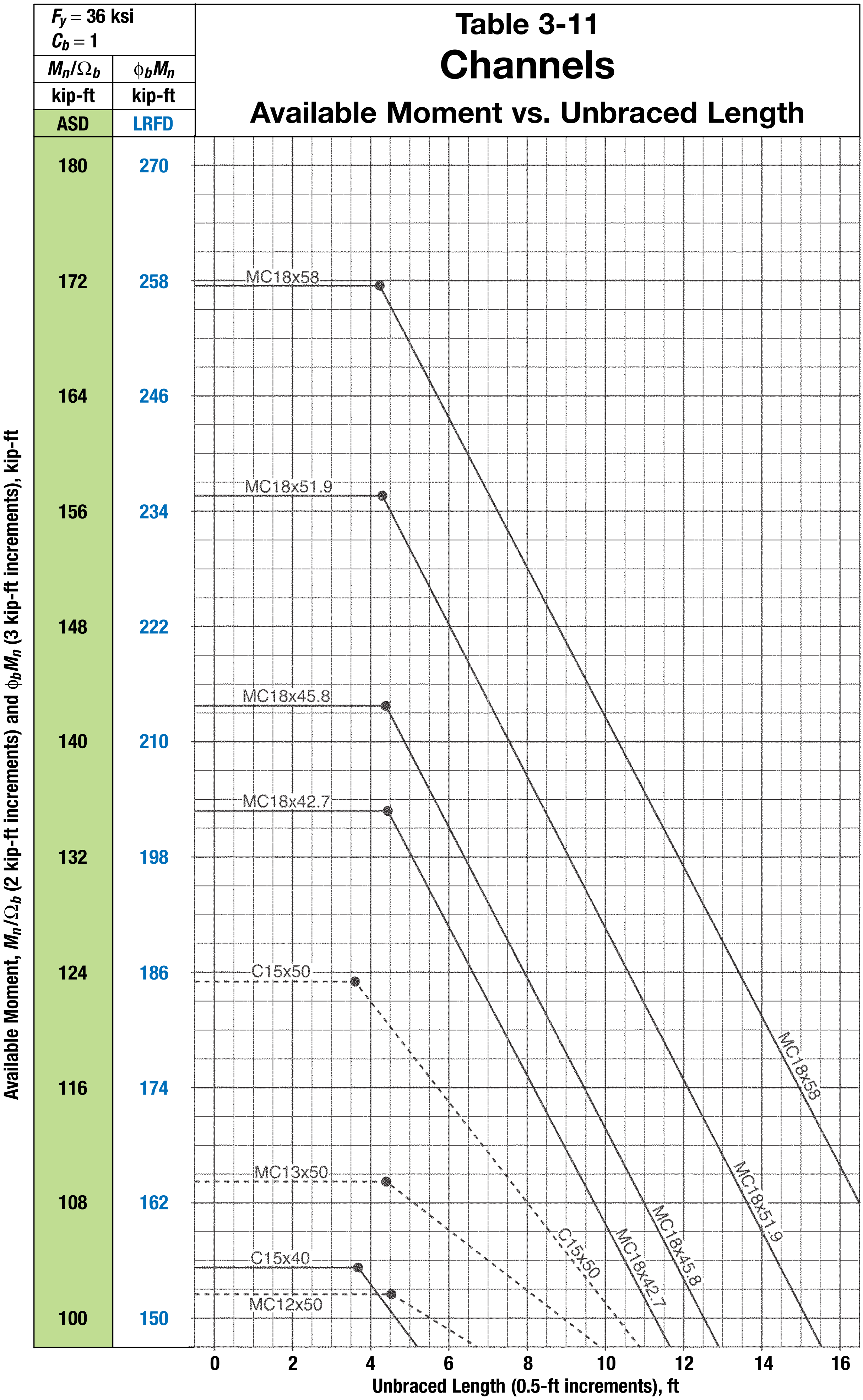


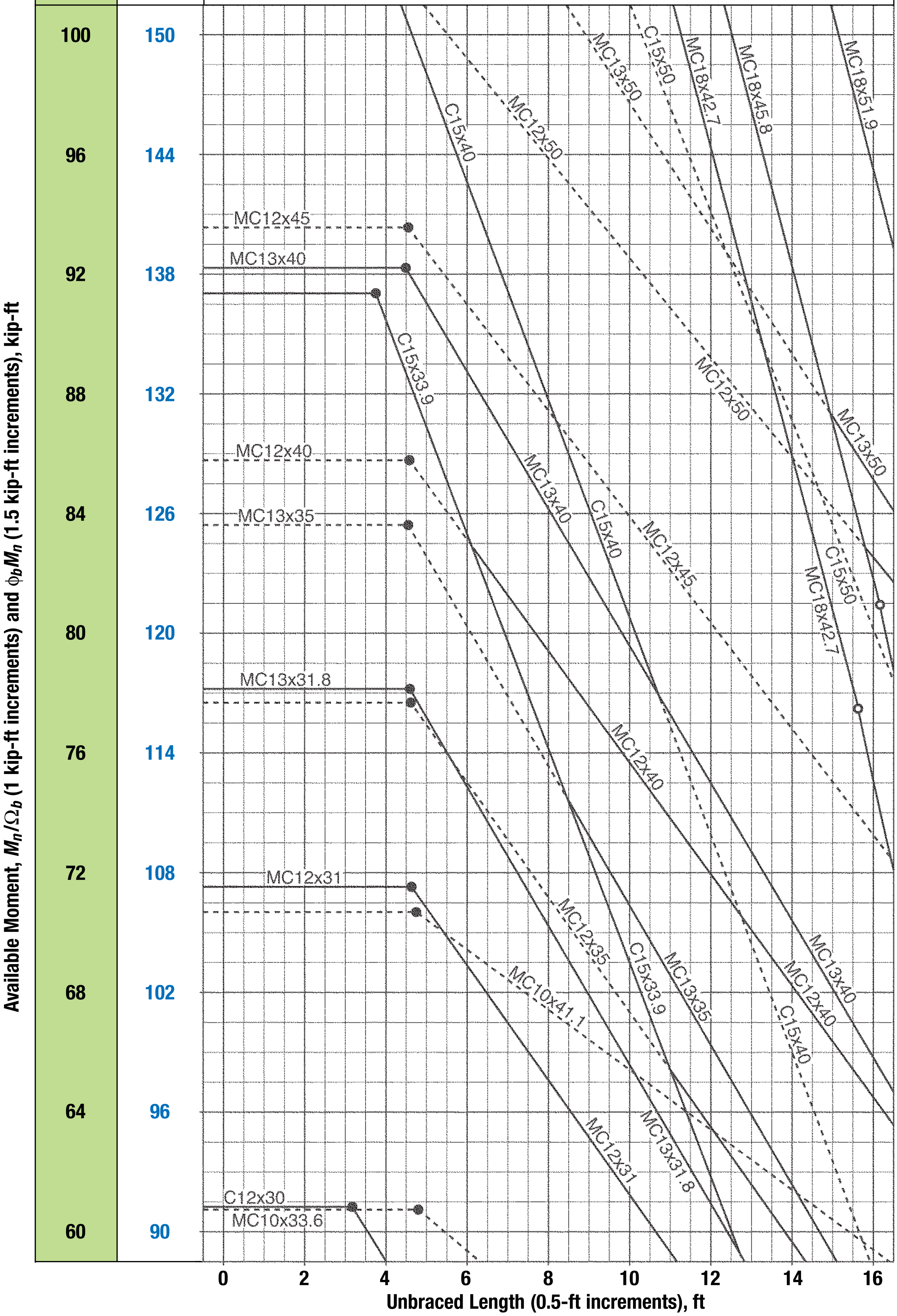
Table 3-11
Channels
Available Moment vs. Unbraced Length



$F_y = 36$ ksi
$C_b = 1$

M_n/Ω_b	$\phi_b M_n$
kip-ft	kip-ft
ASD	LRFD

Table 3-11 (continued)
Channels
Available Moment vs. Unbraced Length



$F_y = 36$ ksi $C_b = 1$	
M_n/Ω_b	$\phi_b M_n$
kip-ft	kip-ft
ASD	LRFD

Table 3-11 (continued)
Channels
Available Moment vs. Unbraced Length

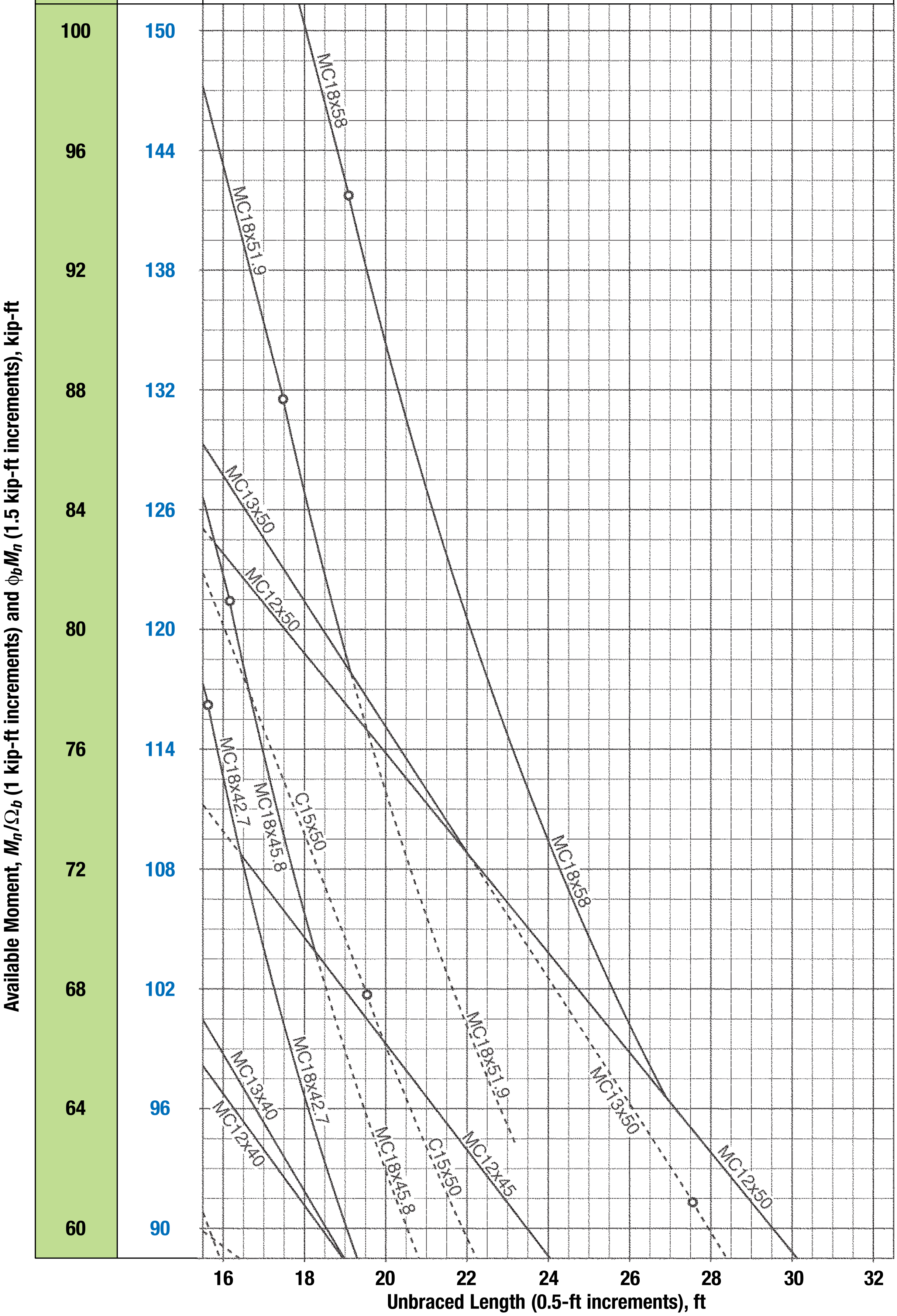
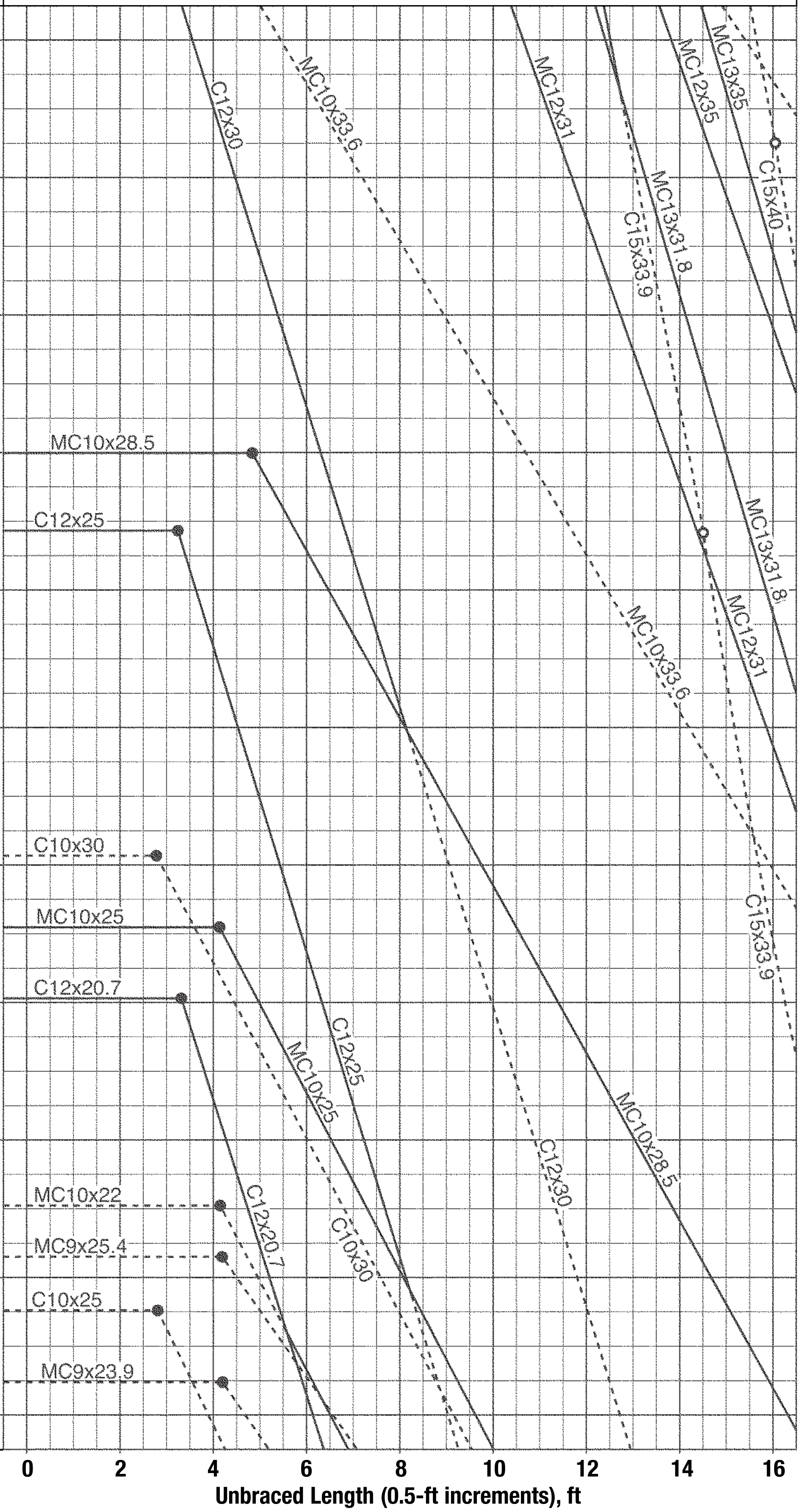


Table 3-11 (continued)
Channels
Available Moment vs. Unbraced Length

$F_y = 36$ ksi
$C_b = 1$

M_n/Ω_b kip-ft ASD	$\phi_b M_n$ kip-ft LRFD
60	90
58	87
56	84
54	81
52	78
50	75
48	72
46	69
44	66
42	63
40	60



F_y = 36 ksi C_b = 1	
M_n/Ω_b	$\phi_b M_n$
kip-ft	kip-ft
ASD	LRFD

Table 3-11 (continued)
Channels
Available Moment vs. Unbraced Length

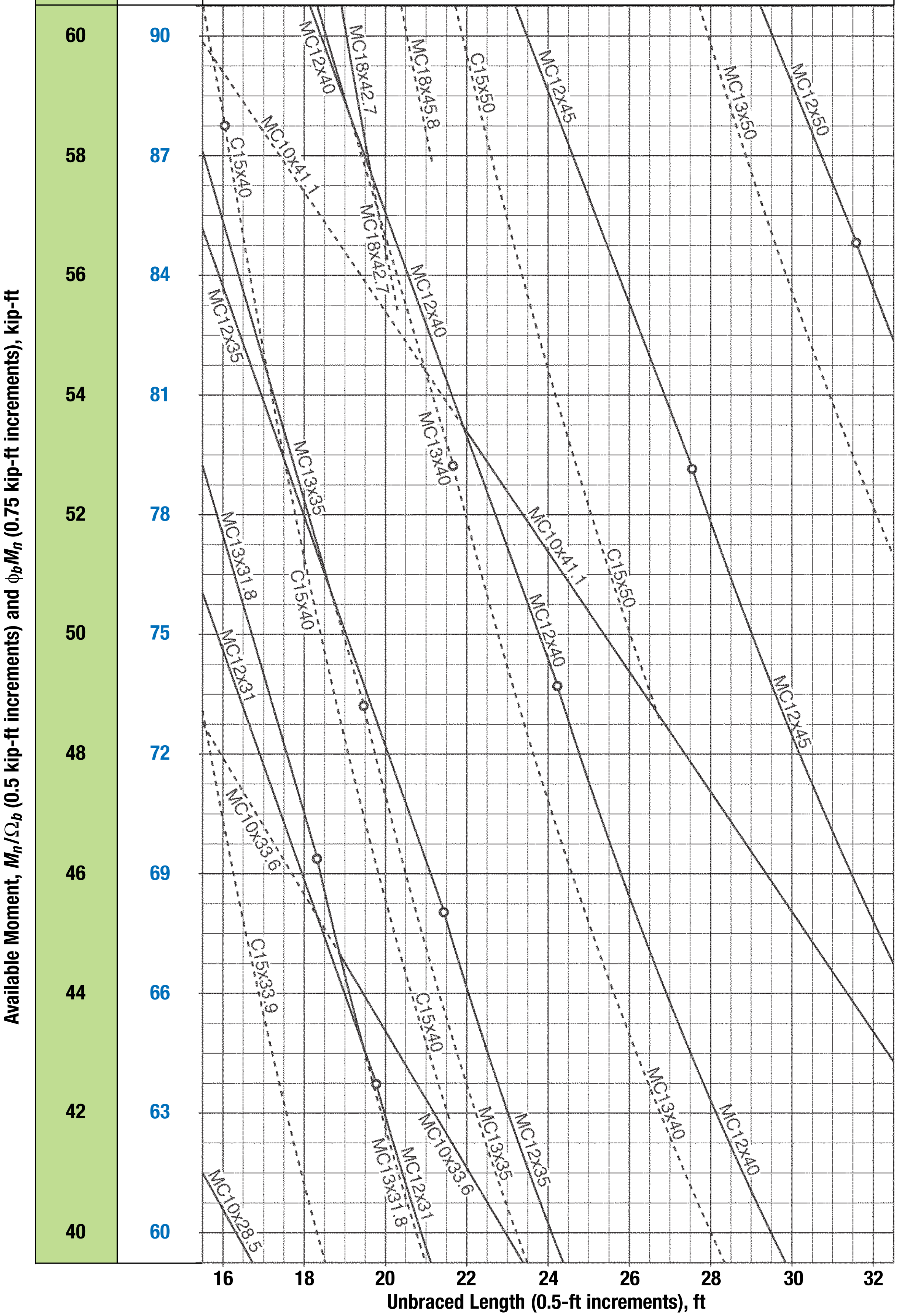
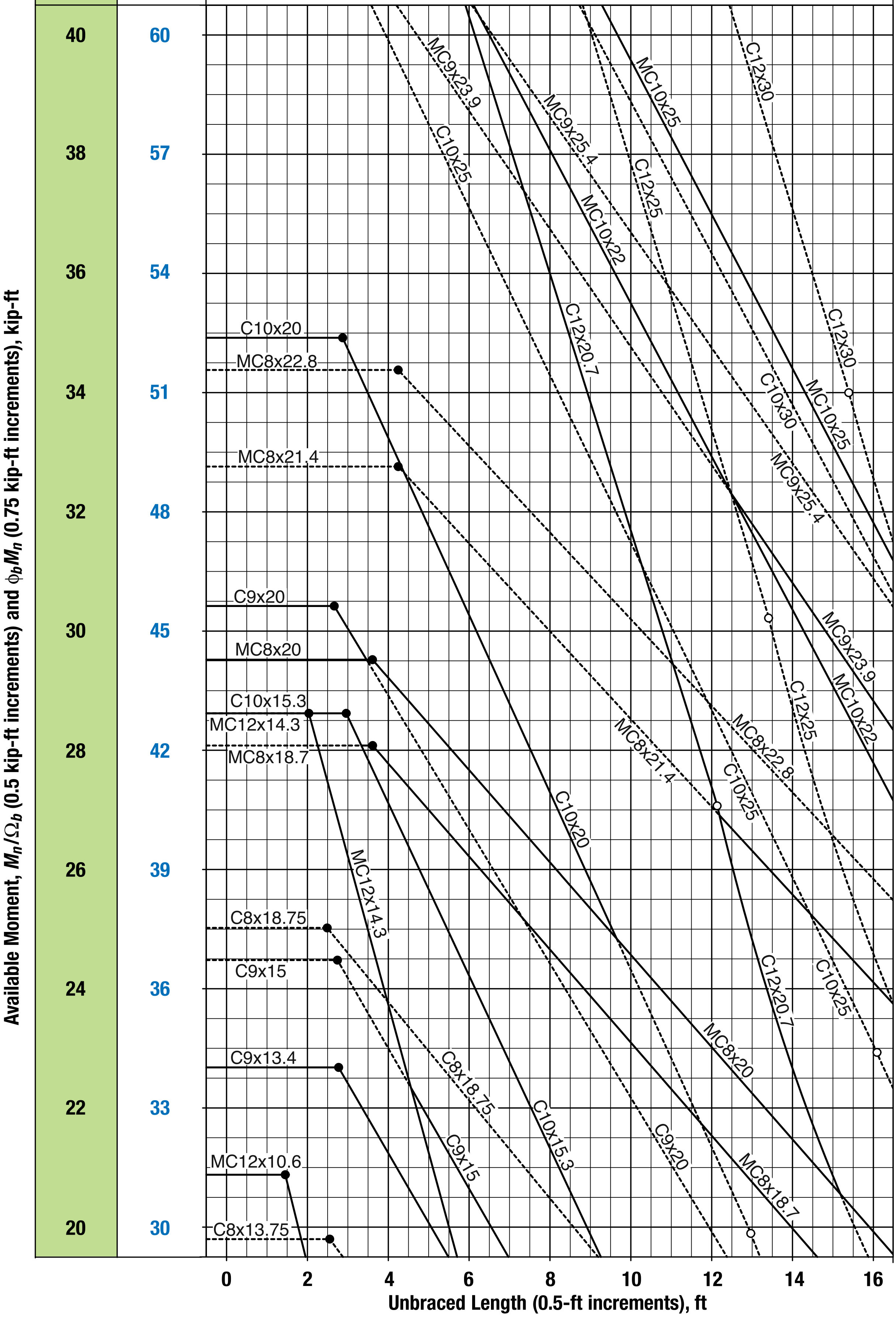


Table 3-11 (continued)
Channels
Available Moment vs. Unbraced Length

$F_y = 36$ ksi
$C_b = 1$



$F_y = 36$ ksi $C_b = 1$	
M_n/Ω_b	$\phi_b M_n$
kip-ft	kip-ft
ASD	LRFD

Table 3-11 (continued)
Channels
Available Moment vs. Unbraced Length

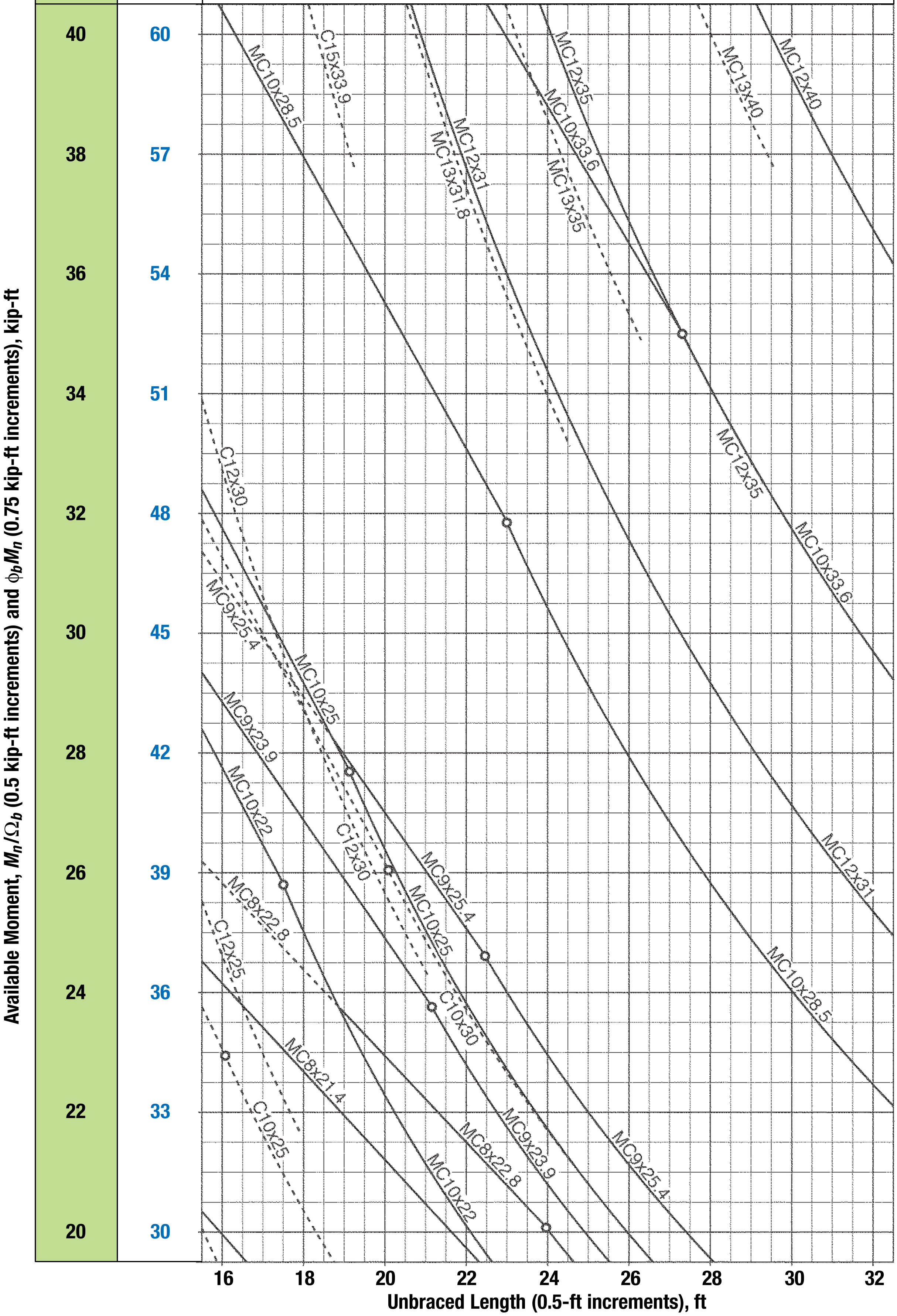
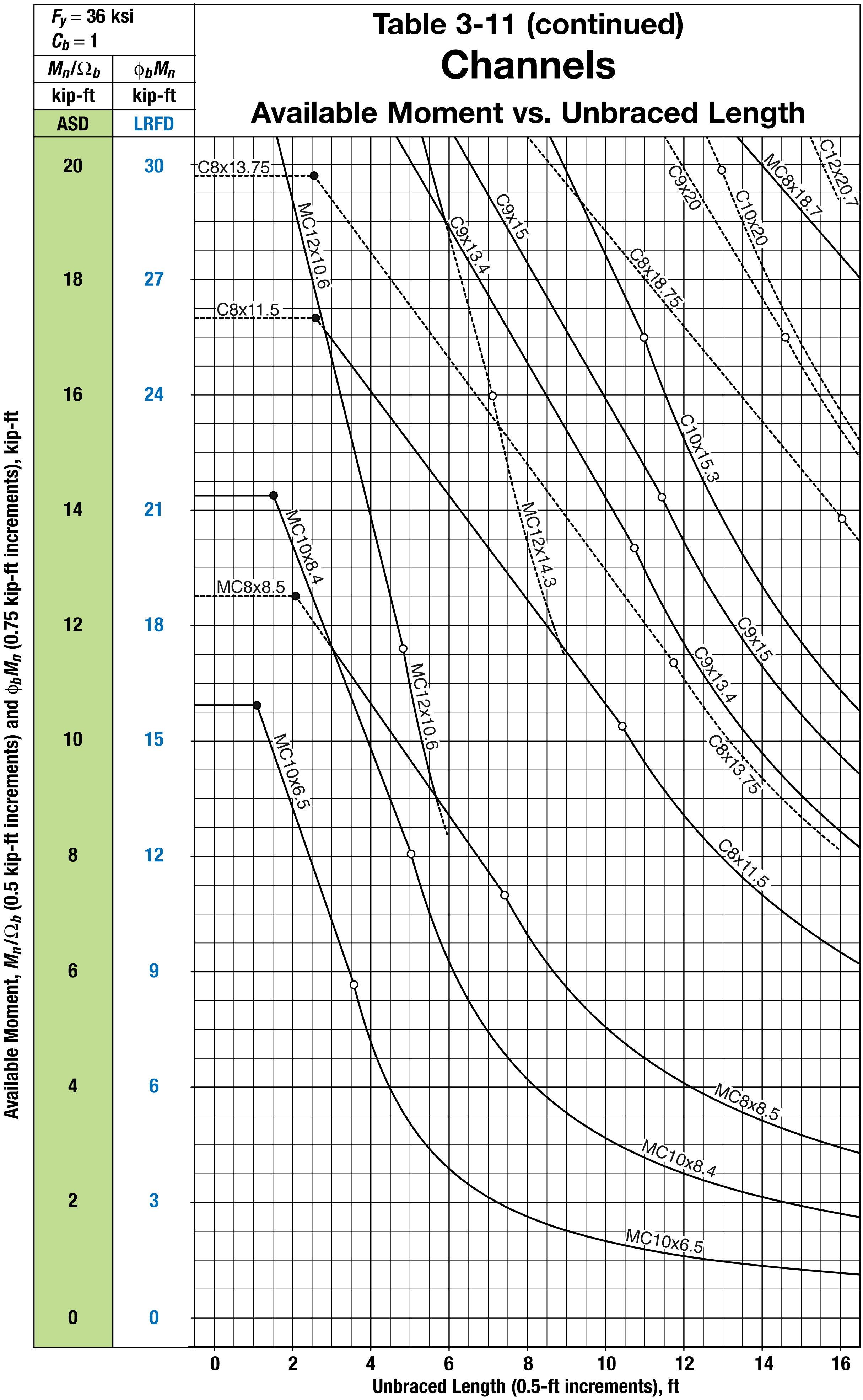
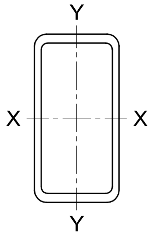


Table 3-11 (continued)
Channels
Available Moment vs. Unbraced Length





HSS20–HSS12

Table 3-12
Available Flexural Strength, kip-ft
Rectangular HSS

F_y = 50 ksi

Shape		X-Axis M_n/Ω_b ASD	X-Axis $\phi_b M_n$ LRFD	Y-Axis M_n/Ω_b ASD	Y-Axis $\phi_b M_n$ LRFD
HSS20×12×	5/8	574	863	371	557
	1/2	469	705	271	407
	3/8	319	480	180	270
	5/16	242	364	139	209
HSS20×8×	5/8	462	694	221	332
	1/2	379	570	162	243
	3/8	292	439	107	160
	5/16	241	363	81.6	123
HSS20×4×	1/2	287	431	66.6	100
	3/8	223	335	44.5	66.9
	5/16	184	277	34.0	51.1
	1/4	141	212	24.1	36.2
HSS18×6×	5/8	337	506	149	224
	1/2	279	420	108	163
	3/8	216	324	72.0	108
	5/16	182	274	55.5	83.4
	1/4	142	213	39.5	59.3
HSS16×12×	5/8	412	619	337	506
	1/2	337	506	254	381
	3/8	232	349	169	254
	5/16	178	267	131	197
HSS16×8×	5/8	322	484	198	297
	1/2	264	398	151	226
	3/8	205	308	100	151
	5/16	173	260	77.7	117
	1/4	124	187	55.9	84.0
HSS16×4×	5/8	232	348	81.1	122
	1/2	193	290	62.0	93.2
	3/8	150	226	41.8	62.8
	5/16	127	192	32.3	48.5
	1/4	102	153	23.1	34.8
	3/16	71.8	108	14.7	22.0
HSS14×10×	5/8	299	450	237	357
	1/2	247	371	195	294
	3/8	190	286	128	192
	5/16	144	217	99.4	149
	1/4	104	157	72.7	109

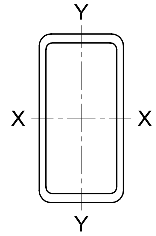
Shape		X-Axis M_n/Ω_b ASD	X-Axis $\phi_b M_n$ LRFD	Y-Axis M_n/Ω_b ASD	Y-Axis $\phi_b M_n$ LRFD
HSS14×6×	5/8	221	333	121	182
	1/2	184	276	101	151
	3/8	143	215	66.6	100
	5/16	121	182	51.8	77.9
	1/4	98.8	149	37.4	56.2
	3/16	65.4	98.3	24.1	36.3
HSS14×4×	5/8	182	274	71.1	107
	1/2	152	229	60.0	90.1
	3/8	119	179	39.8	59.8
	5/16	101	152	31.1	46.7
	1/4	82.8	125	22.5	33.7
	3/16	59.6	89.6	14.4	21.6
HSS12×10×	1/2	197	296	174	261
	3/8	152	229	123	185
	5/16	116	175	94.8	142
	1/4	84.4	127	69.9	105
HSS12×8×	5/8	205	308	154	232
	1/2	170	255	128	193
	3/8	132	199	91.7	138
	5/16	112	168	70.8	106
	1/4	81.7	123	52.0	78.2
	3/16	53.6	80.6	34.2	51.3
HSS12×6×	5/8	172	258	105	158
	1/2	143	215	87.8	132
	3/8	112	168	63.4	95.3
	5/16	95.1	143	49.2	74.0
	1/4	77.6	117	35.9	54.0
	3/16	51.9	78.1	23.4	35.1
HSS12×4×	5/8	138	208	61.1	91.9
	1/2	117	175	52.1	78.4
	3/8	91.6	138	37.9	57.0
	5/16	78.1	117	29.5	44.3
	1/4	63.9	96.0	21.5	32.3
	3/16	47.8	71.9	13.9	20.9
HSS12×3 1/2×	3/8	86.6	130	31.9	48.0
	5/16	73.9	111	24.8	37.3

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

Note: Values are reduced for width-to-thickness criteria, when appropriate. See Table 1-12A for limiting dimensions for compactness.

F_y = 50 ksi

Table 3-12 (continued)
Available Flexural Strength, kip-ft
Rectangular HSS



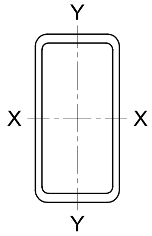
HSS12–HSS8

Shape		X-Axis		Y-Axis	
		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		ASD	LRFD	ASD	LRFD
HSS12×3×	5/16	69.6	105	20.4	30.6
	1/4	57.1	85.9	15.0	22.6
	3/16	42.6	64.1	9.65	14.5
HSS12×2×	5/16	61.1	91.9	11.9	17.9
	1/4	50.1	75.4	8.87	13.3
	3/16	37.6	56.5	5.78	8.68
HSS10×8×	5/8	155	233	133	200
	1/2	129	195	111	167
	3/8	101	152	86.8	131
	5/16	85.8	129	67.5	101
	1/4	63.2	95.1	49.2	73.9
	3/16	41.7	62.7	32.8	49.3
HSS10×6×	5/8	128	192	89.3	134
	1/2	107	161	75.1	113
	3/8	84.3	127	59.1	88.9
	5/16	71.9	108	46.5	69.9
	1/4	58.9	88.5	33.9	51.0
	3/16	39.7	59.7	22.4	33.7
HSS10×5×	3/8	75.8	114	46.7	70.1
	5/16	64.9	97.5	36.8	55.3
	1/4	53.1	79.9	26.9	40.5
	3/16	40.7	61.1	17.7	26.6
HSS10×4×	5/8	101	151	51.4	77.3
	1/2	85.1	128	43.9	66.0
	3/8	67.4	101	34.9	52.5
	5/16	57.6	86.6	27.7	41.7
	1/4	47.4	71.3	20.3	30.5
	3/16	36.4	54.8	13.4	20.1
	1/8	21.6	32.4	7.26	10.9
HSS10×3 1/2×	1/2	79.6	120	36.7	55.1
	3/8	63.1	94.9	29.4	44.3
	5/16	54.1	81.4	23.4	35.1
	1/4	44.7	67.1	17.1	25.7
	3/16	34.2	51.4	11.3	16.9
	1/8	21.8	32.8	6.11	9.18

Shape		X-Axis		Y-Axis	
		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		ASD	LRFD	ASD	LRFD
HSS10×3×	3/8	59.1	88.9	24.3	36.5
	5/16	50.6	76.1	19.2	28.8
	1/4	41.7	62.6	14.1	21.2
	3/16	31.9	48.0	9.27	13.9
	1/8	20.4	30.7	5.01	7.52
HSS10×2×	3/8	50.6	76.1	14.4	21.6
	5/16	43.7	65.6	11.3	17.0
	1/4	35.9	54.0	8.34	12.5
	3/16	27.7	41.6	5.56	8.35
	1/8	17.6	26.4	3.00	4.41
HSS9×7×	5/8	121	181	101	152
	1/2	101	152	84.8	128
	3/8	79.3	119	66.6	100
	5/16	67.6	102	55.9	84.0
	1/4	55.4	83.2	39.8	59.9
	3/16	34.6	52.0	26.7	40.1
HSS9×5×	5/8	96.1	144	63.1	94.9
	1/2	81.1	122	53.6	80.6
	3/8	64.1	96.4	42.7	64.1
	5/16	54.9	82.5	35.8	53.8
	1/4	45.2	67.9	25.9	38.9
	3/16	34.4	51.8	17.2	25.9
HSS9×3×	1/2	61.4	92.3	26.9	40.5
	3/8	49.2	73.9	22.0	33.0
	5/16	42.2	63.4	18.7	28.1
	1/4	34.9	52.5	13.5	20.4
	3/16	26.9	40.5	9.02	13.6
HSS8×6×	5/8	90.1	135	73.6	111
	1/2	76.1	114	62.1	93.4
	3/8	60.1	90.4	49.4	74.3
	5/16	51.4	77.3	42.2	63.4
	1/4	42.2	63.4	31.8	47.8
	3/16	28.9	43.5	21.1	31.7

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

Note: Values are reduced for width-to-thickness criteria, when appropriate. See Table 1-12A for limiting dimensions for compactness.



HSS8–HSS5

Table 3-12 (continued)
Available Flexural Strength, kip-ft
Rectangular HSS

F_y = 50 ksi

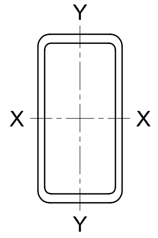
Shape		X-Axis M_n/Ω_b ASD	X-Axis $\phi_b M_n$ LRFD	Y-Axis M_n/Ω_b ASD	Y-Axis $\phi_b M_n$ LRFD
HSS8×4×	5/8	68.4	103	41.4	62.3
	1/2	58.6	88.1	35.7	53.6
	3/8	46.9	70.5	28.7	43.1
	5/16	40.2	60.4	24.7	37.2
	1/4	33.2	49.9	18.9	28.4
	3/16	25.4	38.3	12.5	18.8
	1/8	15.4	23.1	6.94	10.4
HSS8×3×	1/2	49.9	75.0	24.1	36.2
	3/8	40.2	60.4	19.7	29.6
	5/16	34.7	52.1	17.1	25.7
	1/4	28.7	43.1	13.1	19.7
	3/16	22.1	33.3	8.70	13.1
	1/8	14.9	22.4	4.80	7.21
HSS8×2×	3/8	33.4	50.3	11.5	17.3
	5/16	28.9	43.5	10.1	15.2
	1/4	24.2	36.3	7.78	11.7
	3/16	18.7	28.2	5.21	7.83
	1/8	12.6	19.0	2.86	4.30
HSS7×5×	1/2	54.6	82.1	43.2	64.9
	3/8	43.7	65.6	34.4	51.8
	5/16	37.4	56.3	29.7	44.6
	1/4	30.9	46.5	24.5	36.9
	3/16	23.8	35.7	15.9	23.8
	1/8	13.0	19.5	9.05	13.6
HSS7×4×	1/2	46.9	70.5	31.4	47.3
	3/8	37.7	56.6	25.4	38.3
	5/16	32.7	49.1	22.0	33.1
	1/4	26.9	40.5	18.3	27.5
	3/16	20.8	31.2	11.9	17.9
	1/8	12.6	19.0	6.73	10.1
HSS7×3×	1/2	39.4	59.3	21.1	31.7
	3/8	31.9	48.0	17.3	26.1
	5/16	27.7	41.6	15.1	22.7
	1/4	23.0	34.6	12.6	19.0
	3/16	17.8	26.8	8.30	12.5
	1/8	12.3	18.5	4.64	6.98
HSS7×2×	1/4	19.1	28.7	7.53	11.3
	3/16	14.8	22.3	4.97	7.47
	1/8	10.3	15.5	2.79	4.19
HSS6×5×	1/2	42.9	64.5	37.9	57.0
	3/8	34.4	51.8	30.4	45.8
	5/16	29.7	44.6	26.2	39.4
	1/4	24.6	37.0	21.8	32.7
	3/16	19.0	28.6	15.3	23.0
	1/8	10.5	15.7	8.68	13.0
HSS6×4×	1/2	36.4	54.8	27.4	41.3
	3/8	29.7	44.6	22.3	33.5
	5/16	25.7	38.6	19.3	29.1
	1/4	21.3	32.0	16.1	24.2
	3/16	16.5	24.8	11.4	17.2
	1/8	10.1	15.2	6.46	9.71
HSS6×3×	1/2	30.2	45.4	18.2	27.3
	3/8	24.7	37.1	15.0	22.6
	5/16	21.5	32.3	13.1	19.8
	1/4	17.9	27.0	11.0	16.5
	3/16	13.9	21.0	7.91	11.9
	1/8	9.66	14.5	4.46	6.71
HSS6×2×	3/8	19.8	29.7	8.63	13.0
	5/16	17.3	26.1	7.66	11.5
	1/4	14.6	21.9	6.51	9.79
	3/16	11.4	17.2	4.71	7.08
	1/8	7.96	12.0	2.67	4.02
HSS5×4×	1/2	27.2	40.9	23.3	35.1
	3/8	22.4	33.6	19.1	28.8
	5/16	19.4	29.2	16.6	25.0
	1/4	16.2	24.3	13.9	20.9
	3/16	12.6	18.9	10.8	16.3
	1/8	7.85	11.8	6.11	9.18

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

Note: Values are reduced for width-to-thickness criteria, when appropriate. See Table 1-12A for limiting dimensions for compactness.

F_y = 50 ksi

Table 3-12 (continued)
Available Flexural Strength, kip-ft
Rectangular HSS



HSS5–HSS2

Shape		X-Axis		Y-Axis	
		M_n/Ω_b	$\phi_b M_n$	M_n/Ω_b	$\phi_b M_n$
		ASD	LRFD	ASD	LRFD
HSS5×3×	1/2	22.0	33.1	15.2	22.9
	3/8	18.3	27.5	12.7	19.1
	5/16	16.0	24.1	11.2	16.8
	1/4	13.4	20.2	9.41	14.1
	3/16	10.5	15.8	7.39	11.1
	1/8	7.31	11.0	4.22	6.35
HSS5×2½×	1/4	12.1	18.1	7.36	11.1
	3/16	9.46	14.2	5.81	8.74
	1/8	6.61	9.94	3.34	5.02
HSS5×2×	3/8	14.2	21.4	7.19	10.8
	5/16	12.6	18.9	6.41	9.64
	1/4	10.7	16.0	5.49	8.25
	3/16	8.41	12.6	4.37	6.56
	1/8	5.91	8.89	2.51	3.78
HSS4×3×	3/8	12.8	19.2	10.4	15.7
	5/16	11.3	16.9	9.21	13.8
	1/4	9.51	14.3	7.78	11.7
	3/16	7.49	11.3	6.14	9.23
	1/8	5.26	7.91	3.95	5.94
HSS4×2½×	3/8	11.2	16.8	7.98	12.0
	5/16	9.91	14.9	7.11	10.7
	1/4	8.43	12.7	6.06	9.11
	3/16	6.66	10.0	4.82	7.24
	1/8	4.69	7.05	3.11	4.67
HSS4×2×	3/8	9.58	14.4	5.76	8.66
	5/16	8.56	12.9	5.19	7.80
	1/4	7.34	11.0	4.47	6.71
	3/16	5.84	8.78	3.57	5.36
	1/8	4.14	6.23	2.34	3.52
HSS3½×2½×	3/8	8.96	13.5	7.04	10.6
	5/16	7.99	12.0	6.30	9.47
	1/4	6.83	10.3	5.39	8.11
	3/16	5.43	8.16	4.30	6.47
	1/8	3.84	5.78	3.04	4.57
HSS3½×2×	1/4	5.89	8.85	3.94	5.93
	3/16	4.72	7.09	3.17	4.76
	1/8	3.34	5.03	2.27	3.41
HSS3½×1½×	1/4	4.94	7.43	2.64	3.98
	3/16	3.99	6.00	2.16	3.25
	1/8	2.87	4.31	1.57	2.35
HSS3×2½×	5/16	6.26	9.41	5.49	8.25
	1/4	5.39	8.10	4.74	7.13
	3/16	4.32	6.49	3.79	5.70
	1/8	3.07	4.61	2.72	4.09
HSS3×2×	5/16	5.26	7.91	3.94	5.93
	1/4	4.57	6.86	3.44	5.18
	3/16	3.69	5.55	2.79	4.20
	1/8	2.64	3.98	2.00	3.01
HSS3×1½×	1/4	3.77	5.66	2.27	3.42
	3/16	3.09	4.65	1.88	2.82
	1/8	2.23	3.36	1.37	2.06
HSS3×1×	3/16	2.47	3.71	1.08	1.62
	1/8	1.82	2.73	0.811	1.22
HSS2½×2×	1/4	3.42	5.14	2.92	4.39
	3/16	2.79	4.20	2.39	3.59
	1/8	2.02	3.03	1.73	2.60
HSS2½×1½×	1/4	2.77	4.16	1.91	2.87
	3/16	2.28	3.43	1.59	2.39
	1/8	1.67	2.52	1.17	1.76
HSS2½×1×	3/16	1.78	2.67	0.898	1.35
	1/8	1.33	2.00	0.684	1.03
HSS2¼×2×	3/16	2.38	3.57	2.19	3.29
	1/8	1.73	2.60	1.59	2.40
HSS2×1½×	3/16	1.59	2.40	1.30	1.95
	1/8	1.19	1.78	0.971	1.46
HSS2×1×	3/16	1.20	1.80	0.719	1.08
	1/8	0.913	1.37	0.556	0.836

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

Note: Values are reduced for width-to-thickness criteria, when appropriate. See Table 1-12A for limiting dimensions for compactness.

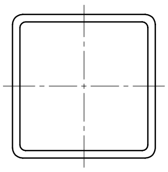


Table 3-13
Available Flexural Strength, kip-ft
Square HSS

HSS16–HSS2

F_y = 50 ksi

Shape		M_n/Ω_b ASD	$\phi_b M_n$ LRFD
HSS16×16×	5/8	499	750
	1/2	372	559
	3/8	247	371
	5/16	193	291
HSS14×14×	5/8	377	566
	1/2	309	464
	3/8	198	297
	5/16	155	234
HSS12×12×	5/8	272	409
	1/2	224	336
	3/8	156	235
	5/16	120	181
	1/4	88.9	134
	3/16	59.5	89.4
HSS10×10×	5/8	183	275
	1/2	151	228
	3/8	118	177
	5/16	90.9	137
	1/4	65.8	98.9
	3/16	44.2	66.5
HSS9×9×	5/8	145	218
	1/2	121	182
	3/8	94.3	142
	5/16	78.6	118
	1/4	55.3	83.2
	3/16	37.3	56.1
	1/8	21.4	32.1
HSS8×8×	5/8	112	168
	1/2	93.6	141
	3/8	73.4	110
	5/16	62.6	94.1
	1/4	46.7	70.2
	3/16	30.8	46.3
	1/8	17.7	26.6
HSS7×7×	5/8	82.6	124
	1/2	69.6	105
	3/8	55.1	82.9
	5/16	47.2	70.9
	1/4	38.7	58.1
	3/16	24.7	37.1
	1/8	14.2	21.4
HSS6×6×	5/8	57.9	87.0
	1/2	49.4	74.3
	3/8	39.4	59.3
	5/16	33.9	51.0
	1/4	27.9	42.0
	3/16	19.5	29.3
	1/8	11.1	16.7
HSS5 1/2×5 1/2×	3/8	32.7	49.1
	5/16	28.2	42.4
	1/4	23.3	35.0
	3/16	17.3	26.0
	1/8	9.58	14.4
HSS5×5×	1/2	32.7	49.1
	3/8	26.4	39.8
	5/16	22.9	34.4
	1/4	19.0	28.5
	3/16	14.7	22.1
	1/8	8.20	12.3
HSS4 1/2×4 1/2×	1/2	25.4	38.3
	3/8	20.9	31.4
	5/16	18.1	27.3
	1/4	15.1	22.7
	3/16	11.8	17.7
	1/8	6.88	10.3
HSS4×4×	1/2	19.2	28.9
	3/8	15.9	24.0
	5/16	13.9	21.0
	1/4	11.7	17.6
	3/16	9.16	13.8
	1/8	5.79	8.70
HSS3 1/2×3 1/2×	3/8	11.7	17.6
	5/16	10.3	15.5
	1/4	8.73	13.1
	3/16	6.89	10.4
	1/8	4.79	7.21
HSS3×3×	3/8	8.11	12.2
	5/16	7.24	10.9
	1/4	6.19	9.30
	3/16	4.92	7.39
	1/8	3.49	5.25
HSS2 1/2×2 1/2×	5/16	4.69	7.05
	1/4	4.07	6.11
	3/16	3.29	4.95
	1/8	2.36	3.55
HSS2 1/4×2 1/4×	1/4	3.19	4.80
	3/16	2.59	3.90
	1/8	1.88	2.83
HSS2×2×	1/4	2.41	3.62
	3/16	1.99	2.99
	1/8	1.46	2.19

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

Note: Values are reduced for width-to-thickness criteria, when appropriate. See Table 1-12A for limiting dimensions for compactness.

F_y = 46 ksi

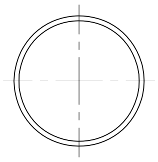
Table 3-14
Available Flexural Strength, kip-ft
Round HSS

HSS20.000–HSS6.625

Shape		M_n/Ω_b ASD	$\phi_b M_n$ LRFD	Shape		M_n/Ω_b ASD	$\phi_b M_n$ LRFD
HSS20.000×	0.500	406	611	HSS8.625×	0.625	86.5	130
	0.375[f]	294	442		0.500	71.2	107
					0.375	54.9	82.5
HSS18.000×	0.500	328	493		0.322	47.7	71.8
	0.375[f]	242	363		0.250	37.6	56.6
					0.188[f]	27.8	41.8
HSS16.000×	0.625	317	476				
	0.500	257	386	HSS7.625×	0.375	42.5	63.8
	0.438	227	342		0.328	37.6	56.6
	0.375	194	292				
	0.312[f]	158	237	HSS7.500×	0.500	52.8	79.4
	0.250[f]	123	184		0.375	41.1	61.8
					0.312	34.7	52.1
HSS14.000×	0.625	241	362		0.250	28.2	42.4
	0.500	196	294		0.188	21.4	32.2
	0.375	149	225				
	0.312	123	185	HSS7.000×	0.500	45.7	68.7
	0.250[f]	95.5	144		0.375	35.6	53.5
					0.312	30.1	45.2
HSS12.750×	0.500	161	242		0.250	24.6	36.9
	0.375	123	185		0.188	18.6	28.0
	0.250[f]	80.4	121		0.125[f]	11.9	17.9
HSS10.750×	0.500	113	170	HSS6.875×	0.500	43.8	65.9
	0.375	86.8	130		0.375	34.2	51.4
	0.250	58.5	87.9		0.312	28.9	43.5
					0.250	23.6	35.5
HSS10.000×	0.625	118	178		0.188	17.9	26.9
	0.500	97.1	146				
	0.375	74.6	112	HSS6.625×	0.500	40.6	61.1
	0.312	62.9	94.5		0.432	35.8	53.8
	0.250	51.0	76.6		0.375	31.7	47.6
	0.188[f]	36.7	55.2		0.312	26.9	40.4
					0.280	24.1	36.2
HSS9.625×	0.500	89.5	135		0.250	21.9	32.8
	0.375	68.9	104		0.188	16.6	25.0
	0.312	58.3	87.6		0.125[f]	10.7	16.1
	0.250	47.3	71.1				
	0.188[f]	34.1	51.3				

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[f] Shape exceeds compact limit for flexure with F_y = 46 ksi; tabulated values have been adjusted accordingly.



HSS6.000–HSS1.660

Table 3-14 (continued)
Available Flexural Strength, kip-ft
Round HSS

F_y = 46 ksi

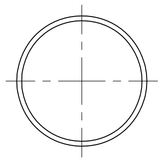
Shape		M_n/Ω_b ASD	$\phi_b M_n$ LRFD
HSS6.000×	0.500	32.8	49.3
	0.375	25.7	38.6
	0.312	21.8	32.7
	0.280	19.7	29.6
	0.250	17.8	26.7
	0.188	13.6	20.4
	0.125[f]	8.91	13.4
HSS5.563×	0.500	27.8	41.7
	0.375	21.8	32.8
	0.258	15.6	23.5
	0.188	11.6	17.4
	0.134	8.38	12.6
HSS5.500×	0.500	27.1	40.7
	0.375	21.3	32.0
	0.258	15.2	22.9
HSS5.000×	0.500	22.0	33.1
	0.375	17.4	26.1
	0.312	14.8	22.3
	0.258	12.5	18.8
	0.250	12.2	18.3
	0.188	9.30	14.0
	0.125	6.36	9.56
HSS4.500×	0.375	13.8	20.8
	0.337	12.6	19.0
	0.237	9.25	13.9
	0.188	7.48	11.2
	0.125	5.12	7.69
HSS4.000×	0.313	9.20	13.8
	0.250	7.60	11.4
	0.237	7.23	10.9
	0.226	6.93	10.4
	0.220	6.79	10.2
	0.188	5.85	8.80
	0.125	4.02	6.04
HSS3.500×	0.313	6.89	10.4
	0.300	6.66	10.0
	0.250	5.72	8.59
	0.216	5.03	7.56
	0.203	4.75	7.14
	0.188	4.43	6.66
	0.125	3.05	4.59
HSS3.000×	0.250	4.11	6.18
	0.216	3.63	5.45
	0.203	3.44	5.18
	0.188	3.19	4.80
	0.152	2.64	3.97
	0.134	2.36	3.55
	0.125	2.22	3.33
HSS2.875×	0.250	3.74	5.62
	0.203	3.14	4.73
	0.188	2.92	4.38
	0.125	2.03	3.05
HSS2.500×	0.250	2.75	4.14
	0.188	2.16	3.25
	0.125	1.51	2.28
HSS2.375×	0.250	2.46	3.69
	0.218	2.20	3.31
	0.188	1.94	2.92
	0.154	1.64	2.46
	0.125	1.36	2.04
HSS1.900×	0.188	1.19	1.79
	0.145	0.966	1.45
	0.120	0.817	1.23
HSS1.660×	0.140	0.700	1.05

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[f] Shape exceeds compact limit for flexure with F_y = 46 ksi; tabulated values have been adjusted accordingly.

F_y = 35 ksi

Table 3-15
Available Flexural Strength, kip-ft
Pipe



Shape	M_n/Ω_b	$\phi_b M_n$	Shape	M_n/Ω_b	$\phi_b M_n$
	ASD	LRFD		ASD	LRFD
Pipe 12 x-Strong	123	184	Pipe $2\frac{1}{2}$ xx-Strong	5.08	7.64
Pipe 12 Std.	93.8	141	Pipe $2\frac{1}{2}$ x-Strong	3.09	4.64
Pipe 10 x-Strong	86.0	129	Pipe $2\frac{1}{2}$ Std.	2.39	3.59
Pipe 10 Std.	64.4	96.8	Pipe 2 xx-Strong	2.79	4.19
Pipe 8 xx-Strong	87.2	131	Pipe 2 x-Strong	1.68	2.53
Pipe 8 x-Strong	54.1	81.4	Pipe 2 Std.	1.25	1.87
Pipe 8 Std.	36.3	54.6	Pipe $1\frac{1}{2}$ x-Strong	0.958	1.44
Pipe 6 xx-Strong	47.9	72.0	Pipe $1\frac{1}{2}$ Std.	0.736	1.11
Pipe 6 x-Strong	27.3	41.0	Pipe $1\frac{1}{4}$ x-Strong	0.686	1.03
Pipe 6 Std.	18.5	27.8	Pipe $1\frac{1}{4}$ Std.	0.533	0.801
Pipe 5 xx-Strong	29.1	43.7	Pipe 1 x-Strong	0.385	0.579
Pipe 5 x-Strong	16.6	24.9	Pipe 1 Std.	0.308	0.463
Pipe 5 Std.	11.9	17.9	Pipe $\frac{3}{4}$ x-Strong	0.207	0.311
Pipe 4 xx-Strong	16.6	24.9	Pipe $\frac{3}{4}$ Std.	0.164	0.247
Pipe 4 x-Strong	9.65	14.5	Pipe $\frac{1}{2}$ x-Strong	0.120	0.180
Pipe 4 Std.	7.07	10.6	Pipe $\frac{1}{2}$ Std.	0.0969	0.146
Pipe $3\frac{1}{2}$ x-Strong	7.11	10.7			
Pipe $3\frac{1}{2}$ Std.	5.30	7.96			
Pipe 3 xx-Strong	8.55	12.8			
Pipe 3 x-Strong	5.08	7.64			
Pipe 3 Std.	3.83	5.75			

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

Table 3-16a
Available Shear Stress, ksi
Tension Field Action NOT Included

F_y = 36 ksi

$\Omega_v = 1.67$	$\phi_v = 0.90$
$\dfrac{V_n}{\Omega_v A_w}$	$\dfrac{\phi_v V_n}{A_w}$
ASD	LRFD

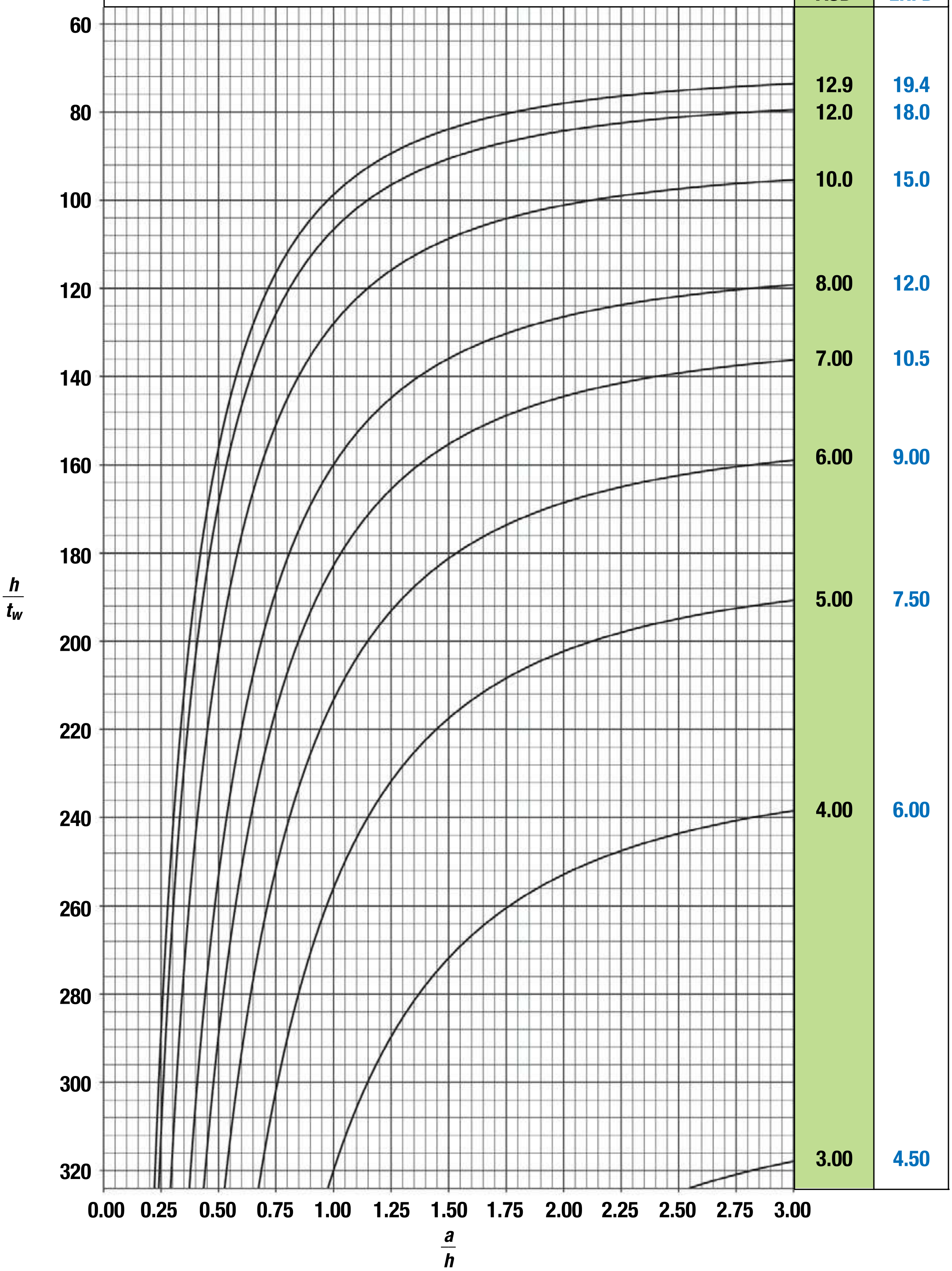


Table 3-16b
Available Shear Stress, ksi
Tension Field Action Included
When $2A_w/(A_{fc}+A_{ft}) \le 2.5$, $h/b_{fc} \le 6.0$ and $h/b_{ft} \le 6.0$

F_y = 36 ksi

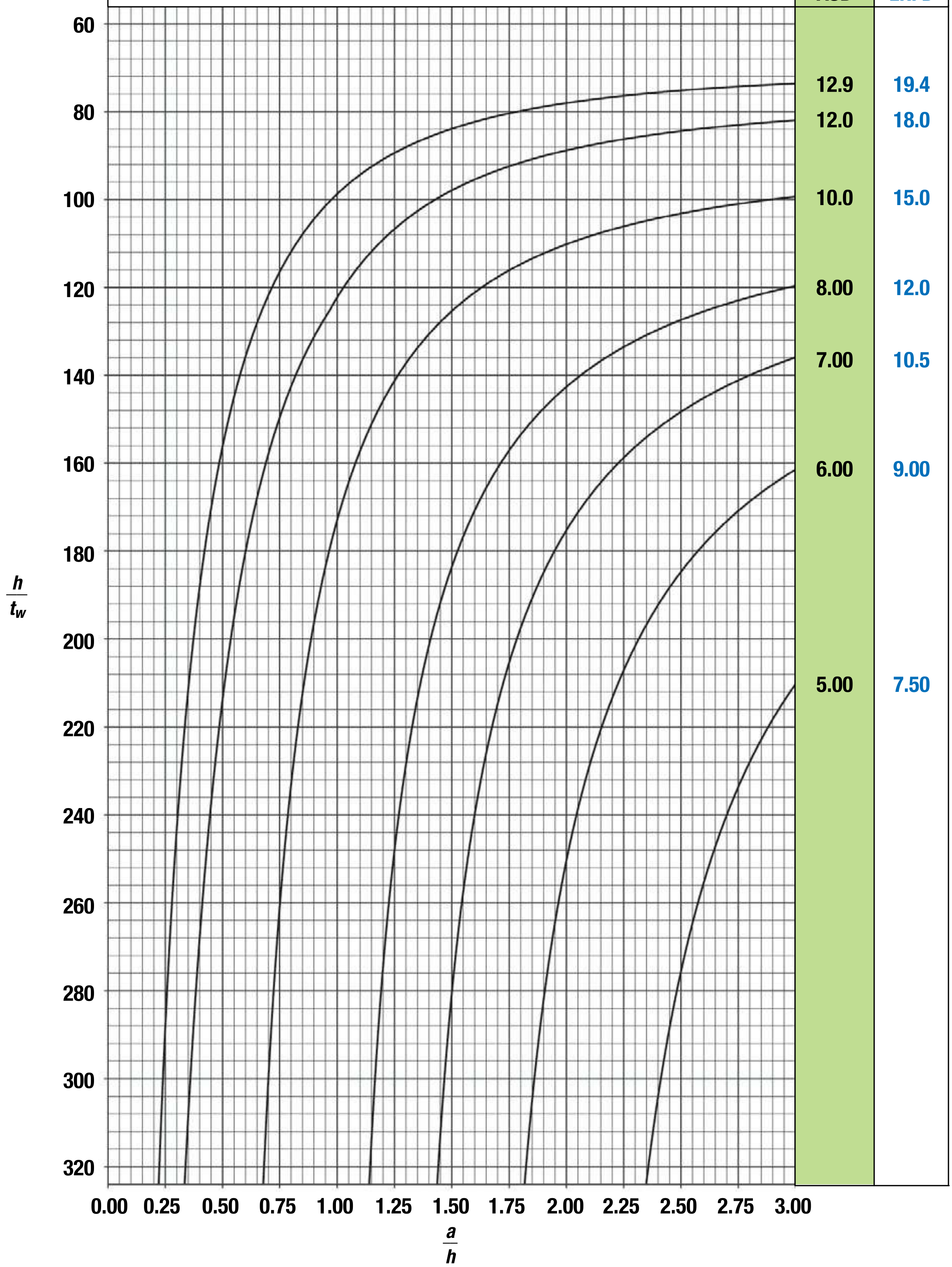


Table 3-16c
Available Shear Stress, ksi
Tension Field Action Included
When $2A_w/(A_{fc} + A_{ft}) > 2.5$ or $h/b_{fc} > 6.0$ or $h/b_{ft} > 6.0$

$F_y = 36$ ksi

$\Omega_v = 1.67$	$\phi_v = 0.90$
$\dfrac{V_n}{\Omega_v A_w}$	$\dfrac{\phi_v V_n}{A_w}$
ASD	LRFD

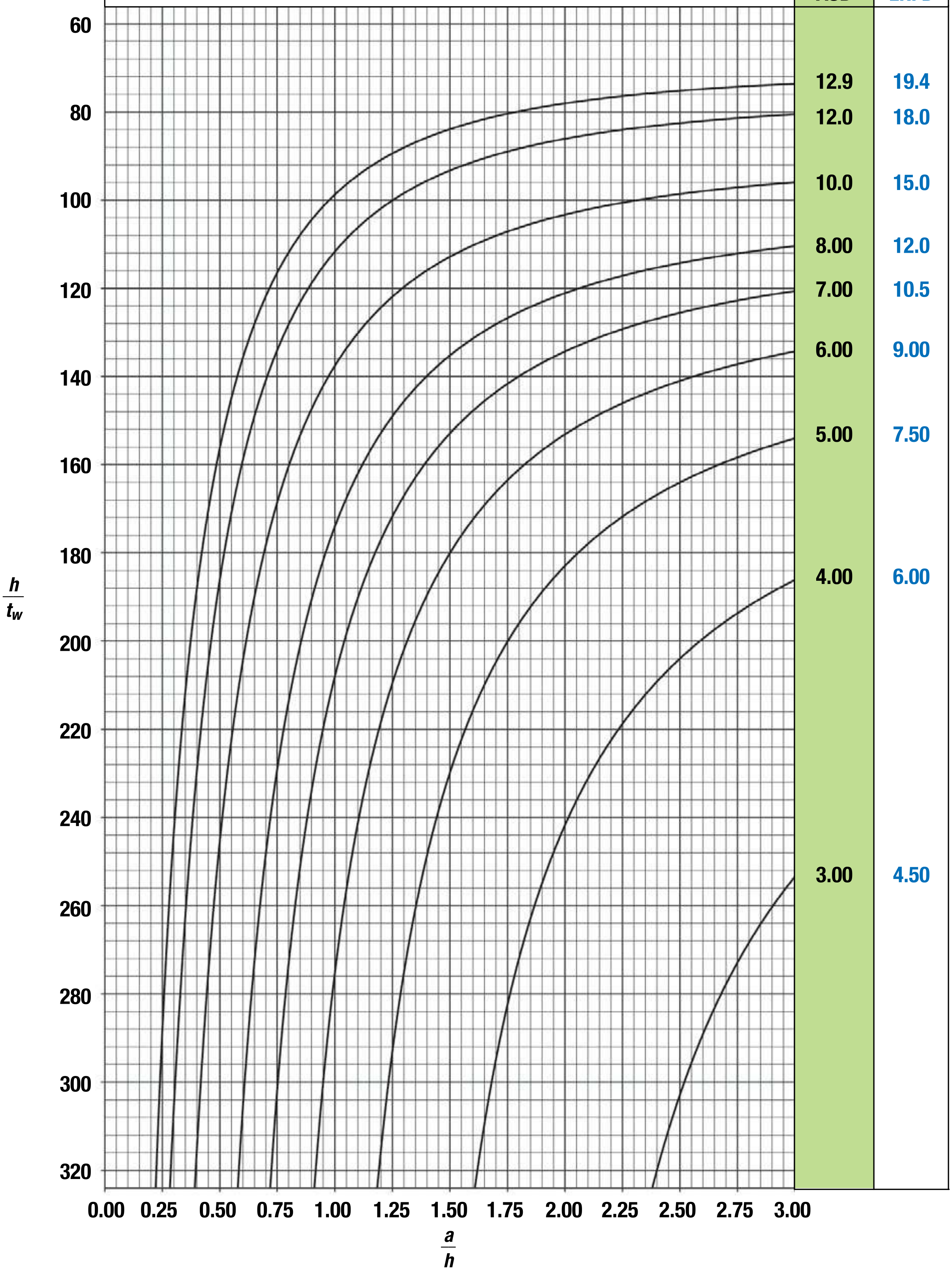


Table 3-17a
Available Shear Stress, ksi
Tension Field Action NOT Included

F_y = 50 ksi

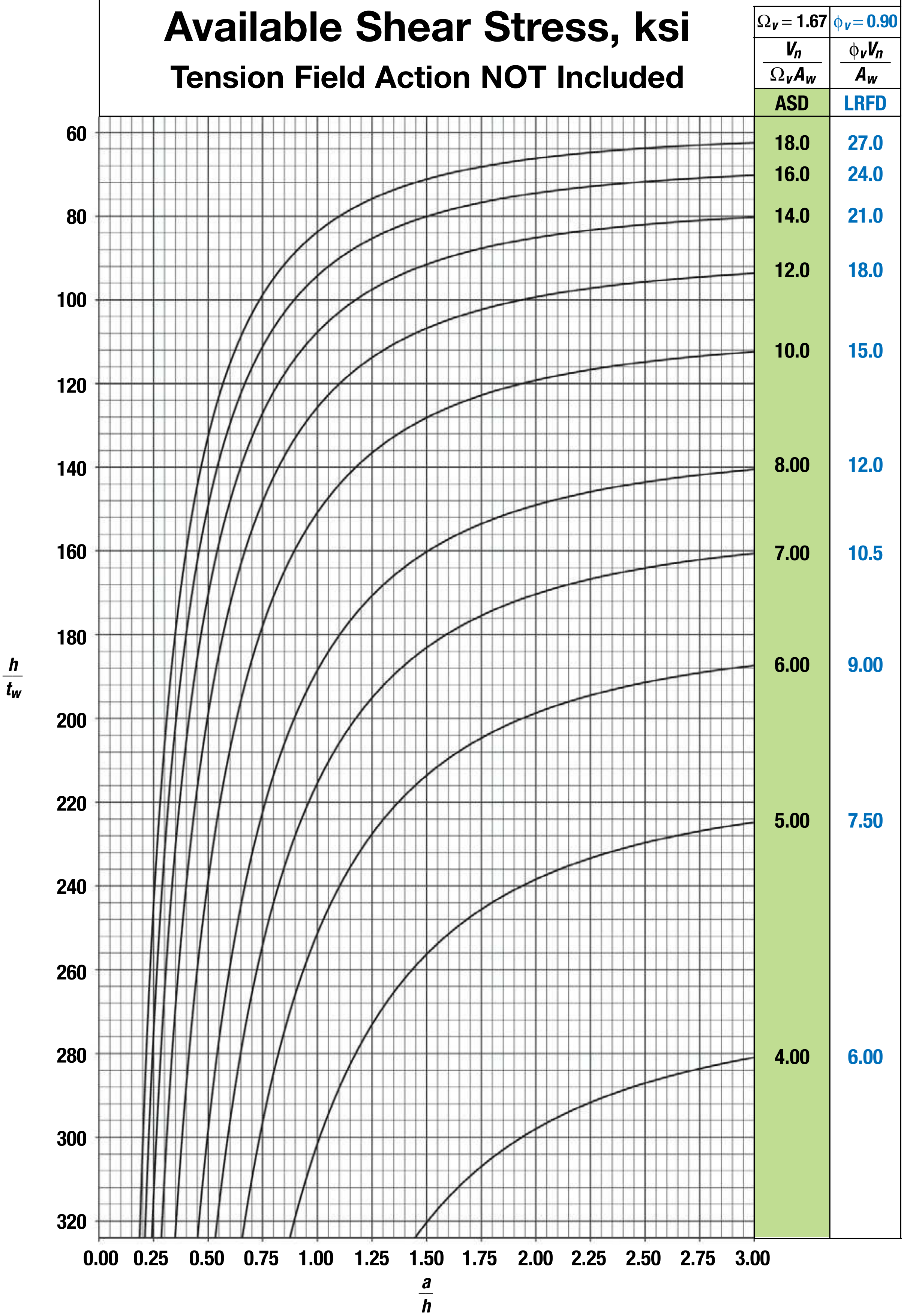


Table 3-17b
Available Shear Stress, ksi
Tension Field Action Included
When $2A_w/(A_{fc}+A_{ft}) \le 2.5$, $h/b_{fc} \le 6.0$ and $h/b_{ft} \le 6.0$

$F_y = 50$ ksi

$\Omega_v = 1.67$	$\phi_v = 0.90$
$\dfrac{V_n}{\Omega_v A_w}$	$\dfrac{\phi_v V_n}{A_w}$
ASD	LRFD

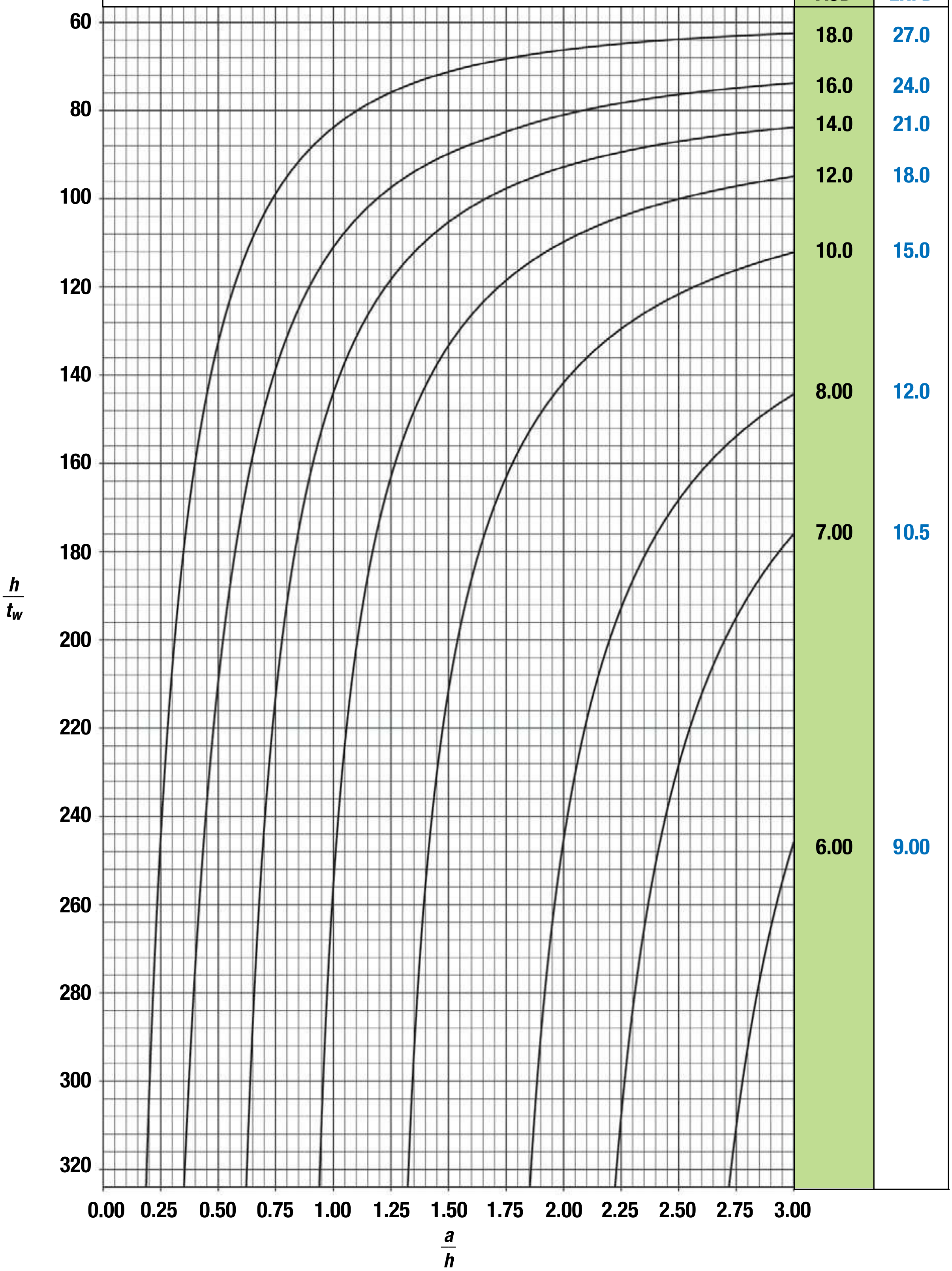
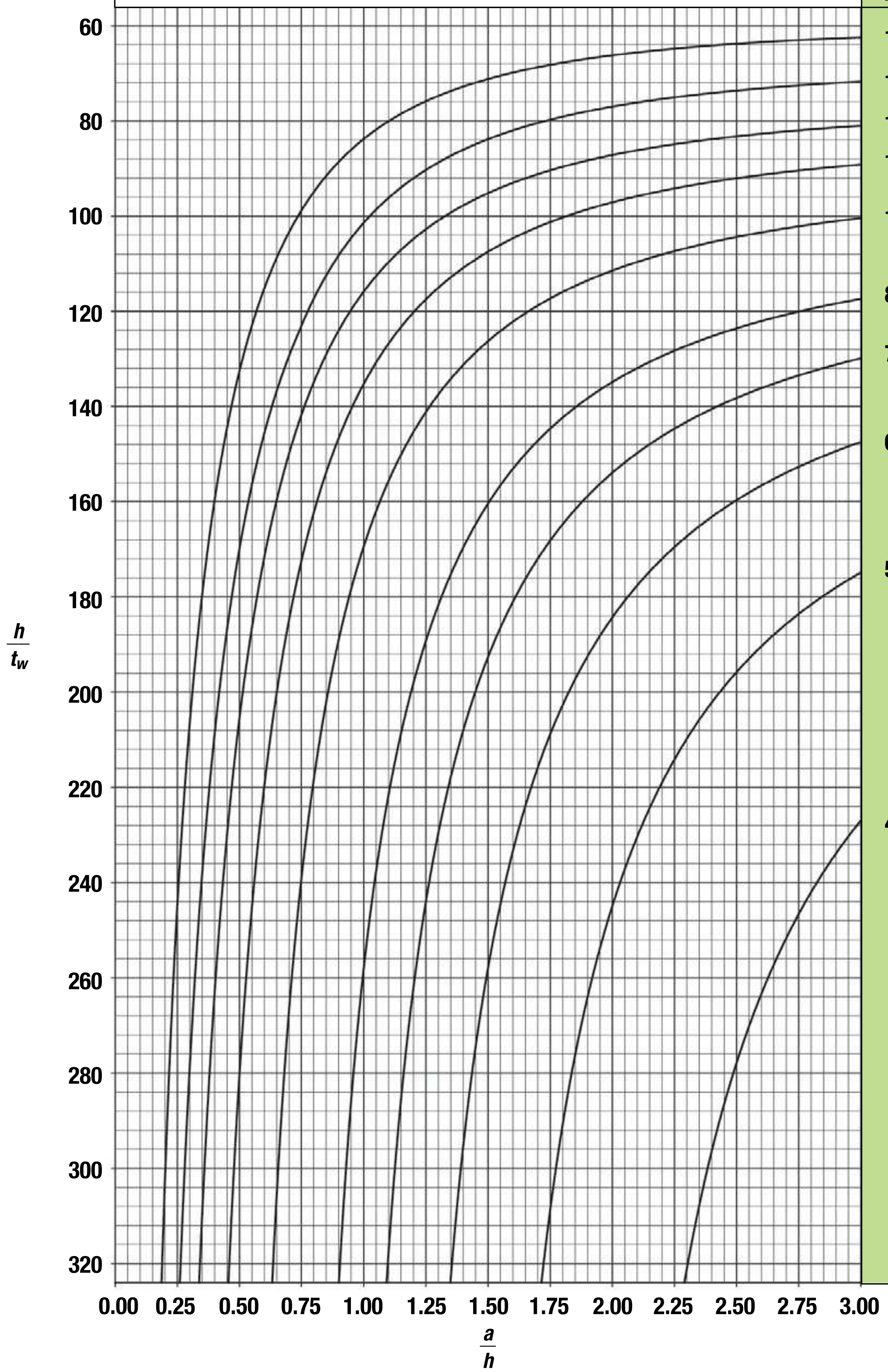


Table 3-17c
Available Shear Stress, ksi
Tension Field Action Included
When $2A_w/(A_{fc}+A_{ft}) > 2.5$ or $h/b_{fc} > 6.0$ or $h/b_{ft} > 6.0$

F_y = 50 ksi

$\Omega_v = 1.67$	$\phi_v = 0.90$
$\dfrac{V_n}{\Omega_v A_w}$	$\dfrac{\phi_v V_n}{A_w}$
ASD	LRFD
18.0	27.0
16.0	24.0
14.0	21.0
12.0	18.0
10.0	15.0
8.00	12.0
7.00	10.5
6.00	9.00
5.00	7.50
4.00	6.00



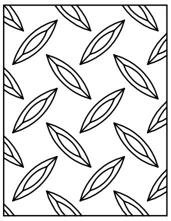


Table 3-18a
Raised Pattern Floor Plate Deflection-Controlled Applications
Recommended Maximum Uniformly Distributed Service Load, lb/ft^2

Plate thickness *t*, in.	Theoretical weight, lb/ft^2	Span, ft					Moment of inertia per ft of width, in.^{4}/ft
		1.5	2	2.5	3	3.5	
1/8	6.15	89.5	37.8	19.3	11.2	7.05	0.00195
3/16	8.70	302	127	65.3	37.8	23.8	0.00659
1/4	11.3	716	302	155	89.5	56.4	0.0156
5/16	13.8	1400	590	302	175	110	0.0305
3/8	16.4	2420	1020	522	302	190	0.0527
1/2	21.5	5730	2420	1240	716	451	0.125
5/8	26.6	11200	4720	2420	1400	881	0.244
3/4	31.7	19300	8160	4180	2420	1520	0.422
7/8	36.8	30700	13000	6630	3840	2420	0.670
1	41.9	45800	19300	9900	5730	3610	1.00
1 1/4	52.1	89500	37800	19300	11200	7050	1.95
1 1/2	62.3	155000	65300	33400	19300	12200	3.38
1 3/4	72.5	246000	104000	53100	30700	19300	5.36
2	82.7	367000	155000	79200	45800	28900	8.00

Plate thickness *t*, in.	Theoretical weight, lb/ft^2	Span, ft					Moment of inertia per ft of width, in.^{4}/ft
		4	4.5	5	6	7	
3/16	8.70	15.9	11.2	8.16	4.72	2.97	0.00659
1/4	11.3	37.8	26.5	19.3	11.2	7.05	0.0156
5/16	13.8	73.8	51.8	37.8	21.9	13.8	0.0305
3/8	16.4	127	89.5	65.3	37.8	23.8	0.0527
1/2	21.5	302	212	155	89.5	56.4	0.125
5/8	26.6	590	414	302	175	110	0.244
3/4	31.7	1020	716	522	302	190	0.422
7/8	36.8	1620	1140	829	480	302	0.670
1	41.9	2420	1700	1240	716	451	1.00
1 1/4	52.1	4720	3320	2420	1400	881	1.95
1 1/2	62.3	8160	5730	4180	2420	1520	3.38
1 3/4	72.5	13000	9100	6630	3840	2420	5.36
2	82.7	19300	13600	9900	5730	3610	8.00

Note: Material conforms to ASTM A786.

Table 3-18b
Raised Pattern Floor Plate Flexural-Strength-Controlled Applications
Recommended Maximum Uniformly Distributed Load, lb/ft²

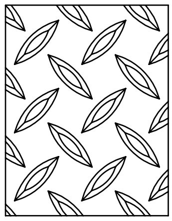
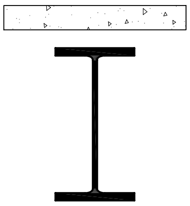


Plate thickness *t*, in.	Theoretical weight, lb/ft²	Span, ft 1.5		2		2.5		3		3.5		Plastic section modulus per ft of width, in.³/ft
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
1/8	6.15	222	333	125	188	79.8	120	55.4	83.3	40.7	61.2	0.0469
3/16	8.70	499	750	281	422	180	270	125	188	91.7	138	0.105
1/4	11.3	887	1330	499	750	319	480	222	333	163	245	0.188
5/16	13.8	1390	2080	780	1170	499	750	347	521	255	383	0.293
3/8	16.4	2000	3000	1120	1690	719	1080	499	750	367	551	0.422
1/2	21.5	3550	5330	2000	3000	1280	1920	887	1330	652	980	0.750
5/8	26.6	5540	8330	3120	4690	2000	3000	1390	2080	1020	1530	1.17
3/4	31.7	7980	12000	4490	6750	2870	4320	2000	3000	1470	2200	1.69
7/8	36.8	10900	16300	6110	9190	3910	5880	2720	4080	2000	3000	2.30
1	41.9	14200	21300	7980	12000	5110	7680	3550	5330	2610	3920	3.00
1 1/4	52.1	22200	33300	12500	18800	7980	12000	5540	8330	4070	6120	4.69
1 1/2	62.3	31900	48000	18000	27000	11500	17300	7980	12000	5870	8820	6.75
1 3/4	72.5	43500	65300	24500	36800	15600	23500	10900	16300	7980	12000	9.19
2	82.7	56800	85300	31900	48000	20400	30700	14200	21300	10400	15700	12.0

Plate thickness *t*, in.	Theoretical weight, lb/ft²	Span, ft 4		4.5		5		6		7		Plastic section modulus per ft of width, in.³/ft
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
3/16	8.70	70.2	105	55.4	83.3	44.9	67.5	31.2	46.9	22.9	34.4	0.105
1/4	11.3	125	188	98.6	148	79.8	120	55.4	83.3	40.7	61.2	0.188
5/16	13.8	195	293	154	231	125	188	86.6	130	63.6	95.7	0.293
3/8	16.4	281	422	222	333	180	270	125	188	91.7	138	0.422
1/2	21.5	499	750	394	593	319	480	222	333	163	245	0.750
5/8	26.6	780	1170	616	926	499	750	347	521	255	383	1.17
3/4	31.7	1120	1690	887	1330	719	1080	499	750	367	551	1.69
7/8	36.8	1530	2300	1210	1810	978	1470	679	1020	499	750	2.30
1	41.9	2000	3000	1580	2370	1280	1920	887	1330	652	980	3.00
1 1/4	52.1	3120	4690	2460	3700	2000	3000	1390	2080	1020	1530	4.69
1 1/2	62.3	4490	6750	3550	5330	2870	4320	2000	3000	1470	2200	6.75
1 3/4	72.5	6110	9190	4830	7260	3910	5880	2720	4080	2000	3000	9.19
2	82.7	7980	12000	6310	9480	5110	7680	3550	5330	2610	3920	12.0

Note: Material conforms to ASTM A786.



W40

Table 3-19
Composite W-Shapes
Available Strength in Flexure, kip-ft

F_y = 50 ksi

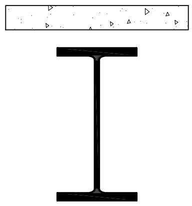
Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n[d]	$Y2$[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W40×297	3320	4990	TFL	0	4370	4770	7170	4880	7330	4990	7500	5100	7660
			2	0.413	3710	4700	7060	4790	7200	4880	7340	4980	7480
			3	0.825	3060	4610	6930	4690	7050	4770	7160	4840	7280
			4	1.24	2410	4510	6790	4570	6880	4630	6970	4700	7060
			BFL	1.65	1760	4400	6620	4450	6680	4490	6750	4530	6820
			6	4.58	1420	4320	6490	4360	6550	4390	6600	4430	6650
			7	8.17	1090	4180	6280	4210	6320	4240	6370	4260	6410
W40×294	3170	4760	TFL	0	4310	4770	7180	4880	7340	4990	7500	5100	7660
			2	0.483	3730	4710	7080	4800	7220	4900	7360	4990	7500
			3	0.965	3150	4630	6960	4710	7080	4790	7200	4870	7320
			4	1.45	2570	4540	6820	4600	6920	4670	7010	4730	7110
			BFL	1.93	1990	4430	6660	4480	6740	4530	6810	4580	6880
			6	5.71	1540	4300	6470	4340	6520	4380	6580	4420	6640
			7	10.0	1080	4080	6130	4110	6170	4130	6210	4160	6250
W40×278	2970	4460	TFL	0	4120	4540	6820	4640	6970	4740	7130	4850	7280
			2	0.453	3570	4480	6730	4570	6860	4660	7000	4750	7130
			3	0.905	3030	4410	6620	4480	6730	4560	6850	4630	6960
			4	1.36	2490	4320	6490	4380	6590	4440	6680	4510	6770
			BFL	1.81	1940	4220	6350	4270	6420	4320	6490	4370	6570
			6	5.67	1490	4100	6160	4130	6210	4170	6270	4210	6320
			7	10.1	1030	3870	5820	3900	5860	3920	5900	3950	5930
W40×277	3120	4690	TFL	0	4080	4440	6680	4540	6830	4650	6980	4750	7140
			2	0.395	3450	4370	6580	4460	6700	4550	6830	4630	6960
			3	0.790	2830	4290	6450	4360	6560	4440	6670	4510	6770
			4	1.19	2200	4200	6310	4260	6400	4310	6480	4370	6560
			BFL	1.58	1580	4100	6160	4130	6210	4170	6270	4210	6330
			6	4.20	1300	4030	6060	4060	6110	4090	6150	4130	6200
			7	7.58	1020	3920	5890	3940	5930	3970	5970	4000	6010
W40×264	2820	4240	TFL	0	3870	4250	6390	4350	6530	4440	6680	4540	6820
			2	0.433	3360	4190	6300	4280	6430	4360	6550	4440	6680
			3	0.865	2840	4120	6200	4190	6300	4270	6410	4340	6520
			4	1.30	2330	4040	6080	4100	6170	4160	6250	4220	6340
			BFL	1.73	1810	3950	5940	4000	6010	4040	6080	4090	6150
			6	5.53	1390	3840	5770	3870	5820	3910	5870	3940	5930
			7	9.92	968	3630	5460	3660	5500	3680	5540	3710	5570

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Ductility (slip capacity) of the shear connection at the beam/concrete interface may control minimum ΣQ_n requirements per AISC *Specification* Section I3.2d.

F_y = 50 ksi

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft

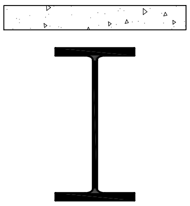


W40

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W40×297	5210	7820	5310	7990	5420	8150	5530	8320	5640	8480	5750	8640	5860	8810
	5070	7620	5160	7760	5250	7900	5350	8040	5440	8180	5530	8310	5620	8450
	4920	7390	5000	7510	5070	7620	5150	7740	5220	7850	5300	7970	5380	8080
	4760	7150	4820	7240	4880	7330	4940	7420	5000	7510	5060	7600	5120	7690
	4580	6880	4620	6950	4670	7010	4710	7080	4750	7140	4800	7210	4840	7280
	4460	6710	4500	6760	4530	6810	4570	6870	4600	6920	4640	6970	4670	7030
	4290	6450	4320	6490	4340	6530	4370	6570	4400	6610	4430	6650	4450	6690
W40×294	5200	7820	5310	7980	5420	8150	5530	8310	5630	8470	5740	8630	5850	8790
	5080	7640	5180	7780	5270	7920	5360	8060	5450	8200	5550	8340	5640	8480
	4950	7430	5020	7550	5100	7670	5180	7790	5260	7910	5340	8020	5420	8140
	4800	7210	4860	7300	4920	7400	4990	7500	5050	7590	5120	7690	5180	7790
	4630	6960	4680	7030	4730	7110	4780	7180	4830	7260	4880	7330	4930	7410
	4460	6700	4490	6760	4530	6810	4570	6870	4610	6930	4650	6990	4690	7040
	4190	6290	4210	6330	4240	6370	4270	6410	4290	6450	4320	6500	4350	6540
W40×278	4950	7440	5050	7590	5150	7750	5260	7900	5360	8060	5460	8210	5560	8360
	4830	7270	4920	7400	5010	7530	5100	7670	5190	7800	5280	7940	5370	8070
	4710	7080	4780	7190	4860	7300	4930	7420	5010	7530	5090	7640	5160	7760
	4570	6870	4630	6960	4690	7050	4750	7150	4820	7240	4880	7330	4940	7430
	4420	6640	4470	6710	4510	6780	4560	6860	4610	6930	4660	7000	4710	7080
	4250	6380	4280	6440	4320	6490	4360	6550	4390	6600	4430	6660	4470	6720
	3970	5970	4000	6010	4030	6050	4050	6090	4080	6130	4100	6170	4130	6200
W40×277	4850	7290	4950	7440	5050	7590	5150	7750	5260	7900	5360	8050	5460	8210
	4720	7090	4810	7220	4890	7350	4980	7480	5060	7610	5150	7740	5240	7870
	4580	6880	4650	6980	4720	7090	4790	7200	4860	7300	4930	7410	5000	7510
	4420	6640	4480	6730	4530	6810	4590	6890	4640	6970	4700	7060	4750	7140
	4250	6390	4290	6450	4330	6510	4370	6570	4410	6630	4450	6690	4490	6750
	4160	6250	4190	6300	4220	6350	4260	6400	4290	6450	4320	6500	4350	6540
	4020	6040	4050	6080	4070	6120	4100	6160	4120	6200	4150	6230	4170	6270
W40×264	4630	6970	4730	7110	4830	7260	4920	7400	5020	7550	5120	7690	5210	7840
	4530	6800	4610	6930	4690	7060	4780	7180	4860	7310	4950	7430	5030	7560
	4410	6620	4480	6730	4550	6840	4620	6940	4690	7050	4760	7160	4830	7260
	4280	6430	4330	6520	4390	6600	4450	6690	4510	6780	4570	6860	4630	6950
	4130	6210	4180	6280	4230	6350	4270	6420	4320	6490	4360	6550	4410	6620
	3980	5980	4010	6030	4050	6080	4080	6140	4120	6190	4150	6240	4190	6290
	3730	5610	3760	5640	3780	5680	3800	5720	3830	5750	3850	5790	3880	5830

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[b] $Y2$ = distance from top of the steel beam to concrete flange force



W40

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft

F_y = 50 ksi

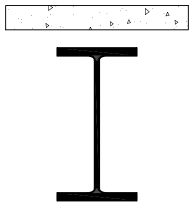
Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n[d]	$Y2$[b], in.							
	kip-ft			in.	kip	2		2.5		3		3.5	
	ASD	LRFD				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W40×249	2790	4200	TFL	0	3680	3980	5980	4070	6120	4160	6260	4250	6390
			2	0.355	3110	3920	5890	4000	6010	4070	6120	4150	6240
			3	0.710	2550	3850	5780	3910	5880	3970	5970	4040	6070
			4	1.07	1990	3770	5660	3820	5740	3870	5810	3920	5890
			BFL	1.42	1430	3680	5520	3710	5580	3750	5630	3780	5690
			6	4.03	1180	3620	5440	3650	5480	3680	5530	3710	5570
			7	7.45	919	3520	5290	3540	5320	3560	5360	3590	5390
W40×235	2520	3790	TFL	0	3460	3770	5660	3850	5790	3940	5920	4030	6050
			2	0.395	2980	3720	5580	3790	5700	3860	5810	3940	5920
			3	0.790	2510	3650	5490	3720	5590	3780	5680	3840	5780
			4	1.19	2040	3580	5390	3640	5460	3690	5540	3740	5620
			BFL	1.58	1570	3510	5270	3540	5330	3580	5390	3620	5450
			6	5.16	1220	3410	5130	3440	5180	3470	5220	3500	5270
			7	9.44	864	3250	4880	3270	4920	3290	4950	3310	4980
W40×215	2410	3620	TFL	0	3180	3410	5120	3490	5240	3560	5360	3640	5480
			2	0.305	2690	3350	5040	3420	5140	3490	5240	3560	5340
			3	0.610	2210	3300	4950	3350	5040	3410	5120	3460	5200
			4	0.915	1730	3230	4850	3270	4920	3320	4980	3360	5050
			BFL	1.22	1250	3160	4740	3190	4790	3220	4840	3250	4880
			6	3.80	1020	3110	4670	3130	4710	3160	4750	3180	4780
			7	7.29	794	3020	4540	3040	4570	3060	4600	3080	4630
W40×211	2260	3400	TFL	0	3110	3360	5050	3440	5170	3520	5290	3590	5400
			2	0.355	2690	3320	4990	3380	5090	3450	5190	3520	5290
			3	0.710	2270	3260	4910	3320	4990	3380	5080	3430	5160
			4	1.07	1850	3200	4810	3250	4880	3300	4950	3340	5020
			BFL	1.42	1430	3140	4710	3170	4770	3210	4820	3240	4870
			6	5.00	1100	3050	4590	3080	4630	3110	4670	3140	4710
			7	9.35	776	2900	4370	2920	4390	2940	4420	2960	4450
W40×199	2170	3260	TFL	0	2940	3130	4710	3210	4820	3280	4930	3350	5040
			2	0.268	2520	3090	4640	3150	4730	3210	4830	3280	4920
			3	0.535	2090	3040	4560	3090	4640	3140	4720	3190	4800
			4	0.803	1670	2980	4480	3020	4540	3060	4600	3110	4670
			BFL	1.07	1250	2920	4390	2950	4430	2980	4480	3010	4530
			6	4.09	992	2860	4300	2890	4340	2910	4380	2940	4410
			7	8.04	735	2760	4150	2780	4170	2800	4200	2810	4230

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Ductility (slip capacity) of the shear connection at the beam/concrete interface may control minimum ΣQ_n requirements per AISC *Specification* Section I3.2d.

F_y = 50 ksi

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft

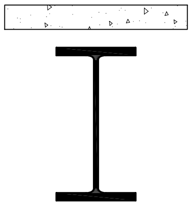


W40

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W40×249	4350	6530	4440	6670	4530	6810	4620	6950	4710	7080	4800	7220	4900	7360
	4230	6360	4310	6470	4380	6590	4460	6710	4540	6820	4620	6940	4700	7060
	4100	6170	4170	6260	4230	6360	4290	6450	4360	6550	4420	6640	4480	6740
	3970	5960	4020	6030	4060	6110	4110	6180	4160	6260	4210	6330	4260	6410
	3820	5740	3850	5790	3890	5850	3930	5900	3960	5950	4000	6010	4030	6060
	3740	5610	3770	5660	3790	5700	3820	5750	3850	5790	3880	5840	3910	5880
	3610	5430	3630	5460	3660	5500	3680	5530	3700	5560	3730	5600	3750	5630
W40×235	4110	6180	4200	6310	4280	6440	4370	6570	4460	6700	4540	6830	4630	6960
	4010	6030	4090	6140	4160	6260	4240	6370	4310	6480	4390	6590	4460	6700
	3910	5870	3970	5960	4030	6060	4090	6150	4160	6250	4220	6340	4280	6440
	3790	5690	3840	5770	3890	5850	3940	5920	3990	6000	4040	6080	4090	6150
	3660	5500	3700	5560	3740	5620	3780	5680	3820	5740	3860	5800	3900	5860
	3540	5310	3570	5360	3600	5410	3630	5450	3660	5500	3690	5540	3720	5590
	3330	5010	3360	5040	3380	5080	3400	5110	3420	5140	3440	5170	3460	5210
W40×215	3720	5600	3800	5720	3880	5830	3960	5950	4040	6070	4120	6190	4200	6310
	3620	5450	3690	5550	3760	5650	3820	5750	3890	5850	3960	5950	4030	6050
	3520	5280	3570	5370	3630	5450	3680	5530	3740	5620	3790	5700	3850	5780
	3400	5110	3440	5180	3490	5240	3530	5310	3570	5370	3620	5440	3660	5500
	3280	4930	3310	4980	3340	5020	3370	5070	3400	5120	3440	5160	3470	5210
	3210	4820	3230	4860	3260	4900	3280	4940	3310	4970	3340	5010	3360	5050
	3100	4660	3120	4690	3140	4720	3160	4750	3180	4780	3200	4810	3220	4840
W40×211	3670	5520	3750	5640	3830	5750	3900	5870	3980	5980	4060	6100	4140	6220
	3580	5390	3650	5490	3720	5590	3790	5690	3850	5790	3920	5890	3990	5990
	3490	5250	3550	5330	3600	5420	3660	5500	3720	5590	3770	5670	3830	5760
	3390	5090	3430	5160	3480	5230	3530	5300	3570	5370	3620	5440	3660	5510
	3280	4930	3310	4980	3350	5030	3390	5090	3420	5140	3460	5200	3490	5250
	3160	4760	3190	4800	3220	4840	3250	4880	3270	4920	3300	4960	3330	5000
	2980	4480	3000	4510	3020	4540	3040	4570	3060	4600	3080	4630	3100	4660
W40×199	3430	5150	3500	5260	3570	5370	3650	5480	3720	5590	3790	5700	3870	5810
	3340	5020	3400	5110	3460	5210	3530	5300	3590	5400	3650	5490	3720	5580
	3250	4880	3300	4960	3350	5030	3400	5110	3450	5190	3510	5270	3560	5350
	3150	4730	3190	4790	3230	4860	3270	4920	3310	4980	3360	5040	3400	5110
	3040	4570	3070	4620	3110	4670	3140	4710	3170	4760	3200	4810	3230	4850
	2960	4450	2990	4490	3010	4530	3040	4560	3060	4600	3090	4640	3110	4670
	2830	4260	2850	4280	2870	4310	2890	4340	2910	4370	2920	4390	2940	4420

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[b] $Y2$ = distance from top of the steel beam to concrete flange force



W40–W36

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft

F_y = 50 ksi

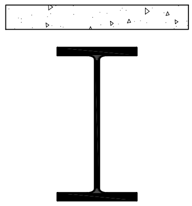
Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n[d]	$Y2$[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W40×183	1930	2900	TFL	0	2670	2860	4300	2930	4400	2990	4500	3060	4600
			2	0.300	2310	2820	4240	2880	4330	2940	4410	2990	4500
			3	0.600	1960	2780	4180	2830	4250	2880	4320	2920	4400
			4	0.900	1600	2730	4100	2770	4160	2810	4220	2850	4280
			BFL	1.20	1250	2680	4020	2710	4070	2740	4110	2770	4160
			6	4.77	958	2610	3920	2630	3950	2650	3990	2680	4030
			7	9.25	666	2480	3720	2490	3750	2510	3770	2530	3800
W40×167	1730	2600	TFL	0	2470	2620	3940	2680	4030	2740	4120	2800	4220
			2	0.258	2160	2590	3890	2640	3970	2700	4050	2750	4130
			3	0.515	1860	2550	3840	2600	3900	2640	3970	2690	4040
			4	0.773	1550	2510	3770	2550	3830	2590	3890	2630	3950
			BFL	1.03	1250	2470	3710	2490	3760	2530	3800	2560	3850
			6	4.95	933	2390	3600	2420	3630	2440	3670	2460	3700
			7	9.82	616	2240	3370	2260	3400	2280	3420	2290	3440
W40×149	1490	2240	TFL	0	2190	2310	3470	2360	3550	2420	3630	2470	3710
			2	0.208	1950	2280	3430	2330	3500	2380	3570	2430	3650
			3	0.415	1700	2250	3380	2290	3450	2340	3510	2380	3580
			4	0.623	1460	2220	3340	2260	3390	2290	3450	2330	3500
			BFL	0.830	1210	2190	3290	2220	3330	2250	3380	2280	3420
			6	5.15	879	2110	3170	2130	3200	2150	3240	2180	3270
			7	10.4	548	1950	2930	1960	2950	1980	2970	1990	2990
W36×302	3190	4800	TFL	0	4450	4590	6890	4700	7060	4810	7230	4920	7390
			2	0.420	3750	4510	6780	4600	6920	4700	7060	4790	7200
			3	0.840	3050	4420	6640	4490	6750	4570	6870	4640	6980
			4	1.26	2350	4310	6480	4370	6570	4430	6650	4490	6740
			BFL	1.68	1640	4190	6290	4230	6360	4270	6420	4310	6480
			6	4.06	1380	4120	6200	4160	6250	4190	6300	4230	6350
			7	6.88	1110	4030	6050	4050	6090	4080	6130	4110	6170
W36×282	2970	4460	TFL	0	4150	4250	6390	4350	6540	4460	6700	4560	6850
			2	0.393	3490	4180	6280	4270	6410	4350	6540	4440	6670
			3	0.785	2840	4090	6150	4170	6260	4240	6370	4310	6470
			4	1.18	2190	4000	6010	4050	6090	4110	6170	4160	6260
			BFL	1.57	1540	3890	5840	3930	5900	3970	5960	4000	6020
			6	4.00	1290	3830	5760	3860	5800	3890	5850	3930	5900
			7	6.84	1040	3740	5620	3760	5660	3790	5690	3810	5730

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Ductility (slip capacity) of the shear connection at the beam/concrete interface may control minimum ΣQ_n requirements per AISC *Specification* Section I3.2d.

F_y = 50 ksi

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft



W40–W36

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W40×183	3130	4700	3190	4800	3260	4900	3320	5000	3390	5100	3460	5200	3520	5300
	3050	4590	3110	4670	3170	4760	3220	4850	3280	4930	3340	5020	3400	5110
	2970	4470	3020	4540	3070	4620	3120	4690	3170	4760	3220	4840	3270	4910
	2890	4340	2930	4400	2970	4460	3010	4520	3050	4580	3090	4640	3130	4700
	2800	4210	2830	4260	2860	4300	2890	4350	2920	4400	2960	4440	2990	4490
	2700	4060	2730	4100	2750	4130	2770	4170	2800	4200	2820	4240	2850	4280
	2540	3820	2560	3850	2580	3870	2590	3900	2610	3920	2630	3950	2640	3970
W40×167	2870	4310	2930	4400	2990	4490	3050	4580	3110	4680	3170	4770	3240	4860
	2800	4210	2860	4290	2910	4380	2970	4460	3020	4540	3070	4620	3130	4700
	2740	4110	2780	4180	2830	4250	2880	4320	2920	4390	2970	4460	3020	4530
	2670	4010	2710	4070	2740	4120	2780	4180	2820	4240	2860	4300	2900	4360
	2590	3900	2620	3940	2650	3990	2690	4040	2720	4080	2750	4130	2780	4180
	2490	3740	2510	3770	2530	3810	2560	3840	2580	3880	2600	3910	2630	3950
	2310	3470	2320	3490	2340	3510	2350	3540	2370	3560	2380	3580	2400	3600
W40×149	2520	3790	2580	3880	2630	3960	2690	4040	2740	4120	2800	4200	2850	4290
	2470	3720	2520	3790	2570	3860	2620	3940	2670	4010	2720	4080	2770	4160
	2420	3640	2460	3700	2510	3770	2550	3830	2590	3890	2630	3960	2680	4020
	2370	3560	2400	3610	2440	3670	2480	3720	2510	3780	2550	3830	2580	3880
	2310	3470	2340	3520	2370	3560	2400	3610	2430	3650	2460	3700	2490	3740
	2200	3300	2220	3340	2240	3370	2260	3400	2290	3430	2310	3470	2330	3500
	2000	3010	2020	3030	2030	3050	2040	3070	2060	3090	2070	3110	2090	3130
W36×302	5030	7560	5140	7730	5250	7890	5360	8060	5470	8230	5580	8390	5700	8560
	4880	7340	4980	7480	5070	7620	5160	7760	5260	7900	5350	8040	5440	8180
	4720	7090	4800	7210	4870	7320	4950	7440	5020	7550	5100	7670	5180	7780
	4540	6830	4600	6920	4660	7010	4720	7090	4780	7180	4840	7270	4900	7360
	4350	6540	4390	6600	4430	6660	4470	6730	4520	6790	4560	6850	4600	6910
	4260	6410	4300	6460	4330	6510	4370	6560	4400	6610	4430	6670	4470	6720
	4140	6220	4160	6260	4190	6300	4220	6340	4250	6380	4270	6420	4300	6470
W36×282	4660	7010	4770	7170	4870	7320	4970	7480	5080	7630	5180	7790	5280	7940
	4530	6810	4610	6940	4700	7070	4790	7200	4880	7330	4960	7460	5050	7590
	4380	6580	4450	6690	4520	6790	4590	6900	4660	7010	4730	7110	4800	7220
	4220	6340	4270	6420	4330	6500	4380	6580	4440	6670	4490	6750	4540	6830
	4040	6080	4080	6130	4120	6190	4160	6250	4200	6310	4230	6360	4270	6420
	3960	5950	3990	6000	4020	6050	4050	6090	4090	6140	4120	6190	4150	6240
	3840	5770	3870	5810	3890	5850	3920	5890	3940	5930	3970	5970	4000	6010

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[b] $Y2$ = distance from top of the steel beam to concrete flange force

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft

W36

F_y = 50 ksi

Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n[d]	$Y2$[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W36×262	2740	4130	TFL	0	3860	3940	5920	4040	6070	4130	6210	4230	6350
			2	0.360	3260	3870	5820	3960	5940	4040	6070	4120	6190
			3	0.720	2660	3800	5710	3860	5810	3930	5910	4000	6010
			4	1.08	2070	3710	5580	3760	5660	3820	5730	3870	5810
			BFL	1.44	1470	3610	5430	3650	5490	3690	5540	3720	5600
			6	3.96	1220	3560	5350	3590	5390	3620	5440	3650	5480
			7	6.96	965	3460	5210	3490	5240	3510	5280	3540	5310
W36×256	2590	3900	TFL	0	3770	3890	5850	3980	5990	4080	6130	4170	6270
			2	0.433	3240	3830	5760	3910	5880	3990	6000	4070	6120
			3	0.865	2710	3760	5650	3830	5750	3900	5860	3960	5960
			4	1.30	2180	3680	5530	3730	5610	3790	5690	3840	5780
			BFL	1.73	1650	3590	5390	3630	5450	3670	5520	3710	5580
			6	5.18	1300	3490	5250	3520	5300	3560	5350	3590	5390
			7	8.90	941	3330	5010	3350	5040	3380	5080	3400	5110
W36×247	2570	3860	TFL	0	3630	3680	5530	3770	5670	3860	5800	3950	5940
			2	0.338	3070	3620	5440	3700	5560	3770	5670	3850	5790
			3	0.675	2510	3550	5340	3610	5430	3680	5530	3740	5620
			4	1.01	1950	3470	5220	3520	5290	3570	5360	3620	5440
			BFL	1.35	1400	3380	5090	3420	5140	3450	5190	3490	5240
			6	3.95	1150	3330	5000	3360	5050	3390	5090	3410	5130
			7	7.02	906	3240	4860	3260	4900	3280	4930	3300	4970
W36×232	2340	3510	TFL	0	3400	3490	5240	3570	5370	3660	5500	3740	5620
			2	0.393	2930	3430	5160	3510	5270	3580	5380	3650	5490
			3	0.785	2450	3370	5070	3430	5160	3500	5250	3560	5350
			4	1.18	1980	3300	4960	3350	5040	3400	5110	3450	5190
			BFL	1.57	1500	3220	4840	3260	4900	3300	4960	3330	5010
			6	5.04	1180	3140	4720	3170	4760	3200	4810	3230	4850
			7	8.78	850	2990	4500	3010	4530	3040	4560	3060	4590
W36×231	2400	3610	TFL	0	3410	3450	5180	3530	5310	3620	5430	3700	5560
			2	0.315	2890	3390	5090	3460	5200	3530	5310	3610	5420
			3	0.630	2370	3330	5000	3380	5090	3440	5180	3500	5270
			4	0.945	1850	3250	4890	3300	4960	3350	5030	3390	5100
			BFL	1.26	1330	3170	4770	3210	4820	3240	4870	3270	4920
			6	3.88	1090	3120	4690	3150	4730	3170	4770	3200	4810
			7	7.03	853	3030	4560	3050	4590	3070	4620	3090	4650

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Ductility (slip capacity) of the shear connection at the beam/concrete interface may control minimum ΣQ_n requirements per AISC *Specification* Section I3.2d.

F_y = 50 ksi

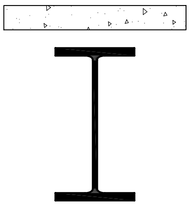
Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft

W36

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W36×262	4320	6500	4420	6640	4520	6790	4610	6930	4710	7080	4810	7220	4900	7370
	4200	6310	4280	6430	4360	6560	4440	6680	4530	6800	4610	6920	4690	7050
	4060	6110	4130	6210	4200	6310	4260	6410	4330	6510	4400	6610	4460	6710
	3920	5890	3970	5970	4020	6040	4070	6120	4120	6200	4180	6280	4230	6350
	3760	5650	3800	5710	3830	5760	3870	5820	3910	5870	3940	5930	3980	5980
	3680	5530	3710	5570	3740	5620	3770	5670	3800	5710	3830	5760	3860	5800
	3560	5350	3580	5390	3610	5420	3630	5460	3660	5490	3680	5530	3700	5570
W36×256	4260	6410	4360	6550	4450	6690	4550	6830	4640	6970	4730	7120	4830	7260
	4150	6240	4230	6360	4320	6490	4400	6610	4480	6730	4560	6850	4640	6970
	4030	6060	4100	6160	4170	6260	4230	6360	4300	6470	4370	6570	4440	6670
	3900	5860	3950	5940	4010	6020	4060	6100	4120	6190	4170	6270	4220	6350
	3750	5640	3790	5700	3830	5760	3880	5830	3920	5890	3960	5950	4000	6010
	3620	5440	3650	5490	3690	5540	3720	5590	3750	5640	3780	5690	3820	5740
	3420	5150	3450	5180	3470	5220	3500	5250	3520	5290	3540	5320	3570	5360
W36×247	4040	6080	4130	6210	4220	6350	4310	6480	4400	6620	4500	6760	4590	6890
	3930	5900	4000	6020	4080	6130	4160	6250	4230	6360	4310	6480	4390	6590
	3800	5710	3860	5810	3930	5900	3990	6000	4050	6090	4110	6180	4180	6280
	3670	5510	3720	5580	3760	5660	3810	5730	3860	5800	3910	5880	3960	5950
	3520	5300	3560	5350	3590	5400	3630	5450	3660	5510	3700	5560	3730	5610
	3440	5170	3470	5220	3500	5260	3530	5300	3560	5350	3590	5390	3620	5430
	3330	5000	3350	5030	3370	5070	3390	5100	3420	5140	3440	5170	3460	5200
W36×232	3830	5750	3910	5880	4000	6010	4080	6130	4170	6260	4250	6390	4330	6520
	3730	5600	3800	5710	3870	5820	3950	5930	4020	6040	4090	6150	4160	6260
	3620	5440	3680	5530	3740	5620	3800	5710	3860	5800	3920	5900	3980	5990
	3500	5260	3550	5330	3600	5410	3650	5480	3700	5560	3750	5630	3800	5710
	3370	5070	3410	5120	3450	5180	3480	5240	3520	5290	3560	5350	3600	5410
	3260	4890	3290	4940	3310	4980	3340	5030	3370	5070	3400	5110	3430	5160
	3080	4630	3100	4660	3120	4690	3140	4720	3160	4750	3180	4790	3210	4820
W36×231	3790	5690	3870	5820	3960	5950	4040	6070	4130	6200	4210	6330	4300	6460
	3680	5530	3750	5640	3820	5750	3890	5850	3970	5960	4040	6070	4110	6180
	3560	5350	3620	5440	3680	5530	3740	5620	3800	5710	3860	5800	3920	5890
	3440	5170	3480	5240	3530	5310	3580	5380	3620	5440	3670	5510	3720	5580
	3310	4970	3340	5020	3370	5070	3410	5120	3440	5170	3470	5220	3500	5270
	3230	4850	3260	4890	3280	4930	3310	4980	3340	5020	3360	5060	3390	5100
	3120	4680	3140	4720	3160	4750	3180	4780	3200	4810	3220	4840	3240	4880

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[b] $Y2$ = distance from top of the steel beam to concrete flange force



W36

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft

F_y = 50 ksi

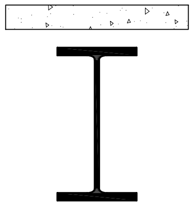
Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n[d]	$Y2$[b], in. 2		2.5		3		3.5	
	kip-ft			in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	ASD	LRFD											
W36×210	2080	3120	TFL	0	3100	3140	4720	3220	4840	3300	4960	3370	5070
			2	0.340	2680	3100	4660	3160	4760	3230	4860	3300	4960
			3	0.680	2270	3050	4580	3100	4660	3160	4750	3220	4830
			4	1.02	1850	2990	4490	3030	4560	3080	4630	3130	4700
			BFL	1.36	1440	2920	4390	2960	4440	2990	4500	3030	4550
			6	5.04	1100	2840	4260	2860	4300	2890	4350	2920	4390
			7	9.03	774	2690	4040	2710	4070	2730	4100	2750	4130
W36×194	1910	2880	TFL	0	2850	2880	4330	2950	4440	3020	4540	3090	4650
			2	0.315	2470	2840	4270	2900	4360	2960	4450	3020	4540
			3	0.630	2090	2790	4200	2840	4270	2900	4350	2950	4430
			4	0.945	1710	2740	4120	2780	4180	2820	4240	2870	4310
			BFL	1.26	1330	2680	4030	2710	4080	2750	4130	2780	4180
			6	4.93	1020	2600	3910	2630	3950	2650	3990	2680	4030
			7	8.94	713	2470	3710	2480	3730	2500	3760	2520	3790
W36×182	1790	2690	TFL	0	2680	2690	4050	2760	4150	2830	4250	2900	4350
			2	0.295	2320	2660	3990	2710	4080	2770	4170	2830	4250
			3	0.590	1970	2610	3930	2660	4000	2710	4070	2760	4150
			4	0.885	1610	2560	3850	2600	3910	2640	3970	2680	4040
			BFL	1.18	1250	2510	3770	2540	3820	2570	3870	2600	3910
			6	4.89	961	2440	3670	2460	3700	2490	3740	2510	3770
			7	8.91	670	2310	3470	2330	3500	2340	3520	2360	3550
W36×170	1670	2510	TFL	0	2500	2510	3770	2570	3860	2630	3960	2690	4050
			2	0.275	2170	2470	3720	2530	3800	2580	3880	2630	3960
			3	0.550	1840	2430	3660	2480	3730	2520	3790	2570	3860
			4	0.825	1510	2390	3590	2430	3650	2460	3700	2500	3760
			BFL	1.10	1180	2340	3520	2370	3560	2400	3600	2430	3650
			6	4.83	903	2270	3420	2300	3450	2320	3480	2340	3520
			7	8.91	625	2150	3230	2170	3250	2180	3280	2200	3300
W36×160	1560	2340	TFL	0	2350	2350	3530	2400	3610	2460	3700	2520	3790
			2	0.255	2040	2310	3480	2360	3550	2410	3630	2470	3710
			3	0.510	1740	2280	3420	2320	3490	2360	3550	2410	3620
			4	0.765	1430	2240	3360	2270	3410	2310	3470	2340	3520
			BFL	1.02	1130	2190	3290	2220	3340	2250	3380	2280	3420
			6	4.82	857	2130	3200	2150	3230	2170	3260	2190	3290
			7	8.96	588	2010	3020	2020	3040	2040	3060	2050	3080

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Ductility (slip capacity) of the shear connection at the beam/concrete interface may control minimum ΣQ_n requirements per AISC *Specification* Section I3.2d.

F_y = 50 ksi

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft

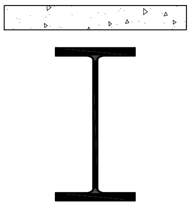


W36

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W36×210	3450	5190	3530	5300	3610	5420	3680	5540	3760	5650	3840	5770	3920	5880
	3370	5060	3430	5160	3500	5260	3570	5360	3630	5460	3700	5560	3770	5660
	3270	4920	3330	5000	3390	5090	3440	5170	3500	5260	3550	5340	3610	5430
	3170	4770	3220	4840	3260	4910	3310	4980	3360	5040	3400	5110	3450	5180
	3060	4610	3100	4660	3140	4710	3170	4770	3210	4820	3240	4880	3280	4930
	2950	4430	2970	4470	3000	4510	3030	4550	3060	4590	3080	4640	3110	4680
	2760	4160	2780	4180	2800	4210	2820	4240	2840	4270	2860	4300	2880	4330
W36×194	3160	4760	3240	4860	3310	4970	3380	5080	3450	5180	3520	5290	3590	5400
	3090	4640	3150	4730	3210	4820	3270	4910	3330	5010	3390	5100	3450	5190
	3000	4510	3050	4590	3100	4670	3160	4740	3210	4820	3260	4900	3310	4980
	2910	4370	2950	4440	2990	4500	3040	4560	3080	4630	3120	4690	3160	4760
	2810	4230	2840	4280	2880	4330	2910	4380	2940	4430	2980	4480	3010	4530
	2710	4070	2730	4100	2760	4140	2780	4180	2810	4220	2830	4260	2860	4300
	2540	3810	2560	3840	2570	3870	2590	3900	2610	3920	2630	3950	2640	3980
W36×182	2960	4450	3030	4550	3100	4650	3160	4750	3230	4850	3300	4950	3360	5060
	2890	4340	2950	4430	3000	4520	3060	4600	3120	4690	3180	4780	3240	4860
	2810	4220	2860	4300	2910	4370	2960	4440	3010	4520	3050	4590	3110	4660
	2720	4100	2760	4160	2810	4220	2850	4280	2890	4340	2930	4400	2970	4460
	2630	3960	2670	4010	2700	4050	2730	4100	2760	4150	2790	4190	2820	4240
	2530	3810	2560	3850	2580	3880	2610	3920	2630	3950	2650	3990	2680	4030
	2380	3570	2390	3600	2410	3620	2430	3650	2440	3670	2460	3700	2480	3720
W36×170	2760	4140	2820	4240	2880	4330	2940	4430	3010	4520	3070	4610	3130	4710
	2690	4040	2740	4120	2800	4200	2850	4290	2910	4370	2960	4450	3010	4530
	2620	3930	2660	4000	2710	4070	2750	4140	2800	4210	2850	4280	2890	4350
	2540	3820	2580	3870	2610	3930	2650	3990	2690	4040	2730	4100	2770	4160
	2460	3690	2490	3740	2520	3780	2550	3830	2580	3870	2600	3910	2630	3960
	2360	3550	2390	3580	2410	3620	2430	3650	2450	3690	2480	3720	2500	3750
	2210	3320	2230	3350	2240	3370	2260	3400	2270	3420	2290	3440	2310	3470
W36×160	2580	3880	2640	3970	2700	4050	2760	4140	2810	4230	2870	4320	2930	4410
	2520	3780	2570	3860	2620	3940	2670	4010	2720	4090	2770	4170	2820	4240
	2450	3680	2490	3750	2540	3810	2580	3880	2620	3940	2670	4010	2710	4070
	2380	3580	2410	3630	2450	3680	2490	3740	2520	3790	2560	3840	2590	3900
	2300	3460	2330	3510	2360	3550	2390	3590	2420	3630	2450	3680	2470	3720
	2210	3330	2230	3360	2260	3390	2280	3420	2300	3450	2320	3490	2340	3520
	2070	3110	2080	3130	2100	3150	2110	3170	2130	3190	2140	3220	2150	3240

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[b] $Y2$ = distance from top of the steel beam to concrete flange force



W36–W33

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft

F_y = 50 ksi

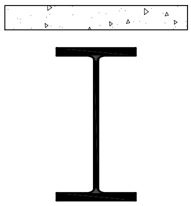
Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	Y1[a]	ΣQ_n[d]	Y2[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W36×150	1450	2180	TFL	0	2220	2210	3310	2260	3400	2320	3480	2370	3560
			2	0.235	1930	2180	3270	2220	3340	2270	3410	2320	3490
			3	0.470	1650	2140	3220	2180	3280	2220	3340	2270	3410
			4	0.705	1370	2110	3160	2140	3220	2170	3270	2210	3320
			BFL	0.940	1090	2070	3110	2090	3150	2120	3190	2150	3230
			6	4.82	820	2000	3010	2020	3040	2040	3070	2060	3100
			7	9.09	554	1880	2830	1900	2850	1910	2870	1930	2890
W36×135	1270	1910	TFL	0	2000	1970	2960	2020	3040	2070	3110	2120	3190
			2	0.198	1760	1950	2930	1990	2990	2030	3060	2080	3120
			3	0.395	1520	1920	2880	1960	2940	2000	3000	2030	3060
			4	0.593	1280	1890	2840	1920	2890	1950	2940	1990	2980
			BFL	0.790	1050	1860	2790	1880	2830	1910	2870	1940	2910
			6	4.92	773	1790	2700	1810	2720	1830	2750	1850	2780
			7	9.49	499	1670	2510	1680	2530	1690	2540	1710	2560
W33×221	2140	3210	TFL	0	3270	3090	4640	3170	4760	3250	4890	3330	5010
			2	0.320	2760	3030	4560	3100	4660	3170	4770	3240	4870
			3	0.640	2250	2970	4460	3030	4550	3080	4630	3140	4720
			4	0.960	1750	2900	4360	2940	4420	2990	4490	3030	4560
			BFL	1.28	1240	2820	4240	2850	4290	2880	4330	2910	4380
			6	3.67	1030	2770	4170	2800	4210	2830	4250	2850	4290
			7	6.42	816	2700	4060	2720	4090	2740	4120	2760	4150
W33×201	1930	2900	TFL	0	2960	2780	4180	2850	4290	2930	4400	3000	4510
			2	0.288	2500	2730	4110	2790	4200	2860	4290	2920	4390
			3	0.575	2050	2680	4020	2730	4100	2780	4180	2830	4250
			4	0.863	1600	2620	3930	2660	3990	2700	4050	2740	4110
			BFL	1.15	1150	2550	3830	2580	3870	2600	3920	2630	3960
			6	3.65	944	2500	3760	2530	3800	2550	3830	2570	3870
			7	6.52	739	2430	3650	2450	3680	2470	3710	2490	3740
W33×169	1570	2360	TFL	0	2480	2330	3510	2400	3600	2460	3690	2520	3790
			2	0.305	2120	2300	3450	2350	3530	2400	3610	2460	3690
			3	0.610	1770	2250	3390	2300	3450	2340	3520	2390	3590
			4	0.915	1420	2210	3310	2240	3370	2280	3420	2310	3470
			BFL	1.22	1070	2150	3230	2180	3270	2200	3310	2230	3350
			6	4.28	845	2100	3150	2120	3190	2140	3220	2160	3250
			7	7.66	619	2010	3020	2020	3040	2040	3070	2060	3090

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Ductility (slip capacity) of the shear connection at the beam/concrete interface may control minimum ΣQ_n requirements per AISC *Specification* Section I3.2d.

F_y = 50 ksi

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft



W36–W33

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W36×150	2430	3650	2480	3730	2540	3810	2590	3900	2650	3980	2700	4060	2760	4140
	2370	3560	2420	3630	2460	3700	2510	3780	2560	3850	2610	3920	2660	3990
	2310	3470	2350	3530	2390	3590	2430	3650	2470	3710	2510	3780	2550	3840
	2240	3370	2280	3420	2310	3470	2340	3520	2380	3580	2410	3630	2450	3680
	2170	3270	2200	3310	2230	3350	2260	3390	2280	3430	2310	3470	2340	3510
	2080	3130	2100	3160	2130	3200	2150	3230	2170	3260	2190	3290	2210	3320
	1940	2910	1950	2940	1970	2960	1980	2980	1990	3000	2010	3020	2020	3040
W36×135	2170	3260	2220	3340	2270	3410	2320	3490	2370	3560	2420	3640	2470	3710
	2120	3190	2170	3250	2210	3320	2250	3390	2300	3450	2340	3520	2380	3580
	2070	3110	2110	3170	2150	3230	2180	3280	2220	3340	2260	3400	2300	3450
	2020	3030	2050	3080	2080	3130	2110	3180	2150	3220	2180	3270	2210	3320
	1960	2950	1990	2990	2010	3030	2040	3070	2070	3110	2090	3150	2120	3190
	1870	2810	1890	2840	1910	2870	1930	2900	1950	2930	1970	2960	1990	2990
	1720	2580	1730	2600	1740	2620	1750	2640	1770	2660	1780	2670	1790	2690
W33×221	3410	5130	3490	5250	3580	5380	3660	5500	3740	5620	3820	5740	3900	5860
	3310	4970	3380	5080	3450	5180	3510	5280	3580	5390	3650	5490	3720	5590
	3200	4800	3250	4890	3310	4970	3360	5060	3420	5140	3480	5220	3530	5310
	3070	4620	3120	4690	3160	4750	3210	4820	3250	4880	3290	4950	3340	5010
	2940	4430	2980	4470	3010	4520	3040	4570	3070	4610	3100	4660	3130	4710
	2880	4320	2900	4360	2930	4400	2950	4440	2980	4480	3010	4520	3030	4560
	2780	4180	2800	4210	2820	4240	2840	4270	2860	4300	2880	4330	2900	4360
W33×201	3070	4620	3150	4730	3220	4840	3300	4950	3370	5060	3440	5170	3520	5290
	2980	4480	3040	4570	3110	4670	3170	4760	3230	4860	3290	4950	3360	5040
	2880	4330	2930	4410	2980	4480	3030	4560	3090	4640	3140	4720	3190	4790
	2770	4170	2810	4230	2850	4290	2890	4350	2930	4410	2970	4470	3010	4530
	2660	4000	2690	4040	2720	4090	2750	4130	2780	4170	2810	4220	2830	4260
	2600	3900	2620	3940	2640	3980	2670	4010	2690	4050	2720	4080	2740	4120
	2500	3760	2520	3790	2540	3820	2560	3850	2580	3880	2600	3900	2620	3930
W33×169	2580	3880	2640	3970	2700	4070	2770	4160	2830	4250	2890	4340	2950	4440
	2510	3770	2560	3850	2610	3930	2670	4010	2720	4090	2770	4170	2830	4250
	2430	3650	2470	3720	2520	3790	2560	3850	2610	3920	2650	3990	2700	4050
	2350	3530	2380	3580	2420	3630	2450	3690	2490	3740	2520	3790	2560	3850
	2260	3390	2290	3430	2310	3470	2340	3510	2370	3550	2390	3600	2420	3640
	2180	3280	2200	3310	2230	3350	2250	3380	2270	3410	2290	3440	2310	3470
	2070	3110	2090	3140	2100	3160	2120	3180	2130	3210	2150	3230	2160	3250

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[b] $Y2$ = distance from top of the steel beam to concrete flange force

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft

W33–W30

F_y = 50 ksi

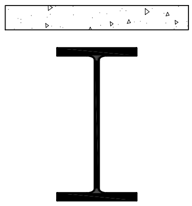
Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n[d]	$Y2$[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W33×152	1390	2100	TFL	0	2250	2100	3160	2160	3240	2210	3330	2270	3410
			2	0.265	1940	2070	3110	2120	3180	2160	3250	2210	3330
			3	0.530	1630	2030	3050	2070	3110	2110	3170	2150	3240
			4	0.795	1320	1990	2990	2020	3040	2060	3090	2090	3140
			BFL	1.06	1020	1950	2920	1970	2960	2000	3000	2020	3040
			6	4.34	788	1890	2850	1910	2870	1930	2900	1950	2930
			7	7.91	561	1800	2710	1820	2730	1830	2750	1840	2770
W33×141	1280	1930	TFL	0	2080	1930	2900	1980	2980	2030	3060	2090	3140
			2	0.240	1800	1900	2860	1950	2930	1990	2990	2040	3060
			3	0.480	1520	1870	2810	1910	2870	1950	2920	1980	2980
			4	0.720	1250	1830	2760	1860	2800	1900	2850	1930	2900
			BFL	0.960	971	1790	2700	1820	2730	1840	2770	1870	2810
			6	4.34	745	1740	2620	1760	2650	1780	2680	1800	2700
			7	8.08	519	1650	2480	1660	2500	1680	2520	1690	2540
W33×130	1170	1750	TFL	0	1920	1770	2660	1820	2740	1870	2810	1920	2880
			2	0.214	1670	1750	2630	1790	2690	1830	2750	1870	2810
			3	0.428	1420	1720	2580	1750	2640	1790	2690	1820	2740
			4	0.641	1180	1690	2540	1720	2580	1750	2620	1780	2670
			BFL	0.855	932	1650	2490	1680	2520	1700	2560	1720	2590
			6	4.39	705	1600	2410	1620	2440	1640	2460	1660	2490
			7	8.30	479	1510	2270	1520	2290	1530	2300	1540	2320
W33×118	1040	1560	TFL	0	1740	1600	2400	1640	2470	1680	2530	1730	2600
			2	0.185	1520	1580	2370	1610	2420	1650	2480	1690	2540
			3	0.370	1310	1550	2330	1580	2380	1620	2430	1650	2480
			4	0.555	1100	1520	2290	1550	2330	1580	2370	1610	2420
			BFL	0.740	884	1500	2250	1520	2280	1540	2320	1560	2350
			6	4.47	659	1450	2170	1460	2200	1480	2220	1500	2250
			7	8.56	434	1350	2030	1360	2050	1370	2060	1380	2080
W30×116	943	1420	TFL	0	1710	1450	2180	1490	2240	1540	2310	1580	2370
			2	0.213	1490	1430	2150	1460	2200	1500	2260	1540	2310
			3	0.425	1260	1400	2110	1430	2150	1460	2200	1500	2250
			4	0.638	1040	1370	2060	1400	2100	1430	2140	1450	2180
			BFL	0.850	818	1340	2020	1360	2050	1380	2080	1400	2110
			6	3.98	623	1300	1960	1320	1980	1330	2000	1350	2030
			7	7.43	428	1230	1840	1240	1860	1250	1870	1260	1890

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Ductility (slip capacity) of the shear connection at the beam/concrete interface may control minimum ΣQ_n requirements per AISC *Specification* Section I3.2d.

F_y = 50 ksi

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft



W33–W30

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W33×152	2320	3490	2380	3580	2440	3660	2490	3750	2550	3830	2600	3910	2660	4000
	2260	3400	2310	3470	2360	3540	2410	3620	2450	3690	2500	3760	2550	3830
	2190	3300	2230	3360	2280	3420	2320	3480	2360	3540	2400	3600	2440	3660
	2120	3190	2160	3240	2190	3290	2220	3340	2250	3390	2290	3440	2320	3490
	2050	3080	2070	3110	2100	3150	2120	3190	2150	3230	2170	3270	2200	3310
	1970	2960	1990	2990	2010	3020	2030	3050	2050	3080	2070	3110	2090	3140
	1860	2790	1870	2810	1890	2830	1900	2850	1910	2880	1930	2900	1940	2920
W33×141	2140	3210	2190	3290	2240	3370	2290	3450	2350	3520	2400	3600	2450	3680
	2080	3130	2130	3200	2170	3260	2220	3330	2260	3400	2310	3470	2350	3530
	2020	3040	2060	3100	2100	3150	2140	3210	2170	3270	2210	3320	2250	3380
	1960	2940	1990	2990	2020	3040	2050	3080	2080	3130	2110	3180	2140	3220
	1890	2840	1920	2880	1940	2920	1960	2950	1990	2990	2010	3020	2040	3060
	1820	2730	1840	2760	1850	2790	1870	2820	1890	2840	1910	2870	1930	2900
	1700	2560	1720	2580	1730	2600	1740	2620	1750	2640	1770	2660	1780	2680
W33×130	1960	2950	2010	3020	2060	3100	2110	3170	2150	3240	2200	3310	2250	3380
	1910	2880	1960	2940	2000	3000	2040	3060	2080	3130	2120	3190	2160	3250
	1860	2800	1900	2850	1930	2900	1970	2960	2000	3010	2040	3060	2070	3120
	1800	2710	1830	2760	1860	2800	1890	2850	1920	2890	1950	2930	1980	2980
	1750	2630	1770	2660	1790	2690	1820	2730	1840	2760	1860	2800	1890	2830
	1670	2510	1690	2540	1710	2570	1730	2590	1740	2620	1760	2650	1780	2670
	1560	2340	1570	2360	1580	2370	1590	2390	1600	2410	1620	2430	1630	2450
W33×118	1770	2660	1810	2730	1860	2790	1900	2860	1940	2920	1990	2990	2030	3050
	1730	2600	1760	2650	1800	2710	1840	2770	1880	2820	1920	2880	1950	2940
	1680	2530	1710	2580	1750	2630	1780	2670	1810	2720	1850	2770	1880	2820
	1630	2460	1660	2500	1690	2540	1720	2580	1740	2620	1770	2660	1800	2700
	1580	2380	1610	2420	1630	2450	1650	2480	1670	2510	1700	2550	1720	2580
	1510	2270	1530	2300	1550	2320	1560	2350	1580	2370	1590	2400	1610	2420
	1390	2100	1410	2110	1420	2130	1430	2140	1440	2160	1450	2180	1460	2190
W30×116	1620	2440	1660	2500	1710	2570	1750	2630	1790	2690	1830	2760	1880	2820
	1580	2370	1610	2420	1650	2480	1690	2540	1720	2590	1760	2650	1800	2700
	1530	2300	1560	2340	1590	2390	1620	2440	1650	2490	1680	2530	1720	2580
	1480	2220	1500	2260	1530	2300	1550	2340	1580	2380	1610	2410	1630	2450
	1420	2140	1440	2170	1470	2200	1490	2230	1510	2260	1530	2290	1550	2320
	1360	2050	1380	2070	1390	2100	1410	2120	1430	2140	1440	2170	1460	2190
	1270	1910	1280	1920	1290	1940	1300	1950	1310	1970	1320	1990	1330	2000

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[b] $Y2$ = distance from top of the steel beam to concrete flange force

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft

W30–W27

F_y = 50 ksi

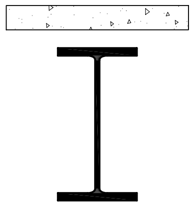
Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n[d]	$Y2$[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W30×108	863	1300	TFL	0	1590	1340	2010	1380	2070	1420	2130	1460	2190
			2	0.190	1390	1320	1980	1350	2030	1380	2080	1420	2130
			3	0.380	1190	1290	1940	1320	1990	1350	2030	1380	2080
			4	0.570	987	1270	1910	1290	1940	1320	1980	1340	2020
			BFL	0.760	787	1240	1870	1260	1900	1280	1930	1300	1960
			6	4.04	592	1200	1800	1210	1830	1230	1850	1240	1870
			7	7.63	396	1120	1690	1130	1700	1140	1720	1150	1730
W30×99	778	1170	TFL	0	1450	1220	1830	1260	1890	1290	1940	1330	2000
			2	0.168	1270	1200	1800	1230	1850	1260	1900	1300	1950
			3	0.335	1100	1180	1780	1210	1820	1240	1860	1260	1900
			4	0.503	922	1160	1740	1180	1780	1210	1810	1230	1850
			BFL	0.670	747	1140	1710	1160	1740	1170	1770	1190	1790
			6	4.19	555	1100	1650	1110	1670	1120	1690	1140	1710
			7	7.88	363	1020	1530	1030	1540	1040	1560	1050	1570
W30×90	706	1060	TFL	0	1320	1100	1650	1130	1700	1160	1750	1200	1800
			2	0.153	1160	1080	1630	1110	1670	1140	1710	1170	1760
			3	0.305	998	1070	1600	1090	1640	1110	1680	1140	1710
			4	0.458	839	1050	1570	1070	1600	1090	1640	1110	1670
			BFL	0.610	681	1030	1540	1040	1570	1060	1590	1080	1620
			6	4.01	505	989	1490	1000	1510	1010	1530	1030	1540
			7	7.76	329	920	1380	928	1400	937	1410	945	1420
W27×102	761	1140	TFL	0	1500	1160	1750	1200	1810	1240	1860	1280	1920
			2	0.208	1290	1140	1720	1170	1770	1210	1810	1240	1860
			3	0.415	1090	1120	1680	1150	1720	1170	1760	1200	1800
			4	0.623	878	1090	1640	1110	1670	1140	1710	1160	1740
			BFL	0.830	670	1060	1600	1080	1620	1100	1650	1110	1670
			6	3.40	523	1030	1550	1050	1570	1060	1590	1070	1610
			7	6.27	375	984	1480	993	1490	1000	1510	1010	1520
W27×94	694	1040	TFL	0	1380	1060	1600	1100	1650	1130	1700	1170	1750
			2	0.186	1190	1040	1570	1070	1610	1100	1660	1130	1700
			3	0.373	1010	1020	1540	1050	1580	1070	1610	1100	1650
			4	0.559	821	1000	1500	1020	1530	1040	1570	1060	1600
			BFL	0.745	635	976	1470	992	1490	1010	1510	1020	1540
			6	3.45	490	947	1420	959	1440	971	1460	983	1480
			7	6.41	345	897	1350	905	1360	914	1370	922	1390

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Ductility (slip capacity) of the shear connection at the beam/concrete interface may control minimum ΣQ_n requirements per AISC *Specification* Section I3.2d.

F_y = 50 ksi

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft

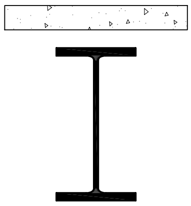


W30–W27

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W30×108	1490	2250	1530	2310	1570	2370	1610	2430	1650	2480	1690	2540	1730	2600
	1450	2190	1490	2240	1520	2290	1560	2340	1590	2390	1630	2450	1660	2500
	1410	2120	1440	2170	1470	2210	1500	2260	1530	2300	1560	2340	1590	2390
	1370	2050	1390	2090	1420	2130	1440	2170	1470	2200	1490	2240	1510	2280
	1320	1980	1340	2010	1360	2040	1380	2070	1400	2100	1420	2130	1440	2160
	1260	1890	1270	1910	1290	1940	1300	1960	1320	1980	1330	2000	1350	2030
	1160	1750	1170	1760	1180	1780	1190	1790	1200	1810	1210	1820	1220	1840
W30×99	1360	2050	1400	2100	1440	2160	1470	2210	1510	2270	1540	2320	1580	2380
	1330	2000	1360	2040	1390	2090	1420	2140	1460	2190	1490	2230	1520	2280
	1290	1940	1320	1980	1350	2020	1370	2060	1400	2100	1430	2150	1460	2190
	1250	1880	1270	1920	1300	1950	1320	1990	1340	2020	1370	2050	1390	2090
	1210	1820	1230	1850	1250	1880	1270	1910	1290	1930	1300	1960	1320	1990
	1150	1730	1160	1750	1180	1770	1190	1790	1210	1810	1220	1830	1230	1850
	1050	1590	1060	1600	1070	1610	1080	1630	1090	1640	1100	1650	1110	1670
W30×90	1230	1850	1260	1900	1300	1950	1330	2000	1360	2050	1390	2100	1430	2150
	1200	1800	1230	1840	1260	1890	1280	1930	1310	1970	1340	2020	1370	2060
	1160	1750	1190	1790	1210	1830	1240	1860	1260	1900	1290	1940	1310	1970
	1130	1700	1150	1730	1170	1760	1190	1790	1210	1820	1230	1860	1260	1890
	1090	1640	1110	1670	1130	1700	1150	1720	1160	1750	1180	1770	1200	1800
	1040	1560	1050	1580	1070	1600	1080	1620	1090	1640	1100	1660	1120	1680
	953	1430	961	1440	969	1460	978	1470	986	1480	994	1490	1000	1510
W27×102	1310	1970	1350	2030	1390	2090	1430	2140	1460	2200	1500	2260	1540	2310
	1270	1910	1300	1960	1340	2010	1370	2060	1400	2100	1430	2150	1460	2200
	1230	1840	1250	1880	1280	1930	1310	1970	1340	2010	1360	2050	1390	2090
	1180	1770	1200	1810	1220	1840	1250	1870	1270	1900	1290	1940	1310	1970
	1130	1700	1150	1720	1160	1750	1180	1770	1200	1800	1210	1830	1230	1850
	1090	1630	1100	1650	1110	1670	1130	1690	1140	1710	1150	1730	1160	1750
	1020	1540	1030	1550	1040	1560	1050	1580	1060	1590	1070	1610	1080	1620
W27×94	1200	1810	1240	1860	1270	1910	1300	1960	1340	2010	1370	2060	1410	2120
	1160	1750	1190	1790	1220	1840	1250	1880	1280	1930	1310	1970	1340	2020
	1120	1690	1150	1730	1170	1760	1200	1800	1220	1840	1250	1880	1270	1920
	1080	1630	1110	1660	1120	1690	1140	1720	1160	1750	1180	1780	1210	1810
	1040	1560	1050	1590	1070	1610	1090	1630	1100	1660	1120	1680	1130	1700
	996	1500	1010	1510	1020	1530	1030	1550	1040	1570	1060	1590	1070	1610
	931	1400	940	1410	948	1430	957	1440	965	1450	974	1460	983	1480

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

b $Y2$ = distance from top of the steel beam to concrete flange force



W27–W24

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft

F_y = 50 ksi

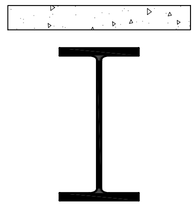
Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n[d]	$Y2$[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W27×84	609	915	TFL	0	1240	946	1420	977	1470	1010	1510	1040	1560
			2	0.160	1080	929	1400	956	1440	983	1480	1010	1520
			3	0.320	915	911	1370	934	1400	957	1440	980	1470
			4	0.480	755	892	1340	911	1370	930	1400	949	1430
			BFL	0.640	595	872	1310	887	1330	902	1360	916	1380
			6	3.53	452	843	1270	855	1280	866	1300	877	1320
			7	6.64	309	793	1190	800	1200	808	1210	816	1230
W24×94	634	953	TFL	0	1390	978	1470	1010	1520	1050	1570	1080	1630
			2	0.219	1190	957	1440	987	1480	1020	1530	1050	1570
			3	0.438	988	934	1400	959	1440	983	1480	1010	1510
			4	0.656	790	909	1370	928	1400	948	1430	968	1450
			BFL	0.875	591	881	1320	896	1350	911	1370	926	1390
			6	3.05	469	858	1290	869	1310	881	1320	893	1340
			7	5.43	346	819	1230	828	1240	837	1260	845	1270
W24×84	559	840	TFL	0	1240	866	1300	897	1350	927	1390	958	1440
			2	0.193	1060	848	1270	874	1310	901	1350	927	1390
			3	0.385	888	828	1240	850	1280	872	1310	894	1340
			4	0.578	714	806	1210	824	1240	842	1270	860	1290
			BFL	0.770	540	783	1180	797	1200	810	1220	824	1240
			6	3.02	425	761	1140	772	1160	782	1180	793	1190
			7	5.48	309	725	1090	733	1100	740	1110	748	1120
W24×76	499	750	TFL	0	1120	780	1170	808	1210	836	1260	863	1300
			2	0.170	967	764	1150	788	1180	812	1220	836	1260
			3	0.340	814	747	1120	767	1150	787	1180	807	1210
			4	0.510	662	728	1090	745	1120	761	1140	778	1170
			BFL	0.680	509	708	1060	721	1080	734	1100	746	1120
			6	2.99	394	687	1030	697	1050	707	1060	716	1080
			7	5.59	280	651	979	658	989	665	1000	672	1010
W24×68	442	664	TFL	0	1010	695	1040	720	1080	745	1120	770	1160
			2	0.146	874	681	1020	703	1060	725	1090	746	1120
			3	0.293	743	666	1000	685	1030	704	1060	722	1090
			4	0.439	611	651	978	666	1000	681	1020	697	1050
			BFL	0.585	480	635	954	647	972	658	990	670	1010
			6	3.04	366	613	922	623	936	632	949	641	963
			7	5.80	251	577	867	583	876	589	886	595	895

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Ductility (slip capacity) of the shear connection at the beam/concrete interface may control minimum ΣQ_n requirements per AISC *Specification* Section I3.2d.

F_y = 50 ksi

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft

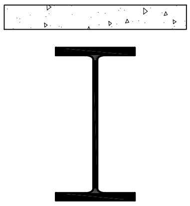


W27–W24

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W27×84	1070	1610	1100	1650	1130	1700	1160	1750	1190	1790	1220	1840	1250	1880
	1040	1560	1060	1600	1090	1640	1120	1680	1140	1720	1170	1760	1200	1800
	1000	1510	1030	1540	1050	1580	1070	1610	1090	1640	1120	1680	1140	1710
	968	1450	987	1480	1010	1510	1020	1540	1040	1570	1060	1600	1080	1620
	931	1400	946	1420	961	1440	976	1470	991	1490	1010	1510	1020	1530
	888	1340	900	1350	911	1370	922	1390	933	1400	945	1420	956	1440
	824	1240	831	1250	839	1260	847	1270	854	1280	862	1300	870	1310
W24×94	1120	1680	1150	1730	1190	1780	1220	1830	1250	1890	1290	1940	1320	1990
	1080	1620	1110	1660	1130	1710	1160	1750	1190	1790	1220	1840	1250	1880
	1030	1550	1060	1590	1080	1630	1110	1660	1130	1700	1160	1740	1180	1770
	988	1480	1010	1510	1030	1540	1050	1570	1070	1600	1090	1630	1110	1660
	940	1410	955	1440	970	1460	985	1480	999	1500	1010	1520	1030	1550
	904	1360	916	1380	928	1390	939	1410	951	1430	963	1450	975	1460
	854	1280	863	1300	871	1310	880	1320	888	1340	897	1350	906	1360
W24×84	989	1490	1020	1530	1050	1580	1080	1630	1110	1670	1140	1720	1170	1760
	954	1430	980	1470	1010	1510	1030	1550	1060	1590	1090	1630	1110	1670
	916	1380	939	1410	961	1440	983	1480	1010	1510	1030	1540	1050	1580
	878	1320	895	1350	913	1370	931	1400	949	1430	967	1450	985	1480
	837	1260	851	1280	864	1300	878	1320	891	1340	904	1360	918	1380
	804	1210	814	1220	825	1240	835	1260	846	1270	856	1290	867	1300
	756	1140	764	1150	771	1160	779	1170	787	1180	794	1190	802	1210
W24×76	891	1340	919	1380	947	1420	975	1470	1000	1510	1030	1550	1060	1590
	860	1290	884	1330	909	1370	933	1400	957	1440	981	1470	1010	1510
	828	1240	848	1270	868	1310	889	1340	909	1370	929	1400	950	1430
	794	1190	811	1220	827	1240	844	1270	860	1290	877	1320	893	1340
	759	1140	772	1160	784	1180	797	1200	810	1220	823	1240	835	1260
	726	1090	736	1110	746	1120	756	1140	766	1150	775	1170	785	1180
	679	1020	686	1030	693	1040	700	1050	707	1060	714	1070	721	1080
W24×68	795	1190	820	1230	845	1270	870	1310	895	1350	920	1380	945	1420
	768	1150	790	1190	812	1220	834	1250	855	1290	877	1320	899	1350
	741	1110	759	1140	778	1170	796	1200	815	1220	833	1250	852	1280
	712	1070	727	1090	742	1120	758	1140	773	1160	788	1180	804	1210
	682	1030	694	1040	706	1060	718	1080	730	1100	742	1120	754	1130
	650	977	659	990	668	1000	677	1020	686	1030	696	1050	705	1060
	602	904	608	914	614	923	620	933	627	942	633	951	639	961

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[b] $Y2$ = distance from top of the steel beam to concrete flange force



W24–W21

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft

F_y = 50 ksi

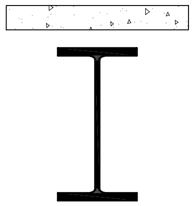
Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n[d]	$Y2$[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W24×62	382	574	TFL	0	910	629	945	652	979	674	1010	697	1050
			2	0.148	806	618	929	638	959	658	990	679	1020
			3	0.295	702	607	912	624	938	642	964	659	991
			4	0.443	598	594	893	609	916	624	938	639	961
			BFL	0.590	495	581	874	594	892	606	911	618	929
			6	3.45	361	555	834	564	848	573	862	582	875
			7	6.56	228	509	764	514	773	520	781	526	790
W24×55	334	503	TFL	0	810	558	838	578	869	598	899	618	929
			2	0.126	721	549	825	567	852	585	879	603	906
			3	0.253	633	539	810	555	834	571	858	586	881
			4	0.379	544	529	795	542	815	556	836	570	856
			BFL	0.505	456	518	779	529	796	541	813	552	830
			6	3.46	329	493	742	502	754	510	766	518	779
			7	6.67	203	449	675	454	682	459	690	464	697
W21×73	429	645	TFL	0	1080	676	1020	703	1060	730	1100	756	1140
			2	0.185	921	660	992	683	1030	706	1060	729	1100
			3	0.370	768	642	966	662	994	681	1020	700	1050
			4	0.555	614	624	937	639	960	654	983	670	1010
			BFL	0.740	461	603	907	615	924	626	941	638	959
			6	2.58	365	586	881	595	895	604	908	613	922
			7	4.69	269	559	840	566	851	573	861	579	871
W21×68	399	600	TFL	0	1000	626	941	651	979	676	1020	701	1050
			2	0.171	858	612	919	633	951	654	983	676	1020
			3	0.343	717	596	895	613	922	631	949	649	976
			4	0.514	575	578	869	593	891	607	912	621	934
			BFL	0.685	434	560	842	571	858	582	874	593	891
			6	2.60	342	544	817	552	830	561	843	569	856
			7	4.74	250	518	778	524	787	530	797	536	806
W21×62	359	540	TFL	0	915	571	858	594	892	616	926	639	961
			2	0.154	788	558	838	577	868	597	897	617	927
			3	0.308	662	544	817	560	842	577	867	593	891
			4	0.461	535	528	794	542	814	555	834	568	854
			BFL	0.615	408	512	770	523	785	533	801	543	816
			6	2.54	318	497	747	505	759	513	771	521	782
			7	4.78	229	472	709	477	717	483	726	489	734

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Ductility (slip capacity) of the shear connection at the beam/concrete interface may control minimum ΣQ_n requirements per AISC *Specification* Section I3.2d.

F_y = 50 ksi

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft



W24–W21

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W24×62	720	1080	742	1120	765	1150	788	1180	811	1220	833	1250	856	1290
	699	1050	719	1080	739	1110	759	1140	779	1170	799	1200	819	1230
	677	1020	694	1040	712	1070	729	1100	747	1120	764	1150	782	1180
	654	983	669	1010	684	1030	699	1050	714	1070	729	1100	744	1120
	631	948	643	967	655	985	668	1000	680	1020	692	1040	705	1060
	591	889	600	902	609	916	618	929	627	943	636	956	645	970
	531	798	537	807	543	816	548	824	554	833	560	841	565	850
W24×55	639	960	659	990	679	1020	699	1050	719	1080	740	1110	760	1140
	621	933	639	960	657	987	675	1010	693	1040	711	1070	729	1100
	602	905	618	929	634	953	650	976	665	1000	681	1020	697	1050
	583	876	597	897	610	917	624	938	637	958	651	978	665	999
	564	847	575	864	586	881	598	898	609	915	620	932	632	950
	526	791	534	803	543	816	551	828	559	840	567	853	576	865
	469	705	474	713	479	720	484	728	489	735	494	743	499	751
W21×73	783	1180	810	1220	837	1260	864	1300	890	1340	917	1380	944	1420
	752	1130	775	1160	798	1200	821	1230	844	1270	867	1300	890	1340
	719	1080	738	1110	757	1140	777	1170	796	1200	815	1220	834	1250
	685	1030	700	1050	715	1080	731	1100	746	1120	761	1140	777	1170
	649	976	661	993	672	1010	684	1030	695	1040	707	1060	718	1080
	623	936	632	949	641	963	650	977	659	990	668	1000	677	1020
	586	881	593	891	599	901	606	911	613	921	620	931	626	941
W21×68	726	1090	751	1130	776	1170	801	1200	826	1240	851	1280	876	1320
	697	1050	719	1080	740	1110	761	1140	783	1180	804	1210	826	1240
	667	1000	685	1030	703	1060	721	1080	739	1110	757	1140	774	1160
	636	956	650	977	664	999	679	1020	693	1040	708	1060	722	1080
	603	907	614	923	625	939	636	956	647	972	657	988	668	1000
	578	868	586	881	595	894	603	907	612	920	620	933	629	945
	543	816	549	825	555	834	561	844	568	853	574	862	580	872
W21×62	662	995	685	1030	708	1060	731	1100	753	1130	776	1170	799	1200
	636	956	656	986	676	1020	695	1050	715	1070	735	1100	754	1130
	610	916	626	941	643	966	659	991	676	1020	692	1040	709	1070
	582	874	595	895	609	915	622	935	635	955	649	975	662	995
	553	831	563	847	573	862	584	877	594	893	604	908	614	923
	529	794	536	806	544	818	552	830	560	842	568	854	576	866
	494	743	500	752	506	760	511	769	517	777	523	786	529	795

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[b] $Y2$ = distance from top of the steel beam to concrete flange force

W21

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft

F_y = 50 ksi

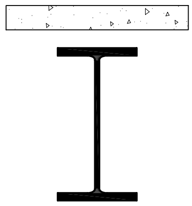
Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n[d]	$Y2$[b], in.							
	kip-ft			in.	kip	2		2.5		3		3.5	
	ASD	LRFD				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W21×57	322	484	TFL	0	835	523	786	544	817	565	849	585	880
			2	0.163	728	512	769	530	797	548	824	566	851
			3	0.325	622	500	751	515	775	531	798	546	821
			4	0.488	515	487	732	500	751	513	771	526	790
			BFL	0.650	409	473	712	484	727	494	742	504	758
			6	2.93	309	455	684	463	695	470	707	478	718
			7	5.40	209	424	637	429	645	435	653	440	661
W21×55	314	473	TFL	0	810	501	753	521	784	542	814	562	844
			2	0.131	703	490	737	508	763	525	789	543	816
			3	0.261	595	478	719	493	741	508	764	523	786
			4	0.392	488	466	700	478	719	490	737	502	755
			BFL	0.522	381	453	681	462	695	472	709	481	723
			6	2.62	292	437	657	445	668	452	679	459	690
			7	5.00	203	411	618	417	626	422	634	427	641
W21×50	274	413	TFL	0	735	455	684	473	711	491	739	510	766
			2	0.134	648	446	670	462	694	478	719	494	743
			3	0.268	560	436	656	450	677	464	698	478	719
			4	0.401	473	426	640	438	658	450	676	461	694
			BFL	0.535	386	415	624	425	639	435	653	444	668
			6	2.91	285	397	597	404	607	411	618	418	629
			7	5.56	184	366	550	370	557	375	563	379	570
W21×48	265	398	TFL	0	705	433	650	450	677	468	703	485	730
			2	0.108	617	424	637	439	660	455	683	470	706
			3	0.215	530	414	623	428	643	441	662	454	682
			4	0.323	442	404	608	415	624	426	641	437	658
			BFL	0.430	355	394	592	403	606	412	619	421	632
			6	2.71	266	379	569	385	579	392	589	398	599
			7	5.26	176	352	529	356	535	361	542	365	549
W21×44	238	358	TFL	0	650	401	602	417	626	433	651	449	675
			2	0.113	577	393	591	407	612	422	634	436	656
			3	0.225	504	385	579	398	598	410	617	423	636
			4	0.338	431	377	566	388	583	398	599	409	615
			BFL	0.450	358	368	553	377	567	386	580	395	594
			6	2.92	260	351	527	357	537	364	547	370	556
			7	5.71	163	320	481	324	487	328	493	332	499

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Ductility (slip capacity) of the shear connection at the beam/concrete interface may control minimum ΣQ_n requirements per AISC *Specification* Section I3.2d.

F_y = 50 ksi

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft

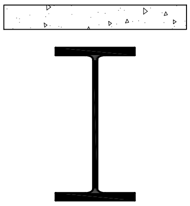


W21

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W21×57	606	911	627	943	648	974	669	1010	690	1040	710	1070	731	1100
	585	879	603	906	621	933	639	960	657	988	675	1020	694	1040
	562	845	577	868	593	891	609	915	624	938	640	961	655	985
	539	809	551	829	564	848	577	867	590	887	603	906	616	925
	514	773	524	788	535	804	545	819	555	834	565	850	575	865
	486	730	493	742	501	753	509	765	517	776	524	788	532	800
	445	669	450	677	455	684	461	692	466	700	471	708	476	716
W21×55	582	875	602	905	622	936	643	966	663	996	683	1030	703	1060
	560	842	578	868	595	895	613	921	630	948	648	974	665	1000
	538	808	553	831	568	853	582	875	597	898	612	920	627	942
	515	774	527	792	539	810	551	828	563	847	576	865	588	883
	491	738	500	752	510	766	519	781	529	795	538	809	548	823
	466	701	474	712	481	723	488	734	496	745	503	756	510	767
	432	649	437	656	442	664	447	672	452	679	457	687	462	695
W21×50	528	794	546	821	565	849	583	876	601	904	620	932	638	959
	510	767	527	791	543	816	559	840	575	864	591	889	607	913
	492	740	506	761	520	782	534	803	548	824	562	845	576	866
	473	711	485	729	497	747	509	764	520	782	532	800	544	818
	454	682	463	696	473	711	483	725	492	740	502	754	512	769
	425	639	433	650	440	661	447	671	454	682	461	693	468	704
	384	577	389	584	393	591	398	598	402	605	407	612	412	619
W21×48	503	756	521	783	538	809	556	835	573	862	591	888	609	915
	485	729	501	753	516	776	532	799	547	822	562	845	578	868
	467	702	480	722	494	742	507	762	520	782	533	802	547	821
	449	674	460	691	471	707	482	724	493	741	504	757	515	774
	429	645	438	659	447	672	456	685	465	699	474	712	483	725
	405	609	412	619	418	629	425	639	432	649	438	659	445	669
	369	555	374	562	378	568	383	575	387	582	391	588	396	595
W21×44	465	700	482	724	498	748	514	773	530	797	547	821	563	846
	451	677	465	699	479	721	494	742	508	764	523	785	537	807
	435	654	448	673	461	692	473	711	486	730	498	749	511	768
	420	631	431	647	441	663	452	679	463	696	474	712	484	728
	404	607	413	620	422	634	431	647	440	661	448	674	457	687
	377	566	383	576	390	586	396	595	403	605	409	615	416	625
	336	505	340	511	344	518	348	524	352	530	357	536	361	542

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[b] $Y2$ = distance from top of the steel beam to concrete flange force



W18

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft

F_y = 50 ksi

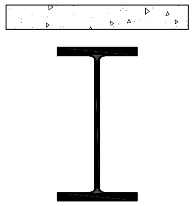
Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n[d]	$Y2$[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W18×60	307	461	TFL	0	880	487	733	509	766	531	799	553	832
			2	0.174	749	474	712	492	740	511	768	530	796
			3	0.348	617	459	690	474	713	490	736	505	759
			4	0.521	486	443	666	455	684	467	702	479	720
			BFL	0.695	355	426	640	435	653	444	667	452	680
			6	2.18	287	414	623	422	634	429	644	436	655
			7	3.80	220	398	598	403	606	409	614	414	623
W18×55	279	420	TFL	0	810	447	671	467	702	487	732	507	762
			2	0.158	691	434	653	452	679	469	705	486	731
			3	0.315	573	421	633	435	654	450	676	464	697
			4	0.473	454	407	612	418	629	430	646	441	663
			BFL	0.630	336	392	589	400	602	409	614	417	627
			6	2.15	269	381	572	387	582	394	592	401	603
			7	3.86	203	364	547	369	555	374	563	379	570
W18×50	252	379	TFL	0	735	403	606	422	634	440	662	458	689
			2	0.143	628	392	590	408	613	424	637	439	660
			3	0.285	521	381	572	394	592	407	611	420	631
			4	0.428	414	368	553	378	569	389	584	399	600
			BFL	0.570	308	355	533	362	545	370	556	378	568
			6	2.08	246	345	518	351	527	357	537	363	546
			7	3.82	184	329	495	334	502	339	509	343	516
W18×46	226	340	TFL	0	675	372	559	389	585	406	610	423	635
			2	0.151	583	363	545	377	567	392	589	406	611
			3	0.303	492	353	530	365	548	377	567	389	585
			4	0.454	400	342	513	352	528	362	543	372	558
			BFL	0.605	308	330	496	338	508	345	519	353	531
			6	2.42	239	318	478	324	487	330	496	336	505
			7	4.36	169	299	450	303	456	308	462	312	469
W18×40	196	294	TFL	0	590	322	485	337	507	352	529	367	551
			2	0.131	511	314	472	327	491	340	511	352	530
			3	0.263	432	306	459	316	475	327	492	338	508
			4	0.394	353	296	445	305	459	314	472	323	485
			BFL	0.525	274	287	431	294	441	300	451	307	462
			6	2.26	211	276	415	282	423	287	431	292	439
			7	4.27	148	260	390	263	396	267	401	271	407

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Ductility (slip capacity) of the shear connection at the beam/concrete interface may control minimum ΣQ_n requirements per AISC *Specification* Section I3.2d.

F_y = 50 ksi

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft

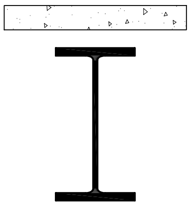


W18

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W18×60	575	865	597	898	619	931	641	964	663	997	685	1030	707	1060
	548	824	567	852	586	880	605	909	623	937	642	965	661	993
	521	782	536	805	551	829	567	852	582	875	598	898	613	921
	491	739	504	757	516	775	528	793	540	812	552	830	564	848
	461	693	470	707	479	720	488	733	497	747	506	760	514	773
	443	666	450	677	457	688	465	698	472	709	479	720	486	731
	420	631	425	639	431	647	436	656	442	664	447	672	453	680
W18×55	527	793	548	823	568	854	588	884	608	914	629	945	649	975
	503	756	521	782	538	808	555	834	572	860	590	886	607	912
	478	719	493	740	507	762	521	783	535	805	550	826	564	848
	452	680	464	697	475	714	486	731	498	748	509	765	520	782
	425	639	434	652	442	664	450	677	459	690	467	702	476	715
	408	613	414	623	421	633	428	643	434	653	441	663	448	673
	384	578	389	585	395	593	400	601	405	608	410	616	415	623
W18×50	477	717	495	744	513	772	532	799	550	827	568	854	587	882
	455	684	471	708	486	731	502	755	518	778	533	802	549	825
	433	650	446	670	459	689	472	709	485	728	498	748	511	767
	409	615	420	631	430	646	440	662	451	677	461	693	471	708
	385	579	393	591	401	602	408	614	416	625	424	637	431	649
	369	555	375	564	381	573	388	583	394	592	400	601	406	610
	348	523	352	530	357	537	362	543	366	550	371	557	375	564
W18×46	440	661	456	686	473	711	490	737	507	762	524	787	541	813
	421	633	435	655	450	676	465	698	479	720	494	742	508	764
	402	604	414	622	426	640	438	659	451	677	463	696	475	714
	382	573	392	588	402	603	412	618	421	633	431	648	441	663
	361	542	369	554	376	565	384	577	392	589	399	600	407	612
	342	514	348	523	354	532	360	541	366	550	372	559	378	568
	316	475	320	481	325	488	329	494	333	500	337	507	341	513
W18×40	381	573	396	595	411	617	425	639	440	662	455	684	470	706
	365	549	378	568	391	587	403	606	416	626	429	645	442	664
	349	524	359	540	370	556	381	573	392	589	403	605	413	621
	332	498	340	512	349	525	358	538	367	551	376	565	384	578
	314	472	321	482	328	493	335	503	341	513	348	523	355	534
	297	447	303	455	308	463	313	471	318	479	324	486	329	494
	274	412	278	418	282	424	286	429	289	435	293	440	297	446

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[b] $Y2$ = distance from top of the steel beam to concrete flange force



W18–W16

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft

F_y = 50 ksi

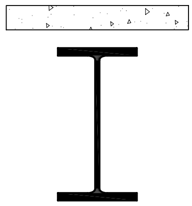
Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n[d]	$Y2$[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W18×35	166	249	TFL	0	515	279	419	292	438	305	458	317	477
			2	0.106	451	272	409	284	426	295	443	306	460
			3	0.213	388	265	399	275	413	285	428	294	443
			4	0.319	324	258	388	266	400	274	412	282	425
			BFL	0.425	260	251	377	257	387	264	396	270	406
			6	2.37	194	240	360	245	368	250	375	254	382
			7	4.56	129	222	334	225	338	228	343	232	348
W16×45	205	309	TFL	0	665	333	501	350	526	367	551	383	576
			2	0.141	566	323	486	337	507	351	528	366	549
			3	0.283	466	312	469	324	487	336	504	347	522
			4	0.424	367	301	452	310	466	319	479	328	493
			BFL	0.565	267	288	433	295	443	302	453	308	463
			6	1.77	217	280	421	286	430	291	438	297	446
			7	3.23	166	269	404	273	411	277	417	281	423
W16×40	182	274	TFL	0	590	294	443	309	465	324	487	339	509
			2	0.126	502	285	429	298	448	310	466	323	485
			3	0.253	413	276	414	286	430	296	445	307	461
			4	0.379	325	265	399	274	411	282	423	290	436
			BFL	0.505	237	255	383	261	392	267	401	272	409
			6	1.70	192	248	373	253	380	258	387	262	394
			7	3.16	148	238	358	242	363	246	369	249	375
W16×36	160	240	TFL	0	530	263	396	276	415	290	435	303	455
			2	0.108	455	255	384	267	401	278	418	289	435
			3	0.215	380	247	372	257	386	266	400	276	414
			4	0.323	305	239	359	246	370	254	382	262	393
			BFL	0.430	229	230	346	236	354	241	363	247	371
			6	1.82	181	223	334	227	341	232	348	236	355
			7	3.46	133	211	318	215	323	218	328	221	333
W16×31	135	203	TFL	0	457	227	341	238	358	249	375	261	392
			2	0.110	396	220	331	230	346	240	361	250	376
			3	0.220	335	214	321	222	334	231	347	239	359
			4	0.330	274	207	311	214	321	221	332	227	342
			BFL	0.440	213	200	300	205	308	210	316	216	324
			6	2.00	164	192	289	196	295	200	301	204	307
			7	3.80	114	180	270	183	275	186	279	188	283

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Ductility (slip capacity) of the shear connection at the beam/concrete interface may control minimum ΣQ_n requirements per AISC *Specification* Section I3.2d.

F_y = 50 ksi

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft

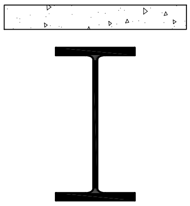


W18–W16

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W18×35	330	496	343	516	356	535	369	554	382	574	394	593	407	612
	317	477	329	494	340	511	351	528	362	545	374	562	385	578
	304	457	314	472	323	486	333	501	343	515	352	530	362	544
	291	437	299	449	307	461	315	473	323	485	331	497	339	510
	277	416	283	426	290	435	296	445	303	455	309	465	316	474
	259	390	264	397	269	404	274	411	279	419	283	426	288	433
	235	353	238	358	241	363	244	367	248	372	251	377	254	382
W16×45	400	601	416	626	433	651	450	676	466	701	483	726	499	751
	380	571	394	592	408	613	422	634	436	655	450	677	464	698
	359	539	370	557	382	574	394	592	405	609	417	627	429	644
	337	507	346	521	355	534	365	548	374	562	383	576	392	589
	315	473	322	483	328	493	335	503	342	513	348	523	355	533
	302	454	307	462	313	470	318	478	324	486	329	495	334	503
	286	429	290	436	294	442	298	448	302	454	306	460	310	467
W16×40	353	531	368	553	383	575	397	597	412	620	427	642	442	664
	335	504	348	523	360	542	373	561	385	579	398	598	410	617
	317	476	327	492	338	507	348	523	358	538	368	554	379	569
	298	448	306	460	314	472	322	484	330	496	338	509	347	521
	278	418	284	427	290	436	296	445	302	454	308	463	314	472
	267	401	272	409	277	416	282	423	286	430	291	438	296	445
	253	380	257	386	260	391	264	397	268	402	271	408	275	413
W16×36	316	475	329	495	342	515	356	535	369	555	382	574	395	594
	301	452	312	469	324	486	335	503	346	520	358	537	369	555
	285	429	295	443	304	457	314	471	323	486	333	500	342	514
	269	405	277	416	284	428	292	439	300	450	307	462	315	473
	253	380	259	389	264	397	270	406	276	414	281	423	287	432
	241	362	245	368	250	375	254	382	259	389	263	396	268	402
	225	338	228	343	231	348	235	353	238	358	241	363	245	367
W16×31	272	409	284	426	295	443	306	460	318	478	329	495	341	512
	260	391	270	405	280	420	290	435	299	450	309	465	319	480
	247	372	256	384	264	397	272	409	281	422	289	434	297	447
	234	352	241	362	248	373	255	383	262	393	268	404	275	414
	221	332	226	340	232	348	237	356	242	364	248	372	253	380
	208	313	212	319	216	325	221	332	225	338	229	344	233	350
	191	287	194	292	197	296	200	300	203	304	205	309	208	313

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[b] $Y2$ = distance from top of the steel beam to concrete flange force



W16–W14

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft

F_y = 50 ksi

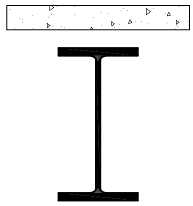
Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n[d]	$Y2$[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W16×26	110	166	TFL	0	384	189	284	198	298	208	312	217	327
			2	0.0863	337	184	276	192	289	201	302	209	314
			3	0.173	289	179	269	186	280	193	291	201	301
			4	0.259	242	174	261	180	270	186	279	192	288
			BFL	0.345	194	168	253	173	260	178	267	183	275
			6	2.05	145	161	241	164	247	168	252	171	258
			7	4.01	96.0	148	223	151	226	153	230	155	234
W14×38	153	231	TFL	0	560	253	380	267	401	281	422	295	443
			2	0.129	473	244	367	256	384	268	402	279	420
			3	0.258	386	234	352	244	367	254	381	263	396
			4	0.386	299	224	337	232	348	239	360	247	371
			BFL	0.515	211	214	321	219	329	224	337	229	345
			6	1.38	176	209	313	213	320	217	327	222	333
			7	2.53	140	201	303	205	308	208	313	212	319
W14×34	136	205	TFL	0	500	225	338	237	356	250	375	262	394
			2	0.114	423	217	326	227	342	238	357	248	373
			3	0.228	346	208	313	217	326	226	339	234	352
			4	0.341	270	200	300	206	310	213	320	220	330
			BFL	0.455	193	190	286	195	293	200	301	205	308
			6	1.42	159	186	279	190	285	193	291	197	297
			7	2.61	125	179	269	182	273	185	278	188	283
W14×30	118	177	TFL	0	443	197	295	208	312	219	329	230	345
			2	0.0963	378	190	285	199	300	209	314	218	328
			3	0.193	313	183	275	191	287	199	298	206	310
			4	0.289	248	176	264	182	273	188	283	194	292
			BFL	0.385	183	168	253	173	260	177	266	182	273
			6	1.46	147	163	245	167	250	170	256	174	261
			7	2.80	111	156	234	158	238	161	242	164	246
W14×26	100	151	TFL	0	385	172	258	181	273	191	287	201	301
			2	0.105	332	166	250	175	262	183	275	191	287
			3	0.210	279	161	241	168	252	175	262	182	273
			4	0.315	226	155	232	160	241	166	249	172	258
			BFL	0.420	173	148	223	153	230	157	236	161	243
			6	1.67	135	143	215	146	220	149	225	153	230
			7	3.18	96.1	134	202	137	205	139	209	141	213

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Ductility (slip capacity) of the shear connection at the beam/concrete interface may control minimum ΣQ_n requirements per AISC *Specification* Section I3.2d.

F_y = 50 ksi

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft



W16–W14

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W16×26	227	341	237	356	246	370	256	384	265	399	275	413	285	428
	218	327	226	340	234	352	243	365	251	377	259	390	268	403
	208	312	215	323	222	334	229	345	237	356	244	366	251	377
	198	297	204	306	210	315	216	324	222	333	228	343	234	352
	188	282	192	289	197	296	202	304	207	311	212	318	217	326
	175	263	179	268	182	274	186	279	189	285	193	290	197	296
	158	237	160	241	163	244	165	248	167	252	170	255	172	259
W14×38	309	464	323	485	337	506	351	527	365	548	379	569	393	590
	291	438	303	455	315	473	327	491	338	508	350	526	362	544
	273	410	283	425	292	439	302	454	311	468	321	482	331	497
	254	382	262	393	269	404	276	416	284	427	291	438	299	449
	235	353	240	361	245	369	250	376	256	384	261	392	266	400
	226	340	230	346	235	353	239	360	244	366	248	373	252	379
	215	324	219	329	222	334	226	340	229	345	233	350	236	355
W14×34	274	413	287	431	299	450	312	469	324	488	337	506	349	525
	259	389	269	405	280	421	291	437	301	453	312	468	322	484
	243	365	252	378	260	391	269	404	277	417	286	430	295	443
	227	340	233	351	240	361	247	371	253	381	260	391	267	401
	210	315	214	322	219	330	224	337	229	344	234	351	239	359
	201	303	205	309	209	315	213	321	217	327	221	333	225	338
	191	287	194	292	197	297	201	301	204	306	207	311	210	316
W14×30	241	362	252	378	263	395	274	412	285	428	296	445	307	461
	228	342	237	356	246	370	256	385	265	399	275	413	284	427
	214	322	222	334	230	345	238	357	245	369	253	381	261	392
	201	301	207	311	213	320	219	329	225	339	231	348	238	357
	186	280	191	287	196	294	200	301	205	308	209	315	214	321
	178	267	181	273	185	278	189	284	192	289	196	295	200	300
	167	250	169	255	172	259	175	263	178	267	180	271	183	275
W14×26	210	316	220	330	229	345	239	359	248	373	258	388	268	402
	199	300	208	312	216	325	224	337	233	349	241	362	249	374
	188	283	195	294	202	304	209	315	216	325	223	336	230	346
	177	266	183	275	188	283	194	292	200	300	205	309	211	317
	166	249	170	256	174	262	179	269	183	275	187	282	192	288
	156	235	160	240	163	245	166	250	170	255	173	260	176	265
	144	216	146	220	149	223	151	227	153	231	156	234	158	238

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[b] $Y2$ = distance from top of the steel beam to concrete flange force

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft

W14–W12

F_y = 50 ksi

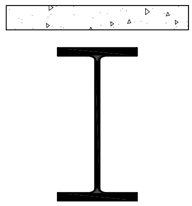
Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n[d]	$Y2$[b], in. 2		2.5		3		3.5	
	kip-ft			in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	ASD	LRFD											
W14×22	82.8	125	TFL	0	325	143	215	151	228	159	240	168	252
			2	0.0838	283	139	209	146	220	153	230	160	241
			3	0.168	241	135	202	141	211	147	220	153	229
			4	0.251	199	130	195	135	203	140	210	145	218
			BFL	0.335	157	125	188	129	194	133	200	137	206
			6	1.67	119	120	180	123	184	126	189	129	193
			7	3.32	81.1	111	167	113	170	115	173	117	176
W12×30	108	162	TFL	0	440	179	269	190	285	201	302	212	318
			2	0.110	368	171	258	181	271	190	285	199	299
			3	0.220	296	164	246	171	257	178	268	186	279
			4	0.330	224	155	234	161	242	167	251	172	259
			BFL	0.440	153	147	221	151	227	155	232	158	238
			6	1.10	131	144	216	147	221	151	226	154	231
			7	1.92	110	140	211	143	215	146	219	149	223
W12×26	92.8	140	TFL	0	383	155	232	164	247	174	261	183	275
			2	0.0950	321	148	223	156	235	164	247	172	259
			3	0.190	259	142	213	148	223	155	232	161	242
			4	0.285	198	135	203	140	210	145	217	150	225
			BFL	0.380	136	128	192	131	197	134	202	138	207
			6	1.07	116	125	188	128	192	131	197	134	201
			7	1.94	95.6	121	183	124	186	126	190	129	193
W12×22	73.1	110	TFL	0	324	132	198	140	210	148	222	156	234
			2	0.106	281	127	191	134	202	141	213	148	223
			3	0.213	238	123	185	129	193	135	202	141	211
			4	0.319	196	118	177	123	185	128	192	133	199
			BFL	0.425	153	113	170	117	175	120	181	124	187
			6	1.66	117	107	162	110	166	113	170	116	175
			7	3.03	81.0	99.8	150	102	153	104	156	106	159
W12×19	61.6	92.6	TFL	0	279	113	169	120	180	126	190	133	201
			2	0.0875	243	109	164	115	173	121	182	127	191
			3	0.175	208	105	158	110	166	116	174	121	182
			4	0.263	173	101	152	106	159	110	165	114	172
			BFL	0.350	138	97.3	146	101	151	104	157	108	162
			6	1.68	104	92.3	139	94.9	143	97.4	146	100	150
			7	3.14	69.6	84.7	127	86.4	130	88.2	133	89.9	135

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Ductility (slip capacity) of the shear connection at the beam/concrete interface may control minimum ΣQ_n requirements per AISC *Specification* Section I3.2d.

F_y = 50 ksi

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft

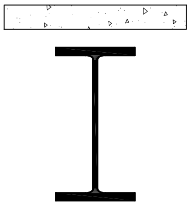


W14–W12

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W14×22	176	264	184	276	192	288	200	301	208	313	216	325	224	337
	167	251	174	262	181	273	188	283	195	294	203	304	210	315
	159	238	165	247	171	256	177	266	183	275	189	284	195	293
	150	225	155	233	160	240	165	248	170	255	175	262	180	270
	141	212	145	218	149	223	153	229	157	235	160	241	164	247
	132	198	135	202	138	207	140	211	143	216	146	220	149	225
	119	179	121	182	123	185	125	188	127	191	129	194	131	198
W12×30	223	335	234	351	245	368	255	384	266	400	277	417	288	433
	208	313	217	327	226	340	236	354	245	368	254	382	263	396
	193	290	201	301	208	313	215	324	223	335	230	346	237	357
	178	267	183	276	189	284	195	293	200	301	206	309	211	318
	162	244	166	250	170	255	174	261	177	267	181	272	185	278
	157	236	160	241	164	246	167	251	170	256	173	261	177	266
	151	227	154	232	157	236	160	240	162	244	165	248	168	252
W12×26	193	290	202	304	212	318	221	333	231	347	240	361	250	376
	180	271	188	283	196	295	204	307	212	319	220	331	228	343
	168	252	174	262	181	271	187	281	193	291	200	300	206	310
	155	232	160	240	164	247	169	255	174	262	179	269	184	277
	141	212	145	217	148	222	151	228	155	233	158	238	162	243
	137	205	139	210	142	214	145	218	148	223	151	227	154	231
	131	197	133	200	136	204	138	208	141	211	143	215	145	218
W12×22	164	247	172	259	180	271	188	283	196	295	205	307	213	320
	155	234	162	244	169	255	176	265	183	276	191	286	198	297
	147	220	152	229	158	238	164	247	170	256	176	265	182	274
	137	207	142	214	147	221	152	229	157	236	162	243	167	251
	128	193	132	198	136	204	140	210	143	215	147	221	151	227
	119	179	122	183	125	188	128	192	131	197	134	201	137	205
	108	162	110	165	112	168	114	171	116	174	118	177	120	180
W12×19	140	211	147	221	154	232	161	242	168	253	175	263	182	274
	133	200	139	209	145	219	151	228	158	237	164	246	170	255
	126	189	131	197	136	205	142	213	147	221	152	228	157	236
	119	178	123	185	127	191	132	198	136	204	140	211	145	217
	111	167	115	172	118	177	121	183	125	188	128	193	132	198
	103	154	105	158	108	162	110	166	113	170	116	174	118	178
	91.7	138	93.4	140	95.1	143	96.9	146	98.6	148	100	151	102	153

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[b] $Y2$ = distance from top of the steel beam to concrete flange force



W12–W10

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft

F_y = 50 ksi

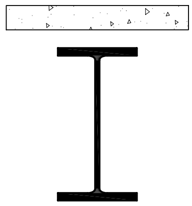
Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n[d]	$Y2$[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W12×16	50.1	75.4	TFL	0	236	94.0	141	99.9	150	106	159	112	168
			2	0.0663	209	91.3	137	96.5	145	102	153	107	161
			3	0.133	183	88.6	133	93.1	140	97.7	147	102	154
			4	0.199	156	85.7	129	89.6	135	93.5	141	97.4	146
			BFL	0.265	130	82.8	124	86.0	129	89.2	134	92.5	139
			6	1.71	94.3	77.6	117	79.9	120	82.3	124	84.6	127
			7	3.32	58.9	69.6	105	71.1	107	72.5	109	74.0	111
W12×14	43.4	65.3	TFL	0	208	82.5	124	87.7	132	92.9	140	98.1	147
			2	0.0563	186	80.3	121	84.9	128	89.5	135	94.2	142
			3	0.113	163	77.9	117	82.0	123	86.1	129	90.2	135
			4	0.169	141	75.5	114	79.1	119	82.6	124	86.1	129
			BFL	0.225	119	73.1	110	76.1	114	79.0	119	82.0	123
			6	1.68	85.3	68.3	103	70.4	106	72.6	109	74.7	112
			7	3.35	52.0	60.8	91.4	62.1	93.3	63.4	95.3	64.7	97.2
W10×26	78.1	117	TFL	0	381	136	204	145	218	155	233	164	247
			2	0.110	317	129	194	137	206	145	218	153	230
			3	0.220	254	122	184	129	193	135	203	141	213
			4	0.330	190	115	173	120	180	125	187	129	195
			BFL	0.440	127	108	162	111	167	114	171	117	176
			6	0.886	111	106	159	108	163	111	167	114	171
			7	1.49	95.1	103	155	105	158	108	162	110	166
W10×22	64.9	97.5	TFL	0	325	115	173	123	185	131	197	139	209
			2	0.0900	273	110	165	116	175	123	185	130	196
			3	0.180	221	104	157	110	165	115	173	121	181
			4	0.270	169	98.4	148	103	154	107	161	111	167
			BFL	0.360	118	92.5	139	95.4	143	98.3	148	101	152
			6	0.962	99.3	90.1	135	92.5	139	95.0	143	97.5	147
			7	1.72	81.1	87.0	131	89.1	134	91.1	137	93.1	140
W10×19	53.9	81.0	TFL	0	281	99.6	150	107	160	114	171	121	181
			2	0.0988	241	95.5	144	102	153	108	162	114	171
			3	0.198	202	91.2	137	96.3	145	101	152	106	160
			4	0.296	162	86.8	130	90.8	137	94.9	143	98.9	149
			BFL	0.395	122	82.1	123	85.2	128	88.2	133	91.3	137
			6	1.25	96.2	78.5	118	80.9	122	83.3	125	85.8	129
			7	2.29	70.3	73.7	111	75.4	113	77.2	116	78.9	119

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Ductility (slip capacity) of the shear connection at the beam/concrete interface may control minimum ΣQ_n requirements per AISC *Specification* Section I3.2d.

F_y = 50 ksi

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft

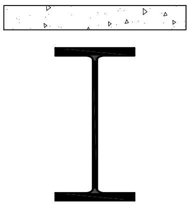


W12–W10

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W12×16	118	177	123	185	129	194	135	203	141	212	147	221	153	230
	112	169	117	176	123	184	128	192	133	200	138	208	143	216
	107	161	111	167	116	174	120	181	125	188	130	195	134	202
	101	152	105	158	109	164	113	170	117	176	121	182	125	187
	95.7	144	99.0	149	102	154	105	158	109	163	112	168	115	173
	87.0	131	89.4	134	91.7	138	94.1	141	96.4	145	98.8	148	101	152
	75.5	113	77.0	116	78.4	118	79.9	120	81.4	122	82.8	125	84.3	127
W12×14	103	155	108	163	114	171	119	179	124	186	129	194	134	202
	98.8	148	103	155	108	162	113	169	117	176	122	183	127	190
	94.2	142	98.3	148	102	154	106	160	111	166	115	172	119	178
	89.6	135	93.1	140	96.7	145	100	151	104	156	107	161	111	166
	85.0	128	87.9	132	90.9	137	93.9	141	96.8	146	99.8	150	103	154
	76.8	115	79.0	119	81.1	122	83.2	125	85.3	128	87.5	131	89.6	135
	66.0	99.2	67.3	101	68.6	103	69.9	105	71.2	107	72.5	109	73.8	111
W10×26	174	261	183	275	193	290	202	304	212	318	221	332	231	347
	161	242	169	254	177	266	185	277	193	289	200	301	208	313
	148	222	154	232	160	241	167	251	173	260	179	270	186	279
	134	202	139	209	144	216	148	223	153	230	158	237	163	244
	120	181	123	186	127	190	130	195	133	200	136	205	139	209
	117	175	119	179	122	184	125	188	128	192	130	196	133	200
	113	169	115	173	117	176	120	180	122	183	124	187	127	191
W10×22	147	221	155	234	164	246	172	258	180	270	188	282	196	294
	137	206	144	216	151	226	157	236	164	247	171	257	178	267
	126	190	132	198	137	206	143	215	148	223	154	231	159	239
	115	173	120	180	124	186	128	192	132	199	136	205	141	211
	104	157	107	161	110	165	113	170	116	174	119	179	122	183
	100	150	102	154	105	158	107	161	110	165	112	169	115	173
	95.1	143	97.1	146	99.2	149	101	152	103	155	105	158	107	161
W10×19	128	192	135	202	142	213	149	223	156	234	163	244	170	255
	120	180	126	189	132	198	138	207	144	216	150	225	156	234
	111	167	116	175	121	183	126	190	132	198	137	205	142	213
	103	155	107	161	111	167	115	173	119	179	123	185	127	191
	94.3	142	97.4	146	100	151	103	156	107	160	110	165	113	169
	88.2	132	90.6	136	93.0	140	95.4	143	97.8	147	100	151	103	154
	80.7	121	82.4	124	84.2	127	85.9	129	87.7	132	89.4	134	91.2	137

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[b] $Y2$ = distance from top of the steel beam to concrete flange force



W10

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft

F_y = 50 ksi

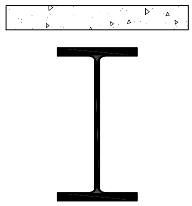
Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n[d]	$Y2$[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W10×17	46.7	70.1	TFL	0	250	87.8	132	94.0	141	100	151	106	160
			2	0.0825	216	84.4	127	89.8	135	95.2	143	101	151
			3	0.165	183	80.9	122	85.5	128	90.0	135	94.6	142
			4	0.248	150	77.2	116	81.0	122	84.7	127	88.5	133
			BFL	0.330	117	73.5	110	76.4	115	79.3	119	82.2	124
			6	1.31	89.8	69.7	105	71.9	108	74.2	111	76.4	115
			7	2.45	62.4	64.4	96.8	65.9	99.1	67.5	101	69.1	104
W10×15	39.9	60.0	TFL	0	221	77.0	116	82.5	124	88.0	132	93.5	140
			2	0.0675	194	74.2	112	79.1	119	83.9	126	88.7	133
			3	0.135	167	71.4	107	75.6	114	79.7	120	83.9	126
			4	0.203	140	68.5	103	72.0	108	75.5	113	78.9	119
			BFL	0.270	113	65.5	98.4	68.3	103	71.1	107	73.9	111
			6	1.35	83.8	61.5	92.5	63.6	95.6	65.7	98.7	67.8	102
			7	2.60	55.1	55.8	83.9	57.2	86.0	58.6	88.0	59.9	90.1
W10×12	31.2	46.9	TFL	0	177	61.3	92.1	65.7	98.7	70.1	105	74.5	112
			2	0.0525	156	59.1	88.9	63.0	94.8	66.9	100	70.8	106
			3	0.105	135	57.0	85.7	60.4	90.7	63.7	95.8	67.1	101
			4	0.158	115	54.8	82.4	57.7	86.7	60.5	91.0	63.4	95.3
			BFL	0.210	93.8	52.5	78.9	54.9	82.4	57.2	86.0	59.5	89.5
			6	1.30	69.0	49.2	73.9	50.9	76.5	52.6	79.1	54.4	81.7
			7	2.61	44.3	44.3	66.6	45.4	68.2	46.5	69.9	47.6	71.5

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Ductility (slip capacity) of the shear connection at the beam/concrete interface may control minimum ΣQ_n requirements per AISC *Specification* Section I3.2d.

F_y = 50 ksi

Table 3-19 (continued)
Composite W-Shapes
Available Strength in Flexure, kip-ft



W10

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W10×17	113	169	119	179	125	188	131	197	138	207	144	216	150	225
	106	159	111	167	117	176	122	184	128	192	133	200	138	208
	99.2	149	104	156	108	163	113	170	117	177	122	183	127	190
	92.2	139	96.0	144	99.7	150	103	156	107	161	111	167	115	172
	85.2	128	88.1	132	91.0	137	93.9	141	96.8	146	99.8	150	103	154
	78.6	118	80.9	122	83.1	125	85.4	128	87.6	132	89.8	135	92.1	138
	70.6	106	72.2	108	73.7	111	75.3	113	76.8	115	78.4	118	80.0	120
W10×15	99.0	149	104	157	110	165	115	174	121	182	126	190	132	198
	93.5	141	98.4	148	103	155	108	162	113	170	118	177	123	184
	88.0	132	92.2	139	96.3	145	100	151	105	157	109	164	113	170
	82.4	124	85.9	129	89.4	134	92.9	140	96.4	145	99.8	150	103	155
	76.7	115	79.5	120	82.3	124	85.2	128	88.0	132	90.8	136	93.6	141
	69.9	105	72.0	108	74.1	111	76.2	114	78.2	118	80.3	121	82.4	124
	61.3	92.2	62.7	94.2	64.1	96.3	65.4	98.3	66.8	100	68.2	102	69.6	105
W10×12	78.9	119	83.3	125	87.7	132	92.2	139	96.6	145	101	152	105	158
	74.7	112	78.6	118	82.5	124	86.4	130	90.3	136	94.2	142	98.1	147
	70.5	106	73.9	111	77.3	116	80.6	121	84.0	126	87.4	131	90.8	136
	66.2	99.6	69.1	104	72.0	108	74.8	112	77.7	117	80.5	121	83.4	125
	61.9	93.0	64.2	96.5	66.6	100	68.9	104	71.2	107	73.6	111	75.9	114
	56.1	84.3	57.8	86.9	59.5	89.5	61.2	92.1	63.0	94.6	64.7	97.2	66.4	99.8
	48.7	73.2	49.8	74.9	50.9	76.5	52.0	78.2	53.1	79.8	54.2	81.5	55.3	83.2

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[b] $Y2$ = distance from top of the steel beam to concrete flange force

I_{LB}

W40

Table 3-20
Lower-Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections, in.^{4}

F_y = 50 ksi

Shape[d]	PNA[c]	$Y1$[a]	$\sum Q_n$	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W40×297	TFL	0	4370	44100	45100	46100	47100	48100	49200	50300	51400	52500	53600	54800
(23200)	2	0.413	3710	42400	43300	44200	45200	46100	47100	48100	49100	50100	51200	52200
	3	0.825	3060	40500	41300	42100	42900	43800	44600	45500	46400	47300	48300	49200
	4	1.24	2410	38100	38800	39500	40200	40900	41700	42500	43200	44000	44800	45700
	BFL	1.65	1760	35200	35800	36400	36900	37500	38100	38800	39400	40000	40700	41400
	6	4.58	1420	33500	34000	34400	34900	35400	36000	36500	37000	37600	38100	38700
	7	8.17	1090	31600	32000	32300	32800	33200	33600	34000	34500	34900	35400	35800
W40×294	TFL	0	4310	43100	44100	45100	46100	47100	48200	49300	50400	51500	52600	53800
(21900)	2	0.483	3730	41600	42500	43400	44400	45300	46300	47300	48300	49400	50400	51500
	3	0.965	3150	39800	40700	41500	42300	43200	44100	45000	45900	46900	47800	48800
	4	1.45	2570	37800	38500	39200	40000	40800	41500	42300	43200	44000	44900	45700
	BFL	1.93	1990	35300	35900	36600	37200	37800	38500	39200	39900	40600	41300	42000
	6	5.71	1540	33100	33600	34100	34600	35200	35700	36300	36900	37500	38100	38700
	7	10.0	1080	30400	30800	31200	31600	32000	32400	32900	33300	33800	34200	34700
W40×278	TFL	0	4120	40600	41500	42500	43400	44400	45400	46400	47500	48500	49600	50700
(20500)	2	0.453	3570	39200	40000	40900	41800	42700	43600	44600	45600	46500	47600	48600
	3	0.905	3030	37500	38300	39100	39900	40800	41600	42500	43400	44300	45200	46100
	4	1.36	2490	35700	36300	37100	37800	38500	39300	40000	40800	41600	42500	43300
	BFL	1.81	1940	33400	34000	34600	35200	35800	36500	37100	37800	38500	39200	39900
	6	5.67	1490	31200	31700	32200	32700	33200	33700	34300	34800	35400	36000	36600
	7	10.1	1030	28500	28900	29300	29700	30100	30500	30900	31300	31700	32200	32600
W40×277	TFL	0	4080	41400	42300	43200	44100	45100	46100	47100	48100	49100	50200	51300
(21900)	2	0.395	3450	39700	40600	41400	42300	43200	44100	45000	45900	46900	47800	48800
	3	0.790	2830	37800	38600	39300	40100	40900	41700	42500	43400	44200	45100	46000
	4	1.19	2200	35500	36200	36800	37500	38200	38800	39500	40300	41000	41700	42500
	BFL	1.58	1580	32800	33300	33800	34300	34900	35400	36000	36500	37100	37700	38300
	6	4.20	1300	31300	31700	32200	32600	33100	33600	34100	34600	35100	35600	36100
	7	7.58	1020	29700	30100	30400	30800	31200	31600	32000	32400	32800	33200	33700
W40×264	TFL	0	3870	38100	39000	39900	40800	41700	42600	43600	44600	45600	46600	47600
(19400)	2	0.433	3360	36800	37600	38400	39300	40100	41000	41900	42800	43700	44700	45600
	3	0.865	2840	35300	36000	36700	37500	38300	39100	39900	40700	41500	42400	43300
	4	1.30	2330	33500	34100	34800	35500	36200	36900	37600	38300	39100	39800	40600
	BFL	1.73	1810	31300	31900	32400	33000	33600	34200	34800	35400	36100	36700	37400
	6	5.53	1390	29300	29800	30200	30700	31200	31700	32200	32700	33200	33800	34300
	7	9.92	968	26900	27200	27600	28000	28300	28700	29100	29500	29900	30300	30700

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Value in parentheses is I_x (in.^{4}) of noncomposite steel shape.

Table 3-20 (continued)
Lower-Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections, in.^{4}

F_y = 50 ksi

I_{LB}

W40

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W40×249	TFL	0	3680	36900	37700	38500	39400	40300	41100	42000	43000	43900	44800	45800
(19600)	2	0.355	3110	35500	36200	37000	37700	38500	39300	40200	41000	41900	42700	43600
	3	0.710	2550	33800	34400	35100	35800	36500	37200	38000	38700	39500	40300	41100
	4	1.07	1990	31800	32300	32900	33500	34100	34700	35400	36000	36700	37300	38000
	BFL	1.42	1430	29300	29700	30200	30700	31200	31700	32200	32700	33200	33700	34300
	6	4.03	1180	28000	28400	28800	29200	29600	30100	30500	30900	31400	31900	32300
	7	7.45	919	26500	26800	27200	27500	27900	28200	28600	28900	29300	29700	30100
W40×235	TFL	0	3460	33900	34700	35500	36300	37100	37900	38800	39600	40500	41400	42300
(17400)	2	0.395	2980	32700	33400	34100	34800	35600	36400	37200	38000	38800	39600	40500
	3	0.790	2510	31300	31900	32600	33300	33900	34600	35400	36100	36800	37600	38400
	4	1.19	2040	29600	30200	30800	31400	32000	32600	33200	33900	34500	35200	35900
	BFL	1.58	1570	27700	28200	28700	29200	29700	30200	30700	31300	31800	32400	33000
	6	5.16	1220	26000	26400	26800	27200	27700	28100	28500	29000	29400	29900	30400
	7	9.44	864	24000	24300	24600	24900	25300	25600	25900	26300	26600	27000	27400
W40×215	TFL	0	3180	31400	32100	32800	33500	34200	35000	35800	36600	37400	38200	39000
(16700)	2	0.305	2690	30200	30800	31400	32100	32800	33500	34200	34900	35600	36400	37200
	3	0.610	2210	28700	29300	29900	30500	31100	31700	32300	33000	33600	34300	35000
	4	0.915	1730	27100	27500	28000	28500	29100	29600	30100	30700	31300	31800	32400
	BFL	1.22	1250	25000	25400	25800	26200	26600	27000	27500	27900	28400	28800	29300
	6	3.80	1020	23800	24200	24500	24900	25200	25600	26000	26300	26700	27100	27500
	7	7.29	794	22600	22800	23100	23400	23700	24000	24300	24600	25000	25300	25600
W40×211	TFL	0	3110	30100	30800	31500	32200	33000	33700	34500	35200	36000	36800	37700
(15500)	2	0.355	2690	29100	29700	30400	31000	31700	32400	33100	33800	34500	35300	36100
	3	0.710	2270	27800	28400	29000	29600	30200	30900	31500	32200	32800	33500	34200
	4	1.07	1850	26400	26900	27400	28000	28500	29100	29600	30200	30800	31400	32000
	BFL	1.42	1430	24700	25200	25600	26000	26500	27000	27400	27900	28400	28900	29500
	6	5.00	1100	23100	23500	23900	24200	24600	25000	25400	25800	26200	26700	27100
	7	9.35	776	21300	21600	21900	22200	22500	22800	23100	23400	23700	24000	24400
W40×199	TFL	0	2940	28300	28900	29600	30300	30900	31600	32300	33100	33800	34500	35300
(14900)	2	0.268	2520	27300	27900	28500	29100	29700	30300	31000	31700	32300	33000	33700
	3	0.535	2090	26000	26600	27100	27700	28200	28800	29400	30000	30600	31200	31900
	4	0.803	1670	24600	25100	25500	26000	26500	27000	27500	28100	28600	29100	29700
	BFL	1.07	1250	22900	23300	23700	24100	24500	24900	25300	25700	26200	26600	27100
	6	4.09	992	21700	22000	22300	22600	23000	23300	23700	24100	24400	24800	25200
	7	8.04	735	20300	20500	20800	21000	21300	21600	21900	22200	22500	22800	23100

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Value in parentheses is I_x (in.^{4}) of noncomposite steel shape.

I_{LB}

W40–W36

Table 3-20 (continued)
Lower-Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections, in.^{4}

F_y = 50 ksi

Shape[d]	PNA[c]	$Y1$[a]	$\sum Q_n$	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W40×183	TFL	0	2670	25500	26100	26700	27300	27900	28600	29200	29900	30500	31200	31900
(13200)	2	0.300	2310	24600	25200	25700	26300	26900	27500	28100	28700	29300	29900	30600
	3	0.600	1960	23600	24100	24600	25100	25700	26200	26800	27300	27900	28500	29100
	4	0.900	1600	22400	22900	23300	23800	24200	24700	25200	25700	26200	26700	27200
	BFL	1.20	1250	21100	21400	21800	22200	22600	23000	23400	23800	24300	24700	25200
	6	4.77	958	19700	20000	20300	20700	21000	21300	21700	22000	22400	22700	23100
	7	9.25	666	18100	18400	18600	18800	19100	19300	19600	19900	20100	20400	20700
W40×167	TFL	0	2470	22800	23300	23900	24400	25000	25600	26200	26800	27400	28000	28700
(11600)	2	0.258	2160	22000	22500	23000	23600	24100	24600	25200	25800	26300	26900	27500
	3	0.515	1860	21200	21700	22100	22600	23100	23600	24100	24600	25200	25700	26300
	4	0.773	1550	20200	20600	21100	21500	21900	22400	22800	23300	23800	24300	24800
	BFL	1.03	1250	19100	19500	19800	20200	20600	21000	21400	21800	22200	22600	23100
	6	4.95	933	17700	18000	18300	18600	18900	19300	19600	19900	20300	20600	21000
	7	9.82	616	16100	16300	16500	16700	17000	17200	17400	17700	17900	18200	18400
W40×149	TFL	0	2190	19600	20000	20500	21000	21500	22000	22500	23100	23600	24200	24700
(9800)	2	0.208	1950	19000	19400	19900	20300	20800	21300	21800	22300	22800	23300	23900
	3	0.415	1700	18300	18700	19100	19600	20000	20500	20900	21400	21900	22300	22800
	4	0.623	1460	17600	18000	18400	18700	19100	19600	20000	20400	20800	21300	21700
	BFL	0.830	1210	16700	17100	17400	17800	18100	18500	18900	19200	19600	20000	20400
	6	5.15	879	15400	15700	15900	16200	16500	16800	17100	17400	17700	18000	18300
	7	10.4	548	13700	13900	14100	14300	14500	14700	14900	15100	15300	15500	15800
W36×302	TFL	0	4450	40100	41000	42000	42900	43900	44900	46000	47100	48100	49200	50400
(21100)	2	0.420	3750	38500	39300	40200	41100	42000	42900	43900	44800	45800	46800	47900
	3	0.840	3050	36500	37300	38100	38900	39700	40500	41300	42200	43100	44000	44900
	4	1.26	2350	34200	34900	35500	36200	36900	37600	38300	39000	39800	40600	41300
	BFL	1.68	1640	31300	31800	32300	32900	33400	33900	34500	35100	35700	36300	36900
	6	4.06	1380	30100	30500	31000	31400	31900	32400	32900	33400	33900	34400	35000
	7	6.88	1110	28700	29000	29400	29800	30200	30600	31000	31500	31900	32300	32800
W36×282	TFL	0	4150	37100	38000	38900	39800	40700	41600	42600	43600	44600	45600	46700
(19600)	2	0.393	3490	35600	36400	37200	38000	38900	39700	40600	41500	42400	43400	44300
	3	0.785	2840	33800	34500	35300	36000	36700	37500	38300	39100	39900	40800	41600
	4	1.18	2190	31700	32300	32900	33500	34200	34800	35500	36200	36900	37600	38300
	BFL	1.57	1540	29100	29600	30000	30500	31000	31500	32100	32600	33100	33700	34300
	6	4.00	1290	27900	28300	28700	29200	29600	30100	30500	31000	31500	31900	32400
	7	6.84	1040	26600	27000	27300	27700	28100	28400	28800	29200	29600	30000	30500

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Value in parentheses is I_x (in.^{4}) of noncomposite steel shape.

Table 3-20 (continued)
Lower-Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections, in.^{4}

F_y = 50 ksi

I_{LB}

W36

Shape[d]	PNA[c]	$Y1$[a]	$\sum Q_n$	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W36×262	TFL	0	3860	34000	34800	35700	36500	37400	38200	39100	40000	41000	41900	42900
(17900)	2	0.360	3260	32700	33400	34200	34900	35700	36500	37300	38200	39000	39900	40800
	3	0.720	2660	31100	31700	32400	33100	33800	34500	35200	36000	36700	37500	38300
	4	1.08	2070	29200	29700	30300	30900	31500	32100	32700	33400	34000	34700	35400
	BFL	1.44	1470	26800	27200	27700	28200	28600	29100	29600	30100	30600	31200	31700
	6	3.96	1220	25700	26000	26400	26800	27200	27700	28100	28500	29000	29400	29900
	7	6.96	965	24400	24700	25000	25300	25700	26000	26400	26800	27100	27500	27900
W36×256	TFL	0	3770	32900	33700	34500	35400	36200	37100	38000	38900	39800	40700	41700
(16800)	2	0.433	3240	31700	32500	33200	34000	34700	35500	36400	37200	38000	38900	39800
	3	0.865	2710	30300	31000	31600	32300	33000	33800	34500	35300	36000	36800	37600
	4	1.30	2180	28600	29200	29800	30400	31000	31700	32300	33000	33600	34300	35000
	BFL	1.73	1650	26600	27100	27600	28100	28600	29100	29700	30200	30800	31400	32000
	6	5.18	1300	25100	25500	25900	26300	26800	27200	27700	28100	28600	29100	29600
	7	8.90	941	23300	23600	23900	24200	24600	24900	25300	25600	26000	26400	26700
W36×247	TFL	0	3630	31700	32500	33200	34000	34800	35600	36500	37300	38200	39100	40000
(16700)	2	0.338	3070	30500	31200	31900	32600	33300	34100	34800	35600	36400	37200	38100
	3	0.675	2510	29000	29600	30200	30900	31500	32200	32900	33600	34300	35000	35800
	4	1.01	1950	27200	27700	28300	28800	29400	29900	30500	31100	31700	32400	33000
	BFL	1.35	1400	25100	25500	25900	26300	26800	27200	27700	28200	28700	29200	29700
	6	3.95	1150	23900	24300	24700	25000	25400	25800	26200	26600	27100	27500	27900
	7	7.02	906	22700	23000	23300	23600	23900	24300	24600	24900	25300	25700	26000
W36×232	TFL	0	3400	29400	30100	30800	31500	32300	33100	33900	34700	35500	36300	37200
(15000)	2	0.393	2930	28300	28900	29600	30300	31000	31700	32500	33200	34000	34800	35500
	3	0.785	2450	27000	27600	28200	28800	29500	30100	30800	31500	32200	32900	33600
	4	1.18	1980	25600	26100	26600	27200	27700	28300	28900	29500	30100	30700	31300
	BFL	1.57	1500	23800	24200	24700	25100	25600	26100	26500	27000	27500	28100	28600
	6	5.04	1180	22400	22800	23100	23500	23900	24300	24700	25100	25600	26000	26400
	7	8.78	850	20700	21000	21300	21600	21900	22200	22500	22900	23200	23500	23900
W36×231	TFL	0	3410	29600	30300	31000	31700	32500	33200	34000	34800	35700	36500	37300
(15600)	2	0.315	2890	28400	29100	29700	30400	31100	31800	32500	33200	34000	34800	35500
	3	0.630	2370	27100	27600	28200	28800	29400	30100	30700	31400	32000	32700	33400
	4	0.945	1850	25400	25900	26400	26900	27500	28000	28600	29100	29700	30300	30900
	BFL	1.26	1330	23400	23800	24200	24700	25100	25500	25900	26400	26900	27300	27800
	6	3.88	1090	22400	22700	23100	23400	23800	24100	24500	24900	25300	25700	26100
	7	7.03	853	21200	21500	21800	22100	22400	22700	23000	23300	23600	24000	24300

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Value in parentheses is I_x (in.^{4}) of noncomposite steel shape.

I_{LB}

W36

Table 3-20 (continued)
Lower-Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections, in.^{4}

F_y = 50 ksi

Shape[d]	PNA[c]	$Y1$[a]	$\sum Q_n$	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W36×210	TFL	0	3100	26000	26700	27300	28000	28700	29400	30100	30800	31600	32300	33100
(13200)	2	0.340	2680	25100	25700	26300	26900	27500	28200	28900	29500	30200	30900	31700
	3	0.680	2270	24000	24600	25100	25700	26300	26900	27500	28100	28700	29400	30000
	4	1.02	1850	22800	23300	23800	24300	24800	25300	25800	26400	26900	27500	28100
	BFL	1.36	1440	21300	21700	22200	22600	23000	23500	23900	24400	24900	25300	25800
	6	5.04	1100	19900	20300	20600	20900	21300	21700	22000	22400	22800	23200	23600
	7	9.03	774	18300	18600	18800	19100	19400	19700	20000	20200	20500	20800	21200
W36×194	TFL	0	2850	23800	24400	25000	25600	26200	26900	27500	28200	28900	29600	30300
(12100)	2	0.315	2470	23000	23500	24100	24600	25200	25800	26400	27000	27700	28300	29000
	3	0.630	2090	22000	22500	23000	23500	24000	24600	25100	25700	26300	26900	27500
	4	0.945	1710	20900	21300	21800	22200	22700	23200	23700	24200	24700	25200	25700
	BFL	1.26	1330	19500	19900	20300	20700	21100	21500	21900	22300	22800	23200	23700
	6	4.93	1020	18300	18600	18900	19200	19500	19900	20200	20600	20900	21300	21700
	7	8.94	713	16800	17000	17300	17500	17700	18000	18300	18500	18800	19100	19400
W36×182	TFL	0	2680	22200	22700	23300	23900	24400	25000	25700	26300	26900	27600	28300
(11300)	2	0.295	2320	21400	21900	22400	23000	23500	24100	24600	25200	25800	26400	27000
	3	0.590	1970	20500	21000	21500	21900	22400	22900	23500	24000	24500	25100	25700
	4	0.885	1610	19500	19900	20300	20700	21200	21600	22100	22600	23000	23500	24000
	BFL	1.18	1250	18200	18600	18900	19300	19700	20000	20400	20800	21200	21700	22100
	6	4.89	961	17000	17300	17600	17900	18200	18600	18900	19200	19600	19900	20200
	7	8.91	670	15700	15900	16100	16300	16600	16800	17000	17300	17600	17800	18100
W36×170	TFL	0	2500	20600	21100	21600	22200	22700	23300	23800	24400	25000	25600	26300
(10500)	2	0.275	2170	19900	20400	20800	21300	21800	22400	22900	23400	24000	24600	25100
	3	0.550	1840	19100	19500	19900	20400	20900	21300	21800	22300	22800	23300	23900
	4	0.825	1510	18100	18500	18900	19300	19700	20100	20500	21000	21400	21900	22400
	BFL	1.10	1180	17000	17300	17600	18000	18300	18700	19100	19400	19800	20200	20600
	6	4.83	903	15900	16100	16400	16700	17000	17300	17600	17900	18200	18500	18900
	7	8.91	625	14500	14700	15000	15200	15400	15600	15800	16100	16300	16600	16800
W36×160	TFL	0	2350	19200	19600	20100	20600	21100	21700	22200	22700	23300	23900	24400
(9760)	2	0.255	2040	18500	18900	19400	19900	20300	20800	21300	21800	22300	22900	23400
	3	0.510	1740	17800	18200	18600	19000	19400	19900	20300	20800	21300	21800	22300
	4	0.765	1430	16900	17200	17600	18000	18400	18800	19200	19600	20000	20400	20900
	BFL	1.02	1130	15900	16200	16500	16800	17100	17500	17800	18200	18600	18900	19300
	6	4.82	857	14800	15000	15300	15600	15800	16100	16400	16700	17000	17300	17600
	7	8.96	588	13500	13700	13900	14100	14300	14500	14700	15000	15200	15400	15600

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Value in parentheses is I_x (in.^{4}) of noncomposite steel shape.

Table 3-20 (continued)
Lower-Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections, in.^{4}

F_y = 50 ksi

I_{LB}

W36–W33

Shape[d]	PNA[c]	$Y1$[a]	$\sum Q_n$	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W36×150	TFL	0	2220	17900	18300	18800	19200	19700	20200	20700	21200	21800	22300	22800
(9040)	2	0.235	1930	17200	17700	18100	18500	19000	19400	19900	20400	20900	21400	21900
	3	0.470	1650	16600	16900	17300	17700	18200	18600	19000	19400	19900	20300	20800
	4	0.705	1370	15800	16100	16500	16800	17200	17600	18000	18300	18800	19200	19600
	BFL	0.940	1090	14900	15200	15500	15800	16100	16400	16700	17100	17400	17800	18100
	6	4.82	820	13800	14000	14300	14500	14800	15100	15300	15600	15900	16200	16500
	7	9.09	554	12600	12700	12900	13100	13300	13500	13700	13900	14100	14300	14600
W36×135	TFL	0	2000	15600	16000	16400	16900	17300	17700	18200	18600	19100	19600	20100
(7800)	2	0.198	1760	15100	15500	15900	16300	16700	17100	17500	18000	18400	18800	19300
	3	0.395	1520	14600	14900	15300	15600	16000	16400	16800	17200	17600	18000	18400
	4	0.593	1280	13900	14200	14500	14900	15200	15600	15900	16300	16600	17000	17400
	BFL	0.790	1050	13200	13500	13800	14000	14300	14600	15000	15300	15600	15900	16300
	6	4.92	773	12200	12400	12600	12900	13100	13300	13600	13800	14100	14400	14700
	7	9.49	499	10900	11100	11300	11400	11600	11800	11900	12100	12300	12500	12700
W33×221	TFL	0	3270	24600	25300	25900	26600	27200	27900	28600	29400	30100	30900	31600
(12900)	2	0.320	2760	23600	24200	24800	25400	26000	26700	27300	28000	28700	29300	30100
	3	0.640	2250	22500	23000	23500	24000	24600	25200	25700	26300	26900	27500	28200
	4	0.960	1750	21100	21500	22000	22400	22900	23400	23900	24400	24900	25400	26000
	BFL	1.28	1240	19400	19700	20100	20400	20800	21200	21600	22000	22400	22800	23200
	6	3.67	1030	18500	18800	19100	19400	19800	20100	20400	20800	21100	21500	21900
	7	6.42	816	17600	17800	18100	18400	18600	18900	19200	19500	19800	20100	20400
W33×201	TFL	0	2960	22100	22700	23300	23800	24500	25100	25700	26400	27000	27700	28400
(11600)	2	0.288	2500	21200	21700	22300	22800	23400	23900	24500	25100	25700	26400	27000
	3	0.575	2050	20200	20700	21100	21600	22100	22600	23200	23700	24200	24800	25400
	4	0.863	1600	19000	19400	19800	20200	20600	21100	21500	22000	22400	22900	23400
	BFL	1.15	1150	17500	17800	18100	18500	18800	19100	19500	19900	20200	20600	21000
	6	3.65	944	16700	17000	17200	17500	17800	18100	18400	18700	19100	19400	19700
	7	6.52	739	15800	16000	16300	16500	16700	17000	17200	17500	17800	18000	18300
W33×169	TFL	0	2480	18100	18600	19100	19600	20100	20600	21200	21700	22300	22900	23400
(9290)	2	0.305	2120	17400	17900	18300	18800	19300	19700	20200	20700	21300	21800	22300
	3	0.610	1770	16700	17100	17500	17900	18300	18700	19200	19600	20100	20600	21100
	4	0.915	1420	15700	16100	16400	16800	17200	17600	17900	18300	18800	19200	19600
	BFL	1.22	1070	14600	14900	15200	15500	15800	16100	16500	16800	17100	17500	17800
	6	4.28	845	13800	14000	14300	14500	14800	15100	15300	15600	15900	16200	16500
	7	7.66	619	12800	13000	13200	13400	13600	13800	14000	14300	14500	14700	14900

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Value in parentheses is I_x (in.^{4}) of noncomposite steel shape.

I_{LB}

W33–W30

Table 3-20 (continued)
Lower-Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections, in.^{4}

F_y = 50 ksi

Shape[d]	PNA[c]	$Y1$[a]	$\sum Q_n$	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W33×152	TFL	0	2250	16100	16500	16900	17400	17800	18300	18800	19300	19800	20300	20800
(8160)	2	0.265	1940	15500	15900	16300	16700	17100	17600	18000	18500	18900	19400	19900
	3	0.530	1630	14800	15200	15500	15900	16300	16700	17100	17500	17900	18400	18800
	4	0.795	1320	14000	14300	14600	15000	15300	15700	16000	16400	16800	17100	17500
	BFL	1.06	1020	13100	13400	13600	13900	14200	14500	14800	15100	15400	15700	16100
	6	4.34	788	12300	12500	12700	12900	13200	13400	13700	13900	14200	14500	14700
	7	7.91	561	11300	11500	11700	11800	12000	12200	12400	12600	12800	13000	13200
W33×141	TFL	0	2080	14700	15100	15500	15900	16300	16700	17200	17600	18100	18600	19100
(7450)	2	0.240	1800	14200	14500	14900	15300	15700	16100	16500	16900	17300	17800	18200
	3	0.480	1520	13600	13900	14200	14600	14900	15300	15700	16100	16500	16900	17300
	4	0.720	1250	12900	13200	13500	13800	14100	14400	14800	15100	15500	15800	16200
	BFL	0.960	971	12100	12300	12600	12800	13100	13400	13700	13900	14200	14500	14800
	6	4.34	745	11300	11500	11700	11900	12100	12400	12600	12800	13100	13300	13600
	7	8.08	519	10300	10500	10700	10800	11000	11200	11300	11500	11700	11900	12100
W33×130	TFL	0	1920	13300	13700	14000	14400	14800	15200	15600	16000	16500	16900	17300
(6710)	2	0.214	1670	12800	13200	13500	13900	14200	14600	15000	15400	15800	16200	16600
	3	0.428	1420	12300	12600	12900	13300	13600	13900	14300	14600	15000	15400	15800
	4	0.641	1180	11700	12000	12300	12600	12900	13200	13500	13800	14100	14500	14800
	BFL	0.855	932	11000	11300	11500	11800	12000	12300	12500	12800	13100	13400	13700
	6	4.39	705	10300	10500	10600	10900	11100	11300	11500	11700	12000	12200	12400
	7	8.30	479	9350	9490	9640	9790	9950	10100	10300	10400	10600	10800	11000
W33×118	TFL	0	1740	11800	12100	12500	12800	13200	13500	13900	14300	14700	15100	15500
(5900)	2	0.185	1520	11400	11700	12000	12300	12700	13000	13400	13700	14100	14400	14800
	3	0.370	1310	11000	11300	11500	11800	12100	12500	12800	13100	13400	13800	14100
	4	0.555	1100	10500	10700	11000	11300	11500	11800	12100	12400	12700	13000	13300
	BFL	0.740	884	9890	10100	10300	10600	10800	11000	11300	11500	11800	12100	12300
	6	4.47	659	9150	9330	9510	9700	9890	10100	10300	10500	10700	10900	11200
	7	8.56	434	8260	8390	8530	8660	8800	8950	9090	9250	9400	9560	9720
W30×116	TFL	0	1710	9870	10200	10500	10800	11100	11400	11800	12100	12500	12800	13200
(4930)	2	0.213	1490	9530	9810	10100	10400	10700	11000	11300	11600	12000	12300	12600
	3	0.425	1260	9120	9370	9630	9900	10200	10400	10700	11000	11300	11600	12000
	4	0.638	1040	8670	8890	9120	9360	9600	9850	10100	10400	10600	10900	11200
	BFL	0.850	818	8130	8320	8520	8720	8920	9140	9360	9580	9810	10000	10300
	6	3.98	623	7570	7730	7890	8060	8230	8400	8580	8770	8960	9150	9350
	7	7.43	428	6910	7030	7150	7270	7400	7530	7670	7810	7950	8090	8240

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Value in parentheses is I_x (in.^{4}) of noncomposite steel shape.

Table 3-20 (continued)
Lower-Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections, in.^{4}

F_y = 50 ksi

I_{LB}

W30–W27

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W30×108	TFL	0	1590	9000	9280	9560	9840	10100	10400	10800	11100	11400	11700	12100
(4470)	2	0.190	1390	8700	8950	9220	9480	9760	10000	10300	10600	10900	11300	11600
	3	0.380	1190	8350	8590	8830	9070	9330	9590	9850	10100	10400	10700	11000
	4	0.570	987	7940	8150	8370	8590	8820	9050	9290	9530	9780	10000	10300
	BFL	0.760	787	7470	7650	7840	8030	8230	8430	8640	8850	9060	9290	9510
	6	4.04	592	6930	7080	7230	7390	7550	7710	7880	8060	8240	8420	8600
	7	7.63	396	6280	6390	6500	6620	6730	6850	6980	7110	7240	7370	7510
W30×99	TFL	0	1450	8110	8350	8610	8870	9140	9420	9700	9990	10300	10600	10900
(3990)	2	0.168	1270	7830	8070	8300	8550	8800	9060	9330	9600	9880	10200	10500
	3	0.335	1100	7540	7760	7980	8200	8440	8670	8920	9170	9430	9690	9960
	4	0.503	922	7190	7380	7580	7790	8000	8210	8430	8660	8890	9130	9370
	BFL	0.670	747	6790	6960	7130	7310	7490	7680	7880	8070	8280	8480	8700
	6	4.19	555	6270	6410	6550	6690	6840	7000	7150	7310	7480	7650	7820
	7	7.88	363	5640	5740	5840	5950	6050	6160	6280	6390	6510	6640	6760
W30×90	TFL	0	1320	7310	7530	7760	8000	8240	8490	8750	9010	9280	9560	9840
(3610)	2	0.153	1160	7070	7280	7490	7720	7940	8180	8420	8660	8920	9180	9440
	3	0.305	998	6790	6990	7190	7390	7600	7820	8040	8260	8500	8730	8980
	4	0.458	839	6480	6660	6840	7020	7210	7410	7610	7810	8020	8240	8460
	BFL	0.610	681	6130	6280	6440	6600	6760	6940	7110	7290	7470	7660	7850
	6	4.01	505	5660	5780	5910	6040	6180	6310	6460	6600	6750	6910	7060
	7	7.76	329	5090	5180	5270	5360	5460	5560	5660	5770	5880	5990	6100
W27×102	TFL	0	1500	7250	7480	7730	7980	8240	8510	8780	9060	9350	9650	9950
(3620)	2	0.208	1290	6970	7190	7420	7650	7890	8140	8390	8650	8920	9200	9480
	3	0.415	1090	6670	6870	7080	7290	7510	7730	7960	8200	8450	8700	8950
	4	0.623	878	6300	6470	6650	6840	7030	7230	7430	7640	7850	8070	8300
	BFL	0.830	670	5860	6010	6160	6310	6470	6640	6810	6980	7160	7340	7530
	6	3.40	523	5500	5620	5740	5870	6010	6150	6290	6430	6580	6740	6900
	7	6.27	375	5070	5170	5260	5360	5470	5570	5680	5800	5910	6030	6150
W27×94	TFL	0	1380	6560	6780	7000	7230	7470	7720	7970	8230	8490	8760	9040
(3270)	2	0.186	1190	6320	6520	6730	6940	7160	7390	7620	7860	8100	8360	8610
	3	0.373	1010	6050	6240	6430	6620	6820	7030	7240	7460	7680	7910	8150
	4	0.559	821	5730	5890	6060	6230	6400	6590	6770	6970	7160	7370	7580
	BFL	0.745	635	5350	5480	5620	5770	5920	6070	6230	6390	6560	6730	6910
	6	3.45	490	5000	5110	5230	5350	5470	5600	5730	5870	6010	6150	6290
	7	6.41	345	4590	4670	4760	4860	4950	5050	5150	5250	5360	5470	5580

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Value in parentheses is I_x (in.^{4}) of noncomposite steel shape.

I_{LB}

W27–W24

Table 3-20 (continued)
Lower-Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections, in.^{4}

F_y = 50 ksi

Shape[d]	PNA[c]	$Y1$[a]	$\sum Q_n$	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W27×84	TFL	0	1240	5770	5960	6160	6360	6580	6790	7020	7250	7480	7730	7970
(2850)	2	0.160	1080	5570	5740	5930	6120	6320	6520	6730	6940	7160	7390	7620
	3	0.320	915	5330	5490	5660	5830	6010	6200	6390	6590	6790	6990	7200
	4	0.480	755	5060	5200	5360	5510	5670	5840	6010	6180	6360	6540	6730
	BFL	0.640	595	4740	4870	5000	5130	5270	5410	5550	5700	5860	6010	6180
	6	3.53	452	4410	4510	4620	4730	4840	4960	5080	5200	5330	5460	5590
	7	6.64	309	4010	4090	4170	4250	4340	4430	4510	4610	4700	4800	4900
W24×94	TFL	0	1390	5480	5680	5880	6100	6320	6550	6780	7020	7270	7530	7790
(2700)	2	0.219	1190	5260	5450	5640	5840	6040	6250	6470	6690	6920	7150	7390
	3	0.438	988	5010	5180	5350	5520	5710	5900	6090	6290	6500	6710	6930
	4	0.656	790	4710	4860	5010	5160	5320	5490	5660	5830	6010	6200	6390
	BFL	0.875	591	4360	4480	4600	4730	4860	5000	5140	5280	5430	5580	5740
	6	3.05	469	4100	4200	4310	4420	4530	4640	4760	4880	5010	5140	5270
	7	5.43	346	3810	3890	3970	4060	4140	4230	4330	4420	4520	4630	4730
W24×84	TFL	0	1240	4810	4990	5170	5360	5560	5760	5970	6180	6400	6630	6860
(2370)	2	0.193	1060	4620	4790	4950	5130	5310	5490	5690	5880	6090	6300	6510
	3	0.385	888	4410	4560	4710	4870	5030	5200	5370	5550	5740	5930	6120
	4	0.578	714	4160	4290	4420	4560	4700	4850	5000	5160	5320	5480	5650
	BFL	0.770	540	3850	3960	4070	4190	4310	4430	4550	4680	4820	4960	5100
	6	3.02	425	3620	3710	3800	3900	4000	4100	4210	4320	4430	4550	4660
	7	5.48	309	3350	3420	3490	3570	3640	3720	3810	3890	3980	4070	4160
W24×76	TFL	0	1120	4280	4440	4600	4770	4950	5130	5320	5510	5710	5910	6120
(2100)	2	0.170	967	4120	4270	4420	4580	4740	4910	5080	5260	5440	5630	5830
	3	0.340	814	3930	4070	4210	4350	4500	4650	4810	4970	5140	5310	5490
	4	0.510	662	3720	3840	3960	4090	4220	4350	4490	4630	4780	4930	5090
	BFL	0.680	509	3460	3560	3660	3770	3880	3990	4110	4230	4360	4480	4610
	6	2.99	394	3230	3320	3400	3490	3580	3680	3770	3880	3980	4080	4190
	7	5.59	280	2970	3040	3100	3170	3240	3310	3390	3460	3540	3630	3710
W24×68	TFL	0	1010	3760	3900	4050	4200	4360	4520	4690	4860	5040	5220	5410
(1830)	2	0.146	874	3620	3760	3890	4030	4180	4330	4480	4640	4810	4980	5150
	3	0.293	743	3470	3590	3710	3840	3980	4110	4260	4400	4550	4710	4870
	4	0.439	611	3290	3390	3510	3620	3740	3860	3990	4120	4250	4390	4530
	BFL	0.585	480	3080	3170	3260	3360	3460	3570	3670	3790	3900	4020	4140
	6	3.04	366	2860	2930	3010	3090	3180	3260	3350	3450	3540	3640	3740
	7	5.80	251	2600	2660	2720	2780	2840	2900	2970	3040	3110	3180	3260

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Value in parentheses is I_x (in.^{4}) of noncomposite steel shape.

Table 3-20 (continued)
Lower-Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections, in.^{4}

F_y = 50 ksi

I_{LB}

W24–W21

Shape[d]	PNA[c]	Y1[a]	$\sum Q_n$	Y2[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W24×62 (1550)	TFL	0	910	3300	3420	3560	3690	3840	3980	4130	4290	4450	4610	4780
	2	0.148	806	3190	3310	3440	3560	3700	3840	3980	4120	4270	4430	4590
	3	0.295	702	3070	3180	3300	3420	3540	3670	3800	3940	4080	4220	4370
	4	0.443	598	2930	3040	3140	3250	3360	3480	3600	3720	3850	3980	4110
	BFL	0.590	495	2780	2870	2960	3060	3160	3260	3370	3480	3590	3710	3830
	6	3.45	361	2540	2610	2690	2770	2850	2930	3020	3110	3200	3290	3390
	7	6.56	228	2250	2300	2350	2410	2470	2520	2590	2650	2710	2780	2850
W24×55 (1350)	TFL	0	810	2890	3010	3120	3250	3370	3500	3640	3770	3920	4060	4210
	2	0.126	721	2800	2910	3020	3140	3250	3380	3500	3630	3770	3900	4050
	3	0.253	633	2700	2800	2910	3010	3120	3240	3360	3480	3600	3730	3860
	4	0.379	544	2590	2680	2780	2870	2970	3080	3190	3300	3410	3530	3650
	BFL	0.505	456	2460	2540	2630	2720	2810	2900	3000	3100	3200	3300	3410
	6	3.46	329	2240	2310	2370	2450	2520	2590	2670	2750	2830	2920	3000
	7	6.67	203	1970	2010	2060	2110	2160	2210	2270	2320	2380	2440	2500
W21×73 (1600)	TFL	0	1080	3310	3450	3590	3740	3900	4060	4220	4390	4570	4750	4940
	2	0.185	921	3170	3300	3430	3570	3710	3860	4010	4170	4330	4500	4670
	3	0.370	768	3020	3140	3260	3380	3510	3640	3780	3920	4070	4220	4380
	4	0.555	614	2840	2940	3050	3150	3270	3380	3500	3630	3750	3890	4020
	BFL	0.740	461	2620	2710	2790	2880	2980	3070	3170	3270	3380	3490	3600
	6	2.58	365	2470	2540	2610	2680	2760	2840	2930	3010	3100	3190	3290
	7	4.69	269	2280	2340	2400	2460	2520	2580	2650	2720	2790	2860	2930
W21×68 (1480)	TFL	0	1000	3060	3180	3320	3450	3600	3750	3900	4060	4220	4390	4560
	2	0.171	858	2930	3050	3180	3300	3440	3570	3710	3860	4010	4160	4320
	3	0.343	717	2800	2900	3010	3130	3250	3370	3500	3630	3770	3910	4050
	4	0.514	575	2630	2720	2820	2920	3030	3130	3250	3360	3480	3600	3730
	BFL	0.685	434	2430	2510	2590	2670	2760	2850	2940	3040	3140	3240	3340
	6	2.60	342	2280	2350	2420	2490	2560	2630	2710	2790	2880	2960	3050
	7	4.74	250	2110	2160	2210	2270	2330	2390	2450	2510	2580	2640	2710
W21×62 (1330)	TFL	0	915	2760	2880	3000	3120	3250	3390	3530	3670	3820	3970	4130
	2	0.154	788	2650	2760	2870	2990	3110	3240	3360	3500	3640	3780	3920
	3	0.308	662	2530	2630	2730	2840	2950	3060	3180	3300	3420	3550	3680
	4	0.461	535	2390	2470	2560	2650	2750	2850	2950	3060	3170	3280	3400
	BFL	0.615	408	2210	2280	2360	2440	2520	2600	2690	2770	2870	2960	3060
	6	2.54	318	2070	2130	2190	2260	2320	2390	2460	2540	2610	2690	2780
	7	4.78	229	1900	1950	2000	2050	2100	2150	2210	2270	2330	2390	2450

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Value in parentheses is I_x (in.^{4}) of noncomposite steel shape.

I_{LB}

W21

Table 3-20 (continued)
Lower-Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections, in.^{4}

F_y = 50 ksi

Shape[d]	PNA[c]	$Y1$[a]	$\sum Q_n$	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W21×57	TFL	0	835	2490	2590	2700	2820	2940	3060	3190	3320	3460	3600	3740
(1170)	2	0.163	728	2400	2490	2600	2710	2820	2930	3050	3170	3300	3430	3570
	3	0.325	622	2290	2380	2480	2580	2680	2780	2890	3010	3120	3240	3370
	4	0.488	515	2170	2250	2340	2430	2520	2610	2710	2810	2910	3020	3130
	BFL	0.650	409	2030	2110	2180	2250	2330	2410	2500	2580	2670	2770	2860
	6	2.93	309	1880	1940	2000	2060	2120	2190	2260	2330	2410	2480	2560
	7	5.40	209	1700	1740	1780	1830	1880	1930	1980	2030	2090	2140	2200
W21×55	TFL	0	810	2390	2490	2590	2710	2820	2940	3060	3190	3320	3450	3590
(1140)	2	0.131	703	2300	2390	2490	2590	2700	2810	2930	3040	3160	3290	3420
	3	0.261	595	2190	2280	2370	2470	2560	2660	2770	2870	2990	3100	3220
	4	0.392	488	2080	2150	2230	2320	2400	2490	2580	2680	2780	2880	2980
	BFL	0.522	381	1940	2000	2070	2140	2210	2290	2370	2450	2530	2620	2710
	6	2.62	292	1800	1850	1910	1970	2030	2090	2160	2230	2290	2370	2440
	7	5.00	203	1640	1680	1720	1770	1810	1860	1910	1960	2010	2070	2120
W21×50	TFL	0	735	2110	2210	2300	2400	2510	2620	2730	2840	2960	3080	3210
(984)	2	0.134	648	2040	2130	2220	2310	2410	2510	2620	2730	2840	2950	3070
	3	0.268	560	1960	2040	2130	2210	2300	2400	2490	2590	2690	2800	2910
	4	0.401	473	1870	1940	2020	2100	2180	2260	2350	2440	2530	2630	2730
	BFL	0.535	386	1760	1830	1890	1960	2030	2110	2180	2260	2350	2430	2520
	6	2.91	285	1620	1670	1720	1780	1840	1900	1960	2020	2090	2160	2230
	7	5.56	184	1440	1470	1510	1550	1590	1640	1680	1730	1780	1820	1880
W21×48	TFL	0	705	2030	2110	2210	2300	2400	2500	2610	2720	2830	2950	3070
(959)	2	0.108	617	1950	2040	2120	2210	2300	2400	2500	2600	2710	2820	2930
	3	0.215	530	1870	1950	2030	2110	2200	2280	2380	2470	2570	2670	2770
	4	0.323	442	1780	1850	1920	1990	2070	2150	2230	2320	2400	2490	2590
	BFL	0.430	355	1670	1730	1790	1860	1920	1990	2060	2140	2210	2290	2370
	6	2.71	266	1540	1590	1640	1690	1750	1810	1860	1920	1990	2050	2120
	7	5.26	176	1390	1420	1460	1500	1540	1580	1620	1660	1710	1750	1800
W21×44	TFL	0	650	1830	1920	2000	2090	2180	2280	2370	2480	2580	2690	2800
(843)	2	0.113	577	1780	1850	1930	2020	2100	2190	2280	2380	2480	2580	2680
	3	0.225	504	1710	1780	1850	1930	2010	2100	2180	2270	2360	2460	2550
	4	0.338	431	1630	1700	1770	1840	1910	1990	2060	2150	2230	2310	2400
	BFL	0.450	358	1550	1610	1670	1730	1790	1860	1930	2000	2080	2150	2230
	6	2.92	260	1410	1460	1500	1560	1610	1660	1720	1780	1840	1900	1960
	7	5.71	163	1240	1270	1310	1340	1380	1420	1460	1500	1540	1580	1630

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Value in parentheses is I_x (in.^{4}) of noncomposite steel shape.

Table 3-20 (continued)
Lower-Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections, in.^{4}

F_y = 50 ksi

I_{LB}

W18

Shape[d]	PNA[c]	Y1[a]	ΣQ_n	Y2[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W18×60	TFL	0	880	2070	2170	2270	2380	2490	2610	2730	2860	2990	3130	3270
(984)	2	0.174	749	1980	2070	2170	2270	2370	2480	2590	2710	2830	2950	3080
	3	0.348	617	1880	1960	2050	2140	2230	2330	2430	2530	2640	2750	2860
	4	0.521	486	1760	1830	1900	1980	2060	2140	2230	2320	2410	2510	2610
	BFL	0.695	355	1610	1660	1720	1790	1850	1920	1990	2060	2140	2220	2300
	6	2.18	287	1520	1570	1620	1670	1730	1780	1840	1910	1970	2040	2110
	7	3.80	220	1420	1460	1500	1540	1590	1640	1680	1730	1790	1840	1900
W18×55	TFL	0	810	1880	1970	2070	2170	2270	2380	2490	2600	2720	2850	2980
(890)	2	0.158	691	1800	1880	1970	2060	2160	2260	2360	2470	2580	2690	2810
	3	0.315	573	1710	1790	1860	1950	2030	2120	2210	2310	2410	2510	2620
	4	0.473	454	1600	1670	1730	1810	1880	1960	2040	2120	2210	2300	2390
	BFL	0.630	336	1470	1520	1580	1640	1700	1760	1830	1900	1970	2040	2110
	6	2.15	269	1380	1430	1480	1530	1580	1630	1690	1750	1800	1870	1930
	7	3.86	203	1290	1320	1360	1400	1440	1490	1530	1580	1630	1670	1730
W18×50	TFL	0	735	1690	1770	1860	1950	2040	2140	2240	2350	2450	2570	2680
(800)	2	0.143	628	1620	1700	1780	1860	1940	2030	2130	2220	2320	2430	2530
	3	0.285	521	1540	1610	1680	1750	1830	1910	2000	2080	2170	2260	2360
	4	0.428	414	1440	1500	1560	1630	1700	1770	1840	1910	1990	2070	2160
	BFL	0.570	308	1330	1370	1430	1480	1530	1590	1650	1710	1780	1840	1910
	6	2.08	246	1250	1290	1330	1380	1420	1470	1520	1580	1630	1690	1740
	7	3.82	184	1160	1190	1220	1260	1300	1340	1380	1420	1460	1510	1550
W18×46	TFL	0	675	1540	1610	1690	1780	1860	1950	2040	2140	2240	2340	2450
(712)	2	0.151	583	1480	1550	1620	1700	1780	1860	1950	2040	2130	2220	2320
	3	0.303	492	1410	1470	1540	1610	1680	1760	1840	1920	2000	2090	2180
	4	0.454	400	1330	1380	1440	1500	1570	1630	1700	1780	1850	1930	2010
	BFL	0.605	308	1230	1280	1330	1380	1430	1490	1550	1610	1670	1730	1800
	6	2.42	239	1140	1180	1220	1270	1310	1360	1410	1460	1510	1570	1620
	7	4.36	169	1040	1070	1100	1140	1170	1210	1250	1280	1320	1370	1410
W18×40	TFL	0	590	1320	1390	1450	1530	1600	1680	1760	1840	1930	2020	2110
(612)	2	0.131	511	1270	1330	1390	1460	1530	1600	1680	1760	1840	1920	2010
	3	0.263	432	1210	1270	1320	1390	1450	1510	1580	1650	1730	1800	1880
	4	0.394	353	1140	1190	1240	1300	1350	1410	1470	1530	1600	1670	1740
	BFL	0.525	274	1060	1100	1150	1190	1240	1290	1340	1390	1450	1510	1560
	6	2.26	211	985	1020	1060	1090	1130	1170	1220	1260	1310	1350	1400
	7	4.27	148	896	922	950	979	1010	1040	1070	1110	1140	1180	1210

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Value in parentheses is I_x (in.^{4}) of noncomposite steel shape.

I_{LB}

W18–W16

Table 3-20 (continued)
Lower-Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections, in.^{4}

F_y = 50 ksi

Shape[d]	PNA[c]	$Y1$[a]	$\sum Q_n$	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W18×35	TFL	0	515	1120	1170	1230	1300	1360	1430	1500	1570	1650	1720	1800
(510)	2	0.106	451	1080	1130	1190	1240	1300	1370	1430	1500	1570	1640	1720
	3	0.213	388	1030	1080	1130	1190	1240	1300	1360	1420	1490	1550	1620
	4	0.319	324	978	1020	1070	1120	1170	1220	1270	1330	1390	1450	1510
	BFL	0.425	260	917	955	995	1040	1080	1130	1170	1220	1270	1320	1380
	6	2.37	194	842	873	906	940	975	1010	1050	1090	1130	1170	1220
	7	4.56	129	753	776	800	825	851	878	906	935	965	996	1030
W16×45	TFL	0	665	1260	1330	1400	1470	1550	1630	1720	1810	1900	1990	2090
(586)	2	0.141	566	1200	1270	1330	1400	1470	1550	1630	1710	1790	1880	1970
	3	0.283	466	1140	1200	1260	1320	1380	1450	1520	1590	1670	1750	1830
	4	0.424	367	1060	1110	1160	1220	1270	1330	1390	1450	1520	1590	1660
	BFL	0.565	267	971	1010	1050	1090	1140	1190	1230	1290	1340	1390	1450
	6	1.77	217	917	950	986	1020	1060	1100	1140	1190	1230	1280	1330
	7	3.23	166	854	882	910	940	972	1000	1040	1070	1110	1150	1190
W16×40	TFL	0	590	1110	1170	1230	1300	1370	1440	1520	1590	1670	1760	1850
(518)	2	0.126	502	1060	1120	1170	1240	1300	1370	1430	1510	1580	1660	1740
	3	0.253	413	1000	1050	1110	1160	1220	1280	1340	1400	1470	1540	1610
	4	0.379	325	937	980	1030	1070	1120	1170	1230	1280	1340	1400	1460
	BFL	0.505	237	856	891	927	965	1000	1050	1090	1130	1180	1230	1280
	6	1.70	192	808	837	869	901	935	971	1010	1050	1090	1130	1170
	7	3.16	148	755	779	804	831	859	888	918	949	982	1020	1050
W16×36	TFL	0	530	973	1030	1080	1140	1200	1270	1340	1410	1480	1550	1630
(448)	2	0.108	455	933	983	1040	1090	1150	1210	1270	1330	1400	1470	1540
	3	0.215	380	886	931	979	1030	1080	1130	1190	1250	1310	1370	1440
	4	0.323	305	831	871	912	956	1000	1050	1100	1150	1200	1260	1310
	BFL	0.430	229	765	797	831	867	905	944	984	1030	1070	1120	1160
	6	1.82	181	715	743	772	802	833	866	901	936	973	1010	1050
	7	3.46	133	659	680	703	727	752	778	805	833	862	892	923
W16×31	TFL	0	457	827	874	923	974	1030	1080	1140	1200	1260	1330	1400
(375)	2	0.110	396	795	838	884	931	981	1030	1090	1140	1200	1260	1320
	3	0.220	335	758	797	838	882	927	974	1020	1070	1130	1180	1240
	4	0.330	274	714	749	786	824	864	906	949	995	1040	1090	1140
	BFL	0.440	213	663	692	723	756	790	825	862	900	940	982	1020
	6	2.00	164	614	639	664	691	720	749	780	812	845	879	914
	7	3.80	114	556	574	594	614	636	658	681	705	730	756	783

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Value in parentheses is I_x (in.^{4}) of noncomposite steel shape.

Table 3-20 (continued)
Lower-Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections, in.^{4}

F_y = 50 ksi

I_{LB}
W16–W14

Shape[d]	PNA[c]	Y1[a]	ΣQn	Y2[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W16×26	TFL	0	384	674	712	753	796	840	887	935	985	1040	1090	1150
(301)	2	0.0863	337	649	686	724	763	805	849	894	941	990	1040	1090
	3	0.173	289	621	654	689	726	764	804	846	889	934	980	1030
	4	0.259	242	589	619	651	683	718	754	791	830	871	912	956
	BFL	0.345	194	551	577	604	633	663	694	727	760	795	832	869
	6	2.05	145	505	527	549	572	597	622	649	676	705	734	765
	7	4.01	96.0	450	466	482	499	517	535	555	575	596	617	640
W14×38	TFL	0	560	844	896	951	1010	1070	1130	1200	1270	1340	1410	1490
(385)	2	0.129	473	805	853	903	956	1010	1070	1130	1190	1260	1330	1400
	3	0.258	386	759	802	847	894	943	995	1050	1100	1160	1220	1290
	4	0.386	299	704	741	779	819	861	905	951	999	1050	1100	1150
	BFL	0.515	211	636	665	695	726	759	794	830	868	907	948	990
	6	1.38	176	604	629	656	683	712	742	774	807	841	877	914
	7	2.53	140	568	589	611	634	659	684	710	738	766	796	827
W14×34	TFL	0	500	745	791	840	891	945	1000	1060	1120	1190	1250	1320
(340)	2	0.114	423	711	754	798	845	895	946	1000	1060	1110	1180	1240
	3	0.228	346	671	709	749	791	835	881	929	979	1030	1090	1140
	4	0.341	270	624	656	691	727	764	804	845	888	933	979	1030
	BFL	0.455	193	566	591	618	647	677	708	741	775	811	848	886
	6	1.42	159	535	558	581	606	632	659	687	717	748	780	813
	7	2.61	125	502	521	540	561	582	605	628	653	678	705	732
W14×30	TFL	0	443	642	682	725	770	817	866	918	972	1030	1090	1150
(291)	2	0.0963	378	614	651	691	732	775	821	868	918	969	1020	1080
	3	0.193	313	581	615	650	688	727	767	810	855	901	949	999
	4	0.289	248	543	572	603	635	669	704	741	780	820	862	905
	BFL	0.385	183	496	520	545	571	599	627	658	689	722	756	791
	6	1.46	147	466	486	507	530	553	578	604	630	658	687	717
	7	2.80	111	432	448	465	483	502	522	542	564	586	610	634
W14×26	TFL	0	385	553	589	626	665	706	749	794	841	890	941	994
(245)	2	0.105	332	530	563	598	634	672	712	754	797	843	890	938
	3	0.210	279	504	534	565	598	633	669	707	746	787	830	874
	4	0.315	226	473	499	527	556	586	618	652	686	722	760	799
	BFL	0.420	173	436	458	481	506	531	558	586	615	645	677	709
	6	1.67	135	405	423	443	463	485	507	530	555	580	607	634
	7	3.18	96.1	368	382	397	413	429	447	465	483	503	523	544

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Value in parentheses is I_x (in.^{4}) of noncomposite steel shape.

Table 3-20 (continued)
Lower-Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections, in.^{4}

I_{LB}

W14–W12

F_y = 50 ksi

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W14×22	TFL	0	325	453	483	514	547	581	617	655	694	735	778	822
(199)	2	0.0838	283	436	463	492	523	555	588	624	660	698	738	779
	3	0.168	241	416	441	467	495	525	555	587	621	656	692	730
	4	0.251	199	392	415	438	463	489	517	545	575	606	639	672
	BFL	0.335	157	365	384	404	426	448	472	496	522	548	576	605
	6	1.67	119	335	351	368	386	404	423	444	465	487	509	533
	7	3.32	81.1	301	312	325	338	352	366	381	397	413	430	448
W12×30	TFL	0	440	530	567	606	648	691	737	785	835	887	942	998
(238)	2	0.110	368	504	538	573	611	651	692	736	782	829	879	931
	3	0.220	296	473	503	534	567	602	639	678	718	760	804	850
	4	0.330	224	435	460	486	514	544	575	607	641	676	713	751
	BFL	0.440	153	389	408	428	449	472	495	520	546	573	601	631
	6	1.10	131	372	389	407	426	446	467	489	512	536	561	587
	7	1.92	110	355	370	385	402	419	438	457	477	498	520	542
W12×26	TFL	0	383	455	487	521	557	594	634	676	719	764	812	861
(204)	2	0.0950	321	433	462	493	526	560	596	634	674	715	758	803
	3	0.190	259	407	432	460	489	519	551	585	620	656	694	734
	4	0.285	198	375	397	420	444	470	497	525	555	586	618	652
	BFL	0.380	136	336	352	370	389	409	429	451	474	498	523	548
	6	1.07	116	321	336	351	368	386	404	423	444	465	487	509
	7	1.94	95.6	304	317	331	345	360	376	392	410	428	447	467
W12×22	TFL	0	324	371	398	427	458	490	523	559	596	634	674	716
(156)	2	0.106	281	356	381	408	436	466	497	530	564	600	638	676
	3	0.213	238	338	361	386	412	439	467	497	528	561	595	631
	4	0.319	196	318	339	360	383	408	433	460	487	517	547	578
	BFL	0.425	153	294	312	330	350	370	392	414	438	463	489	515
	6	1.66	117	270	285	300	316	333	351	370	389	410	431	453
	7	3.03	81.0	242	253	265	277	290	303	317	332	347	363	380
W12×19	TFL	0	279	313	336	361	387	414	443	473	505	538	573	608
(130)	2	0.0875	243	300	322	345	369	395	422	450	479	510	542	575
	3	0.175	208	286	306	327	349	373	398	423	450	479	508	539
	4	0.263	173	270	288	307	327	348	370	393	417	442	469	496
	BFL	0.350	138	251	266	283	300	318	337	357	378	400	423	447
	6	1.68	104	229	242	255	270	284	300	317	334	352	370	390
	7	3.14	69.6	203	212	222	233	244	255	267	280	293	307	321

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Value in parentheses is I_x (in.^{4}) of noncomposite steel shape.

Table 3-20 (continued)
Lower-Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections, in.^{4}

F_y = 50 ksi

I_{LB}

W12–W10

Shape[d]	PNA[c]	$Y1$[a]	$\sum Q_n$	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W12×16	TFL	0	236	254	273	294	316	339	363	388	415	442	471	501
(103)	2	0.0663	209	245	263	282	303	324	347	371	396	422	449	477
	3	0.133	183	235	252	270	289	309	330	352	375	400	425	451
	4	0.199	156	223	239	255	272	291	310	330	351	373	396	420
	BFL	0.265	130	210	224	239	254	271	288	306	325	344	365	386
	6	1.71	94.3	189	200	212	225	238	251	266	281	297	313	331
	7	3.32	58.9	163	171	179	188	197	207	217	228	239	250	262
W12×14	TFL	0	208	220	237	255	274	295	316	338	361	386	411	437
(88.6)	2	0.0563	186	213	229	246	264	283	303	324	346	369	393	418
	3	0.113	163	204	219	235	252	270	288	308	328	350	372	395
	4	0.169	141	195	209	223	239	255	272	290	309	329	349	370
	BFL	0.225	119	184	197	210	224	238	254	270	287	305	323	342
	6	1.68	85.3	165	175	186	197	208	221	234	247	261	276	291
	7	3.35	52.0	141	148	155	163	171	179	188	198	207	218	228
W10×26	TFL	0	381	339	367	397	429	463	499	536	576	617	661	706
(144)	2	0.110	317	321	346	374	403	434	466	500	536	574	613	655
	3	0.220	254	300	322	346	372	399	428	458	490	523	557	594
	4	0.330	190	274	292	312	334	356	380	405	431	459	488	518
	BFL	0.440	127	241	255	270	286	303	321	340	360	381	402	425
	6	0.886	111	232	245	258	273	288	304	321	339	358	377	398
	7	1.49	95.1	222	233	245	258	271	286	301	317	333	351	369
W10×22	TFL	0	325	282	306	331	358	387	417	449	483	518	555	593
(118)	2	0.0900	273	267	289	313	337	364	391	420	451	483	517	552
	3	0.180	221	251	270	291	312	336	360	386	413	442	472	503
	4	0.270	169	230	246	264	282	302	323	345	368	392	417	443
	BFL	0.360	118	205	218	232	246	261	277	295	312	331	351	371
	6	0.962	99.3	195	206	218	230	244	258	273	289	305	323	341
	7	1.72	81.1	183	193	203	214	225	238	250	264	278	293	308
W10×19	TFL	0	281	238	259	281	304	329	355	383	412	443	474	508
(96.3)	2	0.0988	241	227	246	267	288	311	335	361	388	416	445	476
	3	0.198	202	215	232	251	270	291	313	336	360	386	413	440
	4	0.296	162	200	215	231	248	266	286	306	327	350	373	397
	BFL	0.395	122	182	195	208	222	237	253	270	287	306	325	345
	6	1.25	96.2	169	179	190	202	215	228	243	257	273	289	306
	7	2.29	70.3	153	161	170	179	189	200	211	223	235	248	261

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Value in parentheses is I_x (in.^{4}) of noncomposite steel shape.

Table 3-20 (continued)
Lower-Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections, in.^{4}

I_{LB}

W10

F_y = 50 ksi

Shape[d]	PNA[c]	$Y1$[a]	$\sum Q_n$	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W10×17	TFL	0	250	206	224	244	264	286	310	334	360	387	415	445
(81.9)	2	0.0825	216	197	214	232	251	272	293	316	340	365	391	418
	3	0.165	183	187	202	219	236	255	274	295	317	340	364	388
	4	0.248	150	175	189	203	219	235	253	271	290	311	332	354
	BFL	0.330	117	161	173	185	198	212	227	243	259	276	294	313
	6	1.31	89.8	148	157	167	178	190	202	215	229	243	258	274
	7	2.45	62.4	132	139	147	155	164	173	183	193	204	215	227
W10×15	TFL	0	221	177	193	210	228	248	268	289	312	336	361	387
(68.9)	2	0.0675	194	170	185	201	218	236	255	275	296	318	342	366
	3	0.135	167	162	176	190	206	223	240	259	278	299	320	342
	4	0.203	140	153	165	178	192	207	223	240	258	276	295	315
	BFL	0.270	113	142	153	164	177	190	204	218	233	250	266	284
	6	1.35	83.8	128	137	147	157	167	178	190	203	216	229	244
	7	2.60	55.1	112	118	125	133	140	148	157	166	175	185	196
W10×12	TFL	0	177	139	152	165	180	195	211	229	247	265	285	306
(53.8)	2	0.0525	156	134	145	158	172	186	201	217	234	252	271	290
	3	0.105	135	127	138	150	163	176	190	205	221	237	254	272
	4	0.158	115	121	131	142	153	165	178	191	206	221	236	252
	BFL	0.210	93.8	113	122	131	141	152	163	175	187	200	214	228
	6	1.30	69.0	102	109	116	124	133	142	152	162	173	184	195
	7	2.61	44.3	87.9	93.0	98.4	104	110	117	124	131	139	146	155

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis
[b] $Y2$ = distance from top of the steel beam to concrete flange force
[c] See Figure 3-3(c) for PNA locations.
[d] Value in parentheses is I_x (in.^{4}) of noncomposite steel shape.

Table 3-21
Shear Stud Anchor
Nominal Horizontal Shear Strength for One Steel Headed Stud Anchor, Q_n, kips

F_u = 65 ksi

Q_n

Deck Condition			Stud Anchor Diameter, in.	Normal Weight Concrete w_c = 145 pcf, f'_c = 3 ksi	Normal Weight Concrete w_c = 145 pcf, f'_c = 4 ksi	Lightweight Concrete w_c = 110 pcf, f'_c = 3 ksi	Lightweight Concrete w_c = 110 pcf, f'_c = 4 ksi
No Deck			3/8	5.26	5.38	4.28	5.31
			1/2	9.35	9.57	7.60	9.43
			5/8	14.6	15.0	11.9	14.7
			3/4	21.0	21.5	17.1	21.2
			7/8	28.6	29.3	23.3	28.9
			1	37.4	38.3	30.4	37.7
Deck Parallel	$\frac{w_r}{h_r} \geq 1.5$		3/8	5.26	5.38	4.28	5.31
			1/2	9.35	9.57	7.60	9.43
			5/8	14.6	15.0	11.9	14.7
			3/4	21.0	21.5	17.1	21.2
	$\frac{w_r}{h_r} < 1.5$		3/8	4.58	4.58	4.28	4.58
			1/2	8.14	8.14	7.60	8.14
			5/8	12.7	12.7	11.9	12.7
			3/4	18.3	18.3	17.1	18.3
Deck Perpendicular	Weak studs per rib (R_p = 0.60)	1	3/8	4.31	4.31	4.28	4.31
			1/2	7.66	7.66	7.60	7.66
			5/8	12.0	12.0	11.9	12.0
			3/4	17.2	17.2	17.1	17.2
		2	3/8	3.66	3.66	3.66	3.66
			1/2	6.51	6.51	6.51	6.51
			5/8	10.2	10.2	10.2	10.2
			3/4	14.6	14.6	14.6	14.6
		3	3/8	3.02	3.02	3.02	3.02
			1/2	5.36	5.36	5.36	5.36
			5/8	8.38	8.38	8.38	8.38
			3/4	12.1	12.1	12.1	12.1
	Strong studs per rib (R_p = 0.75)	1	3/8	5.26	5.38	4.28	5.31
			1/2	9.35	9.57	7.60	9.43
			5/8	14.6	15.0	11.9	14.7
			3/4	21.0	21.5	17.1	21.2
		2	3/8	4.58	4.58	4.28	4.58
			1/2	8.14	8.14	7.60	8.14
			5/8	12.7	12.7	11.9	12.7
			3/4	18.3	18.3	17.1	18.3
		3	3/8	3.77	3.77	3.77	3.77
			1/2	6.70	6.70	6.70	6.70
			5/8	10.5	10.5	10.5	10.5
			3/4	15.1	15.1	15.1	15.1

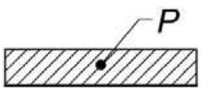
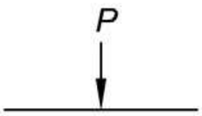
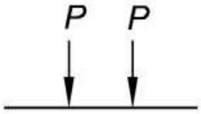
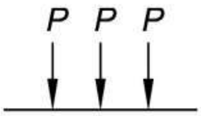
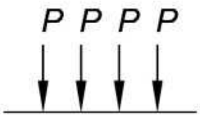
Notes: Tabulated values are applicable only to concrete made with ASTM C33 aggregates for normal weight concrete and ASTM C330 aggregates for lightweight concrete.
After-weld steel headed stud anchor lengths assumed to be ≥ Deck height + 1.5 in.

BEAM DIAGRAMS AND FORMULAS

The following variable definitions apply to Tables 3-22 and 3-23.

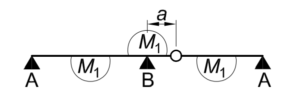
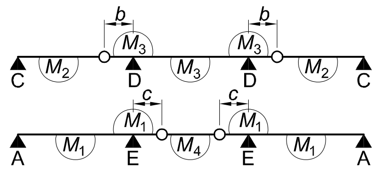
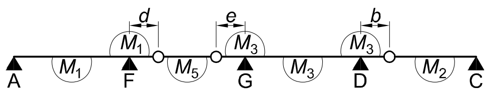
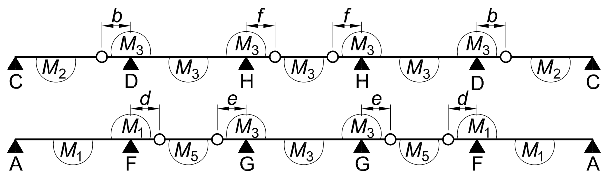
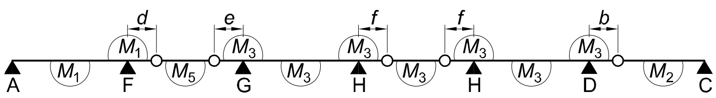
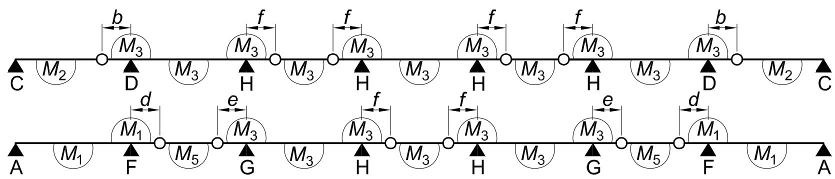
E = Modulus of elasticity of steel = 29,000 ksi
I = Moment of inertia of beam, in.^{4}
L = Total length of beam between reaction points, ft
M_{max} = Maximum moment, kip-in.
M_1 = Maximum moment in left section of beam, kip-in.
M_2 = Maximum moment in right section of beam, kip-in.
M_3 = Maximum positive moment in beam with combined end moment conditions, kip-in.
M_x = Moment at distance x from end of beam, kip-in.
P = Concentrated load, kips
P_1 = Concentrated load nearest left reaction, kips
P_2 = Concentrated load nearest right reaction, and of different magnitude than P_1, kips
R = End beam reaction for any condition of symmetrical loading, kips
R_1 = Left end beam reaction, kips
R_2 = Right end or intermediate beam reaction, kips
R_3 = Right end beam reaction, kips
V = Maximum vertical shear for any condition of symmetrical loading, kips
V_1 = Maximum vertical shear in left section of beam, kips
V_2 = Vertical shear at right reaction point, or to left of intermediate reaction point of beam, kips
V_3 = Vertical shear at right reaction point, or to right of intermediate reaction point of beam, kips
V_x = Vertical shear at distance x from end of beam, kips
W = Total load on beam, kips
a = Measured distance along beam, in.
b = Measured distance along beam which may be greater or less than a, in.
l = Total length of beam between reaction points, in.
w = Uniformly distributed load per unit of length, kip/in.
w_1 = Uniformly distributed load per unit of length nearest left reaction, kip/in.
w_2 = Uniformly distributed load per unit of length nearest right reaction and of different magnitude than w_1, kip/in.
x = Any distance measured along beam from left reaction, in.
x_1 = Any distance measured along overhang section of beam from nearest reaction point, in.
Δ_{max} = Maximum deflection, in.
Δ_a = Deflection at point of load, in.
Δ_x = Deflection at any point x distance from left reaction, in.
Δ_{x1} = Deflection of overhang section of beam at any distance from nearest reaction point, in.

Table 3-22a
Concentrated Load Equivalents

n	Loading	Coeff.	Simple Beam	Beam Fixed One End, Supported at Other	Beam Fixed Both Ends
					
∞		a	0.125	0.070	0.042
		b	—	0.125	0.083
		c	0.500	0.375	—
		d	—	0.625	0.500
		e	0.013	0.005	0.003
		f	1.000	1.000	0.667
		g	1.000	0.415	0.300
2		a	0.250	0.156	0.125
		b	—	0.188	0.125
		c	0.500	0.313	—
		d	—	0.688	0.500
		e	0.021	0.009	0.005
		f	2.000	1.500	1.000
		g	0.800	0.477	0.400
3		a	0.333	0.222	0.111
		b	—	0.333	0.222
		c	1.000	0.667	—
		d	—	1.333	1.000
		e	0.036	0.015	0.008
		f	2.667	2.667	1.778
		g	1.022	0.438	0.333
4		a	0.500	0.266	0.188
		b	—	0.469	0.313
		c	1.500	1.031	—
		d	—	1.969	1.500
		e	0.050	0.021	0.010
		f	4.000	3.750	2.500
		g	0.950	0.428	0.320
5		a	0.600	0.360	0.200
		b	—	0.600	0.400
		c	2.000	1.400	—
		d	—	2.600	2.000
		e	0.063	0.027	0.013
		f	4.800	4.800	3.200
		g	1.008	0.424	0.312

Maximum positive moment (kip-ft): aPL
Maximum negative moment (kip-ft): bPL
Pinned end reaction (kips): cP
Fixed end reaction (kips): dP
Maximum deflection (in.): ePl^3 / EI
Equivalent simple span uniform load (kips): fP
Deflection coefficient for equivalent simple span uniform load: g
Number of equal load spaces: n
Span of beam (ft): L
Span of beam (in.): l

Table 3-22b
Cantilevered Beams
Beam Diagrams and Formulas— Equal Loads, Equally Spaced

No. Spans	System
2	
3	
4	
5	
≥6 (even)	
≥7 (odd)	

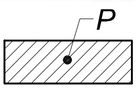
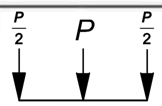
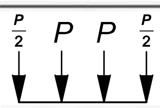
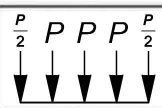
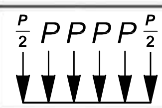
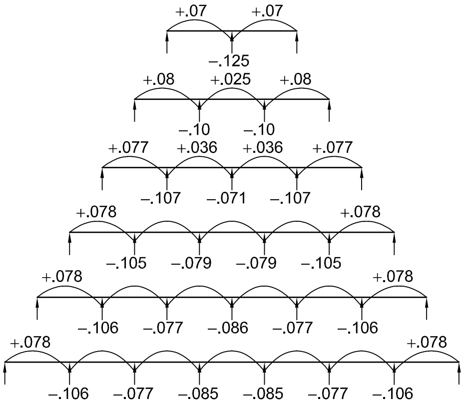
	n	∞	2	3	4	5
	Typical Span Loading					
Moments	M_1	0.086×PL	0.167×PL	0.250×PL	0.333×PL	0.429×PL
	M_2	0.096×PL	0.188×PL	0.278×PL	0.375×PL	0.480×PL
	M_3	0.063×PL	0.125×PL	0.167×PL	0.250×PL	0.300×PL
	M_4	0.039×PL	0.083×PL	0.083×PL	0.167×PL	0.171×PL
	M_5	0.051×PL	0.104×PL	0.139×PL	0.208×PL	0.249×PL
Reactions	A	0.414×P	0.833×P	1.250×P	1.667×P	2.071×P
	B	1.172×P	2.333×P	3.500×P	4.667×P	5.857×P
	C	0.438×P	0.875×P	1.333×P	1.750×P	2.200×P
	D	1.063×P	2.125×P	3.167×P	4.250×P	5.300×P
	E	1.086×P	2.167×P	3.250×P	4.333×P	5.429×P
	F	1.109×P	2.208×P	3.333×P	4.417×P	5.557×P
	G	0.977×P	1.958×P	2.917×P	3.917×P	4.871×P
	H	1.000×P	2.000×P	3.000×P	4.000×P	5.000×P
Cantilever Dimensions	a	0.172×L	0.250×L	0.200×L	0.182×L	0.176×L
	b	0.125×L	0.200×L	0.143×L	0.143×L	0.130×L
	c	0.220×L	0.333×L	0.250×L	0.222×L	0.229×L
	d	0.204×L	0.308×L	0.231×L	0.211×L	0.203×L
	e	0.157×L	0.273×L	0.182×L	0.176×L	0.160×L
	f	0.147×L	0.250×L	0.167×L	0.167×L	0.150×L

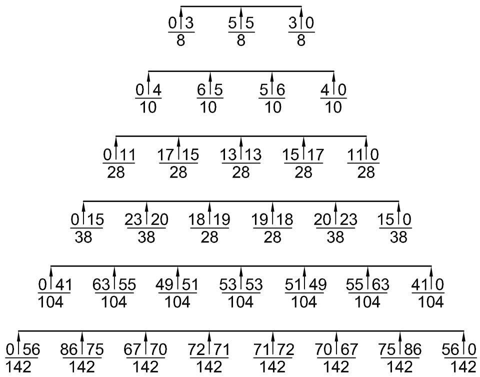
Table 3-22c
Continuous Beams
Moments and Shear Coefficients—Equal Spans, Equally Loaded

Uniform Load

Moment in terms of wl^2

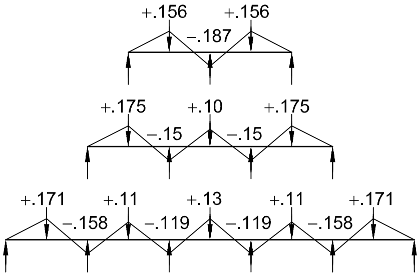


Shear in terms of wl

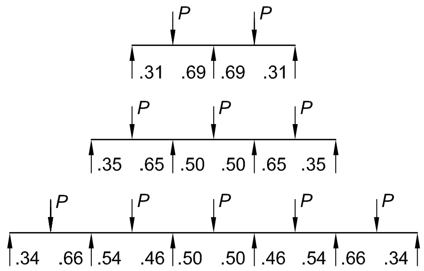


Concentrated Load at Center

Moment in terms of Pl

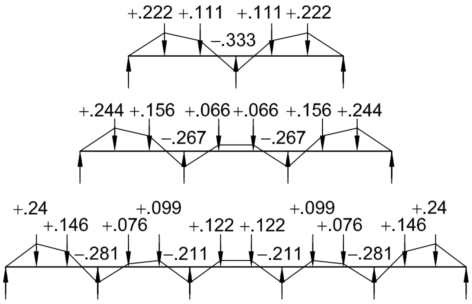


Shear in terms of P

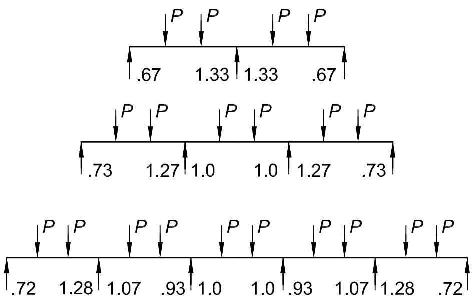


Concentrated Load at Third Points

Moment in terms of Pl

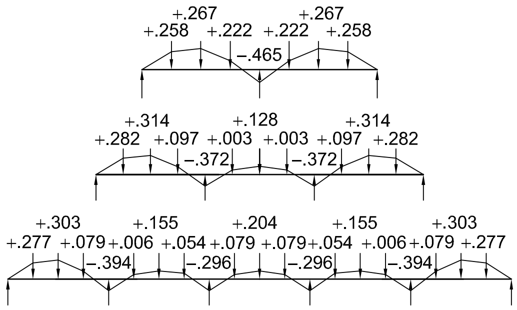


Shear in terms of P



Concentrated Load at Quarter Points

Moment in terms of Pl



Shear in terms of P

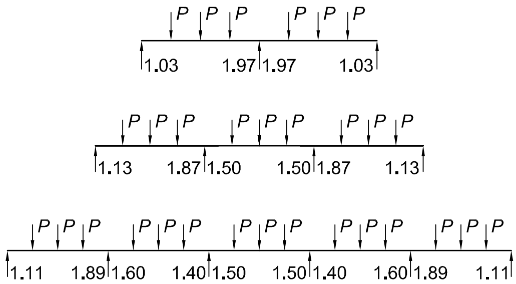
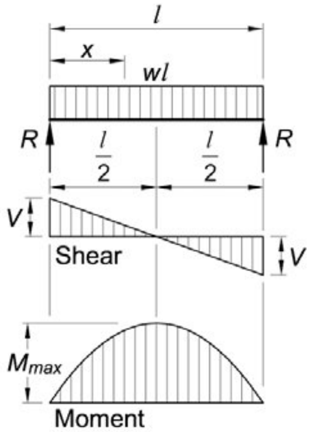


Table 3-23
Shears, Moments and Deflections

1. SIMPLE BEAM — UNIFORMLY DISTRIBUTED LOAD



Total Equiv. Uniform Load .... $= wl$

$R = V$ .... $= \dfrac{wl}{2}$

V_x .... $= w\left(\dfrac{l}{2} - x\right)$

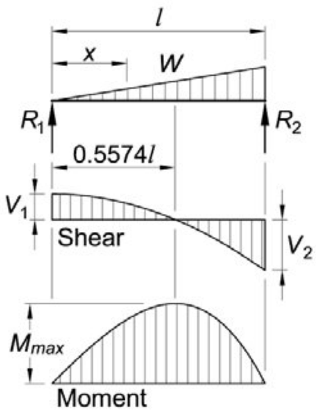
M_{max} (at center) .... $= \dfrac{wl^2}{8}$

M_x .... $= \dfrac{wx}{2}(l - x)$

Δ_{max} (at center) .... $= \dfrac{5wl^4}{384EI}$

Δ_x .... $= \dfrac{wx}{24EI}\left(l^3 - 2lx^2 + x^3\right)$

2. SIMPLE BEAM — LOAD INCREASING UNIFORMLY TO ONE END



Total Equiv. Uniform Load .... $= \dfrac{16W}{9\sqrt{3}} = 1.03\,W$

$R_1 = V_1$ .... $= \dfrac{W}{3}$

$R_2 = V_2 = V_{max}$ .... $= \dfrac{2W}{3}$

V_x .... $= \dfrac{W}{3} - \dfrac{Wx^2}{l^2}$

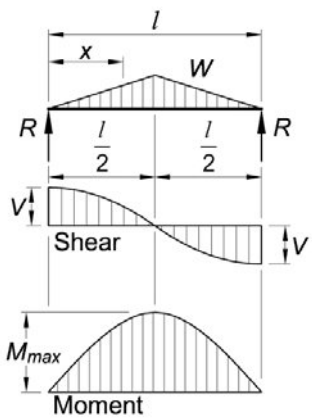
M_{max} $\left(\text{at } x = \dfrac{l}{\sqrt{3}} = 0.557\,l\right)$ .... $= \dfrac{2Wl}{9\sqrt{3}} = 0.128\,Wl$

M_x .... $= \dfrac{Wx}{3l^2}\left(l^2 - x^2\right)$

Δ_{max} $\left(\text{at } x = l\sqrt{1 - \sqrt{\dfrac{8}{15}}} = 0.519\,l\right)$ .... $= 0.0130\,\dfrac{Wl^3}{EI}$

Δ_x .... $= \dfrac{Wx}{180EIl^2}\left(3x^4 - 10l^2x^2 + 7l^4\right)$

3. SIMPLE BEAM — LOAD INCREASING UNIFORMLY TO CENTER



Total Equiv. Uniform Load .... $= \dfrac{4W}{3}$

$R = V$ .... $= \dfrac{W}{2}$

V_x (when $x < \dfrac{l}{2}$) .... $= \dfrac{W}{2l^2}\left(l^2 - 4x^2\right)$

M_{max} (at center) .... $= \dfrac{Wl}{6}$

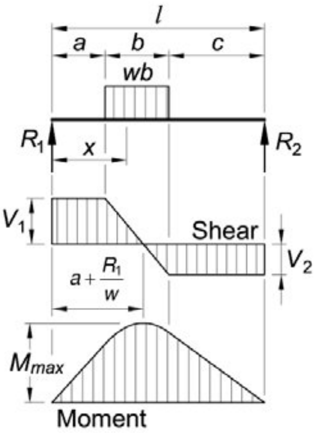
M_x (when $x < \dfrac{l}{2}$) .... $= Wx\left(\dfrac{1}{2} - \dfrac{2x^2}{3l^2}\right)$

Δ_{max} (at center) .... $= \dfrac{Wl^3}{60EI}$

Δ_x (when $x < \dfrac{l}{2}$) .... $= \dfrac{Wx}{480EIl^2}\left(5l^2 - 4x^2\right)^2$

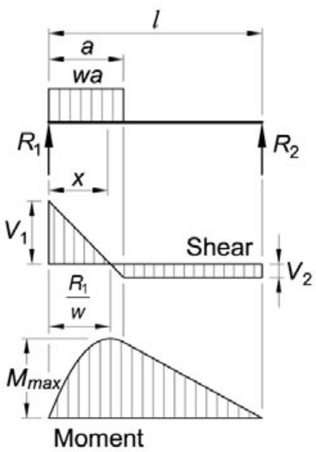
Table 3-23 (continued)
Shears, Moments and Deflections

4. SIMPLE BEAM — UNIFORM LOAD PARTIALLY DISTRIBUTED



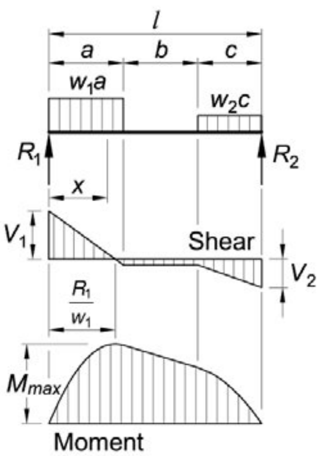
$R_1 = V_1$	(max. when $a < c$)	$= \frac{wb}{2l}(2c+b)$
$R_2 = V_2$	(max. when $a > c$)	$= \frac{wb}{2l}(2a+b)$
V_x	[when $x > a$ and $< (a+b)$]	$= R_1 - w(x-a)$
M_{max}	$\left(\text{at } x = a + \frac{R_1}{w}\right)$	$= R_1\left(a + \frac{R_1}{2w}\right)$
M_x	(when $x < a$)	$= R_1 x$
M_x	[when $x > a$ and $< (a+b)$]	$= R_1 x - \frac{w}{2}(x-a)^2$
M_x	[when $x > (a+b)$]	$= R_2(l-x)$

5. SIMPLE BEAM — UNIFORM LOAD PARTIALLY DISTRIBUTED AT ONE END



$R_1 = V_1 = V_{max}$	....	$= \frac{wa}{2l}(2l-a)$
$R_2 = V_2$	....	$= \frac{wa^2}{2l}$
V_x	(when $x < a$)	$= R_1 - wx$
M_{max}	$\left(\text{at } x = \frac{R_1}{w}\right)$	$= \frac{R_1^2}{2w}$
M_x	(when $x < a$)	$= R_1 x - \frac{wx^2}{2}$
M_x	(when $x > a$)	$= R_2(l-x)$
Δ_x	(when $x < a$)	$= \frac{wx}{24EIl}\left[a^2(2l-a)^2 - 2ax^2(2l-a) + lx^3\right]$
Δ_x	(when $x > a$)	$= \frac{wa^2(l-x)}{24EIl}\left(4xl - 2x^2 - a^2\right)$

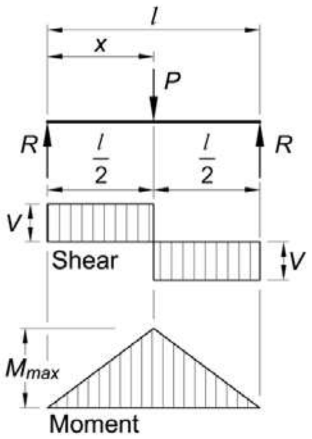
6. SIMPLE BEAM — UNIFORM LOAD PARTIALLY DISTRIBUTED AT EACH END



$R_1 = V_1$	....	$= \frac{w_1 a(2l-a) + w_2 c^2}{2l}$
$R_2 = V_2$	....	$= \frac{w_2 c(2l-c) + w_1 a^2}{2l}$
V_x	(when $x < a$)	$= R_1 - w_1 x$
V_x	[when $a < x < (a+b)$]	$= R_1 - w_1 a$
V_x	[when $x > (a+b)$]	$= R_2 - w_2(l-x)$
M_{max}	$\left(\text{at } x = \frac{R_1}{w_1}, \text{when } R_1 < w_1 a\right)$	$= \frac{R_1^2}{2w_1}$
M_{max}	$\left(\text{at } x = l - \frac{R_2}{w_2}, \text{when } R_2 < w_2 c\right)$	$= \frac{R_2^2}{2w_2}$
M_x	(when $x < a$)	$= R_1 x - \frac{w_1 x^2}{2}$
M_x	[when $a < x < (a+b)$]	$= R_1 x - \frac{w_1 a}{2}(2x-a)$
M_x	[when $x > (a+b)$]	$= R_2(l-x) - \frac{w_2(l-x)^2}{2}$

Table 3-23 (continued)
Shears, Moments and Deflections

7. SIMPLE BEAM — CONCENTRATED LOAD AT CENTER



Total Equiv. Uniform Load .... $= 2P$

$R = V$ .... $= \frac{P}{2}$

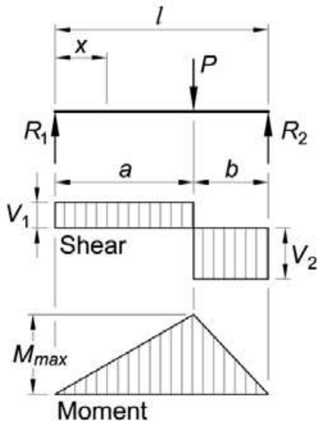
M_{max} (at point of load) .... $= \frac{Pl}{4}$

M_x (when $x < \frac{l}{2}$) .... $= \frac{Px}{2}$

Δ_{max} (at point of load) .... $= \frac{Pl^3}{48EI}$

Δ_x (when $x < \frac{l}{2}$) .... $= \frac{Px}{48EI}\left(3l^2 - 4x^2\right)$

8. SIMPLE BEAM — CONCENTRATED LOAD AT ANY POINT



Total Equiv. Uniform Load .... $= \frac{8Pab}{l^2}$

$R_1 = V_1$ ($= V_{max}$ when $a < b$) .... $= \frac{Pb}{l}$

$R_2 = V_2$ ($= V_{max}$ when $a > b$) .... $= \frac{Pa}{l}$

M_{max} (at point of load) .... $= \frac{Pab}{l}$

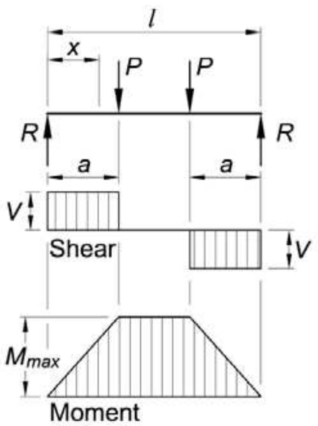
M_x (when $x < a$) .... $= \frac{Pbx}{l}$

Δ_{max} $\left[\text{at } x = \sqrt{\frac{a(a+2b)}{3}}, \text{ when } a > b\right]$ .... $= \frac{Pab(a+2b)\sqrt{3a(a+2b)}}{27EIl}$

Δ_a (at point of load) .... $= \frac{Pa^2b^2}{3EIl}$

Δ_x (when $x < a$) .... $= \frac{Pbx}{6EIl}\left(l^2 - b^2 - x^2\right)$

9. SIMPLE BEAM — TWO EQUAL CONCENTRATED LOADS SYMMETRICALLY PLACED



Total Equiv. Uniform Load .... $= \frac{8Pa}{l}$

$R = V$ .... $= P$

M_{max} (between loads) .... $= Pa$

M_x (when $x < a$) .... $= Px$

Δ_{max} (at center) .... $= \frac{Pa}{24EI}\left(3l^2 - 4a^2\right)$

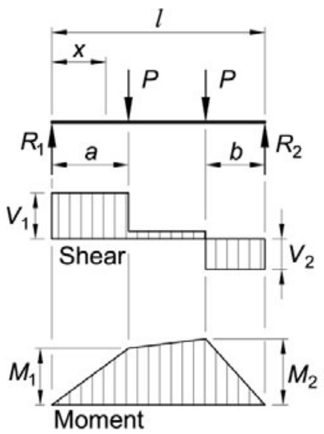
Δ_{max} (at $a = \frac{l}{3}$) .... $= \frac{23Pl^3}{648EI}$

Δ_x (when $x < a$) .... $= \frac{Px}{6EI}\left(3la - 3a^2 - x^2\right)$

Δ_x [when $a < x < (l - a)$] .... $= \frac{Pa}{6EI}\left(3lx - 3x^2 - a^2\right)$

Table 3-23 (continued)
Shears, Moments and Deflections

10. SIMPLE BEAM — TWO EQUAL CONCENTRATED LOADS UNSYMMETRICALLY PLACED



$R_1 = V_1$ ($= V_{max}$ when $a < b$) .... $= \frac{P}{l}(l - a + b)$

$R_2 = V_2$ ($= V_{max}$ when $a > b$) .... $= \frac{P}{l}(l - b + a)$

V_x [when $a < x < (l - b)$] .... $= \frac{P}{l}(b - a)$

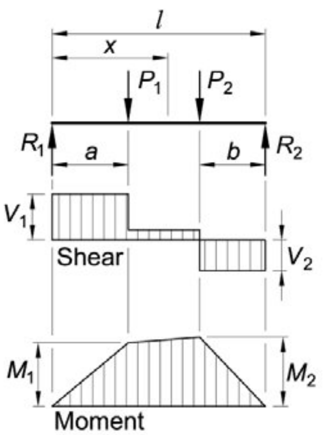
M_1 ($= M_{max}$ when $a > b$) .... $= R_1 a$

M_2 ($= M_{max}$ when $a < b$) .... $= R_2 b$

M_x (when $x < a$) .... $= R_1 x$

M_x [when $a < x < (l - b)$] .... $= R_1 x - P(x - a)$

11. SIMPLE BEAM — TWO UNEQUAL CONCENTRATED LOADS UNSYMMETRICALLY PLACED



$R_1 = V_1$ .... $= \dfrac{P_1(l - a) + P_2 b}{l}$

$R_2 = V_2$ .... $= \dfrac{P_1 a + P_2(l - b)}{l}$

V_x [when $a < x < (l - b)$] .... $= R_1 - P_1$

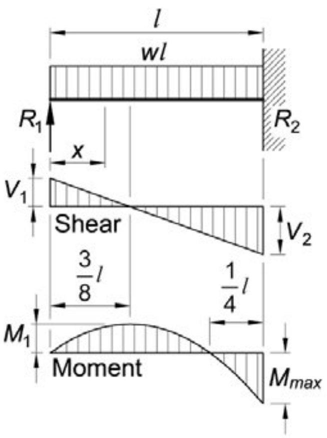
M_1 ($= M_{max}$ when $R_1 < P_1$) .... $= R_1 a$

M_2 ($= M_{max}$ when $R_2 < P_2$) .... $= R_2 b$

M_x (when $x < a$) .... $= R_1 x$

M_x [when $a < x < (l - b)$] .... $= R_1 x - P_1(x - a)$

12. BEAM FIXED AT ONE END, SUPPORTED AT OTHER — UNIFORMLY DISTRIBUTED LOAD



Total Equiv. Uniform Load .... $= wl$

$R_1 = V_1$ .... $= \frac{3wl}{8}$

$R_2 = V_2 = V_{max}$ .... $= \frac{5wl}{8}$

V_x .... $= R_1 - wx$

M_{max} .... $= \frac{wl^2}{8}$

M_1 (at $x = \frac{3}{8}l$) .... $= \frac{9}{128}wl^2$

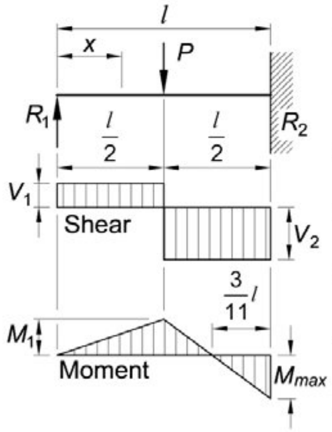
M_x .... $= R_1 x - \frac{wx^2}{2}$

Δ_{max} [at $x = \frac{l}{16}(1 + \sqrt{33}) = 0.422\, l$] .... $= \frac{wl^4}{185EI}$

Δ_x .... $= \frac{wx}{48EI}(l^3 - 3lx^2 + 2x^3)$

Table 3-23 (continued)
Shears, Moments and Deflections

13. BEAM FIXED AT ONE END, SUPPORTED AT OTHER — CONCENTRATED LOAD AT CENTER



Total Equiv. Uniform Load .... $= \frac{3P}{2}$

$R_1 = V_1$ .... $= \frac{5P}{16}$

$R_2 = V_2 = V_{max}$ .... $= \frac{11P}{16}$

M_{max} (at fixed end) .... $= \frac{3Pl}{16}$

M_1 (at point of load) .... $= \frac{5Pl}{32}$

M_x (when $x < \frac{l}{2}$) .... $= \frac{5Px}{16}$

M_x (when $x > \frac{l}{2}$) .... $= P\left(\frac{l}{2} - \frac{11x}{16}\right)$

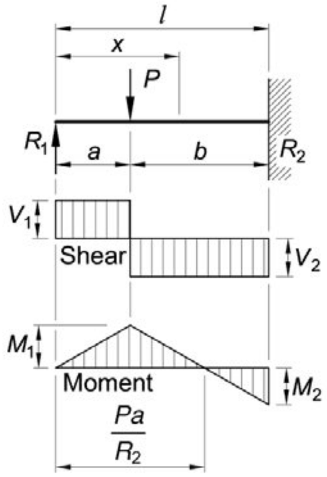
Δ_{max} (at $x = \frac{l}{\sqrt{5}} = 0.447\,l$) .... $= \frac{Pl^3}{48EI\sqrt{5}} = 0.00932\frac{Pl^3}{EI}$

Δ_x (at point of load) .... $= \frac{7Pl^3}{768EI}$

Δ_x (when $x < \frac{l}{2}$) .... $= \frac{Px}{96EI}\left(3l^2 - 5x^2\right)$

Δ_x (when $x > \frac{l}{2}$) .... $= \frac{P}{96EI}(x-l)^2(11x - 2l)$

14. BEAM FIXED AT ONE END, SUPPORTED AT THE OTHER — CONCENTRATED LOAD AT ANY POINT



$R_1 = V_1$ .... $= \frac{Pb^2}{2l^3}(a + 2l)$

$R_2 = V_2$ .... $= \frac{Pa}{2l^3}\left(3l^2 - a^2\right)$

M_1 (at point of load) .... $= R_1 a$

M_2 (at fixed end) .... $= \frac{Pab}{2l^2}(a + l)$

M_x (when $x < a$) .... $= R_1 x$

M_x (when $x > a$) .... $= R_1 x - P(x - a)$

Δ_{max} $\left[\text{when } a < 0.414l \text{ at } x = l\frac{\left(l^2 + a^2\right)}{\left(3l^2 - a^2\right)}\right]$ .. $= \frac{Pa}{3EI}\frac{\left(l^2 - a^2\right)^3}{\left(3l^2 - a^2\right)^2}$

Δ_{max} $\left(\text{when } a > 0.414l \text{ at } x = l\sqrt{\frac{a}{2l + a}}\right)$ ..... $= \frac{Pab^2}{6EI}\sqrt{\frac{a}{2l + a}}$

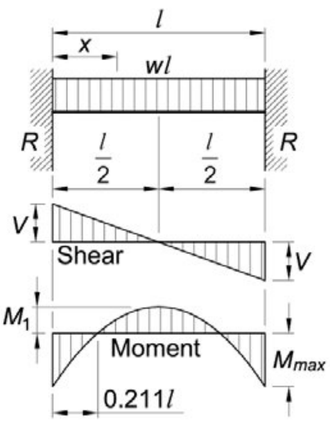
Δ_a (at point of load) .... $= \frac{Pa^2b^3}{12EIl^3}(3l + a)$

Δ_x (when $x < a$) .... $= \frac{Pb^2x}{12EIl^3}\left(3al^2 - 2lx^2 - ax^2\right)$

Δ_x (when $x > a$) .... $= \frac{Pa}{12EIl^3}(l - x)^2\left(3l^2x - a^2x - 2a^2l\right)$

Table 3-23 (continued)
Shears, Moments and Deflections

15. BEAM FIXED AT BOTH ENDS — UNIFORMLY DISTRIBUTED LOADS



Total Equiv. Uniform Load .... $= \frac{2wl}{3}$

$R = V$ .... $= \frac{wl}{2}$

V_x .... $= w\left(\frac{l}{2} - x\right)$

M_{max} (at ends) .... $= \frac{wl^2}{12}$

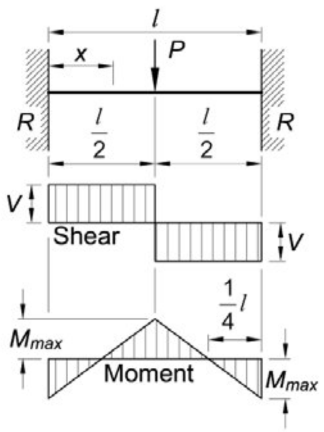
M_1 (at center) .... $= \frac{wl^2}{24}$

M_x .... $= \frac{w}{12}\left(6lx - l^2 - 6x^2\right)$

Δ_{max} (at center) .... $= \frac{wl^4}{384EI}$

Δ_x .... $= \frac{wx^2}{24EI}(l - x)^2$

16. BEAM FIXED AT BOTH ENDS — CONCENTRATED LOAD AT CENTER



Total Equiv. Uniform Load .... $= P$

$R = V$ .... $= \frac{P}{2}$

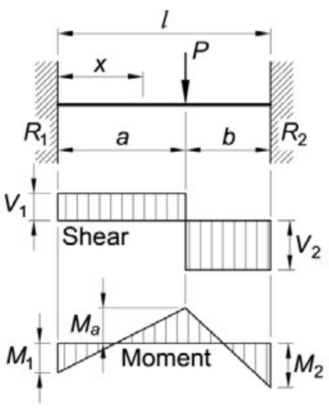
M_{max} (at center and ends) .... $= \frac{Pl}{8}$

M_x (when $x < \frac{l}{2}$) .... $= \frac{P}{8}(4x - l)$

Δ_{max} (at center) .... $= \frac{Pl^3}{192EI}$

Δ_x (when $x < \frac{l}{2}$) .... $= \frac{Px^2}{48EI}(3l - 4x)$

17. BEAM FIXED AT BOTH ENDS — CONCENTRATED LOAD AT ANY POINT



$R_1 = V_1 (= V_{max}$ when $a < b)$ .... $= \frac{Pb^2}{l^3}(3a + b)$

$R_2 = V_2 (= V_{max}$ when $a > b)$ .... $= \frac{Pa^2}{l^3}(a + 3b)$

M_1 $(= M_{max}$ when $a < b)$ .... $= \frac{Pab^2}{l^2}$

M_2 $(= M_{max}$ when $a > b)$ .... $= \frac{Pa^2b}{l^2}$

M_a (at point of load) .... $= \frac{2Pa^2b^2}{l^3}$

M_x (when $x < a$) .... $= R_1 x - \frac{Pab^2}{l^2}$

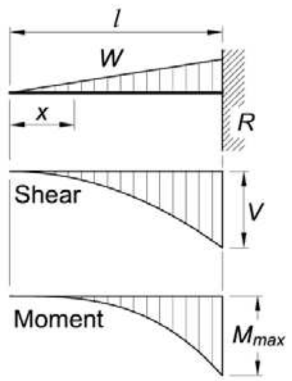
Δ_{max} (when $a > b$ at $x = \frac{2al}{3a + b}$) .... $= \frac{2Pa^3b^2}{3EI(3a + b)^2}$

Δ_a (at point of load) .... $= \frac{Pa^3b^3}{3EIl^3}$

Δ_x (when $x < a$) .... $= \frac{Pb^2x^2}{6EIl^3}(3al - 3ax - bx)$

Table 3-23 (continued)
Shears, Moments and Deflections

18. CANTILEVERED BEAM — LOAD INCREASING UNIFORMLY TO FIXED END



Total Equiv. Uniform Load .... $= \frac{8}{3}W$

$R = V$ .... $= W$

V_x .... $= W\frac{x^2}{l^2}$

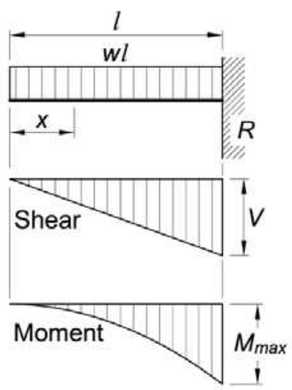
M_{max} (at fixed end) .... $= \frac{Wl}{3}$

M_x .... $= \frac{Wx^3}{3l^2}$

Δ_{max} (at free end) .... $= \frac{Wl^3}{15EI}$

Δ_x .... $= \frac{W}{60EIl^2}\left(x^5 - 5l^4x + 4l^5\right)$

19. CANTILEVERED BEAM — UNIFORMLY DISTRIBUTED LOAD



Total Equiv. Uniform Load .... $= 4wl$

$R = V$ .... $= wl$

V_x .... $= wx$

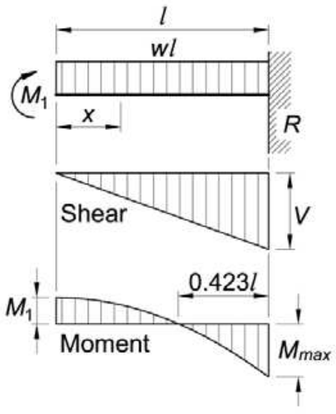
M_{max} (at fixed end) .... $= \frac{wl^2}{2}$

M_x .... $= \frac{wx^2}{2}$

Δ_{max} (at free end) .... $= \frac{wl^4}{8EI}$

Δ_x .... $= \frac{w}{24EI}\left(x^4 - 4l^3x + 3l^4\right)$

20. BEAM FIXED AT ONE END, FREE TO DEFLECT VERTICALLY BUT NOT ROTATE AT OTHER — UNIFORMLY DISTRIBUTED LOAD



Total Equiv. Uniform Load .... $= \frac{8}{3}wl$

$R = V$ .... $= wl$

V_x .... $= wx$

M_1 (at deflected end) .... $= \frac{wl^2}{6}$

M_{max} (at fixed end) .... $= \frac{wl^2}{3}$

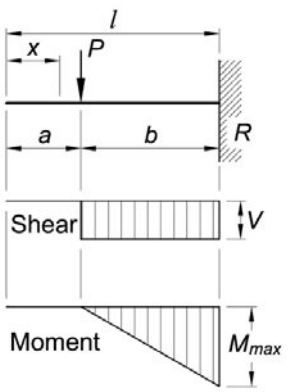
M_x .... $= \frac{w}{6}\left(l^2 - 3x^2\right)$

Δ_{max} (at deflected end) .... $= \frac{wl^4}{24EI}$

Δ_x .... $= \frac{w\left(l^2 - x^2\right)^2}{24EI}$

Table 3-23 (continued)
Shears, Moments and Deflections

21. CANTILEVERED BEAM — CONCENTRATED LOAD AT ANY POINT



Total Equiv. Uniform Load .... $= \dfrac{8Pb}{l}$

$R = V$ .... $= P$

M_{max} (at fixed end) .... $= Pb$

M_x (when $x > a$) .... $= P(x-a)$

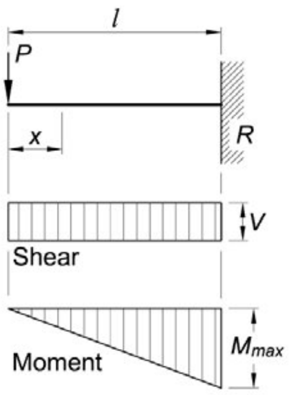
Δ_{max} (at free end) .... $= \dfrac{Pb^2}{6EI}(3l-b)$

Δ_a (at point of load) .... $= \dfrac{Pb^3}{3EI}$

Δ_x (when $x < a$) .... $= \dfrac{Pb^2}{6EI}(3l-3x-b)$

Δ_x (when $x > a$) .... $= \dfrac{P(l-x)^2}{6EI}(3b-l+x)$

22. CANTILEVERED BEAM — CONCENTRATED LOAD AT FREE END



Total Equiv. Uniform Load .... $= 8P$

$R = V$ .... $= P$

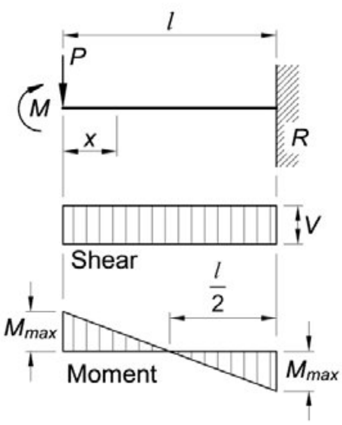
M_{max} (at fixed end) .... $= Pl$

M_x .... $= Px$

Δ_{max} (at free end) .... $= \dfrac{Pl^3}{3EI}$

Δ_x .... $= \dfrac{P}{6EI}\left(2l^3 - 3l^2x + x^3\right)$

23. BEAM FIXED AT ONE END, FREE TO DEFLECT VERTICALLY BUT NOT ROTATE AT OTHER — CONCENTRATED LOAD AT DEFLECTED END



Total Equiv. Uniform Load .... $= 4P$

$R=V$ .... $= P$

M_{max} (at both ends) .... $= \dfrac{Pl}{2}$

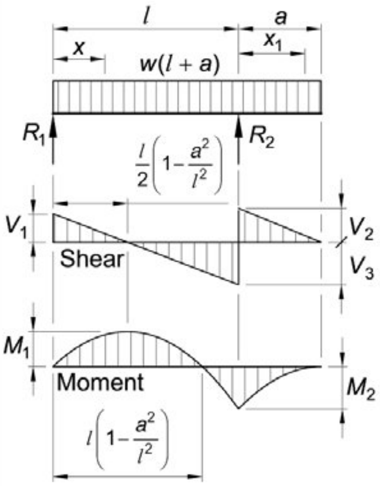
M_x .... $= P\left(\dfrac{l}{2} - x\right)$

Δ_{max} (at deflected end) .... $= \dfrac{Pl^3}{12EI}$

Δ_x .... $= \dfrac{P(l-x)^2}{12EI}(l+2x)$

Table 3-23 (continued)
Shears, Moments and Deflections

24. BEAM OVERHANGING ONE SUPPORT — UNIFORMLY DISTRIBUTED LOAD



$R_1 = V_1$ .... $= \frac{w}{2l}\left(l^2 - a^2\right)$

$R_2 = V_2 + V_3$ .... $= \frac{w}{2l}(l + a)^2$

V_2 .... $= wa$

V_3 .... $= \frac{w}{2l}\left(l^2 + a^2\right)$

V_x (between supports) .... $= R_1 - wx$

V_{x_1} (for overhang) .... $= w(a - x_1)$

M_1 $\left[\text{at } x = \frac{l}{2}\left(1 - \frac{a^2}{l^2}\right)\right]$ .... $= \frac{w}{8l^2}(l + a)^2 (l - a)^2$

M_2 (at R_2) .... $= \frac{wa^2}{2}$

M_x (between supports) .... $= \frac{wx}{2l}\left(l^2 - a^2 - xl\right)$

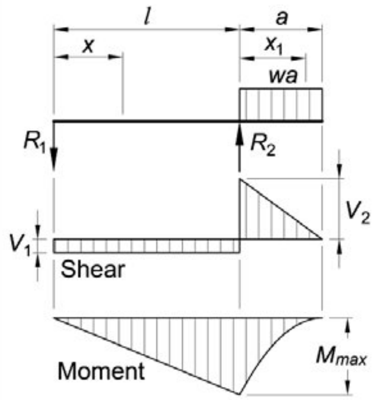
M_{x_1} (for overhang) .... $= \frac{w}{2}(a - x_1)^2$

Δ_x (between supports) .... $= \frac{wx}{24EIl}\left(l^4 - 2l^2x^2 + lx^3 - 2a^2l^2 + 2a^2x^2\right)$

Δ_{x_1} (for overhang) .... $= \frac{wx_1}{24EI}\left(4a^2l - l^3 + 6a^2x_1 - 4ax_1^2 + x_1^3\right)$

NOTE: For a negative value of Δ_x, deflection is upward.

25. BEAM OVERHANGING ONE SUPPORT — UNIFORMLY DISTRIBUTED LOAD ON OVERHANG



$R_1 = V_1$ .... $= \frac{wa^2}{2l}$

$R_2 = V_1 + V_2$ .... $= \frac{wa}{2l}(2l + a)$

V_2 .... $= wa$

V_{x_1} (for overhang) .... $= w(a - x_1)$

M_{max} (at R_2) .... $= \frac{wa^2}{2}$

M_x (between supports) .... $= \frac{wa^2x}{2l}$

M_{x_1} (for overhang) .... $= \frac{w}{2}(a - x_1)^2$

Δ_{max} $\left(\text{between supports at } x = \frac{l}{\sqrt{3}}\right) = \frac{wa^2l^2}{18\sqrt{3}EI} = 0.0321\frac{wa^2l^2}{EI}$

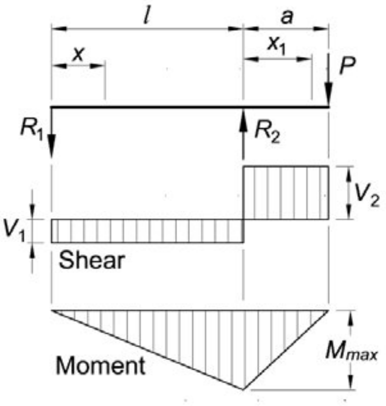
Δ_{max} (for overhang at $x_1 = a$)..... $= \frac{wa^3}{24EI}(4l + 3a)$

Δ_x (between supports) .... $= \frac{wa^2x}{12EIl}\left(l^2 - x^2\right)$

Δ_{x_1} (for overhang) .... $= \frac{wx_1}{24EI}\left(4a^2l + 6a^2x_1 - 4ax_1^2 + x_1^3\right)$

Table 3-23 (continued)
Shears, Moments and Deflections

26. BEAM OVERHANGING ONE SUPPORT — CONCENTRATED LOAD AT END OF OVERHANG



$R_1 = V_1$ .... $= \dfrac{Pa}{l}$

$R_2 = V_1 + V_2$ .... $= \dfrac{P}{l}(l+a)$

V_2 .... $= P$

M_{max} (at R_2) .... $= Pa$

M_x (between supports) .... $= \dfrac{Pax}{l}$

M_{x_1} (for overhang) .... $= P(a - x_1)$

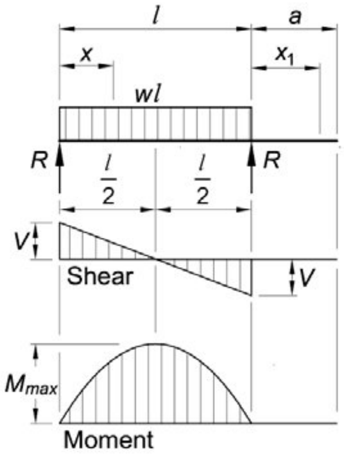
Δ_{max} $\left(\text{between supports at } x = \dfrac{l}{\sqrt{3}}\right)$ .... $= \dfrac{Pal^2}{9\sqrt{3}EI} = 0.0642\dfrac{Pal^2}{EI}$

Δ_{max} (for overhang at $x_1 = a$) .... $= \dfrac{Pa^2}{3EI}(l+a)$

Δ_x (between supports) .... $= \dfrac{Pax}{6EIl}\left(l^2 - x^2\right)$

Δ_{x_1} (for overhang) .... $= \dfrac{Px_1}{6EI}\left(2al + 3ax_1 - x_1^2\right)$

27. BEAM OVERHANGING ONE SUPPORT — UNIFORMLY DISTRIBUTED LOAD BETWEEN SUPPORTS



Total Equiv. Uniform Load .... $= wl$

$R = V$ .... $= \dfrac{wl}{2}$

V_x .... $= w\left(\dfrac{l}{2} - x\right)$

M_{max} (at center) .... $= \dfrac{wl^2}{8}$

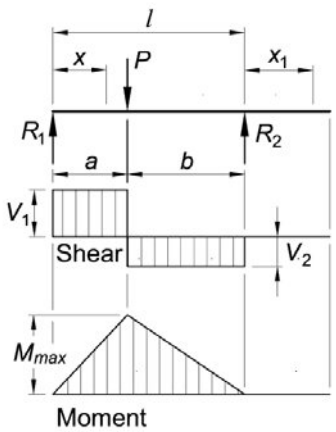
M_x .... $= \dfrac{wx}{2}(l - x)$

Δ_{max} (at center) .... $= \dfrac{5wl^4}{384EI}$

Δ_x .... $= \dfrac{wx}{24EI}\left(l^3 - 2lx^2 + x^3\right)$

Δ_{x_1} .... $= \dfrac{wl^3x_1}{24EI}$

28. BEAM OVERHANGING ONE SUPPORT — CONCENTRATED LOAD AT ANY POINT BETWEEN SUPPORTS



Total Equiv. Uniform Load .... $= \dfrac{8Pab}{l^2}$

$R_1 = V_1$ (= V_{max} when $a < b$) .... $= \dfrac{Pb}{l}$

$R_2 = V_2$ (= V_{max} when $a > b$) .... $= \dfrac{Pa}{l}$

M_{max} (at point of load) .... $= \dfrac{Pab}{l}$

M_x (when $x < a$) .... $= \dfrac{Pbx}{l}$

Δ_{max} $\left[\text{at } x = \sqrt{\dfrac{a(a+2b)}{3}} \text{ when } a > b\right]$ .... $= \dfrac{Pab(a+2b)\sqrt{3a(a+2b)}}{27EIl}$

Δ_a (at point of load) .... $= \dfrac{Pa^2b^2}{3EIl}$

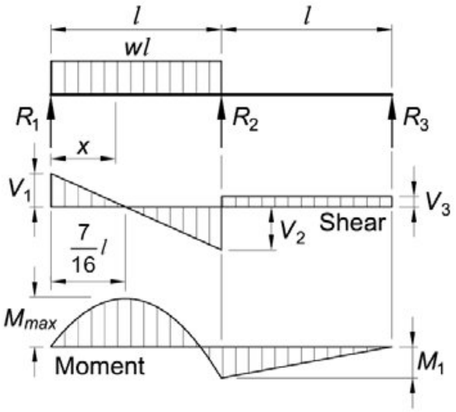
Δ_x (when $x < a$) .... $= \dfrac{Pbx}{6EIl}\left(l^2 - b^2 - x^2\right)$

Δ_x (when $x > a$) .... $= \dfrac{Pa(l-x)}{6EIl}\left(2lx - x^2 - a^2\right)$

Δ_{x_1} .... $= \dfrac{Pabx_1}{6EIl}(l+a)$

Table 3-23 (continued)
Shears, Moments and Deflections

29. CONTINUOUS BEAM — TWO EQUAL SPANS — UNIFORM LOAD ON ONE SPAN



Total Equiv. Uniform Load .... $= \frac{49}{64} wl$

$R_1 = V_1$ .... $= \frac{7}{16} wl$

$R_2 = V_2 + V_3$ .... $= \frac{5}{8} wl$

$R_3 = V_3$ .... $= \frac{1}{16} wl$

V_2 .... $= \frac{9}{16} wl$

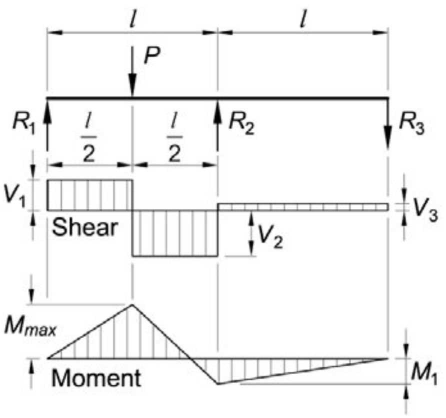
M_{max} (at $x = \frac{7}{16} l$) .... $= \frac{49}{512} wl^2$

M_1 (at support R_2) .... $= \frac{1}{16} wl^2$

M_x (when $x < l$) .... $= \frac{wx}{16}(7l - 8x)$

Δ_{max} (at 0.472 l from R_1) .... $= \frac{0.0092\, wl^4}{EI}$

30. CONTINUOUS BEAM — TWO EQUAL SPANS — CONCENTRATED LOAD AT CENTER OF ONE SPAN



Total Equiv. Uniform Load .... $= \frac{13}{8} P$

$R_1 = V_1$ .... $= \frac{13}{32} P$

$R_2 = V_2 + V_3$ .... $= \frac{11}{16} P$

$R_3 = V_3$ .... $= \frac{3}{32} P$

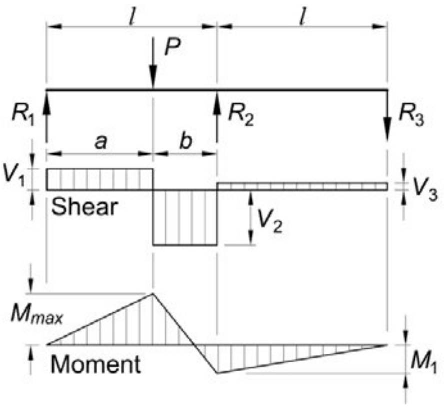
V_2 .... $= \frac{19}{32} P$

M_{max} (at point of load) .... $= \frac{13}{64} Pl$

M_1 (at support R_2) .... $= \frac{3}{32} Pl$

Δ_{max} (at 0.480 l from R_1) .... $= \frac{0.015\, Pl^3}{EI}$

31. CONTINUOUS BEAM — TWO EQUAL SPANS — CONCENTRATED LOAD AT ANY POINT



$R_1 = V_1$ .... $= \frac{Pb}{4l^3}\left[4l^2 - a(l + a)\right]$

$R_2 = V_2 + V_3$ .... $= \frac{Pa}{2l^3}\left[2l^2 + b(l + a)\right]$

$R_3 = V_3$ .... $= \frac{Pab}{4l^3}(l + a)$

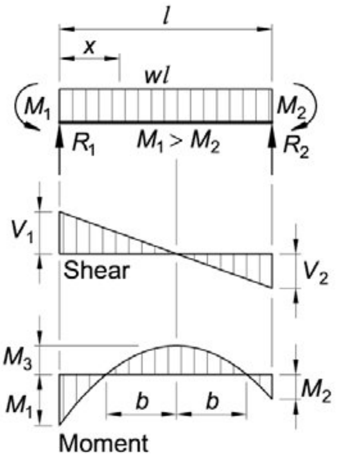
V_2 .... $= \frac{Pa}{4l^3}\left[4l^2 + b(l + a)\right]$

M_{max} (at point of load) .... $= \frac{Pab}{4l^3}\left[4l^2 - a(l + a)\right]$

M_1 (at support R_2) .... $= \frac{Pab}{4l^2}(l + a)$

Table 3-23 (continued)
Shears, Moments and Deflections

32. BEAM — UNIFORMLY DISTRIBUTED LOAD AND VARIABLE END MOMENTS



$R_1 = V_1$ .... $= \dfrac{wl}{2} + \dfrac{M_1 - M_2}{l}$

$R_2 = V_2$ .... $= \dfrac{wl}{2} - \dfrac{M_1 - M_2}{l}$

V_x .... $= w\left(\dfrac{l}{2} - x\right) + \dfrac{M_1 - M_2}{l}$

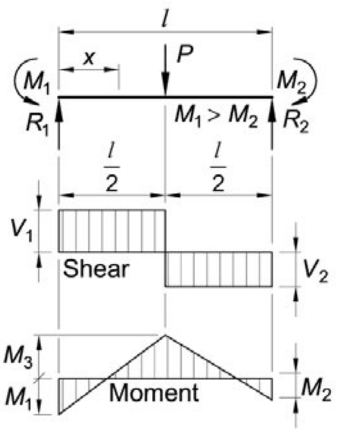
M_3 (at $x = \dfrac{l}{2} + \dfrac{M_1 - M_2}{wl}$) .... $= \dfrac{wl^2}{8} - \dfrac{M_1 + M_2}{2} + \dfrac{(M_1 - M_2)^2}{2wl^2}$

M_x .... $= \dfrac{wx}{2}(l - x) + \left(\dfrac{M_1 - M_2}{l}\right)x - M_1$

b (to locate inflection points) .... $= \sqrt{\dfrac{l^2}{4} - \left(\dfrac{M_1 + M_2}{w}\right) + \left(\dfrac{M_1 - M_2}{wl}\right)^2}$

Δ_x .... $= \dfrac{wx}{24EI}\left[x^3 - \left(2l + \dfrac{4M_1}{wl} - \dfrac{4M_2}{wl}\right)x^2 + \dfrac{12M_1}{w}x + l^3 - \dfrac{8M_1 l}{w} - \dfrac{4M_2 l}{w}\right]$

33. BEAM — CONCENTRATED LOAD AT CENTER AND VARIABLE END MOMENTS



$R_1 = V_1$ .... $= \dfrac{P}{2} + \dfrac{M_1 - M_2}{l}$

$R_2 = V_2$ .... $= \dfrac{P}{2} - \dfrac{M_1 - M_2}{l}$

M_3 (at center) .... $= \dfrac{Pl}{4} - \dfrac{M_1 - M_2}{2}$

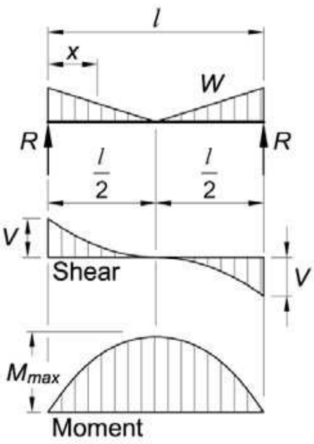
M_x (when $x < \dfrac{l}{2}$) .... $= \left(\dfrac{P}{2} + \dfrac{M_1 - M_2}{l}\right)x - M_1$

M_x (when $x > \dfrac{l}{2}$) .... $= \dfrac{P}{2}(l - x) + \dfrac{(M_1 - M_2)x}{l} - M_1$

Δ_x (when $x < \dfrac{l}{2}$) $= \dfrac{Px}{48EI}\left(3l^2 - 4x^2 - \dfrac{8(l - x)}{Pl}\left[M_1(2l - x) + M_2(l + x)\right]\right)$

Table 3-23 (continued)
Shears, Moments and Deflections

34. SIMPLE BEAM — LOAD INCREASING UNIFORMLY FROM CENTER



Total Equiv. Uniform Load .... $= \frac{2W}{3}$

$R = V$ .... $= \frac{W}{2}$

V_x (when $x < \frac{l}{2}$)........................ $= \frac{W}{2}\left(\frac{l-2x}{l}\right)^2$

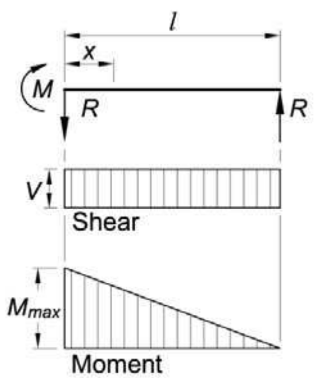
M_{max} (at center)............................ $= \frac{Wl}{12}$

M_x (when $x < \frac{l}{2}$)........................ $= \frac{W}{2}\left(x - \frac{2x^2}{l} + \frac{4x^3}{3l^2}\right)$

Δ_{max} (at center)............................ $= \frac{3Wl^3}{320EI}$

Δ_x (when $x < \frac{l}{2}$)........................ $= \frac{W}{12EI}\left(x^3 - \frac{x^4}{l} + \frac{2x^5}{5l^2} - \frac{3l^2x}{8}\right)$

35. SIMPLE BEAM — CONCENTRATED MOMENT AT END



Total Equiv. Uniform Load .... $= \frac{8M}{l}$

$R = V$ .... $= \frac{M}{l}$

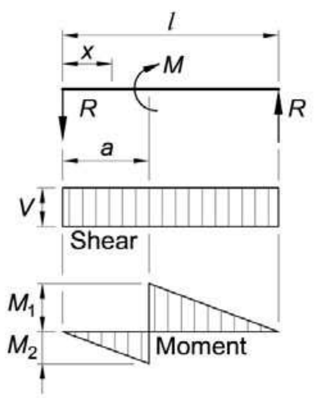
M_{max} .... $= M$

M_x .... $= M\left(1 - \frac{x}{l}\right)$

Δ_{max} (at $x = 0.423\ l$)...................... $= 0.0642\,\frac{Ml^2}{EI}$

Δ_x .... $= \frac{M}{6EI}\left(3x^2 - \frac{x^3}{l} - 2lx\right)$

36. SIMPLE BEAM — CONCENTRATED MOMENT AT ANY POINT



Total Equiv. Uniform Load .... $= \frac{8M}{l}$

$R = V$ .... $= \frac{M}{l}$

M_x (when $x < a$)........................ $= Rx$

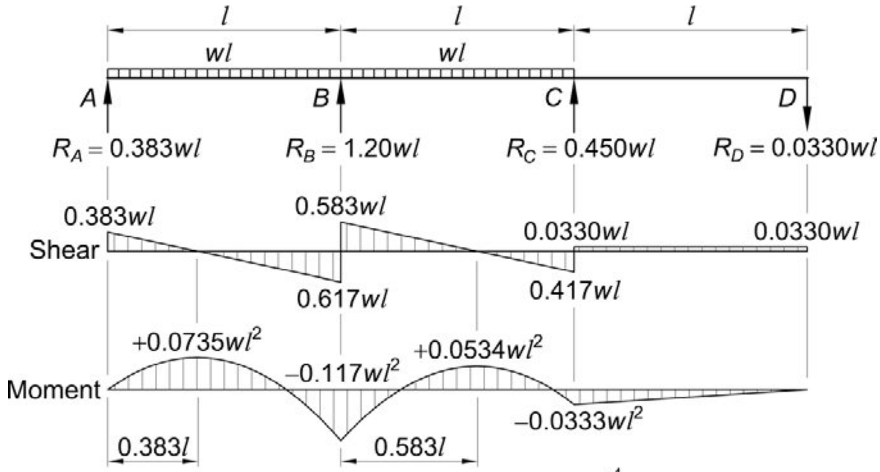
M_x (when $x > a$)........................ $= R\,(l - x)$

Δ_x (when $x < a$)........................ $= \frac{M}{6EI}\left[\left(6a - \frac{3a^2}{l} - 2l\right)x - \frac{x^3}{l}\right]$

Δ_x (when $x > a$)........................ $= \frac{M}{6EI}\left[3\left(a^2 + x^2\right) - \frac{x^3}{l} - \left(2l + \frac{3a^2}{l}\right)x\right]$

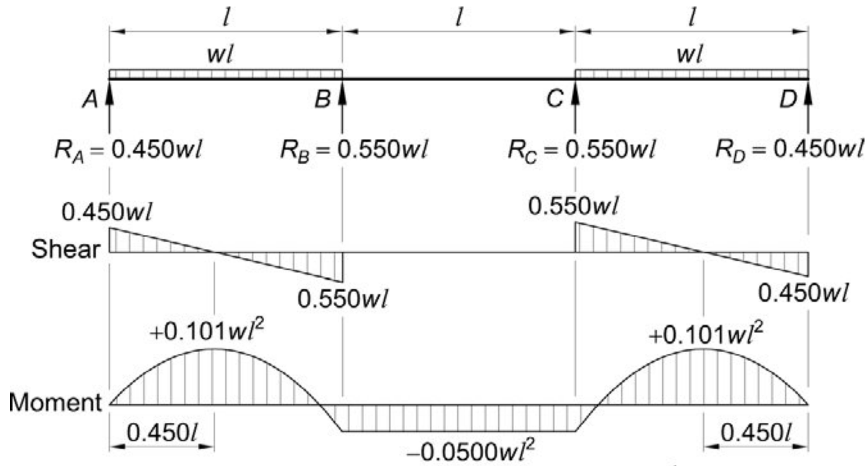
Table 3-23 (continued)
Shears, Moments and Deflections

37. CONTINUOUS BEAM — THREE EQUAL SPANS — ONE END SPAN UNLOADED



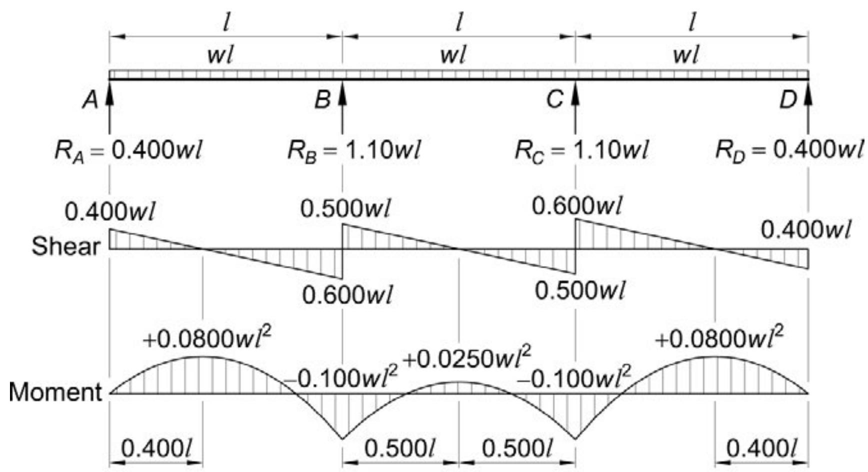
$$\Delta_{max}\left(0.430l \text{ from } A\right) = 0.0059\frac{wl^4}{EI}$$

38. CONTINUOUS BEAM — THREE EQUAL SPANS — END SPANS LOADED



$$\Delta_{max}\left(0.479l \text{ from } A \text{ or } D\right) = 0.0099\frac{wl^4}{EI}$$

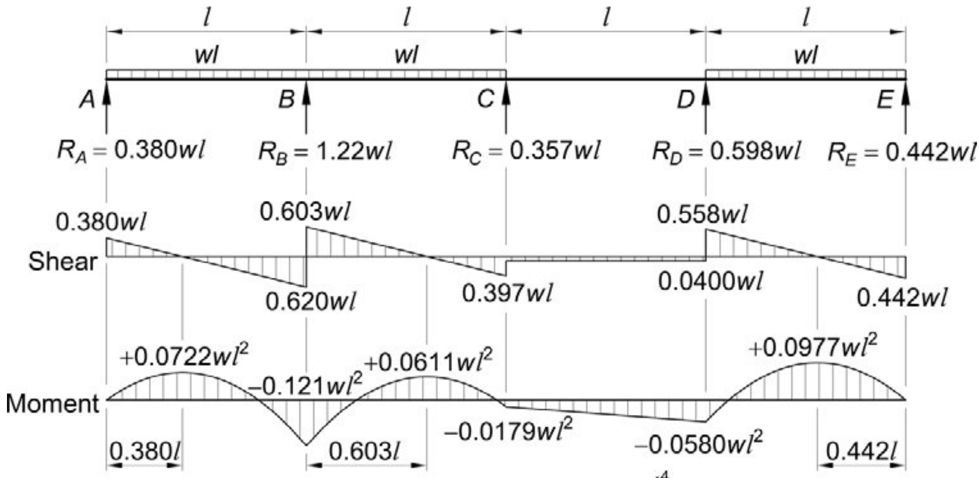
39. CONTINUOUS BEAM — THREE EQUAL SPANS — ALL SPANS LOADED



$$\Delta_{max}\left(0.446l \text{ from } A \text{ or } D\right) = 0.0069\frac{wl^4}{EI}$$

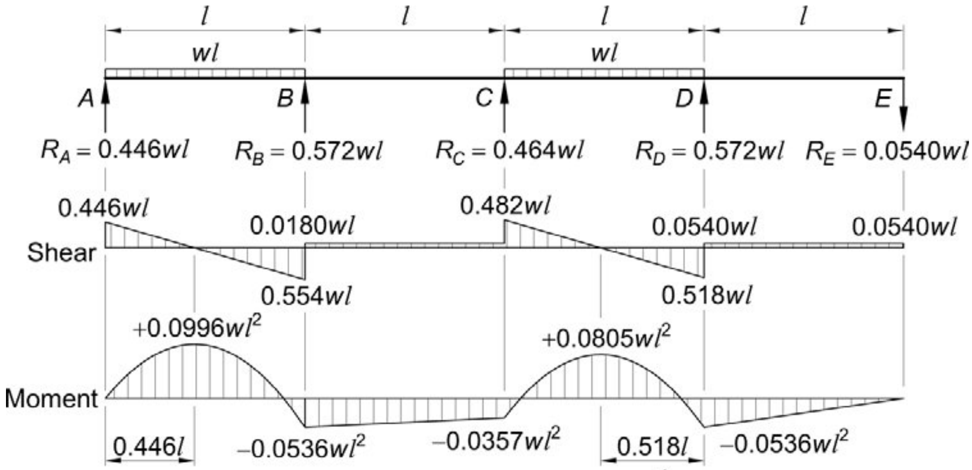
Table 3-23 (continued)
Shears, Moments and Deflections

40. CONTINUOUS BEAM — FOUR EQUAL SPANS — THIRD SPAN UNLOADED



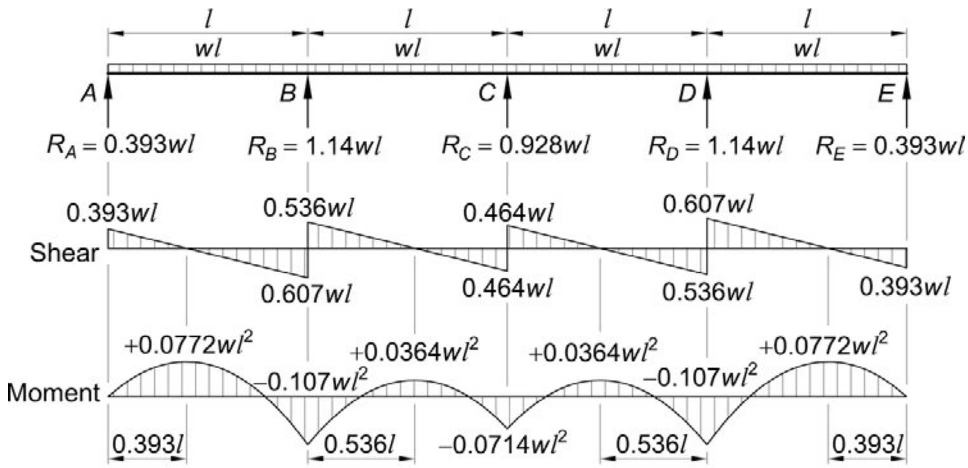
$\Delta_{max}\left(0.475l \text{ from } E\right) = 0.0094\dfrac{wl^4}{EI}$

41. CONTINUOUS BEAM — FOUR EQUAL SPANS — FIRST AND THIRD SPANS LOADED



$\Delta_{max}\left(0.477l \text{ from } A\right) = 0.0097\dfrac{wl^4}{EI}$

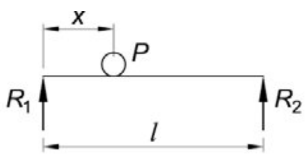
42. CONTINUOUS BEAM — FOUR EQUAL SPANS — ALL SPANS LOADED



$\Delta_{max}\left(0.440l \text{ from } A \text{ or } E\right) = 0.0065\dfrac{wl^4}{EI}$

Table 3-23 (continued)
Shears, Moments and Deflections

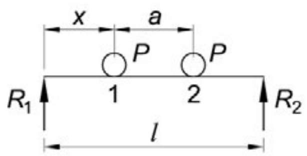
43. SIMPLE BEAM — ONE CONCENTRATED MOVING LOAD



$$R_{1\,max} = V_{1\,max} (\text{at } x = 0) \ldots\ldots = P$$

$$M_{max} \left(\text{at point of load, when } x = \frac{l}{2}\right) \ldots\ldots = \frac{Pl}{4}$$

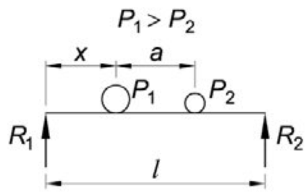
44. SIMPLE BEAM — TWO EQUAL CONCENTRATED MOVING LOADS



$$R_{1\,max} = V_{1\,max} (\text{at } x = 0) \ldots\ldots = P\left(2 - \frac{a}{l}\right)$$

$$M_{max} \begin{cases} \left[\begin{array}{l}\text{when } a < (2-\sqrt{2})l = 0.586l \\ \text{under load 1 at } x = \dfrac{1}{2}\left(l - \dfrac{a}{2}\right)\end{array}\right] \ldots\ldots = \dfrac{P}{2l}\left(l - \dfrac{a}{2}\right)^2 \\ \left[\begin{array}{l}\text{when } a > (2-\sqrt{2})l = 0.586l \\ \text{with one load at center of span (Case 43)}\end{array}\right] \ldots\ldots = \dfrac{Pl}{4} \end{cases}$$

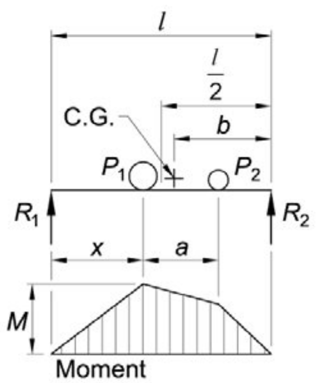
45. SIMPLE BEAM — TWO UNEQUAL CONCENTRATED MOVING LOADS



$$R_{1\,max} = V_{1\,max} (\text{at } x = 0) \ldots\ldots = P_1 + P_2 \frac{l-a}{l}$$

$$M_{max} \begin{cases} \left[\text{under } P_1 \text{, at } x = \dfrac{1}{2}\left(l - \dfrac{P_2 a}{P_1 + P_2}\right)\right] \ldots\ldots = (P_1 + P_2)\dfrac{x^2}{l} \\ \left[\begin{array}{l} M_{max} \text{ may occur with larger} \\ \text{load at center of span and other} \\ \text{load off span (Case 43)}\end{array}\right] \ldots\ldots = \dfrac{P_1 l}{4} \end{cases}$$

GENERAL RULES FOR SIMPLE BEAMS CARRYING MOVING CONCENTRATED LOADS



The maximum shear due to moving concentrated loads occurs at one support when one of the loads is at that support. With several moving loads, the location that will produce maximum shear must be determined by trial.

The maximum bending moment produced by moving concentrated loads occurs under one of the loads when that load is as far from one support as the center of gravity of all the moving loads on the beam is from the other support.

In the accompanying diagram, the maximum bending moment occurs under load P_1 when $x = b$. It should also be noted that this condition occurs when the center-line of the span is midway between the center of gravity of loads and the nearest concentrated load.

PART 4

DESIGN OF COMPRESSION MEMBERS

SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of members subject to axial compression. For the design of members subject to combined axial compression and flexure, see Part 6.

AVAILABLE COMPRESSIVE STRENGTH

The available strength of compression members, $\phi_c P_n$ or P_n/Ω_c, which must equal or exceed the required strength, P_u or P_a, respectively, is determined according to AISC *Specification* Chapter E.

Use of Table 6-2 for Design of Compression Members

Table 6-2 may be used for design of compression members. This table includes all W-shapes, not just those most commonly used as columns. See Part 6 for additional information on using Table 6-2 for design of compression members.

LOCAL BUCKLING

Determining the Width-to-Thickness Ratios of the Cross Section

Steel compression members are classified on the basis of the width-to-thickness ratios of the various elements of the cross section. The width-to-thickness ratio is calculated for each element of the cross section per AISC *Specification* Section B4. Limiting width-to-thickness ratios for various values of F_y of members subjected to axial compression are presented in Table 6-1a.

Determining the Slenderness of the Cross Section

When the width-to-thickness ratios of all compression elements are less than or equal to λ_r, the cross section is nonslender, and the gross area, A_g, is used to determine the nominal compressive strength, P_n; thus, there is no reduction in strength for element slenderness. When the width-to-thickness ratio of any compression element is greater than λ_r, the cross section is slender and A_e may be less than A_g. The effective area used to calculate the nominal compressive strength, P_n, is a function of λ_r and the critical stress, F_{cr}.

EFFECTIVE LENGTH AND COLUMN SLENDERNESS

Columns are designed for their slenderness, L_c/r, per AISC *Specification* Section E2. The effective length, L_c, is equal to the effective length factor, K, multiplied by L, the physical length between braced points (see AISC *Specification* Appendix 6).

When a stability analysis is performed using the direct analysis method per AISC *Specification* Chapter C, $K = 1$.

When a stability analysis is performed using the first-order analysis method in AISC *Specification* Appendix 7, Section 7.3, $K = 1$.

When a stability analysis is performed using the effective length method in AISC *Specification* Appendix 7, Section 7.2, the following applies:

$K = 1$ for columns braced at each end and whose flexural stiffnesses are not considered to contribute to lateral stability and resistance to lateral loads.

$K = 1$ for all columns when the ratio of maximum second-order drift to first-order drift in all stories is less than 1.1.

K shall be determined from a sidesway buckling analysis for all columns whose flexural stiffnesses are considered to contribute to lateral stability and resistance to lateral loads. Guidance on the proper determination of the value of K is given in AISC *Specification* Commentary to Appendix 7, Section 7.2.

As indicated in the User Note in AISC *Specification* Section E2, compression member slenderness, L_c / r, should preferably be limited to a maximum of 200. Note that this recommendation does not apply to members that are primarily tension members but subject to incidental compression under other load combinations.

Additional information is available in the SSRC *Guide to Stability Design Criteria for Metal Structures* (Ziemian, 2010).

COMPOSITE COMPRESSION MEMBERS

For the design of encased composite and filled composite compression members, see AISC *Specification* Section I2. See also AISC Design Guide 6, *Load and Resistance Factor Design of W-Shapes Encased in Concrete* (Griffis, 1992). For further information on composite design and construction, see also Viest et al. (1997).

For the design of filled composite compression members, see AISC *Specification* Section I2 and the design tables provided in Part IV of the AISC *Design Examples* document found at **www.aisc.org/manualresources**.

DESIGN TABLE DISCUSSION

Steel Compression—Member Selection Tables

Tabulated values account for element slenderness effects.

Table 4-1. Available Strength in Axial Compression—W-Shapes

Available strengths in axial compression are given for W-shapes in Tables 4-1a, 4-1b and 4-1c. The tables reflect $F_y = 50$ ksi (ASTM A992 and ASTM A913 where applicable), $F_y = 65$ ksi (ASTM A913) and $F_y = 70$ ksi (ASTM A913), respectively. These tables include W-shapes that are most commonly used in axial compression, and do not reflect the complete range of sections available in the relevant F_y. Available strengths in axial compression for all W-shapes, including those not shown in Table 4-1, are presented in Table 6-2 for $F_y = 50$ ksi.

The tabulated values are given for the effective length with respect to the y-axis, L_{cy}. However, the effective length with respect to the x-axis, L_{cx}, must also be investigated. To determine the available strength in axial compression, the table should be entered at the larger of L_{cy} and $L_{cy\,eq}$, where

$$L_{cy\,eq} = \frac{L_{cx}}{\dfrac{r_x}{r_y}} \tag{4-1}$$

Where the torsional unbraced length and the flexural unbraced lengths are equal, torsional buckling generally does not control. However, where the torsional unbraced length is larger than the flexural unbraced length, AISC *Specification* Section E4 may control the design of W-shape columns. For further information, see Liu et al. (2013).

Values of the ratio r_x/r_y and other properties useful in the design of W-shape compression members are listed at the bottom of Table 4-1.

Variables P_{wo}, P_{wi}, P_{wb} and P_{fb} shown in Table 4-1 can be used to determine the strength of W-shapes without stiffeners to resist concentrated forces applied normal to the face(s) of the flange(s). In these tables it is assumed that the concentrated forces act far enough away from the member ends that end effects are not considered (end effects are addressed in Part 9). When $P_r \le \phi R_n$ or R_n/Ω, column web stiffeners are not required. Figures 4-1, 4-2 and 4-3 illustrate the limit states and the applicable variables for each.

Web Local Yielding

The variables P_{wo} and P_{wi} can be used in the calculation of the available web local yielding strength for the column as follows:

LRFD	ASD
$\phi R_n = P_{wo} + P_{wi} l_b$ (4-2a)	$R_n/\Omega = P_{wo} + P_{wi} l_b$ (4-2b)

where

R_n = $F_{yw} t_w (5k + l_b) = 5F_{yw} t_w k + F_{yw} t_w l_b$, kips (AISC *Specification* Equation J10-2)
P_{wo} = $\phi 5 F_{yw} t_w k$ for LRFD and $5F_{yw} t_w k/\Omega$ for ASD, kips
P_{wi} = $\phi F_{yw} t_w$ for LRFD and $F_{yw} t_w/\Omega$ for ASD, kip/in.
k = distance from outer face of flange to the web toe of fillet, in.
l_b = length of bearing, in.
t_w = thickness of web, in.
ϕ = 1.00
Ω = 1.50

Web Compression Buckling

The variable P_{wb} is the available web compression buckling strength for the column as follows:

LRFD	ASD
$\phi R_n = P_{wb}$ (4-3a)	$R_n/\Omega = P_{wb}$ (4-3b)

where

$$R_n = \frac{24t_w^3\sqrt{EF_{yw}}}{h} Q_f \text{ (AISC \textit{Specification} Equation J10-8)}$$

$$P_{wb} = \frac{\phi 24t_w^3\sqrt{EF_{yw}}}{h} Q_f \text{ for LRFD and } \frac{24t_w^3\sqrt{EF_{yw}}}{\Omega h} Q_f \text{ for ASD, kips}$$

Q_f = 1.0 for W-shapes
F_{yw} = specified minimum yield stress of the web, ksi
h = clear distance between flanges less the fillet or corner radius for rolled shapes, in.
ϕ = 0.90
Ω = 1.67

Flange Local Bending

The variable P_{fb} is the available flange local bending strength for the column as follows:

LRFD	ASD
$\phi R_n = P_{fb}$ (4-4a)	$R_n/\Omega = P_{fb}$ (4-4b)

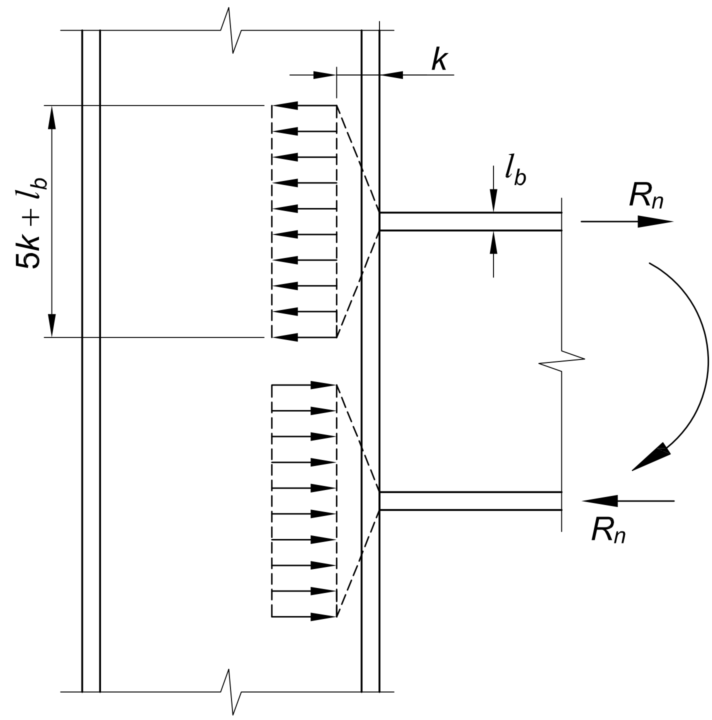


Fig. 4-1. Illustration of web local yielding limit state (AISC Specification *Section J10.2).*

where

R_n = $6.25F_{yf}t_f^2$, kips (AISC *Specification* Equation J10-1)

P_{fb} = $\phi 6.25F_{yf}t_f^2$ for LRFD and $6.25F_{yf}t_f^2/\Omega$ for ASD, kips

ϕ = 0.90

Ω = 1.67

Table 4-2. Available Strength in Axial Compression—HP-Shapes

Table 4-2 is similar to Table 4-1, except it covers HP-shapes with F_y = 50 ksi (ASTM A572 Grade 50).

Table 4-3. Available Strength in Axial Compression—Rectangular HSS

Available strengths in axial compression are given for rectangular HSS with F_y = 50 ksi (ASTM A500 Grade C). The tabulated values are given for the effective length with respect to the *y*-axis, L_{cy}. However, the effective length with respect to the *x*-axis, L_{cx}, must also be

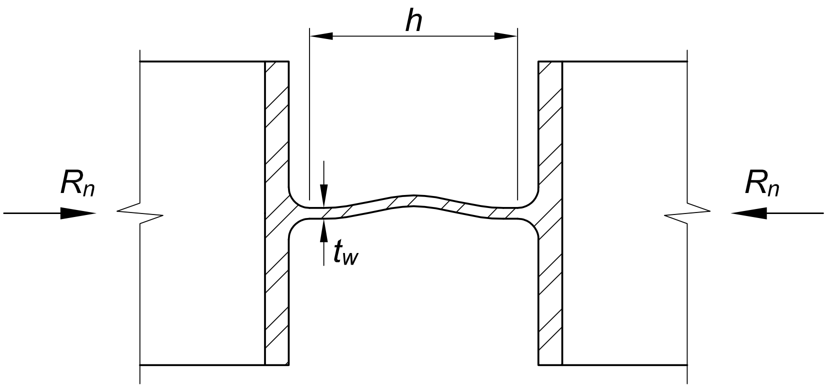


Fig. 4-2. Illustration of web compression buckling limit state (AISC Specification *Section J10.5).*

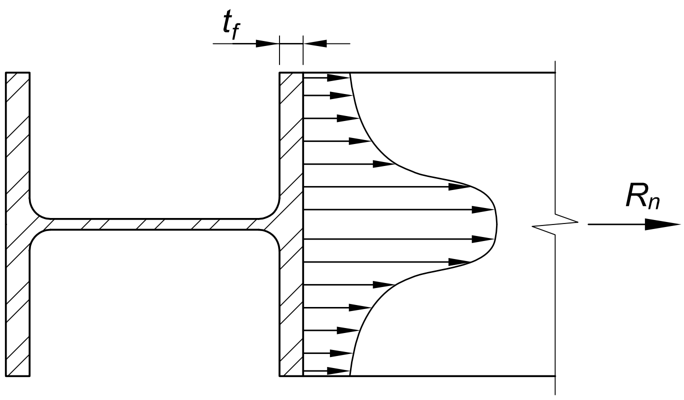


Fig. 4-3. Illustration of flange local bending limit state (AISC Specification *Section J10.1).*

investigated. To determine the available strength in axial compression, the table should be entered at the larger of L_{cy} and $L_{cy\,eq}$, where

$$L_{cy\,eq} = \frac{L_{cx}}{\dfrac{r_x}{r_y}} \tag{4-1}$$

Values of the ratio r_x/r_y and other properties useful in the design of rectangular HSS compression members are listed at the bottom of Table 4-3.

Table 4-4. Available Strength in Axial Compression—Square HSS

Table 4-4 is similar to Table 4-3, except that it covers square HSS.

Table 4-5. Available Strength in Axial Compression—Round HSS

Available strengths in axial compression are given for round HSS with $F_y = 46$ ksi (ASTM A500 Grade C). To determine the available strength in axial compression, the table should be entered at L_c. Other properties useful in the design of compression members are listed at the bottom of the available column strength tables.

Table 4-6. Available Strength in Axial Compression—Pipe

Table 4-6 is similar to Table 4-5, except it covers pipe with $F_y = 35$ ksi (ASTM A53 Grade B).

Table 4-7. Available Strength in Axial Compression—Concentrically Loaded WT-Shapes

Available strengths in axial compression, including the limit state of flexural-torsional buckling with $C_w = 0$ according to the User Note in AISC *Specification* Section E4, are given for concentrically loaded WT-shapes with $F_y = 50$ ksi (ASTM A992). Separate tabulated values are given for the effective lengths with respect to the *x*- and *y*-axes, L_{cx} and L_{cy}, respectively. For the flexural-torsional buckling effective length, use the tabulated values for the *y*-axis. Other properties useful in the design of concentrically loaded WT-shape compression members are listed at the bottom of Table 4-7.

Table 4-8. Available Strength in Axial Compression—Double Angles—Equal Legs

Available strengths in axial compression, including the limit state of flexural-torsional buckling with $C_w = 0$ according to the User Note in AISC *Specification* Section E4, are given for equal-leg double angles with $F_y = 36$ ksi (ASTM A36), assuming 3/8-in. separation between the angles for 2L2 through 2L8 members and 3/4-in. separation between the angles for 2L10 and 2L12 members. These values can be used conservatively when a larger separation is provided. Alternatively, the value of L_{cy} can be multiplied by the ratio of the tabulated r_y to r_y for the actual separation.

Separate tabulated values are given for the effective lengths with respect to the *x*- and *y*-axes, L_{cx} and L_{cy}, respectively. For the flexural-torsional buckling effective length, use the tabulated values for the *y*-axis. For buckling about the *x*-axis, the available strength is not affected by the number of intermediate connectors. However, for buckling about the y-axis, the effects of shear deformations of the intermediate connectors must be considered. The tabulated values for L_{cy} have been adjusted for the shear deformations in accordance with AISC *Specification* Equations E6-2a and E6-2b, which is applicable for intermediate shear connectors that are welded or connected by means of pretensioned bolts with Class A or B faying surfaces. The number of intermediate connectors is given in the table and the line of demarcation between the required connector values is dashed. Intermediate connectors are selected such that the available compression buckling strength about the y-axis is equal to or greater than 90% of that for compression buckling of the two angles as a unit. If fewer connectors or snug-tightened bolted intermediate connectors are used, the available strength must be recalculated per AISC *Specification* Section E6. Per AISC *Specification* Section E6.2, the slenderness of the individual components of the built-up member based upon the distance between intermediate connectors, a, must not exceed three-quarters of the controlling slenderness of the overall built-up compression member.

Other properties useful in the design of double-angle compression members are listed at the bottom of Table 4-8.

Table 4-9. Available Strength in Axial Compression—Double Angles—LLBB

Table 4-9 is the same as Table 4-8, except that it provides available strengths in axial compression for double angles with long legs back-to-back.

Table 4-10. Available Strength in Axial Compression—Double Angles—SLBB

Table 4-10 is the same as Table 4-8, except that it provides available strengths in axial compression for double angles with short legs back-to-back.

Table 4-11. Available Strength in Axial Compression—Concentrically Loaded Single Angles

Available strengths in axial compression are given for single angles, loaded through the centroid of the cross section, with $F_y = 36$ ksi (ASTM A36) based upon the effective length with respect to the z-axis, L_{cz}.

Eccentrically loaded single angles may be assumed to be loaded through the centroid when the requirements of AISC *Specification* Section E5 are met. In these cases, the eccentricity and end restraint are accounted for through a modified slenderness. Table 4-11 can then be entered using an effective length based on the modified slenderness ratio times the radius of gyration about the z-axis, r_z.

Table 4-12. Available Strength in Axial Compression—Eccentrically Loaded Single Angles

Available strengths in axial compression are given for eccentrically loaded single angles with $F_y = 36$ ksi (ASTM A36).

The long leg of the angle is assumed to be attached to a gusset plate with a thickness of $1.5t$. The tabulated values assume a load placed at the mid-width of the long leg of the angle at a distance of $0.75t$ from the face of this leg.

Effective length, L_c, is assumed to be the same on all axes (r_x, r_y, r_z and r_w). Table 4-12 considers the combined bending stresses at the heel and the tips of the angle (points A, B and C in Figure 4-4) produced by axial compression plus biaxial bending moments about the principal *w*- and *z*-axes using AISC *Specification* Equation H2-1. Points A and C are assumed at the angle mid-thickness at distances *b* and *d* (respectively) from the heel.

Note that for some sections, such as L3½×3×5/16, the calculated available strength can increase slightly as the unbraced length increases from zero, and then decrease as the unbraced length further increases.

Table 4-13. Stiffness Reduction Factor

The stiffness reduction factor, τ_b, is the ratio of the tangent modulus, E_T, to the elastic modulus, E. The equations for computing τ_b are provided in AISC *Specification* Section C2.3. Table 4-13 provides values of τ_b for materials with a specified minimum yield strength of 35 ksi, 36 ksi, 46 ksi, 50 ksi, 65 ksi and 70 ksi.

When a stability analysis is performed using the direct analysis method in AISC *Specification* Chapter C, that procedure requires consideration of residual stresses and their adverse effects on column stiffness through the use of a reduced effective stiffness of $0.80EI\tau_b$.

When a stability analysis is performed using the effective length method in AISC *Specification* Appendix 7, Section 7.2, that procedure requires determination of the effective length factor, *K*. A common method of determining *K* is through the use of alignment charts provided in the AISC *Specification* Commentary.

When column buckling occurs in the inelastic range, residual stresses will reduce the effective stiffness of columns and the alignment charts usually give conservative results. For more accurate solutions, inelastic *K*-factors can be determined from the alignment chart by using τ_b times the elastic modulus of the columns in the equation for *G* as discussed in AISC *Specification* Appendix 7 Commentary.

Table 4-14. Available Critical Stress for Compression Members

Table 4-14 provides the available critical stress for various ratios of L_c/r, for materials with a specified minimum yield strength of 35 ksi, 36 ksi, 46 ksi, 50 ksi, 65 ksi and 70 ksi.

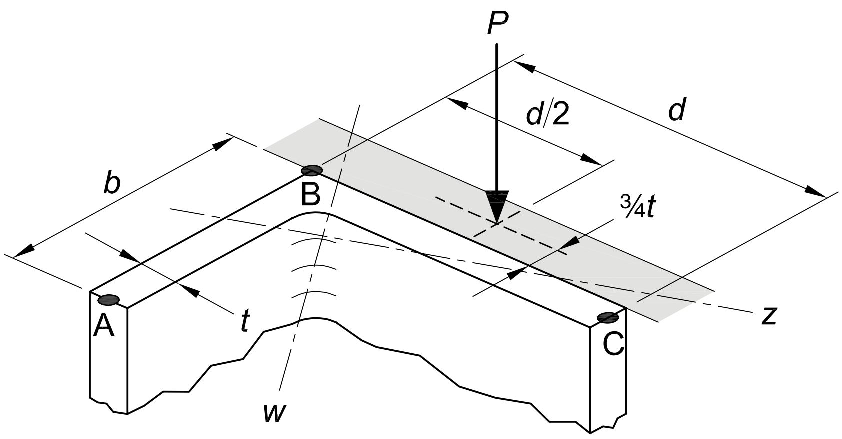


Fig. 4-4. Eccentrically loaded single angle.

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Table 4-1a
Available Strength in Axial Compression, kips
W-Shapes

W14

F_y = 50 ksi

Shape		W14×							
lb/ft		873[h]		808[h]		730[h]		665[h]	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	7690	11600	7130	10700	6440	9670	5870	8820
	11	7300	11000	6750	10100	6070	9130	5530	8310
	12	7220	10900	6680	10000	6010	9030	5470	8220
	13	7140	10700	6600	9920	5940	8920	5400	8110
	14	7060	10600	6520	9800	5860	8810	5330	8010
	15	6970	10500	6440	9680	5780	8690	5250	7890
	16	6880	10300	6350	9540	5690	8560	5170	7770
	17	6780	10200	6250	9400	5610	8430	5090	7650
	18	6680	10000	6160	9250	5510	8290	5000	7520
	19	6570	9870	6050	9100	5420	8140	4910	7380
	20	6460	9700	5950	8940	5320	7990	4820	7240
	22	6220	9350	5730	8610	5110	7670	4620	6950
	24	5980	8980	5490	8260	4890	7340	4420	6640
	26	5720	8600	5250	7890	4660	7000	4200	6320
	28	5460	8200	5000	7520	4420	6650	3990	5990
	30	5190	7790	4750	7130	4180	6290	3760	5660
	32	4910	7380	4490	6750	3940	5930	3540	5320
	34	4630	6970	4230	6360	3700	5560	3320	4990
	36	4360	6550	3970	5970	3460	5200	3100	4650
	38	4080	6140	3710	5580	3220	4850	2880	4330
	40	3810	5730	3460	5200	2990	4500	2670	4010
	42	3550	5340	3210	4830	2770	4160	2460	3690
	44	3290	4950	2970	4470	2550	3830	2260	3390
	46	3040	4570	2740	4120	2330	3510	2060	3100
	48	2800	4200	2520	3780	2140	3220	1900	2850
	50	2580	3870	2320	3480	1970	2970	1750	2630
Properties									
P_{wo}, kips		4010	6010	3560	5340	2820	4230	2410	3620
P_{wi}, kip/in.		131	197	125	187	102	154	94.3	142
P_{wb}, kips		93000	140000	79600	120000	44000	66100	34400	51700
P_{fb}, kips		5680	8540	4910	7370	4510	6780	3820	5750
L_p, ft		17.3		17.1		16.6		16.3	
L_r, ft		329		309		275		253	
A_g, in.^{2}		257		238		215		196	
I_x, in.^{4}		18100		15900		14300		12400	
I_y, in.^{4}		6170		5550		4720		4170	
r_y, in.		4.90		4.83		4.69		4.62	
r_x/r_y		1.71		1.69		1.74		1.73	
$P_{ex}L_c^2/10^4$, k-in.^{2}		518000		455000		409000		355000	
$P_{ey}L_c^2/10^4$, k-in.^{2}		177000		159000		135000		119000	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

F_y = 50 ksi

Table 4-1a (continued)
Available Strength in Axial Compression, kips
W-Shapes

W14

Shape		W14×							
lb/ft		605[h]		550[h]		500[h]		455[h]	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	5330	8010	4850	7290	4400	6610	4010	6030
	11	5010	7530	4550	6840	4120	6200	3750	5640
	12	4950	7440	4500	6760	4070	6120	3710	5570
	13	4890	7350	4440	6670	4020	6040	3660	5500
	14	4820	7250	4380	6580	3960	5950	3600	5420
	15	4750	7140	4310	6480	3900	5860	3550	5330
	16	4680	7030	4240	6380	3840	5770	3490	5240
	17	4600	6920	4170	6270	3770	5660	3420	5150
	18	4520	6790	4100	6160	3700	5560	3360	5050
	19	4440	6670	4020	6040	3630	5450	3290	4950
	20	4350	6540	3940	5920	3550	5340	3220	4840
	22	4170	6260	3770	5660	3390	5100	3080	4620
	24	3980	5980	3590	5400	3230	4860	2920	4400
	26	3780	5680	3410	5120	3060	4600	2770	4160
	28	3580	5380	3220	4840	2890	4340	2610	3920
	30	3370	5070	3030	4560	2720	4080	2450	3680
	32	3170	4760	2840	4270	2540	3820	2290	3440
	34	2960	4450	2650	3990	2370	3560	2130	3200
	36	2760	4140	2460	3700	2200	3300	1970	2960
	38	2560	3840	2280	3430	2030	3050	1820	2730
	40	2360	3550	2100	3160	1870	2800	1670	2510
	42	2170	3270	1930	2900	1710	2570	1520	2290
	44	1990	2990	1760	2650	1560	2340	1390	2080
	46	1820	2730	1610	2420	1420	2140	1270	1910
	48	1670	2510	1480	2220	1310	1960	1160	1750
	50	1540	2310	1360	2050	1200	1810	1070	1610
Properties									
P_{wo}, kips		2060	3090	1750	2630	1500	2240	1280	1920
P_{wi}, kip/in.		86.7	130	79.3	119	73.0	110	67.3	101
P_{wb}, kips		26600	40100	20500	30800	15900	23900	12500	18800
P_{fb}, kips		3240	4870	2730	4100	2290	3450	1930	2900
L_p, ft		16.1		15.9		15.6		15.5	
L_r, ft		232		213		196		179	
A_g, in.^{2}		178		162		147		134	
I_x, in.^{4}		10800		9430		8210		7190	
I_y, in.^{4}		3680		3250		2880		2560	
r_y, in.		4.55		4.49		4.43		4.38	
r_x/r_y		1.71		1.70		1.69		1.67	
$P_{ex}L_c^2/10^4$, k-in.^{2}		309000		270000		235000		206000	
$P_{ey}L_c^2/10^4$, k-in.^{2}		105000		93000		82400		73300	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

W14

Table 4-1a (continued)
Available Strength in Axial Compression, kips
W-Shapes

F_y = 50 ksi

Shape	W14×											
lb/ft	426[h]		398[h]		370[h]		342[h]		311[h]		283[h]	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y: 0	3740	5620	3500	5260	3260	4900	3020	4540	2740	4110	2490	3750
11	3500	5260	3270	4920	3040	4570	2820	4230	2550	3830	2320	3480
12	3450	5190	3230	4850	3000	4510	2780	4180	2510	3770	2290	3440
13	3410	5120	3180	4780	2960	4450	2740	4120	2470	3720	2250	3380
14	3350	5040	3130	4710	2910	4380	2700	4050	2430	3660	2210	3330
15	3300	4960	3080	4630	2870	4310	2650	3980	2390	3600	2180	3270
16	3240	4870	3030	4550	2810	4230	2600	3910	2350	3530	2140	3210
17	3180	4790	2970	4470	2760	4150	2550	3840	2300	3460	2090	3150
18	3120	4690	2920	4380	2710	4070	2500	3760	2260	3390	2050	3080
19	3060	4600	2850	4290	2650	3980	2450	3680	2210	3320	2000	3010
20	2990	4500	2790	4200	2590	3890	2390	3600	2160	3240	1960	2940
22	2860	4290	2660	4000	2470	3710	2280	3420	2050	3080	1860	2800
24	2710	4080	2530	3800	2340	3520	2160	3240	1940	2920	1760	2640
26	2560	3850	2390	3590	2210	3320	2040	3060	1830	2750	1660	2490
28	2410	3630	2250	3380	2080	3120	1910	2870	1710	2580	1550	2330
30	2260	3400	2100	3160	1940	2920	1790	2680	1600	2400	1450	2170
32	2110	3170	1960	2950	1810	2720	1660	2500	1490	2230	1340	2020
34	1960	2950	1820	2730	1670	2520	1540	2310	1370	2060	1240	1860
36	1810	2730	1680	2530	1540	2320	1420	2130	1260	1900	1140	1710
38	1670	2510	1550	2320	1420	2130	1300	1950	1160	1740	1040	1560
40	1530	2300	1410	2130	1300	1950	1180	1780	1050	1580	945	1420
42	1390	2090	1290	1930	1180	1770	1070	1610	954	1430	857	1290
44	1270	1910	1170	1760	1070	1610	979	1470	869	1310	781	1170
46	1160	1750	1070	1610	980	1470	896	1350	795	1200	715	1070
48	1070	1600	985	1480	900	1350	823	1240	730	1100	656	986
50	983	1480	907	1360	830	1250	758	1140	673	1010	605	909

Properties

	426 ASD	426 LRFD	398 ASD	398 LRFD	370 ASD	370 LRFD	342 ASD	342 LRFD	311 ASD	311 LRFD	283 ASD	283 LRFD
P_{wo}, kips	1140	1710	1010	1520	902	1350	788	1180	672	1010	574	861
P_{wi}, kip/in.	62.7	94.0	59.0	88.5	55.3	83.0	51.3	77.0	47.0	70.5	43.0	64.5
P_{wb}, kips	10100	15100	8420	12700	6920	10400	5540	8320	4250	6390	3260	4900
P_{fb}, kips	1730	2600	1520	2280	1320	1990	1140	1720	956	1440	802	1210

	426	398	370	342	311	283
L_p, ft	15.3	15.2	15.1	15.0	14.8	14.7
L_r, ft	168	158	148	138	125	114
A_g, in.^{2}	125	117	109	101	91.4	83.3
I_x, in.^{4}	6600	6000	5440	4900	4330	3840
I_y, in.^{4}	2360	2170	1990	1810	1610	1440
r_y, in.	4.34	4.31	4.27	4.24	4.20	4.17
r_x/r_y	1.67	1.66	1.66	1.65	1.64	1.63
$P_{ex}L_c^2/10^4$, k-in.^{2}	189000	172000	156000	140000	124000	110000
$P_{ey}L_c^2/10^4$, k-in.^{2}	67500	62100	57000	51800	46100	41200

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

F_y = 50 ksi

Table 4-1a (continued)
Available Strength in Axial Compression, kips
W-Shapes

W14

Shape		W14×											
lb/ft		257		233		211		193		176		159	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	2260	3400	2050	3080	1860	2790	1700	2560	1550	2330	1400	2100
	6	2210	3330	2010	3010	1810	2730	1660	2500	1510	2280	1370	2050
	7	2200	3300	1990	2990	1800	2700	1650	2480	1500	2260	1350	2030
	8	2180	3270	1970	2960	1780	2680	1630	2450	1490	2240	1340	2010
	9	2150	3240	1950	2930	1760	2650	1610	2430	1470	2210	1330	1990
	10	2130	3200	1930	2900	1740	2620	1590	2400	1450	2180	1310	1970
	11	2100	3160	1900	2860	1720	2580	1570	2360	1430	2150	1290	1940
	12	2070	3110	1870	2820	1690	2550	1550	2330	1410	2120	1270	1910
	13	2040	3060	1840	2770	1670	2510	1530	2290	1390	2090	1250	1880
	14	2010	3010	1810	2730	1640	2460	1500	2250	1360	2050	1230	1850
	15	1970	2960	1780	2680	1610	2420	1470	2210	1340	2010	1210	1810
	16	1930	2900	1750	2630	1580	2370	1440	2170	1310	1970	1180	1780
	17	1890	2850	1710	2570	1540	2320	1410	2120	1280	1930	1160	1740
	18	1850	2790	1670	2520	1510	2270	1380	2080	1260	1890	1130	1700
	19	1810	2720	1640	2460	1480	2220	1350	2030	1230	1840	1100	1660
	20	1770	2660	1600	2400	1440	2160	1320	1980	1200	1800	1070	1620
	22	1680	2520	1510	2280	1360	2050	1250	1870	1130	1700	1020	1530
	24	1590	2380	1430	2150	1290	1930	1170	1770	1070	1600	957	1440
	26	1490	2240	1340	2020	1210	1820	1100	1660	998	1500	896	1350
	28	1400	2100	1260	1890	1130	1700	1030	1550	931	1400	835	1250
	30	1300	1950	1170	1750	1050	1570	954	1430	863	1300	773	1160
	32	1200	1810	1080	1620	968	1460	881	1320	796	1200	713	1070
	34	1110	1670	994	1490	890	1340	810	1220	730	1100	653	982
	36	1020	1530	911	1370	815	1220	740	1110	667	1000	596	896
	38	928	1400	830	1250	741	1110	673	1010	605	909	540	812
	40	841	1260	751	1130	670	1010	608	914	546	821	487	733

Properties

	257		233		211		193		176		159	
P_{wo}, kips	490	735	414	621	353	529	303	454	264	396	222	333
P_{wi}, kip/in.	39.3	59.0	35.7	53.5	32.7	49.0	29.7	44.5	27.7	41.5	24.8	37.3
P_{wb}, kips	2480	3730	1850	2780	1430	2150	1070	1610	870	1310	628	944
P_{fb}, kips	668	1000	554	832	455	684	388	583	321	483	265	398
L_p, ft	14.6		14.5		14.4		14.3		14.2		14.1	
L_r, ft	104		95.0		86.6		79.4		73.2		66.7	
A_g, in.^{2}	75.6		68.5		62.0		56.8		51.8		46.7	
I_x, in.^{4}	3400		3010		2660		2400		2140		1900	
I_y, in.^{4}	1290		1150		1030		931		838		748	
r_y, in.	4.13		4.10		4.07		4.05		4.02		4.00	
r_x/r_y	1.62		1.62		1.61		1.60		1.60		1.60	
$P_{ex}L_c^2/10^4$, k-in.^{2}	97300		86200		76100		68700		61300		54400	
$P_{ey}L_c^2/10^4$, k-in.^{2}	36900		32900		29500		26600		24000		21400	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

W14

Table 4-1a (continued)
Available Strength in Axial Compression, kips
W-Shapes

F_y = 50 ksi

Shape	W14×											
lb/ft	145		132		120		109		99		90	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y												
0	1280	1920	1160	1750	1060	1590	958	1440	871	1310	793	1190
6	1250	1880	1130	1700	1030	1550	932	1400	848	1270	772	1160
7	1240	1860	1120	1680	1020	1530	923	1390	839	1260	764	1150
8	1230	1840	1110	1660	1010	1510	913	1370	830	1250	755	1140
9	1210	1820	1090	1640	994	1490	901	1350	819	1230	745	1120
10	1200	1800	1080	1620	980	1470	888	1340	807	1210	735	1100
11	1180	1770	1060	1600	965	1450	874	1310	794	1190	723	1090
12	1160	1750	1040	1570	948	1430	859	1290	780	1170	710	1070
13	1140	1720	1020	1540	931	1400	843	1270	766	1150	697	1050
14	1120	1690	1000	1510	912	1370	826	1240	750	1130	682	1030
15	1100	1650	982	1480	892	1340	808	1210	733	1100	667	1000
16	1080	1620	960	1440	872	1310	789	1190	716	1080	652	979
17	1060	1590	937	1410	850	1280	770	1160	698	1050	635	955
18	1030	1550	913	1370	828	1240	750	1130	680	1020	618	929
19	1010	1510	888	1330	805	1210	729	1100	661	994	601	903
20	980	1470	862	1300	782	1180	708	1060	642	964	583	877
22	927	1390	810	1220	734	1100	664	998	602	904	547	822
24	872	1310	756	1140	685	1030	620	931	561	843	509	766
26	816	1230	702	1060	635	955	574	863	519	781	472	709
28	759	1140	648	974	586	880	529	796	478	719	434	653
30	703	1060	594	893	537	807	485	729	438	658	397	597
32	647	973	542	814	489	735	441	663	398	598	361	543
34	593	891	491	738	443	665	399	600	360	541	326	490
36	540	812	442	664	398	598	359	539	323	485	292	439
38	489	735	397	596	357	536	322	484	290	435	262	394
40	441	663	358	538	322	484	290	437	261	393	237	356
Properties												
P_{wo}, kips	192	287	175	263	151	227	128	192	112	167	96.1	144
P_{wi}, kip/in.	22.7	34.0	21.5	32.3	19.7	29.5	17.5	26.3	16.2	24.3	14.7	22.0
P_{wb}, kips	476	716	407	611	312	469	220	330	173	260	129	194
P_{fb}, kips	222	334	199	298	165	249	138	208	114	171	94.3	142
L_p, ft	14.1		13.3		13.2		13.2		13.5		15.1	
L_r, ft	61.7		55.8		51.9		48.5		45.3		42.5	
A_g, in.^{2}	42.7		38.8		35.3		32.0		29.1		26.5	
I_x, in.^{4}	1710		1530		1380		1240		1110		999	
I_y, in.^{4}	677		548		495		447		402		362	
r_y, in.	3.98		3.76		3.74		3.73		3.71		3.70	
r_x/r_y	1.59		1.67		1.67		1.67		1.66		1.66	
$P_{ex}L_c^2/10^4$, k-in.^{2}	48900		43800		39500		35500		31800		28600	
$P_{ey}L_c^2/10^4$, k-in.^{2}	19400		15700		14200		12800		11500		10400	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

F_y = 50 ksi

Table 4-1a (continued)
Available Strength in Axial Compression, kips
W-Shapes

W14

Shape		W14×													
lb/ft		82		74		68		61		53		48		43[c]	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	719	1080	653	981	599	900	536	805	467	702	422	634	374	562
	6	676	1020	614	922	562	845	503	756	421	633	380	572	339	510
	7	661	993	600	902	550	826	492	739	406	610	366	551	327	491
	8	644	968	585	879	536	805	479	720	389	585	351	527	312	470
	9	626	940	568	854	520	782	465	699	371	557	334	502	297	447
	10	606	910	550	827	503	756	450	676	351	528	316	475	281	422
	11	584	878	531	797	485	729	433	651	331	497	298	447	264	397
	12	562	844	510	767	466	701	416	626	310	465	279	419	247	371
	13	538	809	489	735	446	671	398	599	288	433	259	390	229	345
	14	514	772	467	701	426	640	380	571	267	401	240	360	212	318
	15	489	735	444	667	405	608	361	543	246	369	221	331	194	292
	16	464	697	421	633	384	577	342	514	225	338	202	303	177	267
	17	438	659	398	598	362	544	323	485	205	308	183	276	161	242
	18	413	620	375	563	341	512	304	456	185	278	166	249	145	218
	19	387	582	352	529	320	480	285	428	166	250	149	224	130	196
	20	362	545	329	495	299	449	266	399	150	226	134	202	117	177
	22	314	472	285	428	258	388	229	345	124	186	111	167	97.1	146
	24	267	402	243	365	219	330	195	293	104	157	93.2	140	81.6	123
	26	228	343	207	311	187	281	166	249	88.8	133	79.4	119	69.5	104
	28	197	295	179	268	161	242	143	215	76.6	115	68.5	103	59.9	90.1
	30	171	257	156	234	140	211	125	187	66.7	100	59.7	89.7	52.2	78.5
	32	150	226	137	205	123	185	110	165	58.6	88.1				
	34	133	200	121	182	109	164	97.0	146						
	36	119	179	108	162	97.5	147	86.5	130						
	38	107	160	96.9	146	87.5	131	77.7	117						
	40	96.3	145	87.5	131	79.0	119	70.1	105						

Properties

	82 ASD	82 LRFD	74 ASD	74 LRFD	68 ASD	68 LRFD	61 ASD	61 LRFD	53 ASD	53 LRFD	48 ASD	48 LRFD	43 ASD	43 LRFD
P_{wo}, kips	123	185	104	155	90.6	136	77.5	116	77.1	116	67.4	101	56.9	85.4
P_{wi}, kip/in.	17.0	25.5	15.0	22.5	13.8	20.8	12.5	18.8	12.3	18.5	11.3	17.0	10.2	15.3
P_{wb}, kips	201	302	138	207	108	163	80.1	120	76.7	115	59.5	89.5	43.0	64.7
P_{fb}, kips	137	206	115	173	97.0	146	77.8	117	81.5	123	66.2	99.6	52.6	79.0

	82	74	68	61	53	48	43
L_p, ft	8.76	8.76	8.69	8.65	6.78	6.75	6.68
L_r, ft	33.2	31.0	29.3	27.5	22.3	21.1	20.0
A_g, in.^{2}	24.0	21.8	20.0	17.9	15.6	14.1	12.6
I_x, in.^{4}	881	795	722	640	541	484	428
I_y, in.^{4}	148	134	121	107	57.7	51.4	45.2
r_y, in.	2.48	2.48	2.46	2.45	1.92	1.91	1.89
r_x/r_y	2.44	2.44	2.44	2.44	3.07	3.06	3.08
$P_{ex}L_c^2/10^4$, k-in.^{2}	25200	22800	20700	18300	15500	13900	12300
$P_{ey}L_c^2/10^4$, k-in.^{2}	4240	3840	3460	3060	1650	1470	1290

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

W12

Table 4-1a (continued)
Available Strength in Axial Compression, kips
W-Shapes

F_y = 50 ksi

Shape		W12×											
lb/ft		336[h]		305[h]		279[h]		252[h]		230[h]		210	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	2960	4450	2680	4030	2450	3690	2220	3330	2030	3050	1850	2780
	6	2870	4310	2590	3900	2370	3570	2140	3220	1960	2940	1790	2680
	7	2840	4260	2560	3850	2340	3520	2120	3180	1930	2910	1760	2650
	8	2800	4210	2530	3800	2310	3470	2090	3140	1910	2860	1740	2610
	9	2760	4150	2490	3740	2280	3420	2060	3090	1880	2820	1710	2570
	10	2710	4080	2450	3680	2240	3360	2020	3030	1840	2770	1680	2520
	11	2660	4000	2400	3610	2190	3300	1980	2970	1800	2710	1640	2470
	12	2610	3920	2350	3540	2150	3230	1940	2910	1760	2650	1610	2420
	13	2550	3840	2300	3460	2100	3150	1890	2840	1720	2590	1570	2360
	14	2490	3750	2250	3380	2050	3080	1840	2770	1680	2520	1530	2300
	15	2430	3660	2190	3290	1990	3000	1790	2700	1630	2450	1480	2230
	16	2370	3560	2130	3200	1940	2910	1740	2620	1580	2380	1440	2160
	17	2300	3460	2070	3100	1880	2820	1690	2540	1540	2310	1390	2100
	18	2230	3350	2000	3010	1820	2730	1630	2460	1480	2230	1350	2030
	19	2160	3250	1940	2910	1760	2640	1580	2370	1430	2150	1300	1950
	20	2090	3140	1870	2810	1700	2550	1520	2290	1380	2070	1250	1880
	22	1940	2910	1730	2610	1570	2360	1410	2110	1270	1910	1150	1730
	24	1790	2690	1600	2400	1440	2170	1290	1940	1170	1750	1050	1580
	26	1640	2460	1460	2190	1320	1980	1170	1760	1060	1590	955	1440
	28	1490	2240	1320	1990	1190	1790	1060	1590	954	1430	859	1290
	30	1350	2030	1190	1790	1070	1610	949	1430	854	1280	767	1150
	32	1210	1820	1070	1600	954	1430	843	1270	756	1140	678	1020
	34	1080	1620	945	1420	845	1270	746	1120	670	1010	600	902
	36	959	1440	843	1270	754	1130	666	1000	597	898	535	805
	38	861	1290	757	1140	676	1020	598	898	536	806	481	722
	40	777	1170	683	1030	610	917	539	811	484	727	434	652
Properties													
P_{wo}, kips		1050	1580	897	1340	783	1170	665	998	574	861	492	738
P_{wi}, kip/in.		59.3	89.0	54.3	81.5	51.0	76.5	46.7	70.0	43.0	64.5	39.3	59.0
P_{wb}, kips		10000	15100	7690	11600	6380	9590	4870	7320	3810	5730	2930	4400
P_{fb}, kips		1640	2460	1370	2070	1140	1720	947	1420	802	1210	676	1020
L_p, ft		12.3		12.1		11.9		11.8		11.7		11.6	
L_r, ft		150		137		126		114		105		95.8	
A_g, in.^{2}		98.9		89.5		81.9		74.1		67.7		61.8	
I_x, in.^{4}		4060		3550		3110		2720		2420		2140	
I_y, in.^{4}		1190		1050		937		828		742		664	
r_y, in.		3.47		3.42		3.38		3.34		3.31		3.28	
r_x/r_y		1.85		1.84		1.82		1.81		1.80		1.80	
$P_{ex}L_c^2/10^4$, k-in.^{2}		116000		102000		89000		77900		69300		61300	
$P_{ey}L_c^2/10^4$, k-in.^{2}		34100		30100		26800		23700		21200		19000	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

F_y = 50 ksi

Table 4-1a (continued)
Available Strength in Axial Compression, kips
W-Shapes

W12

Shape	W12×											
lb/ft	190		170		152		136		120		106	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y												
0	1680	2520	1500	2250	1340	2010	1190	1800	1050	1580	934	1400
6	1620	2430	1440	2170	1290	1940	1150	1730	1010	1520	898	1350
7	1600	2400	1420	2140	1270	1910	1130	1710	1000	1500	886	1330
8	1570	2360	1400	2110	1250	1880	1120	1680	984	1480	871	1310
9	1550	2320	1380	2070	1230	1850	1100	1650	966	1450	855	1290
10	1520	2280	1350	2030	1210	1810	1080	1620	947	1420	838	1260
11	1490	2230	1320	1990	1180	1770	1050	1580	925	1390	819	1230
12	1450	2180	1290	1940	1150	1730	1030	1540	903	1360	799	1200
13	1420	2130	1260	1900	1120	1690	1000	1500	879	1320	777	1170
14	1380	2070	1230	1840	1090	1640	972	1460	854	1280	755	1130
15	1340	2010	1190	1790	1060	1590	942	1420	828	1240	731	1100
16	1300	1950	1150	1730	1030	1540	912	1370	800	1200	707	1060
17	1260	1890	1120	1680	992	1490	881	1320	773	1160	682	1030
18	1210	1820	1080	1620	957	1440	849	1280	744	1120	656	987
19	1170	1760	1040	1560	921	1380	816	1230	715	1070	631	948
20	1130	1690	997	1500	885	1330	784	1180	686	1030	604	908
22	1030	1560	916	1380	811	1220	717	1080	626	942	552	829
24	944	1420	834	1250	737	1110	651	978	567	853	499	750
26	855	1280	754	1130	665	999	586	880	510	766	448	673
28	767	1150	675	1010	595	894	523	786	454	682	398	598
30	684	1030	600	902	527	793	462	695	400	601	350	526
32	603	906	528	794	464	697	406	610	352	528	308	462
34	534	803	468	704	411	617	360	541	311	468	272	410
36	476	716	418	628	366	551	321	482	278	417	243	365
38	428	643	375	563	329	494	288	433	249	375	218	328
40	386	580	338	508	297	446	260	391	225	338	197	296
Properties												
P_{wo}, kips	412	617	346	518	290	435	244	365	201	302	162	242
P_{wi}, kip/in.	35.3	53.0	32.0	48.0	29.0	43.5	26.3	39.5	23.7	35.5	20.3	30.5
P_{wb}, kips	2120	3190	1580	2370	1170	1760	878	1320	637	957	405	609
P_{fb}, kips	567	852	455	684	367	551	292	439	231	347	183	276
L_p, ft	11.5		11.4		11.3		11.2		11.1		11.0	
L_r, ft	87.3		78.5		70.6		63.2		56.5		50.7	
A_g, in.^{2}	56.0		50.0		44.7		39.9		35.2		31.2	
I_x, in.^{4}	1890		1650		1430		1240		1070		933	
I_y, in.^{4}	589		517		454		398		345		301	
r_y, in.	3.25		3.22		3.19		3.16		3.13		3.11	
r_x/r_y	1.79		1.78		1.77		1.77		1.76		1.76	
$P_{ex}L_c^2/10^4$, k-in.^{2}	54100		47200		40900		35500		30600		26700	
$P_{ey}L_c^2/10^4$, k-in.^{2}	16900		14800		13000		11400		9870		8620	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

W12

Table 4-1a (continued)
Available Strength in Axial Compression, kips
W-Shapes

F_y = 50 ksi

Shape	W12×									
lb/ft	96		87		79		72		65	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y										
0	844	1270	766	1150	695	1040	632	949	572	859
6	811	1220	736	1110	667	1000	606	911	549	825
7	800	1200	726	1090	657	988	597	898	540	812
8	787	1180	714	1070	646	971	587	883	531	798
9	772	1160	700	1050	634	953	576	866	521	783
10	756	1140	685	1030	620	932	564	847	510	766
11	739	1110	670	1010	606	910	550	827	497	747
12	720	1080	653	981	590	887	536	806	484	728
13	701	1050	635	954	574	862	521	783	470	707
14	680	1020	616	925	556	836	505	759	456	685
15	659	990	596	896	538	809	489	735	441	663
16	637	957	576	865	520	781	472	709	426	640
17	614	923	555	834	501	753	455	683	410	616
18	591	888	534	802	481	723	437	656	393	591
19	567	852	512	770	462	694	419	629	377	567
20	543	816	490	737	442	664	401	602	360	542
22	495	744	446	671	402	604	364	547	327	492
24	447	672	403	605	362	544	328	493	294	442
26	401	602	360	541	323	486	292	440	262	394
28	356	535	319	480	286	430	259	389	231	348
30	312	469	280	421	250	376	226	340	202	304
32	274	413	246	370	220	331	199	299	178	267
34	243	365	218	327	195	293	176	265	157	236
36	217	326	194	292	174	261	157	236	140	211
38	195	293	174	262	156	234	141	212	126	189
40	176	264	157	237	141	212	127	191	114	171
Properties										
P_{wo}, kips	138	206	121	182	104	156	91.0	137	78.0	117
P_{wi}, kip/in.	18.3	27.5	17.2	25.8	15.7	23.5	14.3	21.5	13.0	19.5
P_{wb}, kips	296	445	243	365	185	278	142	213	106	159
P_{fb}, kips	152	228	123	185	101	152	84.0	126	68.5	103
L_p, ft	10.9		10.8		10.8		10.7		11.9	
L_r, ft	46.7		43.1		39.9		37.5		35.1	
A_g, in.^{2}	28.2		25.6		23.2		21.1		19.1	
I_x, in.^{4}	833		740		662		597		533	
I_y, in.^{4}	270		241		216		195		174	
r_y, in.	3.09		3.07		3.05		3.04		3.02	
r_x/r_y	1.76		1.75		1.75		1.75		1.75	
$P_{ex}L_c^2/10^4$, k-in.^{2}	23800		21200		18900		17100		15300	
$P_{ey}L_c^2/10^4$, k-in.^{2}	7730		6900		6180		5580		4980	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

F_y = 50 ksi

Table 4-1a (continued)
Available Strength in Axial Compression, kips
W-Shapes

W12

Shape		W12×									
lb/ft		58		53		50		45		40	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	509	765	467	702	437	657	392	589	350	526
	6	479	720	439	660	396	595	355	534	317	476
	7	469	705	429	646	382	574	342	515	305	459
	8	457	687	419	629	367	551	329	494	293	440
	9	445	668	407	611	350	526	313	471	279	420
	10	431	647	394	592	332	500	297	447	265	398
	11	416	625	380	571	314	472	281	422	250	375
	12	400	601	365	549	295	443	263	396	234	352
	13	384	577	350	526	275	413	246	369	218	328
	14	367	551	334	502	255	384	228	343	202	304
	15	349	525	318	478	236	355	210	316	187	281
	16	332	499	301	453	217	326	193	290	171	257
	17	314	472	285	428	198	298	176	265	156	235
	18	296	445	268	403	180	270	160	240	142	213
	19	278	418	252	378	162	244	144	216	127	191
	20	261	392	235	354	146	220	130	195	115	173
	22	227	341	204	307	121	182	107	161	95.0	143
	24	194	292	174	261	102	153	90.3	136	79.8	120
	26	165	249	148	223	86.6	130	76.9	116	68.0	102
	28	143	214	128	192	74.7	112	66.3	99.7	58.6	88.1
	30	124	187	111	167	65.0	97.8	57.8	86.8	51.1	76.8
	32	109	164	97.8	147	57.2	85.9	50.8	76.3	44.9	67.5
	34	96.7	145	86.6	130						
	36	86.3	130	77.3	116						
	38	77.4	116	69.4	104						
	40	69.9	105	62.6	94.1						
Properties											
P_{wo}, kips		74.4	112	67.9	102	70.3	105	60.3	90.5	50.2	75.2
P_{wi}, kip/in.		12.0	18.0	11.5	17.3	12.3	18.5	11.2	16.8	9.83	14.8
P_{wb}, kips		83.1	125	73.3	110	88.4	133	65.6	98.6	44.8	67.4
P_{fb}, kips		76.6	115	61.9	93.0	76.6	115	61.9	93.0	49.6	74.6
L_p, ft		8.87		8.76		6.92		6.89		6.85	
L_r, ft		29.8		28.2		23.8		22.4		21.1	
A_g, in.^{2}		17.0		15.6		14.6		13.1		11.7	
I_x, in.^{4}		475		425		391		348		307	
I_y, in.^{4}		107		95.8		56.3		50.0		44.1	
r_y, in.		2.51		2.48		1.96		1.95		1.94	
r_x/r_y		2.10		2.11		2.64		2.64		2.64	
$P_{ex}L_c^2/10^4$, k-in.^{2}		13600		12200		11200		9960		8790	
$P_{ey}L_c^2/10^4$, k-in.^{2}		3060		2740		1610		1430		1260	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

Table 4-1a (continued)
Available Strength in Axial Compression, kips
W-Shapes

W10

F_y = 50 ksi

Shape		W10×											
lb/ft		112		100		88		77		68		60	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	985	1480	877	1320	778	1170	680	1020	596	895	530	796
	6	934	1400	831	1250	737	1110	643	966	563	846	500	752
	7	917	1380	815	1230	722	1090	630	946	552	829	490	737
	8	897	1350	797	1200	706	1060	615	925	539	810	479	719
	9	875	1310	777	1170	688	1030	599	900	525	789	466	700
	10	851	1280	755	1130	669	1000	582	874	509	765	452	679
	11	825	1240	732	1100	647	973	563	846	493	741	437	657
	12	798	1200	707	1060	625	940	543	816	475	714	421	633
	13	769	1160	681	1020	602	905	522	785	457	687	405	608
	14	739	1110	654	983	578	868	501	753	438	658	388	583
	15	708	1060	626	941	553	831	479	720	419	629	370	556
	16	677	1020	598	898	527	792	456	686	399	599	352	530
	17	645	969	569	855	501	754	433	651	379	569	334	502
	18	613	921	540	811	475	714	410	617	358	539	316	475
	19	580	872	511	767	449	675	387	582	338	508	298	448
	20	548	824	482	724	423	636	365	548	318	478	280	421
	22	485	728	425	638	373	560	320	481	279	419	245	368
	24	423	636	370	556	324	487	277	417	241	363	212	318
	26	365	548	318	478	278	417	237	356	206	310	181	271
	28	315	473	274	412	239	360	204	307	178	267	156	234
	30	274	412	239	359	209	313	178	267	155	233	136	204
	32	241	362	210	315	183	276	156	235	136	205	119	179
	34	213	321	186	279	162	244	139	208	121	181	106	159
	36	190	286	166	249	145	218	124	186	108	162	94.2	142
	38	171	257	149	224	130	195	111	167	96.5	145	84.5	127
	40	154	232	134	202	117	176	100	150	87.1	131	76.3	115

Properties

Property	112 ASD	112 LRFD	100 ASD	100 LRFD	88 ASD	88 LRFD	77 ASD	77 LRFD	68 ASD	68 LRFD	60 ASD	60 LRFD
P_{wo}, kips	220	330	184	275	150	225	121	182	99.5	149	82.6	124
P_{wi}, kip/in.	25.2	37.8	22.7	34.0	20.2	30.3	17.7	26.5	15.7	23.5	14.0	21.0
P_{wb}, kips	949	1430	690	1040	487	732	328	494	229	344	163	245
P_{fb}, kips	292	439	235	353	183	276	142	213	111	167	86.5	130

Property	112	100	88	77	68	60
L_p, ft	9.47	9.36	9.29	9.18	9.15	9.08
L_r, ft	64.1	57.9	51.2	45.3	40.6	36.6
A_g, in.^{2}	32.9	29.3	26.0	22.7	19.9	17.7
I_x, in.^{4}	716	623	534	455	394	341
I_y, in.^{4}	236	207	179	154	134	116
r_y, in.	2.68	2.65	2.63	2.60	2.59	2.57
r_x/r_y	1.74	1.74	1.73	1.73	1.71	1.71
$P_{ex}L_c^2/10^4$, k-in.^{2}	20500	17800	15300	13000	11300	9760
$P_{ey}L_c^2/10^4$, k-in.^{2}	6750	5920	5120	4410	3840	3320

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

F_y = 50 ksi

Table 4-1a (continued)
Available Strength in Axial Compression, kips
W-Shapes

W10

Shape		W10×									
lb/ft		54		49		45		39		33	
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	473	711	431	648	398	598	344	517	291	437
	6	446	671	407	611	363	545	313	470	263	395
	7	437	657	398	598	350	527	302	454	253	381
	8	427	642	388	584	337	507	290	436	243	365
	9	415	624	378	568	322	485	277	416	232	348
	10	403	605	366	550	307	461	263	396	220	330
	11	389	585	354	532	291	437	249	374	207	311
	12	375	564	341	512	274	411	234	352	194	292
	13	361	542	327	492	256	385	219	329	181	272
	14	345	519	313	471	239	359	203	306	168	253
	15	330	495	299	449	222	333	188	283	155	233
	16	314	471	284	427	204	307	173	260	142	214
	17	297	447	269	404	188	282	158	238	130	195
	18	281	422	254	382	171	257	144	217	117	177
	19	265	398	239	360	155	234	130	196	106	159
	20	249	374	224	337	140	211	118	177	95.4	143
	22	217	327	196	294	116	174	97.2	146	78.8	118
	24	188	282	168	253	97.4	146	81.7	123	66.2	99.5
	26	160	240	143	216	83.0	125	69.6	105	56.4	84.8
	28	138	207	124	186	71.5	108	60.0	90.2	48.7	73.1
	30	120	180	108	162	62.3	93.7	52.3	78.6	42.4	63.7
	32	106	159	94.7	142	54.8	82.3	46.0	69.1	37.3	56.0
	34	93.5	141	83.9	126						
	36	83.4	125	74.8	112						
	38	74.8	112	67.2	101						
	40	67.6	102	60.6	91.1						
Properties											
P_{wo}, kips		69.1	104	60.1	90.1	65.3	98.0	54.1	81.1	45.2	67.8
P_{wi}, kip/in.		12.3	18.5	11.3	17.0	11.7	17.5	10.5	15.8	9.67	14.5
P_{wb}, kips		112	168	86.6	130	94.2	142	68.7	103	53.7	80.7
P_{fb}, kips		70.8	106	58.7	88.2	71.9	108	52.6	79.0	35.4	53.2
L_p, ft		9.04		8.97		7.10		6.99		6.85	
L_r, ft		33.6		31.6		26.9		24.2		21.8	
A_g, in.^{2}		15.8		14.4		13.3		11.5		9.71	
I_x, in.^{4}		303		272		248		209		171	
I_y, in.^{4}		103		93.4		53.4		45.0		36.6	
r_y, in.		2.56		2.54		2.01		1.98		1.94	
r_x/r_y		1.71		1.71		2.15		2.16		2.16	
$P_{ex}L_c^2/10^4$, k-in.^{2}		8670		7790		7100		5980		4890	
$P_{ey}L_c^2/10^4$, k-in.^{2}		2950		2670		1530		1290		1050	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

Table 4-1a (continued)
Available Strength in Axial Compression, kips
W-Shapes

W8

F_y = 50 ksi

Shape		W8×											
lb/ft		67		58		48		40		35		31	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	590	886	512	769	422	634	350	526	308	463	273	411
	6	542	815	470	706	387	581	320	481	281	423	249	374
	7	526	790	455	685	375	563	309	465	272	409	241	362
	8	508	763	439	660	361	543	298	448	262	394	232	348
	9	488	733	422	634	347	521	285	429	251	377	222	333
	10	467	701	403	606	331	497	272	409	239	359	211	317
	11	444	668	384	576	314	473	258	388	226	340	200	301
	12	421	633	363	546	297	447	243	366	213	321	189	283
	13	397	597	342	514	280	421	228	343	200	301	177	266
	14	373	560	321	482	262	394	213	321	187	281	165	248
	15	348	523	299	450	244	367	198	298	174	261	153	230
	16	324	487	278	418	226	340	183	275	160	241	141	212
	17	300	450	257	386	209	314	169	253	147	221	130	195
	18	276	415	236	355	192	288	154	232	135	203	118	178
	19	253	381	216	325	175	264	141	211	123	184	108	162
	20	231	347	197	296	159	239	127	191	111	166	97.2	146
	22	191	287	163	244	132	198	105	158	91.5	138	80.3	121
	24	160	241	137	205	111	166	88.2	133	76.9	116	67.5	101
	26	137	205	116	175	94.2	142	75.2	113	65.5	98.5	57.5	86.5
	28	118	177	100	151	81.2	122	64.8	97.4	56.5	84.9	49.6	74.5
	30	103	154	87.5	131	70.7	106	56.5	84.9	49.2	74.0	43.2	64.9
	32	90.3	136	76.9	116	62.2	93.5	49.6	74.6	43.3	65.0	38.0	57.1
	34	79.9	120	68.1	102	55.1	82.8	44.0	66.1				
Properties													
P_{wo}, kips		126	190	102	153	72.0	108	57.2	85.9	45.9	68.9	39.4	59.1
P_{wi}, kip/in.		19.0	28.5	17.0	25.5	13.3	20.0	12.0	18.0	10.3	15.5	9.50	14.3
P_{wb}, kips		507	761	363	546	174	262	127	192	81.1	122	63.0	94.7
P_{fb}, kips		164	246	123	185	87.8	132	58.7	88.2	45.9	68.9	35.4	53.2
L_p, ft		7.49		7.42		7.35		7.21		7.17		7.18	
L_r, ft		47.6		41.6		35.2		29.9		27.0		24.8	
A_g, in.^{2}		19.7		17.1		14.1		11.7		10.3		9.13	
I_x, in.^{4}		272		228		184		146		127		110	
I_y, in.^{4}		88.6		75.1		60.9		49.1		42.6		37.1	
r_y, in.		2.12		2.10		2.08		2.04		2.03		2.02	
r_x/r_y		1.75		1.74		1.74		1.73		1.73		1.72	
$P_{ex}L_c^2/10^4$, k-in.^{2}		7790		6530		5270		4180		3630		3150	
$P_{ey}L_c^2/10^4$, k-in.^{2}		2540		2150		1740		1410		1220		1060	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

Table 4-1b
Available Strength in Axial Compression, kips
W-Shapes

F_y = 65 ksi

W14

Shape		W14×							
lb/ft		873[h]		808[h]		730[h]		665[h]	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	10000	15000	9260	13900	8370	12600	7630	11500
	11	9340	14000	8630	13000	7760	11700	7060	10600
	12	9210	13800	8510	12800	7650	11500	6960	10500
	13	9080	13700	8390	12600	7530	11300	6850	10300
	14	8950	13400	8260	12400	7410	11100	6730	10100
	15	8800	13200	8120	12200	7270	10900	6600	9930
	16	8640	13000	7970	12000	7140	10700	6470	9730
	17	8480	12800	7820	11800	6990	10500	6340	9530
	18	8320	12500	7660	11500	6840	10300	6200	9310
	19	8140	12200	7500	11300	6680	10000	6050	9100
	20	7960	12000	7330	11000	6520	9810	5900	8870
	22	7590	11400	6970	10500	6190	9310	5590	8410
	24	7200	10800	6610	9930	5850	8790	5270	7920
	26	6800	10200	6230	9360	5490	8260	4950	7430
	28	6400	9620	5850	8790	5140	7720	4610	6940
	30	5990	9000	5460	8210	4780	7180	4280	6440
	32	5580	8390	5080	7630	4420	6650	3960	5950
	34	5180	7780	4700	7070	4080	6130	3640	5460
	36	4780	7180	4330	6510	3740	5620	3320	4990
	38	4390	6600	3970	5970	3410	5120	3020	4540
	40	4020	6040	3620	5450	3090	4640	2730	4100
	42	3650	5490	3290	4940	2800	4210	2480	3720
	44	3330	5000	2990	4500	2550	3830	2260	3390
	46	3040	4570	2740	4120	2330	3510	2060	3100
	48	2800	4200	2520	3780	2140	3220	1900	2850
	50	2580	3870	2320	3480	1970	2970	1750	2630
Properties									
P_{wo}, kips		5210	7810	4630	6940	3670	5500	3140	4710
P_{wi}, kip/in.		171	256	162	243	133	200	123	184
P_{wb}, kips		106000	159000	90800	136000	50100	75300	39200	58900
P_{fb}, kips		7390	11100	6380	9580	5860	8810	4970	7470
L_p, ft		15.2		15.0		14.5		14.3	
L_r, ft		253		238		212		195	
A_g, in.^{2}		257		238		215		196	
I_x, in.^{4}		18100		15900		14300		12400	
I_y, in.^{4}		6170		5550		4720		4170	
r_y, in.		4.90		4.83		4.69		4.62	
r_x/r_y		1.71		1.69		1.74		1.73	
$P_{ex}L_c^2/10^4$, k-in.^{2}		518000		455000		409000		355000	
$P_{ey}L_c^2/10^4$, k-in.^{2}		177000		159000		135000		119000	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying, as discussed in Part 2.

Table 4-1b (continued)
Available Strength in Axial Compression, kips
W-Shapes

W14

F_y = 65 ksi

Shape		W14×							
lb/ft		605[h]		550[h]		500[h]		455[h]	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	6930	10400	6310	9480	5720	8600	5220	7840
	11	6400	9610	5810	8730	5260	7900	4780	7190
	12	6300	9470	5720	8590	5170	7780	4710	7070
	13	6200	9310	5620	8450	5090	7640	4620	6950
	14	6090	9150	5520	8300	4990	7500	4530	6820
	15	5970	8970	5410	8130	4890	7350	4440	6680
	16	5850	8790	5300	7970	4790	7190	4340	6530
	17	5720	8600	5180	7790	4680	7030	4240	6380
	18	5590	8410	5060	7610	4560	6860	4140	6220
	19	5460	8200	4930	7420	4450	6690	4030	6060
	20	5320	7990	4810	7220	4330	6510	3920	5890
	22	5030	7560	4540	6820	4080	6140	3690	5550
	24	4730	7120	4260	6410	3830	5750	3460	5200
	26	4430	6660	3980	5990	3570	5370	3220	4840
	28	4130	6200	3700	5570	3310	4980	2980	4480
	30	3820	5740	3420	5140	3050	4590	2740	4120
	32	3520	5290	3150	4730	2800	4210	2510	3780
	34	3230	4850	2880	4320	2550	3840	2290	3440
	36	2940	4420	2620	3930	2320	3480	2070	3110
	38	2660	4000	2360	3550	2090	3130	1860	2790
	40	2400	3610	2130	3200	1880	2830	1680	2520
	42	2180	3280	1930	2900	1710	2570	1520	2290
	44	1990	2990	1760	2650	1560	2340	1390	2080
	46	1820	2730	1610	2420	1420	2140	1270	1910
	48	1670	2510	1480	2220	1310	1960	1160	1750
	50	1540	2310	1360	2050	1200	1810	1070	1610
Properties									
P_{wo}, kips		2680	4020	2280	3420	1950	2920	1670	2500
P_{wi}, kip/in.		113	169	103	155	94.9	142	87.5	131
P_{wb}, kips		30400	45700	23300	35100	18200	27300	14200	21400
P_{fb}, kips		4210	6330	3550	5340	2980	4480	2510	3770
L_p, ft		14.1		13.9		13.7		13.6	
L_r, ft		178		164		151		138	
A_g, in.^{2}		178		162		147		134	
I_x, in.^{4}		10800		9430		8210		7190	
I_y, in.^{4}		3680		3250		2880		2560	
r_y, in.		4.55		4.49		4.43		4.38	
r_x/r_y		1.71		1.70		1.69		1.67	
$P_{ex}L_c^2/10^4$, k-in.^{2}		309000		270000		235000		206000	
$P_{ey}L_c^2/10^4$, k-in.^{2}		105000		93000		82400		73300	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying, as discussed in Part 2.

F_y = 65 ksi

Table 4-1b (continued)
Available Strength in Axial Compression, kips
W-Shapes

W14

Shape	W14×											
lb/ft	426[h]		398[h]		370[h]		342[h]		311[h]		283[h]	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y												
0	4870	7310	4550	6840	4240	6380	3930	5910	3560	5350	3240	4870
11	4460	6700	4170	6260	3870	5820	3590	5390	3240	4870	2950	4430
12	4380	6590	4100	6160	3810	5720	3520	5290	3180	4780	2890	4350
13	4300	6470	4020	6040	3740	5620	3460	5200	3120	4690	2840	4270
14	4220	6340	3940	5920	3660	5500	3390	5090	3060	4590	2780	4180
15	4130	6210	3860	5800	3580	5390	3310	4980	2990	4490	2720	4080
16	4040	6070	3770	5670	3500	5260	3230	4860	2920	4380	2650	3980
17	3940	5930	3680	5530	3420	5130	3150	4740	2840	4270	2580	3880
18	3840	5780	3590	5390	3330	5000	3070	4620	2770	4160	2510	3780
19	3740	5630	3490	5250	3240	4860	2990	4490	2690	4040	2440	3670
20	3640	5470	3390	5100	3140	4720	2900	4360	2610	3920	2370	3560
22	3420	5140	3190	4790	2950	4430	2720	4090	2440	3670	2220	3330
24	3200	4810	2980	4480	2750	4140	2540	3810	2280	3420	2060	3100
26	2980	4470	2770	4160	2550	3840	2350	3530	2110	3160	1900	2860
28	2750	4140	2560	3840	2360	3540	2160	3250	1940	2910	1750	2630
30	2530	3800	2350	3530	2160	3240	1980	2980	1770	2660	1600	2400
32	2310	3470	2140	3220	1970	2960	1800	2710	1610	2420	1450	2180
34	2100	3160	1940	2920	1780	2680	1630	2450	1450	2180	1310	1960
36	1900	2850	1750	2630	1600	2410	1460	2200	1300	1950	1170	1750
38	1700	2560	1570	2360	1440	2160	1310	1970	1170	1750	1050	1570
40	1540	2310	1420	2130	1300	1950	1180	1780	1050	1580	945	1420
42	1390	2090	1290	1930	1180	1770	1070	1610	954	1430	857	1290
44	1270	1910	1170	1760	1070	1610	979	1470	869	1310	781	1170
46	1160	1750	1070	1610	980	1470	896	1350	795	1200	715	1070
48	1070	1600	985	1480	900	1350	823	1240	730	1100	656	986
50	983	1480	907	1360	830	1250	758	1140	673	1010	605	909
Properties												
P_{wo}, kips	1480	2220	1320	1980	1170	1760	1020	1540	874	1310	746	1120
P_{wi}, kip/in.	81.5	122	76.7	115	71.9	108	66.7	100	61.1	91.7	55.9	83.9
P_{wb}, kips	11500	17200	9600	14400	7890	11900	6320	9490	4850	7290	3710	5580
P_{fb}, kips	2250	3380	1980	2970	1720	2590	1480	2230	1240	1870	1040	1570
L_p, ft	13.4		13.4		13.2		13.1		13.0		12.9	
L_r, ft	130		122		114		106		96.7		88.3	
A_g, in.^{2}	125		117		109		101		91.4		83.3	
I_x, in.^{4}	6600		6000		5440		4900		4330		3840	
I_y, in.^{4}	2360		2170		1990		1810		1610		1440	
r_y, in.	4.34		4.31		4.27		4.24		4.20		4.17	
r_x/r_y	1.67		1.66		1.66		1.65		1.64		1.63	
$P_{ex}L_c^2/10^4$, k-in.^{2}	189000		172000		156000		140000		124000		110000	
$P_{ey}L_c^2/10^4$, k-in.^{2}	67500		62100		57000		51800		46100		41200	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying, as discussed in Part 2.

Table 4-1b (continued)
Available Strength in Axial Compression, kips
W-Shapes

W14

F_y = 65 ksi

Shape	W14×											
lb/ft	257		233		211		193		176		159	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y												
0	2940	4420	2670	4010	2410	3630	2210	3320	2020	3030	1820	2730
6	2860	4300	2590	3890	2340	3520	2150	3220	1960	2940	1760	2650
7	2830	4250	2560	3850	2320	3480	2120	3190	1930	2910	1740	2620
8	2800	4200	2530	3800	2290	3440	2100	3150	1910	2870	1720	2590
9	2760	4140	2500	3750	2260	3390	2070	3110	1880	2830	1700	2550
10	2720	4080	2460	3690	2220	3340	2030	3060	1850	2780	1670	2510
11	2670	4010	2420	3630	2180	3280	2000	3000	1820	2740	1640	2460
12	2620	3940	2370	3560	2140	3220	1960	2950	1780	2680	1610	2420
13	2570	3860	2320	3490	2100	3150	1920	2890	1750	2630	1570	2360
14	2510	3780	2270	3420	2050	3080	1880	2820	1710	2570	1540	2310
15	2460	3690	2220	3340	2000	3010	1830	2750	1670	2500	1500	2250
16	2400	3600	2160	3250	1950	2940	1790	2680	1620	2440	1460	2190
17	2330	3510	2110	3170	1900	2860	1740	2610	1580	2370	1420	2130
18	2270	3410	2050	3080	1850	2780	1690	2540	1530	2300	1380	2070
19	2200	3310	1990	2990	1790	2690	1640	2460	1490	2230	1330	2010
20	2130	3210	1930	2890	1730	2610	1580	2380	1440	2160	1290	1940
22	2000	3000	1800	2700	1620	2430	1480	2220	1340	2010	1200	1810
24	1850	2790	1670	2510	1500	2250	1370	2050	1240	1860	1110	1670
26	1710	2570	1540	2310	1380	2070	1260	1890	1140	1710	1020	1530
28	1570	2360	1410	2120	1260	1900	1150	1730	1040	1560	929	1400
30	1430	2150	1280	1930	1150	1720	1040	1570	941	1410	842	1270
32	1290	1940	1160	1740	1040	1560	941	1410	847	1270	757	1140
34	1160	1750	1040	1560	927	1390	841	1260	756	1140	675	1010
36	1040	1560	927	1390	827	1240	750	1130	674	1010	602	905
38	932	1400	832	1250	742	1120	673	1010	605	909	540	812
40	841	1260	751	1130	670	1010	608	914	546	821	487	733
Properties												
P_{wo}, kips	637	955	538	807	459	688	393	590	343	515	289	433
P_{wi}, kip/in.	51.1	76.7	46.4	69.6	42.5	63.7	38.6	57.9	36.0	54.0	32.3	48.4
P_{wb}, kips	2830	4250	2110	3170	1630	2460	1220	1840	992	1490	716	1080
P_{fb}, kips	869	1310	720	1080	592	890	504	758	417	627	344	518
L_p, ft	12.8		12.7		12.6		12.5		12.5		12.4	
L_r, ft	80.7		73.5		67.2		61.8		57.1		52.4	
A_g, in.^{2}	75.6		68.5		62.0		56.8		51.8		46.7	
I_x, in.^{4}	3400		3010		2660		2400		2140		1900	
I_y, in.^{4}	1290		1150		1030		931		838		748	
r_y, in.	4.13		4.10		4.07		4.05		4.02		4.00	
r_x/r_y	1.62		1.62		1.61		1.60		1.60		1.60	
$P_{ex}L_c^2/10^4$, k-in.^{2}	97300		86200		76100		68700		61300		54400	
$P_{ey}L_c^2/10^4$, k-in.^{2}	36900		32900		29500		26600		24000		21400	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Confirm ASTM A913 material availability before specifying, as discussed in Part 2.

F_y = 65 ksi

Table 4-1b (continued)
Available Strength in Axial Compression, kips
W-Shapes

W14

Shape	W14×											
lb/ft	145		132		120		109		99		90	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y												
0	1660	2500	1510	2270	1370	2070	1250	1870	1130	1700	1030	1550
6	1610	2420	1460	2190	1330	1990	1200	1810	1090	1640	995	1500
7	1590	2390	1440	2160	1310	1970	1190	1780	1080	1620	982	1480
8	1570	2360	1420	2130	1290	1940	1170	1760	1060	1600	968	1450
9	1550	2330	1400	2100	1270	1910	1150	1730	1040	1570	951	1430
10	1520	2290	1370	2060	1250	1870	1130	1700	1030	1540	933	1400
11	1500	2250	1340	2020	1220	1830	1110	1660	1000	1510	914	1370
12	1470	2210	1310	1970	1190	1790	1080	1620	982	1480	893	1340
13	1440	2160	1280	1930	1160	1750	1050	1590	957	1440	871	1310
14	1400	2110	1250	1880	1130	1700	1030	1540	932	1400	848	1270
15	1370	2060	1210	1830	1100	1660	998	1500	906	1360	824	1240
16	1330	2000	1180	1770	1070	1610	968	1460	878	1320	799	1200
17	1290	1950	1140	1720	1040	1560	937	1410	850	1280	773	1160
18	1260	1890	1100	1660	1000	1500	906	1360	821	1230	746	1120
19	1220	1830	1060	1600	965	1450	873	1310	791	1190	719	1080
20	1180	1770	1030	1540	929	1400	840	1260	761	1140	691	1040
22	1090	1640	945	1420	856	1290	774	1160	700	1050	636	956
24	1010	1520	865	1300	782	1180	707	1060	639	960	580	872
26	927	1390	785	1180	709	1070	640	963	578	869	525	789
28	844	1270	707	1060	638	959	576	866	519	781	471	708
30	764	1150	632	950	569	856	514	772	463	696	419	630
32	686	1030	559	840	503	756	454	682	408	614	370	556
34	611	918	495	744	446	670	402	604	362	544	328	492
36	545	819	442	664	398	598	359	539	323	485	292	439
38	489	735	397	596	357	536	322	484	290	435	262	394
40	441	663	358	538	322	484	290	437	261	393	237	356
Properties												
P_{wo}, kips	249	373	228	342	197	295	166	249	145	218	125	187
P_{wi}, kip/in.	29.5	44.2	28.0	41.9	25.6	38.4	22.8	34.1	21.0	31.5	19.1	28.6
P_{wb}, kips	543	816	464	697	356	535	251	377	197	297	147	222
P_{fb}, kips	289	434	258	388	215	323	180	270	148	222	123	184
L_p, ft	12.3		11.6		11.6		12.5		14.0		15.3	
L_r, ft	48.7		44.3		41.5		39.1		36.8		34.9	
A_g, in.^{2}	42.7		38.8		35.3		32.0		29.1		26.5	
I_x, in.^{4}	1710		1530		1380		1240		1110		999	
I_y, in.^{4}	677		548		495		447		402		362	
r_y, in.	3.98		3.76		3.74		3.73		3.71		3.70	
r_x/r_y	1.59		1.67		1.67		1.67		1.66		1.66	
$P_{ex}L_c^2/10^4$, k-in.^{2}	48900		43800		39500		35500		31800		28600	
$P_{ey}L_c^2/10^4$, k-in.^{2}	19400		15700		14200		12800		11500		10400	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Confirm ASTM A913 material availability before specifying, as discussed in Part 2.

Table 4-1b (continued)
Available Strength in Axial Compression, kips
W-Shapes

W12

F_y = 65 ksi

Shape	W12×							
lb/ft	230[h]		210		190		170	
Design	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y								
0	2640	3960	2410	3620	2180	3280	1950	2920
6	2520	3790	2300	3450	2080	3130	1860	2790
7	2480	3730	2260	3400	2050	3070	1820	2740
8	2430	3660	2220	3330	2010	3020	1790	2690
9	2380	3580	2170	3260	1960	2950	1750	2630
10	2330	3500	2120	3180	1910	2880	1710	2560
11	2270	3400	2060	3100	1860	2800	1660	2490
12	2200	3310	2000	3010	1810	2720	1610	2420
13	2130	3210	1940	2920	1750	2630	1560	2340
14	2060	3100	1870	2820	1690	2540	1500	2260
15	1990	2990	1810	2720	1630	2450	1450	2170
16	1910	2880	1740	2610	1560	2350	1390	2090
17	1840	2760	1670	2500	1500	2250	1330	2000
18	1760	2640	1590	2390	1430	2150	1270	1910
18	1680	2520	1520	2280	1370	2050	1210	1820
20	1600	2400	1450	2170	1300	1950	1150	1730
22	1440	2160	1300	1950	1160	1750	1030	1540
24	1280	1930	1160	1740	1030	1550	910	1370
26	1130	1700	1020	1530	908	1360	797	1200
28	988	1480	885	1330	788	1180	690	1040
30	860	1290	771	1160	686	1030	601	904
32	756	1140	678	1020	603	906	528	794
34	670	1010	600	902	534	803	468	704
36	597	898	535	805	476	716	418	628
38	536	806	481	722	428	643	375	563
40	484	727	434	652	386	580	338	508
Properties								
P_{wo}, kips	746	1120	639	959	535	803	449	674
P_{wi}, kip/in.	55.9	83.9	51.1	76.7	45.9	68.9	41.6	62.4
P_{wb}, kips	4340	6530	3340	5020	2420	3640	1800	2710
P_{fb}, kips	1040	1570	878	1320	737	1110	592	890
L_p, ft	10.3		10.2		10.1		9.98	
L_r, ft	80.7		73.9		67.4		60.7	
A_g, in.^{2}	67.7		61.8		56.0		50.0	
I_x, in.^{4}	2420		2140		1890		1650	
I_y, in.^{4}	742		664		589		517	
r_y, in.	3.31		3.28		3.25		3.22	
r_x/r_y	1.80		1.80		1.79		1.78	
$P_{ex}L_c^2/10^4$, k-in.^{2}	69300		61300		54100		47200	
$P_{ey}L_c^2/10^4$, k-in.^{2}	21200		19000		16900		14800	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying, as discussed in Part 2.

F_y = 65 ksi

Table 4-1b (continued)
Available Strength in Axial Compression, kips
W-Shapes

W12

Shape		W12×							
lb/ft		152		136		120		106	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	1740	2610	1550	2330	1370	2060	1210	1830
	6	1660	2490	1480	2220	1300	1960	1150	1730
	7	1630	2450	1450	2180	1280	1920	1130	1700
	8	1600	2400	1420	2140	1250	1880	1110	1670
	9	1560	2350	1390	2090	1220	1840	1080	1630
	10	1520	2290	1350	2040	1190	1790	1050	1580
	11	1480	2220	1320	1980	1160	1740	1020	1540
	12	1430	2150	1270	1920	1120	1680	990	1490
	13	1390	2080	1230	1850	1080	1630	956	1440
	14	1340	2010	1190	1780	1040	1570	920	1380
	15	1290	1930	1140	1710	1000	1500	883	1330
	16	1230	1850	1090	1640	958	1440	845	1270
	17	1180	1770	1050	1570	915	1380	807	1210
	18	1130	1690	996	1500	871	1310	768	1150
	19	1070	1610	947	1420	827	1240	729	1100
	20	1020	1530	898	1350	783	1180	689	1040
	22	907	1360	800	1200	697	1050	612	920
	24	802	1210	705	1060	613	921	537	808
	26	701	1050	615	924	532	800	466	700
	28	606	910	530	797	459	690	402	604
	30	528	793	462	695	400	601	350	526
	32	464	697	406	610	352	528	308	462
	34	411	617	360	541	311	468	272	410
	36	366	551	321	482	278	417	243	365
	38	329	494	288	433	249	375	218	328
	40	297	446	260	391	225	338	197	296
Properties									
P_{wo}, kips		377	566	317	475	262	392	210	315
P_{wi}, kip/in.		37.7	56.6	34.2	51.4	30.8	46.2	26.4	39.7
P_{wb}, kips		1330	2000	1000	1500	726	1090	462	694
P_{fb}, kips		477	717	380	571	300	450	238	358
L_p, ft		9.88		9.79		9.70		9.63	
L_r, ft		54.8		49.1		44.2		39.9	
A_g, in.^{2}		44.7		39.9		35.2		31.2	
I_x, in.^{4}		1430		1240		1070		933	
I_y, in.^{4}		454		398		345		301	
r_y, in.		3.19		3.16		3.13		3.11	
r_x/r_y		1.77		1.77		1.76		1.76	
$P_{ex}L_c^2/10^4$, k-in.^{2}		40900		35500		30600		26700	
$P_{ey}L_c^2/10^4$, k-in.^{2}		13000		11400		9870		8620	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Confirm ASTM A913 material availability before specifying, as discussed in Part 2.

Table 4-1b (continued)
Available Strength in Axial Compression, kips
W-Shapes

W12

F_y = 65 ksi

Shape		W12×									
lb/ft		96		87		79		72		65	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	1100	1650	996	1500	903	1360	821	1230	743	1120
	6	1040	1570	946	1420	856	1290	779	1170	704	1060
	7	1020	1540	928	1390	840	1260	764	1150	691	1040
	8	1000	1510	908	1360	822	1240	747	1120	675	1020
	9	977	1470	886	1330	802	1200	728	1090	658	989
	10	951	1430	862	1300	779	1170	708	1060	640	962
	11	923	1390	836	1260	756	1140	687	1030	620	932
	12	893	1340	808	1210	731	1100	664	997	599	900
	13	861	1290	780	1170	704	1060	639	961	577	867
	14	829	1250	750	1130	677	1020	614	923	554	833
	15	795	1190	719	1080	648	975	589	885	530	797
	16	760	1140	687	1030	620	931	562	845	506	761
	17	725	1090	655	984	590	887	535	805	482	724
	18	690	1040	622	936	561	843	508	764	457	687
	19	654	983	590	887	531	798	481	723	432	650
	20	619	930	557	838	501	753	454	683	408	613
	22	548	824	493	742	443	666	401	603	360	540
	24	481	722	432	649	387	582	350	526	313	471
	26	416	625	373	560	333	501	301	453	269	404
	28	358	539	321	483	287	432	260	390	232	349
	30	312	469	280	421	250	376	226	340	202	304
	32	274	413	246	370	220	331	199	299	178	267
	34	243	365	218	327	195	293	176	265	157	236
	36	217	326	194	292	174	261	157	236	140	211
	38	195	293	174	262	156	234	141	212	126	189
	40	176	264	157	237	141	212	127	191	114	171
Properties											
P_{wo}, kips		179	268	157	236	135	203	118	177	101	152
P_{wi}, kip/in.		23.8	35.8	22.3	33.5	20.4	30.6	18.6	28.0	16.9	25.4
P_{wb}, kips		337	507	277	416	211	316	161	243	121	181
P_{fb}, kips		197	296	160	240	131	198	109	164	89.0	134
L_p, ft		9.57		9.51		9.78		11.0		12.2	
L_r, ft		37.0		34.4		32.1		30.4		28.8	
A_g, in.^{2}		28.2		25.6		23.2		21.1		19.1	
I_x, in.^{4}		833		740		662		597		533	
I_y, in.^{4}		270		241		216		195		174	
r_y, in.		3.09		3.07		3.05		3.04		3.02	
r_x/r_y		1.76		1.75		1.75		1.75		1.75	
$P_{ex}L_c^2/10^4$, k-in.^{2}		23800		21200		18900		17100		15300	
$P_{ey}L_c^2/10^4$, k-in.^{2}		7730		6900		6180		5580		4980	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Confirm ASTM A913 material availability before specifying, as discussed in Part 2.

F_y = 70 ksi

Table 4-1c
Available Strength in Axial Compression, kips
W-Shapes

W14

Shape		W14×							
lb/ft		873[h]		808[h]		730[h]		665[h]	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	10800	16200	9980	15000	9010	13500	8220	12300
	11	10000	15000	9240	13900	8310	12500	7560	11400
	12	9860	14800	9110	13700	8180	12300	7440	11200
	13	9710	14600	8970	13500	8050	12100	7310	11000
	14	9550	14400	8810	13200	7900	11900	7180	10800
	15	9380	14100	8650	13000	7750	11600	7030	10600
	16	9210	13800	8490	12800	7590	11400	6880	10300
	17	9020	13600	8310	12500	7430	11200	6730	10100
	18	8830	13300	8130	12200	7250	10900	6570	9870
	19	8630	13000	7940	11900	7080	10600	6400	9620
	20	8430	12700	7750	11600	6890	10400	6230	9370
	22	8000	12000	7350	11000	6520	9790	5880	8840
	24	7560	11400	6930	10400	6130	9210	5520	8300
	26	7110	10700	6510	9780	5730	8610	5150	7740
	28	6660	10000	6080	9140	5330	8010	4780	7190
	30	6200	9320	5650	8490	4930	7410	4410	6630
	32	5740	8630	5220	7850	4540	6820	4050	6090
	34	5300	7960	4810	7220	4150	6240	3700	5560
	36	4860	7310	4400	6610	3780	5680	3360	5050
	38	4440	6670	4010	6020	3420	5140	3020	4550
	40	4030	6050	3620	5440	3090	4640	2730	4100
	42	3650	5490	3290	4940	2800	4210	2480	3720
	44	3330	5000	2990	4500	2550	3830	2260	3390
	46	3040	4570	2740	4120	2330	3510	2060	3100
	48	2800	4200	2520	3780	2140	3220	1900	2850
	50	2580	3870	2320	3480	1970	2970	1750	2630

Properties

	873 ASD	873 LRFD	808 ASD	808 LRFD	730 ASD	730 LRFD	665 ASD	665 LRFD
P_{wo}, kips	5610	8410	4980	7470	3950	5920	3380	5070
P_{wi}, kip/in.	184	276	175	262	143	215	132	198
P_{wb}, kips	110000	165000	94200	142000	52000	78200	40700	61200
P_{fb}, kips	7950	12000	6870	10300	6320	9490	5350	8040
L_p, ft	14.6		14.4		14.0		13.8	
L_r, ft	235		221		197		181	
A_g, in.^{2}	257		238		215		196	
I_x, in.^{4}	18100		15900		14300		12400	
I_y, in.^{4}	6170		5550		4720		4170	
r_y, in.	4.90		4.83		4.69		4.62	
r_x/r_y	1.71		1.69		1.74		1.73	
$P_{ex}L_c^2/10^4$, k-in.^{2}	518000		455000		409000		355000	
$P_{ey}L_c^2/10^4$, k-in.^{2}	177000		159000		135000		119000	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying, as discussed in Part 2.

W14

Table 4-1c (continued)
Available Strength in Axial Compression, kips
W-Shapes

F_y = 70 ksi

Shape		W14×							
lb/ft		605[h]		550[h]		500[h]		455[h]	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	7460	11200	6790	10200	6160	9260	5620	8440
	11	6850	10300	6220	9340	5630	8460	5120	7690
	12	6730	10100	6110	9190	5530	8310	5030	7560
	13	6620	9940	6000	9020	5430	8160	4930	7410
	14	6490	9750	5880	8840	5320	7990	4830	7260
	15	6360	9550	5760	8660	5200	7820	4730	7100
	16	6220	9350	5630	8460	5080	7640	4610	6930
	17	6070	9130	5500	8260	4960	7450	4500	6760
	18	5920	8900	5360	8050	4830	7260	4380	6580
	19	5770	8670	5220	7840	4700	7060	4260	6400
	20	5610	8430	5070	7620	4560	6860	4130	6210
	22	5290	7950	4770	7160	4280	6440	3870	5820
	24	4950	7440	4460	6700	4000	6010	3610	5420
	26	4610	6930	4140	6230	3710	5570	3340	5020
	28	4270	6420	3830	5750	3420	5140	3080	4620
	30	3930	5910	3520	5290	3130	4710	2810	4230
	32	3600	5410	3210	4830	2860	4290	2560	3840
	34	3280	4920	2920	4380	2590	3890	2310	3470
	36	2970	4460	2630	3950	2320	3490	2070	3110
	38	2660	4000	2360	3550	2090	3130	1860	2790
	40	2400	3610	2130	3200	1880	2830	1680	2520
	42	2180	3280	1930	2900	1710	2570	1520	2290
	44	1990	2990	1760	2650	1560	2340	1390	2080
	46	1820	2730	1610	2420	1420	2140	1270	1910
	48	1670	2510	1480	2220	1310	1960	1160	1750
	50	1540	2310	1360	2050	1200	1810	1070	1610

Properties

	605[h] ASD	605[h] LRFD	550[h] ASD	550[h] LRFD	500[h] ASD	500[h] LRFD	455[h] ASD	455[h] LRFD
P_{wo}, kips	2890	4330	2450	3680	2100	3140	1800	2690
P_{wi}, kip/in.	121	182	111	167	102	153	94.3	141
P_{wb}, kips	31500	47400	24200	36400	18800	28300	14800	22200
P_{fb}, kips	4530	6810	3820	5750	3210	4820	2700	4060
L_p, ft	13.6		13.4		13.2		13.1	
L_r, ft	166		153		140		128	
A_g, in.^{2}	178		162		147		134	
I_x, in.^{4}	10800		9430		8210		7190	
I_y, in.^{4}	3680		3250		2880		2560	
r_y, in.	4.55		4.49		4.43		4.38	
r_x/r_y	1.71		1.70		1.69		1.67	
$P_{ex}L_c^2/10^4$, k-in.^{2}	309000		270000		235000		206000	
$P_{ey}L_c^2/10^4$, k-in.^{2}	105000		93000		82400		73300	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying, as discussed in Part 2.

F_y = 70 ksi

Table 4-1c (continued)
Available Strength in Axial Compression, kips
W-Shapes

W14

Shape	W14×											
lb/ft	426[h]		398[h]		370[h]		342[h]		311[h]		283[h]	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y												
0	5240	7870	4900	7370	4570	6870	4230	6360	3830	5760	3490	5250
11	4770	7160	4460	6700	4140	6230	3830	5760	3460	5200	3150	4740
12	4680	7040	4370	6580	4070	6110	3760	5650	3400	5110	3090	4640
13	4590	6900	4290	6450	3990	5990	3690	5540	3330	5000	3030	4550
14	4490	6760	4200	6310	3900	5860	3610	5420	3250	4890	2960	4440
15	4390	6600	4100	6170	3810	5720	3520	5290	3170	4770	2890	4340
16	4290	6450	4000	6020	3710	5580	3430	5160	3090	4650	2810	4220
17	4180	6280	3900	5860	3620	5440	3340	5020	3010	4520	2730	4110
18	4070	6110	3790	5700	3520	5280	3250	4880	2920	4390	2650	3990
19	3950	5940	3680	5540	3410	5130	3150	4730	2830	4260	2570	3860
20	3830	5760	3570	5370	3310	4970	3050	4580	2740	4120	2490	3740
22	3590	5390	3340	5020	3090	4640	2850	4280	2560	3840	2320	3480
24	3340	5020	3110	4670	2870	4310	2640	3970	2370	3560	2140	3220
26	3090	4640	2870	4310	2650	3980	2430	3660	2180	3270	1970	2960
28	2840	4260	2630	3960	2420	3640	2230	3350	1990	2990	1800	2700
30	2590	3890	2400	3610	2210	3320	2020	3040	1810	2710	1630	2450
32	2350	3530	2180	3270	2000	3000	1830	2750	1630	2450	1470	2200
34	2120	3190	1960	2950	1790	2700	1640	2460	1460	2190	1310	1970
36	1900	2850	1750	2630	1600	2410	1460	2200	1300	1950	1170	1750
38	1700	2560	1570	2360	1440	2160	1310	1970	1170	1750	1050	1570
40	1540	2310	1420	2130	1300	1950	1180	1780	1050	1580	945	1420
42	1390	2090	1290	1930	1180	1770	1070	1610	954	1430	857	1290
44	1270	1910	1170	1760	1070	1610	979	1470	869	1310	781	1170
46	1160	1750	1070	1610	980	1470	896	1350	795	1200	715	1070
48	1070	1600	985	1480	900	1350	823	1240	730	1100	656	986
50	983	1480	907	1360	830	1250	758	1140	673	1010	605	909
Properties												
P_{wo}, kips	1590	2390	1420	2130	1260	1890	1100	1650	941	1410	804	1210
P_{wi}, kip/in.	87.7	132	82.6	124	77.5	116	71.9	108	65.8	98.7	60.2	90.3
P_{wb}, kips	11900	17900	9960	15000	8190	12300	6550	9850	5030	7560	3850	5790
P_{fb}, kips	2420	3640	2130	3200	1850	2790	1600	2400	1340	2010	1120	1690
L_p, ft	13.0		12.9		12.7		12.7		12.5		12.4	
L_r, ft	120		113		106		98.7		89.9		82.1	
A_g, in.^{2}	125		117		109		101		91.4		83.3	
I_x, in.^{4}	6600		6000		5440		4900		4330		3840	
I_y, in.^{4}	2360		2170		1990		1810		1610		1440	
r_y, in.	4.34		4.31		4.27		4.24		4.20		4.17	
r_x/r_y	1.67		1.66		1.66		1.65		1.64		1.63	
$P_{ex}L_c^2/10^4$, k-in.^{2}	189000		172000		156000		140000		124000		110000	
$P_{ey}L_c^2/10^4$, k-in.^{2}	67500		62100		57000		51800		46100		41200	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying, as discussed in Part 2.

W14

Table 4-1c (continued)
Available Strength in Axial Compression, kips
W-Shapes

F_y = 70 ksi

Shape	W14×											
lb/ft	257		233		211		193		176		159	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y												
0	3170	4760	2870	4320	2600	3910	2380	3580	2170	3260	1960	2940
6	3070	4620	2780	4180	2520	3780	2310	3460	2100	3160	1890	2850
7	3040	4570	2750	4130	2490	3740	2280	3420	2080	3120	1870	2810
8	3000	4510	2710	4080	2450	3690	2250	3380	2050	3080	1850	2770
9	2950	4440	2670	4020	2420	3630	2210	3330	2020	3030	1820	2730
10	2910	4370	2630	3950	2380	3570	2180	3270	1980	2980	1790	2680
11	2850	4290	2580	3880	2330	3510	2140	3210	1940	2920	1750	2630
12	2800	4210	2530	3800	2290	3440	2090	3140	1900	2860	1710	2580
13	2740	4120	2480	3720	2240	3360	2050	3070	1860	2800	1680	2520
14	2680	4020	2420	3630	2180	3280	2000	3000	1820	2730	1630	2460
15	2610	3920	2360	3540	2130	3200	1940	2920	1770	2660	1590	2390
16	2540	3820	2290	3450	2070	3110	1890	2840	1720	2580	1550	2320
17	2470	3710	2230	3350	2010	3020	1840	2760	1670	2510	1500	2250
18	2390	3600	2160	3250	1950	2930	1780	2670	1620	2430	1450	2180
19	2320	3490	2090	3140	1880	2830	1720	2590	1560	2350	1400	2110
20	2240	3370	2020	3040	1820	2740	1660	2500	1510	2270	1350	2040
22	2090	3130	1880	2820	1690	2540	1540	2320	1400	2100	1250	1880
24	1930	2900	1730	2600	1560	2340	1420	2130	1280	1930	1150	1730
26	1770	2660	1590	2390	1420	2140	1300	1950	1170	1760	1050	1580
28	1610	2420	1440	2170	1290	1940	1180	1770	1060	1600	951	1430
30	1460	2190	1300	1960	1170	1750	1060	1590	955	1440	854	1280
32	1310	1970	1170	1760	1040	1570	949	1430	853	1280	762	1140
34	1160	1750	1040	1560	927	1390	841	1260	756	1140	675	1010
36	1040	1560	927	1390	827	1240	750	1130	674	1010	602	905
38	932	1400	832	1250	742	1120	673	1010	605	909	540	812
40	841	1260	751	1130	670	1010	608	914	546	821	487	733

Properties

Property	257 ASD	257 LRFD	233 ASD	233 LRFD	211 ASD	211 LRFD	193 ASD	193 LRFD	176 ASD	176 LRFD	159 ASD	159 LRFD
P_{wo}, kips	686	1030	579	869	494	741	424	635	370	555	311	467
P_{wi}, kip/in.	55.1	82.6	49.9	74.9	45.7	68.6	41.5	62.3	38.7	58.1	34.8	52.2
P_{wb}, kips	2940	4410	2190	3290	1700	2550	1270	1900	1030	1550	743	1120
P_{fb}, kips	936	1410	775	1160	638	958	543	816	450	676	371	558

Property	257	233	211	193	176	159
L_p, ft	12.3	12.2	12.1	12.1	12.0	11.9
L_r, ft	75.1	68.4	62.6	57.6	53.4	49.0
A_g, in.^{2}	75.6	68.5	62.0	56.8	51.8	46.7
I_x, in.^{4}	3400	3010	2660	2400	2140	1900
I_y, in.^{4}	1290	1150	1030	931	838	748
r_y, in.	4.13	4.10	4.07	4.05	4.02	4.00
r_x/r_y	1.62	1.62	1.61	1.60	1.60	1.60
$P_{ex}L_c^2/10^4$, k-in.^{2}	97300	86200	76100	68700	61300	54400
$P_{ey}L_c^2/10^4$, k-in.^{2}	36900	32900	29500	26600	24000	21400

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Confirm ASTM A913 material availability before specifying, as discussed in Part 2.

F_y = 70 ksi

Table 4-1c (continued)
Available Strength in Axial Compression, kips
W-Shapes

W14

Shape	W14×											
lb/ft	145		132		120		109		99		90	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y												
0	1790	2690	1630	2440	1480	2220	1340	2020	1220	1830	1110	1670
6	1730	2600	1570	2350	1420	2140	1290	1940	1170	1760	1070	1610
7	1710	2570	1550	2320	1410	2110	1270	1910	1160	1740	1050	1580
8	1690	2530	1520	2290	1380	2080	1250	1880	1140	1710	1040	1560
9	1660	2490	1490	2250	1360	2040	1230	1850	1120	1680	1020	1530
10	1630	2450	1470	2200	1330	2000	1210	1810	1100	1650	997	1500
11	1600	2400	1430	2150	1300	1960	1180	1770	1070	1610	975	1470
12	1570	2350	1400	2100	1270	1910	1150	1730	1050	1570	951	1430
13	1530	2300	1360	2050	1240	1860	1120	1690	1020	1530	926	1390
14	1490	2240	1330	1990	1200	1810	1090	1640	989	1490	899	1350
15	1450	2180	1290	1930	1170	1750	1060	1590	959	1440	872	1310
16	1410	2120	1250	1870	1130	1700	1020	1540	927	1390	843	1270
17	1370	2060	1200	1810	1090	1640	988	1480	895	1350	814	1220
18	1320	1990	1160	1740	1050	1580	952	1430	862	1300	784	1180
19	1280	1920	1120	1680	1010	1520	915	1380	829	1250	753	1130
20	1230	1850	1070	1610	971	1460	878	1320	795	1190	722	1090
22	1140	1710	982	1480	888	1340	803	1210	726	1090	660	991
24	1050	1570	892	1340	806	1210	729	1100	658	989	597	898
26	954	1430	804	1210	726	1090	655	985	591	889	536	806
28	863	1300	718	1080	648	973	585	879	527	792	478	718
30	775	1160	636	956	573	861	516	776	465	698	421	632
32	689	1040	559	840	503	756	454	682	408	614	370	556
34	611	918	495	744	446	670	402	604	362	544	328	492
36	545	819	442	664	398	598	359	539	323	485	292	439
38	489	735	397	596	357	536	322	484	290	435	262	394
40	441	663	358	538	322	484	290	437	261	393	237	356
Properties												
P_{wo}, kips	268	402	245	368	212	318	179	268	156	234	134	202
P_{wi}, kip/in.	31.7	47.6	30.1	45.2	27.5	41.3	24.5	36.8	22.6	34.0	20.5	30.8
P_{wb}, kips	564	847	481	723	369	555	260	391	205	308	153	230
P_{fb}, kips	311	468	278	418	231	348	194	291	159	240	132	198
L_p, ft	11.9		11.2		11.3		12.7		14.1		15.3	
L_r, ft	45.7		41.6		39.0		36.8		34.8		33.0	
A_g, in.^{2}	42.7		38.8		35.3		32.0		29.1		26.5	
I_x, in.^{4}	1710		1530		1380		1240		1110		999	
I_y, in.^{4}	677		548		495		447		402		362	
r_y, in.	3.98		3.76		3.74		3.73		3.71		3.70	
r_x/r_y	1.59		1.67		1.67		1.67		1.66		1.66	
$P_{ex}L_c^2/10^4$, k-in.^{2}	48900		43800		39500		35500		31800		28600	
$P_{ey}L_c^2/10^4$, k-in.^{2}	19400		15700		14200		12800		11500		10400	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Confirm ASTM A913 material availability before specifying, as discussed in Part 2.

W12

Table 4-1c (continued)
Available Strength in Axial Compression, kips
W-Shapes

F_y = 70 ksi

Shape		W12×							
lb/ft		230[h]		210		190		170	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	2840	4270	2590	3890	2350	3530	2100	3150
	6	2700	4060	2470	3710	2230	3360	1990	2990
	7	2660	3990	2420	3640	2190	3290	1950	2940
	8	2600	3910	2370	3570	2150	3230	1910	2880
	9	2540	3820	2320	3480	2100	3150	1870	2810
	10	2480	3730	2260	3390	2040	3070	1820	2730
	11	2410	3620	2190	3300	1980	2980	1760	2650
	12	2340	3510	2130	3200	1920	2890	1710	2570
	13	2260	3400	2050	3090	1850	2790	1650	2480
	14	2180	3280	1980	2980	1790	2680	1590	2380
	15	2100	3150	1900	2860	1710	2580	1520	2290
	16	2010	3020	1820	2740	1640	2470	1460	2190
	17	1920	2890	1740	2620	1570	2360	1390	2090
	18	1840	2760	1660	2500	1490	2240	1320	1990
	19	1750	2620	1580	2370	1420	2130	1250	1890
	20	1660	2490	1500	2250	1340	2020	1190	1780
	22	1480	2220	1330	2010	1190	1800	1050	1580
	24	1310	1970	1180	1770	1050	1580	924	1390
	26	1140	1720	1030	1540	913	1370	800	1200
	28	988	1480	885	1330	788	1180	690	1040
	30	860	1290	771	1160	686	1030	601	904
	32	756	1140	678	1020	603	906	528	794
	34	670	1010	600	902	534	803	468	704
	36	597	898	535	805	476	716	418	628
	38	536	806	481	722	428	643	375	563
	40	484	727	434	652	386	580	338	508
Properties									
P_{wo}, kips		804	1210	688	1030	576	864	484	726
P_{wi}, kip/in.		60.2	90.3	55.1	82.6	49.5	74.2	44.8	67.2
P_{wb}, kips		4510	6770	3460	5210	2510	3780	1870	2810
P_{fb}, kips		1120	1690	946	1420	793	1190	638	958
L_p, ft		9.88		9.79		9.70		9.61	
L_r, ft		75.0		68.7		62.7		56.5	
A_g, in.^{2}		67.7		61.8		56.0		50.0	
I_x, in.^{4}		2420		2140		1890		1650	
I_y, in.^{4}		742		664		589		517	
r_y, in.		3.31		3.28		3.25		3.22	
r_x/r_y		1.80		1.80		1.79		1.78	
$P_{ex}L_c^2/10^4$, k-in.^{2}		69300		61300		54100		47200	
$P_{ey}L_c^2/10^4$, k-in.^{2}		21200		19000		16900		14800	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Confirm ASTM A913 material availability before specifying, as discussed in Part 2.

F_y = 70 ksi

Table 4-1c (continued)
Available Strength in Axial Compression, kips
W-Shapes

W12

Shape		W12×							
lb/ft		152		136		120		106	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	1870	2820	1670	2510	1480	2220	1310	1970
	6	1780	2670	1590	2380	1400	2100	1240	1860
	7	1750	2620	1560	2340	1370	2060	1210	1820
	8	1710	2570	1520	2290	1340	2010	1190	1780
	9	1670	2500	1480	2230	1310	1960	1160	1740
	10	1620	2440	1440	2170	1270	1910	1120	1690
	11	1570	2360	1400	2100	1230	1850	1090	1630
	12	1520	2290	1350	2030	1190	1790	1050	1580
	13	1470	2200	1300	1960	1140	1720	1010	1520
	14	1410	2120	1250	1880	1100	1650	970	1460
	15	1350	2030	1200	1800	1050	1580	928	1390
	16	1290	1940	1150	1720	1000	1510	885	1330
	17	1230	1850	1090	1640	955	1440	842	1270
	18	1170	1760	1040	1560	906	1360	798	1200
	19	1110	1670	982	1480	857	1290	754	1130
	20	1050	1580	927	1390	808	1210	711	1070
	22	929	1400	819	1230	712	1070	625	940
	24	813	1220	715	1070	620	932	544	817
	26	702	1060	615	925	532	800	466	700
	28	606	910	530	797	459	690	402	604
	30	528	793	462	695	400	601	350	526
	32	464	697	406	610	352	528	308	462
	34	411	617	360	541	311	468	272	410
	36	366	551	321	482	278	417	243	365
	38	329	494	288	433	249	375	218	328
	40	297	446	260	391	225	338	197	296
Properties									
P_{wo}, kips		406	609	341	512	282	422	226	339
P_{wi}, kip/in.		40.6	60.9	36.9	55.3	33.1	49.7	28.5	42.7
P_{wb}, kips		1380	2080	1040	1560	753	1130	479	720
P_{fb}, kips		513	772	409	615	323	485	257	386
L_p, ft		9.52		9.43		9.34		9.28	
L_r, ft		51.0		45.8		41.3		37.3	
A_g, in.^{2}		44.7		39.9		35.2		31.2	
I_x, in.^{4}		1430		1240		1070		933	
I_y, in.^{4}		454		398		345		301	
r_y, in.		3.19		3.16		3.13		3.11	
r_x/r_y		1.77		1.77		1.76		1.76	
$P_{ex}L_c^2/10^4$, k-in.^{2}		40900		35500		30600		26700	
$P_{ey}L_c^2/10^4$, k-in.^{2}		13000		11400		9870		8620	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Confirm ASTM A913 material availability before specifying, as discussed in Part 2.

Table 4-1c (continued)
Available Strength in Axial Compression, kips
W-Shapes

W12

F_y = 70 ksi

Shape		W12×									
lb/ft		96		87		79		72		65	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	1180	1780	1070	1610	972	1460	884	1330	801	1200
	6	1120	1680	1010	1520	919	1380	835	1260	755	1140
	7	1100	1650	994	1490	900	1350	818	1230	740	1110
	8	1070	1610	971	1460	879	1320	799	1200	722	1090
	9	1040	1570	945	1420	855	1290	777	1170	702	1060
	10	1010	1520	918	1380	830	1250	754	1130	681	1020
	11	981	1470	888	1330	803	1210	729	1100	658	990
	12	946	1420	857	1290	774	1160	703	1060	634	953
	13	911	1370	824	1240	744	1120	675	1020	609	916
	14	873	1310	790	1190	713	1070	647	972	583	877
	15	835	1260	755	1130	681	1020	618	928	557	836
	16	796	1200	719	1080	648	974	588	884	529	796
	17	757	1140	683	1030	615	925	558	838	502	754
	18	717	1080	646	972	582	875	528	793	474	713
	19	677	1020	610	917	549	825	497	747	447	671
	20	637	958	574	863	516	775	467	702	419	630
	22	560	842	503	757	452	679	409	614	366	550
	24	486	730	436	655	390	587	353	530	316	474
	26	416	625	373	560	333	501	301	453	269	404
	28	358	539	321	483	287	432	260	390	232	349
	30	312	469	280	421	250	376	226	340	202	304
	32	274	413	246	370	220	331	199	299	178	267
	34	243	365	218	327	195	293	176	265	157	236
	36	217	326	194	292	174	261	157	236	140	211
	38	195	293	174	262	156	234	141	212	126	189
	40	176	264	157	237	141	212	127	191	114	171

Properties

Property	96 ASD	96 LRFD	87 ASD	87 LRFD	79 ASD	79 LRFD	72 ASD	72 LRFD	65 ASD	65 LRFD
P_{wo}, kips	193	289	169	254	146	219	127	191	109	164
P_{wi}, kip/in.	25.7	38.5	24.0	36.1	21.9	32.9	20.1	30.1	18.2	27.3
P_{wb}, kips	350	526	287	432	219	328	168	252	125	188
P_{fb}, kips	212	319	172	258	142	213	118	177	95.9	144

Property	96	87	79	72	65
L_p, ft	9.22	9.16	9.92	11.0	12.2
L_r, ft	34.7	32.3	30.3	28.8	27.3
A_g, in.^{2}	28.2	25.6	23.2	21.1	19.1
I_x, in.^{4}	833	740	662	597	533
I_y, in.^{4}	270	241	216	195	174
r_y, in.	3.09	3.07	3.05	3.04	3.02
r_x/r_y	1.76	1.75	1.75	1.75	1.75
$P_{ex}L_c^2/10^4$, k-in.^{2}	23800	21200	18900	17100	15300
$P_{ey}L_c^2/10^4$, k-in.^{2}	7730	6900	6180	5580	4980

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Confirm ASTM A913 material availability before specifying, as discussed in Part 2.

F_y = 50 ksi

Table 4-2
Available Strength in Axial Compression, kips
HP-Shapes

HP18

Shape		HP18×							
lb/ft		204		181		157		135	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	1800	2710	1590	2390	1380	2080	1190	1800
	6	1770	2650	1560	2340	1350	2040	1170	1760
	7	1750	2630	1550	2330	1340	2020	1160	1740
	8	1740	2610	1540	2310	1330	2000	1150	1730
	9	1720	2590	1520	2290	1320	1980	1140	1710
	10	1700	2560	1500	2260	1300	1960	1130	1690
	11	1680	2530	1490	2230	1290	1940	1110	1670
	12	1660	2500	1470	2200	1270	1910	1100	1650
	13	1640	2460	1450	2170	1250	1880	1080	1620
	14	1610	2420	1420	2140	1230	1850	1060	1600
	15	1590	2380	1400	2100	1210	1820	1050	1570
	16	1560	2340	1370	2070	1190	1790	1030	1540
	17	1530	2300	1350	2030	1170	1760	1010	1510
	18	1500	2250	1320	1990	1150	1720	985	1480
	19	1470	2210	1290	1950	1120	1680	964	1450
	20	1440	2160	1270	1900	1100	1650	942	1420
	22	1370	2060	1210	1810	1040	1570	896	1350
	24	1300	1950	1140	1720	989	1490	848	1280
	26	1230	1850	1080	1620	933	1400	800	1200
	28	1160	1740	1010	1530	876	1320	750	1130
	30	1080	1630	950	1430	819	1230	700	1050
	32	1010	1520	884	1330	761	1140	650	977
	34	936	1410	820	1230	705	1060	601	904
	36	865	1300	756	1140	650	977	553	831
	38	795	1190	695	1040	596	896	507	761
	40	728	1090	635	954	544	818	461	693
Properties									
P_{wo}, kips		435	653	363	545	297	446	241	362
P_{wi}, kip/in.		37.7	56.5	33.3	50.0	29.0	43.5	25.0	37.5
P_{wb}, kips		1830	2740	1270	1910	840	1260	535	804
P_{fb}, kips		239	359	187	281	142	213	105	158
L_p, ft		15.2		15.1		18.1		21.4	
L_r, ft		67.8		61.3		55.8		50.5	
A_g, in.^{2}		60.2		53.2		46.2		39.9	
I_x, in.^{4}		3480		3020		2570		2200	
I_y, in.^{4}		1120		974		833		706	
r_y, in.		4.31		4.28		4.25		4.21	
r_x/r_y		1.76		1.76		1.75		1.76	
$P_{ex}L_c^2/10^4$, k-in.^{2}		99600		86400		73600		63000	
$P_{ey}L_c^2/10^4$, k-in.^{2}		32100		27900		23800		20200	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Table 4-2 (continued)
Available Strength in Axial Compression, kips
HP-Shapes

HP16

F_y = 50 ksi

Shape	HP16×											
lb/ft	183		162		141		121		101		88[c]	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y												
0	1620	2430	1430	2150	1250	1880	1070	1610	895	1350	753	1130
6	1580	2370	1390	2090	1220	1830	1040	1570	871	1310	736	1110
7	1570	2350	1380	2070	1200	1810	1030	1550	862	1300	730	1100
8	1550	2330	1360	2050	1190	1790	1020	1540	852	1280	723	1090
9	1530	2300	1350	2020	1180	1770	1010	1520	841	1260	715	1070
10	1510	2270	1330	2000	1160	1740	995	1490	829	1250	706	1060
11	1490	2240	1310	1970	1140	1720	979	1470	816	1230	697	1050
12	1470	2200	1290	1930	1120	1690	962	1450	802	1210	687	1030
13	1440	2160	1260	1900	1100	1660	944	1420	787	1180	676	1020
14	1410	2120	1240	1860	1080	1630	926	1390	771	1160	663	997
15	1390	2080	1210	1820	1060	1590	906	1360	754	1130	648	975
16	1360	2040	1190	1780	1030	1560	885	1330	736	1110	633	951
17	1320	1990	1160	1740	1010	1520	863	1300	718	1080	617	927
18	1290	1940	1130	1700	985	1480	841	1260	699	1050	600	902
19	1260	1890	1100	1650	958	1440	818	1230	679	1020	583	877
20	1230	1840	1070	1610	931	1400	794	1190	659	991	566	851
22	1160	1740	1010	1510	876	1320	746	1120	618	929	530	797
24	1080	1630	942	1420	819	1230	696	1050	576	866	494	742
26	1010	1520	877	1320	761	1140	646	971	534	802	457	686
28	939	1410	811	1220	703	1060	596	896	491	739	420	631
30	866	1300	746	1120	645	970	546	821	450	676	384	577
32	794	1190	682	1030	589	886	498	748	409	615	348	524
34	725	1090	620	932	535	804	451	678	370	556	314	473
36	657	988	561	843	482	725	405	609	331	498	281	423
38	592	889	503	756	433	651	364	547	297	447	253	380
40	534	803	454	682	391	587	328	494	268	404	228	343
Properties												
P_{wo}, kips	435	653	363	545	300	451	241	362	189	283	155	232
P_{wi}, kip/in.	37.7	56.5	33.3	50.0	29.2	43.8	25.0	37.5	20.8	31.3	18.0	27.0
P_{wb}, kips	2100	3160	1450	2190	974	1460	612	920	356	535	229	345
P_{fb}, kips	239	359	187	281	143	215	105	158	73.1	110	54.6	82.0
L_p, ft	13.6		13.5		13.4		16.7		20.2		22.9	
L_r, ft	67.6		60.2		54.5		48.6		43.6		40.6	
A_g, in.^{2}	54.1		47.7		41.7		35.8		29.9		25.8	
I_x, in.^{4}	2510		2190		1870		1590		1300		1110	
I_y, in.^{4}	818		697		599		504		412		349	
r_y, in.	3.89		3.82		3.79		3.75		3.71		3.68	
r_x/r_y	1.75		1.77		1.77		1.78		1.78		1.78	
$P_{ex}L_c^2/10^4$, k-in.^{2}	71800		62700		53500		45500		37200		31800	
$P_{ey}L_c^2/10^4$, k-in.^{2}	23400		19900		17100		14400		11800		9990	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 50 ksi; tabulated values have been adjusted accordingly.

F_y = 50 ksi

Table 4-2 (continued)
Available Strength in Axial Compression, kips
HP-Shapes

HP14–HP12

Shape	HP14×								HP12×			
lb/ft	117		102		89		73[c]		89		84	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
Effective length, L_c (ft), with respect to least radius of gyration, r_y	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	1030	1550	901	1350	781	1170	625	940	775	1170	737	1110
6	1000	1500	875	1310	758	1140	610	917	742	1120	705	1060
7	990	1490	865	1300	750	1130	604	908	731	1100	694	1040
8	977	1470	855	1280	740	1110	598	899	717	1080	681	1020
9	964	1450	843	1270	730	1100	591	888	703	1060	667	1000
10	949	1430	829	1250	718	1080	583	876	687	1030	652	980
11	933	1400	815	1220	705	1060	574	863	669	1010	636	955
12	916	1380	800	1200	692	1040	565	849	651	978	618	929
13	897	1350	783	1180	677	1020	554	832	631	949	599	901
14	878	1320	766	1150	662	995	541	813	611	918	580	872
15	857	1290	748	1120	646	971	527	793	590	886	560	842
16	836	1260	729	1100	629	946	514	772	568	853	539	810
17	813	1220	709	1070	612	920	499	750	545	820	518	779
18	790	1190	689	1030	594	893	484	728	523	785	496	746
19	767	1150	668	1000	576	866	469	705	500	751	474	713
20	743	1120	646	972	557	838	453	681	476	716	452	680
22	694	1040	603	906	519	780	422	634	430	646	408	614
24	643	967	558	839	480	722	389	585	384	578	365	549
26	593	891	514	772	441	663	357	537	340	512	323	486
28	543	816	470	706	403	606	325	489	298	448	283	425
30	494	742	427	641	365	549	294	442	260	390	247	371
32	446	671	385	579	329	494	264	397	228	343	217	326
34	400	602	344	518	294	441	235	354	202	304	192	289
36	357	537	307	462	262	394	210	316	180	271	171	257
38	320	482	276	414	235	353	188	283	162	243	154	231
40	289	435	249	374	212	319	170	256	146	220	139	208

Properties

Property	HP14×117		HP14×102		HP14×89		HP14×73		HP12×89		HP12×84	
P_{wo}, kips	201	302	162	243	134	201	100	150	158	238	158	236
P_{wi}, kip/in.	26.8	40.3	23.5	35.3	20.5	30.8	16.8	25.3	24.0	36.0	22.8	34.3
P_{wb}, kips	790	1190	531	798	354	532	195	294	660	991	572	859
P_{fb}, kips	121	182	93.0	140	70.8	106	47.7	71.7	97.0	146	87.8	132
L_p, ft	12.9		15.6		17.8		21.2		10.4		10.4	
L_r, ft	50.5		45.7		41.7		37.6		42.8		41.3	
A_g, in.^{2}	34.4		30.1		26.1		21.4		25.9		24.6	
I_x, in.^{4}	1220		1050		904		729		693		650	
I_y, in.^{4}	443		380		326		261		224		213	
r_y, in.	3.59		3.56		3.53		3.49		2.94		2.94	
r_x/r_y	1.66		1.66		1.67		1.67		1.76		1.75	
$P_{ex}L_c^2/10^4$, k-in.^{2}	34900		30100		25900		20900		19800		18600	
$P_{ey}L_c^2/10^4$, k-in.^{2}	12700		10900		9330		7470		6410		6100	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 50 ksi; tabulated values have been adjusted accordingly.

Table 4-2 (continued)
Available Strength in Axial Compression, kips
HP-Shapes

HP12–HP8

F_y = 50 ksi

Shape	HP12×						HP10×				HP8×	
lb/ft	74		63		53[c]		57		42		36	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y												
0	653	981	551	828	460	692	500	751	371	558	317	477
6	624	938	526	791	443	666	469	706	348	523	287	432
7	614	923	518	778	436	655	459	690	340	511	277	416
8	603	906	508	763	427	642	447	672	331	497	266	400
9	591	888	497	747	418	628	434	652	321	482	254	381
10	577	867	485	729	408	613	420	631	310	465	241	362
11	562	845	472	710	397	597	404	608	298	448	227	341
12	546	821	459	690	386	579	388	584	286	430	213	320
13	530	796	445	668	373	561	372	559	273	411	199	299
14	512	770	430	646	361	542	355	533	260	391	184	277
15	494	743	414	622	347	522	337	506	247	371	170	256
16	476	715	398	598	334	502	319	480	233	351	156	235
17	457	687	382	574	320	481	301	453	220	330	143	214
18	437	658	365	549	306	460	283	426	206	310	129	194
19	418	628	348	524	292	438	265	399	193	290	117	175
20	398	599	332	498	277	417	248	373	180	270	105	158
22	359	540	298	448	249	374	214	322	154	232	86.9	131
24	320	482	265	399	221	332	182	273	131	196	73.0	110
26	283	426	234	351	194	292	155	233	111	167	62.2	93.5
28	247	372	203	305	169	254	133	201	95.9	144	53.7	80.7
30	216	324	177	266	147	221	116	175	83.5	126	46.7	70.3
32	189	285	156	234	129	194	102	154	73.4	110	41.1	61.8
34	168	252	138	207	114	172	90.5	136	65.0	97.7		
36	150	225	123	185	102	153	80.7	121	58.0	87.2		
38	134	202	110	166	91.6	138	72.5	109	52.1	78.2		
40	121	182	99.6	150	82.7	124	65.4	98.3	47.0	70.6		
Properties												
P_{wo}, kips	132	198	107	161	81.9	123	118	177	78.2	117	83.8	126
P_{wi}, kip/in.	20.2	30.3	17.2	25.8	14.5	21.8	18.8	28.3	13.8	20.8	14.8	22.3
P_{wb}, kips	393	591	243	365	147	221	397	597	158	237	241	363
P_{fb}, kips	69.6	105	49.6	74.6	35.4	53.2	59.7	89.8	33.0	49.6	37.1	55.7
L_p, ft	11.9		14.4		16.6		8.65		12.3		6.90	
L_r, ft	37.9		34.0		31.1		34.8		28.3		27.3	
A_g, in.^{2}	21.8		18.4		15.5		16.7		12.4		10.6	
I_x, in.^{4}	569		472		393		294		210		119	
I_y, in.^{4}	186		153		127		101		71.7		40.3	
r_y, in.	2.92		2.88		2.86		2.45		2.41		1.95	
r_x/r_y	1.75		1.76		1.76		1.71		1.71		1.72	
$P_{ex}L_c^2/10^4$, k-in.^{2}	16300		13500		11200		8410		6010		3410	
$P_{ey}L_c^2/10^4$, k-in.^{2}	5320		4380		3630		2890		2050		1150	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 50 ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates KL/r_y equal to or greater than 200.

F_y = 50 ksi

Table 4-3
Available Strength in Axial Compression, kips
Rectangular HSS

HSS20–HSS16

Shape	HSS20×12×								HSS16×12×			
	5/8		1/2[c]		3/8[c]		5/16[c]		5/8		1/2	
t_{des}, in.	0.581		0.465		0.349		0.291		0.581		0.465	
lb/ft	127.37		103.30		78.52		65.87		110.36		89.68	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y												
0	1050	1570	792	1190	528	794	397	597	907	1360	737	1110
6	1030	1550	783	1180	522	785	393	591	892	1340	725	1090
7	1030	1540	779	1170	520	782	392	589	887	1330	721	1080
8	1020	1530	775	1170	518	778	391	587	881	1320	716	1080
9	1010	1520	771	1160	515	774	389	584	874	1310	710	1070
10	1000	1510	766	1150	511	769	387	582	867	1300	704	1060
11	994	1490	760	1140	508	764	385	578	858	1290	698	1050
12	985	1480	755	1130	504	758	383	575	849	1280	691	1040
13	974	1460	748	1120	500	752	380	571	840	1260	683	1030
14	963	1450	741	1110	496	745	377	567	829	1250	675	1010
15	951	1430	734	1100	491	738	375	563	819	1230	666	1000
16	938	1410	727	1090	486	731	371	558	807	1210	657	988
17	925	1390	718	1080	481	723	368	553	795	1190	647	973
18	911	1370	710	1070	475	715	365	548	782	1180	637	958
19	896	1350	701	1050	470	706	361	543	769	1160	627	942
20	881	1320	692	1040	464	697	358	537	756	1140	616	926
21	866	1300	682	1030	458	688	354	532	742	1110	605	909
22	850	1280	672	1010	451	678	350	526	727	1090	594	892
23	833	1250	662	995	445	668	346	519	712	1070	582	874
24	816	1230	652	980	438	658	341	513	697	1050	570	856
25	799	1200	641	963	431	648	336	505	682	1020	557	838
26	782	1180	630	947	424	637	331	497	666	1000	545	819
27	764	1150	619	930	416	626	325	489	650	977	532	800
28	746	1120	607	912	409	615	319	480	634	953	519	780
29	728	1090	594	892	401	603	314	471	618	928	506	761
30	710	1070	579	870	394	592	308	462	601	904	493	741
32	672	1010	550	826	378	568	296	444	568	854	467	701
34	635	955	520	781	362	544	283	426	535	804	440	661
36	598	898	490	736	346	520	271	407	502	754	413	621
38	561	843	460	692	329	495	258	388	469	705	387	582
40	524	788	431	647	313	470	245	369	437	656	361	542

Properties

	HSS20×12×5/8	HSS20×12×1/2	HSS20×12×3/8	HSS20×12×5/16	HSS16×12×5/8	HSS16×12×1/2
A_g, in.^{2}	35.0	28.3	21.5	18.1	30.3	24.6
I_x, in.^{4}	1880	1550	1200	1010	1090	904
I_y, in.^{4}	851	705	547	464	700	581
r_y, in.	4.93	4.99	5.04	5.07	4.80	4.86
r_x/r_y	1.49	1.48	1.48	1.48	1.25	1.25

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Table 4-3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS16

F_y = 50 ksi

Shape		HSS16×12×				HSS16×8×							
		$^3/_8$ c		$^5/_{16}$ c		$^5/_8$		$^1/_2$		$^3/_8$ c		$^5/_{16}$ c	
t_{des}, in.		0.349		0.291		0.581		0.465		0.349		0.291	
lb/ft		68.31		57.36		93.34		76.07		58.10		48.86	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	512	770	386	580	769	1160	626	940	432	649	332	499
	6	506	760	382	574	743	1120	605	909	421	632	324	486
	7	504	757	381	572	733	1100	597	897	417	626	321	482
	8	501	753	379	570	722	1090	589	885	412	619	317	477
	9	498	748	377	567	710	1070	579	870	407	612	314	471
	10	495	743	375	564	697	1050	569	855	402	604	309	465
	11	491	738	373	561	683	1030	557	838	396	594	305	458
	12	487	732	371	557	668	1000	545	820	389	585	300	451
	13	483	725	368	553	652	979	532	800	382	574	295	443
	14	478	718	365	549	634	954	519	780	375	563	289	435
	15	473	711	362	545	617	927	505	759	367	551	283	426
	16	468	703	359	540	598	899	490	736	359	539	277	417
	17	462	695	356	535	579	870	475	714	350	526	271	407
	18	457	687	352	530	559	841	459	690	341	513	264	397
	19	451	678	349	524	539	811	443	666	332	499	257	387
	20	445	668	345	518	519	780	427	642	323	485	250	376
	21	438	658	341	512	498	749	411	617	313	471	243	366
	22	431	648	337	506	478	718	394	592	304	456	236	355
	23	425	638	332	499	457	687	378	567	293	441	229	343
	24	418	628	328	492	436	656	361	543	281	422	221	332
	25	410	617	322	484	416	625	344	518	268	403	213	321
	26	403	606	316	475	395	594	328	493	256	385	206	309
	27	395	594	311	467	375	564	312	469	244	366	198	298
	28	388	583	305	458	356	534	296	445	232	348	190	286
	29	380	571	299	449	336	505	280	421	220	330	183	275
	30	372	559	292	440	317	477	265	398	208	313	175	263
	32	356	534	280	421	280	421	235	353	185	278	158	237
	34	338	508	267	402	248	373	208	313	164	247	140	210
	36	318	478	255	383	221	333	186	279	146	220	125	188
	38	298	448	242	363	199	299	167	250	131	197	112	168
	40	278	418	229	344	179	269	150	226	119	178	101	152

Properties

	HSS16×12×$^3/_8$	HSS16×12×$^5/_{16}$	HSS16×8×$^5/_8$	HSS16×8×$^1/_2$	HSS16×8×$^3/_8$	HSS16×8×$^5/_{16}$
A_g, in.^{2}	18.7	15.7	25.7	20.9	16.0	13.4
I_x, in.^{4}	702	595	815	679	531	451
I_y, in.^{4}	452	384	274	230	181	155
r_y, in.	4.91	4.94	3.27	3.32	3.37	3.40
r_x/r_y	1.25	1.24	1.72	1.72	1.71	1.71

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

c Shape is slender for compression with F_y = 50 ksi; tabulated values have been adjusted accordingly.

F_y = 50 ksi

Table 4-3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS16–HSS14

Shape		HSS16×8×		HSS14×10×									
		$^1/_4{}^c$		$^5/_8$		$^1/_2$		$^3/_8{}^c$		$^5/_{16}{}^c$		$^1/_4{}^c$	
t_{des}, in.		0.233		0.581		0.465		0.349		0.291		0.233	
lb/ft		39.43		93.34		76.07		58.10		48.86		39.43	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	240	360	769	1160	626	940	462	695	360	542	251	377
	6	234	352	751	1130	611	919	454	682	354	532	247	372
	7	232	348	745	1120	606	911	451	678	352	528	246	370
	8	229	345	737	1110	600	902	447	672	349	524	245	368
	9	227	341	729	1100	594	893	444	667	346	520	243	365
	10	224	336	720	1080	587	882	439	660	343	515	241	363
	11	220	331	710	1070	579	870	434	653	339	510	239	360
	12	217	326	699	1050	570	857	429	645	335	504	237	356
	13	213	320	688	1030	561	843	424	637	331	498	235	353
	14	209	314	675	1020	551	829	418	628	327	491	232	349
	15	205	308	663	996	541	813	412	619	322	484	230	345
	16	201	302	649	976	530	797	405	609	317	476	227	341
	17	196	295	635	954	519	781	398	599	312	468	224	336
	18	191	288	620	932	508	763	391	587	306	460	221	332
	19	187	280	605	910	496	745	382	574	301	452	217	327
	20	182	273	590	886	483	727	372	560	295	443	214	322
	21	177	265	574	863	471	708	363	545	289	434	211	317
	22	171	258	558	838	458	688	353	531	283	425	207	311
	23	166	250	541	814	445	669	343	516	276	415	203	305
	24	161	242	525	789	432	649	333	501	270	405	199	299
	25	155	234	508	763	418	628	323	486	263	395	194	292
	26	150	225	491	738	405	608	313	470	256	385	189	284
	27	145	217	474	712	391	588	303	455	250	375	184	277
	28	139	209	457	687	377	567	292	440	243	365	179	270
	29	134	201	440	661	364	547	282	424	236	354	174	262
	30	128	193	423	636	350	526	272	409	229	344	169	255
	32	118	177	390	586	323	486	251	378	213	319	159	239
	34	107	161	357	536	297	446	231	348	196	294	149	224
	36	98.0	147	325	489	271	408	212	318	180	270	139	209
	38	90.0	135	294	442	247	371	193	290	164	246	129	194
	40	82.4	124	266	399	223	334	175	262	148	223	119	179

Properties

	HSS16×8×1/4	HSS14×10×5/8	HSS14×10×1/2	HSS14×10×3/8	HSS14×10×5/16	HSS14×10×1/4
A_g, in.^{2}	10.8	25.7	20.9	16.0	13.4	10.8
I_x, in.^{4}	368	687	573	447	380	310
I_y, in.^{4}	127	407	341	267	227	186
r_y, in.	3.42	3.98	4.04	4.09	4.12	4.14
r_x/r_y	1.70	1.30	1.29	1.29	1.29	1.29

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

HSS12

Table 4-3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

F_y = 50 ksi

Shape		HSS12×10×								HSS12×8×			
		1/2		3/8		5/16[c]		1/4[c]		5/8		1/2	
t_{des}, in.		0.465		0.349		0.291		0.233		0.581		0.465	
lb/ft		69.27		53.00		44.60		36.03		76.33		62.46	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	569	855	437	657	351	527	247	372	629	945	515	774
	6	555	835	427	642	344	517	244	366	605	910	496	746
	7	550	827	423	636	341	513	243	365	597	897	490	736
	8	545	819	419	630	339	509	241	362	588	883	482	725
	9	539	810	415	623	336	505	239	360	577	868	474	713
	10	532	799	409	615	332	499	237	357	566	850	465	699
	11	524	788	404	607	329	494	235	354	553	832	455	684
	12	516	776	398	598	325	488	233	350	540	812	445	668
	13	508	763	391	588	320	481	231	347	526	791	433	651
	14	499	750	384	578	316	474	228	343	511	769	422	634
	15	489	735	377	567	311	467	225	339	496	745	409	615
	16	479	720	370	556	306	459	222	334	480	721	396	596
	17	469	704	362	544	300	451	219	330	464	697	383	576
	18	458	688	354	531	295	443	216	325	447	672	370	556
	19	446	671	345	519	289	434	213	320	430	646	356	535
	20	435	654	336	506	282	424	209	315	412	620	342	514
	21	423	636	327	492	275	413	206	309	395	594	328	493
	22	411	618	318	479	267	402	202	303	377	567	314	472
	23	399	599	309	465	260	390	198	298	360	541	300	451
	24	386	581	300	451	252	379	193	290	343	515	286	430
	25	374	562	290	436	244	367	188	283	325	489	272	409
	26	361	543	281	422	236	355	183	276	308	463	258	388
	27	349	524	271	408	228	343	178	268	292	438	244	367
	28	336	505	262	393	220	331	173	260	275	413	231	347
	29	323	486	252	379	212	319	168	253	259	389	218	328
	30	311	467	242	364	204	307	163	245	243	366	205	309
	32	286	430	224	336	189	284	153	229	214	321	181	272
	34	262	393	205	308	173	260	142	214	189	285	160	241
	36	238	358	187	281	158	238	130	195	169	254	143	215
	38	215	324	170	255	144	216	118	178	152	228	128	193
	40	194	292	153	230	130	195	107	161	137	206	116	174

Properties

	1/2	3/8	5/16[c]	1/4[c]	5/8	1/2
A_g, in.^{2}	19.0	14.6	12.2	9.90	21.0	17.2
I_x, in.^{4}	395	310	264	216	397	333
I_y, in.^{4}	298	234	200	164	210	178
r_y, in.	3.96	4.01	4.04	4.07	3.16	3.21
r_x/r_y	1.15	1.15	1.15	1.15	1.37	1.37

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

F_y = 50 ksi

Table 4-3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS12

Shape		HSS12×8×								HSS12×6×			
		3/8		5/16[c]		1/4[c]		3/16[c]		5/8		1/2	
t_{des}, in.		0.349		0.291		0.233		0.174		0.581		0.465	
lb/ft		47.90		40.35		32.63		24.73		67.82		55.66	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	395	594	318	477	233	350	143	215	560	841	458	688
	6	381	573	309	464	227	341	140	211	524	787	430	646
	7	377	566	306	459	224	337	139	209	512	769	420	631
	8	371	558	302	454	222	333	138	208	498	748	409	615
	9	365	548	298	448	219	329	137	205	482	725	397	597
	10	358	538	293	441	216	324	135	203	466	700	384	577
	11	351	527	289	434	212	319	134	201	448	673	370	556
	12	343	515	283	426	209	314	132	198	429	645	355	534
	13	335	503	278	418	205	308	130	195	410	616	340	511
	14	326	490	272	409	201	301	128	192	390	586	324	487
	15	317	476	266	399	196	295	126	189	370	556	308	462
	16	307	462	259	389	192	288	123	185	349	525	291	438
	17	297	447	251	377	187	281	121	182	329	494	275	413
	18	287	432	242	364	182	273	118	178	308	463	258	388
	19	277	416	234	352	177	266	116	174	288	433	242	364
	20	267	401	225	338	172	258	113	170	268	403	226	339
	21	256	385	216	325	166	250	110	166	248	373	210	316
	22	245	369	208	312	161	242	107	161	229	345	195	293
	23	235	353	199	299	155	233	104	157	211	317	180	270
	24	224	337	190	285	150	225	101	151	194	291	165	248
	25	214	321	181	272	144	217	97.0	146	178	268	152	229
	26	203	305	172	259	139	208	93.4	140	165	248	141	211
	27	193	290	164	246	133	200	89.8	135	153	230	130	196
	28	183	274	155	233	127	191	86.1	129	142	214	121	182
	29	173	260	147	220	120	181	82.5	124	133	199	113	170
	30	163	245	138	208	114	171	79.0	119	124	186	106	159
	32	144	216	122	184	101	151	71.9	108	109	164	92.9	140
	34	127	192	108	163	89.2	134	65.2	98.0	96.4	145	82.2	124
	36	114	171	96.8	145	79.5	120	59.4	89.3	86.0	129	73.4	110
	38	102	153	86.8	131	71.4	107	54.4	81.8	77.2	116	65.8	99.0
	40	92.1	138	78.4	118	64.4	96.8	49.5	74.4			59.4	89.3

Properties

Property	3/8	5/16	1/4	3/16	5/8	1/2
A_g, in.^{2}	13.2	11.1	8.96	6.76	18.7	15.3
I_x, in.^{4}	262	224	184	140	321	271
I_y, in.^{4}	140	120	98.8	75.7	107	91.1
r_y, in.	3.27	3.29	3.32	3.35	2.39	2.44
r_x/r_y	1.37	1.37	1.36	1.36	1.73	1.73

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

Table 4-3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS12–HSS10

$F_y = 50$ ksi

Shape		HSS12×6×								HSS10×8×			
		3/8		5/16[c]		1/4[c]		3/16[c]		5/8		1/2	
t_{des}, in.		0.349		0.291		0.233		0.174		0.581		0.465	
lb/ft		42.79		36.10		29.23		22.18		67.82		55.66	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	353	531	282	424	205	308	134	202	560	841	458	688
	6	332	500	269	405	196	295	128	193	538	809	441	663
	7	325	489	265	398	193	290	126	190	530	797	435	653
	8	317	476	260	390	189	284	124	186	522	784	428	643
	9	308	463	254	382	185	278	121	183	512	770	420	631
	10	298	448	248	372	181	272	119	178	501	754	412	619
	11	288	432	241	362	176	265	116	174	490	736	403	605
	12	277	416	234	351	171	257	112	169	478	718	393	590
	13	265	399	224	337	166	249	109	164	465	698	382	575
	14	253	381	215	323	160	241	105	158	451	678	372	558
	15	241	362	205	307	154	232	102	153	437	657	360	541
	16	229	344	194	292	148	223	97.9	147	422	635	349	524
	17	216	325	184	276	142	214	94.0	141	407	612	336	506
	18	204	306	174	261	136	204	90.1	135	392	589	324	487
	19	191	288	163	245	130	195	86.1	129	376	565	312	468
	20	179	269	153	230	123	185	82.1	123	360	541	299	449
	21	167	251	143	215	117	176	78.0	117	344	517	286	430
	22	155	233	133	200	109	164	74.0	111	328	493	273	411
	23	144	216	124	186	101	152	70.0	105	312	470	260	391
	24	133	199	114	172	93.9	141	66.1	99.3	297	446	248	372
	25	122	184	105	158	86.5	130	62.1	93.3	281	422	235	353
	26	113	170	97.3	146	80.0	120	58.4	87.7	266	399	223	334
	27	105	157	90.2	136	74.2	111	55.0	82.7	251	377	210	316
	28	97.4	146	83.9	126	69.0	104	52.0	78.1	236	354	198	298
	29	90.8	136	78.2	118	64.3	96.6	49.2	73.9	221	333	187	280
	30	84.9	128	73.1	110	60.1	90.3	46.4	69.8	207	311	175	263
	32	74.6	112	64.2	96.5	52.8	79.4	40.8	61.3	182	274	154	231
	34	66.1	99.3	56.9	85.5	46.8	70.3	36.1	54.3	161	242	136	205
	36	58.9	88.6	50.7	76.3	41.7	62.7	32.2	48.5	144	216	121	183
	38	52.9	79.5	45.5	68.4	37.4	56.3	28.9	43.5	129	194	109	164
	40	47.7	71.7	41.1	61.8	33.8	50.8	26.1	39.2	116	175	98.4	148

Properties

Property	3/8	5/16[c]	1/4[c]	3/16[c]	5/8	1/2
A_g, in.^{2}	11.8	9.92	8.03	6.06	18.7	15.3
I_x, in.^{4}	215	184	151	116	253	214
I_y, in.^{4}	72.9	62.8	51.9	40.0	178	151
r_y, in.	2.49	2.52	2.54	2.57	3.09	3.14
r_x/r_y	1.72	1.71	1.71	1.70	1.19	1.19

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

F_y = 50 ksi

Table 4-3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS10

Shape	HSS10×8×								HSS10×6×	
	3/8		5/16		1/4[c]		3/16[c]		5/8	
t_{des}, in.	0.349		0.291		0.233		0.174		0.581	
lb/ft	42.79		36.10		29.23		22.18		59.32	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y										
0	353	531	297	446	227	341	141	212	491	738
6	340	512	286	430	220	331	138	207	458	689
7	336	505	283	425	218	327	137	205	447	672
8	331	497	278	418	215	323	135	203	434	653
9	325	488	274	411	212	319	134	201	420	632
10	319	479	268	403	209	314	132	199	405	609
11	312	469	263	395	205	309	131	196	389	585
12	304	457	257	386	202	303	129	194	372	560
13	297	446	250	376	197	297	127	191	355	533
14	288	434	243	366	193	290	125	187	337	506
15	280	421	236	355	188	283	122	184	319	479
16	271	407	229	344	184	276	120	180	300	451
17	262	394	221	333	179	269	117	177	282	423
18	253	380	214	321	174	261	115	173	263	396
19	243	365	206	309	168	252	112	169	245	369
20	234	351	198	297	161	243	109	164	228	342
21	224	336	190	285	155	233	106	160	210	316
22	214	322	182	273	148	223	103	156	194	291
23	204	307	174	261	142	213	100	150	177	266
24	195	293	165	249	135	204	96.3	145	163	245
25	185	278	157	237	129	194	92.6	139	150	225
26	176	264	149	225	123	184	88.9	134	139	208
27	166	250	142	213	116	175	85.2	128	129	193
28	157	236	134	201	110	165	81.5	122	120	180
29	148	222	126	190	104	156	77.8	117	111	168
30	139	209	119	179	98.0	147	74.2	112	104	157
32	122	184	105	158	86.5	130	66.5	99.9	91.5	138
34	108	163	92.9	140	76.6	115	58.9	88.5	81.1	122
36	96.7	145	82.8	125	68.3	103	52.5	78.9	72.3	109
38	86.8	130	74.3	112	61.3	92.1	47.1	70.8	64.9	97.6
40	78.3	118	67.1	101	55.3	83.2	42.5	63.9		

Properties

	3/8	5/16	1/4[c]	3/16[c]	5/8
A_g, in.^{2}	11.8	9.92	8.03	6.06	16.4
I_x, in.^{4}	169	145	119	91.4	201
I_y, in.^{4}	120	103	84.7	65.1	89.4
r_y, in.	3.19	3.22	3.25	3.28	2.34
r_x/r_y	1.19	1.19	1.18	1.18	1.50

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

Table 4-3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS10

F_y = 50 ksi

Shape		HSS10×6×									
		1/2		3/8		5/16		1/4[c]		3/16[c]	
t_{des}, in.		0.465		0.349		0.291		0.233		0.174	
lb/ft		48.85		37.69		31.84		25.82		19.63	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	404	607	311	468	262	394	199	299	132	198
	6	378	568	292	439	246	370	189	285	126	189
	7	369	555	286	429	241	362	186	280	124	186
	8	359	540	278	418	235	353	182	274	121	182
	9	348	523	270	406	228	343	178	268	119	178
	10	336	505	261	392	221	332	174	261	116	174
	11	323	486	251	378	213	320	169	254	113	169
	12	310	466	241	363	205	307	164	246	109	164
	13	296	445	231	347	196	294	158	238	106	159
	14	282	423	220	331	187	281	152	229	102	154
	15	267	401	209	314	178	267	145	218	98.3	148
	16	252	379	198	298	169	253	138	207	94.4	142
	17	237	357	187	281	159	239	130	196	90.4	136
	18	222	334	176	264	150	225	123	184	86.4	130
	19	208	312	164	247	141	211	115	173	82.2	124
	20	193	291	153	231	132	198	108	162	78.1	117
	21	179	269	143	215	123	184	101	151	74.0	111
	22	166	249	132	199	114	171	93.4	140	69.9	105
	23	152	229	122	184	105	158	86.6	130	65.8	98.9
	24	140	210	112	169	96.8	146	79.8	120	61.7	92.8
	25	129	194	103	155	89.3	134	73.5	110	57.0	85.6
	26	119	179	95.6	144	82.5	124	68.0	102	52.7	79.1
	27	110	166	88.7	133	76.5	115	63.0	94.7	48.8	73.4
	28	103	154	82.4	124	71.2	107	58.6	88.1	45.4	68.2
	29	95.7	144	76.8	116	66.3	99.7	54.6	82.1	42.3	63.6
	30	89.4	134	71.8	108	62.0	93.2	51.1	76.7	39.6	59.4
	32	78.6	118	63.1	94.9	54.5	81.9	44.9	67.4	34.8	52.2
	34	69.6	105	55.9	84.0	48.3	72.5	39.7	59.7	30.8	46.3
	36	62.1	93.3	49.9	75.0	43.0	64.7	35.5	53.3	27.5	41.3
	38	55.7	83.8	44.8	67.3	38.6	58.1	31.8	47.8	24.7	37.0
	40			40.4	60.7	34.9	52.4	28.7	43.2	22.2	33.4

Properties

	1/2	3/8	5/16	1/4[c]	3/16[c]
A_g, in.^{2}	13.5	10.4	8.76	7.10	5.37
I_x, in.^{4}	171	137	118	96.9	74.6
I_y, in.^{4}	76.8	61.8	53.3	44.1	34.1
r_y, in.	2.39	2.44	2.47	2.49	2.52
r_x/r_y	1.49	1.49	1.48	1.48	1.48

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 50 ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

F_y = 50 ksi

Table 4-3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS10–HSS9

Shape		HSS10×5×								HSS9×7×	
		3/8		5/16		1/4[c]		3/16[c]		5/8	
t_{des}, in.		0.349		0.291		0.233		0.174		0.581	
lb/ft		35.13		29.72		24.12		18.35		59.32	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	290	435	245	368	185	278	122	183	491	738
	6	265	398	224	337	173	260	114	171	466	700
	7	256	385	217	326	169	254	111	167	457	687
	8	247	371	209	314	164	246	108	163	447	672
	9	236	355	200	301	159	239	105	158	436	655
	10	225	339	191	288	153	230	102	153	424	637
	11	214	321	182	273	147	221	97.9	147	411	618
	12	202	303	172	258	141	212	93.9	141	398	598
	13	190	285	161	243	133	199	89.8	135	383	576
	14	177	266	151	227	124	187	85.5	129	368	554
	15	165	248	141	212	116	174	81.2	122	353	531
	16	152	229	130	196	108	162	76.7	115	337	507
	17	140	211	120	181	99.6	150	72.3	109	321	483
	18	129	193	110	166	91.6	138	67.8	102	305	459
	19	117	176	101	151	83.8	126	63.4	95.3	289	435
	20	106	159	91.4	137	76.3	115	59.0	88.7	273	411
	21	96.2	145	82.9	125	69.2	104	53.9	81.0	257	387
	22	87.6	132	75.5	113	63.1	94.8	49.1	73.8	242	363
	23	80.2	121	69.1	104	57.7	86.7	44.9	67.5	226	340
	24	73.6	111	63.4	95.3	53.0	79.6	41.3	62.0	211	317
	25	67.9	102	58.5	87.9	48.8	73.4	38.0	57.2	196	295
	26	62.7	94.3	54.1	81.2	45.1	67.9	35.2	52.9	182	273
	27	58.2	87.5	50.1	75.3	41.9	62.9	32.6	49.0	169	253
	28	54.1	81.3	46.6	70.1	38.9	58.5	30.3	45.6	157	236
	29	50.4	75.8	43.4	65.3	36.3	54.5	28.3	42.5	146	220
	30	47.1	70.8	40.6	61.0	33.9	51.0	26.4	39.7	137	205
	32	41.4	62.3	35.7	53.6	29.8	44.8	23.2	34.9	120	180
	34	36.7	55.2	31.6	47.5	26.4	39.7	20.6	30.9	106	160
	36									94.9	143
	38									85.1	128
	40									76.8	115

Properties

	HSS10×5×3/8	HSS10×5×5/16	HSS10×5×1/4[c]	HSS10×5×3/16[c]	HSS9×7×5/8
A_g, in.^{2}	9.67	8.17	6.63	5.02	16.4
I_x, in.^{4}	120	104	85.8	66.2	174
I_y, in.^{4}	40.6	35.2	29.3	22.7	117
r_y, in.	2.05	2.07	2.10	2.13	2.68
r_x/r_y	1.72	1.72	1.71	1.70	1.22

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 50 ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

Table 4-3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS9

F_y = 50 ksi

Shape	HSS9×7× 1/2		3/8		5/16		1/4[c]		3/16[c]	
t_{des}, in.	0.465		0.349		0.291		0.233		0.174	
lb/ft	48.85		37.69		31.84		25.82		19.63	
Design	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y: 0	404	607	311	468	262	394	209	314	137	205
6	384	577	296	446	250	376	201	302	133	199
7	377	567	291	438	246	369	198	297	131	197
8	369	555	285	429	241	362	195	293	129	195
9	360	542	279	419	235	354	191	287	128	192
10	351	527	272	408	230	345	187	280	126	189
11	341	512	264	397	223	335	182	273	123	185
12	330	496	256	385	216	325	176	265	121	182
13	318	478	247	372	209	315	170	256	118	177
14	306	461	238	358	202	304	165	247	115	173
15	294	442	229	344	194	292	158	238	111	167
16	282	423	220	330	186	280	152	229	108	162
17	269	404	210	316	178	268	146	219	104	157
18	256	384	200	301	170	256	139	209	100	151
19	243	365	190	286	162	244	133	199	96.6	145
20	230	345	181	271	154	231	126	190	92.7	139
21	217	326	171	257	146	219	120	180	88.8	133
22	204	307	161	242	138	207	113	170	84.9	128
23	191	288	151	228	130	195	107	160	80.9	122
24	179	269	142	214	122	183	100	151	77.0	116
25	167	251	133	200	114	171	94.0	141	72.3	109
26	155	234	124	186	106	160	88.0	132	67.8	102
27	144	217	115	173	99.0	149	82.0	123	63.3	95.2
28	134	201	107	161	92.1	138	76.2	115	58.9	88.5
29	125	188	99.8	150	85.8	129	71.1	107	54.9	82.5
30	117	175	93.2	140	80.2	121	66.4	99.8	51.3	77.1
32	103	154	81.9	123	70.5	106	58.4	87.7	45.1	67.8
34	90.8	137	72.6	109	62.5	93.9	51.7	77.7	39.9	60.0
36	81.0	122	64.7	97.3	55.7	83.7	46.1	69.3	35.6	53.5
38	72.7	109	58.1	87.3	50.0	75.1	41.4	62.2	32.0	48.1
40	65.6	98.7	52.4	78.8	45.1	67.8	37.4	56.2	28.9	43.4
Properties										
A_g, in.^{2}	13.5		10.4		8.76		7.10		5.37	
I_x, in.^{4}	149		119		102		84.1		64.7	
I_y, in.^{4}	100		80.4		69.2		57.2		44.1	
r_y, in.	2.73		2.78		2.81		2.84		2.87	
r_x/r_y	1.22		1.22		1.21		1.21		1.21	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 50 ksi; tabulated values have been adjusted accordingly.

F_y = 50 ksi

Table 4-3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS9

Shape		HSS9×5× 5/8		1/2		3/8		5/16		1/4[c]		3/16[c]	
t_{des}, in.		0.581		0.465		0.349		0.291		0.233		0.174	
lb/ft		50.81		42.05		32.58		27.59		22.42		17.08	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	419	630	347	522	269	404	227	342	181	272	120	180
	6	378	568	315	473	245	368	208	312	169	253	112	168
	7	364	548	304	457	237	356	201	302	164	246	109	164
	8	349	525	292	439	228	343	194	291	158	238	106	160
	9	333	500	279	419	218	328	186	279	152	228	103	155
	10	315	473	265	398	208	313	177	266	145	218	99.4	149
	11	297	446	250	376	197	296	168	252	138	207	95.5	144
	12	278	418	235	353	186	279	158	238	130	196	91.5	138
	13	259	389	220	330	174	262	149	224	122	184	87.3	131
	14	239	360	204	307	163	245	139	209	115	172	83.0	125
	15	220	331	189	284	151	227	129	194	107	161	78.5	118
	16	202	303	173	261	140	210	120	180	99.1	149	74.0	111
	17	184	276	159	238	128	193	110	166	91.4	137	69.5	104
	18	166	250	144	217	117	176	101	152	84.0	126	64.5	97.0
	19	149	224	130	196	107	160	92.0	138	76.7	115	59.1	88.8
	20	135	202	117	177	96.5	145	83.2	125	69.7	105	53.7	80.8
	21	122	184	107	160	87.5	131	75.5	113	63.2	95.0	48.7	73.3
	22	111	167	97.1	146	79.7	120	68.8	103	57.6	86.5	44.4	66.8
	23	102	153	88.8	134	72.9	110	62.9	94.6	52.7	79.2	40.6	61.1
	24	93.5	141	81.6	123	67.0	101	57.8	86.9	48.4	72.7	37.3	56.1
	25	86.2	130	75.2	113	61.7	92.8	53.3	80.1	44.6	67.0	34.4	51.7
	26	79.7	120	69.5	104	57.1	85.8	49.3	74.0	41.2	62.0	31.8	47.8
	27	73.9	111	64.5	96.9	52.9	79.5	45.7	68.6	38.2	57.4	29.5	44.3
	28	68.7	103	59.9	90.1	49.2	74.0	42.5	63.8	35.5	53.4	27.4	41.2
	29	64.1	96.3	55.9	84.0	45.9	69.0	39.6	59.5	33.1	49.8	25.6	38.4
	30	59.9	90.0	52.2	78.5	42.9	64.4	37.0	55.6	31.0	46.5	23.9	35.9
	32	52.6	79.1	45.9	69.0	37.7	56.6	32.5	48.9	27.2	40.9	21.0	31.6
	34							28.8	43.3	24.1	36.2	18.6	27.9

Properties

	5/8	1/2	3/8	5/16	1/4[c]	3/16[c]
A_g, in.^{2}	14.0	11.6	8.97	7.59	6.17	4.67
I_x, in.^{4}	133	115	92.5	79.8	66.1	51.1
I_y, in.^{4}	52.0	45.2	36.8	32.0	26.6	20.7
r_y, in.	1.92	1.97	2.03	2.05	2.08	2.10
r_x/r_y	1.60	1.59	1.58	1.58	1.57	1.58

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

Table 4-3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS8

F_y = 50 ksi

Shape		HSS8×6×									
		5/8		1/2		3/8		5/16		1/4	
t_{des}, in.		0.581		0.465		0.349		0.291		0.233	
lb/ft		50.81		42.05		32.58		27.59		22.42	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	419	630	347	522	269	404	227	342	185	278
	6	389	585	324	487	251	378	213	320	173	260
	7	379	570	316	474	245	369	208	312	169	254
	8	368	553	306	461	238	358	202	304	165	248
	9	355	534	296	446	231	347	196	295	160	240
	10	342	514	286	429	223	335	189	284	155	232
	11	327	492	274	412	214	322	182	274	149	224
	12	312	469	262	394	205	309	175	263	143	215
	13	297	446	250	375	196	295	167	251	137	205
	14	281	422	237	356	187	280	159	239	130	196
	15	265	398	224	336	177	266	151	226	124	186
	16	248	373	210	316	167	251	142	214	117	176
	17	232	349	197	297	157	236	134	201	110	166
	18	216	325	184	277	147	221	126	189	104	156
	19	200	301	171	258	137	206	117	177	97.0	146
	20	185	278	159	239	128	192	109	164	90.5	136
	21	170	256	147	220	118	178	101	153	84.1	126
	22	156	234	135	202	109	164	93.8	141	77.9	117
	23	142	214	123	185	100	151	86.3	130	71.9	108
	24	131	196	113	170	92.1	138	79.2	119	66.0	99.2
	25	120	181	104	157	84.9	128	73.0	110	60.8	91.5
	26	111	167	96.4	145	78.5	118	67.5	101	56.3	84.6
	27	103	155	89.4	134	72.8	109	62.6	94.1	52.2	78.4
	28	96.0	144	83.1	125	67.6	102	58.2	87.5	48.5	72.9
	29	89.5	135	77.5	116	63.1	94.8	54.3	81.6	45.2	68.0
	30	83.7	126	72.4	109	58.9	88.6	50.7	76.2	42.3	63.5
	32	73.5	111	63.6	95.7	51.8	77.8	44.6	67.0	37.1	55.8
	34	65.1	97.9	56.4	84.7	45.9	69.0	39.5	59.3	32.9	49.4
	36	58.1	87.3	50.3	75.6	40.9	61.5	35.2	52.9	29.3	44.1
	38			45.1	67.8	36.7	55.2	31.6	47.5	26.3	39.6
	40							28.5	42.9	23.8	35.7

Properties

	5/8	1/2	3/8	5/16	1/4
A_g, in.^{2}	14.0	11.6	8.97	7.59	6.17
I_x, in.^{4}	114	98.2	79.1	68.3	56.6
I_y, in.^{4}	72.3	62.5	50.6	43.8	36.4
r_y, in.	2.27	2.32	2.38	2.40	2.43
r_x/r_y	1.26	1.25	1.25	1.25	1.25

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

F_y = 50 ksi

Table 4-3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS8

Shape		HSS8×6×		HSS8×4×							
		3/16[c]		5/8		1/2		3/8		5/16	
t_{des}, in.		0.174		0.581		0.465		0.349		0.291	
lb/ft		17.08		42.30		35.24		27.48		23.34	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	128	192	350	526	292	438	227	341	193	289
	6	122	183	297	446	250	375	196	295	167	251
	7	119	180	279	420	236	355	186	280	159	238
	8	117	176	261	392	221	332	175	263	149	225
	9	114	172	241	362	205	309	163	245	140	210
	10	111	167	221	332	189	284	151	227	130	195
	11	108	162	200	301	173	260	139	209	119	179
	12	104	157	180	271	156	235	126	190	109	164
	13	101	151	161	241	140	211	114	172	98.5	148
	14	96.9	146	142	213	125	188	102	154	88.5	133
	15	93.0	140	124	186	110	165	91.0	137	78.9	119
	16	88.9	134	109	163	96.6	145	80.1	120	69.7	105
	17	84.6	127	96.4	145	85.6	129	71.0	107	61.7	92.7
	18	79.6	120	85.9	129	76.4	115	63.3	95.1	55.0	82.7
	19	74.6	112	77.1	116	68.5	103	56.8	85.4	49.4	74.2
	20	69.7	105	69.6	105	61.9	93.0	51.3	77.1	44.6	67.0
	21	64.9	97.6	63.1	94.9	56.1	84.3	46.5	69.9	40.4	60.8
	22	60.2	90.5	57.5	86.5	51.1	76.8	42.4	63.7	36.8	55.4
	23	55.7	83.7	52.6	79.1	46.8	70.3	38.8	58.3	33.7	50.7
	24	51.2	77.0	48.3	72.7	43.0	64.6	35.6	53.5	31.0	46.5
	25	47.2	70.9	44.6	67.0	39.6	59.5	32.8	49.3	28.5	42.9
	26	43.6	65.6			36.6	55.0	30.3	45.6	26.4	39.6
	27	40.5	60.8							24.5	36.8
	28	37.6	56.6								
	29	35.1	52.7								
	30	32.8	49.3								
	32	28.8	43.3								
	34	25.5	38.4								
	36	22.8	34.2								
	38	20.4	30.7								
	40	18.4	27.7								

Properties

	HSS8×6×3/16	HSS8×4×5/8	HSS8×4×1/2	HSS8×4×3/8	HSS8×4×5/16
A_g, in.^{2}	4.67	11.7	9.74	7.58	6.43
I_x, in.^{4}	43.7	82.0	71.8	58.7	51.0
I_y, in.^{4}	28.2	26.6	23.6	19.6	17.2
r_y, in.	2.46	1.51	1.56	1.61	1.63
r_x/r_y	1.24	1.75	1.74	1.73	1.73

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

Table 4-3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS8–HSS7

F_y = 50 ksi

Shape		HSS8×4×						HSS7×5×			
		1/4		3/16[c]		1/8[c]		1/2		3/8	
t_{des}, in.		0.233		0.174		0.116		0.465		0.349	
lb/ft		19.02		14.53		9.86		35.24		27.48	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	157	236	107	161	60.0	90.1	292	438	227	341
	6	137	205	96.8	146	54.2	81.5	263	395	206	309
	7	130	196	93.3	140	52.3	78.6	253	380	199	299
	8	123	185	89.3	134	50.2	75.4	242	364	191	287
	9	115	173	85.0	128	47.9	71.9	231	347	182	274
	10	107	161	80.4	121	45.4	68.2	219	328	173	260
	11	98.8	149	75.7	114	42.8	64.4	206	309	163	246
	12	90.5	136	70.1	105	40.2	60.4	192	289	154	231
	13	82.3	124	63.9	96.1	37.5	56.3	179	269	143	216
	14	74.2	112	57.9	87.0	34.8	52.3	166	249	133	200
	15	66.4	99.8	52.0	78.1	32.1	48.2	152	229	123	185
	16	58.9	88.5	46.3	69.7	29.4	44.2	139	209	113	170
	17	52.2	78.4	41.1	61.7	26.8	40.2	127	190	104	156
	18	46.5	69.9	36.6	55.0	24.5	36.8	114	172	94.2	142
	19	41.8	62.8	32.9	49.4	22.5	33.8	103	154	85.1	128
	20	37.7	56.6	29.7	44.6	20.6	31.0	92.7	139	76.8	115
	21	34.2	51.4	26.9	40.4	18.7	28.1	84.1	126	69.6	105
	22	31.1	46.8	24.5	36.8	17.0	25.6	76.6	115	63.4	95.4
	23	28.5	42.8	22.4	33.7	15.6	23.4	70.1	105	58.0	87.2
	24	26.2	39.3	20.6	31.0	14.3	21.5	64.4	96.8	53.3	80.1
	25	24.1	36.2	19.0	28.5	13.2	19.8	59.3	89.2	49.1	73.8
	26	22.3	33.5	17.6	26.4	12.2	18.3	54.9	82.5	45.4	68.3
	27	20.7	31.1	16.3	24.5	11.3	17.0	50.9	76.5	42.1	63.3
	28			15.1	22.7	10.5	15.8	47.3	71.1	39.2	58.9
	29							44.1	66.3	36.5	54.9
	30							41.2	61.9	34.1	51.3
	32									30.0	45.1

Properties

	HSS8×4×1/4	HSS8×4×3/16	HSS8×4×1/8	HSS7×5×1/2	HSS7×5×3/8
A_g, in.^{2}	5.24	3.98	2.70	9.74	7.58
I_x, in.^{4}	42.5	33.1	22.9	60.6	49.5
I_y, in.^{4}	14.4	11.3	7.90	35.6	29.3
r_y, in.	1.66	1.69	1.71	1.91	1.97
r_x/r_y	1.72	1.70	1.71	1.31	1.30

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

F_y = 50 ksi

Table 4-3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS7

Shape	HSS7×5×								HSS7×4×	
	5/16		1/4		3/16[c]		1/8[c]		1/2	
t_{des}, in.	0.291		0.233		0.174		0.116		0.465	
lb/ft	23.34		19.02		14.53		9.86		31.84	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y: 0	193	289	157	236	115	173	62.6	94.1	264	396
6	175	263	143	215	107	161	59.2	88.9	224	337
7	169	254	138	208	104	156	58.0	87.1	212	318
8	162	244	133	200	101	152	56.6	85.1	198	297
9	155	233	127	191	97.3	146	55.1	82.8	183	275
10	148	222	121	182	92.8	139	53.4	80.3	168	253
11	140	210	115	173	88.0	132	51.6	77.6	153	230
12	131	197	108	163	83.1	125	49.7	74.7	138	207
13	123	185	101	152	78.0	117	47.3	71.1	123	185
14	114	172	94.6	142	72.9	110	44.8	67.4	109	164
15	106	159	87.8	132	67.8	102	42.3	63.6	95.7	144
16	97.5	146	81.0	122	62.7	94.3	39.8	59.8	84.1	126
17	89.3	134	74.4	112	57.8	86.8	37.3	56.0	74.5	112
18	81.3	122	68.0	102	52.9	79.5	34.7	52.2	66.4	99.9
19	73.6	111	61.8	92.9	48.2	72.5	32.3	48.5	59.6	89.6
20	66.4	99.9	55.8	83.9	43.6	65.6	29.8	44.8	53.8	80.9
21	60.3	90.6	50.6	76.1	39.6	59.5	27.4	41.2	48.8	73.4
22	54.9	82.5	46.1	69.3	36.1	54.2	25.0	37.5	44.5	66.8
23	50.2	75.5	42.2	63.4	33.0	49.6	22.8	34.3	40.7	61.2
24	46.1	69.4	38.7	58.2	30.3	45.6	21.0	31.5	37.4	56.2
25	42.5	63.9	35.7	53.7	27.9	42.0	19.3	29.0	34.4	51.8
26	39.3	59.1	33.0	49.6	25.8	38.8	17.9	26.8		
27	36.5	54.8	30.6	46.0	23.9	36.0	16.6	24.9		
28	33.9	51.0	28.5	42.8	22.3	33.5	15.4	23.2		
29	31.6	47.5	26.5	39.9	20.8	31.2	14.4	21.6		
30	29.5	44.4	24.8	37.3	19.4	29.2	13.4	20.2		
32	26.0	39.0	21.8	32.8	17.0	25.6	11.8	17.7		
34					15.1	22.7	10.4	15.7		

Properties

	HSS7×5×5/16	HSS7×5×1/4	HSS7×5×3/16	HSS7×5×1/8	HSS7×4×1/2
A_g, in.^{2}	6.43	5.24	3.98	2.70	8.81
I_x, in.^{4}	43.0	35.9	27.9	19.3	50.7
I_y, in.^{4}	25.5	21.3	16.6	11.6	20.7
r_y, in.	1.99	2.02	2.05	2.07	1.53
r_x/r_y	1.30	1.30	1.29	1.29	1.57

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

HSS7

Table 4-3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

F_y = 50 ksi

Shape		HSS7×4×									
		3/8		5/16		1/4		3/16[c]		1/8[c]	
t_{des}, in.		0.349		0.291		0.233		0.174		0.116	
lb/ft		24.93		21.21		17.32		13.25		9.01	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	206	310	175	263	143	215	105	157	58.9	88.5
	6	177	266	151	227	124	186	93.6	141	53.1	79.8
	7	168	252	144	216	118	177	90.0	135	51.1	76.9
	8	157	236	135	203	111	167	85.1	128	49.0	73.6
	9	146	220	126	189	104	156	79.8	120	46.6	70.1
	10	135	203	117	175	96.6	145	74.2	111	44.1	66.3
	11	124	186	107	161	88.9	134	68.4	103	41.5	62.4
	12	112	169	97.6	147	81.3	122	62.7	94.2	38.8	58.4
	13	101	152	88.2	133	73.7	111	57.0	85.6	36.1	54.3
	14	90.1	135	79.0	119	66.3	99.7	51.4	77.2	33.4	50.2
	15	79.7	120	70.2	106	59.2	89.0	46.0	69.1	30.7	46.1
	16	70.0	105	61.8	92.9	52.3	78.6	40.8	61.3	28.0	42.1
	17	62.0	93.2	54.8	82.3	46.3	69.6	36.1	54.3	25.4	38.1
	18	55.3	83.2	48.9	73.4	41.3	62.1	32.2	48.4	22.6	34.0
	19	49.7	74.6	43.8	65.9	37.1	55.8	28.9	43.5	20.3	30.5
	20	44.8	67.4	39.6	59.5	33.5	50.3	26.1	39.2	18.3	27.6
	21	40.7	61.1	35.9	53.9	30.4	45.6	23.7	35.6	16.6	25.0
	22	37.0	55.7	32.7	49.2	27.7	41.6	21.6	32.4	15.2	22.8
	23	33.9	50.9	29.9	45.0	25.3	38.0	19.7	29.7	13.9	20.8
	24	31.1	46.8	27.5	41.3	23.2	34.9	18.1	27.2	12.7	19.1
	25	28.7	43.1	25.3	38.1	21.4	32.2	16.7	25.1	11.7	17.6
	26	26.5	39.9	23.4	35.2	19.8	29.8	15.4	23.2	10.8	16.3
	27					18.4	27.6	14.3	21.5	10.1	15.1
	28									9.35	14.1

Properties

	3/8	5/16	1/4	3/16[c]	1/8[c]
A_g, in.^{2}	6.88	5.85	4.77	3.63	2.46
I_x, in.^{4}	41.8	36.5	30.5	23.8	16.6
I_y, in.^{4}	17.3	15.2	12.8	10.0	7.03
r_y, in.	1.58	1.61	1.64	1.66	1.69
r_x/r_y	1.56	1.55	1.54	1.54	1.53

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 50 ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

F_y = 50 ksi

Table 4-3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS6

Shape		HSS6×5×											
		1/2		3/8		5/16		1/4		3/16		1/8[c]	
t_{des}, in.		0.465		0.349		0.291		0.233		0.174		0.116	
lb/ft		31.84		24.93		21.21		17.32		13.25		9.01	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	264	396	206	310	175	263	143	215	109	163	61.3	92.1
	1	263	395	205	309	175	263	142	214	108	163	61.2	92.0
	2	261	392	204	306	173	260	141	212	108	162	60.9	91.5
	3	257	386	201	302	171	257	139	210	106	160	60.4	90.7
	4	251	378	197	296	168	252	137	206	104	157	59.7	89.7
	5	245	368	192	288	163	246	134	201	102	153	58.8	88.4
	6	237	356	186	279	159	238	130	195	98.9	149	57.7	86.8
	7	228	342	179	269	153	230	125	188	95.7	144	56.5	84.9
	8	218	327	172	258	147	220	120	181	92.0	138	55.1	82.8
	9	207	311	163	246	140	210	115	173	88.0	132	53.5	80.4
	10	195	293	155	233	133	200	109	164	83.7	126	51.8	77.9
	11	183	275	146	219	125	188	103	155	79.3	119	50.0	75.1
	12	171	257	137	205	118	177	97.0	146	74.7	112	47.9	71.9
	13	159	238	127	191	110	165	90.7	136	70.0	105	45.4	68.3
	14	146	220	118	177	102	153	84.4	127	65.2	98.0	42.9	64.5
	15	134	201	108	163	93.9	141	78.0	117	60.5	90.9	40.3	60.6
	16	122	183	99.2	149	86.2	130	71.8	108	55.8	83.8	37.8	56.8
	17	110	166	90.2	136	78.7	118	65.7	98.8	51.2	76.9	35.2	52.9
	18	99.3	149	81.6	123	71.4	107	59.8	89.9	46.7	70.2	32.2	48.4
	19	89.1	134	73.3	110	64.3	96.7	54.1	81.3	42.4	63.7	29.3	44.0
	20	80.4	121	66.2	99.5	58.0	87.2	48.8	73.3	38.3	57.5	26.5	39.8
	21	72.9	110	60.0	90.2	52.7	79.1	44.3	66.5	34.7	52.2	24.0	36.1
	22	66.4	99.9	54.7	82.2	48.0	72.1	40.3	60.6	31.6	47.5	21.9	32.9
	23	60.8	91.4	50.0	75.2	43.9	66.0	36.9	55.5	28.9	43.5	20.0	30.1
	24	55.8	83.9	46.0	69.1	40.3	60.6	33.9	50.9	26.6	39.9	18.4	27.6
	25	51.5	77.3	42.4	63.7	37.2	55.8	31.2	46.9	24.5	36.8	16.9	25.4
	26	47.6	71.5	39.2	58.9	34.3	51.6	28.9	43.4	22.6	34.0	15.7	23.5
	27	44.1	66.3	36.3	54.6	31.9	47.9	26.8	40.2	21.0	31.6	14.5	21.8
	28	41.0	61.6	33.8	50.8	29.6	44.5	24.9	37.4	19.5	29.3	13.5	20.3
	29	38.2	57.5	31.5	47.3	27.6	41.5	23.2	34.9	18.2	27.4	12.6	18.9
	30	35.7	53.7	29.4	44.2	25.8	38.8	21.7	32.6	17.0	25.6	11.8	17.7

Properties

A_g, in.^{2}	8.81	6.88	5.85	4.77	3.63	2.46
I_x, in.^{4}	41.1	33.9	29.6	24.7	19.3	13.4
I_y, in.^{4}	30.8	25.5	22.3	18.7	14.6	10.2
r_y, in.	1.87	1.92	1.95	1.98	2.01	2.03
r_x/r_y	1.16	1.16	1.15	1.15	1.15	1.15

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Table 4-3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS6

F_y = 50 ksi

Shape	HSS6×4×											
	1/2		3/8		5/16		1/4		3/16		1/8[c]	
t_{des}, in.	0.465		0.349		0.291		0.233		0.174		0.116	
lb/ft	28.43		22.37		19.08		15.62		11.97		8.16	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y												
0	236	355	185	278	157	237	129	193	98.2	148	57.9	87.0
1	235	353	184	277	157	236	128	193	97.8	147	57.7	86.7
2	232	348	182	273	155	233	127	190	96.7	145	57.2	85.9
3	226	340	178	267	152	228	124	187	94.8	142	56.3	84.6
4	219	329	173	259	147	221	121	181	92.2	139	55.1	82.8
5	210	315	166	249	142	213	116	175	88.9	134	53.6	80.6
6	199	300	158	238	135	203	111	167	85.1	128	51.9	77.9
7	188	282	149	224	128	193	106	159	80.9	122	49.8	74.9
8	175	263	140	210	120	181	99.3	149	76.2	115	47.6	71.5
9	161	243	130	195	112	168	92.6	139	71.2	107	45.2	67.9
10	148	222	119	179	103	155	85.8	129	66.1	99.3	42.6	64.1
11	134	201	109	164	94.5	142	78.8	118	60.8	91.4	39.9	60.0
12	120	181	98.4	148	85.8	129	71.7	108	55.5	83.4	37.2	55.9
13	107	161	88.2	133	77.2	116	64.8	97.4	50.3	75.5	34.4	51.8
14	94.3	142	78.4	118	68.9	104	58.1	87.3	45.2	67.9	31.6	47.5
15	82.3	124	68.9	104	60.9	91.6	51.6	77.6	40.3	60.5	28.3	42.5
16	72.3	109	60.5	91.0	53.5	80.5	45.4	68.3	35.5	53.4	25.1	37.7
17	64.0	96.2	53.6	80.6	47.4	71.3	40.3	60.5	31.5	47.3	22.2	33.4
18	57.1	85.9	47.8	71.9	42.3	63.6	35.9	54.0	28.1	42.2	19.8	29.8
19	51.3	77.1	42.9	64.5	38.0	57.1	32.2	48.4	25.2	37.9	17.8	26.7
20	46.3	69.5	38.7	58.2	34.3	51.5	29.1	43.7	22.7	34.2	16.0	24.1
21	42.0	63.1	35.1	52.8	31.1	46.7	26.4	39.7	20.6	31.0	14.5	21.9
22	38.2	57.5	32.0	48.1	28.3	42.6	24.0	36.1	18.8	28.2	13.3	19.9
23	35.0	52.6	29.3	44.0	25.9	38.9	22.0	33.1	17.2	25.8	12.1	18.2
24	32.1	48.3	26.9	40.4	23.8	35.8	20.2	30.4	15.8	23.7	11.1	16.7
25	29.6	44.5	24.8	37.3	21.9	33.0	18.6	28.0	14.6	21.9	10.3	15.4
26					20.3	30.5	17.2	25.9	13.5	20.2	9.49	14.3
27									12.5	18.8	8.80	13.2
Properties												
A_g, in.^{2}	7.88		6.18		5.26		4.30		3.28		2.23	
I_x, in.^{4}	34.0		28.3		24.8		20.9		16.4		11.4	
I_y, in.^{4}	17.8		14.9		13.2		11.1		8.76		6.15	
r_y, in.	1.50		1.55		1.58		1.61		1.63		1.66	
r_x/r_y	1.39		1.38		1.37		1.37		1.37		1.36	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

F_y = 50 ksi

Table 4-3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS6

Shape	HSS6×3×											
	1/2		3/8		5/16		1/4		3/16		1/8[c]	
t_{des}, in.	0.465		0.349		0.291		0.233		0.174		0.116	
lb/ft	25.03		19.82		16.96		13.91		10.70		7.31	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y												
0	208	313	164	247	140	211	115	173	87.7	132	51.0	76.7
1	206	310	163	245	139	209	114	172	87.1	131	50.7	76.3
2	201	302	159	239	136	204	112	168	85.4	128	50.0	75.1
3	193	290	153	230	131	197	108	162	82.6	124	48.7	73.2
4	182	273	145	218	124	187	103	154	78.8	118	47.0	70.7
5	169	254	135	203	116	175	96.3	145	74.1	111	44.9	67.6
6	154	231	124	187	107	161	89.1	134	68.8	103	42.5	63.9
7	138	207	113	169	97.3	146	81.3	122	63.1	94.8	39.8	59.8
8	122	183	100	151	87.1	131	73.1	110	57.0	85.7	36.9	55.4
9	105	158	88.0	132	76.7	115	64.8	97.4	50.8	76.4	33.8	50.8
10	89.9	135	76.0	114	66.6	100	56.7	85.2	44.7	67.2	30.6	46.1
11	75.2	113	64.7	97.2	57.0	85.7	48.8	73.4	38.8	58.3	27.2	40.9
12	63.2	95.0	54.4	81.7	48.0	72.2	41.4	62.3	33.2	49.9	23.4	35.2
13	53.8	80.9	46.3	69.6	40.9	61.5	35.3	53.1	28.3	42.5	19.9	29.9
14	46.4	69.8	39.9	60.0	35.3	53.0	30.4	45.7	24.4	36.6	17.2	25.8
15	40.4	60.8	34.8	52.3	30.7	46.2	26.5	39.9	21.2	31.9	15.0	22.5
16	35.5	53.4	30.6	46.0	27.0	40.6	23.3	35.0	18.7	28.1	13.2	19.8
17	31.5	47.3	27.1	40.7	23.9	36.0	20.6	31.0	16.5	24.9	11.7	17.5
18	28.1	42.2	24.2	36.3	21.4	32.1	18.4	27.7	14.7	22.2	10.4	15.6
19			21.7	32.6	19.2	28.8	16.5	24.8	13.2	19.9	9.33	14.0
20							14.9	22.4	11.9	18.0	8.42	12.7
21											7.64	11.5
Properties												
A_g, in.^{2}	6.95		5.48		4.68		3.84		2.93		2.00	
I_x, in.^{4}	26.8		22.7		20.1		17.0		13.4		9.43	
I_y, in.^{4}	8.69		7.48		6.67		5.70		4.55		3.23	
r_y, in.	1.12		1.17		1.19		1.22		1.25		1.27	
r_x/r_y	1.76		1.74		1.74		1.72		1.71		1.71	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

Table 4-3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS5

F_y = 50 ksi

Shape		HSS5×4×1/2		HSS5×4×3/8		HSS5×4×5/16		HSS5×4×1/4		HSS5×4×3/16		HSS5×4×1/8[c]	
t_{des}, in.		0.465		0.349		0.291		0.233		0.174		0.116	
lb/ft		25.03		19.82		16.96		13.91		10.70		7.31	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	208	313	164	247	140	211	115	173	87.7	132	56.4	84.8
	1	207	311	163	245	139	210	114	172	87.4	131	56.2	84.5
	2	204	307	161	242	138	207	113	170	86.3	130	55.7	83.6
	3	199	299	157	237	135	202	111	166	84.5	127	54.7	82.3
	4	192	289	153	229	131	196	107	161	82.1	123	53.5	80.4
	5	184	276	146	220	125	188	103	155	79.2	119	51.9	78.0
	6	174	262	139	209	119	179	98.6	148	75.7	114	50.1	75.2
	7	163	246	131	197	113	169	93.3	140	71.7	108	48.0	72.1
	8	152	228	123	184	105	159	87.5	131	67.4	101	45.6	68.6
	9	139	210	113	170	97.8	147	81.3	122	62.9	94.5	43.1	64.8
	10	127	191	104	156	89.9	135	75.0	113	58.1	87.4	40.1	60.3
	11	114	172	94.5	142	81.9	123	68.6	103	53.3	80.2	36.9	55.4
	12	102	154	85.1	128	73.9	111	62.2	93.4	48.5	72.9	33.6	50.5
	13	90.3	136	76.0	114	66.2	99.5	55.9	84.0	43.8	65.8	30.4	45.7
	14	78.9	119	67.2	101	58.7	88.2	49.8	74.8	39.2	58.9	27.3	41.0
	15	68.7	103	58.7	88.3	51.5	77.4	43.9	66.0	34.8	52.3	24.3	36.5
	16	60.4	90.8	51.6	77.6	45.3	68.0	38.6	58.0	30.6	46.0	21.4	32.2
	17	53.5	80.4	45.7	68.7	40.1	60.3	34.2	51.4	27.1	40.7	19.0	28.5
	18	47.7	71.7	40.8	61.3	35.8	53.7	30.5	45.8	24.2	36.3	16.9	25.4
	19	42.8	64.4	36.6	55.0	32.1	48.2	27.4	41.1	21.7	32.6	15.2	22.8
	20	38.7	58.1	33.0	49.7	29.0	43.5	24.7	37.1	19.6	29.4	13.7	20.6
	21	35.1	52.7	30.0	45.0	26.3	39.5	22.4	33.7	17.8	26.7	12.4	18.7
	22	31.9	48.0	27.3	41.0	23.9	36.0	20.4	30.7	16.2	24.3	11.3	17.0
	23	29.2	43.9	25.0	37.5	21.9	32.9	18.7	28.1	14.8	22.2	10.4	15.6
	24	26.8	40.4	22.9	34.5	20.1	30.2	17.2	25.8	13.6	20.4	9.51	14.3
	25			21.1	31.8	18.5	27.9	15.8	23.8	12.5	18.8	8.77	13.2
	26							14.6	22.0	11.6	17.4	8.10	12.2
	27											7.52	11.3

Properties

	1/2	3/8	5/16	1/4	3/16	1/8
A_g, in.^{2}	6.95	5.48	4.68	3.84	2.93	2.00
I_x, in.^{4}	21.2	17.9	15.8	13.4	10.6	7.42
I_y, in.^{4}	14.9	12.6	11.1	9.46	7.48	5.27
r_y, in.	1.46	1.52	1.54	1.57	1.60	1.62
r_x/r_y	1.20	1.19	1.19	1.19	1.19	1.19

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

F_y = 50 ksi

Table 4-3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS5

Shape	HSS5×3×											
	1/2		3/8		5/16		1/4		3/16		1/8[c]	
t_{des}, in.	0.465		0.349		0.291		0.233		0.174		0.116	
lb/ft	21.63		17.27		14.83		12.21		9.42		6.46	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y: 0	180	271	143	215	123	184	101	152	77.2	116	49.5	74.4
1	179	269	142	213	122	183	100	151	76.7	115	49.2	74.0
2	174	261	139	208	119	179	97.9	147	75.1	113	48.5	72.8
3	166	250	133	200	115	172	94.4	142	72.5	109	47.2	70.9
4	156	235	126	189	109	163	89.6	135	69.0	104	45.4	68.3
5	144	217	117	176	101	152	83.8	126	64.7	97.3	43.3	65.0
6	131	197	107	161	93.1	140	77.2	116	59.9	90.0	40.8	61.3
7	117	175	96.2	145	84.2	127	70.1	105	54.6	82.1	38.0	57.1
8	102	154	85.2	128	75.0	113	62.7	94.2	49.1	73.8	34.4	51.7
9	87.9	132	74.2	112	65.8	99.0	55.2	83.0	43.6	65.5	30.7	46.1
10	74.3	112	63.7	95.7	56.9	85.5	48.0	72.1	38.1	57.2	27.0	40.6
11	61.7	92.7	53.6	80.5	48.4	72.7	41.0	61.7	32.8	49.3	23.4	35.2
12	51.8	77.9	45.0	67.7	40.7	61.1	34.6	52.0	27.8	41.8	20.0	30.1
13	44.2	66.4	38.4	57.7	34.7	52.1	29.5	44.3	23.7	35.6	17.1	25.7
14	38.1	57.2	33.1	49.7	29.9	44.9	25.4	38.2	20.5	30.7	14.7	22.1
15	33.2	49.9	28.8	43.3	26.0	39.1	22.1	33.3	17.8	26.8	12.8	19.3
16	29.2	43.8	25.3	38.1	22.9	34.4	19.5	29.2	15.7	23.5	11.3	16.9
17	25.8	38.8	22.4	33.7	20.3	30.5	17.2	25.9	13.9	20.8	9.99	15.0
18	23.0	34.6	20.0	30.1	18.1	27.2	15.4	23.1	12.4	18.6	8.91	13.4
19			18.0	27.0	16.2	24.4	13.8	20.7	11.1	16.7	8.00	12.0
20									10.0	15.1	7.22	10.8

Properties

	1/2	3/8	5/16	1/4	3/16	1/8
A_g, in.^{2}	6.02	4.78	4.10	3.37	2.58	1.77
I_x, in.^{4}	16.4	14.1	12.6	10.7	8.53	6.03
I_y, in.^{4}	7.18	6.25	5.60	4.81	3.85	2.75
r_y, in.	1.09	1.14	1.17	1.19	1.22	1.25
r_x/r_y	1.51	1.51	1.50	1.50	1.49	1.48

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

Table 4-3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS5–HSS4

F_y = 50 ksi

Shape	HSS5×2½×						HSS4×3×					
	1/4		3/16		1/8[c]		3/8		5/16		1/4	
t_{des}, in.	0.233		0.174		0.116		0.349		0.291		0.233	
lb/ft	11.36		8.78		6.03		14.72		12.70		10.51	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y												
0	94.0	141	72.2	108	45.9	69.0	122	184	105	158	87.1	131
1	93.0	140	71.4	107	45.6	68.5	121	182	105	157	86.4	130
2	90.1	135	69.3	104	44.6	67.0	118	178	102	153	84.4	127
3	85.5	129	65.9	99.0	42.9	64.5	113	170	97.9	147	81.2	122
4	79.4	119	61.4	92.2	40.7	61.2	107	161	92.4	139	76.9	116
5	72.2	109	56.0	84.2	38.1	57.2	98.9	149	85.8	129	71.6	108
6	64.3	96.6	50.1	75.3	35.0	52.6	90.0	135	78.3	118	65.7	98.8
7	56.1	84.3	43.9	66.0	30.9	46.5	80.6	121	70.4	106	59.4	89.2
8	47.9	71.9	37.8	56.7	26.8	40.3	70.9	107	62.2	93.4	52.8	79.4
9	40.0	60.1	31.8	47.8	22.8	34.3	61.3	92.1	54.0	81.2	46.2	69.5
10	32.7	49.2	26.2	39.3	19.0	28.5	52.1	78.3	46.2	69.4	39.8	59.9
11	27.0	40.6	21.6	32.5	15.7	23.6	43.5	65.3	38.8	58.3	33.8	50.8
12	22.7	34.1	18.2	27.3	13.2	19.8	36.5	54.9	32.6	49.0	28.4	42.7
13	19.4	29.1	15.5	23.3	11.2	16.9	31.1	46.8	27.8	41.7	24.2	36.3
14	16.7	25.1	13.4	20.1	9.69	14.6	26.8	40.3	23.9	36.0	20.9	31.3
15	14.5	21.9	11.6	17.5	8.44	12.7	23.4	35.1	20.9	31.3	18.2	27.3
16	12.8	19.2	10.2	15.4	7.42	11.1	20.5	30.9	18.3	27.5	16.0	24.0
17			9.06	13.6	6.57	9.88	18.2	27.4	16.2	24.4	14.1	21.3
18							16.2	24.4	14.5	21.8	12.6	19.0
19											11.3	17.0
Properties												
A_g, in.^{2}	3.14		2.41		1.65		4.09		3.52		2.91	
I_x, in.^{4}	9.40		7.51		5.34		7.93		7.14		6.15	
I_y, in.^{4}	3.13		2.53		1.82		5.01		4.52		3.91	
r_y, in.	0.999		1.02		1.05		1.11		1.13		1.16	
r_x/r_y	1.73		1.74		1.71		1.25		1.26		1.25	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

F_y = 50 ksi

Table 4-3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS4

Shape		HSS4×3×				HSS4×2½×							
		3/16		1/8		3/8		5/16		1/4		3/16	
t_{des}, in.		0.174		0.116		0.349		0.291		0.233		0.174	
lb/ft		8.15		5.61		13.44		11.64		9.66		7.51	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	67.1	101	46.1	69.3	112	168	96.7	145	79.9	120	61.7	92.7
	1	66.6	100	45.8	68.8	111	166	95.6	144	79.1	119	61.0	91.7
	2	65.1	97.8	44.8	67.3	107	160	92.3	139	76.5	115	59.1	88.9
	3	62.7	94.3	43.2	65.0	100	151	87.0	131	72.3	109	56.1	84.3
	4	59.5	89.5	41.1	61.8	91.8	138	80.1	120	66.9	101	52.1	78.3
	5	55.7	83.7	38.5	57.9	82.2	123	72.1	108	60.5	91.0	47.4	71.2
	6	51.3	77.1	35.6	53.5	71.7	108	63.4	95.2	53.6	80.5	42.2	63.4
	7	46.6	70.0	32.4	48.7	61.0	91.7	54.4	81.8	46.4	69.7	36.8	55.3
	8	41.7	62.6	29.1	43.7	50.7	76.2	45.6	68.6	39.2	59.0	31.4	47.2
	9	36.7	55.2	25.8	38.7	41.0	61.6	37.3	56.1	32.5	48.8	26.2	39.4
	10	31.9	47.9	22.5	33.8	33.2	49.9	30.2	45.4	26.4	39.7	21.5	32.3
	11	27.3	41.0	19.3	29.0	27.4	41.2	25.0	37.6	21.8	32.8	17.7	26.7
	12	23.0	34.6	16.3	24.6	23.0	34.6	21.0	31.6	18.3	27.5	14.9	22.4
	13	19.6	29.4	13.9	20.9	19.6	29.5	17.9	26.9	15.6	23.5	12.7	19.1
	14	16.9	25.4	12.0	18.0	16.9	25.4	15.4	23.2	13.5	20.2	10.9	16.5
	15	14.7	22.1	10.5	15.7	14.7	22.2	13.4	20.2	11.7	17.6	9.54	14.3
	16	12.9	19.4	9.19	13.8					10.3	15.5	8.38	12.6
	17	11.5	17.2	8.14	12.2								
	18	10.2	15.4	7.26	10.9								
	19	9.17	13.8	6.52	9.80								
	20			5.88	8.84								

Properties

Property	HSS4×3×3/16	HSS4×3×1/8	HSS4×2½×3/8	HSS4×2½×5/16	HSS4×2½×1/4	HSS4×2½×3/16
A_g, in.^{2}	2.24	1.54	3.74	3.23	2.67	2.06
I_x, in.^{4}	4.93	3.52	6.77	6.13	5.32	4.30
I_y, in.^{4}	3.16	2.27	3.17	2.89	2.53	2.06
r_y, in.	1.19	1.21	0.922	0.947	0.973	0.999
r_x/r_y	1.25	1.26	1.46	1.46	1.45	1.44

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

Table 4-3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS4

F_y = 50 ksi

Shape		HSS4×2½×		HSS4×2×									
		1/8		3/8		5/16		1/4		3/16		1/8	
t_{des}, in.		0.116		0.349		0.291		0.233		0.174		0.116	
lb/ft		5.18		12.17		10.58		8.81		6.87		4.75	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	42.5	63.9	101	153	88.0	132	73.1	110	56.6	85.0	38.9	58.5
	1	42.1	63.3	99.5	150	86.4	130	71.8	108	55.7	83.7	38.3	57.6
	2	40.9	61.4	93.8	141	81.7	123	68.2	102	53.0	79.7	36.6	55.0
	3	38.9	58.4	84.9	128	74.5	112	62.5	93.9	48.9	73.5	33.9	51.0
	4	36.3	54.5	73.9	111	65.5	98.4	55.3	83.2	43.6	65.5	30.5	45.8
	5	33.2	49.9	61.9	93.0	55.4	83.3	47.3	71.2	37.7	56.6	26.6	39.9
	6	29.7	44.7	49.7	74.8	45.2	67.9	39.1	58.8	31.5	47.3	22.5	33.7
	7	26.1	39.3	38.4	57.7	35.5	53.4	31.2	46.9	25.5	38.3	18.4	27.7
	8	22.5	33.9	29.4	44.2	27.3	41.0	24.1	36.3	19.9	29.9	14.6	22.0
	9	19.0	28.6	23.2	34.9	21.5	32.4	19.1	28.7	15.7	23.7	11.5	17.3
	10	15.7	23.6	18.8	28.3	17.4	26.2	15.5	23.2	12.8	19.2	9.35	14.1
	11	13.0	19.5	15.5	23.4	14.4	21.7	12.8	19.2	10.5	15.8	7.73	11.6
	12	10.9	16.4	13.1	19.6	12.1	18.2	10.7	16.1	8.86	13.3	6.49	9.76
	13	9.30	14.0					9.15	13.7	7.55	11.3	5.53	8.31
	14	8.02	12.1										
	15	6.99	10.5										
	16	6.14	9.23										
	17	5.44	8.18										

Properties

	HSS4×2½×1/8	HSS4×2×3/8	HSS4×2×5/16	HSS4×2×1/4	HSS4×2×3/16	HSS4×2×1/8
A_g, in.^{2}	1.42	3.39	2.94	2.44	1.89	1.30
I_x, in.^{4}	3.09	5.60	5.13	4.49	3.66	2.65
I_y, in.^{4}	1.49	1.80	1.67	1.48	1.22	0.898
r_y, in.	1.03	0.729	0.754	0.779	0.804	0.830
r_x/r_y	1.43	1.77	1.75	1.75	1.73	1.72

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

F_y = 50 ksi

Table 4-4
Available Strength in Axial Compression, kips
Square HSS

HSS16–HSS14

Shape		HSS16×16×						HSS14×14×					
		1/2		3/8[c]		5/16[c]		5/8		1/2		3/8[c]	
t_{des}, in.		0.465		0.349		0.291		0.581		0.465		0.349	
lb/ft		103.30		78.52		65.87		110.36		89.68		68.31	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	847	1270	549	825	403	606	907	1360	737	1110	527	792
	6	839	1260	546	820	401	603	896	1350	727	1090	522	785
	7	836	1260	544	818	400	601	892	1340	724	1090	521	783
	8	833	1250	543	816	399	600	887	1330	720	1080	519	780
	9	829	1250	541	814	398	598	881	1320	716	1080	517	777
	10	825	1240	540	811	397	596	875	1320	711	1070	515	773
	11	821	1230	538	808	395	594	869	1310	706	1060	512	770
	12	816	1230	536	805	394	592	862	1300	700	1050	509	765
	13	810	1220	533	802	392	590	854	1280	694	1040	506	761
	14	805	1210	531	798	391	587	846	1270	688	1030	503	756
	15	798	1200	528	794	389	585	837	1260	681	1020	500	751
	16	792	1190	526	790	387	582	828	1240	674	1010	496	746
	17	785	1180	523	786	385	578	819	1230	666	1000	492	740
	18	778	1170	520	781	383	575	808	1220	658	989	488	734
	19	770	1160	516	776	380	572	798	1200	649	976	484	727
	20	762	1150	513	771	378	568	787	1180	640	963	480	721
	21	754	1130	509	765	375	564	775	1170	631	949	475	714
	22	746	1120	506	760	373	560	764	1150	622	935	470	707
	23	737	1110	502	754	370	556	752	1130	612	920	465	699
	24	728	1090	498	748	367	552	739	1110	602	905	460	691
	25	718	1080	493	742	364	548	726	1090	592	890	452	680
	26	709	1070	489	735	361	543	713	1070	582	874	444	668
	27	699	1050	485	729	358	538	700	1050	571	858	436	656
	28	689	1040	480	722	355	534	686	1030	560	842	428	644
	29	678	1020	475	715	352	529	673	1010	549	825	420	631
	30	668	1000	471	707	348	523	659	990	538	808	412	619
	32	646	971	460	692	341	513	630	947	515	774	395	593
	34	624	938	450	676	334	502	601	904	492	739	377	567
	36	601	904	439	660	326	490	572	860	468	704	360	540
	38	578	869	428	643	318	478	543	816	445	668	342	514
	40	555	834	416	625	310	466	513	772	421	633	324	487

Properties

	HSS16×16×1/2	HSS16×16×3/8	HSS16×16×5/16	HSS14×14×5/8	HSS14×14×1/2	HSS14×14×3/8
A_g, in.^{2}	28.3	21.5	18.1	30.3	24.6	18.7
$I_x = I_y$, in.^{4}	1130	873	739	897	743	577
$r_x = r_y$, in.	6.31	6.37	6.39	5.44	5.49	5.55

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 50 ksi; tabulated values have been adjusted accordingly.

Table 4-4 (continued)
Available Strength in Axial Compression, kips
Square HSS

HSS14–HSS12

F_y = 50 ksi

Shape	HSS14×14×	HSS12×12×									
	5/16[c]		5/8		1/2		3/8		5/16[c]		1/4[c]
t_{des}, in.	0.291		0.581		0.465		0.349		0.291		0.233
lb/ft	57.36		93.34		76.07		58.10		48.86		39.43
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y: 0	388	584	769	1160	626	940	479	720	372	559	253	380
6	385	579	756	1140	615	924	471	708	368	552	250	376
7	384	577	751	1130	611	919	468	704	366	550	249	375
8	383	576	746	1120	607	912	465	699	364	548	248	373
9	382	573	739	1110	602	905	461	693	362	545	247	371
10	380	571	732	1100	596	896	457	687	360	541	246	369
11	378	568	725	1090	590	887	453	680	358	538	244	367
12	376	566	717	1080	584	878	448	673	355	534	242	364
13	374	563	708	1060	577	867	442	665	352	530	241	362
14	372	559	699	1050	569	856	437	657	349	525	239	359
15	370	556	689	1040	562	844	431	648	346	520	237	356
16	367	552	678	1020	553	832	425	638	343	515	235	353
17	365	548	667	1000	545	819	418	628	339	510	232	349
18	362	544	656	986	535	805	411	618	335	504	230	346
19	359	539	644	968	526	791	404	608	331	498	227	342
20	356	535	632	949	516	776	397	596	327	492	225	338
21	353	530	619	930	506	761	389	585	323	485	222	334
22	349	525	606	911	496	745	381	573	319	479	219	330
23	346	520	593	891	485	729	373	561	314	472	216	325
24	342	514	579	870	474	713	365	549	307	461	213	321
25	338	509	565	850	463	696	357	537	300	451	210	316
26	335	503	551	829	452	680	349	524	293	440	207	311
27	331	497	537	807	441	662	340	511	286	430	204	306
28	327	491	523	786	429	645	331	498	279	419	200	301
29	322	484	508	764	418	628	322	485	271	408	197	296
30	318	478	494	742	406	610	314	471	264	397	194	291
32	309	465	464	698	382	575	296	445	249	375	186	280
34	300	451	435	654	359	540	278	418	234	352	179	269
36	291	437	406	610	336	504	260	391	220	330	171	257
38	281	422	377	567	313	470	243	365	205	308	163	246
40	271	407	349	525	290	436	226	339	191	287	155	233

Properties

	HSS14×14×5/16	HSS12×12×5/8	HSS12×12×1/2	HSS12×12×3/8	HSS12×12×5/16	HSS12×12×1/4
A_g, in.^{2}	15.7	25.7	20.9	16.0	13.4	10.8
$I_x = I_y$, in.^{4}	490	548	457	357	304	248
$r_x = r_y$, in.	5.58	4.62	4.68	4.73	4.76	4.79

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 50 ksi; tabulated values have been adjusted accordingly.

F_y = 50 ksi

Table 4-4 (continued)
Available Strength in Axial Compression, kips
Square HSS

HSS12–HSS10

Shape		HSS12×12×		HSS10×10×									
		$^3/_{16}$ c		$^5/_8$		$^1/_2$		$^3/_8$		$^5/_{16}$		$^1/_4$ c	
t_{des}, in.		0.174		0.581		0.465		0.349		0.291		0.233	
lb/ft		29.84		76.33		62.46		47.90		40.35		32.63	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	149	225	629	945	515	774	395	594	332	499	241	362
	6	148	223	612	921	502	755	386	580	324	487	237	357
	7	148	222	607	912	497	748	382	574	321	483	236	354
	8	147	221	600	902	492	740	378	569	318	478	234	352
	9	146	220	593	891	486	731	374	562	315	473	232	349
	10	146	219	585	879	480	721	369	555	311	467	231	346
	11	145	217	576	865	473	711	364	547	306	460	228	343
	12	144	216	566	851	465	699	358	538	301	453	226	340
	13	143	215	556	835	457	687	352	529	296	445	223	336
	14	142	213	545	819	448	674	346	519	291	437	221	332
	15	141	211	534	802	439	660	339	509	285	429	218	327
	16	139	210	522	784	430	646	332	498	279	420	215	323
	17	138	208	509	765	420	631	324	487	273	411	212	318
	18	137	206	496	746	410	616	317	476	267	401	208	313
	19	136	204	483	726	399	600	309	464	260	391	205	308
	20	134	202	470	706	388	583	300	452	253	381	201	302
	21	133	199	456	685	377	567	292	439	246	370	197	297
	22	131	197	442	664	366	550	284	426	239	360	193	291
	23	129	195	428	643	354	533	275	413	232	349	188	283
	24	128	192	413	621	343	515	266	400	225	338	183	274
	25	126	190	399	599	331	498	258	387	218	327	177	266
	26	124	187	384	577	319	480	249	374	210	316	171	257
	27	123	184	370	555	308	462	240	360	203	305	165	248
	28	121	181	355	534	296	445	231	347	195	293	159	239
	29	119	179	341	512	284	427	222	334	188	282	153	230
	30	117	176	326	490	273	410	213	321	180	271	147	221
	32	113	170	298	448	250	375	196	294	166	249	135	203
	34	109	163	271	407	228	342	179	269	152	228	124	186
	36	105	157	244	367	206	310	163	244	138	207	113	170
	38	100	151	219	329	185	278	147	220	125	187	102	153
	40	95.9	144	198	297	167	251	132	199	112	169	92.1	138

Properties

Property	HSS12×12×3/16	HSS10×10×5/8	HSS10×10×1/2	HSS10×10×3/8	HSS10×10×5/16	HSS10×10×1/4
A_g, in.^{2}	8.15	21.0	17.2	13.2	11.1	8.96
$I_x = I_y$, in.^{4}	189	304	256	202	172	141
$r_x = r_y$, in.	4.82	3.80	3.86	3.92	3.94	3.97

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

c Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Table 4-4 (continued)
Available Strength in Axial Compression, kips
Square HSS

HSS10–HSS9

F_y = 50 ksi

Shape		HSS10×10×		HSS9×9×									
		3/16[c]		5/8		1/2		3/8		5/16		1/4[c]	
t_{des}, in.		0.174		0.581		0.465		0.349		0.291		0.233	
lb/ft		24.73		67.82		55.66		42.79		36.10		29.23	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	145	218	560	841	458	688	353	531	297	446	233	350
	6	143	215	542	814	444	667	343	515	288	433	228	342
	7	142	213	535	805	439	659	339	509	285	428	226	340
	8	141	212	528	794	433	651	334	503	281	423	224	337
	9	140	211	520	782	426	641	330	495	277	417	222	334
	10	139	209	511	768	419	630	324	488	273	410	220	330
	11	138	207	501	754	412	619	319	479	268	403	217	326
	12	137	205	491	738	403	606	312	470	263	396	213	321
	13	135	203	480	721	394	593	306	460	258	387	209	314
	14	134	201	468	704	385	579	299	449	252	379	204	307
	15	132	199	456	686	375	564	291	438	246	370	199	300
	16	130	196	443	666	365	549	284	427	240	360	194	292
	17	129	193	430	647	355	533	276	415	233	350	189	284
	18	127	191	417	626	344	517	268	403	226	340	184	276
	19	125	188	403	606	333	500	260	390	219	330	178	268
	20	123	185	389	585	322	483	251	377	212	319	172	259
	21	121	182	375	563	310	466	242	364	205	308	167	251
	22	119	178	360	542	299	449	234	351	198	297	161	242
	23	116	175	346	520	287	431	225	338	190	286	155	233
	24	114	172	331	498	275	414	216	325	183	275	149	224
	25	112	168	317	476	264	396	207	311	176	264	143	215
	26	109	164	302	455	252	379	198	298	168	253	137	206
	27	107	161	288	433	240	361	189	285	161	242	131	197
	28	105	157	274	412	229	344	181	272	154	231	125	188
	29	102	153	260	391	218	327	172	259	147	220	120	180
	30	99.4	149	247	371	207	311	164	246	139	210	114	171
	32	94.2	142	220	331	185	278	147	221	126	189	103	154
	34	88.8	134	195	293	164	247	131	197	112	169	91.9	138
	36	83.4	125	174	262	147	220	117	176	100	150	82.0	123
	38	78.0	117	156	235	132	198	105	158	89.9	135	73.6	111
	40	70.6	106	141	212	119	179	94.8	143	81.1	122	66.4	99.8

Properties

Property	HSS10×10×3/16	HSS9×9×5/8	HSS9×9×1/2	HSS9×9×3/8	HSS9×9×5/16	HSS9×9×1/4
A_g, in.^{2}	6.76	18.7	15.3	11.8	9.92	8.03
$I_x = I_y$, in.^{4}	108	216	183	145	124	102
$r_x = r_y$, in.	4.00	3.40	3.45	3.51	3.54	3.56

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

F_y = 50 ksi

Table 4-4 (continued)
Available Strength in Axial Compression, kips
Square HSS

HSS9–HSS8

Shape		HSS9×9×				HSS8×8×							
		$^{3}/_{16}$ [c]		$^{1}/_{8}$ [c]		$^{5}/_{8}$		$^{1}/_{2}$		$^{3}/_{8}$		$^{5}/_{16}$	
t_{des}, in.		0.174		0.116		0.581		0.465		0.349		0.291	
lb/ft		22.18		14.96		59.32		48.85		37.69		31.84	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	141	213	68.0	102	491	738	404	607	311	468	262	394
	6	139	209	66.8	100	471	707	388	583	299	450	252	379
	7	138	207	66.4	99.8	463	697	382	575	295	444	249	374
	8	137	206	65.9	99.1	455	684	376	565	290	436	245	368
	9	136	204	65.4	98.3	446	671	369	554	285	428	240	361
	10	134	202	64.8	97.4	436	656	361	542	279	419	236	354
	11	133	200	64.2	96.5	426	640	352	529	273	410	230	346
	12	131	197	63.5	95.4	414	623	343	516	266	400	225	338
	13	130	195	62.8	94.3	402	605	333	501	259	389	219	329
	14	128	192	62.0	93.1	390	586	323	486	251	378	212	319
	15	126	189	61.1	91.8	377	566	313	470	243	366	206	310
	16	124	186	60.2	90.5	363	546	302	454	235	354	199	299
	17	122	183	59.3	89.1	349	525	291	437	227	341	192	289
	18	119	179	58.3	87.6	335	504	279	420	218	328	185	278
	19	117	176	57.3	86.1	321	482	268	403	210	315	178	267
	20	115	172	56.2	84.5	307	461	256	385	201	302	171	256
	21	112	169	55.1	82.8	292	439	245	368	192	289	163	245
	22	110	165	54.0	81.1	278	417	233	350	183	275	156	234
	23	107	161	52.8	79.4	263	396	221	333	174	262	149	223
	24	104	157	51.6	77.6	249	374	210	315	166	249	141	212
	25	101	152	50.4	75.8	235	353	198	298	157	236	134	201
	26	98.6	148	49.2	73.9	221	333	187	281	148	223	127	191
	27	95.8	144	47.9	72.0	208	313	176	265	140	211	120	180
	28	92.9	140	46.6	70.1	195	293	165	249	132	198	113	170
	29	89.9	135	45.3	68.1	182	274	155	233	124	186	106	160
	30	87.0	131	44.0	66.2	170	256	145	217	116	174	99.5	150
	32	78.6	118	41.4	62.2	149	225	127	191	102	153	87.5	131
	34	70.5	106	38.7	58.2	132	199	113	169	90.2	136	77.5	116
	36	62.9	94.5	36.0	54.1	118	177	100	151	80.5	121	69.1	104
	38	56.5	84.9	33.6	50.4	106	159	90.2	136	72.2	109	62.0	93.2
	40	51.0	76.6	31.4	47.2	95.6	144	81.4	122	65.2	98.0	56.0	84.1

Properties

Property	HSS9×9×3/16	HSS9×9×1/8	HSS8×8×5/8	HSS8×8×1/2	HSS8×8×3/8	HSS8×8×5/16
A_g, in.^{2}	6.06	4.09	16.4	13.5	10.4	8.76
$I_x = I_y$, in.^{4}	78.2	53.5	146	125	100	85.6
$r_x = r_y$, in.	3.59	3.62	2.99	3.04	3.10	3.13

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

HSS8–HSS7

Table 4-4 (continued)
Available Strength in Axial Compression, kips
Square HSS

F_y = 50 ksi

Shape	HSS8×8× 1/4		HSS8×8× 3/16^c		HSS8×8× 1/8^c		HSS7×7× 5/8		HSS7×7× 1/2		HSS7×7× 3/8	
t_{des}, in.	0.233		0.174		0.116		0.581		0.465		0.349	
lb/ft	25.82		19.63		13.26		50.81		42.05		32.58	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y												
0	213	319	137	206	66.6	100	419	630	347	522	269	404
6	205	308	134	201	65.2	98.0	396	595	329	494	255	383
7	202	303	133	199	64.7	97.2	388	583	322	484	250	376
8	199	299	131	197	64.1	96.3	379	569	315	474	245	368
9	195	293	130	195	63.4	95.3	369	554	307	461	239	359
10	191	287	128	193	62.7	94.2	358	538	298	448	232	349
11	187	281	126	190	61.9	93.0	346	520	289	434	225	338
12	182	274	124	187	61.0	91.7	334	502	279	419	218	327
13	178	267	122	184	60.1	90.3	321	482	269	404	210	316
14	173	260	120	180	59.1	88.8	307	462	258	387	202	303
15	167	252	118	177	58.0	87.2	294	441	247	371	194	291
16	162	243	115	173	56.9	85.6	280	420	235	354	185	278
17	156	235	112	169	55.8	83.8	265	399	224	336	176	265
18	151	227	110	165	54.6	82.0	251	377	212	319	168	252
19	145	218	107	160	53.3	80.1	237	356	200	301	159	239
20	139	209	104	156	52.0	78.2	223	335	189	284	150	226
21	133	200	101	151	50.7	76.2	209	314	177	267	141	212
22	127	191	97.1	146	49.3	74.2	195	293	166	250	133	200
23	121	182	92.7	139	47.9	72.1	182	273	155	233	124	187
24	115	173	88.3	133	46.5	69.9	169	253	145	217	116	175
25	110	165	83.9	126	45.1	67.8	156	234	134	201	108	163
26	104	156	79.5	120	43.6	65.6	144	216	124	186	100	151
27	98.1	147	75.3	113	42.2	63.4	133	201	115	173	92.9	140
28	92.5	139	71.1	107	40.7	61.1	124	186	107	161	86.4	130
29	87.1	131	67.0	101	39.2	58.9	116	174	99.6	150	80.6	121
30	81.7	123	63.0	94.7	37.7	56.6	108	162	93.1	140	75.3	113
32	71.8	108	55.4	83.2	34.6	52.0	95.0	143	81.8	123	66.2	99.4
34	63.6	95.6	49.0	73.7	31.9	48.0	84.1	126	72.4	109	58.6	88.1
36	56.7	85.3	43.7	65.7	29.5	44.4	75.1	113	64.6	97.1	52.3	78.6
38	50.9	76.5	39.3	59.0	27.0	40.5	67.4	101	58.0	87.2	46.9	70.5
40	46.0	69.1	35.4	53.2	24.3	36.6	60.8	91.4	52.3	78.7	42.3	63.6

Properties

	HSS8×8×1/4	HSS8×8×3/16	HSS8×8×1/8	HSS7×7×5/8	HSS7×7×1/2	HSS7×7×3/8
A_g, in.^{2}	7.10	5.37	3.62	14.0	11.6	8.97
$I_x = I_y$, in.^{4}	70.7	54.4	37.4	93.4	80.5	65.0
$r_x = r_y$, in.	3.15	3.18	3.21	2.58	2.63	2.69

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

F_y = 50 ksi

Table 4-4 (continued)
Available Strength in Axial Compression, kips
Square HSS

HSS7–HSS6

Shape		HSS7×7×								HSS6×6×			
		5/16		1/4		3/16[c]		1/8[c]		5/8		1/2	
t_{des}, in.		0.291		0.233		0.174		0.116		0.581		0.465	
lb/ft		27.59		22.42		17.08		11.56		42.30		35.24	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	227	342	185	278	132	198	65.1	97.9	350	526	292	438
	6	216	324	176	264	127	191	63.2	95.0	323	486	270	406
	7	212	319	173	259	126	189	62.6	94.0	314	472	263	395
	8	207	312	169	254	124	186	61.8	92.9	304	456	255	383
	9	203	304	165	248	122	183	60.9	91.6	292	439	246	369
	10	197	296	161	242	120	180	60.0	90.1	280	421	236	355
	11	191	288	156	235	117	176	58.9	88.6	267	402	226	339
	12	185	278	151	227	115	172	57.8	86.9	254	382	215	323
	13	179	269	146	219	111	167	56.6	85.1	240	361	204	306
	14	172	258	141	211	107	161	55.4	83.2	226	340	193	289
	15	165	248	135	203	103	154	54.0	81.2	212	318	181	272
	16	158	237	129	194	98.4	148	52.6	79.1	198	297	170	255
	17	151	226	124	186	94.0	141	51.2	76.9	184	276	158	238
	18	143	215	118	177	89.6	135	49.7	74.6	170	255	147	221
	19	136	204	112	168	85.2	128	48.1	72.3	156	235	136	204
	20	129	193	106	159	80.8	121	46.5	69.9	143	215	125	188
	21	121	182	100	150	76.3	115	44.9	67.5	130	196	115	172
	22	114	172	94.2	142	72.0	108	43.2	65.0	119	179	104	157
	23	107	161	88.4	133	67.7	102	41.5	62.4	109	163	95.6	144
	24	100	150	82.8	125	63.4	95.3	39.8	59.9	99.8	150	87.8	132
	25	93.4	140	77.4	116	59.3	89.1	38.1	57.3	92.0	138	80.9	122
	26	86.7	130	72.0	108	55.3	83.1	36.4	54.7	85.1	128	74.8	112
	27	80.4	121	66.8	100	51.3	77.1	34.6	52.0	78.9	119	69.4	104
	28	74.8	112	62.1	93.4	47.7	71.7	32.9	49.5	73.4	110	64.5	96.9
	29	69.7	105	57.9	87.0	44.5	66.8	30.7	46.2	68.4	103	60.1	90.4
	30	65.1	97.9	54.1	81.3	41.6	62.5	28.7	43.2	63.9	96.0	56.2	84.4
	32	57.2	86.0	47.6	71.5	36.5	54.9	25.3	38.0	56.2	84.4	49.4	74.2
	34	50.7	76.2	42.1	63.3	32.4	48.6	22.4	33.6	49.7	74.8	43.7	65.7
	36	45.2	68.0	37.6	56.5	28.9	43.4	20.0	30.0	44.4	66.7	39.0	58.6
	38	40.6	61.0	33.7	50.7	25.9	38.9	17.9	26.9				
	40	36.6	55.1	30.4	45.8	23.4	35.1	16.2	24.3				

Properties

	5/16	1/4	3/16	1/8	5/8	1/2
A_g, in.^{2}	7.59	6.17	4.67	3.16	11.7	9.74
$I_x = I_y$, in.^{4}	56.1	46.5	36.0	24.8	55.2	48.3
$r_x = r_y$, in.	2.72	2.75	2.77	2.80	2.17	2.23

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

Table 4-4 (continued)
Available Strength in Axial Compression, kips
Square HSS

HSS6

F_y = 50 ksi

Shape		HSS6×6×									
		3/8		5/16		1/4		3/16		1/8[c]	
t_{des}, in.		0.349		0.291		0.233		0.174		0.116	
lb/ft		27.48		23.34		19.02		14.53		9.86	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	227	341	193	289	157	236	119	179	63.1	94.8
	6	211	317	179	270	146	220	111	167	60.5	90.9
	7	206	309	175	263	143	215	109	163	59.6	89.5
	8	199	300	170	255	139	208	106	159	58.5	87.9
	9	193	289	164	247	134	202	102	154	57.3	86.2
	10	185	279	158	238	129	195	98.8	148	56.1	84.3
	11	178	267	152	228	124	187	95.0	143	54.7	82.2
	12	170	255	145	218	119	179	91.0	137	53.2	80.0
	13	161	242	138	207	113	170	86.8	130	51.6	77.6
	14	153	229	131	197	108	162	82.5	124	50.0	75.1
	15	144	216	123	186	102	153	78.2	117	48.2	72.5
	16	135	203	116	175	95.9	144	73.7	111	46.4	69.8
	17	126	190	109	164	90.0	135	69.3	104	44.5	67.0
	18	118	177	102	153	84.1	126	64.9	97.6	42.6	64.1
	19	109	164	94.4	142	78.4	118	60.6	91.0	40.7	61.1
	20	101	152	87.4	131	72.7	109	56.3	84.6	38.7	58.1
	21	92.9	140	80.6	121	67.2	101	52.1	78.4	35.9	53.9
	22	85.0	128	74.0	111	61.9	93.0	48.1	72.3	33.1	49.8
	23	77.8	117	67.7	102	56.6	85.1	44.1	66.3	30.4	45.7
	24	71.4	107	62.2	93.5	52.0	78.1	40.5	60.9	27.9	42.0
	25	65.8	98.9	57.3	86.1	47.9	72.0	37.3	56.1	25.8	38.7
	26	60.8	91.4	53.0	79.6	44.3	66.6	34.5	51.9	23.8	35.8
	27	56.4	84.8	49.1	73.8	41.1	61.7	32.0	48.1	22.1	33.2
	28	52.5	78.8	45.7	68.7	38.2	57.4	29.8	44.7	20.5	30.9
	29	48.9	73.5	42.6	64.0	35.6	53.5	27.7	41.7	19.1	28.8
	30	45.7	68.7	39.8	59.8	33.3	50.0	25.9	39.0	17.9	26.9
	32	40.2	60.4	35.0	52.6	29.2	44.0	22.8	34.2	15.7	23.6
	34	35.6	53.5	31.0	46.6	25.9	38.9	20.2	30.3	13.9	20.9
	36	31.7	47.7	27.6	41.5	23.1	34.7	18.0	27.1	12.4	18.7
	38	28.5	42.8	24.8	37.3	20.7	31.2	16.2	24.3	11.1	16.8

Properties

	3/8	5/16	1/4	3/16	1/8
A_g, in.^{2}	7.58	6.43	5.24	3.98	2.70
$I_x = I_y$, in.^{4}	39.5	34.3	28.6	22.3	15.5
$r_x = r_y$, in.	2.28	2.31	2.34	2.37	2.39

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 50 ksi; tabulated values have been adjusted accordingly.

F_y = 50 ksi

Table 4-4 (continued)
Available Strength in Axial Compression, kips
Square HSS

HSS5½–HSS5

Shape		HSS5½×5½×										HSS5×5×	
		3/8		5/16		1/4		3/16		1/8[c]		1/2	
t_{des}, in.		0.349		0.291		0.233		0.174		0.116		0.465	
lb/ft		24.93		21.21		17.32		13.25		9.01		28.43	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	206	310	175	263	143	215	109	163	61.4	92.3	236	355
	6	189	284	161	242	131	197	100	151	58.4	87.7	210	316
	7	183	275	156	234	127	192	97.3	146	57.3	86.1	202	303
	8	176	265	151	226	123	185	94.1	141	56.1	84.3	192	289
	9	169	254	145	217	118	178	90.5	136	54.7	82.2	182	274
	10	161	243	138	208	113	170	86.7	130	53.2	80.0	172	258
	11	153	231	132	198	108	162	82.7	124	51.6	77.6	161	241
	12	145	218	125	187	102	154	78.5	118	49.9	75.0	149	224
	13	137	205	117	177	96.5	145	74.2	112	48.1	72.2	138	207
	14	128	192	110	166	90.6	136	69.8	105	46.2	69.4	127	190
	15	119	179	103	155	84.7	127	65.4	98.3	44.2	66.4	115	173
	16	110	166	95.6	144	78.8	118	61.0	91.7	42.0	63.1	105	157
	17	102	153	88.4	133	73.0	110	56.6	85.1	39.1	58.7	94.2	142
	18	93.6	141	81.4	122	67.3	101	52.3	78.6	36.2	54.4	84.1	126
	19	85.6	129	74.6	112	61.8	92.9	48.1	72.3	33.3	50.1	75.5	113
	20	77.7	117	68.0	102	56.4	84.8	44.1	66.2	30.6	46.0	68.1	102
	21	70.5	106	61.6	92.7	51.2	77.0	40.1	60.2	27.9	42.0	61.8	92.9
	22	64.2	96.5	56.2	84.4	46.7	70.1	36.5	54.9	25.4	38.2	56.3	84.6
	23	58.7	88.3	51.4	77.2	42.7	64.2	33.4	50.2	23.3	35.0	51.5	77.4
	24	53.9	81.1	47.2	70.9	39.2	58.9	30.7	46.1	21.4	32.1	47.3	71.1
	25	49.7	74.7	43.5	65.4	36.1	54.3	28.3	42.5	19.7	29.6	43.6	65.5
	26	46.0	69.1	40.2	60.4	33.4	50.2	26.2	39.3	18.2	27.4	40.3	60.6
	27	42.6	64.1	37.3	56.0	31.0	46.6	24.2	36.4	16.9	25.4	37.4	56.2
	28	39.6	59.6	34.7	52.1	28.8	43.3	22.5	33.9	15.7	23.6	34.8	52.2
	29	36.9	55.5	32.3	48.6	26.9	40.4	21.0	31.6	14.6	22.0	32.4	48.7
	30	34.5	51.9	30.2	45.4	25.1	37.7	19.6	29.5	13.7	20.6	30.3	45.5

Properties

Property	3/8	5/16	1/4	3/16	1/8[c]	HSS5×5×1/2
A_g, in.^{2}	6.88	5.85	4.77	3.63	2.46	7.88
$I_x = I_y$, in.^{4}	29.7	25.9	21.7	17.0	11.8	26.0
$r_x = r_y$, in.	2.08	2.11	2.13	2.16	2.19	1.82

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

HSS5–HSS4½

Table 4-4 (continued)
Available Strength in Axial Compression, kips
Square HSS

F_y = 50 ksi

Shape		HSS5×5×										HSS4½×4½×	
		3/8		5/16		1/4		3/16		1/8[c]		1/2	
t_{des}, in.		0.349		0.291		0.233		0.174		0.116		0.465	
lb/ft		22.37		19.08		15.62		11.97		8.16		25.03	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	185	278	157	237	129	193	98.2	148	59.8	89.9	208	313
	1	184	277	157	236	128	193	97.9	147	59.7	89.7	207	311
	2	183	275	156	234	127	191	97.1	146	59.4	89.2	205	308
	3	180	271	153	231	126	189	95.8	144	58.8	88.4	201	302
	4	176	265	150	226	123	185	94.0	141	58.1	87.3	195	293
	5	172	258	146	220	120	180	91.7	138	57.2	86.0	188	283
	6	166	250	142	213	116	175	89.0	134	56.1	84.3	180	270
	7	160	240	137	205	112	168	85.9	129	54.8	82.3	171	256
	8	153	229	131	196	107	161	82.4	124	53.3	80.1	160	241
	9	145	218	124	187	102	154	78.7	118	51.7	77.7	150	225
	10	137	206	118	177	97.0	146	74.7	112	49.9	75.1	139	208
	11	129	193	111	166	91.5	137	70.5	106	48.0	72.2	127	191
	12	120	180	103	156	85.7	129	66.2	99.5	45.5	68.4	116	174
	13	111	167	96.2	145	79.8	120	61.8	92.9	42.6	64.0	105	157
	14	103	154	88.9	134	74.0	111	57.4	86.3	39.6	59.6	93.9	141
	15	94.0	141	81.7	123	68.2	102	53.0	79.7	36.7	55.2	83.4	125
	16	85.6	129	74.6	112	62.4	93.8	48.7	73.2	33.8	50.8	73.5	110
	17	75.5	116	67.8	102	56.9	85.5	44.5	66.8	31.0	46.5	65.1	97.8
	18	69.6	105	61.2	91.9	51.5	77.4	40.4	60.7	28.2	42.4	58.0	87.2
	19	62.5	93.9	54.9	82.5	46.3	69.6	36.4	54.8	25.5	38.4	52.1	78.3
	20	56.4	84.8	49.6	74.5	41.8	62.8	32.9	49.4	23.0	34.6	47.0	70.7
	21	51.2	76.9	44.9	67.6	37.9	57.0	29.8	44.8	20.9	31.4	42.6	64.1
	22	46.6	70.0	41.0	61.5	34.5	51.9	27.2	40.8	19.0	28.6	38.9	58.4
	23	42.6	64.1	37.5	56.3	31.6	47.5	24.9	37.4	17.4	26.2	35.5	53.4
	24	39.2	58.9	34.4	51.7	29.0	43.6	22.8	34.3	16.0	24.1	32.6	49.1
	25	36.1	54.2	31.7	47.7	26.7	40.2	21.0	31.6	14.7	22.2	30.1	45.2
	26	33.4	50.2	29.3	44.1	24.7	37.2	19.5	29.2	13.6	20.5	27.8	41.8
	27	30.9	46.5	27.2	40.9	22.9	34.5	18.0	27.1	12.6	19.0		
	28	28.8	43.2	25.3	38.0	21.3	32.1	16.8	25.2	11.8	17.7		
	29	26.8	40.3	23.6	35.4	19.9	29.9	15.6	23.5	11.0	16.5		

Properties

	3/8	5/16	1/4	3/16	1/8[c]	1/2
A_g, in.^{2}	6.18	5.26	4.30	3.28	2.23	6.95
$I_x = I_y$, in.^{4}	21.7	19.0	16.0	12.6	8.80	18.1
$r_x = r_y$, in.	1.87	1.90	1.93	1.96	1.99	1.61

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 50 ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

F_y = 50 ksi

Table 4-4 (continued)
Available Strength in Axial Compression, kips
Square HSS

HSS4½–HSS4

Shape		HSS4½×4½×										HSS4×4×	
		3/8		5/16		1/4		3/16		1/8[c]		1/2	
t_{des}, in.		0.349		0.291		0.233		0.174		0.116		0.465	
lb/ft		19.82		16.96		13.91		10.70		7.31		21.63	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	164	247	140	211	115	173	87.7	132	57.7	86.8	180	271
	1	163	246	140	210	115	172	87.4	131	57.6	86.6	179	269
	2	162	243	138	208	113	170	86.5	130	57.2	86.0	176	265
	3	159	238	136	204	111	167	85.1	128	56.6	85.0	172	258
	4	154	232	132	199	109	163	83.0	125	55.6	83.6	166	249
	5	149	224	128	192	105	158	80.5	121	54.5	81.9	158	237
	6	143	215	123	185	101	152	77.5	117	53.1	79.8	149	224
	7	136	205	117	176	96.8	145	74.1	111	50.9	76.5	139	209
	8	129	194	111	167	91.8	138	70.4	106	48.4	72.8	128	193
	9	121	182	104	157	86.5	130	66.4	99.8	45.7	68.8	117	176
	10	112	169	97.3	146	80.9	122	62.2	93.5	43.0	64.6	106	160
	11	104	156	90.2	136	75.1	113	57.9	87.0	40.1	60.2	95.0	143
	12	95.3	143	82.9	125	69.3	104	53.5	80.4	37.1	55.8	84.1	126
	13	86.7	130	75.7	114	63.4	95.4	49.1	73.7	34.1	51.3	73.6	111
	14	78.3	118	68.6	103	57.7	86.7	44.7	67.2	31.2	46.9	63.7	95.8
	15	70.2	105	61.7	92.8	52.1	78.3	40.5	60.8	28.4	42.6	55.5	83.5
	16	62.3	93.7	55.1	82.9	46.7	70.2	36.4	54.7	25.6	38.4	48.8	73.3
	17	55.2	83.0	48.8	73.4	41.5	62.4	32.4	48.7	22.9	34.4	43.2	65.0
	18	49.2	74.0	43.6	65.5	37.0	55.6	28.9	43.4	20.4	30.7	38.6	58.0
	19	44.2	66.4	39.1	58.8	33.2	49.9	25.9	39.0	18.3	27.5	34.6	52.0
	20	39.9	59.9	35.3	53.0	30.0	45.1	23.4	35.2	16.5	24.9	31.2	46.9
	21	36.2	54.4	32.0	48.1	27.2	40.9	21.2	31.9	15.0	22.5	28.3	42.6
	22	33.0	49.5	29.2	43.8	24.8	37.3	19.4	29.1	13.7	20.5	25.8	38.8
	23	30.2	45.3	26.7	40.1	22.7	34.1	17.7	26.6	12.5	18.8	23.6	35.5
	24	27.7	41.6	24.5	36.8	20.8	31.3	16.3	24.4	11.5	17.3		
	25	25.5	38.4	22.6	34.0	19.2	28.8	15.0	22.5	10.6	15.9		
	26	23.6	35.5	20.9	31.4	17.7	26.7	13.9	20.8	9.78	14.7		
	27	21.9	32.9	19.4	29.1	16.5	24.7	12.8	19.3	9.07	13.6		
	28			18.0	27.1	15.3	23.0	11.9	18.0	8.44	12.7		
	29							11.1	16.7	7.86	11.8		

Properties

	3/8	5/16	1/4	3/16	1/8[c]	HSS4×4×1/2
A_g, in.^{2}	5.48	4.68	3.84	2.93	2.00	6.02
$I_x = I_y$, in.^{4}	15.3	13.5	11.4	9.02	6.35	11.9
$r_x = r_y$, in.	1.67	1.70	1.73	1.75	1.78	1.41

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

HSS4

Table 4-4 (continued)
Available Strength in Axial Compression, kips
Square HSS

F_y = 50 ksi

Shape		HSS4×4×									
		3/8		5/16		1/4		3/16		1/8	
t_{des}, in.		0.349		0.291		0.233		0.174		0.116	
lb/ft		17.27		14.83		12.21		9.42		6.46	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	143	215	123	184	101	152	77.2	116	53.0	79.6
	1	142	214	122	184	100	151	76.9	116	52.8	79.3
	2	140	211	120	181	99.1	149	75.9	114	52.1	78.3
	3	137	206	118	177	96.8	146	74.3	112	51.0	76.7
	4	132	199	114	171	93.8	141	72.0	108	49.5	74.5
	5	127	190	109	164	90.0	135	69.2	104	47.7	71.7
	6	120	180	103	156	85.6	129	66.0	99.2	45.5	68.4
	7	113	169	97.3	146	80.7	121	62.3	93.7	43.1	64.8
	8	105	157	90.6	136	75.4	113	58.4	87.7	40.5	60.8
	9	96.4	145	83.6	126	69.8	105	54.2	81.4	37.7	56.6
	10	87.9	132	76.4	115	64.0	96.1	49.8	74.9	34.8	52.2
	11	79.4	119	69.2	104	58.1	87.4	45.5	68.3	31.8	47.8
	12	71.0	107	62.0	93.2	52.3	78.7	41.1	61.8	28.9	43.4
	13	62.8	94.4	55.1	82.8	46.7	70.2	36.8	55.4	26.0	39.1
	14	55.0	82.7	48.5	72.8	41.3	62.1	32.7	49.2	23.2	34.8
	15	47.9	72.0	42.2	63.5	36.1	54.3	28.8	43.2	20.5	30.8
	16	42.1	63.3	37.1	55.8	31.7	47.7	25.3	38.0	18.0	27.1
	17	37.3	56.1	32.9	49.4	28.1	42.3	22.4	33.6	16.0	24.0
	18	33.3	50.0	29.3	44.1	25.1	37.7	20.0	30.0	14.2	21.4
	19	29.9	44.9	26.3	39.6	22.5	33.8	17.9	26.9	12.8	19.2
	20	27.0	40.5	23.8	35.7	20.3	30.5	16.2	24.3	11.5	17.3
	21	24.4	36.7	21.5	32.4	18.4	27.7	14.7	22.1	10.5	15.7
	22	22.3	33.5	19.6	29.5	16.8	25.2	13.4	20.1	9.53	14.3
	23	20.4	30.6	18.0	27.0	15.4	23.1	12.2	18.4	8.72	13.1
	24	18.7	28.1	16.5	24.8	14.1	21.2	11.2	16.9	8.01	12.0
	25					13.0	19.5	10.4	15.6	7.38	11.1
	26									6.82	10.3

Properties

	3/8	5/16	1/4	3/16	1/8
A_g, in.^{2}	4.78	4.10	3.37	2.58	1.77
$I_x = I_y$, in.^{4}	10.3	9.14	7.80	6.21	4.40
$r_x = r_y$, in.	1.47	1.49	1.52	1.55	1.58

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

F_y = 50 ksi

Table 4-4 (continued)
Available Strength in Axial Compression, kips
Square HSS

HSS3½

Shape		HSS3½×3½×									
		3/8		5/16		1/4		3/16		1/8	
t_{des}, in.		0.349		0.291		0.233		0.174		0.116	
lb/ft		14.72		12.70		10.51		8.15		5.61	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	122	184	105	158	87.1	131	67.1	101	46.1	69.3
	1	122	183	105	157	86.6	130	66.7	100	45.8	68.9
	2	119	179	103	154	85.0	128	65.5	98.5	45.1	67.8
	3	115	173	99.6	150	82.5	124	63.7	95.7	43.8	65.9
	4	110	166	95.2	143	79.1	119	61.1	91.9	42.1	63.4
	5	104	156	90.0	135	74.9	113	58.0	87.2	40.1	60.2
	6	96.4	145	83.9	126	70.1	105	54.5	81.9	37.7	56.6
	7	88.5	133	77.3	116	64.8	97.4	50.5	75.9	35.0	52.6
	8	80.1	120	70.3	106	59.2	89.0	46.3	69.6	32.2	48.4
	9	71.6	108	63.1	94.9	53.4	80.3	42.0	63.1	29.3	44.0
	10	63.1	94.8	56.0	84.1	47.6	71.6	37.6	56.6	26.3	39.5
	11	54.9	82.5	49.0	73.7	41.9	63.0	33.3	50.1	23.4	35.2
	12	47.1	70.7	42.4	63.7	36.5	54.9	29.2	43.9	20.6	30.9
	13	40.1	60.3	36.2	54.4	31.3	47.1	25.2	37.9	17.9	26.8
	14	34.6	52.0	31.2	46.9	27.0	40.6	21.7	32.7	15.4	23.1
	15	30.1	45.3	27.2	40.8	23.5	35.4	18.9	28.5	13.4	20.2
	16	26.5	39.8	23.9	35.9	20.7	31.1	16.6	25.0	11.8	17.7
	17	23.5	35.2	21.2	31.8	18.3	27.5	14.7	22.2	10.4	15.7
	18	20.9	31.4	18.9	28.4	16.3	24.6	13.2	19.8	9.31	14.0
	19	18.8	28.2	16.9	25.5	14.7	22.0	11.8	17.7	8.36	12.6
	20	16.9	25.5	15.3	23.0	13.2	19.9	10.7	16.0	7.54	11.3
	21	15.4	23.1	13.9	20.8	12.0	18.0	9.66	14.5	6.84	10.3
	22					10.9	16.4	8.80	13.2	6.23	9.37
Properties											
A_g, in.^{2}		4.09		3.52		2.91		2.24		1.54	
$I_x = I_y$, in.^{4}		6.49		5.84		5.04		4.05		2.90	
$r_x = r_y$, in.		1.26		1.29		1.32		1.35		1.37	

ASD	LRFD	Note: Heavy line indicates L_c/r_y equal to or greater than 200.
$\Omega_c = 1.67$	$\phi_c = 0.90$	

Table 4-4 (continued)
Available Strength in Axial Compression, kips
Square HSS

HSS3

F_y = 50 ksi

Shape		HSS3×3× 3/8		5/16		1/4		3/16		1/8	
t_{des}, in.		0.349		0.291		0.233		0.174		0.116	
lb/ft		12.17		10.58		8.81		6.87		4.75	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	101	153	88.0	132	73.1	110	56.6	85.0	38.9	58.5
	1	101	151	87.2	131	72.4	109	56.1	84.4	38.6	58.1
	2	97.8	147	84.9	128	70.6	106	54.8	82.3	37.7	56.7
	3	93.3	140	81.2	122	67.6	102	52.6	79.1	36.3	54.6
	4	87.4	131	76.2	115	63.7	95.8	49.7	74.7	34.4	51.7
	5	80.3	121	70.2	106	59.0	88.7	46.2	69.5	32.1	48.3
	6	72.4	109	63.6	95.6	53.7	80.7	42.3	63.5	29.5	44.4
	7	64.1	96.4	56.6	85.0	48.1	72.2	38.0	57.2	26.7	40.1
	8	55.7	83.7	49.4	74.2	42.3	63.5	33.7	50.6	23.8	35.8
	9	47.5	71.4	42.4	63.7	36.6	55.0	29.4	44.1	20.9	31.4
	10	39.8	59.8	35.7	53.6	31.1	46.7	25.2	37.8	18.0	27.1
	11	32.9	49.4	29.6	44.5	25.9	39.0	21.2	31.8	15.3	23.1
	12	27.6	41.5	24.9	37.4	21.8	32.8	17.8	26.8	12.9	19.4
	13	23.5	35.4	21.2	31.8	18.6	27.9	15.2	22.8	11.0	16.5
	14	20.3	30.5	18.3	27.4	16.0	24.1	13.1	19.7	9.48	14.2
	15	17.7	26.6	15.9	23.9	13.9	21.0	11.4	17.1	8.26	12.4
	16	15.5	23.3	14.0	21.0	12.3	18.4	10.0	15.1	7.26	10.9
	17	13.8	20.7	12.4	18.6	10.9	16.3	8.87	13.3	6.43	9.66
	18			11.0	16.6	9.69	14.6	7.91	11.9	5.73	8.62
	19							7.10	10.7	5.15	7.73

Properties

	3/8	5/16	1/4	3/16	1/8
A_g, in.^{2}	3.39	2.94	2.44	1.89	1.30
$I_x = I_y$, in.^{4}	3.78	3.45	3.02	2.46	1.78
$r_x = r_y$, in.	1.06	1.08	1.11	1.14	1.17

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

Table 4-4 (continued)
Available Strength in Axial Compression, kips
Square HSS

F_y = 50 ksi

HSS2½–HSS2¼

Shape		HSS2½×2½×								HSS2¼×2¼×	
		5/16		1/4		3/16		1/8		1/4	
t_{des}, in.		0.291		0.233		0.174		0.116		0.233	
lb/ft		8.45		7.11		5.59		3.90		6.26	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	70.4	106	59.0	88.6	46.1	69.3	32.0	48.1	52.1	78.3
	1	69.4	104	58.2	87.5	45.6	68.5	31.7	47.6	51.3	77.0
	2	66.6	100	56.0	84.2	43.9	66.1	30.6	46.0	48.8	73.4
	3	62.3	93.6	52.6	79.0	41.4	62.2	28.9	43.5	45.0	67.7
	4	56.6	85.1	48.1	72.3	38.1	57.2	26.7	40.2	40.2	60.4
	5	50.1	75.3	42.9	64.4	34.2	51.3	24.1	36.3	34.7	52.2
	6	43.1	64.8	37.2	56.0	29.9	45.0	21.3	32.0	29.1	43.7
	7	36.1	54.3	31.5	47.4	25.6	38.5	18.4	27.7	23.5	35.4
	8	29.5	44.3	26.0	39.1	21.4	32.2	15.5	23.4	18.4	27.7
	9	23.5	35.2	20.9	31.5	17.4	26.2	12.8	19.3	14.6	21.9
	10	19.0	28.6	17.0	25.5	14.1	21.2	10.4	15.6	11.8	17.7
	11	15.7	23.6	14.0	21.1	11.7	17.5	8.60	12.9	9.75	14.7
	12	13.2	19.8	11.8	17.7	9.80	14.7	7.22	10.9	8.19	12.3
	13	11.2	16.9	10.0	15.1	8.35	12.6	6.15	9.25	6.98	10.5
	14	9.69	14.6	8.65	13.0	7.20	10.8	5.31	7.98		
	15			7.53	11.3	6.27	9.43	4.62	6.95		
	16							4.06	6.11		

Properties

	HSS2½×2½×5/16	HSS2½×2½×1/4	HSS2½×2½×3/16	HSS2½×2½×1/8	HSS2¼×2¼×1/4
A_g, in.^{2}	2.35	1.97	1.54	1.07	1.74
$I_x = I_y$, in.^{4}	1.82	1.63	1.35	0.998	1.13
$r_x = r_y$, in.	0.880	0.908	0.937	0.965	0.806

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

Table 4-4 (continued)
Available Strength in Axial Compression, kips
Square HSS

HSS2¼–HSS2

F_y = 50 ksi

Shape		HSS2¼×2¼×				HSS2×2×					
		3/16		1/8		1/4		3/16		1/8	
t_{des}, in.		0.174		0.116		0.233		0.174		0.116	
lb/ft		4.96		3.48		5.41		4.32		3.05	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_y	0	41.0	61.6	28.6	43.0	45.2	67.9	35.6	53.5	25.1	37.8
	1	40.4	60.7	28.2	42.4	44.3	66.5	34.9	52.5	24.7	37.1
	2	38.6	58.0	27.0	40.7	41.5	62.4	32.9	49.5	23.4	35.1
	3	35.8	53.8	25.2	37.9	37.3	56.1	29.9	44.9	21.4	32.1
	4	32.2	48.4	22.8	34.3	32.2	48.4	26.0	39.1	18.8	28.3
	5	28.1	42.3	20.1	30.2	26.6	40.0	21.8	32.8	16.0	24.0
	6	23.8	35.8	17.2	25.9	21.0	31.6	17.6	26.4	13.1	19.6
	7	19.6	29.4	14.3	21.5	15.9	24.0	13.6	20.5	10.3	15.5
	8	15.6	23.4	11.6	17.4	12.2	18.3	10.4	15.7	7.93	11.9
	9	12.3	18.5	9.18	13.8	9.64	14.5	8.24	12.4	6.27	9.42
	10	9.97	15.0	7.43	11.2	7.81	11.7	6.67	10.0	5.08	7.63
	11	8.24	12.4	6.14	9.23	6.46	9.70	5.52	8.29	4.20	6.31
	12	6.92	10.4	5.16	7.76			4.63	6.97	3.53	5.30
	13	5.90	8.87	4.40	6.61						
	14			3.79	5.70						
Properties											
A_g, in.^{2}		1.37		0.956		1.51		1.19		0.840	
$I_x = I_y$, in.^{4}		0.953		0.712		0.747		0.641		0.486	
$r_x = r_y$, in.		0.835		0.863		0.704		0.733		0.761	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates L_c/r_y equal to or greater than 200.

F_y = 46 ksi

Table 4-5
Available Strength in Axial Compression, kips
Round HSS

HSS20.000–HSS16.000

Shape		HSS20.000×				HSS18.000×				HSS16.000×			
		0.500		0.375		0.500		0.375		0.625		0.500	
t_{des}, in.		0.465		0.349		0.465		0.349		0.581		0.465	
lb/ft		104.00		78.67		93.54		70.66		103.00		82.85	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to radius of gyration, r	0	785	1180	592	890	705	1060	534	803	774	1160	625	940
	6	779	1170	588	884	699	1050	530	796	765	1150	618	929
	7	777	1170	586	881	696	1050	528	793	762	1140	615	925
	8	775	1160	585	879	694	1040	526	790	758	1140	613	921
	9	772	1160	583	876	691	1040	524	787	754	1130	609	916
	10	769	1160	580	872	688	1030	521	783	749	1130	605	910
	11	766	1150	578	869	684	1030	519	779	744	1120	601	904
	12	762	1150	575	865	680	1020	516	775	739	1110	597	897
	13	759	1140	572	860	676	1020	512	770	733	1100	592	890
	14	754	1130	569	856	671	1010	509	765	726	1090	587	882
	15	750	1130	566	851	666	1000	505	759	719	1080	582	874
	16	745	1120	563	846	661	994	501	754	712	1070	576	866
	17	740	1110	559	840	656	985	497	747	705	1060	570	856
	18	735	1100	555	834	650	977	493	741	697	1050	563	847
	19	730	1100	551	828	644	968	488	734	688	1030	557	837
	20	724	1090	547	821	638	958	484	727	680	1020	550	826
	21	718	1080	542	815	631	948	479	720	671	1010	543	816
	22	712	1070	537	808	624	938	474	712	661	994	535	804
	23	705	1060	533	801	617	928	468	704	652	980	528	793
	24	698	1050	528	793	610	917	463	696	642	965	520	781
	25	692	1040	522	785	602	905	457	688	632	950	511	769
	26	684	1030	517	777	595	894	452	679	621	934	503	756
	27	677	1020	512	769	587	882	446	670	611	918	495	743
	28	670	1010	506	761	579	870	440	661	600	902	486	730
	29	662	995	500	752	570	857	433	652	589	885	477	717
	30	654	983	494	743	562	845	427	642	578	868	468	704
	32	638	959	482	725	545	819	414	623	555	834	450	676
	34	621	933	470	706	527	792	401	602	532	799	431	648
	36	604	907	457	686	509	765	387	582	508	764	412	620
	38	586	880	443	666	490	737	373	561	484	728	393	591
	40	567	853	430	646	471	708	359	539	460	692	374	562

Properties

	HSS20.000×0.500	HSS20.000×0.375	HSS18.000×0.500	HSS18.000×0.375	HSS16.000×0.625	HSS16.000×0.500
A_g, in.^{2}	28.5	21.5	25.6	19.4	28.1	22.7
I, in.^{4}	1360	1040	985	754	838	685
r, in.	6.91	6.95	6.20	6.24	5.46	5.49

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

HSS16.000–
HSS14.000

Table 4-5 (continued)
Available Strength in Axial Compression, kips
Round HSS

F_y = 46 ksi

Shape		HSS16.000×								HSS14.000×			
		0.438		0.375		0.312		0.250		0.625		0.500	
t_{des}, in.		0.407		0.349		0.291		0.233		0.581		0.465	
lb/ft		72.87		62.64		52.32		42.09		89.36		72.16	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to radius of gyration, r	0	548	824	474	712	397	596	317	476	675	1010	545	820
	6	542	814	468	704	392	589	313	471	665	999	537	807
	7	540	811	466	701	391	587	312	469	661	993	534	803
	8	537	807	464	698	389	584	311	467	657	987	531	798
	9	534	803	462	694	387	581	309	464	652	980	527	792
	10	531	798	459	690	384	578	307	462	646	972	523	786
	11	527	793	456	685	382	574	305	459	641	963	518	779
	12	524	787	453	680	379	570	303	455	634	953	513	771
	13	519	781	449	675	376	565	301	452	628	943	508	763
	14	515	774	445	669	373	561	298	448	620	932	502	755
	15	510	767	441	663	370	555	295	444	613	921	496	745
	16	505	759	437	657	366	550	293	440	605	909	490	736
	17	500	751	432	650	362	544	290	435	596	896	483	726
	18	494	743	428	643	358	538	286	430	587	883	476	715
	19	489	734	423	635	354	532	283	426	578	869	468	704
	20	482	725	417	627	350	526	280	420	568	854	461	692
	21	476	716	412	619	345	519	276	415	558	839	453	680
	22	470	706	406	611	341	512	272	410	548	824	445	668
	23	463	696	401	602	336	505	269	404	538	808	436	656
	24	456	686	395	593	331	497	265	398	527	792	428	643
	25	449	675	389	584	326	490	261	392	516	776	419	630
	26	442	664	382	575	321	482	257	386	505	759	410	616
	27	434	653	376	565	315	474	252	379	493	742	401	603
	28	427	642	370	555	310	466	248	373	482	724	392	589
	29	419	630	363	546	304	458	244	366	470	707	382	575
	30	411	618	356	535	299	449	239	360	459	689	373	561
	32	395	594	343	515	287	432	230	346	435	653	354	532
	34	379	570	329	494	276	414	221	332	411	617	335	503
	36	363	545	314	472	264	397	212	318	387	581	316	474
	38	346	520	300	451	252	379	202	304	363	546	296	446
	40	329	494	285	429	240	360	193	289	340	510	278	417

Properties

	0.438	0.375	0.312	0.250	0.625	0.500
A_g, in.^{2}	19.9	17.2	14.4	11.5	24.5	19.8
I, in.^{4}	606	526	443	359	552	453
r, in.	5.51	5.53	5.55	5.58	4.75	4.79

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

F_y = 46 ksi

Table 4-5 (continued)
Available Strength in Axial Compression, kips
Round HSS

HSS14.000–HSS12.750

Shape		HSS14.000×						HSS12.750×					
		0.375		0.312		0.250		0.500		0.375		0.250	
t_{des}, in.		0.349		0.291		0.233		0.465		0.349		0.233	
lb/ft		54.62		45.65		36.75		65.48		49.61		33.41	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to radius of gyration, r	0	413	621	344	517	278	418	493	741	375	563	252	379
	6	407	612	339	510	274	412	484	728	368	553	248	373
	7	405	608	337	507	273	410	481	723	365	549	246	370
	8	402	605	335	504	271	407	477	717	363	545	244	367
	9	400	600	333	501	269	405	473	711	360	541	242	364
	10	396	596	330	497	267	401	468	704	356	535	240	361
	11	393	591	328	492	265	398	463	697	353	530	238	357
	12	389	585	324	488	262	394	458	688	348	524	235	353
	13	385	579	321	483	260	390	452	680	344	517	232	349
	14	381	572	318	477	257	386	446	670	339	510	229	344
	15	376	566	314	472	254	381	439	660	335	503	226	339
	16	372	558	310	466	251	377	432	650	329	495	222	334
	17	366	551	306	459	247	372	425	639	324	487	219	329
	18	361	543	301	453	244	366	418	628	318	478	215	323
	19	356	535	297	446	240	361	410	616	312	470	211	317
	20	350	526	292	439	236	355	402	604	306	460	207	311
	21	344	517	287	432	232	349	393	591	300	451	203	305
	22	338	508	282	424	228	343	385	578	294	441	199	299
	23	332	499	277	416	224	337	376	565	287	432	194	292
	24	325	489	272	408	220	330	367	552	280	422	190	285
	25	319	479	266	400	216	324	358	538	274	411	185	279
	26	312	469	261	392	211	317	349	524	267	401	181	272
	27	305	459	255	383	207	310	339	510	260	390	176	265
	28	298	448	249	375	202	304	330	496	253	380	171	258
	29	291	438	244	366	197	297	321	482	245	369	167	250
	30	284	427	238	357	193	290	311	467	238	358	162	243
	32	270	406	226	339	183	275	292	439	224	337	152	229
	34	256	384	214	321	174	261	273	410	210	315	143	214
	36	241	363	202	303	164	246	254	382	195	294	133	200
	38	227	341	190	286	154	232	235	354	181	272	124	186
	40	213	320	178	268	145	218	217	327	168	252	115	172
Properties													
A_g, in.^{2}		15.0		12.5		10.1		17.9		13.6		9.16	
I, in.^{4}		349		295		239		339		262		180	
r, in.		4.83		4.85		4.87		4.35		4.39		4.43	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

HSS10.750–
HSS10.000

Table 4-5 (continued)
Available Strength in Axial Compression, kips
Round HSS

F_y = 46 ksi

Shape	HSS10.750×						HSS10.000×					
	0.500		0.375		0.250		0.625		0.500		0.375	
t_{des}, in.	0.465		0.349		0.233		0.581		0.465		0.349	
lb/ft	54.79		41.59		28.06		62.64		50.78		38.58	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to radius of gyration, r												
0	413	621	314	472	212	319	474	712	383	575	292	439
6	402	605	306	460	207	311	459	690	371	558	283	426
7	399	599	303	456	205	308	454	682	367	552	280	421
8	394	593	300	451	203	305	448	674	363	545	277	416
9	389	585	296	445	200	301	442	664	357	537	273	410
10	384	577	292	439	198	297	434	653	352	529	269	404
11	378	568	288	433	195	293	427	641	346	519	264	397
12	372	559	283	426	192	288	418	628	339	509	259	389
13	365	549	278	418	188	283	409	615	332	499	254	381
14	358	538	273	410	185	278	400	601	324	487	248	373
15	351	527	267	402	181	272	390	586	316	476	242	364
16	343	515	261	393	177	266	379	570	308	463	236	355
17	334	503	255	384	173	260	369	554	300	450	230	345
18	326	490	249	374	169	254	358	537	291	437	223	335
19	317	477	243	365	165	248	346	520	282	424	216	325
20	308	464	236	355	160	241	335	503	273	410	209	314
21	299	450	229	344	156	234	323	486	263	396	202	304
22	290	436	222	334	151	227	311	468	254	382	195	293
23	281	422	215	323	146	220	299	450	244	367	188	282
24	271	408	208	313	142	213	287	432	235	353	181	272
25	262	393	201	302	137	206	275	414	225	339	173	261
26	252	379	194	291	132	199	263	396	216	324	166	250
27	242	364	186	280	127	191	252	378	206	310	159	239
28	233	350	179	269	123	184	240	360	197	296	152	228
29	223	336	172	259	118	177	228	343	188	282	145	218
30	214	322	165	248	113	170	217	326	179	268	138	207
32	195	294	151	227	104	156	195	293	161	242	124	187
34	177	267	137	206	94.4	142	173	260	143	216	111	167
36	160	241	124	187	85.6	129	155	232	128	192	99.3	149
38	144	216	112	168	77.0	116	139	208	115	173	89.1	134
40	130	195	101	151	69.5	104	125	188	104	156	80.4	121
Properties												
A_g, in.^{2}	15.0		11.4		7.70		17.2		13.9		10.6	
I, in.^{4}	199		154		106		191		159		123	
r, in.	3.64		3.68		3.72		3.34		3.38		3.41	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

F_y = 46 ksi

Table 4-5 (continued)
Available Strength in Axial Compression, kips
Round HSS

HSS10.000–HSS9.625

Shape	HSS10.000×						HSS9.625×					
	0.312		0.250		0.188		0.500		0.375		0.312	
t_{des}, in.	0.291		0.233		0.174		0.465		0.349		0.291	
lb/ft	32.31		26.06		19.72		48.77		37.08		31.06	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to radius of gyration, r												
0	245	368	197	296	148	222	369	555	281	422	235	353
6	237	357	191	287	144	216	357	537	272	409	228	342
7	235	353	189	284	142	214	353	530	269	404	225	338
8	232	349	187	281	140	211	348	523	265	399	222	334
9	229	344	184	277	139	208	343	515	261	393	219	329
10	225	339	182	273	136	205	337	506	257	386	215	323
11	221	333	178	268	134	202	330	496	252	379	211	317
12	217	327	175	263	132	198	323	486	247	371	207	311
13	213	320	172	258	129	194	316	475	241	363	202	304
14	208	313	168	252	126	190	308	463	236	354	197	297
15	203	305	164	246	123	186	300	451	229	345	192	289
16	198	298	160	240	120	181	291	438	223	335	187	281
17	193	290	156	234	117	176	283	425	217	326	182	273
18	187	282	151	227	114	171	274	411	210	315	176	265
19	182	273	147	221	111	166	265	398	203	305	170	256
20	176	264	142	214	107	161	255	384	196	295	165	247
21	170	256	138	207	104	156	246	369	189	284	159	239
22	164	247	133	200	100	151	236	355	182	273	153	230
23	158	238	128	192	96.6	145	227	340	174	262	147	221
24	152	229	123	185	93.1	140	217	326	167	251	141	212
25	146	220	118	178	89.5	134	207	312	160	241	135	203
26	140	211	114	171	85.9	129	198	297	153	230	129	194
27	134	202	109	164	82.3	124	188	283	146	219	123	185
28	128	193	104	156	78.7	118	179	269	139	208	117	176
29	122	184	99.3	149	75.2	113	170	255	132	198	111	167
30	117	175	94.7	142	71.7	108	161	242	125	188	106	159
32	105	158	85.6	129	64.9	97.5	143	216	112	168	94.5	142
34	94.3	142	76.8	115	58.4	87.7	127	191	99.1	149	83.9	126
36	84.1	126	68.5	103	52.1	78.3	113	170	88.4	133	74.8	112
38	75.5	114	61.5	92.5	46.7	70.2	102	153	79.3	119	67.1	101
40	68.2	102	55.5	83.4	42.2	63.4	91.8	138	71.6	108	60.6	91.1
Properties												
A_g, in.^{2}	8.88		7.15		5.37		13.4		10.2		8.53	
I, in.^{4}	105		85.3		64.8		141		110		93.0	
r, in.	3.43		3.45		3.47		3.24		3.28		3.30	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Table 4-5 (continued)
Available Strength in Axial Compression, kips
Round HSS

F_y = 46 ksi

HSS9.625–HSS8.625

Shape		HSS9.625×				HSS8.625×							
		0.250		0.188		0.625		0.500		0.375		0.322	
t_{des}, in.		0.233		0.174		0.581		0.465		0.349		0.300	
lb/ft		25.06		18.97		53.45		43.43		33.07		28.58	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to radius of gyration, r	0	189	284	142	214	405	609	328	493	250	375	216	325
	6	183	276	138	207	388	583	314	473	240	361	208	312
	7	181	272	136	205	382	574	310	465	236	355	205	308
	8	179	269	135	202	375	564	304	457	232	349	201	303
	9	176	265	133	200	368	553	298	448	228	343	198	297
	10	173	260	131	196	359	540	292	439	223	335	193	291
	11	170	256	128	193	351	527	285	428	218	328	189	284
	12	167	251	126	189	341	513	277	417	212	319	184	277
	13	163	245	123	185	331	497	269	405	206	310	179	269
	14	159	239	120	181	321	482	261	392	200	301	174	261
	15	155	233	117	176	310	465	252	380	194	291	168	253
	16	151	227	114	171	298	448	244	366	187	281	163	244
	17	147	221	111	167	287	431	234	352	180	271	157	236
	18	142	214	107	162	275	414	225	338	173	261	151	227
	19	138	207	104	156	263	396	216	324	166	250	145	217
	20	133	200	101	151	251	378	206	310	159	239	139	208
	21	128	193	97.1	146	239	360	197	295	152	228	132	199
	22	124	186	93.5	141	227	342	187	281	145	217	126	190
	23	119	179	90.0	135	215	324	177	267	138	207	120	180
	24	114	171	86.4	130	204	306	168	253	130	196	114	171
	25	109	164	82.8	124	192	289	159	239	123	185	108	162
	26	104	157	79.2	119	181	272	150	225	117	175	102	153
	27	99.7	150	75.6	114	170	255	141	212	110	165	96.1	144
	28	95.0	143	72.1	108	159	239	132	198	103	155	90.3	136
	29	90.4	136	68.6	103	148	223	123	185	96.6	145	84.8	127
	30	85.8	129	65.2	98.0	138	208	115	173	90.3	136	79.2	119
	32	76.9	116	58.5	88.0	122	183	101	152	79.4	119	69.6	105
	34	68.4	103	52.1	78.3	108	162	89.7	135	70.3	106	61.7	92.7
	36	61.0	91.7	46.5	69.8	96.2	145	80.0	120	62.7	94.3	55.0	82.7
	38	54.7	82.3	41.7	62.7	86.3	130	71.8	108	56.3	84.6	49.4	74.2
	40	49.4	74.2	37.6	56.6	77.9	117	64.8	97.5	50.8	76.3	44.6	67.0

Properties

Property	0.250	0.188	0.625	0.500	0.375	0.322
A_g, in.^{2}	6.87	5.17	14.7	11.9	9.07	7.85
I, in.^{4}	75.9	57.7	119	100	77.8	68.1
r, in.	3.32	3.34	2.85	2.89	2.93	2.95

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

F_y = 46 ksi

Table 4-5 (continued)
Available Strength in Axial Compression, kips
Round HSS

HSS8.625–HSS7.500

Shape		HSS8.625×				HSS7.625×				HSS7.500×			
		0.250		0.188		0.375		0.328		0.500		0.375	
t_{des}, in.		0.233		0.174		0.349		0.305		0.465		0.349	
lb/ft		22.38		16.96		29.06		25.59		37.42		28.56	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to radius of gyration, r	0	169	254	127	191	220	330	193	290	284	426	216	325
	6	163	244	122	184	209	314	183	276	268	403	205	307
	7	160	241	121	181	205	308	180	270	263	395	201	301
	8	158	237	119	178	200	301	176	265	257	386	196	295
	9	155	233	117	175	195	294	172	258	250	376	191	287
	10	152	228	114	172	190	286	167	251	243	365	186	279
	11	148	223	112	168	184	277	162	244	235	353	180	270
	12	144	217	109	164	178	268	157	236	227	341	174	261
	13	140	211	106	159	172	258	151	227	218	327	167	251
	14	136	205	103	155	165	248	145	219	209	314	161	241
	15	132	199	99.7	150	158	238	140	210	200	300	154	231
	16	128	192	96.4	145	151	228	133	201	190	286	147	220
	17	123	185	93.0	140	144	217	127	191	181	271	139	210
	18	118	178	89.6	135	137	206	121	182	171	257	132	199
	19	114	171	86.1	129	130	195	115	172	161	243	125	188
	20	109	164	82.5	124	123	185	108	163	152	228	118	177
	21	104	157	78.9	119	116	174	102	154	142	214	111	167
	22	99.4	149	75.3	113	109	163	96.0	144	133	200	104	156
	23	94.6	142	71.7	108	102	153	90.0	135	124	187	97.0	146
	24	89.8	135	68.2	102	95.1	143	84.0	126	115	173	90.3	136
	25	85.1	128	64.7	97.2	88.5	133	78.3	118	107	160	83.8	126
	26	80.5	121	61.2	91.9	82.0	123	72.6	109	98.6	148	77.5	116
	27	76.0	114	57.8	86.8	76.1	114	67.3	101	91.4	137	71.9	108
	28	71.5	107	54.4	81.8	70.7	106	62.6	94.1	85.0	128	66.8	100
	29	67.2	101	51.2	76.9	65.9	99.1	58.4	87.7	79.3	119	62.3	93.6
	30	62.8	94.4	47.9	72.0	61.6	92.6	54.5	82.0	74.1	111	58.2	87.5
	32	55.2	83.0	42.1	63.3	54.1	81.4	47.9	72.0	65.1	97.8	51.2	76.9
	34	48.9	73.5	37.3	56.1	48.0	72.1	42.5	63.8	57.7	86.7	45.3	68.1
	36	43.6	65.6	33.3	50.0	42.8	64.3	37.9	56.9	51.4	77.3	40.4	60.7
	38	39.2	58.8	29.9	44.9	38.4	57.7	34.0	51.1	46.2	69.4	36.3	54.5
	40	35.3	53.1	26.9	40.5	34.7	52.1	30.7	46.1	41.7	62.6	32.7	49.2

Properties

Property	HSS8.625×0.250	HSS8.625×0.188	HSS7.625×0.375	HSS7.625×0.328	HSS7.500×0.500	HSS7.500×0.375
A_g, in.^{2}	6.14	4.62	7.98	7.01	10.3	7.84
I, in.^{4}	54.1	41.3	52.9	47.1	63.9	50.2
r, in.	2.97	2.99	2.58	2.59	2.49	2.53

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Table 4-5 (continued)
Available Strength in Axial Compression, kips
Round HSS

HSS7.500–HSS7.000

F_y = 46 ksi

Shape		HSS7.500×						HSS7.000×					
		0.312		0.250		0.188		0.500		0.375		0.312	
t_{des}, in.		0.291		0.233		0.174		0.465		0.349		0.291	
lb/ft		23.97		19.38		14.70		34.74		26.56		22.31	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to radius of gyration, r	0	182	273	147	220	110	166	263	395	201	302	169	254
	6	172	259	139	209	105	157	247	371	189	283	159	239
	7	169	254	136	205	103	154	241	362	184	277	155	233
	8	165	248	133	201	100	151	234	352	179	270	151	227
	9	161	242	130	196	98.0	147	227	342	174	262	147	221
	10	156	235	127	190	95.4	143	220	330	168	253	142	214
	11	152	228	123	184	92.5	139	212	318	162	244	137	206
	12	146	220	119	178	89.5	135	203	305	156	234	132	198
	13	141	212	114	172	86.3	130	194	292	149	224	126	190
	14	136	204	110	165	83.0	125	185	278	142	214	120	181
	15	130	195	105	158	79.6	120	175	264	135	203	115	172
	16	124	186	101	151	76.1	114	166	249	128	193	109	163
	17	118	177	95.9	144	72.6	109	156	235	121	182	103	154
	18	112	168	91.1	137	69.0	104	147	221	114	171	96.6	145
	19	106	159	86.3	130	65.4	98.3	137	206	107	160	90.6	136
	20	100	150	81.5	123	61.8	92.9	128	192	99.6	150	84.7	127
	21	94.1	141	76.7	115	58.3	87.6	119	179	92.6	139	78.9	119
	22	88.3	133	72.1	108	54.8	82.3	110	165	85.9	129	73.3	110
	23	82.5	124	67.5	101	51.3	77.1	101	152	79.4	119	67.8	102
	24	77.0	116	63.0	94.6	48.0	72.1	93.1	140	73.0	110	62.4	93.8
	25	71.5	108	58.6	88.1	44.7	67.2	85.8	129	67.2	101	57.5	86.4
	26	66.2	99.4	54.3	81.5	41.4	62.3	79.4	119	62.2	93.4	53.2	79.9
	27	61.4	92.2	50.3	75.6	38.4	57.7	73.6	111	57.6	86.6	49.3	74.1
	28	57.1	85.7	46.8	70.3	35.7	53.7	68.4	103	53.6	80.6	45.8	68.9
	29	53.2	79.9	43.6	65.5	33.3	50.1	63.8	95.9	50.0	75.1	42.7	64.2
	30	49.7	74.7	40.8	61.3	31.1	46.8	59.6	89.6	46.7	70.2	39.9	60.0
	32	43.7	65.7	35.8	53.8	27.4	41.1	52.4	78.8	41.0	61.7	35.1	52.8
	34	38.7	58.2	31.7	47.7	24.2	36.4	46.4	69.8	36.4	54.6	31.1	46.7
	36	34.5	51.9	28.3	42.5	21.6	32.5	41.4	62.2	32.4	48.7	27.7	41.7
	38	31.0	46.6	25.4	38.2	19.4	29.2	37.2	55.8	29.1	43.7	24.9	37.4
	40	28.0	42.0	22.9	34.5	17.5	26.3						

Properties

	HSS7.500×0.312	HSS7.500×0.250	HSS7.500×0.188	HSS7.000×0.500	HSS7.000×0.375	HSS7.000×0.312
A_g, in.^{2}	6.59	5.32	4.00	9.55	7.29	6.13
I, in.^{4}	42.9	35.2	26.9	51.2	40.4	34.6
r, in.	2.55	2.57	2.59	2.32	2.35	2.37

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 4-5 (continued)
Available Strength in Axial Compression, kips
Round HSS

F_y = 46 ksi

HSS7.000–HSS6.875

Shape		HSS7.000×						HSS6.875×					
		0.250		0.188		0.125		0.500		0.375		0.312	
t_{des}, in.		0.233		0.174		0.116		0.465		0.349		0.291	
lb/ft		18.04		13.69		9.19		34.07		26.06		21.89	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to radius of gyration, r	0	136	205	103	154	69.1	104	258	388	197	296	166	249
	6	128	193	96.8	145	65.2	98.0	241	362	185	278	156	234
	7	125	189	94.7	142	63.8	95.9	235	353	180	271	152	228
	8	122	184	92.3	139	62.2	93.6	229	344	176	264	148	222
	9	119	179	89.8	135	60.5	91.0	221	333	170	256	144	216
	10	115	173	87.0	131	58.7	88.2	214	321	164	247	139	209
	11	111	167	84.0	126	56.7	85.2	205	309	158	238	134	201
	12	107	161	80.8	121	54.6	82.1	197	296	152	228	128	193
	13	102	154	77.5	116	52.4	78.8	188	282	145	218	123	184
	14	97.8	147	74.1	111	50.1	75.3	178	268	138	208	117	176
	15	93.1	140	70.6	106	47.8	71.8	169	254	131	197	111	167
	16	88.3	133	67.0	101	45.4	68.3	159	239	124	186	105	158
	17	83.5	126	63.4	95.4	43.0	64.7	150	225	117	175	99.0	149
	18	78.7	118	59.9	90.0	40.6	61.1	140	211	110	165	93.0	140
	19	73.9	111	56.3	84.6	38.2	57.5	131	197	102	154	87.1	131
	20	69.2	104	52.7	79.2	35.9	53.9	122	183	95.4	143	81.2	122
	21	64.5	97.0	49.2	74.0	33.5	50.4	113	169	88.6	133	75.5	113
	22	60.0	90.2	45.8	68.9	31.3	47.0	104	156	81.9	123	69.9	105
	23	55.6	83.6	42.5	63.9	29.0	43.6	95.2	143	75.4	113	64.5	96.9
	24	51.2	77.0	39.3	59.0	26.9	40.4	87.4	131	69.2	104	59.2	89.0
	25	47.2	71.0	36.2	54.4	24.8	37.2	80.6	121	63.8	95.9	54.6	82.0
	26	43.7	65.6	33.5	50.3	22.9	34.4	74.5	112	59.0	88.7	50.5	75.8
	27	40.5	60.8	31.0	46.6	21.2	31.9	69.1	104	54.7	82.2	46.8	70.3
	28	37.6	56.6	28.8	43.4	19.7	29.7	64.2	96.5	50.9	76.5	43.5	65.4
	29	35.1	52.7	26.9	40.4	18.4	27.6	59.9	90.0	47.4	71.3	40.6	61.0
	30	32.8	49.3	25.1	37.8	17.2	25.8	55.9	84.1	44.3	66.6	37.9	57.0
	32	28.8	43.3	22.1	33.2	15.1	22.7	49.2	73.9	38.9	58.5	33.3	50.1
	34	25.5	38.4	19.6	29.4	13.4	20.1	43.5	65.5	34.5	51.9	29.5	44.4
	36	22.8	34.2	17.4	26.2	11.9	17.9	38.8	58.4	30.8	46.2	26.3	39.6
	38	20.4	30.7	15.7	23.5	10.7	16.1			27.6	41.5	23.6	35.5
	40			14.1	21.2	9.67	14.5						
Properties													
A_g, in.^{2}		4.95		3.73		2.51		9.36		7.16		6.02	
I, in.^{4}		28.4		21.7		14.9		48.3		38.2		32.7	
r, in.		2.39		2.41		2.43		2.27		2.31		2.33	

ASD	LRFD	
$\Omega_c = 1.67$	$\phi_c = 0.90$	Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 4-5 (continued)
Available Strength in Axial Compression, kips
Round HSS

F_y = 46 ksi

HSS6.875–HSS6.625

Shape		HSS6.875×				HSS6.625×					
		0.250		0.188		0.500		0.432		0.375	
t_{des}, in.		0.233		0.174		0.465		0.402		0.349	
lb/ft		17.71		13.44		32.74		28.60		25.06	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to radius of gyration, r	0	134	201	101	152	248	373	217	325	190	285
	6	126	189	94.7	142	230	346	201	303	177	265
	7	123	185	92.6	139	224	337	196	295	172	259
	8	120	180	90.3	136	218	327	190	286	167	251
	9	116	175	87.7	132	210	316	184	277	162	243
	10	112	169	84.8	128	202	304	177	266	156	234
	11	108	163	81.8	123	194	291	170	255	149	225
	12	104	156	78.6	118	185	278	162	244	143	215
	13	99.5	150	75.3	113	176	264	154	232	136	204
	14	94.9	143	71.9	108	166	250	146	220	129	194
	15	90.2	136	68.4	103	157	236	138	207	122	183
	16	85.4	128	64.8	97.4	147	221	130	195	115	172
	17	80.6	121	61.2	92.1	138	207	121	182	107	161
	18	75.8	114	57.7	86.7	128	193	113	170	100	151
	19	71.1	107	54.1	81.3	119	179	105	158	93.2	140
	20	66.4	99.8	50.6	76.0	110	165	97.2	146	86.3	130
	21	61.8	92.8	47.1	70.8	101	152	89.6	135	79.7	120
	22	57.3	86.1	43.8	65.8	92.2	139	82.0	123	73.1	110
	23	52.9	79.6	40.5	60.9	84.4	127	75.1	113	66.9	101
	24	48.6	73.1	37.3	56.0	77.5	116	68.9	104	61.4	92.4
	25	44.8	67.4	34.3	51.6	71.4	107	63.5	95.5	56.6	85.1
	26	41.4	62.3	31.7	47.7	66.0	99.3	58.7	88.3	52.4	78.7
	27	38.4	57.8	29.4	44.2	61.2	92.0	54.5	81.9	48.5	73.0
	28	35.7	53.7	27.4	41.1	56.9	85.6	50.6	76.1	45.1	67.9
	29	33.3	50.1	25.5	38.3	53.1	79.8	47.2	71.0	42.1	63.3
	30	31.1	46.8	23.8	35.8	49.6	74.6	44.1	66.3	39.3	59.1
	32	27.4	41.1	21.0	31.5	43.6	65.5	38.8	58.3	34.6	51.9
	34	24.2	36.4	18.6	27.9	38.6	58.0	34.4	51.6	30.6	46.0
	36	21.6	32.5	16.6	24.9	34.4	51.8	30.6	46.1	27.3	41.0
	38	19.4	29.2	14.9	22.3						
Properties											
A_g, in.^{2}		4.86		3.66		9.00		7.86		6.88	
I, in.^{4}		26.8		20.6		42.9		38.2		34.0	
r, in.		2.35		2.37		2.18		2.20		2.22	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 46 ksi

Table 4-5 (continued)
Available Strength in Axial Compression, kips
Round HSS

HSS6.625

Shape	HSS6.625×									
	0.312		0.280		0.250		0.188		0.125	
t_{des}, in.	0.291		0.260		0.233		0.174		0.116	
lb/ft	21.06		18.99		17.04		12.94		8.69	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to radius of gyration, r										
0	159	240	143	215	129	194	97.2	146	65.3	98.1
6	149	224	134	201	120	181	90.9	137	61.1	91.9
7	145	218	130	196	117	177	88.7	133	59.7	89.7
8	141	212	127	190	114	172	86.3	130	58.1	87.3
9	136	205	123	184	111	166	83.6	126	56.3	84.6
10	131	198	118	178	107	160	80.7	121	54.4	81.7
11	126	190	114	171	102	154	77.6	117	52.3	78.6
12	121	182	109	163	98.1	147	74.4	112	50.2	75.4
13	115	173	104	156	93.6	141	71.0	107	47.9	72.0
14	109	164	98.4	148	88.9	134	67.5	101	45.6	68.5
15	103	155	93.1	140	84.1	126	63.9	96.1	43.2	65.0
16	97.3	146	87.8	132	79.3	119	60.3	90.7	40.9	61.4
17	91.3	137	82.4	124	74.5	112	56.7	85.3	38.5	57.8
18	85.3	128	77.1	116	69.7	105	53.2	79.9	36.1	54.2
19	79.4	119	71.8	108	65.0	97.7	49.6	74.6	33.7	50.7
20	73.7	111	66.6	100	60.4	90.7	46.1	69.4	31.4	47.2
21	68.1	102	61.6	92.6	55.9	84.0	42.7	64.3	29.1	43.8
22	62.7	94.2	56.7	85.3	51.5	77.4	39.5	59.3	26.9	40.4
23	57.3	86.2	51.9	78.1	47.2	70.9	36.2	54.4	24.7	37.2
24	52.6	79.1	47.7	71.7	43.3	65.1	33.3	50.0	22.7	34.1
25	48.5	72.9	44.0	66.1	39.9	60.0	30.6	46.1	20.9	31.5
26	44.9	67.4	40.6	61.1	36.9	55.5	28.3	42.6	19.4	29.1
27	41.6	62.5	37.7	56.7	34.2	51.4	26.3	39.5	18.0	27.0
28	38.7	58.1	35.0	52.7	31.8	47.8	24.4	36.7	16.7	25.1
29	36.1	54.2	32.7	49.1	29.7	44.6	22.8	34.2	15.6	23.4
30	33.7	50.6	30.5	45.9	27.7	41.7	21.3	32.0	14.5	21.9
32	29.6	44.5	26.8	40.3	24.4	36.6	18.7	28.1	12.8	19.2
34	26.2	39.4	23.8	35.7	21.6	32.4	16.6	24.9	11.3	17.0
36	23.4	35.2	21.2	31.9	19.3	28.9	14.8	22.2	10.1	15.2
38							13.3	19.9	9.06	13.6

Properties

	0.312	0.280	0.250	0.188	0.125
A_g, in.^{2}	5.79	5.20	4.68	3.53	2.37
I, in.^{4}	29.1	26.4	23.9	18.4	12.6
r, in.	2.24	2.25	2.26	2.28	2.30

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 4-5 (continued)
Available Strength in Axial Compression, kips
Round HSS

HSS6.000

F_y = 46 ksi

Shape	HSS6.000×											
	0.500		0.375		0.312		0.280		0.250		0.188	
t_{des}, in.	0.465		0.349		0.291		0.260		0.233		0.174	
lb/ft	29.40		22.55		18.97		17.12		15.37		11.68	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to radius of gyration, r												
0	223	335	171	257	144	216	129	194	116	175	87.6	132
1	222	334	170	256	143	216	129	194	116	174	87.4	131
2	221	332	169	254	142	214	128	192	115	173	86.8	130
3	218	327	167	251	141	212	126	190	114	171	85.8	129
4	214	322	164	247	138	208	124	187	112	168	84.5	127
5	209	314	161	242	135	204	122	183	110	165	82.7	124
6	204	306	157	235	132	198	119	178	107	161	80.7	121
7	197	296	152	228	128	192	115	173	104	156	78.3	118
8	190	285	146	220	124	186	111	167	100	151	75.7	114
9	182	273	140	211	119	178	107	161	96.3	145	72.8	109
10	173	260	134	201	113	170	102	153	92.1	138	69.7	105
11	164	247	127	191	108	162	97.2	146	87.7	132	66.5	99.9
12	155	233	121	181	102	154	92.1	138	83.1	125	63.1	94.8
13	146	219	113	170	96.3	145	86.8	131	78.4	118	59.6	89.5
14	136	204	106	160	90.3	136	81.5	122	73.7	111	56.0	84.2
15	126	190	99.0	149	84.3	127	76.1	114	68.9	103	52.4	78.8
16	117	176	91.9	138	78.3	118	70.8	106	64.1	96.3	48.8	73.4
17	108	162	84.8	127	72.4	109	65.5	98.4	59.3	89.2	45.3	68.1
18	98.4	148	77.9	117	66.6	100	60.3	90.7	54.7	82.2	41.8	62.8
19	89.7	135	71.2	107	61.0	91.7	55.3	83.1	50.2	75.4	38.4	57.8
20	81.1	122	64.7	97.3	55.6	83.5	50.5	75.8	45.8	68.9	35.2	52.8
21	73.6	111	58.7	88.2	50.4	75.8	45.7	68.8	41.6	62.5	31.9	48.0
22	67.0	101	53.5	80.4	45.9	69.0	41.7	62.6	37.9	56.9	29.1	43.7
23	61.3	92.2	48.9	73.5	42.0	63.2	38.1	57.3	34.7	52.1	26.6	40.0
24	56.3	84.6	44.9	67.5	38.6	58.0	35.0	52.6	31.8	47.8	24.5	36.8
25	51.9	78.0	41.4	62.3	35.6	53.5	32.3	48.5	29.3	44.1	22.5	33.9
26	48.0	72.1	38.3	57.6	32.9	49.4	29.8	44.9	27.1	40.8	20.8	31.3
28	41.4	62.2	33.0	49.6	28.4	42.6	25.7	38.7	23.4	35.1	18.0	27.0
30	36.0	54.2	28.8	43.2	24.7	37.1	22.4	33.7	20.4	30.6	15.7	23.5
32	31.7	47.6	25.3	38.0	21.7	32.6	19.7	29.6	17.9	26.9	13.8	20.7
34									15.9	23.8	12.2	18.3

Properties

	0.500	0.375	0.312	0.280	0.250	0.188
A_g, in.^{2}	8.09	6.20	5.22	4.69	4.22	3.18
I, in.^{4}	31.2	24.8	21.3	19.3	17.6	13.5
r, in.	1.96	2.00	2.02	2.03	2.04	2.06

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 46 ksi

Table 4-5 (continued)
Available Strength in Axial Compression, kips
Round HSS

HSS6.000–HSS5.563

Shape	HSS6.000×		HSS5.563×									
	0.125		0.500		0.375		0.258		0.188		0.134	
t_{des}, in.	0.116		0.465		0.349		0.240		0.174		0.124	
lb/ft	7.85		27.06		20.80		14.63		10.80		7.78	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to radius of gyration, r: 0	58.9	88.6	205	308	158	237	110	166	81.3	122	58.4	87.8
1	58.8	88.4	205	308	157	236	110	166	81.0	122	58.2	87.5
2	58.4	87.8	203	305	156	234	109	164	80.4	121	57.8	86.9
3	57.8	86.8	200	300	154	231	108	162	79.3	119	57.0	85.7
4	56.9	85.5	196	294	151	226	106	159	77.9	117	56.0	84.2
5	55.7	83.8	191	286	147	221	103	155	76.0	114	54.7	82.2
6	54.4	81.7	184	277	142	214	100	150	73.8	111	53.1	79.8
7	52.8	79.4	178	267	137	206	96.6	145	71.3	107	51.3	77.2
8	51.1	76.8	170	255	131	198	92.7	139	68.6	103	49.4	74.2
9	49.2	73.9	162	243	125	188	88.5	133	65.5	98.5	47.2	70.9
10	47.1	70.8	153	229	119	178	84.0	126	62.3	93.6	44.9	67.5
11	45.0	67.6	143	216	112	168	79.3	119	58.9	88.6	42.5	63.9
12	42.7	64.2	134	201	105	158	74.4	112	55.4	83.3	40.0	60.1
13	40.4	60.7	125	187	97.7	147	69.5	104	51.9	78.0	37.5	56.3
14	38.0	57.1	115	173	90.5	136	64.6	97.0	48.3	72.6	34.9	52.4
15	35.6	53.5	106	159	83.3	125	59.6	89.6	44.7	67.2	32.3	48.6
16	33.2	49.9	96.3	145	76.3	115	54.8	82.3	41.2	61.9	29.8	44.8
17	30.9	46.4	87.3	131	69.5	105	50.0	75.2	37.7	56.7	27.3	41.1
18	28.5	42.9	78.6	118	63.0	94.7	45.5	68.3	34.4	51.7	24.9	37.5
19	26.3	39.5	70.6	106	56.6	85.1	41.0	61.6	31.1	46.8	22.6	34.0
20	24.1	36.2	63.7	95.7	51.1	76.8	37.0	55.6	28.1	42.2	20.4	30.7
21	21.9	32.9	57.8	86.8	46.3	69.6	33.5	50.4	25.5	38.3	18.5	27.8
22	20.0	30.0	52.6	79.1	42.2	63.5	30.6	45.9	23.2	34.9	16.9	25.3
23	18.3	27.5	48.2	72.4	38.6	58.1	28.0	42.0	21.2	31.9	15.4	23.2
24	16.8	25.2	44.2	66.5	35.5	53.3	25.7	38.6	19.5	29.3	14.2	21.3
25	15.5	23.2	40.8	61.3	32.7	49.1	23.7	35.6	18.0	27.0	13.1	19.6
26	14.3	21.5	37.7	56.6	30.2	45.4	21.9	32.9	16.6	25.0	12.1	18.1
28	12.3	18.5	32.5	48.8	26.1	39.2	18.9	28.4	14.3	21.5	10.4	15.6
30	10.7	16.1	28.3	42.5	22.7	34.1	16.4	24.7	12.5	18.8	9.06	13.6
32	9.44	14.2									7.97	12.0
34	8.36	12.6										

Properties

	HSS6.000×0.125	HSS5.563×0.500	HSS5.563×0.375	HSS5.563×0.258	HSS5.563×0.188	HSS5.563×0.134
A_g, in.^{2}	2.14	7.45	5.72	4.01	2.95	2.12
I, in.^{4}	9.28	24.4	19.5	14.2	10.7	7.84
r, in.	2.08	1.81	1.85	1.88	1.91	1.92

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 4-5 (continued)
Available Strength in Axial Compression, kips
Round HSS

F_y = 46 ksi

HSS5.500–HSS5.000

Shape	HSS5.500×						HSS5.000×					
	0.500		0.375		0.258		0.500		0.375		0.312	
t_{des}, in.	0.465		0.349		0.240		0.465		0.349		0.291	
lb/ft	26.73		20.55		14.46		24.05		18.54		15.64	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to radius of gyration, r												
0	203	305	156	234	109	164	182	274	140	211	118	178
1	202	304	155	233	109	164	182	273	140	210	118	177
2	200	301	154	231	108	163	180	270	138	208	117	176
3	197	297	152	228	107	160	176	265	136	204	115	173
4	193	290	149	223	105	157	172	258	133	199	112	168
5	188	283	145	218	102	153	166	250	129	193	109	163
6	182	273	140	211	98.9	149	159	240	124	186	105	157
7	175	263	135	203	95.3	143	152	228	118	177	99.9	150
8	167	251	129	194	91.4	137	144	216	112	168	94.8	143
9	159	239	123	185	87.2	131	135	202	105	158	89.4	134
10	150	225	117	175	82.6	124	125	189	98.4	148	83.7	126
11	141	211	110	165	77.9	117	116	174	91.3	137	77.8	117
12	131	197	103	154	73.1	110	106	160	84.2	126	71.8	108
13	122	183	95.5	143	68.1	102	97.0	146	77.0	116	65.9	99.0
14	112	168	88.3	133	63.2	94.9	87.7	132	69.9	105	60.0	90.1
15	103	154	81.2	122	58.2	87.5	78.7	118	63.1	94.8	54.2	81.5
16	93.5	141	74.2	112	53.4	80.3	70.0	105	56.5	84.9	48.7	73.2
17	84.6	127	67.5	101	48.7	73.2	62.0	93.2	50.1	75.4	43.3	65.1
18	76.0	114	61.0	91.6	44.1	66.3	55.3	83.1	44.7	67.2	38.6	58.1
19	68.2	102	54.7	82.2	39.7	59.7	49.6	74.6	40.1	60.3	34.7	52.1
20	61.5	92.5	49.4	74.2	35.8	53.9	44.8	67.3	36.2	54.5	31.3	47.0
21	55.8	83.9	44.8	67.3	32.5	48.9	40.6	61.0	32.9	49.4	28.4	42.7
22	50.9	76.4	40.8	61.3	29.6	44.5	37.0	55.6	29.9	45.0	25.9	38.9
23	46.5	69.9	37.3	56.1	27.1	40.7	33.9	50.9	27.4	41.2	23.7	35.6
24	42.7	64.2	34.3	51.5	24.9	37.4	31.1	46.7	25.2	37.8	21.7	32.7
25	39.4	59.2	31.6	47.5	22.9	34.5	28.7	43.1	23.2	34.9	20.0	30.1
26	36.4	54.7	29.2	43.9	21.2	31.9	26.5	39.8	21.4	32.2	18.5	27.8
28	31.4	47.2	25.2	37.9	18.3	27.5						
30			21.9	33.0	15.9	23.9						
Properties												
A_g, in.^{2}	7.36		5.65		3.97		6.62		5.10		4.30	
I, in.^{4}	23.5		18.8		13.7		17.2		13.9		12.0	
r, in.	1.79		1.83		1.86		1.61		1.65		1.67	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 46 ksi

Table 4-5 (continued)
Available Strength in Axial Compression, kips
Round HSS

HSS5.000–HSS4.500

Shape		HSS5.000×								HSS4.500×			
		0.258		0.250		0.188		0.125		0.375		0.337	
t_{des}, in.		0.240		0.233		0.174		0.116		0.349		0.313	
lb/ft		13.08		12.69		9.67		6.51		16.54		15.00	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to radius of gyration, r	0	98.9	149	96.1	144	72.7	109	49.0	73.7	125	188	113	171
	1	98.6	148	95.8	144	72.5	109	48.9	73.5	125	188	113	170
	2	97.6	147	94.8	143	71.8	108	48.4	72.7	123	185	111	168
	3	95.9	144	93.2	140	70.6	106	47.6	71.6	120	181	109	164
	4	93.7	141	91.1	137	69.0	104	46.6	70.0	117	175	106	159
	5	90.8	137	88.3	133	66.9	101	45.2	68.0	112	168	102	153
	6	87.5	132	85.1	128	64.5	97.0	43.6	65.6	107	160	96.8	145
	7	83.7	126	81.4	122	61.8	92.9	41.8	62.9	101	151	91.4	137
	8	79.6	120	77.4	116	58.8	88.4	39.9	59.9	94.1	141	85.5	129
	9	75.1	113	73.0	110	55.6	83.6	37.7	56.7	87.2	131	79.3	119
	10	70.4	106	68.5	103	52.2	78.5	35.5	53.3	80.1	120	72.9	110
	11	65.6	98.6	63.8	95.9	48.7	73.2	33.1	49.8	72.9	110	66.5	99.9
	12	60.7	91.2	59.0	88.7	45.1	67.8	30.8	46.2	65.7	98.8	60.0	90.2
	13	55.7	83.8	54.2	81.5	41.5	62.4	28.4	42.6	58.8	88.3	53.7	80.8
	14	50.9	76.5	49.5	74.3	38.0	57.1	26.0	39.1	52.1	78.2	47.7	71.7
	15	46.1	69.3	44.8	67.4	34.5	51.9	23.7	35.6	45.6	68.6	41.9	62.9
	16	41.5	62.4	40.3	60.6	31.1	46.8	21.4	32.2	40.1	60.3	36.8	55.3
	17	37.0	55.7	36.0	54.1	27.9	41.9	19.2	28.9	35.5	53.4	32.6	49.0
	18	33.0	49.6	32.1	48.3	24.9	37.4	17.2	25.8	31.7	47.6	29.1	43.7
	19	29.6	44.6	28.8	43.3	22.3	33.5	15.4	23.2	28.4	42.7	26.1	39.2
	20	26.8	40.2	26.0	39.1	20.1	30.3	13.9	20.9	25.7	38.6	23.5	35.4
	21	24.3	36.5	23.6	35.5	18.3	27.5	12.6	19.0	23.3	35.0	21.4	32.1
	22	22.1	33.2	21.5	32.3	16.6	25.0	11.5	17.3	21.2	31.9	19.5	29.3
	23	20.2	30.4	19.7	29.6	15.2	22.9	10.5	15.8	19.4	29.2	17.8	26.8
	24	18.6	27.9	18.1	27.1	14.0	21.0	9.65	14.5	17.8	26.8	16.4	24.6
	25	17.1	25.7	16.6	25.0	12.9	19.4	8.90	13.4				
	26	15.8	23.8	15.4	23.1	11.9	17.9	8.23	12.4				
	28	13.7	20.5	13.3	19.9	10.3	15.4	7.09	10.7				

Properties

	HSS5.000×0.258	HSS5.000×0.250	HSS5.000×0.188	HSS5.000×0.125	HSS4.500×0.375	HSS4.500×0.337
A_g, in.^{2}	3.59	3.49	2.64	1.78	4.55	4.12
I, in.^{4}	10.2	9.94	7.69	5.31	9.87	9.07
r, in.	1.69	1.69	1.71	1.73	1.47	1.48

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates L_c/r equal to or greater than 200.

HSS4.500–
HSS4.000

Table 4-5 (continued)
Available Strength in Axial Compression, kips
Round HSS

F_y = 46 ksi

Shape		HSS4.500× 0.237		HSS4.500× 0.188		HSS4.500× 0.125		HSS4.000× 0.313		HSS4.000× 0.250	
t_{des}, in.		0.220		0.174		0.116		0.291		0.233	
lb/ft		10.80		8.67		5.85		12.34		10.00	
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length, L_c (ft), with respect to radius of gyration, r	0	81.5	123	65.0	97.7	44.1	66.2	93.4	140	76.0	114
	1	81.2	122	64.7	97.3	43.9	66.0	92.9	140	75.6	114
	2	80.2	121	63.9	96.1	43.4	65.2	91.3	137	74.4	112
	3	78.5	118	62.6	94.1	42.5	63.9	88.8	133	72.4	109
	4	76.2	115	60.8	91.4	41.3	62.1	85.4	128	69.6	105
	5	73.4	110	58.6	88.1	39.8	59.9	81.3	122	66.3	99.6
	6	70.1	105	56.0	84.2	38.1	57.3	76.4	115	62.4	93.8
	7	66.4	99.8	53.1	79.8	36.2	54.4	71.1	107	58.1	87.4
	8	62.3	93.7	49.9	75.0	34.0	51.2	65.4	98.3	53.5	80.5
	9	58.1	87.3	46.5	69.9	31.8	47.8	59.5	89.5	48.8	73.3
	10	53.6	80.6	43.0	64.6	29.4	44.3	53.6	80.5	44.0	66.1
	11	49.1	73.8	39.4	59.2	27.1	40.7	47.7	71.6	39.2	58.9
	12	44.6	67.0	35.8	53.8	24.7	37.1	41.9	63.0	34.6	51.9
	13	40.1	60.3	32.3	48.6	22.3	33.5	36.5	54.8	30.1	45.3
	14	35.8	53.9	28.9	43.4	20.0	30.1	31.5	47.3	26.0	39.1
	15	31.7	47.7	25.6	38.5	17.8	26.7	27.4	41.2	22.6	34.0
	16	27.9	41.9	22.5	33.9	15.7	23.6	24.1	36.2	19.9	29.9
	17	24.7	37.1	20.0	30.0	13.9	20.9	21.3	32.1	17.6	26.5
	18	22.0	33.1	17.8	26.8	12.4	18.6	19.0	28.6	15.7	23.6
	19	19.8	29.7	16.0	24.0	11.1	16.7	17.1	25.7	14.1	21.2
	20	17.8	26.8	14.4	21.7	10.0	15.1	15.4	23.2	12.7	19.1
	21	16.2	24.3	13.1	19.7	9.10	13.7	14.0	21.0	11.6	17.4
	22	14.7	22.2	11.9	17.9	8.29	12.5	12.7	19.1	10.5	15.8
	23	13.5	20.3	10.9	16.4	7.58	11.4				
	24	12.4	18.6	10.0	15.0	6.97	10.5				
	25	11.4	17.2	9.23	13.9	6.42	9.65				

Properties

	HSS4.500× 0.237	HSS4.500× 0.188	HSS4.500× 0.125	HSS4.000× 0.313	HSS4.000× 0.250
A_g, in.^{2}	2.96	2.36	1.60	3.39	2.76
I, in.^{4}	6.79	5.54	3.84	5.87	4.91
r, in.	1.52	1.53	1.55	1.32	1.33

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 46 ksi

Table 4-5 (continued)
Available Strength in Axial Compression, kips
Round HSS

HSS4.000

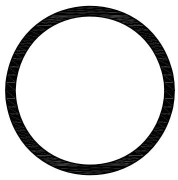
Shape		HSS4.000× 0.237		HSS4.000× 0.226		HSS4.000× 0.220		HSS4.000× 0.188		HSS4.000× 0.125	
t_{des}, in.		0.220		0.210		0.205		0.174		0.116	
lb/ft		9.53		9.12		8.89		7.66		5.18	
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length, L_c (ft), with respect to radius of gyration, r	0	71.9	108	68.9	103	67.2	101	57.6	86.5	39.1	58.8
	1	71.5	107	68.5	103	66.8	100	57.3	86.1	38.9	58.5
	2	70.4	106	67.4	101	65.8	98.9	56.4	84.7	38.3	57.6
	3	68.5	103	65.6	98.6	64.0	96.2	54.9	82.5	37.3	56.1
	4	65.9	99.1	63.2	94.9	61.7	92.7	52.9	79.5	36.0	54.1
	5	62.8	94.4	60.2	90.4	58.7	88.3	50.4	75.8	34.4	51.7
	6	59.2	89.0	56.7	85.2	55.3	83.2	47.5	71.5	32.5	48.8
	7	55.2	83.0	52.9	79.5	51.6	77.6	44.4	66.7	30.4	45.7
	8	50.9	76.5	48.8	73.3	47.6	71.5	41.0	61.6	28.1	42.3
	9	46.4	69.8	44.5	66.9	43.4	65.3	37.4	56.3	25.8	38.7
	10	41.9	63.0	40.2	60.3	39.2	58.9	33.8	50.9	23.3	35.1
	11	37.4	56.3	35.9	53.9	35.0	52.6	30.3	45.5	20.9	31.5
	12	33.1	49.7	31.7	47.6	30.9	46.5	26.8	40.2	18.6	28.0
	13	28.9	43.4	27.7	41.6	27.0	40.6	23.4	35.2	16.4	24.6
	14	25.0	37.5	23.9	35.9	23.3	35.1	20.3	30.5	14.2	21.3
	15	21.7	32.7	20.8	31.3	20.3	30.5	17.7	26.6	12.4	18.6
	16	19.1	28.7	18.3	27.5	17.9	26.8	15.5	23.3	10.9	16.3
	17	16.9	25.4	16.2	24.4	15.8	23.8	13.8	20.7	9.63	14.5
	18	15.1	22.7	14.5	21.7	14.1	21.2	12.3	18.4	8.59	12.9
	19	13.6	20.4	13.0	19.5	12.7	19.0	11.0	16.6	7.71	11.6
	20	12.2	18.4	11.7	17.6	11.4	17.2	9.94	14.9	6.95	10.5
	21	11.1	16.7	10.6	16.0	10.4	15.6	9.02	13.6	6.31	9.48
	22	10.1	15.2	9.68	14.6	9.45	14.2	8.21	12.3	5.75	8.64

Properties

	0.237	0.226	0.220	0.188	0.125
A_g, in.^{2}	2.61	2.50	2.44	2.09	1.42
I, in.^{4}	4.68	4.50	4.41	3.83	2.67
r, in.	1.34	1.34	1.34	1.35	1.37

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates L_c/r equal to or greater than 200.



PIPE 12–
PIPE 8

Table 4-6
Available Strength in Axial Compression, kips
Pipe

F_y = 35 ksi

Shape	Pipe 12				Pipe 10				Pipe 8			
	x-Strong		Std		x-Strong		Std		xx-Strong		x-Strong	
t_{des}, in.	0.465		0.349		0.465		0.340		0.816		0.465	
lb/ft	65.5		49.6		54.8		40.5		72.5		43.4	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to radius of gyration, r												
0	367	551	287	432	316	476	241	362	419	630	249	375
6	362	544	283	426	310	466	236	355	405	609	242	363
7	360	541	282	424	308	463	235	353	400	601	239	359
8	358	538	280	421	305	459	233	350	394	593	236	354
9	355	534	278	418	303	455	231	347	388	583	232	349
10	353	530	276	415	299	450	228	343	381	573	228	343
11	350	526	274	412	296	445	226	339	373	561	224	337
12	347	521	272	408	292	439	223	335	365	549	220	330
13	343	516	269	405	288	433	220	330	357	536	215	323
14	340	511	266	400	284	427	217	326	348	523	210	315
15	336	505	263	396	279	420	213	320	338	508	204	307
16	332	499	260	391	274	413	210	315	328	494	199	299
17	328	493	257	386	269	405	206	310	318	478	193	290
18	323	486	254	381	264	397	202	304	308	463	187	282
19	319	479	250	376	259	389	198	298	297	447	181	273
20	314	472	246	370	253	381	194	291	286	430	175	263
21	309	464	243	365	248	372	190	285	275	414	169	254
22	304	457	239	359	242	363	185	278	264	397	163	245
23	298	449	235	353	236	354	181	272	253	380	156	235
24	293	440	230	346	230	345	176	265	242	364	150	225
25	288	432	226	340	224	336	172	258	231	347	144	216
26	282	424	222	333	217	327	167	251	220	331	137	206
27	276	415	217	327	211	317	162	244	209	314	131	197
28	270	406	213	320	205	308	157	236	198	298	125	188
29	264	397	208	313	198	298	153	229	188	283	119	178
30	258	388	204	306	192	288	148	222	178	267	113	169
32	246	370	194	292	179	269	138	207	158	237	101	152
34	234	351	185	277	166	250	128	193	140	210	89.7	135
36	221	333	175	263	154	231	119	179	124	187	80.0	120
38	209	314	165	248	142	213	110	165	112	168	71.8	108
40	197	296	156	234	130	195	101	152	101	152	64.8	97.5

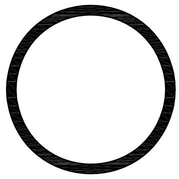
Properties

	Pipe 12 x-Strong	Pipe 12 Std	Pipe 10 x-Strong	Pipe 10 Std	Pipe 8 xx-Strong	Pipe 8 x-Strong
A_g, in.^{2}	17.5	13.7	15.1	11.5	20.0	11.9
I, in.^{4}	339	262	199	151	154	100
r, in.	4.35	4.39	3.64	3.68	2.78	2.89

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

F_y = 35 ksi

Table 4-6 (continued)
Available Strength in Axial Compression, kips
Pipe



PIPE 8–
PIPE 5

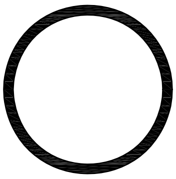
Shape		Pipe 8		Pipe 6						Pipe 5	
		Std		xx-Strong		x-Strong		Std		xx-Strong	
t_{des}, in.		0.300		0.805		0.403		0.261		0.699	
lb/ft		28.6		53.2		28.6		19.0		38.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to radius of gyration, r	0	165	247	308	463	164	247	109	164	224	337
	6	160	240	290	436	155	233	103	155	205	309
	7	158	237	283	426	152	229	101	153	199	299
	8	156	234	276	415	149	224	99.3	149	192	288
	9	154	231	268	403	145	218	96.9	146	184	277
	10	151	227	260	391	141	212	94.2	142	176	264
	11	148	223	251	377	136	205	91.4	137	167	251
	12	146	219	241	362	132	198	88.4	133	158	237
	13	143	214	231	347	127	191	85.2	128	149	223
	14	139	209	221	332	122	183	81.9	123	139	209
	15	136	204	210	316	116	175	78.5	118	130	195
	16	132	199	199	299	111	167	75.1	113	120	181
	17	129	194	188	283	106	159	71.6	108	111	167
	18	125	188	177	267	100	151	68.0	102	102	153
	19	121	182	167	250	94.7	142	64.4	96.8	93.1	140
	20	117	176	156	234	89.2	134	60.9	91.5	84.5	127
	21	113	170	145	218	83.8	126	57.3	86.2	76.7	115
	22	109	164	135	203	78.5	118	53.9	81.0	69.9	105
	23	105	158	125	188	73.3	110	50.5	75.8	63.9	96.1
	24	101	152	115	173	68.3	103	47.1	70.8	58.7	88.2
	25	96.9	146	106	160	63.3	95.1	43.9	65.9	54.1	81.3
	26	92.8	139	98.2	148	58.5	88.0	40.6	61.1	50.0	75.2
	27	88.7	133	91.1	137	54.3	81.6	37.7	56.7	46.4	69.7
	28	84.7	127	84.7	127	50.5	75.8	35.0	52.7	43.1	64.8
	29	80.7	121	78.9	119	47.0	70.7	32.7	49.1	40.2	60.4
	30	76.8	115	73.8	111	44.0	66.1	30.5	45.9		
	32	69.1	104	64.8	97.4	38.6	58.1	26.8	40.3		
	34	61.7	92.7	57.4	86.3	34.2	51.4	23.8	35.7		
	36	55.0	82.7			30.5	45.9	21.2	31.9		
	38	49.4	74.2								
	40	44.6	67.0								

Properties

	Pipe 8 Std	Pipe 6 xx-Strong	Pipe 6 x-Strong	Pipe 6 Std	Pipe 5 xx-Strong
A_g, in.^{2}	7.85	14.7	7.83	5.20	10.7
I, in.^{4}	68.1	63.5	38.3	26.5	32.2
r, in.	2.95	2.08	2.20	2.25	1.74

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates L_c/r equal to or greater than 200.



PIPE 5–
PIPE 4

Table 4-6 (continued)
Available Strength in Axial Compression, kips
Pipe

F_y = 35 ksi

Shape		Pipe 5				Pipe 4					
		x-Strong		Std		xx-Strong		x-Strong		Std	
t_{des}, in.		0.349		0.241		0.628		0.315		0.221	
lb/ft		20.8		14.6		27.6		15.0		10.8	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to radius of gyration, r	0	120	180	84.0	126	161	241	86.8	130	62.0	93.2
	6	111	167	78.0	117	140	210	76.9	116	55.2	83.0
	7	108	162	75.9	114	133	200	73.6	111	52.9	79.6
	8	105	157	73.5	111	126	189	70.0	105	50.4	75.8
	9	101	152	71.0	107	118	177	66.1	99.3	47.7	71.8
	10	96.8	146	68.2	103	110	165	62.0	93.1	44.9	67.5
	11	92.5	139	65.3	98.1	101	152	57.7	86.8	42.0	63.1
	12	88.1	132	62.2	93.6	92.7	139	53.4	80.3	38.9	58.5
	13	83.5	125	59.1	88.8	84.3	127	49.1	73.8	35.9	54.0
	14	78.7	118	55.8	83.9	76.0	114	44.9	67.4	32.9	49.5
	15	74.0	111	52.6	79.0	68.1	102	40.7	61.2	30.0	45.1
	16	69.2	104	49.3	74.1	60.3	90.7	36.7	55.1	27.1	40.8
	17	64.4	96.9	46.0	69.1	53.5	80.3	32.8	49.2	24.4	36.6
	18	59.8	89.8	42.8	64.3	47.7	71.7	29.2	43.9	21.7	32.7
	19	55.2	83.0	39.6	59.5	42.8	64.3	26.2	39.4	19.5	29.3
	20	50.7	76.3	36.5	54.9	38.6	58.0	23.7	35.6	17.6	26.5
	21	46.4	69.8	33.5	50.4	35.0	52.6	21.5	32.3	16.0	24.0
	22	42.3	63.6	30.6	45.9	31.9	48.0	19.6	29.4	14.6	21.9
	23	38.7	58.2	28.0	42.0	29.2	43.9	17.9	26.9	13.3	20.0
	24	35.5	53.4	25.7	38.6			16.4	24.7	12.2	18.4
	25	32.8	49.2	23.7	35.6					11.3	16.9
	26	30.3	45.5	21.9	32.9						
	27	28.1	42.2	20.3	30.5						
	28	26.1	39.2	18.9	28.4						
	29	24.3	36.6	17.6	26.4						
	30	22.7	34.2	16.4	24.7						

Properties

	Pipe 5 x-Strong	Pipe 5 Std	Pipe 4 xx-Strong	Pipe 4 x-Strong	Pipe 4 Std
A_g, in.^{2}	5.73	4.01	7.66	4.14	2.96
I, in.^{4}	19.5	14.3	14.7	9.12	6.82
r, in.	1.85	1.88	1.39	1.48	1.51

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 35 ksi

Table 4-6 (continued)
Available Strength in Axial Compression, kips
Pipe

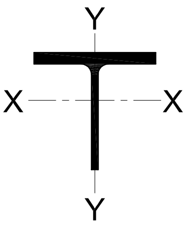
PIPE 3½–
PIPE 3

Shape		Pipe 3½				Pipe 3					
		x-Strong		Std		xx-Strong		x-Strong		Std	
t_{des}, in.		0.296		0.211		0.559		0.280		0.201	
lb/ft		12.5		9.12		18.6		10.3		7.58	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to radius of gyration, r	0	71.9	108	52.4	78.7	108	163	59.3	89.1	43.4	65.2
	6	61.6	92.6	45.2	67.9	85.6	129	48.4	72.7	35.7	53.7
	7	58.2	87.5	42.8	64.4	78.6	118	44.9	67.5	33.3	50.1
	8	54.6	82.1	40.3	60.6	71.2	107	41.3	62.0	30.7	46.2
	9	50.8	76.3	37.6	56.5	63.7	95.7	37.5	56.3	28.0	42.2
	10	46.8	70.3	34.8	52.2	56.2	84.5	33.6	50.6	25.3	38.1
	11	42.8	64.3	31.9	47.9	49.0	73.6	29.9	44.9	22.6	34.0
	12	38.7	58.2	29.0	43.6	42.1	63.3	26.2	39.4	20.0	30.0
	13	34.8	52.3	26.2	39.4	35.9	53.9	22.7	34.1	17.5	26.2
	14	31.0	46.6	23.4	35.2	30.9	46.5	19.6	29.4	15.1	22.7
	15	27.3	41.0	20.8	31.3	26.9	40.5	17.1	25.6	13.1	19.8
	16	24.0	36.1	18.3	27.5	23.7	35.6	15.0	22.5	11.6	17.4
	17	21.3	32.0	16.2	24.4	21.0	31.5	13.3	20.0	10.2	15.4
	18	19.0	28.5	14.5	21.7			11.8	17.8	9.13	13.7
	19	17.0	25.6	13.0	19.5			10.6	16.0	8.19	12.3
	20	15.4	23.1	11.7	17.6						
	21	13.9	20.9	10.6	16.0						
	22			9.68	14.6						

Properties					
A_g, in.^{2}	3.43	2.50	5.17	2.83	2.07
I, in.^{4}	5.94	4.52	5.79	3.70	2.85
r, in.	1.31	1.34	1.06	1.14	1.17

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates L_c/r equal to or greater than 200.



WT18

Table 4-7
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes

F_y = 50 ksi

Shape		WT18×									
lb/ft		151[c]		141[c]		131[c]		123.5[c]		115.5[c]	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to indicated axis											
X-X Axis	0	1310	1960	1200	1810	1100	1660	1030	1550	957	1440
	10	1260	1900	1160	1750	1070	1610	997	1500	927	1390
	12	1250	1870	1150	1720	1050	1580	983	1480	914	1370
	14	1230	1840	1130	1700	1040	1560	967	1450	898	1350
	16	1200	1810	1110	1660	1020	1530	948	1420	881	1320
	18	1180	1770	1080	1630	994	1490	927	1390	862	1300
	20	1150	1720	1060	1590	970	1460	905	1360	841	1260
	22	1120	1680	1030	1540	944	1420	881	1320	819	1230
	24	1080	1620	997	1500	916	1380	855	1280	795	1190
	26	1040	1560	964	1450	887	1330	828	1240	770	1160
	28	1000	1500	931	1400	856	1290	799	1200	743	1120
	30	959	1440	893	1340	824	1240	769	1160	716	1080
	32	917	1380	854	1280	791	1190	739	1110	687	1030
	34	874	1310	813	1220	755	1130	708	1060	659	990
	36	830	1250	773	1160	717	1080	676	1020	629	946
	40	743	1120	691	1040	641	964	605	909	568	854
Y-Y Axis	0	1310	1960	1200	1810	1100	1660	1030	1550	957	1440
	10	1130	1690	1020	1540	916	1380	836	1260	758	1140
	12	1110	1670	1010	1520	904	1360	826	1240	749	1130
	14	1080	1630	987	1480	887	1330	811	1220	736	1110
	16	1050	1570	959	1440	863	1300	791	1190	719	1080
	18	1000	1510	924	1390	833	1250	765	1150	696	1050
	20	955	1430	880	1320	798	1200	733	1100	669	1010
	22	901	1350	831	1250	755	1130	698	1050	637	957
	24	845	1270	780	1170	708	1060	657	988	602	905
	26	788	1180	726	1090	659	991	612	920	563	846
	28	730	1100	672	1010	610	916	566	850	520	782
	30	672	1010	619	930	560	842	520	781	478	718
	32	615	924	566	850	512	769	474	713	435	654
	34	559	841	514	773	464	698	430	647	394	592
	36	505	759	464	697	418	628	387	582	355	533
	40	412	619	379	569	342	514	317	477	291	438

Properties

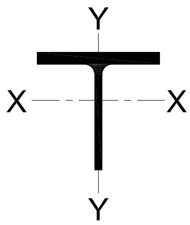
	151	141	131	123.5	115.5
A_g, in.^{2}	44.5	41.5	38.5	36.3	34.1
r_x, in.	5.37	5.36	5.36	5.36	5.36
r_y, in.	3.82	3.80	3.76	3.74	3.71

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 50 ksi; tabulated values have been adjusted accordingly.

F_y = 50 ksi

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes



WT18

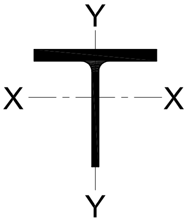
Shape		WT18×									
lb/ft		128[c]		116[c]		105[c]		97[c]		91[c]	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to indicated axis: X-X Axis	0	1100	1660	973	1460	876	1320	790	1190	730	1100
	10	1070	1610	946	1420	852	1280	768	1150	710	1070
	12	1060	1590	934	1400	841	1260	758	1140	701	1050
	14	1040	1570	920	1380	829	1250	747	1120	691	1040
	16	1030	1540	905	1360	815	1230	734	1100	679	1020
	18	1010	1510	887	1330	800	1200	720	1080	666	1000
	20	984	1480	868	1300	783	1180	705	1060	652	980
	22	960	1440	847	1270	764	1150	688	1030	637	957
	24	932	1400	825	1240	745	1120	671	1010	620	933
	26	901	1350	802	1200	724	1090	652	979	603	907
	28	870	1310	777	1170	702	1050	632	950	585	879
	30	837	1260	751	1130	679	1020	611	919	566	851
	32	804	1210	724	1090	655	985	590	887	546	821
	34	770	1160	693	1040	631	948	568	854	526	791
	36	735	1110	662	995	603	907	545	820	505	760
	40	665	1000	598	899	546	820	499	751	463	696
Y-Y Axis	0	1100	1660	973	1460	876	1320	790	1190	730	1100
	10	895	1340	780	1170	672	1010	590	887	531	798
	12	847	1270	745	1120	643	966	566	851	510	767
	14	789	1190	697	1050	604	907	535	804	483	727
	16	724	1090	640	962	555	834	498	748	451	677
	18	656	986	579	870	502	755	451	678	413	621
	20	587	882	517	777	448	673	402	604	369	554
	22	518	778	455	684	393	591	353	531	324	487
	24	452	679	396	595	340	511	305	459	280	421
	26	389	585	341	512	294	442	264	397	244	366
	28	338	508	296	445	257	386	231	347	213	320
	30	296	445	260	391	226	339	203	306	188	283
	32	261	393	230	345	200	300	180	271	167	251
	34	232	349	204	307	178	267	161	242	149	224
	36	208	312	183	275	160	240	144	217	134	201
	40	169	254	149	224	130	196	118	177	109	164

Properties

	128[c]	116[c]	105[c]	97[c]	91[c]
A_g, in.^{2}	37.6	34.0	30.9	28.5	26.8
r_x, in.	5.66	5.63	5.65	5.62	5.62
r_y, in.	2.65	2.62	2.58	2.56	2.55

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 50 ksi; tabulated values have been adjusted accordingly.



WT18

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes

F_y = 50 ksi

Shape		WT18×							
lb/ft		85[c]		80[c]		75[c]		67.5[c]	
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length, L_c (ft), with respect to indicated axis: X-X Axis	0	669	1010	620	932	576	866	508	763
	10	651	978	603	906	560	842	494	742
	12	642	966	596	895	553	832	488	733
	14	633	952	587	882	545	820	481	723
	16	622	936	577	867	536	806	473	711
	18	611	918	566	851	526	791	465	698
	20	598	898	554	833	515	774	455	684
	22	584	877	541	813	503	756	445	669
	24	568	854	527	792	490	737	434	652
	26	553	830	513	770	477	716	422	634
	28	536	805	497	747	462	695	410	616
	30	518	779	481	723	448	673	397	597
	32	500	752	464	698	432	650	384	577
	34	482	724	447	672	417	626	370	557
	36	463	696	430	646	400	602	356	536
	40	424	638	394	592	367	552	328	493
Y-Y Axis	0	669	1010	620	932	576	866	508	763
	10	470	707	418	628	367	552	282	424
	12	453	681	402	605	354	533	272	408
	14	430	646	382	575	337	507	258	388
	16	401	603	358	538	316	475	242	364
	18	369	555	329	495	291	438	223	335
	20	333	500	298	448	264	397	200	300
	22	292	439	261	392	231	347	178	267
	24	253	380	227	341	203	304	158	237
	26	220	331	199	299	178	268	140	211
	28	193	291	175	263	157	237	125	188
	30	171	257	155	233	140	210	112	168
	32	152	228	138	207	125	187	100	151
	34	136	204	123	185	112	168	90.5	136
	36	122	183	111	167	101	151	81.9	123
	40	99.9	150	91.1	137	82.9	125		

Properties

	85	80	75	67.5
A_g, in.^{2}	25.0	23.5	22.1	19.9
r_x, in.	5.61	5.61	5.62	5.66
r_y, in.	2.53	2.50	2.47	2.38

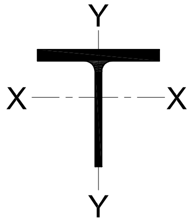
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 50 ksi

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes



WT16.5

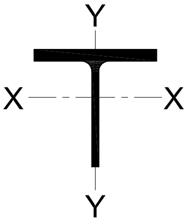
Shape		WT16.5×									
lb/ft		193.5[h]		177[h]		159		145.5[c]		131.5[c]	
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	1710	2570	1560	2340	1400	2110	1280	1930	1140	1710
	10	1640	2460	1500	2250	1340	2020	1230	1850	1090	1640
	12	1610	2420	1470	2210	1320	1980	1200	1810	1070	1610
	14	1570	2370	1440	2160	1290	1940	1180	1770	1050	1580
	16	1540	2310	1400	2110	1260	1890	1150	1730	1030	1550
	18	1490	2250	1360	2050	1220	1840	1120	1680	1000	1510
	20	1450	2180	1320	1980	1180	1780	1080	1620	972	1460
	22	1400	2100	1280	1920	1140	1720	1040	1570	940	1410
	24	1350	2030	1230	1840	1100	1650	1000	1510	903	1360
	26	1290	1940	1180	1770	1050	1580	960	1440	865	1300
	28	1240	1860	1130	1690	1010	1510	916	1380	825	1240
	30	1180	1770	1070	1610	958	1440	872	1310	785	1180
	32	1120	1690	1020	1530	909	1370	827	1240	744	1120
	34	1060	1600	964	1450	859	1290	781	1170	702	1060
	36	1000	1510	910	1370	810	1220	736	1110	661	993
	40	886	1330	802	1200	712	1070	646	971	579	871
Y-Y Axis	0	1710	2570	1560	2340	1400	2110	1280	1930	1140	1710
	10	1540	2310	1390	2090	1230	1850	1110	1670	979	1470
	12	1500	2250	1360	2040	1200	1810	1090	1630	962	1450
	14	1440	2170	1310	1970	1170	1750	1050	1580	936	1410
	16	1380	2080	1260	1890	1120	1680	1010	1520	900	1350
	18	1320	1980	1200	1800	1060	1600	962	1450	858	1290
	20	1250	1870	1130	1700	1010	1510	909	1370	811	1220
	22	1170	1760	1060	1600	944	1420	853	1280	761	1140
	24	1100	1650	992	1490	881	1320	796	1200	709	1070
	26	1020	1530	920	1380	816	1230	737	1110	656	986
	28	940	1410	849	1280	752	1130	678	1020	603	906
	30	863	1300	778	1170	689	1030	620	932	551	828
	32	787	1180	709	1070	626	942	563	846	500	751
	34	714	1070	642	965	566	851	508	764	450	676
	36	642	966	577	866	508	763	455	684	403	606
	40	522	785	469	704	413	621	371	557	328	494

Properties

	193.5	177	159	145.5	131.5
A_g, in.^{2}	57.0	52.1	46.8	42.8	38.7
r_x, in.	5.07	5.03	4.99	4.96	4.93
r_y, in.	3.77	3.74	3.71	3.68	3.65

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
[c] Shape is slender for compression with F_y = 50 ksi; tabulated values have been adjusted accordingly.



WT16.5

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes

F_y = 50 ksi

Shape		WT16.5×							
lb/ft		120.5[c]		110.5[c]		100.5[c]		84.5[c]	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	1040	1560	935	1400	837	1260	677	1020
	10	997	1500	900	1350	806	1210	654	983
	12	981	1470	885	1330	793	1190	644	968
	14	961	1450	868	1300	777	1170	633	951
	16	940	1410	848	1270	760	1140	620	931
	18	916	1380	827	1240	741	1110	605	910
	20	890	1340	803	1210	720	1080	590	886
	22	861	1290	778	1170	697	1050	573	861
	24	832	1250	751	1130	673	1010	555	834
	26	798	1200	723	1090	648	974	536	805
	28	762	1150	693	1040	622	935	516	776
	30	725	1090	663	996	595	894	496	745
	32	688	1030	629	945	567	853	475	714
	34	650	977	594	893	539	811	453	681
	36	612	920	559	841	510	766	432	649
	40	537	808	491	738	447	672	388	583
Y-Y Axis	0	1040	1560	935	1400	837	1260	677	1020
	10	867	1300	761	1140	656	987	516	775
	12	853	1280	751	1130	648	974	493	741
	14	834	1250	735	1100	636	956	463	696
	16	807	1210	714	1070	619	930	428	643
	18	771	1160	686	1030	597	898	389	585
	20	729	1100	653	982	571	858	345	519
	22	685	1030	614	923	541	813	301	452
	24	638	959	572	860	507	761	258	388
	26	590	887	529	795	469	704	223	335
	28	542	815	486	730	430	646	194	292
	30	495	743	443	665	392	589	171	256
	32	448	674	401	602	354	532	151	227
	34	403	605	359	540	318	477	134	202
	36	361	543	323	485	286	429	120	181
	40	295	443	264	397	234	352	98.1	147

Properties

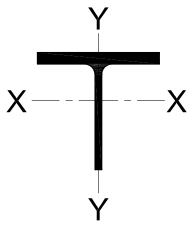
	120.5	110.5	100.5	84.5
A_g, in.^{2}	35.6	32.6	29.7	24.7
r_x, in.	4.96	4.95	4.95	5.12
r_y, in.	3.62	3.59	3.56	2.50

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

F_y = 50 ksi

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes



WT16.5

Shape		WT16.5×							
lb/ft		76[c]		70.5[c]		65[c]		59[c]	
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	607	913	548	824	500	751	446	670
	10	587	882	530	797	484	727	432	649
	12	578	869	523	785	477	716	425	639
	14	568	854	513	772	468	704	418	629
	16	557	837	503	756	459	690	410	617
	18	544	818	492	739	449	675	401	603
	20	530	797	479	721	438	658	391	588
	22	515	774	466	700	426	640	381	573
	24	499	750	452	679	413	621	370	556
	26	482	725	437	657	400	601	358	538
	28	465	699	421	633	386	580	345	519
	30	447	672	405	609	371	558	333	500
	32	428	644	388	584	356	535	319	480
	34	409	615	371	558	341	512	306	460
	36	390	586	354	532	325	489	292	439
	40	351	528	319	480	294	442	264	397
Y-Y Axis	0	607	913	548	824	500	751	446	670
	10	434	652	374	562	315	474	250	376
	12	416	626	359	539	303	455	240	361
	14	393	591	339	510	287	431	228	343
	16	365	548	315	474	267	401	212	319
	18	333	500	288	433	244	367	194	292
	20	296	446	258	387	217	326	173	260
	22	258	387	223	336	189	284	153	230
	24	222	334	193	291	165	248	135	203
	26	193	290	169	253	145	218	120	180
	28	169	254	148	222	128	192	106	160
	30	149	223	131	196	113	170	94.7	142
	32	132	198	116	174	101	152	84.8	127
	34	118	177	104	156	90.3	136	76.3	115
	36	106	159	93.2	140	81.3	122	68.9	103
	40	86.3	130	76.3	115				

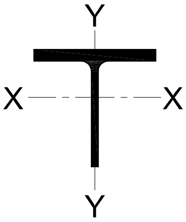
Properties

	76	70.5	65	59
A_g, in.^{2}	22.5	20.7	19.1	17.4
r_x, in.	5.14	5.15	5.18	5.20
r_y, in.	2.47	2.43	2.38	2.32

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r equal to or greater than 200.



WT15

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes

F_y = 50 ksi

Shape		WT15×											
lb/ft		195.5[h]		178.5[h]		163[h]		146		130.5		117.5[c]	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	1720	2590	1570	2360	1440	2160	1290	1940	1150	1730	1030	1550
	10	1640	2470	1490	2250	1360	2050	1220	1840	1090	1640	981	1470
	12	1610	2410	1460	2200	1330	2010	1190	1790	1070	1610	960	1440
	14	1560	2350	1420	2140	1300	1950	1160	1750	1040	1560	934	1400
	16	1520	2280	1380	2080	1260	1890	1130	1690	1010	1510	904	1360
	18	1470	2210	1330	2010	1220	1830	1090	1630	971	1460	872	1310
	20	1410	2130	1280	1930	1170	1760	1040	1570	933	1400	837	1260
	22	1360	2040	1230	1850	1120	1680	999	1500	892	1340	799	1200
	24	1300	1950	1170	1760	1070	1610	952	1430	850	1280	761	1140
	26	1230	1850	1120	1680	1010	1520	903	1360	806	1210	721	1080
	28	1170	1760	1060	1590	959	1440	853	1280	761	1140	680	1020
	30	1100	1660	997	1500	904	1360	803	1210	716	1080	638	959
	32	1040	1560	936	1410	848	1270	752	1130	670	1010	597	897
	34	973	1460	875	1320	792	1190	702	1060	625	940	556	835
	36	907	1360	815	1230	737	1110	652	980	580	872	515	774
	40	781	1170	699	1050	630	947	556	836	494	743	437	657
Y-Y Axis	0	1720	2590	1570	2360	1440	2160	1290	1940	1150	1730	1030	1550
	10	1570	2360	1420	2130	1290	1940	1140	1720	1000	1510	891	1340
	12	1520	2280	1380	2070	1250	1880	1110	1670	979	1470	870	1310
	14	1460	2190	1320	1990	1200	1810	1070	1610	944	1420	841	1260
	16	1390	2100	1260	1900	1150	1720	1020	1530	901	1350	804	1210
	18	1320	1990	1200	1800	1090	1630	966	1450	853	1280	761	1140
	20	1250	1870	1130	1700	1020	1540	909	1370	801	1200	715	1070
	22	1170	1760	1060	1590	955	1440	849	1280	747	1120	667	1000
	24	1090	1630	982	1480	887	1330	788	1180	692	1040	617	928
	26	1010	1510	907	1360	818	1230	726	1090	636	956	567	852
	28	925	1390	833	1250	749	1130	664	999	581	873	518	778
	30	845	1270	760	1140	682	1020	604	908	527	792	469	705
	32	767	1150	688	1030	617	927	546	820	475	714	422	634
	34	692	1040	620	932	553	832	489	735	424	637	376	566
	36	619	930	554	833	494	743	437	657	379	570	337	506
	40	502	755	450	676	402	604	355	534	308	463	274	412

Properties

Property	195.5	178.5	163	146	130.5	117.5
A_g, in.^{2}	57.6	52.5	48.0	43.0	38.5	34.7
r_x, in.	4.61	4.56	4.52	4.48	4.46	4.41
r_y, in.	3.67	3.64	3.60	3.58	3.53	3.51

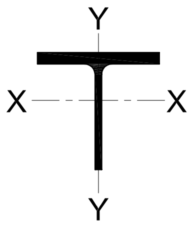
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

[c] Shape is slender for compression with F_y = 50 ksi; tabulated values have been adjusted accordingly.

F_y = 50 ksi

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes



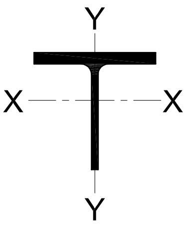
WT15

Shape		WT15×									
lb/ft		105.5[c]		95.5[c]		86.5[c]		74[c]		66[c]	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	912	1370	808	1210	719	1080	610	916	535	805
	10	869	1310	770	1160	686	1030	584	878	514	772
	12	851	1280	754	1130	672	1010	573	862	504	758
	14	830	1250	735	1110	655	985	561	843	494	742
	16	806	1210	714	1070	637	957	547	822	482	724
	18	780	1170	691	1040	617	927	531	799	468	704
	20	751	1130	667	1000	595	894	514	773	454	682
	22	718	1080	640	962	571	858	496	746	438	658
	24	684	1030	612	920	546	821	477	717	422	634
	26	648	974	582	875	521	783	457	687	404	608
	28	611	919	549	826	494	743	437	656	387	581
	30	575	864	516	776	467	703	415	624	368	553
	32	538	808	483	726	438	658	394	592	350	525
	34	501	753	450	676	408	613	370	556	331	497
	36	465	698	417	627	378	568	345	519	311	468
	40	395	593	354	532	321	483	297	447	269	404
Y-Y Axis	0	912	1370	808	1210	719	1080	610	916	535	805
	10	770	1160	664	998	570	857	464	697	384	577
	12	756	1140	654	982	562	845	435	654	362	545
	14	734	1100	638	959	550	827	401	602	335	504
	16	704	1060	616	926	533	801	359	540	302	455
	18	669	1000	589	886	511	769	314	472	265	398
	20	629	945	555	834	486	730	270	406	227	342
	22	587	882	518	779	456	686	228	343	193	289
	24	543	817	479	721	422	635	194	292	165	248
	26	499	750	440	662	388	583	167	251	142	214
	28	455	684	401	603	353	530	145	218	124	186
	30	412	620	363	545	318	479	127	191	109	164
	32	371	557	325	489	285	428	112	168	96.3	145
	34	330	496	290	436	254	382	99.6	150	85.8	129
	36	296	445	260	391	228	343	89.1	134	76.8	115
	40	241	362	212	319	187	281				
Properties											
A_g, in.^{2}		31.1		28.0		25.4		21.8		19.5	
r_x, in.		4.43		4.42		4.42		4.63		4.66	
r_y, in.		3.49		3.46		3.42		2.28		2.25	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 50 ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r equal to or greater than 200.



WT15

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes

F_y = 50 ksi

Shape		WT15×									
lb/ft		62[c]		58[c]		54[c]		49.5[c]		45[c]	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	493	740	458	688	420	632	376	565	331	498
	10	473	710	440	661	404	607	361	543	318	478
	12	464	698	432	649	397	596	355	534	313	470
	14	454	683	423	635	388	584	348	523	306	460
	16	443	666	412	620	379	570	340	510	299	449
	18	431	648	401	603	369	554	331	497	291	437
	20	418	628	389	585	358	538	321	482	282	424
	22	403	606	376	565	346	520	310	466	273	410
	24	388	584	362	544	333	501	299	450	263	396
	26	373	560	347	522	320	481	288	432	253	380
	28	356	535	332	499	306	461	276	414	242	364
	30	339	510	317	476	292	439	263	396	231	348
	32	322	484	301	452	278	418	250	376	220	331
	34	305	458	285	428	263	396	238	357	209	314
	36	288	432	269	404	249	374	225	338	197	297
	40	251	377	236	355	220	330	199	299	175	262
Y-Y Axis	0	493	740	458	688	420	632	376	565	331	498
	10	342	514	303	455	258	388	210	316	168	253
	12	324	486	286	430	245	368	199	300	160	241
	14	300	451	266	399	227	341	185	279	150	225
	16	272	409	241	362	206	310	167	252	137	206
	18	239	359	211	317	180	270	147	220	123	185
	20	205	307	180	271	155	232	128	192	109	163
	22	174	261	154	232	133	200	111	167	95.6	144
	24	149	224	133	200	116	174	97.1	146	84.3	127
	26	129	194	115	173	101	152	85.2	128	74.4	112
	28	113	169	101	152	88.5	133	75.2	113	66.0	99.2
	30	99.1	149	88.8	133	78.2	118	66.6	100	58.7	88.3
	32	87.7	132	78.8	118	69.5	105	59.4	89.3	52.5	79.0
	34	78.2	118	70.3	106	62.2	93.4	53.2	80.0	47.2	70.9
	36	70.1	105	63.1	94.8						

Properties

	62	58	54	49.5	45
A_g, in.^{2}	18.2	17.1	15.9	14.5	13.2
r_x, in.	4.66	4.67	4.69	4.71	4.69
r_y, in.	2.23	2.19	2.15	2.10	2.09

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 50 ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 50 ksi

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes

WT13.5

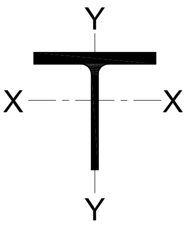
Shape			WT13.5×									
lb/ft			129		117.5		108.5		97[c]		89[c]		80.5[c]	
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to indicated axis	X-X Axis	0	1140	1710	1040	1560	958	1440	850	1280	778	1170	691	1040
		10	1070	1610	973	1460	896	1350	799	1200	732	1100	651	978
		12	1040	1560	945	1420	870	1310	777	1170	713	1070	634	953
		14	1000	1510	913	1370	840	1260	750	1130	691	1040	614	923
		16	965	1450	878	1320	807	1210	720	1080	664	997	592	890
		18	924	1390	839	1260	771	1160	687	1030	634	953	568	854
		20	879	1320	798	1200	732	1100	653	981	603	906	543	816
		22	832	1250	756	1140	692	1040	617	927	570	857	514	773
		24	784	1180	711	1070	651	978	579	871	536	805	483	726
		26	734	1100	666	1000	609	915	541	814	501	753	452	679
		28	684	1030	620	932	566	851	503	756	466	701	420	631
		30	635	954	575	864	524	787	465	699	432	649	388	583
		32	585	880	530	796	482	724	428	643	397	597	357	537
		34	537	807	486	730	441	663	391	588	364	547	327	491
		36	490	737	443	666	401	603	356	534	331	498	297	447
		40	402	604	362	544	327	492	290	435	270	406	242	364
	Y-Y Axis	0	1140	1710	1040	1560	958	1440	850	1280	778	1170	691	1040
		10	1010	1520	911	1370	833	1250	730	1100	650	978	568	854
		12	975	1470	880	1320	806	1210	709	1070	634	953	557	837
		14	932	1400	841	1260	772	1160	680	1020	610	917	539	811
		16	883	1330	797	1200	731	1100	645	969	580	871	515	774
		18	829	1250	748	1120	687	1030	606	910	545	819	485	728
		20	773	1160	697	1050	639	961	564	847	507	762	451	678
		22	715	1080	644	968	591	888	520	782	467	702	416	626
		24	657	987	590	887	541	814	476	716	427	642	380	572
		26	598	899	537	807	492	740	432	650	387	582	344	518
		28	541	813	485	729	444	668	390	585	348	523	309	465
		30	486	730	434	653	398	598	348	523	310	466	275	414
		32	432	649	385	579	353	530	308	463	274	412	243	366
		34	383	576	342	514	313	471	274	411	244	366	217	326
		36	342	515	306	459	280	421	245	368	218	328	194	292
		40	278	418	248	373	227	342	199	299	178	267	158	238

Properties

	129	117.5	108.5	97[c]	89[c]	80.5[c]
A_g, in.^{2}	38.1	34.7	32.0	28.6	26.3	23.8
r_x, in.	4.02	4.00	3.96	3.94	3.97	3.95
r_y, in.	3.36	3.33	3.32	3.29	3.25	3.23

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 50 ksi; tabulated values have been adjusted accordingly.



WT13.5

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes

F_y = 50 ksi

Shape		WT13.5×											
lb/ft		73[c]		64.5[c]		57[c]		51[c]		47[c]		42[c]	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to indicated axis													
X-X Axis	0	616	926	535	804	467	702	406	610	366	551	322	484
	10	580	872	507	762	443	666	385	579	348	523	306	460
	12	565	850	495	744	433	651	376	566	340	511	300	450
	14	548	824	482	724	421	633	366	551	331	498	292	439
	16	529	794	466	701	408	614	355	533	321	482	283	425
	18	507	763	450	676	394	592	342	515	310	466	273	411
	20	485	729	431	649	378	569	329	494	298	448	263	395
	22	461	693	412	620	362	544	315	473	285	429	252	379
	24	436	655	392	590	345	518	300	450	272	409	240	361
	26	410	616	371	558	327	491	284	427	258	388	229	343
	28	381	573	349	524	308	464	268	403	244	367	216	325
	30	352	530	325	488	290	436	252	379	230	345	204	306
	32	324	487	301	452	269	404	236	355	215	323	191	287
	34	296	446	277	417	248	373	220	330	201	302	179	268
	36	270	405	254	382	228	342	203	304	186	280	166	250
	40	220	330	210	316	189	284	168	252	156	234	141	212
Y-Y Axis	0	616	926	535	804	467	702	406	610	366	551	322	484
	10	490	737	409	614	338	509	280	421	239	359	189	284
	12	481	724	380	572	317	476	264	396	225	339	179	269
	14	468	704	345	519	290	436	242	364	208	312	165	249
	16	450	676	305	458	257	386	218	327	187	281	149	225
	18	427	642	264	397	223	335	189	284	163	245	130	196
	20	400	601	225	338	189	283	160	240	138	208	112	168
	22	369	554	188	283	159	239	135	203	118	177	96.6	145
	24	337	506	160	240	135	204	116	174	101	152	83.7	126
	26	305	458	137	206	117	175	100	150	87.9	132	73.0	110
	28	273	411	119	179	101	152	87.1	131	76.8	115	64.1	96.4
	30	243	365	104	156	88.9	134	76.5	115	67.6	102	56.6	85.1
	32	212	323	91.8	138	78.6	118	67.7	102	59.9	90.0	50.3	75.7
	34	192	288	81.6	123	69.9	105	60.3	90.6	53.4	80.3	45.0	67.6
	36	172	258	72.9	110	62.6	94.0						
	40	140	211										
Properties													
A_g, in.^{2}		21.6		18.9		16.8		15.0		13.8		12.4	
r_x, in.		3.95		4.13		4.15		4.14		4.16		4.18	
r_y, in.		3.20		2.21		2.18		2.15		2.12		2.07	

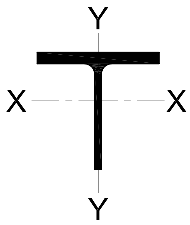
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 50 ksi

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes



WT12

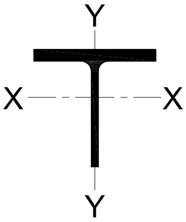
Shape		WT12×											
lb/ft		185[h]		167.5[h]		153[h]		139.5[h]		125		114.5	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to indicated axis													
X-X Axis	0	1630	2450	1470	2210	1340	2020	1230	1850	1100	1660	1010	1510
	10	1520	2280	1360	2050	1240	1870	1130	1700	1020	1530	927	1390
	12	1470	2210	1320	1980	1200	1810	1100	1650	981	1470	894	1340
	14	1410	2120	1270	1900	1160	1740	1050	1580	940	1410	856	1290
	16	1350	2030	1210	1820	1100	1660	1000	1510	896	1350	815	1230
	18	1290	1930	1150	1730	1050	1570	950	1430	848	1270	771	1160
	20	1220	1830	1090	1630	987	1480	895	1340	798	1200	724	1090
	22	1140	1720	1020	1530	925	1390	837	1260	745	1120	676	1020
	24	1070	1600	951	1430	861	1290	779	1170	692	1040	627	942
	26	992	1490	881	1320	797	1200	719	1080	638	959	577	868
	28	916	1380	812	1220	733	1100	661	993	585	879	528	794
	30	841	1260	744	1120	670	1010	603	906	532	800	480	722
	32	767	1150	677	1020	609	915	546	821	482	724	434	652
	34	696	1050	613	921	550	827	492	740	433	651	389	584
	36	627	943	550	827	492	740	440	661	386	581	347	521
	40	508	764	446	670	399	599	356	536	313	470	281	422
Y-Y Axis	0	1630	2450	1470	2210	1340	2020	1230	1850	1100	1660	1010	1510
	10	1470	2210	1320	1980	1200	1810	1090	1640	976	1470	885	1330
	12	1410	2120	1260	1900	1150	1730	1040	1570	933	1400	846	1270
	14	1340	2010	1200	1800	1090	1640	990	1490	883	1330	801	1200
	16	1260	1900	1130	1700	1030	1540	930	1400	829	1250	751	1130
	18	1180	1770	1050	1580	956	1440	867	1300	771	1160	698	1050
	20	1090	1650	976	1470	885	1330	800	1200	711	1070	643	966
	22	1010	1510	896	1350	811	1220	733	1100	651	978	587	882
	24	921	1380	817	1230	738	1110	666	1000	590	886	531	798
	26	834	1250	739	1110	666	1000	600	901	530	797	476	716
	28	750	1130	662	996	596	896	536	805	472	710	424	637
	30	669	1010	589	886	529	795	474	712	417	626	373	560
	32	591	888	519	780	466	700	417	627	367	551	328	493
	34	524	787	460	692	413	620	370	555	325	489	291	437
	36	468	703	411	617	368	554	330	496	290	436	260	390
	40	379	570	333	500	299	449	268	402	235	354	211	317

Properties

	185	167.5	153	139.5	125	114.5
A_g, in.^{2}	54.5	49.1	44.9	41.0	36.8	33.6
r_x, in.	3.78	3.73	3.69	3.65	3.61	3.58
r_y, in.	3.27	3.23	3.20	3.17	3.14	3.11

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.



WT12

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes

F_y = 50 ksi

Shape		WT12×									
lb/ft		103.5		96		88		81		73[c]	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to indicated axis: X-X Axis	0	907	1360	844	1270	772	1160	716	1080	637	957
	10	834	1250	776	1170	709	1070	657	987	589	886
	12	804	1210	748	1120	683	1030	632	950	569	855
	14	770	1160	715	1080	653	982	605	909	544	818
	16	733	1100	680	1020	621	933	574	863	517	776
	18	692	1040	642	965	586	880	542	814	487	732
	20	649	976	602	905	549	825	507	763	456	686
	22	605	910	561	843	511	768	472	709	425	638
	24	561	843	519	780	472	710	436	656	392	590
	26	516	775	477	717	433	652	400	602	360	541
	28	471	708	435	654	395	594	365	548	328	493
	30	428	643	395	593	358	538	330	496	297	446
	32	386	580	355	534	322	484	297	446	267	401
	34	345	518	317	477	287	431	264	397	238	357
	36	308	462	283	425	256	385	236	354	212	319
	40	249	374	229	345	207	312	191	287	172	258
Y-Y Axis	0	907	1360	844	1270	772	1160	716	1080	637	957
	10	792	1190	732	1100	662	995	605	909	530	797
	12	758	1140	701	1050	635	955	582	875	512	770
	14	717	1080	664	998	602	904	553	831	488	733
	16	672	1010	622	935	563	847	519	780	458	688
	18	623	937	577	868	522	785	482	724	425	639
	20	573	862	531	798	479	721	443	666	390	586
	22	522	785	483	727	436	655	403	606	354	533
	24	472	709	436	656	393	590	364	547	319	479
	26	422	635	390	587	351	527	325	488	284	427
	28	375	563	346	520	310	466	288	432	251	377
	30	329	494	303	456	272	408	252	379	220	330
	32	290	435	267	402	239	360	222	334	194	291
	34	257	386	237	356	212	319	197	297	172	259
	36	229	345	212	318	190	285	176	265	154	231
	40	186	280	172	258	154	231	143	215	125	188

Properties

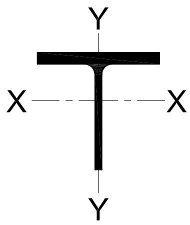
	103.5	96	88	81	73
A_g, in.^{2}	30.3	28.2	25.8	23.9	21.5
r_x, in.	3.55	3.53	3.51	3.50	3.50
r_y, in.	3.08	3.07	3.04	3.05	3.01

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 50 ksi; tabulated values have been adjusted accordingly.

F_y = 50 ksi

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes



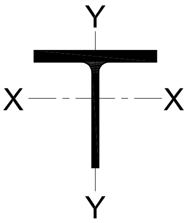
WT12

Shape		WT12×									
lb/ft		65.5[c]		58.5[c]		52[c]		51.5[c]		47[c]	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to indicated axis											
X-X Axis	0	565	849	494	742	430	646	428	644	385	579
	10	523	787	457	688	399	599	400	601	360	541
	12	506	761	442	665	386	580	388	584	349	525
	14	486	731	425	639	371	557	375	563	337	507
	16	465	698	406	611	354	533	360	541	324	487
	18	439	659	386	580	337	506	344	516	309	465
	20	411	618	364	547	318	478	326	490	294	441
	22	383	576	341	512	298	448	308	463	277	417
	24	354	532	315	473	278	418	288	433	260	391
	26	325	489	289	434	257	386	267	401	243	365
	28	297	446	264	396	234	352	245	368	224	336
	30	269	404	239	359	212	319	224	336	204	307
	32	242	364	215	323	191	287	203	305	186	279
	34	216	325	191	288	170	256	183	275	167	252
	36	193	289	171	257	152	228	164	246	150	225
	40	156	234	138	208	123	185	133	199	121	182
Y-Y Axis	0	565	849	494	742	430	646	428	644	385	579
	10	457	687	384	577	316	476	315	474	276	415
	12	444	667	375	564	310	466	287	431	252	379
	14	424	638	362	544	301	452	251	378	223	336
	16	399	600	344	517	288	432	215	323	191	287
	18	371	558	320	481	271	408	180	270	160	240
	20	340	512	294	442	251	378	148	222	132	198
	22	309	464	267	402	228	343	124	186	110	166
	24	277	417	240	360	205	308	105	157	93.7	141
	26	247	371	213	320	182	274	89.6	135	80.4	121
	28	217	326	187	281	160	240	77.6	117	69.7	105
	30	190	286	164	247	141	212	67.8	102	61.0	91.7
	32	168	252	145	218	125	188	59.8	89.8	53.8	80.9
	34	149	224	129	194	111	167				
	36	134	201	116	174	100	150				
	40	109	164	94.5	142	81.7	123				
Properties											
A_g, in.^{2}		19.3		17.2		15.3		15.1		13.8	
r_x, in.		3.52		3.51		3.51		3.67		3.67	
r_y, in.		2.97		2.94		2.91		1.99		1.98	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 50 ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r equal to or greater than 200.



WT12

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes

$F_y = 50$ ksi

Shape			WT12×									
lb/ft			42[c]		38[c]		34[c]		31[c]		27.5[c]	
Design			P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length, L_c (ft), with respect to indicated axis	X-X Axis	0	338	508	299	449	261	393	236	355	203	306
		10	316	475	280	421	245	368	223	334	192	288
		12	307	461	272	408	238	358	217	326	187	281
		14	296	445	263	395	230	346	210	316	181	272
		16	285	428	252	379	221	333	203	304	175	263
		18	272	409	241	362	212	318	194	292	168	252
		20	258	388	229	345	202	303	186	279	160	241
		22	244	367	217	326	191	287	176	265	153	229
		24	229	345	204	306	180	270	167	251	144	217
		26	214	322	191	287	168	253	157	236	136	204
		28	199	299	177	266	157	236	147	221	128	192
		30	184	276	164	246	145	218	137	206	119	179
		32	167	251	151	227	134	201	127	191	110	166
		34	150	226	137	205	122	184	117	175	102	153
		36	135	202	122	184	110	166	105	158	93.3	140
		40	109	164	98.9	149	89.3	134	85.4	128	76.3	115
	Y-Y Axis	0	338	508	299	449	261	393	236	355	203	306
		10	231	347	192	289	152	229	114	171	85.7	129
		12	212	318	177	267	141	212	92.4	139	71.2	107
		14	189	285	159	239	127	190	74.2	112	58.5	87.9
		16	163	245	138	207	109	164	60.1	90.4	48.1	72.3
		18	136	204	115	172	92.2	139	49.3	74.1	39.9	59.9
		20	113	169	96.0	144	78.0	117	41.0	61.6	33.4	50.2
		22	94.6	142	81.2	122	66.5	100	34.5	51.9	28.3	42.5
		24	80.5	121	69.3	104	57.2	85.9				
		26	69.3	104	59.8	89.9	49.5	74.5				
		28	60.2	90.4	52.1	78.3	43.3	65.1				
		30	52.7	79.3	45.7	68.7	38.1	57.3				
		32	46.6	70.0	40.4	60.7						

Properties

	42[c]	38[c]	34[c]	31[c]	27.5[c]
A_g, in.^{2}	12.4	11.2	10.0	9.11	8.10
r_x, in.	3.67	3.68	3.70	3.79	3.80
r_y, in.	1.95	1.92	1.87	1.38	1.34

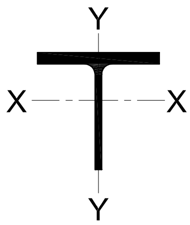
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 50 ksi

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes



WT10.5

Shape		WT10.5×											
lb/ft		100.5		91		83		73.5		66		61	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	886	1330	802	1210	731	1100	647	972	581	873	536	806
	10	794	1190	718	1080	652	980	579	870	519	780	478	719
	12	757	1140	683	1030	620	932	551	828	494	742	455	684
	14	715	1070	645	969	584	878	520	782	466	700	429	644
	16	669	1010	603	906	546	820	487	732	436	655	400	602
	18	621	934	559	840	505	759	451	678	403	606	371	557
	20	572	859	513	771	463	696	415	624	370	557	340	511
	22	521	784	467	702	421	633	378	568	337	507	309	464
	24	471	709	422	634	379	570	341	513	304	457	278	418
	26	423	635	377	567	338	508	305	459	272	408	248	373
	28	375	564	334	502	299	449	271	407	241	362	219	330
	30	330	496	293	440	262	393	238	357	211	317	192	288
	32	290	436	257	387	230	345	209	314	185	278	169	253
	34	257	386	228	343	204	306	185	278	164	247	149	225
	36	229	344	203	306	182	273	165	248	146	220	133	200
	40	186	279	165	248	147	221	134	201	119	178	108	162
Y-Y Axis	0	886	1330	802	1210	731	1100	647	972	581	873	536	806
	10	779	1170	701	1050	635	954	550	826	485	728	440	661
	12	742	1110	668	1000	606	910	526	791	466	700	424	637
	14	699	1050	630	946	571	858	497	746	440	662	402	604
	16	653	981	587	883	533	800	463	696	411	618	375	564
	18	604	908	543	816	492	739	427	642	379	570	346	520
	20	553	832	497	747	450	676	390	586	346	520	316	474
	22	502	755	450	677	408	613	353	530	313	470	285	428
	24	452	679	405	608	366	550	316	475	279	420	254	382
	26	402	605	360	541	325	489	280	421	247	372	225	338
	28	355	534	317	477	287	431	245	369	216	325	196	295
	30	310	467	277	416	250	376	214	322	189	284	172	258
	32	273	411	244	366	220	331	189	284	167	251	151	227
	34	242	364	216	325	195	293	168	252	148	222	134	202
	36	216	325	193	290	174	262	150	225	132	199	120	181
	40	175	263	156	235	141	213	121	183	107	161	97.6	147

Properties

	100.5	91	83	73.5	66	61
A_g, in.^{2}	29.6	26.8	24.4	21.6	19.4	17.9
r_x, in.	3.10	3.07	3.04	3.08	3.06	3.04
r_y, in.	3.02	3.00	2.99	2.95	2.93	2.91

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes

WT10.5

F_y = 50 ksi

Shape			WT10.5×											
lb/ft			55.5[c]		50.5[c]		46.5[c]		41.5[c]		36.5[c]		34[c]	
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to indicated axis	X-X Axis	0	480	722	431	648	407	612	353	530	300	451	276	414
		10	433	651	389	584	371	558	323	485	275	413	252	379
		12	414	622	371	558	355	534	310	467	264	397	243	365
		14	390	586	352	528	337	507	296	445	252	379	232	348
		16	364	547	330	496	318	478	281	422	239	360	220	330
		18	337	506	306	460	297	446	263	395	225	338	207	311
		20	308	464	280	421	275	414	243	366	210	316	193	290
		22	280	421	254	382	253	381	223	336	195	293	179	269
		24	252	379	228	343	231	347	204	306	178	267	165	248
		26	225	338	203	306	209	314	184	276	161	241	149	225
		28	199	298	179	270	188	282	165	248	144	216	134	201
		30	174	261	157	235	167	251	146	220	128	192	119	178
		32	153	229	138	207	148	222	129	194	112	169	104	157
		34	135	203	122	183	131	196	114	172	99.6	150	92.5	139
		36	121	181	109	163	117	175	102	153	88.8	133	82.5	124
		40	97.6	147	88.1	132	94.4	142	82.5	124	71.9	108	66.8	100
	Y-Y Axis	0	480	722	431	648	407	612	353	530	300	451	276	414
		10	390	587	343	516	282	424	245	369	206	309	185	277
		12	378	568	335	503	248	372	216	324	183	275	166	249
		14	360	541	322	483	211	318	184	277	156	235	142	214
		16	337	506	303	455	176	264	153	230	130	195	118	177
		18	311	468	280	421	142	214	124	187	105	158	95.8	144
		20	284	427	256	385	117	175	102	153	86.4	130	79.0	119
		22	256	385	231	347	97.0	146	84.9	128	72.2	108	66.2	99.5
		24	229	344	206	310	81.9	123	71.8	108	61.1	91.9	56.1	84.4
		26	202	304	182	274	70.1	105	61.4	92.4	52.4	78.8	48.2	72.4
		28	176	265	159	239	60.6	91.1	53.2	79.9	45.4	68.2	41.8	62.8
		30	154	232	139	210	52.9	79.6	46.5	69.8	39.7	59.7	36.5	54.9
		32	136	205	123	185								
		34	121	182	109	165								
		36	108	163	97.9	147								
		40	88.1	132	79.7	120								

Properties

	55.5	50.5	46.5	41.5	36.5	34
A_g, in.^{2}	16.3	14.9	13.7	12.2	10.7	10.0
r_x, in.	3.03	3.01	3.25	3.22	3.21	3.20
r_y, in.	2.90	2.89	1.84	1.83	1.81	1.80

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 50 ksi

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes

WT10.5

Shape		WT10.5×											
lb/ft		31[c]		27.5[c]		24[c]		28.5[c]		25[c]		22[c]	
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length, L_c (ft), with respect to indicated axis: X-X Axis	0	247	371	215	323	182	274	225	338	193	290	165	248
	10	226	340	197	296	168	252	207	312	178	268	153	229
	12	218	327	190	285	162	243	200	301	172	258	147	221
	14	208	313	182	273	155	233	192	288	165	248	141	213
	16	197	297	172	259	147	222	182	274	157	236	135	203
	18	186	279	163	244	139	209	173	259	149	223	128	192
	20	174	261	152	229	131	197	162	244	140	210	120	181
	22	161	243	142	213	122	183	151	227	131	196	113	169
	24	149	224	131	197	113	170	140	210	121	182	105	157
	26	136	204	120	180	104	156	129	194	112	168	96.5	145
	28	123	184	109	164	94.7	142	117	176	102	154	88.5	133
	30	109	164	97.8	147	85.8	129	104	157	92.3	139	80.5	121
	32	95.9	144	86.1	129	76.6	115	92.3	139	81.7	123	72.5	109
	34	84.9	128	76.3	115	67.8	102	81.8	123	72.4	109	64.2	96.5
	36	75.8	114	68.1	102	60.5	91.0	73.0	110	64.6	97.0	57.3	86.1
	40	61.4	92.2	55.1	82.9	49.0	73.7	59.1	88.8	52.3	78.6	46.4	69.7
Y-Y Axis	0	247	371	215	323	182	274	225	338	193	290	165	248
	10	158	237	124	187	90.4	136	120	180	91.0	137	67.9	102
	12	142	213	113	169	81.9	123	94.1	141	71.5	107	54.8	82.4
	14	123	185	97.8	147	70.6	106	72.5	109	56.2	84.5	44.1	66.2
	16	101	152	81.0	122	59.9	90.0	57.1	85.9	44.9	67.5	35.7	53.6
	18	82.9	125	67.2	101	50.7	76.1	46.0	69.2	36.5	54.8	29.3	44.0
	20	68.7	103	56.2	84.5	43.0	64.6	37.8	56.8	30.1	45.3	24.3	36.6
	22	57.7	86.7	47.6	71.5	36.7	55.2	31.5	47.4				
	24	49.0	73.7	40.6	61.1	31.6	47.5						
	26	42.2	63.4	35.1	52.7	27.4	41.2						
	28	36.6	55.0	30.5	45.9								

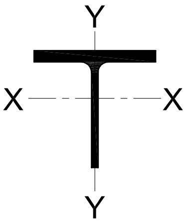
Properties

	31	27.5	24	28.5	25	22
A_g, in.^{2}	9.13	8.10	7.07	8.37	7.36	6.49
r_x, in.	3.21	3.23	3.26	3.29	3.30	3.31
r_y, in.	1.77	1.73	1.66	1.35	1.30	1.26

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r equal to or greater than 200.



WT9

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes

$F_y = 50$ ksi

Shape		WT9×											
lb/ft		87.5		79		71.5		65		59.5		53	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	769	1160	695	1040	629	945	575	864	527	792	467	702
	10	663	997	597	897	538	809	491	738	451	678	399	600
	12	621	933	558	839	502	755	458	688	421	633	373	560
	14	575	864	515	775	463	696	422	634	388	584	343	516
	16	526	790	470	707	422	634	383	576	354	532	313	470
	18	475	714	424	638	380	571	344	518	318	478	281	422
	20	424	638	378	568	337	507	305	459	283	425	249	375
	22	374	563	332	500	296	445	267	402	248	373	219	328
	24	327	491	289	434	256	385	231	347	215	323	189	284
	26	281	422	248	372	219	329	197	297	184	276	162	243
	28	242	364	214	321	189	284	170	256	158	238	139	209
	30	211	317	186	280	165	247	148	223	138	207	121	182
	32	185	279	164	246	145	217	130	196	121	182	107	160
	34	164	247	145	218	128	193	115	173	107	161	94.5	142
	36	146	220	129	194	114	172	103	155	95.8	144	84.3	127
	40	119	178	105	157	92.6	139	83.4	125	77.6	117	68.3	103
Y-Y Axis	0	769	1160	695	1040	629	945	575	864	527	792	467	702
	10	664	999	597	898	538	809	489	735	443	666	387	581
	12	626	940	562	845	506	761	460	692	418	628	365	549
	14	583	876	523	786	471	708	427	642	389	584	340	511
	16	536	806	481	723	432	650	392	590	357	536	312	468
	18	488	734	437	657	393	590	356	535	323	486	282	424
	20	440	661	393	591	353	530	319	479	290	435	252	379
	22	392	589	350	525	313	470	283	425	256	385	223	335
	24	345	518	307	462	275	413	247	372	224	337	194	292
	26	300	451	267	401	238	357	214	321	194	291	167	251
	28	259	389	230	346	205	308	185	277	167	252	145	217
	30	226	340	201	302	179	269	161	242	146	220	126	190
	32	199	299	177	265	157	237	142	213	129	193	111	167
	34	176	265	157	235	140	210	126	189	114	171	98.6	148
	36	157	236	140	210	125	187	112	168	102	153	88.1	132
	40	127	191	113	170	101	152	90.9	137	82.6	124	71.5	107

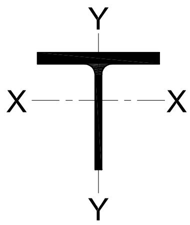
Properties

	87.5	79	71.5	65	59.5	53
A_g, in.^{2}	25.7	23.2	21.0	19.2	17.6	15.6
r_x, in.	2.66	2.63	2.60	2.58	2.60	2.59
r_y, in.	2.76	2.74	2.72	2.70	2.69	2.66

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

F_y = 50 ksi

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes



WT9

Shape		WT9×											
lb/ft		48.5		43[c]		38[c]		35.5[c]		32.5[c]		30[c]	
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	425	639	376	565	322	483	309	464	278	418	252	379
	10	362	544	323	486	278	417	271	407	245	368	222	334
	12	337	507	301	453	260	391	254	382	232	349	210	316
	14	310	466	277	416	241	362	237	356	216	325	197	296
	16	282	424	251	378	219	329	217	327	199	299	182	274
	18	253	380	225	338	196	294	198	297	180	271	166	249
	20	224	336	199	299	173	260	178	267	162	243	149	224
	22	195	294	174	261	151	227	158	237	144	216	132	198
	24	169	253	150	225	130	195	139	209	126	189	116	174
	26	144	216	128	192	111	166	121	181	109	164	100	150
	28	124	186	110	165	95.3	143	104	156	94.1	141	86.2	130
	30	108	162	95.8	144	83.1	125	90.6	136	81.9	123	75.1	113
	32	94.9	143	84.2	127	73.0	110	79.6	120	72.0	108	66.0	99.2
	34	84.0	126	74.6	112	64.7	97.2	70.5	106	63.8	95.9	58.5	87.9
	36	75.0	113	66.5	100	57.7	86.7	62.9	94.5	56.9	85.5	52.2	78.4
	40	60.7	91.2	53.9	81.0	46.7	70.2	50.9	76.6	46.1	69.3	42.3	63.5
Y-Y Axis	0	425	639	376	565	322	483	309	464	278	418	252	379
	10	348	523	302	454	252	379	205	308	186	279	168	253
	12	330	496	288	433	243	366	176	264	159	239	144	217
	14	307	461	269	405	230	345	146	219	132	198	119	179
	16	282	423	248	372	212	318	117	176	106	159	95.6	144
	18	255	383	224	337	192	288	93.5	140	84.4	127	76.6	115
	20	228	342	200	301	171	257	76.3	115	68.9	104	62.6	94.1
	22	201	302	176	265	151	227	63.3	95.2	57.3	86.1	52.1	78.3
	24	175	263	153	231	131	197	53.4	80.3	48.4	72.7	44.0	66.1
	26	151	226	132	198	113	169	45.7	68.6	41.3	62.1	37.6	56.6
	28	130	196	114	172	97.7	147	39.5	59.3	35.7	53.7	32.6	48.9
	30	114	171	99.8	150	85.4	128						
	32	100	151	88.0	132	75.4	113						
	34	89.0	134	78.1	117	66.9	101						
	36	79.5	119	69.8	105	59.9	90.0						
	40	64.5	96.9	56.7	85.2	48.7	73.1						

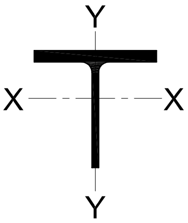
Properties

	48.5	43[c]	38[c]	35.5[c]	32.5[c]	30[c]
A_g, in.^{2}	14.2	12.7	11.1	10.4	9.55	8.82
r_x, in.	2.56	2.55	2.54	2.74	2.72	2.71
r_y, in.	2.65	2.63	2.61	1.70	1.69	1.68

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r equal to or greater than 200.



WT9

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes

F_y = 50 ksi

Shape		WT9×									
lb/ft		27.5[c]		25[c]		23[c]		20[c]		17.5[c]	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	228	343	202	303	185	278	155	233	133	199
	10	201	302	178	267	165	247	138	207	118	178
	12	190	286	168	253	156	235	131	196	113	169
	14	178	268	158	237	147	221	123	185	106	160
	16	165	249	146	220	137	206	115	172	99.2	149
	18	152	228	134	202	126	190	106	159	91.8	138
	20	137	205	122	183	115	174	96.6	145	84.2	127
	22	121	182	109	164	104	157	87.4	131	76.5	115
	24	106	160	95.6	144	92.0	138	78.3	118	68.8	103
	26	91.9	138	82.6	124	80.2	120	69.2	104	61.2	92.0
	28	79.2	119	71.2	107	69.2	104	59.6	89.6	53.4	80.2
	30	69.0	104	62.1	93.3	60.2	90.5	51.9	78.1	46.5	69.9
	32	60.6	91.1	54.5	82.0	53.0	79.6	45.7	68.6	40.9	61.4
	34	53.7	80.7	48.3	72.6	46.9	70.5	40.4	60.8	36.2	54.4
	36	47.9	72.0	43.1	64.8	41.8	62.9	36.1	54.2	32.3	48.5
	40	38.8	58.3	34.9	52.5	33.9	50.9	29.2	43.9	26.2	39.3
Y-Y Axis	0	228	343	202	303	185	278	155	233	133	199
	10	150	225	128	193	95.9	144	77.1	116	57.3	86.1
	12	129	194	112	169	73.2	110	59.6	89.5	44.6	67.0
	14	107	161	93.1	140	55.5	83.5	45.7	68.7	35.0	52.6
	16	85.4	128	74.4	112	43.4	65.2	35.9	53.9	27.9	41.9
	18	68.7	103	60.1	90.3	34.7	52.2	28.8	43.4	22.6	34.0
	20	56.3	84.7	49.4	74.2	28.4	42.6	23.6	35.5	18.7	28.0
	22	47.0	70.6	41.2	62.0						
	24	39.7	59.7	34.9	52.5						
	26	34.0	51.1	29.9	45.0						

Properties

	27.5	25	23	20	17.5
A_g, in.^{2}	8.10	7.34	6.77	5.88	5.15
r_x, in.	2.71	2.70	2.77	2.76	2.79
r_y, in.	1.67	1.65	1.29	1.27	1.22

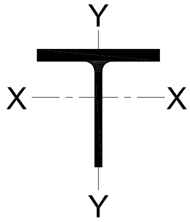
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 50 ksi

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes



WT8

Shape			WT8×											
lb/ft			50		44.5		38.5[c]		33.5[c]		28.5[c]		25[c]	
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to indicated axis	X-X Axis	0	440	662	392	590	338	508	287	431	248	373	212	319
		10	359	540	320	481	274	412	236	355	210	315	181	271
		12	329	494	292	439	250	376	216	325	193	291	168	253
		14	296	445	263	395	224	337	193	290	176	265	154	232
		16	262	394	232	349	198	297	170	255	158	237	138	208
		18	228	343	202	304	171	258	147	221	140	210	122	183
		20	196	294	173	260	146	220	125	188	122	183	106	160
		22	165	248	146	219	122	184	104	157	104	157	91.1	137
		24	138	208	122	184	103	154	87.6	132	88.3	133	76.9	116
		26	118	177	104	157	87.5	132	74.7	112	75.2	113	65.5	98.5
		28	102	153	89.9	135	75.5	113	64.4	96.7	64.9	97.5	56.5	84.9
		30	88.6	133	78.3	118	65.8	98.8	56.1	84.3	56.5	84.9	49.2	74.0
		32	77.9	117	68.8	103	57.8	86.9	49.3	74.1	49.7	74.7	43.3	65.0
		34	69.0	104	61.0	91.6	51.2	76.9	43.7	65.6	44.0	66.1	38.3	57.6
		36	61.5	92.5	54.4	81.7	45.7	68.6	38.9	58.5	39.2	59.0	34.2	51.4
		40									31.8	47.8	27.7	41.6
	Y-Y Axis	0	440	662	392	590	338	508	287	431	248	373	212	319
		10	365	548	321	483	272	408	229	344	159	238	136	205
		12	340	511	300	451	255	383	217	326	133	200	114	172
		14	312	470	276	414	234	352	200	301	108	162	92.4	139
		16	283	425	249	375	212	319	181	273	84.4	127	72.4	109
		18	253	380	222	334	189	284	162	243	67.2	101	57.9	87.0
		20	223	335	196	294	166	249	142	214	54.8	82.3	47.2	71.0
		22	194	291	170	255	144	216	123	185	45.5	68.3	39.2	59.0
		24	166	249	145	218	122	184	105	157	38.3	57.6	33.1	49.8
		26	141	213	124	186	105	157	89.6	135	32.7	49.2	28.3	42.5
		28	122	184	107	161	90.4	136	77.5	117				
		30	107	160	93.3	140	78.9	119	67.7	102				
		32	93.7	141	82.1	123	69.5	104	59.7	89.7				
		34	83.1	125	72.8	109	61.6	92.6	52.9	79.6				
		36	74.2	112	65.0	97.7	55.0	82.7	47.3	71.1				
		40	60.2	90.4	52.7	79.2	44.7	67.1	38.4	57.7				

Properties

	50	44.5	38.5	33.5	28.5	25
A_g, in.^{2}	14.7	13.1	11.3	9.81	8.39	7.37
r_x, in.	2.28	2.27	2.24	2.22	2.41	2.40
r_y, in.	2.51	2.49	2.47	2.46	1.60	1.59

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes

WT8

F_y = 50 ksi

Shape		WT8×									
lb/ft		22.5[c]		20[c]		18[c]		15.5[c]		13[c]	
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length, L_c (ft), with respect to indicated axis: X-X Axis	0	187	280	161	242	143	215	119	180	97.0	146
	10	159	239	137	206	122	184	103	155	83.9	126
	12	148	222	127	191	114	171	96.4	145	78.7	118
	14	136	204	117	176	105	158	89.2	134	73.0	110
	16	123	185	106	159	95.6	144	81.5	122	67.0	101
	18	109	164	94.6	142	85.9	129	73.6	111	60.7	91.2
	20	95.0	143	83.3	125	76.1	114	65.6	98.6	54.3	81.6
	22	81.3	122	71.2	107	65.9	99.0	57.7	86.8	48.0	72.2
	24	68.6	103	60.0	90.1	55.7	83.7	49.6	74.6	41.8	62.9
	26	58.5	87.9	51.1	76.8	47.4	71.3	42.3	63.5	36.2	54.4
	28	50.4	75.8	44.0	66.2	40.9	61.5	36.4	54.8	31.2	46.9
	30	43.9	66.0	38.4	57.7	35.6	53.6	31.7	47.7	27.2	40.8
	32	38.6	58.0	33.7	50.7	31.3	47.1	27.9	41.9	23.9	35.9
	34	34.2	51.4	29.9	44.9	27.7	41.7	24.7	37.1	21.2	31.8
	36	30.5	45.8	26.6	40.0	24.7	37.2	22.0	33.1	18.9	28.4
	40					20.0	30.1	17.9	26.8	15.3	23.0
Y-Y Axis	0	187	280	161	242	143	215	119	180	97.0	146
	10	119	178	99.7	150	81.8	123	53.4	80.3	36.6	55.1
	12	99.5	150	85.5	128	70.1	105	39.7	59.7	28.4	42.6
	14	80.2	120	69.1	104	56.2	84.5	30.3	45.6	22.2	33.3
	16	62.8	94.4	54.4	81.7	44.8	67.4	23.8	35.7	17.6	26.5
	18	50.3	75.7	43.7	65.7	36.3	54.6	19.1	28.7	14.3	21.5
	20	41.2	61.9	35.8	53.8	29.9	45.0				
	22	34.2	51.5	29.8	44.9	25.0	37.6				
	24	28.9	43.5	25.2	37.9	21.2	31.9				
	26	24.7	37.2	21.6	32.5						

Properties	22.5	20	18	15.5	13
A_g, in.^{2}	6.63	5.89	5.29	4.56	3.84
r_x, in.	2.39	2.37	2.41	2.45	2.47
r_y, in.	1.57	1.56	1.52	1.17	1.12

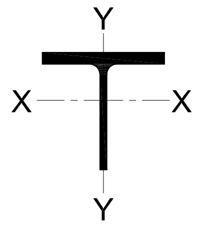
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 50 ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 50 ksi

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes



WT7

Shape		WT7×											
lb/ft		66		60		54.5		49.5		45		41	
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	581	873	530	797	479	720	437	657	395	594	359	540
	10	409	614	370	556	330	496	300	450	270	405	264	397
	12	350	526	316	474	280	421	254	381	228	343	231	347
	14	291	438	262	393	231	347	209	313	187	281	197	295
	16	236	355	211	317	184	277	166	250	148	223	163	246
	18	187	281	167	251	145	219	131	197	117	176	132	199
	20	152	228	135	203	118	177	106	160	94.9	143	107	161
	22	125	188	112	168	97.4	146	87.8	132	78.4	118	88.6	133
	24	105	158	93.8	141	81.8	123	73.8	111	65.9	99.1	74.4	112
	26	89.7	135	79.9	120	69.7	105	62.9	94.5	56.2	84.4	63.4	95.3
	28	77.3	116	68.9	104	60.1	90.4					54.7	82.2
	30											47.6	71.6
Y-Y Axis	0	581	873	530	797	479	720	437	657	395	594	359	540
	10	519	781	485	699	412	619	366	549	320	481	298	446
	12	512	770	461	693	410	616	364	547	319	480	277	417
	14	497	747	450	676	403	605	360	541	317	476	254	382
	16	477	716	433	650	389	585	351	527	312	468	229	345
	18	454	682	412	619	371	558	336	505	301	452	204	307
	20	429	645	389	585	351	527	318	478	286	429	179	270
	22	403	606	366	550	330	495	299	449	269	404	155	234
	24	376	566	342	513	308	462	279	419	251	377	133	199
	26	350	525	317	476	285	429	258	388	233	349	113	170
	28	323	485	292	439	263	395	238	358	214	322	97.6	147
	30	296	445	268	402	241	362	218	328	196	295	85.1	128
	32	270	406	244	367	219	330	198	298	178	268	74.9	113
	34	245	368	221	332	199	299	179	269	161	242	66.4	99.7
	36	220	331	198	298	178	268	161	242	144	217	59.2	89.0
	40	178	268	161	242	145	217	130	196	117	176	48.0	72.1

Properties

	66	60	54.5	49.5	45	41
A_g, in.^{2}	19.4	17.7	16.0	14.6	13.2	12.0
r_x, in.	1.73	1.71	1.68	1.67	1.66	1.85
r_y, in.	3.76	3.74	3.73	3.71	3.70	2.48

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes

WT7

$F_y = 50$ ksi

Shape		WT7×											
lb/ft		37		34		30.5[c]		26.5[c]		24[c]		21.5[c]	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to indicated axis													
X-X Axis	0	326	491	299	450	267	402	232	349	207	312	182	273
	10	237	357	217	326	194	291	173	261	157	236	138	207
	12	206	310	188	283	168	253	152	229	138	207	122	183
	14	175	263	159	240	142	213	130	196	118	177	104	156
	16	145	217	132	198	117	175	109	164	98.7	148	86.7	130
	18	116	175	106	159	93.5	141	88.8	133	80.5	121	70.3	106
	20	94.2	142	85.5	128	75.8	114	71.9	108	65.2	98.0	57.0	85.6
	22	77.9	117	70.7	106	62.6	94.1	59.5	89.4	53.9	81.0	47.1	70.8
	24	65.4	98.3	59.4	89.2	52.6	79.1	50.0	75.1	45.3	68.1	39.6	59.5
	26	55.7	83.8	50.6	76.0	44.8	67.4	42.6	64.0	38.6	58.0	33.7	50.7
	28	48.1	72.2	43.6	65.6	38.7	58.1	36.7	55.2	33.3	50.0	29.1	43.7
	30	41.9	62.9	38.0	57.1	33.7	50.6	32.0	48.1	29.0	43.6	25.3	38.1
Y-Y Axis	0	326	491	299	450	267	402	232	349	207	312	182	273
	10	270	405	245	369	217	326	171	257	153	230	133	200
	12	251	378	229	344	203	305	151	228	136	204	119	179
	14	230	346	210	315	186	280	131	196	117	176	102	154
	16	208	313	189	284	168	253	110	166	98.7	148	86.1	129
	18	185	279	168	253	149	224	90.8	136	81.1	122	70.4	106
	20	163	245	147	222	131	197	73.8	111	66.0	99.2	57.4	86.3
	22	141	212	127	191	113	170	61.2	92.0	54.8	82.3	47.7	71.7
	24	120	181	108	163	96.0	144	51.5	77.5	46.1	69.3	40.2	60.4
	26	103	154	92.5	139	82.0	123	44.0	66.1	39.4	59.2	34.3	51.6
	28	88.6	133	79.9	120	70.9	106	38.0	57.1	34.0	51.1	29.7	44.6
	30	77.3	116	69.7	105	61.8	92.9	33.1	49.8	29.7	44.6	25.9	38.9
	32	68.0	102	61.3	92.1	54.4	81.8	29.1	43.8				
	34	60.2	90.5	54.3	81.7	48.2	72.5						
	36	53.8	80.8	48.5	72.9	43.1	64.7						
	40	43.6	65.5	39.3	59.1	34.9	52.5						
Properties													
A_g, in.^{2}		10.9		10.0		8.96		7.80		7.07		6.31	
r_x, in.		1.82		1.81		1.80		1.88		1.88		1.86	
r_y, in.		2.48		2.46		2.45		1.92		1.91		1.89	

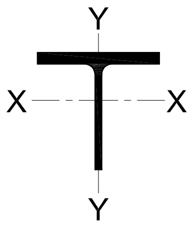
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 50 ksi

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes



WT7

Shape		WT7×									
lb/ft		19[c]		17[c]		15[c]		13[c]		11[c]	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	159	239	140	210	122	183	103	155	84.1	126
	10	127	191	112	168	98.1	147	84.3	127	69.2	104
	12	115	173	101	152	89.2	134	77.2	116	63.5	95.4
	14	102	153	90.1	135	79.8	120	69.5	104	57.4	86.2
	16	87.4	131	78.3	118	70.0	105	61.5	92.4	51.0	76.7
	18	73.6	111	66.0	99.1	59.7	89.7	53.5	80.4	44.6	67.0
	20	60.6	91.1	54.3	81.6	49.4	74.3	45.2	67.9	38.4	57.6
	22	50.1	75.3	44.9	67.4	40.8	61.4	37.3	56.1	32.1	48.2
	24	42.1	63.2	37.7	56.7	34.3	51.6	31.4	47.1	27.0	40.5
	26	35.9	53.9	32.1	48.3	29.2	44.0	26.7	40.2	23.0	34.5
	28	30.9	46.5	27.7	41.6	25.2	37.9	23.0	34.6	19.8	29.8
	30	26.9	40.5	24.1	36.3	22.0	33.0	20.1	30.2	17.3	25.9
	32	23.7	35.6	21.2	31.9	19.3	29.0	17.6	26.5	15.2	22.8
	34	21.0	31.5	18.8	28.2	17.1	25.7	15.6	23.5	13.4	20.2
Y-Y Axis	0	159	239	140	210	122	183	103	155	84.1	126
	10	101	152	86.4	130	70.2	106	41.4	62.3	30.3	45.5
	12	83.9	126	72.3	109	58.8	88.4	30.0	45.1	22.5	33.8
	14	67.1	101	57.6	86.5	46.5	69.9	22.6	33.9	17.1	25.8
	16	52.3	78.6	45.0	67.7	36.8	55.3	17.5	26.4	13.4	20.2
	18	41.8	62.8	36.1	54.2	29.7	44.6	14.0	21.0		
	20	34.1	51.2	29.5	44.3	24.4	36.6				
	22	28.3	42.5	24.5	36.9	20.3	30.5				
	24	23.9	35.9	20.7	31.1	17.2	25.9				

Properties

	19	17	15	13	11
A_g, in.^{2}	5.58	5.00	4.42	3.85	3.25
r_x, in.	2.04	2.04	2.07	2.12	2.14
r_y, in.	1.55	1.53	1.49	1.08	1.04

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes

WT6

F_y = 50 ksi

Shape		WT6×											
lb/ft		29		26.5		25		22.5		20[c]		17.5[c]	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	255	383	233	350	219	329	196	295	172	258	151	226
	4	237	356	216	325	205	308	184	276	161	242	143	215
	6	216	324	197	296	188	283	169	254	149	224	135	203
	8	189	284	173	261	168	252	150	226	133	200	124	186
	10	160	240	147	221	145	218	130	195	114	171	110	166
	12	130	195	120	180	121	182	108	162	94.5	142	94.9	143
	14	102	153	94.2	142	97.6	147	86.8	130	75.7	114	79.5	120
	16	78.2	117	72.3	109	76.2	115	67.6	102	58.7	88.2	64.8	97.5
	18	61.8	92.8	57.1	85.9	60.2	90.5	53.4	80.3	46.4	69.7	51.6	77.5
	20	50.0	75.2	46.3	69.6	48.8	73.3	43.3	65.0	37.6	56.5	41.8	62.8
	22	41.3	62.1	38.3	57.5	40.3	60.6	35.8	53.8	31.0	46.7	34.5	51.9
	24	34.7	52.2	32.1	48.3	33.9	50.9	30.1	45.2	26.1	39.2	29.0	43.6
	26					28.9	43.4	25.6	38.5	22.2	33.4	24.7	37.2
	28											21.3	32.0
Y-Y Axis	0	255	383	233	350	219	329	196	295	172	258	151	226
	4	222	334	197	296	194	291	170	255	147	220	127	191
	6	221	333	196	295	190	285	167	251	145	217	122	183
	8	219	329	194	292	179	269	159	239	139	209	111	167
	10	211	317	189	284	163	245	145	218	128	192	95.4	143
	12	197	297	178	267	145	218	129	194	114	171	78.8	118
	14	182	273	164	246	126	189	112	168	98.8	149	62.7	94.2
	16	165	247	148	222	107	161	95.0	143	83.7	126	48.6	73.1
	18	147	221	132	198	88.8	133	78.8	118	69.3	104	38.7	58.1
	20	130	195	116	174	72.3	109	64.2	96.5	56.4	84.8	31.4	47.3
	22	113	169	100	151	59.9	90.1	53.2	79.9	46.8	70.3	26.1	39.2
	24	96.4	145	85.7	129	50.4	75.8	44.8	67.3	39.4	59.2	22.0	33.0
	26	82.3	124	73.1	110	43.0	64.7	38.2	57.4	33.6	50.5		
	28	71.0	107	63.2	94.9	37.1	55.8	33.0	49.6	29.0	43.6		
	30	61.9	93.1	55.1	82.8	32.4	48.6	28.8	43.2	25.3	38.0		
	32	54.5	81.8	48.5	72.8	28.5	42.8	25.3	38.0	22.3	33.5		

Properties

	29	26.5	25	22.5	20[c]	17.5[c]
A_g, in.^{2}	8.52	7.78	7.30	6.56	5.84	5.17
r_x, in.	1.50	1.51	1.60	1.59	1.57	1.76
r_y, in.	2.51	2.48	1.96	1.95	1.94	1.54

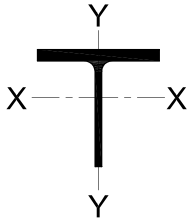
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 50 ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 50 ksi

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes



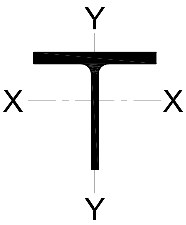
WT6

Shape		WT6×											
lb/ft		15[c]		13[c]		11[c]		9.5[c]		8[c]		7[c]	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to indicated axis													
X-X Axis	0	125	187	105	158	89.8	135	75.0	113	61.6	92.6	52.4	78.7
	4	119	178	100	151	86.3	130	72.1	108	59.3	89.1	50.4	75.8
	6	112	168	94.4	142	82.2	123	68.6	103	56.6	85.1	48.1	72.4
	8	102	154	86.6	130	76.6	115	64.1	96.3	53.0	79.6	45.1	67.8
	10	91.6	138	77.6	117	70.0	105	58.6	88.1	48.7	73.2	41.5	62.3
	12	79.9	120	67.8	102	62.7	94.3	52.6	79.1	43.9	65.9	37.4	56.2
	14	67.2	101	57.7	86.8	54.8	82.3	46.2	69.5	38.7	58.2	33.1	49.8
	16	54.6	82.1	47.4	71.3	46.0	69.1	39.6	59.5	33.5	50.4	28.7	43.2
	18	43.4	65.2	37.7	56.6	37.7	56.6	32.4	48.8	28.0	42.1	24.4	36.7
	20	35.2	52.9	30.5	45.9	30.5	45.9	26.3	39.5	22.7	34.1	20.0	30.1
	22	29.1	43.7	25.2	37.9	25.2	37.9	21.7	32.6	18.8	28.2	16.5	24.9
	24	24.4	36.7	21.2	31.9	21.2	31.9	18.3	27.4	15.8	23.7	13.9	20.9
	26	20.8	31.3	18.1	27.2	18.1	27.1	15.6	23.4	13.4	20.2	11.8	17.8
	28	17.9	27.0	15.6	23.4	15.6	23.4	13.4	20.2	11.6	17.4	10.2	15.3
	30					13.6	20.4	11.7	17.6	10.1	15.2	8.89	13.4
	32									8.87	13.3	7.82	11.7
Y-Y Axis	0	125	187	105	158	89.8	135	75.0	113	61.6	92.6	52.4	78.7
	4	99.7	150	79.0	119	64.4	96.7	49.1	73.9	33.5	50.4	24.6	36.9
	6	96.5	145	76.9	116	50.9	76.6	39.5	59.3	26.2	39.4	19.5	29.3
	8	89.2	134	72.1	108	34.2	51.5	26.4	39.8	18.2	27.3	14.2	21.3
	10	78.2	118	64.0	96.2	22.8	34.3	17.9	27.0	12.7	19.2	10.2	15.3
	12	64.7	97.3	54.1	81.3	16.2	24.3	12.8	19.3	9.28	14.0	7.54	11.3
	14	51.3	77.1	43.0	64.6	12.0	18.0						
	16	39.8	59.9	33.6	50.5								
	18	31.8	47.8	26.9	40.5								
	20	25.9	38.9	22.0	33.1								
	22	21.5	32.3	18.3	27.5								
	24	18.1	27.2	15.4	23.2								
Properties													
A_g, in.^{2}		4.40		3.82		3.24		2.79		2.36		2.08	
r_x, in.		1.75		1.75		1.90		1.90		1.92		1.92	
r_y, in.		1.52		1.51		0.847		0.821		0.773		0.753	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 50 ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r equal to or greater than 200.



WT5

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes

F_y = 50 ksi

Shape		WT5×									
lb/ft		22.5		19.5		16.5		15		13[c]	
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	199	298	172	258	145	218	132	199	112	168
	4	178	267	154	231	131	196	122	184	104	156
	6	155	233	134	202	114	172	111	166	94.9	143
	8	128	192	111	166	95.0	143	96.0	144	82.4	124
	10	100	150	86.5	130	74.8	112	80.2	121	68.7	103
	12	73.9	111	63.9	96.0	55.8	83.9	64.3	96.7	54.9	82.5
	14	54.3	81.6	46.9	70.5	41.0	61.6	49.5	74.4	42.1	63.2
	16	41.6	62.5	35.9	54.0	31.4	47.2	37.9	57.0	32.2	48.4
	18	32.8	49.4	28.4	42.7	24.8	37.3	29.9	45.0	25.5	38.3
	20	26.6	40.0	23.0	34.6	20.1	30.2	24.3	36.4	20.6	31.0
	22							20.0	30.1	17.0	25.6
	24							16.8	25.3	14.3	21.5
Y-Y Axis	0	199	298	172	258	145	218	132	199	112	168
	4	179	270	150	226	121	182	114	171	94.5	142
	6	176	265	148	223	120	180	104	157	88.1	132
	8	166	250	141	212	116	174	90.0	135	76.3	115
	10	152	228	129	194	107	160	73.8	111	62.5	93.9
	12	135	203	115	173	95.0	143	57.7	86.8	48.8	73.3
	14	118	178	100	151	82.4	124	43.4	65.2	36.6	55.0
	16	101	152	85.4	128	69.8	105	33.4	50.1	28.2	42.4
	18	84.8	127	71.2	107	57.7	86.8	26.4	39.7	22.4	33.6
	20	69.5	104	58.1	87.4	47.0	70.6	21.5	32.3	18.2	27.3
	22	57.5	86.4	48.1	72.3	38.9	58.5	17.8	26.7	15.1	22.6
	24	48.3	72.7	40.5	60.8	32.8	49.3				
	26	41.2	61.9	34.5	51.9	28.0	42.0				
	28	35.6	53.4	29.8	44.8	24.1	36.3				
	30	31.0	46.6	26.0	39.0	21.1	31.6				
	32	27.2	40.9	22.8	34.3	18.5	27.8				

Properties

	22.5	19.5	16.5	15	13
A_g, in.^{2}	6.63	5.73	4.85	4.42	3.81
r_x, in.	1.24	1.24	1.26	1.45	1.44
r_y, in.	2.01	1.98	1.94	1.37	1.36

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 50 ksi

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes

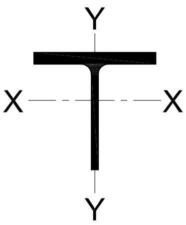
WT5

Shape		WT5×									
lb/ft		11[c]		9.5[c]		8.5[c]		7.5[c]		6[c]	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to indicated axis											
X-X Axis	0	93.8	141	81.5	123	71.8	108	62.7	94.3	47.3	71.1
	4	87.4	131	76.6	115	67.6	102	59.2	89.0	44.7	67.2
	6	80.0	120	70.9	107	62.8	94.4	55.1	82.7	41.6	62.5
	8	70.7	106	63.3	95.2	56.5	84.9	49.7	74.7	37.6	56.6
	10	59.2	89.0	54.0	81.1	48.6	73.0	43.2	64.9	33.1	49.7
	12	47.6	71.6	44.4	66.7	40.1	60.3	35.8	53.8	28.2	42.4
	14	36.8	55.3	35.2	53.0	32.1	48.2	28.6	43.1	22.9	34.5
	16	28.2	42.3	27.2	40.8	24.8	37.3	22.2	33.4	17.8	26.7
	18	22.2	33.4	21.5	32.3	19.6	29.5	17.5	26.4	14.1	21.1
	20	18.0	27.1	17.4	26.1	15.9	23.9	14.2	21.4	11.4	17.1
	22	14.9	22.4	14.4	21.6	13.1	19.7	11.7	17.7	9.41	14.1
	24	12.5	18.8	12.1	18.2	11.0	16.6	9.87	14.8	7.91	11.9
	26					9.39	14.1	8.41	12.6	6.74	10.1
Y-Y Axis	0	93.8	141	81.5	123	71.8	108	62.7	94.3	47.3	71.1
	4	74.3	112	61.3	92.1	50.7	76.1	40.1	60.3	26.4	39.6
	6	70.5	106	47.7	71.7	39.3	59.0	31.0	46.5	20.8	31.2
	8	61.7	92.7	32.7	49.1	26.3	39.6	20.6	30.9	14.3	21.5
	10	50.5	75.9	21.5	32.3	17.5	26.3	13.9	20.9	9.99	15.0
	12	39.1	58.8	15.1	22.7	12.4	18.6	9.93	14.9	7.25	10.9
	14	29.3	44.1	11.2	16.8	9.21	13.8				
	16	22.7	34.1								
	18	18.0	27.1								
	20	14.7	22.1								
	22	12.2	18.3								
Properties											
A_g, in.^{2}		3.24		2.81		2.50		2.21		1.77	
r_x, in.		1.46		1.54		1.56		1.57		1.57	
r_y, in.		1.33		0.874		0.844		0.810		0.785	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r equal to or greater than 200.



WT4

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes

F_y = 50 ksi

Shape		WT4×									
lb/ft		33.5		29		24		20		17.5	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	295	443	256	384	211	317	176	264	154	231
	4	253	380	218	328	177	267	148	222	129	193
	6	209	314	179	269	143	215	119	179	103	154
	8	160	240	135	204	106	159	88.1	132	75.0	113
	10	113	170	94.6	142	71.5	108	59.8	89.9	50.3	75.6
	12	78.6	118	65.7	98.7	49.7	74.7	41.5	62.4	34.9	52.5
	14	57.8	86.8	48.2	72.5	36.5	54.9	30.5	45.9	25.6	38.6
	16	44.2	66.5	36.9	55.5	27.9	42.0	23.4	35.1	19.6	29.5
Y-Y Axis	0	295	443	256	384	211	317	176	264	154	231
	4	281	422	241	362	196	294	158	237	135	202
	6	270	406	233	351	192	288	156	234	134	201
	8	253	380	219	329	180	270	148	222	129	193
	10	233	350	201	302	165	248	136	204	118	177
	12	210	315	181	272	148	223	121	182	106	159
	14	186	279	160	240	131	196	106	160	92.7	139
	16	161	243	138	208	113	170	91.5	138	79.5	120
	18	138	207	118	177	95.7	144	77.1	116	66.9	100
	20	115	173	98.0	147	79.4	119	63.5	95.4	55.0	82.6
	22	95.2	143	81.1	122	65.6	98.6	52.5	78.9	45.5	68.4
	24	80.0	120	68.1	102	55.2	82.9	44.1	66.3	38.2	57.5
	26	68.2	103	58.1	87.3	47.0	70.7	37.6	56.5	32.6	49.0
	28	58.8	88.4	50.1	75.3	40.6	61.0	32.5	48.8	28.1	42.3
	30	51.3	77.0	43.6	65.6	35.3	53.1	28.3	42.5	24.5	36.8
	32	45.0	67.7	38.4	57.6	31.1	46.7	24.9	37.4	21.5	32.4

Properties

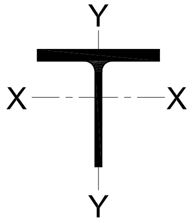
Property	33.5	29	24	20	17.5
A_g, in.^{2}	9.84	8.54	7.05	5.87	5.14
r_x, in.	1.05	1.03	0.986	0.988	0.968
r_y, in.	2.12	2.10	2.08	2.04	2.03

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 50 ksi

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes

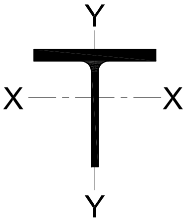


WT4

Shape		WT4×							
lb/ft		15.5		14		12		10.5	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	137	205	123	185	106	159	92.2	139
	4	114	171	105	157	89.5	135	80.6	121
	6	91.2	137	85.1	128	72.5	109	68.2	102
	8	66.6	100	63.7	95.8	54.0	81.1	53.9	81.0
	10	44.7	67.2	43.9	65.9	36.9	55.4	39.8	59.9
	12	31.0	46.6	30.5	45.8	25.6	38.5	28.0	42.1
	14	22.8	34.3	22.4	33.6	18.8	28.3	20.6	30.9
	16	17.5	26.2	17.1	25.8	14.4	21.7	15.8	23.7
	18							12.4	18.7
Y-Y Axis	0	137	205	123	185	106	159	92.2	139
	4	115	173	109	164	90.9	137	78.4	118
	6	115	173	105	157	88.2	133	70.6	106
	8	112	168	94.3	142	80.2	121	59.0	88.7
	10	104	156	81.8	123	69.6	105	46.6	70.0
	12	93.1	140	68.6	103	58.3	87.7	34.8	52.3
	14	81.6	123	55.7	83.7	47.3	71.0	25.7	38.6
	16	69.9	105	43.7	65.7	37.0	55.7	19.7	29.7
	18	58.7	88.2	34.6	52.0	29.3	44.1	15.6	23.5
	20	48.2	72.4	28.1	42.2	23.8	35.8	12.7	19.1
	22	39.9	60.0	23.2	34.9	19.7	29.6		
	24	33.6	50.4	19.5	29.4	16.6	24.9		
	26	28.6	43.0	16.6	25.0	14.1	21.2		
	28	24.7	37.1						
	30	21.5	32.3						
	32	18.9	28.4						
Properties									
A_g, in.^{2}		4.56		4.12		3.54		3.08	
r_x, in.		0.969		1.01		0.999		1.12	
r_y, in.		2.02		1.62		1.61		1.26	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates L_c/r equal to or greater than 200.



WT4

Table 4-7 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded WT-Shapes

$F_y = 50$ ksi

Shape		WT4×							
lb/ft		9		7.5		6.5		5[c]	
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	78.7	118	66.5	99.9	57.5	86.4	41.5	62.4
	4	69.2	104	59.4	89.2	51.4	77.3	37.5	56.4
	6	58.8	88.4	51.5	77.4	44.7	67.3	33.1	49.7
	8	46.9	70.5	42.3	63.5	36.8	55.3	27.6	41.6
	10	35.0	52.6	32.8	49.2	28.7	43.1	21.3	32.1
	12	24.8	37.2	24.0	36.0	21.1	31.6	15.4	23.2
	14	18.2	27.4	17.6	26.4	15.5	23.3	11.3	17.1
	16	13.9	20.9	13.5	20.2	11.8	17.8	8.69	13.1
	18	11.0	16.5	10.6	16.0	9.36	14.1	6.87	10.3
	20			8.62	13.0	7.58	11.4	5.56	8.36
Y-Y Axis	0	78.7	118	66.5	99.9	57.5	86.4	41.5	62.4
	4	63.6	95.6	49.3	74.0	39.7	59.7	27.5	41.4
	6	58.1	87.3	38.3	57.5	30.7	46.1	22.1	33.2
	8	48.5	73.0	26.3	39.5	20.5	30.8	15.0	22.6
	10	38.0	57.1	17.2	25.8	13.5	20.4	10.1	15.2
	12	28.0	42.1	12.1	18.1	9.56	14.4	7.21	10.8
	14	20.8	31.2	8.92	13.4	7.09	10.7	5.37	8.07
	16	16.0	24.0						
	18	12.7	19.0						
	20	10.3	15.5						

Properties

	9	7.5	6.5	5
A_g, in.^{2}	2.63	2.22	1.92	1.48
r_x, in.	1.14	1.22	1.23	1.20
r_y, in.	1.23	0.876	0.843	0.840

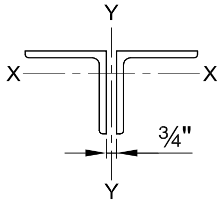
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 36 ksi

Table 4-8
Available Strength in Axial Compression, kips
Double Angles—Equal Legs

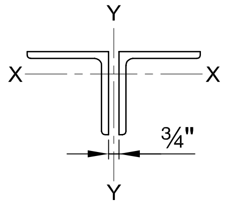


2L12

Shape		2L12×12×								
		1 3/8		1 1/4		1 1/8		1		No. of connectors[a]
lb/ft		210		193		174		156		
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	1340	2020	1220	1840	1110	1670	992	1490	b
	2	1340	2010	1220	1840	1110	1670	989	1490	
	4	1330	2000	1210	1820	1100	1660	983	1480	
	6	1310	1970	1200	1800	1090	1640	972	1460	
	8	1290	1940	1180	1770	1070	1610	957	1440	
	10	1270	1900	1160	1740	1050	1580	938	1410	
	12	1230	1860	1130	1700	1030	1540	916	1380	
	14	1200	1800	1100	1650	997	1500	890	1340	
	16	1160	1740	1060	1590	964	1450	861	1290	
	18	1110	1670	1020	1530	928	1390	829	1250	
	20	1070	1600	976	1470	889	1340	795	1190	
	22	1020	1530	931	1400	848	1280	758	1140	
	24	964	1450	884	1330	806	1210	721	1080	
	26	911	1370	835	1260	762	1150	682	1030	
	28	856	1290	786	1180	717	1080	642	966	
	30	801	1200	736	1110	672	1010	602	905	
	32	746	1120	686	1030	627	942	562	845	
	34	692	1040	637	957	582	875	523	786	
	36	639	960	588	884	538	809	484	727	
	38	587	882	541	813	496	745	446	670	
	40	537	807	495	744	454	683	409	614	
Y-Y Axis	0	1340	2020	1220	1840	1110	1670	992	1490	2
	6	1210	1810	1080	1620	951	1430	814	1220	
	9	1200	1810	1070	1610	949	1430	812	1220	
	12	1200	1800	1070	1610	945	1420	809	1220	
	15	1190	1790	1060	1600	939	1410	804	1210	
	18	1170	1760	1050	1580	930	1400	797	1200	
	21	1150	1720	1030	1550	915	1380	786	1180	
	24	1110	1670	1000	1510	894	1340	760	1140	
	27	1070	1610	968	1460	866	1300	735	1100	
	30	1030	1540	928	1400	832	1250	703	1060	
	33	956	1440	864	1300	777	1170	667	1000	
	36	901	1350	815	1220	733	1100	628	944	
	39	845	1270	764	1150	687	1030	587	882	
	42	788	1180	712	1070	641	963	545	820	
	45	731	1100	660	992	594	893	504	757	
	48	674	1010	608	914	548	824	462	694	
	51	619	930	558	839	503	756	421	633	
	54	565	849	509	765	459	689	401	603	3
	57	512	770	461	693	415	624	363	546	
	60	463	696	417	627	376	565	329	495	
	63	421	632	379	570	342	514	300	450	
Properties of 2 angles—3/4 in. back to back										
A_g, in.^{2}		62.2		56.8		51.6		46.0		
r_x, in.		3.64		3.66		3.68		3.70		
r_y, in.		5.32		5.29		5.28		5.25		
Properties of single angle										
r_z, in.		2.30		2.31		2.33		2.34		

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.



2L10

Table 4-8 (continued)
Available Strength in Axial Compression, kips
Double Angles—Equal Legs

F_y = 36 ksi

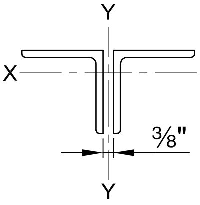
Shape		2L10×10×1 3/8		2L10×10×1 1/4		2L10×10×1 1/8		2L10×10×1		2L10×10×7/8		2L10×10×3/4[c]		No. of connectors[a]
lb/ft		174		160		145		129		114		98.2		
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	1100	1660	1010	1520	918	1380	819	1230	724	1090	611	919	
	2	1100	1650	1010	1510	915	1380	816	1230	722	1090	610	917	
	4	1090	1640	996	1500	906	1360	809	1220	715	1070	606	911	
	6	1070	1610	979	1470	891	1340	795	1200	704	1060	599	900	
	8	1050	1570	957	1440	871	1310	778	1170	688	1030	590	886	
	10	1010	1520	928	1400	846	1270	755	1130	668	1000	577	868	
	12	978	1470	895	1350	815	1230	728	1090	645	970	558	839	
	14	936	1410	857	1290	781	1170	698	1050	619	930	536	805	
	16	890	1340	815	1230	743	1120	665	999	590	886	511	768	
	18	840	1260	771	1160	703	1060	629	945	558	839	484	728	
	20	788	1180	724	1090	660	992	591	889	525	789	456	685	b
	22	734	1100	675	1010	616	926	552	830	491	738	427	641	
	24	679	1020	625	939	571	858	512	770	456	685	397	597	
	26	625	939	575	865	526	790	472	710	421	632	367	551	
	28	570	857	526	790	481	722	432	650	386	579	337	506	
	30	517	777	477	718	437	656	393	591	351	528	307	462	
	32	466	700	431	647	394	593	356	534	318	478	279	419	
	34	416	625	385	579	353	531	319	480	286	430	251	377	
	36	371	558	344	517	315	473	285	428	255	383	224	337	
	38	333	501	309	464	283	425	256	384	229	344	201	303	
	40	301	452	278	419	255	383	231	347	207	311	182	273	
Y-Y Axis	0	1100	1660	1010	1520	918	1380	819	1230	724	1090	611	919	
	6	1020	1540	921	1380	823	1240	712	1070	605	909	490	736	
	9	1020	1530	917	1380	819	1230	710	1070	603	906	488	734	
	12	1010	1520	909	1370	813	1220	705	1060	599	901	485	730	
	15	990	1490	894	1340	801	1200	696	1050	593	892	481	723	
	18	960	1440	869	1310	781	1170	682	1020	583	876	475	713	
	21	922	1390	835	1260	752	1130	660	992	567	853	459	689	
	24	879	1320	796	1200	718	1080	632	949	545	820	441	662	2
	27	812	1220	736	1110	664	997	585	880	508	763	418	628	
	30	758	1140	687	1030	619	931	547	822	475	714	391	587	
	33	703	1060	636	956	573	862	506	761	440	662	362	544	
	36	647	973	585	879	527	792	465	699	404	608	331	498	
	39	592	889	534	802	480	722	424	638	369	554	301	452	
	42	537	807	483	726	435	653	384	577	333	501	271	407	
	45	483	726	434	653	390	587	345	518	299	449	253	381	
	48	432	649	387	582	347	522	306	461	266	399	225	339	
	51	383	576	344	517	308	463	272	409	236	355	201	302	
	54	342	514	307	462	276	414	244	366	212	318	180	271	3
	57	308	462	276	415	248	372	219	329	190	286	162	244	
	70	278	418	249	375	224	336	198	298	172	259	147	221	
	63	252	379	226	340	203	306	180	271	157	235	134	201	
Properties of 2 angles—3/4 in. back to back														
A_g, in.^{2}		51.2		46.8		42.6		38.0		33.6		29.0		
r_x, in.		3.00		3.02		3.03		3.05		3.07		3.10		
r_y, in.		4.53		4.49		4.46		4.45		4.42		4.41		
Properties of single angle														
r_z, in.		1.91		1.91		1.92		1.92		1.93		1.96		

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.

F_y = 36 ksi

Table 4-8 (continued)
Available Strength in Axial Compression, kips
Double Angles—Equal Legs



2L8

Shape		2L8×8× 1 1/8		1		7/8		3/4		5/8		9/16[c]		No. of connectors[a]
lb/ft		114		102		90.0		77.8		65.4		59.2		
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	724	1090	651	978	573	862	496	745	418	628	355	534	b
	2	721	1080	648	973	571	857	493	741	416	625	354	532	
	4	709	1070	638	959	562	845	486	730	410	616	350	527	
	6	691	1040	622	934	548	824	474	712	400	601	344	518	
	8	666	1000	600	901	529	795	458	688	386	581	336	505	
	10	636	955	573	861	505	760	437	657	370	556	326	490	
	12	600	902	541	813	478	719	414	622	350	526	313	471	
	14	561	843	506	761	448	673	388	583	328	494	297	446	
	16	519	779	469	704	415	624	360	541	305	458	276	415	
	18	475	713	429	646	381	572	330	497	281	422	254	382	
	20	430	646	390	586	346	520	300	451	255	384	231	348	
	22	385	579	350	526	311	468	270	406	230	346	209	314	
	24	342	513	311	467	277	416	241	362	206	309	187	280	
	26	300	451	273	411	244	367	213	320	182	273	165	248	
	28	260	391	237	357	213	320	185	279	159	239	144	217	
	30	226	340	207	311	185	278	161	243	138	208	126	189	
	32	199	299	182	273	163	245	142	213	122	183	111	166	
	34	176	265	161	242	144	217	126	189	108	162	98.0	147	
	36	157	236	144	216	129	193	112	168	96.1	144	87.4	131	
	38	141	212	129	194	115	173	101	151	86.2	130	78.4	118	
	40	127	191	116	175	104	157	90.8	136	77.8	117	70.8	106	
Y-Y Axis	0	724	1090	651	978	573	862	496	745	418	628	355	534	2
	6	673	1010	594	893	510	767	424	637	335	503	288	433	
	9	666	1000	589	885	506	761	421	633	333	500	286	431	
	12	649	975	576	866	497	747	415	624	329	494	283	426	
	15	621	934	553	832	480	721	403	606	322	484	278	418	
	18	587	882	523	786	444	667	376	565	304	457	265	398	
	21	536	805	478	718	410	616	348	523	284	427	249	375	
	24	491	738	438	658	373	561	317	477	260	391	229	345	
	27	444	668	396	595	336	505	285	428	234	352	207	312	
	30	397	597	354	532	298	448	253	380	208	313	184	277	
	33	351	528	312	469	261	392	221	332	182	274	161	242	
	36	307	461	272	409	237	356	200	301	165	247	146	219	3
	39	264	397	234	352	203	306	172	258	142	213	126	189	
	42	228	343	203	304	176	264	149	224	123	185	109	164	
	45	199	299	177	266	154	231	130	195	108	162	95.8	144	
	48	175	263	156	234	135	203	115	172	94.9	143	84.6	127	
	51	155	234	138	207	120	180	102	153	84.3	127	75.2	113	
	54	139	208	123	185	107	161	90.9	137	75.4	113	67.3	101	
	57	125	187	111	166	96.3	145	81.7	123	67.8	102	60.6	91.1	

Properties of 2 angles—3/8 in. back to back

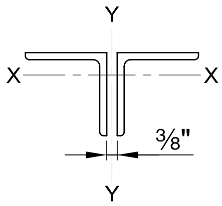
	1 1/8	1	7/8	3/4	5/8	9/16
A_g, in.^{2}	33.6	30.2	26.6	23.0	19.4	17.5
r_x, in.	2.41	2.43	2.45	2.46	2.48	2.49
r_y, in.	3.54	3.52	3.50	3.47	3.45	3.44

Properties of single angle

	1 1/8	1	7/8	3/4	5/8	9/16
r_z, in.	1.56	1.56	1.57	1.57	1.58	1.58

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.



2L8–2L6

Table 4-8 (continued)
Available Strength in Axial Compression, kips
Double Angles—Equal Legs

F_y = 36 ksi

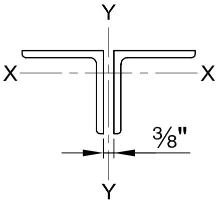
Shape		2L8×8× 1/2[c]		No. of connectors[a]	2L6×6× 1		2L6×6× 7/8		2L6×6× 3/4		2L6×6× 5/8		No. of connectors[a]
lb/ft		52.8			74.8		66.2		57.4		48.4		
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	296	445		474	713	420	632	364	548	308	463	
	2	295	443		470	706	416	626	361	543	306	459	
	4	292	439		457	686	405	609	351	528	297	447	
	6	287	432		436	655	387	581	335	504	284	427	
	8	281	422		408	613	362	545	315	473	267	401	
	10	272	409		374	563	334	501	290	436	246	370	
	12	262	394		337	507	301	453	262	394	223	336	
	14	251	377		298	448	267	401	233	350	199	299	
	16	238	358		259	389	232	349	203	305	174	261	
	18	224	337		220	331	199	299	174	261	149	224	
	20	208	312	b	184	276	167	250	146	219	126	189	b
	22	187	281		152	228	138	207	121	181	104	157	
	24	167	252		128	192	116	174	101	152	87.7	132	
	26	148	223		109	164	98.6	148	86.4	130	74.8	112	
	28	130	195		93.8	141	85.1	128	74.5	112	64.5	96.9	
	30	113	170				74.1	111	64.9	97.6	56.1	84.4	
	32	99.2	149										
	34	87.9	132										
	36	78.4	118										
	38	70.4	106										
	40	63.5	95.4										
Y-Y Axis	0	296	445		474	713	420	632	364	548	308	463	
	6	234	352		446	670	390	586	331	497	270	406	
	9	233	350		431	647	378	569	323	485	265	398	
	12	231	348		405	609	357	536	305	459	253	380	
	15	229	344		373	561	328	494	281	423	234	352	
	18	222	334		328	493	288	434	247	371	206	309	
	21	213	320	2	288	433	253	380	216	325	180	270	
	24	198	298		248	373	217	327	185	278	154	231	
	27	180	271		209	315	183	275	155	233	129	194	3
	30	161	241		173	260	151	226	127	191	106	159	
	33	141	212		143	215	125	187	106	159	87.6	132	
	36	127	192		120	181	105	158	88.9	134	73.8	111	
	39	110	166		103	154	89.5	135	75.8	114	63.0	94.8	
	42	96.2	145		88.5	133	77.3	116	65.5	98.4	54.5	81.8	
	45	84.5	127		77.1	116	67.3	101					
	48	74.8	112	3									
	51	66.6	100										
	54	59.7	89.7										
	57	53.8	80.8										
Properties of 2 angles—3/8 in. back to back													
A_g, in.^{2}		15.7			22.0		19.5		16.9		14.3		
r_x, in.		2.49			1.79		1.81		1.82		1.84		
r_y, in.		3.43			2.72		2.70		2.67		2.65		
Properties of single angle													
r_z, in.		1.59			1.17		1.17		1.17		1.17		

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 36 ksi

Table 4-8 (continued)
Available Strength in Axial Compression, kips
Double Angles—Equal Legs



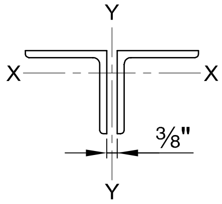
2L6

Shape			2L6×6×									No. of connectors[a]
			9/16		1/2		7/16[c]		3/8[c]		5/16[c]	
lb/ft			43.8		39.2		34.4		29.8		24.8	
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length, L_c (ft), with respect to indicated axis	X-X Axis	0	278	418	248	373	211	318	165	248	122	183	b
		2	276	414	246	369	210	316	164	246	121	182	
		4	268	403	239	360	206	310	161	242	119	179	
		6	257	386	229	344	200	300	156	235	116	174	
		8	241	363	215	324	191	287	150	226	111	167	
		10	223	335	199	299	177	265	142	214	106	159	
		12	202	304	181	272	160	241	133	200	99.6	150	
		14	180	271	161	243	143	215	123	184	92.5	139	
		16	158	237	141	213	125	189	108	163	84.7	127	
		18	136	204	122	183	108	162	93.6	141	76.4	115	
		20	115	172	103	155	91.5	138	79.3	119	67.1	101	
		22	95.2	143	85.8	129	76.1	114	66.1	99.3	55.9	84.1	
		24	80.0	120	72.1	108	63.9	96.1	55.5	83.4	47.0	70.7	
		26	68.2	102	61.4	92.3	54.5	81.9	47.3	71.1	40.1	60.2	
		28	58.8	88.3	53.0	79.6	47.0	70.6	40.8	61.3	34.5	51.9	
		30	51.2	77.0	46.1	69.4	40.9	61.5	35.5	53.4	30.1	45.2	
	Y-Y Axis	0	278	418	248	373	211	318	165	248	122	183	2
		6	237	356	203	305	170	255	130	196	88.9	134	
		9	233	350	200	301	168	252	129	194	88.2	133	
		12	220	331	191	287	162	243	126	190	86.7	130	
		15	202	304	176	265	151	227	121	183	84.2	127	
		18	180	270	158	237	136	205	111	167	80.2	121	
		21	156	234	137	206	119	178	97.9	147	74.3	112	
		24	132	199	116	175	101	152	83.5	126	65.7	98.7	
		27	115	173	101	152	87.6	132	72.4	109	54.9	82.5	
		30	94.2	142	82.8	124	72.0	108	59.9	90.0	48.0	72.2	3
		33	78.2	118	68.8	103	60.0	90.3	50.2	75.4	40.6	61.0	
		36	66.0	99.2	58.1	87.4	50.8	76.3	42.6	64.0	34.6	52.1	
		39	56.4	84.7	49.7	74.7	43.5	65.4	36.5	54.9	29.9	44.9	
		42	48.7	73.2	43.0	64.6	37.6	56.6	31.7	47.6	26.0	39.0	

Properties of 2 angles—3/8 in. back to back					
A_g, in.^{2}	12.9	11.5	10.2	8.76	7.34
r_x, in.	1.85	1.86	1.86	1.87	1.88
r_y, in.	2.64	2.63	2.62	2.60	2.59
Properties of single angle					
r_z, in.	1.18	1.18	1.18	1.19	1.19

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.



2L5

Table 4-8 (continued)
Available Strength in Axial Compression, kips
Double Angles—Equal Legs

F_y = 36 ksi

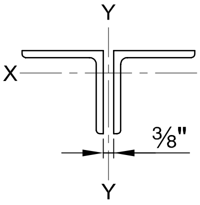
Shape		2L5×5×7/8		2L5×5×3/4		2L5×5×5/8		2L5×5×1/2		2L5×5×7/16		2L5×5×3/8[c]		2L5×5×5/16[c]		No. of connectors[a]
lb/ft		54.4		47.2		40.0		32.4		28.6		24.6		20.6		
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	345	518	302	454	254	382	207	310	182	273	154	231	116	174	
	2	340	511	298	448	251	377	204	306	180	270	153	229	115	172	
	4	327	491	286	430	241	363	196	295	173	260	148	223	112	168	
	6	305	458	267	402	226	340	184	276	162	244	140	211	107	161	
	8	277	417	243	366	206	310	168	252	148	223	129	193	101	152	
	10	245	368	215	324	183	275	149	225	132	199	115	173	93.3	140	
	12	211	317	186	279	159	238	130	195	115	173	99.9	150	84.5	127	b
	14	177	265	156	234	134	201	109	165	97.2	146	84.8	127	71.9	108	
	16	144	216	127	191	110	165	90.1	135	80.3	121	70.2	105	59.6	89.6	
	18	114	172	101	153	87.8	132	72.2	109	64.5	96.9	56.5	84.9	48.1	72.4	
	20	92.7	139	82.2	124	71.1	107	58.5	88.0	52.2	78.5	45.8	68.8	39.0	58.6	
	22	76.6	115	67.9	102	58.8	88.4	48.4	72.7	43.2	64.9	37.8	56.8	32.2	48.4	
	24	64.4	96.7	57.1	85.8	49.4	74.3	40.6	61.1	36.3	54.5	31.8	47.8	27.1	40.7	
	26													23.1	34.7	
Y-Y Axis	0	345	518	302	454	254	382	207	310	182	273	154	231	116	174	
	2	329	495	283	426	233	350	181	271	153	230	125	187	92.0	138	
	4	328	493	282	424	232	349	180	270	152	229	124	187	91.7	138	
	6	322	485	279	419	230	345	178	268	151	227	123	185	91.3	137	
	8	311	468	270	406	224	336	175	263	149	224	122	183	90.6	136	
	10	296	445	257	386	214	322	169	254	142	214	118	177	88.9	134	
	12	278	418	241	363	201	302	159	240	134	201	112	168	86.4	130	2
	14	252	379	218	328	182	273	145	217	123	185	103	155	81.9	123	
	16	230	345	198	298	165	248	131	197	111	167	93.9	141	75.2	113	
	18	207	311	178	268	148	223	117	176	98.9	149	83.7	126	67.5	101	
	20	184	276	158	237	131	197	104	156	86.6	130	73.3	110	59.4	89.3	
	22	161	242	138	207	114	172	90.2	136	74.7	112	63.3	95.1	51.4	77.2	
	24	140	210	119	179	98.4	148	77.3	116	67.0	101	56.7	85.2	46.1	69.3	
	26	119	179	102	153	84.1	126	66.1	99.4	57.4	86.2	48.7	73.2	39.8	59.8	
	28	103	155	87.8	132	72.6	109	57.2	85.9	49.7	74.7	42.2	63.5	34.7	52.1	
	30	89.8	135	76.6	115	63.3	95.2	49.9	75.0	43.4	65.3	37.0	55.6	30.4	45.7	
	32	79.0	119	67.4	101	55.7	83.8	44.0	66.1	38.3	57.5	32.6	49.0	26.9	40.4	3
	34	70.0	105	59.7	89.8	49.4	74.3	39.0	58.6	34.0	51.1	29.0	43.6	23.9	36.0	
	36	62.5	93.9	53.3	80.1	44.1	66.3	34.8	52.4	30.4	45.6	25.9	39.0	21.4	32.2	
	38	56.1	84.3													
Properties of 2 angles—3/8 in. back to back																
A_g, in.^{2}		16.0		14.0		11.8		9.58		8.44		7.30		6.14		
r_x, in.		1.49		1.50		1.52		1.53		1.54		1.55		1.56		
r_y, in.		2.30		2.27		2.25		2.22		2.21		2.20		2.19		
Properties of single angle																
r_z, in.		0.971		0.972		0.975		0.980		0.983		0.986		0.990		

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 36 ksi

Table 4-8 (continued)
Available Strength in Axial Compression, kips
Double Angles—Equal Legs



2L4

Shape		2L4×4× 3/4		5/8		1/2		7/16		3/8		5/16		1/4[c]		No. of connectors[a]
lb/ft		37.0		31.4		25.6		22.6		19.6		16.4		13.2		
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	235	353	199	299	162	243	142	214	123	185	103	155	72.6	109	
	2	230	346	195	293	158	238	139	210	121	182	101	152	71.6	108	
	4	215	324	183	275	149	224	131	197	114	171	95.6	144	68.9	104	
	6	193	290	164	247	134	202	118	178	103	155	86.6	130	64.5	96.9	
	8	166	249	142	213	116	174	103	154	89.5	134	75.5	113	58.7	88.2	
	10	136	205	117	176	96.3	145	85.5	128	74.7	112	63.2	95.0	51.2	77.0	b
	12	107	161	93.1	140	76.7	115	68.3	103	59.9	90.1	50.9	76.5	41.4	62.2	
	14	80.8	121	70.7	106	58.5	87.9	52.3	78.6	46.1	69.3	39.3	59.1	32.1	48.3	
	16	61.9	93.0	54.1	81.4	44.8	67.3	40.1	60.2	35.3	53.0	30.1	45.2	24.6	37.0	
	18	48.9	73.5	42.8	64.3	35.4	53.2	31.6	47.6	27.9	41.9	23.8	35.7	19.4	29.2	
	20			34.6	52.1	28.7	43.1	25.6	38.5	22.6	33.9	19.3	28.9	15.7	23.7	
Y-Y Axis	0	235	353	199	299	162	243	142	214	123	185	103	155	72.6	109	
	2	225	338	187	281	148	222	127	190	105	159	83.1	125	57.7	86.7	
	4	223	335	186	279	147	221	126	189	105	158	82.6	124	57.4	86.3	
	6	215	324	180	271	144	216	124	186	103	155	81.6	123	57.0	85.7	
	8	203	306	170	256	137	205	118	178	99.7	150	79.4	119	56.2	84.4	
	10	188	283	158	237	126	190	110	165	93.2	140	75.2	113	54.7	82.2	
	12	167	250	139	209	112	168	96.8	146	82.6	124	67.3	101	50.7	76.2	
	14	148	222	123	184	98.2	148	85.1	128	72.6	109	59.5	89.4	45.4	68.2	
	16	128	193	106	160	84.8	127	73.3	110	62.5	94.0	51.3	77.0	39.4	59.2	3
	18	109	164	90.1	135	71.8	108	61.8	92.9	52.7	79.2	43.2	64.9	33.3	50.1	
	20	91.5	137	74.9	113	59.4	89.3	51.0	76.7	43.4	65.3	35.6	53.5	27.6	41.5	
	22	75.7	114	62.0	93.2	49.2	74.0	42.3	63.6	36.1	54.2	29.7	44.6	23.1	34.8	
	24	63.6	95.7	52.2	78.4	41.5	62.3	35.7	53.6	30.4	45.8	25.1	37.7	19.6	29.5	
	26	54.3	81.6	44.5	66.9	35.4	53.2	30.4	45.8	26.0	39.1	21.5	32.3	16.9	25.3	
	28	46.8	70.4	38.4	57.7	30.5	45.9	26.3	39.5	22.5	33.8	18.6	27.9	14.6	22.0	
	30	40.8	61.3	33.5	50.3	26.6	40.0	22.9	34.5	19.6	29.5					

Properties of 2 angles—3/8 in. back to back

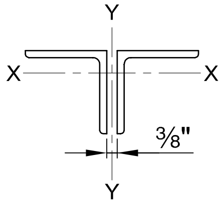
	3/4	5/8	1/2	7/16	3/8	5/16	1/4
A_g, in.^{2}	10.9	9.22	7.50	6.60	5.72	4.80	3.86
r_x, in.	1.18	1.20	1.21	1.22	1.23	1.24	1.25
r_y, in.	1.88	1.85	1.83	1.81	1.80	1.79	1.78

Properties of single angle

	3/4	5/8	1/2	7/16	3/8	5/16	1/4
r_z, in.	0.774	0.774	0.776	0.777	0.779	0.781	0.783

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.



2L3 1/2

Table 4-8 (continued)
Available Strength in Axial Compression, kips
Double Angles—Equal Legs

F_y = 36 ksi

Shape		2L3 1/2×3 1/2×										No. of connectors[a]
		1/2		7/16		3/8		5/16		1/4[c]		
lb/ft		22.2		19.6		17.0		14.4		11.6		
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	140	211	125	187	108	162	90.5	136	69.6	105	b
	1	139	209	124	186	107	161	90.0	135	69.3	104	
	2	136	205	121	182	105	158	88.2	133	68.4	103	
	3	132	198	117	176	102	153	85.4	128	66.8	100	
	4	126	189	112	168	96.9	146	81.6	123	64.7	97.3	
	5	118	177	105	158	91.3	137	77.0	116	62.1	93.3	
	6	109	164	97.7	147	84.9	128	71.7	108	58.3	87.6	
	7	100	150	89.5	135	77.9	117	65.8	99.0	53.6	80.6	
	8	90.2	136	80.9	122	70.6	106	59.7	89.8	48.7	73.2	
	9	80.3	121	72.1	108	63.0	94.8	53.5	80.4	43.7	65.7	
	10	70.4	106	63.5	95.4	55.6	83.6	47.3	71.0	38.7	58.2	
	11	61.0	91.6	55.1	82.8	48.4	72.7	41.2	62.0	33.9	50.9	
	12	51.9	78.1	47.1	70.8	41.5	62.4	35.5	53.4	29.2	44.0	
	13	44.3	66.5	40.1	60.3	35.4	53.1	30.3	45.5	24.9	37.5	
	14	38.2	57.4	34.6	52.0	30.5	45.8	26.1	39.2	21.5	32.3	
	15	33.2	50.0	30.1	45.3	26.6	39.9	22.7	34.2	18.7	28.2	
	16	29.2	43.9	26.5	39.8	23.3	35.1	20.0	30.0	16.5	24.8	
	17	25.9	38.9	23.5	35.3	20.7	31.1	17.7	26.6	14.6	21.9	
	18							15.8	23.7	13.0	19.6	
Y-Y Axis	0	140	211	125	187	108	162	90.5	136	69.6	105	3
	2	131	196	114	171	95.5	143	76.3	115	56.6	85.1	
	4	129	194	113	169	94.7	142	75.7	114	56.2	84.5	
	6	124	186	109	164	92.1	138	74.2	111	55.3	83.1	
	8	115	173	101	152	86.3	130	70.4	106	53.2	79.9	
	10	101	152	89.1	134	76.2	114	62.7	94.2	48.3	72.6	
	12	88.5	133	77.6	117	66.3	99.7	54.7	82.2	42.5	63.8	
	14	75.3	113	65.8	98.9	56.2	84.4	46.3	69.6	36.0	54.2	
	16	62.5	94.0	54.4	81.8	46.4	69.7	38.2	57.4	29.7	44.6	
	18	50.5	75.9	43.8	65.8	37.3	56.0	30.7	46.1	23.9	35.9	
	20	41.0	61.6	35.6	53.5	30.3	45.5	25.0	37.6	19.5	29.3	
	22	33.9	51.0	29.5	44.3	25.1	37.7	20.7	31.2	16.2	24.4	
	24	28.5	42.9	24.8	37.2	21.1	31.8	17.5	26.3	13.7	20.6	
	26	24.3	36.6	21.1	31.8	18.0	27.1	14.9	22.4	11.7	17.6	

Properties of 2 angles—3/8 in. back to back

	1/2	7/16	3/8	5/16	1/4
A_g, in.^{2}	6.50	5.78	5.00	4.20	3.40
r_x, in.	1.05	1.06	1.07	1.08	1.09
r_y, in.	1.63	1.61	1.60	1.59	1.57

Properties of single angle

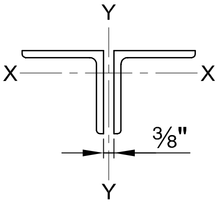
	1/2	7/16	3/8	5/16	1/4
r_z, in.	0.679	0.681	0.683	0.685	0.688

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 36 ksi

Table 4-8 (continued)
Available Strength in Axial Compression, kips
Double Angles—Equal Legs



2L3

Shape		2L3×3×												No. of connectors[a]
		1/2		7/16		3/8		5/16		1/4		3/16[c]		
lb/ft		18.8		16.6		14.4		12.2		9.80		7.42		
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	119	179	105	157	91.0	137	76.7	115	62.1	93.3	41.0	61.6	b
	1	118	177	104	156	90.1	135	76.1	114	61.5	92.5	40.7	61.2	
	2	115	172	101	152	87.7	132	74.0	111	59.9	90.1	40.0	60.2	
	3	109	164	96.4	145	83.8	126	70.8	106	57.3	86.2	38.9	58.5	
	4	102	154	90.3	136	78.6	118	66.5	99.9	53.9	81.0	37.3	56.1	
	5	93.9	141	83.0	125	72.4	109	61.3	92.1	49.8	74.8	35.4	53.1	
	6	84.6	127	75.0	113	65.4	98.3	55.5	83.4	45.2	67.9	33.1	49.7	
	7	74.8	112	66.4	99.8	58.1	87.3	49.4	74.2	40.3	60.5	30.5	45.8	
	8	64.9	97.6	57.8	86.9	50.6	76.1	43.2	64.9	35.3	53.0	26.9	40.5	
	9	55.3	83.1	49.3	74.2	43.3	65.1	37.0	55.7	30.3	45.6	23.2	34.9	
	10	46.2	69.4	41.3	62.1	36.4	54.7	31.2	46.9	25.6	38.5	19.7	29.6	
	11	38.1	57.3	34.2	51.4	30.1	45.3	25.9	38.9	21.3	32.0	16.4	24.6	
	12	32.1	48.2	28.7	43.2	25.3	38.1	21.7	32.7	17.9	26.9	13.8	20.7	
	13	27.3	41.0	24.5	36.8	21.6	32.4	18.5	27.9	15.3	22.9	11.7	17.6	
	14	23.5	35.4	21.1	31.7	18.6	28.0	16.0	24.0	13.2	19.8	10.1	15.2	
	15			18.4	27.6	16.2	24.4	13.9	20.9	11.5	17.2	8.80	13.2	
Y-Y Axis	0	119	179	105	157	91.0	137	76.7	115	62.1	93.3	41.0	61.6	3
	2	112	169	97.5	147	82.8	124	67.1	101	50.6	76.0	32.2	48.3	
	4	110	165	95.8	144	81.6	123	66.3	99.6	50.0	75.2	31.9	48.0	
	6	103	155	90.0	135	77.3	116	63.5	95.4	48.5	72.9	31.5	47.3	
	8	90.9	137	79.5	120	68.4	103	56.6	85.0	44.2	66.4	30.1	45.2	
	10	78.6	118	68.7	103	59.0	88.7	48.8	73.3	38.4	57.7	27.0	40.5	
	12	65.8	98.9	57.3	86.2	49.2	73.9	40.5	60.9	31.9	48.0	22.8	34.2	
	14	53.3	80.1	46.3	69.6	39.6	59.6	32.5	48.8	25.6	38.4	18.4	27.6	
	16	41.7	62.7	36.2	54.4	30.9	46.4	25.2	37.9	19.9	29.9	14.4	21.7	
	18	33.0	49.6	28.6	43.1	24.5	36.8	20.0	30.1	15.9	23.8	11.6	17.4	
	20	26.8	40.2	23.2	34.9	19.9	29.9	16.3	24.5	12.9	19.4	9.48	14.3	
	22	22.1	33.3	19.2	28.9	16.5	24.7	13.5	20.3	10.7	16.1	7.89	11.9	

Properties of 2 angles—3/8 in. back to back

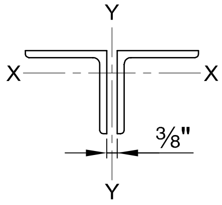
	1/2	7/16	3/8	5/16	1/4	3/16
A_g, in.^{2}	5.52	4.86	4.22	3.56	2.88	2.18
r_x, in.	0.895	0.903	0.910	0.918	0.926	0.933
r_y, in.	1.43	1.42	1.41	1.39	1.38	1.37

Properties of single angle

	1/2	7/16	3/8	5/16	1/4	3/16
r_z, in.	0.580	0.580	0.581	0.583	0.585	0.586

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.



$2L2^{1}/_{2}$

Table 4-8 (continued)
Available Strength in Axial Compression, kips
Double Angles—Equal Legs

F_y = 36 ksi

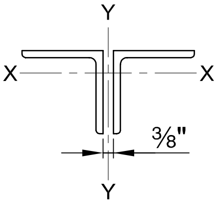
Shape			2L2½×2½×										No. of connectors[a]
			½		3/8		5/16		¼		3/16[c]		
lb/ft			15.4		11.8		10.0		8.20		6.14		
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length, L_c (ft), with respect to indicated axis	X-X Axis	0	97.4	146	74.6	112	62.9	94.6	51.3	77.1	37.9	57.0	b
		1	96.1	144	73.6	111	62.1	93.4	50.6	76.1	37.6	56.5	
		2	92.1	138	70.7	106	59.7	89.7	48.7	73.2	36.6	55.0	
		3	85.9	129	66.0	99.3	55.9	84.0	45.6	68.6	34.6	52.0	
		4	77.8	117	60.1	90.3	50.9	76.5	41.7	62.6	31.6	47.6	
		5	68.6	103	53.2	80.0	45.2	67.9	37.1	55.7	28.2	42.4	
		6	58.8	88.4	45.9	68.9	39.0	58.7	32.1	48.3	24.5	36.8	
		7	49.0	73.6	38.5	57.8	32.9	49.4	27.2	40.8	20.8	31.2	
		8	39.7	59.7	31.4	47.2	26.9	40.5	22.3	33.6	17.2	25.8	
		9	31.5	47.3	25.0	37.6	21.5	32.3	17.9	26.9	13.8	20.7	
		10	25.5	38.3	20.3	30.5	17.4	26.2	14.5	21.8	11.2	16.8	
		11	21.1	31.7	16.7	25.2	14.4	21.6	12.0	18.0	9.23	13.9	
		12	17.7	26.6	14.1	21.1	12.1	18.2	10.1	15.1	7.76	11.7	
	Y-Y Axis	0	97.4	146	74.6	112	62.9	94.6	51.3	77.1	37.9	57.0	3
		1	93.7	141	69.8	105	57.3	86.1	44.5	66.9	30.3	45.6	
		2	93.4	140	69.6	105	57.1	85.9	44.4	66.7	30.2	45.5	
		3	92.0	138	68.9	104	56.7	85.2	44.1	66.3	30.1	45.2	
		4	89.2	134	67.3	101	55.6	83.6	43.5	65.4	29.8	44.8	
		5	85.4	128	64.6	97.0	53.6	80.6	42.3	63.6	29.3	44.0	
		6	80.9	122	61.1	91.9	50.8	76.4	40.4	60.8	28.4	42.7	
		7	73.9	111	55.8	83.9	46.4	69.7	37.1	55.7	26.6	39.9	
		8	68.1	102	51.3	77.1	42.5	63.9	34.0	51.1	24.6	37.0	
		9	62.0	93.1	46.6	70.0	38.5	57.9	30.8	46.3	22.4	33.6	
		10	55.8	83.9	41.8	62.8	34.5	51.8	27.6	41.4	20.1	30.2	
		11	49.7	74.7	37.1	55.8	30.5	45.8	24.3	36.6	17.7	26.7	
		12	43.8	65.8	32.6	48.9	26.7	40.1	21.2	31.9	15.5	23.3	
		13	38.1	57.3	28.2	42.4	23.0	34.5	18.3	27.5	13.3	20.0	
		14	32.9	49.4	24.3	36.6	19.9	29.8	15.8	23.8	11.6	17.4	
		15	28.7	43.1	21.2	31.9	17.3	26.0	13.8	20.8	10.1	15.2	
		16	25.2	37.9	18.7	28.1	15.2	22.9	12.2	18.3	8.95	13.5	
		17	22.3	33.6	16.6	24.9	13.5	20.3	10.8	16.2	7.96	12.0	
		18	19.9	30.0	14.8	22.2	12.1	18.1	9.65	14.5	7.12	10.7	
		19	17.9	26.9	13.3	19.9	10.8	16.3	8.67	13.0	6.40	9.62	
		20	16.2	24.3	12.0	18.0							
Properties of 2 angles—3/8 in. back to back													
A_g, in.^{2}			4.52		3.46		2.92		2.38		1.80		
r_x, in.			0.735		0.749		0.756		0.764		0.771		
r_y, in.			1.23		1.21		1.19		1.18		1.17		
Properties of single angle													
r_z, in.			0.481		0.481		0.481		0.482		0.482		

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 36 ksi

Table 4-8 (continued)
Available Strength in Axial Compression, kips
Double Angles—Equal Legs

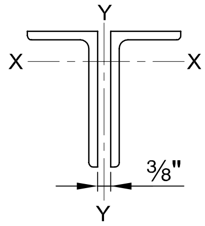


2L2

Shape		2L2×2× 3/8		2L2×2× 5/16		2L2×2× 1/4		2L2×2× 3/16		2L2×2× 1/8[c]		No. of connectors[a]
lb/ft		9.40		7.84		6.38		4.88		3.30		
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	
Effective length, L_c (ft), with respect to indicated axis: X-X Axis	0	59.1	88.8	50.0	75.2	40.7	61.2	31.0	46.7	18.5	27.8	b
	1	57.8	86.9	49.0	73.6	39.9	60.0	30.4	45.7	18.3	27.4	
	2	54.2	81.4	45.9	69.1	37.5	56.4	28.6	43.0	17.5	26.4	
	3	48.6	73.0	41.3	62.1	33.8	50.8	25.9	38.9	16.4	24.6	
	4	41.7	62.7	35.6	53.5	29.3	44.0	22.5	33.7	14.9	22.4	
	5	34.3	51.6	29.4	44.2	24.3	36.5	18.7	28.1	12.9	19.4	
	6	27.0	40.6	23.3	35.0	19.3	29.1	15.0	22.5	10.4	15.6	
	7	20.4	30.6	17.7	26.6	14.7	22.1	11.5	17.3	8.04	12.1	
	8	15.6	23.5	13.5	20.3	11.3	17.0	8.80	13.2	6.16	9.25	
	9	12.3	18.5	10.7	16.1	8.91	13.4	6.95	10.4	4.86	7.31	
	10					7.22	10.9	5.63	8.46	3.94	5.92	
Y-Y Axis	0	59.1	88.8	50.0	75.2	40.7	61.2	31.0	46.7	18.5	27.8	3
	1	56.5	84.9	47.0	70.6	37.1	55.7	26.4	39.6	14.6	21.9	
	2	56.1	84.3	46.7	70.2	36.9	55.4	26.3	39.5	14.6	21.9	
	3	54.6	82.1	45.7	68.7	36.3	54.6	26.0	39.1	14.5	21.7	
	4	52.0	78.2	43.7	65.6	35.0	52.5	25.3	38.1	14.3	21.5	
	5	48.7	73.2	40.9	61.4	32.8	49.3	24.1	36.2	14.0	21.1	
	6	43.6	65.6	36.5	54.9	29.3	44.1	21.7	32.6	13.3	20.0	
	7	39.2	59.0	32.8	49.3	26.2	39.4	19.4	29.2	12.2	18.3	
	8	34.7	52.1	28.9	43.4	23.1	34.7	17.1	25.6	10.8	16.2	
	9	30.2	45.3	25.0	37.6	19.9	29.9	14.7	22.1	9.33	14.0	
	10	25.8	38.8	21.3	32.1	16.9	25.4	12.4	18.7	7.87	11.8	
	11	21.7	32.6	17.8	26.8	14.1	21.2	10.3	15.5	6.62	9.94	
	12	18.2	27.4	15.0	22.6	11.9	17.8	8.72	13.1	5.62	8.45	
	13	15.5	23.3	12.8	19.2	10.1	15.2	7.46	11.2	4.83	7.26	
	14	13.4	20.1	11.0	16.6	8.75	13.1	6.45	9.69	4.19	6.30	
	15	11.7	17.6	9.63	14.5	7.63	11.5	5.63	8.46	3.67	5.51	
	16	10.3	15.4	8.47	12.7	6.71	10.1	4.95	7.44			
Properties of 2 angles—3/8 in. back to back												
A_g, in.^{2}		2.74		2.32		1.89		1.44		0.982		
r_x, in.		0.591		0.598		0.605		0.612		0.620		
r_y, in.		1.01		0.996		0.982		0.967		0.951		
Properties of single angle												
r_z, in.		0.386		0.386		0.387		0.389		0.391		

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.



2L8 LLBB

Table 4-9
Available Strength in Axial Compression, kips
Double Angles—LLBB

F_y = 36 ksi

Shape		2L8×6×														
		1		7/8		3/4		5/8		9/16[c]		1/2[c]		7/16[c]		No. of connectors[a]
lb/ft		88.4		78.2		67.6		57.0		51.4		46.0		40.4		
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	565	849	496	745	431	648	362	544	317	476	272	409	224	337	b
	4	554	832	486	731	423	636	355	534	312	469	268	402	222	333	
	6	540	812	475	713	413	621	347	522	306	460	263	395	218	328	
	8	522	785	459	690	399	600	336	505	298	447	256	384	213	321	
	10	500	751	439	660	383	575	322	484	287	432	247	371	207	312	
	12	474	712	416	626	363	546	306	460	275	414	237	356	199	300	
	14	444	668	391	588	341	513	288	432	261	392	225	338	190	285	
	16	413	621	363	546	318	477	268	403	243	365	212	319	179	270	
	18	380	571	335	503	293	440	247	372	225	338	199	299	168	253	
	20	346	521	305	459	267	402	226	340	206	309	184	276	156	235	
	22	313	470	276	414	242	364	205	308	186	280	167	251	144	217	
	24	279	420	247	371	217	326	184	277	167	252	150	225	132	198	
	26	247	371	218	328	192	289	164	246	149	224	133	200	118	178	
	28	216	325	191	288	169	254	144	217	131	197	118	177	104	157	
	30	188	283	167	251	147	221	126	189	115	172	103	154	91.2	137	
	32	166	249	147	220	129	195	110	166	101	151	90.1	135	80.2	120	
	34	147	220	130	195	115	172	97.9	147	89.2	134	79.9	120	71.0	107	
	36	131	197	116	174	102	154	87.3	131	79.6	120	71.2	107	63.3	95.2	
	38	117	176	104	156	91.8	138	78.3	118	71.4	107	63.9	96.1	56.8	85.4	
	40	106	159	93.8	141	82.9	125	70.7	106	64.5	96.9	57.7	86.7	51.3	77.1	
	42					75.2	113	64.1	96.4	58.5	87.9	52.3	78.6	46.5	69.9	
Y-Y Axis	0	565	849	496	745	431	648	362	544	317	476	272	409	224	337	2
	4	524	787	451	677	379	570	302	454	263	395	216	325	170	256	
	6	517	778	446	670	376	564	300	450	260	391	215	323	169	254	
	8	506	760	436	656	369	554	295	443	256	385	212	319	167	251	
	10	487	732	422	634	357	537	287	431	250	376	208	313	165	247	
	12	464	697	402	604	342	513	276	415	241	363	203	305	161	242	
	14	426	640	369	555	315	473	256	384	225	338	192	289	153	230	
	16	394	592	341	513	291	437	237	356	209	314	180	271	146	219	
	18	360	542	312	469	266	399	216	325	191	288	165	249	136	205	
	20	326	490	282	424	240	360	195	294	173	260	150	225	126	189	
	22	292	439	252	379	214	321	174	262	154	232	134	201	113	170	
	24	258	389	223	335	189	284	154	231	136	205	118	178	99.8	150	
	26	227	340	195	293	164	247	134	201	118	178	103	155	87.3	131	
	28	196	295	169	254	143	214	116	175	103	155	90.0	135	76.8	115	
	30	171	258	148	222	125	188	102	153	90.7	136	79.3	119	68.0	102	
	32	151	227	130	195	110	165	90.1	135	80.3	121	70.4	106	60.5	90.9	
	34	134	201	115	174	97.8	147	80.1	120	71.5	107	62.8	94.4	54.1	81.3	
	36	120	180	103	155	87.5	131	71.7	108	64.1	96.3	56.3	84.7	48.7	73.2	
	38	108	162	92.7	139	78.7	118	64.6	97.1	57.7	86.8	50.8	76.4	44.0	66.1	
	40	97.1	146	83.8	126	71.1	107	58.4	87.8	52.3	78.6	46.1	69.2	39.9	60.0	
	42	88.2	133													

Properties of 2 angles—3/8 in. back to back

	1	7/8	3/4	5/8	9/16	1/2	7/16
A_g, in.^{2}	26.2	23.0	20.0	16.8	15.2	13.6	12.0
r_x, in.	2.49	2.50	2.52	2.54	2.55	2.55	2.56
r_y, in.	2.52	2.50	2.47	2.45	2.44	2.43	2.42

Properties of single angle

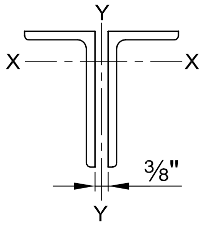
	1	7/8	3/4	5/8	9/16	1/2	7/16
r_z, in.	1.28	1.28	1.29	1.29	1.30	1.30	1.31

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 36 ksi

Table 4-9 (continued)
Available Strength in Axial Compression, kips
Double Angles—LLBB



2L8 LLBB

Shape		2L8×4× 1		7/8		3/4		5/8		9/16[c]		1/2[c]		7/16[c]		No. of connectors[a]
lb/ft		74.8		66.2		57.4		48.4		43.8		39.2		34.4		
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	479	719	423	635	366	551	308	463	269	405	229	344	190	285	
	4	469	706	415	623	360	541	303	455	265	399	225	339	187	281	
	6	458	689	405	609	351	528	296	444	260	391	221	333	184	276	
	8	443	666	392	589	340	511	286	430	254	381	216	325	179	270	
	10	424	638	375	564	326	490	275	413	245	369	209	314	174	261	
	12	402	605	356	535	310	466	261	392	236	354	201	302	167	251	
	14	378	568	335	503	292	438	246	369	224	336	192	288	160	240	
	16	352	529	312	469	272	409	229	345	209	314	181	273	151	227	
	18	324	487	288	433	251	378	212	319	193	290	170	256	142	214	
	20	296	444	263	395	230	346	194	292	177	266	159	238	133	200	
	22	267	402	238	358	208	313	176	265	161	242	144	217	123	185	b
	24	239	360	214	321	187	281	158	238	145	217	130	195	113	170	
	26	212	319	190	285	167	250	141	212	129	194	116	174	102	154	
	28	186	280	167	251	147	221	124	187	114	171	102	154	90.7	136	
	30	162	244	146	219	128	193	109	163	99.6	150	89.6	135	79.4	119	
	32	143	214	128	192	113	169	95.5	144	87.5	132	78.7	118	69.7	105	
	34	126	190	113	170	99.8	150	84.6	127	77.5	117	69.7	105	61.8	92.9	
	36	113	169	101	152	89.0	134	75.5	113	69.2	104	62.2	93.5	55.1	82.8	
	38	101	152	90.7	136	79.9	120	67.7	102	62.1	93.3	55.8	83.9	49.5	74.3	
	40	91.2	137	81.8	123	72.1	108	61.1	91.9	56.0	84.2	50.4	75.7	44.6	67.1	
	42			74.2	112	65.4	98.3	55.5	83.3	50.8	76.4	45.7	68.7	40.5	60.9	
Y-Y Axis	0	479	719	423	635	366	551	308	463	269	405	229	344	190	285	
	4	437	657	379	570	320	480	257	386	225	338	185	278	145	218	
	6	415	624	360	542	305	458	245	369	216	324	179	269	141	212	
	8	384	577	333	500	282	423	227	342	200	301	169	254	134	201	
	10	336	504	290	436	246	369	198	298	175	264	150	226	121	182	
	12	291	438	251	377	212	318	170	256	151	227	130	195	108	162	
	14	246	370	210	316	177	266	142	213	126	189	108	162	90.0	135	2
	16	202	304	172	259	144	217	115	172	101	152	87.0	131	72.9	110	
	18	162	244	138	207	115	173	92.1	138	81.6	123	70.6	106	59.7	89.7	
	20	132	199	112	169	94.2	142	75.5	113	67.1	101	58.3	87.6	49.5	74.4	
	22	110	165	93.1	140	78.4	118	63.0	94.6	56.1	84.3	48.8	73.4	41.7	62.6	
	24	92.3	139	78.5	118	66.2	99.4	53.2	80.0	47.5	71.4	41.4	62.3	35.5	53.3	
	26	78.8	118	67.1	101											

Properties of 2 angles—3/8 in. back to back

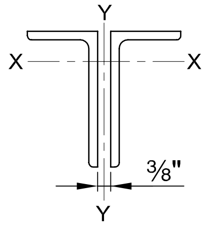
	1	7/8	3/4	5/8	9/16	1/2	7/16
A_g, in.^{2}	22.2	19.6	17.0	14.3	13.0	11.6	10.2
r_x, in.	2.51	2.53	2.55	2.56	2.57	2.58	2.59
r_y, in.	1.60	1.57	1.55	1.52	1.51	1.50	1.49

Properties of single angle

	1	7/8	3/4	5/8	9/16	1/2	7/16
r_z, in.	0.844	0.846	0.850	0.856	0.859	0.863	0.867

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.



2L7 LLBB

Table 4-9 (continued)
Available Strength in Axial Compression, kips
Double Angles—LLBB

F_y = 36 ksi

Shape		2L7×4×3/4		2L7×4×5/8		2L7×4×1/2[c]		2L7×4×7/16[c]		2L7×4×3/8[c]		No. of connectors[a]
lb/ft		52.4		44.2		35.8		31.4		27.2		
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	
Effective length, L_c (ft), with respect to indicated axis: X-X Axis	0	334	502	280	421	219	329	183	275	148	223	b
	4	326	490	273	411	215	323	180	270	146	219	
	6	316	475	265	399	210	315	176	264	142	214	
	8	303	455	254	382	203	304	170	255	138	207	
	10	286	430	241	362	194	291	163	245	132	199	
	12	267	402	225	338	182	274	154	232	126	189	
	14	246	370	208	312	169	254	145	218	118	178	
	16	225	338	190	285	154	232	135	203	110	166	
	18	202	304	171	257	139	209	123	185	102	153	
	20	180	270	152	229	124	187	110	166	92.8	140	
	22	158	237	134	201	110	165	97.3	146	83.9	126	
	24	137	205	116	175	95.5	144	84.9	128	73.9	111	
	26	117	176	99.8	150	82.1	123	73.0	110	63.7	95.7	
	28	101	151	86.1	129	70.8	106	63.0	94.6	54.9	82.5	
	30	87.8	132	75.0	113	61.6	92.7	54.9	82.4	47.8	71.9	
	32	77.2	116	65.9	99.0	54.2	81.4	48.2	72.5	42.0	63.2	
	34	68.4	103	58.4	87.7	48.0	72.1	42.7	64.2	37.2	55.9	
	36	61.0	91.6	52.1	78.3	42.8	64.3	38.1	57.3	33.2	49.9	
Y-Y Axis	0	334	502	280	421	219	329	183	275	148	223	2
	4	300	450	243	365	183	276	148	222	112	169	
	6	286	430	233	350	177	266	144	216	109	164	
	8	266	399	216	325	165	249	137	206	105	157	
	10	233	351	190	285	146	220	123	186	95.8	144	
	12	203	305	165	247	127	191	108	162	86.0	129	
	14	172	258	139	209	107	161	90.7	136	73.9	111	
	16	142	213	114	171	87.4	131	74.0	111	60.5	90.9	
	18	114	172	91.4	137	70.4	106	60.1	90.3	49.6	74.5	
	20	93.0	140	74.7	112	57.8	86.9	49.6	74.5	41.2	62.0	
	22	77.2	116	62.1	93.4	48.3	72.6	41.5	62.4	34.7	52.2	
	24	65.1	97.9	52.5	78.8	40.9	61.5	35.2	53.0	29.6	44.4	
	26	55.6	83.6	44.9	67.4	35.0	52.7					

Properties of 2 angles—3/8 in. back to back

	3/4	5/8	1/2[c]	7/16[c]	3/8[c]
A_g, in.^{2}	15.5	13.0	10.5	9.26	8.00
r_x, in.	2.21	2.23	2.25	2.26	2.27
r_y, in.	1.61	1.58	1.56	1.55	1.54

Properties of single angle

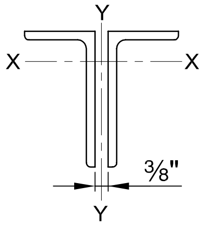
	3/4	5/8	1/2[c]	7/16[c]	3/8[c]
r_z, in.	0.855	0.860	0.866	0.869	0.873

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 36 ksi

Table 4-9 (continued)
Available Strength in Axial Compression, kips
Double Angles—LLBB

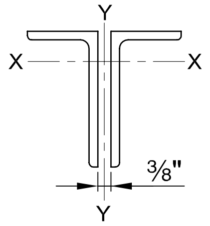


2L6 LLBB

Shape		2L6×4×								No. of connectors[a]
		7/8		3/4		5/8		9/16		
lb/ft		54.4		47.2		40.0		36.2		
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	345	518	300	450	252	379	229	343	b
	4	333	501	290	435	244	366	221	332	
	6	319	479	277	417	234	351	212	318	
	8	300	451	261	393	220	331	200	300	
	10	277	416	242	363	204	307	185	278	
	12	252	378	220	331	186	279	169	254	
	14	224	337	197	296	166	250	151	228	
	16	197	296	173	260	146	220	133	201	
	18	170	255	150	225	127	191	116	174	
	20	144	216	127	191	108	162	98.6	148	
	22	119	179	106	159	90.1	135	82.5	124	
	24	100	151	89.0	134	75.7	114	69.3	104	
	26	85.5	128	75.9	114	64.5	97.0	59.1	88.8	
	28	73.7	111	65.4	98.3	55.6	83.6	50.9	76.6	
	30	64.2	96.5	57.0	85.6	48.5	72.9	44.4	66.7	
Y-Y Axis	0	345	518	300	450	252	379	229	343	2
	4	321	483	275	413	225	338	200	300	
	6	308	462	264	396	217	326	193	290	
	8	287	431	246	369	203	305	181	272	
	10	254	382	217	327	179	270	160	241	
	12	224	337	191	287	157	236	141	211	
	14	193	290	164	246	135	202	120	180	
	16	162	244	137	206	112	169	100	150	
	18	133	200	112	168	91.2	137	81.1	122	
	20	108	163	90.8	137	74.3	112	66.2	99.5	
	22	89.8	135	75.3	113	61.7	92.7	55.0	82.7	
	24	75.6	114	63.4	95.3	52.0	78.2	46.4	69.8	
	26	64.5	96.9	54.2	81.4	44.4	66.8	39.7	59.7	
	28	55.7	83.7	46.8	70.3					
Properties of 2 angles—3/8 in. back to back										
A_g, in.^{2}		16.0		13.9		11.7		10.6		
r_x, in.		1.86		1.88		1.89		1.90		
r_y, in.		1.71		1.68		1.66		1.65		
Properties of single angle										
r_z, in.		0.854		0.856		0.859		0.861		

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
Note: Heavy line indicates L_c/r equal to or greater than 200.



2L6 LLBB

Table 4-9 (continued)
Available Strength in Axial Compression, kips
Double Angles—LLBB

F_y = 36 ksi

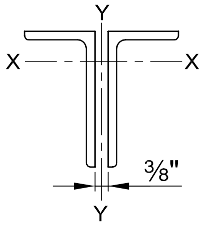
Shape		2L6×4×								No. of connectors[a]
		1/2		7/16[c]		3/8[c]		5/16[c]		
lb/ft		32.4		28.6		24.6		20.6		
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length, L_c (ft), with respect to indicated axis: X-X Axis	0	205	308	176	265	144	216	112	169	b
	4	198	298	171	257	140	210	109	164	
	6	190	286	166	249	135	203	106	159	
	8	179	269	158	237	129	194	101	152	
	10	166	250	147	221	122	183	95.6	144	
	12	152	228	134	201	113	170	89.0	134	
	14	136	205	120	181	104	156	81.7	123	
	16	120	181	106	160	92.4	139	74.1	111	
	18	104	157	92.6	139	80.5	121	66.2	99.5	
	20	89.2	134	79.2	119	69.0	104	58.3	87.6	
	22	74.7	112	66.5	99.9	58.0	87.2	49.2	73.9	
	24	62.8	94.4	55.8	83.9	48.7	73.2	41.3	62.1	
	26	53.5	80.4	47.6	71.5	41.5	62.4	35.2	52.9	
	28	46.1	69.4	41.0	61.7	35.8	53.8	30.4	45.6	
	30	40.2	60.4	35.7	53.7	31.2	46.9	26.5	39.8	
	32			31.4	47.2	27.4	41.2	23.2	34.9	
Y-Y Axis	0	205	308	176	265	144	216	112	169	2
	4	174	261	146	220	115	173	82.5	124	
	6	168	253	142	213	112	169	80.8	121	
	8	158	238	134	202	108	162	78.0	117	
	10	141	212	120	180	98.8	148	72.4	109	
	12	124	186	105	158	87.1	131	65.9	99.1	
	14	106	159	89.8	135	74.5	112	58.2	87.5	
	16	87.8	132	74.5	112	61.9	93.0	48.5	72.9	
	18	71.2	107	60.3	90.7	50.3	75.6	40.0	60.1	
	20	58.2	87.5	49.5	74.4	41.5	62.4	33.3	50.1	
	22	48.5	72.9	41.3	62.1	34.8	52.3	28.1	42.2	
	24	41.0	61.6	35.0	52.6	29.5	44.3	24.0	36.0	
	26	35.0	52.7	30.0	45.0	25.3	38.1	20.7	31.0	
Properties of 2 angles—3/8 in. back to back										
A_g, in.^{2}		9.50		8.36		7.22		6.06		
r_x, in.		1.91		1.92		1.93		1.94		
r_y, in.		1.64		1.62		1.61		1.60		
Properties of single angle										
r_z, in.		0.864		0.867		0.870		0.874		

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 36 ksi

Table 4-9 (continued)
Available Strength in Axial Compression, kips
Double Angles—LLBB

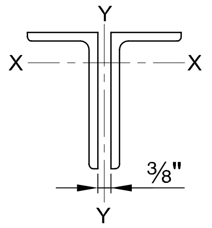


2L6 LLBB

Shape		2L6×3½×						No. of connectors[a]
		½		3/8[c]		5/16[c]		
lb/ft		30.6		23.4		19.6		
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	
Effective length, L_c (ft), with respect to indicated axis: X-X Axis	0	194	292	136	205	106	160	b
	2	192	289	135	204	106	159	
	4	188	282	133	200	104	156	
	6	180	271	129	193	100	151	
	8	170	256	123	185	96.0	144	
	10	158	237	116	174	90.6	136	
	12	144	217	108	162	84.5	127	
	14	130	195	98.6	148	77.7	117	
	16	115	172	88.1	132	70.5	106	
	18	99.6	150	76.7	115	63.1	94.8	
	20	85.2	128	65.7	98.8	55.6	83.5	
	22	71.6	108	55.3	83.1	46.9	70.5	
	24	60.1	90.4	46.4	69.8	39.4	59.2	
	28	44.2	66.4	34.1	51.3	29.0	43.5	
	30	38.5	57.8	29.7	44.7	25.2	37.9	
	32	33.8	50.8	26.1	39.3	22.2	33.3	
Y-Y Axis	0	194	292	136	205	106	160	2
	2	168	252	111	167	79.9	120	
	4	164	246	109	164	78.6	118	
	6	155	233	105	158	75.9	114	
	8	138	207	96.6	145	70.6	106	
	10	119	179	84.5	127	63.6	95.6	
	12	99.3	149	70.6	106	55.1	82.9	
	14	79.8	120	56.6	85.1	44.4	66.7	
	16	62.4	93.8	44.7	67.2	35.6	53.5	
	18	49.9	75.0	36.1	54.2	29.0	43.5	
	20	40.7	61.2	29.6	44.5	24.0	36.0	
	22	33.8	50.9	24.7	37.2	20.1	30.2	
Properties of 2 angles—3/8 in. back to back								
A_g, in.^{2}		9.00		6.88		5.78		
r_x, in.		1.92		1.93		1.94		
r_y, in.		1.40		1.38		1.37		
Properties of single angle								
r_z, in.		0.756		0.763		0.767		

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.



2L5 LLBB

Table 4-9 (continued)
Available Strength in Axial Compression, kips
Double Angles—LLBB

F_y = 36 ksi

Shape		2L5×3½×										No. of connectors[a]
		3/4		5/8		1/2		3/8[c]		5/16[c]		
lb/ft		39.6		33.6		27.2		20.8		17.4		
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	252	379	213	319	172	259	130	195	102	153	
	2	249	374	210	316	170	256	128	193	101	152	
	4	240	360	202	304	164	247	125	187	98.2	148	
	6	225	338	190	286	155	232	118	177	93.5	141	
	8	206	310	174	262	142	213	109	163	87.4	131	
	10	184	276	156	234	127	191	97.4	146	80.0	120	
	12	160	241	136	204	111	167	85.4	128	71.8	108	b
	14	136	204	115	173	95.1	143	73.1	110	61.8	92.8	
	16	112	169	95.7	144	79.3	119	61.0	91.7	51.7	77.7	
	18	90.6	136	77.3	116	64.3	96.7	49.7	74.7	42.2	63.5	
	20	73.4	110	62.6	94.1	52.1	78.3	40.2	60.5	34.2	51.4	
	22	60.6	91.1	51.7	77.8	43.1	64.7	33.3	50.0	28.3	42.5	
	24	50.9	76.6	43.5	65.4	36.2	54.4	27.9	42.0	23.8	35.7	
Y-Y Axis	0	252	379	213	319	172	259	130	195	102	153	
	2	239	360	198	297	155	232	109	164	82.2	124	
	4	234	351	194	291	152	228	107	161	81.1	122	
	6	220	331	183	275	144	217	103	155	78.7	118	
	8	197	296	163	245	129	195	93.7	141	73.4	110	
	10	173	260	143	215	113	170	82.4	124	65.5	98.4	
	12	148	222	121	182	95.8	144	69.8	105	55.6	83.6	2
	14	122	184	99.9	150	78.6	118	57.1	85.8	45.5	68.3	
	16	98.5	148	79.6	120	62.4	93.7	45.2	68.0	36.1	54.2	
	18	81.7	123	63.2	95.0	49.6	74.6	36.2	54.5	29.1	43.7	
	20	66.4	99.8	51.4	77.2	40.4	60.7	29.6	44.5	23.9	35.9	
	22	55.0	82.6	42.5	63.9	33.5	50.4	24.6	37.0	19.9	30.0	
	24	46.2	69.5	37.5	56.3	28.2	42.5	20.8	31.3	16.9	25.4	
												3

Properties of 2 angles—3/8 in. back to back

	3/4	5/8	1/2	3/8	5/16
A_g, in.^{2}	11.7	9.86	8.00	6.10	5.12
r_x, in.	1.55	1.56	1.58	1.59	1.60
r_y, in.	1.53	1.50	1.48	1.46	1.44

Properties of single angle

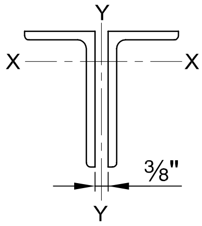
	3/4	5/8	1/2	3/8	5/16
r_z, in.	0.744	0.746	0.750	0.755	0.758

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 36 ksi

Table 4-9 (continued)
Available Strength in Axial Compression, kips
Double Angles—LLBB

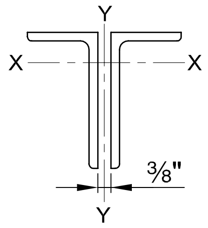


2L5 LLBB

Shape		2L5×3×										
		1/2		7/16		3/8[c]		5/16[c]		1/4[c]		No. of connectors[a]
lb/ft		25.6		22.6		19.6		16.4		13.2		
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	162	243	143	214	122	183	95.6	144	70.2	106	b
	2	160	240	141	212	120	181	94.7	142	69.6	105	
	4	154	231	136	204	117	176	92.1	138	67.7	102	
	6	145	218	128	193	111	167	87.8	132	64.7	97.2	
	8	133	200	118	177	102	153	82.2	124	60.7	91.2	
	10	119	179	106	159	91.7	138	75.4	113	55.9	84.0	
	12	104	157	92.7	139	80.5	121	67.9	102	50.5	75.9	
	14	89.2	134	79.3	119	69.0	104	58.6	88.0	44.7	67.2	
	16	74.3	112	66.2	99.5	57.8	86.8	49.1	73.9	38.9	58.4	
	18	60.3	90.7	53.9	81.0	47.2	70.9	40.3	60.5	32.8	49.3	
	20	48.9	73.4	43.7	65.6	38.2	57.4	32.6	49.0	26.6	39.9	
	22	40.4	60.7	36.1	54.2	31.6	47.5	26.9	40.5	22.0	33.0	
	24	33.9	51.0	30.3	45.6	26.5	39.9	22.6	34.0	18.5	27.7	
Y-Y Axis	0	162	243	143	214	122	183	95.6	144	70.2	106	2
	2	145	218	125	187	103	155	77.8	117	51.7	77.7	
	4	140	211	121	181	99.9	150	76.0	114	50.6	76.0	
	6	129	194	111	167	92.8	140	72.0	108	48.2	72.5	
	8	109	165	94.7	142	79.4	119	63.4	95.2	43.4	65.3	
	10	90.2	136	78.0	117	65.4	98.3	52.4	78.8	37.5	56.3	
	12	71.1	107	61.3	92.1	51.3	77.1	41.2	61.9	30.1	45.2	
	14	53.8	80.8	46.3	69.6	38.8	58.4	31.4	47.2	23.4	35.1	
	16	41.5	62.4	35.9	53.9	30.2	45.4	24.6	36.9	18.5	27.8	
	18	33.0	49.6	28.5	42.9	24.1	36.2	19.7	29.6	15.0	22.5	
	20	26.8	40.3	23.2	34.9	19.6	29.5	16.1	24.2			
Properties of 2 angles—3/8 in. back to back												
A_g, in.^{2}		7.50		6.62		5.72		4.82		3.88		
r_x, in.		1.58		1.59		1.60		1.61		1.62		
r_y, in.		1.24		1.23		1.22		1.21		1.19		
Properties of single angle												
r_z, in.		0.642		0.644		0.646		0.649		0.652		

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.



2L4 LLBB

Table 4-9 (continued)
Available Strength in Axial Compression, kips
Double Angles—LLBB

F_y = 36 ksi

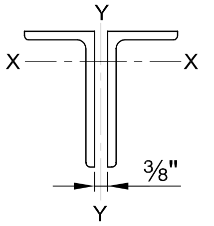
Shape		2L4×3[1]/2× 1/2		3/8		5/16[c]		1/4[c]		No. of connectors[a]
lb/ft		23.8		18.2		15.4		12.4		
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	
Effective length, L_c (ft), with respect to indicated axis										
X-X Axis	0	151	227	116	174	96.9	146	71.3	107	b
	2	148	222	113	170	95.1	143	70.4	106	
	4	139	209	107	161	89.8	135	67.6	102	
	6	126	189	97.0	146	81.5	122	63.2	95.1	
	8	109	165	84.7	127	71.1	107	56.6	85.0	
	10	91.4	137	71.1	107	59.7	89.8	48.7	73.2	
	12	73.3	110	57.5	86.4	48.2	72.5	39.5	59.3	
	14	56.4	84.8	44.6	67.0	37.4	56.3	30.8	46.3	
	16	43.2	64.9	34.1	51.3	28.7	43.1	23.6	35.4	
	18	34.1	51.3	27.0	40.6	22.7	34.0	18.6	28.0	
	20	27.6	41.5	21.9	32.8	18.3	27.6	15.1	22.7	
Y-Y Axis	0	151	227	116	174	96.9	146	71.3	107	2
	2	139	209	101	151	79.8	120	57.1	85.8	
	4	137	206	99.5	149	79.0	119	56.6	85.0	
	6	131	197	96.3	145	76.9	116	55.5	83.5	
	8	118	178	87.9	132	71.0	107	52.8	79.3	
	10	105	157	77.9	117	63.3	95.2	48.3	72.6	
	12	89.8	135	66.8	100	54.4	81.7	41.9	62.9	
	14	74.9	113	55.6	83.6	45.2	67.9	35.0	52.6	
	16	60.8	91.4	45.0	67.6	36.3	54.6	28.2	42.3	
	18	50.9	76.5	37.6	56.5	30.5	45.8	23.8	35.7	3
	20	41.4	62.2	30.6	46.1	24.9	37.4	19.5	29.3	
	22	34.3	51.5	25.4	38.2	20.7	31.1	16.3	24.5	
	24	28.8	43.3	21.4	32.2	17.5	26.2	13.8	20.7	
	26	24.6	37.0							
Properties of 2 angles—3/8 in. back to back										
A_g, in.^{2}		7.00		5.36		4.50		3.64		
r_x, in.		1.23		1.25		1.25		1.26		
r_y, in.		1.57		1.55		1.53		1.52		
Properties of single angle										
r_z, in.		0.716		0.719		0.721		0.723		

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 36 ksi

Table 4-9 (continued)
Available Strength in Axial Compression, kips
Double Angles—LLBB



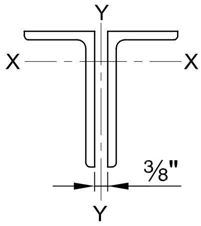
2L4 LLBB

Shape		2L4×3×										
		5/8		1/2		3/8		5/16[c]		1/4[c]		No. of connectors[a]
lb/ft		27.2		22.2		17.0		14.4		11.6		
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	172	259	140	211	107	161	90.0	135	67.5	102	
	2	169	253	137	206	105	158	88.4	133	66.5	100	
	4	159	239	129	195	99.5	149	83.6	126	63.5	95.4	
	6	144	216	117	176	90.4	136	76.1	114	58.8	88.3	
	8	125	188	102	154	79.1	119	66.7	100	52.7	79.2	
	10	104	157	85.6	129	66.6	100	56.3	84.6	45.5	68.4	
	12	83.6	126	68.9	104	54.0	81.1	45.8	68.8	37.0	55.7	b
	14	64.3	96.6	53.2	80.0	42.1	63.3	35.9	53.9	29.0	43.6	
	16	49.2	74.0	40.8	61.2	32.2	48.5	27.5	41.3	22.2	33.4	
	18	38.9	58.5	32.2	48.4	25.5	38.3	21.7	32.6	17.6	26.4	
	20	31.5	47.4	26.1	39.2	20.6	31.0	17.6	26.4	14.2	21.4	
Y-Y Axis	0	172	259	140	211	107	161	90.0	135	67.5	102	
	2	164	246	130	195	94.6	142	75.5	114	54.1	81.4	
	4	158	237	126	190	92.4	139	73.9	111	53.3	80.1	
	6	146	219	117	176	86.5	130	69.9	105	51.2	76.9	
	8	126	189	100	151	74.7	112	60.9	91.5	46.2	69.4	
	10	106	160	84.4	127	62.6	94.1	51.2	76.9	39.0	58.7	2
	12	86.5	130	68.1	102	50.3	75.7	41.1	61.8	31.4	47.2	
	14	67.8	102	52.8	79.3	38.8	58.3	31.6	47.5	24.1	36.3	
	16	54.8	82.3	42.6	64.0	31.3	47.1	24.5	36.9	18.9	28.4	
	18	43.4	65.2	33.8	50.7	24.9	37.4	19.5	29.4	15.1	22.7	
	20	35.2	52.9	27.4	41.2	20.3	30.5	15.9	23.9	12.4	18.6	
	22	29.1	43.8	22.7	34.1							3

Properties of 2 angles—3/8 in. back to back					
A_g, in.^{2}	7.98	6.50	4.98	4.18	3.38
r_x, in.	1.23	1.24	1.26	1.27	1.27
r_y, in.	1.35	1.32	1.30	1.29	1.27
Properties of single angle					
r_z, in.	0.631	0.633	0.636	0.638	0.639

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.



2L3½ LLBB

Table 4-9 (continued)
Available Strength in Axial Compression, kips
Double Angles—LLBB

F_y = 36 ksi

Shape		2L3½×3× ½		2L3½×3× 7/16		2L3½×3× 3/8		2L3½×3× 5/16		2L3½×3× ¼[c]		No. of connectors[a]
lb/ft		20.4		18.2		15.8		13.2		10.8		
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	
Effective length, L_c (ft), with respect to indicated axis: X-X Axis	0	130	196	115	173	100	150	84.1	126	66.3	99.6	b
	2	127	191	112	169	97.5	147	82.0	123	64.9	97.5	
	4	117	176	104	156	90.3	136	75.9	114	60.9	91.5	
	6	103	154	91.1	137	79.5	119	66.8	100	54.4	81.7	
	8	85.2	128	75.9	114	66.5	99.9	55.9	84.0	45.6	68.6	
	10	67.2	101	60.1	90.3	52.8	79.4	44.4	66.8	36.4	54.7	
	12	50.1	75.3	45.2	67.9	39.9	60.0	33.5	50.4	27.6	41.5	
	14	36.8	55.4	33.2	49.9	29.4	44.1	24.7	37.1	20.4	30.6	
	16	28.2	42.4	25.4	38.2	22.5	33.8	18.9	28.4	15.6	23.4	
	18			20.1	30.2	17.8	26.7	14.9	22.4	12.3	18.5	
Y-Y Axis	0	130	196	115	173	100	150	84.1	126	66.3	99.6	2
	2	122	184	106	160	89.9	135	72.4	109	54.3	81.7	
	4	119	179	104	156	88.1	132	71.2	107	53.5	80.4	
	6	111	166	97.0	146	82.9	125	67.6	102	51.4	77.3	
	8	95.7	144	83.9	126	72.0	108	59.0	88.7	45.8	68.8	
	10	81.2	122	71.1	107	60.9	91.6	49.9	75.0	39.0	58.6	
	12	66.4	99.8	58.0	87.1	49.6	74.6	40.5	60.8	31.7	47.7	
	14	54.8	82.4	47.7	71.7	40.7	61.2	33.0	49.6	25.8	38.8	3
	16	42.5	63.8	36.9	55.5	31.5	47.3	25.5	38.4	20.1	30.2	
	18	33.6	50.5	29.3	44.0	25.0	37.6	20.3	30.5	16.0	24.1	
	20	27.3	41.0	23.8	35.7	20.3	30.5	16.5	24.8	13.1	19.7	
	22	22.6	33.9	19.7	29.6	16.8	25.3	13.7	20.6	10.9	16.3	

Properties of 2 angles—3/8 in. back to back

	½	7/16	3/8	5/16	¼
A_g, in.^{2}	6.04	5.34	4.64	3.90	3.16
r_x, in.	1.07	1.08	1.09	1.09	1.10
r_y, in.	1.37	1.36	1.35	1.33	1.32

Properties of single angle

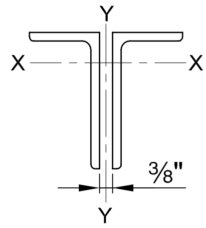
	½	7/16	3/8	5/16	¼
r_z, in.	0.618	0.620	0.622	0.624	0.628

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 36 ksi

Table 4-9 (continued)
Available Strength in Axial Compression, kips
Double Angles—LLBB



2L3½ LLBB

Shape			2L3½×2½× ½		3/8		5/16		¼[c]		No. of connectors[a]
lb/ft			18.8		14.4		12.2		9.80		
Design			P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	
Effective length, L_c (ft), with respect to indicated axis	X-X Axis	0	119	179	91.4	137	77.2	116	60.7	91.2	b
		1	119	178	90.8	137	76.7	115	60.4	90.7	
		2	116	175	89.1	134	75.3	113	59.5	89.4	
		3	113	169	86.4	130	73.0	110	58.0	87.2	
		4	108	162	82.7	124	69.9	105	56.0	84.1	
		5	102	153	78.2	117	66.2	99.5	53.5	80.4	
		6	94.5	142	72.9	110	61.8	92.9	50.3	75.6	
		7	86.9	131	67.2	101	57.1	85.8	46.5	69.9	
		8	78.8	118	61.2	92.0	52.1	78.2	42.5	63.8	
		9	70.5	106	55.0	82.7	46.9	70.5	38.3	57.6	
		10	62.3	93.7	48.8	73.4	41.7	62.7	34.2	51.3	
		11	54.4	81.8	42.8	64.4	36.7	55.1	30.1	45.2	
		12	46.8	70.4	37.1	55.7	31.8	47.8	26.2	39.4	
		13	39.9	60.0	31.7	47.6	27.2	40.9	22.5	33.8	
		14	34.4	51.7	27.3	41.1	23.5	35.3	19.4	29.1	
		15	30.0	45.1	23.8	35.8	20.5	30.8	16.9	25.4	
		16	26.3	39.6	20.9	31.4	18.0	27.0	14.8	22.3	
		17	23.3	35.1	18.5	27.9	15.9	23.9	13.1	19.7	
		18	20.8	31.3	16.5	24.8	14.2	21.4	11.7	17.6	
	Y-Y Axis	0	119	179	91.4	137	77.2	116	60.7	91.2	2
		1	113	170	83.2	125	67.5	101	50.9	76.6	
		2	112	169	82.6	124	67.1	101	50.6	76.1	
		3	110	166	81.4	122	66.2	99.4	50.0	75.2	
		4	106	160	79.0	119	64.4	96.9	48.9	73.5	
		5	101	152	75.4	113	61.7	92.7	47.2	70.9	
		6	92.3	139	69.0	104	56.6	85.1	43.8	65.8	
		7	84.7	127	63.2	95.0	51.9	78.0	40.3	60.5	
		8	76.7	115	57.1	85.8	46.8	70.3	36.4	54.7	
		9	68.5	103	50.8	76.4	41.5	62.4	32.4	48.6	
		10	60.3	90.6	44.6	67.1	36.3	54.6	28.3	42.5	
		11	52.4	78.8	38.6	58.1	31.3	47.1	24.4	36.7	
		12	47.1	70.8	34.5	51.8	26.6	40.0	20.8	31.2	
		13	40.3	60.5	29.5	44.4	22.8	34.3	17.9	26.8	
		14	34.8	52.3	25.5	38.4	19.8	29.7	15.5	23.3	
		15	30.3	45.6	22.3	33.5	17.3	26.0	13.6	20.4	
		16	26.7	40.1	19.6	29.5	15.9	24.0	12.0	18.1	
		17	23.7	35.6	17.4	26.2	14.2	21.3	10.7	16.1	
		18	21.1	31.8	15.6	23.4	12.7	19.0	9.56	14.4	
											3

Properties of 2 angles—3/8 in. back to back

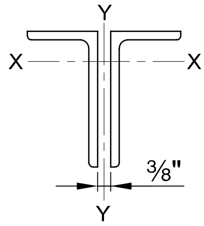
	½	3/8	5/16	¼
A_g, in.^{2}	5.54	4.24	3.58	2.90
r_x, in.	1.08	1.10	1.11	1.12
r_y, in.	1.13	1.11	1.09	1.08

Properties of single angle

	½	3/8	5/16	¼
r_z, in.	0.532	0.535	0.538	0.541

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.



2L3 LLBB

Table 4-9 (continued)
Available Strength in Axial Compression, kips
Double Angles—LLBB

F_y = 36 ksi

Shape		2L3×2½×												No. of connectors[a]
		½		7/16		3/8		5/16		1/4		3/16[c]		
lb/ft		17.0		15.2		13.2		11.2		9.00		6.78		
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	108	162	95.7	144	83.2	125	70.3	106	56.9	85.5	39.7	59.6	b
	1	107	161	94.9	143	82.5	124	69.7	105	56.4	84.8	39.4	59.3	
	2	104	156	92.3	139	80.3	121	67.9	102	55.0	82.7	38.8	58.3	
	3	99.3	149	88.3	133	76.8	115	65.0	97.6	52.7	79.2	37.6	56.6	
	4	93.1	140	82.9	125	72.2	109	61.1	91.9	49.6	74.6	35.8	53.9	
	5	85.7	129	76.4	115	66.6	100	56.5	84.9	45.9	69.0	33.6	50.6	
	6	77.5	117	69.2	104	60.4	90.8	51.3	77.1	41.8	62.8	31.1	46.8	
	7	68.8	103	61.5	92.5	53.9	80.9	45.8	68.9	37.4	56.2	28.3	42.6	
	8	60.0	90.2	53.8	80.8	47.1	70.8	40.2	60.4	32.9	49.4	25.1	37.7	
	9	51.3	77.2	46.1	69.3	40.5	60.9	34.7	52.1	28.4	42.7	21.7	32.7	
	10	43.2	64.9	38.9	58.4	34.2	51.5	29.4	44.1	24.1	36.3	18.5	27.8	
	11	35.7	53.7	32.2	48.4	28.4	42.7	24.4	36.7	20.1	30.2	15.5	23.3	
	12	30.0	45.1	27.1	40.7	23.9	35.9	20.5	30.9	16.9	25.4	13.0	19.5	
	13	25.6	38.4	23.1	34.7	20.4	30.6	17.5	26.3	14.4	21.7	11.1	16.7	
	14	22.1	33.1	19.9	29.9	17.6	26.4	15.1	22.7	12.4	18.7	9.55	14.4	
	15	19.2	28.9	17.3	26.0	15.3	23.0	13.1	19.7	10.8	16.3	8.32	12.5	
Y-Y Axis	0	108	162	95.7	144	83.2	125	70.3	106	56.9	85.5	39.7	59.6	2
	1	103	155	90.4	136	77.0	116	63.0	94.7	48.2	72.4	31.7	47.6	
	2	103	154	89.9	135	76.7	115	62.7	94.3	48.0	72.1	31.5	47.4	
	3	101	151	88.5	133	75.7	114	62.0	93.2	47.5	71.4	31.3	47.1	
	4	97.5	147	85.8	129	73.6	111	60.6	91.1	46.6	70.1	30.9	46.5	
	5	93.0	140	81.8	123	70.3	106	58.2	87.5	45.1	67.8	30.3	45.6	
	6	87.7	132	74.9	113	64.4	96.9	53.5	80.5	41.8	62.9	28.9	43.4	
	7	80.0	120	68.9	104	59.3	89.1	49.3	74.1	38.6	58.0	27.2	40.9	
	8	73.2	110	62.6	94.1	53.8	80.9	44.7	67.2	35.0	52.6	24.9	37.5	
	9	66.3	99.6	56.2	84.4	48.2	72.4	40.0	60.1	31.3	47.0	22.4	33.7	
	10	59.3	89.1	49.7	74.7	42.6	64.0	35.3	53.1	27.5	41.4	19.8	29.7	
	11	52.4	78.7	43.5	65.3	37.2	55.9	30.7	46.2	23.9	35.9	17.2	25.8	
	12	45.7	68.7	39.4	59.2	33.6	50.5	27.8	41.7	21.4	32.2	15.4	23.1	3
	13	39.4	59.2	33.8	50.8	28.8	43.3	23.8	35.7	18.4	27.6	13.3	20.0	
	14	34.0	51.1	29.2	43.9	24.9	37.4	20.6	30.9	15.9	24.0	11.6	17.4	
	15	29.7	44.6	25.5	38.3	21.7	32.6	18.0	27.0	13.9	21.0	10.2	15.3	
	16	26.1	39.2	22.4	33.7	19.1	28.7	15.8	23.8	12.3	18.5	9.00	13.5	
	17	23.1	34.8	19.9	29.9	17.0	25.5	14.0	21.1	10.9	16.4	8.01	12.0	
	18	20.6	31.0	17.7	26.7	15.1	22.8	12.5	18.8	9.77	14.7	7.18	10.8	
	19	18.5	27.9	15.9	23.9	13.6	20.4	11.3	16.9					

Properties of 2 angles—3/8 in. back to back

	½	7/16	3/8	5/16	1/4	3/16
A_g, in.²	5.00	4.44	3.86	3.26	2.64	2.00
r_x, in.	0.910	0.917	0.924	0.932	0.940	0.947
r_y, in.	1.18	1.16	1.15	1.14	1.12	1.11

Properties of single angle

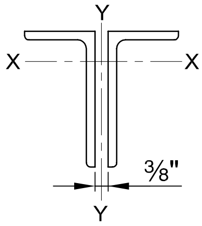
	½	7/16	3/8	5/16	1/4	3/16
r_z, in.	0.516	0.516	0.517	0.518	0.520	0.521

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 36 ksi

Table 4-9 (continued)
Available Strength in Axial Compression, kips
Double Angles—LLBB



2L3 LLBB

Shape			2L3×2×									
			1/2		3/8		5/16		1/4		3/16[c]		No. of connectors[a]
lb/ft			15.4		11.8		10.0		8.20		6.14		
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length, L_c (ft), with respect to indicated axis	X-X Axis	0	97.4	146	75.4	113	63.8	95.9	51.7	77.8	36.4	54.8	
		1	96.6	145	74.8	112	63.3	95.1	51.3	77.1	36.2	54.4	
		2	94.0	141	72.9	110	61.7	92.7	50.0	75.2	35.5	53.3	
		3	89.9	135	69.8	105	59.1	88.8	48.0	72.1	34.3	51.6	
		4	84.5	127	65.7	98.8	55.7	83.7	45.3	68.0	32.7	49.2	
		5	78.0	117	60.8	91.4	51.6	77.6	42.0	63.1	30.8	46.3	
		6	70.7	106	55.3	83.1	47.0	70.7	38.3	57.6	28.6	43.0	
		7	62.9	94.6	49.4	74.3	42.1	63.3	34.4	51.7	26.2	39.3	
		8	55.1	82.8	43.4	65.3	37.1	55.7	30.3	45.6	23.3	35.1	b
		9	47.3	71.1	37.5	56.3	32.1	48.2	26.3	39.5	20.3	30.5	
		10	39.9	60.0	31.8	47.8	27.3	41.0	22.5	33.7	17.4	26.1	
		11	33.1	49.8	26.5	39.8	22.8	34.3	18.8	28.3	14.6	21.9	
		12	27.9	41.9	22.3	33.5	19.2	28.8	15.8	23.7	12.3	18.4	
		13	23.7	35.7	19.0	28.5	16.3	24.5	13.5	20.2	10.4	15.7	
		14	20.5	30.8	16.4	24.6	14.1	21.2	11.6	17.4	9.00	13.5	
		15	17.8	26.8	14.3	21.4	12.3	18.4	10.1	15.2	7.84	11.8	
		16									6.89	10.4	
	Y-Y Axis	0	97.4	146	75.4	113	63.8	95.9	51.7	77.8	36.4	54.8	
		1	93.5	141	70.4	106	57.9	86.9	44.6	67.0	29.7	44.6	
		2	92.2	139	69.5	104	57.2	85.9	44.1	66.3	29.4	44.2	
		3	88.8	134	67.3	101	55.5	83.4	43.0	64.6	28.9	43.4	
		4	83.8	126	63.5	95.4	52.6	79.0	41.0	61.6	28.0	42.0	
		5	75.5	113	57.0	85.7	47.2	71.0	37.0	55.7	26.0	39.1	
		6	67.8	102	50.9	76.6	42.2	63.4	33.1	49.7	23.6	35.4	
		7	59.8	89.8	44.6	67.0	36.8	55.3	28.8	43.3	20.6	31.0	
		8	51.6	77.6	38.2	57.5	31.4	47.3	24.5	36.9	17.5	26.4	2
		9	43.8	65.8	32.1	48.3	26.3	39.5	20.4	30.7	14.6	21.9	
		10	38.3	57.5	27.7	41.7	22.6	33.9	16.7	25.2	12.0	18.1	
		11	31.7	47.6	23.0	34.6	18.8	28.2	14.6	21.9	10.1	15.2	
		12	26.7	40.1	19.4	29.1	15.8	23.8	12.3	18.5	8.57	12.9	
		13	22.8	34.2	16.5	24.9	13.5	20.3	10.6	15.9	7.36	11.1	
		14	19.6	29.5	14.3	21.5	11.7	17.6	9.14	13.7	6.39	9.60	
		15	17.1	25.7	12.5	18.7							3

Properties of 2 angles—3/8 in. back to back

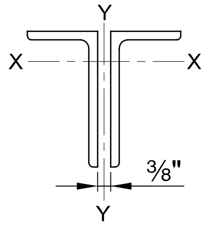
	1/2	3/8	5/16	1/4	3/16
A_g, in.^{2}	4.52	3.50	2.96	2.40	1.83
r_x, in.	0.922	0.937	0.945	0.953	0.961
r_y, in.	0.940	0.911	0.897	0.883	0.869

Properties of single angle

	1/2	3/8	5/16	1/4	3/16
r_z, in.	0.425	0.426	0.428	0.431	0.435

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.



2L2½ LLBB

Table 4-9 (continued)
Available Strength in Axial Compression, kips
Double Angles—LLBB

F_y = 36 ksi

Shape		2L2½×2×3/8		2L2½×2×5/16		2L2½×2×1/4		2L2½×2×3/16[c]		2L2½×1½×1/4		2L2½×1½×3/16[c]		No. of connectors[a]
lb/ft		10.6		9.00		7.24		5.50		6.38		4.88		
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	
Effective length, L_c (ft), with respect to indicated axis: X-X Axis	0	66.8	100	56.9	85.5	46.1	69.3	34.9	52.5	40.7	61.2	30.8	46.3	
	1	66.0	99.2	56.2	84.5	45.6	68.5	34.6	52.0	40.3	60.5	30.5	45.9	
	2	63.5	95.4	54.1	81.3	43.9	66.0	33.5	50.4	38.8	58.3	29.6	44.6	
	3	59.5	89.4	50.8	76.3	41.3	62.0	31.7	47.6	36.5	54.9	28.1	42.2	
	4	54.3	81.7	46.5	69.9	37.8	56.9	29.1	43.8	33.5	50.4	25.9	38.9	
	5	48.4	72.7	41.5	62.3	33.8	50.9	26.1	39.2	30.1	45.2	23.2	34.9	
	6	42.0	63.1	36.1	54.2	29.5	44.4	22.8	34.3	26.3	39.5	20.4	30.7	b
	7	35.5	53.3	30.6	46.0	25.1	37.8	19.5	29.3	22.5	33.8	17.5	26.3	
	8	29.2	43.9	25.3	38.1	20.9	31.4	16.2	24.4	18.7	28.1	14.6	22.0	
	9	23.4	35.2	20.4	30.6	16.9	25.3	13.2	19.8	15.2	22.8	12.0	18.0	
	10	19.0	28.5	16.5	24.8	13.7	20.5	10.7	16.1	12.3	18.5	9.69	14.6	
	11	15.7	23.6	13.6	20.5	11.3	17.0	8.83	13.3	10.2	15.3	8.01	12.0	
	12	13.2	19.8	11.5	17.2	9.49	14.3	7.42	11.2	8.55	12.9	6.73	10.1	
	13					8.08	12.1	6.32	9.50	7.29	10.9	5.73	8.61	
Y-Y Axis	0	66.8	100	56.9	85.5	46.1	69.3	34.9	52.5	40.7	61.2	30.8	46.3	
	1	63.2	95.0	52.6	79.1	41.0	61.6	28.8	43.2	36.7	55.1	26.1	39.2	
	2	62.6	94.1	52.2	78.4	40.6	61.1	28.6	42.9	35.7	53.7	25.5	38.3	
	3	60.8	91.3	50.9	76.5	39.8	59.9	28.1	42.2	33.6	50.5	24.2	36.3	2
	4	57.6	86.6	48.4	72.7	38.1	57.3	27.2	40.9	29.3	44.1	21.3	32.0	
	5	53.6	80.5	43.7	65.7	34.6	52.0	25.1	37.8	25.1	37.7	18.2	27.4	
	6	47.8	71.8	39.3	59.1	31.2	46.8	22.8	34.2	20.6	31.0	14.9	22.5	
	7	42.6	64.0	34.7	52.1	27.5	41.3	20.1	30.2	16.3	24.6	11.8	17.7	
	8	37.2	55.9	30.0	45.1	23.7	35.6	17.3	26.1	13.3	20.0	9.59	14.4	
	9	32.0	48.1	25.4	38.2	20.0	30.1	14.6	22.0	10.6	15.9	7.67	11.5	
	10	27.0	40.5	22.3	33.5	17.4	26.2	12.7	19.0	8.61	12.9	6.27	9.42	
	11	22.4	33.7	18.5	27.8	14.5	21.8	10.6	15.9	7.14	10.7	5.21	7.84	3
	12	18.9	28.3	15.6	23.4	12.2	18.4	8.97	13.5					
	13	16.1	24.2	13.3	20.0	10.4	15.7	7.68	11.5					
	14	13.9	20.9	11.5	17.3	9.03	13.6	6.66	10.0					
	15	12.1	18.2	10.0	15.0	7.88	11.8	5.82	8.75					

Properties of 2 angles—3/8 in. back to back

Property	2L2½×2×3/8	2L2½×2×5/16	2L2½×2×1/4	2L2½×2×3/16	2L2½×1½×1/4	2L2½×1½×3/16
A_g, in.^{2}	3.10	2.64	2.14	1.64	1.89	1.45
r_x, in.	0.766	0.774	0.782	0.790	0.790	0.800
r_y, in.	0.957	0.943	0.930	0.916	0.691	0.677

Properties of single angle

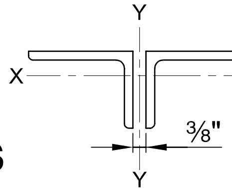
Property	2L2½×2×3/8	2L2½×2×5/16	2L2½×2×1/4	2L2½×2×3/16	2L2½×1½×1/4	2L2½×1½×3/16
r_z, in.	0.419	0.420	0.423	0.426	0.321	0.324

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 36 ksi

Table 4-10
Available Strength in Axial Compression, kips
Double Angles—SLBB



2L8 SLBB

Shape			2L8×6× 1		7/8		3/4		5/8		9/16[c]		1/2[c]		7/16[c]		No. of connectors[a]
lb/ft			88.4		78.2		67.6		57.0		51.4		46.0		40.4		
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length, L_c (ft), with respect to indicated axis	X-X Axis	0	565	849	496	745	431	648	362	544	317	476	272	409	224	337	b
		4	542	815	476	716	414	623	348	524	307	461	264	396	219	329	
		6	515	774	453	681	394	593	332	499	295	443	253	381	212	318	
		8	479	720	422	635	368	553	310	466	279	419	240	361	202	304	
		10	437	657	386	580	337	506	284	427	258	388	224	336	189	284	
		12	391	587	346	520	302	454	256	384	232	349	205	308	173	261	
		14	342	514	304	456	265	399	225	339	205	308	184	277	157	236	
		16	293	441	261	393	229	344	195	293	178	267	160	240	139	210	
		18	246	370	220	331	193	291	165	249	151	227	136	205	121	182	
		20	202	304	182	273	160	240	137	206	126	189	114	171	101	152	
		22	167	251	150	226	132	199	114	171	104	156	94.0	141	83.8	126	
		24	140	211	126	190	111	167	95.4	143	87.3	131	79.0	119	70.5	106	
		26	120	180	108	162	94.6	142	81.3	122	74.4	112	67.3	101	60.0	90.2	
		28	103	155	92.7	139	81.5	123	70.1	105	64.1	96.4	58.0	87.2	51.8	77.8	
		30													45.1	67.8	
	Y-Y Axis	0	565	849	496	745	431	648	362	544	317	476	272	409	224	337	3
		4	524	787	450	676	378	569	301	453	261	392	215	323	169	254	
		6	523	786	450	676	378	568	301	452	261	392	215	323	169	253	
		8	522	784	449	674	377	567	300	451	260	391	214	322	168	253	
		10	519	780	447	672	376	565	300	450	260	390	214	322	168	253	
		12	513	771	443	666	374	562	298	448	259	389	213	321	168	252	
		16	488	733	425	638	363	545	293	440	255	383	211	318	166	250	
		20	440	661	384	577	331	497	272	409	241	362	204	307	162	244	
		24	396	595	345	519	297	447	246	370	220	330	191	287	155	233	
		28	349	525	304	457	262	394	217	326	194	292	171	257	143	214	
		32	302	454	263	395	226	340	187	281	168	252	148	223	127	191	
		36	256	385	223	335	191	287	158	238	142	213	125	188	108	162	
		40	222	334	193	290	165	248	136	205	122	183	108	162	92.9	140	4
		44	184	276	160	240	136	205	113	170	101	152	89.7	135	77.6	117	
		48	155	232	134	202	115	173	95.1	143	85.5	128	75.7	114	65.7	98.7	
		52	132	198	114	172	98.0	147	81.2	122	73.0	110	64.7	97.3	56.3	84.6	
		56	114	171	98.7	148	84.6	127	70.1	105	63.1	94.8	56.0	84.1	48.7	73.2	
		60	99.1	149	86.1	129	73.7	111	61.2	91.9	55.0	82.7	48.9	73.4	42.5	63.9	

Properties of 2 angles—3/8 in. back to back

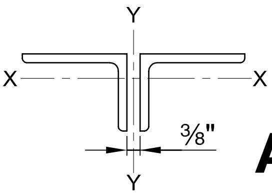
	1	7/8	3/4	5/8	9/16	1/2	7/16
A_g, in.^{2}	26.2	23.0	20.0	16.8	15.2	13.6	12.0
r_x, in.	1.72	1.74	1.75	1.77	1.78	1.79	1.80
r_y, in.	3.77	3.75	3.72	3.70	3.69	3.68	3.66

Properties of single angle

	1	7/8	3/4	5/8	9/16	1/2	7/16
r_z, in.	1.28	1.28	1.29	1.29	1.30	1.30	1.31

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.



2L8 SLBB

Table 4-10 (continued)
Available Strength in Axial Compression, kips
Double Angles—SLBB

F_y = 36 ksi

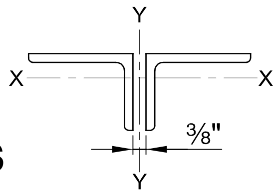
Shape		2L8×4×1		2L8×4×7/8		2L8×4×3/4		2L8×4×5/8		2L8×4×9/16[c]		2L8×4×1/2[c]		2L8×4×7/16[c]		No. of connectors[a]
lb/ft		74.8		66.2		57.4		48.4		43.8		39.2		34.4		
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	479	719	423	635	366	551	308	463	269	405	229	344	190	285	b
	4	427	642	378	568	328	493	277	416	247	372	211	317	175	263	
	6	370	556	328	493	286	430	242	363	221	332	190	286	159	238	
	8	303	455	270	405	236	355	200	301	183	276	164	246	138	207	
	10	234	352	210	315	184	277	157	236	145	217	131	196	114	172	
	12	171	257	154	231	136	204	116	175	108	162	98.1	147	87.7	132	
	14	125	189	113	170	99.8	150	85.6	129	79.3	119	72.1	108	64.5	97.0	
	16	96.0	144	86.4	130	76.4	115	65.5	98.5	60.7	91.2	55.2	82.9	49.4	74.3	
	18											43.6	65.5	39.0	58.7	
Y-Y Axis	0	479	719	423	635	366	551	308	463	269	405	229	344	190	285	6
	4	445	668	385	578	323	486	258	388	226	339	185	277	145	217	
	6	445	668	385	578	323	486	258	388	226	339	185	277	144	217	
	8	444	668	384	578	323	486	258	388	226	339	185	277	144	217	
	10	444	667	384	577	323	485	258	388	226	339	184	277	144	217	
	12	442	665	383	576	322	485	258	388	225	339	184	277	144	217	
	16	425	638	373	561	319	479	257	386	225	338	184	276	144	217	
	20	390	586	343	515	296	445	246	370	222	333	183	275	143	216	
	24	356	535	313	471	270	407	226	339	204	307	175	263	141	212	
	28	320	482	282	424	243	365	203	305	184	276	162	243	133	200	
	32	284	426	249	374	214	322	179	269	162	243	143	215	121	182	
	36	247	371	217	326	186	280	155	233	140	211	124	187	108	162	
	40	211	318	185	279	159	239	132	199	120	180	106	159	92.3	139	
	44	182	274	160	240	136	205	111	166	100	150	88.3	133	77.1	116	
	48	153	230	134	202	115	172	93.0	140	84.1	126	74.3	112	64.9	97.6	
	52	131	196	114	172	97.8	147	79.2	119	71.7	108	63.4	95.3	55.4	83.3	
	56	113	169	98.5	148	84.3	127	68.4	103	61.9	93.0	54.7	82.2	47.8	71.9	
	60	98.1	147	85.9	129	73.5	110	61.0	91.7	53.9	81.0	47.7	71.7	41.7	62.7	
	64	86.2	130	75.5	113	64.6	97.1	53.6	80.6	47.4	71.3	41.9	63.0	36.7	55.1	
	68	76.4	115													7
Properties of 2 angles—3/8 in. back to back																
A_g, in.^{2}		22.2		19.6		17.0		14.3		13.0		11.6		10.2		
r_x, in.		1.03		1.04		1.05		1.06		1.07		1.08		1.09		
r_y, in.		4.08		4.06		4.03		4.00		3.99		3.97		3.96		
Properties of single angle																
r_z, in.		0.844		0.846		0.850		0.856		0.859		0.863		0.867		

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 36 ksi

Table 4-10 (continued)
Available Strength in Axial Compression, kips
Double Angles—SLBB

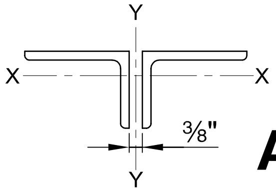


2L7 SLBB

Shape		2L7×4× 3/4		2L7×4× 5/8		2L7×4× 1/2[c]		2L7×4× 7/16[c]		2L7×4× 3/8[c]		No. of connectors[a]
lb/ft		52.4		44.2		35.8		31.4		27.2		
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	
Effective length, L_c (ft), with respect to indicated axis: X-X Axis	0	334	502	280	421	219	329	183	275	148	223	
	4	301	453	254	381	202	304	170	255	138	207	
	6	264	397	224	336	181	273	154	231	125	188	
	8	220	331	188	282	153	229	134	202	109	164	
	10	174	262	150	225	122	184	109	164	91.6	138	
	12	131	197	114	171	93.3	140	83.6	126	72.2	109	b
	14	96.3	145	83.8	126	68.9	104	61.9	93.0	53.4	80.3	
	16	73.7	111	64.1	96.4	52.7	79.3	47.4	71.2	40.9	61.5	
	18	58.2	87.5	50.7	76.2	41.7	62.6	37.4	56.2	32.3	48.6	
Y-Y Axis	0	334	502	280	421	219	329	183	275	148	223	
	4	303	456	244	367	184	276	148	222	111	167	
	6	303	455	244	367	184	276	148	222	111	167	
	8	303	455	244	367	184	276	147	222	111	167	
	10	302	454	244	366	183	276	147	221	111	167	
	12	299	450	243	365	183	275	147	221	111	167	
	16	283	425	234	352	181	272	146	220	111	166	
	20	252	378	210	315	167	251	141	211	108	163	
	24	223	335	186	279	148	223	129	194	102	154	
	28	193	290	160	241	128	192	112	168	92.1	138	5
	32	163	245	136	204	108	162	94.4	142	80.3	121	
	36	135	203	112	168	88.9	134	77.7	117	66.0	99.2	
	40	113	169	93.4	140	72.2	108	63.2	94.9	53.8	80.8	
	44	93.1	140	77.2	116	59.7	89.8	52.3	78.6	44.6	67.0	
	48	78.3	118	64.9	97.6	50.2	75.5	44.0	66.2	37.6	56.4	
	52	66.7	100	55.4	83.2	42.8	64.4	37.6	56.4	32.1	48.2	
	56	57.5	86.5	47.7	71.8	37.0	55.5	32.4	48.7	27.7	41.6	
												6
Properties of 2 angles—3/8 in. back to back												
A_g, in.^{2}		15.5		13.0		10.5		9.26		8.00		
r_x, in.		1.08		1.10		1.11		1.12		1.12		
r_y, in.		3.48		3.46		3.43		3.42		3.40		
Properties of single angle												
r_z, in.		0.855		0.860		0.866		0.869		0.873		

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.



2L6 SLBB

Table 4-10 (continued)
Available Strength in Axial Compression, kips
Double Angles—SLBB

F_y = 36 ksi

Shape		2L6×4× 7/8		2L6×4× 3/4		2L6×4× 5/8		2L6×4× 9/16		No. of connectors[a]
lb/ft		54.4		47.2		40.0		36.2		
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	
Effective length, L_c (ft), with respect to indicated axis										
X-X Axis	0	345	518	300	450	252	379	229	343	b
	4	312	469	272	409	229	345	208	313	
	6	275	414	241	362	204	306	185	278	
	8	231	347	204	306	172	259	157	236	
	10	184	277	164	246	139	209	128	192	
	12	140	210	126	189	107	161	98.6	148	
	14	103	155	92.9	140	79.6	120	73.4	110	
	16	78.9	119	71.1	107	60.9	91.6	56.2	84.4	
	18	62.4	93.7	56.2	84.4	48.1	72.3	44.4	66.7	
Y-Y Axis	0	345	518	300	450	252	379	229	343	4
	4	326	490	278	418	227	341	201	302	
	6	325	489	278	417	227	341	201	302	
	8	323	485	276	415	226	339	200	301	
	10	315	473	271	408	224	336	199	299	
	12	303	456	262	394	218	328	195	293	
	16	268	403	232	349	194	291	175	262	
	20	233	351	202	303	168	253	152	228	
	24	197	296	170	255	141	212	127	191	
	28	161	242	138	208	115	172	103	155	
	32	132	198	113	169	93.0	140	83.6	126	5
	36	104	156	89.2	134	73.6	111	66.2	99.5	
	40	84.3	127	72.3	109	59.6	89.7	53.7	80.7	
	44	69.7	105	59.8	89.8	49.3	74.1	44.4	66.7	
	48	58.6	88.0	50.2	75.5	41.5	62.3	37.3	56.1	

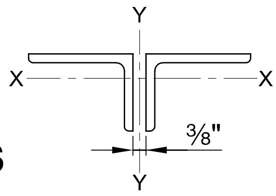
Properties of 2 angles—3/8 in. back to back	7/8	3/4	5/8	9/16
A_g, in.^{2}	16.0	13.9	11.7	10.6
r_x, in.	1.10	1.12	1.13	1.14
r_y, in.	2.96	2.94	2.91	2.90
Properties of single angle				
r_z, in.	0.854	0.856	0.859	0.861

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 36 ksi

Table 4-10 (continued)
Available Strength in Axial Compression, kips
Double Angles—SLBB

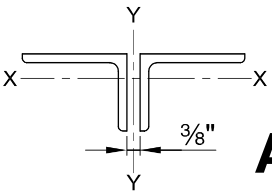


2L6 SLBB

Shape		2L6×4×								
		1/2		7/16[c]		3/8[c]		5/16[c]		No. of connectors[a]
lb/ft		32.4		28.6		24.6		20.6		
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	
Effective length, L_c (ft), with respect to indicated axis: X-X Axis	0	205	308	176	265	144	216	112	169	
	4	187	280	163	245	134	201	105	157	
	6	166	249	147	220	122	183	95.7	144	
	8	141	212	125	188	107	161	84.5	127	
	10	114	172	102	153	88.6	133	71.8	108	
	12	88.4	133	78.9	119	69.2	104	58.7	88.2	b
	14	65.7	98.8	58.9	88.5	51.7	77.8	44.2	66.4	
	16	50.3	75.7	45.1	67.8	39.6	59.5	33.8	50.8	
	18	39.8	59.8	35.6	53.5	31.3	47.0	26.7	40.2	
Y-Y Axis	0	205	308	176	265	144	216	112	169	
	4	174	262	146	220	114	172	81.8	123	
	6	174	262	146	220	114	172	81.7	123	
	8	174	261	146	219	114	172	81.6	123	
	10	173	260	145	218	114	171	81.4	122	
	12	171	257	144	217	113	170	81.1	122	
	16	155	233	134	202	109	164	79.5	119	
	20	135	203	118	177	99.4	149	75.1	113	4
	24	113	170	98.8	148	83.9	126	66.5	100	
	28	91.9	138	80.1	120	68.0	102	55.9	84.0	
	32	72.1	108	62.8	94.4	53.3	80.1	44.0	66.1	
	36	57.1	85.8	49.8	74.8	42.3	63.6	35.0	52.7	
	40	46.3	69.6	40.4	60.8	34.4	51.7	28.5	42.9	
	44	39.5	59.4	33.5	50.3	28.5	42.8	23.7	35.6	
	48	33.2	49.9	28.1	42.3					
										5
Properties of 2 angles—3/8 in. back to back										
A_g, in.^{2}		9.50		8.36		7.22		6.06		
r_x, in.		1.14		1.15		1.16		1.17		
r_y, in.		2.89		2.88		2.86		2.85		
Properties of single angle										
r_z, in.		0.864		0.867		0.870		0.874		

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.



2L6 SLBB

Table 4-10 (continued)
Available Strength in Axial Compression, kips
Double Angles—SLBB

F_y = 36 ksi

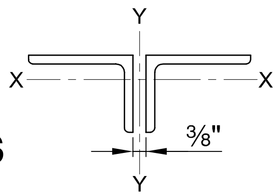
Shape		2L6×3½×						No. of connectors[a]
		1/2		3/8[c]		5/16[c]		
lb/ft		30.6		23.4		19.6		
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length, L_c (ft), with respect to indicated axis: X-X Axis	0	194	292	136	205	106	160	b
	1	192	289	135	204	106	159	
	2	188	282	133	200	104	156	
	3	180	271	129	194	101	151	
	4	170	256	123	185	96.4	145	
	5	158	238	116	175	91.2	137	
	6	145	218	109	163	85.3	128	
	7	131	196	99.9	150	78.7	118	
	8	116	174	89.9	135	71.7	108	
	9	101	151	78.7	118	64.5	96.9	
	10	86.4	130	67.8	102	57.2	85.9	
	11	72.7	109	57.5	86.4	49.0	73.6	
	12	61.1	91.9	48.3	72.6	41.1	61.8	
	13	52.1	78.3	41.1	61.8	35.1	52.7	
	14	44.9	67.5	35.5	53.3	30.2	45.4	
	15	39.1	58.8	30.9	46.4	26.3	39.6	
	16	34.4	51.7	27.2	40.8	23.1	34.8	
Y-Y Axis	0	194	292	136	205	106	160	5
	6	166	249	109	164	78.1	117	
	8	165	248	109	164	78.0	117	
	10	165	248	109	164	77.9	117	
	12	163	246	109	163	77.7	117	
	14	160	240	108	162	77.5	116	
	16	150	225	106	159	76.8	115	
	18	141	212	103	154	75.7	114	
	20	131	197	97.5	146	73.5	111	
	22	121	182	90.9	137	70.1	105	
	24	111	167	83.5	125	65.8	98.9	
	26	101	152	75.9	114	61.1	91.8	
	28	91.0	137	68.5	103	56.1	84.4	
	30	81.5	122	61.2	92.1	50.4	75.8	
	32	72.2	109	54.2	81.5	44.7	67.1	
	34	64.0	96.2	48.1	72.4	39.7	59.7	
	38	51.3	77.1	38.6	58.1	31.9	48.0	
	42	42.0	63.2	31.7	47.6	26.2	39.4	
	46	35.1	52.7	26.5	39.8	21.9	32.9	
	48	32.2	48.4	24.3	36.5	20.1	30.3	
Properties of 2 angles—3/8 in. back to back								
A_g, in.^{2}		9.00		6.88		5.78		
r_x, in.		0.968		0.984		0.991		
r_y, in.		2.96		2.94		2.92		
Properties of single angle								
r_z, in.		0.756		0.763		0.767		

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.

F_y = 36 ksi

Table 4-10 (continued)
Available Strength in Axial Compression, kips
Double Angles—SLBB



2L5 SLBB

Shape		2L5×3½×										No. of connectors[a]
		3/4		5/8		1/2		3/8[c]		5/16[c]		
lb/ft		39.6		33.6		27.2		20.8		17.4		
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length, L_c (ft), with respect to indicated axis: X-X Axis	0	252	379	213	319	172	259	130	195	102	153	b
	1	250	376	211	317	171	257	129	194	101	152	
	2	244	367	206	310	167	251	127	190	99.6	150	
	3	235	353	198	298	161	242	123	185	96.7	145	
	4	222	334	188	282	153	230	117	176	92.8	139	
	5	207	310	175	263	143	214	110	165	87.9	132	
	6	189	284	161	241	131	197	101	152	82.3	124	
	7	170	256	145	218	119	179	92.0	138	76.1	114	
	8	151	227	129	194	106	160	82.5	124	69.2	104	
	9	132	198	113	170	93.3	140	72.9	110	61.2	91.9	
	10	113	170	97.6	147	80.8	121	63.5	95.4	53.3	80.1	
	11	95.7	144	82.9	125	68.9	104	54.5	81.8	45.7	68.7	
	12	80.5	121	69.6	105	58.0	87.2	46.0	69.1	38.6	58.0	
	13	68.6	103	59.3	89.2	49.4	74.3	39.2	58.9	32.9	49.4	
	14	59.1	88.8	51.2	76.9	42.6	64.0	33.8	50.8	28.4	42.6	
	15	51.5	77.4	44.6	67.0	37.1	55.8	29.4	44.3	24.7	37.1	
	16	45.3	68.0	39.2	58.9	32.6	49.0	25.9	38.9	21.7	32.6	
	17							22.9	34.5	19.2	28.9	
Y-Y Axis	0	252	379	213	319	172	259	130	195	102	153	4
	6	237	357	196	295	153	230	107	162	80.9	122	
	8	231	348	193	290	152	228	107	161	80.6	121	
	10	222	333	186	279	148	223	106	159	80.1	120	
	12	210	316	176	265	141	212	103	155	79.2	119	
	14	192	288	161	242	129	194	95.9	144	76.2	114	
	16	177	265	148	222	119	178	88.4	133	72.0	108	
	18	161	242	134	202	108	162	80.3	121	66.1	99.4	
	20	145	218	121	182	96.6	145	72.0	108	59.4	89.3	
	22	129	194	107	162	85.7	129	63.8	95.9	52.7	79.2	
	24	114	171	94.5	142	75.1	113	55.8	83.9	46.1	69.4	
	26	99.0	149	82.1	123	65.0	97.7	48.2	72.4	39.8	59.8	
	28	85.4	128	70.8	106	56.1	84.4	41.6	62.6	34.5	51.8	
	30	74.4	112	61.7	92.8	48.9	73.6	36.3	54.6	30.1	45.3	
	32	65.4	98.3	54.3	81.6	43.0	64.7	32.0	48.1	26.5	39.9	
	34	58.0	87.1	48.1	72.3	38.2	57.3	28.4	42.6	23.5	35.4	
	38	46.4	69.8	38.5	57.9	30.6	45.9	22.7	34.2	18.9	28.4	

Properties of 2 angles—3/8 in. back to back

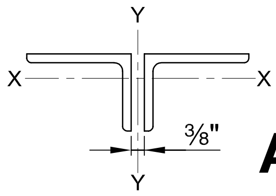
	3/4	5/8	1/2	3/8[c]	5/16[c]
A_g, in.^{2}	11.7	9.86	8.00	6.10	5.12
r_x, in.	0.974	0.987	1.00	1.02	1.02
r_y, in.	2.47	2.45	2.42	2.39	2.38

Properties of single angle

	3/4	5/8	1/2	3/8[c]	5/16[c]
r_z, in.	0.744	0.746	0.750	0.755	0.758

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.



2L5 SLBB

Table 4-10 (continued)
Available Strength in Axial Compression, kips
Double Angles—SLBB

F_y = 36 ksi

Shape			2L5×3× 1/2		7/16		3/8[c]		5/16[c]		1/4[c]		No. of connectors[a]
lb/ft			25.6		22.6		19.6		16.4		13.2		
Design			P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	
Effective length, L_c (ft), with respect to indicated axis	X-X Axis	0	162	243	143	214	122	183	95.6	144	70.2	106	b
		1	160	240	141	212	121	181	94.8	142	69.6	105	
		2	155	232	137	205	117	176	92.4	139	67.9	102	
		3	146	220	129	194	112	168	88.5	133	65.2	98.0	
		4	135	203	120	180	104	156	83.4	125	61.5	92.5	
		5	122	184	108	163	94.1	141	77.2	116	57.1	85.9	
		6	108	163	96.1	144	83.6	126	70.1	105	52.2	78.4	
		7	93.6	141	83.3	125	72.7	109	61.8	92.9	46.8	70.3	
		8	79.1	119	70.7	106	61.8	92.9	52.8	79.3	41.2	62.0	
		9	65.4	98.4	58.7	88.2	51.4	77.3	44.1	66.2	35.6	53.6	
		10	53.2	79.9	47.7	71.7	41.9	63.0	36.0	54.1	29.5	44.3	
		11	43.9	66.0	39.4	59.3	34.7	52.1	29.8	44.7	24.4	36.6	
		12	36.9	55.5	33.1	49.8	29.1	43.8	25.0	37.6	20.5	30.8	
		13	31.5	47.3	28.2	42.4	24.8	37.3	21.3	32.0	17.4	26.2	
		14					21.4	32.2	18.4	27.6	15.0	22.6	
	Y-Y Axis	0	162	243	143	214	122	183	95.6	144	70.2	106	4
		6	144	217	123	185	101	152	76.5	115	50.4	75.8	
		8	143	216	123	185	101	152	76.3	115	50.3	75.7	
		10	141	211	121	183	101	151	76.1	114	50.2	75.5	
		12	135	202	117	176	98.7	148	75.6	114	49.9	75.0	
		14	124	186	108	163	92.4	139	73.5	110	49.4	74.2	
		16	114	171	99.9	150	85.6	129	69.9	105	48.2	72.4	
		18	104	157	91.2	137	78.2	118	64.9	97.6	45.9	69.0	
		20	94.1	141	82.3	124	70.6	106	58.7	88.3	42.6	64.1	
		22	84.1	126	73.4	110	62.9	94.6	52.4	78.8	38.9	58.5	
		24	74.4	112	64.8	97.4	55.5	83.5	46.3	69.5	35.0	52.6	
		26	65.0	97.8	56.6	85.0	48.4	72.8	40.3	60.6	31.6	47.5	5
		28	56.2	84.5	48.9	73.5	41.8	62.9	34.9	52.4	27.4	41.2	
		30	49.0	73.7	42.6	64.0	36.5	54.9	30.4	45.7	24.0	36.0	
		32	43.1	64.8	37.5	56.3	32.1	48.3	26.8	40.3	21.1	31.8	
		34	38.2	57.4	33.2	49.9	28.5	42.8	23.8	35.7	18.8	28.2	
		38	30.6	46.0	26.6	40.0	22.8	34.3	19.1	28.6	15.1	22.6	

Properties of 2 angles—3/8 in. back to back

	1/2	7/16	3/8[c]	5/16[c]	1/4[c]
A_g, in.^{2}	7.50	6.62	5.72	4.82	3.88
r_x, in.	0.824	0.831	0.838	0.846	0.853
r_y, in.	2.50	2.48	2.47	2.46	2.44

Properties of single angle

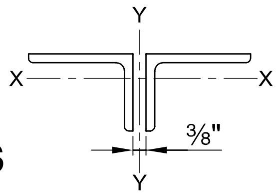
	1/2	7/16	3/8[c]	5/16[c]	1/4[c]
r_z, in.	0.642	0.644	0.646	0.649	0.652

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 36 ksi

Table 4-10 (continued)
Available Strength in Axial Compression, kips
Double Angles—SLBB



2L4 SLBB

Shape		2L4×3½× 1/2		2L4×3½× 3/8		2L4×3½× 5/16		2L4×3½× 1/4[c]		No. of connectors[a]
lb/ft		23.8		18.2		15.4		12.4		
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	
Effective length, L_c (ft), with respect to indicated axis: X-X Axis	0	151	227	116	174	96.9	146	71.3	107	b
	1	150	225	115	172	96.4	145	71.0	107	
	2	147	221	112	169	94.4	142	70.0	105	
	3	142	213	109	163	91.3	137	68.4	103	
	4	135	203	104	156	87.1	131	66.3	99.6	
	5	127	190	97.3	146	81.9	123	63.5	92.5	
	6	117	176	90.2	136	76.1	114	59.9	90.0	
	7	107	161	82.5	124	69.7	105	55.7	83.7	
	8	96.4	145	74.4	112	63.0	94.7	51.2	76.9	
	9	85.5	129	66.2	99.5	56.2	84.4	45.9	69.0	
	10	74.9	113	58.1	87.3	49.4	74.3	40.5	60.8	
	11	64.6	97.1	50.3	75.6	42.9	64.4	35.2	52.9	
	12	54.9	82.5	42.8	64.4	36.7	55.1	30.2	45.4	
	13	46.8	70.3	36.5	54.9	31.2	46.9	25.7	38.7	
	14	40.3	60.6	31.5	47.3	26.9	40.5	22.2	33.4	
	15	35.1	52.8	27.4	41.2	23.5	35.3	19.3	29.1	
	16	30.9	46.4	24.1	36.2	20.6	31.0	17.0	25.5	
	17	27.3	41.1	21.3	32.1	18.3	27.4	15.1	22.6	
Y-Y Axis	0	151	227	116	174	96.9	146	71.3	107	3
	6	136	204	98.8	148	78.5	118	56.1	84.3	
	8	130	195	95.8	144	76.8	115	55.4	83.2	
	10	118	177	88.1	132	71.9	108	53.4	80.2	
	12	106	160	79.6	120	65.5	98.4	50.1	75.4	
	14	94.2	142	70.4	106	58.1	87.3	45.0	67.6	
	16	81.8	123	61.0	91.6	50.3	75.6	39.1	58.8	
	18	69.7	105	51.8	77.8	42.7	64.2	33.3	50.0	
	20	58.3	87.6	43.0	64.6	35.4	53.3	27.6	41.5	
	22	48.2	72.5	35.7	53.6	29.5	44.3	23.0	34.6	
	24	40.6	61.0	30.0	45.2	24.8	37.3	19.5	29.3	
	26	34.6	52.0	25.7	38.6	21.2	31.9	16.7	25.1	
	28	29.9	44.9	22.2	33.3	18.4	27.6	14.4	21.7	
	30	26.0	39.1	19.3	29.0	16.0	24.1	12.6	19.0	

Properties of 2 angles—3/8 in. back to back

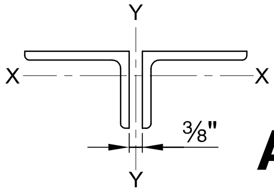
	1/2	3/8	5/16	1/4
A_g, in.^{2}	7.00	5.36	4.50	3.64
r_x, in.	1.04	1.05	1.06	1.07
r_y, in.	1.89	1.86	1.85	1.83

Properties of single angle

	1/2	3/8	5/16	1/4
r_z, in.	0.716	0.719	0.721	0.723

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.



2L4 SLBB

Table 4-10 (continued)
Available Strength in Axial Compression, kips
Double Angles—SLBB

F_y = 36 ksi

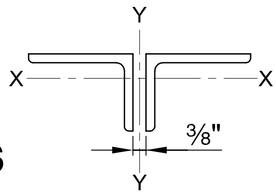
Shape		2L4×3× 5/8		1/2		3/8		5/16		1/4[c]		No. of connectors[a]
lb/ft		27.2		22.2		17.0		14.4		11.6		
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	
Effective length, L_c (ft), with respect to indicated axis: X-X Axis	0	172	259	140	211	107	161	90.0	135	67.5	102	b
	1	170	256	139	208	106	160	89.2	134	67.0	101	
	2	165	248	134	202	103	155	86.6	130	65.4	98.3	
	3	156	235	128	192	98.2	148	82.5	124	62.9	94.5	
	4	145	218	119	179	91.6	138	77.0	116	59.5	89.4	
	5	132	198	108	163	83.7	126	70.5	106	55.4	83.2	
	6	117	176	96.7	145	75.0	113	63.3	95.2	50.7	76.2	
	7	102	154	84.6	127	65.9	99.1	55.8	83.8	45.4	68.3	
	8	87.2	131	72.5	109	56.8	85.4	48.2	72.4	39.3	59.1	
	9	72.8	109	60.8	91.5	48.0	72.1	40.8	61.3	33.4	50.2	
	10	59.5	89.4	49.9	75.1	39.6	59.5	33.8	50.8	27.8	41.7	
	11	49.2	73.9	41.3	62.0	32.7	49.2	27.9	42.0	22.9	34.5	
	12	41.3	62.1	34.7	52.1	27.5	41.3	23.5	35.3	19.3	29.0	
	13	35.2	52.9	29.6	44.4	23.4	35.2	20.0	30.0	16.4	24.7	
	14	30.3	45.6	25.5	38.3	20.2	30.4	17.2	25.9	14.2	21.3	
Y-Y Axis	0	172	259	140	211	107	161	90.0	135	67.5	102	3
	6	159	239	128	192	93.1	140	74.3	112	53.3	80.0	
	8	151	227	122	183	90.9	137	73.1	110	52.8	79.4	
	10	141	212	114	171	85.8	129	68.6	103	51.2	77.0	
	12	126	190	102	153	76.8	115	62.4	93.9	48.3	72.6	
	14	113	170	90.8	136	68.5	103	55.5	83.4	43.7	65.7	
	16	99.5	150	79.7	120	60.0	90.3	48.2	72.5	38.1	57.3	
	18	86.2	129	68.7	103	51.7	77.6	41.1	61.8	32.6	48.9	
	20	73.3	110	58.2	87.5	43.6	65.6	35.8	53.8	28.3	42.6	4
	22	61.2	92.0	48.4	72.8	36.3	54.5	29.7	44.7	23.6	35.5	
	24	51.5	77.4	40.7	61.2	30.5	45.9	25.1	37.7	19.9	29.9	
	26	43.9	65.9	34.7	52.2	26.0	39.2	21.4	32.2	17.0	25.6	
	28	37.8	56.9	30.0	45.0	22.5	33.8	18.5	27.8	14.7	22.1	
	30	33.0	49.6	26.1	39.2	19.6	29.5	16.1	24.2	12.9	19.3	
	32	29.0	43.6	23.0	34.5	17.2	25.9					
Properties of 2 angles—3/8 in. back to back												
A_g, in.^{2}		7.98		6.50		4.98		4.18		3.38		
r_x, in.		0.845		0.858		0.873		0.880		0.887		
r_y, in.		1.98		1.95		1.93		1.91		1.90		
Properties of single angle												
r_z, in.		0.631		0.633		0.636		0.638		0.639		

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 36 ksi

Table 4-10 (continued)
Available Strength in Axial Compression, kips
Double Angles—SLBB

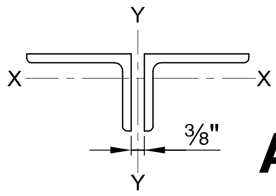


2L3½ SLBB

Shape		2L3½×3×½		2L3½×3×7/16		2L3½×3×3/8		2L3½×3×5/16		2L3½×3×1/4[c]		No. of connectors[a]
lb/ft		20.4		18.2		15.8		13.2		10.8		
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	130	196	115	173	100	150	84.1	126	66.3	99.6	b
	1	129	194	114	171	99.1	149	83.3	125	65.8	98.8	
	2	125	188	111	166	96.3	145	81.0	122	64.2	96.6	
	3	119	179	106	159	91.8	138	77.3	116	61.8	92.9	
	4	111	167	98.6	148	85.9	129	72.4	109	58.5	87.9	
	5	102	153	90.4	136	78.8	118	66.5	100	54.1	81.4	
	6	91.3	137	81.2	122	71.0	107	60.0	90.2	48.9	73.5	
	7	80.3	121	71.6	108	62.7	94.3	53.1	79.9	43.4	65.2	
	8	69.3	104	62.0	93.1	54.4	81.7	46.2	69.4	37.8	56.8	
	9	58.6	88.1	52.6	79.0	46.2	69.5	39.4	59.2	32.3	48.6	
	10	48.5	72.9	43.7	65.6	38.5	57.9	33.0	49.6	27.2	40.8	
	11	40.1	60.2	36.1	54.2	31.8	47.9	27.3	41.0	22.5	33.8	
	12	33.7	50.6	30.3	45.6	26.8	40.2	22.9	34.4	18.9	28.4	
	13	28.7	43.1	25.8	38.8	22.8	34.3	19.5	29.3	16.1	24.2	
	14	24.7	37.2	22.3	33.5	19.7	29.6	16.8	25.3	13.9	20.9	
	15							14.7	22.0	12.1	18.2	
Y-Y Axis	0	130	196	115	173	100	150	84.1	126	66.3	99.6	3
	6	117	176	102	154	87.2	131	70.7	106	53.1	79.8	
	8	109	164	95.5	144	82.1	123	67.5	101	51.6	77.5	
	10	96.1	144	84.2	127	72.5	109	60.1	90.3	47.0	70.6	
	12	84.4	127	73.8	111	63.5	95.4	52.7	79.2	41.5	62.3	
	14	72.3	109	63.0	94.7	54.2	81.5	45.0	67.6	35.4	53.2	
	16	60.6	91.0	52.6	79.0	45.2	67.9	37.4	56.2	29.4	44.2	
	18	49.4	74.3	42.7	64.2	36.6	55.0	30.3	45.5	23.8	35.7	
	20	41.7	62.6	36.0	54.1	29.7	44.7	24.6	37.0	19.4	29.1	
	22	34.5	51.8	29.8	44.7	24.6	37.0	20.4	30.7	16.1	24.2	
	24	29.0	43.6	25.0	37.6	20.7	31.1	17.2	25.8	13.6	20.4	
	26	24.7	37.1	21.4	32.1	17.7	26.6	14.7	22.0	11.6	17.4	
	28	21.3	32.0									4
Properties of 2 angles—3/8 in. back to back												
A_g, in.^{2}		6.04		5.34		4.64		3.90		3.16		
r_x, in.		0.877		0.885		0.892		0.900		0.908		
r_y, in.		1.69		1.67		1.66		1.65		1.63		
Properties of single angle												
r_z, in.		0.618		0.620		0.622		0.624		0.628		

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.



2L3½ SLBB

Table 4-10 (continued)
Available Strength in Axial Compression, kips
Double Angles—SLBB

F_y = 36 ksi

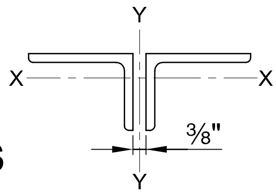
Shape		2L3½×2½×								
		½		⅜		5/16		¼[c]		No. of connectors[a]
lb/ft		18.8		14.4		12.2		9.80		
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length, L_c (ft), with respect to indicated axis										
X-X Axis	0	119	179	91.4	137	77.2	116	60.7	91.2	b
	1	118	177	90.1	135	76.1	114	60.0	90.1	
	2	112	169	86.2	129	72.8	109	57.9	87.0	
	3	104	156	80.0	120	67.7	102	54.5	82.0	
	4	93.3	140	72.1	108	61.2	92.0	49.8	74.9	
	5	81.2	122	63.2	94.9	53.7	80.7	43.8	65.9	
	6	68.5	103	53.7	80.7	45.8	68.8	37.5	56.4	
	7	56.1	84.3	44.3	66.6	37.9	57.0	31.2	46.9	
	8	44.4	66.7	35.5	53.3	30.5	45.8	25.2	37.9	
	9	35.1	52.7	28.0	42.1	24.1	36.2	20.0	30.0	
	10	28.4	42.7	22.7	34.1	19.5	29.4	16.2	24.3	
	11	23.5	35.3	18.8	28.2	16.1	24.3	13.4	20.1	
	12					13.6	20.4	11.2	16.9	
Y-Y Axis	0	119	179	91.4	137	77.2	116	60.7	91.2	4
	2	112	169	82.4	124	66.7	100	50.0	75.1	
	4	112	168	82.1	123	66.5	100	49.8	74.9	
	6	108	163	80.9	122	65.9	99.1	49.5	74.4	
	8	102	153	76.7	115	63.6	95.5	48.6	73.1	
	10	90.7	136	68.5	102	57.2	86.0	45.1	67.7	
	12	80.5	121	60.6	91.1	50.7	76.2	40.2	60.4	
	14	69.9	105	52.4	78.8	43.9	65.9	34.8	52.3	
	16	59.4	89.3	44.3	66.7	37.0	55.7	29.3	44.1	
	18	49.4	74.2	36.7	55.1	30.6	46.0	24.1	36.2	
	20	40.2	60.5	29.8	44.8	24.9	37.4	19.6	29.5	
	22	33.3	50.0	24.7	37.1	20.6	30.9	16.3	24.4	
	24	28.0	42.0	20.7	31.2	17.3	26.0	13.7	20.6	
	26	23.8	35.8	17.7	26.6	14.8	22.2	11.7	17.6	
	28	20.6	30.9	15.3	22.9	12.7	19.1	10.1	15.2	
Properties of 2 angles—⅜ in. back to back										
A_g, in.^{2}		5.54		4.24		3.58		2.90		
r_x, in.		0.701		0.716		0.723		0.731		
r_y, in.		1.76		1.73		1.72		1.70		
Properties of single angle										
r_z, in.		0.532		0.535		0.538		0.541		

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 36 ksi

Table 4-10 (continued)
Available Strength in Axial Compression, kips
Double Angles—SLBB



2L3 SLBB

Shape		2L3×2½×												No. of connectors[a]
		1/2		7/16		3/8		5/16		1/4		3/16[c]		
lb/ft		17.0		15.2		13.2		11.2		9.00		6.78		
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	108	162	95.7	144	83.2	125	70.3	106	56.9	85.5	39.7	59.6	
	1	106	160	94.3	142	82.0	123	69.3	104	56.1	84.4	39.3	59.1	
	2	102	153	90.3	136	78.6	118	66.5	99.9	53.9	81.0	38.2	57.5	
	3	94.4	142	84.0	126	73.2	110	62.0	93.2	50.3	75.7	36.3	54.6	
	4	85.2	128	75.9	114	66.3	99.7	56.3	84.6	45.8	68.8	33.6	50.4	
	5	74.6	112	66.7	100	58.4	87.7	49.7	74.7	40.5	60.8	30.3	45.6	
	6	63.5	95.4	56.9	85.5	49.9	75.0	42.6	64.1	34.9	52.4	26.6	40.0	b
	7	52.4	78.8	47.1	70.8	41.5	62.4	35.6	53.5	29.2	43.9	22.4	33.7	
	8	42.0	63.2	37.9	57.0	33.6	50.4	28.9	43.4	23.8	35.8	18.3	27.5	
	9	33.2	49.9	30.0	45.1	26.6	39.9	22.9	34.5	18.9	28.5	14.6	22.0	
	10	26.9	40.4	24.3	36.5	21.5	32.4	18.6	27.9	15.3	23.0	11.8	17.8	
	11	22.2	33.4	20.1	30.2	17.8	26.7	15.4	23.1	12.7	19.0	9.78	14.7	
	12			16.9	25.4	15.0	22.5	12.9	19.4	10.6	16.0	8.22	12.4	
Y-Y Axis	0	108	162	95.7	144	83.2	125	70.3	106	56.9	85.5	39.7	59.6	
	2	103	154	90.0	135	76.6	115	62.6	94.1	47.8	71.8	31.2	46.9	
	4	101	152	88.8	133	75.8	114	62.1	93.3	47.5	71.4	31.1	46.7	
	6	94.8	142	83.8	126	72.1	108	59.9	90.0	46.4	69.8	30.7	46.1	
	8	83.8	126	74.1	111	63.8	95.9	53.3	80.1	42.3	63.5	29.3	44.1	3
	10	73.0	110	64.4	96.8	55.3	83.1	46.2	69.5	36.8	55.3	26.5	39.9	
	12	61.6	92.5	54.2	81.5	46.4	69.7	38.7	58.2	30.9	46.4	22.4	33.7	
	14	50.4	75.7	44.3	66.5	37.7	56.7	31.4	47.2	25.0	37.6	18.1	27.3	
	16	41.6	62.5	36.4	54.8	30.8	46.3	25.6	38.5	20.3	30.6	14.2	21.4	
	18	32.9	49.4	28.8	43.3	24.4	36.7	20.3	30.5	16.1	24.3	11.8	17.7	
	20	26.7	40.1	23.4	35.1	19.8	29.7	16.5	24.7	13.1	19.7	9.59	14.4	
	22	22.0	33.1	19.3	29.0	16.4	24.6	13.6	20.5	10.9	16.3	7.96	12.0	4
	24	18.5	27.9	16.2	24.4	13.8	20.7	11.5	17.2	9.14	13.7			

Properties of 2 angles—3/8 in. back to back

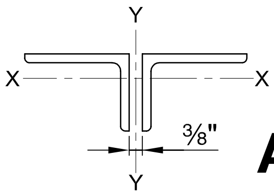
	1/2	7/16	3/8	5/16	1/4	3/16
A_g, in.^{2}	5.00	4.44	3.86	3.26	2.64	2.00
r_x, in.	0.718	0.724	0.731	0.739	0.746	0.753
r_y, in.	1.49	1.48	1.46	1.45	1.44	1.42

Properties of single angle

	1/2	7/16	3/8	5/16	1/4	3/16
r_z, in.	0.516	0.516	0.517	0.518	0.520	0.521

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.



2L3 SLBB

Table 4-10 (continued)
Available Strength in Axial Compression, kips
Double Angles—SLBB

F_y = 36 ksi

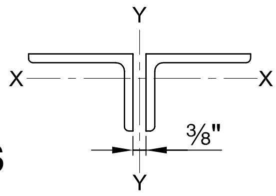
Shape		2L3×2× 1/2		3/8		5/16		1/4		3/16[c]		No. of connectors[a]
lb/ft		15.4		11.8		10.0		8.20		6.14		
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	
Effective length, L_c (ft), with respect to indicated axis: X-X Axis	0	97.4	146	75.4	113	63.8	95.9	51.7	77.8	36.4	54.8	
	1	95.0	143	73.6	111	62.3	93.6	50.5	76.0	35.8	53.8	
	2	87.9	132	68.4	103	58.0	87.1	47.1	70.8	33.8	50.9	
	3	77.3	116	60.5	90.9	51.4	77.3	41.9	63.0	30.8	46.3	
	4	64.6	97.1	50.9	76.5	43.5	65.3	35.6	53.5	27.0	40.6	
	5	51.2	77.0	40.8	61.3	35.0	52.6	28.8	43.3	22.3	33.6	b
	6	38.6	58.0	31.1	46.8	26.9	40.4	22.3	33.5	17.4	26.1	
	7	28.4	42.7	23.0	34.5	19.9	29.9	16.6	24.9	13.0	19.5	
	8	21.7	32.7	17.6	26.4	15.2	22.9	12.7	19.0	9.94	14.9	
	9	17.2	25.8	13.9	20.9	12.0	18.1	10.0	15.0	7.85	11.8	
Y-Y Axis	0	97.4	146	75.4	113	63.8	95.9	51.7	77.8	36.4	54.8	
	2	93.1	140	69.8	105	57.3	86.1	43.9	66.0	29.0	43.6	
	4	92.0	138	69.5	104	57.1	85.7	43.8	65.8	28.9	43.5	
	6	86.9	131	66.7	100	55.6	83.6	43.3	65.0	28.8	43.3	
	8	77.5	117	59.6	89.6	49.9	75.1	40.0	60.1	28.0	42.1	4
	10	68.3	103	52.3	78.7	43.8	65.8	35.2	52.8	25.8	38.8	
	12	58.5	87.9	44.7	67.1	37.3	56.0	29.9	44.9	22.2	33.4	
	14	48.6	73.1	37.0	55.6	30.8	46.2	24.6	37.0	18.3	27.5	
	16	40.7	61.1	30.8	46.3	25.5	38.3	20.4	30.6	14.5	21.9	
	18	32.4	48.6	24.4	36.7	20.2	30.3	16.1	24.2	11.9	17.9	
	20	26.2	39.4	19.8	29.8	16.3	24.6	13.1	19.7	9.69	14.6	
	22	21.7	32.6	16.4	24.6	13.5	20.3	10.8	16.3	8.03	12.1	5
	24	18.2	27.4	13.8	20.7	11.4	17.1	9.10	13.7	6.76	10.2	
	26	15.5	23.3									
Properties of 2 angles—3/8 in. back to back												
A_g, in.^{2}		4.52		3.50		2.96		2.40		1.83		
r_x, in.		0.543		0.555		0.562		0.569		0.577		
r_y, in.		1.56		1.54		1.52		1.51		1.49		
Properties of single angle												
r_z, in.		0.425		0.426		0.428		0.431		0.435		

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 36 ksi

Table 4-10 (continued)
Available Strength in Axial Compression, kips
Double Angles—SLBB

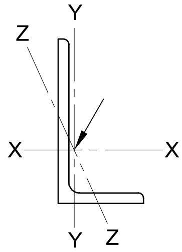


2L2$^1/_2$ SLBB

Shape		2L2$^1/_2$×2×								2L2$^1/_2$×1$^1/_2$×				No. of connectors[a]
		3/8		5/16		1/4		3/16[c]		1/4		3/16[c]		
lb/ft		10.6		9.0		7.24		5.50		6.38		4.88		
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length, L_c (ft), with respect to indicated axis — X-X Axis	0	66.8	100	56.9	85.5	46.1	69.3	34.9	52.5	40.7	61.2	30.8	46.3	
	1	65.3	98.2	55.6	83.6	45.1	67.8	34.3	51.6	38.9	58.5	29.7	44.7	
	2	61.0	91.6	52.0	78.2	42.3	63.5	32.5	48.8	34.0	51.1	26.2	39.4	
	3	54.3	81.7	46.5	69.9	37.9	57.0	29.2	43.9	27.1	40.7	21.1	31.7	
	4	46.2	69.5	39.7	59.7	32.5	48.9	25.2	37.8	19.7	29.7	15.5	23.3	
	5	37.6	56.5	32.5	48.8	26.7	40.2	20.8	31.2	13.2	19.8	10.5	15.7	
	6	29.2	43.9	25.4	38.1	21.0	31.6	16.4	24.7	9.17	13.8	7.28	10.9	b
	7	21.8	32.7	19.0	28.5	15.8	23.8	12.5	18.7					
	8	16.7	25.0	14.5	21.8	12.1	18.2	9.53	14.3					
	9	13.2	19.8	11.5	17.3	9.57	14.4	7.53	11.3					
Y-Y Axis	0	66.8	100	56.9	85.5	46.1	69.3	34.9	52.5	40.7	61.2	30.8	46.3	
	2	62.9	94.5	52.3	78.6	40.6	61.1	28.3	42.6	36.1	54.3	25.5	38.3	
	4	61.3	92.1	51.4	77.2	40.2	60.4	28.1	42.3	36.0	54.1	25.4	38.2	
	6	56.1	84.3	46.3	69.6	36.9	55.5	26.9	40.4	34.4	51.7	25.0	37.6	5
	8	47.9	72.0	39.8	59.9	31.8	47.8	23.7	35.6	29.6	44.5	22.4	33.6	
	10	39.8	59.8	32.8	49.2	26.0	39.1	19.5	29.3	24.9	37.4	18.8	28.3	
	12	31.8	47.7	25.8	38.8	20.4	30.6	15.3	22.9	20.0	30.1	15.1	22.7	
	14	24.3	36.5	20.3	30.6	16.0	24.0	11.9	18.0	15.5	23.3	11.7	17.5	
	16	18.6	28.0	15.6	23.4	12.3	18.4	9.20	13.8	12.2	18.4	9.20	13.8	
	18	14.7	22.1	12.3	18.5	9.70	14.6	7.29	11.0	9.67	14.5	7.28	10.9	
	20	11.9	17.9	10.0	15.0	7.87	11.8	5.92	8.90	7.84	11.8	5.90	8.87	6
Properties of 2 angles—3/8 in. back to back														
A_g, in.^{2}		3.10		2.64		2.14		1.64		1.89		1.45		
r_x, in.		0.574		0.581		0.589		0.597		0.409		0.416		
r_y, in.		1.27		1.26		1.24		1.23		1.32		1.30		
Properties of single angle														
r_z, in.		0.419		0.420		0.423		0.426		0.321		0.324		

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors with Class A or B faying surfaces must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r equal to or greater than 200.



L12

Table 4-11
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles

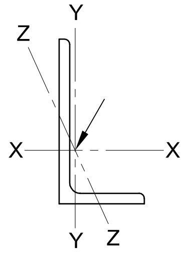
F_y = 36 ksi

Shape	L12×12× 1 3/8		L12×12× 1 1/4		L12×12× 1 1/8		L12×12× 1	
lb/ft	105		96.4		87.2		77.8	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z								
0	670	1010	612	920	556	836	496	745
1	669	1010	611	919	555	835	495	744
2	667	1000	609	915	553	831	493	741
3	662	995	604	908	549	825	490	736
4	655	985	598	899	544	817	485	729
5	647	972	591	888	537	807	479	720
6	637	957	582	874	529	795	472	709
7	625	939	571	858	519	781	463	696
8	612	919	559	840	509	764	454	682
9	597	897	546	820	497	747	443	666
10	581	873	531	798	484	727	432	649
11	564	847	516	775	470	706	419	630
12	545	820	499	750	455	684	406	611
13	526	791	482	724	439	660	392	590
14	506	761	463	697	423	636	378	568
15	486	730	445	668	406	611	363	546
16	465	698	426	640	389	585	348	523
17	443	666	406	610	371	558	332	499
18	421	633	386	581	354	532	317	476
19	400	601	367	551	336	505	301	452
20	378	568	347	521	318	478	285	428
21	356	536	327	492	300	452	269	405
22	335	504	308	463	283	425	254	381
23	314	472	289	434	266	399	238	358
24	294	441	270	406	249	374	223	336
25	274	411	252	379	232	349	209	314
26	254	382	234	352	216	325	194	292
27	236	354	217	326	201	301	180	271
28	219	329	202	303	186	280	168	252
Properties								
A_g, in.^{2}	31.1		28.4		25.8		23.0	
r_z, in.	2.30		2.31		2.33		2.34	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

F_y = 36 ksi

Table 4-11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles

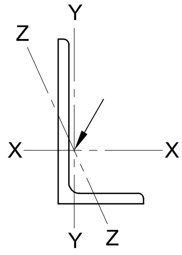


L10

Shape	L10×10× 1 3/8		1 1/4		1 1/8		1		7/8		3/4 c	
lb/ft	87.1		79.9		72.3		64.7		56.9		49.1	
Design	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z												
0	552	829	504	758	459	690	410	616	362	544	306	460
1	551	828	503	757	458	689	409	614	361	543	305	459
2	547	823	500	752	455	684	406	611	359	540	304	457
3	542	814	495	744	451	677	402	604	356	534	302	454
4	534	802	488	733	444	668	396	596	351	527	299	449
5	524	787	479	720	436	656	389	585	344	517	295	443
6	512	770	468	704	426	641	380	572	337	506	290	436
7	498	749	456	685	415	624	370	557	328	493	284	427
8	483	726	442	664	403	605	359	540	318	478	275	414
9	466	701	426	641	389	584	347	521	307	462	266	400
10	448	674	410	616	374	562	333	501	295	444	257	386
11	429	645	392	590	358	538	319	480	283	426	246	370
12	409	615	374	562	341	513	305	458	270	406	235	354
13	388	584	355	534	324	488	289	435	257	386	224	337
14	367	552	336	505	307	461	274	411	243	365	212	319
15	346	520	316	475	289	434	258	388	229	344	201	301
16	324	487	296	445	271	408	242	364	215	323	189	283
17	303	455	277	416	253	381	226	340	201	302	177	266
18	281	423	257	387	236	354	210	316	187	282	165	248
19	261	392	238	358	219	328	195	293	174	261	153	230
20	240	361	220	330	202	303	180	270	160	241	142	213
21	221	332	202	303	185	279	165	249	148	222	131	197
22	201	303	184	277	169	255	151	227	135	203	120	181
23	184	277	168	253	155	233	138	208	123	186	110	165
24	169	254	155	233	142	214	127	191	113	170	101	152
25	156	234	143	214	131	197	117	176	105	157	93.0	140
26	144	217	132	198	121	182	108	163	96.6	145	86.0	129
27	134	201	122	184	112	169	100	151	89.6	135	79.8	120
28	124	187	114	171	105	157	93.3	140	83.3	125	74.2	111
Properties												
A_g, in.^{2}	25.6		23.4		21.3		19.0		16.8		14.5	
r_z, in.	1.91		1.91		1.92		1.92		1.93		1.96	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.



L8

Table 4-11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles

F_y = 36 ksi

Shape	L8×8×											
	1 1/8		1		7/8		3/4		5/8		9/16[c]	
lb/ft	56.9		51.0		45.0		38.9		32.7		29.6	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z												
0	362	544	326	489	287	431	248	373	208	313	178	268
1	361	543	324	488	286	430	247	371	208	312	178	267
2	358	538	321	483	283	426	245	368	206	309	177	265
3	352	529	317	476	279	419	241	362	203	305	175	263
4	345	518	310	465	273	410	236	355	198	298	172	259
5	335	504	301	453	265	399	230	345	193	290	169	254
6	324	487	291	437	257	386	222	334	187	281	165	248
7	311	467	279	420	247	371	213	320	180	270	160	241
8	297	446	267	401	235	354	204	306	172	258	155	233
9	281	423	253	380	223	336	193	290	163	245	148	222
10	265	399	238	358	211	317	182	274	154	231	140	210
11	248	373	223	336	198	297	171	257	144	217	131	197
12	231	348	208	312	184	277	159	239	135	202	122	183
13	214	322	192	289	170	256	147	222	125	188	113	170
14	197	296	177	266	157	236	136	204	115	173	104	157
15	180	270	161	243	144	216	124	187	105	158	95.5	143
16	163	245	147	220	130	196	113	170	95.9	144	86.9	131
17	147	221	132	199	118	177	102	153	86.8	130	78.6	118
18	132	198	118	178	106	159	91.3	137	77.9	117	70.5	106
19	118	178	106	160	94.8	142	82.0	123	69.9	105	63.3	95.1
20	107	160	95.9	144	85.5	129	74.0	111	63.1	94.9	57.1	85.9
21	96.8	145	87.0	131	77.6	117	67.1	101	57.3	86.1	51.8	77.9
22	88.2	133	79.2	119	70.7	106	61.1	91.9	52.2	78.4	47.2	71.0
23	80.7	121	72.5	109	64.7	97.2	55.9	84.1	47.7	71.7	43.2	64.9
24	74.1	111	66.6	100	59.4	89.3	51.4	77.2	43.8	65.9	39.7	59.6
25	68.3	103	61.4	92.2	54.8	82.3	47.3	71.2	40.4	60.7	36.6	55.0
26	63.1	94.9	56.7	85.3	50.6	76.1	43.8	65.8	37.4	56.1	33.8	50.8
Properties												
A_g, in.^{2}	16.8		15.1		13.3		11.5		9.69		8.77	
r_z, in.	1.56		1.56		1.57		1.57		1.58		1.58	

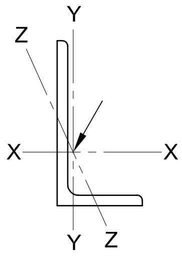
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_z equal to or greater than 200.

F_y = 36 ksi

Table 4-11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles



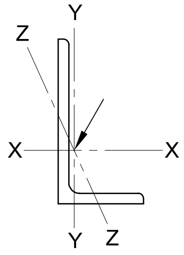
L8

Shape	L8×8×		L8×6×									
	1/2[c]		1		7/8		3/4		5/8		9/16[c]	
lb/ft	26.4		44.2		39.1		33.8		28.5		25.7	
Design	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z: 0	148	222	282	424	248	373	215	324	181	272	159	238
1	147	222	281	422	247	371	214	322	180	270	158	237
2	147	220	277	417	243	366	211	318	177	267	156	235
3	145	218	271	407	238	357	207	311	174	261	153	230
4	143	215	262	394	230	346	200	301	168	253	149	224
5	140	211	252	378	221	332	192	289	161	243	144	217
6	137	206	239	359	210	315	183	275	153	231	139	208
7	134	201	225	338	198	297	172	259	145	217	132	198
8	130	195	210	316	184	277	161	242	135	203	123	185
9	125	188	194	292	170	256	149	224	125	188	114	171
10	120	181	178	267	156	235	137	205	115	172	105	157
11	115	173	161	242	142	213	124	187	104	157	95.3	143
12	109	164	145	218	127	191	112	168	94.0	141	86.0	129
13	102	153	129	194	113	170	99.7	150	83.9	126	76.9	116
14	93.9	141	114	171	100	150	88.2	133	74.2	112	68.1	102
15	86.1	129	99.6	150	87.4	131	77.1	116	64.9	97.6	59.7	89.7
16	78.4	118	87.5	132	76.8	115	67.8	102	57.1	85.8	52.4	78.8
17	71.0	107	77.5	117	68.1	102	60.0	90.2	50.5	76.0	46.5	69.8
18	63.9	96.0	69.1	104	60.7	91.2	53.6	80.5	45.1	67.8	41.4	62.3
19	57.3	86.1	62.1	93.3	54.5	81.9	48.1	72.2	40.5	60.8	37.2	55.9
20	51.7	77.7	56.0	84.2	49.2	73.9	43.4	65.2	36.5	54.9	33.6	50.4
21	46.9	70.5	50.8	76.4	44.6	67.0	39.3	59.1	33.1	49.8	30.4	45.8
22	42.7	64.2										
23	39.1	58.8										
24	35.9	54.0										
25	33.1	49.8										
26	30.6	46.0										
Properties												
A_g, in.^{2}	7.84		13.1		11.5		9.99		8.41		7.61	
r_z, in.	1.59		1.28		1.28		1.29		1.29		1.30	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_z equal to or greater than 200.



L8

Table 4-11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles

F_y = 36 ksi

Shape	L8×6×				L8×4×							
	1/2[c]		7/16[c]		1		7/8		3/4		5/8	
lb/ft	23.0		20.2		37.4		33.1		28.7		24.2	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z												
0	136	204	112	168	239	360	211	317	183	275	154	231
1	135	204	112	168	237	356	209	314	181	272	152	229
2	134	201	111	166	229	345	202	304	175	264	148	222
3	132	198	109	164	217	327	192	288	167	250	140	211
4	128	193	107	160	202	303	178	268	155	233	130	196
5	124	186	104	156	183	276	162	243	141	212	119	179
6	119	179	100	151	163	245	144	217	125	189	106	160
7	113	170	95.6	144	142	214	126	189	109	165	92.8	140
8	107	161	90.5	136	121	182	107	161	93.5	141	79.5	120
9	101	151	85.1	128	101	152	89.5	135	78.2	118	66.7	100
10	93.6	141	79.4	119	82.5	124	73.1	110	64.0	96.2	54.8	82.3
11	85.2	128	73.5	110	68.2	103	60.4	90.8	52.9	79.5	45.3	68.0
12	76.8	115	67.5	101	57.3	86.1	50.8	76.3	44.5	66.8	38.0	57.2
13	68.7	103	61.2	92.0	48.8	73.4	43.3	65.0	37.9	56.9	32.4	48.7
14	60.9	91.5	54.3	81.6	42.1	63.3	37.3	56.1	32.7	49.1	27.9	42.0
15	53.3	80.1	47.7	71.7								
16	46.9	70.4	41.9	63.0								
17	41.5	62.4	37.1	55.8								
18	37.0	55.6	33.1	49.8								
19	33.2	49.9	29.7	44.7								
20	30.0	45.1	26.8	40.3								
21	27.2	40.9	24.3	36.6								
Properties												
A_g, in.^{2}	6.80		5.99		11.1		9.79		8.49		7.16	
r_z, in.	1.30		1.31		0.844		0.846		0.850		0.856	

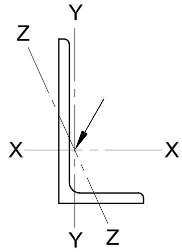
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_z equal to or greater than 200.

F_y = 36 ksi

Table 4-11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles



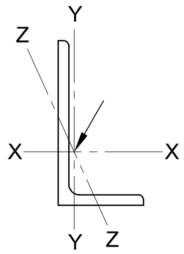
L8–L7

Shape	L8×4×						L7×4×					
	9/16[c]		1/2[c]		7/16[c]		3/4		5/8		1/2[c]	
lb/ft	21.9		19.6		17.2		26.2		22.1		17.9	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z												
0	134	202	114	172	95.1	143	167	251	140	211	110	165
1	133	200	113	171	94.3	142	165	248	139	208	109	164
2	130	195	111	166	92.1	138	160	241	134	202	106	160
3	125	188	106	160	88.6	133	152	228	128	192	102	153
4	118	177	101	151	83.8	126	141	212	119	179	96.1	144
5	108	163	93.4	140	78.1	117	129	194	108	163	88.1	132
6	96.7	145	85.4	128	71.5	108	115	173	96.9	146	78.8	118
7	84.6	127	75.9	114	64.4	96.8	100	151	84.8	127	69.1	104
8	72.5	109	65.2	98.0	57.0	85.7	85.9	129	72.7	109	59.4	89.2
9	60.9	91.5	54.8	82.4	48.7	73.1	72.0	108	61.1	91.8	50.0	75.2
10	50.0	75.1	45.1	67.8	40.1	60.3	59.1	88.8	50.2	75.4	41.2	61.9
11	41.3	62.1	37.3	56.0	33.1	49.8	48.8	73.4	41.5	62.3	34.0	51.1
12	34.7	52.2	31.3	47.1	27.8	41.8	41.0	61.6	34.8	52.4	28.6	43.0
13	29.6	44.5	26.7	40.1	23.7	35.7	34.9	52.5	29.7	44.6	24.4	36.6
14	25.5	38.3	23.0	34.6	20.5	30.7	30.1	45.3	25.6	38.5	21.0	31.6
Properties												
A_g, in.^{2}	6.49		5.80		5.11		7.74		6.50		5.26	
r_z, in.	0.859		0.863		0.867		0.855		0.860		0.866	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_z equal to or greater than 200.



L7–L6

Table 4-11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles

F_y = 36 ksi

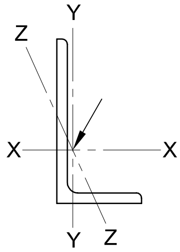
Shape	L7×4×				L6×6×							
	$^7/_{16}$[c]		$^3/_8$[c]		1		$^7/_8$		$^3/_4$		$^5/_8$	
lb/ft	15.7		13.6		37.4		33.1		28.7		24.2	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z												
0	91.6	138	74.2	112	237	356	210	316	182	274	154	231
1	90.9	137	73.7	111	236	354	209	314	181	273	153	230
2	88.8	133	71.9	108	232	349	206	309	178	268	150	226
3	85.3	128	69.2	104	226	339	200	301	174	261	146	220
4	80.6	121	65.5	98.4	217	326	192	289	167	251	141	211
5	74.9	113	61.0	91.7	206	310	183	275	159	239	134	201
6	68.5	103	55.9	84.0	194	292	172	259	149	225	126	189
7	61.0	91.7	50.4	75.7	181	272	160	241	139	209	117	176
8	52.5	78.9	44.6	67.1	166	250	147	222	128	192	108	162
9	44.3	66.5	38.5	57.9	151	228	134	202	116	175	98.1	148
10	36.5	54.9	31.8	47.8	136	205	121	182	105	158	88.3	133
11	30.2	45.3	26.3	39.5	121	182	108	162	93.3	140	78.6	118
12	25.3	38.1	22.1	33.2	107	161	94.7	142	82.2	123	69.2	104
13	21.6	32.5	18.8	28.3	93.0	140	82.4	124	71.5	108	60.3	90.6
14	18.6	28.0	16.2	24.4	80.2	121	71.1	107	61.7	92.7	52.0	78.1
15					69.9	105	61.9	93.1	53.7	80.7	45.3	68.1
16					61.4	92.3	54.4	81.8	47.2	71.0	39.8	59.8
17					54.4	81.7	48.2	72.5	41.8	62.9	35.3	53.0
18					48.5	72.9	43.0	64.6	37.3	56.1	31.4	47.3
19					43.5	65.4	38.6	58.0	33.5	50.3	28.2	42.4
Properties												
A_g, in.^{2}	4.63		4.00		11.0		9.75		8.46		7.13	
r_z, in.	0.869		0.873		1.17		1.17		1.17		1.17	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r_z equal to or greater than 200.

F_y = 36 ksi

Table 4-11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles

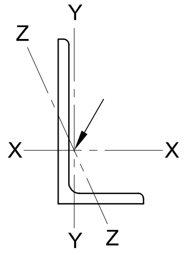


L6

Shape		L6×6×										L6×4×	
		9/16		1/2		7/16[c]		3/8[c]		5/16[c]		7/8	
lb/ft		21.9		19.6		17.2		14.9		12.4		27.2	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z	0	139	209	124	187	105	158	82.4	124	60.8	91.3	172	259
	1	138	208	124	186	105	158	82.1	123	60.6	91.0	171	257
	2	136	204	122	183	104	156	81.3	122	59.9	90.1	165	249
	3	132	199	118	178	102	153	79.8	120	58.9	88.6	157	236
	4	127	192	114	171	98.9	149	77.8	117	57.6	86.5	146	219
	5	121	182	109	163	95.5	143	75.3	113	55.8	83.9	133	200
	6	114	172	102	154	90.0	135	72.3	109	53.7	80.8	119	178
	7	106	160	95.3	143	83.9	126	68.9	104	51.4	77.2	104	156
	8	98.1	147	87.8	132	77.3	116	65.1	97.8	48.7	73.2	88.7	133
	9	89.5	134	80.0	120	70.5	106	60.9	91.6	45.8	68.9	74.3	112
	10	80.7	121	72.2	108	63.5	95.5	55.3	83.1	42.7	64.2	60.9	91.5
	11	72.0	108	64.4	96.7	56.7	85.2	49.4	74.3	39.5	59.4	50.3	75.6
	12	63.5	95.4	56.8	85.4	50.0	75.1	43.7	65.6	36.2	54.4	42.3	63.6
	13	55.4	83.3	49.6	74.5	43.6	65.6	38.2	57.4	32.0	48.1	36.0	54.2
	14	47.8	71.9	42.8	64.3	37.7	56.6	33.0	49.6	27.7	41.6	31.1	46.7
	15	41.7	62.6	37.3	56.0	32.8	49.3	28.8	43.2	24.1	36.2		
	16	36.6	55.0	32.8	49.2	28.8	43.3	25.3	38.0	21.2	31.8		
	17	32.4	48.8	29.0	43.6	25.5	38.4	22.4	33.7	18.8	28.2		
	18	28.9	43.5	25.9	38.9	22.8	34.2	20.0	30.0	16.7	25.2		
	19	26.0	39.0	23.2	34.9	20.5	30.7	17.9	27.0	15.0	22.6		
Properties													
A_g, in.^{2}		6.45		5.77		5.08		4.38		3.67		8.00	
r_z, in.		1.18		1.18		1.18		1.19		1.19		0.854	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r_z equal to or greater than 200.



L6

Table 4-11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles

F_y = 36 ksi

Shape	L6×4×									
	3/4		5/8		9/16		1/2		7/16[c]	
lb/ft	23.6		20.0		18.1		16.2		14.3	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z: 0	150	225	126	190	114	172	102	154	88.0	132
1	148	223	125	188	113	170	101	152	87.3	131
2	144	216	121	182	110	165	98.3	148	85.1	128
3	136	205	115	173	104	157	93.5	140	81.6	123
4	127	191	107	161	97.2	146	87.0	131	76.7	115
5	116	174	97.7	147	88.6	133	79.4	119	70.0	105
6	103	155	87.3	131	79.2	119	71.0	107	62.7	94.2
7	90.1	135	76.4	115	69.4	104	62.3	93.6	55.0	82.6
8	77.2	116	65.5	98.4	59.5	89.4	53.5	80.3	47.3	71.0
9	64.7	97.3	55.0	82.6	50.0	75.1	45.0	67.6	39.8	59.8
10	53.1	79.8	45.1	67.8	41.1	61.8	37.0	55.6	32.8	49.3
11	43.9	65.9	37.3	56.1	34.0	51.0	30.6	46.0	27.1	40.7
12	36.9	55.4	31.3	47.1	28.5	42.9	25.7	38.6	22.8	34.2
13	31.4	47.2	26.7	40.1	24.3	36.5	21.9	32.9	19.4	29.2
14	27.1	40.7	23.0	34.6	21.0	31.5	18.9	28.4	16.7	25.1

Properties

	3/4	5/8	9/16	1/2	7/16
A_g, in.^{2}	6.94	5.86	5.31	4.75	4.18
r_z, in.	0.856	0.859	0.861	0.864	0.867

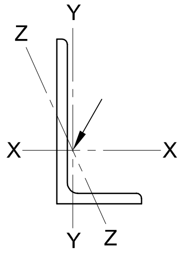
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_z equal to or greater than 200.

F_y = 36 ksi

Table 4-11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles



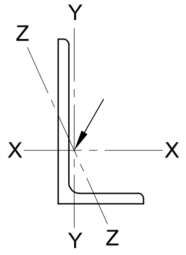
L6

Shape	L6×4×				L6×3½×					
	3/8[c]		5/16[c]		1/2		3/8[c]		5/16[c]	
lb/ft	12.3		10.3		15.3		11.7		9.80	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z										
0	71.8	108	56.1	84.3	97.0	146	68.2	102	53.1	79.8
1	71.2	107	55.7	83.7	95.7	144	67.5	101	52.6	79.0
2	69.5	105	54.4	81.7	92.0	138	65.4	98.3	51.0	76.7
3	66.8	100	52.3	78.6	86.1	129	62.1	93.3	48.5	72.9
4	63.1	94.8	49.4	74.3	78.5	118	57.6	86.6	45.1	67.8
5	58.6	88.0	46.0	69.2	69.6	105	52.4	78.7	41.2	61.9
6	53.5	80.4	42.1	63.3	60.2	90.4	46.4	69.7	36.7	55.2
7	47.6	71.6	37.9	57.0	50.6	76.1	39.2	58.9	32.1	48.2
8	41.0	61.6	33.6	50.5	41.5	62.4	32.2	48.4	27.3	41.0
9	34.6	52.0	29.2	43.9	33.1	49.8	25.8	38.8	21.9	32.9
10	28.5	42.9	24.2	36.3	26.8	40.3	20.9	31.4	17.7	26.7
11	23.6	35.4	20.0	30.0	22.2	33.3	17.3	26.0	14.7	22.0
12	19.8	29.8	16.8	25.2	18.6	28.0	14.5	21.8	12.3	18.5
13	16.9	25.4	14.3	21.5						
14	14.6	21.9	12.3	18.5						
Properties										
A_g, in.^{2}	3.61		3.03		4.50		3.44		2.89	
r_z, in.	0.870		0.874		0.756		0.763		0.767	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_z equal to or greater than 200.



L5

Table 4-11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles

F_y = 36 ksi

Shape	L5×5× 7/8		3/4		5/8		1/2		7/16		3/8[c]	
lb/ft	27.2		23.6		20.0		16.2		14.3		12.3	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z												
0	172	259	150	226	127	191	103	155	91.0	137	77.0	116
1	171	257	149	224	126	190	102	154	90.3	136	76.5	115
2	167	251	146	219	123	185	100	150	88.2	133	75.3	113
3	160	241	140	210	118	178	96.2	145	84.8	127	73.1	110
4	152	228	132	199	112	168	91.0	137	80.2	121	69.5	104
5	141	212	123	185	104	157	84.8	127	74.8	112	64.7	97.3
6	129	194	113	169	95.4	143	77.7	117	68.6	103	59.4	89.3
7	116	175	102	153	86.0	129	70.1	105	61.9	93.1	53.7	80.7
8	103	155	90.0	135	76.3	115	62.3	93.6	55.1	82.8	47.8	71.8
9	89.9	135	78.6	118	66.7	100	54.5	81.9	48.2	72.4	41.8	62.9
10	77.2	116	67.4	101	57.3	86.1	46.9	70.5	41.5	62.4	36.1	54.2
11	65.1	97.8	56.9	85.5	48.4	72.7	39.7	59.6	35.2	52.9	30.6	46.0
12	54.7	82.2	47.8	71.8	40.7	61.1	33.3	50.1	29.6	44.4	25.7	38.7
13	46.6	70.0	40.7	61.2	34.6	52.1	28.4	42.7	25.2	37.9	21.9	32.9
14	40.2	60.4	35.1	52.8	29.9	44.9	24.5	36.8	21.7	32.6	18.9	28.4
15	35.0	52.6	30.6	46.0	26.0	39.1	21.3	32.1	18.9	28.4	16.5	24.7
16	30.8	46.2	26.9	40.4	22.9	34.4	18.8	28.2	16.6	25.0	14.5	21.7
Properties												
A_g, in.^{2}	8.00		6.98		5.90		4.79		4.22		3.65	
r_z, in.	0.971		0.972		0.975		0.980		0.983		0.986	

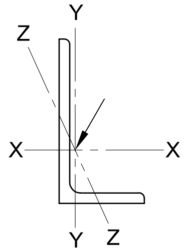
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_z equal to or greater than 200.

F_y = 36 ksi

Table 4-11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles



L5

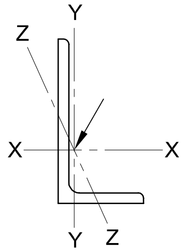
Shape		L5×5×		L5×3½×									
		5/16[c]		3/4		5/8		1/2		3/8[c]		5/16[c]	
lb/ft		10.3		19.8		16.8		13.6		10.4		8.70	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z	0	57.8	86.9	126	190	106	160	86.2	130	64.9	97.5	51.0	76.7
	1	57.5	86.5	124	187	105	158	85.1	128	64.2	96.5	50.5	75.9
	2	56.7	85.1	119	179	101	151	81.7	123	62.1	93.3	48.9	73.5
	3	55.2	83.0	111	168	94.0	141	76.4	115	58.3	87.7	46.3	69.6
	4	53.2	79.9	101	152	85.5	128	69.5	104	53.1	79.9	42.9	64.5
	5	50.7	76.2	89.5	135	75.6	114	61.6	92.5	47.2	70.9	38.9	58.5
	6	47.8	71.8	77.0	116	65.1	97.8	53.1	79.8	40.7	61.2	34.3	51.6
	7	44.4	66.8	64.5	96.9	54.5	81.9	44.6	67.0	34.3	51.5	26.9	43.5
	8	40.3	60.6	52.5	78.9	44.4	66.8	36.4	54.7	28.1	42.2	23.7	35.7
	9	35.4	53.2	41.7	62.7	35.4	53.1	29.0	43.6	22.4	33.7	19.0	28.5
	10	30.5	45.9	33.8	50.8	28.6	43.0	23.5	35.3	18.1	27.3	15.4	23.1
	11	26.0	39.0	27.9	42.0	23.7	35.6	19.4	29.2	15.0	22.5	12.7	19.1
	12	21.8	32.8	23.5	35.3	19.9	29.9	16.3	24.5	12.6	18.9	10.7	16.0
	13	18.6	27.9										
	14	16.0	24.1										
	15	14.0	21.0										
	16	12.3	18.4										

Properties

	L5×5×5/16	L5×3½×3/4	L5×3½×5/8	L5×3½×1/2	L5×3½×3/8	L5×3½×5/16
A_g, in.^{2}	3.07	5.85	4.93	4.00	3.05	2.56
r_z, in.	0.990	0.744	0.746	0.750	0.755	0.758

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r_z equal to or greater than 200.



L5

Table 4-11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles

F_y = 36 ksi

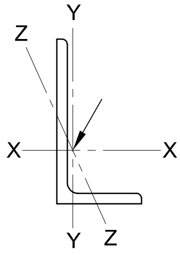
Shape		L5×3½×¼[c]		L5×3×½		L5×3×7/16		L5×3×⅜[c]		L5×3×5/16[c]		L5×3×¼[c]	
lb/ft		7.00		12.8		11.3		9.80		8.20		6.60	
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z	0	37.0	55.6	80.8	122	71.4	107	60.8	91.4	47.8	71.8	35.1	52.8
	1	36.7	55.1	79.4	119	70.1	105	59.9	90.0	47.1	70.8	34.6	52.0
	2	35.7	53.7	75.1	113	66.3	99.7	57.3	86.0	45.1	67.8	33.2	49.9
	3	34.2	51.4	68.5	103	60.5	91.0	52.4	78.7	41.9	63.0	30.9	46.5
	4	32.0	48.1	60.2	90.5	53.3	80.0	46.1	69.3	37.9	56.9	28.0	42.1
	5	29.1	43.8	51.0	76.7	45.2	67.9	39.1	58.8	33.1	49.8	24.6	37.0
	6	25.9	38.9	41.7	62.7	37.0	55.5	32.1	48.2	27.2	40.8	21.0	31.6
	7	22.5	33.8	32.8	49.3	29.1	43.8	25.3	38.0	21.5	32.3	17.4	26.1
	8	19.1	28.7	25.2	37.9	22.4	33.7	19.5	29.3	16.6	24.9	13.5	20.2
	9	15.4	23.2	19.9	29.9	17.7	26.6	15.4	23.1	13.1	19.7	10.6	16.0
	10	12.5	18.8	16.1	24.2	14.3	21.5	12.5	18.7	10.6	15.9	8.61	12.9
	11	10.3	15.5										
	12	8.69	13.1										
Properties													
A_g, in.²		2.07		3.75		3.31		2.86		2.41		1.94	
r_z, in.		0.761		0.642		0.644		0.646		0.649		0.652	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r_z equal to or greater than 200.

F_y = 36 ksi

Table 4-11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles

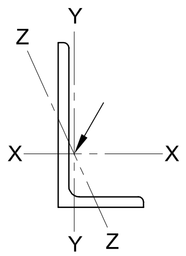


L4

Shape		L4×4×											
		3/4		5/8		1/2		7/16		3/8		5/16	
lb/ft		18.5		15.7		12.8		11.3		9.80		8.20	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z	0	117	176	99.4	149	80.8	122	71.1	107	61.7	92.7	51.6	77.5
	1	116	174	98.1	147	79.8	120	70.3	106	60.9	91.5	50.9	76.6
	2	111	168	94.5	142	76.9	116	67.7	102	58.6	88.1	49.1	73.8
	3	105	157	88.7	133	72.2	108	63.5	95.5	55.1	82.8	46.1	69.3
	4	95.8	144	81.2	122	66.1	99.3	58.2	87.5	50.5	75.9	42.3	63.6
	5	85.5	128	72.4	109	59.0	88.7	52.0	78.1	45.1	67.8	37.8	56.9
	6	74.4	112	63.0	94.7	51.4	77.2	45.3	68.0	39.3	59.1	33.0	49.6
	7	63.1	94.8	53.5	80.3	43.6	65.6	38.4	57.8	33.4	50.2	28.1	42.2
	8	52.2	78.4	44.2	66.5	36.1	54.3	31.8	47.9	27.7	41.7	23.3	35.1
	9	42.0	63.1	35.6	53.5	29.1	43.7	25.7	38.6	22.4	33.6	18.9	28.4
	10	34.0	51.1	28.8	43.3	23.6	35.4	20.8	31.3	18.1	27.2	15.3	23.0
	11	28.1	42.3	23.8	35.8	19.5	29.3	17.2	25.8	15.0	22.5	12.6	19.0
	12	23.6	35.5	20.0	30.1	16.4	24.6	14.4	21.7	12.6	18.9	10.6	15.9
	13											9.04	13.6
Properties													
A_g, in.^{2}		5.44		4.61		3.75		3.30		2.86		2.40	
r_z, in.		0.774		0.774		0.776		0.777		0.779		0.781	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates L_c/r_z equal to or greater than 200.



L4

Table 4-11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles

F_y = 36 ksi

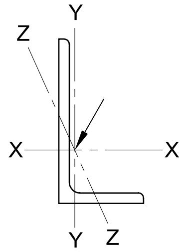
Shape	L4×4×		L4×3½×								L4×3×	
	¼[c]		½		⅜		5/16		¼[c]		⅝	
lb/ft	6.60		11.9		9.10		7.70		6.20		13.6	
Design	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z												
0	36.3	54.5	75.4	113	57.8	86.8	48.4	72.7	35.6	53.6	86.0	129
1	36.0	54.1	74.3	112	56.9	85.6	47.7	71.6	35.3	53.0	84.4	127
2	35.1	52.8	71.1	107	54.5	81.9	45.6	68.6	34.3	51.5	79.7	120
3	33.7	50.6	66.0	99.3	50.6	76.1	42.4	63.8	32.6	48.9	72.5	109
4	31.7	47.7	59.6	89.5	45.7	68.7	38.3	57.6	30.1	45.3	63.4	95.3
5	29.4	44.1	52.1	78.4	40.0	60.2	33.6	50.5	27.0	40.5	53.4	80.3
6	26.6	40.0	44.3	66.6	34.1	51.2	28.7	43.1	23.3	35.0	43.3	65.1
7	22.7	34.1	36.6	54.9	28.2	42.3	23.7	35.6	19.3	29.0	33.8	50.9
8	18.9	28.3	29.3	44.0	22.6	34.0	19.1	28.7	15.5	23.3	25.9	38.9
9	15.2	22.9	23.1	34.8	17.9	26.8	15.1	22.7	12.3	18.4	20.5	30.8
10	12.4	18.6	18.7	28.1	14.5	21.7	12.2	18.3	9.93	14.9	16.6	24.9
11	10.2	15.3	15.5	23.3	12.0	18.0	10.1	15.2	8.21	12.3		
12	8.58	12.9					8.48	12.7	6.90	10.4		
13	7.31	11.0										
Properties												
A_g, in.²	1.93		3.50		2.68		2.25		1.82		3.99	
r_z, in.	0.783		0.716		0.719		0.721		0.723		0.631	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r_z equal to or greater than 200.

F_y = 36 ksi

Table 4-11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles



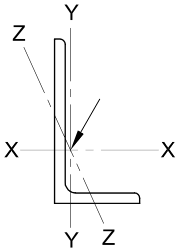
L4–L3½

Shape	L4×3×								L3½×3½×			
	1/2		3/8		5/16		1/4[c]		1/2		7/16	
lb/ft	11.1		8.50		7.20		5.80		11.1		9.80	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z												
0	70.1	105	53.7	80.7	44.9	67.5	33.8	50.8	70.1	105	62.3	93.6
1	68.7	103	52.7	79.2	44.1	66.3	33.3	50.0	68.9	104	61.3	92.1
2	65.0	97.6	49.8	74.8	41.7	62.7	31.8	47.8	65.6	98.6	58.4	87.7
3	59.1	88.8	45.3	68.2	38.0	57.1	29.4	44.2	60.4	90.8	53.8	80.8
4	51.8	77.8	39.8	59.8	33.4	50.2	26.4	39.7	53.9	80.9	48.0	72.1
5	43.7	65.6	33.6	50.5	28.2	42.4	22.9	34.4	46.4	69.8	41.4	62.2
6	35.5	53.3	27.3	41.1	23.0	34.6	18.7	28.1	38.8	58.3	34.6	52.0
7	27.7	41.7	21.4	32.2	18.1	27.2	14.7	22.0	31.3	47.0	28.0	42.0
8	21.2	31.9	16.4	24.7	13.9	20.9	11.3	16.9	24.4	36.7	21.9	32.9
9	16.8	25.2	13.0	19.5	11.0	16.5	8.89	13.4	19.3	29.0	17.3	26.0
10	13.6	20.4	10.5	15.8	8.88	13.3	7.20	10.8	15.6	23.5	14.0	21.0
11									12.9	19.4	11.6	17.4
Properties												
A_g, in.^{2}	3.25		2.49		2.09		1.69		3.25		2.89	
r_z, in.	0.633		0.636		0.638		0.639		0.679		0.681	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_z equal to or greater than 200.



L3½

Table 4-11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles

F_y = 36 ksi

Shape		L3½×3½×						L3½×3×					
		3/8		5/16		1/4[c]		1/2		7/16		3/8	
lb/ft		8.50		7.20		5.80		10.2		9.10		7.90	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z	0	53.9	81.0	45.3	68.0	34.8	52.3	65.1	97.8	57.6	86.5	50.0	75.2
	1	53.0	79.7	44.5	66.9	34.4	51.7	63.8	95.9	56.4	84.8	49.0	73.7
	2	50.5	75.9	42.4	63.8	33.3	50.0	60.1	90.4	53.2	79.9	46.2	69.5
	3	46.6	70.0	39.1	58.8	31.4	47.2	54.5	81.8	48.2	72.4	41.9	63.0
	4	41.6	62.5	35.0	52.5	28.4	42.6	47.4	71.2	42.0	63.1	36.6	54.9
	5	35.9	54.0	30.2	45.4	24.6	36.9	39.6	59.6	35.2	52.8	30.6	46.1
	6	30.0	45.1	25.3	38.0	20.6	30.9	31.9	47.9	28.3	42.5	24.7	37.1
	7	24.3	36.5	20.5	30.8	16.7	25.1	24.6	36.9	21.9	32.9	19.1	28.7
	8	19.0	28.6	16.1	24.2	13.1	19.7	18.8	28.3	16.7	25.2	14.6	22.0
	9	15.0	22.6	12.7	19.1	10.4	15.6	14.9	22.3	13.2	19.9	11.6	17.4
	10	12.2	18.3	10.3	15.5	8.40	12.6	12.0	18.1	10.7	16.1	9.37	14.1
	11	10.1	15.1	8.50	12.8	6.94	10.4						
Properties													
A_g, in.^{2}		2.50		2.10		1.70		3.02		2.67		2.32	
r_z, in.		0.683		0.685		0.688		0.618		0.620		0.622	

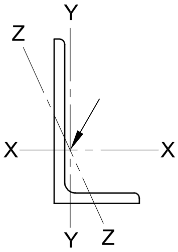
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_z equal to or greater than 200.

F_y = 36 ksi

Table 4-11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles

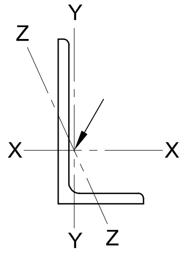


L3½

Shape		L3½×3×				L3½×2½×							
		5/16		1/4^c		1/2		3/8		5/16		1/4^c	
lb/ft		6.60		5.40		9.40		7.20		6.10		4.90	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z	0	42.0	63.2	33.1	49.8	59.7	89.7	45.7	68.7	38.6	58.0	30.3	45.6
	1	41.2	62.0	32.6	49.0	58.1	87.4	44.5	66.9	37.6	56.5	29.7	44.6
	2	38.9	58.4	31.0	46.7	53.6	80.6	41.1	61.8	34.7	52.2	27.8	41.8
	3	35.3	53.0	28.6	43.0	46.9	70.5	36.0	54.1	30.5	45.8	24.8	37.2
	4	30.8	46.3	25.0	37.6	38.9	58.5	29.9	45.0	25.4	38.1	20.7	31.0
	5	25.8	38.8	21.1	31.7	30.6	45.9	23.6	35.4	20.0	30.1	16.4	24.6
	6	20.9	31.3	17.0	25.6	22.7	34.2	17.6	26.4	15.0	22.6	12.3	18.5
	7	16.2	24.3	13.3	20.0	16.7	25.1	12.9	19.4	11.0	16.6	9.04	13.6
	8	12.4	18.6	10.2	15.3	12.8	19.2	9.90	14.9	8.45	12.7	6.92	10.4
	9	9.78	14.7	8.03	12.1							5.47	8.22
	10	7.93	11.9	6.50	9.78								
Properties													
A_g, in.²		1.95		1.58		2.77		2.12		1.79		1.45	
r_z, in.		0.624		0.628		0.532		0.535		0.538		0.541	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r_z equal to or greater than 200.



L3

Table 4-11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles

F_y = 36 ksi

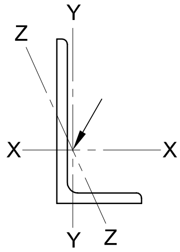
Shape		L3×3×											
		1/2		7/16		3/8		5/16		1/4		3/16[c]	
lb/ft		9.40		8.30		7.20		6.10		4.90		3.71	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z	0	59.5	89.4	52.4	78.7	45.5	68.4	38.4	57.7	31.0	46.7	20.5	30.8
	1	58.2	87.4	51.2	77.0	44.5	66.8	37.5	56.4	30.4	45.6	20.2	30.4
	2	54.4	81.7	47.9	71.9	41.6	62.5	35.1	52.7	28.4	42.7	19.3	29.0
	3	48.6	73.0	42.8	64.3	37.2	55.9	31.4	47.2	25.4	38.2	17.9	26.9
	4	41.5	62.4	36.5	54.9	31.8	47.7	26.9	40.4	21.8	32.7	16.1	24.2
	5	33.9	50.9	29.8	44.8	25.9	39.0	22.0	33.0	17.8	26.8	13.5	20.3
	6	26.4	39.7	23.3	35.0	20.3	30.5	17.2	25.8	14.0	21.0	10.6	16.0
	7	19.8	29.7	17.4	26.2	15.2	22.8	12.9	19.4	10.5	15.8	7.97	12.0
	8	15.1	22.8	13.3	20.0	11.6	17.5	9.87	14.8	8.04	12.1	6.10	9.18
	9	12.0	18.0	10.5	15.8	9.18	13.8	7.80	11.7	6.35	9.54	4.82	7.25
Properties													
A_g, in.^{2}		2.76		2.43		2.11		1.78		1.44		1.09	
r_z, in.		0.580		0.580		0.581		0.583		0.585		0.586	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r_z equal to or greater than 200.

F_y = 36 ksi

Table 4-11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles



L3

Shape	L3×2$^1/_2$×											
	$^1/_2$		$^7/_{16}$		$^3/_8$		$^5/_{16}$		$^1/_4$		$^3/_{16}$[c]	
lb/ft	8.50		7.60		6.60		5.60		4.50		3.39	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z												
0	53.9	81.0	47.9	71.9	41.6	62.5	35.1	52.8	28.5	42.8	19.8	29.8
1	52.4	78.7	46.5	69.9	40.4	60.8	34.2	51.3	27.7	41.6	19.5	29.2
2	48.1	72.3	42.7	64.2	37.1	55.8	31.4	47.2	25.4	38.2	18.3	27.5
3	41.7	62.7	37.0	55.7	32.2	48.4	27.2	41.0	22.1	33.2	16.3	24.4
4	34.2	51.4	30.3	45.6	26.4	39.7	22.4	33.6	18.2	27.3	13.8	20.7
5	26.4	39.8	23.5	35.3	20.5	30.8	17.3	26.1	14.1	21.2	10.7	16.1
6	19.3	29.0	17.1	25.8	15.0	22.5	12.7	19.1	10.3	15.6	7.87	11.8
7	14.2	21.3	12.6	18.9	11.0	16.5	9.32	14.0	7.60	11.4	5.78	8.69
8	10.9	16.3	9.64	14.5	8.41	12.6	7.13	10.7	5.82	8.75	4.43	6.65

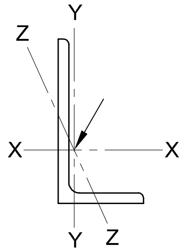
Properties

	$^1/_2$	$^7/_{16}$	$^3/_8$	$^5/_{16}$	$^1/_4$	$^3/_{16}$
A_g, in.^{2}	2.50	2.22	1.93	1.63	1.32	1.00
r_z, in.	0.516	0.516	0.517	0.518	0.520	0.521

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_z equal to or greater than 200.



L3–L2 1/2

Table 4-11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles

F_y = 36 ksi

Shape		L3×2×										L2 1/2×2 1/2×	
		1/2		3/8		5/16		1/4		3/16[c]		1/2	
lb/ft		7.70		5.90		5.00		4.10		3.07		7.70	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z	0	48.7	73.2	37.7	56.7	31.9	48.0	25.9	38.9	18.3	27.5	48.7	73.2
	1	46.7	70.2	36.2	54.4	30.6	46.0	24.8	37.3	17.7	26.6	47.1	70.9
	2	41.2	61.9	31.9	48.0	27.0	40.6	22.0	33.0	16.0	24.1	42.7	64.2
	3	33.4	50.2	25.9	38.9	22.0	33.0	17.9	26.9	13.6	20.4	36.3	54.5
	4	24.9	37.4	19.3	29.1	16.5	24.7	13.5	20.2	10.4	15.7	28.8	43.3
	5	17.0	25.6	13.3	19.9	11.3	17.0	9.31	14.0	7.24	10.9	21.5	32.3
	6	11.8	17.8	9.21	13.8	7.86	11.8	6.46	9.71	5.03	7.56	15.2	22.8
	7	8.70	13.1	6.77	10.2	5.78	8.68	4.75	7.14	3.70	5.56	11.1	16.7
	8											8.53	12.8
Properties													
A_g, in.^{2}		2.26		1.75		1.48		1.20		0.917		2.26	
r_z, in.		0.425		0.426		0.428		0.431		0.435		0.481	

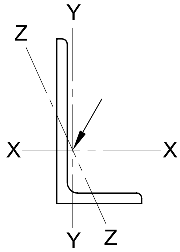
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_z equal to or greater than 200.

F_y = 36 ksi

Table 4-11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles



L2 1/2

Shape		L2 1/2×2 1/2×								L2 1/2×2×	
		3/8		5/16		1/4		3/16[c]		3/8	
lb/ft		5.90		5.00		4.10		3.07		5.30	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z	0	37.3	56.1	31.5	47.3	25.7	38.6	19.0	28.5	33.4	50.2
	1	36.1	54.2	30.5	45.8	24.8	37.3	18.6	27.9	32.0	48.1
	2	32.7	49.2	27.6	41.5	22.5	33.8	17.0	25.6	28.1	42.3
	3	27.8	41.7	23.4	35.2	19.1	28.7	14.5	21.8	22.7	34.0
	4	22.1	33.2	18.6	28.0	15.2	22.9	11.5	17.3	16.7	25.2
	5	16.4	24.7	13.9	20.9	11.3	17.1	8.59	12.9	11.4	17.1
	6	11.6	17.4	9.79	14.7	8.02	12.0	6.07	9.12	7.89	11.9
	7	8.53	12.8	7.20	10.8	5.89	8.85	4.46	6.70		
	8	6.53	9.81	5.51	8.28	4.51	6.78	3.41	5.13		

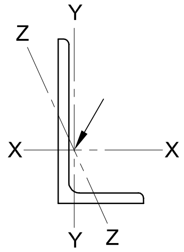
Properties

	L2 1/2×2 1/2×3/8	L2 1/2×2 1/2×5/16	L2 1/2×2 1/2×1/4	L2 1/2×2 1/2×3/16	L2 1/2×2×3/8
A_g, in.^{2}	1.73	1.46	1.19	0.901	1.55
r_z, in.	0.481	0.481	0.482	0.482	0.419

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_z equal to or greater than 200.



L2½

Table 4-11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles

F_y = 36 ksi

Shape	L2½×2×						L2½×1½×			
	5/16		1/4		3/16[c]		1/4		3/16[c]	
lb/ft	4.50		3.62		2.75		3.19		2.44	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z: 0	28.5	42.8	23.1	34.7	17.4	26.2	20.4	30.7	15.4	23.1
1	27.3	41.0	22.1	33.2	16.8	25.3	19.0	28.5	14.5	21.8
2	24.0	36.0	19.5	29.3	14.9	22.4	15.2	22.9	11.7	17.6
3	19.3	29.0	15.8	23.7	12.1	18.2	10.5	15.8	8.15	12.2
4	14.3	21.5	11.7	17.6	9.04	13.6	6.37	9.57	4.96	7.45
5	9.72	14.6	7.99	12.0	6.20	9.32	4.07	6.12	3.17	4.77
6	6.75	10.1	5.55	8.34	4.30	6.47				
7	4.96	7.46	4.08	6.13	3.16	4.75				

Properties

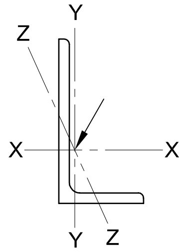
	L2½×2×5/16	L2½×2×1/4	L2½×2×3/16	L2½×1½×1/4	L2½×1½×3/16
A_g, in.^{2}	1.32	1.07	0.818	0.947	0.724
r_z, in.	0.420	0.423	0.426	0.321	0.324

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r_z equal to or greater than 200.

F_y = 36 ksi

Table 4-11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles



L2

Shape		L2×2×									
		3/8		5/16		1/4		3/16		1/8[c]	
lb/ft		4.70		3.92		3.19		2.44		1.65	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z	0	29.5	44.4	25.0	37.6	20.3	30.6	15.6	23.4	9.25	13.9
	1	28.1	42.2	23.8	35.7	19.3	29.1	14.8	22.2	8.95	13.5
	2	24.1	36.2	20.4	30.7	16.6	25.0	12.7	19.1	8.08	12.1
	3	18.7	28.1	15.8	23.8	12.9	19.4	9.92	14.9	6.77	10.2
	4	13.1	19.7	11.1	16.7	9.05	13.6	6.98	10.5	4.79	7.20
	5	8.52	12.8	7.22	10.8	5.90	8.87	4.56	6.86	3.13	4.71
	6	5.92	8.90	5.01	7.53	4.10	6.16	3.17	4.76	2.18	3.27

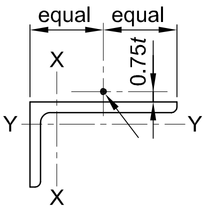
Properties

	3/8	5/16	1/4	3/16	1/8
A_g, in.^{2}	1.37	1.16	0.944	0.722	0.491
r_z, in.	0.386	0.386	0.387	0.389	0.391

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_z equal to or greater than 200.



L12

Table 4-12
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

F_y = 36 ksi

Shape		L12×12×							
		1 3/8		1 1/4		1 1/8		1	
lb/ft		105		96.4		87.2		77.8	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z	0	355	534	343	516	328	494	311	467
	1	354	533	342	515	328	493	310	466
	2	353	530	341	513	326	491	309	464
	3	350	526	338	509	324	487	306	461
	4	346	521	334	503	320	482	303	456
	5	341	514	330	496	316	475	298	449
	6	336	505	324	488	310	467	293	442
	7	329	496	318	479	304	458	287	433
	8	322	485	310	468	297	448	281	423
	9	313	473	302	457	289	437	273	413
	10	305	461	294	444	281	425	266	401
	11	296	447	285	431	273	412	257	389
	12	286	433	276	417	263	399	249	376
	13	276	419	266	403	254	385	240	363
	14	266	404	256	388	245	371	230	349
	15	256	389	246	373	235	357	221	336
	16	246	373	236	359	225	342	212	322
	17	235	358	226	344	215	328	203	308
	18	225	343	216	329	206	314	193	295
	19	215	328	206	314	196	299	184	281
	20	205	313	197	300	187	286	175	268
	21	196	299	187	286	178	272	167	255
	22	186	285	178	272	169	259	158	242
	23	177	271	169	259	161	246	150	230
	24	168	258	161	246	153	233	142	218
	25	160	245	152	233	145	221	135	206
	26	151	232	144	221	137	210	127	195
	27	143	220	136	209	129	198	120	184
	28	136	208	129	198	122	187	114	174

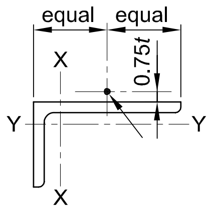
Properties

	1 3/8	1 1/4	1 1/8	1
A_g, in.^{2}	31.1	28.4	25.8	23.0
r_z, in.	2.30	2.31	2.33	2.34

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

F_y = 36 ksi

Table 4-12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

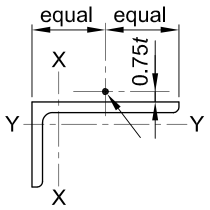


L10

Shape	L10×10×											
	1 3/8		1 1/4		1 1/8		1		7/8		3/4 [c]	
lb/ft	87.1		79.9		72.3		64.7		56.9		49.1	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z												
0	262	393	254	383	247	371	234	352	223	336	187	282
1	261	393	254	382	246	370	233	351	223	335	187	282
2	259	390	252	379	245	368	232	349	221	333	187	281
3	257	386	249	375	242	364	229	345	219	329	187	281
4	253	380	246	370	238	358	225	339	215	324	186	280
5	248	373	241	363	233	351	221	333	211	317	185	278
6	242	365	235	354	228	343	215	325	205	310	184	277
7	236	356	229	345	221	334	209	316	199	301	181	272
8	229	345	221	335	214	324	202	306	193	291	176	265
9	221	334	214	323	207	312	195	295	186	281	172	258
10	213	322	206	311	199	301	187	284	178	270	165	250
11	204	310	197	299	190	288	179	272	170	258	158	239
12	196	297	189	286	182	276	171	260	162	246	150	228
13	187	284	180	274	173	263	163	247	154	234	143	217
14	178	271	171	261	165	250	154	235	146	223	135	206
15	170	258	163	248	156	238	146	223	138	211	128	195
16	161	245	154	235	148	225	138	211	130	199	121	184
17	152	233	146	222	140	213	130	199	123	188	113	173
18	144	220	138	210	132	201	123	188	115	176	107	163
19	136	208	130	198	124	189	115	177	108	166	100	153
20	128	196	122	187	116	178	108	166	101	155	94.1	143
21	121	185	115	176	109	167	102	156	95.4	146	88.0	134
22	113	174	108	165	102	157	95.4	145	89.1	136	82.2	125
23	107	163	101	155	96.4	147	89.3	136	83.2	127	76.7	117
24	100	154	95.4	145	90.5	138	83.7	128	77.9	119	71.7	109
25	95.1	145	89.8	137	85.1	130	78.6	120	73.1	111	67.1	102
26	89.8	137	84.8	129	80.2	122	74.0	113	68.7	105	63.0	96.4
27	84.9	130	80.1	122	75.7	115	69.8	106	64.7	99.0	59.2	90.7
28	80.5	123	75.8	116	71.6	109	65.9	100	61.0	93.3	55.8	85.4
Properties												
A_g, in.^{2}	25.6		23.4		21.3		19.0		16.8		14.5	
r_z, in.	1.91		1.91		1.92		1.92		1.93		1.96	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.



L8

Table 4-12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

F_y = 36 ksi

Shape		L8×8×											
		1 1/8		1		7/8		3/4		5/8		9/16 [c]	
lb/ft		56.9		51.0		45.0		38.9		32.7		29.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z	0	174	262	167	251	160	240	150	225	127	192	108	163
	1	173	261	167	251	159	239	149	225	127	191	108	163
	2	172	258	165	248	157	237	148	222	127	191	108	162
	3	169	254	162	244	155	233	145	219	126	189	107	161
	4	165	249	158	239	151	228	142	213	125	188	107	160
	5	160	242	154	232	147	221	137	207	124	186	105	159
	6	155	234	148	224	141	213	132	200	121	181	103	154
	7	149	225	142	215	135	205	127	191	116	175	100	150
	8	142	216	136	206	129	196	120	182	110	167	97.4	146
	9	135	205	129	196	122	186	114	173	104	158	94.2	141
	10	128	195	122	186	116	176	108	163	98.3	149	90.7	135
	11	121	184	115	175	109	166	101	154	92.1	140	86.1	129
	12	114	174	108	165	102	155	94.8	144	86.0	130	80.3	122
	13	107	163	101	154	95.6	145	88.3	134	80.0	121	74.6	113
	14	100	153	94.9	144	89.1	136	82.1	125	74.1	113	69.1	105
	15	93.7	143	88.4	134	82.8	126	76.1	116	68.6	104	63.8	97.5
	16	87.3	133	82.1	125	76.8	117	70.4	107	63.3	96.8	58.8	90.0
	17	81.1	123	76.1	116	71.1	108	64.9	99.4	58.3	89.2	54.1	82.8
	18	75.1	114	70.3	107	65.5	100	59.7	91.4	53.5	81.9	49.6	75.9
	19	69.6	106	65.1	99.6	60.5	92.6	55.0	84.2	49.1	75.2	45.5	69.6
	20	64.7	99.0	60.4	92.4	56.0	85.8	50.8	77.8	45.3	69.3	41.9	64.1
	21	60.2	92.2	56.2	85.9	52.0	79.6	47.1	72.0	41.9	64.1	38.7	59.2
	22	56.2	86.1	52.3	80.1	48.4	74.1	43.7	66.9	38.8	59.4	35.8	54.8
	23	52.6	80.5	48.9	74.8	45.1	69.1	40.7	62.3	36.1	55.2	33.2	50.9
	24	49.3	75.5	45.8	70.1	42.2	64.6	38.0	58.1	33.6	51.4	30.9	47.4
	25	46.3	70.9	42.9	65.7	39.5	60.5	35.5	54.4	31.4	48.0	28.9	44.2
	26	43.6	66.7	40.3	61.8	37.1	56.8	33.3	50.9	29.4	44.9	27.0	41.3

Properties

	1 1/8	1	7/8	3/4	5/8	9/16
A_g, in.^{2}	16.8	15.1	13.3	11.5	9.69	8.77
r_z, in.	1.56	1.56	1.57	1.57	1.58	1.58

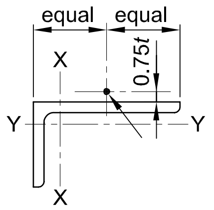
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_z equal to or greater than 200.

F_y = 36 ksi

Table 4-12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

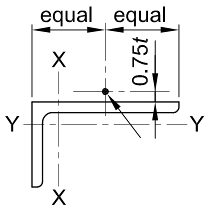


L8

Shape	L8×8×		L8×6×									
	$^1/_2$ [c,f]		1		$^7/_8$		$^3/_4$		$^5/_8$		$^9/_{16}$ [c]	
lb/ft	26.4		44.2		39.1		33.8		28.5		25.7	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z												
0	89.1	134	160	240	159	239	157	237	154	232	153	230
1	89.1	133	159	239	158	238	157	236	153	230	151	228
2	88.8	133	157	237	156	235	154	232	149	225	146	220
3	88.4	132	154	233	153	231	150	227	143	216	139	210
4	87.8	131	150	227	148	224	145	219	135	205	130	198
5	87.0	130	145	220	139	210	135	205	127	194	121	185
6	84.7	127	134	204	128	195	125	190	119	182	112	172
7	82.3	123	123	188	118	180	114	174	109	167	104	160
8	79.8	119	113	173	108	165	104	159	98.3	150	94.5	145
9	77.3	115	104	159	98.9	151	94.1	144	88.1	135	84.3	129
10	74.9	112	95.1	145	89.8	137	85.0	130	78.9	121	75.1	115
11	72.5	108	86.6	132	81.4	125	76.6	118	70.6	108	67.0	103
12	70.1	104	78.7	121	73.7	113	69.0	106	63.2	97.6	59.8	92.5
13	67.1	99.8	71.5	110	66.6	102	62.2	96.0	56.6	87.5	53.4	82.7
14	63.9	94.6	64.8	99.8	60.2	92.8	56.0	86.5	50.7	78.5	47.8	74.0
15	59.3	89.2	58.7	90.4	54.3	83.8	50.4	77.8	45.4	70.3	42.7	66.2
16	54.6	83.6	53.3	82.1	49.2	75.9	45.5	70.3	40.8	63.2	38.4	59.5
17	50.2	76.8	48.7	75.0	44.8	69.1	41.3	63.8	36.9	57.2	34.7	53.7
18	46.0	70.4	44.6	68.7	41.0	63.1	37.6	58.1	33.6	52.0	31.5	48.7
19	42.1	64.5	41.0	63.1	37.6	57.9	34.5	53.2	30.7	47.4	28.7	44.4
20	38.7	59.3	37.8	58.2	34.6	53.3	31.7	48.9	28.1	43.5	26.3	40.7
21	35.7	54.7	35.0	53.9	32.0	49.3	29.2	45.1	25.9	40.0	24.2	37.4
22	33.1	50.6										
23	30.7	46.9										
24	28.5	43.6										
25	26.6	40.7										
26	24.9	38.0										
Properties												
A_g, in.^{2}	7.84		13.1		11.5		9.99		8.41		7.61	
r_z, in.	1.59		1.28		1.28		1.29		1.29		1.30	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
[f] Shape exceeds compact limit for flexure with $F_y = 36$ ksi.
Note: Heavy line indicates L_c/r_z equal to or greater than 200.



L8

Table 4-12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

F_y = 36 ksi

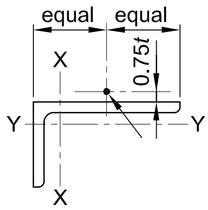
Shape		L8×6×				L8×4×							
		1/2[c,f]		7/16[c,f]		1		7/8		3/4		5/8	
lb/ft		23.0		20.2		37.4		33.1		28.7		24.2	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z	0	125	188	91.8	138	67.8	102	65.9	99.0	64.0	96.2	62.2	93.5
	1	125	187	91.9	138	67.2	101	65.2	98.1	63.4	95.3	61.5	92.6
	2	124	186	91.9	138	65.4	98.5	63.3	95.3	61.6	92.8	59.6	89.8
	3	124	186	91.2	136	62.6	94.4	60.5	91.2	58.8	88.7	56.5	85.3
	4	125	186	90.7	135	59.0	89.2	57.0	86.2	55.1	83.3	52.6	79.7
	5	118	179	90.7	135	55.1	83.5	53.0	80.3	50.9	77.2	48.3	73.3
	6	108	166	92.1	135	50.9	77.3	48.7	73.9	46.5	70.7	43.7	66.6
	7	99.4	152	92.7	141	46.6	70.9	44.3	67.4	42.0	64.1	39.2	59.9
	8	89.5	137	84.2	130	42.2	64.4	40.0	61.0	37.7	57.6	34.9	53.4
	9	79.9	123	75.0	115	38.0	58.1	35.8	54.7	33.6	51.4	30.9	47.4
	10	71.2	110	66.5	103	34.0	52.0	31.8	48.7	29.7	45.5	27.2	41.7
	11	63.2	97.9	59.1	91.8	30.4	46.5	28.4	43.5	26.4	40.4	24.0	36.8
	12	56.2	85.1	52.6	81.8	27.4	41.9	25.4	39.0	23.5	36.1	21.3	32.7
	13	50.0	77.6	46.8	72.9	24.7	37.8	22.9	35.1	21.1	32.3	19.0	29.2
	14	44.6	69.3	41.6	64.8	22.4	34.3	20.7	31.7	19.0	29.2	17.1	26.2
	15	39.8	61.8	37.0	57.7								
	16	35.7	55.4	33.1	51.6								
	17	32.2	49.9	29.8	46.4								
	18	29.1	45.2	27.0	42.0								
	19	26.5	41.2	24.5	38.1								
	20	24.3	37.6	22.4	34.8								
	21	22.3	34.6	20.5	31.9								
Properties													
A_g, in.^{2}		6.80		5.99		11.1		9.79		8.49		7.16	
r_z, in.		1.30		1.31		0.844		0.846		0.850		0.856	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
[f] Shape exceeds compact limit for flexure with $F_y = 36$ ksi.
Note: Heavy line indicates L_c/r_z equal to or greater than 200.

F_y = 36 ksi

Table 4-12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

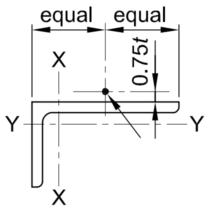


L8–L7

Shape	L8×4×						L7×4×					
	9/16[c]		1/2[c,f]		7/16[c,f]		3/4		5/8		1/2[c]	
lb/ft	21.9		19.6		17.2		26.2		22.1		17.9	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z												
0	60.0	90.2	57.8	87.0	55.5	83.4	65.1	97.9	62.5	94.0	59.5	89.4
1	59.4	89.3	57.2	86.0	54.8	82.5	64.4	96.8	61.8	93.0	58.8	88.4
2	57.6	86.7	55.3	83.4	52.9	79.7	62.1	93.5	59.8	90.0	56.7	85.4
3	54.7	82.5	52.4	79.2	50.0	75.5	58.7	88.6	56.5	85.3	53.4	80.6
4	51.0	77.2	48.7	73.8	46.2	70.1	54.8	82.9	52.3	79.3	49.2	74.6
5	46.7	70.9	44.6	67.7	42.1	64.0	50.3	76.4	47.7	72.4	44.4	67.5
6	42.1	64.1	40.3	61.4	37.9	57.8	45.7	69.5	42.9	65.4	39.5	60.3
7	37.6	57.4	36.0	55.0	33.7	51.7	41.0	62.6	38.2	58.4	34.8	53.3
8	33.3	51.0	31.7	48.6	29.9	45.9	36.6	56.0	33.9	51.9	30.5	46.8
9	29.4	45.1	27.9	42.8	26.2	40.3	32.5	49.7	29.8	45.7	26.7	41.0
10	25.8	39.5	24.3	37.4	22.8	35.1	28.6	43.8	26.1	40.1	23.2	35.6
11	22.6	34.8	21.3	32.7	19.9	30.6	25.3	38.8	22.9	35.2	20.2	31.1
12	20.0	30.8	18.8	28.9	17.5	26.9	22.5	34.5	20.3	31.2	17.8	27.4
13	17.9	27.5	16.7	25.7	15.5	23.8	20.1	30.8	18.0	27.7	15.7	24.2
14	16.0	24.6	14.9	23.0	13.8	21.3	18.0	27.7	16.2	24.9	14.0	21.6
Properties												
A_g, in.^{2}	6.49		5.80		5.11		7.74		6.50		5.26	
r_z, in.	0.859		0.863		0.867		0.855		0.860		0.866	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
[f] Shape exceeds compact limit for flexure with $F_y = 36$ ksi.
Note: Heavy line indicates L_c/r_z equal to or greater than 200.



L7–L6

Table 4-12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

F_y = 36 ksi

Shape	L7×4×				L6×6×							
	7/16[c,f]		3/8[c,f]		1		7/8		3/4		5/8	
lb/ft	15.7		13.6		37.4		33.1		28.7		24.2	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z												
0	56.9	85.5	54.0	81.2	101	152	99.2	149	94.0	141	87.8	132
1	56.1	84.5	53.3	80.1	100	151	98.6	148	93.4	140	87.3	131
2	54.0	81.4	51.1	77.0	99.3	149	97.0	145	91.7	138	85.7	128
3	50.7	76.6	47.7	72.2	96.7	145	94.2	141	89.1	134	83.1	125
4	46.6	70.6	43.6	66.1	93.1	140	90.6	136	85.5	128	79.6	120
5	42.0	64.0	39.1	59.5	88.9	134	86.3	130	81.3	122	75.5	114
6	37.4	57.2	34.6	52.9	84.1	127	81.4	123	76.5	115	70.8	107
7	33.0	50.5	30.4	46.6	79.0	119	76.2	115	71.4	108	65.9	99.0
8	28.7	44.1	26.5	40.9	73.7	111	70.8	107	66.1	100	60.8	92.3
9	24.9	38.4	23.1	35.6	68.3	103	65.4	99.5	60.8	92.6	55.7	84.8
10	21.6	33.2	19.9	30.7	63.0	95.9	60.1	91.5	55.7	84.9	50.8	77.4
11	18.7	28.9	17.2	26.5	57.8	88.2	54.9	83.8	50.7	77.4	46.0	70.3
12	16.4	25.3	15.0	23.1	52.8	80.7	50.0	76.4	46.0	70.3	41.6	63.6
13	14.5	22.4	13.2	20.4	48.1	73.5	45.3	69.3	41.6	63.6	37.4	57.2
14	12.9	19.9	11.7	18.1	43.6	66.7	40.9	62.6	37.4	57.2	33.5	51.3
15					39.7	60.7	37.1	56.8	33.8	51.8	30.2	46.2
16					36.2	55.5	33.8	51.8	30.7	47.0	27.3	41.8
17					33.2	50.9	30.9	47.3	28.0	42.9	24.8	38.0
18					30.5	46.8	28.4	43.4	25.6	39.3	22.7	34.7
19					28.2	43.2	26.1	40.0	23.5	36.1	20.8	31.8

Properties

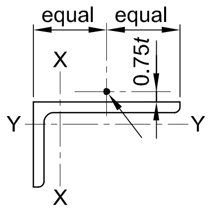
	L7×4×7/16	L7×4×3/8	L6×6×1	L6×6×7/8	L6×6×3/4	L6×6×5/8
A_g, in.^{2}	4.63	4.00	11.0	9.75	8.46	7.13
r_z, in.	0.869	0.873	1.17	1.17	1.17	1.17

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
[f] Shape exceeds compact limit for flexure with $F_y = 36$ ksi.
Note: Heavy line indicates L_c/r_z equal to or greater than 200.

F_y = 36 ksi

Table 4-12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles



L6

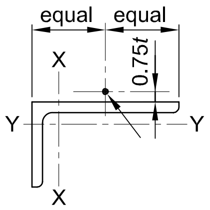
Shape	L6×6×										L6×4×	
	9/16		1/2		7/16[c]		3/8[c,f]		5/16[c,f]		7/8	
lb/ft	21.9		19.6		17.2		14.9		12.4		27.2	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z												
0	83.3	125	77.9	117	64.6	97.2	49.6	74.5	34.6	52.1	72.1	108
1	82.8	124	77.7	116	64.5	97.0	49.5	74.4	34.6	52.0	71.1	107
2	81.3	122	77.3	116	64.3	96.6	49.2	74.0	34.4	51.7	68.4	103
3	78.8	118	75.2	113	63.8	95.9	48.8	73.4	34.0	51.2	64.3	97.1
4	75.5	113	72.0	108	62.8	94.3	48.0	72.1	33.6	50.5	59.3	89.8
5	71.5	108	68.1	102	60.6	90.9	46.1	69.3	33.0	49.7	53.9	81.9
6	67.1	101	63.8	96.5	58.0	86.8	44.3	66.5	32.4	48.6	48.6	73.9
7	62.3	94.5	59.2	89.8	54.4	82.4	42.5	63.7	31.2	46.8	43.5	66.4
8	57.5	87.3	54.5	82.8	49.9	75.8	40.6	60.8	29.7	44.5	38.7	59.2
9	52.6	80.1	49.8	75.7	45.5	69.3	38.8	58.0	28.2	42.2	34.2	52.4
10	47.9	73.0	45.2	68.9	41.2	62.9	36.5	54.3	26.6	39.9	30.0	46.1
11	43.4	66.2	40.8	62.3	37.2	56.8	34.1	50.3	25.1	37.5	26.5	40.6
12	39.1	59.8	36.7	56.2	33.3	51.0	30.6	46.3	23.5	35.1	23.5	36.1
13	35.1	53.8	32.9	50.4	29.8	45.6	27.3	41.9	21.5	32.0	21.0	32.2
14	31.4	48.2	29.4	44.9	26.5	40.7	24.3	37.2	19.3	28.6	18.8	28.9
15	28.3	43.3	26.3	40.3	23.8	36.4	21.7	33.2	17.3	25.7		
16	25.6	39.1	23.7	36.4	21.4	32.8	19.5	29.8	15.5	23.1		
17	23.2	35.5	21.5	32.9	19.4	29.6	17.6	26.9	14.0	20.9		
18	21.2	32.4	19.6	30.0	17.6	26.9	15.9	24.4	12.7	18.9		
19	19.4	29.7	17.9	27.4	16.0	24.6	14.5	22.2	11.6	17.2		

Properties

	9/16	1/2	7/16	3/8	5/16	7/8
A_g, in.^{2}	6.45	5.77	5.08	4.38	3.67	8.00
r_z, in.	1.18	1.18	1.18	1.19	1.19	0.854

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
[f] Shape exceeds compact limit for flexure with $F_y = 36$ ksi.
Note: Heavy line indicates L_c/r_z equal to or greater than 200.



L6

Table 4-12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

F_y = 36 ksi

Shape	L6×4×									
	3/4		5/8		9/16		1/2		7/16[c]	
lb/ft	23.6		20.0		18.1		16.2		14.3	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z										
0	70.4	105	67.6	101	66.5	99.0	64.0	96.3	62.8	94.4
1	69.4	104	66.5	100	65.3	98.2	63.1	95.0	61.9	93.1
2	66.5	100	63.4	95.6	62.1	93.5	60.5	91.2	59.2	89.3
3	62.1	93.8	58.8	88.9	57.9	87.5	56.2	85.0	54.9	83.1
4	56.9	86.2	53.9	81.8	52.8	80.1	50.9	77.3	49.5	75.3
5	51.5	78.3	48.6	73.9	47.2	71.9	45.2	68.9	43.6	66.5
6	46.2	70.4	43.1	65.9	41.7	63.7	39.7	60.7	37.9	58.1
7	41.0	62.7	38.0	58.2	36.5	55.9	34.6	53.0	32.7	50.3
8	36.2	55.5	33.3	51.1	31.8	48.8	30.0	46.1	28.2	43.4
9	31.8	48.8	29.0	44.6	27.6	42.5	25.9	39.9	24.2	37.4
10	27.8	42.7	25.2	38.8	23.9	36.8	22.3	34.4	20.8	32.1
11	24.4	37.5	22.0	33.8	20.7	32.0	19.3	29.8	17.9	27.7
12	21.5	33.1	19.3	29.8	18.2	28.0	16.9	26.1	15.6	24.1
13	19.2	29.5	17.1	26.4	16.1	24.8	14.9	23.0	13.7	21.2
14	17.2	26.4	15.3	23.5	14.3	22.0	13.2	20.4	12.2	18.8

Properties

A_g, in.^{2}	6.94	5.86	5.31	4.75	4.18
r_z, in.	0.856	0.859	0.861	0.864	0.867

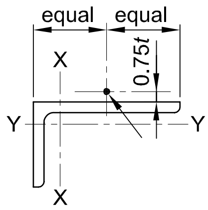
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_z equal to or greater than 200.

F_y = 36 ksi

Table 4-12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

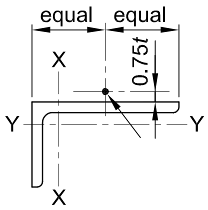


L6

Shape	L6×4×				L6×3½×					
	3/8[c,f]		5/16[c,f]		1/2		3/8[c,f]		5/16[c,f]	
lb/ft	12.3		10.3		15.3		11.7		9.80	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z										
0	59.9	90.0	55.7	83.7	47.7	71.6	44.1	66.3	41.5	62.5
1	58.9	88.7	55.8	83.8	46.9	70.6	43.3	65.2	40.8	61.3
2	56.2	84.8	53.2	80.3	44.7	67.5	41.1	62.1	38.5	58.1
3	51.9	78.6	48.7	73.9	41.4	62.6	37.8	57.3	35.2	53.3
4	46.6	70.9	43.4	66.1	37.3	56.6	33.9	51.5	31.2	47.5
5	41.0	62.6	37.8	57.8	33.0	50.3	29.7	45.4	27.1	41.5
6	35.6	54.6	32.5	50.0	28.8	44.1	25.7	39.4	23.3	35.8
7	30.6	47.2	27.8	42.9	24.9	38.3	21.9	33.7	19.9	30.7
8	26.2	40.4	23.8	36.8	21.5	33.0	18.6	28.7	17.0	26.2
9	22.3	34.5	20.3	31.5	18.3	28.2	15.7	24.3	14.2	22.0
10	19.1	29.5	17.2	26.8	15.8	24.3	13.4	20.7	12.1	18.7
11	16.4	25.3	14.7	22.9	13.7	21.1	11.6	17.9	10.4	16.0
12	14.2	22.0	12.7	19.7	12.0	18.5	10.1	15.6	9.04	13.9
13	12.4	19.3	11.1	17.2						
14	11.0	17.0	9.83	15.2						
Properties										
A_g, in.^{2}	3.61		3.03		4.50		3.44		2.89	
r_z, in.	0.870		0.874		0.756		0.763		0.767	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
[f] Shape exceeds compact limit for flexure with $F_y = 36$ ksi.
Note: Heavy line indicates L_c/r_z equal to or greater than 200.



L5

Table 4-12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

F_y = 36 ksi

Shape		L5×5×											
		$^7/_8$		$^3/_4$		$^5/_8$		$^1/_2$		$^7/_{16}$		$^3/_8$[c]	
lb/ft		27.2		23.6		20.0		16.2		14.3		12.3	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z	0	72.2	108	69.0	103	66.0	99.2	61.0	91.8	56.6	85.1	46.6	70.0
	1	71.6	107	68.5	102	65.4	98.3	60.5	91.0	56.1	84.4	46.5	69.8
	2	70.0	105	66.8	100	63.7	95.9	58.9	88.6	54.6	82.2	46.2	69.4
	3	67.4	101	64.2	96.7	61.1	92.1	56.3	84.9	52.2	78.7	45.7	68.7
	4	64.0	96.6	60.7	91.7	57.7	87.1	53.0	80.1	49.1	74.1	43.8	65.7
	5	60.0	90.7	56.7	85.8	53.7	81.2	49.2	74.4	45.5	68.8	41.4	62.1
	6	55.6	84.3	52.4	79.4	49.3	74.8	45.0	68.3	41.5	63.0	38.7	58.5
	7	51.1	77.6	47.9	72.8	44.9	68.2	40.8	62.0	37.5	57.0	34.8	53.0
	8	46.6	70.9	43.4	66.1	40.5	61.6	36.6	55.7	33.5	51.1	31.1	47.4
	9	42.1	64.2	39.1	59.6	36.2	55.3	32.5	49.7	29.7	45.4	27.5	42.0
	10	37.9	57.8	35.0	53.4	32.2	49.3	28.8	44.0	26.2	40.1	24.2	37.0
	11	33.8	51.7	31.1	47.5	28.5	43.6	25.3	38.7	23.0	35.2	21.1	32.3
	12	30.1	46.1	27.6	42.2	25.1	38.5	22.2	34.0	20.1	30.8	18.4	28.2
	13	27.0	41.3	24.6	37.7	22.3	34.2	19.6	30.0	17.7	27.2	16.1	24.7
	14	24.3	37.2	22.1	33.8	19.9	30.5	17.4	26.7	15.7	24.1	14.3	21.9
	15	22.0	33.6	19.9	30.5	17.9	27.4	15.6	23.9	14.1	21.6	12.7	19.5
	16	20.0	30.5	18.0	27.6	16.2	24.8	14.0	21.5	12.6	19.4	11.4	17.5

Properties

	$^7/_8$	$^3/_4$	$^5/_8$	$^1/_2$	$^7/_{16}$	$^3/_8$
A_g, in.^{2}	8.00	6.98	5.90	4.79	4.22	3.65
r_z, in.	0.971	0.972	0.975	0.980	0.983	0.986

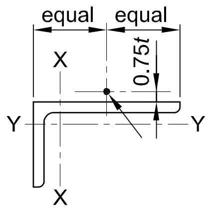
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_z equal to or greater than 200.

F_y = 36 ksi

Table 4-12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

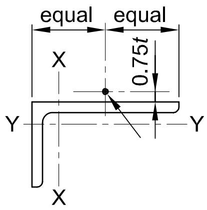


L5

Shape		L5×5×		L5×3½×									
		5/16[c,f]		3/4		5/8		1/2		3/8[c]		5/16[c,f]	
lb/ft		10.3		19.8		16.8		13.6		10.4		8.70	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z	0	34.7	52.2	55.5	83.4	54.8	82.3	53.3	80.2	49.9	75.0	47.6	71.6
	1	34.6	52.0	55.0	82.8	54.2	81.5	52.0	78.3	48.9	73.6	46.6	70.2
	2	34.3	51.6	53.5	80.6	51.2	77.3	48.5	73.2	45.9	69.4	43.6	65.8
	3	33.9	51.0	49.0	74.2	46.6	70.5	43.9	66.5	41.2	62.5	38.9	59.1
	4	32.6	48.9	43.9	66.7	41.3	62.7	38.8	59.1	35.6	54.3	33.5	51.2
	5	31.0	46.5	38.7	59.0	36.3	55.3	33.7	51.4	30.2	46.2	28.2	43.4
	6	29.5	44.2	33.9	51.8	31.5	48.2	28.8	44.2	25.3	39.0	23.6	36.4
	7	27.9	41.8	29.5	45.1	27.1	41.6	24.5	37.7	21.2	32.7	19.5	30.1
	8	26.3	39.1	25.4	39.0	23.2	35.6	20.8	32.0	17.7	27.4	16.1	25.0
	9	24.2	35.8	21.8	33.5	19.7	30.4	17.5	27.0	14.8	22.9	13.4	20.7
	10	21.3	32.4	18.9	29.0	17.0	26.1	15.0	23.1	12.5	19.3	11.2	17.4
	11	18.6	28.5	16.5	25.3	14.7	22.7	12.9	19.9	10.7	16.6	9.62	14.8
	12	16.1	24.7	14.5	22.3	12.9	19.9	11.2	17.3	9.32	14.3	8.31	12.8
	13	14.1	21.7										
	14	12.5	19.1										
	15	11.1	17.0										
	16	9.97	15.2										
Properties													
A_g, in.^{2}		3.07		5.85		4.93		4.00		3.05		2.56	
r_z, in.		0.990		0.744		0.746		0.750		0.755		0.758	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
[f] Shape exceeds compact limit for flexure with $F_y = 36$ ksi.
Note: Heavy line indicates L_c/r_z equal to or greater than 200.



L5

Table 4-12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

F_y = 36 ksi

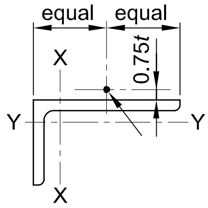
Shape	L5×3½×		L5×3×									
	1/4[c,f]		1/2		7/16		3/8[c]		5/16[c,f]		1/4[c,f]	
lb/ft	7.00		12.8		11.3		9.80		8.20		6.60	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z												
0	32.1	48.3	36.2	54.4	34.7	52.2	34.2	51.4	31.9	47.9	30.0	45.1
1	32.3	48.5	35.3	53.2	34.0	51.1	33.4	50.3	31.1	46.8	29.1	43.9
2	32.4	48.6	33.0	49.8	31.8	48.0	31.2	47.1	28.9	43.7	26.8	40.6
3	33.1	49.1	29.9	45.3	28.6	43.4	27.8	42.2	25.7	39.0	23.5	35.8
4	30.0	46.0	26.3	40.1	25.0	38.1	24.0	36.5	22.1	33.8	20.0	30.5
5	25.0	38.5	22.7	34.7	21.4	32.7	20.3	31.0	18.7	28.6	16.6	25.6
6	20.6	31.9	19.3	29.6	18.1	27.7	16.9	26.0	15.4	23.7	13.7	21.2
7	17.0	26.5	16.3	25.0	15.1	23.2	14.0	21.6	12.7	19.6	11.3	17.5
8	14.1	22.0	13.6	20.9	12.6	19.4	11.6	17.9	10.4	16.1	9.23	14.2
9	11.7	18.2	11.5	17.8	10.6	16.4	9.79	15.0	8.72	13.4	7.64	11.8
10	9.79	15.2	9.95	15.2	9.12	14.0	8.33	12.8	7.38	11.3	6.43	9.94
11	8.31	12.9										
12	7.14	11.0										
Properties												
A_g, in.^{2}	2.07		3.75		3.31		2.86		2.41		1.94	
r_z, in.	0.761		0.642		0.644		0.646		0.649		0.652	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.
[f] Shape exceeds compact limit for flexure with F_y = 36 ksi.
Note: Heavy line indicates L_c/r_z equal to or greater than 200.

F_y = 36 ksi

Table 4-12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles



L4

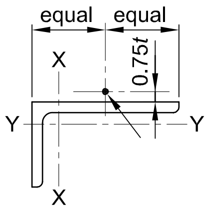
Shape		L4×4×											
		3/4		5/8		1/2		7/16		3/8		5/16	
lb/ft		18.5		15.7		12.8		11.3		9.80		8.20	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z	0	46.0	69.1	44.7	67.1	41.3	62.2	39.6	59.6	36.9	55.5	32.1	48.2
	1	45.4	68.3	44.1	66.3	40.8	61.4	39.1	58.8	36.4	54.8	31.9	48.0
	2	43.8	66.0	42.4	63.9	39.2	59.0	37.5	56.5	34.9	52.5	31.5	47.3
	3	41.4	62.5	39.9	60.2	36.7	55.4	35.0	52.9	32.5	49.1	29.4	44.4
	4	38.4	58.0	36.7	55.6	33.6	50.8	32.0	48.4	29.6	44.8	26.7	40.4
	5	35.0	53.0	33.2	50.4	30.2	45.8	28.6	43.4	26.4	40.1	23.7	36.0
	6	31.4	47.7	29.6	45.0	26.7	40.6	25.1	38.3	23.1	35.2	20.7	31.5
	7	27.9	42.5	26.0	39.7	23.3	35.5	21.8	33.3	20.0	30.6	17.8	27.2
	8	24.5	37.4	22.7	34.7	20.1	30.8	18.8	28.7	17.1	26.2	15.2	23.3
	9	21.3	32.5	19.6	29.9	17.2	26.4	16.0	24.5	14.5	22.2	12.8	19.6
	10	18.5	28.4	16.9	25.9	14.8	22.6	13.7	20.9	12.3	18.9	10.8	16.6
	11	16.3	24.9	14.7	22.6	12.8	19.6	11.8	18.1	10.6	16.3	9.34	14.2
	12	14.4	22.0	12.9	19.8	11.2	17.2	10.3	15.7	9.26	14.1	8.09	12.3
	13									8.12	12.4	7.07	10.8

Properties

	3/4	5/8	1/2	7/16	3/8	5/16
A_g, in.^{2}	5.44	4.61	3.75	3.30	2.86	2.40
r_z, in.	0.774	0.774	0.776	0.777	0.779	0.781

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates L_c/r_z equal to or greater than 200.



L4

Table 4-12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

F_y = 36 ksi

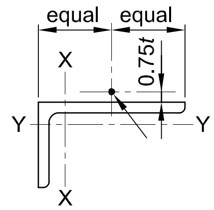
Shape	L4×4×		L4×3½×								L4×3×	
	¼[c,f]		½		⅜		5/16		¼[c,f]		⅝	
lb/ft	6.60		11.9		9.10		7.70		6.20		13.6	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z												
0	21.9	33.0	50.4	75.8	48.0	72.1	36.0	54.1	24.5	36.9	39.4	59.2
1	21.8	32.8	49.7	74.8	48.1	72.3	36.0	54.0	24.5	36.8	38.9	58.5
2	21.6	32.5	47.7	72.1	48.5	73.4	35.8	53.7	23.8	35.8	37.5	56.6
3	20.8	31.2	44.5	67.8	43.7	66.6	34.8	51.8	22.8	34.2	34.6	52.5
4	19.6	29.4	40.6	62.3	37.9	58.3	34.8	52.5	21.9	32.6	29.7	45.2
5	18.3	27.5	34.8	53.6	32.3	49.9	29.4	45.7	21.9	31.6	25.1	38.3
6	17.2	25.6	28.6	44.2	25.8	40.0	23.7	36.9	21.4	33.5	21.0	32.3
7	15.5	23.0	23.5	36.4	20.8	32.3	18.8	29.4	16.8	26.3	17.5	26.9
8	13.2	20.2	19.3	29.9	16.8	26.2	15.1	23.6	13.3	20.9	14.6	22.4
9	11.1	16.9	16.0	24.8	13.7	21.4	12.3	19.1	10.7	16.8	12.3	18.9
10	9.34	14.2	13.4	20.8	11.4	17.8	10.1	15.8	8.88	13.8	10.5	16.1
11	7.97	12.1	11.5	17.7	9.72	15.0	8.59	13.3	7.45	11.6		
12	6.87	10.5			8.33	12.9	7.34	11.3	6.34	9.88		
13	5.99	9.15										
Properties												
A_g, in.^{2}	1.93		3.50		2.68		2.25		1.82		3.99	
r_z, in.	0.783		0.716		0.719		0.721		0.723		0.631	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
[f] Shape exceeds compact limit for flexure with $F_y = 36$ ksi.
Note: Heavy line indicates L_c/r_z equal to or greater than 200.

F_y = 36 ksi

Table 4-12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

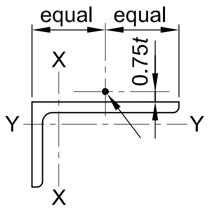


L4–L3½

Shape	L4×3×								L3½×3½×			
	1/2		3/8		5/16		1/4[c,f]		1/2		7/16	
lb/ft	11.1		8.50		7.20		5.80		11.1		9.80	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z: 0	39.3	59.1	38.0	57.2	37.5	56.3	31.2	46.9	33.7	50.6	32.0	48.1
1	38.7	58.2	37.3	56.1	36.2	54.6	31.0	46.6	33.1	49.8	31.4	47.3
2	36.9	55.8	35.0	53.0	32.9	49.8	31.0	46.7	31.4	47.4	29.8	45.0
3	32.8	49.9	30.5	46.4	28.9	44.1	26.3	40.3	29.0	43.8	27.4	41.4
4	27.8	42.4	25.2	38.6	23.6	36.2	21.8	33.5	26.0	39.4	24.5	37.2
5	23.2	35.5	20.6	31.6	18.9	29.2	17.2	26.7	22.8	34.7	21.4	32.6
6	19.2	29.5	16.7	25.7	15.1	23.4	13.5	21.0	19.7	30.1	18.4	28.1
7	15.7	24.3	13.5	20.8	12.1	18.8	10.7	16.7	16.8	25.7	15.6	23.8
8	12.9	19.9	10.9	16.9	9.83	15.2	8.59	13.3	14.1	21.6	13.0	20.0
9	10.8	16.7	9.09	14.0	8.09	12.5	7.02	10.8	11.9	18.3	11.0	16.8
10	9.21	14.1	7.65	11.8	6.78	10.4	5.85	9.07	10.2	15.6	9.39	14.3
11									8.84	13.5	8.09	12.3
Properties												
A_g, in.^{2}	3.25		2.49		2.09		1.69		3.25		2.89	
r_z, in.	0.633		0.636		0.638		0.639		0.679		0.681	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.
[f] Shape exceeds compact limit for flexure with F_y = 36 ksi.
Note: Heavy line indicates L_c/r_z equal to or greater than 200.



L3½

Table 4-12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

F_y = 36 ksi

Shape	L3½×3½×3/8		L3½×3½×5/16		L3½×3½×1/4[c]		L3½×3×1/2		L3½×3×7/16		L3½×3×3/8	
lb/ft	8.50		7.20		5.80		10.2		9.10		7.90	
Design	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z												
0	30.8	46.3	28.1	42.3	21.0	31.7	36.8	55.3	37.8	56.8	38.7	58.3
1	30.2	45.5	27.6	41.5	21.0	31.5	36.2	54.6	37.1	55.8	37.9	57.1
2	28.6	43.1	26.1	39.3	20.7	31.1	34.6	52.3	35.1	53.2	35.5	53.8
3	26.2	39.6	23.8	36.0	19.5	29.3	32.1	48.9	32.1	49.0	31.7	48.6
4	23.3	35.4	21.1	32.0	18.1	27.1	28.5	43.6	28.1	43.1	27.3	41.9
5	20.3	30.8	18.3	27.8	16.1	24.5	23.2	35.6	22.5	34.6	21.4	33.1
6	17.3	26.4	15.5	23.7	13.6	20.8	18.7	28.9	17.9	27.7	16.9	26.2
7	14.6	22.3	13.0	19.8	11.3	17.3	15.1	23.4	14.3	22.2	13.4	20.7
8	12.1	18.5	10.7	16.4	9.31	14.2	12.3	19.0	11.6	17.9	10.7	16.6
9	10.1	15.5	8.95	13.7	7.70	11.7	10.2	15.7	9.56	14.7	8.80	13.6
10	8.61	13.1	7.56	11.5	6.46	9.89	8.62	13.2	8.01	12.3	7.34	11.3
11	7.39	11.3	6.46	9.89	5.50	8.42						
Properties												
A_g, in.^{2}	2.50		2.10		1.70		3.02		2.67		2.32	
r_z, in.	0.683		0.685		0.688		0.618		0.620		0.622	

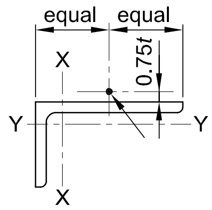
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_z equal to or greater than 200.

F_y = 36 ksi

Table 4-12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

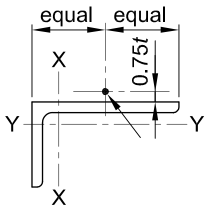


L3$^1/_2$

Shape	L3$^1/_2$×3×				L3$^1/_2$×2$^1/_2$×							
	$^5/_{16}$		$^1/_4$^{c}		$^1/_2$		$^3/_8$		$^5/_{16}$		$^1/_4$^{c}	
lb/ft	6.60		5.40		9.40		7.20		6.10		4.90	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z												
0	34.6	52.0	24.6	37.0	28.2	42.4	27.7	41.6	27.2	40.9	26.3	39.5
1	34.8	52.2	24.6	37.0	27.7	41.7	27.0	40.7	25.9	39.1	25.2	38.0
2	35.2	53.4	23.8	35.6	25.5	38.5	23.9	36.1	22.8	34.6	22.0	33.4
3	30.2	46.3	23.3	34.5	21.6	32.9	20.0	30.5	18.9	28.8	17.9	27.4
4	25.4	39.3	22.6	35.0	17.9	27.4	16.2	24.8	15.0	23.1	13.9	21.4
5	20.0	30.9	18.2	28.3	14.6	22.4	12.9	19.9	11.8	18.2	10.7	16.6
6	15.5	24.1	13.8	21.6	11.8	18.1	10.2	15.8	9.31	14.3	8.30	12.8
7	12.1	18.9	10.7	16.7	9.54	14.6	8.16	12.5	7.34	11.3	6.47	10.0
8	9.67	15.0	8.48	13.2	7.86	12.0	6.64	10.2	5.93	9.15	5.19	8.02
9	7.87	12.2	6.86	10.6							4.25	6.57
10	6.54	10.1	5.67	8.81								
Properties												
A_g, in.^{2}	1.95		1.58		2.77		2.12		1.79		1.45	
r_z, in.	0.624		0.628		0.532		0.535		0.538		0.541	

ASD	LRFD
Ω_c = 1.67	ϕ_c = 0.90

[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.
Note: Heavy line indicates L_c/r_z equal to or greater than 200.



L3

Table 4-12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

F_y = 36 ksi

Shape		L3×3×											
		1/2		7/16		3/8		5/16		1/4		3/16[c,f]	
lb/ft		9.40		8.30		7.20		6.10		4.90		3.71	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z	0	25.3	38.1	24.4	36.7	23.4	35.1	21.8	32.8	19.5	29.4	12.4	18.6
	1	24.7	37.2	23.8	35.8	22.8	34.3	21.2	32.0	19.1	28.7	12.3	18.5
	2	23.2	35.0	22.2	33.6	21.2	32.0	19.7	29.8	17.7	26.7	12.0	18.0
	3	20.9	31.7	20.0	30.2	19.0	28.7	17.5	26.6	15.6	23.7	11.1	16.6
	4	18.3	27.8	17.3	26.4	16.4	24.9	15.0	22.9	13.3	20.3	10.2	15.2
	5	15.6	23.8	14.7	22.4	13.8	21.0	12.5	19.1	11.0	16.8	9.19	13.6
	6	13.1	20.0	12.2	18.7	11.3	17.4	10.3	15.7	8.97	13.7	7.38	11.2
	7	10.8	16.5	10.0	15.3	9.24	14.1	8.30	12.7	7.16	10.9	5.84	8.93
	8	8.98	13.7	8.29	12.6	7.59	11.6	6.77	10.3	5.80	8.88	4.68	7.17
	9	7.57	11.5	6.95	10.6	6.33	9.69	5.62	8.60	4.78	7.32	3.84	5.87

Properties

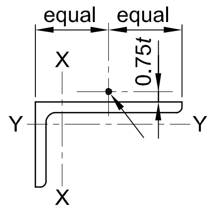
	1/2	7/16	3/8	5/16	1/4	3/16
A_g, in.^{2}	2.76	2.43	2.11	1.78	1.44	1.09
r_z, in.	0.580	0.580	0.581	0.583	0.585	0.586

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.
[f] Shape exceeds compact limit for flexure with F_y = 36 ksi.
Note: Heavy line indicates L_c/r_z equal to or greater than 200.

F_y = 36 ksi

Table 4-12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles



L3

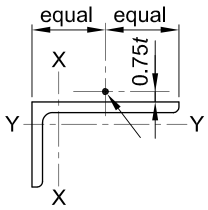
Shape		L3×2 1/2× 1/2		7/16		3/8		5/16		1/4		3/16[c,f]	
lb/ft		8.50		7.60		6.60		5.60		4.50		3.39	
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z	0	24.8	37.3	25.8	38.7	26.1	39.3	26.7	40.1	24.5	36.8	14.9	22.5
	1	24.4	36.7	25.2	38.0	25.5	38.4	25.8	39.0	24.8	37.2	14.7	22.1
	2	23.1	35.0	23.7	35.9	23.6	35.9	23.5	35.8	21.9	33.4	14.1	21.1
	3	21.3	32.5	21.4	32.8	20.8	31.8	20.0	30.7	18.1	27.8	14.3	20.7
	4	17.4	26.7	16.9	26.0	16.2	24.9	15.3	23.6	13.9	21.5	12.1	18.8
	5	13.8	21.2	13.3	20.4	12.5	19.3	11.6	17.9	10.3	16.0	8.77	13.6
	6	10.9	16.8	10.3	15.9	9.63	14.8	8.81	13.6	7.74	12.0	6.46	10.0
	7	8.71	13.4	8.20	12.6	7.54	11.6	6.82	10.5	5.94	9.21	4.90	7.64
	8	7.10	10.9	6.64	10.2	6.07	9.37	5.45	8.42	4.70	7.29	3.85	5.99

Properties

	1/2	7/16	3/8	5/16	1/4	3/16
A_g, in.^{2}	2.50	2.22	1.93	1.63	1.32	1.00
r_z, in.	0.516	0.516	0.517	0.518	0.520	0.521

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.
[f] Shape exceeds compact limit for flexure with F_y = 36 ksi.
Note: Heavy line indicates L_c/r_z equal to or greater than 200.



L3–L2 1/2

Table 4-12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

F_y = 36 ksi

Shape		L3×2×1/2		L3×2×3/8		L3×2×5/16		L3×2×1/4		L3×2×3/16[c,f]		L2 1/2×2 1/2×1/2	
lb/ft		7.70		5.90		5.00		4.10		3.07		7.70	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z	0	18.3	27.6	17.7	26.7	17.0	25.6	16.3	24.6	15.4	23.2	18.2	27.4
	1	17.5	26.3	16.7	25.2	15.9	24.0	15.4	23.2	14.4	21.8	17.7	26.6
	2	15.3	23.2	14.3	21.7	13.5	20.5	12.9	19.6	11.9	18.2	16.2	24.5
	3	12.7	19.3	11.6	17.7	10.8	16.5	10.0	15.4	9.11	13.9	14.2	21.5
	4	10.2	15.6	9.10	13.9	8.37	12.8	7.60	11.6	6.68	10.3	12.0	18.2
	5	8.00	12.2	6.98	10.7	6.33	9.74	5.65	8.71	4.86	7.52	9.85	15.0
	6	6.31	9.68	5.41	8.31	4.86	7.47	4.28	6.60	3.62	5.60	7.90	12.0
	7	5.09	7.82	4.31	6.62	3.84	5.91	3.35	5.17	2.80	4.33	6.42	9.82
	8											5.31	8.12

Properties

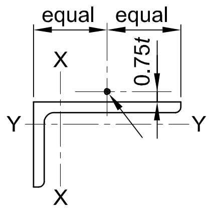
	L3×2×1/2	L3×2×3/8	L3×2×5/16	L3×2×1/4	L3×2×3/16	L2 1/2×2 1/2×1/2
A_g, in.^{2}	2.26	1.75	1.48	1.20	0.917	2.26
r_z, in.	0.425	0.426	0.428	0.431	0.435	0.481

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
[f] Shape exceeds compact limit for flexure with $F_y = 36$ ksi.
Note: Heavy line indicates L_c/r_z equal to or greater than 200.

F_y = 36 ksi

Table 4-12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles



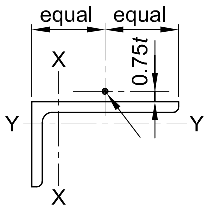
L2 1/2

Shape		L2 1/2×2 1/2×								L2 1/2×2×	
		3/8		5/16		1/4		3/16[c]		3/8	
lb/ft		5.90		5.00		4.10		3.07		5.30	
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z	0	17.0	25.6	16.1	24.2	14.8	22.3	11.6	17.5	16.9	25.4
	1	16.5	24.8	15.5	23.4	14.3	21.5	11.6	17.4	16.4	24.8
	2	14.9	22.6	14.0	21.2	12.8	19.4	11.0	16.4	15.0	22.8
	3	12.9	19.5	11.9	18.1	10.8	16.4	9.24	14.0	11.9	18.2
	4	10.6	16.2	9.80	14.9	8.78	13.3	7.37	11.2	8.99	13.8
	5	8.57	13.0	7.78	11.8	6.89	10.5	5.70	8.72	6.68	10.2
	6	6.74	10.3	6.06	9.27	5.30	8.11	4.32	6.62	5.08	7.82
	7	5.40	8.26	4.81	7.36	4.17	6.38	3.36	5.15	3.99	6.14
	8	4.41	6.75	3.90	5.97	3.36	5.14	2.69	4.11		
Properties											
A_g, in.^{2}		1.73		1.46		1.19		0.901		1.55	
r_z, in.		0.481		0.481		0.482		0.482		0.419	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_z equal to or greater than 200.



L2½

Table 4-12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

F_y = 36 ksi

Shape		L2½×2×						L2½×1½×			
		5/16		1/4		3/16[c]		1/4		3/16[c]	
lb/ft		4.50		3.62		2.75		3.19		2.44	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z	0	17.4	26.2	17.6	26.4	15.6	23.4	9.14	13.7	8.60	12.9
	1	16.8	25.3	16.7	25.3	15.6	23.5	8.34	12.5	7.86	11.8
	2	15.0	22.9	14.3	21.9	12.5	19.2	6.65	10.1	6.04	9.20
	3	11.5	17.7	10.7	16.5	9.57	14.7	4.88	7.47	4.27	6.56
	4	8.48	13.0	7.68	11.8	6.62	10.2	3.44	5.29	2.94	4.52
	5	6.19	9.55	5.51	8.53	4.66	7.23	2.50	3.85	2.10	3.23
	6	4.64	7.16	4.08	6.31	3.40	5.28				
	7	3.61	5.57	3.15	4.86	2.60	4.03				
Properties											
A_g, in.^{2}		1.32		1.07		0.818		0.947		0.724	
r_z, in.		0.420		0.423		0.426		0.321		0.324	

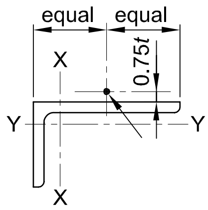
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi; tabulated values have been adjusted accordingly.

Note: Heavy line indicates L_c/r_z equal to or greater than 200.

F_y = 36 ksi

Table 4-12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles



L2

Shape		L2×2×									
		3/8		5/16		1/4		3/16		1/8[c,f]	
lb/ft		4.70		3.92		3.19		2.44		1.65	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c (ft), with respect to least radius of gyration, r_z	0	11.6	17.4	11.2	16.8	10.5	15.8	9.46	14.2	5.46	8.21
	1	11.0	16.7	10.6	16.0	10.0	15.0	8.93	13.4	5.37	8.07
	2	9.70	14.6	9.23	13.9	8.57	12.9	7.57	11.4	4.86	7.29
	3	7.92	12.0	7.42	11.2	6.79	10.3	5.91	8.99	4.23	6.32
	4	6.17	9.42	5.69	8.69	5.12	7.82	4.36	6.68	3.37	4.98
	5	4.67	7.14	4.24	6.48	3.75	5.74	3.14	4.81	2.40	3.58
	6	3.61	5.53	3.25	4.97	2.84	4.35	2.35	3.59	1.76	2.63
Properties											
A_g, in.^{2}		1.37		1.16		0.944		0.722		0.491	
r_z, in.		0.386		0.386		0.387		0.389		0.391	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi; tabulated values have been adjusted accordingly.
[f] Shape exceeds compact limit for flexure with $F_y = 36$ ksi.
Note: Heavy line indicates L_c/r_z equal to or greater than 200.

Table 4-13
Stiffness Reduction Factor τ_b

ASD $\frac{P_a}{A}$ / LRFD $\frac{P_u}{A}$	F_y, ksi 35		36		46		50		65		70	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
50	–	–	–	–	–	–	–	–	–	0.710	–	0.816
49	–	–	–	–	–	–	–	0.0784	–	0.742	–	0.840
48	–	–	–	–	–	–	–	0.154	–	0.773	–	0.862
47	–	–	–	–	–	–	–	0.226	–	0.801	–	0.882
46	–	–	–	–	–	–	–	0.294	–	0.827	–	0.901
45	–	–	–	–	–	0.0851	–	0.360	–	0.852	–	0.918
44	–	–	–	–	–	0.166	–	0.422	–	0.875	–	0.934
43	–	–	–	–	–	0.244	–	0.482	–	0.896	0.0674	0.948
42	–	–	–	–	–	0.318	–	0.538	–	0.915	0.154	0.960
41	–	–	–	–	–	0.388	–	0.590	–	0.932	0.236	0.971
40	–	–	–	–	–	0.454	–	0.640	0.0606	0.947	0.313	0.980
39	–	–	–	–	–	0.516	–	0.686	0.154	0.960	0.387	0.987
38	–	–	–	–	–	0.575	–	0.730	0.242	0.971	0.457	0.993
37	–	–	–	–	–	0.629	–	0.770	0.325	0.981	0.522	0.997
36	–	–	–	–	–	0.681	–	0.806	0.404	0.988	0.583	0.999
35	–	–	–	0.108	–	0.728	–	0.840	0.477	0.994	0.640	1.00
34	–	0.111	–	0.210	–	0.771	–	0.870	0.546	0.998	0.693	↓
33	–	0.216	–	0.306	–	0.811	–	0.898	0.610	1.00	0.741	
32	–	0.313	–	0.395	–	0.847	–	0.922	0.669	↓	0.786	
31	–	0.405	–	0.478	–	0.879	0.0317	0.942	0.723		0.826	
30	–	0.490	–	0.556	–	0.907	0.154	0.960	0.773		0.862	
29	–	0.568	–	0.627	–	0.932	0.267	0.974	0.817		0.894	
28	–	0.640	–	0.691	0.102	0.953	0.373	0.986	0.857		0.922	
27	–	0.705	–	0.750	0.229	0.970	0.470	0.994	0.892		0.945	
26	–	0.764	–	0.802	0.346	0.983	0.559	0.998	0.922		0.964	
25	–	0.816	–	0.849	0.454	0.992	0.640	1.00	0.947		0.980	
24	–	0.862	–	0.889	0.552	0.998	0.713	↓	0.967		0.991	
23	–	0.901	–	0.923	0.640	1.00	0.777		0.982		0.997	
22	–	0.934	0.087	0.951	0.719	↓	0.834		0.993		1.00	
21	0.154	0.960	0.249	0.972	0.788		0.882		0.999		↓	
20	0.313	0.980	0.395	0.988	0.847		0.922		1.00			
19	0.457	0.993	0.525	0.997	0.896		0.953		↓			
18	0.583	0.999	0.640	1.00	0.936		0.977					
17	0.693	1.00	0.739	↓	0.967		0.992					
16	0.786	1.00	0.822		0.987		0.999					
15	0.862	1.00	0.889		0.998		1.00					
14	0.922	1.00	0.940		0.999		↓					
13	0.964	1.00	0.976		1.00							
12	0.991	1.00	0.996		↓							
11	1.00	1.00	1.00									
10	1.00	1.00	1.00									

– Indicates the stiffness reduction parameter is not applicable because the required strength exceeds the available strength for $L_c/r = 0$.
$A = A_g$ for members not controlled by slender element buckling, in.^{2}
$= A_e$ as defined in AISC *Specification* Section E7 for members controlled by slender element buckling, in.^{2}

Table 4-14
Available Critical Stress for Compression Members

	F_y = 35 ksi		F_y = 36 ksi		F_y = 46 ksi		F_y = 50 ksi		F_y = 65 ksi		F_y = 70 ksi	
$\frac{L_c}{r}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$
	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
1	21.0	31.5	21.6	32.4	27.5	41.4	29.9	45.0	38.9	58.5	41.9	63.0
2	21.0	31.5	21.6	32.4	27.5	41.4	29.9	45.0	38.9	58.5	41.9	63.0
3	20.9	31.5	21.5	32.4	27.5	41.4	29.9	45.0	38.9	58.4	41.9	62.9
4	20.9	31.5	21.5	32.4	27.5	41.4	29.9	44.9	38.9	58.4	41.8	62.9
5	20.9	31.5	21.5	32.4	27.5	41.3	29.9	44.9	38.8	58.4	41.8	62.8
6	20.9	31.4	21.5	32.3	27.5	41.3	29.9	44.9	38.8	58.3	41.8	62.8
7	20.9	31.4	21.5	32.3	27.5	41.3	29.8	44.8	38.7	58.2	41.7	62.7
8	20.9	31.4	21.5	32.3	27.4	41.2	29.8	44.8	38.7	58.1	41.6	62.6
9	20.9	31.4	21.5	32.3	27.4	41.2	29.8	44.7	38.6	58.1	41.6	62.5
10	20.9	31.3	21.4	32.2	27.4	41.1	29.7	44.7	38.6	57.9	41.5	62.4
11	20.8	31.3	21.4	32.2	27.3	41.1	29.7	44.6	38.5	57.8	41.4	62.2
12	20.8	31.3	21.4	32.2	27.3	41.0	29.6	44.5	38.4	57.7	41.3	62.1
13	20.8	31.2	21.4	32.1	27.2	40.9	29.6	44.4	38.3	57.6	41.2	61.9
14	20.7	31.2	21.3	32.1	27.2	40.9	29.5	44.4	38.2	57.4	41.1	61.7
15	20.7	31.1	21.3	32.0	27.1	40.8	29.5	44.3	38.1	57.3	41.0	61.6
16	20.7	31.1	21.3	32.0	27.1	40.7	29.4	44.2	38.0	57.1	40.8	61.4
17	20.7	31.0	21.2	31.9	27.0	40.6	29.3	44.1	37.9	56.9	40.7	61.2
18	20.6	31.0	21.2	31.9	27.0	40.5	29.2	43.9	37.7	56.7	40.5	60.9
19	20.6	30.9	21.2	31.8	26.9	40.4	29.2	43.8	37.6	56.5	40.4	60.7
20	20.5	30.9	21.1	31.7	26.8	40.3	29.1	43.7	37.5	56.3	40.2	60.5
21	20.5	30.8	21.1	31.7	26.7	40.2	29.0	43.6	37.3	56.1	40.1	60.2
22	20.4	30.7	21.0	31.6	26.7	40.1	28.9	43.4	37.2	55.9	39.9	60.0
23	20.4	30.7	21.0	31.5	26.6	40.0	28.8	43.3	37.0	55.6	39.7	59.7
24	20.3	30.6	20.9	31.4	26.5	39.8	28.7	43.1	36.8	55.4	39.5	59.4
25	20.3	30.5	20.9	31.4	26.4	39.7	28.6	43.0	36.7	55.1	39.3	59.1
26	20.2	30.4	20.8	31.3	26.3	39.6	28.5	42.8	36.5	54.9	39.1	58.8
27	20.2	30.3	20.7	31.2	26.2	39.4	28.4	42.7	36.3	54.6	38.9	58.5
28	20.1	30.3	20.7	31.1	26.1	39.3	28.3	42.5	36.1	54.3	38.7	58.1
29	20.1	30.2	20.6	31.0	26.0	39.1	28.2	42.3	35.9	54.0	38.5	57.8
30	20.0	30.1	20.6	30.9	25.9	39.0	28.0	42.1	35.7	53.7	38.2	57.5
31	20.0	30.0	20.5	30.8	25.8	38.8	27.9	41.9	35.5	53.4	38.0	57.1
32	19.9	29.9	20.4	30.7	25.7	38.6	27.8	41.8	35.3	53.1	37.7	56.7
33	19.8	29.8	20.4	30.6	25.6	38.5	27.7	41.6	35.1	52.7	37.5	56.4
34	19.8	29.7	20.3	30.5	25.5	38.3	27.5	41.4	34.9	52.4	37.2	56.0
35	19.7	29.6	20.2	30.4	25.4	38.1	27.4	41.2	34.6	52.1	37.0	55.6
36	19.6	29.5	20.1	30.3	25.2	37.9	27.2	40.9	34.4	51.7	36.7	55.2
37	19.5	29.4	20.1	30.1	25.1	37.8	27.1	40.7	34.2	51.4	36.4	54.8
38	19.5	29.3	20.0	30.0	25.0	37.6	26.9	40.5	33.9	51.0	36.2	54.3
39	19.4	29.1	19.9	29.9	24.9	37.4	26.8	40.3	33.7	50.6	35.9	53.9
40	19.3	29.0	19.8	29.8	24.7	37.2	26.6	40.0	33.4	50.2	35.6	53.5

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Table 4-14 (continued)
Available Critical Stress for Compression Members

L_c/r	F_y = 35 ksi		F_y = 36 ksi		F_y = 46 ksi		F_y = 50 ksi		F_y = 65 ksi		F_y = 70 ksi	
	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$
	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
41	19.2	28.9	19.7	29.7	24.6	37.0	26.5	39.8	33.2	49.9	35.3	53.0
42	19.2	28.8	19.6	29.5	24.5	36.8	26.3	39.5	32.9	49.5	35.0	52.6
43	19.1	28.7	19.6	29.4	24.3	36.6	26.2	39.3	32.6	49.1	34.7	52.1
44	19.0	28.5	19.5	29.3	24.2	36.3	26.0	39.1	32.4	48.7	34.4	51.7
45	18.9	28.4	19.4	29.1	24.0	36.1	25.8	38.8	32.1	48.3	34.1	51.2
46	18.8	28.3	19.3	29.0	23.9	35.9	25.6	38.5	31.8	47.8	33.8	50.7
47	18.7	28.1	19.2	28.9	23.8	35.7	25.5	38.3	31.6	47.4	33.4	50.3
48	18.6	28.0	19.1	28.7	23.6	35.4	25.3	38.0	31.3	47.0	33.1	49.8
49	18.5	27.9	19.0	28.5	23.4	35.2	25.1	37.7	31.0	46.6	32.8	49.3
50	18.4	27.7	18.9	28.4	23.3	35.0	24.9	37.5	30.7	46.1	32.5	48.8
51	18.3	27.6	18.8	28.3	23.1	34.8	24.8	37.2	30.4	45.7	32.1	48.3
52	18.3	27.4	18.7	28.1	23.0	34.5	24.6	36.9	30.1	45.2	31.8	47.8
53	18.2	27.3	18.6	28.0	22.8	34.3	24.4	36.7	29.8	44.8	31.4	47.3
54	18.1	27.1	18.5	27.8	22.6	34.0	24.2	36.4	29.5	44.3	31.1	46.7
55	18.0	27.0	18.4	27.6	22.5	33.8	24.0	36.1	29.2	43.9	30.8	46.2
56	17.9	26.8	18.3	27.5	22.3	33.5	23.8	35.8	28.9	43.4	30.4	45.7
57	17.7	26.7	18.2	27.3	22.1	33.3	23.6	35.5	28.6	43.0	30.1	45.2
58	17.6	26.5	18.1	27.1	22.0	33.0	23.4	35.2	28.3	42.5	29.7	44.6
59	17.5	26.4	17.9	27.0	21.8	32.8	23.2	34.9	28.0	42.0	29.4	44.1
60	17.4	26.2	17.8	26.8	21.6	32.5	23.0	34.6	27.6	41.5	29.0	43.6
61	17.3	26.0	17.7	26.6	21.4	32.2	22.8	34.3	27.3	41.1	28.6	43.0
62	17.2	25.9	17.6	26.5	21.3	32.0	22.6	34.0	27.0	40.6	28.3	42.5
63	17.1	25.7	17.5	26.3	21.1	31.7	22.4	33.7	26.7	40.1	27.9	42.0
64	17.0	25.5	17.4	26.1	20.9	31.4	22.2	33.4	26.4	39.6	27.6	41.4
65	16.9	25.4	17.3	25.9	20.7	31.2	22.0	33.0	26.0	39.2	27.2	40.9
66	16.8	25.2	17.1	25.8	20.5	30.9	21.8	32.7	25.7	38.7	26.8	40.3
67	16.7	25.0	17.0	25.6	20.4	30.6	21.6	32.4	25.4	38.2	26.5	39.8
68	16.5	24.9	16.9	25.4	20.2	30.3	21.4	32.1	25.1	37.7	26.1	39.2
69	16.4	24.7	16.8	25.2	20.0	30.1	21.1	31.8	24.8	37.2	25.7	38.7
70	16.3	24.5	16.7	25.0	19.8	29.8	20.9	31.4	24.4	36.7	25.4	38.2
71	16.2	24.3	16.5	24.8	19.6	29.5	20.7	31.1	24.1	36.2	25.0	37.6
72	16.1	24.2	16.4	24.7	19.4	29.2	20.5	30.8	23.8	35.7	24.7	37.1
73	16.0	24.0	16.3	24.5	19.2	28.9	20.3	30.5	23.5	35.3	24.3	36.5
74	15.8	23.8	16.2	24.3	19.1	28.6	20.1	30.2	23.1	34.8	23.9	36.0
75	15.7	23.6	16.0	24.1	18.9	28.4	19.8	29.8	22.8	34.3	23.6	35.4
76	15.6	23.4	15.9	23.9	18.7	28.1	19.6	29.5	22.5	33.8	23.2	34.9
77	15.5	23.3	15.8	23.7	18.5	27.8	19.4	29.2	22.2	33.3	22.8	34.3
78	15.4	23.1	15.6	23.5	18.3	27.5	19.2	28.8	21.8	32.8	22.5	33.8
79	15.2	22.9	15.5	23.3	18.1	27.2	19.0	28.5	21.5	32.3	22.1	33.3
80	15.1	22.7	15.4	23.1	17.9	26.9	18.8	28.2	21.2	31.8	21.8	32.7

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Table 4-14 (continued)
Available Critical Stress for Compression Members

$\frac{L_c}{r}$	F_y = 35 ksi		F_y = 36 ksi		F_y = 46 ksi		F_y = 50 ksi		F_y = 65 ksi		F_y = 70 ksi	
	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$
	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
81	15.0	22.5	15.3	22.9	17.7	26.6	18.5	27.9	20.9	31.4	21.4	32.2
82	14.9	22.3	15.1	22.7	17.5	26.3	18.3	27.5	20.5	30.9	21.1	31.7
83	14.7	22.1	15.0	22.5	17.3	26.0	18.1	27.2	20.2	30.4	20.7	31.1
84	14.6	22.0	14.9	22.3	17.1	25.8	17.9	26.9	19.9	29.9	20.4	30.6
85	14.5	21.8	14.7	22.1	16.9	25.5	17.7	26.5	19.6	29.4	20.0	30.1
86	14.4	21.6	14.6	22.0	16.7	25.2	17.4	26.2	19.3	29.0	19.7	29.5
87	14.2	21.4	14.5	21.8	16.6	24.9	17.2	25.9	19.0	28.5	19.3	29.0
88	14.1	21.2	14.3	21.6	16.4	24.6	17.0	25.5	18.6	28.0	19.0	28.5
89	14.0	21.0	14.2	21.4	16.2	24.3	16.8	25.2	18.3	27.6	18.6	28.0
90	13.8	20.8	14.1	21.2	16.0	24.0	16.6	24.9	18.0	27.1	18.3	27.5
91	13.7	20.6	13.9	21.0	15.8	23.7	16.3	24.6	17.7	26.6	18.0	27.0
92	13.6	20.4	13.8	20.8	15.6	23.4	16.1	24.2	17.4	26.2	17.6	26.5
93	13.5	20.2	13.7	20.5	15.4	23.1	15.9	23.9	17.1	25.7	17.3	26.0
94	13.3	20.0	13.5	20.3	15.2	22.8	15.7	23.6	16.8	25.3	17.0	25.5
95	13.2	19.9	13.4	20.1	15.0	22.6	15.5	23.3	16.5	24.8	16.6	25.0
96	13.1	19.7	13.3	19.9	14.8	22.3	15.3	22.9	16.2	24.4	16.3	24.5
97	13.0	19.5	13.1	19.7	14.6	22.0	15.0	22.6	15.9	23.9	16.0	24.0
98	12.8	19.3	13.0	19.5	14.4	21.7	14.8	22.3	15.6	23.5	15.7	23.5
99	12.7	19.1	12.9	19.3	14.2	21.4	14.6	22.0	15.3	23.0	15.3	23.0
100	12.6	18.9	12.7	19.1	14.1	21.1	14.4	21.7	15.0	22.6	15.0	22.6
101	12.4	18.7	12.6	18.9	13.9	20.8	14.2	21.3	14.7	22.1	14.7	22.1
102	12.3	18.5	12.5	18.7	13.7	20.6	14.0	21.0	14.4	21.7	14.4	21.7
103	12.2	18.3	12.3	18.5	13.5	20.3	13.8	20.7	14.2	21.3	14.2	21.3
104	12.1	18.1	12.2	18.3	13.3	20.0	13.6	20.4	13.9	20.9	13.9	20.9
105	11.9	17.9	12.1	18.1	13.1	19.7	13.4	20.1	13.6	20.5	13.6	20.5
106	11.8	17.7	11.9	17.9	12.9	19.4	13.2	19.8	13.4	20.1	13.4	20.1
107	11.7	17.5	11.8	17.7	12.8	19.2	13.0	19.5	13.1	19.7	13.1	19.7
108	11.5	17.3	11.7	17.5	12.6	18.9	12.8	19.2	12.9	19.4	12.9	19.4
109	11.4	17.2	11.5	17.3	12.4	18.6	12.6	18.9	12.7	19.0	12.7	19.0
110	11.3	17.0	11.4	17.1	12.2	18.3	12.4	18.6	12.4	18.7	12.4	18.7
111	11.2	16.8	11.3	16.9	12.0	18.1	12.2	18.3	12.2	18.3	12.2	18.3
112	11.0	16.6	11.1	16.7	11.8	17.8	12.0	18.0	12.0	18.0	12.0	18.0
113	10.9	16.4	11.0	16.5	11.7	17.5	11.8	17.7	11.8	17.7	11.8	17.7
114	10.8	16.2	10.9	16.3	11.5	17.3	11.6	17.4	11.6	17.4	11.6	17.4
115	10.7	16.0	10.7	16.2	11.3	17.0	11.4	17.1	11.4	17.1	11.4	17.1
116	10.5	15.8	10.6	16.0	11.1	16.7	11.2	16.8	11.2	16.8	11.2	16.8
117	10.4	15.6	10.5	15.8	11.0	16.5	11.0	16.5	11.0	16.5	11.0	16.5
118	10.3	15.5	10.4	15.6	10.8	16.2	10.8	16.2	10.8	16.2	10.8	16.2
119	10.2	15.3	10.2	15.4	10.6	16.0	10.6	16.0	10.6	16.0	10.6	16.0
120	10.0	15.1	10.1	15.2	10.4	15.7	10.4	15.7	10.4	15.7	10.4	15.7

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Table 4-14 (continued)
Available Critical Stress for Compression Members

L_c/r	$F_y = 35$ ksi		$F_y = 36$ ksi		$F_y = 46$ ksi		$F_y = 50$ ksi		$F_y = 65$ ksi		$F_y = 70$ ksi	
	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$
	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
121	9.91	14.9	10.0	15.0	10.3	15.4	10.3	15.4	10.3	15.4	10.3	15.4
122	9.79	14.7	9.85	14.8	10.1	15.2	10.1	15.2	10.1	15.2	10.1	15.2
123	9.67	14.5	9.72	14.6	9.94	14.9	9.94	14.9	9.94	14.9	9.94	14.9
124	9.55	14.3	9.59	14.4	9.78	14.7	9.78	14.7	9.78	14.7	9.78	14.7
125	9.43	14.2	9.47	14.2	9.62	14.5	9.62	14.5	9.62	14.5	9.62	14.5
126	9.31	14.0	9.35	14.0	9.47	14.2	9.47	14.2	9.47	14.2	9.47	14.2
127	9.19	13.8	9.22	13.9	9.32	14.0	9.32	14.0	9.32	14.0	9.32	14.0
128	9.07	13.6	9.10	13.7	9.17	13.8	9.17	13.8	9.17	13.8	9.17	13.8
129	8.95	13.4	8.98	13.5	9.03	13.6	9.03	13.6	9.03	13.6	9.03	13.6
130	8.83	13.3	8.86	13.3	8.89	13.4	8.89	13.4	8.89	13.4	8.89	13.4
131	8.71	13.1	8.73	13.1	8.76	13.2	8.76	13.2	8.76	13.2	8.76	13.2
132	8.60	12.9	8.61	12.9	8.63	13.0	8.63	13.0	8.63	13.0	8.63	13.0
133	8.48	12.7	8.49	12.8	8.50	12.8	8.50	12.8	8.50	12.8	8.50	12.8
134	8.37	12.6	8.37	12.6	8.37	12.6	8.37	12.6	8.37	12.6	8.37	12.6
135	8.25	12.4	8.25	12.4	8.25	12.4	8.25	12.4	8.25	12.4	8.25	12.4
136	8.13	12.2	8.13	12.2	8.13	12.2	8.13	12.2	8.13	12.2	8.13	12.2
137	8.01	12.0	8.01	12.0	8.01	12.0	8.01	12.0	8.01	12.0	8.01	12.0
138	7.89	11.9	7.89	11.9	7.89	11.9	7.89	11.9	7.89	11.9	7.89	11.9
139	7.78	11.7	7.78	11.7	7.78	11.7	7.78	11.7	7.78	11.7	7.78	11.7
140	7.67	11.5	7.67	11.5	7.67	11.5	7.67	11.5	7.67	11.5	7.67	11.5
141	7.56	11.4	7.56	11.4	7.56	11.4	7.56	11.4	7.56	11.4	7.56	11.4
142	7.45	11.2	7.45	11.2	7.45	11.2	7.45	11.2	7.45	11.2	7.45	11.2
143	7.35	11.0	7.35	11.0	7.35	11.0	7.35	11.0	7.35	11.0	7.35	11.0
144	7.25	10.9	7.25	10.9	7.25	10.9	7.25	10.9	7.25	10.9	7.25	10.9
145	7.15	10.7	7.15	10.7	7.15	10.7	7.15	10.7	7.15	10.7	7.15	10.7
146	7.05	10.6	7.05	10.6	7.05	10.6	7.05	10.6	7.05	10.6	7.05	10.6
147	6.96	10.5	6.96	10.5	6.96	10.5	6.96	10.5	6.96	10.5	6.96	10.5
148	6.86	10.3	6.86	10.3	6.86	10.3	6.86	10.3	6.86	10.3	6.86	10.3
149	6.77	10.2	6.77	10.2	6.77	10.2	6.77	10.2	6.77	10.2	6.77	10.2
150	6.68	10.0	6.68	10.0	6.68	10.0	6.68	10.0	6.68	10.0	6.68	10.0
151	6.59	9.91	6.59	9.91	6.59	9.91	6.59	9.91	6.59	9.91	6.59	9.91
152	6.51	9.78	6.51	9.78	6.51	9.78	6.51	9.78	6.51	9.78	6.51	9.78
153	6.42	9.65	6.42	9.65	6.42	9.65	6.42	9.65	6.42	9.65	6.42	9.65
154	6.34	9.53	6.34	9.53	6.34	9.53	6.34	9.53	6.34	9.53	6.34	9.53
155	6.26	9.40	6.26	9.40	6.26	9.40	6.26	9.40	6.26	9.40	6.26	9.40
156	6.18	9.28	6.18	9.28	6.18	9.28	6.18	9.28	6.18	9.28	6.18	9.28
157	6.10	9.17	6.10	9.17	6.10	9.17	6.10	9.17	6.10	9.17	6.10	9.17
158	6.02	9.05	6.02	9.05	6.02	9.05	6.02	9.05	6.02	9.05	6.02	9.05
159	5.95	8.94	5.95	8.94	5.95	8.94	5.95	8.94	5.95	8.94	5.95	8.94
160	5.87	8.82	5.87	8.82	5.87	8.82	5.87	8.82	5.87	8.82	5.87	8.82

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Table 4-14 (continued)
Available Critical Stress for Compression Members

	F_y = 35 ksi		F_y = 36 ksi		F_y = 46 ksi		F_y = 50 ksi		F_y = 65 ksi		F_y = 70 ksi	
$\frac{L_c}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD
161	5.80	8.72	5.80	8.72	5.80	8.72	5.80	8.72	5.80	8.72	5.80	8.72
162	5.73	8.61	5.73	8.61	5.73	8.61	5.73	8.61	5.73	8.61	5.73	8.61
163	5.66	8.50	5.66	8.50	5.66	8.50	5.66	8.50	5.66	8.50	5.66	8.50
164	5.59	8.40	5.59	8.40	5.59	8.40	5.59	8.40	5.59	8.40	5.59	8.40
165	5.52	8.30	5.52	8.30	5.52	8.30	5.52	8.30	5.52	8.30	5.52	8.30
166	5.45	8.20	5.45	8.20	5.45	8.20	5.45	8.20	5.45	8.20	5.45	8.20
167	5.39	8.10	5.39	8.10	5.39	8.10	5.39	8.10	5.39	8.10	5.39	8.10
168	5.33	8.00	5.33	8.00	5.33	8.00	5.33	8.00	5.33	8.00	5.33	8.00
169	5.25	7.89	5.25	7.89	5.25	7.89	5.25	7.89	5.25	7.89	5.25	7.89
170	5.20	7.82	5.20	7.82	5.20	7.82	5.20	7.82	5.20	7.82	5.20	7.82
171	5.14	7.73	5.14	7.73	5.14	7.73	5.14	7.73	5.14	7.73	5.14	7.73
172	5.08	7.64	5.08	7.64	5.08	7.64	5.08	7.64	5.08	7.64	5.08	7.64
173	5.02	7.55	5.02	7.55	5.02	7.55	5.02	7.55	5.02	7.55	5.02	7.55
174	4.96	7.46	4.96	7.46	4.96	7.46	4.96	7.46	4.96	7.46	4.96	7.46
175	4.91	7.38	4.91	7.38	4.91	7.38	4.91	7.38	4.91	7.38	4.91	7.38
176	4.85	7.29	4.85	7.29	4.85	7.29	4.85	7.29	4.85	7.29	4.85	7.29
177	4.80	7.21	4.80	7.21	4.80	7.21	4.80	7.21	4.80	7.21	4.80	7.21
178	4.74	7.13	4.74	7.13	4.74	7.13	4.74	7.13	4.74	7.13	4.74	7.13
179	4.69	7.05	4.69	7.05	4.69	7.05	4.69	7.05	4.69	7.05	4.69	7.05
180	4.64	6.97	4.64	6.97	4.64	6.97	4.64	6.97	4.64	6.97	4.64	6.97
181	4.59	6.90	4.59	6.90	4.59	6.90	4.59	6.90	4.59	6.90	4.59	6.90
182	4.54	6.82	4.54	6.82	4.54	6.82	4.54	6.82	4.54	6.82	4.54	6.82
183	4.49	6.75	4.49	6.75	4.49	6.75	4.49	6.75	4.49	6.75	4.49	6.75
184	4.44	6.67	4.44	6.67	4.44	6.67	4.44	6.67	4.44	6.67	4.44	6.67
185	4.39	6.60	4.39	6.60	4.39	6.60	4.39	6.60	4.39	6.60	4.39	6.60
186	4.34	6.53	4.34	6.53	4.34	6.53	4.34	6.53	4.34	6.53	4.34	6.53
187	4.30	6.46	4.30	6.46	4.30	6.46	4.30	6.46	4.30	6.46	4.30	6.46
188	4.25	6.39	4.25	6.39	4.25	6.39	4.25	6.39	4.25	6.39	4.25	6.39
189	4.21	6.32	4.21	6.32	4.21	6.32	4.21	6.32	4.21	6.32	4.21	6.32
190	4.16	6.26	4.16	6.26	4.16	6.26	4.16	6.26	4.16	6.26	4.16	6.26
191	4.12	6.19	4.12	6.19	4.12	6.19	4.12	6.19	4.12	6.19	4.12	6.19
192	4.08	6.13	4.08	6.13	4.08	6.13	4.08	6.13	4.08	6.13	4.08	6.13
193	4.04	6.06	4.04	6.06	4.04	6.06	4.04	6.06	4.04	6.06	4.04	6.06
194	3.99	6.00	3.99	6.00	3.99	6.00	3.99	6.00	3.99	6.00	3.99	6.00
195	3.95	5.94	3.95	5.94	3.95	5.94	3.95	5.94	3.95	5.94	3.95	5.94
196	3.91	5.88	3.91	5.88	3.91	5.88	3.91	5.88	3.91	5.88	3.91	5.88
197	3.87	5.82	3.87	5.82	3.87	5.82	3.87	5.82	3.87	5.82	3.87	5.82
198	3.83	5.76	3.83	5.76	3.83	5.76	3.83	5.76	3.83	5.76	3.83	5.76
199	3.80	5.70	3.80	5.70	3.80	5.70	3.80	5.70	3.80	5.70	3.80	5.70
200	3.76	5.65	3.76	5.65	3.76	5.65	3.76	5.65	3.76	5.65	3.76	5.65

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

PART 5

DESIGN OF TENSION MEMBERS

SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of members subject to static axial tension. For fatigue applications, see AISC *Specification* Appendix 3. For the design of members subject to eccentric tension or combined tension and flexure, see Part 6.

GROSS AREA, NET AREA AND EFFECTIVE NET AREA

In the determination of the available strength of a tension member, the gross area, A_g, is needed for the tensile yielding limit state and the effective net area, A_e, is needed for the tensile rupture limit state, as stipulated in AISC *Specification* Section D2.

Gross Area

The gross area, A_g, is determined as specified in AISC *Specification* Section B4.3a.

Effective Net Area

The effective net area, A_e, is determined from AISC *Specification* Section D3 by multiplying the net area, A_n, by the shear lag coefficient, U, where A_n is determined for tension members per AISC *Specification* Section B4.3b and U is determined from AISC *Specification* Table D3.1. Shear lag parameters are illustrated in AISC *Specification* Commentary Figures C-D3.1, C-D3.2 and C-D3.4.

TENSILE STRENGTH

The limit state of tensile yielding will control the available tensile strength over tensile rupture when the following relationship is satisfied:

LRFD	ASD
$0.90F_yA_g \le 0.75F_uA_e$ (5-1a)	$\dfrac{F_yA_g}{1.67} \le \dfrac{F_uA_e}{2.00}$ (5-1b)

These expressions are both reduced to:

$$\frac{A_e}{A_g} \ge 1.2\frac{F_y}{F_u} \tag{5-2}$$

Otherwise, the limit state of tensile rupture will control over tensile yielding.

Design of tension members without consideration of the tensile rupture limit state may require connections with reinforcement, resulting in a design that may have a lower tonnage but a higher overall cost. It is generally more economical to design larger members that do not require reinforcement.

Yielding Limit State

The available tensile strength due to tensile yielding, which must equal or exceed the required strength, P_u or P_a, is determined for tension members, per AISC *Specification* Section D2(a), using Equation D2-1.

Rupture Limit State

The available tensile strength due to tensile rupture, which must equal or exceed the required strength, P_u or P_a, is determined for tension members, per AISC *Specification* Section D2(b), using Equation D2-2.

Use of Table 6-2 for Design of Tension Members

Table 6-2 may be used for design of tension members. This table includes all W-shapes. Values of available strength for the tensile rupture limit state are based on the assumption that $A_e = 0.75A_g$. Therefore, if $A_e < 0.75A_g$, the available strength based on the rupture limit state must be calculated. See Part 6 for additional information on using Table 6-2 for design of tension members.

OTHER SPECIFICATION REQUIREMENTS AND DESIGN CONSIDERATIONS

Special Requirements for Heavy Shapes and Plates

For tension members with complete-joint-penetration groove welded joints and made from heavy shapes with a flange thickness exceeding 2 in. or built-up sections consisting of plates with a thickness exceeding 2 in., see AISC *Specification* Section A3.1c and Section A3.1d.

Slenderness

Tension member slenderness ratio, L/r, should preferably be limited to a maximum of 300 per the User Note in AISC *Specification* Section D1. The intent of this recommendation is explained in the corresponding Commentary.

DESIGN TABLE DISCUSSION

Available tensile strengths for various types of tension members (see individual descriptions in the text to follow) are given in Tables 5-1 through 5-8 and in Table 6-2 for the limit states of tensile yielding and tensile rupture. In each case, the tabulated values for available tensile rupture strength are based upon the assumption that $A_e = 0.75A_g$, which is arbitrarily selected as a value that is practical to achieve with typical end connections. Such consideration of the effective net area during the design of the member will simplify the design of its end connections, which can be difficult to configure and costly if tension members are selected based upon available tensile yielding strength only, without considering the reduction in strength due to the connection.

When $A_e > 0.75A_g$, either the tabulated values for available tensile rupture strength can be used conservatively or the available tensile rupture strength can be calculated based upon the actual value of A_e. When $A_e < 0.75A_g$, the tabulated values of the available tensile rupture strength cannot be used but rather must be calculated based upon the actual value of A_e.

Table 5-1. Available Strength in Axial Tension—W-Shapes

Available strengths in axial tension are given for W-shapes with $F_y = 50$ ksi and $F_u = 65$ ksi (ASTM A992). Note that tensile rupture will control over tensile yielding for W-shapes with $F_y = 50$ ksi and $F_u = 65$ ksi when $A_e/A_g < 0.923$. Otherwise, tensile yielding will control over tensile rupture.

Table 5-2. Available Strength in Axial Tension—Angles

Available strengths in axial tension are given for single angles with F_y = 36 ksi and F_u = 58 ksi (ASTM A36). Note that tensile rupture will control over tensile yielding for single angles with F_y = 36 ksi and F_u = 58 ksi when A_e/A_g < 0.745. Otherwise, tensile yielding will control over tensile rupture.

Table 5-3. Available Strength in Axial Tension—WT-Shapes

Table 5-3 is similar to Table 5-1, except that it covers WT-shapes with F_y = 50 ksi and F_u = 65 ksi (ASTM A992).

Table 5-4. Available Strength in Axial Tension—Rectangular HSS

Available strengths in axial tension are given for rectangular HSS with F_y = 50 ksi and F_u = 62 ksi (ASTM A500 Grade C). Note that tensile rupture will control over tensile yielding for rectangular HSS with F_y = 50 ksi and F_u = 62 ksi when A_e/A_g < 0.968. Otherwise, tensile yielding will control over tensile rupture.

Table 5-5. Available Strength in Axial Tension—Square HSS

Table 5-5 is similar to Table 5-4, except that it covers square HSS with F_y = 50 ksi and F_u = 62 ksi (ASTM A500 Grade C).

Table 5-6. Available Strength in Axial Tension—Round HSS

Available strengths in axial tension are given for round HSS with F_y = 46 ksi and F_u = 62 ksi (ASTM A500 Grade C). Note that tensile rupture will control over tensile yielding for round HSS with F_y = 46 ksi and F_u = 62 ksi when A_e/A_g < 0.890. Otherwise, tensile yielding will control over tensile rupture.

Table 5-7. Available Strength in Axial Tension—Pipe

Available strengths in axial tension are given for pipe with F_y = 35 ksi and F_u = 60 ksi (ASTM A53 Grade B). Note that tensile rupture will control over tensile yielding for pipe with F_y = 35 ksi and F_u = 60 ksi when A_e/A_g < 0.700. Otherwise, tensile yielding will control over tensile rupture.

Table 5-8. Available Strength in Axial Tension—Double Angles

Available strengths in axial tension are given for double angles with F_y = 36 ksi and F_u = 58 ksi (ASTM A36). Note that tensile rupture will control over tensile yielding for double angles with F_y = 36 ksi and F_u = 58 ksi when A_e/A_g < 0.745. Otherwise, tensile yielding will control over tensile rupture.

F_y = 50 ksi
F_u = 65 ksi

Table 5-1
Available Strength in Axial Tension
W-Shapes

W44–W40

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips P_n/Ω_t	Yielding kips $\phi_t P_n$	Rupture kips P_n/Ω_t	Rupture kips $\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
W44×335	98.5	73.9	2950	4430	2400	3600
×290	85.4	64.1	2560	3840	2080	3120
×262	77.2	57.9	2310	3470	1880	2820
×230	67.8	50.9	2030	3050	1650	2480
W40×655[h]	193	145	5780	8690	4710	7070
×593[h]	174	131	5210	7830	4260	6390
×503[h]	148	111	4430	6660	3610	5410
×431[h]	127	95.3	3800	5720	3100	4650
×397[h]	117	87.8	3500	5270	2850	4280
×372[h]	110	82.5	3290	4950	2680	4020
×362[h]	106	79.5	3170	4770	2580	3880
×324	95.3	71.5	2850	4290	2320	3490
×297	87.3	65.5	2610	3930	2130	3190
×277	81.5	61.1	2440	3670	1990	2980
×249	73.5	55.1	2200	3310	1790	2690
×215	63.5	47.6	1900	2860	1550	2320
×199	58.8	44.1	1760	2650	1430	2150
W40×392[h]	116	87.0	3470	5220	2830	4240
×331[h]	97.7	73.3	2930	4400	2380	3570
×327[h]	95.9	71.9	2870	4320	2340	3510
×294	86.2	64.7	2580	3880	2100	3150
×278	82.3	61.7	2460	3700	2010	3010
×264	77.4	58.1	2320	3480	1890	2830
×235	69.1	51.8	2070	3110	1680	2530
×211	62.1	46.6	1860	2790	1510	2270
×183	53.3	40.0	1600	2400	1300	1950
×167	49.3	37.0	1480	2220	1200	1800
×149	43.8	32.9	1310	1970	1070	1600

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

Table 5-1 (continued)
Available Strength in Axial Tension
W-Shapes

W36–W33

F_y = 50 ksi
F_u = 65 ksi

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
W36×925[h]	272	204	8140	12200	6630	9950
×853[h]	251	188	7510	11300	6110	9170
×802[h]	236	177	7070	10600	5750	8630
×723[h]	213	160	6380	9590	5200	7800
×652[h]	192	144	5750	8640	4680	7020
×529[h]	156	117	4670	7020	3800	5700
×487[h]	143	107	4280	6440	3480	5220
×441[h]	130	97.5	3890	5850	3170	4750
×395[h]	116	87.0	3470	5220	2830	4240
×361[h]	106	79.5	3170	4770	2580	3880
×330	96.9	72.7	2900	4360	2360	3540
×302	89.0	66.8	2660	4010	2170	3260
×282	82.9	62.2	2480	3730	2020	3030
×262	77.2	57.9	2310	3470	1880	2820
×247	72.5	54.4	2170	3260	1770	2650
×231	68.2	51.2	2040	3070	1660	2500
W36×256	75.3	56.5	2250	3390	1840	2750
×232	68.0	51.0	2040	3060	1660	2490
×210	61.9	46.4	1850	2790	1510	2260
×194	57.0	42.8	1710	2570	1390	2090
×182	53.6	40.2	1600	2410	1310	1960
×170	50.0	37.5	1500	2250	1220	1830
×160	47.0	35.3	1410	2120	1150	1720
×150	44.3	33.2	1330	1990	1080	1620
×135	39.9	29.9	1190	1800	972	1460
W33×387[h]	114	85.5	3410	5130	2780	4170
×354[h]	104	78.0	3110	4680	2540	3800
×318	93.7	70.3	2810	4220	2280	3430
×291	85.6	64.2	2560	3850	2090	3130
×263	77.4	58.1	2320	3480	1890	2830
×241	71.1	53.3	2130	3200	1730	2600
×221	65.3	49.0	1960	2940	1590	2390
×201	59.1	44.3	1770	2660	1440	2160

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

F_y = 50 ksi
F_u = 65 ksi

Table 5-1 (continued)
Available Strength in Axial Tension
W-Shapes

W33–W27

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
W33×169	49.5	37.1	1480	2230	1210	1810
×152	44.9	33.7	1340	2020	1100	1640
×141	41.5	31.1	1240	1870	1010	1520
×130	38.3	28.7	1150	1720	933	1400
×118	34.7	26.0	1040	1560	845	1270
W30×391[h]	115	86.3	3440	5180	2800	4210
×357[h]	105	78.8	3140	4730	2560	3840
×326[h]	95.9	71.9	2870	4320	2340	3510
×292	86.0	64.5	2570	3870	2100	3140
×261	77.0	57.8	2310	3470	1880	2820
×235	69.3	52.0	2070	3120	1690	2540
×211	62.3	46.7	1870	2800	1520	2280
×191	56.1	42.1	1680	2520	1370	2050
×173	50.9	38.2	1520	2290	1240	1860
W30×148	43.6	32.7	1310	1960	1060	1590
×132	38.8	29.1	1160	1750	946	1420
×124	36.5	27.4	1090	1640	891	1340
×116	34.2	25.7	1020	1540	835	1250
×108	31.7	23.8	949	1430	774	1160
×99	29.0	21.8	868	1310	709	1060
×90	26.3	19.7	787	1180	640	960
W27×539[h]	159	119	4760	7160	3870	5800
×368[h]	109	81.8	3260	4910	2660	3990
×336[h]	99.2	74.4	2970	4460	2420	3630
×307[h]	90.2	67.7	2700	4060	2200	3300
×281	83.1	62.3	2490	3740	2020	3040
×258	76.1	57.1	2280	3420	1860	2780
×235	69.4	52.1	2080	3120	1690	2540
×217	63.9	47.9	1910	2880	1560	2340
×194	57.1	42.8	1710	2570	1390	2090
×178	52.5	39.4	1570	2360	1280	1920
×161	47.6	35.7	1430	2140	1160	1740
×146	43.2	32.4	1290	1940	1050	1580

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

W27–W24

Table 5-1 (continued)
Available Strength in Axial Tension
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding (kips)		Rupture (kips)	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
W27×129	37.8	28.4	1130	1700	923	1380
×114	33.6	25.2	1010	1510	819	1230
×102	30.0	22.5	898	1350	731	1100
×94	27.6	20.7	826	1240	673	1010
×84	24.7	18.5	740	1110	601	902
W24×370[h]	109	81.8	3260	4910	2660	3990
×335[h]	98.3	73.7	2940	4420	2400	3590
×306[h]	89.7	67.3	2690	4040	2190	3280
×279[h]	81.9	61.4	2450	3690	2000	2990
×250	73.5	55.1	2200	3310	1790	2690
×229	67.2	50.4	2010	3020	1640	2460
×207	60.7	45.5	1820	2730	1480	2220
×192	56.5	42.4	1690	2540	1380	2070
×176	51.7	38.8	1550	2330	1260	1890
×162	47.8	35.9	1430	2150	1170	1750
×146	43.0	32.3	1290	1940	1050	1570
×131	38.6	29.0	1160	1740	943	1410
×117	34.4	25.8	1030	1550	839	1260
×104	30.7	23.0	919	1380	748	1120
W24×103	30.3	22.7	907	1360	738	1110
×94	27.7	20.8	829	1250	676	1010
×84	24.7	18.5	740	1110	601	902
×76	22.4	16.8	671	1010	546	819
×68	20.1	15.1	602	905	491	736
W24×62	18.2	13.7	545	819	445	668
×55	16.2	12.2	485	729	397	595

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

F_y = 50 ksi
F_u = 65 ksi

Table 5-1 (continued)
Available Strength in Axial Tension
W-Shapes

W21

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
W21×275[h]	81.8	61.4	2450	3680	2000	2990
×248	73.8	55.4	2210	3320	1800	2700
×223	66.5	49.9	1990	2990	1620	2430
×201	59.3	44.5	1780	2670	1450	2170
×182	53.6	40.2	1600	2410	1310	1960
×166	48.8	36.6	1460	2200	1190	1780
×147	43.2	32.4	1290	1940	1050	1580
×132	38.8	29.1	1160	1750	946	1420
×122	35.9	26.9	1070	1620	874	1310
×111	32.6	24.5	976	1470	796	1190
×101	29.8	22.4	892	1340	728	1090
W21×93	27.3	20.5	817	1230	666	999
×83	24.4	18.3	731	1100	595	892
×73	21.5	16.1	644	968	523	785
×68	20.0	15.0	599	900	488	731
×62	18.3	13.7	548	824	445	668
×55	16.2	12.2	485	729	397	595
×48	14.1	10.6	422	635	345	517
W21×57	16.7	12.5	500	752	406	609
×50	14.7	11.0	440	662	358	536
×44	13.0	9.75	389	585	317	475

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

W18–W16

Table 5-1 (continued)
Available Strength in Axial Tension
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
W18×311[h]	91.6	68.7	2740	4120	2230	3350
×283[h]	83.3	62.5	2490	3750	2030	3050
×258[h]	76.0	57.0	2280	3420	1850	2780
×234[h]	68.6	51.5	2050	3090	1670	2510
×211	62.3	46.7	1870	2800	1520	2280
×192	56.2	42.2	1680	2530	1370	2060
×175	51.4	38.6	1540	2310	1250	1880
×158	46.3	34.7	1390	2080	1130	1690
×143	42.0	31.5	1260	1890	1020	1540
×130	38.3	28.7	1150	1720	933	1400
×119	35.1	26.3	1050	1580	855	1280
×106	31.1	23.3	931	1400	757	1140
×97	28.5	21.4	853	1280	696	1040
×86	25.3	19.0	757	1140	618	926
×76	22.3	16.7	668	1000	543	814
W18×71	20.9	15.7	626	941	510	765
×65	19.1	14.3	572	860	465	697
×60	17.6	13.2	527	792	429	644
×55	16.2	12.2	485	729	397	595
×50	14.7	11.0	440	662	358	536
W18×46	13.5	10.1	404	608	328	492
×40	11.8	8.85	353	531	288	431
×35	10.3	7.73	308	464	251	377
W16×100	29.4	22.1	880	1320	718	1080
×89	26.2	19.7	784	1180	640	960
×77	22.6	17.0	677	1020	553	829
×67	19.6	14.7	587	882	478	717
W16×57	16.8	12.6	503	756	410	614
×50	14.7	11.0	440	662	358	536
×45	13.3	9.98	398	599	324	487
×40	11.8	8.85	353	531	288	431
×36	10.6	7.95	317	477	258	388
W16×31	9.13	6.85	273	411	223	334
×26	7.68	5.76	230	346	187	281

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

F_y = 50 ksi
F_u = 65 ksi

Table 5-1 (continued)
Available Strength in Axial Tension
W-Shapes

W14

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
W14×873[h]	257	193	7690	11600	6270	9410
×808[h]	238	179	7130	10700	5820	8730
×730[h]	215	161	6440	9680	5230	7850
×665[h]	196	147	5870	8820	4780	7170
×605[h]	178	134	5330	8010	4360	6530
×550[h]	162	122	4850	7290	3970	5950
×500[h]	147	110	4400	6620	3580	5360
×455[h]	134	101	4010	6030	3280	4920
×426[h]	125	93.8	3740	5630	3050	4570
×398[h]	117	87.8	3500	5270	2850	4280
×370[h]	109	81.8	3260	4910	2660	3990
×342[h]	101	75.8	3020	4550	2460	3700
×311[h]	91.4	68.6	2740	4110	2230	3340
×283[h]	83.3	62.5	2490	3750	2030	3050
×257	75.6	56.7	2260	3400	1840	2760
×233	68.5	51.4	2050	3080	1670	2510
×211	62.0	46.5	1860	2790	1510	2270
×193	56.8	42.6	1700	2560	1380	2080
×176	51.8	38.9	1550	2330	1260	1900
×159	46.7	35.0	1400	2100	1140	1710
×145	42.7	32.0	1280	1920	1040	1560
W14×132	38.8	29.1	1160	1750	946	1420
×120	35.3	26.5	1060	1590	861	1290
×109	32.0	24.0	958	1440	780	1170
×99	29.1	21.8	871	1310	709	1060
×90	26.5	19.9	793	1190	647	970
W14×82	24.0	18.0	719	1080	585	878
×74	21.8	16.4	653	981	533	800
×68	20.0	15.0	599	900	488	731
×61	17.9	13.4	536	806	436	653
W14×53	15.6	11.7	467	702	380	570
×48	14.1	10.6	422	635	345	517
×43	12.6	9.45	377	567	307	461

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

W14–W12

Table 5-1 (continued)
Available Strength in Axial Tension
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

Shape	Gross Area, A_g	A_e = 0.75A_g	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
W14×38	11.2	8.40	335	504	273	410
×34	10.0	7.50	299	450	244	366
×30	8.85	6.64	265	398	216	324
W14×26	7.69	5.77	230	346	188	281
×22	6.49	4.87	194	292	158	237
W12×336[h]	98.9	74.2	2960	4450	2410	3620
×305[h]	89.5	67.1	2680	4030	2180	3270
×279[h]	81.9	61.4	2450	3690	2000	2990
×252[h]	74.1	55.6	2220	3330	1810	2710
×230[h]	67.7	50.8	2030	3050	1650	2480
×210	61.8	46.4	1850	2780	1510	2260
×190	56.0	42.0	1680	2520	1370	2050
×170	50.0	37.5	1500	2250	1220	1830
×152	44.7	33.5	1340	2010	1090	1630
×136	39.9	29.9	1190	1800	972	1460
×120	35.2	26.4	1050	1580	858	1290
×106	31.2	23.4	934	1400	761	1140
×96	28.2	21.2	844	1270	689	1030
×87	25.6	19.2	766	1150	624	936
×79	23.2	17.4	695	1040	566	848
×72	21.1	15.8	632	950	514	770
×65	19.1	14.3	572	860	465	697
W12×58	17.0	12.8	509	765	416	624
×53	15.6	11.7	467	702	380	570
W12×50	14.6	11.0	437	657	358	536
×45	13.1	9.83	392	590	319	479
×40	11.7	8.78	350	527	285	428
W12×35	10.3	7.73	308	464	251	377
×30	8.79	6.59	263	396	214	321
×26	7.65	5.74	229	344	187	280
W12×22	6.48	4.86	194	292	158	237
×19	5.57	4.18	167	251	136	204
×16	4.71	3.53	141	212	115	172
×14	4.16	3.12	125	187	101	152

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

F_y = 50 ksi
F_u = 65 ksi

Table 5-1 (continued)
Available Strength in Axial Tension
W-Shapes

W10–W8

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding, kips		Rupture, kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
W10×112	32.9	24.7	985	1480	803	1200
×100	29.3	22.0	877	1320	715	1070
×88	26.0	19.5	778	1170	634	951
×77	22.7	17.0	680	1020	553	829
×68	19.9	14.9	596	896	484	726
×60	17.7	13.3	530	797	432	648
×54	15.8	11.9	473	711	387	580
×49	14.4	10.8	431	648	351	527
W10×45	13.3	9.98	398	599	324	487
×39	11.5	8.63	344	518	280	421
×33	9.71	7.28	291	437	237	355
W10×30	8.84	6.63	265	398	215	323
×26	7.61	5.71	228	342	186	278
×22	6.49	4.87	194	292	158	237
W10×19	5.62	4.22	168	253	137	206
×17	4.99	3.74	149	225	122	182
×15	4.41	3.31	132	198	108	161
×12	3.54	2.66	106	159	86.5	130
W8×67	19.7	14.8	590	887	481	722
×58	17.1	12.8	512	770	416	624
×48	14.1	10.6	422	635	345	517
×40	11.7	8.78	350	527	285	428
×35	10.3	7.73	308	464	251	377
×31	9.13	6.85	273	411	223	334
W8×28	8.25	6.19	247	371	201	302
×24	7.08	5.31	212	319	173	259
W8×21	6.16	4.62	184	277	150	225
×18	5.26	3.95	157	237	128	193
W8×15	4.44	3.33	133	200	108	162
×13	3.84	2.88	115	173	93.6	140
×10	2.96	2.22	88.6	133	72.2	108

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

Table 5-2
Available Strength in Axial Tension
Angles

L12–L8

F_y = 36 ksi
F_u = 58 ksi

Shape	Gross Area, A_g	A_e = $0.75A_g$	Yielding kips P_n/Ω_t	Yielding kips $\phi_t P_n$	Rupture kips P_n/Ω_t	Rupture kips $\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
L12×12×1$^3/_8$	31.1	23.3	670	1010	676	1010
×1$^1/_4$	28.4	21.3	612	920	618	927
×1$^1/_8$	25.8	19.4	556	836	563	844
×1	23.0	17.3	496	745	502	753
L10×10×1$^3/_8$	25.6	19.2	552	829	557	835
×1$^1/_4$	23.4	17.6	504	758	510	766
×1$^1/_8$	21.3	16.0	459	690	464	696
×1	19.0	14.3	410	616	415	622
×$^7/_8$	16.8	12.6	362	544	365	548
×$^3/_4$	14.5	10.9	313	470	316	474
L8×8×1$^1/_8$	16.8	12.6	362	544	365	548
×1	15.1	11.3	326	489	328	492
×$^7/_8$	13.3	9.98	287	431	289	434
×$^3/_4$	11.5	8.63	248	373	250	375
×$^5/_8$	9.69	7.27	209	314	211	316
×$^9/_{16}$	8.77	6.58	189	284	191	286
×$^1/_2$	7.84	5.88	169	254	171	256
L8×6×1	13.1	9.83	282	424	285	428
×$^7/_8$	11.5	8.63	248	373	250	375
×$^3/_4$	9.99	7.49	215	324	217	326
×$^5/_8$	8.41	6.31	181	272	183	274
×$^9/_{16}$	7.61	5.71	164	247	166	248
×$^1/_2$	6.80	5.10	147	220	148	222
×$^7/_{16}$	5.99	4.49	129	194	130	195
L8×4×1	11.1	8.33	239	360	242	362
×$^7/_8$	9.79	7.34	211	317	213	319
×$^3/_4$	8.49	6.37	183	275	185	277
×$^5/_8$	7.16	5.37	154	232	156	234
×$^9/_{16}$	6.49	4.87	140	210	141	212
×$^1/_2$	5.80	4.35	125	188	126	189
×$^7/_{16}$	5.11	3.83	110	166	111	167

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.745A_g$.

F_y = 36 ksi
F_u = 58 ksi

Table 5-2 (continued)
Available Strength in Axial Tension
Angles

L7–L5

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding, kips P_n/Ω_t	Yielding, kips $\phi_t P_n$	Rupture, kips P_n/Ω_t	Rupture, kips $\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
L7×4×3/4	7.74	5.81	167	251	168	253
×5/8	6.50	4.88	140	211	142	212
×1/2	5.26	3.95	113	170	115	172
×7/16	4.63	3.47	99.8	150	101	151
×3/8	4.00	3.00	86.2	130	87.0	131
L6×6×1	11.0	8.25	237	356	239	359
×7/8	9.75	7.31	210	316	212	318
×3/4	8.46	6.35	182	274	184	276
×5/8	7.13	5.35	154	231	155	233
×9/16	6.45	4.84	139	209	140	211
×1/2	5.77	4.33	124	187	126	188
×7/16	5.08	3.81	110	165	110	166
×3/8	4.38	3.29	94.4	142	95.4	143
×5/16	3.67	2.75	79.1	119	79.8	120
L6×4×7/8	8.00	6.00	172	259	174	261
×3/4	6.94	5.21	150	225	151	227
×5/8	5.86	4.40	126	190	128	191
×9/16	5.31	3.98	114	172	115	173
×1/2	4.75	3.56	102	154	103	155
×7/16	4.18	3.14	90.1	135	91.1	137
×3/8	3.61	2.71	77.8	117	78.6	118
×5/16	3.03	2.27	65.3	98.2	65.8	98.7
L6×3 1/2×1/2	4.50	3.38	97.0	146	98.0	147
×3/8	3.44	2.58	74.2	111	74.8	112
×5/16	2.89	2.17	62.3	93.6	62.9	94.4
L5×5×7/8	8.00	6.00	172	259	174	261
×3/4	6.98	5.24	150	226	152	228
×5/8	5.90	4.43	127	191	128	193
×1/2	4.79	3.59	103	155	104	156
×7/16	4.22	3.17	91.0	137	91.9	138
×3/8	3.65	2.74	78.7	118	79.5	119
×5/16	3.07	2.30	66.2	99.5	66.7	100

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.745A_g$.

Table 5-2 (continued)
Available Strength in Axial Tension
Angles

L5–L3$^1/_2$

F_y = 36 ksi
F_u = 58 ksi

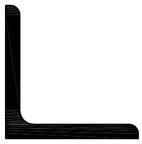
Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
L5×3$^1/_2$×$^3/_4$	5.85	4.39	126	190	127	191
×$^5/_8$	4.93	3.70	106	160	107	161
×$^1/_2$	4.00	3.00	86.2	130	87.0	131
×$^3/_8$	3.05	2.29	65.7	98.8	66.4	99.6
×$^5/_{16}$	2.56	1.92	55.2	82.9	55.7	83.5
×$^1/_4$	2.07	1.55	44.6	67.1	45.0	67.4
L5×3×$^1/_2$	3.75	2.81	80.8	122	81.5	122
×$^7/_{16}$	3.31	2.48	71.4	107	71.9	108
×$^3/_8$	2.86	2.15	61.7	92.7	62.4	93.5
×$^5/_{16}$	2.41	1.81	52.0	78.1	52.5	78.7
×$^1/_4$	1.94	1.46	41.8	62.9	42.3	63.5
L4×4×$^3/_4$	5.44	4.08	117	176	118	177
×$^5/_8$	4.61	3.46	99.4	149	100	151
×$^1/_2$	3.75	2.81	80.8	122	81.5	122
×$^7/_{16}$	3.30	2.48	71.1	107	71.9	108
×$^3/_8$	2.86	2.15	61.7	92.7	62.4	93.5
×$^5/_{16}$	2.40	1.80	51.7	77.8	52.2	78.3
×$^1/_4$	1.93	1.45	41.6	62.5	42.1	63.1
L4×3$^1/_2$×$^1/_2$	3.50	2.63	75.4	113	76.3	114
×$^3/_8$	2.68	2.01	57.8	86.8	58.3	87.4
×$^5/_{16}$	2.25	1.69	48.5	72.9	49.0	73.5
×$^1/_4$	1.82	1.37	39.2	59.0	39.7	59.6
L4×3×$^5/_8$	3.99	2.99	86.0	129	86.7	130
×$^1/_2$	3.25	2.44	70.1	105	70.8	106
×$^3/_8$	2.49	1.87	53.7	80.7	54.2	81.3
×$^5/_{16}$	2.09	1.57	45.1	67.7	45.5	68.3
×$^1/_4$	1.69	1.27	36.4	54.8	36.8	55.2
L3$^1/_2$×3$^1/_2$×$^1/_2$	3.25	2.44	70.1	105	70.8	106
×$^7/_{16}$	2.89	2.17	62.3	93.6	62.9	94.4
×$^3/_8$	2.50	1.88	53.9	81.0	54.5	81.8
×$^5/_{16}$	2.10	1.58	45.3	68.0	45.8	68.7
×$^1/_4$	1.70	1.28	36.6	55.1	37.1	55.7

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.745A_g$.

F_y = 36 ksi
F_u = 58 ksi

Table 5-2 (continued)
Available Strength in Axial Tension
Angles



L3$^1/_2$–L2$^1/_2$

Shape	Gross Area, A_g	A_e = 0.75A_g	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
L3$^1/_2$×3×$^1/_2$	3.02	2.27	65.1	97.8	65.8	98.7
×$^7/_{16}$	2.67	2.00	57.6	86.5	58.0	87.0
×$^3/_8$	2.32	1.74	50.0	75.2	50.5	75.7
×$^5/_{16}$	1.95	1.46	42.0	63.2	42.3	63.5
×$^1/_4$	1.58	1.19	34.1	51.2	34.5	51.8
L3$^1/_2$×2$^1/_2$×$^1/_2$	2.77	2.08	59.7	89.7	60.3	90.5
×$^3/_8$	2.12	1.59	45.7	68.7	46.1	69.2
×$^5/_{16}$	1.79	1.34	38.6	58.0	38.9	58.3
×$^1/_4$	1.45	1.09	31.3	47.0	31.6	47.4
L3×3×$^1/_2$	2.76	2.07	59.5	89.4	60.0	90.0
×$^7/_{16}$	2.43	1.82	52.4	78.7	52.8	79.2
×$^3/_8$	2.11	1.58	45.5	68.4	45.8	68.7
×$^5/_{16}$	1.78	1.34	38.4	57.7	38.9	58.3
×$^1/_4$	1.44	1.08	31.0	46.7	31.3	47.0
×$^3/_{16}$	1.09	0.818	23.5	35.3	23.7	35.6
L3×2$^1/_2$×$^1/_2$	2.50	1.88	53.9	81.0	54.5	81.8
×$^7/_{16}$	2.22	1.67	47.9	71.9	48.4	72.6
×$^3/_8$	1.93	1.45	41.6	62.5	42.1	63.1
×$^5/_{16}$	1.63	1.22	35.1	52.8	35.4	53.1
×$^1/_4$	1.32	0.990	28.5	42.8	28.7	43.1
×$^3/_{16}$	1.00	0.750	21.6	32.4	21.8	32.6
L3×2×$^1/_2$	2.26	1.70	48.7	73.2	49.3	74.0
×$^3/_8$	1.75	1.31	37.7	56.7	38.0	57.0
×$^5/_{16}$	1.48	1.11	31.9	48.0	32.2	48.3
×$^1/_4$	1.20	0.900	25.9	38.9	26.1	39.2
×$^3/_{16}$	0.917	0.688	19.8	29.7	20.0	29.9
L2$^1/_2$×2$^1/_2$×$^1/_2$	2.26	1.70	48.7	73.2	49.3	74.0
×$^3/_8$	1.73	1.30	37.3	56.1	37.7	56.6
×$^5/_{16}$	1.46	1.10	31.5	47.3	31.9	47.9
×$^1/_4$	1.19	0.893	25.7	38.6	25.9	38.8
×$^3/_{16}$	0.901	0.676	19.4	29.2	19.6	29.4

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.745A_g$.

L2½–L2

Table 5-2 (continued)
Available Strength in Axial Tension
Angles

F_y = 36 ksi
F_u = 58 ksi

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips P_n/Ω_t	Yielding kips $\phi_t P_n$	Rupture kips P_n/Ω_t	Rupture kips $\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
L2½×2×3/8	1.55	1.16	33.4	50.2	33.6	50.5
×5/16	1.32	0.990	28.5	42.8	28.7	43.1
×1/4	1.07	0.803	23.1	34.7	23.3	34.9
×3/16	0.818	0.614	17.6	26.5	17.8	26.7
L2½×1½×1/4	0.947	0.710	20.4	30.7	20.6	30.9
×3/16	0.724	0.543	15.6	23.5	15.7	23.6
L2×2×3/8	1.37	1.03	29.5	44.4	29.9	44.8
×5/16	1.16	0.870	25.0	37.6	25.2	37.8
×1/4	0.944	0.708	20.3	30.6	20.5	30.8
×3/16	0.722	0.542	15.6	23.4	15.7	23.6
×1/8	0.491	0.368	10.6	15.9	10.7	16.0

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.745A_g$.

F_y = 50 ksi
F_u = 65 ksi

Table 5-3
Available Strength in Axial Tension
WT-Shapes

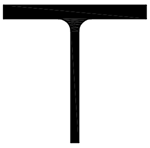
WT22–WT20

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
WT22×167.5	49.2	36.9	1470	2210	1200	1800
×145	42.6	32.0	1280	1920	1040	1560
×131	38.5	28.9	1150	1730	939	1410
×115	33.9	25.4	1010	1530	826	1240
WT20×327.5[h]	96.4	72.3	2890	4340	2350	3520
×296.5[h]	87.2	65.4	2610	3920	2130	3190
×251.5[h]	74.0	55.5	2220	3330	1800	2710
×215.5[h]	63.3	47.5	1900	2850	1540	2320
×198.5[h]	58.3	43.7	1750	2620	1420	2130
×186[h]	54.7	41.0	1640	2460	1330	2000
×181[h]	53.2	39.9	1590	2390	1300	1950
×162	47.7	35.8	1430	2150	1160	1750
×148.5	43.6	32.7	1310	1960	1060	1590
×138.5	40.7	30.5	1220	1830	991	1490
×124.5	36.7	27.5	1100	1650	894	1340
×107.5	31.8	23.9	952	1430	777	1170
×99.5	29.2	21.9	874	1310	712	1070
WT20×196[h]	57.8	43.4	1730	2600	1410	2120
×165.5[h]	48.8	36.6	1460	2200	1190	1780
×163.5[h]	47.9	35.9	1430	2160	1170	1750
×147	43.1	32.3	1290	1940	1050	1570
×139	41.0	30.8	1230	1850	1000	1500
×132	38.7	29.0	1160	1740	943	1410
×117.5	34.6	26.0	1040	1560	845	1270
×105.5	31.1	23.3	931	1400	757	1140
×91.5	26.7	20.0	799	1200	650	975
×83.5	24.5	18.4	734	1100	598	897
×74.5	21.9	16.4	656	986	533	800

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.



WT18–WT16.5

Table 5-3 (continued)
Available Strength in Axial Tension
WT-Shapes

F_y = 50 ksi
F_u = 65 ksi

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
WT18×462.5[h]	136	102	4070	6120	3320	4970
×426.5[h]	126	94.5	3770	5670	3070	4610
×401[h]	118	88.5	3530	5310	2880	4310
×361.5[h]	107	80.3	3200	4820	2610	3910
×326[h]	96.2	72.2	2880	4330	2350	3520
×264.5[h]	77.8	58.4	2330	3500	1900	2850
×243.5[h]	71.7	53.8	2150	3230	1750	2620
×220.5[h]	64.9	48.7	1940	2920	1580	2370
×197.5[h]	58.1	43.6	1740	2610	1420	2130
×180.5[h]	53.0	39.8	1590	2390	1290	1940
×165	48.4	36.3	1450	2180	1180	1770
×151	44.5	33.4	1330	2000	1090	1630
×141	41.5	31.1	1240	1870	1010	1520
×131	38.5	28.9	1150	1730	939	1410
×123.5	36.3	27.2	1090	1630	884	1330
×115.5	34.1	25.6	1020	1530	832	1250
WT18×128	37.6	28.2	1130	1690	917	1370
×116	34.0	25.5	1020	1530	829	1240
×105	30.9	23.2	925	1390	754	1130
×97	28.5	21.4	853	1280	696	1040
×91	26.8	20.1	802	1210	653	980
×85	25.0	18.8	749	1130	611	917
×80	23.5	17.6	704	1060	572	858
×75	22.1	16.6	662	995	540	809
×67.5	19.9	14.9	596	896	484	726
WT16.5×193.5[h]	57.0	42.8	1710	2570	1390	2090
×177[h]	52.1	39.1	1560	2340	1270	1910
×159	46.8	35.1	1400	2110	1140	1710
×145.5	42.8	32.1	1280	1930	1040	1560
×131.5	38.7	29.0	1160	1740	943	1410
×120.5	35.6	26.7	1070	1600	868	1300
×110.5	32.6	24.5	976	1470	796	1190
×100.5	29.7	22.3	889	1340	725	1090

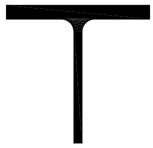
Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

F_y = 50 ksi
F_u = 65 ksi

Table 5-3 (continued)
Available Strength in Axial Tension
WT-Shapes

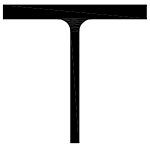


WT16.5–WT13.5

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
WT16.5×84.5	24.7	18.5	740	1110	601	902
×76	22.5	16.9	674	1010	549	824
×70.5	20.7	15.5	620	932	504	756
×65	19.1	14.3	572	860	465	697
×59	17.4	13.1	521	783	426	639
WT15×195.5^h	57.6	43.2	1720	2590	1400	2110
×178.5^h	52.5	39.4	1570	2360	1280	1920
×163^h	48.0	36.0	1440	2160	1170	1760
×146	43.0	32.3	1290	1940	1050	1570
×130.5	38.5	28.9	1150	1730	939	1410
×117.5	34.7	26.0	1040	1560	845	1270
×105.5	31.1	23.3	931	1400	757	1140
×95.5	28.0	21.0	838	1260	683	1020
×86.5	25.4	19.1	760	1140	621	931
WT15×74	21.8	16.4	653	981	533	800
×66	19.5	14.6	584	878	475	712
×62	18.2	13.7	545	819	445	668
×58	17.1	12.8	512	770	416	624
×54	15.9	11.9	476	716	387	580
×49.5	14.5	10.9	434	653	354	531
×45	13.2	9.90	395	594	322	483
WT13.5×269.5^h	79.3	59.5	2370	3570	1930	2900
×184^h	54.2	40.7	1620	2440	1320	1980
×168^h	49.5	37.1	1480	2230	1210	1810
×153.5^h	45.2	33.9	1350	2030	1100	1650
×140.5	41.5	31.1	1240	1870	1010	1520
×129	38.1	28.6	1140	1710	930	1390
×117.5	34.7	26.0	1040	1560	845	1270
×108.5	32.0	24.0	958	1440	780	1170
×97	28.6	21.5	856	1290	699	1050
×89	26.3	19.7	787	1180	640	960
×80.5	23.8	17.9	713	1070	582	873
×73	21.6	16.2	647	972	527	790

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.



WT13.5–WT12

Table 5-3 (continued)
Available Strength in Axial Tension
WT-Shapes

F_y = 50 ksi
F_u = 65 ksi

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
WT13.5×64.5	18.9	14.2	566	851	462	692
×57	16.8	12.6	503	756	410	614
×51	15.0	11.3	449	675	367	551
×47	13.8	10.4	413	621	338	507
×42	12.4	9.30	371	558	302	453
WT12×185[h]	54.5	40.9	1630	2450	1330	1990
×167.5[h]	49.1	36.8	1470	2210	1200	1790
×153[h]	44.9	33.7	1340	2020	1100	1640
×139.5[h]	41.0	30.8	1230	1850	1000	1500
×125	36.8	27.6	1100	1660	897	1350
×114.5	33.6	25.2	1010	1510	819	1230
×103.5	30.3	22.7	907	1360	738	1110
×96	28.2	21.2	844	1270	689	1030
×88	25.8	19.4	772	1160	631	946
×81	23.9	17.9	716	1080	582	873
×73	21.5	16.1	644	968	523	785
×65.5	19.3	14.5	578	869	471	707
×58.5	17.2	12.9	515	774	419	629
×52	15.3	11.5	458	689	374	561
WT12×51.5	15.1	11.3	452	680	367	551
×47	13.8	10.4	413	621	338	507
×42	12.4	9.30	371	558	302	453
×38	11.2	8.40	335	504	273	410
×34	10.0	7.50	299	450	244	366
WT12×31	9.11	6.83	273	410	222	333
×27.5	8.10	6.08	243	365	198	296

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

F_y = 50 ksi
F_u = 65 ksi

Table 5-3 (continued)
Available Strength in Axial Tension
WT-Shapes

WT10.5

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips P_n/Ω_t	Yielding kips $\phi_t P_n$	Rupture kips P_n/Ω_t	Rupture kips $\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
WT10.5×137.5[h]	40.9	30.7	1220	1840	998	1500
×124	37.0	27.8	1110	1670	904	1360
×111.5	33.2	24.9	994	1490	809	1210
×100.5	29.6	22.2	886	1330	722	1080
×91	26.8	20.1	802	1210	653	980
×83	24.4	18.3	731	1100	595	892
×73.5	21.6	16.2	647	972	527	790
×66	19.4	14.6	581	873	475	712
×61	17.9	13.4	536	806	436	653
×55.5	16.3	12.2	488	734	397	595
×50.5	14.9	11.2	446	671	364	546
WT10.5×46.5	13.7	10.3	410	617	335	502
×41.5	12.2	9.15	365	549	297	446
×36.5	10.7	8.03	320	482	261	391
×34	10.0	7.50	299	450	244	366
×31	9.13	6.85	273	411	223	334
×27.5	8.10	6.08	243	365	198	296
×24	7.07	5.30	212	318	172	258
WT10.5×28.5	8.37	6.28	251	377	204	306
×25	7.36	5.52	220	331	179	269
×22	6.49	4.87	194	292	158	237

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

WT9–WT8

Table 5-3 (continued)
Available Strength in Axial Tension
WT-Shapes

F_y = 50 ksi
F_u = 65 ksi

Shape	Gross Area, A_g	A_e = 0.75A_g	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
WT9×155.5[h]	45.8	34.4	1370	2060	1120	1680
×141.5[h]	41.7	31.3	1250	1880	1020	1530
×129[h]	38.0	28.5	1140	1710	926	1390
×117[h]	34.3	25.7	1030	1540	835	1250
×105.5	31.2	23.4	934	1400	761	1140
×96	28.1	21.1	841	1260	686	1030
×87.5	25.7	19.3	769	1160	627	941
×79	23.2	17.4	695	1040	566	848
×71.5	21.0	15.8	629	945	514	770
×65	19.2	14.4	575	864	468	702
×59.5	17.6	13.2	527	792	429	644
×53	15.6	11.7	467	702	380	570
×48.5	14.2	10.7	425	639	348	522
×43	12.7	9.53	380	572	310	465
×38	11.1	8.33	332	500	271	406
WT9×35.5	10.4	7.80	311	468	254	380
×32.5	9.55	7.16	286	430	233	349
×30	8.82	6.62	264	397	215	323
×27.5	8.10	6.08	243	365	198	296
×25	7.34	5.51	220	330	179	269
WT9×23	6.77	5.08	203	305	165	248
×20	5.88	4.41	176	265	143	215
×17.5	5.15	3.86	154	232	125	188
WT8×50	14.7	11.0	440	662	358	536
×44.5	13.1	9.83	392	590	319	479
×38.5	11.3	8.48	338	509	276	413
×33.5	9.81	7.36	294	441	239	359
WT8×28.5	8.39	6.29	251	378	204	307
×25	7.37	5.53	221	332	180	270
×22.5	6.63	4.97	199	298	162	242
×20	5.89	4.42	176	265	144	215
×18	5.29	3.97	158	238	129	194
WT8×15.5	4.56	3.42	137	205	111	167
×13	3.84	2.88	115	173	93.6	140

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

F_y = 50 ksi
F_u = 65 ksi

Table 5-3 (continued)
Available Strength in Axial Tension
WT-Shapes

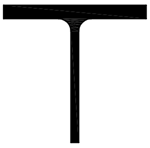
WT7

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
WT7×436.5[h]	129	96.8	3860	5810	3150	4720
×404[h]	119	89.3	3560	5360	2900	4350
×365[h]	107	80.3	3200	4820	2610	3910
×332.5[h]	97.8	73.4	2930	4400	2390	3580
×302.5[h]	89.0	66.8	2660	4010	2170	3260
×275[h]	80.9	60.7	2420	3640	1970	2960
×250[h]	73.5	55.1	2200	3310	1790	2690
×227.5[h]	66.9	50.2	2000	3010	1630	2450
×213[h]	62.7	47.0	1880	2820	1530	2290
×199[h]	58.4	43.8	1750	2630	1420	2140
×185[h]	54.4	40.8	1630	2450	1330	1990
×171[h]	50.3	37.7	1510	2260	1230	1840
×155.5[h]	45.7	34.3	1370	2060	1110	1670
×141.5[h]	41.6	31.2	1250	1870	1010	1520
×128.5	37.8	28.4	1130	1700	923	1380
×116.5	34.2	25.7	1020	1540	835	1250
×105.5	31.0	23.3	928	1400	757	1140
×96.5	28.4	21.3	850	1280	692	1040
×88	25.9	19.4	775	1170	631	946
×79.5	23.4	17.6	701	1050	572	858
×72.5	21.3	16.0	638	959	520	780
WT7×66	19.4	14.6	581	873	475	712
×60	17.7	13.3	530	797	432	648
×54.5	16.0	12.0	479	720	390	585
×49.5	14.6	11.0	437	657	358	536
×45	13.2	9.90	395	594	322	483
WT7×41	12.0	9.00	359	540	293	439
×37	10.9	8.18	326	491	266	399
×34	10.0	7.50	299	450	244	366
×30.5	8.96	6.72	268	403	218	328
WT7×26.5	7.80	5.85	234	351	190	285
×24	7.07	5.30	212	318	172	258
×21.5	6.31	4.73	189	284	154	231

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.



WT7–WT6

Table 5-3 (continued)
Available Strength in Axial Tension
WT-Shapes

F_y = 50 ksi
F_u = 65 ksi

Shape	Gross Area, A_g	A_e = $0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
WT7×19	5.58	4.19	167	251	136	204
×17	5.00	3.75	150	225	122	183
×15	4.42	3.32	132	199	108	162
WT7×13	3.85	2.89	115	173	93.9	141
×11	3.25	2.44	97.3	146	79.3	119
WT6×168[h]	49.5	37.1	1480	2230	1210	1810
×152.5[h]	44.7	33.5	1340	2010	1090	1630
×139.5[h]	41.0	30.8	1230	1850	1000	1500
×126[h]	37.1	27.8	1110	1670	904	1360
×115[h]	33.8	25.4	1010	1520	826	1240
×105	30.9	23.2	925	1390	754	1130
×95	28.0	21.0	838	1260	683	1020
×85	25.0	18.8	749	1130	611	917
×76	22.4	16.8	671	1010	546	819
×68	20.0	15.0	599	900	488	731
×60	17.6	13.2	527	792	429	644
×53	15.6	11.7	467	702	380	570
×48	14.1	10.6	422	635	345	517
×43.5	12.8	9.60	383	576	312	468
×39.5	11.6	8.70	347	522	283	424
×36	10.6	7.95	317	477	258	388
×32.5	9.54	7.16	286	429	233	349
WT6×29	8.52	6.39	255	383	208	312
×26.5	7.78	5.84	233	350	190	285
WT6×25	7.30	5.48	219	329	178	267
×22.5	6.56	4.92	196	295	160	240
×20	5.84	4.38	175	263	142	214
WT6×17.5	5.17	3.88	155	233	126	189
×15	4.40	3.30	132	198	107	161
×13	3.82	2.87	114	172	93.3	140
WT6×11	3.24	2.43	97.0	146	79.0	118
×9.5	2.79	2.09	83.5	126	67.9	102
×8	2.36	1.77	70.7	106	57.5	86.3
×7	2.08	1.56	62.3	93.6	50.7	76.1

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

F_y = 50 ksi
F_u = 65 ksi

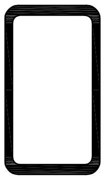
Table 5-3 (continued)
Available Strength in Axial Tension
WT-Shapes

WT5–WT4

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips P_n/Ω_t	Yielding kips $\phi_t P_n$	Rupture kips P_n/Ω_t	Rupture kips $\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
WT5×56	16.5	12.4	494	743	403	605
×50	14.7	11.0	440	662	358	536
×44	13.0	9.75	389	585	317	475
×38.5	11.3	8.48	338	509	276	413
×34	10.0	7.50	299	450	244	366
×30	8.84	6.63	265	398	215	323
×27	7.90	5.93	237	356	193	289
×24.5	7.21	5.41	216	324	176	264
WT5×22.5	6.63	4.97	199	298	162	242
×19.5	5.73	4.30	172	258	140	210
×16.5	4.85	3.64	145	218	118	177
WT5×15	4.42	3.32	132	199	108	162
×13	3.81	2.86	114	171	93.0	139
×11	3.24	2.43	97.0	146	79.0	118
WT5×9.5	2.81	2.11	84.1	126	68.6	103
×8.5	2.50	1.88	74.9	113	61.1	91.7
×7.5	2.21	1.66	66.2	99.5	54.0	80.9
×6	1.77	1.33	53.0	79.7	43.2	64.8
WT4×33.5	9.84	7.38	295	443	240	360
×29	8.54	6.41	256	384	208	312
×24	7.05	5.29	211	317	172	258
×20	5.87	4.40	176	264	143	215
×17.5	5.14	3.86	154	231	125	188
×15.5	4.56	3.42	137	205	111	167
WT4×14	4.12	3.09	123	185	100	151
×12	3.54	2.66	106	159	86.5	130
WT4×10.5	3.08	2.31	92.2	139	75.1	113
×9	2.63	1.97	78.7	118	64.0	96.0
WT4×7.5	2.22	1.67	66.5	99.9	54.3	81.4
×6.5	1.92	1.44	57.5	86.4	46.8	70.2
×5	1.48	1.11	44.3	66.6	36.1	54.1

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.



HSS20–HSS16

Table 5-4
Available Strength in Axial Tension
Rectangular HSS

F_y = 50 ksi
F_u = 62 ksi

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips P_n/Ω_t	Yielding kips $\phi_t P_n$	Rupture kips P_n/Ω_t	Rupture kips $\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
HSS20×12×5/8	35.0	26.3	1050	1580	815	1220
×1/2	28.3	21.2	847	1270	657	986
×3/8	21.5	16.1	644	968	499	749
×5/16	18.1	13.6	542	815	422	632
HSS20×8×5/8	30.3	22.7	907	1360	704	1060
×1/2	24.6	18.5	737	1110	574	860
×3/8	18.7	14.0	560	842	434	651
×5/16	15.7	11.8	470	707	366	549
HSS20×4×1/2	20.9	15.7	626	941	487	730
×3/8	16.0	12.0	479	720	372	558
×5/16	13.4	10.1	401	603	313	470
×1/4	10.8	8.10	323	486	251	377
HSS18×6×5/8	25.7	19.3	729	1160	598	897
×1/2	20.9	15.7	626	941	487	730
×3/8	16.0	12.0	479	720	372	558
×5/16	13.4	10.1	401	603	313	470
×1/4	10.8	8.10	323	486	251	377
HSS16×12×5/8	30.3	22.7	907	1360	704	1060
×1/2	24.6	18.5	737	1110	574	860
×3/8	18.7	14.0	560	842	434	651
×5/16	15.7	11.8	470	707	366	549
HSS16×8×5/8	25.7	19.3	769	1160	598	897
×1/2	20.9	15.7	626	941	487	730
×3/8	16.0	12.0	479	720	372	558
×5/16	13.4	10.1	401	603	313	470
×1/4	10.8	8.10	323	486	251	377
HSS16×4×5/8	21.0	15.8	629	945	490	735
×1/2	17.2	12.9	515	774	400	600
×3/8	13.2	9.90	395	594	307	460
×5/16	11.1	8.32	332	500	258	387
×1/4	8.96	6.72	268	403	208	312
×3/16	6.76	5.07	202	304	157	236

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.968A_g$.

F_y = 50 ksi
F_u = 62 ksi

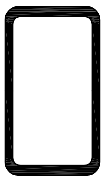
Table 5-4 (continued)
Available Strength in Axial Tension
Rectangular HSS

HSS14–HSS12

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips P_n/Ω_t	Yielding kips $\phi_t P_n$	Rupture kips P_n/Ω_t	Rupture kips $\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
HSS14×10×5/8	25.7	19.3	769	1160	598	897
×1/2	20.9	15.7	626	941	487	730
×3/8	16.0	12.0	479	720	372	558
×5/16	13.4	10.1	401	603	313	470
×1/4	10.8	8.10	323	486	251	377
HSS14×6×5/8	21.0	15.8	629	945	490	735
×1/2	17.2	12.9	515	774	400	600
×3/8	13.2	9.90	395	594	307	460
×5/16	11.1	8.32	332	500	258	387
×1/4	8.96	6.72	268	403	208	312
×3/16	6.76	5.07	202	304	157	236
HSS14×4×5/8	18.7	14.0	560	842	434	651
×1/2	15.3	11.5	458	689	357	535
×3/8	11.8	8.85	353	531	274	412
×5/16	9.92	7.44	297	446	231	346
×1/4	8.03	6.02	240	361	187	280
×3/16	6.06	4.55	181	273	141	212
HSS12×10×1/2	19.0	14.3	569	855	443	665
×3/8	14.6	10.9	437	657	341	512
×5/16	12.2	9.15	365	549	284	425
×1/4	9.90	7.43	296	446	230	345
HSS12×8×5/8	21.0	15.8	629	945	490	735
×1/2	17.2	12.9	515	774	400	600
×3/8	13.2	9.90	395	594	307	460
×5/16	11.1	8.32	332	500	258	387
×1/4	8.96	6.72	268	403	208	312
×3/16	6.76	5.07	202	304	157	236
HSS12×6×5/8	18.7	14.0	560	842	434	651
×1/2	15.3	11.5	458	689	357	535
×3/8	11.8	8.85	353	531	274	412
×5/16	9.92	7.44	297	446	231	346
×1/4	8.03	6.02	240	361	187	280
×3/16	6.06	4.55	181	273	141	212

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.968A_g$.



HSS12–HSS10

Table 5-4 (continued)
Available Strength in Axial Tension
Rectangular HSS

F_y = 50 ksi
F_u = 62 ksi

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
HSS12×4×5/8	16.4	12.3	491	738	381	572
×1/2	13.5	10.1	404	608	313	470
×3/8	10.4	7.80	311	468	242	363
×5/16	8.76	6.57	262	394	204	306
×1/4	7.10	5.33	213	320	165	248
×3/16	5.37	4.03	161	242	125	187
HSS12×3 1/2×3/8	10.0	7.50	299	450	233	349
×5/16	8.46	6.34	253	381	197	295
HSS12×3×5/16	8.17	6.13	245	368	190	285
×1/4	6.63	4.97	199	298	154	231
×3/16	5.02	3.76	150	226	117	175
HSS12×2×5/16	7.59	5.69	227	342	176	265
×1/4	6.17	4.63	185	278	144	215
×3/16	4.67	3.50	140	210	109	163
HSS10×8×5/8	18.7	14.0	560	842	434	651
×1/2	15.3	11.5	458	689	357	535
×3/8	11.8	8.85	353	531	274	412
×5/16	9.92	7.44	297	446	231	346
×1/4	8.03	6.02	240	361	187	280
×3/16	6.06	4.55	181	273	141	212
HSS10×6×5/8	16.4	12.3	491	738	381	572
×1/2	13.5	10.1	404	608	313	470
×3/8	10.4	7.80	311	468	242	363
×5/16	8.76	6.57	262	394	204	306
×1/4	7.10	5.33	213	320	165	248
×3/16	5.37	4.03	161	242	125	187
HSS10×5×3/8	9.67	7.25	290	435	225	337
×5/16	8.17	6.13	245	368	190	285
×1/4	6.63	4.97	199	298	154	231
×3/16	5.02	3.76	150	226	117	175

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.968A_g$.

F_y = 50 ksi
F_u = 62 ksi

Table 5-4 (continued)
Available Strength in Axial Tension
Rectangular HSS

HSS10–HSS9

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding (kips)		Rupture (kips)	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
HSS10×4×5/8	14.0	10.5	419	630	326	488
×1/2	11.6	8.70	347	522	270	405
×3/8	8.97	6.73	269	404	209	313
×5/16	7.59	5.69	227	342	176	265
×1/4	6.17	4.63	185	278	144	215
×3/16	4.67	3.50	140	210	109	163
×1/8	3.16	2.37	94.6	142	73.5	110
HSS10×3 1/2×1/2	11.1	8.32	332	500	258	387
×3/8	8.62	6.47	258	388	201	301
×5/16	7.30	5.48	219	329	170	255
×1/4	5.93	4.45	178	267	138	207
×3/16	4.50	3.38	135	203	105	157
×1/8	3.04	2.28	91.0	137	70.7	106
HSS10×3×3/8	8.27	6.20	248	372	192	288
×5/16	7.01	5.26	210	315	163	245
×1/4	5.70	4.27	171	257	133	199
×3/16	4.32	3.24	129	194	100	151
×1/8	2.93	2.20	87.7	132	68.2	102
HSS10×2×3/8	7.58	5.69	227	341	176	265
×5/16	6.43	4.82	193	289	149	224
×1/4	5.24	3.93	157	236	122	183
×3/16	3.98	2.99	119	179	92.7	139
×1/8	2.70	2.03	80.8	122	62.9	94.4
HSS9×7×5/8	16.4	12.3	491	738	381	572
×1/2	13.5	10.1	404	608	313	470
×3/8	10.4	7.80	311	468	242	363
×5/16	8.76	6.57	262	394	204	306
×1/4	7.10	5.33	213	320	165	248
×3/16	5.37	4.03	161	242	125	187

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.968A_g$.

HSS9–HSS8

Table 5-4 (continued)
Available Strength in Axial Tension
Rectangular HSS

F_y = 50 ksi
F_u = 62 ksi

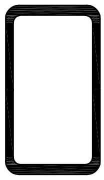
Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding, kips		Rupture, kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
HSS9×5×5/8	14.0	10.5	419	630	326	488
×1/2	11.6	8.70	347	522	270	405
×3/8	8.97	6.73	269	404	209	313
×5/16	7.59	5.69	227	342	176	265
×1/4	6.17	4.63	185	278	144	215
×3/16	4.67	3.50	140	210	109	163
HSS9×3×1/2	9.74	7.30	292	438	227	340
×3/8	7.58	5.69	227	341	176	265
×5/16	6.43	4.82	193	289	149	224
×1/4	5.24	3.93	157	236	122	183
×3/16	3.98	2.99	119	179	92.7	139
HSS8×6×5/8	14.0	10.5	419	630	326	488
×1/2	11.6	8.70	347	522	270	405
×3/8	8.97	6.73	269	404	209	313
×5/16	7.59	5.69	227	342	176	265
×1/4	6.17	4.63	185	278	144	215
×3/16	4.67	3.50	140	210	109	163
HSS8×4×5/8	11.7	8.78	350	527	272	408
×1/2	9.74	7.30	292	438	227	340
×3/8	7.58	5.69	227	341	176	265
×5/16	6.43	4.82	193	289	149	224
×1/4	5.24	3.93	157	236	122	183
×3/16	3.98	2.99	119	179	92.7	139
×1/8	2.70	2.03	80.8	122	62.9	94.4
HSS8×3×1/2	8.81	6.61	264	396	205	307
×3/8	6.88	5.16	206	310	160	240
×5/16	5.85	4.39	175	263	136	204
×1/4	4.77	3.58	143	215	111	166
×3/16	3.63	2.72	109	163	84.3	126
×1/8	2.46	1.85	73.7	111	57.4	86.0

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.968A_g$.

F_y = 50 ksi
F_u = 62 ksi

Table 5-4 (continued)
Available Strength in Axial Tension
Rectangular HSS



HSS8–HSS6

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
HSS8×2×3/8	6.18	4.63	185	278	144	216
×5/16	5.26	3.94	157	237	122	184
×1/4	4.30	3.22	129	194	100	150
×3/16	3.28	2.46	98.2	148	76.3	114
×1/8	2.23	1.67	66.8	100	51.8	77.7
HSS7×5×1/2	9.74	7.30	292	438	227	340
×3/8	7.58	5.69	227	341	176	265
×5/16	6.43	4.82	193	289	149	224
×1/4	5.24	3.93	157	236	122	183
×3/16	3.98	2.99	119	179	92.7	139
×1/8	2.70	2.03	80.8	122	62.9	94.4
HSS7×4×1/2	8.81	6.61	264	396	205	307
×3/8	6.88	5.16	206	310	160	240
×5/16	5.85	4.39	175	263	136	204
×1/4	4.77	3.58	143	215	111	166
×3/16	3.63	2.72	109	163	84.3	126
×1/8	2.46	1.85	73.7	111	57.4	86.0
HSS7×3×1/2	7.88	5.91	236	355	183	275
×3/8	6.18	4.63	185	278	144	216
×5/16	5.26	3.94	157	237	122	184
×1/4	4.30	3.22	129	194	100	150
×3/16	3.28	2.46	98.2	148	76.3	114
×1/8	2.23	1.67	66.8	100	51.8	77.7
HSS7×2×1/4	3.84	2.88	115	173	89.3	134
×3/16	2.93	2.20	87.7	132	68.2	102
×1/8	2.00	1.50	59.9	90.0	46.5	69.8
HSS6×5×1/2	8.81	6.61	264	396	205	307
×3/8	6.88	5.16	206	310	160	240
×5/16	5.85	4.39	175	263	136	204
×1/4	4.77	3.58	143	215	111	166
×3/16	3.63	2.72	109	163	84.3	126
×1/8	2.46	1.85	73.7	111	57.4	86.0

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.968A_g$.

HSS6–HSS5

Table 5-4 (continued)
Available Strength in Axial Tension
Rectangular HSS

F_y = 50 ksi
F_u = 62 ksi

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips P_n/Ω_t	Yielding kips $\phi_t P_n$	Rupture kips P_n/Ω_t	Rupture kips $\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
HSS6×4×1/2	7.88	5.91	236	355	183	275
×3/8	6.18	4.63	185	278	144	216
×5/16	5.26	3.94	157	237	122	184
×1/4	4.30	3.22	129	194	100	150
×3/16	3.28	2.46	98.2	148	76.3	114
×1/8	2.23	1.67	66.8	100	51.8	77.7
HSS6×3×1/2	6.95	5.21	208	313	162	242
×3/8	5.48	4.11	164	247	127	191
×5/16	4.68	3.51	140	211	109	163
×1/4	3.84	2.88	115	173	89.3	134
×3/16	2.93	2.20	87.7	132	68.2	102
×1/8	2.00	1.50	59.9	90.0	46.5	69.8
HSS6×2×3/8	4.78	3.58	143	215	111	167
×5/16	4.10	3.08	123	185	95.5	143
×1/4	3.37	2.53	101	152	78.4	118
×3/16	2.58	1.94	77.2	116	60.1	90.2
×1/8	1.77	1.33	53.0	79.7	41.2	61.8
HSS5×4×1/2	6.95	5.21	208	313	162	242
×3/8	5.48	4.11	164	247	127	191
×5/16	4.68	3.51	140	211	109	163
×1/4	3.84	2.88	115	173	89.3	134
×3/16	2.93	2.20	87.7	132	68.2	102
×1/8	2.00	1.50	59.9	90.0	46.5	69.8
HSS5×3×1/2	6.02	4.51	180	271	140	210
×3/8	4.78	3.58	143	215	111	167
×5/16	4.10	3.08	123	185	95.5	143
×1/4	3.37	2.53	101	152	78.4	118
×3/16	2.58	1.94	77.2	116	60.1	90.2
×1/8	1.77	1.33	53.0	79.7	41.2	61.8
HSS5×2 1/2×1/4	3.14	2.36	94.0	141	73.2	110
×3/16	2.41	1.81	72.2	108	56.1	84.2
×1/8	1.65	1.24	49.4	74.3	38.4	57.7

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.968A_g$.

F_y = 50 ksi
F_u = 62 ksi

Table 5-4 (continued)
Available Strength in Axial Tension
Rectangular HSS

HSS5–HSS3 1/2

Shape	Gross Area, A_g	A_e = 0.75A_g	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
HSS5×2×3/8	4.09	3.07	122	184	95.2	143
×5/16	3.52	2.64	105	158	81.8	123
×1/4	2.91	2.18	87.1	131	67.6	101
×3/16	2.24	1.68	67.1	101	52.1	78.1
×1/8	1.54	1.16	46.1	69.3	36.0	53.9
HSS4×3×3/8	4.09	3.07	122	184	95.2	143
×5/16	3.52	2.64	105	158	81.8	123
×1/4	2.91	2.18	87.1	131	67.6	101
×3/16	2.24	1.68	67.1	101	52.1	78.1
×1/8	1.54	1.16	46.1	69.3	36.0	53.9
HSS4×2 1/2×3/8	3.74	2.81	112	168	87.1	131
×5/16	3.23	2.42	96.7	145	75.0	113
×1/4	2.67	2.00	79.9	120	62.0	93.0
×3/16	2.06	1.55	61.7	92.7	48.1	72.1
×1/8	1.42	1.07	42.5	63.9	33.2	49.8
HSS4×2×3/8	3.39	2.54	101	153	78.7	118
×5/16	2.94	2.21	88.0	132	68.5	103
×1/4	2.44	1.83	73.1	110	56.7	85.1
×3/16	1.89	1.42	56.6	85.1	44.0	66.0
×1/8	1.30	0.975	38.9	58.5	30.2	45.3
HSS3 1/2×2 1/2×3/8	3.39	2.54	101	153	78.7	118
×5/16	2.94	2.21	88.0	132	68.5	103
×1/4	2.44	1.83	73.1	110	56.7	85.1
×3/16	1.89	1.42	56.6	85.1	44.0	66.0
×1/8	1.30	0.975	38.9	58.5	30.2	45.3
HSS3 1/2×2×1/4	2.21	1.66	66.2	99.5	51.5	77.2
×3/16	1.71	1.28	51.2	77.0	39.7	59.5
×1/8	1.19	0.892	35.6	53.6	27.7	41.5
HSS3 1/2×1 1/2×1/4	1.97	1.48	59.0	88.7	45.9	68.8
×3/16	1.54	1.16	46.1	69.3	36.0	53.9
×1/8	1.07	0.803	32.0	48.2	24.9	37.3

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.968A_g$.

Table 5-4 (continued)
Available Strength in Axial Tension
Rectangular HSS

HSS3–HSS2

F_y = 50 ksi
F_u = 62 ksi

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding, kips		Rupture, kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
HSS3×2 1/2×5/16	2.64	1.98	79.0	119	61.4	92.1
×1/4	2.21	1.66	66.2	99.5	51.5	77.2
×3/16	1.71	1.28	51.2	77.0	39.7	59.5
×1/8	1.19	0.892	35.6	53.6	27.7	41.5
HSS3×2×5/16	2.35	1.76	70.4	106	54.6	81.8
×1/4	1.97	1.48	59.0	88.7	45.9	68.8
×3/16	1.54	1.16	46.1	69.3	36.0	53.9
×1/8	1.07	0.803	32.0	48.2	24.9	37.3
HSS3×1 1/2×1/4	1.74	1.30	52.1	78.3	40.6	60.9
×3/16	1.37	1.03	41.0	61.7	31.9	47.9
×1/8	0.956	0.717	28.6	43.0	22.2	33.3
HSS3×1×3/16	1.19	0.892	35.6	53.6	27.7	41.5
×1/8	0.840	0.630	25.1	37.8	19.5	29.3
HSS2 1/2×2×1/4	1.74	1.30	52.1	78.3	40.6	60.9
×3/16	1.37	1.03	41.0	61.7	31.9	47.9
×1/8	0.956	0.717	28.6	43.0	22.2	33.3
HSS2 1/2×1 1/2×1/4	1.51	1.13	45.2	68.0	35.0	52.5
×3/16	1.19	0.892	35.6	53.6	27.7	41.5
×1/8	0.840	0.630	25.1	37.8	19.5	29.3
HSS2 1/2×1×3/16	1.02	0.765	30.5	45.9	23.7	35.6
×1/8	0.724	0.543	21.7	32.6	16.8	25.2
HSS2 1/4×2×3/16	1.28	0.960	38.3	57.6	29.8	44.6
×1/8	0.898	0.674	26.9	40.4	20.9	31.3
HSS2×1 1/2×3/16	1.02	0.765	30.5	45.9	23.7	35.6
×1/8	0.724	0.543	21.7	32.6	16.8	25.2
HSS2×1×3/16	0.845	0.634	25.3	38.0	19.7	29.5
×1/8	0.608	0.456	18.2	27.4	14.1	21.2

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.968A_g$.

F_y = 50 ksi
F_u = 62 ksi

Table 5-5
Available Strength in Axial Tension
Square HSS

HSS16–HSS8

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips P_n/Ω_t	Yielding kips $\phi_t P_n$	Rupture kips P_n/Ω_t	Rupture kips $\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
HSS16×16×5/8	35.0	26.3	1050	1580	815	1220
×1/2	28.3	21.2	847	1270	657	986
×3/8	21.5	16.1	644	968	499	749
×5/16	18.1	13.6	542	815	422	632
HSS14×14×5/8	30.3	22.7	907	1360	704	1060
×1/2	24.6	18.5	737	1110	574	860
×3/8	18.7	14.0	560	842	434	651
×5/16	15.7	11.8	470	707	366	549
HSS12×12×5/8	25.7	19.3	769	1160	598	897
×1/2	20.9	15.7	626	941	487	730
×3/8	16.0	12.0	479	720	372	558
×5/16	13.4	10.1	401	603	313	470
×1/4	10.8	8.10	323	486	251	377
×3/16	8.15	6.11	244	367	189	284
HSS10×10×5/8	21.0	15.8	629	945	490	735
×1/2	17.2	12.9	515	774	400	600
×3/8	13.2	9.90	395	594	307	460
×5/16	11.1	8.32	332	500	258	387
×1/4	8.96	6.72	268	403	208	312
×3/16	6.76	5.07	202	304	157	236
HSS9×9×5/8	18.7	14.0	560	842	434	651
×1/2	15.3	11.5	458	689	357	535
×3/8	11.8	8.85	353	531	274	412
×5/16	9.92	7.44	297	446	231	346
×1/4	8.03	6.02	240	361	187	280
×3/16	6.06	4.55	181	273	141	212
×1/8	4.09	3.07	122	184	95.2	143
HSS8×8×5/8	16.4	12.3	491	738	381	572
×1/2	13.5	10.1	404	608	313	470
×3/8	10.4	7.80	311	468	242	363
×5/16	8.76	6.57	262	394	204	306
×1/4	7.10	5.33	213	320	165	248
×3/16	5.37	4.03	161	242	125	187
×1/8	3.62	2.71	108	163	84.3	126

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.968A_g$.

Table 5-5 (continued)
Available Strength in Axial Tension
Square HSS

HSS7–HSS4-1/2

F_y = 50 ksi
F_u = 62 ksi

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding, kips		Rupture, kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
HSS7×7×5/8	14.0	10.5	419	630	326	488
×1/2	11.6	8.70	347	522	270	405
×3/8	8.97	6.73	269	404	209	313
×5/16	7.59	5.69	227	342	176	265
×1/4	6.17	4.63	185	278	144	215
×3/16	4.67	3.50	140	210	109	163
×1/8	3.16	2.37	94.6	142	73.5	110
HSS6×6×5/8	11.7	8.78	350	527	272	408
×1/2	9.74	7.30	292	438	227	340
×3/8	7.58	5.69	227	341	176	265
×5/16	6.43	4.82	193	289	149	224
×1/4	5.24	3.93	157	236	122	183
×3/16	3.98	2.99	119	179	92.7	139
×1/8	2.70	2.03	80.8	122	62.9	94.4
HSS5-1/2×5-1/2×3/8	6.88	5.16	206	310	160	240
×5/16	5.85	4.39	175	263	136	204
×1/4	4.77	3.58	143	215	111	166
×3/16	3.63	2.72	109	163	84.3	126
×1/8	2.46	1.85	73.7	111	57.4	86.0
HSS5×5×1/2	7.88	5.91	236	355	183	275
×3/8	6.18	4.63	185	278	144	216
×5/16	5.26	3.94	157	237	122	184
×1/4	4.30	3.22	129	194	100	150
×3/16	3.28	2.46	98.2	148	76.3	114
×1/8	2.23	1.67	66.8	100	51.8	77.7
HSS4-1/2×4-1/2×1/2	6.95	5.21	208	313	162	242
×3/8	5.48	4.11	164	247	127	191
×5/16	4.68	3.51	140	211	109	163
×1/4	3.84	2.88	115	173	89.3	134
×3/16	2.93	2.20	87.7	132	68.2	102
×1/8	2.00	1.50	59.9	90.0	46.5	69.8

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.968A_g$.

Table 5-5 (continued)
Available Strength in Axial Tension
Square HSS

F_y = 50 ksi
F_u = 62 ksi

HSS4–HSS2

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding, kips		Rupture, kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
HSS4×4×1/2	6.02	4.51	180	271	140	210
×3/8	4.78	3.58	143	215	111	167
×5/16	4.10	3.08	123	185	95.5	143
×1/4	3.37	2.53	101	152	78.4	118
×3/16	2.58	1.94	77.2	116	60.1	90.2
×1/8	1.77	1.33	53.0	79.7	41.2	61.8
HSS3 1/2×3 1/2×3/8	4.09	3.07	122	184	95.2	143
×5/16	3.52	2.64	105	158	81.8	123
×1/4	2.91	2.18	87.1	131	67.6	101
×3/16	2.24	1.68	67.1	101	52.1	78.1
×1/8	1.54	1.16	46.1	69.3	36.0	53.9
HSS3×3×3/8	3.39	2.54	101	153	78.7	118
×5/16	2.94	2.21	88.0	132	68.5	103
×1/4	2.44	1.83	73.1	110	56.7	85.1
×3/16	1.89	1.42	56.6	85.1	44.0	66.0
×1/8	1.30	0.975	38.9	58.5	30.2	45.3
HSS2 1/2×2 1/2×5/16	2.35	1.76	70.4	106	54.6	81.8
×1/4	1.97	1.48	59.0	88.7	45.9	68.8
×3/16	1.54	1.16	46.1	69.3	36.0	53.9
×1/8	1.07	0.803	32.0	48.2	24.9	37.3
HSS2 1/4×2 1/4×1/4	1.74	1.30	52.1	78.3	40.6	60.9
×3/16	1.37	1.03	41.0	61.7	31.9	47.9
×1/8	0.956	0.717	28.6	43.0	22.2	33.3
HSS2×2×1/4	1.51	1.13	45.2	68.0	35.0	52.5
×3/16	1.19	0.892	35.6	53.6	27.7	41.5
×1/8	0.840	0.630	25.1	37.8	19.5	29.3

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.968A_g$.

HSS20.000–
HSS10.000

Table 5-6
Available Strength in Axial Tension
Round HSS

F_y = 46 ksi
F_u = 62 ksi

Shape	Gross Area, A_g	A_e = $0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
HSS20.000×0.500	28.5	21.4	785	1180	663	995
×0.375	21.5	16.1	592	890	499	749
HSS18.000×0.500	25.6	19.2	705	1060	595	893
×0.375	19.4	14.6	534	803	453	679
HSS16.000×0.625	28.1	21.1	774	1160	654	981
×0.500	22.7	17.0	625	940	527	791
×0.438	19.9	14.9	548	824	462	693
×0.375	17.2	12.9	474	712	400	600
×0.312	14.4	10.8	397	596	335	502
×0.250	11.5	8.63	317	476	268	401
HSS14.000×0.625	24.5	18.4	675	1010	570	856
×0.500	19.8	14.9	545	820	462	693
×0.375	15.0	11.3	413	621	350	525
×0.312	12.5	9.38	344	518	291	436
×0.250	10.1	7.58	278	418	235	352
HSS12.750×0.500	17.9	13.4	493	741	415	623
×0.375	13.6	10.2	375	563	316	474
×0.250	9.16	6.87	252	379	213	319
HSS10.750×0.500	15.0	11.3	413	621	350	525
×0.375	11.4	8.55	314	472	265	398
×0.250	7.70	5.78	212	319	179	269
HSS10.000×0.625	17.2	12.9	474	712	400	600
×0.500	13.9	10.4	383	575	322	484
×0.375	10.6	7.95	292	439	246	370
×0.312	8.88	6.66	245	368	206	310
×0.250	7.15	5.36	197	296	166	249
×0.188	5.37	4.03	148	222	125	187

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.890A_g$.

F_y = 46 ksi
F_u = 62 ksi

Table 5-6 (continued)
Available Strength in Axial Tension
Round HSS

HSS9.625–HSS6.875

Shape	Gross Area, A_g	A_e = $0.75A_g$	Yielding kips P_n/Ω_t	Yielding kips $\phi_t P_n$	Rupture kips P_n/Ω_t	Rupture kips $\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
HSS9.625×0.500	13.4	10.1	369	555	313	470
×0.375	10.2	7.65	281	422	237	356
×0.312	8.53	6.40	235	353	198	298
×0.250	6.87	5.15	189	284	160	239
×0.188	5.17	3.88	142	214	120	180
HSS8.625×0.625	14.7	11.0	405	609	341	512
×0.500	11.9	8.92	328	493	277	415
×0.375	9.07	6.80	250	375	211	316
×0.322	7.85	5.89	216	325	183	274
×0.250	6.14	4.60	169	254	143	214
×0.188	4.62	3.47	127	191	108	161
HSS7.625×0.375	7.98	5.99	220	330	186	279
×0.328	7.01	5.26	193	290	163	245
HSS7.500×0.500	10.3	7.73	284	426	240	359
×0.375	7.84	5.88	216	325	182	273
×0.312	6.59	4.94	182	273	153	230
×0.250	5.32	3.99	147	220	124	186
×0.188	4.00	3.00	110	166	93.0	140
HSS7.000×0.500	9.55	7.16	263	395	222	333
×0.375	7.29	5.47	201	302	170	254
×0.312	6.13	4.60	169	254	143	214
×0.250	4.95	3.71	136	205	115	173
×0.188	3.73	2.80	103	154	86.8	130
×0.125	2.51	1.88	69.1	104	58.3	87.4
HSS6.875×0.500	9.36	7.02	258	388	218	326
×0.375	7.16	5.37	197	296	166	250
×0.312	6.02	4.51	166	249	140	210
×0.250	4.86	3.64	134	201	113	170
×0.188	3.66	2.75	101	152	85.3	128

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.890A_g$.

Table 5-6 (continued)
Available Strength in Axial Tension
Round HSS

HSS6.625–HSS5.000

F_y = 46 ksi
F_u = 62 ksi

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips P_n/Ω_t	Yielding kips $\phi_t P_n$	Rupture kips P_n/Ω_t	Rupture kips $\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
HSS6.625×0.500	9.00	6.75	248	373	209	314
×0.432	7.86	5.90	217	325	183	274
×0.375	6.88	5.16	190	285	160	240
×0.312	5.79	4.34	159	240	135	202
×0.280	5.20	3.90	143	215	121	181
×0.250	4.68	3.51	129	194	109	163
×0.188	3.53	2.65	97.2	146	82.2	123
×0.125	2.37	1.78	65.3	98.1	55.2	82.8
HSS6.000×0.500	8.09	6.07	223	335	188	282
×0.375	6.20	4.65	171	257	144	216
×0.312	5.22	3.92	144	216	122	182
×0.280	4.69	3.52	129	194	109	164
×0.250	4.22	3.17	116	175	98.3	147
×0.188	3.18	2.39	87.6	132	74.1	111
×0.125	2.14	1.61	58.9	88.6	49.9	74.9
HSS5.563×0.500	7.45	5.59	205	308	173	260
×0.375	5.72	4.29	158	237	133	199
×0.258	4.01	3.01	110	166	93.3	140
×0.188	2.95	2.21	81.3	122	68.5	103
×0.134	2.12	1.59	58.4	87.8	49.3	73.9
HSS5.500×0.500	7.36	5.52	203	305	171	257
×0.375	5.65	4.24	156	234	131	197
×0.258	3.97	2.98	109	164	92.4	139
HSS5.000×0.500	6.62	4.97	182	274	154	231
×0.375	5.10	3.82	140	211	119	178
×0.312	4.30	3.22	118	178	100	150
×0.258	3.59	2.69	98.9	149	83.4	125
×0.250	3.49	2.62	96.1	144	81.2	122
×0.188	2.64	1.98	72.7	109	61.4	92.1
×0.125	1.78	1.34	49.0	73.7	41.5	62.3

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.890A_g$.

F_y = 46 ksi
F_u = 62 ksi

Table 5-6 (continued)
Available Strength in Axial Tension
Round HSS

HSS4.500–HSS2.500

Shape	Gross Area, A_g	A_e = $0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
HSS4.500×0.375	4.55	3.41	125	188	106	159
×0.337	4.12	3.09	113	171	95.8	144
×0.237	2.96	2.22	81.5	123	68.8	103
×0.188	2.36	1.77	65.0	97.7	54.9	82.3
×0.125	1.60	1.20	44.1	66.2	37.2	55.8
HSS4.000×0.313	3.39	2.54	93.4	140	78.7	118
×0.250	2.76	2.07	76.0	114	64.2	96.3
×0.237	2.61	1.96	71.9	108	60.8	91.1
×0.226	2.50	1.88	68.9	104	58.3	87.4
×0.220	2.44	1.83	67.2	101	56.7	85.1
×0.188	2.09	1.57	57.6	86.5	48.7	73.0
×0.125	1.42	1.07	39.1	58.8	33.2	49.8
HSS3.500×0.313	2.93	2.20	80.7	121	68.2	102
×0.300	2.82	2.11	77.7	117	65.7	98.6
×0.250	2.39	1.79	65.8	98.9	55.5	83.2
×0.216	2.08	1.56	57.3	86.1	48.4	72.5
×0.203	1.97	1.48	54.3	81.6	45.9	68.8
×0.188	1.82	1.36	50.1	75.3	42.5	63.7
×0.125	1.23	0.923	33.9	50.9	28.6	42.9
HSS3.000×0.250	2.03	1.52	55.9	84.0	47.1	70.7
×0.216	1.77	1.33	48.8	73.3	41.2	61.8
×0.203	1.67	1.25	46.0	69.1	38.8	58.1
×0.188	1.54	1.16	42.4	63.8	36.0	53.9
×0.152	1.27	0.953	35.0	52.6	29.5	44.3
×0.134	1.12	0.840	30.9	46.4	26.0	39.1
×0.125	1.05	0.788	28.9	43.5	24.4	36.6
HSS2.875×0.250	1.93	1.45	53.2	79.9	45.0	67.4
×0.203	1.59	1.19	43.8	65.8	36.9	55.3
×0.188	1.48	1.11	40.8	61.3	34.4	51.6
×0.125	1.01	0.758	27.8	41.8	23.5	35.2
HSS2.500×0.250	1.66	1.25	45.7	68.7	38.8	58.1
×0.188	1.27	0.953	35.0	52.6	29.5	44.3
×0.125	0.869	0.652	23.9	36.0	20.2	30.3

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.890A_g$.

Table 5-6 (continued)
Available Strength in Axial Tension
Round HSS

HSS2.375–HSS1.660

F_y = 46 ksi
F_u = 62 ksi

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding, kips		Rupture, kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
HSS2.375×0.250	1.57	1.18	43.2	65.0	36.6	54.9
×0.218	1.39	1.04	38.3	57.5	32.2	48.4
×0.188	1.20	0.900	33.1	49.7	27.9	41.9
×0.154	1.00	0.750	27.5	41.4	23.3	34.9
×0.125	0.823	0.617	22.7	34.1	19.1	28.7
HSS1.900×0.188	0.943	0.707	26.0	39.0	21.9	32.9
×0.145	0.749	0.562	20.6	31.0	17.4	26.1
×0.120	0.624	0.468	17.2	25.8	14.5	21.8
HSS1.660×0.140	0.625	0.469	17.2	25.9	14.5	21.8

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.890A_g$.

F_y = 35 ksi
F_u = 60 ksi

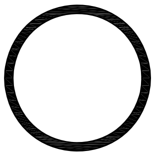
Table 5-7
Available Strength in Axial Tension
Pipe

PIPE12–PIPE1¼

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
Pipe 12 X-Strong	17.5	13.1	367	551	393	590
Std.	13.7	10.3	287	432	309	464
Pipe 10 X-Strong	15.1	11.3	316	476	339	509
Std.	11.5	8.63	241	362	259	388
Pipe 8 XX-Strong	20.0	15.0	419	630	450	675
X-Strong	11.9	8.93	249	375	268	402
Std.	7.85	5.89	165	247	177	265
Pipe 6 XX-Strong	14.7	11.0	308	463	330	495
X-Strong	7.83	5.87	164	247	176	264
Std.	5.20	3.90	109	164	117	176
Pipe 5 XX-Strong	10.7	8.03	224	337	241	361
X-Strong	5.73	4.30	120	180	129	194
Std.	4.01	3.01	84.0	126	90.3	135
Pipe 4 XX-Strong	7.66	5.75	161	241	173	259
X-Strong	4.14	3.11	86.8	130	93.3	140
Std.	2.96	2.22	62.0	93.2	66.6	99.9
Pipe 3½ X-Strong	3.43	2.57	71.9	108	77.1	116
Std.	2.50	1.88	52.4	78.8	56.4	84.6
Pipe 3 XX-Strong	5.17	3.88	108	163	116	175
X-Strong	2.83	2.12	59.3	89.1	63.6	95.4
Std.	2.07	1.55	43.4	65.2	46.5	69.8
Pipe 2½ XX-Strong	3.83	2.87	80.3	121	86.1	129
X-Strong	2.10	1.58	44.0	66.2	47.4	71.1
Std.	1.61	1.21	33.7	50.7	36.3	54.5
Pipe 2 XX-Strong	2.51	1.88	52.6	79.1	56.4	84.6
X-Strong	1.40	1.05	29.3	44.1	31.5	47.3
Std.	1.02	0.765	21.4	32.1	23.0	34.4
Pipe 1½ X-Strong	1.00	0.750	21.0	31.5	22.5	33.8
Std.	0.749	0.562	15.7	23.6	16.9	25.3
Pipe 1¼ X-Strong	0.837	0.628	17.5	26.4	18.8	28.3
Std.	0.625	0.469	13.1	19.7	14.1	21.1

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.700A_g$.



PIPE1–
PIPE$^1/_2$

Table 5-7 (continued)
Available Strength in Axial Tension
Pipe

F_y = 35 ksi
F_u = 60 ksi

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
Pipe 1 X-Strong	0.602	0.452	12.6	19.0	13.6	20.3
Std.	0.469	0.352	9.83	14.8	10.6	15.8
Pipe $^3/_4$ X-Strong	0.407	0.305	8.53	12.8	9.15	13.7
Std.	0.312	0.234	6.54	9.83	7.02	10.5
Pipe $^1/_2$ X-Strong	0.303	0.227	6.35	9.54	6.81	10.2
Std.	0.234	0.176	4.90	7.37	5.28	7.92

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.700A_g$.

F_y = 36 ksi
F_u = 58 ksi

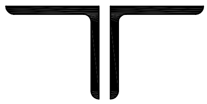
Table 5-8
Available Strength in Axial Tension
Double Angles

2L12–2L8

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
2L12×12×1 3/8	62.2	46.7	1340	2020	1350	2030
×1 1/4	56.8	42.6	1220	1840	1240	1850
×1 1/8	51.6	38.7	1110	1670	1120	1680
×1	46.0	34.5	992	1490	1000	1500
2L10×10×1 3/8	51.2	38.4	1100	1660	1110	1670
×1 1/4	46.8	35.1	1010	1520	1020	1530
×1 1/8	42.6	32.0	918	1380	928	1390
×1	38.0	28.5	819	1230	827	1240
×7/8	33.6	25.2	724	1090	731	1100
×3/4	29.0	21.8	625	940	632	948
2L8×8×1 1/8	33.6	25.2	724	1090	731	1100
×1	30.2	22.7	651	978	658	987
×7/8	26.6	20.0	573	862	580	870
×3/4	23.0	17.3	496	745	502	753
×5/8	19.4	14.6	418	629	423	635
×9/16	17.5	13.1	377	567	380	570
×1/2	15.7	11.8	338	509	342	513
2L8×6×1	26.2	19.7	565	849	571	857
×7/8	23.0	17.3	496	745	502	753
×3/4	20.0	15.0	431	648	435	653
×5/8	16.8	12.6	362	544	365	548
×9/16	15.2	11.4	328	492	331	496
×1/2	13.6	10.2	293	441	296	444
×7/16	12.0	9.00	259	389	261	392
2L8×4×1	22.2	16.7	479	719	484	726
×7/8	19.6	14.7	423	635	426	639
×3/4	17.0	12.8	366	551	371	557
×5/8	14.3	10.7	308	463	310	465
×9/16	13.0	9.75	280	421	283	424
×1/2	11.6	8.70	250	376	252	378
×7/16	10.2	7.65	220	330	222	333

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.745A_g$.



2L7–2L5

Table 5-8 (continued)
Available Strength in Axial Tension
Double Angles

F_y = 36 ksi
F_u = 58 ksi

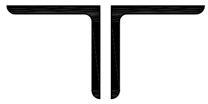
Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
2L7×4×3/4	15.5	11.6	334	502	336	505
×5/8	13.0	9.75	280	421	283	424
×1/2	10.5	7.88	226	340	229	343
×7/16	9.26	6.95	200	300	202	302
×3/8	8.00	6.00	172	259	174	261
2L6×6×1	22.0	16.5	474	713	479	718
×7/8	19.5	14.6	420	632	423	635
×3/4	16.9	12.7	364	548	368	552
×5/8	14.3	10.7	308	463	310	465
×9/16	12.9	9.68	278	418	281	421
×1/2	11.5	8.63	248	373	250	375
×7/16	10.2	7.65	220	330	222	333
×3/8	8.76	6.57	189	284	191	286
×5/16	7.34	5.51	158	238	160	240
2L6×4×7/8	16.0	12.0	345	518	348	522
×3/4	13.9	10.4	300	450	302	452
×5/8	11.7	8.78	252	379	255	382
×9/16	10.6	7.95	229	343	231	346
×1/2	9.50	7.13	205	308	207	310
×7/16	8.36	6.27	180	271	182	273
×3/8	7.22	5.42	156	234	157	236
×5/16	6.06	4.55	131	196	132	198
2L6×3 1/2×1/2	9.00	6.75	194	292	196	294
×3/8	6.88	5.16	148	223	150	224
×5/16	5.78	4.34	125	187	126	189
2L5×5×7/8	16.0	12.0	345	518	348	522
×3/4	14.0	10.5	302	454	305	457
×5/8	11.8	8.85	254	382	257	385
×1/2	9.58	7.19	207	310	209	313
×7/16	8.44	6.33	182	273	184	275
×3/8	7.30	5.48	157	237	159	238
×5/16	6.14	4.61	132	199	134	201

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.745A_g$.

F_y = 36 ksi
F_u = 58 ksi

Table 5-8 (continued)
Available Strength in Axial Tension
Double Angles



2L5–2L3½

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
2L5×3½×¾	11.7	8.78	252	379	255	382
×⅝	9.86	7.40	213	319	215	322
×½	8.00	6.00	172	259	174	261
×⅜	6.10	4.58	131	198	133	199
×5/16	5.12	3.84	110	166	111	167
×¼	4.14	3.11	89.2	134	90.2	135
2L5×3×½	7.50	5.63	162	243	163	245
×7/16	6.62	4.97	143	214	144	216
×⅜	5.72	4.29	123	185	124	187
×5/16	4.82	3.62	104	156	105	157
×¼	3.88	2.91	83.6	126	84.4	127
2L4×4×¾	10.9	8.18	235	353	237	356
×⅝	9.22	6.92	199	299	201	301
×½	7.50	5.63	162	243	163	245
×7/16	6.60	4.95	142	214	144	215
×⅜	5.72	4.29	123	185	124	187
×5/16	4.80	3.60	103	156	104	157
×¼	3.86	2.90	83.2	125	84.1	126
2L4×3½×½	7.00	5.25	151	227	152	228
×⅜	5.36	4.02	116	174	117	175
×5/16	4.50	3.38	97.0	146	98.0	147
×¼	3.64	2.73	78.5	118	79.2	119
2L4×3×⅝	7.98	5.99	172	259	174	261
×½	6.50	4.88	140	211	142	212
×⅜	4.98	3.74	107	161	108	163
×5/16	4.18	3.14	90.1	135	91.1	137
×¼	3.38	2.54	72.9	110	73.7	110
2L3½×3½×½	6.50	4.88	140	211	142	212
×7/16	5.78	4.34	125	187	126	189
×⅜	5.00	3.75	108	162	109	163
×5/16	4.20	3.15	90.5	136	91.4	137
×¼	3.40	2.55	73.3	110	74.0	111

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.745A_g$.

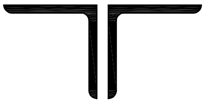

2L3 1/2–2L2 1/2

Table 5-8 (continued)
Available Strength in Axial Tension
Double Angles

F_y = 36 ksi
F_u = 58 ksi

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding, kips		Rupture, kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
2L3 1/2×3×1/2	6.04	4.53	130	196	131	197
×7/16	5.34	4.01	115	173	116	174
×3/8	4.64	3.48	100	150	101	151
×5/16	3.90	2.93	84.1	126	85.0	127
×1/4	3.16	2.37	68.1	102	68.7	103
2L3 1/2×2 1/2×1/2	5.54	4.16	119	179	121	181
×3/8	4.24	3.18	91.4	137	92.2	138
×5/16	3.58	2.69	77.2	116	78.0	117
×1/4	2.90	2.18	62.5	94.0	63.2	94.8
2L3×3×1/2	5.52	4.14	119	179	120	180
×7/16	4.86	3.65	105	157	106	159
×3/8	4.22	3.17	91.0	137	91.9	138
×5/16	3.56	2.67	76.7	115	77.4	116
×1/4	2.88	2.16	62.1	93.3	62.6	94.0
×3/16	2.18	1.64	47.0	70.6	47.6	71.3
2L3×2 1/2×1/2	5.00	3.75	108	162	109	163
×7/16	4.44	3.33	95.7	144	96.6	145
×3/8	3.86	2.90	83.2	125	84.1	126
×5/16	3.26	2.45	70.3	106	71.1	107
×1/4	2.64	1.98	56.9	85.5	57.4	86.1
×3/16	2.00	1.50	43.1	64.8	43.5	65.3
2L3×2×1/2	4.52	3.39	97.4	146	98.3	147
×3/8	3.50	2.63	75.4	113	76.3	114
×5/16	2.96	2.22	63.8	95.9	64.4	96.6
×1/4	2.40	1.80	51.7	77.8	52.2	78.3
×3/16	1.83	1.37	39.4	59.3	39.7	59.6
2L2 1/2×2 1/2×1/2	4.52	3.39	97.4	146	98.3	147
×3/8	3.46	2.60	74.6	112	75.4	113
×5/16	2.92	2.19	62.9	94.6	63.5	95.3
×1/4	2.38	1.79	51.3	77.1	51.9	77.9
×3/16	1.80	1.35	38.8	58.3	39.2	58.7

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.745A_g$.

F_y = 36 ksi
F_u = 58 ksi

Table 5-8 (continued)
Available Strength in Axial Tension
Double Angles

2L2 1/2–2L2

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding (kips) P_n/Ω_t	Yielding (kips) $\phi_t P_n$	Rupture (kips) P_n/Ω_t	Rupture (kips) $\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
2L2 1/2×2×3/8	3.10	2.33	66.8	100	67.6	101
×5/16	2.64	1.98	56.9	85.5	57.4	86.1
×1/4	2.14	1.61	46.1	69.3	46.7	70.0
×3/16	1.64	1.23	35.4	53.1	35.7	53.5
2L2 1/2×1 1/2×1/4	1.89	1.42	40.7	61.2	41.2	61.8
×3/16	1.45	1.09	31.3	47.0	31.6	47.4
2L2×2×3/8	2.74	2.06	59.1	88.8	59.7	89.6
×5/16	2.32	1.74	50.0	75.2	50.5	75.7
×1/4	1.89	1.42	40.7	61.2	41.2	61.8
×3/16	1.44	1.08	31.0	46.7	31.3	47.0
×1/8	0.982	0.737	21.2	31.8	21.4	32.1

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.745A_g$.

PART 6

DESIGN OF MEMBERS SUBJECT TO COMBINED FORCES

SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of W-shape and composite members subject to biaxial flexure and/or flexure in combination with axial tension or compression and/or torsion.

LOCAL BUCKLING CONSIDERATIONS

Width-to-thickness ratio limits for classification of shapes as compact, noncompact or slender-element are provided in AISC *Specification* Chapter B. Discussions of width-to-thickness ratios in Parts 3 and 6 of the Manual apply based upon the available strength being determined. Limiting width-to-thickness ratios for various values of F_y of members subjected to flexure and axial compression are presented in Table 6-1.

MEMBERS SUBJECT TO FLEXURE AND SHEAR

AISC *Specification* Chapters F and G apply to members subject to flexure and shear, respectively. Part 3 addresses design of flexural members.

The available moment strength, $\phi_b M_n$ or M_n/Ω_b, which must equal or exceed the required moment strength, M_u or M_a, respectively, can be found in Table 6-2.

The values given in Table 6-2 are based on $C_b = 1.0$. For situations where lateral-torsional buckling controls flexural design, appropriate adjustments to the nominal flexural strength may be made for $C_b > 1.0$ as follows:

$$M_{n(C_b>1.0)} = C_b M_{n(C_b=1.0)} \leq \begin{cases} M_p \text{ for compact sections} \\ M'_p \text{ for noncompact sections} \end{cases}$$

For flexural members, the available shear strength, $\phi_v V_n$ or V_n/Ω_v, which must equal or exceed the required shear strength, V_u or V_a, respectively, can be found in Table 6-2.

MEMBERS SUBJECT TO AXIAL COMPRESSION

AISC *Specification* Chapter E applies to members subject to axial compression. Part 4 addresses design of compression members.

For compression members, the available strength, $\phi_c P_n$ or P_n/Ω_c, which must equal or exceed the required strength, P_u or P_a, respectively, can be found in Table 6-2.

MEMBERS SUBJECT TO TENSION

AISC *Specification* Chapter D applies to members subject to tension. Part 5 of the Manual addresses design of tension members.

For tension members, the available strength, $\phi_t P_n$ or P_n/Ω_t, which must equal or exceed the required strength, P_u or P_a, respectively, can be found in Table 6-2.

MEMBERS SUBJECT TO COMBINED AXIAL FORCE AND FLEXURE

The interaction of required strengths for members subject to combined axial (tensile or compressive) forces and flexure must satisfy the interaction equations of AISC *Specification* Chapter H as follows:

1. Doubly symmetric and singly symmetric members: AISC *Specification* Section H1
2. Unsymmetric and other members: AISC *Specification* Section H2

The requirements of AISC *Specification* Chapters D, E and F and design considerations given in Parts 3, 4 and 5 apply to the design of members subject to combined axial force and flexure.

The adequacy of W-shapes subject to combined axial force and flexure is governed by either Equation H1-1a or Equation H1-1b of the AISC *Specification* as follows:

(a) When $\dfrac{P_r}{P_c} \geq 0.2$

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0 \qquad \text{(}\textit{Spec}\text{. Eq. H1-1a)}$$

(b) When $\dfrac{P_r}{P_c} < 0.2$

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0 \qquad \text{(}\textit{Spec}\text{. Eq. H1-1b)}$$

where

M_{cx} = available flexural strength about the x-axis, $\phi_b M_{nx}$ or M_{nx}/Ω_b, determined in accordance with AISC *Specification* Chapter F, kip-in.

M_{cy} = available flexural strength about the y-axis, $\phi_b M_{ny}$ or M_{ny}/Ω_b, determined in accordance with AISC *Specification* Chapter F, kip-in.

M_{rx} = required flexural strength about the x-axis, determined in accordance with AISC *Specification* Chapter C, using LRFD (M_{ux}) or ASD (M_{ax}) load combinations, kip-in.

M_{ry} = required flexural strength about the y-axis, determined in accordance with AISC *Specification* Chapter C, using LRFD (M_{uy}) or ASD (M_{ay}) load combinations, kip-in.

P_c = available axial strength, ϕP_n or P_n/Ω, kips

P_r = required axial strength, determined in accordance with AISC *Specification* Chapter C, using LRFD (P_u) or ASD (P_a) load combinations, kips

Parts 3, 4 and 5 address ϕ and Ω for members subject to flexure, compression and tension alone, respectively.

For W-shaped members subject to compression and flexure about the major principal axis only, the provisions of AISC *Specification* Section H1.3 may produce a more economical design than the provisions of Section H1.1.

MEMBERS SUBJECT TO COMBINED TORSION, FLEXURE, SHEAR AND/OR AXIAL FORCE

The interaction of the required strengths for members subject to torsion, flexure, shear, and/or axial force must satisfy the requirements of AISC *Specification* Section H3.

See also AISC Design Guide 9, *Torsional Analysis of Structural Steel Members* (Seaburg and Carter, 1997).

COMPOSITE MEMBERS SUBJECT TO FLEXURE, AXIAL OR COMBINED FORCES

Requirements for the design of composite members subject to axial force, flexure, shear and combined forces are given in AISC *Specification* Chapter I.

SELECTION TABLE FOR DESIGN OF FLEXURE, COMPRESSION, TENSION OR COMBINED FORCES: W-SHAPES

Steel W-shapes with F_y = 50 ksi and F_u = 65 ksi (ASTM A992) subject to flexure, compression, tension, or combined axial force and flexure may be checked for compliance with the provisions of the appropriate chapters of the AISC *Specification* using Table 6-2.

All W-shapes given in Table 1-1 are included in Table 6-2.

COEFFICIENTS FOR DESIGN OF W-SHAPES SUBJECT TO COMBINED FORCES

Previous editions of this Manual included a table in Part 6 that offered coefficients for design of W-shapes subject to combined forces; that table is now available at **www.aisc.org/manualresources** in Part IV of the *Design Examples*.

DESIGN TABLE DISCUSSION

Table 6-1. Width-to-Thickness Ratios

Values for limiting width-to-thickness ratios of various elements of the cross section subject to compression are given for a range of F_y values for use in the classification of members subject to axial compression in Table 6-1a and members subject to flexure in Table 6-1b.

Table 6-2. Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces, W-Shapes

The available strengths of the W-shapes for F_y = 50 ksi and F_u = 65 ksi (ASTM A992) given in Table 6-2 may be used to design members with only compression, tension, flexure and shear or may be used to design members subject to combined effects. All of the information presented here has already been presented in Parts 3, 4 and 5, as appropriate, but has been grouped here for ease of use.

W-Shapes Subject to Flexure

The available flexural strengths of W-shapes bent about their major principal axis are given in Table 6-2.

For flexural design, the numerical values given in the center column of Table 6-2 represent the laterally unbraced length of the beam, L_b, in feet. All applicable limit states are addressed and C_b = 1. Values of L_p and M_p listed for noncompact sections represent L'_p and M'_p, as defined in Part 3.

The available flexural strength of the W-shapes bent about minor principal axis are given in the lower portion of Table 6-2. Because the limit state of lateral-torsional buckling does not apply to bending of W-shapes about their minor axis, the available strength is a single value based on the limit state of yielding or flange local buckling.

W-Shapes Subject to Shear

The available shear strengths of W-shapes are given in the lower portion of Table 6-2.

All W-shapes with $F_y = 50$ ksi meet the requirements of either Section G2.1(a) or Section G2.1(b)(1)(i) of the AISC *Specification*. Available shear strengths listed in Table 6-2 take into consideration these provisions. W-shapes not meeting the requirements of Section G2.1(a) are identified in the table with footnotes.

W-Shapes Subject to Compression

The available compressive strengths of W-shapes are given in Table 6-2.

For compression the numerical values given in the center column of the table represent the effective length, L_c, of the column in feet with respect to the least radius of gyration, r_y. Therefore, the table should be entered with the larger of L_{cy} and $L_{cy\,eq}$, where

$$L_{cy\,eq} = \frac{L_{cx}}{\dfrac{r_x}{r_y}}$$

The available compressive strengths listed in Table 6-2 account for flexural buckling and local buckling as appropriate for W-shapes with $F_y = 50$ ksi. Compressive strengths are given for a range of effective lengths up to a slenderness ratio not exceeding 200. Those W-shapes with elements initially defined as slender are identified in the table with footnotes.

W-Shapes Subject to Tension

The available tensile strengths of W-shapes are given in the lower portion of Table 6-2 for the limit states of tensile yielding and tensile rupture.

Strengths given for the limit state of tensile rupture are based on the assumption that $A_e = 0.75A_g$.

W-Shapes Subject to Combined Forces

AISC *Specification* Equation H1-1a or Equation H1-1b governs the design of W-shapes subject to combined axial force and flexure. The values of available strengths in tension, compression or flexure obtained from Table 6-2 may be used to check interaction through these equations or the equations given in AISC *Specification* Section H1.3.

Table 6-3. Cross-Section Strength for Rectangular Encased W-Shapes

Tables 6-3a and 6-3b present equations applicable to the design of W-shape members encased in concrete subject to combined compression and flexure according to the plastic stress distribution method defined in AISC *Specification* Section I1.2a and Geschwindner (2010). The nominal axial and flexural strengths as well as equations for the pertinent properties are given for encased composite members subjected to flexure about the *x*-axis and *y*-axis in Tables 6-3a and 6-3b, respectively, depending on where the plastic neutral axis is located in the member. The given equations may be used with the interaction diagram (Method 2) or the simplified interaction equations (Method 2—Simplified) as discussed in AISC *Specification* Commentary Section I5.

Table 6-4. Cross-Section Strength for Composite Filled Rectangular HSS

Table 6-4 presents equations applicable to the design of concrete filled rectangular members subject to combined compression and flexure according to the plastic stress distribution method defined in AISC *Specification* Section I1.2a and Geschwindner (2010). The nominal axial and flexural strengths as well as equations for the pertinent properties are given for composite members subjected to flexure about either principal axis. The table is only applicable to filled composite members classified as compact in accordance with AISC *Specification* Section I1.4. The given equations may be used with the interaction diagram (Method 2) or the simplified interaction equations (Method 2—Simplified) as discussed in AISC *Specification* Commentary Section I5.

Table 6-5. Cross-Section Strength for Composite Filled Round HSS

Table 6-5 presents equations applicable to the design of concrete filled circular members subject to combined compression and flexure according to the plastic stress distribution method defined in AISC *Specification* Section I1.2a, Geschwindner (2010) and Denavit et al. (2015). The nominal axial and flexural strengths as well as equations for the pertinent properties are given for filled composite members bent about any axis. The table is only applicable to filled composite members classified as compact in accordance with AISC *Specification* Section I1.4. The given equations may be used with the interaction diagram (Method 2) or the simplified interaction equations (Method 2—Simplified) as discussed in AISC *Specification* Commentary Section I5.

PART 6 REFERENCES

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Table 6-1a
Width-to-Thickness Ratios: Compression Elements Members Subject to Axial Compression

	Case	Description of Element	Width-to-Thickness Ratio	F_y, ksi 32	36	42	46	50
				λ_r	λ_r	λ_r	λ_r	λ_r
Unstiffened Elements	1	Flanges of rolled I-shaped sections, plates projecting from rolled I-shaped sections, outstanding legs of pairs of angles connected with continuous contact, flanges of channels, and flanges of tees	b/t	–	15.9	14.7	–	13.5
	2	Flanges of built-up I-shaped sections and plates or angle legs projecting from built-up I-shaped sections	b/t	$19.3\sqrt{k_c}$	$18.2\sqrt{k_c}$	$16.8\sqrt{k_c}$	–	$15.4\sqrt{k_c}$
	3	Legs of single angles, legs of double angles with separators, and all other unstiffened elements	b/t	–	12.8	11.8	–	10.8
	4	Stems of tees	d/t	–	21.3	19.7	–	18.1
Stiffened Elements	5	Webs of doubly symmetric rolled and built-up I-shaped sections and channels	h/t_w	–	42.3	39.2	–	35.9
	6	Walls of rectangular HSS	b/t	–	39.7	–	35.2	33.7
	7	Flange cover plates and diaphragm plates between lines of fasteners or welds	b/t	42.1	39.7	36.8	–	33.7
	8	All other stiffened elements	b/t	44.9	42.3	39.2	37.4	35.9
	9	Round HSS	D/t	–	88.6	76.0	69.3	63.8

Note: See Tables 2-4 and 2-5 for preferred material specification.
– Indicates that element is not available with specified F_y.
$k_c = 4/\sqrt{h/t_w}$, but shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes.

Table 6-1a (continued)
Width-to-Thickness Ratios: Compression Elements Members Subject to Axial Compression

	Case	Description of Element	Width-to-Thickness Ratio	F_y, ksi				
				55	58	60	65	70
				λ_r	λ_r	λ_r	λ_r	λ_r
Unstiffened Elements	1	Flanges of rolled I-shaped sections, plates projecting from rolled I-shaped sections, outstanding legs of pairs of angles connected with continuous contact, flanges of channels, and flanges of tees	b/t	12.9	–	12.3	11.8	11.4
	2	Flanges of built-up I-shaped sections and plates or angle legs projecting from built-up I-shaped sections	b/t	$14.7\sqrt{k_c}$	–	$14.1\sqrt{k_c}$	$13.5\sqrt{k_c}$	$13.0\sqrt{k_c}$
	3	Legs of single angles, legs of double angles with separators, and all other unstiffened elements	b/t	10.3	–	9.89	9.51	9.16
	4	Stems of tees	d/t	17.2	–	16.5	15.8	15.3
Stiffened Elements	5	Webs of doubly symmetric rolled and built-up I-shaped sections and channels	h/t_w	34.2	–	32.8	31.5	30.3
	6	Walls of rectangular HSS	b/t	32.1	31.3	–	–	–
	7	Flange cover plates and diaphragm plates between lines of fasteners or welds	b/t	32.1	–	30.8	29.6	28.5
	8	All other stiffened elements	b/t	34.2	–	32.8	31.5	30.3
	9	Round HSS	D/t	–	–	–	–	–

Note: See Tables 2-4 and 2-5 for preferred material specification.
– Indicates that element is not available with specified F_y.
$k_c = 4/\sqrt{h/t_w}$, but shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes.

Table 6-1b
Width-to-Thickness Ratios: Compression Elements Members Subject to Flexure

	Case	Description of Element	Width-to-Thickness Ratio	F_y, ksi									
				32		36		42		46		50	
				λ_p	λ_r	λ_p	λ_r	λ_p	λ_r	λ_p	λ_r	λ_p	λ_r
Unstiffened Elements	10	Flanges of rolled I-shaped sections, channels, and tees	b/t	–	–	10.8	28.4	9.99	26.3	–	–	9.15	24.1
	11	Flanges of doubly and singly symmetric I-shaped built-up sections	b/t	11.4	[a]	10.8	[a]	9.99	[a]	–	–	9.15	[a]
	12	Legs of single angles	b/t	–	–	15.3	25.8	14.2	23.9	–	–	13.0	21.9
	13	Flanges of all I-shaped sections and channels in flexure about the minor axis	b/t	11.4	30.1	10.8	28.4	9.99	26.3	–	–	9.15	24.1
	14	Stems of tees	d/t	–	–	23.8	43.1	22.1	39.9	–	–	20.2	36.6
Stiffened Elements	15	Webs of doubly symmetric I-shaped sections and channels	h/t_w	113	172	107	162	98.8	150	–	–	90.6	137
	16	Webs of singly symmetric I-shaped sections	h_c/t_w	[a]	172	[a]	162	[a]	150	–	–	[a]	137
	17	Flanges of rectangular HSS	b/t	–	–	31.8	39.7	–	–	28.1	35.2	27.0	33.7
	18	Flange cover plates and diaphragm plates between lines of fasteners or welds	b/t	33.7	42.1	31.8	39.7	29.4	36.8	–	–	27.0	33.7
	19	Webs of rectangular HSS and box sections	h/t	–	–	68.7	162	–	–	60.8	143	58.3	137
	20	Round HSS	D/t	–	–	56.4	250	48.3	214	44.1	195	40.6	180
	21	Flanges of box sections	b/t	33.7	44.9	31.8	42.3	–	–	28.1	37.4	27.0	35.9

[a] See AISC *Specification* Table B4.1b.
– Indicates that element is not available with specified F_y.
Note: See Tables 2-4 and 2-5 for preferred material specification.

Table 6-1b (continued)
Width-to-Thickness Ratios: Compression Elements Members Subject to Flexure

	Case	Description of Element	Width-to-Thickness Ratio	F_y, ksi									
				55		58		60		65		70	
				λ_p	λ_r	λ_p	λ_r	λ_p	λ_r	λ_p	λ_r	λ_p	λ_r
Unstiffened Elements	10	Flanges of rolled I-shaped sections, channels, and tees	b/t	8.73	23.0	–	–	8.35	22.0	8.03	21.1	7.73	20.4
	11	Flanges of doubly and singly symmetric I-shaped built-up sections	b/t	8.73	[a]	–	–	8.35	[a]	8.03	[a]	7.73	[a]
	12	Legs of single angles	b/t	12.4	20.9	–	–	11.9	20.0	11.4	19.2	11.0	18.5
	13	Flanges of all I-shaped sections and channels in flexure about the minor axis	b/t	8.73	23.0	–	–	8.35	22.0	8.03	21.1	7.73	20.4
	14	Stems of tees	d/t	19.3	34.9	–	–	18.5	33.4	17.7	32.1	17.1	30.9
Stiffened Elements	15	Webs of doubly symmetric I-shaped sections and channels	h/t_w	86.3	131	–	–	82.7	125	79.4	120	76.5	116
	16	Webs of singly symmetric I-shaped sections	h_c/t_w	[a]	131	–	–	[a]	125	[a]	120	[a]	116
	17	Flanges of rectangular HSS	b/t	–	–	25.0	31.3	–	–	–	–	–	–
	18	Flange cover plates and diaphragm plates between lines of fasteners or welds	b/t	25.7	32.1	–	–	24.6	30.8	23.7	29.6	22.8	28.5
	19	Webs of rectangular HSS and box sections	h/t	55.6	131	–	–	–	–	–	–	–	–
	20	Round HSS	D/t	–	–	35.0	155	–	–	–	–	–	–
	21	Flanges of box sections	b/t	–	–	–	–	–	–	–	–	–	–

[a] See AISC *Specification* Table B4.1b.
– Indicates that element is not available with specified F_y.
Note: See Tables 2-4 and 2-5 for preferred material specification.

Table 6-2
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W44

W44×						Shape	W44×					
335[c]		290[c]		262[c]		lb/ft	335		290		262	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
2900	4360	2400	3610	2110	3180	0	4040	6080	3520	5290	3170	4760
2830	4250	2340	3520	2060	3090	6	4040	6080	3520	5290	3170	4760
2800	4210	2320	3480	2040	3060	7	4040	6080	3520	5290	3170	4760
2770	4160	2290	3440	2010	3030	8	4040	6080	3520	5290	3170	4760
2730	4110	2260	3400	1990	2990	9	4040	6080	3520	5290	3170	4760
2690	4050	2230	3350	1960	2950	10	4040	6080	3520	5290	3170	4760
2650	3990	2200	3300	1930	2900	11	4040	6080	3520	5290	3170	4760
2600	3910	2160	3240	1900	2850	12	4040	6080	3520	5290	3170	4760
2550	3830	2120	3180	1860	2800	13	4000	6010	3480	5230	3130	4700
2490	3740	2080	3120	1820	2740	14	3940	5930	3430	5150	3080	4620
2430	3650	2030	3050	1780	2680	15	3880	5840	3370	5070	3020	4550
2360	3550	1990	2980	1740	2620	16	3820	5750	3320	4980	2970	4470
2300	3450	1940	2910	1700	2560	17	3760	5660	3260	4900	2920	4390
2230	3350	1890	2840	1660	2490	18	3700	5570	3210	4820	2870	4310
2160	3240	1840	2760	1610	2420	19	3640	5480	3150	4740	2810	4230
2090	3140	1780	2680	1560	2350	20	3590	5390	3100	4650	2760	4150
1940	2920	1670	2520	1470	2210	22	3470	5210	2990	4490	2660	3990
1790	2690	1550	2340	1370	2060	24	3350	5030	2880	4320	2550	3840
1640	2470	1430	2140	1270	1910	26	3230	4850	2770	4160	2450	3680
1500	2250	1300	1950	1160	1750	28	3110	4670	2660	3990	2340	3520
1350	2040	1170	1770	1050	1580	30	2990	4490	2550	3830	2240	3360
1220	1830	1060	1590	944	1420	32	2870	4320	2440	3660	2130	3200
1080	1630	939	1410	839	1260	34	2750	4140	2330	3500	2030	3050
966	1450	838	1260	749	1130	36	2630	3960	2220	3330	1910	2870
867	1300	752	1130	672	1010	38	2510	3780	2070	3110	1750	2630
783	1180	679	1020	606	911	40	2360	3550	1920	2880	1620	2430
710	1070	616	925	550	827	42	2200	3310	1780	2680	1500	2260
647	972	561	843	501	753	44	2060	3100	1660	2500	1400	2100
592	890	513	771	459	689	46	1940	2910	1560	2350	1310	1970
544	817	471	708	421	633	48	1830	2750	1470	2210	1230	1850
501	753	434	653	388	583	50	1730	2600	1390	2090	1160	1740

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2950	4430	2560	3840	2310	3470
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2400	3600	2080	3120	1880	2820
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
906	1360	754	1130	680	1020
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
589	885	511	769	454	683

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
12.3	38.9	12.3	36.9	12.3	35.7
Area, in.^{2}					
98.5		85.4		77.2	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
31100	1200	27000	1040	24100	923
r_y, in.					
3.49		3.49		3.47	
r_x/r_y					
5.10		5.10		5.10	

[c] Shape is slender for compression with F_y = 50 ksi.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W44–W40

F_y = 50 ksi
F_u = 65 ksi

Shape	W44×		W40×				Shape	W44×		W40×			
	230[c]		655[h]		593[h]		lb/ft	230[v]		655[h]		593[h]	
	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	1800	2710	5780	8680	5210	7830	0	2740	4130	7680	11600	6890	10400
	1750	2630	5630	8470	5070	7630	6	2740	4130	7680	11600	6890	10400
	1740	2610	5580	8390	5030	7560	7	2740	4130	7680	11600	6890	10400
	1720	2580	5520	8300	4970	7470	8	2740	4130	7680	11600	6890	10400
	1690	2540	5460	8200	4910	7380	9	2740	4130	7680	11600	6890	10400
	1670	2510	5380	8090	4840	7280	10	2740	4130	7680	11600	6890	10400
	1640	2470	5300	7970	4770	7170	11	2740	4130	7680	11600	6890	10400
	1610	2420	5220	7840	4690	7050	12	2740	4130	7680	11600	6890	10400
	1580	2380	5130	7710	4610	6920	13	2700	4060	7680	11600	6890	10400
	1550	2330	5030	7560	4520	6790	14	2660	3990	7660	11500	6850	10300
	1510	2280	4930	7410	4420	6650	15	2610	3920	7610	11400	6800	10200
	1480	2220	4820	7250	4320	6500	16	2560	3850	7550	11400	6740	10100
	1440	2170	4710	7080	4220	6340	17	2510	3780	7500	11300	6690	10100
	1400	2110	4600	6910	4110	6180	18	2470	3710	7440	11200	6630	9970
	1360	2050	4480	6730	4000	6020	19	2420	3640	7380	11100	6580	9880
	1320	1990	4360	6550	3890	5850	20	2370	3570	7330	11000	6520	9800
	1240	1870	4100	6170	3660	5500	22	2280	3420	7210	10800	6410	9630
	1160	1740	3850	5780	3420	5140	24	2180	3280	7100	10700	6300	9470
	1070	1610	3580	5390	3180	4780	26	2090	3140	6990	10500	6190	9300
	986	1480	3320	4990	2940	4420	28	1990	3000	6880	10300	6080	9130
	902	1360	3060	4600	2700	4060	30	1900	2860	6770	10200	5970	8970
	812	1220	2800	4210	2470	3710	32	1810	2710	6650	10000	5860	8800
	720	1080	2550	3840	2240	3370	34	1710	2570	6540	9830	5740	8630
	642	966	2310	3480	2020	3040	36	1570	2350	6430	9660	5630	8470
	577	867	2080	3120	1820	2730	38	1430	2150	6320	9490	5520	8300
	520	782	1880	2820	1640	2460	40	1320	1980	6200	9320	5410	8130
	472	709	1700	2560	1490	2230	42	1220	1830	6090	9160	5300	7970
	430	646	1550	2330	1350	2040	44	1130	1700	5980	8990	5190	7800
	393	591	1420	2130	1240	1860	46	1060	1590	5870	8820	5080	7630
	361	543	1300	1960	1140	1710	48	991	1490	5750	8650	4970	7470
	333	501	1200	1800	1050	1580	50	932	1400	5640	8480	4860	7300

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2030	3050	5780	8690	5210	7830
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1650	2480	4710	7070	4260	6390
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
547	822	1720	2580	1540	2310
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
392	589	1350	2030	1200	1800

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
12.1	34.3	13.6	69.9	13.4	63.9
Area, in.²					
67.8		193		174	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
20800	796	56500	2870	50400	2520
r_y, in.					
3.43		3.86		3.80	
r_x/r_y					
5.10		4.43		4.47	

[c] Shape is slender for compression with F_y = 50 ksi.
[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 50 ksi; therefore, ϕ_v = 0.90 and Ω_v = 1.67.

F_y = 50 ksi
F_u = 65 ksi

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W40

Shape	W40×						W40×					
lb/ft	503[h]		431[h]		397[h]		503[h]		431[h]		397[h]	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips						Available Flexural Strength, kip-ft					
Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	4430	6660	3800	5710	3500	5260	5790	8700	4890	7350	4490	6750
6	4310	6480	3700	5550	3400	5120	5790	8700	4890	7350	4490	6750
7	4270	6420	3660	5500	3370	5060	5790	8700	4890	7350	4490	6750
8	4220	6340	3610	5430	3330	5000	5790	8700	4890	7350	4490	6750
9	4170	6260	3570	5360	3280	4940	5790	8700	4890	7350	4490	6750
10	4110	6170	3510	5280	3240	4860	5790	8700	4890	7350	4490	6750
11	4040	6070	3460	5190	3180	4780	5790	8700	4890	7350	4490	6750
12	3970	5970	3390	5100	3120	4700	5790	8700	4890	7350	4490	6750
13	3900	5860	3330	5000	3060	4600	5790	8700	4880	7340	4480	6740
14	3820	5740	3260	4890	3000	4510	5740	8630	4830	7260	4430	6660
15	3730	5610	3180	4780	2930	4400	5690	8550	4780	7180	4380	6580
16	3650	5480	3110	4670	2860	4300	5630	8460	4720	7100	4330	6500
17	3560	5350	3030	4550	2780	4180	5570	8380	4670	7020	4270	6420
18	3460	5200	2940	4420	2710	4070	5520	8300	4620	6940	4220	6350
19	3370	5060	2860	4300	2630	3950	5460	8210	4560	6860	4170	6270
20	3270	4910	2770	4170	2550	3830	5410	8130	4510	6780	4120	6190
22	3070	4610	2590	3900	2380	3580	5300	7960	4400	6620	4010	6030
24	2860	4300	2410	3630	2220	3330	5190	7800	4290	6460	3910	5880
26	2650	3980	2230	3350	2050	3080	5080	7630	4190	6290	3800	5720
28	2440	3670	2050	3080	1880	2820	4970	7460	4080	6130	3700	5560
30	2230	3360	1870	2810	1710	2580	4850	7300	3970	5970	3600	5400
32	2030	3060	1690	2540	1550	2330	4740	7130	3870	5810	3490	5250
34	1840	2760	1530	2290	1400	2100	4630	6960	3760	5650	3390	5090
36	1650	2480	1360	2050	1250	1880	4520	6800	3650	5490	3280	4930
38	1480	2230	1220	1840	1120	1680	4410	6630	3540	5330	3180	4780
40	1340	2010	1100	1660	1010	1520	4300	6460	3440	5170	3070	4620
42	1210	1820	1000	1500	917	1380	4190	6300	3330	5010	2970	4460
44	1100	1660	912	1370	836	1260	4080	6130	3220	4840	2860	4300
46	1010	1520	835	1250	765	1150	3970	5960	3120	4680	2760	4150
48	928	1390	767	1150	702	1060	3860	5800	3010	4520	2630	3950
50	855	1290	706	1060	647	973	3750	5630	2880	4330	2500	3760

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
4430	6660	3800	5720	3500	5270
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3610	5410	3100	4650	2850	4280
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
1300	1950	1110	1660	1000	1500
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
983	1480	818	1230	749	1130

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
13.1	55.2	12.9	49.1	12.9	46.7
Area, in.²					
148		127		117	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
41600	2040	34800	1690	32000	1540
r_y, in.					
3.72		3.65		3.64	
r_x/r_y					
4.52		4.55		4.56	

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W40

F_y = 50 ksi
F_u = 65 ksi

Shape	W40× 392[h]		W40× 372[h]		W40× 362[h]			W40× 392[h]		W40× 372[h]		W40× 362[h]	
lb/ft	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
	3470	5220	3290	4950	3170	4770	0	4270	6410	4190	6300	4090	6150
	3290	4940	3200	4810	3080	4630	6	4270	6410	4190	6300	4090	6150
	3230	4850	3160	4760	3050	4580	7	4270	6410	4190	6300	4090	6150
	3150	4740	3130	4700	3010	4530	8	4270	6410	4190	6300	4090	6150
	3070	4620	3080	4630	2970	4470	9	4270	6410	4190	6300	4090	6150
	2990	4490	3040	4560	2930	4400	10	4230	6350	4190	6300	4090	6150
	2890	4350	2990	4490	2880	4320	11	4170	6260	4190	6300	4090	6150
	2790	4200	2930	4400	2820	4240	12	4100	6170	4190	6300	4090	6150
	2690	4040	2870	4310	2770	4160	13	4040	6080	4180	6280	4080	6130
	2580	3880	2810	4220	2710	4070	14	3980	5990	4130	6200	4030	6050
	2470	3720	2740	4120	2640	3970	15	3920	5900	4070	6120	3970	5970
	2360	3550	2670	4020	2580	3870	16	3860	5810	4020	6040	3920	5900
	2240	3370	2600	3910	2510	3770	17	3800	5720	3970	5970	3870	5820
	2130	3200	2530	3800	2440	3670	18	3740	5620	3920	5890	3820	5740
	2010	3030	2460	3690	2370	3560	19	3680	5530	3870	5810	3770	5660
	1900	2850	2380	3580	2290	3450	20	3620	5440	3810	5730	3720	5590
	1670	2510	2220	3340	2140	3220	22	3500	5260	3710	5580	3610	5430
	1450	2190	2060	3100	1990	2990	24	3380	5080	3610	5420	3510	5280
	1250	1880	1900	2860	1830	2750	26	3260	4900	3500	5270	3410	5120
	1080	1620	1740	2620	1680	2520	28	3140	4720	3400	5110	3310	4970
	938	1410	1590	2380	1530	2300	30	3020	4530	3300	4960	3200	4810
	824	1240	1430	2150	1380	2080	32	2900	4350	3190	4800	3100	4660
	730	1100	1290	1940	1240	1860	34	2780	4170	3090	4640	3000	4500
	651	979	1150	1730	1110	1660	36	2650	3990	2990	4490	2890	4350
	584	878	1030	1550	993	1490	38	2530	3810	2880	4330	2790	4190
	527	793	930	1400	896	1350	40	2390	3590	2780	4180	2690	4040
	478	719	844	1270	813	1220	42	2260	3390	2680	4020	2580	3880
	436	655	769	1160	741	1110	44	2140	3220	2570	3870	2480	3730
			703	1060	678	1020	46	2040	3060	2440	3660	2340	3520
			646	971	622	935	48	1940	2920	2310	3470	2220	3330
			595	895	574	862	50	1850	2790	2190	3300	2110	3170

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3470	5220	3290	4950	3170	4770
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2830	4240	2680	4020	2580	3880
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
1180	1770	942	1410	909	1360
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
519	780	691	1040	674	1010

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.33	38.3	12.7	44.4	12.7	44.0
Area, in.²					
116		110		106	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
29900	803	29600	1420	28900	1380
r_y, in.					
2.64		3.60		3.60	
r_x/r_y					
6.10		4.58		4.58	

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W40

W40× 331[h]		327[h]		324		Shape lb/ft	W40× 331[h]		327[h]		324	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
2930	4400	2870	4320	2850	4290	0	3570	5360	3520	5290	3640	5480
2760	4150	2710	4080	2770	4160	6	3570	5360	3520	5290	3640	5480
2710	4070	2660	3990	2740	4120	7	3570	5360	3520	5290	3640	5480
2640	3970	2590	3900	2710	4070	8	3570	5360	3520	5290	3640	5480
2570	3860	2530	3800	2670	4010	9	3570	5360	3520	5290	3640	5480
2490	3750	2450	3680	2630	3950	10	3510	5280	3470	5210	3640	5480
2410	3630	2370	3560	2580	3880	11	3450	5190	3410	5120	3640	5480
2330	3490	2290	3440	2530	3810	12	3400	5100	3350	5040	3640	5480
2230	3360	2200	3300	2480	3730	13	3340	5010	3290	4950	3630	5450
2140	3220	2110	3170	2430	3650	14	3280	4930	3230	4860	3580	5370
2040	3070	2010	3020	2370	3560	15	3220	4840	3180	4770	3530	5300
1940	2920	1920	2880	2310	3480	16	3160	4750	3120	4690	3480	5230
1850	2770	1820	2730	2250	3380	17	3100	4660	3060	4600	3430	5150
1750	2620	1720	2580	2190	3290	18	3040	4570	3000	4510	3380	5080
1650	2470	1620	2440	2120	3190	19	2980	4480	2940	4430	3330	5000
1550	2320	1530	2290	2050	3090	20	2920	4390	2890	4340	3280	4930
1350	2030	1340	2010	1920	2880	22	2810	4220	2770	4160	3180	4780
1170	1760	1150	1740	1780	2670	24	2690	4040	2660	3990	3080	4640
996	1500	986	1480	1640	2460	26	2570	3860	2540	3820	2990	4490
859	1290	850	1280	1500	2250	28	2450	3690	2420	3640	2890	4340
748	1120	740	1110	1360	2050	30	2330	3510	2310	3470	2790	4190
658	989	651	978	1230	1850	32	2220	3330	2190	3290	2690	4040
583	876	576	866	1100	1660	34	2090	3140	2070	3110	2590	3900
520	781	514	773	984	1480	36	1950	2930	1920	2890	2490	3750
466	701	461	694	883	1330	38	1820	2740	1800	2710	2390	3600
421	633	416	626	797	1200	40	1710	2570	1690	2540	2300	3450
382	574	378	568	723	1090	42	1620	2430	1600	2400	2180	3270
				659	990	44	1530	2300	1510	2270	2050	3070
				603	906	46	1450	2180	1430	2150	1930	2900
				553	832	48	1380	2080	1360	2050	1820	2740
				510	766	50	1320	1980	1300	1960	1730	2600

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2930	4400	2870	4320	2850	4290
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2380	3570	2340	3510	2320	3490
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
996	1490	963	1440	804	1210
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
423	636	419	630	596	896

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.08	33.8	9.11	33.6	12.6	41.2
Area, in.²					
97.7		95.9		95.3	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
24700	644	24500	640	25600	1220
r_y, in.					
2.57		2.58		3.58	
r_x/r_y					
6.19		6.20		4.58	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W40

$F_y = 50$ ksi
$F_u = 65$ ksi

W40× 297[c] P_n/Ω_c ASD	297[c] $\phi_c P_n$ LRFD	294 P_n/Ω_c ASD	294 $\phi_c P_n$ LRFD	278 P_n/Ω_c ASD	278 $\phi_c P_n$ LRFD	Design: Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	W40× 297 M_{nx}/Ω_b ASD	297 $\phi_b M_{nx}$ LRFD	294 M_{nx}/Ω_b ASD	294 $\phi_b M_{nx}$ LRFD	278 M_{nx}/Ω_b ASD	278 $\phi_b M_{nx}$ LRFD
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
2600	3900	2580	3880	2460	3700	0	3320	4990	3170	4760	2970	4460
2530	3800	2430	3660	2320	3490	6	3320	4990	3170	4760	2970	4460
2510	3770	2380	3580	2270	3410	7	3320	4990	3170	4760	2970	4460
2480	3720	2330	3500	2220	3330	8	3320	4990	3170	4760	2970	4460
2440	3670	2260	3400	2150	3240	9	3320	4990	3170	4760	2960	4450
2400	3610	2200	3300	2090	3140	10	3320	4990	3110	4680	2910	4370
2360	3550	2120	3190	2020	3030	11	3320	4990	3060	4590	2850	4290
2320	3480	2040	3070	1940	2920	12	3320	4990	3000	4510	2800	4210
2270	3410	1960	2950	1860	2800	13	3290	4950	2940	4420	2740	4120
2220	3330	1880	2820	1780	2680	14	3250	4880	2880	4330	2690	4040
2160	3250	1790	2690	1700	2550	15	3200	4810	2830	4250	2630	3960
2110	3170	1710	2560	1610	2420	16	3150	4740	2770	4160	2580	3870
2050	3080	1620	2430	1530	2290	17	3100	4670	2710	4080	2520	3790
1990	2990	1530	2300	1440	2160	18	3060	4600	2660	3990	2470	3710
1930	2900	1440	2160	1350	2040	19	3010	4520	2600	3910	2410	3620
1870	2810	1350	2030	1270	1910	20	2960	4450	2540	3820	2360	3540
1740	2620	1180	1770	1100	1660	22	2870	4310	2430	3650	2250	3380
1610	2420	1020	1530	947	1420	24	2770	4170	2310	3480	2140	3210
1480	2230	865	1300	807	1210	26	2680	4020	2200	3310	2030	3040
1350	2030	746	1120	696	1050	28	2580	3880	2090	3130	1910	2880
1230	1840	650	977	606	911	30	2490	3740	1970	2960	1800	2710
1110	1660	571	859	533	801	32	2390	3600	1850	2770	1660	2500
988	1480	506	761	472	709	34	2300	3450	1710	2560	1530	2300
881	1320	451	679	421	633	36	2200	3310	1580	2380	1420	2140
791	1190	405	609	378	568	38	2110	3170	1480	2220	1330	2000
714	1070	366	550	341	512	40	1990	3000	1390	2090	1250	1870
647	973	332	499	309	465	42	1860	2800	1310	1970	1170	1760
590	887					44	1740	2620	1240	1860	1110	1670
540	811					46	1640	2470	1170	1760	1050	1580
496	745					48	1550	2330	1120	1680	998	1500
457	687					50	1470	2210	1060	1600	951	1430

Properties

W40×	297[c]		294		278	
Available Strength in Tensile Yielding, kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	2610	3930	2580	3880	2460	3700
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	2130	3190	2100	3150	2010	3010
Available Strength in Shear, kips	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	740	1110	856	1280	828	1240
Available Strength in Flexure about Y-Y Axis, kip-ft	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
	536	806	373	561	348	523

W40×	297		294		278	
Limiting Unbraced Lengths, ft	L_p	L_r	L_p	L_r	L_p	L_r
	12.5	39.3	9.01	31.5	8.90	30.4
Area, in.²	87.3		86.2		82.3	
Moment of Inertia, in.⁴	I_x	I_y	I_x	I_y	I_x	I_y
	23200	1090	21900	562	20500	521
r_y, in.	3.54		2.55		2.52	
r_x/r_y	4.60		6.24		6.27	

[c] Shape is slender for compression with $F_y = 50$ ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W40

W40×						Shape	W40×					
277[c]		264		249[c]		lb/ft	277		264		249	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
2360	3550	2320	3480	2080	3120	0	3120	4690	2820	4240	2790	4200
2300	3460	2180	3280	2020	3040	6	3120	4690	2820	4240	2790	4200
2280	3420	2140	3210	2000	3010	7	3120	4690	2820	4240	2790	4200
2250	3390	2080	3130	1980	2980	8	3120	4690	2820	4240	2790	4200
2230	3350	2030	3050	1960	2940	9	3120	4690	2810	4230	2790	4200
2200	3300	1960	2950	1930	2900	10	3120	4690	2760	4150	2790	4200
2160	3250	1900	2850	1900	2860	11	3120	4690	2710	4070	2790	4200
2130	3200	1830	2740	1870	2810	12	3120	4690	2650	3990	2790	4200
2090	3140	1750	2630	1830	2760	13	3100	4660	2600	3900	2770	4170
2050	3080	1670	2520	1800	2700	14	3060	4590	2540	3820	2730	4110
2010	3020	1600	2400	1760	2650	15	3010	4530	2490	3740	2690	4040
1960	2950	1520	2280	1720	2590	16	2970	4460	2440	3660	2650	3980
1920	2880	1440	2160	1680	2520	17	2920	4390	2380	3580	2600	3910
1870	2810	1350	2040	1640	2460	18	2870	4320	2330	3500	2560	3850
1810	2730	1270	1910	1590	2390	19	2830	4250	2270	3420	2520	3780
1760	2640	1190	1790	1550	2330	20	2780	4180	2220	3340	2470	3720
1640	2460	1040	1560	1460	2190	22	2690	4040	2110	3170	2390	3590
1520	2280	891	1340	1360	2040	24	2600	3910	2000	3010	2300	3460
1400	2100	759	1140	1250	1880	26	2510	3770	1890	2850	2220	3330
1280	1930	654	984	1140	1720	28	2420	3630	1790	2690	2130	3200
1160	1750	570	857	1040	1560	30	2320	3490	1670	2510	2040	3070
1050	1580	501	753	935	1410	32	2230	3360	1530	2300	1960	2940
943	1420	444	667	836	1260	34	2140	3220	1410	2120	1870	2810
841	1260	396	595	746	1120	36	2050	3080	1310	1960	1790	2680
755	1130	355	534	670	1010	38	1960	2940	1220	1830	1680	2520
681	1020	321	482	604	908	40	1840	2760	1140	1710	1550	2330
618	929	291	437	548	824	42	1710	2570	1070	1610	1440	2170
563	846			499	751	44	1600	2410	1010	1520	1350	2030
515	774			457	687	46	1510	2260	959	1440	1270	1900
473	711			420	631	48	1420	2140	911	1370	1190	1790
436	655			387	581	50	1340	2020	867	1300	1130	1690

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2440	3670	2320	3480	2200	3310
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1990	2980	1890	2830	1790	2690
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
659	989	768	1150	591	887
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
509	765	329	495	454	683

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
12.6	38.8	8.90	29.7	12.5	37.2
Area, in.[2]					
81.5		77.4		73.5	
Moment of Inertia, in.[4]					
I_x	I_y	I_x	I_y	I_x	I_y
21900	1040	19400	493	19600	926
r_y, in.					
3.58		2.52		3.55	
r_x/r_y					
4.58		6.27		4.59	

[c] Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W40

F_y = 50 ksi
F_u = 65 ksi

W40×						Shape	W40×					
235[c]		215[c]		211[c]		lb/ft	235		215		211	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1990	2990	1730	2610	1740	2610	0	2520	3790	2410	3620	2260	3400
1890	2840	1690	2540	1650	2480	6	2520	3790	2410	3620	2260	3400
1860	2790	1670	2510	1620	2430	7	2520	3790	2410	3620	2260	3400
1820	2730	1650	2490	1580	2380	8	2520	3790	2410	3620	2260	3400
1780	2670	1630	2450	1550	2320	9	2520	3790	2410	3620	2250	3390
1730	2600	1610	2420	1500	2260	10	2470	3710	2410	3620	2210	3310
1680	2520	1590	2380	1460	2190	11	2420	3630	2410	3620	2160	3240
1630	2440	1560	2340	1410	2120	12	2370	3560	2410	3620	2110	3170
1570	2360	1530	2300	1360	2050	13	2310	3480	2390	3590	2060	3100
1500	2260	1500	2250	1310	1970	14	2260	3400	2350	3530	2010	3020
1430	2150	1470	2210	1260	1890	15	2210	3330	2310	3470	1960	2950
1360	2050	1430	2160	1200	1810	16	2160	3250	2270	3410	1910	2880
1290	1940	1400	2100	1150	1720	17	2110	3170	2230	3350	1870	2800
1220	1830	1360	2050	1080	1630	18	2060	3100	2190	3290	1820	2730
1150	1730	1330	1990	1020	1530	19	2010	3020	2150	3230	1770	2660
1080	1620	1290	1940	953	1430	20	1960	2940	2110	3170	1720	2590
939	1410	1210	1820	828	1240	22	1860	2790	2030	3060	1620	2440
808	1210	1130	1700	709	1070	24	1750	2640	1960	2940	1530	2290
688	1030	1050	1580	604	908	26	1650	2480	1880	2820	1430	2150
594	892	971	1460	521	783	28	1550	2330	1800	2700	1310	1970
517	777	892	1340	454	682	30	1410	2120	1720	2590	1180	1770
454	683	804	1210	399	599	32	1290	1940	1640	2470	1070	1610
403	605	719	1080	353	531	34	1180	1780	1560	2350	985	1480
359	540	641	963	315	474	36	1100	1650	1470	2220	909	1370
322	484	575	865	283	425	38	1020	1530	1350	2030	844	1270
291	437	519	780	255	384	40	953	1430	1250	1880	787	1180
264	396	471	708			42	895	1350	1160	1740	738	1110
		429	645			44	844	1270	1080	1620	694	1040
		393	590			46	798	1200	1010	1520	656	986
		361	542			48	757	1140	947	1420	621	934
		332	499			50	720	1080	892	1340	590	887

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2070	3110	1900	2860	1860	2790
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1680	2530	1550	2320	1510	2270
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
659	989	507	761	591	887
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
294	443	389	585	262	394

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
8.97	28.4	12.5	35.7	8.87	27.2
Area, in.^{2}					
69.1		63.5		62.1	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
17400	444	16700	803	15500	390
r_y, in.					
2.54		3.54		2.51	
r_x/r_y					
6.26		4.58		6.29	

[c] Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued) Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W40

W40×						Shape	W40×					
199[c]		183[c]		167[c]		lb/ft	199		183		167	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1590	2390	1430	2150	1310	1970	0	2170	3260	1930	2900	1730	2600
1550	2330	1360	2040	1240	1860	6	2170	3260	1930	2900	1730	2600
1530	2310	1330	2000	1210	1820	7	2170	3260	1930	2900	1730	2600
1520	2280	1300	1960	1190	1780	8	2170	3260	1930	2900	1730	2600
1500	2250	1270	1910	1160	1740	9	2170	3260	1920	2890	1710	2570
1480	2220	1240	1860	1120	1690	10	2170	3260	1880	2820	1670	2500
1450	2180	1200	1800	1090	1630	11	2170	3260	1830	2760	1620	2440
1430	2140	1160	1740	1050	1580	12	2170	3260	1790	2690	1580	2380
1400	2100	1120	1680	1010	1520	13	2140	3210	1750	2620	1540	2320
1370	2060	1070	1610	968	1450	14	2100	3160	1700	2560	1500	2250
1340	2010	1030	1550	925	1390	15	2060	3100	1660	2490	1460	2190
1310	1970	985	1480	882	1330	16	2030	3050	1610	2420	1420	2130
1280	1920	939	1410	839	1260	17	1990	2990	1570	2360	1370	2060
1240	1870	892	1340	795	1190	18	1950	2930	1520	2290	1330	2000
1210	1810	846	1270	751	1130	19	1910	2880	1480	2230	1290	1940
1170	1760	799	1200	707	1060	20	1880	2820	1440	2160	1250	1880
1100	1650	701	1050	609	916	22	1800	2710	1350	2030	1160	1750
1020	1540	599	900	515	773	24	1730	2600	1260	1890	1080	1620
947	1420	510	767	438	659	26	1650	2490	1170	1750	967	1450
872	1310	440	661	378	568	28	1580	2380	1040	1560	857	1290
794	1190	383	576	329	495	30	1510	2260	929	1400	767	1150
712	1070	337	506	289	435	32	1430	2150	842	1270	693	1040
632	950	298	448	256	385	34	1360	2040	769	1160	631	949
564	847	266	400	229	344	36	1240	1870	707	1060	579	870
506	760	239	359	205	309	38	1140	1710	654	982	535	803
457	686	216	324	185	278	40	1050	1570	608	914	496	746
414	622					42	969	1460	568	854	463	695
377	567					44	901	1350	533	801	433	651
345	519					46	841	1260	502	754	407	612
317	477					48	788	1190	474	713	384	578
292	439					50	742	1110	449	676	364	547

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1760	2650	1600	2400	1480	2220
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1430	2150	1300	1950	1200	1800
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
503	755	507	761	502	753
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
342	514	220	331	190	285

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
12.2	34.3	8.80	25.8	8.48	24.8
Area, in.²					
58.8		53.3		49.3	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
14900	695	13200	331	11600	283
r_y, in.					
3.45		2.49		2.40	
r_x/r_y					
4.64		6.31		6.38	

[c] Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

W40–W36

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

Shape	W40×		W36×				Shape	W40×		W36×			
	149[c]		925[h]		853[h]		lb/ft	149[v]		925[h]		853[h]	
	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
	1140	1710	8140	12200	7510	11300	0	1490	2240	10300	15500	9780	14700
	1070	1610	7980	12000	7360	11100	6	1490	2240	10300	15500	9780	14700
	1050	1570	7920	11900	7310	11000	7	1490	2240	10300	15500	9780	14700
	1020	1540	7850	11800	7240	10900	8	1490	2240	10300	15500	9780	14700
	994	1490	7770	11700	7170	10800	9	1460	2190	10300	15500	9780	14700
	963	1450	7680	11600	7100	10700	10	1420	2130	10300	15500	9780	14700
	930	1400	7590	11400	7010	10500	11	1380	2070	10300	15500	9780	14700
	895	1340	7490	11300	6920	10400	12	1340	2020	10300	15500	9780	14700
	858	1290	7380	11100	6820	10200	13	1300	1960	10300	15500	9780	14700
	821	1230	7270	10900	6710	10100	14	1260	1900	10300	15500	9780	14700
	782	1180	7150	10700	6600	9920	15	1230	1840	10300	15500	9780	14700
	743	1120	7020	10600	6490	9750	16	1190	1780	10300	15400	9740	14600
	703	1060	6890	10400	6360	9570	17	1150	1730	10200	15300	9690	14600
	664	997	6750	10100	6240	9380	18	1110	1670	10200	15300	9640	14500
	624	938	6600	9930	6110	9180	19	1070	1610	10100	15200	9590	14400
	585	879	6460	9700	5970	8980	20	1030	1550	10100	15100	9540	14300
	495	745	6150	9240	5690	8550	22	956	1440	9970	15000	9450	14200
	416	626	5830	8760	5400	8110	24	866	1300	9880	14800	9350	14100
	355	533	5500	8270	5100	7660	26	755	1140	9780	14700	9250	13900
	306	460	5170	7770	4790	7200	28	667	1000	9680	14600	9160	13800
	266	400	4830	7260	4480	6730	30	595	895	9590	14400	9060	13600
	234	352	4500	6760	4170	6270	32	537	806	9490	14300	8960	13500
	207	312	4160	6260	3870	5810	34	487	733	9400	14100	8870	13300
	185	278	3840	5770	3570	5360	36	446	670	9300	14000	8770	13200
	166	250	3520	5300	3280	4930	38	411	617	9200	13800	8670	13000
			3220	4840	3000	4500	40	380	572	9110	13700	8580	12900
			2920	4390	2720	4090	42	354	532	9010	13500	8480	12700
			2660	4000	2480	3730	44	331	497	8920	13400	8380	12600
			2430	3660	2270	3410	46	310	466	8820	13300	8290	12500
			2240	3360	2080	3130	48	292	439	8730	13100	8190	12300
			2060	3100	1920	2890	50	276	415	8630	13000	8090	12200

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1310	1970	8140	12200	7510	11300
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1070	1600	6630	9950	6110	9170
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
432	650	2600	3900	2170	3260
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
155	233	2120	3190	2010	3020

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
8.09	23.6	15.0	107	15.1	100
Area, in.²					
43.8		272		251	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
9800	229	73000	4940	70000	4600
r_y, in.					
2.29		4.26		4.28	
r_x/r_y					
6.55		3.85		3.90	

[c] Shape is slender for compression with F_y = 50 ksi.
[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 50 ksi; therefore, ϕ_v = 0.90 and Ω_v = 1.67.
Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 50 ksi
F_u = 65 ksi

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W36

Shape	W36× 802[h]		W36× 723[h]		W36× 652[h]		W36× 802[h]		W36× 723[h]		W36× 652[h]	
lb/ft	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Design	Available Compressive Strength, kips						Available Flexural Strength, kip-ft					
Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	7070	10600	6380	9580	5750	8640	9130	13700	8160	12300	7260	10900
6	6920	10400	6240	9380	5620	8450	9130	13700	8160	12300	7260	10900
7	6860	10300	6190	9300	5570	8380	9130	13700	8160	12300	7260	10900
8	6800	10200	6130	9220	5520	8300	9130	13700	8160	12300	7260	10900
9	6740	10100	6070	9130	5460	8210	9130	13700	8160	12300	7260	10900
10	6660	10000	6000	9020	5400	8120	9130	13700	8160	12300	7260	10900
11	6580	9890	5930	8910	5330	8010	9130	13700	8160	12300	7260	10900
12	6490	9750	5840	8780	5250	7890	9130	13700	8160	12300	7260	10900
13	6390	9610	5760	8650	5170	7770	9130	13700	8160	12300	7260	10900
14	6290	9460	5660	8510	5080	7640	9130	13700	8160	12300	7260	10900
15	6190	9300	5570	8360	4990	7500	9130	13700	8150	12200	7240	10900
16	6070	9130	5460	8210	4900	7360	9080	13600	8100	12200	7190	10800
17	5960	8950	5350	8050	4800	7210	9030	13600	8050	12100	7140	10700
18	5830	8770	5240	7880	4690	7050	8980	13500	8000	12000	7100	10700
19	5710	8580	5130	7700	4590	6890	8940	13400	7960	12000	7050	10600
20	5580	8380	5010	7520	4470	6730	8890	13400	7910	11900	7000	10500
22	5310	7980	4760	7150	4250	6380	8790	13200	7810	11700	6910	10400
24	5030	7550	4500	6760	4010	6020	8690	13100	7720	11600	6810	10200
26	4740	7120	4240	6370	3760	5660	8600	12900	7620	11500	6720	10100
28	4440	6680	3970	5960	3520	5290	8500	12800	7530	11300	6630	9960
30	4150	6240	3700	5560	3270	4920	8410	12600	7430	11200	6530	9820
32	3860	5800	3430	5160	3030	4550	8310	12500	7340	11000	6440	9680
34	3570	5360	3170	4760	2790	4190	8210	12300	7240	10900	6350	9540
36	3280	4940	2910	4370	2550	3840	8120	12200	7140	10700	6250	9400
38	3010	4520	2660	4000	2330	3500	8020	12100	7050	10600	6160	9260
40	2740	4120	2420	3630	2110	3160	7930	11900	6950	10500	6060	9120
42	2490	3740	2190	3290	1910	2870	7830	11800	6860	10300	5970	8970
44	2270	3410	2000	3000	1740	2620	7730	11600	6760	10200	5880	8830
46	2070	3120	1830	2750	1590	2390	7640	11500	6670	10000	5780	8690
48	1900	2860	1680	2520	1460	2200	7540	11300	6570	9880	5690	8550
50	1750	2640	1550	2320	1350	2030	7450	11200	6480	9740	5600	8410

Properties

W36×	802[h]		723[h]		652[h]	
Available Strength in Tensile Yielding, kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	7070	10600	6380	9590	5750	8640
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	5750	8630	5200	7800	4680	7020
Available Strength in Shear, kips	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	2030	3040	1810	2720	1620	2430
Available Strength in Flexure about Y-Y Axis, kip-ft	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
	1860	2790	1640	2470	1450	2180
Limiting Unbraced Lengths, ft	L_p	L_r	L_p	L_r	L_p	L_r
	14.9	94.5	14.7	85.5	14.5	77.7
Area, in.^{2}	236		213		192	
Moment of Inertia, in.^{4}	I_x	I_y	I_x	I_y	I_x	I_y
	64800	4210	57300	3700	50600	3230
r_y, in.	4.22		4.17		4.10	
r_x/r_y	3.93		3.93		3.95	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W36

F_y = 50 ksi
F_u = 65 ksi

W36×						Shape	W36×					
529[h]		487[h]		441[h]		lb/ft	529[h]		487[h]		441[h]	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
4670	7020	4280	6430	3890	5850	0	5810	8740	5310	7990	4770	7160
4560	6860	4180	6280	3800	5710	6	5810	8740	5310	7990	4770	7160
4520	6800	4140	6230	3760	5660	7	5810	8740	5310	7990	4770	7160
4480	6730	4100	6160	3730	5600	8	5810	8740	5310	7990	4770	7160
4430	6660	4050	6090	3680	5530	9	5810	8740	5310	7990	4770	7160
4370	6570	4000	6020	3630	5460	10	5810	8740	5310	7990	4770	7160
4310	6480	3950	5930	3580	5380	11	5810	8740	5310	7990	4770	7160
4250	6390	3890	5840	3530	5300	12	5810	8740	5310	7990	4770	7160
4180	6280	3820	5740	3470	5210	13	5810	8740	5310	7990	4770	7160
4110	6170	3750	5640	3400	5110	14	5810	8740	5310	7990	4760	7150
4030	6050	3680	5530	3340	5010	15	5770	8680	5270	7920	4710	7080
3950	5930	3610	5420	3270	4910	16	5730	8610	5220	7850	4670	7020
3860	5800	3530	5300	3190	4800	17	5680	8540	5180	7780	4620	6950
3770	5670	3440	5180	3120	4690	18	5630	8470	5130	7710	4580	6880
3680	5540	3360	5050	3040	4570	19	5590	8400	5080	7640	4530	6810
3590	5400	3270	4920	2960	4450	20	5540	8330	5040	7570	4490	6740
3400	5110	3090	4650	2790	4200	22	5450	8190	4950	7430	4400	6610
3200	4810	2910	4370	2620	3940	24	5350	8050	4850	7290	4310	6470
2990	4500	2720	4090	2450	3680	26	5260	7910	4760	7160	4220	6340
2790	4190	2530	3800	2270	3420	28	5170	7770	4670	7020	4130	6200
2580	3880	2340	3520	2100	3160	30	5070	7630	4580	6880	4030	6060
2380	3580	2150	3240	1930	2900	32	4980	7490	4480	6740	3940	5930
2180	3280	1970	2960	1760	2650	34	4890	7350	4390	6600	3850	5790
1990	2990	1790	2700	1600	2410	36	4790	7210	4300	6460	3760	5660
1800	2710	1620	2440	1440	2170	38	4700	7070	4210	6320	3670	5520
1630	2450	1460	2200	1300	1960	40	4610	6930	4120	6190	3580	5380
1480	2220	1330	1990	1180	1780	42	4510	6790	4020	6050	3490	5250
1350	2020	1210	1820	1080	1620	44	4420	6650	3930	5910	3400	5110
1230	1850	1110	1660	985	1480	46	4330	6510	3840	5770	3310	4980
1130	1700	1020	1530	905	1360	48	4240	6370	3750	5630	3220	4840
1040	1570	936	1410	834	1250	50	4140	6230	3650	5490	3130	4700

Properties

Available Strength in Tensile Yielding, kips							Limiting Unbraced Lengths, ft					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$		L_p	L_r	L_p	L_r	L_p	L_r
4670	7020	4280	6440	3890	5850		14.1	64.3	14.0	59.9	13.8	55.5
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips							Area, in.^{2}					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$		156		143		130	
3800	5700	3480	5220	3170	4750		Moment of Inertia, in.^{4}					
Available Strength in Shear, kips							I_x	I_y	I_x	I_y	I_x	I_y
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$		39600	2490	36000	2250	32100	1990
1280	1920	1180	1770	1060	1590		r_y, in.					
Available Strength in Flexure about Y-Y Axis, kip-ft							4.00		3.96		3.92	
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$		r_x/r_y					
1130	1700	1030	1550	918	1380		4.00		3.99		4.01	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W36

W36×						Shape	W36×					
395[h]		361[h]		330		lb/ft	395[h]		361[h]		330	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
3470	5220	3170	4770	2900	4360	0	4270	6410	3870	5810	3520	5290
3390	5090	3090	4650	2830	4250	6	4270	6410	3870	5810	3520	5290
3360	5040	3070	4610	2800	4210	7	4270	6410	3870	5810	3520	5290
3320	4990	3030	4560	2770	4160	8	4270	6410	3870	5810	3520	5290
3280	4930	3000	4500	2740	4110	9	4270	6410	3870	5810	3520	5290
3240	4870	2960	4440	2700	4060	10	4270	6410	3870	5810	3520	5290
3190	4800	2910	4380	2660	4000	11	4270	6410	3870	5810	3520	5290
3140	4720	2870	4310	2620	3930	12	4270	6410	3870	5810	3520	5290
3090	4640	2810	4230	2570	3860	13	4270	6410	3870	5810	3520	5290
3030	4550	2760	4150	2520	3790	14	4250	6390	3850	5790	3500	5260
2970	4460	2700	4070	2470	3710	15	4210	6330	3810	5720	3460	5190
2900	4360	2650	3980	2410	3630	16	4160	6260	3760	5650	3410	5130
2840	4260	2580	3880	2360	3540	17	4120	6190	3720	5590	3370	5070
2770	4160	2520	3790	2300	3460	18	4070	6120	3680	5520	3330	5000
2700	4060	2460	3690	2240	3370	19	4030	6060	3630	5460	3290	4940
2630	3950	2390	3590	2180	3270	20	3980	5990	3590	5390	3240	4880
2480	3720	2250	3380	2050	3080	22	3900	5850	3500	5260	3160	4750
2320	3490	2110	3170	1920	2880	24	3810	5720	3410	5130	3070	4620
2160	3250	1960	2950	1790	2680	26	3720	5590	3330	5000	2990	4490
2010	3020	1820	2730	1650	2480	28	3630	5450	3240	4870	2910	4370
1850	2780	1670	2520	1520	2290	30	3540	5320	3150	4740	2820	4240
1700	2550	1530	2300	1390	2090	32	3450	5180	3060	4600	2740	4110
1550	2330	1400	2100	1270	1900	34	3360	5050	2980	4470	2650	3990
1400	2110	1260	1900	1140	1720	36	3270	4910	2890	4340	2570	3860
1260	1900	1140	1710	1030	1540	38	3180	4780	2800	4210	2480	3730
1140	1710	1030	1540	927	1390	40	3090	4640	2710	4080	2400	3600
1030	1550	930	1400	841	1260	42	3000	4510	2630	3950	2310	3480
942	1420	847	1270	766	1150	44	2910	4380	2540	3820	2230	3350
861	1290	775	1160	701	1050	46	2820	4240	2450	3690	2130	3200
791	1190	712	1070	644	968	48	2730	4110	2360	3550	2020	3030
729	1100	656	986	593	892	50	2640	3970	2250	3380	1910	2880

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3470	5220	3170	4770	2900	4360
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2830	4240	2580	3880	2360	3540
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
937	1410	851	1280	769	1150
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
811	1220	731	1100	661	994

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
13.7	50.9	13.6	48.2	13.5	45.5
Area, in.^{2}					
116		106		96.9	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
28500	1750	25700	1570	23300	1420
r_y, in.					
3.88		3.85		3.83	
r_x/r_y					
4.05		4.05		4.05	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

W36

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W36×						Shape	W36×					
302		282[c]		262[c]		lb/ft	302		282		262	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
2660	4000	2480	3720	2280	3420	0	3190	4800	2970	4460	2740	4130
2600	3900	2420	3630	2220	3340	6	3190	4800	2970	4460	2740	4130
2570	3870	2390	3600	2200	3310	7	3190	4800	2970	4460	2740	4130
2540	3820	2370	3560	2180	3280	8	3190	4800	2970	4460	2740	4130
2510	3780	2340	3520	2160	3250	9	3190	4800	2970	4460	2740	4130
2480	3730	2310	3470	2130	3200	10	3190	4800	2970	4460	2740	4130
2440	3670	2270	3420	2100	3160	11	3190	4800	2970	4460	2740	4130
2400	3610	2230	3360	2070	3110	12	3190	4800	2970	4460	2740	4130
2360	3550	2190	3300	2040	3060	13	3190	4800	2970	4460	2740	4130
2310	3480	2150	3230	2000	3000	14	3170	4770	2950	4430	2720	4080
2270	3400	2110	3170	1950	2940	15	3130	4710	2910	4370	2680	4030
2220	3330	2060	3100	1910	2870	16	3090	4650	2870	4310	2640	3970
2160	3250	2010	3020	1860	2800	17	3050	4590	2830	4250	2600	3910
2110	3170	1960	2950	1820	2730	18	3010	4530	2790	4190	2560	3850
2050	3090	1910	2870	1770	2660	19	2970	4470	2750	4130	2530	3800
2000	3000	1850	2790	1720	2580	20	2930	4400	2710	4070	2490	3740
1880	2820	1740	2620	1610	2420	22	2850	4280	2630	3950	2410	3620
1760	2640	1630	2450	1510	2260	24	2770	4160	2550	3840	2330	3510
1640	2460	1520	2280	1400	2100	26	2690	4040	2470	3720	2260	3390
1510	2270	1400	2110	1290	1940	28	2610	3920	2390	3600	2180	3280
1390	2090	1290	1940	1180	1780	30	2530	3800	2320	3480	2100	3160
1270	1910	1180	1770	1080	1620	32	2440	3670	2240	3360	2030	3050
1160	1740	1070	1610	977	1470	34	2360	3550	2160	3240	1950	2930
1050	1570	964	1450	879	1320	36	2280	3430	2080	3120	1870	2820
939	1410	865	1300	789	1190	38	2200	3310	2000	3010	1800	2700
847	1270	781	1170	712	1070	40	2120	3190	1920	2890	1720	2590
768	1160	708	1060	646	971	42	2040	3070	1840	2770	1610	2420
700	1050	645	970	588	884	44	1950	2930	1730	2600	1510	2270
641	963	591	888	538	809	46	1840	2760	1630	2440	1420	2130
588	884	542	815	494	743	48	1730	2610	1530	2300	1330	2000
542	815	500	751	456	685	50	1640	2470	1450	2180	1260	1900

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2660	4010	2480	3730	2310	3470
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2170	3260	2020	3030	1880	2820
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
705	1060	657	985	620	930
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
601	904	556	836	509	765

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
13.5	43.6	13.4	42.2	13.3	40.6
Area, in.^{2}					
89.0		82.9		77.2	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
21100	1300	19600	1200	17900	1090
r_y, in.					
3.82		3.80		3.76	
r_x/r_y					
4.03		4.05		4.07	

[c] Shape is slender for compression with F_y = 50 ksi.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W36

Shape	W36× 256		W36× 247[c]		W36× 232[c]		W36× 256		W36× 247		W36× 232	
lb/ft	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Design	Available Compressive Strength, kips						Available Flexural Strength, kip-ft					
Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	2250	3390	2110	3170	2010	3030	2590	3900	2570	3860	2340	3510
6	2140	3210	2060	3100	1920	2890	2590	3900	2570	3860	2340	3510
7	2090	3150	2040	3070	1890	2840	2590	3900	2570	3860	2340	3510
8	2050	3080	2020	3040	1850	2770	2590	3900	2570	3860	2340	3510
9	2000	3000	2000	3010	1800	2700	2590	3900	2570	3860	2340	3510
10	1940	2920	1980	2970	1750	2620	2560	3860	2570	3860	2300	3460
11	1880	2830	1950	2930	1690	2540	2520	3780	2570	3860	2260	3390
12	1820	2730	1920	2890	1630	2450	2470	3710	2570	3860	2210	3330
13	1750	2630	1890	2840	1570	2360	2420	3640	2570	3860	2170	3260
14	1680	2530	1860	2790	1510	2270	2380	3570	2540	3820	2120	3190
15	1610	2420	1820	2740	1440	2170	2330	3500	2500	3760	2080	3130
16	1540	2310	1780	2680	1370	2070	2280	3430	2470	3710	2030	3060
17	1460	2200	1750	2620	1310	1960	2240	3360	2430	3650	1990	2990
18	1390	2080	1700	2560	1240	1860	2190	3290	2390	3590	1950	2920
19	1310	1970	1650	2490	1170	1760	2140	3220	2350	3540	1900	2860
20	1240	1860	1610	2410	1100	1660	2100	3150	2320	3480	1860	2790
22	1090	1640	1510	2270	969	1460	2000	3010	2240	3370	1770	2660
24	951	1430	1410	2110	842	1260	1910	2870	2170	3260	1680	2520
26	817	1230	1300	1960	721	1080	1820	2730	2090	3150	1590	2390
28	704	1060	1200	1810	621	934	1720	2590	2020	3040	1500	2260
30	613	922	1100	1660	541	814	1630	2450	1950	2920	1410	2120
32	539	810	1000	1510	476	715	1530	2290	1870	2810	1290	1930
34	477	718	909	1370	421	633	1410	2120	1800	2700	1180	1780
36	426	640	817	1230	376	565	1310	1960	1720	2590	1100	1650
38	382	575	733	1100	337	507	1220	1830	1650	2480	1020	1530
40	345	518	662	994	305	458	1140	1710	1560	2340	954	1430
42	313	470	600	902	276	415	1070	1610	1450	2180	896	1350
44	285	429	547	822			1010	1520	1350	2030	844	1270
46			500	752			960	1440	1270	1910	799	1200
48			459	691			912	1370	1200	1800	758	1140
50			423	636			868	1310	1130	1700	721	1080

Properties

W36× 256		W36× 247		W36× 232	
Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2250	3390	2170	3260	2040	3060
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1840	2750	1770	2650	1660	2490
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
718	1080	587	881	646	968
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
342	514	474	713	304	458

W36× 256		W36× 247		W36× 232	
Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.36	31.5	13.2	39.4	9.25	30.0
Area, in.^{2}					
75.3		72.5		68.0	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
16800	528	16700	1010	15000	468
r_y, in.					
2.65		3.74		2.62	
r_x/r_y					
5.62		4.06		5.65	

[c] Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W36

F_y = 50 ksi
F_u = 65 ksi

Shape W36×	231[c]		210[c]		194[c]		Shape lb/ft	231		210		194	
	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
	1960	2950	1810	2710	1620	2440	0	2400	3610	2080	3120	1910	2880
	1910	2880	1720	2590	1540	2320	6	2400	3610	2080	3120	1910	2880
	1900	2850	1690	2540	1520	2280	7	2400	3610	2080	3120	1910	2880
	1880	2820	1660	2490	1490	2230	8	2400	3610	2080	3120	1910	2880
	1860	2790	1620	2430	1450	2180	9	2400	3610	2080	3120	1910	2880
	1830	2760	1580	2370	1420	2130	10	2400	3610	2040	3070	1870	2820
	1810	2720	1530	2300	1380	2070	11	2400	3610	2000	3000	1830	2760
	1780	2680	1480	2220	1330	2000	12	2400	3610	1960	2940	1790	2700
	1750	2630	1420	2130	1290	1940	13	2400	3610	1910	2880	1750	2630
	1720	2590	1360	2040	1240	1870	14	2370	3560	1870	2810	1710	2570
	1690	2540	1300	1950	1190	1790	15	2330	3510	1830	2750	1670	2510
	1650	2480	1240	1860	1130	1700	16	2300	3460	1790	2680	1630	2450
	1620	2430	1170	1760	1070	1610	17	2260	3400	1740	2620	1590	2390
	1580	2370	1110	1670	1010	1520	18	2230	3350	1700	2560	1550	2330
	1540	2310	1050	1570	956	1440	19	2190	3290	1660	2490	1510	2270
	1500	2250	984	1480	898	1350	20	2160	3240	1620	2430	1470	2210
	1410	2120	862	1300	784	1180	22	2080	3130	1530	2300	1390	2080
	1310	1980	745	1120	676	1020	24	2010	3020	1450	2170	1310	1960
	1220	1830	636	956	577	867	26	1940	2920	1360	2050	1220	1840
	1120	1680	549	825	497	748	28	1870	2810	1280	1920	1130	1700
	1030	1540	478	718	433	651	30	1800	2700	1160	1750	1020	1530
	933	1400	420	631	381	572	32	1730	2590	1060	1590	925	1390
	843	1270	372	559	337	507	34	1650	2490	970	1460	847	1270
	756	1140	332	499	301	452	36	1580	2380	895	1350	780	1170
	679	1020	298	448	270	406	38	1510	2270	831	1250	723	1090
	612	920	269	404	244	366	40	1410	2120	776	1170	674	1010
	555	835	244	366	221	332	42	1310	1960	727	1090	630	948
	506	761					44	1220	1830	684	1030	593	891
	463	696					46	1140	1720	646	971	559	840
	425	639					48	1070	1610	612	920	529	795
	392	589					50	1010	1520	581	874	502	754

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2040	3070	1850	2790	1710	2570
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1660	2500	1510	2260	1390	2090
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
555	832	609	914	558	838
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
439	660	267	401	244	366

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
13.1	38.6	9.11	28.5	9.04	27.6
Area, in.²					
68.2		61.9		57.0	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
15600	940	13200	411	12100	375
r_y, in.					
3.71		2.58		2.56	
r_x/r_y					
4.07		5.66		5.70	

[c] Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W36

W36×						Shape	W36×					
182[c]		170[c]		160[c]		lb/ft	182		170		160	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1500	2250	1370	2060	1270	1910	0	1790	2690	1670	2510	1560	2340
1430	2140	1300	1960	1210	1810	6	1790	2690	1670	2510	1560	2340
1400	2110	1280	1930	1180	1780	7	1790	2690	1670	2510	1560	2340
1370	2060	1250	1890	1160	1740	8	1790	2690	1670	2510	1560	2340
1340	2020	1230	1840	1130	1700	9	1790	2690	1660	2500	1550	2330
1310	1960	1190	1790	1100	1650	10	1750	2630	1630	2450	1510	2280
1270	1910	1160	1740	1070	1610	11	1710	2580	1590	2390	1480	2220
1230	1850	1120	1690	1030	1550	12	1670	2520	1550	2330	1440	2170
1190	1790	1080	1630	998	1500	13	1640	2460	1510	2280	1410	2120
1150	1720	1040	1570	961	1440	14	1600	2400	1480	2220	1370	2060
1100	1660	1000	1510	922	1390	15	1560	2340	1440	2160	1340	2010
1060	1590	960	1440	882	1330	16	1520	2280	1400	2110	1300	1950
1010	1510	917	1380	842	1270	17	1480	2220	1370	2050	1260	1900
950	1430	874	1310	801	1200	18	1440	2160	1330	2000	1230	1850
894	1340	827	1240	760	1140	19	1400	2100	1290	1940	1190	1790
840	1260	775	1170	717	1080	20	1360	2050	1250	1880	1160	1740
733	1100	675	1010	623	936	22	1280	1930	1180	1770	1080	1630
631	949	580	872	532	800	24	1200	1810	1100	1660	1010	1520
538	809	494	743	454	682	26	1130	1690	1030	1550	936	1410
464	697	426	640	391	588	28	1020	1540	920	1380	829	1250
404	608	371	558	341	512	30	921	1380	825	1240	742	1120
355	534	326	490	299	450	32	835	1250	746	1120	670	1010
315	473	289	434	265	399	34	763	1150	681	1020	610	917
281	422	258	387	237	356	36	702	1050	625	940	560	841
252	379	231	348	212	319	38	650	976	578	869	516	776
227	342	209	314	192	288	40	604	908	537	807	479	720
206	310	189	285			42	565	849	501	753	447	671
						44	530	797	470	706	418	629
						46	500	751	442	665	393	591
						48	473	710	418	628	371	557
						50	448	674	396	595	351	528

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1600	2410	1500	2250	1410	2120
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1310	1960	1220	1830	1150	1720
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
526	790	492	738	468	702
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
226	340	209	314	193	290

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.01	27.0	8.94	26.4	8.83	25.8
Area, in.²					
53.6		50.0		47.0	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
11300	347	10500	320	9760	295
r_y, in.					
2.55		2.53		2.50	
r_x/r_y					
5.69		5.73		5.76	

[c] Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W36–W33

W36×				W33×		Shape	W36×				W33×	
150[c]		135[c]		387[h]		lb/ft	150		135[v]		387[h]	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1180	1770	1040	1560	3410	5130	0	1450	2180	1270	1910	3890	5850
1120	1680	982	1480	3320	4990	6	1450	2180	1270	1910	3890	5850
1100	1650	963	1450	3290	4950	7	1450	2180	1270	1910	3890	5850
1070	1610	941	1410	3260	4890	8	1450	2180	1270	1910	3890	5850
1050	1580	916	1380	3210	4830	9	1440	2160	1250	1880	3890	5850
1020	1530	890	1340	3170	4760	10	1410	2110	1220	1830	3890	5850
989	1490	861	1290	3120	4690	11	1370	2060	1190	1780	3890	5850
956	1440	831	1250	3070	4610	12	1340	2010	1160	1740	3890	5850
922	1390	800	1200	3010	4530	13	1300	1960	1120	1690	3890	5850
887	1330	767	1150	2950	4440	14	1270	1910	1090	1640	3870	5810
851	1280	734	1100	2890	4340	15	1230	1850	1060	1590	3830	5750
813	1220	700	1050	2820	4240	16	1200	1800	1030	1550	3790	5700
775	1170	665	1000	2760	4140	17	1160	1750	997	1500	3750	5640
737	1110	630	947	2680	4040	18	1130	1700	965	1450	3710	5580
698	1050	595	895	2610	3930	19	1100	1650	934	1400	3670	5520
660	992	561	842	2540	3810	20	1060	1600	902	1360	3640	5460
575	865	486	730	2380	3580	22	993	1490	838	1260	3560	5350
490	736	410	616	2230	3350	24	924	1390	775	1160	3480	5230
417	627	349	525	2070	3110	26	838	1260	679	1020	3410	5120
360	541	301	452	1910	2870	28	741	1110	598	899	3330	5000
313	471	262	394	1750	2630	30	662	994	533	801	3250	4890
275	414	230	346	1600	2400	32	597	897	479	720	3180	4770
244	367	204	307	1450	2180	34	542	815	435	653	3100	4660
218	327	182	274	1300	1960	36	497	746	397	597	3020	4540
195	294	163	246	1170	1760	38	457	688	365	548	2940	4430
176	265			1060	1590	40	424	637	337	507	2870	4310
				959	1440	42	395	593	313	471	2790	4200
				874	1310	44	369	555	292	439	2710	4080
				799	1200	46	346	521	274	412	2640	3960
				734	1100	48	326	491	258	387	2560	3850
				676	1020	50	309	464	243	365	2480	3730

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1330	1990	1190	1800	3410	5130
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1080	1620	972	1460	2780	4170
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
449	673	384	577	907	1360
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
177	266	149	224	778	1170

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
8.72	25.3	8.41	24.3	13.3	53.3
Area, in.²					
44.3		39.9		114	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
9040	270	7800	225	24300	1620
r_y, in.					
2.47		2.38		3.77	
r_x/r_y					
5.79		5.88		3.87	

[c] Shape is slender for compression with F_y = 50 ksi.
[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 50 ksi; therefore, ϕ_v = 0.90 and Ω_v = 1.67.
Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 50 ksi
F_u = 65 ksi

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W33

Shape	W33× 354[h]		W33× 318		W33× 291			W33× 354[h]		W33× 318		W33× 291	
lb/ft	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Design	Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
	3110	4680	2810	4220	2560	3850	0	3540	5330	3170	4760	2890	4350
	3030	4550	2730	4100	2490	3750	6	3540	5330	3170	4760	2890	4350
	3000	4510	2700	4060	2470	3710	7	3540	5330	3170	4760	2890	4350
	2970	4460	2670	4020	2440	3670	8	3540	5330	3170	4760	2890	4350
	2930	4400	2640	3960	2410	3620	9	3540	5330	3170	4760	2890	4350
	2890	4340	2600	3910	2370	3560	10	3540	5330	3170	4760	2890	4350
	2840	4270	2560	3840	2330	3510	11	3540	5330	3170	4760	2890	4350
	2790	4200	2510	3780	2290	3440	12	3540	5330	3170	4760	2890	4350
	2740	4120	2470	3710	2250	3380	13	3540	5330	3170	4760	2890	4350
	2690	4040	2410	3630	2200	3310	14	3510	5280	3140	4710	2860	4300
	2630	3950	2360	3550	2150	3230	15	3480	5220	3100	4660	2820	4240
	2570	3860	2310	3470	2100	3160	16	3440	5170	3060	4600	2790	4190
	2510	3770	2250	3380	2050	3080	17	3400	5110	3030	4550	2750	4130
	2440	3670	2190	3290	1990	2990	18	3360	5050	2990	4490	2710	4080
	2370	3570	2130	3200	1940	2910	19	3330	5000	2950	4440	2680	4020
	2300	3460	2070	3110	1880	2820	20	3290	4940	2910	4380	2640	3970
	2160	3250	1940	2910	1760	2640	22	3210	4830	2840	4270	2570	3860
	2020	3030	1810	2710	1640	2460	24	3140	4720	2770	4160	2500	3750
	1870	2810	1670	2510	1520	2280	26	3060	4600	2690	4050	2420	3640
	1730	2590	1540	2310	1390	2090	28	2990	4490	2620	3940	2350	3530
	1580	2380	1410	2120	1270	1910	30	2910	4380	2550	3830	2280	3430
	1440	2170	1280	1930	1160	1740	32	2840	4260	2470	3720	2210	3320
	1300	1960	1160	1740	1040	1570	34	2760	4150	2400	3610	2130	3210
	1170	1760	1040	1560	934	1400	36	2690	4040	2330	3500	2060	3100
	1050	1580	932	1400	838	1260	38	2610	3920	2250	3380	1990	2990
	949	1430	841	1260	756	1140	40	2540	3810	2180	3270	1920	2880
	861	1290	763	1150	686	1030	42	2460	3700	2100	3160	1850	2770
	784	1180	695	1050	625	939	44	2390	3590	2030	3050	1770	2660
	718	1080	636	956	572	859	46	2310	3470	1960	2940	1670	2510
	659	991	584	878	525	789	48	2240	3360	1860	2800	1580	2370
	607	913	538	809	484	727	50	2160	3240	1770	2660	1500	2260

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3110	4680	2810	4220	2560	3850
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2540	3800	2280	3430	2090	3130
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
826	1240	732	1100	668	1000
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
704	1060	624	938	564	848

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
13.2	49.8	13.1	46.5	13.0	43.8
Area, in.²					
104		93.7		85.6	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
22000	1460	19500	1290	17700	1160
r_y, in.					
3.74		3.71		3.68	
r_x/r_y					
3.88		3.91		3.91	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W33

$F_y = 50$ ksi
$F_u = 65$ ksi

W33×						Shape		W33×					
263		241[c]		221[c]		lb/ft		263		241		221	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
2320	3480	2130	3200	1920	2890	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	2590	3900	2350	3530	2140	3210
2250	3390	2070	3110	1870	2810		6	2590	3900	2350	3530	2140	3210
2230	3350	2050	3080	1860	2790		7	2590	3900	2350	3530	2140	3210
2200	3310	2020	3040	1840	2760		8	2590	3900	2350	3530	2140	3210
2170	3270	1990	3000	1810	2730		9	2590	3900	2350	3530	2140	3210
2140	3220	1960	2950	1790	2690		10	2590	3900	2350	3530	2140	3210
2110	3170	1930	2900	1760	2650		11	2590	3900	2350	3530	2140	3210
2070	3110	1900	2850	1730	2600		12	2590	3900	2350	3530	2140	3210
2030	3050	1860	2790	1700	2560		13	2590	3900	2340	3510	2130	3200
1990	2990	1820	2730	1670	2500		14	2560	3840	2310	3460	2100	3150
1940	2920	1780	2670	1630	2450		15	2520	3790	2270	3410	2060	3100
1890	2850	1730	2600	1590	2380		16	2490	3740	2240	3360	2030	3050
1850	2780	1690	2540	1540	2320		17	2450	3690	2210	3320	2000	3010
1800	2700	1640	2470	1500	2260		18	2420	3640	2170	3270	1970	2960
1740	2620	1590	2390	1460	2190		19	2390	3580	2140	3220	1940	2910
1690	2540	1540	2320	1410	2120		20	2350	3530	2110	3170	1910	2860
1580	2380	1440	2170	1320	1980		22	2280	3430	2040	3070	1840	2770
1470	2210	1340	2010	1220	1840		24	2210	3330	1970	2970	1780	2670
1360	2050	1240	1860	1130	1690		26	2140	3220	1910	2870	1710	2580
1250	1880	1130	1700	1030	1550		28	2070	3120	1840	2770	1650	2480
1140	1720	1030	1550	937	1410		30	2010	3010	1770	2670	1590	2380
1040	1560	935	1410	847	1270		32	1940	2910	1710	2570	1520	2290
934	1400	841	1260	760	1140		34	1870	2810	1640	2470	1460	2190
835	1260	750	1130	678	1020		36	1800	2700	1580	2370	1400	2100
749	1130	674	1010	608	914		38	1730	2600	1510	2270	1330	2000
676	1020	608	914	549	825		40	1660	2500	1440	2160	1240	1860
614	922	551	829	498	748		42	1580	2380	1340	2010	1150	1730
559	840	502	755	454	682		44	1490	2230	1260	1890	1080	1620
511	769	460	691	415	624		46	1400	2100	1180	1780	1010	1520
470	706	422	634	381	573		48	1320	1990	1110	1680	953	1430
433	651	389	585	351	528		50	1260	1890	1060	1590	901	1350

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2320	3480	2130	3200	1960	2940
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1890	2830	1730	2600	1590	2390
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
600	900	568	852	525	788
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
504	758	454	683	409	615

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
12.9	41.6	12.8	39.7	12.7	38.2
Area, in.^{2}					
77.4		71.1		65.3	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
15900	1040	14200	933	12900	840
r_y, in.					
3.66		3.62		3.59	
r_x/r_y					
3.91		3.90		3.93	

[c] Shape is slender for compression with $F_y = 50$ ksi.

$F_y = 50$ ksi
$F_u = 65$ ksi

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W33

Shape	W33× 201[c]		169[c]		152[c]			W33× 201		169		152	
lb/ft	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Design	Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	1700	2560	1390	2090	1240	1860	0	1930	2900	1570	2360	1390	2100
	1660	2500	1320	1990	1180	1770	6	1930	2900	1570	2360	1390	2100
	1640	2470	1300	1950	1150	1730	7	1930	2900	1570	2360	1390	2100
	1630	2440	1270	1910	1130	1700	8	1930	2900	1570	2360	1390	2100
	1610	2410	1240	1860	1100	1650	9	1930	2900	1560	2350	1390	2080
	1580	2380	1210	1810	1070	1610	10	1930	2900	1530	2300	1350	2030
	1560	2350	1170	1760	1040	1560	11	1930	2900	1500	2250	1320	1990
	1530	2310	1130	1700	1000	1510	12	1930	2900	1460	2200	1290	1940
	1510	2270	1090	1640	967	1450	13	1920	2880	1430	2140	1260	1890
	1480	2220	1050	1580	929	1400	14	1890	2830	1390	2090	1230	1840
	1450	2170	1010	1510	890	1340	15	1860	2790	1360	2040	1190	1790
	1410	2130	962	1450	851	1280	16	1830	2740	1320	1990	1160	1750
	1380	2080	911	1370	810	1220	17	1790	2700	1290	1940	1130	1700
	1350	2020	859	1290	769	1160	18	1760	2650	1260	1890	1100	1650
	1310	1970	807	1210	721	1080	19	1730	2610	1220	1840	1070	1600
	1270	1910	755	1140	674	1010	20	1700	2560	1190	1780	1030	1550
	1180	1780	656	986	583	876	22	1640	2470	1120	1680	969	1460
	1100	1650	561	843	496	746	24	1580	2380	1050	1580	905	1360
	1010	1520	478	718	423	636	26	1520	2290	982	1480	834	1250
	923	1390	412	619	365	548	28	1460	2200	890	1340	741	1110
	838	1260	359	539	318	478	30	1400	2110	803	1210	666	1000
	756	1140	315	474	279	420	32	1340	2020	730	1100	604	908
	676	1020	279	420	247	372	34	1280	1930	670	1010	552	830
	603	907	249	375	221	332	36	1220	1830	618	929	508	763
	541	814	224	336	198	298	38	1140	1710	574	862	470	707
	489	734	202	303	179	269	40	1050	1580	535	805	438	658
	443	666					42	976	1470	502	754	409	615
	404	607					44	911	1370	472	710	384	577
	369	555					46	854	1280	446	670	362	544
	339	510					48	804	1210	422	635	342	514
	313	470					50	759	1140	401	603	324	488

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1770	2660	1480	2230	1340	2020
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1440	2160	1210	1810	1100	1640
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
482	723	453	679	425	638
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
367	551	211	317	184	277

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
12.6	36.7	8.83	26.7	8.72	25.7
Area, in.²					
59.1		49.5		44.9	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
11600	749	9290	310	8160	273
r_y, in.					
3.56		2.50		2.47	
r_x/r_y					
3.93		5.48		5.47	

[c] Shape is slender for compression with $F_y = 50$ ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W33

F_y = 50 ksi
F_u = 65 ksi

Shape	W33×							W33×					
lb/ft	141[c]		130[c]		118[c]			141		130		118[v]	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	1130	1690	1020	1540	905	1360	0	1280	1930	1170	1750	1040	1560
	1070	1600	966	1450	853	1280	6	1280	1930	1170	1750	1040	1560
	1050	1570	947	1420	835	1250	7	1280	1930	1170	1750	1040	1560
	1020	1540	925	1390	815	1220	8	1280	1930	1170	1750	1040	1560
	996	1500	901	1350	792	1190	9	1270	1910	1150	1730	1010	1520
	968	1450	874	1310	768	1150	10	1240	1860	1120	1680	987	1480
	937	1410	846	1270	742	1120	11	1210	1820	1090	1640	960	1440
	905	1360	817	1230	715	1070	12	1180	1770	1060	1600	934	1400
	872	1310	786	1180	686	1030	13	1150	1730	1030	1550	907	1360
	837	1260	754	1130	657	987	14	1120	1680	1000	1510	880	1320
	801	1200	720	1080	626	941	15	1090	1630	976	1470	853	1280
	765	1150	687	1030	596	895	16	1060	1590	947	1420	826	1240
	727	1090	652	981	564	848	17	1030	1540	918	1380	800	1200
	690	1040	618	929	533	801	18	995	1500	889	1340	773	1160
	652	981	583	877	502	754	19	965	1450	860	1290	746	1120
	609	915	549	825	471	708	20	935	1400	832	1250	719	1080
	524	788	470	706	403	605	22	874	1310	774	1160	666	1000
	444	667	396	596	338	509	24	813	1220	716	1080	601	903
	378	569	338	508	288	433	26	732	1100	630	946	524	787
	326	490	291	438	249	374	28	649	976	557	837	462	694
	284	427	254	381	217	326	30	582	875	498	749	412	619
	250	375	223	335	190	286	32	527	792	450	676	371	558
	221	333	198	297	169	253	34	480	722	409	615	337	506
	197	297	176	265	150	226	36	441	663	375	564	308	463
	177	266	158	238	135	203	38	408	613	346	520	283	426
	160	240					40	379	569	321	482	262	394
							42	353	531	299	449	244	366
							44	331	498	280	420	228	342
							46	312	468	263	395	213	321
							48	294	442	248	372	201	302
							50	279	419	234	352	190	285

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1240	1870	1150	1720	1040	1560
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1010	1520	933	1400	845	1270
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
403	604	384	576	325	489
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
167	251	148	223	128	192

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
8.58	25.0	8.44	24.2	8.19	23.4
Area, in.^{2}					
41.5		38.3		34.7	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
7450	246	6710	218	5900	187
r_y, in.					
2.43		2.39		2.32	
r_x/r_y					
5.51		5.52		5.60	

[c] Shape is slender for compression with $F_y = 50$ ksi.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W30

W30× 391[h] P_n/Ω_c ASD	391[h] $\phi_c P_n$ LRFD	357[h] P_n/Ω_c ASD	357[h] $\phi_c P_n$ LRFD	326[h] P_n/Ω_c ASD	326[h] $\phi_c P_n$ LRFD	Shape lb/ft / Design: Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	W30× 391[h] M_{nx}/Ω_b ASD	391[h] $\phi_b M_{nx}$ LRFD	357[h] M_{nx}/Ω_b ASD	357[h] $\phi_b M_{nx}$ LRFD	326[h] M_{nx}/Ω_b ASD	326[h] $\phi_b M_{nx}$ LRFD
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
3440	5170	3140	4720	2870	4320	0	3620	5440	3290	4950	2970	4460
3350	5030	3060	4590	2790	4190	6	3620	5440	3290	4950	2970	4460
3310	4980	3020	4540	2760	4150	7	3620	5440	3290	4950	2970	4460
3280	4920	2990	4490	2730	4100	8	3620	5440	3290	4950	2970	4460
3230	4860	2950	4430	2690	4040	9	3620	5440	3290	4950	2970	4460
3180	4790	2900	4360	2650	3980	10	3620	5440	3290	4950	2970	4460
3130	4710	2860	4290	2600	3910	11	3620	5440	3290	4950	2970	4460
3080	4620	2800	4210	2550	3840	12	3620	5440	3290	4950	2970	4460
3020	4530	2750	4130	2500	3760	13	3620	5440	3290	4940	2960	4450
2950	4440	2690	4040	2450	3680	14	3590	5390	3260	4900	2930	4400
2890	4340	2630	3950	2390	3590	15	3550	5340	3230	4850	2900	4360
2820	4240	2570	3860	2330	3510	16	3520	5290	3190	4800	2870	4310
2750	4130	2500	3760	2270	3410	17	3490	5250	3160	4750	2840	4270
2670	4020	2430	3650	2210	3320	18	3460	5200	3130	4710	2810	4220
2600	3900	2360	3550	2140	3220	19	3430	5150	3100	4660	2780	4180
2520	3790	2290	3440	2070	3120	20	3400	5110	3070	4610	2750	4130
2360	3540	2140	3220	1940	2910	22	3330	5010	3010	4520	2690	4040
2190	3300	1990	2990	1800	2700	24	3270	4920	2940	4430	2630	3950
2030	3050	1840	2760	1660	2490	26	3210	4820	2880	4330	2560	3850
1870	2800	1690	2530	1520	2280	28	3150	4730	2820	4240	2500	3760
1700	2560	1540	2310	1380	2080	30	3080	4640	2760	4140	2440	3670
1550	2320	1390	2090	1250	1880	32	3020	4540	2690	4050	2380	3580
1390	2100	1250	1890	1120	1690	34	2960	4450	2630	3950	2320	3490
1250	1880	1120	1680	1000	1500	36	2900	4350	2570	3860	2260	3400
1120	1680	1010	1510	898	1350	38	2830	4260	2510	3770	2200	3310
1010	1520	908	1360	811	1220	40	2770	4160	2440	3670	2140	3210
917	1380	823	1240	735	1110	42	2710	4070	2380	3580	2080	3120
835	1260	750	1130	670	1010	44	2650	3980	2320	3480	2020	3030
764	1150	686	1030	613	921	46	2580	3880	2260	3390	1960	2940
702	1050	630	947	563	846	48	2520	3790	2190	3300	1900	2850
647	972	581	873	519	780	50	2460	3690	2130	3200	1830	2760

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3440	5180	3140	4730	2870	4320
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2800	4210	2560	3840	2340	3510
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
903	1350	813	1220	739	1110
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
773	1160	696	1050	629	945

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
13.0	58.8	12.9	54.4	12.7	50.6
Area, in.^{2}					
115		105		95.9	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
20700	1550	18700	1390	16800	1240
r_y, in.					
3.67		3.64		3.60	
r_x/r_y					
3.65		3.65		3.67	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W30

F_y = 50 ksi
F_u = 65 ksi

Shape	W30× 292		W30× 261		W30× 235		W30× 292		W30× 261		W30× 235	
lb/ft	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Design	Available Compressive Strength, kips						Available Flexural Strength, kip-ft					
Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	2570	3870	2310	3460	2070	3120	2640	3980	2350	3540	2110	3180
6	2500	3760	2240	3360	2010	3020	2640	3980	2350	3540	2110	3180
7	2470	3720	2210	3320	1990	2990	2640	3980	2350	3540	2110	3180
8	2440	3670	2180	3280	1960	2950	2640	3980	2350	3540	2110	3180
9	2410	3620	2150	3240	1940	2910	2640	3980	2350	3540	2110	3180
10	2370	3560	2120	3180	1900	2860	2640	3980	2350	3540	2110	3180
11	2330	3500	2080	3130	1870	2810	2640	3980	2350	3540	2110	3180
12	2290	3440	2040	3070	1830	2760	2640	3980	2350	3540	2110	3180
13	2240	3370	2000	3000	1800	2700	2630	3960	2340	3510	2100	3150
14	2190	3290	1950	2940	1750	2640	2600	3910	2310	3470	2070	3110
15	2140	3220	1910	2870	1710	2570	2570	3870	2280	3420	2040	3070
16	2090	3140	1860	2790	1670	2510	2540	3820	2250	3380	2010	3020
17	2030	3050	1810	2710	1620	2440	2510	3780	2220	3340	1980	2980
18	1970	2970	1750	2640	1570	2360	2490	3730	2190	3290	1960	2940
19	1910	2880	1700	2550	1520	2290	2460	3690	2160	3250	1930	2900
20	1850	2790	1640	2470	1470	2220	2430	3650	2130	3200	1900	2850
22	1730	2600	1530	2300	1370	2060	2370	3560	2070	3120	1840	2770
24	1600	2410	1420	2130	1270	1910	2310	3470	2020	3030	1790	2680
26	1480	2220	1300	1960	1160	1750	2250	3380	1960	2940	1730	2600
28	1350	2030	1190	1790	1060	1600	2190	3290	1900	2850	1670	2510
30	1230	1850	1080	1620	962	1450	2130	3200	1840	2760	1620	2430
32	1110	1670	970	1460	865	1300	2070	3110	1780	2680	1560	2340
34	995	1500	866	1300	771	1160	2010	3020	1720	2590	1500	2260
36	888	1330	773	1160	688	1030	1950	2930	1660	2500	1450	2170
38	797	1200	694	1040	617	928	1890	2840	1600	2410	1390	2090
40	719	1080	626	941	557	837	1830	2750	1550	2320	1330	2000
42	652	980	568	853	505	759	1770	2660	1490	2240	1260	1900
44	594	893	517	778	460	692	1710	2570	1420	2130	1190	1790
46	544	817	473	711	421	633	1650	2480	1340	2020	1120	1690
48	499	751	435	653	387	581	1580	2370	1270	1910	1060	1600
50	460	692	401	602	356	536	1500	2260	1210	1820	1010	1520

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2570	3870	2310	3470	2070	3120
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2100	3140	1880	2820	1690	2540
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
653	979	588	882	520	779
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
556	836	489	735	437	656

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
12.6	46.9	12.5	43.4	12.4	41.0
Area, in.^{2}					
86.0		77.0		69.3	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
14900	1100	13100	959	11700	855
r_y, in.					
3.58		3.53		3.51	
r_x/r_y					
3.69		3.71		3.70	

F_y = 50 ksi
F_u = 65 ksi

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W30

Shape	W30×							W30×					
lb/ft	211		191[c]		173[c]			211		191		173	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
	1870	2800	1660	2500	1480	2220	0	1870	2820	1680	2530	1510	2280
	1810	2720	1610	2430	1440	2160	6	1870	2820	1680	2530	1510	2280
	1790	2690	1600	2400	1420	2140	7	1870	2820	1680	2530	1510	2280
	1760	2650	1580	2370	1400	2110	8	1870	2820	1680	2530	1510	2280
	1740	2610	1560	2340	1390	2080	9	1870	2820	1680	2530	1510	2280
	1710	2570	1540	2310	1370	2050	10	1870	2820	1680	2530	1510	2280
	1680	2530	1510	2270	1340	2020	11	1870	2820	1680	2530	1510	2280
	1650	2480	1480	2220	1320	1980	12	1870	2820	1680	2530	1510	2280
	1610	2420	1450	2180	1290	1940	13	1860	2790	1660	2500	1490	2240
	1570	2370	1410	2120	1270	1900	14	1830	2750	1640	2460	1470	2210
	1540	2310	1380	2070	1240	1860	15	1800	2710	1610	2420	1440	2170
	1490	2250	1340	2020	1210	1810	16	1770	2670	1590	2380	1420	2130
	1450	2180	1300	1960	1170	1770	17	1750	2630	1560	2350	1390	2100
	1410	2120	1260	1900	1140	1710	18	1720	2590	1530	2310	1370	2060
	1370	2050	1220	1840	1100	1660	19	1690	2550	1510	2270	1350	2020
	1320	1980	1180	1780	1060	1600	20	1670	2510	1480	2230	1320	1990
	1230	1850	1100	1650	986	1480	22	1610	2420	1430	2150	1270	1910
	1130	1700	1010	1520	907	1360	24	1560	2340	1380	2070	1220	1840
	1040	1560	927	1390	829	1250	26	1500	2260	1330	2000	1180	1770
	947	1420	843	1270	752	1130	28	1450	2180	1280	1920	1130	1690
	857	1290	761	1140	678	1020	30	1400	2100	1220	1840	1080	1620
	770	1160	682	1030	606	911	32	1340	2020	1170	1760	1030	1550
	685	1030	606	911	538	808	34	1290	1940	1120	1690	981	1470
	611	919	541	813	479	721	36	1230	1860	1070	1610	922	1390
	549	824	485	730	430	647	38	1180	1770	1000	1500	850	1280
	495	744	438	659	388	584	40	1110	1670	928	1400	787	1180
	449	675	397	597	352	529	42	1040	1560	866	1300	733	1100
	409	615	362	544	321	482	44	973	1460	812	1220	685	1030
	374	563	331	498	294	441	46	917	1380	763	1150	643	967
	344	517	304	457	270	405	48	866	1300	720	1080	606	911
	317	476	280	421	249	374	50	822	1230	682	1030	573	861

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1870	2800	1680	2520	1520	2290
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1520	2280	1370	2050	1240	1860
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
479	718	436	654	398	597
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
387	581	344	518	307	461

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
12.3	38.7	12.2	36.8	12.1	35.5
Area, in.²					
62.3		56.1		50.9	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
10300	757	9200	673	8230	598
r_y, in.					
3.49		3.46		3.42	
r_x/r_y					
3.70		3.70		3.71	

[c] Shape is slender for compression with F_y = 50 ksi.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W30

$F_y = 50$ ksi
$F_u = 65$ ksi

W30×						Shape	W30×					
148[c]		132[c]		124[c]		lb/ft	148		132		124	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1250	1880	1090	1640	1010	1520	0	1250	1880	1090	1640	1020	1530
1180	1770	1030	1540	949	1430	6	1250	1880	1090	1640	1020	1530
1150	1730	1000	1510	927	1390	7	1250	1880	1090	1640	1020	1530
1120	1680	977	1470	902	1360	8	1250	1880	1090	1640	1010	1530
1090	1630	948	1420	875	1320	9	1220	1830	1060	1600	989	1490
1050	1580	916	1380	846	1270	10	1190	1790	1040	1560	963	1450
1010	1520	883	1330	815	1220	11	1160	1750	1010	1520	937	1410
975	1460	848	1270	782	1170	12	1130	1700	981	1470	911	1370
927	1390	811	1220	747	1120	13	1100	1660	954	1430	885	1330
878	1320	773	1160	712	1070	14	1080	1620	927	1390	859	1290
828	1240	728	1090	676	1020	15	1050	1570	900	1350	833	1250
777	1170	682	1030	636	955	16	1020	1530	874	1310	808	1210
727	1090	637	957	593	891	17	990	1490	847	1270	782	1170
677	1020	592	890	550	827	18	961	1440	820	1230	756	1140
628	944	548	824	509	765	19	932	1400	793	1190	730	1100
581	873	506	760	469	704	20	903	1360	766	1150	704	1060
489	735	424	637	391	588	22	845	1270	712	1070	652	980
411	617	356	535	329	494	24	788	1180	654	983	588	884
350	526	303	456	280	421	26	714	1070	578	868	518	779
302	454	262	393	242	363	28	641	963	516	776	462	695
263	395	228	342	211	316	30	581	874	466	701	417	626
231	347	200	301	185	278	32	531	799	425	638	379	569
205	308	177	267	164	246	34	489	736	390	586	347	522
183	274	158	238	146	220	36	454	682	360	541	320	481
164	246					38	423	635	335	503	297	446
						40	396	595	312	469	277	416
						42	372	559	293	440	259	390
						44	351	528	276	415	244	367
						46	332	500	261	392	230	346
						48	316	474	247	371	218	328
						50	300	452	235	353	207	311

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1310	1960	1160	1750	1090	1640
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1060	1590	946	1420	891	1340
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
399	599	373	559	353	530
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
170	255	146	219	135	203

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
8.05	24.9	7.95	23.8	7.88	23.2
Area, in.^{2}					
43.6		38.8		36.5	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
6680	227	5770	196	5360	181
r_y, in.					
2.28		2.25		2.23	
r_x/r_y					
5.44		5.42		5.43	

[c] Shape is slender for compression with $F_y = 50$ ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 50 ksi
F_u = 65 ksi

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W30

Design: Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	W30×116[c] P_n/Ω_c ASD	W30×116[c] ϕ_cP_n LRFD	W30×108[c] P_n/Ω_c ASD	W30×108[c] ϕ_cP_n LRFD	W30×99[c] P_n/Ω_c ASD	W30×99[c] ϕ_cP_n LRFD	W30×116 M_{nx}/Ω_b ASD	W30×116 ϕ_bM_{nx} LRFD	W30×108 M_{nx}/Ω_b ASD	W30×108 ϕ_bM_{nx} LRFD	W30×99 M_{nx}/Ω_b ASD	W30×99 ϕ_bM_{nx} LRFD
	Available Compressive Strength, kips						Available Flexural Strength, kip-ft					
0	936	1410	854	1280	766	1150	943	1420	863	1300	778	1170
6	876	1320	798	1200	713	1070	943	1420	863	1300	778	1170
7	855	1290	778	1170	695	1040	943	1420	863	1300	778	1170
8	832	1250	756	1140	675	1010	937	1410	854	1280	766	1150
9	806	1210	732	1100	652	981	912	1370	830	1250	743	1120
10	778	1170	706	1060	628	944	887	1330	807	1210	721	1080
11	748	1120	678	1020	603	906	862	1300	783	1180	699	1050
12	717	1080	649	976	576	865	838	1260	759	1140	677	1020
13	685	1030	619	930	548	824	813	1220	736	1110	655	984
14	651	979	588	884	519	781	788	1180	712	1070	632	951
15	617	928	556	836	490	737	763	1150	689	1040	610	917
16	583	876	524	788	461	693	739	1110	665	1000	588	884
17	543	816	491	739	432	649	714	1070	642	964	566	851
18	503	756	454	682	401	602	689	1040	618	929	544	817
19	464	697	417	627	367	551	665	999	594	893	522	784
20	426	640	382	574	334	502	640	962	571	858	499	751
22	354	532	316	475	276	415	590	887	524	787	445	669
24	297	447	266	399	232	348	521	784	453	681	384	577
26	253	381	226	340	197	297	458	689	397	597	336	504
28	218	328	195	293	170	256	408	613	353	530	297	447
30	190	286	170	255	148	223	367	552	317	476	266	400
32	167	251	149	224	130	196	333	501	287	431	241	362
34	148	223	132	199	115	174	305	458	262	393	219	329
36	132	199					281	422	241	362	201	302
38							260	391	222	334	186	279
40							242	364	207	311	172	259
42							226	340	193	290	161	241
44							213	320	181	272	150	226
46							201	301	171	257	142	213
48							190	285	161	242	134	201
50							180	271	153	230	126	190

Properties

	W30×116 ASD	W30×116 LRFD	W30×108 ASD	W30×108 LRFD	W30×99 ASD	W30×99 LRFD
Available Strength in Tensile Yielding, kips	P_n/Ω_t	ϕ_tP_n	P_n/Ω_t	ϕ_tP_n	P_n/Ω_t	ϕ_tP_n
	1020	1540	949	1430	868	1310
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips	P_n/Ω_t	ϕ_tP_n	P_n/Ω_t	ϕ_tP_n	P_n/Ω_t	ϕ_tP_n
	835	1250	774	1160	709	1060
Available Strength in Shear, kips	V_n/Ω_v	ϕ_vV_n	V_n/Ω_v	ϕ_vV_n	V_n/Ω_v	ϕ_vV_n
	339	509	325	487	309	463
Available Strength in Flexure about Y-Y Axis, kip-ft	M_{ny}/Ω_b	ϕ_bM_{ny}	M_{ny}/Ω_b	ϕ_bM_{ny}	M_{ny}/Ω_b	ϕ_bM_{ny}
	123	185	110	165	96.3	145

	W30×116		W30×108		W30×99	
Limiting Unbraced Lengths, ft	L_p	L_r	L_p	L_r	L_p	L_r
	7.74	22.6	7.59	22.1	7.42	21.3
Area, in.^{2}	34.2		31.7		29.0	
Moment of Inertia, in.^{4}	I_x	I_y	I_x	I_y	I_x	I_y
	4930	164	4470	146	3990	128
r_y, in.	2.19		2.15		2.10	
r_x/r_y	5.48		5.53		5.57	

[c] Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W30–W27

F_y = 50 ksi
F_u = 65 ksi

Shape	W30×		W27×					W30×		W27×			
lb/ft	90[c]		539[h]		368[h]			90[v]		539[h]		368[h]	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
	672	1010	4760	7150	3260	4900	0	706	1060	4720	7090	3090	4650
	625	939	4630	6950	3160	4750	6	706	1060	4720	7090	3090	4650
	609	915	4580	6880	3130	4700	7	706	1060	4720	7090	3090	4650
	591	888	4530	6800	3090	4640	8	693	1040	4720	7090	3090	4650
	571	858	4470	6710	3040	4570	9	673	1010	4720	7090	3090	4650
	549	826	4400	6610	2990	4500	10	652	980	4720	7090	3090	4650
	527	792	4330	6500	2940	4420	11	632	949	4720	7090	3090	4650
	503	756	4250	6390	2880	4330	12	611	918	4720	7090	3090	4650
	479	719	4170	6260	2820	4230	13	590	887	4710	7080	3080	4620
	453	681	4080	6130	2750	4140	14	570	857	4690	7040	3050	4590
	428	643	3980	5990	2680	4030	15	549	826	4660	7000	3030	4550
	402	604	3890	5840	2610	3930	16	529	795	4630	6970	3000	4510
	376	566	3790	5690	2540	3820	17	508	764	4610	6930	2980	4470
	351	527	3690	5540	2460	3700	18	488	733	4580	6890	2950	4440
	326	490	3580	5380	2380	3580	19	467	702	4560	6850	2930	4400
	300	451	3470	5220	2300	3460	20	446	671	4530	6810	2900	4360
	248	372	3250	4880	2140	3220	22	390	587	4480	6730	2850	4290
	208	313	3020	4540	1980	2970	24	335	504	4430	6650	2800	4210
	177	267	2790	4190	1810	2730	26	293	440	4370	6570	2750	4130
	153	230	2560	3850	1650	2480	28	259	389	4320	6500	2700	4060
	133	200	2340	3510	1490	2240	30	231	347	4270	6420	2650	3980
	117	176	2120	3190	1340	2010	32	208	313	4220	6340	2600	3910
	104	156	1910	2870	1190	1790	34	189	284	4160	6260	2550	3830
			1710	2560	1060	1600	36	173	260	4110	6180	2500	3760
			1530	2300	954	1430	38	159	240	4060	6100	2450	3680
			1380	2080	861	1290	40	148	222	4010	6020	2400	3610
			1250	1880	781	1170	42	137	206	3960	5950	2350	3530
			1140	1720	712	1070	44	128	193	3900	5870	2300	3460
			1040	1570	651	979	46	121	181	3850	5790	2250	3380
			960	1440	598	899	48	114	171	3800	5710	2200	3310
			884	1330	551	828	50	107	161	3750	5630	2150	3230

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
787	1180	4760	7160	3260	4910
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
640	960	3870	5800	2660	3990
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
249	374	1280	1920	839	1260
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
86.6	130	1090	1640	696	1050

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
7.38	20.9	12.9	88.5	12.3	62.0
Area, in.^{2}					
26.3		159		109	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
3610	115	25600	2110	16200	1310
r_y, in.					
2.09		3.65		3.48	
r_x/r_y					
5.60		3.48		3.51	

[c] Shape is slender for compression with F_y = 50 ksi.
[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 50 ksi; therefore, ϕ_v = 0.90 and Ω_v = 1.67.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued) Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W27

W27×						Shape	W27×					
336[h]		307[h]		281		lb/ft	336[h]		307[h]		281	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
2970	4460	2700	4060	2490	3740	0	2820	4240	2570	3860	2340	3510
2880	4320	2610	3930	2410	3620	6	2820	4240	2570	3860	2340	3510
2840	4270	2580	3880	2380	3580	7	2820	4240	2570	3860	2340	3510
2810	4220	2550	3830	2350	3530	8	2820	4240	2570	3860	2340	3510
2760	4160	2510	3770	2310	3470	9	2820	4240	2570	3860	2340	3510
2720	4090	2470	3710	2270	3410	10	2820	4240	2570	3860	2340	3510
2670	4010	2420	3640	2230	3350	11	2820	4240	2570	3860	2340	3510
2610	3930	2370	3560	2180	3280	12	2820	4240	2570	3860	2330	3510
2560	3840	2320	3480	2130	3200	13	2800	4210	2550	3830	2310	3470
2500	3750	2260	3400	2080	3120	14	2770	4170	2520	3790	2290	3440
2430	3660	2200	3310	2020	3040	15	2750	4130	2500	3750	2260	3400
2370	3560	2140	3220	1970	2960	16	2720	4090	2470	3710	2240	3360
2300	3460	2080	3120	1910	2870	17	2700	4060	2450	3670	2210	3320
2230	3350	2010	3030	1850	2780	18	2670	4020	2420	3640	2190	3290
2160	3240	1950	2930	1790	2690	19	2650	3980	2390	3600	2160	3250
2080	3130	1880	2830	1720	2590	20	2620	3940	2370	3560	2140	3210
1940	2910	1740	2620	1600	2400	22	2570	3870	2320	3490	2090	3140
1780	2680	1600	2410	1470	2210	24	2520	3790	2270	3410	2040	3070
1630	2450	1460	2200	1340	2010	26	2470	3720	2220	3330	1990	2990
1480	2230	1330	2000	1210	1820	28	2420	3640	2170	3260	1940	2920
1340	2010	1200	1800	1090	1640	30	2370	3570	2120	3180	1890	2840
1200	1800	1070	1610	974	1460	32	2320	3490	2070	3110	1840	2770
1070	1600	947	1420	862	1300	34	2270	3420	2020	3030	1790	2700
951	1430	845	1270	769	1160	36	2220	3340	1970	2960	1740	2620
853	1280	758	1140	690	1040	38	2170	3270	1920	2880	1690	2550
770	1160	684	1030	623	936	40	2120	3190	1870	2800	1650	2470
699	1050	621	933	565	849	42	2070	3120	1820	2730	1600	2400
637	957	565	850	515	774	44	2020	3040	1760	2650	1550	2330
582	875	517	778	471	708	46	1970	2960	1710	2580	1500	2250
535	804	475	714	433	650	48	1920	2890	1660	2500	1450	2180
493	741	438	658	399	599	50	1870	2810	1610	2430	1390	2090

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2970	4460	2700	4060	2490	3740
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2420	3630	2200	3300	2020	3040
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
756	1130	687	1030	621	932
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
629	945	566	851	514	773

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
12.2	57.0	12.0	52.6	12.0	49.1
Area, in.²					
99.2		90.2		83.1	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
14600	1180	13100	1050	11900	953
r_y, in.					
3.45		3.41		3.39	
r_x/r_y					
3.51		3.52		3.54	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W27

F_y = 50 ksi
F_u = 65 ksi

W27× 258 P_n/Ω_c ASD	258 $\phi_c P_n$ LRFD	235 P_n/Ω_c ASD	235 $\phi_c P_n$ LRFD	217 P_n/Ω_c ASD	217 $\phi_c P_n$ LRFD	Shape lb/ft — Design: Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	W27× 258 M_{nx}/Ω_b ASD	258 $\phi_b M_{nx}$ LRFD	235 M_{nx}/Ω_b ASD	235 $\phi_b M_{nx}$ LRFD	217 M_{nx}/Ω_b ASD	217 $\phi_b M_{nx}$ LRFD
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
2280	3420	2080	3120	1910	2880	0	2130	3200	1930	2900	1770	2670
2200	3310	2010	3020	1850	2780	6	2130	3200	1930	2900	1770	2670
2180	3270	1980	2980	1830	2740	7	2130	3200	1930	2900	1770	2670
2150	3230	1960	2940	1800	2700	8	2130	3200	1930	2900	1770	2670
2110	3180	1920	2890	1770	2660	9	2130	3200	1930	2900	1770	2670
2080	3120	1890	2840	1740	2610	10	2130	3200	1930	2900	1770	2670
2040	3060	1850	2780	1700	2560	11	2130	3200	1930	2900	1770	2670
1990	2990	1810	2720	1670	2510	12	2120	3190	1920	2890	1770	2660
1950	2930	1770	2660	1630	2450	13	2100	3150	1900	2850	1740	2620
1900	2850	1730	2590	1590	2380	14	2070	3120	1870	2810	1720	2590
1850	2780	1680	2520	1540	2320	15	2050	3080	1850	2780	1700	2550
1790	2700	1630	2450	1500	2250	16	2030	3040	1820	2740	1670	2520
1740	2620	1580	2370	1450	2180	17	2000	3010	1800	2710	1650	2480
1680	2530	1530	2300	1400	2110	18	1980	2970	1780	2670	1630	2450
1630	2450	1470	2220	1360	2040	19	1950	2940	1750	2640	1600	2410
1570	2360	1420	2140	1310	1960	20	1930	2900	1730	2600	1580	2380
1450	2180	1310	1970	1200	1810	22	1880	2830	1680	2530	1530	2310
1330	2000	1200	1810	1100	1660	24	1830	2750	1630	2460	1490	2240
1210	1820	1090	1640	1000	1510	26	1780	2680	1590	2380	1440	2170
1100	1650	987	1480	905	1360	28	1730	2610	1540	2310	1390	2100
984	1480	884	1330	810	1220	30	1690	2530	1490	2240	1350	2030
876	1320	784	1180	718	1080	32	1640	2460	1440	2170	1300	1960
776	1170	695	1040	636	956	34	1590	2390	1400	2100	1250	1890
692	1040	620	932	567	853	36	1540	2320	1350	2030	1210	1820
621	933	556	836	509	765	38	1490	2240	1300	1950	1160	1750
560	842	502	755	459	691	40	1440	2170	1250	1880	1110	1680
508	764	455	684	417	626	42	1400	2100	1200	1810	1060	1590
463	696	415	624	380	571	44	1350	2020	1150	1720	997	1500
424	637	380	571	347	522	46	1300	1950	1090	1630	944	1420
389	585	349	524	319	480	48	1230	1850	1030	1550	896	1350
359	539	321	483	294	442	50	1180	1770	984	1480	853	1280

Properties

	258	258	235	235	217	217
Available Strength in Tensile Yielding, kips						
	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	2280	3420	2080	3120	1910	2880
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips						
	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	1860	2780	1690	2540	1560	2340
Available Strength in Shear, kips						
	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	568	853	522	784	471	707
Available Strength in Flexure about Y-Y Axis, kip-ft						
	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
	467	701	419	630	384	578

	258	258	235	235	217	217
Limiting Unbraced Lengths, ft						
	L_p	L_r	L_p	L_r	L_p	L_r
	11.9	45.9	11.8	42.9	11.7	40.8
Area, in.^{2}	76.1		69.4		63.9	
Moment of Inertia, in.^{4}						
	I_x	I_y	I_x	I_y	I_x	I_y
	10800	859	9700	769	8910	704
r_y, in.	3.36		3.33		3.32	
r_x/r_y	3.54		3.54		3.55	

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

$F_y = 50$ ksi
$F_u = 65$ ksi

W27

W27× 194 P_n/Ω_c ASD	194 $\phi_c P_n$ LRFD	178 P_n/Ω_c ASD	178 $\phi_c P_n$ LRFD	161[c] P_n/Ω_c ASD	161[c] $\phi_c P_n$ LRFD	Design: Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	W27× 194 M_{nx}/Ω_b ASD	194 $\phi_b M_{nx}$ LRFD	178 M_{nx}/Ω_b ASD	178 $\phi_b M_{nx}$ LRFD	161 M_{nx}/Ω_b ASD	161 $\phi_b M_{nx}$ LRFD
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
1710	2570	1570	2360	1420	2140	0	1570	2370	1420	2140	1280	1930
1650	2480	1520	2280	1370	2070	6	1570	2370	1420	2140	1280	1930
1630	2450	1500	2250	1360	2040	7	1570	2370	1420	2140	1280	1930
1610	2410	1470	2220	1340	2010	8	1570	2370	1420	2140	1280	1930
1580	2370	1450	2180	1310	1970	9	1570	2370	1420	2140	1280	1930
1550	2330	1420	2140	1290	1940	10	1570	2370	1420	2140	1280	1930
1520	2280	1390	2090	1260	1900	11	1570	2370	1420	2140	1280	1930
1490	2230	1360	2050	1230	1850	12	1570	2350	1410	2120	1270	1910
1450	2180	1330	2000	1200	1810	13	1540	2320	1390	2090	1250	1880
1410	2120	1290	1940	1170	1760	14	1520	2290	1370	2060	1230	1850
1370	2060	1260	1890	1140	1710	15	1500	2250	1350	2020	1210	1820
1330	2000	1220	1830	1100	1650	16	1480	2220	1320	1990	1190	1790
1290	1940	1180	1770	1060	1600	17	1450	2180	1300	1960	1170	1760
1250	1870	1140	1710	1030	1540	18	1430	2150	1280	1920	1150	1720
1200	1810	1100	1650	990	1490	19	1410	2120	1260	1890	1130	1690
1160	1740	1050	1590	952	1430	20	1390	2080	1240	1860	1110	1660
1070	1600	970	1460	874	1310	22	1340	2020	1190	1790	1060	1600
976	1470	885	1330	797	1200	24	1300	1950	1150	1730	1020	1540
886	1330	801	1200	720	1080	26	1250	1880	1110	1660	981	1470
797	1200	719	1080	646	971	28	1210	1810	1060	1600	939	1410
712	1070	641	963	575	864	30	1160	1740	1020	1530	898	1350
630	947	565	850	506	761	32	1120	1680	977	1470	856	1290
558	839	501	753	448	674	34	1070	1610	933	1400	814	1220
498	748	447	671	400	601	36	1030	1540	890	1340	757	1140
447	671	401	602	359	540	38	981	1470	829	1250	701	1050
403	606	362	544	324	487	40	918	1380	773	1160	652	980
366	550	328	493	294	442	42	861	1290	724	1090	610	916
333	501	299	449	268	402	44	811	1220	681	1020	572	860
305	458	274	411	245	368	46	767	1150	643	966	539	811
280	421	251	378	225	338	48	727	1090	608	914	510	766
258	388	232	348	207	312	50	692	1040	578	868	484	727

Properties

Available Strength in Tensile Yielding, kips

194 P_n/Ω_t	194 $\phi_t P_n$	178 P_n/Ω_t	178 $\phi_t P_n$	161 P_n/Ω_t	161 $\phi_t P_n$
1710	2570	1570	2360	1430	2140

Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips

194 P_n/Ω_t	194 $\phi_t P_n$	178 P_n/Ω_t	178 $\phi_t P_n$	161 P_n/Ω_t	161 $\phi_t P_n$
1390	2090	1280	1920	1160	1740

Available Strength in Shear, kips

194 V_n/Ω_v	194 $\phi_v V_n$	178 V_n/Ω_v	178 $\phi_v V_n$	161 V_n/Ω_v	161 $\phi_v V_n$
422	632	403	605	364	546

Available Strength in Flexure about Y-Y Axis, kip-ft

194 M_{ny}/Ω_b	194 $\phi_b M_{ny}$	178 M_{ny}/Ω_b	178 $\phi_b M_{ny}$	161 M_{ny}/Ω_b	161 $\phi_b M_{ny}$
339	510	304	458	272	409

Limiting Unbraced Lengths, ft

194 L_p	194 L_r	178 L_p	178 L_r	161 L_p	161 L_r
11.6	38.2	11.5	36.4	11.4	34.7

Property	194	178	161
Area, in.^{2}	57.1	52.5	47.6
r_y, in.	3.29	3.25	3.23
r_x/r_y	3.56	3.57	3.56

Moment of Inertia, in.^{4}

194 I_x	194 I_y	178 I_x	178 I_y	161 I_x	161 I_y
7860	619	7020	555	6310	497

[c] Shape is slender for compression with $F_y = 50$ ksi.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W27

F_y = 50 ksi
F_u = 65 ksi

W27×						Shape	W27×					
146[c]		129[c]		114[c]		lb/ft	146		129		114	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1270	1900	1100	1650	958	1440	0	1160	1740	986	1480	856	1290
1220	1840	1030	1550	895	1350	6	1160	1740	986	1480	856	1290
1210	1820	1010	1510	873	1310	7	1160	1740	986	1480	856	1290
1190	1790	977	1470	849	1280	8	1160	1740	981	1470	849	1280
1180	1770	947	1420	822	1240	9	1160	1740	958	1440	828	1240
1160	1740	912	1370	793	1190	10	1160	1740	934	1400	806	1210
1130	1700	872	1310	762	1140	11	1160	1740	911	1370	784	1180
1110	1670	830	1250	729	1100	12	1140	1720	888	1330	763	1150
1090	1630	786	1180	692	1040	13	1120	1690	864	1300	741	1110
1060	1590	742	1110	652	979	14	1100	1660	841	1260	720	1080
1030	1540	697	1050	611	918	15	1080	1630	818	1230	698	1050
994	1490	652	980	571	858	16	1070	1600	795	1190	676	1020
961	1440	607	912	530	797	17	1050	1570	771	1160	655	984
927	1390	563	846	491	738	18	1030	1540	748	1120	633	951
892	1340	520	781	452	680	19	1010	1510	725	1090	611	919
857	1290	478	718	415	623	20	986	1480	701	1050	590	886
786	1180	398	598	344	518	22	947	1420	655	984	547	821
715	1080	335	503	289	435	24	907	1360	608	914	493	740
645	970	285	428	247	371	26	868	1300	544	817	436	655
578	868	246	369	213	320	28	828	1250	489	736	391	587
513	771	214	322	185	278	30	789	1190	445	669	354	532
451	678	188	283	163	245	32	750	1130	408	613	323	485
399	600	167	251	144	217	34	701	1050	376	566	297	446
356	535	149	223	129	193	36	644	967	349	525	275	413
320	481					38	594	893	326	490	256	384
289	434					40	552	830	306	460	239	359
262	393					42	515	774	288	433	225	338
239	358					44	483	725	272	409	212	318
218	328					46	454	682	258	388	200	301
200	301					48	429	644	245	368	190	286
185	278					50	406	610	234	351	181	272

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1290	1940	1130	1700	1010	1510
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1050	1580	923	1380	819	1230
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
332	497	337	505	311	467
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
244	366	144	216	123	185

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
11.3	33.3	7.81	24.2	7.70	23.1
Area, in.²					
43.2		37.8		33.6	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
5660	443	4760	184	4080	159
r_y, in.					
3.20		2.21		2.18	
r_x/r_y					
3.59		5.07		5.05	

[c] Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W27

W27×						Shape	W27×					
102[c]		94[c]		84[c]		lb/ft	102		94		84	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
830	1250	750	1130	655	985	0	761	1140	694	1040	609	915
774	1160	698	1050	608	914	6	761	1140	694	1040	609	915
754	1130	680	1020	592	890	7	761	1140	694	1040	609	915
733	1100	660	993	574	862	8	753	1130	684	1030	597	897
709	1070	638	960	554	833	9	733	1100	665	999	579	870
683	1030	615	924	533	800	10	713	1070	646	970	561	844
656	986	590	886	510	766	11	693	1040	627	942	544	817
627	943	563	847	486	731	12	672	1010	608	913	526	791
597	898	536	805	462	694	13	652	980	588	884	509	764
567	852	508	763	437	656	14	632	950	569	856	491	738
536	805	479	721	411	618	15	612	920	550	827	473	712
501	754	451	677	386	580	16	592	890	531	798	456	685
465	699	420	631	360	541	17	572	860	512	770	438	659
429	645	387	581	334	501	18	552	829	493	741	421	632
395	593	355	533	305	458	19	532	799	474	712	403	606
361	543	324	487	276	415	20	512	769	455	684	385	579
299	450	268	402	228	343	22	471	708	411	618	336	505
251	378	225	338	192	288	24	413	620	356	535	290	437
214	322	192	288	163	246	26	364	547	313	471	255	383
185	277	165	248	141	212	28	325	488	279	419	226	340
161	242	144	216	123	184	30	293	441	251	377	203	305
141	212	126	190	108	162	32	267	401	228	343	184	276
125	188	112	168	95.6	144	34	245	368	209	314	167	252
						36	226	340	192	289	154	231
						38	210	315	178	268	142	214
						40	196	294	166	249	132	199
						42	183	276	155	233	123	186
						44	173	260	146	219	116	174
						46	163	245	138	207	109	164
						48	155	232	130	196	103	155
						50	147	221	124	186	97.5	147

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
898	1350	826	1240	740	1110
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
731	1100	673	1010	601	902
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
279	419	264	395	246	368
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
108	163	96.8	146	82.8	125

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
7.59	22.3	7.49	21.6	7.31	20.8
Area, in.²					
30.0		27.6		24.7	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
3620	139	3270	124	2850	106
r_y, in.					
2.15		2.12		2.07	
r_x/r_y					
5.12		5.14		5.17	

[c] Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W24

F_y = 50 ksi
F_u = 65 ksi

Shape	W24×						lb/ft	W24×					
	370[h]		335[h]		306[h]			370[h]		335[h]		306[h]	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
	3260	4900	2940	4420	2690	4040	0	2820	4240	2540	3830	2300	3460
	3150	4730	2840	4270	2590	3890	6	2820	4240	2540	3830	2300	3460
	3110	4670	2800	4210	2550	3840	7	2820	4240	2540	3830	2300	3460
	3060	4610	2760	4150	2510	3780	8	2820	4240	2540	3830	2300	3460
	3010	4530	2710	4080	2470	3710	9	2820	4240	2540	3830	2300	3460
	2960	4450	2660	4000	2420	3640	10	2820	4240	2540	3830	2300	3460
	2900	4350	2600	3920	2370	3560	11	2820	4240	2540	3830	2300	3460
	2830	4260	2550	3830	2320	3480	12	2810	4220	2530	3810	2290	3440
	2760	4150	2480	3730	2260	3390	13	2790	4190	2510	3780	2270	3410
	2690	4040	2410	3630	2200	3300	14	2770	4160	2490	3750	2250	3380
	2610	3930	2350	3520	2130	3200	15	2750	4130	2470	3720	2230	3350
	2540	3810	2270	3420	2060	3100	16	2730	4100	2450	3690	2210	3320
	2460	3690	2200	3300	2000	3000	17	2710	4070	2430	3660	2190	3290
	2370	3570	2120	3190	1920	2890	18	2690	4040	2410	3630	2170	3260
	2290	3440	2040	3070	1850	2780	19	2670	4010	2390	3600	2150	3230
	2200	3310	1970	2950	1780	2680	20	2650	3980	2370	3570	2130	3200
	2030	3050	1810	2710	1630	2450	22	2610	3920	2330	3510	2090	3140
	1850	2780	1650	2470	1490	2230	24	2570	3860	2290	3450	2050	3080
	1680	2520	1490	2240	1340	2010	26	2530	3800	2250	3390	2010	3020
	1510	2270	1330	2010	1200	1800	28	2490	3740	2210	3330	1970	2960
	1350	2020	1190	1780	1060	1600	30	2450	3690	2170	3270	1930	2900
	1190	1790	1050	1570	936	1410	32	2410	3630	2130	3200	1890	2840
	1050	1580	926	1390	829	1250	34	2370	3570	2090	3140	1850	2780
	939	1410	826	1240	740	1110	36	2330	3510	2050	3080	1810	2720
	843	1270	741	1110	664	998	38	2290	3450	2010	3020	1770	2660
	760	1140	669	1010	599	901	40	2250	3390	1970	2960	1730	2600
	690	1040	607	912	544	817	42	2210	3330	1930	2900	1690	2540
	628	944	553	831	495	744	44	2170	3270	1890	2840	1650	2480
	575	864	506	760	453	681	46	2130	3210	1850	2780	1610	2420
	528	794	465	698	416	625	48	2090	3150	1810	2720	1570	2370
	487	731	428	644	384	576	50	2050	3090	1770	2660	1530	2310

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3260	4910	2940	4420	2690	4040
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2660	3990	2400	3590	2190	3280
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
851	1280	759	1140	683	1020
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
666	1000	594	893	534	803

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
11.6	69.2	11.4	63.1	11.3	57.9
Area, in.^{2}					
109		98.3		89.7	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
13400	1160	11900	1030	10700	919
r_y, in.					
3.27		3.23		3.20	
r_x/r_y					
3.39		3.41		3.41	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W24

W24× 279[h] P_n/Ω_c ASD	279[h] $\phi_c P_n$ LRFD	250 P_n/Ω_c ASD	250 $\phi_c P_n$ LRFD	229 P_n/Ω_c ASD	229 $\phi_c P_n$ LRFD	Shape lb/ft; Design: Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	W24× 279[h] M_{nx}/Ω_b ASD	279[h] $\phi_b M_{nx}$ LRFD	250 M_{nx}/Ω_b ASD	250 $\phi_b M_{nx}$ LRFD	229 M_{nx}/Ω_b ASD	229 $\phi_b M_{nx}$ LRFD
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
2450	3690	2200	3310	2010	3020	0	2080	3130	1860	2790	1680	2530
2360	3550	2120	3180	1930	2910	6	2080	3130	1860	2790	1680	2530
2330	3500	2090	3140	1910	2870	7	2080	3130	1860	2790	1680	2530
2290	3450	2060	3090	1880	2820	8	2080	3130	1860	2790	1680	2530
2250	3390	2020	3030	1840	2770	9	2080	3130	1860	2790	1680	2530
2210	3320	1980	2970	1800	2710	10	2080	3130	1860	2790	1680	2530
2160	3250	1930	2910	1760	2650	11	2080	3130	1860	2790	1680	2530
2110	3170	1890	2840	1720	2590	12	2070	3110	1840	2760	1660	2500
2050	3090	1840	2760	1670	2520	13	2050	3080	1820	2730	1650	2470
2000	3000	1790	2680	1630	2440	14	2030	3050	1800	2700	1630	2440
1940	2910	1730	2600	1570	2370	15	2010	3020	1780	2680	1610	2420
1880	2820	1670	2520	1520	2290	16	1990	2990	1760	2650	1590	2390
1810	2720	1620	2430	1470	2210	17	1970	2960	1740	2620	1570	2360
1750	2620	1560	2340	1410	2130	18	1950	2930	1720	2590	1550	2330
1680	2520	1500	2250	1360	2040	19	1930	2900	1700	2560	1530	2300
1610	2420	1440	2160	1300	1960	20	1910	2870	1680	2530	1510	2270
1480	2220	1310	1970	1190	1790	22	1870	2810	1640	2470	1470	2210
1340	2020	1190	1790	1070	1620	24	1830	2750	1610	2410	1430	2160
1210	1820	1070	1610	964	1450	26	1790	2690	1570	2350	1400	2100
1080	1620	953	1430	857	1290	28	1750	2640	1530	2300	1360	2040
955	1430	840	1260	754	1130	30	1710	2580	1490	2240	1320	1980
839	1260	739	1110	663	996	32	1670	2520	1450	2180	1280	1920
743	1120	654	983	587	882	34	1640	2460	1410	2120	1240	1870
663	996	584	877	523	787	36	1600	2400	1370	2060	1200	1810
595	894	524	787	470	706	38	1560	2340	1330	2000	1160	1750
537	807	473	711	424	637	40	1520	2280	1290	1950	1130	1690
487	732	429	645	385	578	42	1480	2220	1260	1890	1090	1640
444	667	391	587	350	527	44	1440	2160	1220	1830	1050	1580
406	610	357	537	321	482	46	1400	2100	1180	1770	1010	1510
373	560	328	493	294	443	48	1360	2040	1140	1710	958	1440
344	516	303	455	271	408	50	1320	1990	1090	1640	915	1380

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2450	3690	2200	3310	2010	3020
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2000	2990	1790	2690	1640	2460
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
619	929	547	821	499	749
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
482	724	427	641	384	578

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
11.2	53.4	11.1	48.7	11.0	45.2
Area, in.²					
81.9		73.5		67.2	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
9600	823	8490	724	7650	651
r_y, in.					
3.17		3.14		3.11	
r_x/r_y					
3.41		3.41		3.44	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W24

F_y = 50 ksi
F_u = 65 ksi

W24×						Shape	W24×					
207		192		176		lb/ft	207		192		176	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1820	2730	1690	2540	1550	2330	0	1510	2270	1390	2100	1270	1920
1750	2620	1620	2440	1490	2230	6	1510	2270	1390	2100	1270	1920
1720	2590	1600	2410	1460	2200	7	1510	2270	1390	2100	1270	1920
1690	2540	1570	2370	1440	2160	8	1510	2270	1390	2100	1270	1920
1660	2500	1550	2320	1410	2120	9	1510	2270	1390	2100	1270	1920
1630	2440	1510	2270	1380	2080	10	1510	2270	1390	2100	1270	1920
1590	2390	1480	2220	1350	2030	11	1510	2270	1390	2090	1270	1910
1550	2330	1440	2160	1310	1970	12	1490	2240	1370	2060	1250	1880
1510	2260	1400	2110	1280	1920	13	1470	2210	1350	2040	1230	1850
1460	2200	1360	2040	1240	1860	14	1450	2180	1340	2010	1220	1830
1420	2130	1320	1980	1200	1800	15	1430	2160	1320	1980	1200	1800
1370	2060	1270	1910	1160	1740	16	1410	2130	1300	1950	1180	1770
1320	1980	1220	1840	1110	1670	17	1400	2100	1280	1920	1160	1740
1270	1910	1180	1770	1070	1610	18	1380	2070	1260	1900	1140	1720
1220	1830	1130	1700	1030	1540	19	1360	2040	1240	1870	1120	1690
1170	1750	1080	1630	981	1470	20	1340	2010	1220	1840	1110	1660
1060	1600	985	1480	892	1340	22	1300	1960	1190	1780	1070	1610
959	1440	889	1340	803	1210	24	1260	1900	1150	1730	1030	1550
858	1290	795	1190	717	1080	26	1230	1840	1110	1670	995	1500
761	1140	705	1060	634	952	28	1190	1780	1070	1620	959	1440
668	1000	618	928	554	833	30	1150	1730	1040	1560	922	1390
587	882	543	816	487	732	32	1110	1670	1000	1500	886	1330
520	781	481	723	431	648	34	1070	1610	963	1450	849	1280
464	697	429	645	385	578	36	1040	1560	926	1390	812	1220
416	626	385	579	345	519	38	998	1500	889	1340	771	1160
376	565	347	522	312	468	40	960	1440	848	1270	723	1090
341	512	315	474	283	425	42	920	1380	800	1200	681	1020
310	467	287	432	258	387	44	871	1310	757	1140	643	967
284	427	263	395	236	354	46	827	1240	718	1080	610	916
261	392	241	363	216	325	48	787	1180	683	1030	580	871
240	361	222	334	199	300	50	752	1130	652	979	552	830

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1820	2730	1690	2540	1550	2330
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1480	2220	1380	2070	1260	1890
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
447	671	413	620	378	567
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
342	514	314	473	287	431

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
10.9	41.7	10.8	39.7	10.7	37.4
Area, in.^{2}					
60.7		56.5		51.7	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
6820	578	6260	530	5680	479
r_y, in.					
3.08		3.07		3.04	
r_x/r_y					
3.44		3.42		3.45	

F_y = 50 ksi
F_u = 65 ksi

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W24

W24× 162 P_n/Ω_c	162 $\phi_c P_n$	146 P_n/Ω_c	146 $\phi_c P_n$	131 P_n/Ω_c	131 $\phi_c P_n$	Shape lb/ft; Design	W24× 162 M_{nx}/Ω_b	162 $\phi_b M_{nx}$	146 M_{nx}/Ω_b	146 $\phi_b M_{nx}$	131 M_{nx}/Ω_b	131 $\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1430	2150	1290	1930	1160	1740	0	1170	1760	1040	1570	923	1390
1370	2070	1230	1860	1110	1660	6	1170	1760	1040	1570	923	1390
1350	2030	1220	1830	1090	1640	7	1170	1760	1040	1570	923	1390
1330	2000	1200	1800	1070	1610	8	1170	1760	1040	1570	923	1390
1310	1960	1170	1760	1050	1580	9	1170	1760	1040	1570	923	1390
1280	1920	1150	1720	1030	1540	10	1170	1760	1040	1570	923	1390
1250	1880	1120	1680	1000	1500	11	1160	1750	1040	1560	915	1380
1220	1830	1090	1640	973	1460	12	1150	1720	1020	1530	899	1350
1180	1780	1060	1590	945	1420	13	1130	1700	1000	1510	882	1330
1150	1720	1030	1540	915	1370	14	1110	1670	985	1480	866	1300
1110	1670	991	1490	883	1330	15	1090	1640	968	1460	850	1280
1070	1610	956	1440	851	1280	16	1070	1620	951	1430	833	1250
1030	1550	920	1380	819	1230	17	1060	1590	934	1400	817	1230
992	1490	883	1330	785	1180	18	1040	1560	917	1380	801	1200
951	1430	846	1270	751	1130	19	1020	1530	900	1350	784	1180
910	1370	809	1220	717	1080	20	1000	1510	882	1330	768	1150
828	1240	734	1100	649	975	22	968	1450	848	1270	735	1110
746	1120	659	991	581	873	24	932	1400	814	1220	703	1060
666	1000	587	882	516	775	26	897	1350	780	1170	670	1010
589	886	518	778	453	681	28	861	1290	745	1120	638	958
516	775	452	679	395	594	30	826	1240	711	1070	605	909
453	681	397	597	347	522	32	790	1190	677	1020	570	857
402	603	352	529	307	462	34	754	1130	639	960	523	785
358	538	314	472	274	412	36	716	1080	591	888	482	724
321	483	282	423	246	370	38	667	1000	549	826	447	672
290	436	254	382	222	334	40	625	939	513	771	417	626
263	395	231	346	201	303	42	587	883	482	724	390	586
240	360	210	316	184	276	44	554	833	454	682	367	551
219	330	192	289	168	252	46	525	789	429	645	346	520
201	303	176	265	154	232	48	498	749	407	611	328	493
186	279	163	244			50	474	713	387	581	311	468

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1430	2150	1290	1940	1160	1740
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1170	1750	1050	1570	943	1410
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
353	529	321	482	296	445
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
262	394	233	350	203	306

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
10.8	35.8	10.6	33.7	10.5	31.9
Area, in.²					
47.8		43.0		38.6	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
5170	443	4580	391	4020	340
r_y, in.					
3.05		3.01		2.97	
r_x/r_y					
3.41		3.42		3.43	

Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued) Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces W-Shapes

W24

F_y = 50 ksi
F_u = 65 ksi

W24×						Shape	W24×					
117[c]		104[c]		103[c]		lb/ft	117		104		103	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1010	1520	879	1320	886	1330	0	816	1230	721	1080	699	1050
970	1460	845	1270	815	1230	6	816	1230	721	1080	699	1050
957	1440	833	1250	791	1190	7	816	1230	721	1080	699	1050
942	1420	820	1230	765	1150	8	816	1230	721	1080	681	1050
924	1390	805	1210	731	1100	9	816	1230	721	1080	663	996
906	1360	788	1180	695	1050	10	816	1230	721	1080	645	969
886	1330	770	1160	658	988	11	806	1210	711	1070	626	941
864	1300	751	1130	619	930	12	791	1190	696	1050	608	914
838	1260	731	1100	579	870	13	776	1170	682	1030	590	887
811	1220	710	1070	539	810	14	760	1140	668	1000	572	859
783	1180	688	1030	499	750	15	745	1120	654	982	554	832
754	1130	665	999	459	690	16	730	1100	639	961	535	805
724	1090	641	964	421	632	17	714	1070	625	939	517	777
694	1040	614	923	383	576	18	699	1050	611	918	499	750
663	997	587	882	347	521	19	684	1030	596	896	481	723
633	951	559	840	313	471	20	668	1000	582	875	463	695
571	858	504	757	259	389	22	638	959	553	832	425	639
511	767	449	675	217	327	24	607	912	525	789	375	563
452	679	397	596	185	278	26	576	866	496	746	335	504
396	595	346	520	160	240	28	546	820	467	703	303	455
345	518	302	453	139	209	30	515	774	430	647	276	415
303	456	265	398	122	184	32	470	707	389	584	254	382
268	404	235	353			34	430	646	354	532	235	353
239	360	209	315			36	395	594	324	488	219	329
215	323	188	282			38	365	549	299	450	205	308
194	292	170	255			40	340	511	278	417	192	289
176	264	154	231			42	317	477	259	389	181	273
160	241	140	211			44	298	448	242	364	172	258
147	220	128	193			46	281	422	228	342	163	245
135	202	118	177			48	265	398	215	323	155	233
						50	251	378	203	305	148	222

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1030	1550	919	1380	907	1360
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
839	1260	748	1120	738	1110
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
267	401	241	362	270	404
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
178	268	156	234	104	156

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
10.4	30.4	10.3	29.2	7.03	21.9
Area, in.^{2}					
34.4		30.7		30.3	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
3540	297	3100	259	3000	119
r_y, in.					
2.94		2.91		1.99	
r_x/r_y					
3.44		3.47		5.03	

[c] Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W24

W24×						Shape	W24×					
94[c]		84[c]		76[c]		lb/ft	94		84		76	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
794	1190	689	1040	612	919	0	634	953	559	840	499	750
731	1100	633	951	560	842	6	634	953	559	840	499	750
709	1070	614	922	543	816	7	634	952	557	837	496	745
685	1030	592	890	523	787	8	616	926	541	813	481	722
659	990	569	855	502	755	9	599	900	525	789	466	700
630	947	544	817	480	721	10	582	874	509	765	451	677
599	901	517	777	456	685	11	564	848	493	740	435	654
563	847	490	736	431	648	12	547	822	476	716	420	632
527	792	462	694	405	609	13	530	796	460	692	405	609
490	736	430	646	380	571	14	513	770	444	668	390	587
453	681	397	596	353	530	15	495	744	428	643	375	564
417	627	364	547	323	485	16	478	718	412	619	360	541
382	574	332	499	294	442	17	461	692	396	595	345	519
347	522	302	453	266	400	18	443	666	380	571	330	496
314	472	272	408	239	359	19	426	640	363	546	315	473
283	426	245	368	215	324	20	409	614	347	522	294	442
234	352	203	304	178	268	22	366	551	301	453	252	379
197	296	170	256	150	225	24	322	484	264	396	220	330
168	252	145	218	128	192	26	287	431	234	351	194	292
145	217	125	188	110	165	28	258	388	210	315	174	261
126	189	109	164	95.8	144	30	235	353	190	286	157	236
111	166	95.7	144	84.2	127	32	215	324	174	261	143	215
						34	199	299	160	241	131	198
						36	185	277	148	223	121	183
						38	172	259	138	208	113	170
						40	162	243	129	194	106	159
						42	152	229	122	183	99.0	149
						44	144	216	115	172	93.2	140
						46	136	205	109	163	88.1	132
						48	130	195	103	155	83.6	126
						50	124	186	98.2	148	79.5	119

Properties

Available Strength in Tensile Yielding, kips							Limiting Unbraced Lengths, ft					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$		L_p	L_r	L_p	L_r	L_p	L_r
829	1250	740	1110	671	1010		6.99	21.2	6.89	20.3	6.78	19.5
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips							Area, in.^{2}					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$		27.7		24.7		22.4	
676	1010	601	902	546	819		Moment of Inertia, in.^{4}					
Available Strength in Shear, kips							I_x	I_y	I_x	I_y	I_x	I_y
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$		2700	109	2370	94.4	2100	82.5
250	375	227	340	210	315		r_y, in.					
Available Strength in Flexure about Y-Y Axis, kip-ft							1.98		1.95		1.92	
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$		r_x/r_y					
93.6	141	81.3	122	71.4	107		4.98		5.02		5.05	

[c] Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

W24

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W24×						Shape	W24×					
68[c]		62[c]		55[c]		lb/ft	68		62		55[v]	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
536	806	484	727	415	624	0	442	664	382	574	334	503
489	735	411	617	350	525	6	442	664	364	547	316	475
473	711	387	582	329	494	7	436	655	348	523	301	452
456	685	362	544	306	460	8	422	634	332	499	286	430
436	656	335	503	282	424	9	408	613	316	475	272	408
416	625	307	462	258	387	10	394	592	300	451	257	386
394	592	279	420	233	350	11	380	571	284	426	242	364
372	559	246	369	208	313	12	366	549	268	402	227	342
349	524	214	322	180	270	13	351	528	252	378	213	320
326	490	185	277	155	233	14	337	507	236	354	197	296
303	455	161	242	135	203	15	323	486	214	322	175	263
278	418	141	212	119	178	16	309	465	192	289	157	235
252	379	125	188	105	158	17	295	444	174	262	141	212
226	340	112	168	93.7	141	18	281	422	159	239	129	193
203	305	100	151	84.1	126	19	265	398	146	219	118	177
183	276	90.4	136	75.9	114	20	243	365	135	202	108	163
152	228	74.7	112	62.7	94.3	22	207	311	116	175	93.3	140
127	191					24	180	270	102	154	81.7	123
109	163					26	158	238	91.2	137	72.6	109
93.6	141					28	141	212	82.2	124	65.2	98.0
81.5	123					30	127	191	74.8	112	59.1	88.9
						32	116	174	68.7	103	54.1	81.3
						34	106	159	63.4	95.3	49.8	74.9
						36	97.5	147	58.9	88.6	46.2	69.4
						38	90.4	136	55.1	82.8	43.1	64.7
						40	84.3	127	51.7	77.7	40.3	60.6
						42	78.9	119	48.7	73.2	37.9	57.0
						44	74.2	112	46.0	69.2	35.8	53.8
						46	70.0	105	43.6	65.6	33.9	51.0
						48	66.3	99.6	41.5	62.4	32.2	48.4
						50	62.9	94.6	39.6	59.5	30.7	46.1

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
602	905	545	819	485	729
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
491	736	445	668	397	595
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
197	295	204	306	167	252
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
61.1	91.9	39.1	58.8	33.1	49.8

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
6.61	18.9	4.87	14.4	4.73	13.9
Area, in.²					
20.1		18.2		16.2	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
1830	70.4	1550	34.5	1350	29.1
r_y, in.					
1.87		1.38		1.34	
r_x/r_y					
5.11		6.69		6.80	

[c] Shape is slender for compression with $F_y = 50$ ksi.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.
Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 50 ksi
F_u = 65 ksi

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W21

W21×						Shape	W21×					
275[h]		248		223		lb/ft	275[h]		248		223	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
2450	3680	2210	3320	1990	2990	0	1870	2810	1670	2520	1500	2250
2350	3540	2120	3190	1910	2870	6	1870	2810	1670	2520	1500	2250
2320	3490	2090	3150	1880	2830	7	1870	2810	1670	2520	1500	2250
2280	3430	2060	3090	1850	2780	8	1870	2810	1670	2520	1500	2250
2240	3370	2020	3040	1820	2730	9	1870	2810	1670	2520	1500	2250
2190	3300	1980	2970	1780	2670	10	1870	2810	1670	2520	1500	2250
2150	3220	1930	2900	1730	2610	11	1870	2810	1670	2510	1500	2250
2090	3140	1880	2830	1690	2540	12	1850	2790	1660	2490	1480	2230
2040	3060	1830	2750	1640	2470	13	1840	2760	1640	2470	1470	2200
1980	2970	1780	2670	1590	2390	14	1820	2740	1630	2450	1450	2180
1910	2880	1720	2590	1540	2320	15	1810	2720	1610	2430	1440	2160
1850	2780	1660	2500	1490	2240	16	1790	2700	1600	2400	1420	2140
1780	2680	1600	2410	1430	2150	17	1780	2680	1590	2380	1410	2120
1720	2580	1540	2320	1380	2070	18	1770	2650	1570	2360	1390	2100
1650	2480	1480	2220	1320	1980	19	1750	2630	1560	2340	1380	2070
1580	2370	1420	2130	1260	1900	20	1740	2610	1540	2320	1360	2050
1440	2170	1290	1940	1150	1720	22	1710	2570	1510	2270	1340	2010
1300	1960	1170	1750	1030	1550	24	1680	2520	1480	2230	1310	1960
1170	1760	1040	1570	922	1390	26	1650	2480	1460	2190	1280	1920
1040	1560	926	1390	815	1220	28	1620	2430	1430	2140	1250	1880
912	1370	812	1220	713	1070	30	1590	2390	1400	2100	1220	1830
801	1200	714	1070	626	942	32	1560	2350	1370	2060	1190	1790
710	1070	632	950	555	834	34	1530	2300	1340	2010	1160	1750
633	952	564	847	495	744	36	1500	2260	1310	1970	1130	1700
568	854	506	761	444	668	38	1470	2210	1280	1930	1100	1660
513	771	457	686	401	603	40	1440	2170	1250	1880	1070	1610
465	699	414	623	364	547	42	1410	2130	1220	1840	1050	1570
424	637	377	567	331	498	44	1390	2080	1200	1800	1020	1530
388	583	345	519	303	456	46	1360	2040	1170	1750	987	1480
356	535	317	477	278	418	48	1330	1990	1140	1710	958	1440
328	493	292	439	257	386	50	1300	1950	1110	1670	929	1400

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2450	3680	2210	3320	1990	2990
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2000	2990	1800	2700	1620	2430
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
588	882	521	782	468	702
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
477	716	424	638	374	563

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
10.9	62.5	10.9	57.1	10.7	51.4
Area, in.^{2}					
81.8		73.8		66.5	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
7690	787	6830	699	6080	614
r_y, in.					
3.10		3.08		3.04	
r_x/r_y					
3.13		3.12		3.14	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W21

F_y = 50 ksi
F_u = 65 ksi

Shape	W21×201		W21×182		W21×166			W21×201		W21×182		W21×166	
lb/ft	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Design	Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
	1780	2670	1600	2410	1460	2200	0	1320	1990	1190	1790	1080	1620
	1700	2560	1540	2310	1400	2100	6	1320	1990	1190	1790	1080	1620
	1680	2520	1520	2280	1380	2070	7	1320	1990	1190	1790	1080	1620
	1650	2480	1490	2240	1350	2040	8	1320	1990	1190	1790	1080	1620
	1620	2430	1460	2190	1330	2000	9	1320	1990	1190	1790	1080	1620
	1580	2380	1430	2150	1300	1950	10	1320	1990	1190	1790	1080	1620
	1540	2320	1390	2090	1270	1900	11	1320	1980	1180	1780	1070	1610
	1500	2260	1360	2040	1230	1850	12	1300	1960	1170	1750	1060	1590
	1460	2200	1320	1980	1200	1800	13	1290	1940	1150	1730	1040	1570
	1420	2130	1280	1920	1160	1740	14	1270	1910	1140	1710	1030	1550
	1370	2060	1230	1850	1120	1680	15	1260	1890	1120	1690	1020	1530
	1320	1990	1190	1790	1080	1620	16	1240	1870	1110	1670	1000	1500
	1270	1910	1140	1720	1040	1560	17	1230	1850	1100	1650	987	1480
	1220	1840	1100	1650	998	1500	18	1220	1830	1080	1630	973	1460
	1170	1760	1050	1580	955	1440	19	1200	1810	1070	1600	959	1440
	1120	1680	1010	1510	912	1370	20	1190	1780	1050	1580	945	1420
	1020	1530	911	1370	826	1240	22	1160	1740	1020	1540	916	1380
	913	1370	818	1230	741	1110	24	1130	1700	996	1500	888	1330
	814	1220	728	1090	659	991	26	1100	1650	967	1450	860	1290
	718	1080	641	964	580	872	28	1070	1610	938	1410	832	1250
	627	943	559	841	506	760	30	1040	1560	910	1370	803	1210
	551	829	492	739	445	668	32	1010	1520	881	1320	775	1160
	488	734	436	655	394	592	34	983	1480	852	1280	747	1120
	436	655	389	584	351	528	36	954	1430	824	1240	719	1080
	391	588	349	524	315	474	38	925	1390	795	1200	690	1040
	353	530	315	473	285	428	40	895	1350	767	1150	661	993
	320	481	285	429	258	388	42	866	1300	738	1110	624	938
	292	438	260	391	235	354	44	837	1260	703	1060	591	888
	267	401	238	358	215	323	46	808	1210	668	1000	562	844
	245	368	219	328	198	297	48	771	1160	637	957	535	804
	226	339	201	303			50	737	1110	609	915	511	768

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1780	2670	1600	2410	1460	2200
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1450	2170	1310	1960	1190	1780
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
419	628	377	565	338	506
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
332	499	297	446	269	405

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
10.7	46.2	10.6	42.7	10.6	39.9
Area, in.²					
59.3		53.6		48.8	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
5310	542	4730	483	4280	435
r_y, in.					
3.02		3.00		2.99	
r_x/r_y					
3.14		3.13		3.13	

Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W21

Shape	W21×							W21×					
lb/ft	147		132		122			147		132		122	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
	1290	1940	1160	1750	1070	1620	0	931	1400	831	1250	766	1150
	1240	1860	1110	1670	1030	1550	6	931	1400	831	1250	766	1150
	1220	1830	1090	1640	1010	1520	7	931	1400	831	1250	766	1150
	1200	1800	1070	1610	993	1490	8	931	1400	831	1250	766	1150
	1170	1760	1050	1580	973	1460	9	931	1400	831	1250	766	1150
	1150	1720	1030	1540	950	1430	10	931	1400	831	1250	766	1150
	1120	1680	1000	1510	926	1390	11	923	1390	822	1240	757	1140
	1090	1630	974	1460	900	1350	12	909	1370	809	1220	744	1120
	1050	1580	944	1420	872	1310	13	895	1350	796	1200	731	1100
	1020	1530	913	1370	844	1270	14	881	1320	783	1180	718	1080
	985	1480	882	1320	814	1220	15	868	1300	769	1160	705	1060
	949	1430	849	1280	784	1180	16	854	1280	756	1140	693	1040
	912	1370	815	1220	752	1130	17	840	1260	743	1120	680	1020
	874	1310	781	1170	720	1080	18	826	1240	730	1100	667	1000
	836	1260	746	1120	688	1030	19	813	1220	716	1080	654	983
	797	1200	711	1070	656	986	20	799	1200	703	1060	641	963
	720	1080	642	964	591	889	22	771	1160	677	1020	615	924
	644	968	573	861	528	793	24	744	1120	650	977	589	886
	571	858	507	762	466	701	26	716	1080	624	937	563	847
	501	752	443	667	408	613	28	689	1040	597	898	538	808
	436	655	386	581	355	534	30	661	994	571	858	512	769
	383	576	340	510	312	469	32	634	953	544	818	486	730
	339	510	301	452	276	415	34	606	912	518	778	452	679
	303	455	268	403	247	371	36	579	870	481	723	418	629
	272	408	241	362	221	333	38	542	815	448	674	390	585
	245	369	217	327	200	300	40	509	765	420	631	364	548
	222	334	197	296	181	272	42	480	721	395	594	342	515
	203	305	180	270	165	248	44	453	681	373	561	323	485
	185	279	164	247	151	227	46	430	646	353	531	306	459
	170	256	151	227	139	208	48	409	615	336	505	290	436
							50	390	586	320	481	276	415

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1290	1940	1160	1750	1070	1620
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1050	1580	946	1420	874	1310
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
318	477	283	425	260	391
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
231	347	205	309	189	284

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
10.4	36.3	10.3	34.2	10.3	32.7
Area, in.²					
43.2		38.8		35.9	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
3630	376	3220	333	2960	305
r_y, in.					
2.95		2.93		2.92	
r_x/r_y					
3.11		3.11		3.11	

Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued) Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces W-Shapes

W21

F_y = 50 ksi
F_u = 65 ksi

Shape	W21×111		W21×101[c]		W21×93		W21×111		W21×101		W21×93	
lb/ft	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Design	Available Compressive Strength, kips						Available Flexural Strength, kip-ft					
Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	976	1470	884	1330	817	1230	696	1050	631	949	551	829
6	933	1400	849	1280	731	1100	696	1050	631	949	551	829
7	918	1380	836	1260	702	1050	696	1050	631	949	544	818
8	901	1350	822	1240	670	1010	696	1050	631	949	530	796
9	882	1330	806	1210	635	955	696	1050	631	949	515	774
10	861	1290	787	1180	599	900	696	1050	631	949	500	752
11	839	1260	766	1150	561	843	687	1030	622	935	486	730
12	815	1220	744	1120	522	785	674	1010	610	917	471	708
13	790	1190	721	1080	483	726	662	995	598	899	457	686
14	764	1150	697	1050	444	668	649	976	586	881	442	665
15	736	1110	672	1010	406	610	637	957	575	864	428	643
16	708	1060	646	971	369	554	624	939	563	846	413	621
17	680	1020	620	932	333	500	612	920	551	828	398	599
18	651	978	593	891	298	448	600	901	539	810	384	577
19	621	934	566	851	267	402	587	883	527	793	369	555
20	592	889	539	810	241	363	575	864	515	775	355	533
22	532	800	485	729	199	300	550	826	492	739	321	482
24	475	713	432	649	167	252	525	789	468	704	285	429
26	419	629	381	572	143	215	500	752	445	668	257	386
28	365	549	331	498	123	185	475	714	421	633	233	351
30	318	478	289	434	107	161	450	677	397	597	214	321
32	279	420	254	381			420	631	361	543	197	297
34	248	372	225	338			385	579	331	497	183	276
36	221	332	200	301			356	535	305	458	171	258
38	198	298	180	270			331	497	283	425	161	242
40	179	269	162	244			309	464	263	396	151	228
42	162	244	147	221			290	436	247	371	143	215
44	148	222	134	202			273	410	232	349	136	204
46	135	203	123	185			258	388	219	329	129	194
48	124	187	113	169			245	367	207	311	123	185
50							232	349	197	296	118	177

Properties

	W21×111		W21×101		W21×93	
Available Strength in Tensile Yielding, kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	976	1470	892	1340	817	1230
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	796	1190	728	1090	666	999
Available Strength in Shear, kips	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	237	355	214	321	251	376
Available Strength in Flexure about Y-Y Axis, kip-ft	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
	170	256	154	231	86.6	130

	W21×111		W21×101		W21×93	
Limiting Unbraced Lengths, ft	L_p	L_r	L_p	L_r	L_p	L_r
	10.2	31.2	10.2	30.1	6.50	21.3
Area, in.^{2}	32.6		29.8		27.3	
Moment of Inertia, in.^{4}	I_x	I_y	I_x	I_y	I_x	I_y
	2670	274	2420	248	2070	92.9
r_y, in.	2.90		2.89		1.84	
r_x/r_y	3.12		3.12		4.73	

[c] Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 50 ksi
F_u = 65 ksi

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W21

Shape	W21× 83[c]		W21× 73[c]		W21× 68[c]		W21× 83		W21× 73		W21× 68	
lb/ft	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Design: Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	Available Compressive Strength, kips						Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	728	1090	620	931	567	852	489	735	429	645	399	600
6	652	980	561	843	513	771	489	735	429	645	399	600
7	626	941	541	813	494	743	482	724	421	633	391	588
8	597	898	519	780	474	713	468	703	408	614	379	569
9	566	851	495	744	452	680	454	682	396	595	366	550
10	533	802	467	702	429	644	440	661	383	575	354	532
11	499	751	436	656	404	607	426	640	370	556	341	513
12	465	698	405	609	375	564	412	620	357	537	329	494
13	429	645	374	562	346	520	398	599	344	517	316	475
14	394	593	343	515	317	476	384	578	331	498	304	456
15	360	541	312	469	288	433	371	557	318	479	291	438
16	327	491	283	425	261	392	357	536	306	459	279	419
17	294	443	254	382	234	352	343	515	293	440	266	400
18	263	396	227	341	209	314	329	495	280	421	254	381
19	236	355	204	306	187	282	315	474	267	401	239	359
20	213	320	184	276	169	254	301	453	248	373	220	331
22	176	265	152	228	140	210	264	396	215	324	191	286
24	148	223	128	192	117	176	233	351	190	285	168	252
26	126	190	109	163	100	150	209	314	169	255	149	224
28	109	164	93.8	141	86.3	130	190	285	153	230	135	202
30	94.8	142	81.7	123	75.2	113	173	261	140	210	122	184
32							160	240	128	193	112	169
34							148	223	119	178	104	156
36							138	208	110	166	96.4	145
38							129	195	103	155	90.0	135
40							122	183	96.9	146	84.5	127
42							115	173	91.3	137	79.6	120
44							109	164	86.4	130	75.2	113
46							104	156	82.0	123	71.3	107
48							98.6	148	78.0	117	67.8	102
50							94.2	142	74.4	112	64.7	97.2

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
731	1100	644	968	599	900
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
595	892	523	785	488	731
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
220	331	193	289	181	272
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
76.1	114	66.4	99.8	60.9	91.5

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
6.46	20.2	6.39	19.2	6.36	18.7
Area, in.^{2}					
24.4		21.5		20.0	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
1830	81.4	1600	70.6	1480	64.7
r_y, in.					
1.83		1.81		1.80	
r_x/r_y					
4.74		4.77		4.78	

[c] Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W21

F_y = 50 ksi
F_u = 65 ksi

W21×						Shape	W21×					
62[c]		57[c]		55[c]		lb/ft	62		57		55	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
508	763	461	693	439	659	0	359	540	322	484	314	473
458	688	387	582	394	592	6	359	540	305	459	314	473
441	662	363	546	379	570	7	351	527	292	439	305	458
422	634	338	508	362	545	8	339	510	279	419	294	442
402	604	311	468	344	518	9	327	492	265	399	283	425
381	572	281	422	325	489	10	316	475	252	378	272	409
358	538	249	374	306	459	11	304	457	238	358	261	393
335	504	218	327	285	429	12	293	440	225	338	250	376
310	467	188	283	265	398	13	281	423	212	318	240	360
284	426	162	244	243	366	14	270	405	198	298	229	344
257	387	141	212	220	330	15	258	388	180	270	218	328
232	348	124	187	197	296	16	246	370	163	244	207	311
207	311	110	165	175	263	17	235	353	148	222	196	295
185	278	98.1	147	156	235	18	223	336	136	204	181	272
166	249	88.0	132	140	211	19	205	309	125	188	165	248
150	225	79.4	119	127	190	20	189	284	116	175	152	228
124	186	65.6	98.7	105	157	22	163	245	101	153	130	195
104	156			87.9	132	24	143	214	90.0	135	113	170
88.5	133			74.9	113	26	127	190	80.8	121	100	151
76.3	115			64.6	97.0	28	114	171	73.4	110	89.8	135
						30	103	155	67.2	101	81.2	122
						32	94.4	142	62.0	93.1	74.0	111
						34	87.0	131	57.5	86.4	68.0	102
						36	80.7	121	53.7	80.7	62.9	94.6
						38	75.2	113	50.3	75.6	58.5	87.9
						40	70.4	106	47.4	71.2	54.7	82.2
						42	66.2	99.6	44.8	67.3	51.3	77.1
						44	62.5	94.0	42.4	63.8	48.4	72.7
						46	59.2	89.0	40.3	60.6	45.7	68.7
						48	56.3	84.5	38.5	57.8	43.4	65.2
						50	53.6	80.5	36.7	55.2	41.2	62.0

Properties

W21×62		W21×57		W21×55	
Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
548	824	500	752	485	729
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
445	668	406	609	397	595
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
168	252	171	256	156	234
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
54.1	81.4	36.9	55.5	45.9	69.0

W21×62		W21×57		W21×55	
Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
6.25	18.1	4.77	14.3	6.11	17.4
Area, in.^{2}					
18.3		16.7		16.2	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
1330	57.5	1170	30.6	1140	48.4
r_y, in.					
1.77		1.35		1.73	
r_x/r_y					
4.82		6.19		4.86	

[c] Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W21

Shape	W21× 50[c]		W21× 48[c]		W21× 44[c]			W21× 50		W21× 48[f]		W21× 44	
lb/ft	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
	395	593	371	557	338	507	0	274	413	265	398	238	358
	328	493	330	496	277	417	6	257	387	265	398	221	332
	306	461	317	476	258	388	7	245	368	256	385	210	315
	284	426	302	454	238	358	8	233	350	246	370	198	298
	260	390	286	430	217	326	9	221	332	236	355	187	281
	235	354	269	404	196	294	10	209	314	226	340	176	264
	207	311	252	378	174	262	11	197	295	217	326	165	248
	179	270	234	351	150	225	12	184	277	207	311	154	231
	153	231	216	324	127	192	13	172	259	197	296	142	214
	132	199	198	297	110	165	14	157	236	187	282	125	188
	115	173	179	269	95.7	144	15	140	210	178	267	111	167
	101	152	158	238	84.2	126	16	126	189	168	252	99.9	150
	89.7	135	140	211	74.5	112	17	114	172	155	233	90.4	136
	80.0	120	125	188	66.5	99.9	18	105	157	140	210	82.4	124
	71.8	108	112	169	59.7	89.7	19	96.2	145	128	192	75.6	114
	64.8	97.4	101	152	53.9	80.9	20	89.0	134	117	176	69.8	105
			83.8	126			22	77.3	116	99.8	150	60.3	90.7
			70.4	106			24	68.2	103	86.7	130	53.0	79.7
			60.0	90.2			26	61.0	91.7	76.3	115	47.3	71.0
							28	55.2	82.9	68.1	102	42.6	64.0
							30	50.4	75.7	61.3	92.2	38.8	58.3
							32	46.3	69.6	55.8	83.8	35.6	53.4
							34	42.9	64.5	51.1	76.8	32.8	49.4
							36	39.9	60.0	47.1	70.8	30.5	45.9
							38	37.4	56.2	43.7	65.7	28.5	42.8
							40	35.1	52.8	40.7	61.2	26.7	40.2
							42	33.1	49.8	38.2	57.4	25.2	37.9
							44	31.4	47.2	35.9	53.9	23.8	35.8
							46	29.8	44.8	33.9	50.9	22.6	33.9
							48	28.4	42.6	32.1	48.2	21.5	32.3
							50	27.1	40.7	30.4	45.8	20.5	30.8

Properties

	50		48		44	
Available Strength in Tensile Yielding, kips						
	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	440	662	422	635	389	585
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips						
	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	358	536	345	517	317	475
Available Strength in Shear, kips						
	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	158	237	144	216	145	217
Available Strength in Flexure about Y-Y Axis, kip-ft						
	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
	30.4	45.8	36.7	55.2	25.4	38.2

	50		48		44	
Limiting Unbraced Lengths, ft						
	L_p	L_r	L_p	L_r	L_p	L_r
	4.59	13.6	6.09	16.5	4.45	13.0
Area, in.²	14.7		14.1		13.0	
Moment of Inertia, in.⁴						
	I_x	I_y	I_x	I_y	I_x	I_y
	984	24.9	959	38.7	843	20.7
r_y, in.	1.30		1.66		1.26	
r_x/r_y	6.29		4.96		6.40	

[c] Shape is slender for compression with F_y = 50 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W18

F_y = 50 ksi
F_u = 65 ksi

W18×						Shape	W18×					
311[h]		283[h]		258[h]		lb/ft	311[h]		283[h]		258[h]	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips						Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
2740	4120	2490	3750	2280	3420	0	1880	2830	1690	2540	1520	2290
2630	3950	2380	3580	2170	3270	6	1880	2830	1690	2540	1520	2290
2580	3880	2350	3530	2140	3210	7	1880	2830	1690	2540	1520	2290
2540	3810	2300	3460	2100	3150	8	1880	2830	1690	2540	1520	2290
2490	3740	2260	3390	2050	3090	9	1880	2830	1690	2540	1520	2290
2430	3650	2200	3310	2000	3010	10	1880	2830	1690	2540	1520	2290
2370	3560	2150	3220	1950	2930	11	1870	2820	1680	2520	1520	2280
2300	3460	2090	3130	1900	2850	12	1860	2800	1670	2510	1500	2260
2240	3360	2020	3040	1840	2760	13	1850	2780	1660	2490	1490	2240
2160	3250	1950	2940	1770	2670	14	1840	2770	1650	2470	1480	2230
2090	3140	1890	2830	1710	2570	15	1830	2750	1630	2460	1470	2210
2010	3020	1810	2730	1640	2470	16	1820	2730	1620	2440	1460	2200
1930	2910	1740	2620	1580	2370	17	1810	2720	1610	2420	1450	2180
1850	2790	1670	2510	1510	2270	18	1800	2700	1600	2410	1440	2160
1770	2660	1590	2390	1440	2160	19	1790	2680	1590	2390	1430	2150
1690	2540	1520	2280	1370	2060	20	1770	2670	1580	2370	1420	2130
1530	2300	1370	2050	1230	1850	22	1750	2630	1560	2340	1390	2100
1370	2050	1220	1830	1100	1650	24	1730	2600	1540	2310	1370	2060
1210	1820	1080	1620	965	1450	26	1710	2570	1510	2270	1350	2030
1060	1600	939	1410	839	1260	28	1680	2530	1490	2240	1330	2000
925	1390	818	1230	731	1100	30	1660	2500	1470	2210	1310	1960
813	1220	719	1080	643	966	32	1640	2460	1450	2170	1280	1930
720	1080	637	957	569	855	34	1620	2430	1420	2140	1260	1900
642	965	568	854	508	763	36	1590	2400	1400	2110	1240	1870
576	866	510	766	456	685	38	1570	2360	1380	2070	1220	1830
520	782	460	692	411	618	40	1550	2330	1360	2040	1200	1800
472	709	417	627	373	561	42	1530	2300	1340	2010	1180	1770
430	646	380	572	340	511	44	1510	2260	1310	1980	1150	1730
393	591	348	523	311	467	46	1480	2230	1290	1940	1130	1700
361	543	320	480	286	429	48	1460	2200	1270	1910	1110	1670
						50	1440	2160	1250	1880	1090	1630

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2740	4120	2490	3750	2280	3420
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2230	3350	2030	3050	1850	2780
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
678	1020	613	920	550	826
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
516	776	462	694	414	623

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
10.4	81.1	10.3	73.6	10.2	67.3
Area, in.^{2}					
91.6		83.3		76.0	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
6970	795	6170	704	5510	628
r_y, in.					
2.95		2.91		2.88	
r_x/r_y					
2.96		2.96		2.96	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W18

Shape	W18× 234[h]		W18× 211		W18× 192		W18× 234[h]		W18× 211		W18× 192	
lb/ft	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Design	Available Compressive Strength, kips						Available Flexural Strength, kip-ft					
Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	2050	3090	1870	2800	1680	2530	1370	2060	1220	1840	1100	1660
6	1960	2950	1780	2670	1600	2410	1370	2060	1220	1840	1100	1660
7	1930	2900	1750	2630	1570	2370	1370	2060	1220	1840	1100	1660
8	1890	2840	1710	2580	1540	2320	1370	2060	1220	1840	1100	1660
9	1850	2780	1680	2520	1510	2270	1370	2060	1220	1840	1100	1660
10	1800	2710	1630	2460	1470	2210	1370	2060	1220	1840	1100	1660
11	1760	2640	1590	2390	1430	2150	1360	2040	1210	1820	1090	1640
12	1700	2560	1540	2320	1380	2080	1350	2030	1200	1800	1080	1620
13	1650	2480	1490	2240	1340	2010	1340	2010	1190	1790	1070	1610
14	1590	2390	1440	2160	1290	1940	1330	1990	1180	1770	1060	1590
15	1530	2310	1380	2080	1240	1870	1320	1980	1170	1760	1050	1570
16	1470	2220	1330	2000	1190	1790	1310	1960	1160	1740	1040	1560
17	1410	2120	1270	1910	1140	1710	1290	1950	1150	1720	1030	1540
18	1350	2030	1210	1830	1090	1630	1280	1930	1140	1710	1020	1530
19	1290	1930	1160	1740	1030	1550	1270	1910	1130	1690	1010	1510
20	1220	1840	1100	1650	980	1470	1260	1900	1110	1680	994	1490
22	1100	1650	983	1480	874	1310	1240	1860	1090	1640	973	1460
24	973	1460	870	1310	772	1160	1220	1830	1070	1610	952	1430
26	855	1290	762	1150	674	1010	1200	1800	1050	1580	930	1400
28	742	1120	660	991	582	875	1180	1770	1030	1550	909	1370
30	646	971	575	864	507	763	1150	1730	1010	1510	888	1330
32	568	854	505	759	446	670	1130	1700	986	1480	866	1300
34	503	756	447	672	395	594	1110	1670	964	1450	845	1270
36	449	675	399	600	352	530	1090	1640	943	1420	824	1240
38	403	605	358	538	316	475	1070	1600	922	1390	802	1210
40	364	546	323	486	285	429	1050	1570	900	1350	781	1170
42	330	496	293	441	259	389	1020	1540	879	1320	759	1140
44	300	452	267	401	236	355	1000	1510	857	1290	738	1110
46	275	413	244	367	216	324	981	1470	836	1260	717	1080
48							959	1440	814	1220	695	1050
50							937	1410	793	1190	674	1010

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2050	3090	1870	2800	1680	2530
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1670	2510	1520	2280	1370	2060
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
490	734	439	658	392	588
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
372	559	329	495	297	446

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
10.1	61.4	9.96	55.7	9.85	51.0
Area, in.²					
68.6		62.3		56.2	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
4900	558	4330	493	3870	440
r_y, in.					
2.85		2.82		2.79	
r_x/r_y					
2.96		2.96		2.97	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W18

F_y = 50 ksi
F_u = 65 ksi

W18×						Shape	W18×					
175		158		143		lb/ft	175		158		143	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1540	2310	1390	2080	1260	1890	0	993	1490	888	1340	803	1210
1460	2200	1320	1980	1190	1800	6	993	1490	888	1340	803	1210
1440	2160	1290	1950	1170	1760	7	993	1490	888	1340	803	1210
1410	2120	1270	1900	1150	1730	8	993	1490	888	1340	803	1210
1380	2070	1240	1860	1120	1680	9	993	1490	888	1340	803	1210
1340	2010	1200	1810	1090	1640	10	990	1490	885	1330	799	1200
1300	1960	1170	1760	1060	1590	11	980	1470	874	1310	789	1190
1260	1900	1130	1700	1020	1540	12	969	1460	864	1300	779	1170
1220	1830	1090	1640	989	1490	13	959	1440	853	1280	768	1150
1170	1760	1050	1580	951	1430	14	948	1420	843	1270	758	1140
1130	1690	1010	1520	913	1370	15	938	1410	833	1250	748	1120
1080	1620	968	1460	874	1310	16	927	1390	822	1240	737	1110
1030	1550	924	1390	833	1250	17	916	1380	812	1220	727	1090
983	1480	880	1320	793	1190	18	906	1360	801	1200	717	1080
934	1400	836	1260	752	1130	19	895	1350	791	1190	706	1060
885	1330	791	1190	712	1070	20	885	1330	780	1170	696	1050
788	1180	703	1060	631	949	22	864	1300	759	1140	675	1010
694	1040	618	929	554	833	24	842	1270	738	1110	654	984
605	909	537	807	480	721	26	821	1230	717	1080	634	952
521	784	463	696	414	622	28	800	1200	697	1050	613	921
454	683	403	606	360	542	30	779	1170	676	1020	592	890
399	600	354	533	317	476	32	758	1140	655	984	572	859
354	531	314	472	281	422	34	737	1110	634	953	551	828
315	474	280	421	250	376	36	716	1080	613	921	530	797
283	425	251	378	225	338	38	694	1040	592	890	509	766
255	384	227	341	203	305	40	673	1010	571	858	487	732
232	348	206	309	184	276	42	652	980	550	827	461	693
211	317	187	282	168	252	44	631	948	525	790	438	658
193	290					46	610	917	500	752	417	626
						48	585	879	478	718	398	598
						50	560	842	457	687	380	572

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1540	2310	1390	2080	1260	1890
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1250	1880	1130	1690	1020	1540
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
356	534	319	479	285	427
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
264	398	237	356	213	320

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.75	46.9	9.68	42.8	9.61	39.6
Area, in.^{2}					
51.4		46.3		42.0	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
3450	391	3060	347	2750	311
r_y, in.					
2.76		2.74		2.72	
r_x/r_y					
2.97		2.96		2.97	

Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W18

Shape	W18×							W18×					
lb/ft	130		119		106			130		119		106	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	1150	1720	1050	1580	931	1400	0	724	1090	654	983	574	863
	1090	1640	997	1500	883	1330	6	724	1090	654	983	574	863
	1070	1610	979	1470	866	1300	7	724	1090	654	983	574	863
	1050	1570	957	1440	847	1270	8	724	1090	654	983	574	863
	1020	1530	934	1400	825	1240	9	724	1090	654	983	574	863
	992	1490	909	1370	802	1210	10	719	1080	649	975	568	854
	963	1450	881	1320	778	1170	11	709	1070	639	960	558	839
	931	1400	852	1280	752	1130	12	698	1050	628	945	549	825
	898	1350	822	1240	724	1090	13	688	1030	618	929	539	810
	864	1300	790	1190	696	1050	14	678	1020	608	914	529	795
	829	1250	757	1140	666	1000	15	668	1000	598	899	519	781
	792	1190	724	1090	636	956	16	658	988	588	884	510	766
	755	1140	690	1040	606	910	17	647	973	578	869	500	752
	718	1080	656	986	575	864	18	637	958	568	853	490	737
	681	1020	621	934	544	818	19	627	942	558	838	481	722
	644	967	587	883	513	772	20	617	927	548	823	471	708
	570	857	520	781	453	681	22	596	896	527	793	452	679
	499	750	455	683	395	594	24	576	866	507	762	432	650
	431	648	392	589	340	511	26	556	835	487	732	413	620
	372	559	338	508	293	440	28	535	805	467	702	393	591
	324	487	295	443	255	384	30	515	774	447	671	374	562
	285	428	259	389	224	337	32	494	743	426	641	353	531
	252	379	229	345	199	299	34	474	713	406	611	327	492
	225	338	205	307	177	266	36	454	682	380	571	305	458
	202	303	184	276	159	239	38	428	644	356	535	285	428
	182	274	166	249	144	216	40	404	607	335	504	268	402
	165	248	150	226	130	196	42	382	574	316	476	252	379
	151	226	137	206	119	178	44	362	544	300	451	239	359
							46	345	518	285	428	227	341
							48	329	494	272	408	216	325
							50	314	472	259	390	206	310

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1150	1720	1050	1580	931	1400
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
933	1400	855	1280	757	1140
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
259	388	249	373	221	331
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
191	288	172	259	151	227

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.54	36.6	9.50	34.3	9.40	31.8
Area, in.^{2}					
38.3		35.1		31.1	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
2460	278	2190	253	1910	220
r_y, in.					
2.70		2.69		2.66	
r_x/r_y					
2.97		2.94		2.95	

Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W18

F_y = 50 ksi
F_u = 65 ksi

W18×						Shape	W18×					
97		86		76[c]		lb/ft	97		86		76	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
853	1280	757	1140	660	992	0	526	791	464	698	407	611
808	1220	717	1080	628	944	6	526	791	464	698	407	611
793	1190	703	1060	617	928	7	526	791	464	698	407	611
775	1170	687	1030	604	908	8	526	791	464	698	407	611
756	1140	670	1010	589	885	9	526	791	464	698	407	611
734	1100	651	978	572	860	10	520	782	458	688	400	601
712	1070	630	947	554	832	11	511	768	449	674	392	589
688	1030	608	914	534	803	12	502	754	440	661	383	576
662	995	586	880	514	773	13	492	740	431	647	375	563
636	956	562	845	493	741	14	483	725	422	634	366	550
609	915	538	808	472	709	15	473	711	412	620	358	537
581	874	513	771	449	676	16	464	697	403	606	349	525
553	832	488	733	427	642	17	454	683	394	593	341	512
525	789	463	695	405	608	18	445	669	385	579	332	499
497	746	437	657	382	574	19	436	655	376	566	324	486
468	704	412	619	360	541	20	426	640	367	552	315	474
413	621	363	545	316	475	22	407	612	349	525	298	448
360	541	315	474	274	412	24	388	584	331	498	281	423
309	464	270	406	235	353	26	369	555	313	471	264	397
266	400	233	350	202	304	28	351	527	295	444	242	364
232	349	203	305	176	265	30	332	499	270	406	219	329
204	307	178	268	155	233	32	306	460	247	372	200	300
181	272	158	237	137	206	34	283	425	228	343	183	276
161	242	141	212	122	184	36	263	395	211	318	170	255
145	217	126	190	110	165	38	245	369	197	296	158	237
131	196	114	172	99.1	149	40	230	346	184	277	147	221
118	178	104	156	89.9	135	42	217	326	173	261	138	208
108	162					44	205	308	164	246	130	196
						46	194	292	155	233	123	185
						48	185	278	147	221	117	175
						50	176	265	140	211	111	167

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
853	1280	757	1140	668	1000
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
696	1040	618	926	543	814
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
199	299	177	265	155	232
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
138	207	121	182	105	158

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.36	30.4	9.29	28.6	9.22	27.1
Area, in.^{2}					
28.5		25.3		22.3	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
1750	201	1530	175	1330	152
r_y, in.					
2.65		2.63		2.61	
r_x/r_y					
2.95		2.95		2.96	

[c] Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W18

Shape	W18×						W18×					
lb/ft	71		65		60^c		71		65		60	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips						Available Flexural Strength, kip-ft					
Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	626	940	572	859	517	776	364	548	332	499	307	461
6	549	825	501	753	460	691	364	548	332	498	306	460
7	523	787	477	717	439	660	354	532	322	483	297	446
8	496	745	452	679	415	624	343	516	312	468	287	431
9	466	700	424	638	390	585	333	500	302	453	277	417
10	435	653	396	594	363	545	322	485	292	438	268	402
11	403	605	366	550	336	504	312	469	282	424	258	388
12	370	557	336	505	308	463	302	453	272	409	249	374
13	338	508	307	461	281	422	291	438	262	394	239	359
14	306	461	278	417	254	381	281	422	252	379	229	345
15	276	414	249	375	228	342	270	406	242	364	220	330
16	246	370	222	334	203	304	260	390	232	349	210	316
17	218	328	197	296	179	270	249	375	222	334	200	301
18	195	292	176	264	160	241	239	359	212	319	191	287
19	175	262	158	237	144	216	228	343	201	302	177	266
20	158	237	142	214	130	195	216	324	187	281	164	247
22	130	196	118	177	107	161	190	285	164	246	144	216
24	109	165	98.9	149	90.0	135	169	254	145	219	127	192
26	93.3	140	84.2	127	76.7	115	153	230	131	197	115	172
28	80.4	121	72.6	109	66.1	99.4	139	209	119	179	104	156
30							128	192	109	164	95.2	143
32							118	178	101	152	87.9	132
34							110	166	93.9	141	81.6	123
36							103	155	87.8	132	76.1	114
38							96.9	146	82.4	124	71.4	107
40							91.4	137	77.6	117	67.2	101
42							86.6	130	73.4	110	63.5	95.5
44							82.2	124	69.7	105	60.2	90.5
46							78.2	118	66.3	99.6	57.3	86.1
48							74.7	112	63.2	95.0	54.6	82.1
50							71.4	107	60.4	90.8	52.2	78.4

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
626	941	572	860	527	792
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
510	765	465	697	429	644
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
183	275	166	248	151	227
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
61.6	92.6	56.1	84.4	51.4	77.3

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
6.00	19.6	5.97	18.8	5.93	18.2
Area, in.²					
20.9		19.1		17.6	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
1170	60.3	1070	54.8	984	50.1
r_y, in.					
1.70		1.69		1.68	
r_x/r_y					
4.41		4.43		4.45	

[c] Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W18

F_y = 50 ksi
F_u = 65 ksi

W18× 55[c] P_n/Ω_c ASD	55[c] $\phi_c P_n$ LRFD	50[c] P_n/Ω_c ASD	50[c] $\phi_c P_n$ LRFD	46[c] P_n/Ω_c ASD	46[c] $\phi_c P_n$ LRFD	Shape lb/ft — Design: Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	W18× 55 M_{nx}/Ω_b ASD	55 $\phi_b M_{nx}$ LRFD	50 M_{nx}/Ω_b ASD	50 $\phi_b M_{nx}$ LRFD	46 M_{nx}/Ω_b ASD	46 $\phi_b M_{nx}$ LRFD
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
468	703	414	622	379	569	0	279	420	252	379	226	340
416	625	367	551	312	469	6	279	419	251	377	212	319
398	599	351	528	291	437	7	269	405	242	363	203	304
379	570	334	502	268	403	8	260	391	233	350	193	290
357	537	315	474	242	364	9	251	377	224	337	183	275
333	500	296	445	215	323	10	242	363	216	324	173	261
307	462	276	414	188	283	11	232	349	207	311	164	246
282	423	252	379	163	244	12	223	335	198	298	154	232
256	385	229	344	139	209	13	214	321	190	285	144	217
231	348	206	310	120	180	14	205	307	181	272	133	200
207	312	184	277	104	157	15	195	293	172	259	119	179
184	277	163	245	91.6	138	16	186	280	163	246	108	163
163	245	145	217	81.1	122	17	177	266	154	232	99.0	149
146	219	129	194	72.4	109	18	165	248	141	212	91.1	137
131	196	116	174	65.0	97.6	19	152	228	130	195	84.4	127
118	177	104	157	58.6	88.1	20	141	212	120	180	78.5	118
97.4	146	86.3	130			22	123	184	104	156	69.0	104
81.9	123	72.5	109			24	108	163	91.5	137	61.5	92.4
69.8	105	61.8	92.9			26	97.1	146	81.7	123	55.4	83.3
						28	87.9	132	73.8	111	50.5	75.9
						30	80.4	121	67.3	101	46.4	69.7
						32	74.0	111	61.8	92.9	42.9	64.5
						34	68.6	103	57.2	85.9	39.9	60.0
						36	63.9	96.1	53.2	79.9	37.4	56.2
						38	59.9	90.0	49.7	74.8	35.1	52.8
						40	56.3	84.6	46.7	70.2	33.1	49.8
						42	53.2	79.9	44.0	66.2	31.3	47.1
						44	50.3	75.7	41.7	62.6	29.7	44.7
						46	47.8	71.9	39.5	59.4	28.3	42.5
						48	45.6	68.5	37.6	56.6	27.0	40.6
						50	43.5	65.4	35.9	54.0	25.8	38.8

Properties

55[c] ASD	55[c] LRFD	50[c] ASD	50[c] LRFD	46[c] ASD	46[c] LRFD
Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
485	729	440	662	404	608
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
397	595	358	536	328	492
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
141	212	128	192	130	195
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
46.2	69.4	41.4	62.3	29.2	43.9

55		50		46	
Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
5.90	17.6	5.83	16.9	4.56	13.7
Area, in.²					
16.2		14.7		13.5	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
890	44.9	800	40.1	712	22.5
r_y, in.					
1.67		1.65		1.29	
r_x/r_y					
4.44		4.47		5.62	

[c] Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued) Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces

W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W18–W16

Shape	W18×				W16×			W18×				W16×	
lb/ft	40[c]		35[c]		100			40		35		100	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
	318	479	271	407	880	1320	0	196	294	166	249	494	743
	261	392	219	329	829	1250	6	182	274	152	229	494	743
	242	364	202	304	811	1220	7	173	261	144	216	494	743
	223	335	185	279	791	1190	8	165	247	136	204	494	743
	203	305	168	252	769	1160	9	156	234	128	192	493	741
	183	275	150	225	745	1120	10	147	221	120	180	485	729
	160	241	131	197	719	1080	11	138	207	112	168	477	717
	138	207	111	167	692	1040	12	129	194	103	155	469	705
	118	177	94.7	142	664	997	13	120	181	91.9	138	461	693
	101	152	81.6	123	634	953	14	107	161	81.1	122	454	682
	88.3	133	71.1	107	604	908	15	95.9	144	72.4	109	446	670
	77.6	117	62.5	93.9	574	862	16	86.6	130	65.2	97.9	438	658
	68.7	103	55.4	83.2	543	816	17	78.9	119	59.2	88.9	430	646
	61.3	92.2	49.4	74.2	512	770	18	72.4	109	54.1	81.3	422	634
	55.0	82.7	44.3	66.6	481	724	19	66.8	100	49.8	74.8	414	622
	49.7	74.6	40.0	60.1	451	678	20	62.0	93.2	46.1	69.2	406	611
					392	589	22	54.2	81.4	40.0	60.1	390	587
					336	504	24	48.0	72.2	35.3	53.1	375	563
					286	430	26	43.2	64.9	31.6	47.5	359	539
					247	371	28	39.2	58.9	28.6	43.0	343	516
					215	323	30	35.9	53.9	26.1	39.2	327	492
					189	284	32	33.1	49.8	24.0	36.1	312	468
					167	251	34	30.7	46.2	22.2	33.4	292	439
					149	224	36	28.7	43.1	20.7	31.1	273	410
					134	201	38	26.9	40.4	19.4	29.1	256	385
					121	182	40	25.3	38.1	18.2	27.4	242	363
							42	23.9	36.0	17.2	25.8	228	343
							44	22.7	34.1	16.3	24.4	217	326
							46	21.6	32.4	15.4	23.2	206	310
							48	20.6	30.9	14.7	22.1	197	296
							50	19.6	29.5	14.0	21.1	188	283

Properties

	W18×40		W18×35		W16×100	
Available Strength in Tensile Yielding, kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	353	531	308	464	880	1320
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	288	431	251	377	718	1080
Available Strength in Shear, kips	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	113	169	106	159	199	298
Available Strength in Flexure about Y-Y Axis, kip-ft	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
	25.0	37.5	20.1	30.2	137	206

	W18×40		W18×35		W16×100	
Limiting Unbraced Lengths, ft	L_p	L_r	L_p	L_r	L_p	L_r
	4.49	13.1	4.31	12.3	8.87	32.8
Area, in.²	11.8		10.3		29.4	
Moment of Inertia, in.⁴	I_x	I_y	I_x	I_y	I_x	I_y
	612	19.1	510	15.3	1490	186
r_y, in.	1.27		1.22		2.51	
r_x/r_y	5.68		5.77		2.83	

[c] Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W16

F_y = 50 ksi
F_u = 65 ksi

Shape	W16× 89		W16× 77		W16× 67[c]		W16× 89		W16× 77		W16× 67	
lb/ft	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Design	Available Compressive Strength, kips						Available Flexural Strength, kip-ft					
Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	784	1180	677	1020	587	882	437	656	374	563	324	488
6	738	1110	636	956	551	828	437	656	374	563	324	488
7	722	1080	622	935	539	810	437	656	374	563	324	488
8	704	1060	606	911	525	789	437	656	374	563	324	488
9	684	1030	588	884	510	766	435	654	372	559	322	484
10	662	995	569	856	493	741	427	642	365	548	315	474
11	639	960	549	825	475	715	420	631	358	537	308	464
12	614	923	528	793	457	687	412	619	350	526	301	453
13	589	885	505	760	437	657	404	607	343	515	295	443
14	562	845	482	725	417	627	396	596	336	504	288	432
15	535	805	459	690	397	596	388	584	328	493	281	422
16	508	763	435	654	376	565	381	572	321	482	274	412
17	480	722	411	618	355	533	373	560	314	471	267	401
18	452	680	387	581	334	502	365	549	306	460	260	391
19	425	639	363	545	313	471	357	537	299	449	253	380
20	398	598	339	510	293	440	350	525	292	438	246	370
22	345	518	293	441	253	380	334	502	277	416	232	349
24	294	442	250	376	215	323	319	479	262	394	219	328
26	251	377	213	320	183	275	303	455	248	372	205	308
28	216	325	184	276	158	237	287	432	232	349	184	277
30	188	283	160	240	138	207	272	409	212	318	167	252
32	166	249	141	211	121	182	251	377	194	292	153	230
34	147	220	124	187	107	161	233	350	180	270	141	213
36	131	197	111	167	95.5	144	217	327	167	252	131	197
38	117	176	99.7	150	85.7	129	204	306	157	235	122	184
40	106	159	89.9	135	77.4	116	192	288	147	221	115	172
42							181	272	139	209	108	162
44							172	258	131	197	102	153
46							163	245	125	187	96.7	145
48							156	234	119	178	91.9	138
50							149	223	113	170	87.5	132

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
784	1180	677	1020	587	882
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
640	960	553	829	478	717
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
176	265	150	225	129	193
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
120	180	103	154	88.6	133

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
8.80	30.2	8.72	27.8	8.69	26.1
Area, in.^{2}					
26.2		22.6		19.6	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
1300	163	1110	138	954	119
r_y, in.					
2.49		2.47		2.46	
r_x/r_y					
2.83		2.83		2.83	

[c] Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W16

Shape	W16×						W16×					
lb/ft	57		50[c]		45[c]		57		50		45	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips						Available Flexural Strength, kip-ft					
Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	503	756	436	655	385	578	262	394	230	345	205	309
6	434	652	379	569	336	506	259	390	227	341	202	304
7	411	618	359	539	320	482	251	378	219	329	195	293
8	387	581	337	507	303	455	243	366	211	318	188	282
9	360	542	314	472	282	423	235	354	204	306	181	271
10	333	501	290	436	260	390	227	342	196	295	173	261
11	306	460	266	400	237	357	219	330	189	283	166	250
12	278	418	242	363	215	324	211	318	181	272	159	239
13	251	377	218	327	193	291	203	306	173	260	152	228
14	225	338	195	292	172	259	195	294	166	249	145	217
15	199	300	172	259	152	229	188	282	158	237	137	207
16	175	264	152	228	134	201	180	270	150	226	130	196
17	155	233	134	202	118	178	172	258	143	215	121	181
18	139	208	120	180	106	159	164	246	132	198	111	166
19	124	187	107	162	94.8	142	153	230	122	183	102	154
20	112	169	97.0	146	85.5	129	143	215	114	171	94.9	143
22	92.8	139	80.1	120	70.7	106	126	189	99.6	150	82.9	125
24	77.9	117	67.3	101	59.4	89.3	112	169	88.6	133	73.5	111
26	66.4	99.8	57.4	86.2	50.6	76.1	102	153	79.9	120	66.1	99.3
28							92.9	140	72.7	109	60.0	90.2
30							85.5	128	66.8	100	55.0	82.6
32							79.2	119	61.7	92.7	50.7	76.2
34							73.8	111	57.4	86.3	47.1	70.8
36							69.1	104	53.7	80.6	43.9	66.0
38							65.0	97.7	50.4	75.7	41.2	61.9
40							61.3	92.2	47.5	71.4	38.8	58.3
42							58.1	87.3	44.9	67.5	36.7	55.1
44							55.2	83.0	42.6	64.1	34.8	52.3
46							52.6	79.0	40.6	61.0	33.1	49.7
48							50.2	75.4	38.7	58.2	31.5	47.4
50							48.0	72.2	37.0	55.6	30.1	45.2

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
503	756	440	662	398	599
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
410	614	358	536	324	487
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
141	212	124	186	111	167
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
47.2	70.9	40.7	61.1	36.2	54.4

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
5.65	18.3	5.62	17.2	5.55	16.5
Area, in.²					
16.8		14.7		13.3	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
758	43.1	659	37.2	586	32.8
r_y, in.					
1.60		1.59		1.57	
r_x/r_y					
4.20		4.20		4.24	

[c] Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W16

F_y = 50 ksi
F_u = 65 ksi

W16×						Shape	W16×					
40[c]		36[c]		31[c]		lb/ft	40		36		31	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
331	497	293	440	245	369	0	182	274	160	240	135	203
289	435	254	382	194	291	6	179	269	156	234	122	183
276	414	241	363	178	268	7	172	259	150	225	115	173
261	392	228	342	161	243	8	166	249	143	216	108	163
245	368	213	320	144	217	9	159	239	137	206	102	153
228	342	198	297	127	190	10	152	229	131	197	94.9	143
211	317	182	274	108	162	11	146	219	125	188	88.1	132
191	287	165	247	90.6	136	12	139	209	119	178	80.4	121
172	258	147	221	77.2	116	13	132	199	112	169	70.5	106
153	230	130	195	66.6	100	14	126	189	106	160	62.6	94.0
135	203	114	171	58.0	87.1	15	119	179	100	150	56.1	84.4
119	178	99.9	150	51.0	76.6	16	112	168	90.8	136	50.8	76.4
105	158	88.5	133	45.1	67.8	17	101	152	82.2	124	46.4	69.7
93.7	141	78.9	119	40.3	60.5	18	92.8	139	75.0	113	42.6	64.0
84.1	126	70.8	106	36.1	54.3	19	85.4	128	68.8	103	39.4	59.2
75.9	114	63.9	96.1			20	79.0	119	63.6	95.5	36.6	55.0
62.7	94.3	52.8	79.4			22	68.7	103	55.0	82.6	32.0	48.2
52.7	79.2	44.4	66.7			24	60.7	91.2	48.4	72.7	28.5	42.8
44.9	67.5					26	54.3	81.7	43.1	64.8	25.6	38.5
						28	49.2	73.9	38.9	58.4	23.3	35.1
						30	44.9	67.5	35.4	53.2	21.4	32.1
						32	41.3	62.1	32.5	48.8	19.8	29.7
						34	38.3	57.5	30.0	45.1	18.4	27.6
						36	35.7	53.6	27.9	41.9	17.2	25.8
						38	33.4	50.2	26.0	39.1	16.1	24.2
						40	31.4	47.2	24.4	36.7	15.2	22.8
						42	29.6	44.5	23.0	34.6	14.4	21.6
						44	28.0	42.1	21.8	32.7	13.6	20.5
						46	26.6	40.0	20.6	31.0	13.0	19.5
						48	25.4	38.1	19.6	29.5	12.4	18.6
						50	24.2	36.4	18.7	28.1	11.8	17.7

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
353	531	317	477	273	411
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
288	431	258	388	223	334
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
97.6	146	93.8	141	87.5	131
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
31.7	47.6	26.9	40.5	17.5	26.4

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
5.55	15.9	5.37	15.2	4.13	11.8
Area, in.²					
11.8		10.6		9.13	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
518	28.9	448	24.5	375	12.4
r_y, in.					
1.57		1.52		1.17	
r_x/r_y					
4.22		4.28		5.48	

[c] Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W16–W14

Shape	W16×		W14×					W16×		W14×			
lb/ft	26[c]		873[h]		808[h]			26[v]		873[h]		808[h]	
	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
	198	298	7690	11600	7130	10700	0	110	166	5060	7610	4570	6860
	154	231	7570	11400	7010	10500	6	98.0	147	5060	7610	4570	6860
	140	211	7530	11300	6970	10500	7	92.0	138	5060	7610	4570	6860
	126	190	7480	11200	6920	10400	8	86.0	129	5060	7610	4570	6860
	112	168	7430	11200	6870	10300	9	80.1	120	5060	7610	4570	6860
	98.1	147	7360	11100	6810	10200	10	74.1	111	5060	7610	4570	6860
	83.1	125	7300	11000	6750	10100	11	68.1	102	5060	7610	4570	6860
	69.8	105	7220	10900	6680	10000	12	59.1	88.8	5060	7610	4570	6860
	59.5	89.4	7140	10700	6600	9920	13	51.5	77.5	5060	7610	4570	6860
	51.3	77.1	7060	10600	6520	9800	14	45.5	68.4	5060	7610	4570	6860
	44.7	67.2	6970	10500	6440	9680	15	40.6	61.1	5060	7610	4570	6860
	39.3	59.0	6880	10300	6350	9540	16	36.6	55.0	5060	7610	4570	6860
	34.8	52.3	6780	10200	6250	9400	17	33.2	50.0	5060	7610	4570	6860
	31.0	46.6	6680	10000	6160	9250	18	30.4	45.7	5060	7600	4560	6850
			6570	9870	6050	9100	19	28.0	42.1	5050	7590	4550	6840
			6460	9700	5950	8940	20	25.9	39.0	5040	7580	4540	6830
			6220	9350	5730	8610	22	22.6	33.9	5030	7560	4530	6810
			5980	8980	5490	8260	24	19.9	30.0	5010	7540	4520	6790
			5720	8600	5250	7890	26	17.8	26.8	5000	7510	4500	6760
			5460	8200	5000	7520	28	16.2	24.3	4980	7490	4490	6740
			5190	7790	4750	7130	30	14.8	22.2	4970	7470	4470	6720
			4910	7380	4490	6750	32	13.6	20.4	4950	7440	4460	6700
			4630	6970	4230	6360	34	12.6	18.9	4940	7420	4440	6680
			4360	6550	3970	5970	36	11.7	17.6	4920	7400	4430	6650
			4080	6140	3710	5580	38	11.0	16.5	4910	7370	4410	6630
			3810	5730	3460	5200	40	10.3	15.5	4890	7350	4400	6610
			3550	5340	3210	4830	42	9.74	14.6	4880	7330	4380	6590
			3290	4950	2970	4470	44	9.22	13.9	4860	7300	4370	6570
			3040	4570	2740	4120	46	8.76	13.2	4840	7280	4350	6540
			2800	4200	2520	3780	48	8.34	12.5	4830	7260	4340	6520
			2580	3870	2320	3480	50	7.96	12.0	4810	7240	4320	6500

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
230	346	7690	11600	7130	10700
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
187	281	6270	9410	5820	8730
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
70.5	106	1860	2790	1710	2560
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
13.7	20.6	2540	3830	2320	3490

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
3.96	11.2	17.3	329	17.1	309
Area, in.^{2}					
7.68		257		238	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
301	9.59	18100	6170	15900	5550
r_y, in.					
1.12		4.90		4.83	
r_x/r_y					
5.59		1.71		1.69	

[c] Shape is slender for compression with F_y = 50 ksi.
[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 50 ksi; therefore, ϕ_v = 0.90 and Ω_v = 1.67.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W14

F_y = 50 ksi
F_u = 65 ksi

Shape	W14×730[h]		W14×665[h]		W14×605[h]		Design	W14×730[h]		W14×665[h]		W14×605[h]	
lb/ft	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
	6440	9670	5870	8820	5330	8010	0	4140	6230	3690	5550	3290	4950
	6330	9510	5760	8660	5230	7860	6	4140	6230	3690	5550	3290	4950
	6290	9450	5730	8610	5200	7810	7	4140	6230	3690	5550	3290	4950
	6240	9380	5690	8550	5160	7750	8	4140	6230	3690	5550	3290	4950
	6190	9310	5640	8470	5110	7690	9	4140	6230	3690	5550	3290	4950
	6140	9220	5590	8400	5070	7610	10	4140	6230	3690	5550	3290	4950
	6070	9130	5530	8310	5010	7530	11	4140	6230	3690	5550	3290	4950
	6010	9030	5470	8220	4950	7440	12	4140	6230	3690	5550	3290	4950
	5940	8920	5400	8110	4890	7350	13	4140	6230	3690	5550	3290	4950
	5860	8810	5330	8010	4820	7250	14	4140	6230	3690	5550	3290	4950
	5780	8690	5250	7890	4750	7140	15	4140	6230	3690	5550	3290	4950
	5690	8560	5170	7770	4680	7030	16	4140	6230	3690	5550	3290	4950
	5610	8430	5090	7650	4600	6920	17	4140	6220	3690	5540	3290	4940
	5510	8290	5000	7520	4520	6790	18	4130	6210	3680	5530	3280	4930
	5420	8140	4910	7380	4440	6670	19	4120	6200	3670	5520	3270	4920
	5320	7990	4820	7240	4350	6540	20	4120	6190	3670	5510	3270	4910
	5110	7670	4620	6950	4170	6260	22	4100	6160	3650	5490	3250	4890
	4890	7340	4420	6640	3980	5980	24	4090	6140	3640	5470	3240	4870
	4660	7000	4200	6320	3780	5680	26	4070	6120	3620	5450	3230	4850
	4420	6650	3990	5990	3580	5380	28	4060	6100	3610	5430	3210	4830
	4180	6290	3760	5660	3370	5070	30	4040	6080	3600	5400	3200	4810
	3940	5930	3540	5320	3170	4760	32	4030	6050	3580	5380	3180	4790
	3700	5560	3320	4990	2960	4450	34	4010	6030	3570	5360	3170	4770
	3460	5200	3100	4650	2760	4140	36	4000	6010	3550	5340	3160	4740
	3220	4850	2880	4330	2560	3840	38	3980	5990	3540	5320	3140	4720
	2990	4500	2670	4010	2360	3550	40	3970	5970	3520	5300	3130	4700
	2770	4160	2460	3690	2170	3270	42	3950	5940	3510	5280	3120	4680
	2550	3830	2260	3390	1990	2990	44	3940	5920	3500	5250	3100	4660
	2330	3510	2060	3100	1820	2730	46	3920	5900	3480	5230	3090	4640
	2140	3220	1900	2850	1670	2510	48	3910	5880	3470	5210	3070	4620
	1970	2970	1750	2630	1540	2310	50	3900	5850	3450	5190	3060	4600

Properties

W14×	730[h]		665[h]		605[h]	
Available Strength in Tensile Yielding, kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	6440	9680	5870	8820	5330	8010
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	5230	7850	4780	7170	4360	6530
Available Strength in Shear, kips	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	1380	2060	1220	1830	1090	1630
Available Strength in Flexure about Y-Y Axis, kip-ft	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
	2040	3060	1820	2740	1630	2450

W14×	730[h]		665[h]		605[h]	
Limiting Unbraced Lengths, ft	L_p	L_r	L_p	L_r	L_p	L_r
	16.6	275	16.3	253	16.1	232
Area, in.²	215		196		178	
Moment of Inertia, in.⁴	I_x	I_y	I_x	I_y	I_x	I_y
	14300	4720	12400	4170	10800	3680
r_y, in.	4.69		4.62		4.55	
r_x/r_y	1.74		1.73		1.71	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

F_y = 50 ksi
F_u = 65 ksi

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W14

W14×						Shape	W14×					
550[h]		500[h]		455[h]		lb/ft	550[h]		500[h]		455[h]	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
4850	7290	4400	6610	4010	6030	0	2940	4430	2620	3940	2340	3510
4760	7150	4320	6490	3930	5910	6	2940	4430	2620	3940	2340	3510
4730	7110	4290	6440	3910	5870	7	2940	4430	2620	3940	2340	3510
4690	7050	4250	6390	3870	5820	8	2940	4430	2620	3940	2340	3510
4650	6990	4210	6330	3840	5770	9	2940	4430	2620	3940	2340	3510
4600	6920	4170	6270	3800	5710	10	2940	4430	2620	3940	2340	3510
4550	6840	4120	6200	3750	5640	11	2940	4430	2620	3940	2340	3510
4500	6760	4070	6120	3710	5570	12	2940	4430	2620	3940	2340	3510
4440	6670	4020	6040	3660	5500	13	2940	4430	2620	3940	2340	3510
4380	6580	3960	5950	3600	5420	14	2940	4430	2620	3940	2340	3510
4310	6480	3900	5860	3550	5330	15	2940	4430	2620	3940	2340	3510
4240	6380	3840	5770	3490	5240	16	2940	4420	2620	3930	2330	3510
4170	6270	3770	5660	3420	5150	17	2940	4410	2610	3920	2330	3500
4100	6160	3700	5560	3360	5050	18	2930	4400	2600	3910	2320	3490
4020	6040	3630	5450	3290	4950	19	2920	4390	2600	3910	2310	3480
3940	5920	3550	5340	3220	4840	20	2920	4380	2590	3900	2310	3470
3770	5660	3390	5100	3080	4620	22	2900	4360	2580	3880	2290	3450
3590	5400	3230	4860	2920	4400	24	2890	4340	2570	3860	2280	3430
3410	5120	3060	4600	2770	4160	26	2880	4320	2550	3840	2270	3410
3220	4840	2890	4340	2610	3920	28	2860	4300	2540	3820	2260	3390
3030	4560	2720	4080	2450	3680	30	2850	4280	2530	3800	2250	3370
2840	4270	2540	3820	2290	3440	32	2840	4260	2510	3780	2230	3360
2650	3990	2370	3560	2130	3200	34	2820	4240	2500	3760	2220	3340
2460	3700	2200	3300	1970	2960	36	2810	4220	2490	3740	2210	3320
2280	3430	2030	3050	1820	2730	38	2800	4200	2480	3720	2200	3300
2100	3160	1870	2800	1670	2510	40	2780	4180	2460	3700	2180	3280
1930	2900	1710	2570	1520	2290	42	2770	4160	2450	3680	2170	3260
1760	2650	1560	2340	1390	2080	44	2760	4140	2440	3660	2160	3240
1610	2420	1420	2140	1270	1910	46	2740	4120	2420	3640	2150	3230
1480	2220	1310	1960	1160	1750	48	2730	4100	2410	3630	2130	3210
1360	2050	1200	1810	1070	1610	50	2720	4080	2400	3610	2120	3190

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
4850	7290	4400	6620	4010	6030
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3970	5950	3580	5360	3280	4920
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
962	1440	858	1290	768	1150
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
1450	2190	1300	1960	1170	1760

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
15.9	213	15.6	196	15.5	179
Area, in.²					
162		147		134	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
9430	3250	8210	2880	7190	2560
r_y, in.					
4.49		4.43		4.38	
r_x/r_y					
1.70		1.69		1.67	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W14

F_y = 50 ksi
F_u = 65 ksi

W14×						Shape	W14×					
426[h]		398[h]		370[h]		lb/ft	426[h]		398[h]		370[h]	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
3740	5620	3500	5260	3260	4900	0	2170	3260	2000	3000	1840	2760
3670	5510	3430	5160	3200	4800	6	2170	3260	2000	3000	1840	2760
3640	5470	3410	5120	3170	4770	7	2170	3260	2000	3000	1840	2760
3610	5430	3380	5080	3150	4730	8	2170	3260	2000	3000	1840	2760
3580	5380	3350	5030	3110	4680	9	2170	3260	2000	3000	1840	2760
3540	5320	3310	4970	3080	4630	10	2170	3260	2000	3000	1840	2760
3500	5260	3270	4920	3040	4570	11	2170	3260	2000	3000	1840	2760
3450	5190	3230	4850	3000	4510	12	2170	3260	2000	3000	1840	2760
3410	5120	3180	4780	2960	4450	13	2170	3260	2000	3000	1840	2760
3350	5040	3130	4710	2910	4380	14	2170	3260	2000	3000	1840	2760
3300	4960	3080	4630	2870	4310	15	2170	3260	2000	3000	1840	2760
3240	4870	3030	4550	2810	4230	16	2160	3250	1990	3000	1830	2750
3180	4790	2970	4470	2760	4150	17	2160	3240	1990	2990	1830	2740
3120	4690	2920	4380	2710	4070	18	2150	3230	1980	2980	1820	2730
3060	4600	2850	4290	2650	3980	19	2150	3230	1980	2970	1810	2730
2990	4500	2790	4200	2590	3890	20	2140	3220	1970	2960	1810	2720
2860	4290	2660	4000	2470	3710	22	2130	3200	1960	2940	1800	2700
2710	4080	2530	3800	2340	3520	24	2120	3180	1950	2920	1780	2680
2560	3850	2390	3590	2210	3320	26	2100	3160	1930	2910	1770	2660
2410	3630	2250	3380	2080	3120	28	2090	3140	1920	2890	1760	2650
2260	3400	2100	3160	1940	2920	30	2080	3120	1910	2870	1750	2630
2110	3170	1960	2950	1810	2720	32	2070	3110	1900	2850	1740	2610
1960	2950	1820	2730	1670	2520	34	2050	3090	1890	2830	1730	2590
1810	2730	1680	2530	1540	2320	36	2040	3070	1870	2820	1710	2580
1670	2510	1550	2320	1420	2130	38	2030	3050	1860	2800	1700	2560
1530	2300	1410	2130	1300	1950	40	2020	3030	1850	2780	1690	2540
1390	2090	1290	1930	1180	1770	42	2010	3010	1840	2760	1680	2520
1270	1910	1170	1760	1070	1610	44	1990	3000	1830	2750	1670	2510
1160	1750	1070	1610	980	1470	46	1980	2980	1810	2730	1660	2490
1070	1600	985	1480	900	1350	48	1970	2960	1800	2710	1640	2470
983	1480	907	1360	830	1250	50	1960	2940	1790	2690	1630	2450

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3740	5630	3500	5270	3260	4910
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3050	4570	2850	4280	2660	3990
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
703	1050	648	972	594	891
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
1080	1630	1000	1510	923	1390

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
15.3	168	15.2	158	15.1	148
Area, in.^{2}					
125		117		109	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
6600	2360	6000	2170	5440	1990
r_y, in.					
4.34		4.31		4.27	
r_x/r_y					
1.67		1.66		1.66	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W14

Shape	W14× 342[h]		W14× 311[h]		W14× 283[h]		Design	W14× 342[h]		W14× 311[h]		W14× 283[h]	
lb/ft	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips						Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
	3020	4540	2740	4110	2490	3750	0	1680	2520	1500	2260	1350	2030
	2960	4450	2680	4030	2440	3670	6	1680	2520	1500	2260	1350	2030
	2940	4420	2660	3990	2420	3640	7	1680	2520	1500	2260	1350	2030
	2910	4380	2630	3960	2400	3610	8	1680	2520	1500	2260	1350	2030
	2880	4330	2610	3920	2370	3570	9	1680	2520	1500	2260	1350	2030
	2850	4290	2580	3870	2350	3530	10	1680	2520	1500	2260	1350	2030
	2820	4230	2550	3830	2320	3480	11	1680	2520	1500	2260	1350	2030
	2780	4180	2510	3770	2290	3440	12	1680	2520	1500	2260	1350	2030
	2740	4120	2470	3720	2250	3380	13	1680	2520	1500	2260	1350	2030
	2700	4050	2430	3660	2210	3330	14	1680	2520	1500	2260	1350	2030
	2650	3980	2390	3600	2180	3270	15	1680	2520	1500	2260	1350	2030
	2600	3910	2350	3530	2140	3210	16	1670	2510	1500	2250	1350	2020
	2550	3840	2300	3460	2090	3150	17	1670	2500	1490	2240	1340	2010
	2500	3760	2260	3390	2050	3080	18	1660	2490	1490	2230	1330	2010
	2450	3680	2210	3320	2000	3010	19	1650	2490	1480	2230	1330	2000
	2390	3600	2160	3240	1960	2940	20	1650	2480	1480	2220	1320	1990
	2280	3420	2050	3080	1860	2800	22	1640	2460	1460	2200	1310	1970
	2160	3240	1940	2920	1760	2640	24	1630	2440	1450	2180	1300	1960
	2040	3060	1830	2750	1660	2490	26	1610	2430	1440	2170	1290	1940
	1910	2870	1710	2580	1550	2330	28	1600	2410	1430	2150	1280	1920
	1790	2680	1600	2400	1450	2170	30	1590	2390	1420	2130	1270	1910
	1660	2500	1490	2230	1340	2020	32	1580	2370	1410	2120	1260	1890
	1540	2310	1370	2060	1240	1860	34	1570	2360	1400	2100	1250	1870
	1420	2130	1260	1900	1140	1710	36	1560	2340	1390	2080	1230	1860
	1300	1950	1160	1740	1040	1560	38	1550	2320	1370	2070	1220	1840
	1180	1780	1050	1580	945	1420	40	1530	2310	1360	2050	1210	1820
	1070	1610	954	1430	857	1290	42	1520	2290	1350	2030	1200	1810
	979	1470	869	1310	781	1170	44	1510	2270	1340	2020	1190	1790
	896	1350	795	1200	715	1070	46	1500	2250	1330	2000	1180	1770
	823	1240	730	1100	656	986	48	1490	2240	1320	1980	1170	1760
	758	1140	673	1010	605	909	50	1480	2220	1310	1960	1160	1740

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
3020	4550	2740	4110	2490	3750
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2460	3700	2230	3340	2030	3050
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
539	809	482	723	431	646
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
843	1270	758	1140	684	1030

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
15.0	138	14.8	125	14.7	114
Area, in.^{2}					
101		91.4		83.3	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
4900	1810	4330	1610	3840	1440
r_y, in.					
4.24		4.20		4.17	
r_x/r_y					
1.65		1.64		1.63	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

W14

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

Shape	W14× 257		W14× 233		W14× 211		W14× 257		W14× 233		W14× 211	
lb/ft	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Design	Available Compressive Strength, kips						Available Flexural Strength, kip-ft					
Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	2260	3400	2050	3080	1860	2790	1220	1830	1090	1640	973	1460
6	2210	3330	2010	3010	1810	2730	1220	1830	1090	1640	973	1460
7	2200	3300	1990	2990	1800	2700	1220	1830	1090	1640	973	1460
8	2180	3270	1970	2960	1780	2680	1220	1830	1090	1640	973	1460
9	2150	3240	1950	2930	1760	2650	1220	1830	1090	1640	973	1460
10	2130	3200	1930	2900	1740	2620	1220	1830	1090	1640	973	1460
11	2100	3160	1900	2860	1720	2580	1220	1830	1090	1640	973	1460
12	2070	3110	1870	2820	1690	2550	1220	1830	1090	1640	973	1460
13	2040	3060	1840	2770	1670	2510	1220	1830	1090	1640	973	1460
14	2010	3010	1810	2730	1640	2460	1220	1830	1090	1640	973	1460
15	1970	2960	1780	2680	1610	2420	1210	1820	1090	1630	970	1460
16	1930	2900	1750	2630	1580	2370	1210	1810	1080	1620	964	1450
17	1890	2850	1710	2570	1540	2320	1200	1810	1070	1610	959	1440
18	1850	2790	1670	2520	1510	2270	1200	1800	1070	1610	954	1430
19	1810	2720	1640	2460	1480	2220	1190	1790	1060	1600	949	1430
20	1770	2660	1600	2400	1440	2160	1190	1780	1060	1590	943	1420
22	1680	2520	1510	2280	1360	2050	1170	1770	1050	1570	933	1400
24	1590	2380	1430	2150	1290	1930	1160	1750	1040	1560	922	1390
26	1490	2240	1340	2020	1210	1820	1150	1730	1030	1540	911	1370
28	1400	2100	1260	1890	1130	1700	1140	1720	1020	1530	901	1350
30	1300	1950	1170	1750	1050	1570	1130	1700	1000	1510	890	1340
32	1200	1810	1080	1620	968	1460	1120	1680	994	1490	880	1320
34	1110	1670	994	1490	890	1340	1110	1670	983	1480	869	1310
36	1020	1530	911	1370	815	1220	1100	1650	972	1460	859	1290
38	928	1400	830	1250	741	1110	1090	1630	961	1440	848	1270
40	841	1260	751	1130	670	1010	1080	1620	951	1430	837	1260
42	763	1150	681	1020	608	913	1070	1600	940	1410	827	1240
44	695	1040	621	933	554	832	1050	1590	929	1400	816	1230
46	636	956	568	854	507	761	1040	1570	918	1380	806	1210
48	584	878	522	784	465	699	1030	1550	908	1360	795	1190
50	538	809	481	723	429	644	1020	1540	897	1350	784	1180

Properties

W14×	257		233		211	
Available Strength in Tensile Yielding, kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	2260	3400	2050	3080	1860	2790
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	1840	2760	1670	2510	1510	2270
Available Strength in Shear, kips	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	387	581	342	514	308	462
Available Strength in Flexure about Y-Y Axis, kip-ft	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
	614	923	551	829	494	743
Limiting Unbraced Lengths, ft	L_p	L_r	L_p	L_r	L_p	L_r
	14.6	104	14.5	95.0	14.4	86.6
Area, in.²	75.6		68.5		62.0	
Moment of Inertia, in.⁴	I_x	I_y	I_x	I_y	I_x	I_y
	3400	1290	3010	1150	2660	1030
r_y, in.	4.13		4.10		4.07	
r_x/r_y	1.62		1.62		1.61	

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W14

Shape	W14×							W14×					
lb/ft	193		176		159			193		176		159	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	1700	2560	1550	2330	1400	2100		886	1330	798	1200	716	1080
6	1660	2500	1510	2280	1370	2050		886	1330	798	1200	716	1080
7	1650	2480	1500	2260	1350	2030		886	1330	798	1200	716	1080
8	1630	2450	1490	2240	1340	2010		886	1330	798	1200	716	1080
9	1610	2430	1470	2210	1330	1990		886	1330	798	1200	716	1080
10	1590	2400	1450	2180	1310	1970		886	1330	798	1200	716	1080
11	1570	2360	1430	2150	1290	1940		886	1330	798	1200	716	1080
12	1550	2330	1410	2120	1270	1910		886	1330	798	1200	716	1080
13	1530	2290	1390	2090	1250	1880		886	1330	798	1200	716	1080
14	1500	2250	1360	2050	1230	1850		886	1330	798	1200	716	1080
15	1470	2210	1340	2010	1210	1810		882	1330	794	1190	712	1070
16	1440	2170	1310	1970	1180	1780		877	1320	789	1190	706	1060
17	1410	2120	1280	1930	1160	1740		871	1310	784	1180	701	1050
18	1380	2080	1260	1890	1130	1700		866	1300	779	1170	696	1050
19	1350	2030	1230	1840	1100	1660		861	1290	773	1160	691	1040
20	1320	1980	1200	1800	1070	1620		856	1290	768	1150	686	1030
22	1250	1870	1130	1700	1020	1530		845	1270	758	1140	675	1010
24	1170	1770	1070	1600	957	1440		834	1250	747	1120	665	999
26	1100	1660	998	1500	896	1350		824	1240	737	1110	655	984
28	1030	1550	931	1400	835	1250		813	1220	726	1090	644	968
30	954	1430	863	1300	773	1160		803	1210	716	1080	634	953
32	881	1320	796	1200	713	1070		792	1190	706	1060	623	937
34	810	1220	730	1100	653	982		782	1170	695	1040	613	921
36	740	1110	667	1000	596	896		771	1160	685	1030	603	906
38	673	1010	605	909	540	812		760	1140	674	1010	592	890
40	608	914	546	821	487	733		750	1130	664	998	582	875
42	551	829	495	744	442	665		739	1110	653	982	572	859
44	502	755	451	678	403	605		729	1100	643	966	561	843
46	460	691	413	621	369	554		718	1080	632	951	551	828
48	422	634	379	570	339	509		708	1060	622	935	540	812
50	389	585	350	525	312	469		697	1050	612	919	530	797

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1700	2560	1550	2330	1400	2100
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1380	2080	1260	1900	1140	1710
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
276	414	252	378	224	335
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
449	675	407	611	364	548

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
14.3	79.4	14.2	73.2	14.1	66.7
Area, in.^{2}					
56.8		51.8		46.7	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
2400	931	2140	838	1900	748
r_y, in.					
4.05		4.02		4.00	
r_x/r_y					
1.60		1.60		1.60	

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W14

F_y = 50 ksi
F_u = 65 ksi

Shape	W14× 145		W14× 132		W14× 120		W14× 145		W14× 132		W14× 120	
lb/ft	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Design	Available Compressive Strength, kips						Available Flexural Strength, kip-ft					
Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	1280	1920	1160	1750	1060	1590	649	975	584	878	529	795
6	1250	1880	1130	1700	1030	1550	649	975	584	878	529	795
7	1240	1860	1120	1680	1020	1530	649	975	584	878	529	795
8	1230	1840	1110	1660	1010	1510	649	975	584	878	529	795
9	1210	1820	1090	1640	994	1490	649	975	584	878	529	795
10	1200	1800	1080	1620	980	1470	649	975	584	878	529	795
11	1180	1770	1060	1600	965	1450	649	975	584	878	529	795
12	1160	1750	1040	1570	948	1430	649	975	584	878	529	795
13	1140	1720	1020	1540	931	1400	649	975	584	878	529	795
14	1120	1690	1000	1510	912	1370	649	975	580	872	525	789
15	1100	1650	982	1480	892	1340	644	968	575	864	520	781
16	1080	1620	960	1440	872	1310	639	960	570	856	515	774
17	1060	1590	937	1410	850	1280	634	952	565	849	510	766
18	1030	1550	913	1370	828	1240	629	945	560	841	505	758
19	1010	1510	888	1330	805	1210	623	937	554	833	499	751
20	980	1470	862	1300	782	1180	618	929	549	826	494	743
22	927	1390	810	1220	734	1100	608	914	539	810	484	728
24	872	1310	756	1140	685	1030	598	899	529	795	474	712
26	816	1230	702	1060	635	955	588	883	518	779	464	697
28	759	1140	648	974	586	880	577	868	508	764	454	682
30	703	1060	594	893	537	807	567	852	498	748	443	666
32	647	973	542	814	489	735	557	837	488	733	433	651
34	593	891	491	738	443	665	547	822	477	717	423	636
36	540	812	442	664	398	598	536	806	467	702	413	620
38	489	735	397	596	357	536	526	791	457	687	403	605
40	441	663	358	538	322	484	516	776	446	671	392	590
42	400	602	325	488	292	439	506	760	436	656	382	575
44	365	548	296	445	266	400	496	745	426	640	372	559
46	334	501	271	407	244	366	485	729	416	625	362	544
48	306	461	249	374	224	336	475	714	405	609	352	529
50	282	424	229	344	206	310	465	699	395	594	341	513

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1280	1920	1160	1750	1060	1590
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1040	1560	946	1420	861	1290
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
201	302	190	284	171	257
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
332	499	282	424	254	383

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
14.1	61.7	13.3	55.8	13.2	51.9
Area, in.^{2}					
42.7		38.8		35.3	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
1710	677	1530	548	1380	495
r_y, in.					
3.98		3.76		3.74	
r_x/r_y					
1.59		1.67		1.67	

F_y = 50 ksi
F_u = 65 ksi

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W14

W14×						Shape	W14×					
109		99		90		lb/ft	109		99[f]		90[f]	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
958	1440	871	1310	793	1190	0	479	720	430	646	382	574
932	1400	848	1270	772	1160	6	479	720	430	646	382	574
923	1390	839	1260	764	1150	7	479	720	430	646	382	574
913	1370	830	1250	755	1140	8	479	720	430	646	382	574
901	1350	819	1230	745	1120	9	479	720	430	646	382	574
888	1340	807	1210	735	1100	10	479	720	430	646	382	574
874	1310	794	1190	723	1090	11	479	720	430	646	382	574
859	1290	780	1170	710	1070	12	479	720	430	646	382	574
843	1270	766	1150	697	1050	13	479	720	430	646	382	574
826	1240	750	1130	682	1030	14	475	714	427	642	382	574
808	1210	733	1100	667	1000	15	470	706	422	635	382	574
789	1190	716	1080	652	979	16	465	699	417	627	378	568
770	1160	698	1050	635	955	17	460	691	413	620	373	560
750	1130	680	1020	618	929	18	455	684	408	613	368	553
729	1100	661	994	601	903	19	450	676	403	605	363	546
708	1060	642	964	583	877	20	445	669	398	598	358	539
664	998	602	904	547	822	22	435	654	388	583	349	524
620	931	561	843	509	766	24	425	639	378	569	339	510
574	863	519	781	472	709	26	415	623	369	554	329	495
529	796	478	719	434	653	28	405	608	359	539	320	481
485	729	438	658	397	597	30	395	593	349	524	310	466
441	663	398	598	361	543	32	385	578	339	510	300	452
399	600	360	541	326	490	34	375	563	329	495	291	437
359	539	323	485	292	439	36	365	548	320	480	281	423
322	484	290	435	262	394	38	355	533	310	466	271	408
290	437	261	393	237	356	40	345	518	300	451	262	394
263	396	237	356	215	323	42	335	503	290	436	252	379
240	361	216	325	196	294	44	325	488	280	422	239	359
220	330	198	297	179	269	46	315	473	269	404	226	339
202	303	181	273	164	247	48	305	458	255	384	214	322
186	279	167	251	151	228	50	291	438	243	365	204	306

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
958	1440	871	1310	793	1190
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
780	1170	709	1060	647	970
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
150	225	138	207	123	185
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
231	348	207	311	181	273

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
13.2	48.5	13.5	45.3	15.1	42.5
Area, in.²					
32.0		29.1		26.5	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
1240	447	1110	402	999	362
r_y, in.					
3.73		3.71		3.70	
r_x/r_y					
1.67		1.66		1.66	

[f] Shape exceeds compact limit for flexure with F_y = 50 ksi.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W14

F_y = 50 ksi
F_u = 65 ksi

Shape	W14×						W14×					
lb/ft	82		74		68		82		74		68	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips						Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending: 0	719	1080	653	981	599	900	347	521	314	473	287	431
6	676	1020	614	922	562	845	347	521	314	473	287	431
7	661	993	600	902	550	826	347	521	314	473	287	431
8	644	968	585	879	536	805	347	521	314	473	287	431
9	626	940	568	854	520	782	346	519	313	471	285	429
10	606	910	550	827	503	756	340	511	308	463	280	421
11	584	878	531	797	485	729	335	503	302	455	275	413
12	562	844	510	767	466	701	329	495	297	447	270	405
13	538	809	489	735	446	671	324	487	292	439	265	398
14	514	772	467	701	426	640	318	479	286	431	259	390
15	489	735	444	667	405	608	313	471	281	423	254	382
16	464	697	421	633	384	577	308	462	276	415	249	374
17	438	659	398	598	362	544	302	454	270	407	244	366
18	413	620	375	563	341	512	297	446	265	399	239	358
19	387	582	352	529	320	480	291	438	260	390	233	351
20	362	545	329	495	299	449	286	430	254	382	228	343
22	314	472	285	428	258	388	275	414	244	366	218	327
24	267	402	243	365	219	330	264	397	233	350	207	312
26	228	343	207	311	187	281	254	381	223	334	197	296
28	197	295	179	268	161	242	243	365	212	318	186	280
30	171	257	156	234	140	211	232	349	201	302	174	262
32	150	226	137	205	123	185	221	332	188	283	161	242
34	133	200	121	182	109	164	209	313	175	263	149	224
36	119	179	108	162	97.5	147	195	293	164	246	139	209
38	107	160	96.9	146	87.5	131	183	276	154	231	131	196
40	96.3	145	87.5	131	79.0	119	173	260	145	218	123	185
42							164	246	137	206	116	175
44							156	234	130	195	110	166
46							148	223	124	186	105	157
48							141	212	118	177	99.9	150
50							135	203	113	169	95.4	143

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
719	1080	653	981	599	900
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
585	878	533	800	488	731
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
146	219	128	192	116	174
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
112	168	101	152	92.1	138

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
8.76	33.2	8.76	31.0	8.69	29.3
Area, in.²					
24.0		21.8		20.0	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
881	148	795	134	722	121
r_y, in.					
2.48		2.48		2.46	
r_x/r_y					
2.44		2.44		2.44	

Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W14

W14×						Shape	W14×					
61		53		48		lb/ft	61		53		48	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
536	805	467	702	422	634	0	254	383	217	327	196	294
503	756	421	633	380	572	6	254	383	217	327	196	294
492	739	406	610	366	551	7	254	383	216	325	194	292
479	720	389	585	351	527	8	254	383	211	317	189	284
465	699	371	557	334	502	9	253	380	206	309	184	277
450	676	351	528	316	475	10	248	372	200	301	179	269
433	651	331	497	298	447	11	243	365	195	293	174	261
416	626	310	465	279	419	12	238	358	190	285	169	254
398	599	288	433	259	390	13	233	350	185	277	164	246
380	571	267	401	240	360	14	228	343	179	270	159	239
361	543	246	369	221	331	15	223	335	174	262	154	231
342	514	225	338	202	303	16	218	328	169	254	149	223
323	485	205	308	183	276	17	213	320	164	246	143	216
304	456	185	278	166	249	18	208	313	158	238	138	208
285	428	166	250	149	224	19	203	305	153	230	133	200
266	399	150	226	134	202	20	198	298	148	222	128	193
229	345	124	186	111	167	22	188	283	137	206	116	174
195	293	104	157	93.2	140	24	178	268	123	185	103	155
166	249	88.8	133	79.4	119	26	168	253	111	167	92.7	139
143	215	76.6	115	68.5	103	28	157	236	101	153	84.3	127
125	187	66.7	100	59.7	89.7	30	143	215	93.3	140	77.4	116
110	165	58.6	88.1			32	132	198	86.4	130	71.5	107
97.0	146					34	122	184	80.4	121	66.5	99.9
86.5	130					36	114	171	75.3	113	62.1	93.4
77.7	117					38	107	160	70.8	106	58.3	87.6
70.1	105					40	100	151	66.8	100	55.0	82.6
						42	94.6	142	63.2	95.0	52.0	78.1
						44	89.5	135	60.0	90.2	49.3	74.1
						46	85.0	128	57.1	85.9	46.9	70.5
						48	81.0	122	54.5	82.0	44.8	67.3
						50	77.3	116	52.2	78.4	42.8	64.3

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
536	806	467	702	422	635
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
436	653	380	570	345	517
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
104	156	103	154	93.8	141
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
81.8	123	54.9	82.5	48.9	73.5

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
8.65	27.5	6.78	22.3	6.75	21.1
Area, in.^{2}					
17.9		15.6		14.1	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
640	107	541	57.7	484	51.4
r_y, in.					
2.45		1.92		1.91	
r_x/r_y					
2.44		3.07		3.06	

Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued) Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces W-Shapes

W14

$F_y = 50$ ksi
$F_u = 65$ ksi

Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending

Shape	W14×							W14×					
lb/ft	43[c]		38[c]		34[c]			43		38		34	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
	374	563	328	492	286	430	0	174	261	153	231	136	205
	339	510	285	428	248	373	6	174	261	151	226	133	200
	327	491	271	407	236	355	7	172	259	145	218	128	193
	312	470	253	381	222	334	8	167	251	140	210	123	185
	297	447	235	353	208	313	9	162	244	134	202	118	177
	281	422	216	325	191	287	10	158	237	129	194	113	170
	264	397	197	297	174	261	11	153	230	124	186	108	162
	247	371	178	268	157	235	12	148	222	118	178	103	155
	229	345	160	240	140	210	13	143	215	113	170	97.8	147
	212	318	142	213	124	186	14	138	208	107	161	92.8	139
	194	292	125	188	109	163	15	134	201	102	153	87.7	132
	177	267	110	165	95.4	143	16	129	193	96.6	145	81.2	122
	161	242	97.2	146	84.5	127	17	124	186	88.9	134	73.9	111
	145	218	86.7	130	75.4	113	18	119	179	81.8	123	67.7	102
	130	196	77.8	117	67.7	102	19	114	172	75.6	114	62.5	93.9
	117	177	70.2	106	61.1	91.8	20	109	165	70.3	106	57.9	87.1
	97.1	146	58.0	87.2	50.5	75.9	22	95.5	144	61.6	92.6	50.6	76.0
	81.6	123	48.8	73.3	42.4	63.8	24	84.6	127	54.8	82.4	44.8	67.4
	69.5	104					26	76.0	114	49.4	74.2	40.2	60.5
	59.9	90.1					28	68.9	104	44.9	67.5	36.5	54.9
	52.2	78.5					30	63.1	94.8	41.2	62.0	33.4	50.2
							32	58.2	87.4	38.1	57.3	30.8	46.3
							34	54.0	81.1	35.4	53.2	28.6	43.0
							36	50.4	75.7	33.1	49.8	26.7	40.1
							38	47.2	70.9	31.1	46.7	25.0	37.6
							40	44.4	66.8	29.3	44.0	23.5	35.4
							42	42.0	63.1	27.7	41.7	22.2	33.4
							44	39.8	59.8	26.3	39.5	21.1	31.7
							46	37.8	56.8	25.0	37.6	20.0	30.1
							48	36.0	54.2	23.9	35.9	19.1	28.7
							50	34.4	51.7	22.8	34.3	18.2	27.4

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
377	567	335	504	299	450
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
307	461	273	410	244	366
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
83.6	125	87.4	131	79.8	120
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
43.2	64.9	30.2	45.4	26.4	39.8

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
6.68	20.0	5.47	16.2	5.40	15.6
Area, in.^{2}					
12.6		11.2		10.0	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
428	45.2	385	26.7	340	23.3
r_y, in.					
1.89		1.55		1.53	
r_x/r_y					
3.08		3.79		3.81	

[c] Shape is slender for compression with $F_y = 50$ ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 50 ksi
F_u = 65 ksi

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W14

Shape	W14×						W14×					
lb/ft	30^c		26^c		22^c		30		26		22	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips						Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending												
0	249	375	212	319	172	259	118	177	100	151	82.8	125
6	215	323	160	241	128	192	115	172	88.6	133	71.7	108
7	204	306	145	218	115	173	110	165	83.3	125	67.0	101
8	191	288	129	194	102	153	105	158	77.9	117	62.2	93.5
9	178	268	111	167	88.3	133	101	151	72.6	109	57.5	86.3
10	165	248	93.4	140	73.3	110	96.0	144	67.2	101	52.7	79.2
11	149	224	77.4	116	60.6	91.0	91.3	137	61.9	93.0	46.2	69.4
12	134	201	65.0	97.7	50.9	76.5	86.6	130	53.9	81.0	39.9	60.0
13	119	179	55.4	83.3	43.4	65.2	82.0	123	47.5	71.5	35.0	52.7
14	105	157	47.8	71.8	37.4	56.2	77.3	116	42.5	63.8	31.1	46.8
15	91.1	137	41.6	62.5	32.6	48.9	72.1	108	38.3	57.6	28.0	42.0
16	80.1	120	36.6	55.0	28.6	43.0	64.9	97.5	34.9	52.4	25.3	38.1
17	71.0	107	32.4	48.7	25.4	38.1	58.9	88.5	32.0	48.1	23.1	34.8
18	63.3	95.1	28.9	43.4			53.8	80.9	29.5	44.4	21.3	32.0
19	56.8	85.4					49.5	74.4	27.4	41.2	19.7	29.6
20	51.3	77.1					45.8	68.8	25.6	38.5	18.3	27.5
22	42.4	63.7					39.7	59.7	22.6	34.0	16.1	24.1
24	35.6	53.5					35.1	52.7	20.2	30.4	14.3	21.5
26							31.3	47.1	18.3	27.5	12.9	19.4
28							28.3	42.6	16.7	25.1	11.7	17.6
30							25.9	38.9	15.4	23.2	10.8	16.2
32							23.8	35.7	14.3	21.5	9.94	14.9
34							22.0	33.1	13.3	20.0	9.25	13.9
36							20.5	30.8	12.5	18.8	8.64	13.0
38							19.2	28.8	11.7	17.6	8.12	12.2
40							18.0	27.1	11.1	16.7	7.65	11.5
42							17.0	25.5	10.5	15.8	7.24	10.9
44							16.1	24.2	9.98	15.0	6.87	10.3
46							15.3	22.9	9.51	14.3	6.54	9.82
48							14.5	21.8	9.08	13.6	6.23	9.37
50							13.9	20.8	8.69	13.1	5.96	8.96

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
265	398	230	346	194	292
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
216	324	188	281	158	237
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
74.5	112	70.9	106	63.0	94.5
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
22.4	33.7	13.8	20.8	11.0	16.5

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
5.26	14.9	3.81	11.0	3.67	10.4
Area, in.^{2}					
8.85		7.69		6.49	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
291	19.6	245	8.91	199	7.00
r_y, in.					
1.49		1.08		1.04	
r_x/r_y					
3.85		5.23		5.33	

[c] Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

W12

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W12×						Shape	W12×					
336[h]		305[h]		279[h]		lb/ft	336[h]		305[h]		279[h]	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
2960	4450	2680	4030	2450	3690	0	1500	2260	1340	2010	1200	1800
2870	4310	2590	3900	2370	3570	6	1500	2260	1340	2010	1200	1800
2840	4260	2560	3850	2340	3520	7	1500	2260	1340	2010	1200	1800
2800	4210	2530	3800	2310	3470	8	1500	2260	1340	2010	1200	1800
2760	4150	2490	3740	2280	3420	9	1500	2260	1340	2010	1200	1800
2710	4080	2450	3680	2240	3360	10	1500	2260	1340	2010	1200	1800
2660	4000	2400	3610	2190	3300	11	1500	2260	1340	2010	1200	1800
2610	3920	2350	3540	2150	3230	12	1500	2260	1340	2010	1200	1800
2550	3840	2300	3460	2100	3150	13	1500	2260	1340	2010	1200	1800
2490	3750	2250	3380	2050	3080	14	1500	2250	1330	2000	1190	1790
2430	3660	2190	3290	1990	3000	15	1490	2240	1330	1990	1190	1780
2370	3560	2130	3200	1940	2910	16	1490	2230	1320	1990	1180	1780
2300	3460	2070	3100	1880	2820	17	1480	2230	1320	1980	1180	1770
2230	3350	2000	3010	1820	2730	18	1480	2220	1310	1970	1170	1760
2160	3250	1940	2910	1760	2640	19	1470	2210	1310	1970	1170	1760
2090	3140	1870	2810	1700	2550	20	1470	2210	1300	1960	1160	1750
1940	2910	1730	2610	1570	2360	22	1460	2190	1290	1940	1150	1740
1790	2690	1600	2400	1440	2170	24	1450	2180	1280	1930	1150	1720
1640	2460	1460	2190	1320	1980	26	1440	2160	1270	1920	1140	1710
1490	2240	1320	1990	1190	1790	28	1430	2150	1270	1900	1130	1690
1350	2030	1190	1790	1070	1610	30	1420	2130	1260	1890	1120	1680
1210	1820	1070	1600	954	1430	32	1410	2120	1250	1870	1110	1670
1080	1620	945	1420	845	1270	34	1400	2100	1240	1860	1100	1650
959	1440	843	1270	754	1130	36	1390	2090	1230	1850	1090	1640
861	1290	757	1140	676	1020	38	1380	2080	1220	1830	1080	1630
777	1170	683	1030	610	917	40	1370	2060	1210	1820	1070	1610
705	1060	619	931	554	832	42	1360	2050	1200	1800	1060	1600
642	965	564	848	504	758	44	1350	2030	1190	1790	1060	1590
587	883	516	776	462	694	46	1340	2020	1180	1780	1050	1570
539	811	474	713	424	637	48	1330	2000	1170	1760	1040	1560
497	747	437	657	391	587	50	1320	1990	1160	1750	1030	1550

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2960	4450	2680	4030	2450	3690
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2410	3620	2180	3270	2000	2990
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
598	897	531	797	487	730
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
684	1030	609	915	549	825

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
12.3	150	12.1	137	11.9	126
Area, in.^{2}					
98.9		89.5		81.9	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
4060	1190	3550	1050	3110	937
r_y, in.					
3.47		3.42		3.38	
r_x/r_y					
1.85		1.84		1.82	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W12

Shape	W12× 252[h]		W12× 230[h]		W12× 210			W12× 252[h]		W12× 230[h]		W12× 210	
lb/ft	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
	2220	3330	2030	3050	1850	2780	0	1070	1610	963	1450	868	1310
	2140	3220	1960	2940	1790	2680	6	1070	1610	963	1450	868	1310
	2120	3180	1930	2910	1760	2650	7	1070	1610	963	1450	868	1310
	2090	3140	1910	2860	1740	2610	8	1070	1610	963	1450	868	1310
	2060	3090	1880	2820	1710	2570	9	1070	1610	963	1450	868	1310
	2020	3030	1840	2770	1680	2520	10	1070	1610	963	1450	868	1310
	1980	2970	1800	2710	1640	2470	11	1070	1610	963	1450	868	1310
	1940	2910	1760	2650	1610	2420	12	1070	1600	962	1450	867	1300
	1890	2840	1720	2590	1570	2360	13	1060	1600	957	1440	862	1300
	1840	2770	1680	2520	1530	2300	14	1060	1590	953	1430	858	1290
	1790	2700	1630	2450	1480	2230	15	1050	1580	949	1430	854	1280
	1740	2620	1580	2380	1440	2160	16	1050	1580	944	1420	849	1280
	1690	2540	1540	2310	1390	2100	17	1040	1570	940	1410	845	1270
	1630	2460	1480	2230	1350	2030	18	1040	1560	936	1410	841	1260
	1580	2370	1430	2150	1300	1950	19	1040	1560	931	1400	837	1260
	1520	2290	1380	2070	1250	1880	20	1030	1550	927	1390	832	1250
	1410	2110	1270	1910	1150	1730	22	1020	1540	918	1380	824	1240
	1290	1940	1170	1750	1050	1580	24	1010	1520	910	1370	815	1230
	1170	1760	1060	1590	955	1440	26	1010	1510	901	1350	807	1210
	1060	1590	954	1430	859	1290	28	996	1500	893	1340	798	1200
	949	1430	854	1280	767	1150	30	988	1480	884	1330	790	1190
	843	1270	756	1140	678	1020	32	979	1470	875	1320	781	1170
	746	1120	670	1010	600	902	34	970	1460	867	1300	773	1160
	666	1000	597	898	535	805	36	961	1440	858	1290	764	1150
	598	898	536	806	481	722	38	952	1430	849	1280	756	1140
	539	811	484	727	434	652	40	944	1420	841	1260	747	1120
	489	735	439	660	393	591	42	935	1400	832	1250	739	1110
	446	670	400	601	358	539	44	926	1390	823	1240	730	1100
	408	613	366	550	328	493	46	917	1380	815	1220	722	1090
	374	563	336	505	301	453	48	908	1370	806	1210	713	1070
	345	519	310	465	278	417	50	899	1350	797	1200	705	1060

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
2220	3330	2030	3050	1850	2780
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1810	2710	1650	2480	1510	2260
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
431	647	390	584	347	520
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
489	735	442	664	397	596

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
11.8	114	11.7	105	11.6	95.8
Area, in.^{2}					
74.1		67.7		61.8	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
2720	828	2420	742	2140	664
r_y, in.					
3.34		3.31		3.28	
r_x/r_y					
1.81		1.80		1.80	

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC *Specification* Section A3.1c.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W12

F_y = 50 ksi
F_u = 65 ksi

W12×						Shape	W12×					
190		170		152		lb/ft	190		170		152	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
1680	2520	1500	2250	1340	2010	0	776	1170	686	1030	606	911
1620	2430	1440	2170	1290	1940	6	776	1170	686	1030	606	911
1600	2400	1420	2140	1270	1910	7	776	1170	686	1030	606	911
1570	2360	1400	2110	1250	1880	8	776	1170	686	1030	606	911
1550	2320	1380	2070	1230	1850	9	776	1170	686	1030	606	911
1520	2280	1350	2030	1210	1810	10	776	1170	686	1030	606	911
1490	2230	1320	1990	1180	1770	11	776	1170	686	1030	606	911
1450	2180	1290	1940	1150	1730	12	774	1160	684	1030	603	907
1420	2130	1260	1900	1120	1690	13	770	1160	679	1020	599	901
1380	2070	1230	1840	1090	1640	14	765	1150	675	1020	595	895
1340	2010	1190	1790	1060	1590	15	761	1140	671	1010	591	888
1300	1950	1150	1730	1030	1540	16	757	1140	667	1000	587	882
1260	1890	1120	1680	992	1490	17	753	1130	663	997	583	876
1210	1820	1080	1620	957	1440	18	749	1130	659	990	579	870
1170	1760	1040	1560	921	1380	19	745	1120	655	984	575	864
1130	1690	997	1500	885	1330	20	740	1110	651	978	571	858
1030	1560	916	1380	811	1220	22	732	1100	642	966	563	846
944	1420	834	1250	737	1110	24	724	1090	634	953	555	833
855	1280	754	1130	665	999	26	715	1080	626	941	546	821
767	1150	675	1010	595	894	28	707	1060	618	929	538	809
684	1030	600	902	527	793	30	699	1050	610	916	530	797
603	906	528	794	464	697	32	690	1040	601	904	522	785
534	803	468	704	411	617	34	682	1020	593	892	514	772
476	716	418	628	366	551	36	674	1010	585	879	506	760
428	643	375	563	329	494	38	665	1000	577	867	498	748
386	580	338	508	297	446	40	657	987	569	855	490	736
350	526	307	461	269	405	42	648	975	560	842	481	724
319	479	280	420	245	369	44	640	962	552	830	473	711
292	439	256	384	224	337	46	632	949	544	818	465	699
268	403	235	353	206	310	48	623	937	536	805	457	687
247	371	216	325	190	285	50	615	924	527	793	449	675

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1680	2520	1500	2250	1340	2010
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
1370	2050	1220	1830	1090	1630
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
305	458	269	403	238	358
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
357	536	314	473	277	416

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
11.5	87.3	11.4	78.5	11.3	70.6
Area, in.^{2}					
56.0		50.0		44.7	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
1890	589	1650	517	1430	454
r_y, in.					
3.25		3.22		3.19	
r_x/r_y					
1.79		1.78		1.77	

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W12

W12×						Shape		W12×					
136		120		106		lb/ft		136		120		106	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
1190	1800	1050	1580	934	1400	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	534	803	464	698	409	615
1150	1730	1010	1520	898	1350		6	534	803	464	698	409	615
1130	1710	1000	1500	886	1330		7	534	803	464	698	409	615
1120	1680	984	1480	871	1310		8	534	803	464	698	409	615
1100	1650	966	1450	855	1290		9	534	803	464	698	409	615
1080	1620	947	1420	838	1260		10	534	803	464	698	409	615
1050	1580	925	1390	819	1230		11	534	803	464	698	409	615
1030	1540	903	1360	799	1200		12	531	797	460	692	405	609
1000	1500	879	1320	777	1170		13	527	791	456	686	401	603
972	1460	854	1280	755	1130		14	523	785	452	680	397	597
942	1420	828	1240	731	1100		15	519	779	449	674	393	591
912	1370	800	1200	707	1060		16	514	773	445	668	389	585
881	1320	773	1160	682	1030		17	510	767	441	662	386	580
849	1280	744	1120	656	987		18	506	761	437	656	382	574
816	1230	715	1070	631	948		19	502	755	433	650	378	568
784	1180	686	1030	604	908		20	498	749	429	644	374	562
717	1080	626	942	552	829		22	490	737	421	633	366	550
651	978	567	853	499	750		24	482	725	413	621	358	538
586	880	510	766	448	673		26	474	713	405	609	350	526
523	786	454	682	398	598		28	466	701	397	597	342	515
462	695	400	601	350	526		30	458	689	389	585	335	503
406	610	352	528	308	462		32	450	677	381	573	327	491
360	541	311	468	272	410		34	442	664	374	561	319	479
321	482	278	417	243	365		36	434	652	366	550	311	467
288	433	249	375	218	328		38	426	640	358	538	303	456
260	391	225	338	197	296		40	418	628	350	526	295	444
236	354	204	307	179	268		42	410	616	342	514	287	432
215	323	186	279	163	245		44	402	604	334	502	280	420
197	295	170	256	149	224		46	394	592	326	490	272	408
181	271	156	235	137	205		48	386	580	318	478	264	397
166	250	144	216	126	189		50	378	568	310	466	256	385

Properties

Available Strength in Tensile Yielding, kips						Limiting Unbraced Lengths, ft					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	L_p	L_r	L_p	L_r	L_p	L_r
1190	1800	1050	1580	934	1400	11.2	63.2	11.1	56.5	11.0	50.7
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips						Area, in.^{2}					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	39.9		35.2		31.2	
972	1460	858	1290	761	1140	Moment of Inertia, in.^{4}					
Available Strength in Shear, kips						I_x	I_y	I_x	I_y	I_x	I_y
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	1240	398	1070	345	933	301
212	318	186	279	157	236	r_y, in.					
Available Strength in Flexure about Y-Y Axis, kip-ft						3.16		3.13		3.11	
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	r_x/r_y					
245	368	213	320	187	282	1.77		1.76		1.76	

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W12

F_y = 50 ksi
F_u = 65 ksi

W12× 96 P_n/Ω_c ASD	96 $\phi_c P_n$ LRFD	87 P_n/Ω_c ASD	87 $\phi_c P_n$ LRFD	79 P_n/Ω_c ASD	79 $\phi_c P_n$ LRFD	Design: Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	W12× 96 M_{nx}/Ω_b ASD	96 $\phi_b M_{nx}$ LRFD	87 M_{nx}/Ω_b ASD	87 $\phi_b M_{nx}$ LRFD	79 M_{nx}/Ω_b ASD	79 $\phi_b M_{nx}$ LRFD
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
844	1270	766	1150	695	1040	0	367	551	329	495	297	446
811	1220	736	1110	667	1000	6	367	551	329	495	297	446
800	1200	726	1090	657	988	7	367	551	329	495	297	446
787	1180	714	1070	646	971	8	367	551	329	495	297	446
772	1160	700	1050	634	953	9	367	551	329	495	297	446
756	1140	685	1030	620	932	10	367	551	329	495	297	446
739	1110	670	1010	606	910	11	366	551	329	494	296	445
720	1080	653	981	590	887	12	363	545	325	488	292	439
701	1050	635	954	574	862	13	359	539	321	483	288	434
680	1020	616	925	556	836	14	355	533	317	477	285	428
659	990	596	896	538	809	15	351	528	313	471	281	422
637	957	576	865	520	781	16	347	522	310	465	277	417
614	923	555	834	501	753	17	343	516	306	460	273	411
591	888	534	802	481	723	18	339	510	302	454	270	405
567	852	512	770	462	694	19	336	504	298	448	266	400
543	816	490	737	442	664	20	332	499	294	442	262	394
495	744	446	671	402	604	22	324	487	287	431	254	382
447	672	403	605	362	544	24	316	475	279	419	247	371
401	602	360	541	323	486	26	309	464	271	408	239	360
356	535	319	480	286	430	28	301	452	264	396	232	348
312	469	280	421	250	376	30	293	441	256	385	224	337
274	413	246	370	220	331	32	285	429	248	373	217	326
243	365	218	327	195	293	34	278	417	241	362	209	314
217	326	194	292	174	261	36	270	406	233	350	202	303
195	293	174	262	156	234	38	262	394	225	339	194	292
176	264	157	237	141	212	40	255	383	218	327	186	280
159	239	143	215	128	192	42	247	371	210	316	176	264
145	218	130	196	116	175	44	239	359	201	302	167	250
133	200	119	179	106	160	46	231	378	191	287	158	238
122	183	109	164	97.8	147	48	222	333	182	274	151	227
112	169	101	151	90.1	135	50	212	319	174	262	144	217

Properties

	96 ASD	96 LRFD	87 ASD	87 LRFD	79 ASD	79 LRFD
Available Strength in Tensile Yielding, kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	844	1270	766	1150	695	1040
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	689	1030	624	936	566	848
Available Strength in Shear, kips	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	140	210	129	193	117	175
Available Strength in Flexure about Y-Y Axis, kip-ft	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
	168	253	151	227	135	204

	96		87		79	
Limiting Unbraced Lengths, ft	L_p	L_r	L_p	L_r	L_p	L_r
	10.9	46.7	10.8	43.1	10.8	39.9
Area, in.²	28.2		25.6		23.2	
Moment of Inertia, in.⁴	I_x	I_y	I_x	I_y	I_x	I_y
	833	270	740	241	662	216
r_y, in.	3.09		3.07		3.05	
r_x/r_y	1.76		1.75		1.75	

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W12

W12×						Shape	W12×					
72		65		58		lb/ft	72		65[f]		58	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
632	949	572	859	509	765	0	269	405	237	356	216	324
606	911	549	825	479	720	6	269	405	237	356	216	324
597	898	540	812	469	705	7	269	405	237	356	216	324
587	883	531	798	457	687	8	269	405	237	356	216	324
576	866	521	783	445	668	9	269	405	237	356	215	323
564	847	510	766	431	647	10	269	405	237	356	211	318
550	827	497	747	416	625	11	268	404	237	356	207	312
536	806	484	728	400	601	12	265	398	237	356	204	306
521	783	470	707	384	577	13	261	392	233	350	200	301
505	759	456	685	367	551	14	257	387	230	345	196	295
489	735	441	663	349	525	15	254	381	226	340	192	289
472	709	426	640	332	499	16	250	376	222	334	189	283
455	683	410	616	314	472	17	246	370	219	329	185	278
437	656	393	591	296	445	18	242	364	215	323	181	272
419	629	377	567	278	418	19	239	359	212	318	177	266
401	602	360	542	261	392	20	235	353	208	313	173	261
364	547	327	492	227	341	22	228	342	201	302	166	249
328	493	294	442	194	292	24	220	331	194	291	158	238
292	440	262	394	165	249	26	213	320	186	280	151	227
259	389	231	348	143	214	28	205	309	179	269	143	215
226	340	202	304	124	187	30	198	297	172	259	135	203
199	299	178	267	109	164	32	190	286	165	248	125	188
176	265	157	236	96.7	145	34	183	275	158	237	116	174
157	236	140	211	86.3	130	36	176	264	149	224	108	163
141	212	126	189	77.4	116	38	167	251	139	209	102	153
127	191	114	171	69.9	105	40	157	236	130	196	95.7	144
115	173	103	155			42	148	223	123	185	90.5	136
105	158	93.9	141			44	140	211	116	175	85.8	129
96.2	145	85.9	129			46	133	200	110	166	81.6	123
88.3	133	78.9	119			48	127	191	105	158	77.8	117
81.4	122	72.7	109			50	121	182	100	150	74.3	112

Properties

W12×72		W12×65		W12×58	
Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
632	950	572	860	509	765
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
514	770	465	697	416	624
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
106	159	94.4	142	87.8	132
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
123	185	107	161	81.1	122

W12×72		W12×65		W12×58	
Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
10.7	37.5	11.9	35.1	8.87	29.8
Area, in.^{2}					
21.1		19.1		17.0	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
597	195	533	174	475	107
r_y, in.					
3.04		3.02		2.51	
r_x/r_y					
1.75		1.75		2.10	

[f] Shape exceeds compact limit for flexure with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W12

F_y = 50 ksi
F_u = 65 ksi

Shape	W12×							W12×					
lb/ft	53		50		45			53		50		45	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
	467	702	437	657	392	589	0	194	292	179	270	160	241
	439	660	396	595	355	534	6	194	292	179	270	160	241
	429	646	382	574	342	515	7	194	292	179	269	160	240
	419	629	367	551	329	494	8	194	292	175	263	156	234
	407	611	350	526	313	471	9	193	291	171	257	152	229
	394	592	332	500	297	447	10	190	285	167	251	148	223
	380	571	314	472	281	422	11	186	280	163	245	144	217
	365	549	295	443	263	396	12	183	274	159	239	141	211
	350	526	275	413	246	369	13	179	269	155	233	137	206
	334	502	255	384	228	343	14	175	263	151	227	133	200
	318	478	236	355	210	316	15	172	258	147	221	129	194
	301	453	217	326	193	290	16	168	252	143	215	125	188
	285	428	198	298	176	265	17	164	247	139	209	121	183
	268	403	180	270	160	240	18	161	241	135	203	118	177
	252	378	162	244	144	216	19	157	236	131	197	114	171
	235	354	146	220	130	195	20	153	230	127	191	110	165
	204	307	121	182	107	161	22	146	219	119	179	102	154
	174	261	102	153	90.3	136	24	139	208	111	167	92.0	138
	148	223	86.6	130	76.9	116	26	131	197	101	151	83.1	125
	128	192	74.7	112	66.3	99.7	28	124	186	91.9	138	75.8	114
	111	167	65.0	97.8	57.8	86.8	30	114	171	84.6	127	69.7	105
	97.8	147	57.2	85.9	50.8	76.3	32	105	157	78.5	118	64.5	97.0
	86.6	130					34	97.2	146	73.2	110	60.1	90.3
	77.3	116					36	90.6	136	68.6	103	56.2	84.5
	69.4	104					38	84.9	128	64.5	97.0	52.9	79.4
	62.6	94.1					40	79.8	120	60.9	91.6	49.9	75.0
							42	75.4	113	57.7	86.8	47.2	71.0
							44	71.4	107	54.9	82.5	44.8	67.4
							46	67.9	102	52.3	78.6	42.7	64.2
							48	64.7	97.2	49.9	75.1	40.7	61.2
							50	61.7	92.8	47.8	71.8	39.0	58.6

Properties

Available Strength in Tensile Yielding, kips							Limiting Unbraced Lengths, ft					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$		L_p	L_r	L_p	L_r	L_p	L_r
467	702	437	657	392	590		8.76	28.2	6.92	23.8	6.89	22.4
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips							Area, in.^{2}					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$		15.6		14.6		13.1	
380	570	358	536	319	479		Moment of Inertia, in.^{4}					
Available Strength in Shear, kips							I_x	I_y	I_x	I_y	I_x	I_y
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$		425	95.8	391	56.3	348	50.0
83.5	125	90.3	135	81.1	122		r_y, in.					
Available Strength in Flexure about Y-Y Axis, kip-ft							2.48		1.96		1.95	
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$		r_x/r_y					
72.6	109	53.1	79.9	47.4	71.3		2.11		2.64		2.64	

Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 50 ksi
F_u = 65 ksi

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W12

W12×						Shape		W12×					
40		35[c]		30[c]		lb/ft		40		35		30	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips								Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD			ASD	LRFD	ASD	LRFD	ASD	LRFD
350	526	308	463	254	382	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	142	214	128	192	108	162
317	476	263	395	220	330		6	142	214	125	188	105	158
305	459	248	373	209	314		7	142	213	121	182	101	152
293	440	232	349	196	295		8	138	207	117	175	97.2	146
279	420	215	324	182	273		9	134	202	112	169	93.3	140
265	398	198	297	167	251		10	131	196	108	163	89.4	134
250	375	180	271	152	228		11	127	191	104	156	85.5	128
234	352	163	245	137	205		12	123	185	99.6	150	81.5	123
218	328	146	219	122	183		13	120	180	95.3	143	77.6	117
202	304	129	194	108	162		14	116	174	91.0	137	73.7	111
187	281	113	170	94.2	142		15	112	169	86.7	130	69.8	105
171	257	99.6	150	82.8	124		16	109	163	82.4	124	64.8	97.4
156	235	88.2	133	73.3	110		17	105	158	77.2	116	59.1	88.9
142	213	78.7	118	65.4	98.3		18	101	152	71.3	107	54.4	81.7
127	191	70.6	106	58.7	88.3		19	97.7	147	66.1	99.4	50.3	75.5
115	173	63.7	95.8	53.0	79.7		20	94.1	141	61.7	92.7	46.7	70.2
95.0	143	52.7	79.2	43.8	65.8		22	85.0	128	54.4	81.7	40.9	61.5
79.8	120	44.3	66.5	36.8	55.3		24	75.6	114	48.6	73.1	36.4	54.7
68.0	102						26	68.0	102	44.0	66.1	32.8	49.3
58.6	88.1						28	61.9	93.0	40.2	60.4	29.8	44.8
51.1	76.8						30	56.8	85.3	37.0	55.6	27.3	41.1
44.9	67.5						32	52.4	78.8	34.3	51.5	25.3	38.0
							34	48.7	73.3	31.9	48.0	23.5	35.3
							36	45.6	68.5	29.9	44.9	21.9	33.0
							38	42.8	64.3	28.1	42.3	20.6	31.0
							40	40.3	60.6	26.6	39.9	19.4	29.2
							42	38.1	57.3	25.2	37.8	18.4	27.6
							44	36.2	54.3	23.9	35.9	17.4	26.2
							46	34.4	51.7	22.8	34.2	16.6	24.9
							48	32.8	49.3	21.7	32.7	15.8	23.8
							50	31.4	47.1	20.8	31.3	15.1	22.7

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
350	527	308	464	263	396
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
285	428	251	377	214	321
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
70.2	105	75.0	113	64.0	95.9
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
41.9	63.0	28.7	43.1	23.9	35.9

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
6.85	21.1	5.44	16.6	5.37	15.6
Area, in.²					
11.7		10.3		8.79	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
307	44.1	285	24.5	238	20.3
r_y, in.					
1.94		1.54		1.52	
r_x/r_y					
2.64		3.41		3.43	

[c] Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W12

F_y = 50 ksi
F_u = 65 ksi

Shape	W12× 26[c]		W12× 22[c]		W12× 19[c]		W12× 26		W12× 22		W12× 19	
lb/ft	Available Compressive Strength, kips						Available Flexural Strength, kip-ft					
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	215	324	185	278	154	231	92.8	140	73.1	110	61.6	92.6
6	186	279	115	172	95.2	143	90.4	136	59.0	88.7	48.4	72.7
7	176	265	94.7	142	77.7	117	86.8	130	54.3	81.7	44.1	66.3
8	166	249	76.0	114	61.4	92.3	83.2	125	49.7	74.6	39.8	59.8
9	154	232	60.0	90.3	48.5	72.9	79.6	120	45.0	67.6	34.5	51.9
10	143	215	48.6	73.1	39.3	59.0	76.0	114	38.6	58.0	29.2	43.9
11	131	197	40.2	60.4	32.5	48.8	72.4	109	33.5	50.4	25.2	37.9
12	118	177	33.8	50.8	27.3	41.0	68.7	103	29.6	44.5	22.1	33.3
13	105	158	28.8	43.3	23.2	34.9	65.1	97.9	26.5	39.8	19.7	29.6
14	92.7	139	24.8	37.3			61.5	92.5	24.0	36.0	17.7	26.7
15	80.9	122					57.5	86.5	21.9	32.9	16.1	24.2
16	71.1	107					51.9	78.0	20.1	30.3	14.8	22.2
17	63.0	94.7					47.2	70.9	18.6	28.0	13.6	20.5
18	56.2	84.5					43.2	64.9	17.4	26.1	12.7	19.0
19	50.4	75.8					39.8	59.9	16.2	24.4	11.8	17.8
20	45.5	68.4					36.9	55.5	15.3	23.0	11.1	16.7
22	37.6	56.5					32.1	48.3	13.6	20.5	9.86	14.8
24	31.6	47.5					28.4	42.8	12.3	18.5	8.88	13.3
26							25.5	38.3	11.3	16.9	8.08	12.2
28							23.1	34.7	10.4	15.6	7.42	11.2
30							21.1	31.8	9.60	14.4	6.86	10.3
32							19.5	29.3	8.95	13.4	6.39	9.60
34							18.0	27.1	8.38	12.6	5.97	8.97
36							16.8	25.3	7.88	11.8	5.61	8.43
38							15.8	23.7	7.44	11.2	5.29	7.95
40							14.8	22.3	7.04	10.6	5.00	7.52
42							14.0	21.0	6.69	10.1	4.75	7.14
44							13.3	19.9	6.37	9.58	4.52	6.79
46							12.6	18.9	6.08	9.14	4.31	6.48
48							12.0	18.0	5.82	8.74	4.12	6.19
50							11.5	17.2	5.58	8.38	3.95	5.93

Properties

	W12×26		W12×22		W12×19	
Available Strength in Tensile Yielding, kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	229	344	194	292	167	251
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	187	280	158	237	136	204
Available Strength in Shear, kips	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	56.1	84.2	64.0	95.9	57.3	86.0
Available Strength in Flexure about Y-Y Axis, kip-ft	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
	20.4	30.6	9.13	13.7	7.44	11.2

	W12×26		W12×22		W12×19	
Limiting Unbraced Lengths, ft	L_p	L_r	L_p	L_r	L_p	L_r
	5.33	14.9	3.00	9.13	2.90	8.61
Area, in.²	7.65		6.48		5.57	
Moment of Inertia, in.⁴	I_x	I_y	I_x	I_y	I_x	I_y
	204	17.3	156	4.66	130	3.76
r_y, in.	1.51		0.848		0.822	
r_x/r_y	3.42		5.79		5.86	

[c] Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 50 ksi
F_u = 65 ksi

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W12–W10

Shape	W12×				W10×			W12×				W10×	
lb/ft	16[c]		14[c]		112			16		14[v]		112	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
	126	189	107	161	985	1480	0	50.1	75.4	43.4	65.3	367	551
	74.7	112	62.0	93.2	934	1400	6	37.7	56.6	32.0	48.0	367	551
	59.5	89.4	50.1	75.4	917	1380	7	33.9	50.9	28.5	42.9	367	551
	45.9	69.0	38.5	57.8	897	1350	8	30.0	45.1	24.4	36.7	367	551
	36.3	54.5	30.4	45.7	875	1310	9	24.7	37.1	19.9	29.9	367	551
	29.4	44.2	24.6	37.0	851	1280	10	20.7	31.1	16.7	25.0	365	549
	24.3	36.5	20.3	30.6	825	1240	11	17.8	26.7	14.2	21.4	363	545
	20.4	30.7	17.1	25.7	798	1200	12	15.5	23.3	12.4	18.6	360	541
					769	1160	13	13.8	20.7	10.9	16.4	357	537
					739	1110	14	12.3	18.5	9.76	14.7	355	533
					708	1060	15	11.2	16.8	8.80	13.2	352	529
					677	1020	16	10.2	15.3	8.01	12.0	349	525
					645	969	17	9.37	14.1	7.35	11.0	347	521
					613	921	18	8.67	13.0	6.78	10.2	344	517
					580	872	19	8.07	12.1	6.30	9.46	341	513
					548	824	20	7.55	11.3	5.87	8.83	338	509
					485	728	22	6.68	10.0	5.18	7.79	333	501
					423	636	24	6.00	9.02	4.64	6.97	328	493
					365	548	26	5.45	8.18	4.20	6.31	322	485
					315	473	28	4.99	7.50	3.83	5.76	317	476
					274	412	30	4.60	6.92	3.53	5.31	312	468
					241	362	32	4.27	6.42	3.27	4.92	306	460
					213	321	34	3.99	6.00	3.05	4.59	301	452
					190	286	36	3.74	5.62	2.86	4.30	296	444
					171	257	38	3.52	5.30	2.69	4.04	290	436
					154	232	40	3.33	5.01	2.54	3.82	285	428
					140	210	42	3.16	4.75	2.41	3.61	279	420
					127	191	44	3.00	4.51	2.29	3.43	274	412
							46	2.86	4.30	2.18	3.27	269	404
							48	2.74	4.11	2.08	3.12	263	396
							50	2.62	3.94	1.99	2.99	258	388

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
141	212	125	187	985	1480
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
115	172	101	152	803	1200
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
52.8	79.2	42.8	64.3	172	258
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
5.63	8.46	4.74	7.13	173	260

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
2.73	8.05	2.66	7.73	9.47	64.1
Area, in.²					
4.71		4.16		32.9	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
103	2.82	88.6	2.36	716	236
r_y, in.					
0.773		0.753		2.68	
r_x/r_y					
6.04		6.14		1.74	

[c] Shape is slender for compression with F_y = 50 ksi.
[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with F_y = 50 ksi; therefore, ϕ_v = 0.90 and Ω_v = 1.67.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W10

F_y = 50 ksi
F_u = 65 ksi

Shape	W10×100		W10×88		W10×77		W10×100		W10×88		W10×77	
lb/ft	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Design	Available Compressive Strength, kips						Available Flexural Strength, kip-ft					
Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	877	1320	778	1170	680	1020	324	488	282	424	244	366
6	831	1250	737	1110	643	966	324	488	282	424	244	366
7	815	1230	722	1090	630	946	324	488	282	424	244	366
8	797	1200	706	1060	615	925	324	488	282	424	244	366
9	777	1170	688	1030	599	900	324	488	282	424	244	366
10	755	1130	669	1000	582	874	323	485	280	421	241	363
11	732	1100	647	973	563	846	320	481	277	417	239	359
12	707	1060	625	940	543	816	317	477	275	413	236	355
13	681	1020	602	905	522	785	315	473	272	409	234	351
14	654	983	578	868	501	753	312	469	270	405	231	347
15	626	941	553	831	479	720	309	465	267	401	228	343
16	598	898	527	792	456	686	307	461	264	397	226	339
17	569	855	501	754	433	651	304	457	262	393	223	336
18	540	811	475	714	410	617	301	453	259	389	221	332
19	511	767	449	675	387	582	299	449	256	386	218	328
20	482	724	423	636	365	548	296	445	254	382	215	324
22	425	638	373	560	320	481	291	437	249	374	210	316
24	370	556	324	487	277	417	286	429	243	366	205	308
26	318	478	278	417	237	356	280	421	238	358	200	301
28	274	412	239	360	204	307	275	413	233	350	195	293
30	239	359	209	313	178	267	270	405	228	342	190	285
32	210	315	183	276	156	235	264	397	222	334	184	277
34	186	279	162	244	139	208	259	389	217	326	179	269
36	166	249	145	218	124	186	254	381	212	319	174	262
38	149	224	130	195	111	167	248	373	207	311	169	254
40	134	202	117	176	100	150	243	365	201	303	164	246
42	122	183	106	160	90.8	136	238	357	196	295	158	238
44	111	167					232	349	191	287	153	230
46							227	341	186	279	147	222
48							222	333	180	271	141	212
50							217	325	175	263	135	203

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
877	1320	778	1170	680	1020
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
715	1070	634	951	553	829
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
151	226	131	196	112	169
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
152	229	132	199	115	172

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.36	57.9	9.29	51.2	9.18	45.3
Area, in.^{2}					
29.3		26.0		22.7	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
623	207	534	179	455	154
r_y, in.					
2.65		2.63		2.60	
r_x/r_y					
1.74		1.73		1.73	

Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 50 ksi
F_u = 65 ksi

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W10

Shape W10×	68		60		54			68		60		54	
lb/ft	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Design	Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
	596	895	530	796	473	711	0	213	320	186	280	166	250
	563	846	500	752	446	671	6	213	320	186	280	166	250
	552	829	490	737	437	657	7	213	320	186	280	166	250
	539	810	479	719	427	642	8	213	320	186	280	166	250
	525	789	466	700	415	624	9	213	320	186	280	166	250
	509	765	452	679	403	605	10	211	317	184	276	164	246
	493	741	437	657	389	585	11	208	313	181	272	161	242
	475	714	421	633	375	564	12	206	309	179	269	159	239
	457	687	405	608	361	542	13	203	305	176	265	156	235
	438	658	388	583	345	519	14	200	301	174	261	154	231
	419	629	370	556	330	495	15	198	297	171	257	151	227
	399	599	352	530	314	471	16	195	294	169	253	149	224
	379	569	334	502	297	447	17	193	290	166	250	146	220
	358	539	316	475	281	422	18	190	286	164	246	144	216
	338	508	298	448	265	398	19	188	282	161	242	141	212
	318	478	280	421	249	374	20	185	278	159	238	139	209
	279	419	245	368	217	327	22	180	270	153	231	134	201
	241	363	212	318	188	282	24	175	263	148	223	129	194
	206	310	181	271	160	240	26	170	255	143	215	124	186
	178	267	156	234	138	207	28	165	247	138	208	119	179
	155	233	136	204	120	180	30	159	240	133	200	114	171
	136	205	119	179	106	159	32	154	232	128	193	109	164
	121	181	106	159	93.5	141	34	149	224	123	185	103	155
	108	162	94.2	142	83.4	125	36	144	217	118	177	96.7	145
	96.5	145	84.5	127	74.8	112	38	139	209	112	168	91.0	137
	87.1	131	76.3	115	67.6	102	40	134	201	105	159	85.8	129
	79.0	119	69.2	104	61.3	92.1	42	128	192	100	150	81.3	122
							44	121	182	95.0	143	77.2	116
							46	116	174	90.5	136	73.5	110
							48	111	166	86.5	130	70.2	105
							50	106	159	82.8	124	67.1	101

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
596	896	530	797	473	711
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
484	726	432	648	387	580
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
97.8	147	85.7	129	74.7	112
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
100	150	87.3	131	78.1	117

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
9.15	40.6	9.08	36.6	9.04	33.6
Area, in.²					
19.9		17.7		15.8	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
394	134	341	116	303	103
r_y, in.					
2.59		2.57		2.56	
r_x/r_y					
1.71		1.71		1.71	

Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W10

F_y = 50 ksi
F_u = 65 ksi

Shape	W10×							W10×					
lb/ft	49		45		39			49		45		39	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	431	648	398	598	344	517	0	151	227	137	206	117	176
	407	611	363	545	313	470	6	151	227	137	206	117	176
	398	598	350	527	302	454	7	151	227	137	206	117	175
	388	584	337	507	290	436	8	151	227	135	202	114	172
	378	568	322	485	277	416	9	151	226	132	198	112	168
	366	550	307	461	263	396	10	148	223	129	195	109	164
	354	532	291	437	249	374	11	146	219	127	191	107	160
	341	512	274	411	234	352	12	143	215	124	187	104	157
	327	492	256	385	219	329	13	141	212	122	183	102	153
	313	471	239	359	203	306	14	138	208	119	179	99.2	149
	299	449	222	333	188	283	15	136	204	117	175	96.7	145
	284	427	204	307	173	260	16	134	201	114	171	94.2	142
	269	404	188	282	158	238	17	131	197	111	167	91.7	138
	254	382	171	257	144	217	18	129	193	109	164	89.2	134
	239	360	155	234	130	196	19	126	190	106	160	86.7	130
	224	337	140	211	118	177	20	124	186	104	156	84.2	127
	196	294	116	174	97.2	146	22	119	179	98.5	148	79.2	119
	168	253	97.4	146	81.7	123	24	114	171	93.3	140	74.1	111
	143	216	83.0	125	69.6	105	26	109	164	88.1	132	67.4	101
	124	186	71.5	108	60.0	90.2	28	104	157	81.9	123	61.7	92.8
	108	162	62.3	93.7	52.3	78.6	30	99.3	149	75.7	114	56.9	85.5
	94.7	142	54.8	82.3	46.0	69.1	32	93.9	141	70.3	106	52.8	79.4
	83.9	126					34	87.3	131	65.8	98.8	49.3	74.0
	74.8	112					36	81.6	123	61.7	92.8	46.2	69.4
	67.2	101					38	76.7	115	58.2	87.5	43.5	65.4
	60.6	91.1					40	72.3	109	55.0	82.7	41.1	61.7
	55.0	82.6					42	68.4	103	52.2	78.5	38.9	58.5
							44	64.9	97.5	49.7	74.7	37.0	55.6
							46	61.7	92.8	47.4	71.2	35.3	53.0
							48	58.9	88.5	45.3	68.1	33.7	50.7
							50	56.3	84.6	43.4	65.2	32.3	48.5

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
431	648	398	599	344	518
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
351	527	324	487	280	421
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
68.0	102	70.7	106	62.5	93.7
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
70.6	106	50.6	76.1	42.9	64.5

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
8.97	31.6	7.10	26.9	6.99	24.2
Area, in.^{2}					
14.4		13.3		11.5	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
272	93.4	248	53.4	209	45.0
r_y, in.					
2.54		2.01		1.98	
r_x/r_y					
1.71		2.15		2.16	

Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W10

Shape	W10× 33		W10× 30		W10× 26			W10× 33		W10× 30		W10× 26	
lb/ft	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
	291	437	265	398	228	342	0	96.8	146	91.3	137	78.1	117
	263	395	216	325	186	279	6	96.8	146	87.7	132	74.6	112
	253	381	201	302	172	259	7	96.5	145	84.7	127	71.7	108
	243	365	185	278	158	238	8	94.1	141	81.6	123	68.8	103
	232	348	168	253	144	216	9	91.7	138	78.5	118	65.9	99.1
	220	330	151	227	129	194	10	89.3	134	75.4	113	63.0	94.7
	207	311	134	202	114	172	11	86.9	131	72.4	109	60.1	90.4
	194	292	118	177	100	151	12	84.5	127	69.3	104	57.2	86.0
	181	272	102	154	86.9	131	13	82.1	123	66.2	99.5	54.3	81.6
	168	253	88.4	133	75.0	113	14	79.7	120	63.1	94.9	51.4	77.3
	155	233	77.0	116	65.3	98.1	15	77.3	116	60.0	90.2	48.3	72.7
	142	214	67.7	102	57.4	86.3	16	74.9	113	57.0	85.6	44.2	66.4
	130	195	59.9	90.1	50.8	76.4	17	72.5	109	52.8	79.3	40.7	61.2
	117	177	53.5	80.3	45.3	68.2	18	70.2	105	49.0	73.7	37.7	56.7
	106	159	48.0	72.1	40.7	61.2	19	67.8	102	45.8	68.9	35.1	52.8
	95.4	143	43.3	65.1	36.7	55.2	20	65.4	98.3	43.0	64.6	32.9	49.5
	78.8	118	35.8	53.8	30.4	45.6	22	60.2	90.5	38.3	57.6	29.2	43.9
	66.2	99.5					24	53.7	80.8	34.6	51.9	26.2	39.4
	56.4	84.8					26	48.5	72.9	31.5	47.3	23.8	35.8
	48.7	73.1					28	44.2	66.5	28.9	43.5	21.9	32.9
	42.4	63.7					30	40.6	61.1	26.8	40.3	20.2	30.3
	37.3	56.0					32	37.6	56.5	24.9	37.5	18.8	28.2
							34	35.0	52.6	23.3	35.1	17.5	26.3
							36	32.8	49.2	21.9	32.9	16.4	24.7
							38	30.8	46.3	20.7	31.1	15.5	23.3
							40	29.0	43.7	19.6	29.4	14.7	22.0
							42	27.5	41.3	18.6	27.9	13.9	20.9
							44	26.1	39.2	17.7	26.6	13.2	19.9
							46	24.9	37.4	16.9	25.4	12.6	18.9
							48	23.7	35.6	16.1	24.3	12.0	18.1
							50	22.7	34.1	15.5	23.2	11.5	17.3

Properties

W10×	33		30		26	
Available Strength in Tensile Yielding, kips						
	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	291	437	265	398	228	342
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips						
	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	237	355	215	323	186	278
Available Strength in Shear, kips						
	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	56.4	84.7	63.0	94.5	53.6	80.3
Available Strength in Flexure about Y-Y Axis, kip-ft						
	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
	34.9	52.5	22.1	33.2	18.7	28.1

W10×	33		30		26	
Limiting Unbraced Lengths, ft						
	L_p	L_r	L_p	L_r	L_p	L_r
	6.85	21.8	4.84	16.1	4.80	14.9
Area, in.^{2}	9.71		8.84		7.61	
Moment of Inertia, in.^{4}						
	I_x	I_y	I_x	I_y	I_x	I_y
	171	36.6	170	16.7	144	14.1
r_y, in.	1.94		1.37		1.36	
r_x/r_y	2.16		3.20		3.20	

Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W10

F_y = 50 ksi
F_u = 65 ksi

W10×						Shape	W10×					
22[c]		19		17[c]		lb/ft	22		19		17	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
193	290	168	253	148	223	0	64.9	97.5	53.9	81.0	46.7	70.1
157	236	102	154	87.9	132	6	61.4	92.2	44.7	67.1	37.7	56.6
145	218	85.6	129	72.5	109	7	58.7	88.2	41.5	62.4	34.7	52.2
133	200	69.6	105	58.1	87.3	8	56.0	84.2	38.3	57.6	31.7	47.7
120	180	55.3	83.1	45.9	69.0	9	53.3	80.1	35.1	52.8	28.8	43.2
107	161	44.8	67.3	37.2	55.9	10	50.6	76.1	31.5	47.4	24.7	37.2
94.6	142	37.0	55.7	30.7	46.2	11	48.0	72.1	27.5	41.4	21.5	32.3
82.5	124	31.1	46.8	25.8	38.8	12	45.3	68.0	24.4	36.7	19.0	28.5
70.9	107	26.5	39.9	22.0	33.1	13	42.6	64.0	21.9	33.0	17.0	25.5
61.1	91.9	22.9	34.4	19.0	28.5	14	39.5	59.3	19.9	30.0	15.4	23.1
53.3	80.0					15	35.6	53.5	18.3	27.4	14.0	21.1
46.8	70.4					16	32.4	48.7	16.8	25.3	12.9	19.4
41.5	62.3					17	29.7	44.7	15.6	23.5	12.0	18.0
37.0	55.6					18	27.4	41.2	14.6	21.9	11.1	16.7
33.2	49.9					19	25.5	38.3	13.7	20.6	10.4	15.7
30.0	45.0					20	23.8	35.7	12.9	19.4	9.81	14.7
24.8	37.2					22	21.0	31.5	11.5	17.4	8.76	13.2
						24	18.8	28.2	10.5	15.7	7.92	11.9
						26	17.0	25.5	9.57	14.4	7.23	10.9
						28	15.5	23.3	8.82	13.3	6.66	10.0
						30	14.3	21.5	8.19	12.3	6.17	9.27
						32	13.2	19.9	7.64	11.5	5.75	8.64
						34	12.3	18.5	7.16	10.8	5.39	8.09
						36	11.6	17.4	6.74	10.1	5.06	7.61
						38	10.9	16.3	6.36	9.56	4.78	7.19
						40	10.3	15.4	6.03	9.06	4.53	6.81
						42	9.73	14.6	5.73	8.61	4.30	6.46
						44	9.24	13.9	5.46	8.21	4.10	6.16
						46	8.80	13.2	5.21	7.84	3.91	5.88
						48	8.41	12.6	4.99	7.50	3.74	5.62
						50	8.05	12.1	4.78	7.19	3.58	5.39

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
194	292	168	253	149	225
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
158	237	137	206	122	182
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
49.0	73.4	51.0	76.5	48.5	72.7
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
15.2	22.9	8.36	12.6	6.99	10.5

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
4.70	13.8	3.09	9.73	2.98	9.16
Area, in.²					
6.49		5.62		4.99	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
118	11.4	96.3	4.29	81.9	3.56
r_y, in.					
1.33		0.874		0.845	
r_x/r_y					
3.21		4.74		4.79	

[c] Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W10–W8

Shape	W10×				W8×			W10×				W8×	
lb/ft	15[c]		12[c]		67			15		12[f]		67	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
	129	194	97.2	146	590	886	0	39.9	60.0	31.2	46.9	175	263
	74.1	111	57.3	86.1	542	815	6	31.3	47.0	23.9	35.9	175	263
	60.1	90.4	45.9	69.0	526	790	7	28.5	42.9	21.5	32.3	175	263
	47.2	70.9	35.6	53.5	508	763	8	25.8	38.7	19.2	28.8	174	262
	37.3	56.0	28.1	42.3	488	733	9	22.4	33.7	15.7	23.7	172	259
	30.2	45.4	22.8	34.2	467	701	10	19.0	28.6	13.2	19.9	171	256
	25.0	37.5	18.8	28.3	444	668	11	16.5	24.7	11.4	17.1	169	254
	21.0	31.5	15.8	23.8	421	633	12	14.5	21.8	9.92	14.9	167	251
	17.9	26.9	13.5	20.3	397	597	13	12.9	19.4	8.79	13.2	165	249
					373	560	14	11.6	17.5	7.88	11.8	164	246
					348	523	15	10.6	15.9	7.13	10.7	162	243
					324	487	16	9.73	14.6	6.51	9.79	160	241
					300	450	17	9.00	13.5	5.99	9.00	158	238
					276	415	18	8.36	12.6	5.54	8.33	157	236
					253	381	19	7.81	11.7	5.16	7.76	155	233
					231	347	20	7.33	11.0	4.83	7.25	153	230
					191	287	22	6.54	9.82	4.27	6.43	150	225
					160	241	24	5.90	8.86	3.84	5.77	146	220
					137	205	26	5.37	8.08	3.48	5.24	143	215
					118	177	28	4.94	7.42	3.19	4.80	139	210
					103	154	30	4.57	6.87	2.94	4.43	136	204
					90.3	136	32	4.26	6.40	2.73	4.11	133	199
					79.9	120	34	3.98	5.98	2.55	3.84	129	194
							36	3.74	5.62	2.39	3.60	126	189
							38	3.53	5.31	2.26	3.39	122	184
							40	3.34	5.02	2.13	3.20	119	178
							42	3.17	4.77	2.02	3.04	115	173
							44	3.02	4.54	1.92	2.89	112	168
							46	2.88	4.33	1.83	2.75	108	163
							48	2.76	4.14	1.75	2.63	105	157
							50	2.64	3.97	1.68	2.52	100	151

Properties

W10×15[c]		W10×12[c]		W8×67	
Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
132	198	106	159	590	887
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
108	161	86.5	130	481	722
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
46.0	68.9	37.5	56.3	103	154
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
5.74	8.63	4.30	6.46	81.6	123

W10×15		W10×12[f]		W8×67	
Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
2.86	8.61	2.87	8.05	7.49	47.6
Area, in.^{2}					
4.41		3.54		19.7	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
68.9	2.89	53.8	2.18	272	88.6
r_y, in.					
0.810		0.785		2.12	
r_x/r_y					
4.88		4.97		1.75	

[c] Shape is slender for compression with F_y = 50 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

W8

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W8×						Shape	W8×					
58		48		40		lb/ft	58		48		40	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
512	769	422	634	350	526	0	149	224	122	184	99.3	149
470	706	387	581	320	481	6	149	224	122	184	99.3	149
455	685	375	563	309	465	7	149	224	122	184	99.3	149
439	660	361	543	298	448	8	148	223	121	182	98.0	147
422	634	347	521	285	429	9	146	220	119	180	96.4	145
403	606	331	497	272	409	10	145	218	118	177	94.7	142
384	576	314	473	258	388	11	143	215	116	175	93.1	140
363	546	297	447	243	366	12	141	212	114	172	91.4	137
342	514	280	421	228	343	13	140	210	113	169	89.8	135
321	482	262	394	213	321	14	138	207	111	167	88.2	132
299	450	244	367	198	298	15	136	205	109	164	86.5	130
278	418	226	340	183	275	16	135	202	108	162	84.9	128
257	386	209	314	169	253	17	133	200	106	159	83.2	125
236	355	192	288	154	232	18	131	197	104	157	81.6	123
216	325	175	264	141	211	19	129	195	103	154	80.0	120
197	296	159	239	127	191	20	128	192	101	152	78.3	118
163	244	132	198	105	158	22	124	187	97.7	147	75.0	113
137	205	111	166	88.2	133	24	121	182	94.3	142	71.7	108
116	175	94.2	142	75.2	113	26	117	177	90.9	137	68.5	103
100	151	81.2	122	64.8	97.4	28	114	171	87.6	132	65.2	98.0
87.5	131	70.7	106	56.5	84.9	30	111	166	84.2	127	61.8	92.9
76.9	116	62.2	93.5	49.6	74.6	32	107	161	80.9	122	57.6	86.5
68.1	102	55.1	82.8	44.0	66.1	34	104	156	77.5	117	53.9	81.0
						36	100	151	73.7	111	50.6	76.1
						38	97	146	69.6	105	47.8	71.8
						40	93.6	141	65.9	99.1	45.2	67.9
						42	89.9	135	62.7	94.2	42.9	64.5
						44	85.7	129	59.7	89.7	40.9	61.4
						46	81.8	123	57.0	85.7	39.0	58.6
						48	78.3	118	54.5	82.0	37.3	56.0
						50	75.1	113	52.3	78.6	35.7	53.7

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
512	770	422	635	350	527
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
416	624	345	517	285	428
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
89.3	134	68.0	102	59.4	89.1
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
69.6	105	57.1	85.9	46.2	69.4

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
7.42	41.6	7.35	35.2	7.21	29.9
Area, in.^{2}					
17.1		14.1		11.7	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
228	75.1	184	60.9	146	49.1
r_y, in.					
2.10		2.08		2.04	
r_x/r_y					
1.74		1.74		1.73	

Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W8

Shape	W8×							W8×					
lb/ft	35		31		28			35		31[f]		28	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	308	463	273	411	247	371	0	86.6	130	75.8	114	67.9	102
	281	423	249	374	214	321	6	86.6	130	75.8	114	67.4	101
	272	409	241	362	203	305	7	86.6	130	75.8	114	65.7	98.8
	262	394	232	348	191	287	8	85.2	128	74.5	112	64.1	96.3
	251	377	222	333	178	268	9	83.6	126	72.9	110	62.4	93.8
	239	359	211	317	165	249	10	82.0	123	71.3	107	60.7	91.3
	226	340	200	301	152	228	11	80.4	121	69.8	105	59.1	88.8
	213	321	189	283	139	208	12	78.8	118	68.2	102	57.4	86.3
	200	301	177	266	125	188	13	77.1	116	66.6	100	55.8	83.8
	187	281	165	248	113	169	14	75.5	114	65.0	97.7	54.1	81.3
	174	261	153	230	100	151	15	73.9	111	63.5	95.4	52.4	78.8
	160	241	141	212	88.3	133	16	72.3	109	61.9	93.0	50.8	76.3
	147	221	130	195	78.2	118	17	70.7	106	60.3	90.6	49.1	73.8
	135	203	118	178	69.8	105	18	69.1	104	58.7	88.3	47.4	71.3
	123	184	108	162	62.6	94.1	19	67.4	101	57.1	85.9	45.8	68.8
	111	166	97.2	146	56.5	84.9	20	65.8	98.9	55.6	83.5	44.1	66.3
	91.5	138	80.3	121	46.7	70.2	22	62.6	94.1	52.4	78.8	40.1	60.3
	76.9	116	67.5	101	39.2	59.0	24	59.3	89.2	49.3	74.0	36.3	54.5
	65.5	98.5	57.5	86.5	33.4	50.2	26	56.1	84.3	45.3	68.1	33.1	49.8
	56.5	84.9	49.6	74.5			28	52.2	78.5	41.4	62.3	30.5	45.8
	49.2	74.0	43.2	64.9			30	48.2	72.5	38.2	57.4	28.2	42.4
	43.3	65.0	38.0	57.1			32	44.8	67.4	35.5	53.3	26.3	39.5
							34	41.9	63.0	33.1	49.8	24.6	37.0
							36	39.3	59.1	31.0	46.7	23.1	34.8
							38	37.1	55.7	29.2	43.9	21.8	32.8
							40	35.1	52.7	27.6	41.5	20.7	31.1
							42	33.3	50.0	26.2	39.3	19.6	29.5
							44	31.7	47.6	24.9	37.4	18.7	28.1
							46	30.2	45.4	23.7	35.7	17.8	26.8
							48	28.9	43.4	22.7	34.1	17.1	25.7
							50	27.6	41.5	21.7	32.6	16.4	24.6

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
308	464	273	411	247	371
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
251	377	223	334	201	302
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
50.3	75.5	45.6	68.4	45.9	68.9
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
40.2	60.4	35.1	52.8	25.2	37.9

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
7.17	27.0	7.18	24.8	5.72	21.0
Area, in.^{2}					
10.3		9.13		8.25	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
127	42.6	110	37.1	98.0	21.7
r_y, in.					
2.03		2.02		1.62	
r_x/r_y					
1.73		1.72		2.13	

[f] Shape exceeds compact limit for flexure with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W8

F_y = 50 ksi
F_u = 65 ksi

Shape	W8×24		W8×21		W8×18		W8×24		W8×21		W8×18	
lb/ft	24		21		18		24		21		18	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips						Available Flexural Strength, kip-ft					
Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	212	319	184	277	157	237	57.6	86.6	50.9	76.5	42.4	63.8
6	183	275	145	218	123	184	57.1	85.9	48.0	72.2	39.5	59.4
7	174	261	133	200	112	168	55.5	83.5	46.2	69.4	37.8	56.8
8	163	246	121	181	101	152	53.9	81.1	44.3	66.6	36.1	54.2
9	153	229	108	162	89.6	135	52.3	78.7	42.5	63.9	34.3	51.6
10	141	212	95.0	143	78.5	118	50.7	76.3	40.7	61.1	32.6	49.0
11	130	195	82.7	124	67.8	102	49.2	73.9	38.8	58.3	30.9	46.4
12	118	178	70.9	107	57.7	86.7	47.6	71.5	37.0	55.5	29.1	43.8
13	107	160	60.4	90.8	49.2	73.9	46.0	69.1	35.1	52.8	27.4	41.2
14	95.6	144	52.1	78.3	42.4	63.7	44.4	66.7	33.3	50.0	25.2	37.8
15	85.0	128	45.4	68.2	36.9	55.5	42.8	64.3	31.2	46.9	22.9	34.4
16	74.8	112	39.9	59.9	32.4	48.8	41.2	61.9	28.7	43.2	21.0	31.5
17	66.3	99.6	35.3	53.1	28.7	43.2	39.6	59.5	26.6	40.0	19.4	29.1
18	59.1	88.9	31.5	47.4	25.6	38.5	38.0	57.1	24.8	37.3	18.0	27.1
19	53.1	79.8	28.3	42.5	23.0	34.6	36.3	54.5	23.2	34.9	16.8	25.3
20	47.9	72.0	25.5	38.4	20.8	31.2	34.0	51.1	21.8	32.8	15.8	23.7
22	39.6	59.5					30.3	45.5	19.5	29.3	14.0	21.1
24	33.3	50.0					27.2	41.0	17.6	26.5	12.6	19.0
26	28.3	42.6					24.8	37.3	16.1	24.2	11.5	17.3
28							22.8	34.2	14.8	22.3	10.6	15.9
30							21.1	31.6	13.7	20.6	9.79	14.7
32							19.6	29.4	12.8	19.2	9.11	13.7
34							18.3	27.5	12.0	18.0	8.52	12.8
36							17.2	25.8	11.3	17.0	8.00	12.0
38							16.2	24.4	10.6	16.0	7.55	11.3
40							15.3	23.1	10.1	15.2	7.14	10.7
42							14.6	21.9	9.58	14.4	6.78	10.2
44							13.9	20.8	9.12	13.7	6.45	9.69
46							13.2	19.9	8.71	13.1	6.15	9.25
48							12.6	19.0	8.33	12.5	5.88	8.84
50							12.1	18.2	7.98	12.0	5.64	8.47

Properties

	W8×24		W8×21		W8×18	
Available Strength in Tensile Yielding, kips						
	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	212	319	184	277	157	237
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips						
	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	173	259	150	225	128	193
Available Strength in Shear, kips						
	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	38.9	58.3	41.4	62.1	37.4	56.2
Available Strength in Flexure about Y-Y Axis, kip-ft						
	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
	21.4	32.1	14.2	21.3	11.6	17.5

	W8×24		W8×21		W8×18	
Limiting Unbraced Lengths, ft						
	L_p	L_r	L_p	L_r	L_p	L_r
	5.69	18.9	4.45	14.8	4.34	13.5
Area, in.^{2}	7.08		6.16		5.26	
Moment of Inertia, in.^{4}						
	I_x	I_y	I_x	I_y	I_x	I_y
	82.7	18.3	75.3	9.77	61.9	7.97
r_y, in.	1.61		1.26		1.23	
r_x/r_y	2.12		2.77		2.79	

Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 50 ksi
F_u = 65 ksi

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W8

W8×						Shape	W8×					
15		13		10[c]		lb/ft	15		13		10[f]	
P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	Design	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
133	200	115	173	85.7	129	0	33.9	51.0	28.4	42.8	21.9	32.9
81.1	122	67.4	101	51.9	77.9	6	28.4	42.6	23.1	34.7	17.5	26.3
67.9	102	55.6	83.6	42.7	64.2	7	26.5	39.8	21.3	32.1	16.0	24.0
55.2	83.0	44.5	66.9	34.1	51.3	8	24.5	36.9	19.6	29.4	14.4	21.7
43.9	66.0	35.2	52.9	27.0	40.5	9	22.6	34.0	17.8	26.7	12.5	18.7
35.6	53.5	28.5	42.8	21.9	32.8	10	20.7	31.1	15.5	23.3	10.5	15.8
29.4	44.2	23.5	35.4	18.1	27.1	11	18.2	27.3	13.5	20.3	9.11	13.7
24.7	37.1	19.8	29.7	15.2	22.8	12	16.2	24.3	12.0	18.0	8.00	12.0
21.0	31.6	16.9	25.3	12.9	19.4	13	14.6	21.9	10.7	16.1	7.12	10.7
18.1	27.3	14.5	21.8	11.1	16.8	14	13.3	20.0	9.75	14.6	6.41	9.64
						15	12.2	18.3	8.92	13.4	5.83	8.76
						16	11.3	16.9	8.23	12.4	5.35	8.03
						17	10.5	15.7	7.63	11.5	4.93	7.42
						18	9.79	14.7	7.12	10.7	4.58	6.89
						19	9.19	13.8	6.67	10.0	4.28	6.43
						20	8.67	13.0	6.28	9.44	4.01	6.03
						22	7.78	11.7	5.63	8.46	3.57	5.36
						24	7.06	10.6	5.10	7.66	3.22	4.83
						26	6.47	9.72	4.66	7.00	2.93	4.40
						28	5.97	8.97	4.29	6.45	2.69	4.04
						30	5.54	8.33	3.98	5.99	2.49	3.74
						32	5.17	7.78	3.72	5.58	2.31	3.48
						34	4.85	7.29	3.48	5.23	2.16	3.25
						36	4.57	6.87	3.28	4.92	2.03	3.06
						38	4.32	6.49	3.09	4.65	1.92	2.88
						40	4.09	6.15	2.93	4.41	1.81	2.73
						42	3.89	5.85	2.79	4.19	1.72	2.59
						44	3.71	5.57	2.65	3.99	1.64	2.46
						46	3.54	5.32	2.53	3.81	1.56	2.35
						48	3.39	5.10	2.42	3.64	1.49	2.25
						50	3.25	4.89	2.32	3.49	1.43	2.15

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
133	200	115	173	88.6	133
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
108	162	93.6	140	72.2	108
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
39.7	59.6	36.8	55.1	26.8	40.2
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
6.66	10.0	5.36	8.06	4.07	6.12

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
3.09	10.1	2.98	9.27	3.14	8.52
Area, in.²					
4.44		3.84		2.96	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
48.0	3.41	39.6	2.73	30.8	2.09
r_y, in.					
0.876		0.843		0.841	
r_x/r_y					
3.76		3.81		3.83	

[c] Shape is slender for compression with F_y = 50 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W6

F_y = 50 ksi
F_u = 65 ksi

Shape	W6× 25		W6× 20		W6× 16		Shape	W6× 25		W6× 20		W6× 16	
lb/ft	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	lb/ft	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
Design	Available Compressive Strength, kips						Design	Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending													
0	220	330	176	264	142	213	0	47.2	70.9	37.4	56.3	29.2	43.9
6	187	280	149	223	94.6	142	6	46.5	69.9	36.7	55.2	26.4	39.7
7	176	264	140	210	81.7	123	7	45.6	68.5	35.8	53.8	25.4	38.1
8	164	247	130	196	69.0	104	8	44.6	67.0	34.8	52.3	24.3	36.5
9	152	228	120	181	57.0	85.7	9	43.6	65.5	33.8	50.9	23.2	34.9
10	139	209	110	165	46.3	69.5	10	42.6	64.1	32.9	49.4	22.2	33.3
11	127	190	99.8	150	38.2	57.5	11	41.6	62.6	31.9	47.9	21.1	31.7
12	114	171	89.6	135	32.1	48.3	12	40.7	61.1	30.9	46.5	20.0	30.1
13	102	153	79.7	120	27.4	41.1	13	39.7	59.6	30.0	45.0	19.0	28.5
14	90.0	135	70.2	106	23.6	35.5	14	38.7	58.2	29.0	43.6	17.9	26.9
15	78.7	118	61.3	92.1	20.6	30.9	15	37.7	56.7	28.0	42.1	16.6	24.9
16	69.1	104	53.9	80.9	18.1	27.2	16	36.7	55.2	27.1	40.7	15.4	23.2
17	61.2	92.1	47.7	71.7			17	35.8	53.8	26.1	39.2	14.5	21.7
18	54.6	82.1	42.5	64.0			18	34.8	52.3	25.1	37.7	13.6	20.4
19	49.0	73.7	38.2	57.4			19	33.8	50.8	24.1	36.3	12.8	19.2
20	44.3	66.5	34.5	51.8			20	32.8	49.3	23.1	34.7	12.1	18.2
22	36.6	55.0	28.5	42.8			22	30.9	46.4	20.6	31.0	10.9	16.5
24	30.7	46.2	23.9	36.0			24	28.8	43.3	18.7	28.1	9.99	15
26							26	26.4	39.7	17.1	25.7	9.19	13.8
28							28	24.4	36.7	15.8	23.7	8.5	12.8
30							30	22.7	34.1	14.6	22.0	7.92	11.9
32							32	21.2	31.8	13.6	20.5	7.41	11.1
34							34	19.9	29.9	12.8	19.2	6.96	10.5
36							36	18.7	28.1	12.0	18.1	6.57	9.87
38							38	17.7	26.6	11.4	17.1	6.21	9.34
40							40	16.8	25.2	10.8	16.2	5.9	8.86
42							42	15.9	24.0	10.2	15.4	5.61	8.43
44							44	15.2	22.8	9.73	14.6	5.35	8.04
46							46	14.5	21.8	9.30	14.0	5.12	7.69
48							48	13.9	20.9	8.89	13.4	4.9	7.36
50							50	13.3	20.0	8.53	12.8	4.7	7.07

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
220	330	176	264	142	213
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
179	269	143	215	116	174
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
40.8	61.2	32.2	48.4	32.7	49.0
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
21.4	32.1	16.8	25.2	8.46	12.7

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
5.37	23.7	5.30	19.8	3.42	14.1
Area, in.^{2}					
7.34		5.87		4.74	
Moment of Inertia, in.^{4}					
I_x	I_y	I_x	I_y	I_x	I_y
53.4	17.1	41.4	13.3	32.1	4.43
r_y, in.					
1.52		1.50		0.967	
r_x/r_y					
1.78		1.77		2.69	

Note: Heavy line indicates L_c/r equal to or greater than 200.

F_y = 50 ksi
F_u = 65 ksi

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W6

Design: Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	W6×15 P_n/Ω_c ASD	W6×15 $\phi_c P_n$ LRFD	W6×12 P_n/Ω_c ASD	W6×12 $\phi_c P_n$ LRFD	W6×9 P_n/Ω_c ASD	W6×9 $\phi_c P_n$ LRFD	W6×15[f] M_{nx}/Ω_b ASD	W6×15[f] $\phi_b M_{nx}$ LRFD	W6×12 M_{nx}/Ω_b ASD	W6×12 $\phi_b M_{nx}$ LRFD	W6×9[f] M_{nx}/Ω_b ASD	W6×9[f] $\phi_b M_{nx}$ LRFD
	Available Compressive Strength, kips						Available Flexural Strength, kip-ft					
0	133	199	106	160	80.2	121	25.4	38.1	20.7	31.1	15.5	23.4
6	111	166	67.8	102	50.5	75.9	25.4	38.1	18.0	27.0	13.0	19.6
7	104	156	57.6	86.6	42.7	64.2	25.3	38.0	17.0	25.5	12.2	18.3
8	96.3	145	47.8	71.8	35.2	53.0	24.4	36.7	16.0	24.0	11.3	16.9
9	88.4	133	38.6	57.9	28.3	42.5	23.5	35.4	15.0	22.5	10.4	15.6
10	80.4	121	31.2	46.9	22.9	34.4	22.7	34.1	14.0	21.0	9.36	14.1
11	72.4	109	25.8	38.8	18.9	28.5	21.8	32.7	13.0	19.5	8.17	12.3
12	64.5	96.9	21.7	32.6	15.9	23.9	20.9	31.4	11.7	17.6	7.25	10.9
13	56.9	85.5	18.5	27.8	13.6	20.4	20.0	30.1	10.6	15.9	6.52	9.80
14	49.6	74.6	15.9	23.9	11.7	17.6	19.2	28.8	9.70	14.6	5.92	8.90
15	43.2	64.9	13.9	20.9	10.2	15.3	18.3	27.5	8.94	13.4	5.42	8.15
16	38.0	57.1					17.4	26.2	8.29	12.5	5.01	7.52
17	33.6	50.6					16.3	24.5	7.73	11.6	4.65	6.98
18	30.0	45.1					15.1	22.7	7.24	10.9	4.34	6.52
19	26.9	40.5					14.1	21.2	6.82	10.2	4.07	6.12
20	24.3	36.5					13.2	19.9	6.44	9.68	3.83	5.76
22	20.1	30.2					11.7	17.7	5.80	8.71	3.43	5.16
24	16.9	25.4					10.6	15.9	5.28	7.93	3.11	4.68
26							9.62	14.5	4.84	7.28	2.85	4.28
28							8.83	13.3	4.48	6.73	2.63	3.95
30							8.17	12.3	4.16	6.26	2.44	3.66
32							7.59	11.4	3.89	5.85	2.27	3.42
34							7.10	10.7	3.65	5.49	2.13	3.20
36							6.67	10.0	3.44	5.17	2.00	3.01
38							6.29	9.45	3.25	4.89	1.89	2.85
40							5.95	8.94	3.09	4.64	1.79	2.70
42							5.64	8.48	2.94	4.41	1.71	2.56
44							5.37	8.07	2.80	4.21	1.62	2.44
46							5.12	7.70	2.67	4.02	1.55	2.33
48							4.90	7.36	2.56	3.85	1.48	2.23
50							4.69	7.05	2.46	3.69	1.42	2.14

Properties

	W6×15		W6×12		W6×9	
Available Strength in Tensile Yielding, kips						
	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	133	199	106	160	80.2	121
Available Strength in Tensile Rupture (A_e = 0.75A_g), kips						
	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	108	162	86.5	130	65.3	98.0
Available Strength in Shear, kips						
	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
	27.6	41.3	27.7	41.6	20.1	30.1
Available Strength in Flexure about Y-Y Axis, kip-ft						
	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
	10.8	16.3	5.79	8.70	4.29	6.45

	W6×15[f]		W6×12		W6×9[f]	
Limiting Unbraced Lengths, ft						
	L_p	L_r	L_p	L_r	L_p	L_r
	6.91	16.5	3.24	11.2	3.20	9.75
Area, in.^{2}	4.43		3.55		2.68	
Moment of Inertia, in.^{4}						
	I_x	I_y	I_x	I_y	I_x	I_y
	29.1	9.32	22.1	2.99	16.4	2.20
r_y, in.	1.45		0.918		0.905	
r_x/r_y	1.77		2.71		2.73	

[f] Shape exceeds compact limit for flexure with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

W6–W5

F_y = 50 ksi
F_u = 65 ksi

Shape	W6×		W5×					W6×		W5×			
lb/ft	8.5		19		16			8.5[f]		19		16	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$		M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$	M_{nx}/Ω_b	$\phi_b M_{nx}$
	Available Compressive Strength, kips							Available Flexural Strength, kip-ft					
	ASD	LRFD	ASD	LRFD	ASD	LRFD	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD
	75.4	113	166	250	141	212	0	14.0	21.0	28.9	43.5	24.0	36.1
	46.8	70.3	132	199	111	167	6	11.9	17.8	28.1	42.2	23.1	34.7
	39.3	59.1	121	183	102	153	7	11.0	16.6	27.4	41.3	22.5	33.8
	32.2	48.4	110	166	92.2	139	8	10.2	15.3	26.8	40.3	21.9	33.0
	25.7	38.7	98.9	149	82.4	124	9	9.32	14.0	26.2	39.4	21.3	32.1
	20.8	31.3	87.5	132	72.7	109	10	8.23	12.4	25.6	38.5	20.7	31.2
	17.2	25.9	76.5	115	63.2	95.0	11	7.18	10.8	25.0	37.6	20.1	30.3
	14.5	21.7	66.0	99.2	54.2	81.5	12	6.36	9.56	24.4	36.7	19.6	29.4
	12.3	18.5	56.3	84.6	46.2	69.4	13	5.71	8.58	23.8	35.8	19.0	28.5
	10.6	16.0	48.5	72.9	39.8	59.9	14	5.18	7.78	23.2	34.9	18.4	27.6
			42.3	63.5	34.7	52.1	15	4.74	7.12	22.6	34.0	17.8	26.7
			37.1	55.8	30.5	45.8	16	4.37	6.56	22.0	33.1	17.2	25.8
			32.9	49.5	27.0	40.6	17	4.05	6.09	21.4	32.2	16.6	24.9
			29.3	44.1	24.1	36.2	18	3.78	5.68	20.8	31.3	16.0	24.1
			26.3	39.6	21.6	32.5	19	3.54	5.32	20.2	30.4	15.4	23.2
			23.8	35.7	19.5	29.3	20	3.33	5.01	19.6	29.5	14.8	22.2
							22	2.98	4.49	18.4	27.7	13.3	20.0
							24	2.70	4.06	17.0	25.6	12.1	18.3
							26	2.47	3.71	15.6	23.5	11.2	16.8
							28	2.28	3.42	14.5	21.8	10.3	15.5
							30	2.11	3.17	13.5	20.3	9.61	14.4
							32	1.97	2.96	12.6	19.0	8.99	13.5
							34	1.85	2.77	11.9	17.8	8.44	12.7
							36	1.74	2.61	11.2	16.8	7.96	12.0
							38	1.64	2.46	10.6	15.9	7.53	11.3
							40	1.55	2.34	10.0	15.1	7.14	10.7
							42	1.48	2.22	9.56	14.4	6.80	10.2
							44	1.41	2.11	9.12	13.7	6.48	9.74
							46	1.34	2.02	8.72	13.1	6.19	9.31
							48	1.28	1.93	8.35	12.5	5.93	8.92
							50	1.23	1.85	8.01	12.0	5.69	8.55

Properties

Available Strength in Tensile Yielding, kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
75.4	113	166	250	141	212
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips					
P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
61.4	92.1	136	203	115	172
Available Strength in Shear, kips					
V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$	V_n/Ω_v	$\phi_v V_n$
19.8	29.7	27.8	41.7	24.0	36.1
Available Strength in Flexure about Y-Y Axis, kip-ft					
M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$	M_{ny}/Ω_b	$\phi_b M_{ny}$
3.76	5.65	13.8	20.7	11.4	17.2

Limiting Unbraced Lengths, ft					
L_p	L_r	L_p	L_r	L_p	L_r
3.55	9.49	4.52	23.0	4.45	19.8
Area, in.²					
2.52		5.56		4.71	
Moment of Inertia, in.⁴					
I_x	I_y	I_x	I_y	I_x	I_y
14.9	1.99	26.3	9.13	21.4	7.51
r_y, in.					
0.890		1.28		1.26	
r_x/r_y					
2.73		1.70		1.69	

[f] Shape exceeds compact limit for flexure with F_y = 50 ksi.
Note: Heavy line indicates L_c/r equal to or greater than 200.

Table 6-2 (continued)
Available Strength for Members Subject to Axial, Shear, Flexural and Combined Forces
W-Shapes

F_y = 50 ksi
F_u = 65 ksi

W4

W4×		Shape		W4×	
13		lb/ft		13	
P_n/Ω_c	$\phi_c P_n$	Design		M_{nx}/Ω_b	$\phi_b M_{nx}$
Available Compressive Strength, kips				Available Flexural Strength, kip-ft	
ASD	LRFD			ASD	LRFD
115	172	Effective length, L_c, ft, with respect to least radius of gyration, r_y, or unbraced length, L_b, ft, for X-X axis bending	0	15.7	23.6
78.5	118		6	14.7	22.1
68.5	103		7	14.3	21.5
58.5	87.9		8	13.9	20.9
48.9	73.5		9	13.5	20.3
40.0	60.1		10	13.1	19.7
33.0	49.7		11	12.7	19.2
27.8	41.7		12	12.4	18.6
23.7	35.6		13	12.0	18.0
20.4	30.7		14	11.6	17.4
17.8	26.7		15	11.2	16.8
15.6	23.5		16	10.8	16.2
			17	10.4	15.6
			18	10.0	15.0
			19	9.62	14.5
			20	9.14	13.7
			22	8.28	12.4
			24	7.57	11.4
			26	6.97	10.5
			28	6.46	9.72
			30	6.03	9.06
			32	5.64	8.48
			34	5.31	7.98
			36	5.01	7.53
			38	4.74	7.13
			40	4.50	6.77
			42	4.29	6.44
			44	4.09	6.15
			46	3.91	5.88
			48	3.75	5.63
			50	3.59	5.40

Properties

Available Strength in Tensile Yielding, kips		Limiting Unbraced Lengths, ft	
P_n/Ω_t	$\phi_t P_n$	L_p	L_r
115	172	3.53	19.2
Available Strength in Tensile Rupture ($A_e = 0.75A_g$), kips		Area, in.^{2}	
P_n/Ω_t	$\phi_t P_n$	3.83	
93.3	140	Moment of Inertia, in.^{4}	
Available Strength in Shear, kips		I_x	I_y
V_n/Ω_v	$\phi_v V_n$	11.3	3.86
23.3	34.9	r_y, in.	
Available Strength in Flexure about Y-Y Axis, kip-ft		1.00	
M_{ny}/Ω_b	$\phi_b M_{ny}$	r_x/r_y	
7.29	11.0	1.72	

Note: Heavy line indicates L_c/r equal to or greater than 200.

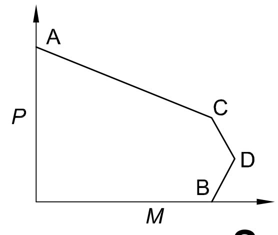
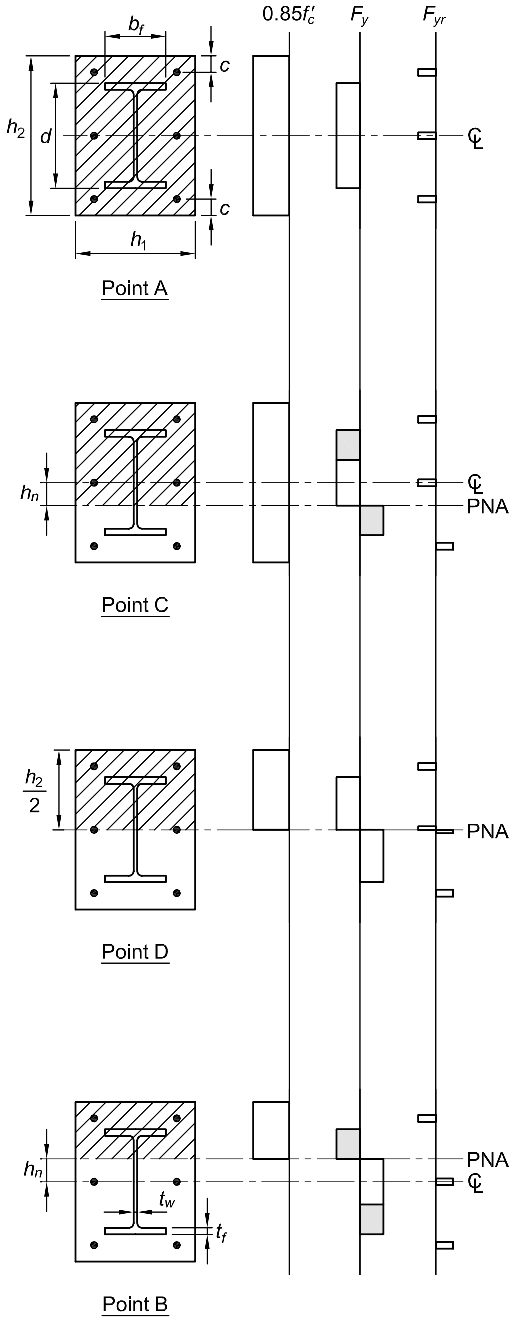
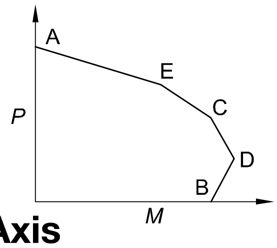


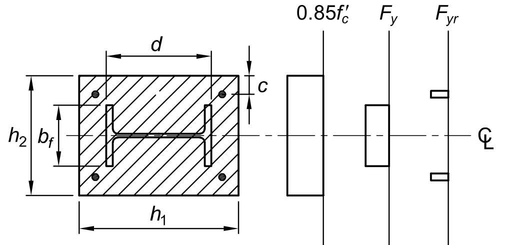
Table 6-3a
Cross-Section Strength for Rectangular Encased W-Shapes
Subject to Flexure about the Major Axis

Section	Stress Distribution	Pt.	Defining Equation
		A	$P_A = F_y A_s + F_{yr} A_{sr} + 0.85 f'_c A_c$ $M_A = 0$ A_s = area of steel shape A_{sr} = area of all continuous reinforcing bars $A_c = h_1 h_2 - A_s - A_{sr}$
		C	$P_C = 0.85 f'_c A_c$ $M_C = M_B$
		D	$P_D = \dfrac{0.85 f'_c A_c}{2}$ $M_D = F_y Z_s + F_{yr} Z_r + 0.85 f'_c \left(\dfrac{Z_c}{2}\right)$ Z_s = full x-axis plastic section modulus of steel shape A_{srs} = area of continuous reinforcing bars at the centerline $Z_r = (A_{sr} - A_{srs})\left(\dfrac{h_2}{2} - c\right)$ $Z_c = \dfrac{h_1 h_2^2}{4} - Z_s - Z_r$
		B	$P_B = 0$ $M_B = M_D - F_y Z_{sn} - 0.85 f'_c \left(\dfrac{Z_{cn}}{2}\right)$ $Z_{cn} = h_1 h_n^2 - Z_{sn}$ For h_n below the flange $\left(h_n \le \dfrac{d}{2} - t_f\right)$ $h_n = \dfrac{0.85 f'_c (A_c + A_{srs}) - 2F_{yr} A_{srs}}{2[0.85 f'_c (h_1 - t_w) + 2F_y t_w]}$ $Z_{sn} = t_w h_n^2$ For h_n within the flange $\left(\dfrac{d}{2} - t_f < h_n \le \dfrac{d}{2}\right)$ $h_n = \dfrac{0.85 f'_c (A_c + A_s - d b_f + A_{srs}) - 2F_y (A_s - d b_f) - 2F_{yr} A_{srs}}{2[0.85 f'_c (h_1 - b_f) + 2F_y b_f]}$ $Z_{sn} = Z_s - b_f \left(\dfrac{d}{2} - h_n\right)\left(\dfrac{d}{2} + h_n\right)$ For h_n above the flange $\left(h_n > \dfrac{d}{2}\right)$ $h_n = \dfrac{0.85 f'_c (A_c + A_s + A_{srs}) - 2F_y A_s - 2F_{yr} A_{srs}}{2(0.85 f'_c h_1)}$ $Z_{sn} = Z_s$

F_y = specified minimum yield stress of steel shape.
F_{yr} = specified minimum yield stress of reinforcing steel.

Table 6-3b
Cross-Section Strength for Rectangular Encased W-Shapes
Subject to Flexure about the Minor Axis



<table>
<tr><th>Section</th><th>Stress Distribution</th><th>Pt.</th><th>Defining Equation</th></tr>
<tr><td colspan="2">


Point A</td><td>A</td><td>

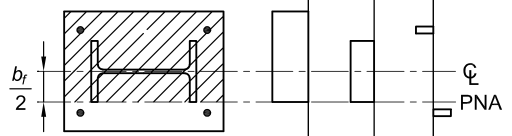
$P_A = F_y A_s + F_{yr} A_{sr} + 0.85 f'_c A_c$

$M_A = 0$

A_s = area of steel shape

A_{sr} = area of all continuous reinforcing bars

$A_c = h_1 h_2 - A_s - A_{sr}$

</td></tr>
<tr><td colspan="2">


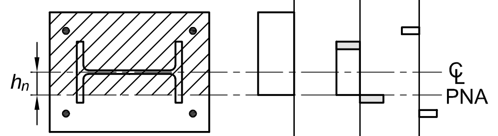
Point E</td><td>E</td><td>

$P_E = F_y A_s + 0.85 f'_c \left[A_c - \frac{h_1}{2}(h_2 - b_f) + \frac{A_{sr}}{2}\right]$

$M_E = M_D - Z_{sE} F_y - 0.85 f'_c \left(\frac{Z_{cE}}{2}\right)$

$Z_{sE} = Z_s$ = plastic section modulus of the steel shape about the y-axis

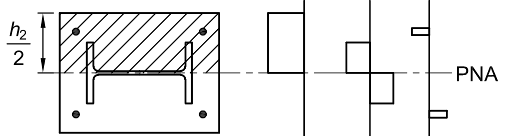
$Z_{cE} = \frac{h_1 b_f^2}{4} - Z_{sE}$

</td></tr>
<tr><td colspan="2">


Point C</td><td>C</td><td>

$P_C = 0.85 f'_c A_c$

$M_C = M_B$

</td></tr>
<tr><td colspan="2">


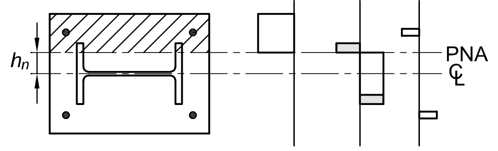
Point D</td><td>D</td><td>

$P_D = \frac{0.85 f'_c A_c}{2}$

$M_D = F_y Z_s + F_{yr} Z_r + 0.85 f'_c \left(\frac{Z_{cn}}{2}\right)$

$Z_r = A_{sr}\left(\frac{h_2}{2} - c\right)$

$Z_{cn} = \frac{h_1 h_n^2}{4} - Z_s - Z_r$

</td></tr>
<tr><td colspan="2">


Point B</td><td>B</td><td>

$P_B = 0$

$M_B = M_D - F_y Z_{sn} - 0.85 f'_c \left(\frac{Z_{cn}}{2}\right)$

$Z_{cn} = h_1 h_n^2 - Z_{sn}$

For h_n within the flange $\left(\frac{t_w}{2} < h_n \le \frac{b_f}{2}\right)$

$h_n = \frac{0.85 f'_c (A_c + A_s - 2 t_f b_f) - 2 F_y (A_s - 2 t_f b_f)}{2[4 F_y t_f + 0.85 f'_c (h_1 - 2 t_f)]}$

$Z_{sn} = Z_s - 2 t_f \left(\frac{b_f}{2} + h_n\right)\left(\frac{b_f}{2} - h_n\right)$

For h_n above the flange $\left(h_n > \frac{b_f}{2}\right)$

$h_n = \frac{0.85 f'_c (A_c + A_s) - 2 F_y A_s}{2(0.85 f'_c h_1)}$

$Z_{sn} = Z_s$

</td></tr>
</table>

F_y = specified minimum yield stress of steel shape.
F_{yr} = specified minimum yield stress of reinforcing steel.

PART 7

DESIGN CONSIDERATIONS FOR BOLTS

SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of bolts in steel-to-steel structural connections. Additional guidance on bolt design is available in AISC Design Guide 17, *High Strength Bolts—A Primer for Structural Engineers,* (Kulak, 2002). For the design of steel-to-concrete anchorage, see Part 14. For the design of connection elements, see Part 9. For the design of simple shear, moment, bracing and other connections, see Parts 10 through 15.

GENERAL REQUIREMENTS FOR BOLTED JOINTS

Fastener Components

The applicable material specifications for fastener components are given in Part 2. In this Part, for convenience in referencing and consistent with AISC *Specification* Section J3.1, ASTM F3125 Grades A325 and F1852 have been labeled Group A bolts, ASTM F3125 Grades A490 and F2280 have been labeled Group B bolts, and ASTM F3043 and ASTM F3111 assemblies have been labeled Group C bolts.

Material and storage requirements for fastener components are given in AISC *Specification* Section A3.3 and RCSC *Specification* Section 2. The compatibility of ASTM A563 nuts and ASTM F436 washers with Grades A325, F1852, A490 and F2280 bolts is given in RCSC *Specification* Table 2.1. These products are given identifying marks, as illustrated in RCSC *Specification* Figure C-2.1. ASTM F3043 and ASTM F3111 assemblies use nuts and washers as defined in their standard, and are marked according to that standard. Alternative-design fasteners and alternative washer-type indicating devices are permitted, subject to the requirements in RCSC *Specification* Sections 2.8 and 2.6.2, respectively.

Mixing grades of fasteners raises inventory and quality control issues associated with the use of multiple fastener grades. When Group A, Group B and/or Group C bolts are used together on a project, different diameters can be specified for each to help ensure that the bolts are installed in the proper location.

Regardless of the bolt type selected, the typical sizes of $^3/_4$-in.-, $^7/_8$-in.-, 1-in.-, $1^1/_8$-in.- and $1^1/_4$-in.-diameter are usually preferred. Diameters above 1 in. may require special consideration for availability, as well as installation, when pretensioned installation is required. Installation wrenches with high torque capacity and special equipment may be required to pretension large diameter Group B and Group C bolts. The use of Group C fasteners is limited as stated in AISC *Specification* Commentary Section J3.1.

Proper Selection of Bolt Length

Per RCSC *Specification* Section 2.3.2, adequate thread engagement is developed when the end of the bolt is at least flush with or projects beyond the face of the nut. To provide for this, the ordered length of Group A and Group B bolts should be calculated as the grip (see Figure 7-1) plus the nominal thickness of washers and/or direct-tension indicators, if used, plus the allowance from Table 7-14, with the total rounded to the next higher increment of $^1/_4$ in. up to a 5-in. length and the next higher $^1/_2$ in. over a 5-in. length. Note that bolts longer than 5 in. are generally available only in $^1/_2$-in. increments, except by special arrangement with the manufacturer or vendor. While longer lengths may be ordered, an 8-in. length is generally the maximum stock length available. Requirements for a minimum stick-through greater than zero are discouraged because of the risk of jamming the nut on the thread runout, particularly in the bolt length range available only in 1/2-in. increments. See Carter (1996) for further information.

For ASTM F3043 and F3111 assemblies, refer to the manufacturer's literature for selection of bolt length.

Washer Requirements

Requirements for the use of ASTM F436 washers and/or plate washers are given in RCSC *Specification* Section 6.

Nut Requirements

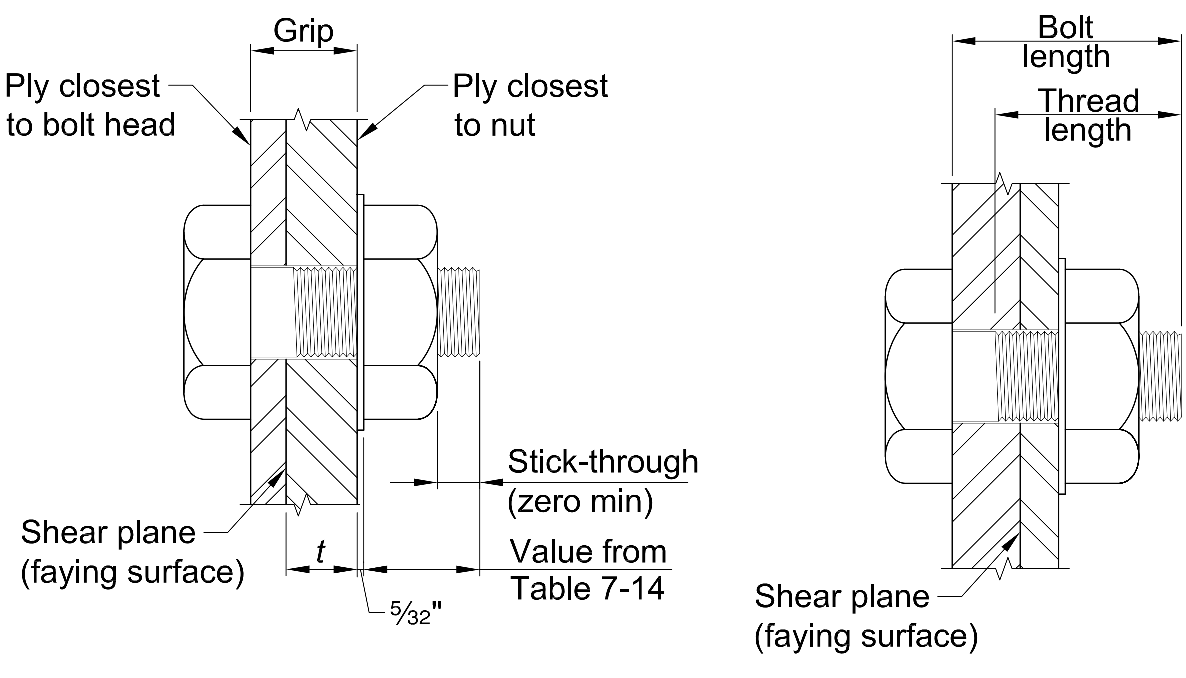
The compatibility of ASTM A563 nuts with Group A and Group B bolts is given in RCSC *Specification* Table 2.1.

Bolted Parts

The requirements for connected plies, faying surfaces, bolt holes and burrs are given in AISC *Specification* Sections J3.2 and M2.5, and RCSC *Specification* Section 3. Spacing and edge distance requirements are given in AISC *Specification* Sections J3.3, J3.4 and J3.5.

PROPER SPECIFICATION OF JOINT TYPE

When Group A or Group B high-strength bolts are to be used, the joint type must be specified as snug-tightened, pretensioned or slip-critical, per AISC *Specification* Section J3.1.



(a) Shear plane location when threads are excluded

(b) Shear plane location when threads are included

Fig. 7-1. Grip and other parameters for bolt length selection.

Snug-Tightened Joints

Snug-tightened joints simplify design, installation and inspection and should be specified whenever pretensioned joints and slip-critical joints are not required. The applicability is summarized and design requirements, installation requirements and inspection requirements are stipulated for snug-tightened joints per RCSC *Specification* Section 4.1. Faying surfaces in snug-tightened joints must meet the requirements in RCSC *Specification* Sections 3.2 and 3.2.1, but not those for slip-critical joints in RCSC *Specification* Section 3.2.2. Note that there is generally no need to limit the actual level of pretension provided in snug-tightened joints, per RCSC *Specification* Section 9.1.

Pretensioned Joints

When pretension is required but slip-resistance is not of concern, a pretensioned joint should be specified. The applicability is summarized and design requirements, installation requirements and inspection requirements are stipulated for pretensioned joints per RCSC *Specification* Section 4.2. Faying surfaces in pretensioned joints must meet the requirements in RCSC *Specification* Sections 3.2 and 3.2.1, but not those for slip-critical joints in RCSC *Specification* Section 3.2.2.

Slip-Critical Joints

The applicability of slip-critical joints is summarized and design requirements, installation requirements, and inspection requirements are stipulated in RCSC *Specification* Section 4.3, except as modified by AISC *Specification* Sections J3.8 and J3.9. Faying surfaces in slip-critical joints must meet the requirements in RCSC *Specification* Sections 3.2 and 3.2.2. The RCSC *Specification* defines a faying surface as "the plane of contact between two plies of a joint." Note that the surfaces under the bolt head, washer and/or nut are not faying surfaces.

Subject to the requirements in RCSC *Specification* Section 4.3, slip-critical joints are rarely required in building design. Slip-critical joints are appreciably more expensive because of the associated costs of faying surface preparation and installation and inspection requirements.

When slip resistance is required and the steel is painted, the fabricator should be consulted to determine the most economical approach to providing the necessary slip resistance. Special paint systems that are rated for slip resistance can be specified. Alternatively, a paint system that is not rated for slip resistance can be used with the faying surfaces masked.

DESIGN REQUIREMENTS

Design requirements are found in the AISC *Specification* as follows. In each case, the available strength determined in accordance with these provisions must equal or exceed the required strength. These requirements are derived from those in the RCSC *Specification*.

Shear

Available shear strength is determined as given in RCSC *Specification* Section 5.1 and AISC *Specification* Section J3.6, with consideration of the presence of fillers or shims, per RCSC *Specification* Section 5.1 and AISC *Specification* Section J5. The nominal shear strengths given in AISC *Specification* Table J3.2 have been reduced by approximately 10% from statistical results of tests to account for uneven force distributions associated with end loading and other effects normally neglected in the design process.

When the length of a bolted joint measured parallel to the line of force exceeds 38 in., a 16.7% strength reduction may be applicable, per AISC *Specification* Table J3.2 footnote b.

The force that can be resisted by a snug-tightened or pretensioned high-strength bolt may also be limited by the bearing or tearout strength at the bolt hole per AISC *Specification* Section J3.10. The effective strength of an individual bolt may be taken as the lesser of the shear strength per Section J3.6 or the controlling bearing and tearout strength at the bolt hole per Section J3.10. The strength of the bolt group may be taken as the sum of the effective strengths of the individual fasteners.

Tension

Available tensile strength is determined as given in RCSC *Specification* Section 5.1 and AISC *Specification* Section J3.6, with consideration of the effects of prying action, if any. Prying action is a phenomenon (in bolted construction only) whereby the deformation of a fitting under a tensile force increases the tensile force in the bolt. While the effect of prying action is relevant to the design of the bolts, it is primarily a function of the strength and stiffness of the connection elements. Prying action is addressed in Part 9.

Combined Shear and Tension

Available strength for combined shear and tension in bearing-type connections is determined as given in RCSC *Specification* Section 5.2 and AISC *Specification* Section J3.7.

Bearing and Tearout Strength at Bolt Holes

Available bearing and tearout strength at bolt holes is determined as given in RCSC *Specification* Section 5.3 and AISC *Specification* Section J3.10.

Slip Resistance

The available slip resistance of slip-critical connections is determined in accordance with AISC *Specification* Section J3.8. The available strength, ϕR_n or R_n/Ω, is determined by applying the resistance factor or safety factor appropriate for the hole type used.

ECCENTRICALLY LOADED BOLT GROUPS

Eccentricity in the Plane of the Faying Surface

When eccentricity occurs in the plane of the faying surface, the bolts must be designed to resist the combined effect of the direct shear, P_u or P_a, and the additional shear from the induced moment, $P_u e$ or $P_a e$. Two analysis methods for this type of eccentricity are the instantaneous center of rotation method and the elastic method.

The instantaneous center of rotation method is more accurate, but generally requires the use of tabulated values or an iterative solution. The elastic method is simplified, but may be excessively conservative because it neglects the ductility of the bolt group and the potential for load redistribution.

Instantaneous Center of Rotation Method

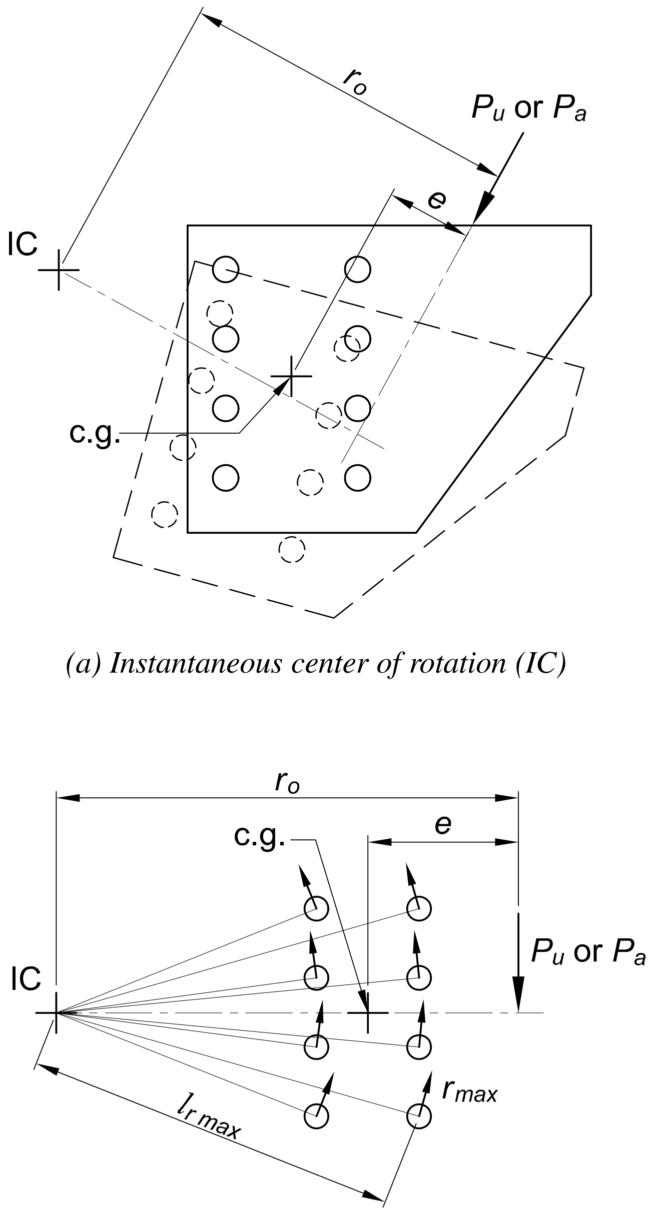
Eccentricity produces both a rotation and a translation of one connection element with respect to the other. The combined effect of this rotation and translation is equivalent to a rotation about a point defined as the instantaneous center of rotation (IC), as illustrated in Figure 7-2(a). The location of the IC depends upon the geometry of the bolt group as well as the direction and point of application of the load.

The load-deformation relationship for one bolt is illustrated in Figure 7-3, where

$$R = R_{ult}(1 - e^{-10\Delta})^{0.55} \tag{7-1}$$

where

R = nominal shear strength of one bolt at a deformation Δ, kips
R_{ult} = ultimate shear strength of one bolt, kips
e = 2.718…, base of the natural logarithm
Δ = total deformation, including shear, bearing and bending deformation in the bolt and bearing deformation of the connection elements, in.



(a) Instantaneous center of rotation (IC)

(b) Forces on bolts in group for case of $\theta = 0°$ for simplicity

Fig. 7-2. Illustration for instantaneous center of rotation method.

The shear strength of the bolt most remote from the IC can be determined by applying a maximum deformation, Δ_{max}, to that bolt. The load-deformation relationship is based upon data obtained experimentally for 3/4-in.-diameter ASTM F3125 Grade A325 bolts in double shear, where $R_{ult} = 74$ kips and $\Delta_{max} = 0.34$ in.

The nominal shear strengths of the other bolts in the joint can be determined by applying a deformation Δ that varies linearly with distance from the IC. The nominal shear strength of the bolt group is, then, the sum of the individual strengths of all bolts.

The individual resistance of each bolt is assumed to act on a line perpendicular to a ray passing through the IC and the centroid of that bolt, as illustrated in Figure 7-2(b). If the correct location of the IC has been selected, the three equations of in-plane static equilibrium ($\Sigma F_x = 0$, $\Sigma F_y = 0$, and $\Sigma M = 0$) will be satisfied.

For further information, see Crawford and Kulak (1971).

Elastic Method

For a force applied as illustrated in Figure 7-4, the eccentric force, P_u or P_a, is resolved into a direct shear, P_u or P_a, acting through the center of gravity (c.g.) of the bolt group and a moment, $P_u e$ or $P_a e$, where e is the eccentricity. Each bolt is then assumed to resist an equal share of the direct shear and a share of the eccentric moment proportional to its distance from the c.g. The resultant vectorial sum of these forces is the required strength for the bolt, r_u or r_a.

The shear per bolt due to the concentric force, P_u or P_a, is r_{pu} or r_{pa}, where

LRFD	ASD
$r_{pu} = \dfrac{P_u}{n}$ (7-2a)	$r_{pa} = \dfrac{P_a}{n}$ (7-2b)

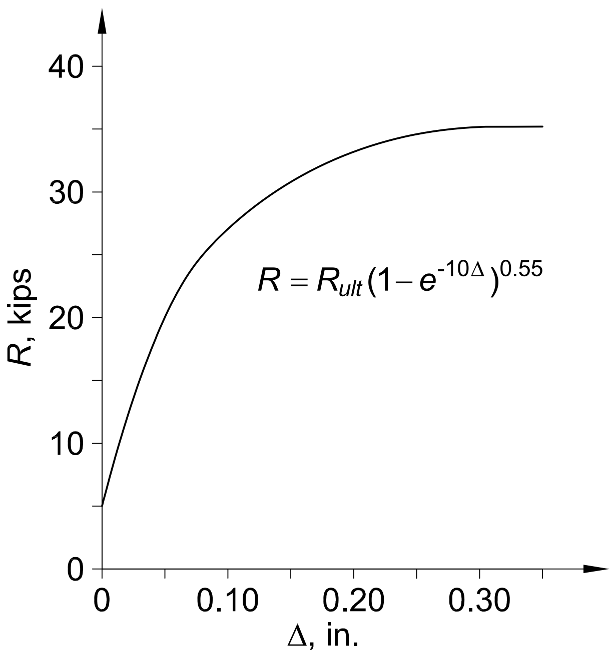


Fig. 7-3. Load-deformation relationship for one 3/4-in.-diameter ASTM F3125 Grade A325 bolt in single shear.

and n is the number of bolts. To determine the resultant forces on each bolt when P_u or P_a is applied at an angle θ with respect to the vertical, r_{pu} or r_{pa} must be resolved into horizontal component, r_{pxu} or r_{pxa}, and vertical component, r_{pyu} or r_{pya}, where

LRFD		ASD	
$r_{pxu} = r_{pu}\sin\theta$	(7-3a)	$r_{pxa} = r_{pa}\sin\theta$	(7-3b)
$r_{pyu} = r_{pu}\cos\theta$	(7-4a)	$r_{pya} = r_{pa}\cos\theta$	(7-4b)

The shear on the bolt most remote from the c.g. due to the moment, P_ue or P_ae, is r_{mu} or r_{ma}, where

LRFD		ASD	
$r_{mu} = \dfrac{P_uec}{I_p}$	(7-5a)	$r_{ma} = \dfrac{P_aec}{I_p}$	(7-5b)

where

$I_p = I_x + I_y$ = polar moment of inertia of the bolt group, in.^{4} per in.^{2}

c = radial distance from c.g. to center of bolt most remote from c.g., in.

To determine the resultant force on the most highly stressed bolt, r_{mu} or r_{ma} must be resolved into horizontal component r_{mxu} or r_{mxa} and vertical component r_{myu} or r_{mya}, where

LRFD		ASD	
$r_{mxu} = \dfrac{P_uec_y}{I_p}$	(7-6a)	$r_{mxa} = \dfrac{P_aec_y}{I_p}$	(7-6b)
$r_{myu} = \dfrac{P_uec_x}{I_p}$	(7-7a)	$r_{mya} = \dfrac{P_aec_x}{I_p}$	(7-7b)

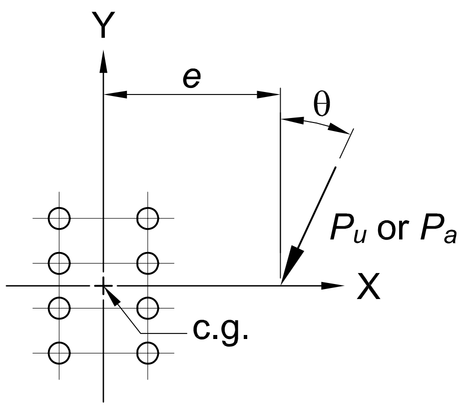


Fig. 7-4. Illustration for elastic method.

In the preceding equations, c_x and c_y are the horizontal and vertical components of the diagonal distance c. Thus, the required strength per bolt is r_u or r_a, where

LRFD	ASD
$r_u = \sqrt{\left(r_{pxu} + r_{mxu}\right)^2 + \left(r_{pyu} + r_{myu}\right)^2}$ (7-8a)	$r_a = \sqrt{\left(r_{pxa} + r_{mxa}\right)^2 + \left(r_{pya} + r_{mya}\right)^2}$ (7-8b)

For further information, see Higgins (1971).

Eccentricity Normal to the Plane of the Faying Surface

Eccentricity normal to the plane of the faying surface produces tension above and compression below the neutral axis for a bracket connection as shown in Figure 7-5. The eccentric force, P_u or P_a, is resolved into a direct shear, P_u or P_a, acting at the faying surface of the joint and a moment normal to the plane of the faying surface, $P_u e$ or $P_a e$, where e is the eccentricity. Each bolt is then assumed to resist an equal share of the concentric force, P_u or P_a, and the moment is resisted by tension in the bolts above the neutral axis and compression below the neutral axis.

Two design approaches for this type of eccentricity are available: Case I, in which the neutral axis is not taken at the center of gravity (c.g.) of the bolt group, and Case II, in which the neutral axis is taken at the c.g.

Case I—Neutral Axis Not at Center of Gravity

The shear per bolt due to the concentric force, r_{uv} or r_{av}, is determined as

LRFD	ASD
$r_{uv} = \dfrac{P_u}{n}$ (7-9a)	$r_{av} = \dfrac{P_a}{n}$ (7-9b)

where n is the number of bolts in the connection.

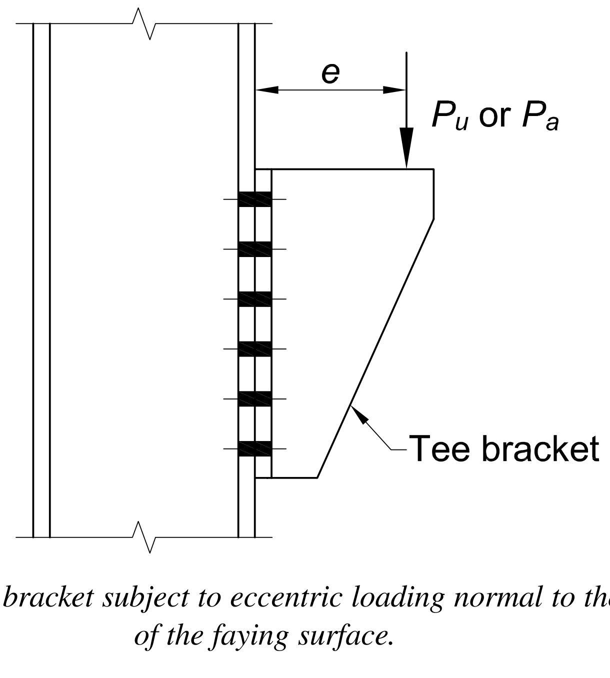


Fig. 7-5. Tee bracket subject to eccentric loading normal to the plane of the faying surface.

A trial position for the neutral axis can be selected at one-sixth of the total bracket depth, measured upward from the bottom [line X-X in Figure 7-6(a)]. To provide for reasonable proportions and to account for the bending stiffness of the connection elements, the effective width of the compression block, b_{eff}, should be taken as

$$b_{eff} = 8t_f \leq b_f \tag{7-10}$$

where

b_f = connection element width, in.
t_f = lesser connection element thickness, in.

This effective width is valid for bracket flanges made from W-shapes, S-shapes, welded plates and angles. Where the bracket flange thickness is not constant, the average flange thickness should be used.

The assumed location of the neutral axis can be evaluated by checking static equilibrium assuming an elastic stress distribution. Equating the moment of the bolt area above the neutral axis with the moment of the compression block area below the neutral axis,

$$(\Sigma A_b)y = b_{eff} d\,(d/2) \tag{7-11}$$

where

ΣA_b = sum of the areas of all bolts above the neutral axis, in.^{2}
d = depth of compression block, in.
y = distance from line X-X to the c.g. of the bolt group above the neutral axis, in.

The value of d may then be adjusted until a reasonable equality exists.

Once the neutral axis has been located, the tensile force per bolt, r_{ut} or r_{at}, as illustrated in Figure 7-6(b), may be determined as

LRFD	ASD
$r_{ut} = \left(\dfrac{P_u ec}{I_x}\right)A_b$ (7-12a)	$r_{at} = \left(\dfrac{P_a ec}{I_x}\right)A_b$ (7-12b)

where

I_x = combined moment of inertia of the bolt group and compression block about the neutral axis, in.^{4}
c = distance from neutral axis to the most remote bolt in the group, in.

Bolts above the neutral axis are subjected to the shear force, the tensile force, and the effect of prying action (see Part 9); bolts below the neutral axis are subjected to the shear force, r_{uv} or r_{av}, only.

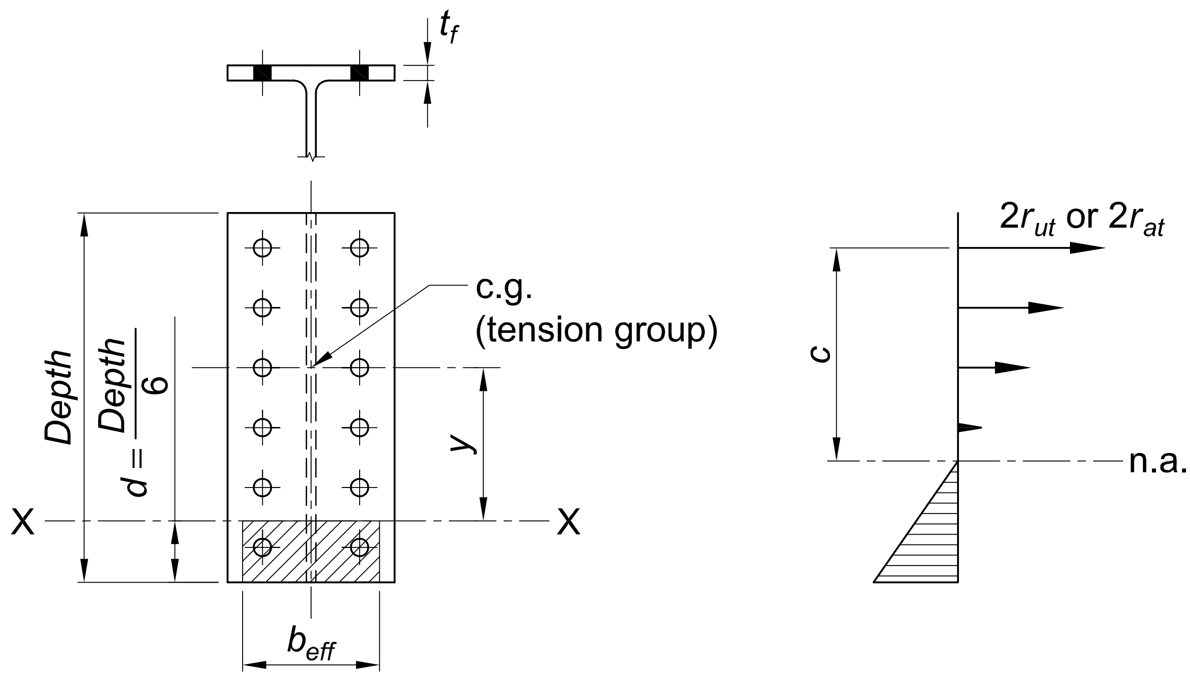
Case II—Neutral Axis at Center of Gravity

This method provides a more direct, but also a more conservative result. As for Case I, the shear force per bolt, r_{uv} or r_{av}, due to the concentric force, P_u *or* P_a, is determined as

LRFD	ASD
$r_{uv} = \dfrac{P_u}{n}$ (7-13a)	$r_{av} = \dfrac{P_a}{n}$ (7-13b)

where n is the number of bolts in the connection.

The neutral axis is assumed to be located at the c.g. of the bolt group as illustrated in Figure 7-7. The bolts above the neutral axis are in tension and the bolts below the neutral axis are said to be in compression. To obtain a more accurate result, a plastic stress distribution is assumed; this assumption is justified because this method is still more conservative than Case I. Accordingly, the tensile force in each bolt above the neutral axis, r_{ut} or r_{at}, due to the moment, P_ue or P_ae, is determined as



(a) Initial approximation of location of n.a. *(b) Force diagram with final location of n.a.*

Fig. 7-6. Location of neutral axis (n.a.) for out-of-plane eccentric loading using Case I.

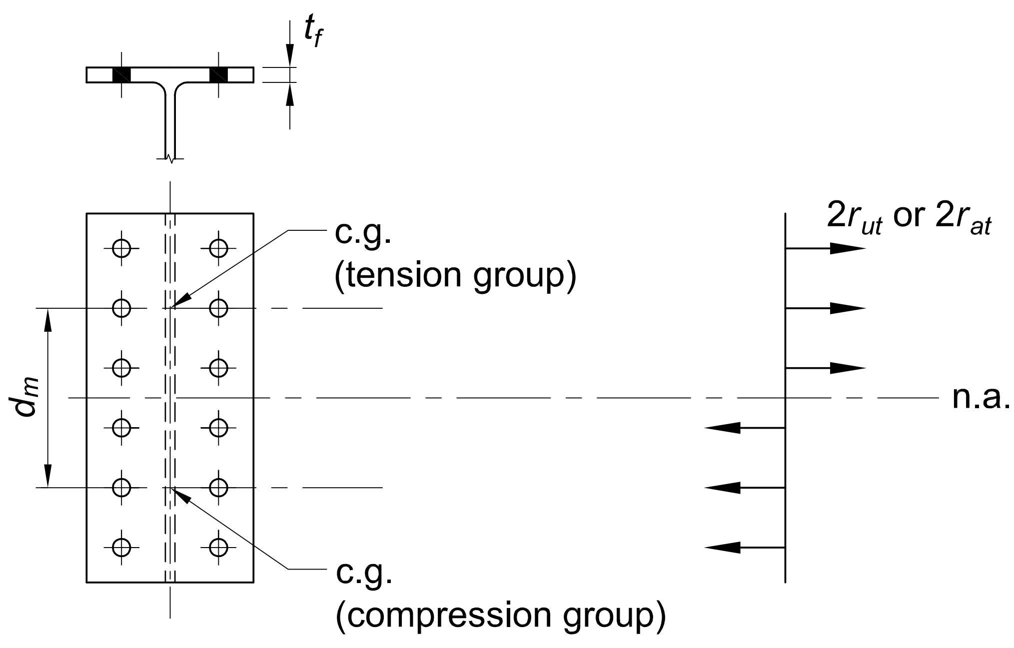


Fig. 7-7. Location of neutral axis (n.a.) for out-of-plane eccentric loading using Case II.

LRFD		ASD	
$r_{ut} = \dfrac{P_u e}{n' d_m}$	(7-14a)	$r_{at} = \dfrac{P_a e}{n' d_m}$	(7-14b)

where

d_m = moment arm between resultant tensile force and resultant compressive force, in.
n' = number of bolts above the neutral axis

Bolts above the neutral axis are subjected to the shear force, the tensile force, and the effect of prying action (see Part 9); bolts below the neutral axis are subjected to the shear force, r_{uv} or r_{av}, only.

SPECIAL CONSIDERATIONS FOR HOLLOW STRUCTURAL SECTIONS

Through-Bolting to HSS

Long bolts that extend through the entire HSS are satisfactory for shear connections that do not require a pretensioned installation. The flexibility of the walls of the HSS precludes installation of pretensioned bolts. Standard structural bolts may be used, although ASTM A449 bolts may be required for longer lengths. The bolts are designed for static shear and the only limit-state involving the HSS is bolt bearing. The available bearing strength is determined as ϕR_n or R_n/Ω, where

$$R_n = 1.8nF_y dt_{des} \tag{7-15}$$

$$\phi = 0.75 \qquad \Omega = 2.00$$

where

F_y = specified minimum yield strength of HSS, ksi
d = fastener diameter, in.
n = number of fasteners
t_{des} = design wall thickness of HSS, in.

Blind Bolts

Special fasteners are available that eliminate the need for access to install a nut (Korol et al., 1993; Henderson, 1996). The shank of the fastener is inserted through holes in the parts to be connected until the head bears on the outer ply (see Figure 7-8). In some cases, a special wrench is used on the open side to keep the outer part of the shank from rotating and simultaneously turn the threaded part of the shank. A wedge or other mechanism on the blind side causes the fixed part of the shank to expand and form a contact with the inside of the HSS. Some fasteners contain a break-off mechanism when the fastener is pretensioned. Recent versions of these fasteners meet the requirements for a pretensioned ASTM F3125 Grade A325 bolt (Henderson, 1996) and could be used in slip-critical or tension conditions. HSS limit states are bolt bearing and tearout in shear, tear-out of the bolt in tension, and wall distortion. Manufacturers' literature must be consulted to determine the available strength of blind bolts.

Flow-Drilling

Flow-drilling is a process that can be used to produce a threaded hole in an HSS to permit blind bolting when the inside of the HSS is inaccessible (Sherman, 1995; Henderson, 1996).

The process is to force a hole through the HSS with a carbide conical tool rotating at sufficient speed to produce high rapid heating, which softens the material in a local area. The material that is displaced as the tool is forced through the plate forms a truncated hollow cone (bushing) on the inner surface and a small upset on the outer surface. Tools can be obtained with a milling collar so that the material on the outer surface is removed, producing a flat surface allowing parts to be brought in close contact. A cold-formed tap is then used to roll a thread into the hole without any chips or removal of material. The resulting threaded hole has the approximate dimensions and hardness of a heavy hex nut. Shear and tension strengths of ASTM F3125 Grade A325 bolts can be developed for certain combinations of bolt size and HSS thickness (see Figure 7-9).

Drilling equipment with suitable rotational speed, torque and thrust is required, but with small sizes and thicknesses, field installation with conventional tools is possible. The bolts are designed with the normal criteria and the HSS limit states are bolt bearing and tearout in shear and distortion of the HSS wall in tension. HSS strength is not affected by the process except for the reduction in area due to the holes.

Threaded Studs to HSS

Threaded studs are available in 3/8-in. to 7/8-in. diameters and can be shop- or field-welded to an HSS with a stud-welding gun. The connection is similar to a bolted connection with an

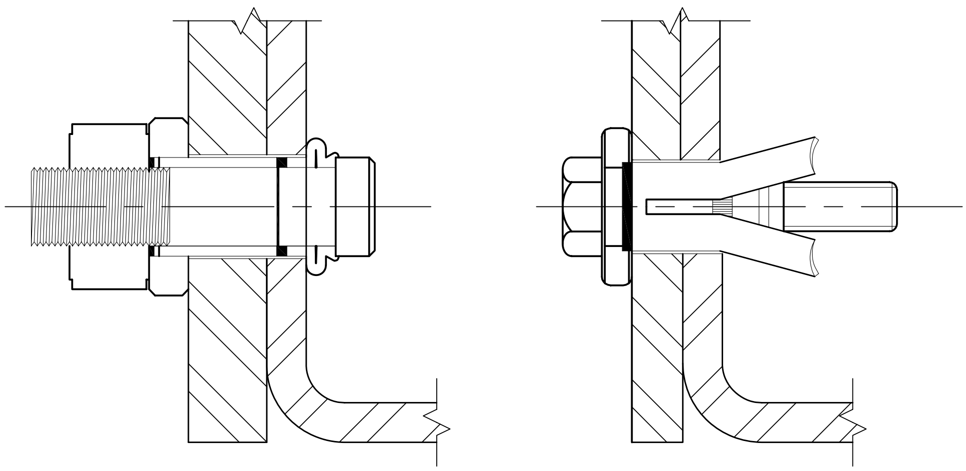


Fig. 7-8. Two types of blind bolts.

HSS Thickness (in.)	Bolt Diameter (in.)				
	1/2	5/8	3/4	7/8	1
3/16	X	X			
1/4	X	X	X		
5/16		X	X	X	
3/8			X	X	X
1/2					X

Fig. 7-9. HSS thickness and bolt diameter combinations for flow-drilling.

external nut. The strength of the stud in tension or shear is based on manufacturer's recommendations and tests. The HSS limit state is distortion of the wall. When using threaded studs, countersunk holes must be used in the attached element to clear the weld fillet at the base of the stud.

Nailing to HSS

Power-driven nails that are installed with a power-actuated gun are satisfactory for pure shear connections where the combined thickness of the attachment and the HSS does not exceed $^1/_2$ in. This system was tested as splices between telescoping round HSS loaded with an axial force (Packer, 1996). The shear resistance of the fasteners is taken as the number of nails times the shear strength of a single nail and ignores any secondary contribution from a dimpling effect between the materials. The limit state for the HSS is shear-bearing. See Packer (1996).

Screwing to HSS

Self-tapping screws with or without self-drilling points are available for connecting materials with combined thicknesses up to $^1/_2$ in. The screws have diameters from 0.08 in. to 0.25 in. The limit states for these connections are given in the AISI *North American Specification for the Design of Cold-Formed Steel Structural Members* (AISI, 2012).

OTHER SPECIFICATION REQUIREMENTS AND DESIGN CONSIDERATIONS

The following other specification requirements and design considerations apply to the design of bolts.

Placement of Bolt Groups

For the required placement of bolt groups at the ends of axially loaded members, see AISC *Specification* Section J1.7.

Bolts in Combination with Welds

For bolts used in combination with welds, see AISC *Specification* Section J1.8.

Coating High-Strength Bolts and Nuts

Coatings can affect the installation and performance of high-strength bolt assemblies. Coatings have a finite thickness and surface properties that can affect thread fit and the torque-tension relationship. Coatings and the process of applying them can have an effect on hydrogen embrittlement. Service environment can have an effect on hydrogen embrittlement or stress corrosion cracking. Where bolts are approved for galvanizing or zinc/aluminum (Zn/Al) coating, nuts and washers are available with corresponding coatings. See ASTM F3125 Annex A1 for requirements regarding nuts, washers and thread fit. Nuts for Grades A325 and F1852 bolts must be galvanized by the same process as the bolt with which they are used. Zn/Al coatings have been used on high strength fasteners in automotive applications and have been tested for use in structural applications on 150-ksi bolts, nuts and washers. The tests evaluate the coated fasteners for hydrogen embrittlement susceptibility using Industrial Fasteners Institute IFI-144 (IFI, 2013), ASTM F1940 and F2660. The tests do not assure durability or corrosion resistance over any length of time. The purchaser should evaluate any other performance characteristics of these coatings. Galvanized ASTM A449 may require an anti-galling lubricant. See Figure 7-10 for permitted coatings for fasteners.

Reuse of Bolts

The reuse of high-strength bolts is limited, per RCSC *Specification* Section 2.3.3. See also Bowman and Betancourt (1991) and AISC Design Guide 17 (Kulak, 2002).

Fatigue Applications

For applications involving fatigue, see RCSC *Specification* Sections 4.2, 4.3 and 5.5, and AISC *Specification* Appendix 3.

Entering and Tightening Clearances

Clearances must be provided for the entering and tightening of the bolts with an impact wrench. The clearance requirements for conventional high-strength bolts (ASTM F3125 Grades A325 and A490) are given in Table 7-15. When high-strength tension-control bolts (ASTM F3125 Grades F1852 and F2280) are specified, the clearance requirements are given in Table 7-16.

ASTM Designation		Fastener Description	Coating Type		
			Mechanical Galvanizing, ASTM B695	Hot Dip Galvanizing, ASTM F2329	Zinc/ Aluminum
ASTM F3125	Gr. A325	Heavy hex, F_u = 120 ksi	Class 55	50 µm	a
	Gr. F1852	Tension control, F_u = 120 ksi	Class 55	–	–
	Gr. A490	Heavy hex, F_u = 150 ksi	–	–	a
	Gr. F2280	Tension control, F_u = 150 ksi	–	–	–
A449		Heavy hex, F_u = 90, 105 and 120 ksi	Class 55	50 µm	–
A354 BC		Heavy hex, F_u = 115 ksi and 125 ksi	Class 55	50 µm	–
A354 BD		Heavy hex, F_u = 140 ksi and 150 ksi	b	b	–

– Indicates this coating is not qualified.

[a] See ASTM F3125 Table 1.1 for approved zinc/aluminum coating standards and grades.

[b] Galvanizing of ASTM A354 BD is not prohibited but may cause susceptibility to hydrogen embrittlement. Precautions to avoid embrittlement, such as those in ASTM A143, should be considered.

Fig. 7-10. Permitted coatings for structural fasteners.

Fully Threaded ASTM F3125 Grade A325 Bolts

ASTM F3125 Grade A325 bolts with length equal to or less than four times the nominal bolt diameter may be ordered as fully threaded with the designation Grade A325T. Fully threaded Grade A325T bolts are not for use in bearing-type "X" connections since it would be impossible to exclude the threads from the shear plane. While this supplementary provision exists for Grade A325 bolts, the supplementary provision does not apply to ASTM F3125 Grade A490 for full-length threading.

ASTM A307 Bolts

AISC *Specification* Section J3 provides limitations on the use of ASTM A307 bolts. ASTM A307 bolts are available with both hex and square heads in diameters from 1/4 in. to 4 in. in Grade A for general applications and Grade B for cast-iron-flanged piping joints. ASTM A563 Grade A nuts are recommended for use with ASTM A307 bolts. Other suitable grades are listed in ASTM A563 Table X1.1.

ASTM A449 and A354 Bolts

Limitations are provided on the use of ASTM A354 and A449 bolts, per AISC *Specification* Section J3.1. The tensile strength of ASTM A354 bolts decreases in bolts over 2 1/2 in. in diameter. The tensile strength of ASTM A449 bolts decreases in bolts over 1 in. in diameter and again over 1 1/2 in. in diameter. ASTM A354 and A449 are available in a variety of product forms. AISC *Specification* Section J3 permits their use as high-strength bolts in applications where the required diameter or length is outside the ranges permitted by ASTM F3125. When ASTM A354 and A449 are used in bolting applications they are ordered to conform to the dimensions of ASME B18.2.6 (ASME, 2010) heavy hex bolts and nuts.

DESIGN TABLE DISCUSSION

Table 7-1. Available Shear Strength of Bolts

The available shear strengths of various grades and sizes of bolts are summarized in Table 7-1.

Table 7-2. Available Tensile Strength of Bolts

The available tensile strengths of various grades and sizes of bolts are summarized in Table 7-2.

Table 7-3. Slip-Critical Connections—Available Slip Resistance

The available slip resistance of various grades and sizes of bolts are summarized in Table 7-3.

Tables 7-4 and 7-5. Available Bearing and Tearout Strength at Bolt Holes

The available bearing and tearout strength at bolt holes is tabulated for various spacings and edge distances in Tables 7-4 and 7-5, respectively. Note that these tables may be applied to bolts with countersunk heads, by subtracting one-half the depth of the countersink from the material thickness, t. As illustrated in Figure 7-11, this is equivalent to subtracting $d_b/4$ from the material thickness, t. Values in Table 7-4 and Table 7-5 are the lesser of $1.2l_ctF_u$ and $2.4dtF_u$ based on AISC *Specification* Section J3.10. Interpolation between values in these tables may produce an incorrect result.

Tables 7-6 through 7-13. Coefficients C for Eccentrically Loaded Bolt Groups

Tables 7-6 through 7-13 employ the instantaneous center of rotation method for the bolt patterns and eccentric conditions indicated, and inclined loads at 0°, 15°, 30°, 45°, 60° and 75°. The tabulated non-dimensional coefficient, C, represents the number of bolts that are effective in resisting the eccentric shear force. In the following discussion, r_n is the least nominal strength of one bolt determined from the limit states of bolt shear strength, bearing and tearout strength at bolt holes, and slip resistance (if the connection is to be slip-critical).

When Analyzing a Known Bolt Group Geometry

For any of the bolt group geometries shown, the available strength of the eccentrically loaded bolt group, ϕR_n or R_n/Ω, is determined as

$$R_n = Cr_n \tag{7-16}$$

For bolts in bearing:

$$\phi = 0.75 \qquad \Omega = 2.00$$

For bolts in slip-critical connections, see AISC *Specification* Section J3.8 for the appropriate resistance and safety factors.

When Selecting a Bolt Group

The available strength must be greater than or equal to the required strength, P_u or P_a. Thus, by dividing the required strength, P_u or P_a, by ϕr_n or r_n/Ω, the minimum coefficient, C, is obtained. The bolt group can then be selected from the table corresponding to the appropri-

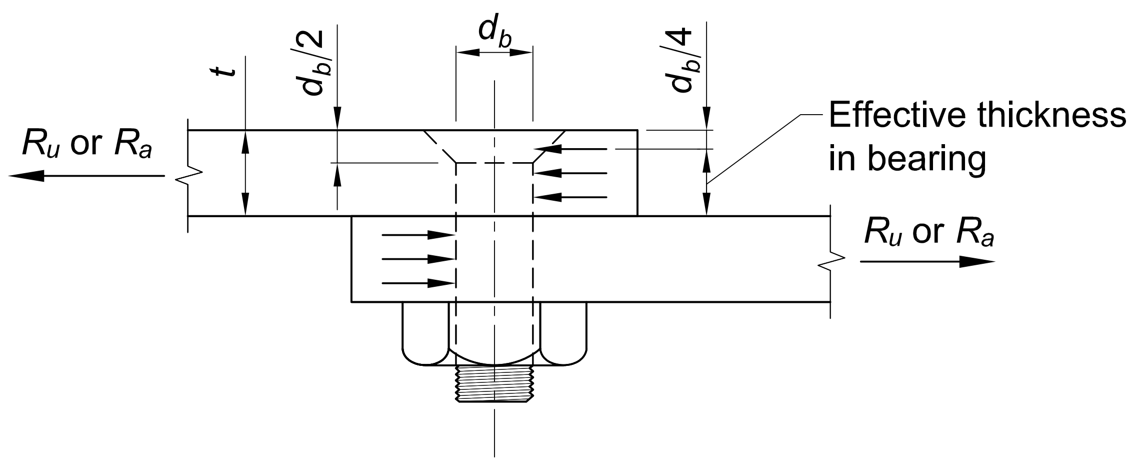


Fig. 7-11. Effective bearing-thickness for bolts with countersunk heads.

ate load angle, at the appropriate eccentricity, e_x, for which the coefficient is of that magnitude or greater.

These tables may be used with any bolt diameter and are conservative when used with Group B or Group C bolts (see Kulak, 1975). Linear interpolation within a given table between adjacent values of e_x is permitted. Although this procedure is based on bearing connections, both load tests and analytical studies indicate that it may be conservatively extended to slip-critical connections (Kulak, 1975).

A convergence criterion of 1% was employed for the tabulated iterative solutions. Straight-line interpolation between values for loads at different angles may be significantly unconservative. Either a direct analysis should be performed or the values for the next lower angle increment in the tables should be used for design. For bolt group patterns not treated in these tables, a direct analysis is required if the instantaneous center of rotation method is to be used.

In some cases, it is necessary to calculate the pure moment strength of a bolt group for purposes of linear interpolation. For these cases, the value of C' has been provided for a load angle of 0°. This moment strength of the bolt group is based on the instantaneous center of rotation method and, since a moment-only condition is assumed, the instantaneous center of rotation coincides with the center of gravity of the bolt group. In this case, the strength is:

$$M_{max} = C' r_n \tag{7-17}$$

where

$$C' = \sum l_i \left[1 - e^{-\left(\frac{10 l_i \Delta_{max}}{l_{max}}\right)}\right]^{0.55}, \text{ in.} \tag{7-18}$$

l_i = distance from the center of gravity of the bolt group to the ith bolt, in.
l_{max} = distance from the center of gravity of the bolt group to the center of the farthest bolt, in.
Δ_{max} = maximum deformation on the bolt farthest from the center of gravity = 0.34 in.

Table 7-14. Dimensions of High-Strength Fasteners

Dimensions of ASTM F3125 Grades A325, F1852, A490 and F2280 bolts, ASTM A563 nuts, and ASTM F436 washers are given in Table 7-14.

Tables 7-15 and 7-16. Entering and Tightening Clearances

Clearance is required for entering and tightening bolts with an impact wrench. The required clearances are given for conventional high-strength bolts and twist-off-type tension-control bolt assemblies in Tables 7-15 and 7-16, respectively.

Table 7-17. Threading Dimensions for High-Strength and Non-High-Strength Bolts

Threading dimensions, properties and standard designations for high-strength and non-high-strength bolts are provided in Table 7-17.

Table 7-18. Weights of High-Strength Fasteners

Weights of conventional ASTM F3125 Grade A325 and Grade A490 bolts, ASTM A563 nuts, and ASTM F436 washers are given in Table 7-18. For dimensions and weights of tension-control ASTM F3125 Grade F1852 and Grade F2280 bolts, refer to manufacturers' literature or the Industrial Fasteners Institute (IFI).

Table 7-19. Dimensions of Non-High-Strength Fasteners

Typical non-high-strength bolt head and nut dimensions are given in Table 7-19. Thread lengths listed in this table may be calculated for non-high-strength bolts as $2d + 1/4$ in. for bolts up to 6-in. long and $2d + 1/2$ in. for bolts over 6-in. long, where d is the bolt diameter. Note that these thread lengths are longer than those given previously for high-strength bolts in Table 7-14. Threading dimensions are given in Table 7-17.

Tables 7-20, 7-21 and 7-22. Weights of Non-High-Strength Fasteners

Weights of non-high-strength fasteners are given in Tables 7-20, 7-21 and 7-22.

PART 7 REFERENCES

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Table 7-1
Available Shear Strength of Bolts, kips

Nominal Bolt Diameter, d, in.					5/8		3/4		7/8		1	
Nominal Bolt Area, in.^{2}					0.307		0.442		0.601		0.785	
Designation	Thread Cond.	F_{nv}/Ω (ksi)	ϕF_{nv} (ksi)	Loading	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
		ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Group A	N	27.0	40.5	S	8.29	12.4	11.9	17.9	16.2	24.3	21.2	31.8
				D	16.6	24.9	23.9	35.8	32.5	48.7	42.4	63.6
	X	34.0	51.0	S	10.4	15.7	15.0	22.5	20.4	30.7	26.7	40.0
				D	20.9	31.3	30.1	45.1	40.9	61.3	53.4	80.1
Group B	N	34.0	51.0	S	10.4	15.7	15.0	22.5	20.4	30.7	26.7	40.0
				D	20.9	31.3	30.1	45.1	40.9	61.3	53.4	80.1
	X	42.0	63.0	S	12.9	19.3	18.6	27.8	25.2	37.9	33.0	49.5
				D	25.8	38.7	37.1	55.7	50.5	75.7	65.9	98.9
Group C	N	45.0	67.5	S	–	–	–	–	–	–	35.3	53.0
				D	–	–	–	–	–	–	70.7	106
	X	56.5	84.8	S	–	–	–	–	–	–	44.4	66.6
				D	–	–	–	–	–	–	88.7	133
A307	Not applicable	13.5	20.3	S	4.14	6.23	5.97	8.97	8.11	12.2	10.6	15.9
				D	8.29	12.5	11.9	17.9	16.2	24.4	21.2	31.9

Nominal Bolt Diameter, d, in.					1 1/8		1 1/4		1 3/8		1 1/2	
Nominal Bolt Area, in.^{2}					0.994		1.23		1.48		1.77	
Designation	Thread Cond.	F_{nv}/Ω (ksi)	ϕF_{nv} (ksi)	Loading	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
		ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Group A	N	27.0	40.5	S	26.8	40.3	33.2	49.8	40.0	59.9	47.8	71.7
				D	53.7	80.5	66.4	99.6	79.9	120	95.6	143
	X	34.0	51.0	S	33.8	50.7	41.8	62.7	50.3	75.5	60.2	90.3
				D	67.6	101	83.6	125	101	151	120	181
Group B	N	34.0	51.0	S	33.8	50.7	41.8	62.7	50.3	75.5	60.2	90.3
				D	67.6	101	83.6	125	101	151	120	181
	X	42.0	63.0	S	41.7	62.6	51.7	77.5	62.2	93.2	74.3	112
				D	83.5	125	103	155	124	186	149	223
Group C	N	45.0	67.5	S	44.7	67.1	55.4	83.0	–	–	–	–
				D	89.5	134	111	166	–	–	–	–
	X	56.5	84.8	S	56.2	84.3	69.5	104	–	–	–	–
				D	112	169	139	209	–	–	–	–
A307	Not applicable	13.5	20.3	S	13.4	20.2	16.6	25.0	20.0	30.0	23.9	35.9
				D	26.8	40.4	33.2	49.9	40.0	60.1	47.8	71.9

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

– Indicates that this grade is unavailable in the given diameter.
For end loaded connections greater than 38 in., see AISC *Specification* Table J3.2 footnote b.
Group A includes ASTM F3125 Grades A325 and F1852 bolts.
Group B includes ASTM F3125 Grades A490 and F2280 bolts.
Group C includes ASTM F3043 and ASTM F3111.
Thread condition "N" indicates that threads are included in the shear plane.
Thread condition "X" indicates that threads are excluded from the shear plane.
S = single shear D = double shear

Table 7-2
Available Tensile Strength of Bolts, kips

Nominal Bolt Diameter, d, in.			5/8		3/4		7/8		1	
Nominal Bolt Area, in.^{2}			0.307		0.442		0.601		0.785	
Designation	F_{nt}/Ω (ksi)	ϕF_{nt} (ksi)	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Group A	45.0	67.5	13.8	20.7	19.9	29.8	27.1	40.6	35.3	53.0
Group B	56.5	84.8	17.3	26.0	25.0	37.4	34.0	51.0	44.4	66.6
Group C	75.0	113	–	–	–	–	–	–	58.9	88.4
A307	22.5	33.8	6.90	10.4	9.94	14.9	13.5	20.3	17.7	26.5

Nominal Bolt Diameter, d, in.			1 1/8		1 1/4		1 3/8		1 1/2	
Nominal Bolt Area, in.^{2}			0.994		1.23		1.48		1.77	
Designation	F_{nt}/Ω (ksi)	ϕF_{nt} (ksi)	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Group A	45.0	67.5	44.7	67.1	55.2	82.8	66.8	100	79.5	119
Group B	56.5	84.8	56.2	84.2	69.3	104	83.9	126	99.8	150
Group C	75.0	113	74.6	112	92.0	138	–	–	–	–
A307	22.5	33.8	22.4	33.5	27.6	41.4	33.4	50.1	39.8	59.6

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

– Indicates that this grade is unavailable in the given diameter.
Group A includes ASTM F3125 Grades A325 and F1852 bolts.
Group B includes ASTM F3125 Grades A490 and F2280 bolts.
Group C includes ASTM F3043 and ASTM F3111.

Group A Bolts (Includes A325 and F1852 bolts)

Table 7-3
Slip-Critical Connections
Available Slip Resistance, kips
(Class A Faying Surface, μ = 0.30)

Group A Bolts

Hole Type	Loading	Nominal Bolt Diameter, *d*, in.							
		5/8		3/4		7/8		1	
		Minimum Group A Bolt Pretension, kips							
		19		28		39		51	
		r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
STD/SSLT	S	4.29	6.44	6.33	9.49	8.81	13.2	11.5	17.3
	D	8.59	12.9	12.7	19.0	17.6	26.4	23.1	34.6
OVS/SSLP	S	3.66	5.47	5.39	8.07	7.51	11.2	9.82	14.7
	D	7.32	10.9	10.8	16.1	15.0	22.5	19.6	29.4
LSL	S	3.01	4.51	4.44	6.64	6.18	9.25	8.08	12.1
	D	6.02	9.02	8.87	13.3	12.4	18.5	16.2	24.2

Hole Type	Loading	Nominal Bolt Diameter, *d*, in.							
		1 1/8		1 1/4		1 3/8		1 1/2	
		Minimum Group A Bolt Pretension, kips							
		64		81		97		118	
		r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
STD/SSLT	S	14.5	21.7	18.3	27.5	21.9	32.9	26.7	40.0
	D	28.9	43.4	36.6	54.9	43.8	65.8	53.3	80.0
OVS/SSLP	S	12.3	18.4	15.6	23.3	18.7	28.0	22.7	34.0
	D	24.7	36.9	31.2	46.7	37.4	55.9	45.5	68.0
LSL	S	10.1	15.2	12.8	19.2	15.4	23.0	18.7	28.0
	D	20.3	30.4	25.7	38.4	30.7	46.0	37.4	56.0

STD = standard hole
OVS = oversized hole
SSLT = short-slotted hole with length transverse to the line of force
SSLP = short-slotted hole with length parallel to the line of force
LSL = long-slotted hole with length transverse or parallel to the line of force

S = single shear
D = double shear

Hole Type	ASD	LRFD
STD and SSLT	Ω = 1.50	ϕ = 1.00
OVS and SSLP	Ω = 1.76	ϕ = 0.85
LSL	Ω = 2.14	ϕ = 0.70

Note: Slip-critical bolt values assume no more than one filler has been provided or bolts have been added to distribute loads in the fillers.

See AISC *Specification* Sections J3.8 and J5 for provisions when fillers are present.

For Class B faying surfaces, multiply the tabulated available strength by 1.67.

Table 7-3 (continued)
Slip-Critical Connections
Available Slip Resistance, kips
(Class A Faying Surface, μ = 0.30)

Group B Bolts (Includes A490 and F2280 bolts)

Group B Bolts

Hole Type	Loading	Nominal Bolt Diameter, d, in.							
		5/8		3/4		7/8		1	
		Minimum Group B Bolt Pretension, kips							
		24		35		49		64	
		r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
STD/SSLT	S	5.42	8.14	7.91	11.9	11.1	16.6	14.5	21.7
	D	10.8	16.3	15.8	23.7	22.1	33.2	28.9	43.4
OVS/SSLP	S	4.62	6.92	6.74	10.1	9.44	14.1	12.3	18.4
	D	9.25	13.8	13.5	20.2	18.9	28.2	24.7	36.9
LSL	S	3.80	5.70	5.54	8.31	7.76	11.6	10.1	15.2
	D	7.60	11.4	11.1	16.6	15.5	23.3	20.3	30.4

Hole Type	Loading	Nominal Bolt Diameter, d, in.							
		1 1/8		1 1/4		1 3/8		1 1/2	
		Minimum Group B Bolt Pretension, kips							
		80		102		121		148	
		r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
STD/SSLT	S	18.1	27.1	23.1	34.6	27.3	41.0	33.4	50.2
	D	36.2	54.2	46.1	69.2	54.7	82.0	66.9	100
OVS/SSLP	S	15.4	23.1	19.6	29.4	23.3	34.9	28.5	42.6
	D	30.8	46.1	39.3	58.8	46.6	69.7	57.0	85.3
LSL	S	12.7	19.0	16.2	24.2	19.2	28.7	23.4	35.1
	D	25.3	38.0	32.3	48.4	38.3	57.4	46.9	70.2

STD = standard hole
OVS = oversized hole
SSLT = short-slotted hole with length transverse to the line of force
SSLP = short-slotted hole with length parallel to the line of force
LSL = long-slotted hole with length transverse or parallel to the line of force

S = single shear
D = double shear

Hole Type	ASD	LRFD
STD and SSLT	$\Omega = 1.50$	$\phi = 1.00$
OVS and SSLP	$\Omega = 1.76$	$\phi = 0.85$
LSL	$\Omega = 2.14$	$\phi = 0.70$

Note: Slip-critical bolt values assume no more than one filler has been provided or bolts have been added to distribute loads in the fillers.

See AISC *Specification* Sections J3.8 and J5 for provisions when fillers are present.

For Class B faying surfaces, multiply the tabulated available strength by 1.67.

Group C, Grade 2 Bolts

Table 7-3 (continued)
Slip-Critical Connections
Available Slip Resistance, kips
(Class A Faying Surface, μ = 0.30)

Group C Bolts

Hole Type	Loading	5/8		3/4		7/8		1	
		Minimum Group C Grade 2 Bolt Pretension, kips							
		–		–		–		90	
		r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
STD/SSLT	S	–	–	–	–	–	–	20.3	30.5
	D	–	–	–	–	–	–	40.7	61.0
OVS/SSLP	S	–	–	–	–	–	–	17.3	25.9
	D	–	–	–	–	–	–	34.7	51.9
LSL	S	–	–	–	–	–	–	14.3	21.4
	D	–	–	–	–	–	–	28.5	42.7

Hole Type	Loading	1 1/8		1 1/4		1 3/8		1 1/2	
		Minimum Group C Grade 2 Bolt Pretension, kips							
		113		143		–		–	
		r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
STD/SSLT	S	25.5	38.3	32.3	48.5	–	–	–	–
	D	51.1	76.6	64.6	97.0	–	–	–	–
OVS/SSLP	S	21.8	32.6	27.5	41.2	–	–	–	–
	D	43.5	65.1	55.1	82.4	–	–	–	–
LSL	S	17.9	26.8	22.7	33.9	–	–	–	–
	D	35.8	53.6	45.3	67.9	–	–	–	–

(Nominal Bolt Diameter, *d*, in.)

STD = standard hole
OVS = oversized hole
SSLT = short-slotted hole with length transverse to the line of force
SSLP = short-slotted hole with length parallel to the line of force
LSL = long-slotted hole with length transverse or parallel to the line of force

S = single shear
D = double shear

Hole Type	ASD	LRFD
STD and SSLT	Ω = 1.50	ϕ = 1.00
OVS and SSLP	Ω = 1.76	ϕ = 0.85
LSL	Ω = 2.14	ϕ = 0.70

– Indicates that this grade is unavailable for the given diameter.

Note: Slip-critical bolt values assume no more than one filler has been provided or bolts have been added to distribute loads in the fillers.

See AISC *Specification* Sections J3.8 and J5 for provisions when fillers are present.

For Class B faying surfaces, multiply the tabulated available strength by 1.67.

Table 7-4
Available Bearing and Tearout Strength at Bolt Holes Based on Bolt Spacing
kip/in. thickness

Hole Type	Bolt Spacing, s, in.	F_u, ksi	Nominal Bolt Diameter, d, in. 5/8		3/4		7/8		1	
			r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
STD SSLT	$2^2/_3\, d_b$	58	34.1	51.1	41.3	62.0	48.6	72.9	53.7	80.5
		65	38.2	57.3	46.3	69.5	54.4	81.7	60.1	90.2
	3 in.	58	43.5	65.3	52.2	78.3	60.9	91.4	65.3	97.9
		65	48.8	73.1	58.5	87.8	68.3	102	73.1	110
SSLP	$2^2/_3\, d_b$	58	27.6	41.3	34.8	52.2	42.1	63.1	47.1	70.7
		65	30.9	46.3	39.0	58.5	47.1	70.7	52.8	79.2
	3 in.	58	43.5	65.3	52.2	78.3	60.9	91.4	58.7	88.1
		65	48.8	73.1	58.5	87.8	68.3	102	65.8	98.7
OVS	$2^2/_3\, d_b$	58	29.7	44.6	37.0	55.5	44.2	66.3	49.3	74.0
		65	33.3	50.0	41.4	62.2	49.6	74.3	55.3	82.9
	3 in.	58	43.5	65.3	52.2	78.3	60.9	91.4	60.9	91.4
		65	48.8	73.1	58.5	87.8	68.3	102	68.3	102
LSLP	$2^2/_3\, d_b$	58	3.62	5.44	4.35	6.53	5.08	7.61	5.80	8.70
		65	4.06	6.09	4.88	7.31	5.69	8.53	6.50	9.75
	3 in.	58	43.5	65.3	39.2	58.7	28.3	42.4	17.4	26.1
		65	48.8	73.1	43.9	65.8	31.7	47.5	19.5	29.3
LSLT	$2^2/_3\, d_b$	58	28.4	42.6	34.4	51.7	40.5	60.7	44.7	67.1
		65	31.8	47.7	38.6	57.9	45.4	68.0	50.1	75.2
	3 in.	58	36.3	54.4	43.5	65.3	50.8	76.1	54.4	81.6
		65	40.6	60.9	48.8	73.1	56.9	85.3	60.9	91.4
STD, SSLT, SSLP, OVS, LSLP	$s \geq s_{full}$	58	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104
		65	48.8	73.1	58.5	87.8	68.3	102	78.0	117
LSLT	$s \geq s_{full}$	58	36.3	54.4	43.5	65.3	50.8	76.1	58.0	87.0
		65	40.6	60.9	48.8	73.1	56.9	85.3	65.0	97.5
Spacing for full bearing and tearout strength, s_{full}[a], in.		STD, SSLT, LSLT	$1^{15}/_{16}$		$2^5/_{16}$		$2^{11}/_{16}$		$3^1/_8$	
		OVS	$2^1/_{16}$		$2^7/_{16}$		$2^{13}/_{16}$		$3^1/_4$	
		SSLP	$2^1/_8$		$2^1/_2$		$2^7/_8$		$3^5/_{16}$	
		LSLP	$2^{13}/_{16}$		$3^3/_8$		$3^{15}/_{16}$		$4^1/_2$	
Minimum Spacing[a] $= 2^2/_3 d$, in.			$1^{11}/_{16}$		2		$2^5/_{16}$		$2^{11}/_{16}$	

STD = standard hole
SSLT = short-slotted hole oriented with length transverse to the line of force
SSLP = short-slotted hole oriented with length parallel to the line of force
OVS = oversized hole
LSLP = long-slotted hole oriented with length parallel to the line of force
LSLT = long-slotted hole oriented with length transverse to the line of force

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

Note: Spacing indicated is from the center of the hole or slot to the center of the adjacent hole or slot in the line of force. Hole deformation is considered. When hole deformation is not considered, see AISC *Specification* Section J3.10.

[a] Decimal value has been rounded to the nearest sixteenth of an inch.

Table 7-4 (continued)
Available Bearing and Tearout Strength at Bolt Holes Based on Bolt Spacing
kip/in. thickness

Hole Type	Bolt Spacing, s, in.	F_u, ksi	Nominal Bolt Diameter, d, in.							
			$1\frac{1}{8}$		$1\frac{1}{4}$		$1\frac{3}{8}$		$1\frac{1}{2}$	
			r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
STD SSLT	$2\frac{2}{3}d_b$	58	60.9	91.4	68.2	102	75.4	113	82.7	124
		65	68.3	102	76.4	115	84.5	127	92.6	139
	3 in.	58	60.9	91.4	–	–	–	–	–	–
		65	68.3	102	–	–	–	–	–	–
SSLP	$2\frac{2}{3}d_b$	58	52.2	78.3	59.5	89.2	66.7	100	74.0	111
		65	58.5	87.8	66.6	99.9	74.8	112	82.9	124
	3 in.	58	52.2	78.3	–	–	–	–	–	–
		65	58.5	87.8	–	–	–	–	–	–
OVS	$2\frac{2}{3}d_b$	58	54.4	81.6	61.6	92.4	68.9	103	76.1	114
		65	60.9	91.4	69.1	104	77.2	116	85.3	128
	3 in.	58	54.4	81.6	–	–	–	–	–	–
		65	60.9	91.4	–	–	–	–	–	–
LSLP	$2\frac{2}{3}d_b$	58	6.53	9.79	7.25	10.9	7.98	12.0	8.70	13.1
		65	7.31	11.0	8.13	12.2	8.94	13.4	9.75	14.6
	3 in.	58	6.53	9.79	–	–	–	–	–	–
		65	7.31	11.0	–	–	–	–	–	–
LSLT	$2\frac{2}{3}d_b$	58	50.8	76.1	56.8	85.2	62.8	94.3	68.9	103
		65	56.9	85.3	63.6	95.5	70.4	106	77.2	116
	3 in.	58	50.8	76.1	–	–	–	–	–	–
		65	56.9	85.3	–	–	–	–	–	–
STD, SSLT, SSLP, OVS, LSLP	$s \ge s_{full}$	58	78.3	117	87.0	131	95.7	144	104	157
		65	87.8	132	97.5	146	107	161	117	176
LSLT	$s \ge s_{full}$	58	65.3	97.9	72.5	109	79.8	120	87.0	131
		65	73.1	110	81.3	122	89.4	134	97.5	146
Spacing for full bearing and tearout strength s_{full}[a], in.		STD, SSLT, LSLT	$3\frac{1}{2}$		$3\frac{7}{8}$		$4\frac{1}{4}$		$4\frac{5}{8}$	
		OVS	$3\frac{11}{16}$		$4\frac{1}{16}$		$4\frac{7}{16}$		$4\frac{13}{16}$	
		SSLP	$3\frac{3}{4}$		$4\frac{1}{8}$		$4\frac{1}{2}$		$4\frac{7}{8}$	
		LSLP	$5\frac{1}{16}$		$5\frac{5}{8}$		$6\frac{3}{16}$		$6\frac{3}{4}$	
Minimum Spacing[a] = $2\frac{2}{3}d$, in.			3		$3\frac{5}{16}$		$3\frac{11}{16}$		4	

STD = standard hole
SSLT = short-slotted hole oriented with length transverse to the line of force
SSLP = short-slotted hole oriented with length parallel to the line of force
OVS = oversized hole
LSLP = long-slotted hole oriented with length parallel to the line of force
LSLT = long-slotted hole oriented with length transverse to the line of force

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

– Indicates spacing less than minimum spacing required per AISC *Specification* Section J3.3.

Note: Spacing indicated is from the center of the hole or slot to the center of the adjacent hole or slot in the line of force. Hole deformation is considered. When hole deformation is not considered, see AISC *Specification* Section J3.10.

[a] Decimal value has been rounded to the nearest sixteenth of an inch.

Table 7-5
Available Bearing and Tearout Strength at Bolt Holes Based on Edge Distance
kip/in. thickness

Hole Type	Edge Distance, l_e, in.	F_u, ksi	Nominal Bolt Diameter, d, in. 5/8 r_n/Ω ASD	5/8 ϕr_n LRFD	3/4 r_n/Ω ASD	3/4 ϕr_n LRFD	7/8 r_n/Ω ASD	7/8 ϕr_n LRFD	1 r_n/Ω ASD	1 ϕr_n LRFD
STD SSLT	1 1/4	58	31.5	47.3	29.4	44.0	27.2	40.8	23.9	35.9
		65	35.3	53.0	32.9	49.4	30.5	45.7	26.8	40.2
	2	58	43.5	65.3	52.2	78.3	53.3	79.9	50.0	75.0
		65	48.8	73.1	58.5	87.8	59.7	89.6	56.1	84.1
SSLP	1 1/4	58	28.3	42.4	26.1	39.2	23.9	35.9	20.7	31.0
		65	31.7	47.5	29.3	43.9	26.8	40.2	23.2	34.7
	2	58	43.5	65.3	52.2	78.3	50.0	75.0	46.8	70.1
		65	48.8	73.1	58.5	87.8	56.1	84.1	52.4	78.6
OVS	1 1/4	58	29.4	44.0	27.2	40.8	25.0	37.5	21.8	32.6
		65	32.9	49.4	30.5	45.7	28.0	42.0	24.4	36.6
	2	58	43.5	65.3	52.2	78.3	51.1	76.7	47.9	71.8
		65	48.8	73.1	58.5	87.8	57.3	85.9	53.6	80.4
LSLP	1 1/4	58	16.3	24.5	10.9	16.3	5.44	8.16	–	–
		65	18.3	27.4	12.2	18.3	6.09	9.14	–	–
	2	58	42.4	63.6	37.0	55.5	31.5	47.3	26.1	39.2
		65	47.5	71.3	41.4	62.2	35.3	53.0	29.3	43.9
LSLT	1 1/4	58	26.3	39.4	24.5	36.7	22.7	34.0	19.9	29.9
		65	29.5	44.2	27.4	41.1	25.4	38.1	22.3	33.5
	2	58	36.3	54.4	43.5	65.3	44.4	66.6	41.7	62.5
		65	40.6	60.9	48.8	73.1	49.8	74.6	46.7	70.1
STD, SSLT, SSLP, OVS, LSLP	$l_e \geq l_{e\,full}$	58	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104
		65	48.8	73.1	58.5	87.8	68.3	102	78.0	117
LSLT	$l_e \geq l_{e\,full}$	58	36.3	54.4	43.5	65.3	50.8	76.1	58.0	87.0
		65	40.6	60.9	48.8	73.1	56.9	85.3	65.0	97.5
Edge distance for full bearing and tearout strength $l_e \geq l_{e\,full}$[a], in.		STD, SSLT, LSLT	1 5/8		1 15/16		2 1/4		2 9/16	
		OVS	1 11/16		2		2 5/16		2 5/8	
		SSLP	1 11/16		2		2 5/16		2 11/16	
		LSLP	2 1/16		2 7/16		2 7/8		3 1/4	

STD = standard hole
SSLT = short-slotted hole oriented with length transverse to the line of force
SSLP = short-slotted hole oriented with length parallel to the line of force
OVS = oversized hole
LSLP = long-slotted hole oriented with length parallel to the line of force
LSLT = long-slotted hole oriented with length transverse to the line of force

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

– Indicates edge distance less than minimum required per AISC *Specification* Section J3.4.

Note: Edge distance indicated is from the center of the hole or slot to the edge of the element in the line of force. Hole deformation is considered. When hole deformation is not considered, see AISC *Specification* Section J3.10.

[a] Decimal value has been rounded to the nearest sixteenth of an inch.

Table 7-5 (continued)
Available Bearing and Tearout Strength at Bolt Holes Based on Edge Distance
kip/in. thickness

Hole Type	Edge Distance, l_e, in.	F_u, ksi	Nominal Bolt Diameter, d, in.							
			$1\frac{1}{8}$		$1\frac{1}{4}$		$1\frac{3}{8}$		$1\frac{1}{2}$	
			r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
STD SSLT	$1\frac{1}{4}$	58	21.8	32.6	19.6	29.4	17.4	26.1	15.2	22.8
		65	24.4	36.6	21.9	32.9	19.5	29.3	17.1	25.6
	2	58	47.9	71.8	45.7	68.5	43.5	65.3	41.3	62.0
		65	53.6	80.4	51.2	76.8	48.8	73.1	46.3	69.5
SSLP	$1\frac{1}{4}$	58	17.4	26.1	15.2	22.8	13.1	19.6	10.9	16.3
		65	19.5	29.3	17.1	25.6	14.6	21.9	12.2	18.3
	2	58	43.5	65.3	41.3	62.0	39.2	58.7	37.0	55.5
		65	48.8	73.1	46.3	69.5	43.9	65.8	41.4	62.2
OVS	$1\frac{1}{4}$	58	18.5	27.7	16.3	24.5	14.1	21.2	12.0	17.9
		65	20.7	31.1	18.3	27.4	15.8	23.8	13.4	20.1
	2	58	44.6	66.9	42.4	63.6	40.2	60.4	38.1	57.1
		65	50.0	75.0	47.5	71.3	45.1	67.6	42.7	64.0
LSLP	$1\frac{1}{4}$	58	–	–	–	–	–	–	–	–
		65	–	–	–	–	–	–	–	–
	2	58	20.7	31.0	15.2	22.8	9.79	14.7	4.35	6.53
		65	23.2	34.7	17.1	25.6	11.0	16.5	4.88	7.31
LSLT	$1\frac{1}{4}$	58	18.1	27.2	16.3	24.5	14.5	21.8	12.7	19.0
		65	20.3	30.5	18.3	27.4	16.3	24.4	14.2	21.3
	2	58	39.9	59.8	38.1	57.1	36.3	54.4	34.4	51.7
		65	44.7	67.0	42.7	64.0	40.6	60.9	38.6	57.9
STD, SSLT, SSLP, OVS, LSLP	$l_e \ge l_{e\,full}$	58	78.3	117	87.0	131	95.7	144	104	157
		65	87.8	132	97.5	146	107	161	117	176
LSLT	$l_e \ge l_{e\,full}$	58	65.3	97.9	72.5	109	79.8	120	87.0	131
		65	73.1	110	81.3	122	89.4	134	97.5	146

Edge distance for full bearing and tearout strength $l_e \ge l_{e\,full}$[a], in.	Hole Type	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$
	STD, SSLT, LSLT	$2\frac{7}{8}$	$3\frac{3}{16}$	$3\frac{1}{2}$	$3\frac{13}{16}$
	OVS	3	$3\frac{5}{16}$	$3\frac{5}{8}$	$3\frac{15}{16}$
	SSLP	3	$3\frac{5}{16}$	$3\frac{5}{8}$	$3\frac{15}{16}$
	LSLP	$3\frac{11}{16}$	$4\frac{1}{16}$	$4\frac{1}{2}$	$4\frac{7}{8}$

STD = standard hole
SSLT = short-slotted hole oriented with length transverse to the line of force
SSLP = short-slotted hole oriented with length parallel to the line of force
OVS = oversized hole
LSLP = long-slotted hole oriented with length parallel to the line of force
LSLT = long-slotted hole oriented with length transverse to the line of force

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

– Indicates edge distance less than minimum required per AISC *Specification* Section J3.4.

Note: Edge distance indicated is from the center of the hole or slot to the edge of the element in the line of force. Hole deformation is considered. When hole deformation is not considered, see AISC *Specification* Section J3.10.

[a] Decimal value has been rounded to the nearest sixteenth of an inch.

Table 7-6
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 0°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

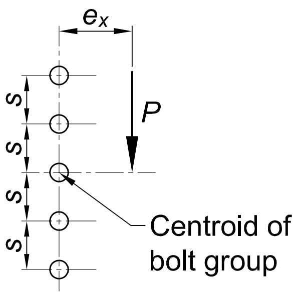
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*										
		2	3	4	5	6	7	8	9	10	11	12
3	1	1.63	2.71	3.75	4.77	5.77	6.77	7.76	8.75	9.74	10.7	11.7
	2	1.18	2.23	3.32	4.39	5.45	6.48	7.51	8.52	9.53	10.5	11.5
	3	0.88	1.75	2.81	3.90	4.98	6.06	7.12	8.17	9.21	10.2	11.3
	4	0.69	1.40	2.36	3.40	4.47	5.56	6.64	7.72	8.78	9.84	10.9
	5	0.56	1.15	2.01	2.96	3.98	5.05	6.13	7.22	8.30	9.38	10.4
	6	0.48	0.97	1.73	2.59	3.55	4.57	5.63	6.70	7.79	8.87	9.96
	7	0.41	0.83	1.51	2.28	3.17	4.13	5.15	6.20	7.28	8.36	9.44
	8	0.36	0.73	1.34	2.04	2.85	3.75	4.72	5.73	6.78	7.85	8.93
	9	0.32	0.65	1.21	1.83	2.59	3.42	4.34	5.31	6.32	7.36	8.42
	10	0.29	0.59	1.09	1.66	2.36	3.14	4.00	4.92	5.89	6.90	7.94
	12	0.24	0.49	0.92	1.40	2.00	2.68	3.44	4.27	5.15	6.09	7.06
	14	0.21	0.42	0.79	1.21	1.74	2.33	3.01	3.75	4.55	5.41	6.31
	16	0.18	0.37	0.70	1.06	1.53	2.06	2.67	3.33	4.06	4.85	5.68
	18	0.16	0.33	0.62	0.95	1.37	1.84	2.39	3.00	3.66	4.38	5.15
	20	0.15	0.29	0.56	0.85	1.24	1.67	2.16	2.72	3.33	3.99	4.70
	24	0.12	0.25	0.47	0.71	1.03	1.40	1.82	2.29	2.81	3.37	3.99
	28	0.11	0.21	0.40	0.61	0.89	1.20	1.57	1.97	2.42	2.92	3.45
	32	0.09	0.18	0.35	0.54	0.78	1.05	1.37	1.73	2.13	2.57	3.04
	36	0.08	0.16	0.31	0.48	0.69	0.94	1.22	1.54	1.90	2.29	2.72
	C′, in.	2.94	5.89	11.3	17.1	25.1	33.8	44.4	55.9	69.2	83.5	100
6	1	1.86	2.88	3.88	4.87	5.86	6.84	7.83	8.81	9.80	10.8	11.8
	2	1.63	2.71	3.75	4.77	5.77	6.77	7.76	8.75	9.74	10.7	11.7
	3	1.39	2.48	3.56	4.60	5.63	6.65	7.65	8.66	9.66	10.7	11.6
	4	1.18	2.23	3.32	4.39	5.45	6.48	7.51	8.52	9.53	10.5	11.5
	5	1.01	1.98	3.07	4.15	5.23	6.28	7.33	8.36	9.38	10.4	11.4
	6	0.88	1.75	2.81	3.90	4.98	6.06	7.12	8.17	9.21	10.2	11.3
	7	0.77	1.56	2.58	3.64	4.73	5.81	6.89	7.95	9.00	10.1	11.1
	8	0.69	1.40	2.36	3.40	4.47	5.56	6.64	7.72	8.78	9.84	10.9
	9	0.62	1.26	2.17	3.17	4.22	5.30	6.39	7.47	8.55	9.61	10.7
	10	0.56	1.15	2.01	2.96	3.98	5.05	6.13	7.22	8.30	9.38	10.4
	12	0.48	0.97	1.73	2.59	3.55	4.57	5.63	6.70	7.79	8.87	9.96
	14	0.41	0.83	1.51	2.28	3.17	4.13	5.15	6.20	7.28	8.36	9.44
	16	0.36	0.73	1.34	2.04	2.85	3.75	4.72	5.73	6.78	7.85	8.93
	18	0.32	0.65	1.21	1.83	2.59	3.42	4.34	5.31	6.32	7.36	8.42
	20	0.29	0.59	1.09	1.66	2.36	3.14	4.00	4.92	5.89	6.90	7.94
	24	0.24	0.49	0.92	1.40	2.00	2.68	3.44	4.27	5.15	6.09	7.06
	28	0.21	0.42	0.79	1.21	1.74	2.33	3.01	3.75	4.55	5.41	6.31
	32	0.18	0.37	0.70	1.06	1.53	2.06	2.67	3.33	4.06	4.85	5.68
	36	0.16	0.33	0.62	0.95	1.37	1.84	2.39	3.00	3.66	4.38	5.15
	C′, in.	5.89	11.8	22.5	34.3	50.2	67.6	88.8	112	138	167	199

Table 7-6 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 15°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

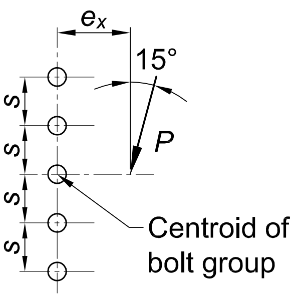
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n										
		2	3	4	5	6	7	8	9	10	11	12
	1	1.61	2.69	3.72	4.74	5.74	6.74	7.73	8.72	9.71	10.7	11.7
	2	1.15	2.20	3.28	4.34	5.39	6.42	7.45	8.46	9.47	10.5	11.5
	3	0.86	1.76	2.78	3.85	4.92	5.98	7.03	8.08	9.11	10.1	11.2
	4	0.67	1.42	2.35	3.36	4.41	5.48	6.55	7.61	8.67	9.72	10.8
	5	0.55	1.17	2.00	2.94	3.94	4.98	6.04	7.11	8.18	9.24	10.3
	6	0.47	0.99	1.73	2.58	3.52	4.52	5.55	6.61	7.67	8.74	9.81
	7	0.41	0.86	1.52	2.30	3.16	4.11	5.10	6.13	7.18	8.24	9.30
	8	0.36	0.75	1.35	2.06	2.86	3.74	4.69	5.68	6.70	7.74	8.80
	9	0.32	0.67	1.22	1.86	2.60	3.43	4.32	5.27	6.26	7.28	8.31
3	10	0.29	0.61	1.10	1.69	2.38	3.16	4.00	4.90	5.85	6.84	7.85
	12	0.24	0.51	0.93	1.43	2.03	2.71	3.46	4.28	5.15	6.06	7.01
	14	0.21	0.43	0.81	1.24	1.76	2.37	3.04	3.78	4.57	5.41	6.30
	16	0.19	0.38	0.71	1.09	1.56	2.10	2.70	3.37	4.09	4.87	5.69
	18	0.17	0.34	0.63	0.97	1.39	1.88	2.43	3.04	3.70	4.42	5.18
	20	0.15	0.30	0.57	0.88	1.26	1.70	2.20	2.76	3.37	4.03	4.74
	24	0.12	0.25	0.48	0.73	1.06	1.43	1.86	2.33	2.86	3.43	4.04
	28	0.11	0.22	0.41	0.63	0.91	1.23	1.60	2.02	2.47	2.97	3.51
	32	0.09	0.19	0.36	0.55	0.80	1.08	1.41	1.77	2.18	2.62	3.10
	36	0.08	0.17	0.32	0.49	0.71	0.96	1.26	1.58	1.95	2.34	2.78
	1	1.85	2.87	3.87	4.86	5.84	6.83	7.81	8.80	9.78	10.8	11.7
	2	1.61	2.69	3.72	4.74	5.74	6.74	7.73	8.73	9.71	10.7	11.7
	3	1.36	2.45	3.52	4.56	5.59	6.60	7.61	8.61	9.61	10.6	11.6
	4	1.15	2.20	3.28	4.34	5.39	6.42	7.45	8.46	9.47	10.5	11.5
	5	0.98	1.96	3.03	4.10	5.16	6.21	7.25	8.28	9.30	10.3	11.3
	6	0.86	1.76	2.78	3.85	4.92	5.98	7.03	8.08	9.11	10.1	11.2
	7	0.75	1.57	2.55	3.60	4.66	5.73	6.80	7.85	8.90	9.94	11.0
	8	0.67	1.42	2.35	3.36	4.41	5.48	6.55	7.61	8.67	9.72	10.8
	9	0.61	1.29	2.16	3.14	4.17	5.23	6.30	7.36	8.43	9.49	10.5
6	10	0.55	1.17	2.00	2.94	3.94	4.98	6.04	7.11	8.18	9.24	10.3
	12	0.47	0.99	1.73	2.58	3.52	4.52	5.55	6.61	7.67	8.74	9.81
	14	0.41	0.86	1.52	2.30	3.16	4.11	5.10	6.13	7.18	8.24	9.30
	16	0.36	0.75	1.35	2.06	2.86	3.74	4.69	5.68	6.70	7.74	8.80
	18	0.32	0.67	1.22	1.86	2.60	3.43	4.32	5.27	6.26	7.28	8.31
	20	0.29	0.61	1.10	1.69	2.38	3.16	4.00	4.90	5.85	6.84	7.85
	24	0.24	0.51	0.93	1.43	2.03	2.71	3.46	4.28	5.15	6.06	7.01
	28	0.21	0.43	0.81	1.24	1.76	2.37	3.04	3.78	4.57	5.41	6.30
	32	0.19	0.38	0.71	1.09	1.56	2.10	2.70	3.37	4.09	4.87	5.69
	36	0.17	0.34	0.63	0.97	1.39	1.88	2.43	3.04	3.70	4.42	5.18

Table 7-6 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 30°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

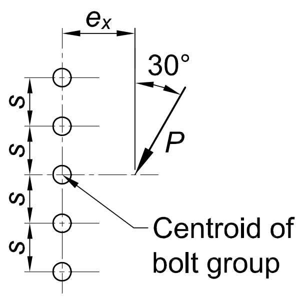
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*										
		2	3	4	5	6	7	8	9	10	11	12
	1	1.58	2.66	3.69	4.70	5.70	6.70	7.69	8.69	9.67	10.7	11.6
	2	1.14	2.20	3.25	4.30	5.33	6.36	7.38	8.39	9.40	10.4	11.4
	3	0.86	1.80	2.79	3.83	4.87	5.92	6.96	7.99	9.02	10.0	11.1
	4	0.69	1.50	2.40	3.39	4.41	5.45	6.49	7.53	8.57	9.61	10.6
	5	0.57	1.27	2.08	3.00	3.98	4.99	6.02	7.06	8.11	9.15	10.2
	6	0.49	1.09	1.82	2.68	3.60	4.57	5.58	6.60	7.64	8.68	9.72
	7	0.43	0.95	1.61	2.40	3.27	4.20	5.17	6.17	7.18	8.21	9.25
	8	0.38	0.83	1.44	2.17	2.98	3.86	4.79	5.76	6.75	7.77	8.79
	9	0.34	0.75	1.30	1.98	2.74	3.57	4.46	5.39	6.35	7.34	8.35
3	10	0.31	0.67	1.19	1.82	2.52	3.31	4.15	5.05	5.98	6.95	7.93
	12	0.26	0.56	1.01	1.55	2.17	2.87	3.64	4.46	5.33	6.24	7.17
	14	0.23	0.48	0.87	1.35	1.90	2.53	3.23	3.98	4.78	5.63	6.51
	16	0.20	0.42	0.77	1.20	1.69	2.26	2.89	3.58	4.33	5.11	5.94
	18	0.18	0.38	0.69	1.07	1.52	2.04	2.62	3.25	3.94	4.67	5.45
	20	0.16	0.34	0.62	0.97	1.37	1.85	2.38	2.97	3.61	4.30	5.02
	24	0.14	0.28	0.52	0.81	1.16	1.57	2.02	2.53	3.09	3.69	4.33
	28	0.12	0.24	0.45	0.70	1.00	1.36	1.75	2.20	2.69	3.22	3.79
	32	0.10	0.21	0.40	0.61	0.88	1.19	1.54	1.94	2.38	2.85	3.37
	36	0.09	0.19	0.35	0.55	0.78	1.07	1.38	1.74	2.13	2.56	3.03
	1	1.83	2.85	3.85	4.84	5.83	6.81	7.80	8.78	9.76	10.7	11.7
	2	1.59	2.66	3.69	4.70	5.71	6.70	7.70	8.69	9.68	10.7	11.7
	3	1.34	2.43	3.48	4.52	5.54	6.55	7.55	8.56	9.55	10.6	11.5
	4	1.14	2.20	3.25	4.30	5.33	6.36	7.38	8.39	9.40	10.4	11.4
	5	0.98	1.99	3.02	4.06	5.11	6.14	7.17	8.20	9.22	10.2	11.2
	6	0.86	1.80	2.79	3.83	4.87	5.92	6.96	7.99	9.02	10.0	11.1
	7	0.77	1.64	2.59	3.60	4.64	5.68	6.73	7.77	8.80	9.83	10.9
	8	0.69	1.50	2.40	3.39	4.41	5.45	6.49	7.53	8.57	9.61	10.6
	9	0.63	1.37	2.23	3.19	4.19	5.22	6.26	7.30	8.34	9.38	10.4
6	10	0.57	1.27	2.08	3.00	3.98	4.99	6.02	7.06	8.11	9.15	10.2
	12	0.49	1.09	1.82	2.68	3.60	4.57	5.58	6.60	7.64	8.68	9.72
	14	0.43	0.95	1.61	2.40	3.27	4.20	5.17	6.17	7.18	8.21	9.25
	16	0.38	0.83	1.44	2.17	2.98	3.86	4.79	5.76	6.75	7.77	8.79
	18	0.34	0.75	1.30	1.98	2.74	3.57	4.46	5.39	6.35	7.34	8.35
	20	0.31	0.67	1.19	1.82	2.52	3.31	4.15	5.05	5.98	6.95	7.93
	24	0.26	0.56	1.01	1.55	2.17	2.87	3.64	4.46	5.33	6.24	7.17
	28	0.23	0.48	0.87	1.35	1.90	2.53	3.23	3.98	4.78	5.63	6.51
	32	0.20	0.42	0.77	1.20	1.69	2.26	2.89	3.58	4.33	5.11	5.94
	36	0.18	0.38	0.69	1.07	1.52	2.04	2.62	3.25	3.94	4.67	5.45

Table 7-6 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 45°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

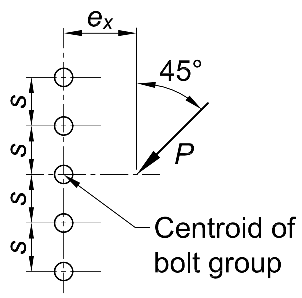
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*										
		2	3	4	5	6	7	8	9	10	11	12
3	1	1.57	2.64	3.66	4.66	5.66	6.66	7.65	8.64	9.63	10.6	11.6
	2	1.17	2.23	3.26	4.28	5.29	6.30	7.31	8.32	9.32	10.3	11.3
	3	0.92	1.89	2.87	3.87	4.88	5.90	6.91	7.93	8.94	9.95	11.0
	4	0.75	1.63	2.54	3.50	4.49	5.49	6.51	7.52	8.53	9.55	10.6
	5	0.64	1.42	2.25	3.17	4.13	5.11	6.11	7.11	8.12	9.14	10.2
	6	0.55	1.25	2.01	2.88	3.80	4.76	5.73	6.73	7.73	8.73	9.74
	7	0.49	1.11	1.81	2.63	3.51	4.43	5.38	6.36	7.34	8.34	9.34
	8	0.44	0.99	1.64	2.41	3.25	4.14	5.06	6.01	6.98	7.96	8.96
	9	0.40	0.90	1.49	2.22	3.02	3.87	4.77	5.69	6.64	7.61	8.58
	10	0.36	0.81	1.37	2.06	2.82	3.63	4.50	5.39	6.32	7.27	8.23
	12	0.31	0.68	1.17	1.79	2.47	3.22	4.02	4.87	5.74	6.65	7.58
	14	0.27	0.59	1.03	1.58	2.20	2.88	3.62	4.41	5.24	6.11	6.99
	16	0.24	0.52	0.91	1.41	1.97	2.60	3.29	4.03	4.81	5.63	6.48
	18	0.21	0.46	0.82	1.27	1.78	2.36	3.00	3.70	4.43	5.21	6.02
	20	0.19	0.41	0.74	1.16	1.62	2.16	2.76	3.41	4.10	4.84	5.61
	24	0.16	0.35	0.63	0.98	1.38	1.85	2.37	2.94	3.56	4.22	4.92
	28	0.14	0.30	0.54	0.85	1.19	1.61	2.08	2.58	3.14	3.73	4.37
	32	0.12	0.26	0.48	0.75	1.05	1.43	1.84	2.30	2.80	3.34	3.92
	36	0.11	0.23	0.43	0.67	0.94	1.28	1.65	2.07	2.53	3.02	3.55
6	1	1.83	2.85	3.85	4.84	5.83	6.81	7.80	8.78	9.76	10.7	11.7
	2	1.57	2.64	3.66	4.67	5.67	6.66	7.66	8.65	9.64	10.6	11.6
	3	1.35	2.43	3.46	4.48	5.49	6.49	7.50	8.49	9.49	10.5	11.5
	4	1.17	2.23	3.26	4.28	5.29	6.30	7.31	8.32	9.32	10.3	11.3
	5	1.03	2.05	3.06	4.07	5.09	6.10	7.12	8.13	9.13	10.1	11.1
	6	0.92	1.89	2.87	3.87	4.88	5.90	6.91	7.93	8.94	9.95	11.0
	7	0.83	1.75	2.70	3.68	4.68	5.69	6.71	7.72	8.74	9.75	10.8
	8	0.75	1.63	2.54	3.50	4.49	5.49	6.51	7.52	8.53	9.55	10.6
	9	0.69	1.52	2.39	3.33	4.30	5.30	6.30	7.31	8.33	9.34	10.4
	10	0.64	1.42	2.25	3.17	4.13	5.11	6.11	7.11	8.12	9.14	10.2
	12	0.55	1.25	2.01	2.88	3.80	4.76	5.73	6.73	7.73	8.73	9.74
	14	0.49	1.11	1.81	2.63	3.51	4.43	5.38	6.36	7.34	8.34	9.34
	16	0.44	0.99	1.64	2.41	3.25	4.14	5.06	6.01	6.98	7.96	8.96
	18	0.40	0.90	1.49	2.22	3.02	3.87	4.77	5.69	6.64	7.61	8.58
	20	0.36	0.81	1.37	2.06	2.82	3.63	4.50	5.39	6.32	7.27	8.23
	24	0.31	0.68	1.17	1.79	2.47	3.22	4.02	4.87	5.74	6.65	7.58
	28	0.27	0.59	1.03	1.58	2.20	2.88	3.62	4.41	5.24	6.11	6.99
	32	0.24	0.52	0.91	1.41	1.97	2.60	3.29	4.03	4.81	5.63	6.48
	36	0.21	0.46	0.82	1.27	1.78	2.36	3.00	3.70	4.43	5.21	6.02

Table 7-6 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 60°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

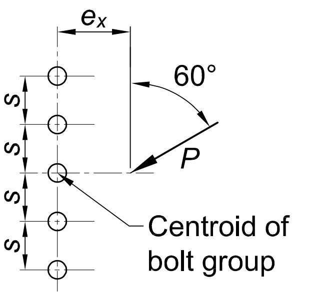
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*										
		2	3	4	5	6	7	8	9	10	11	12
3	1	1.61	2.65	3.65	4.64	5.63	6.62	7.60	8.59	9.57	10.6	11.5
	2	1.27	2.32	3.32	4.31	5.30	6.30	7.29	8.27	9.27	10.3	11.3
	3	1.05	2.05	3.02	4.00	4.98	5.97	6.96	7.94	8.94	9.93	10.9
	4	0.89	1.83	2.77	3.72	4.69	5.66	6.64	7.62	8.61	9.60	10.6
	5	0.77	1.65	2.54	3.47	4.41	5.37	6.34	7.32	8.29	9.28	10.3
	6	0.68	1.49	2.34	3.24	4.16	5.10	6.06	7.02	7.99	8.97	9.95
	7	0.61	1.37	2.17	3.03	3.93	4.85	5.79	6.74	7.71	8.67	9.64
	8	0.56	1.26	2.01	2.83	3.71	4.61	5.54	6.48	7.43	8.39	9.35
	9	0.51	1.16	1.87	2.66	3.51	4.39	5.30	6.23	7.17	8.12	9.07
	10	0.47	1.07	1.74	2.50	3.32	4.19	5.08	5.99	6.92	7.86	8.81
	12	0.40	0.93	1.52	2.22	3.00	3.82	4.67	5.55	6.45	7.37	8.30
	14	0.35	0.81	1.35	2.00	2.73	3.50	4.32	5.16	6.03	6.92	7.83
	16	0.32	0.72	1.21	1.81	2.49	3.23	4.00	4.81	5.65	6.51	7.40
	18	0.29	0.65	1.09	1.66	2.30	2.98	3.72	4.50	5.31	6.14	7.00
	20	0.26	0.58	1.00	1.53	2.12	2.77	3.47	4.21	4.99	5.80	6.63
	24	0.22	0.49	0.85	1.32	1.84	2.41	3.05	3.73	4.45	5.21	5.99
	28	0.19	0.42	0.74	1.15	1.61	2.13	2.71	3.34	4.00	4.70	5.44
	32	0.17	0.37	0.65	1.02	1.43	1.91	2.44	3.02	3.63	4.28	4.97
	36	0.15	0.33	0.59	0.92	1.29	1.72	2.21	2.74	3.31	3.92	4.57
6	1	1.81	2.82	3.81	4.79	5.78	6.76	7.74	8.73	9.71	10.7	11.7
	2	1.60	2.65	3.65	4.64	5.64	6.63	7.62	8.61	9.60	10.6	11.6
	3	1.42	2.48	3.48	4.48	5.47	6.46	7.45	8.44	9.44	10.4	11.4
	4	1.27	2.32	3.32	4.31	5.30	6.30	7.29	8.27	9.27	10.3	11.3
	5	1.15	2.18	3.17	4.15	5.14	6.13	7.12	8.11	9.10	10.1	11.1
	6	1.05	2.05	3.02	4.00	4.98	5.97	6.96	7.94	8.94	9.93	10.9
	7	0.96	1.93	2.89	3.86	4.83	5.81	6.80	7.78	8.77	9.76	10.8
	8	0.89	1.83	2.77	3.72	4.69	5.66	6.64	7.62	8.61	9.60	10.6
	9	0.83	1.73	2.65	3.59	4.55	5.51	6.49	7.47	8.45	9.43	10.4
	10	0.77	1.65	2.54	3.47	4.41	5.37	6.34	7.32	8.29	9.28	10.3
	12	0.68	1.49	2.34	3.24	4.16	5.10	6.06	7.02	7.99	8.97	9.95
	14	0.61	1.37	2.17	3.03	3.93	4.85	5.79	6.74	7.71	8.67	9.64
	16	0.56	1.26	2.01	2.83	3.71	4.61	5.54	6.48	7.43	8.39	9.35
	18	0.51	1.16	1.87	2.66	3.51	4.39	5.30	6.23	7.17	8.12	9.07
	20	0.47	1.07	1.74	2.50	3.32	4.19	5.08	5.99	6.92	7.86	8.81
	24	0.40	0.93	1.52	2.22	3.00	3.82	4.67	5.55	6.45	7.37	8.30
	28	0.35	0.81	1.35	2.00	2.73	3.50	4.32	5.16	6.03	6.92	7.83
	32	0.32	0.72	1.21	1.81	2.49	3.23	4.00	4.81	5.65	6.51	7.40
	36	0.29	0.65	1.09	1.66	2.30	2.98	3.72	4.50	5.31	6.14	7.00

Table 7-6 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 75°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

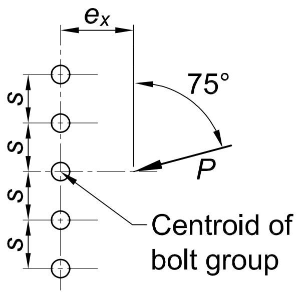
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*										
		2	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**
	1	1.72	2.72	3.70	4.68	5.66	6.64	7.62	8.59	9.58	10.6	11.5
	2	1.49	2.51	3.49	4.46	5.44	6.42	7.40	8.38	9.36	10.3	11.3
	3	1.32	2.33	3.30	4.27	5.24	6.21	7.18	8.15	9.13	10.1	11.1
	4	1.18	2.18	3.14	4.09	5.05	6.01	6.98	7.95	8.92	9.89	10.9
	5	1.07	2.04	2.99	3.93	4.88	5.84	6.79	7.75	8.72	9.68	10.7
	6	0.98	1.92	2.85	3.79	4.73	5.67	6.62	7.57	8.53	9.49	10.5
	7	0.90	1.82	2.73	3.65	4.58	5.52	6.46	7.40	8.36	9.31	10.3
	8	0.84	1.72	2.62	3.52	4.44	5.37	6.30	7.24	8.19	9.14	10.1
	9	0.78	1.63	2.51	3.40	4.31	5.23	6.16	7.09	8.03	8.97	9.92
3	10	0.73	1.55	2.41	3.29	4.19	5.10	6.02	6.94	7.88	8.81	9.76
	12	0.65	1.41	2.23	3.08	3.95	4.84	5.75	6.66	7.59	8.51	9.45
	14	0.58	1.30	2.06	2.88	3.73	4.60	5.50	6.40	7.31	8.23	9.16
	16	0.53	1.20	1.92	2.70	3.52	4.38	5.26	6.15	7.05	7.96	8.88
	18	0.48	1.11	1.78	2.53	3.33	4.17	5.03	5.91	6.80	7.70	8.61
	20	0.44	1.03	1.66	2.38	3.16	3.97	4.82	5.69	6.56	7.45	8.35
	24	0.38	0.89	1.46	2.12	2.85	3.63	4.44	5.27	6.13	6.99	7.87
	28	0.34	0.79	1.29	1.90	2.59	3.33	4.11	4.91	5.73	6.57	7.43
	32	0.30	0.70	1.16	1.73	2.38	3.08	3.81	4.58	5.37	6.19	7.02
	36	0.27	0.62	1.05	1.58	2.19	2.85	3.55	4.28	5.05	5.84	6.65
	1	1.84	2.83	3.81	4.79	5.77	6.75	7.70	8.71	9.70	10.7	11.7
	2	1.71	2.72	3.70	4.69	5.67	6.66	7.64	8.79	9.78	10.8	11.7
	3	1.60	2.61	3.59	4.57	5.55	6.53	7.52	8.50	9.48	10.5	11.5
	4	1.49	2.51	3.49	4.46	5.44	6.42	7.40	8.38	9.36	10.3	11.3
	5	1.40	2.42	3.39	4.37	5.34	6.31	7.29	8.26	9.24	10.2	11.2
	6	1.32	2.33	3.30	4.27	5.24	6.21	7.18	8.15	9.13	10.1	11.1
	7	1.25	2.25	3.22	4.18	5.14	6.11	7.07	8.05	9.01	10.0	11.0
	8	1.18	2.18	3.14	4.09	5.05	6.01	6.98	7.95	8.92	9.89	10.9
	9	1.13	2.11	3.06	4.01	4.97	5.92	6.88	7.85	8.81	9.78	10.8
6	10	1.07	2.04	2.99	3.93	4.88	5.84	6.79	7.75	8.72	9.68	10.7
	12	0.98	1.92	2.85	3.79	4.73	5.67	6.62	7.57	8.53	9.49	10.5
	14	0.90	1.82	2.73	3.65	4.58	5.52	6.46	7.40	8.36	9.31	10.3
	16	0.84	1.72	2.62	3.52	4.44	5.37	6.30	7.24	8.19	9.14	10.1
	18	0.78	1.63	2.51	3.40	4.31	5.23	6.16	7.09	8.03	8.97	9.92
	20	0.73	1.55	2.41	3.29	4.19	5.10	6.02	6.94	7.88	8.81	9.76
	24	0.65	1.41	2.23	3.08	3.95	4.84	5.75	6.66	7.59	8.51	9.45
	28	0.58	1.30	2.06	2.88	3.73	4.60	5.50	6.40	7.31	8.23	9.16
	32	0.53	1.20	1.92	2.70	3.52	4.38	5.26	6.15	7.05	7.96	8.88
	36	0.48	1.11	1.78	2.53	3.33	4.17	5.03	5.91	6.80	7.70	8.61

Table 7-7
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 0°

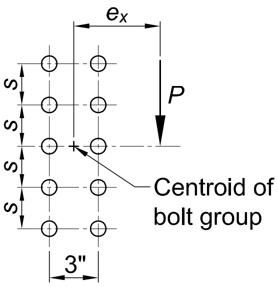
Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where
- P = required force, P_u or P_a, kips
- r_n = nominal strength per bolt, kips
- e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
- s = bolt spacing, in.
- C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	0.84	2.54	4.48	6.59	8.72	10.8	12.9	15.0	17.0	19.0	21.0	23.0
	3	0.65	2.03	3.68	5.67	7.77	9.91	12.1	14.2	16.3	18.3	20.4	22.5
	4	0.54	1.67	3.06	4.86	6.84	8.93	11.1	13.2	15.4	17.5	19.6	21.7
	5	0.45	1.42	2.59	4.21	6.01	8.00	10.1	12.2	14.4	16.5	18.7	20.8
	6	0.39	1.22	2.25	3.69	5.32	7.17	9.16	11.2	13.4	15.5	17.7	19.8
	7	0.35	1.08	1.99	3.27	4.74	6.46	8.33	10.3	12.4	14.5	16.7	18.8
	8	0.31	0.96	1.78	2.93	4.27	5.86	7.60	9.50	11.5	13.6	15.7	17.8
	9	0.28	0.86	1.60	2.65	3.87	5.34	6.97	8.75	10.7	12.7	14.7	16.8
	10	0.26	0.78	1.46	2.42	3.53	4.90	6.42	8.10	9.91	11.8	13.8	15.9
	12	0.22	0.66	1.24	2.06	3.01	4.19	5.51	7.01	8.63	10.4	12.2	14.2
	14	0.19	0.57	1.08	1.78	2.62	3.66	4.82	6.15	7.61	9.19	10.9	12.7
	16	0.17	0.51	0.95	1.57	2.32	3.24	4.27	5.47	6.79	8.23	9.78	11.4
	18	0.15	0.45	0.85	1.41	2.07	2.90	3.83	4.92	6.11	7.43	8.85	10.4
	20	0.14	0.41	0.77	1.27	1.88	2.63	3.48	4.47	5.55	6.76	8.07	9.48
	24	0.12	0.34	0.65	1.07	1.58	2.21	2.93	3.77	4.69	5.72	6.85	8.06
	28	0.10	0.29	0.56	0.92	1.36	1.90	2.53	3.25	4.05	4.95	5.93	7.00
	32	0.09	0.26	0.49	0.80	1.19	1.67	2.22	2.86	3.57	4.36	5.23	6.18
	36	0.08	0.23	0.43	0.72	1.06	1.49	1.98	2.55	3.18	3.90	4.67	5.52
	C', in.	2.94	8.33	15.8	26.0	38.7	54.2	72.2	93.1	117	143	172	204
6	2	0.84	3.24	5.39	7.47	9.51	11.5	13.5	15.5	17.5	19.5	21.5	23.4
	3	0.65	2.79	4.93	7.08	9.17	11.2	13.3	15.3	17.3	19.3	21.3	23.3
	4	0.54	2.41	4.44	6.60	8.75	10.9	12.9	15.0	17.0	19.1	21.1	23.1
	5	0.45	2.10	3.97	6.11	8.27	10.4	12.5	14.6	16.7	18.7	20.8	22.8
	6	0.39	1.85	3.55	5.62	7.77	9.93	12.1	14.2	16.3	18.4	20.4	22.5
	7	0.35	1.64	3.18	5.17	7.27	9.43	11.6	13.7	15.9	18.0	20.1	22.1
	8	0.31	1.47	2.87	4.75	6.79	8.92	11.1	13.3	15.4	17.5	19.6	21.7
	9	0.28	1.34	2.61	4.39	6.34	8.43	10.6	12.7	14.9	17.1	19.2	21.3
	10	0.26	1.22	2.39	4.06	5.92	7.96	10.1	12.2	14.4	16.6	18.7	20.9
	12	0.22	1.04	2.04	3.52	5.20	7.10	9.12	11.2	13.4	15.5	17.7	19.9
	14	0.19	0.90	1.77	3.09	4.61	6.36	8.27	10.3	12.4	14.5	16.7	18.9
	16	0.17	0.80	1.57	2.75	4.12	5.74	7.52	9.44	11.5	13.5	15.7	17.8
	18	0.15	0.71	1.41	2.48	3.72	5.21	6.87	8.68	10.6	12.6	14.7	16.8
	20	0.14	0.64	1.28	2.25	3.38	4.77	6.31	8.02	9.85	11.8	13.8	15.9
	24	0.12	0.54	1.07	1.90	2.86	4.06	5.40	6.91	8.55	10.3	12.2	14.1
	28	0.10	0.46	0.93	1.64	2.47	3.52	4.70	6.05	7.52	9.12	10.8	12.6
	32	0.09	0.41	0.81	1.44	2.18	3.11	4.16	5.37	6.69	8.15	9.71	11.4
	36	0.08	0.36	0.73	1.29	1.94	2.78	3.72	4.81	6.02	7.34	8.78	10.3
	C', in.	2.94	13.2	26.5	47.0	71.4	103	138	180	226	279	337	400

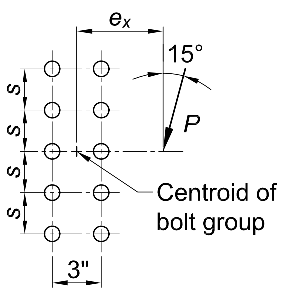
Table 7-7 (continued)
Coefficients C for Eccentrically Loaded Bolt Groups
Angle = 15°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = Cr_n$$
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where
- P = required force, P_u or P_a, kips
- r_n = nominal strength per bolt, kips
- e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
- s = bolt spacing, in.
- C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	0.87	2.54	4.47	6.54	8.63	10.7	12.8	14.8	16.9	18.9	20.9	22.9
	3	0.68	2.04	3.71	5.63	7.69	9.80	11.9	14.0	16.1	18.2	20.2	22.3
	4	0.55	1.69	3.11	4.85	6.79	8.84	10.9	13.1	15.2	17.3	19.4	21.5
	5	0.47	1.44	2.66	4.21	6.00	7.94	9.98	12.1	14.2	16.3	18.4	20.5
	6	0.41	1.25	2.31	3.70	5.34	7.15	9.09	11.1	13.2	15.3	17.4	19.6
	7	0.36	1.10	2.04	3.29	4.79	6.46	8.30	10.2	12.3	14.3	16.4	18.6
	8	0.32	0.98	1.83	2.96	4.32	5.87	7.60	9.45	11.4	13.4	15.5	17.6
	9	0.29	0.88	1.65	2.68	3.94	5.37	6.99	8.74	10.6	12.6	14.6	16.6
	10	0.27	0.81	1.51	2.45	3.61	4.93	6.45	8.11	9.88	11.8	13.7	15.7
	12	0.23	0.68	1.28	2.09	3.08	4.24	5.58	7.05	8.66	10.4	12.2	14.1
	14	0.20	0.59	1.11	1.82	2.69	3.71	4.90	6.21	7.67	9.23	10.9	12.7
	16	0.17	0.52	0.98	1.61	2.38	3.29	4.36	5.54	6.86	8.29	9.83	11.5
	18	0.16	0.47	0.88	1.44	2.13	2.96	3.92	4.99	6.20	7.51	8.93	10.4
	20	0.14	0.42	0.79	1.31	1.93	2.68	3.56	4.54	5.65	6.85	8.17	9.57
	24	0.12	0.35	0.67	1.10	1.62	2.26	3.00	3.84	4.79	5.82	6.96	8.17
	28	0.10	0.30	0.57	0.94	1.40	1.95	2.60	3.32	4.15	5.05	6.05	7.12
	32	0.09	0.27	0.50	0.83	1.23	1.72	2.28	2.93	3.66	4.46	5.34	6.29
	36	0.08	0.24	0.45	0.74	1.10	1.53	2.04	2.61	3.27	3.98	4.78	5.64
6	2	0.87	3.21	5.35	7.42	9.45	11.5	13.5	15.5	17.4	19.4	21.4	23.4
	3	0.68	2.76	4.88	7.00	9.09	11.1	13.2	15.2	17.2	19.2	21.2	23.2
	4	0.55	2.38	4.40	6.53	8.65	10.7	12.8	14.9	16.9	18.9	20.9	22.9
	5	0.47	2.07	3.96	6.04	8.17	10.3	12.4	14.5	16.5	18.6	20.6	22.6
	6	0.41	1.83	3.56	5.56	7.67	9.80	11.9	14.0	16.1	18.2	20.3	22.3
	7	0.36	1.63	3.22	5.12	7.19	9.30	11.4	13.6	15.7	17.8	19.9	21.9
	8	0.32	1.47	2.92	4.73	6.72	8.81	10.9	13.1	15.2	17.3	19.4	21.5
	9	0.29	1.34	2.66	4.37	6.29	8.33	10.4	12.6	14.7	16.8	18.9	21.0
	10	0.27	1.23	2.45	4.05	5.90	7.88	9.95	12.1	14.2	16.3	18.5	20.6
	12	0.23	1.05	2.09	3.53	5.21	7.06	9.04	11.1	13.2	15.3	17.5	19.6
	14	0.20	0.91	1.83	3.11	4.64	6.35	8.22	10.2	12.2	14.3	16.5	18.6
	16	0.17	0.81	1.62	2.78	4.17	5.75	7.51	9.38	11.4	13.4	15.5	17.6
	18	0.16	0.72	1.45	2.50	3.77	5.24	6.88	8.66	10.5	12.5	14.5	16.6
	20	0.14	0.66	1.32	2.28	3.45	4.80	6.34	8.02	9.82	11.7	13.7	15.7
	24	0.12	0.55	1.11	1.93	2.93	4.10	5.46	6.95	8.57	10.3	12.1	14.0
	28	0.10	0.48	0.96	1.67	2.54	3.57	4.78	6.11	7.58	9.15	10.8	12.6
	32	0.09	0.42	0.84	1.47	2.24	3.16	4.24	5.44	6.77	8.21	9.75	11.4
	36	0.08	0.37	0.75	1.32	2.00	2.83	3.80	4.89	6.10	7.42	8.85	10.4

Table 7-7 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 30°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

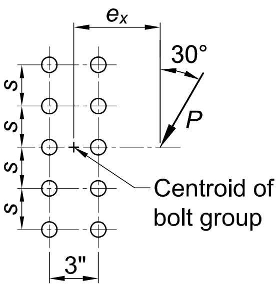
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	0.97	2.60	4.52	6.54	8.59	10.6	12.7	14.7	16.7	18.8	20.8	22.8
	3	0.75	2.12	3.83	5.71	7.71	9.75	11.8	13.9	15.9	18.0	20.0	22.1
	4	0.62	1.78	3.29	4.99	6.88	8.87	10.9	13.0	15.1	17.1	19.2	21.3
	5	0.52	1.53	2.85	4.39	6.16	8.06	10.0	12.1	14.1	16.2	18.3	20.4
	6	0.45	1.34	2.51	3.89	5.54	7.33	9.23	11.2	13.2	15.3	17.3	19.4
	7	0.40	1.19	2.23	3.48	5.01	6.70	8.51	10.4	12.4	14.4	16.4	18.5
	8	0.36	1.07	2.00	3.15	4.57	6.14	7.86	9.68	11.6	13.6	15.6	17.6
	9	0.32	0.97	1.81	2.87	4.19	5.66	7.28	9.02	10.9	12.8	14.7	16.7
	10	0.30	0.88	1.66	2.64	3.87	5.24	6.77	8.43	10.2	12.0	13.9	15.9
	12	0.25	0.75	1.41	2.27	3.34	4.54	5.92	7.43	9.04	10.8	12.5	14.4
	14	0.22	0.65	1.23	1.98	2.93	3.99	5.24	6.61	8.09	9.67	11.4	13.1
	16	0.19	0.58	1.08	1.76	2.60	3.56	4.69	5.94	7.30	8.77	10.3	12.0
	18	0.17	0.52	0.97	1.58	2.34	3.21	4.24	5.38	6.64	8.0	9.45	11.0
	20	0.16	0.47	0.88	1.43	2.12	2.92	3.87	4.92	6.08	7.3	8.70	10.1
	24	0.13	0.39	0.74	1.21	1.79	2.48	3.29	4.18	5.19	6.3	7.48	8.75
	28	0.12	0.34	0.64	1.04	1.55	2.14	2.85	3.63	4.52	5.5	6.54	7.68
	32	0.10	0.30	0.56	0.92	1.36	1.89	2.51	3.21	4.00	4.9	5.81	6.83
	36	0.09	0.26	0.50	0.82	1.21	1.69	2.25	2.87	3.59	4.4	5.22	6.15
6	2	0.97	3.20	5.31	7.37	9.39	11.4	13.4	15.4	17.4	19.4	21.3	23.3
	3	0.75	2.75	4.86	6.95	9.01	11.1	13.1	15.1	17.1	19.1	21.1	23.1
	4	0.62	2.39	4.42	6.49	8.57	10.6	12.7	14.7	16.8	18.8	20.8	22.8
	5	0.52	2.10	4.02	6.04	8.11	10.2	12.3	14.3	16.4	18.4	20.4	22.5
	6	0.45	1.87	3.67	5.61	7.66	9.73	11.8	13.9	16.0	18.0	20.1	22.1
	7	0.40	1.69	3.36	5.21	7.21	9.27	11.4	13.4	15.5	17.6	19.6	21.7
	8	0.36	1.53	3.08	4.84	6.79	8.82	10.9	13.0	15.1	17.1	19.2	21.3
	9	0.32	1.40	2.84	4.51	6.40	8.39	10.4	12.5	14.6	16.7	18.7	20.8
	10	0.30	1.29	2.63	4.21	6.04	7.98	9.99	12.0	14.1	16.2	18.3	20.4
	12	0.25	1.12	2.28	3.70	5.39	7.23	9.16	11.2	13.2	15.3	17.3	19.4
	14	0.22	0.98	2.00	3.29	4.86	6.57	8.41	10.3	12.3	14.4	16.4	18.5
	16	0.19	0.87	1.78	2.95	4.40	6.01	7.75	9.6	11.5	13.5	15.5	17.6
	18	0.17	0.79	1.60	2.68	4.02	5.52	7.17	8.9	10.8	12.7	14.7	16.7
	20	0.16	0.71	1.45	2.45	3.70	5.09	6.65	8.3	10.1	12.0	13.9	15.9
	24	0.13	0.60	1.23	2.08	3.17	4.39	5.79	7.3	8.95	10.7	12.5	14.4
	28	0.12	0.52	1.06	1.82	2.77	3.85	5.11	6.5	7.99	9.59	11.3	13.0
	32	0.10	0.46	0.93	1.61	2.45	3.42	4.56	5.8	7.20	8.68	10.3	11.9
	36	0.09	0.41	0.83	1.44	2.20	3.08	4.12	5.3	6.53	7.91	9.37	10.9

Table 7-7 (continued)
Coefficients C for Eccentrically Loaded Bolt Groups
Angle = 45°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

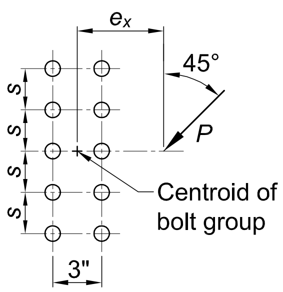
P = required force, P_u or P_a, kips

r_n = nominal strength per bolt, kips

e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.17	2.79	4.67	6.62	8.61	10.6	12.6	14.6	16.6	18.6	20.6	22.6
	3	0.92	2.32	4.06	5.92	7.86	9.83	11.8	13.9	15.9	17.9	19.9	21.9
	4	0.75	1.99	3.57	5.31	7.16	9.09	11.1	13.1	15.1	17.1	19.1	21.1
	5	0.64	1.74	3.17	4.78	6.53	8.39	10.3	12.3	14.3	16.3	18.3	20.3
	6	0.55	1.54	2.84	4.33	5.98	7.76	9.63	11.6	13.5	15.5	17.5	19.5
	7	0.49	1.38	2.57	3.93	5.49	7.20	9.00	10.9	12.8	14.8	16.7	18.7
	8	0.44	1.25	2.33	3.60	5.06	6.70	8.43	10.3	12.1	14.0	16.0	18.0
	9	0.40	1.14	2.13	3.31	4.69	6.25	7.91	9.67	11.5	13.4	15.3	17.2
	10	0.36	1.05	1.96	3.06	4.36	5.85	7.44	9.14	10.9	12.7	14.6	16.5
	12	0.31	0.90	1.68	2.65	3.83	5.17	6.63	8.20	9.86	11.6	13.4	15.2
	14	0.27	0.78	1.47	2.33	3.40	4.61	5.95	7.41	8.97	10.6	12.3	14.1
	16	0.24	0.69	1.31	2.08	3.05	4.16	5.38	6.74	8.20	9.75	11.4	13.1
	18	0.21	0.62	1.17	1.88	2.76	3.77	4.91	6.18	7.55	9.00	10.5	12.1
	20	0.19	0.56	1.06	1.71	2.52	3.45	4.51	5.69	6.97	8.34	9.80	11.3
	24	0.16	0.48	0.90	1.45	2.14	2.94	3.87	4.91	6.04	7.26	8.57	9.95
	28	0.14	0.41	0.77	1.26	1.86	2.56	3.38	4.30	5.30	6.41	7.59	8.85
	32	0.12	0.36	0.68	1.11	1.64	2.27	3.00	3.82	4.73	5.73	6.80	7.94
	36	0.11	0.32	0.61	0.99	1.47	2.03	2.70	3.44	4.26	5.17	6.15	7.20
6	2	1.17	3.24	5.30	7.32	9.33	11.3	13.3	15.3	17.3	19.3	21.3	23.2
	3	0.92	2.84	4.90	6.93	8.96	11.0	13.0	15.0	17.0	19.0	21.0	23.0
	4	0.75	2.51	4.52	6.53	8.56	10.6	12.6	14.6	16.6	18.6	20.6	22.6
	5	0.64	2.24	4.17	6.15	8.15	10.2	12.2	14.2	16.2	18.3	20.3	22.3
	6	0.55	2.03	3.86	5.78	7.76	9.77	11.8	13.8	15.8	17.9	19.9	21.9
	7	0.49	1.85	3.59	5.45	7.39	9.38	11.4	13.4	15.4	17.5	19.5	21.5
	8	0.44	1.70	3.35	5.13	7.03	9.00	11.0	13.0	15.0	17.1	19.1	21.1
	9	0.40	1.57	3.13	4.85	6.70	8.63	10.6	12.6	14.6	16.7	18.7	20.7
	10	0.36	1.46	2.94	4.58	6.38	8.28	10.2	12.2	14.2	16.3	18.3	20.3
	12	0.31	1.28	2.60	4.11	5.81	7.64	9.54	11.5	13.5	15.5	17.5	19.5
	14	0.27	1.13	2.32	3.71	5.31	7.06	8.89	10.8	12.7	14.7	16.7	18.7
	16	0.24	1.01	2.09	3.36	4.88	6.55	8.31	10.2	12.0	14.0	15.9	17.9
	18	0.21	0.92	1.90	3.07	4.50	6.09	7.78	9.56	11.4	13.3	15.2	17.2
	20	0.19	0.84	1.73	2.83	4.18	5.69	7.31	9.02	10.8	12.7	14.6	16.5
	24	0.16	0.72	1.47	2.43	3.64	5.00	6.48	8.08	9.76	11.5	13.3	15.2
	28	0.14	0.62	1.28	2.13	3.22	4.45	5.80	7.28	8.86	10.5	12.2	14.0
	32	0.12	0.55	1.13	1.90	2.88	3.99	5.24	6.62	8.09	9.65	11.3	13.0
	36	0.11	0.49	1.01	1.71	2.61	3.62	4.77	6.05	7.43	8.90	10.4	12.1

Table 7-7 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 60°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

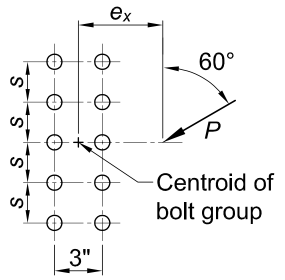
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.51	3.17	4.97	6.85	8.77	10.7	12.7	14.6	16.6	18.6	20.6	22.5
	3	1.24	2.76	4.47	6.30	8.19	10.1	12.0	14.0	16.0	17.9	19.9	21.9
	4	1.04	2.43	4.04	5.81	7.65	9.53	11.5	13.4	15.3	17.3	19.3	21.2
	5	0.89	2.16	3.70	5.39	7.17	9.01	10.9	12.8	14.7	16.7	18.6	20.6
	6	0.77	1.95	3.40	5.01	6.73	8.52	10.4	12.3	14.2	16.1	18.0	20.0
	7	0.68	1.77	3.13	4.67	6.33	8.07	9.88	11.7	13.6	15.5	17.4	19.4
	8	0.61	1.62	2.90	4.37	5.96	7.65	9.42	11.2	13.1	15.0	16.9	18.8
	9	0.56	1.49	2.70	4.09	5.62	7.26	8.98	10.8	12.6	14.5	16.3	18.2
	10	0.51	1.38	2.52	3.84	5.31	6.89	8.58	10.3	12.1	14.0	15.8	17.7
	12	0.43	1.20	2.21	3.40	4.76	6.25	7.85	9.53	11.3	13.0	14.9	16.7
	14	0.38	1.06	1.96	3.05	4.30	5.71	7.23	8.83	10.5	12.2	14.0	15.8
	16	0.34	0.95	1.76	2.75	3.92	5.24	6.68	8.20	9.79	11.5	13.2	14.9
	18	0.30	0.85	1.60	2.51	3.59	4.84	6.19	7.64	9.16	10.8	12.4	14.1
	20	0.27	0.78	1.46	2.30	3.32	4.48	5.76	7.14	8.60	10.1	11.7	13.4
	24	0.23	0.66	1.24	1.97	2.87	3.90	5.04	6.29	7.64	9.06	10.6	12.1
	28	0.20	0.57	1.07	1.72	2.52	3.44	4.47	5.61	6.85	8.17	9.55	11.0
	32	0.18	0.50	0.95	1.52	2.24	3.07	4.01	5.06	6.20	7.41	8.70	10.1
	36	0.16	0.45	0.85	1.37	2.02	2.77	3.63	4.59	5.65	6.77	7.98	9.26
6	2	1.51	3.39	5.36	7.33	9.31	11.3	13.3	15.2	17.2	19.2	21.2	23.2
	3	1.24	3.08	5.04	7.01	8.98	11.0	12.9	14.9	16.9	18.9	20.9	22.8
	4	1.04	2.80	4.73	6.69	8.66	10.6	12.6	14.6	16.6	18.6	20.5	22.5
	5	0.89	2.57	4.45	6.39	8.35	10.3	12.3	14.3	16.2	18.2	20.2	22.2
	6	0.77	2.37	4.20	6.11	8.05	10.0	12.0	13.9	15.9	17.9	19.9	21.8
	7	0.68	2.19	3.98	5.85	7.76	9.70	11.7	13.6	15.6	17.6	19.5	21.5
	8	0.61	2.04	3.77	5.61	7.49	9.41	11.4	13.3	15.3	17.2	19.2	21.2
	9	0.56	1.91	3.59	5.38	7.24	9.13	11.1	13.0	15.0	16.9	18.9	20.9
	10	0.51	1.80	3.42	5.17	7.00	8.87	10.8	12.7	14.7	16.6	18.6	20.5
	12	0.43	1.60	3.11	4.78	6.54	8.37	10.2	12.1	14.1	16.0	18.0	19.9
	14	0.38	1.44	2.85	4.43	6.13	7.91	9.74	11.6	13.5	15.4	17.4	19.3
	16	0.34	1.31	2.63	4.12	5.74	7.48	9.27	11.1	13.0	14.9	16.8	18.7
	18	0.30	1.20	2.43	3.84	5.40	7.08	8.84	10.7	12.5	14.4	16.3	18.2
	20	0.27	1.10	2.26	3.58	5.08	6.71	8.43	10.2	12.0	13.9	15.7	17.6
	24	0.23	0.95	1.97	3.15	4.53	6.06	7.69	9.39	11.2	12.9	14.8	16.6
	28	0.20	0.84	1.73	2.80	4.08	5.52	7.06	8.68	10.4	12.1	13.9	15.7
	32	0.18	0.74	1.54	2.52	3.71	5.05	6.51	8.05	9.66	11.3	13.1	14.8
	36	0.16	0.67	1.39	2.28	3.39	4.65	6.02	7.49	9.03	10.7	12.3	14.0

Table 7-7 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 75°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

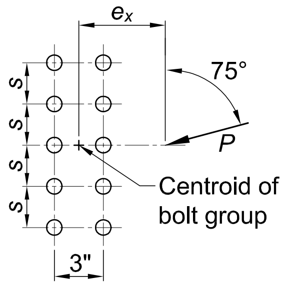
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.84	3.63	5.44	7.29	9.17	11.1	13.0	14.9	16.9	18.8	20.8	22.7
	3	1.71	3.41	5.17	6.97	8.82	10.7	12.6	14.5	16.4	18.4	20.3	22.3
	4	1.57	3.19	4.90	6.67	8.50	10.4	12.2	14.1	16.0	18.0	19.9	21.8
	5	1.44	2.98	4.65	6.39	8.19	10.0	11.9	13.8	15.7	17.6	19.5	21.4
	6	1.31	2.79	4.41	6.12	7.90	9.71	11.6	13.4	15.3	17.2	19.1	21.0
	7	1.20	2.61	4.19	5.88	7.62	9.42	11.3	13.1	15.0	16.9	18.8	20.7
	8	1.10	2.45	3.99	5.65	7.37	9.14	11.0	12.8	14.7	16.5	18.4	20.3
	9	1.01	2.31	3.81	5.43	7.14	8.89	10.7	12.5	14.3	16.2	18.1	20.0
	10	0.93	2.18	3.63	5.23	6.91	8.65	10.4	12.2	14.1	15.9	17.8	19.6
	12	0.81	1.95	3.33	4.86	6.49	8.19	9.94	11.7	13.5	15.3	17.2	19.0
	14	0.71	1.77	3.06	4.53	6.11	7.76	9.47	11.2	13.0	14.8	16.6	18.4
	16	0.63	1.61	2.83	4.23	5.75	7.36	9.03	10.8	12.5	14.3	16.1	17.9
	18	0.57	1.48	2.63	3.96	5.42	6.98	8.61	10.3	12.0	13.8	15.6	17.4
	20	0.52	1.36	2.45	3.72	5.12	6.63	8.23	9.88	11.6	13.3	15.1	16.9
	24	0.44	1.18	2.15	3.30	4.60	6.02	7.53	9.12	10.8	12.4	14.2	15.9
	28	0.38	1.04	1.91	2.95	4.16	5.49	6.93	8.45	10.0	11.7	13.3	15.0
	32	0.34	0.92	1.71	2.67	3.78	5.04	6.41	7.86	9.37	10.9	12.6	14.2
	36	0.30	0.83	1.55	2.43	3.47	4.65	5.94	7.32	8.78	10.3	11.9	13.5
6	2	1.84	3.66	5.55	7.48	9.42	11.4	13.3	15.3	17.6	19.6	21.5	23.5
	3	1.71	3.49	5.36	7.27	9.20	11.2	13.1	15.1	17.0	19.0	21.0	22.9
	4	1.57	3.32	5.18	7.08	9.00	10.9	12.9	14.8	16.8	18.7	20.7	22.7
	5	1.44	3.16	5.01	6.89	8.81	10.7	12.7	14.6	16.6	18.5	20.5	22.4
	6	1.31	3.02	4.84	6.72	8.62	10.5	12.5	14.4	16.3	18.3	20.2	22.2
	7	1.20	2.88	4.69	6.55	8.44	10.4	12.3	14.2	16.1	18.1	20.0	22.0
	8	1.10	2.75	4.54	6.39	8.27	10.2	12.1	14.0	15.9	17.9	19.8	21.8
	9	1.01	2.63	4.40	6.24	8.11	10.0	11.9	13.8	15.7	17.7	19.6	21.5
	10	0.93	2.52	4.27	6.09	7.95	9.83	11.7	13.6	15.6	17.5	19.4	21.3
	12	0.81	2.32	4.03	5.82	7.66	9.52	11.4	13.3	15.2	17.1	19.0	20.9
	14	0.71	2.15	3.82	5.57	7.38	9.22	11.1	13.0	14.9	16.7	18.7	20.6
	16	0.63	2.00	3.62	5.35	7.13	8.95	10.8	12.7	14.5	16.4	18.3	20.2
	18	0.57	1.87	3.44	5.14	6.90	8.69	10.5	12.4	14.2	16.1	18.0	19.9
	20	0.52	1.75	3.28	4.94	6.67	8.45	10.3	12.1	13.9	15.8	17.7	19.5
	24	0.44	1.55	2.98	4.57	6.24	7.98	9.75	11.6	13.4	15.2	17.1	18.9
	28	0.38	1.40	2.74	4.24	5.85	7.54	9.28	11.1	12.9	14.7	16.5	18.3
	32	0.34	1.27	2.52	3.95	5.49	7.13	8.83	10.6	12.4	14.1	16.0	17.8
	36	0.30	1.16	2.33	3.68	5.16	6.75	8.41	10.1	11.9	13.7	15.4	17.3

Table 7-8
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 0°

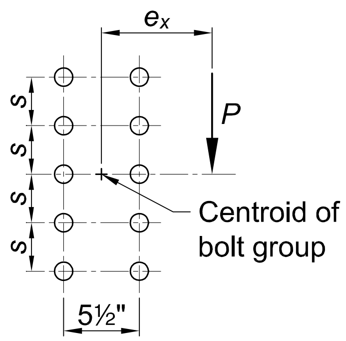
Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where
P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.14	2.75	4.59	6.61	8.69	10.8	12.9	14.9	17.0	19.0	21.0	23.0
	3	0.94	2.32	3.92	5.80	7.82	9.90	12.0	14.1	16.2	18.3	20.4	22.4
	4	0.80	1.99	3.39	5.10	6.98	9.00	11.1	13.2	15.3	17.4	19.6	21.7
	5	0.70	1.74	2.96	4.51	6.24	8.15	10.2	12.3	14.4	16.5	18.6	20.8
	6	0.62	1.54	2.62	4.03	5.60	7.39	9.30	11.3	13.4	15.5	17.7	19.8
	7	0.55	1.38	2.36	3.63	5.07	6.72	8.53	10.5	12.5	14.6	16.7	18.8
	8	0.50	1.25	2.14	3.30	4.61	6.15	7.84	9.67	11.6	13.6	15.7	17.8
	9	0.46	1.14	1.96	3.01	4.22	5.66	7.23	8.97	10.8	12.8	14.8	16.9
	10	0.42	1.04	1.80	2.78	3.89	5.23	6.70	8.34	10.1	12.0	13.9	15.9
	12	0.37	0.90	1.55	2.39	3.36	4.53	5.82	7.28	8.87	10.6	12.4	14.3
	14	0.32	0.79	1.36	2.10	2.96	3.99	5.13	6.44	7.87	9.42	11.1	12.8
	16	0.29	0.70	1.21	1.87	2.64	3.55	4.58	5.76	7.05	8.47	9.99	11.6
	18	0.26	0.63	1.09	1.68	2.37	3.20	4.14	5.21	6.38	7.68	9.08	10.6
	20	0.24	0.57	0.99	1.53	2.16	2.91	3.77	4.75	5.82	7.02	8.30	9.69
	24	0.20	0.48	0.84	1.29	1.83	2.46	3.19	4.03	4.94	5.97	7.07	8.28
	28	0.18	0.42	0.73	1.11	1.58	2.13	2.77	3.49	4.29	5.19	6.15	7.21
	32	0.16	0.37	0.64	0.98	1.39	1.88	2.44	3.08	3.79	4.58	5.44	6.38
	36	0.14	0.33	0.57	0.88	1.24	1.68	2.18	2.75	3.39	4.10	4.87	5.72
	C', in.	5.40	12.3	21.2	32.3	45.8	61.8	80.3	102	125	152	181	213
6	2	1.14	3.25	5.37	7.45	9.49	11.5	13.5	15.5	17.5	19.5	21.4	23.4
	3	0.94	2.86	4.93	7.05	9.14	11.2	13.2	15.3	17.3	19.3	21.3	23.3
	4	0.80	2.52	4.47	6.59	8.72	10.8	12.9	15.0	17.0	19.0	21.0	23.1
	5	0.70	2.24	4.04	6.12	8.25	10.4	12.5	14.6	16.7	18.7	20.8	22.8
	6	0.62	2.00	3.65	5.66	7.77	9.91	12.1	14.2	16.3	18.4	20.4	22.5
	7	0.55	1.80	3.31	5.23	7.29	9.42	11.6	13.7	15.8	17.9	20.0	22.1
	8	0.50	1.64	3.02	4.84	6.83	8.93	11.1	13.2	15.4	17.5	19.6	21.7
	9	0.46	1.50	2.77	4.49	6.39	8.45	10.6	12.7	14.9	17.0	19.2	21.3
	10	0.42	1.38	2.56	4.18	5.99	7.99	10.1	12.2	14.4	16.5	18.7	20.8
	12	0.37	1.19	2.21	3.65	5.29	7.16	9.15	11.2	13.4	15.5	17.7	19.8
	14	0.32	1.04	1.95	3.24	4.72	6.44	8.32	10.3	12.4	14.5	16.7	18.8
	16	0.29	0.93	1.74	2.90	4.24	5.83	7.59	9.48	11.5	13.6	15.7	17.8
	18	0.26	0.84	1.57	2.62	3.84	5.31	6.95	8.74	10.7	12.6	14.7	16.8
	20	0.24	0.76	1.43	2.39	3.50	4.87	6.39	8.08	9.89	11.8	13.8	15.9
	24	0.20	0.64	1.21	2.02	2.98	4.16	5.49	6.99	8.61	10.4	12.2	14.1
	28	0.18	0.55	1.05	1.76	2.59	3.63	4.80	6.13	7.59	9.18	10.9	12.7
	32	0.16	0.49	0.93	1.55	2.29	3.21	4.25	5.45	6.77	8.21	9.76	11.4
	36	0.14	0.43	0.83	1.38	2.05	2.88	3.81	4.90	6.09	7.41	8.83	10.4
	C', in.	5.40	16.0	30.6	51.0	76.2	107	143	185	232	284	342	406

Table 7-8 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 15°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

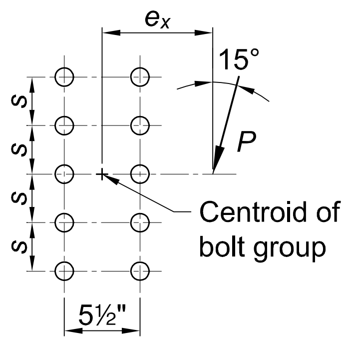
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.18	2.78	4.61	6.59	8.64	10.7	12.8	14.8	16.8	18.9	20.9	22.9
	3	0.97	2.34	3.97	5.80	7.78	9.83	11.9	14.0	16.1	18.1	20.2	22.2
	4	0.83	2.02	3.45	5.11	6.97	8.94	11.0	13.1	15.2	17.3	19.3	21.4
	5	0.72	1.77	3.03	4.54	6.26	8.12	10.1	12.1	14.2	16.3	18.4	20.5
	6	0.64	1.57	2.70	4.06	5.65	7.39	9.27	11.2	13.3	15.4	17.5	19.6
	7	0.57	1.41	2.43	3.66	5.13	6.74	8.52	10.4	12.4	14.4	16.5	18.6
	8	0.52	1.28	2.20	3.34	4.68	6.18	7.86	9.65	11.6	13.5	15.6	17.6
	9	0.48	1.17	2.01	3.06	4.30	5.70	7.27	8.97	10.8	12.7	14.7	16.7
	10	0.44	1.07	1.85	2.82	3.98	5.27	6.76	8.36	10.1	11.9	13.8	15.8
	12	0.38	0.93	1.60	2.44	3.44	4.58	5.90	7.34	8.91	10.6	12.4	14.2
	14	0.33	0.81	1.40	2.15	3.03	4.05	5.22	6.51	7.94	9.47	11.1	12.8
	16	0.30	0.72	1.25	1.91	2.70	3.62	4.68	5.84	7.14	8.54	10.1	11.7
	18	0.27	0.65	1.13	1.72	2.44	3.27	4.23	5.28	6.48	7.77	9.16	10.7
	20	0.25	0.59	1.02	1.57	2.22	2.98	3.86	4.83	5.93	7.11	8.40	9.78
	24	0.21	0.50	0.87	1.33	1.88	2.53	3.27	4.11	5.05	6.07	7.19	8.39
	28	0.18	0.43	0.75	1.15	1.63	2.19	2.84	3.57	4.39	5.29	6.28	7.33
	32	0.16	0.38	0.66	1.01	1.43	1.93	2.50	3.15	3.88	4.68	5.56	6.50
	36	0.14	0.34	0.59	0.90	1.28	1.73	2.24	2.82	3.48	4.19	4.99	5.84
6	2	1.18	3.24	5.34	7.40	9.43	11.5	13.5	15.4	17.4	19.4	21.4	23.4
	3	0.97	2.85	4.90	6.99	9.07	11.1	13.2	15.2	17.2	19.2	21.2	23.2
	4	0.83	2.51	4.45	6.53	8.63	10.7	12.8	14.8	16.9	18.9	20.9	22.9
	5	0.72	2.23	4.05	6.07	8.16	10.3	12.4	14.5	16.5	18.6	20.6	22.6
	6	0.64	2.00	3.68	5.62	7.69	9.80	11.9	14.0	16.1	18.2	20.2	22.3
	7	0.57	1.81	3.36	5.20	7.22	9.31	11.4	13.5	15.7	17.7	19.8	21.9
	8	0.52	1.65	3.08	4.82	6.78	8.83	10.9	13.1	15.2	17.3	19.4	21.5
	9	0.48	1.52	2.83	4.48	6.36	8.37	10.5	12.6	14.7	16.8	18.9	21.0
	10	0.44	1.40	2.62	4.18	5.98	7.93	9.97	12.1	14.2	16.3	18.4	20.6
	12	0.38	1.21	2.27	3.66	5.31	7.13	9.08	11.1	13.2	15.3	17.4	19.6
	14	0.33	1.07	2.00	3.25	4.76	6.44	8.28	10.2	12.3	14.3	16.4	18.6
	16	0.30	0.95	1.79	2.92	4.29	5.85	7.58	9.43	11.4	13.4	15.5	17.6
	18	0.27	0.86	1.62	2.65	3.90	5.34	6.97	8.72	10.6	12.5	14.6	16.6
	20	0.25	0.78	1.47	2.42	3.58	4.91	6.43	8.09	9.87	11.7	13.7	15.7
	24	0.21	0.66	1.25	2.06	3.05	4.21	5.55	7.03	8.64	10.4	12.2	14.1
	28	0.18	0.57	1.08	1.79	2.66	3.68	4.87	6.19	7.65	9.22	10.9	12.6
	32	0.16	0.50	0.95	1.58	2.35	3.26	4.33	5.52	6.84	8.27	9.81	11.4
	36	0.14	0.45	0.85	1.42	2.11	2.93	3.90	4.97	6.18	7.49	8.91	10.4

Table 7-8 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 30°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

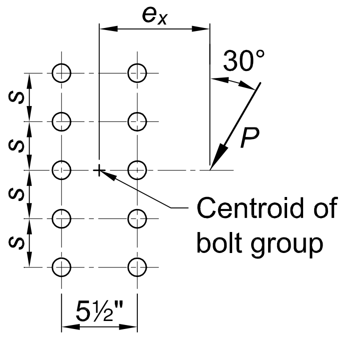
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.30	2.90	4.72	6.66	8.65	10.7	12.7	14.7	16.7	18.7	20.8	22.8
	3	1.08	2.47	4.13	5.94	7.86	9.85	11.9	13.9	16.0	18.0	20.0	22.1
	4	0.92	2.14	3.64	5.30	7.12	9.04	11.0	13.0	15.1	17.1	19.2	21.2
	5	0.80	1.89	3.24	4.76	6.46	8.29	10.2	12.2	14.2	16.3	18.3	20.4
	6	0.71	1.69	2.91	4.29	5.88	7.61	9.45	11.4	13.4	15.4	17.4	19.5
	7	0.64	1.53	2.63	3.90	5.38	7.01	8.76	10.6	12.5	14.5	16.5	18.6
	8	0.58	1.39	2.40	3.57	4.95	6.49	8.14	9.92	11.8	13.7	15.7	17.7
	9	0.53	1.28	2.20	3.29	4.58	6.02	7.59	9.29	11.1	12.9	14.9	16.8
	10	0.49	1.18	2.03	3.04	4.26	5.61	7.09	8.72	10.4	12.2	14.1	16.0
	12	0.42	1.02	1.76	2.65	3.72	4.92	6.25	7.73	9.31	11.0	12.8	14.6
	14	0.37	0.90	1.55	2.34	3.29	4.37	5.58	6.93	8.38	9.93	11.6	13.3
	16	0.33	0.80	1.38	2.09	2.95	3.92	5.03	6.26	7.59	9.03	10.6	12.2
	18	0.30	0.72	1.25	1.89	2.67	3.55	4.57	5.70	6.93	8.27	9.70	11.2
	20	0.27	0.66	1.13	1.73	2.43	3.25	4.19	5.23	6.36	7.62	8.95	10.4
	24	0.23	0.56	0.96	1.46	2.07	2.77	3.57	4.47	5.47	6.56	7.73	8.99
	28	0.20	0.48	0.83	1.27	1.79	2.41	3.11	3.90	4.78	5.75	6.78	7.91
	32	0.18	0.43	0.73	1.12	1.58	2.13	2.76	3.46	4.25	5.11	6.04	7.06
	36	0.16	0.38	0.66	1.00	1.42	1.91	2.47	3.10	3.81	4.59	5.44	6.36
6	2	1.30	3.27	5.33	7.36	9.38	11.4	13.4	15.4	17.4	19.3	21.3	23.3
	3	1.08	2.89	4.91	6.96	9.01	11.0	13.1	15.1	17.1	19.1	21.1	23.1
	4	0.92	2.56	4.50	6.53	8.58	10.6	12.7	14.7	16.8	18.8	20.8	22.8
	5	0.80	2.29	4.13	6.10	8.14	10.2	12.3	14.3	16.4	18.4	20.4	22.5
	6	0.71	2.08	3.80	5.69	7.70	9.75	11.8	13.9	15.9	18.0	20.0	22.1
	7	0.64	1.89	3.51	5.31	7.27	9.30	11.4	13.4	15.5	17.6	19.6	21.7
	8	0.58	1.74	3.25	4.96	6.86	8.86	10.9	13.0	15.0	17.1	19.2	21.3
	9	0.53	1.61	3.02	4.64	6.49	8.44	10.5	12.5	14.6	16.7	18.7	20.8
	10	0.49	1.49	2.81	4.35	6.13	8.04	10.0	12.1	14.1	16.2	18.3	20.4
	12	0.42	1.30	2.47	3.85	5.51	7.31	9.22	11.2	13.2	15.3	17.3	19.4
	14	0.37	1.15	2.19	3.44	4.98	6.67	8.49	10.4	12.4	14.4	16.4	18.5
	16	0.33	1.03	1.96	3.11	4.54	6.12	7.83	9.66	11.6	13.5	15.6	17.6
	18	0.30	0.93	1.78	2.83	4.16	5.63	7.26	9.00	10.8	12.8	14.7	16.7
	20	0.27	0.85	1.62	2.60	3.83	5.21	6.74	8.41	10.2	12.0	13.9	15.9
	24	0.23	0.72	1.38	2.23	3.30	4.51	5.89	7.40	9.02	10.7	12.5	14.4
	28	0.20	0.63	1.20	1.95	2.89	3.96	5.21	6.59	8.07	9.66	11.3	13.1
	32	0.18	0.55	1.06	1.73	2.57	3.53	4.67	5.92	7.28	8.75	10.3	12.0
	36	0.16	0.50	0.95	1.55	2.31	3.18	4.22	5.36	6.61	7.98	9.43	11.0

Table 7-8 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 45°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

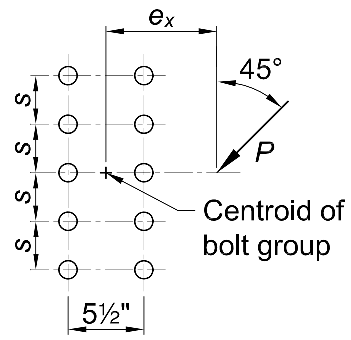
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.53	3.18	4.96	6.84	8.77	10.7	12.7	14.7	16.7	18.7	20.7	22.6
	3	1.30	2.76	4.42	6.22	8.09	10.0	12.0	14.0	15.9	17.9	19.9	21.9
	4	1.11	2.43	3.97	5.67	7.46	9.32	11.2	13.2	15.2	17.2	19.2	21.2
	5	0.98	2.17	3.60	5.19	6.89	8.68	10.6	12.5	14.4	16.4	18.4	20.4
	6	0.87	1.95	3.28	4.77	6.37	8.09	9.90	11.8	13.7	15.6	17.6	19.6
	7	0.78	1.78	3.01	4.40	5.91	7.56	9.31	11.1	13.0	14.9	16.9	18.8
	8	0.71	1.63	2.77	4.07	5.50	7.07	8.76	10.5	12.4	14.2	16.2	18.1
	9	0.65	1.50	2.57	3.78	5.13	6.64	8.26	9.97	11.8	13.6	15.5	17.4
	10	0.60	1.39	2.39	3.52	4.81	6.25	7.81	9.45	11.2	13.0	14.8	16.7
	12	0.52	1.22	2.08	3.09	4.26	5.58	7.01	8.54	10.2	11.9	13.6	15.4
	14	0.45	1.08	1.85	2.75	3.82	5.02	6.34	7.76	9.28	10.9	12.6	14.3
	16	0.41	0.96	1.65	2.48	3.45	4.55	5.77	7.09	8.53	10.1	11.6	13.3
	18	0.37	0.87	1.50	2.25	3.14	4.16	5.29	6.53	7.87	9.30	10.8	12.4
	20	0.33	0.79	1.37	2.06	2.88	3.82	4.87	6.04	7.30	8.65	10.1	11.6
	24	0.28	0.68	1.16	1.76	2.47	3.28	4.21	5.23	6.35	7.55	8.85	10.2
	28	0.25	0.59	1.01	1.53	2.15	2.87	3.69	4.61	5.61	6.69	7.87	9.11
	32	0.22	0.52	0.89	1.35	1.91	2.55	3.29	4.11	5.01	6.00	7.07	8.20
	36	0.20	0.46	0.80	1.21	1.71	2.29	2.96	3.70	4.53	5.43	6.40	7.44
6	2	1.53	3.39	5.36	7.35	9.35	11.3	13.3	15.3	17.3	19.3	21.3	23.2
	3	1.30	3.04	4.99	6.98	8.98	11.0	13.0	15.0	17.0	19.0	21.0	22.9
	4	1.11	2.74	4.64	6.60	8.60	10.6	12.6	14.6	16.6	18.6	20.6	22.6
	5	0.98	2.49	4.31	6.24	8.21	10.2	12.2	14.2	16.3	18.3	20.3	22.3
	6	0.87	2.28	4.02	5.89	7.84	9.82	11.8	13.8	15.9	17.9	19.9	21.9
	7	0.78	2.10	3.76	5.57	7.48	9.44	11.4	13.4	15.5	17.5	19.5	21.5
	8	0.71	1.94	3.53	5.28	7.13	9.07	11.0	13.0	15.1	17.1	19.1	21.1
	9	0.65	1.81	3.32	5.00	6.81	8.71	10.7	12.7	14.7	16.7	18.7	20.7
	10	0.60	1.69	3.13	4.74	6.50	8.37	10.3	12.3	14.3	16.3	18.3	20.3
	12	0.52	1.50	2.80	4.29	5.94	7.74	9.61	11.5	13.5	15.5	17.5	19.5
	14	0.45	1.34	2.52	3.89	5.45	7.17	8.98	10.9	12.8	14.7	16.7	18.7
	16	0.41	1.21	2.29	3.55	5.02	6.67	8.41	10.2	12.1	14.0	16.0	17.9
	18	0.37	1.10	2.09	3.26	4.65	6.22	7.89	9.65	11.5	13.4	15.3	17.2
	20	0.33	1.01	1.92	3.01	4.33	5.82	7.42	9.11	10.9	12.7	14.6	16.5
	24	0.28	0.86	1.64	2.61	3.79	5.13	6.60	8.17	9.84	11.6	13.4	15.2
	28	0.25	0.75	1.44	2.30	3.36	4.58	5.92	7.38	8.95	10.6	12.3	14.1
	32	0.22	0.67	1.27	2.05	3.02	4.12	5.35	6.72	8.18	9.73	11.4	13.0
	36	0.20	0.60	1.14	1.85	2.73	3.74	4.88	6.15	7.52	8.98	10.5	12.1

Table 7-8 (continued)
Coefficients C for Eccentrically Loaded Bolt Groups
Angle = 60°

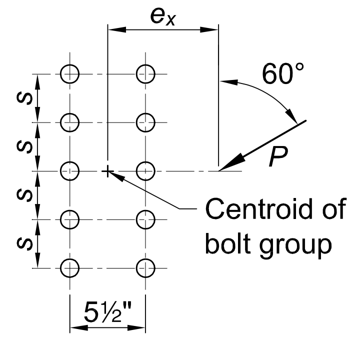
Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where
P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.78	3.55	5.34	7.17	9.04	10.9	12.9	14.8	16.7	18.7	20.6	22.6
	3	1.62	3.26	4.95	6.71	8.53	10.4	12.3	14.2	16.1	18.1	20.0	22.0
	4	1.45	2.97	4.57	6.27	8.04	9.86	11.7	13.6	15.5	17.5	19.4	21.4
	5	1.31	2.71	4.23	5.86	7.58	9.36	11.2	13.1	15.0	16.9	18.8	20.7
	6	1.18	2.48	3.93	5.50	7.16	8.90	10.7	12.5	14.4	16.3	18.2	20.1
	7	1.07	2.28	3.66	5.18	6.79	8.48	10.2	12.0	13.9	15.7	17.6	19.5
	8	0.98	2.11	3.43	4.88	6.45	8.09	9.80	11.6	13.4	15.2	17.1	19.0
	9	0.90	1.97	3.22	4.61	6.12	7.72	9.39	11.1	12.9	14.7	16.6	18.4
	10	0.83	1.84	3.03	4.37	5.82	7.37	9.00	10.7	12.5	14.2	16.1	17.9
	12	0.72	1.62	2.70	3.93	5.28	6.73	8.28	9.91	11.6	13.4	15.1	16.9
	14	0.64	1.45	2.43	3.56	4.81	6.19	7.66	9.22	10.9	12.5	14.3	16.0
	16	0.57	1.31	2.21	3.24	4.42	5.71	7.11	8.60	10.2	11.8	13.5	15.2
	18	0.52	1.19	2.02	2.98	4.07	5.29	6.63	8.05	9.55	11.1	12.7	14.4
	20	0.47	1.09	1.85	2.75	3.77	4.93	6.19	7.55	8.98	10.5	12.1	13.7
	24	0.40	0.93	1.59	2.37	3.28	4.32	5.46	6.69	8.01	9.41	10.9	12.4
	28	0.35	0.82	1.39	2.08	2.90	3.83	4.86	5.99	7.21	8.51	9.88	11.3
	32	0.31	0.72	1.24	1.86	2.59	3.43	4.37	5.41	6.54	7.75	9.02	10.4
	36	0.28	0.65	1.11	1.67	2.34	3.11	3.97	4.93	5.98	7.10	8.29	9.55
6	2	1.78	3.59	5.48	7.41	9.36	11.3	13.3	15.3	17.2	19.2	21.2	23.2
	3	1.62	3.35	5.20	7.12	9.06	11.0	13.0	15.0	16.9	18.9	20.9	22.9
	4	1.45	3.11	4.93	6.82	8.75	10.7	12.7	14.6	16.6	18.6	20.6	22.5
	5	1.31	2.89	4.66	6.53	8.45	10.4	12.3	14.3	16.3	18.2	20.2	22.2
	6	1.18	2.70	4.42	6.26	8.16	10.1	12.0	14.0	15.9	17.9	19.9	21.9
	7	1.07	2.52	4.19	6.01	7.88	9.79	11.7	13.7	15.6	17.6	19.6	21.5
	8	0.98	2.36	3.99	5.77	7.62	9.51	11.4	13.4	15.3	17.3	19.2	21.2
	9	0.90	2.23	3.81	5.55	7.37	9.24	11.1	13.1	15.0	17.0	18.9	20.9
	10	0.83	2.10	3.64	5.35	7.13	8.98	10.9	12.8	14.7	16.7	18.6	20.6
	12	0.72	1.89	3.34	4.97	6.70	8.49	10.3	12.2	14.1	16.1	18.0	19.9
	14	0.64	1.71	3.08	4.63	6.29	8.04	9.85	11.7	13.6	15.5	17.4	19.3
	16	0.57	1.57	2.85	4.32	5.92	7.62	9.39	11.2	13.1	15.0	16.9	18.8
	18	0.52	1.44	2.65	4.04	5.58	7.22	8.95	10.7	12.6	14.4	16.3	18.2
	20	0.47	1.33	2.47	3.79	5.26	6.86	8.55	10.3	12.1	13.9	15.8	17.7
	24	0.40	1.16	2.17	3.36	4.71	6.21	7.82	9.50	11.2	13.0	14.8	16.7
	28	0.35	1.02	1.92	3.00	4.26	5.67	7.19	8.80	10.5	12.2	14.0	15.8
	32	0.31	0.91	1.72	2.71	3.88	5.20	6.64	8.17	9.77	11.4	13.1	14.9
	36	0.28	0.82	1.56	2.46	3.55	4.80	6.16	7.61	9.14	10.7	12.4	14.1

Table 7-8 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 75°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

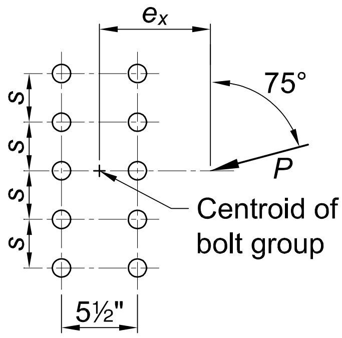
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.92	3.82	5.70	7.57	9.45	11.3	13.2	15.2	17.1	19.0	20.9	22.9
	3	1.87	3.72	5.54	7.36	9.19	11.1	12.9	14.8	16.7	18.6	20.5	22.5
	4	1.82	3.60	5.37	7.14	8.94	10.8	12.6	14.5	16.3	18.2	20.1	22.1
	5	1.75	3.47	5.18	6.92	8.68	10.5	12.3	14.1	16.0	17.9	19.8	21.7
	6	1.68	3.33	5.00	6.69	8.42	10.2	12.0	13.8	15.7	17.5	19.4	21.3
	7	1.60	3.19	4.81	6.47	8.17	9.92	11.7	13.5	15.3	17.2	19.1	20.9
	8	1.52	3.06	4.63	6.26	7.93	9.66	11.4	13.2	15.0	16.9	18.7	20.6
	9	1.45	2.93	4.46	6.05	7.70	9.41	11.2	12.9	14.7	16.5	18.4	20.3
	10	1.38	2.80	4.29	5.85	7.48	9.16	10.9	12.6	14.4	16.2	18.1	19.9
	12	1.25	2.57	3.98	5.48	7.07	8.71	10.4	12.1	13.9	15.7	17.5	19.3
	14	1.13	2.36	3.70	5.15	6.69	8.29	9.96	11.7	13.4	15.2	16.9	18.7
	16	1.03	2.18	3.45	4.85	6.34	7.90	9.53	11.2	12.9	14.7	16.4	18.2
	18	0.95	2.02	3.23	4.57	6.01	7.54	9.13	10.8	12.5	14.2	15.9	17.7
	20	0.87	1.88	3.03	4.32	5.71	7.19	8.75	10.4	12.0	13.7	15.4	17.2
	24	0.75	1.65	2.69	3.87	5.17	6.57	8.05	9.60	11.2	12.9	14.5	16.2
	28	0.66	1.46	2.42	3.50	4.71	6.03	7.44	8.93	10.5	12.1	13.7	15.4
	32	0.59	1.31	2.18	3.19	4.32	5.56	6.90	8.32	9.81	11.4	12.9	14.6
	36	0.53	1.19	1.99	2.92	3.98	5.15	6.42	7.78	9.21	10.7	12.2	13.8
6	2	1.92	3.80	5.69	7.59	9.51	11.5	13.4	15.4	17.6	19.6	21.5	23.5
	3	1.87	3.70	5.55	7.42	9.32	11.2	13.2	15.1	17.1	19.0	21.0	23.0
	4	1.82	3.59	5.40	7.25	9.14	11.1	13.0	14.9	16.9	18.8	20.8	22.7
	5	1.75	3.48	5.26	7.09	8.96	10.9	12.8	14.7	16.6	18.6	20.5	22.5
	6	1.68	3.36	5.11	6.93	8.78	10.7	12.6	14.5	16.4	18.4	20.3	22.2
	7	1.60	3.24	4.97	6.77	8.62	10.5	12.4	14.3	16.2	18.1	20.1	22.0
	8	1.52	3.13	4.84	6.62	8.45	10.3	12.2	14.1	16.0	17.9	19.9	21.8
	9	1.45	3.02	4.71	6.47	8.29	10.2	12.0	13.9	15.8	17.7	19.7	21.6
	10	1.38	2.91	4.58	6.33	8.14	9.98	11.9	13.7	15.6	17.6	19.5	21.4
	12	1.25	2.72	4.34	6.07	7.85	9.67	11.5	13.4	15.3	17.2	19.1	21.0
	14	1.13	2.54	4.13	5.82	7.57	9.38	11.2	13.1	15.0	16.8	18.7	20.6
	16	1.03	2.38	3.92	5.59	7.32	9.10	10.9	12.8	14.6	16.5	18.4	20.3
	18	0.95	2.24	3.74	5.38	7.09	8.85	10.7	12.5	14.3	16.2	18.1	19.9
	20	0.87	2.11	3.57	5.17	6.87	8.61	10.4	12.2	14.0	15.9	17.7	19.6
	24	0.75	1.88	3.27	4.80	6.44	8.15	9.90	11.7	13.5	15.3	17.1	19.0
	28	0.66	1.70	3.00	4.47	6.06	7.72	9.43	11.2	13.0	14.8	16.6	18.4
	32	0.59	1.55	2.77	4.17	5.70	7.31	8.99	10.7	12.5	14.3	16.1	17.9
	36	0.53	1.42	2.57	3.90	5.37	6.93	8.57	10.3	12.0	13.8	15.5	17.3

Table 7-9
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 0°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

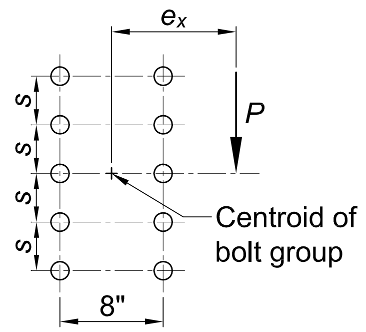
P = required force, P_u or P_a, kips

r_n = nominal strength per bolt, kips

e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.31	2.91	4.71	6.66	8.69	10.8	12.8	14.9	16.9	18.9	21.0	23.0
	3	1.12	2.54	4.14	5.95	7.90	9.93	12.0	14.1	16.2	18.2	20.3	22.4
	4	0.98	2.24	3.66	5.33	7.15	9.10	11.1	13.2	15.3	17.4	19.5	21.6
	5	0.87	1.99	3.27	4.80	6.48	8.33	10.3	12.3	14.4	16.5	18.6	20.7
	6	0.79	1.80	2.95	4.35	5.90	7.63	9.49	11.5	13.5	15.6	17.7	19.8
	7	0.71	1.63	2.68	3.97	5.40	7.02	8.77	10.7	12.6	14.6	16.7	18.8
	8	0.65	1.49	2.46	3.65	4.97	6.48	8.13	9.91	11.8	13.8	15.8	17.9
	9	0.60	1.38	2.27	3.37	4.59	6.01	7.55	9.24	11.1	13.0	14.9	17.0
	10	0.56	1.28	2.11	3.13	4.27	5.59	7.04	8.64	10.4	12.2	14.1	16.1
	12	0.49	1.11	1.84	2.73	3.73	4.90	6.19	7.63	9.18	10.9	12.6	14.5
	14	0.44	0.99	1.64	2.42	3.31	4.36	5.50	6.80	8.20	9.73	11.4	13.1
	16	0.39	0.89	1.47	2.17	2.98	3.91	4.95	6.13	7.40	8.80	10.3	11.9
	18	0.36	0.80	1.33	1.97	2.70	3.55	4.50	5.57	6.73	8.02	9.39	10.9
	20	0.33	0.73	1.22	1.80	2.47	3.25	4.12	5.10	6.17	7.35	8.62	9.99
	24	0.28	0.63	1.04	1.53	2.10	2.77	3.51	4.35	5.28	6.30	7.39	8.59
	28	0.25	0.55	0.91	1.33	1.83	2.41	3.06	3.79	4.60	5.50	6.46	7.51
	32	0.22	0.48	0.80	1.18	1.62	2.13	2.71	3.36	4.08	4.87	5.73	6.67
	36	0.20	0.43	0.72	1.06	1.45	1.91	2.43	3.01	3.66	4.37	5.15	5.99
	C', in.	7.85	16.8	27.3	39.9	54.6	71.5	90.9	113	137	164	194	226
6	2	1.31	3.28	5.35	7.42	9.47	11.5	13.5	15.5	17.5	19.5	21.4	23.4
	3	1.12	2.93	4.94	7.03	9.12	11.2	13.2	15.3	17.3	19.3	21.3	23.3
	4	0.98	2.63	4.52	6.59	8.70	10.8	12.9	14.9	17.0	19.0	21.0	23.0
	5	0.87	2.37	4.13	6.15	8.25	10.4	12.5	14.6	16.6	18.7	20.7	22.8
	6	0.79	2.15	3.78	5.72	7.78	9.90	12.0	14.1	16.2	18.3	20.4	22.4
	7	0.71	1.97	3.47	5.32	7.33	9.43	11.6	13.7	15.8	17.9	20.0	22.1
	8	0.65	1.81	3.19	4.95	6.89	8.95	11.1	13.2	15.4	17.5	19.6	21.7
	9	0.60	1.67	2.95	4.62	6.48	8.49	10.6	12.7	14.9	17.0	19.1	21.3
	10	0.56	1.55	2.75	4.33	6.10	8.05	10.1	12.2	14.4	16.5	18.7	20.8
	12	0.49	1.35	2.40	3.82	5.43	7.25	9.21	11.3	13.4	15.5	17.7	19.8
	14	0.44	1.20	2.14	3.41	4.86	6.56	8.40	10.4	12.4	14.5	16.7	18.8
	16	0.39	1.08	1.92	3.07	4.40	5.96	7.69	9.56	11.5	13.6	15.7	17.8
	18	0.36	0.97	1.75	2.79	4.00	5.46	7.06	8.83	10.7	12.7	14.7	16.8
	20	0.33	0.89	1.60	2.56	3.67	5.02	6.52	8.18	9.97	11.9	13.9	15.9
	24	0.28	0.76	1.37	2.18	3.14	4.32	5.62	7.11	8.71	10.4	12.3	14.2
	28	0.25	0.66	1.19	1.90	2.75	3.78	4.93	6.26	7.70	9.27	11.0	12.7
	32	0.22	0.58	1.05	1.68	2.44	3.35	4.38	5.58	6.88	8.31	9.85	11.5
	36	0.20	0.52	0.95	1.51	2.19	3.01	3.94	5.02	6.21	7.52	8.93	10.4
	C', in.	7.85	19.6	35.6	56.6	82.5	114	150	192	239	292	350	414

Table 7-9 (continued)
Coefficients C for Eccentrically Loaded Bolt Groups
Angle = 15°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

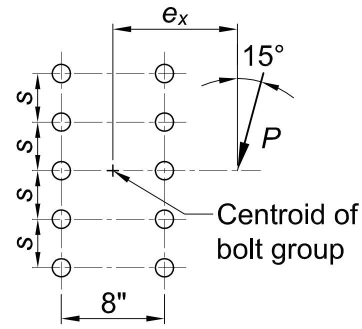
P = required force, P_u or P_a, kips

r_n = nominal strength per bolt, kips

e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.35	2.96	4.75	6.67	8.67	10.7	12.7	14.8	16.8	18.8	20.9	22.9
	3	1.16	2.58	4.20	5.98	7.90	9.89	11.9	14.0	16.0	18.1	20.2	22.2
	4	1.02	2.28	3.73	5.37	7.17	9.08	11.1	13.1	15.2	17.3	19.3	21.4
	5	0.90	2.03	3.35	4.85	6.53	8.34	10.3	12.2	14.3	16.3	18.4	20.5
	6	0.81	1.84	3.03	4.40	5.96	7.66	9.48	11.4	13.4	15.4	17.5	19.6
	7	0.74	1.67	2.76	4.02	5.48	7.06	8.79	10.6	12.6	14.5	16.6	18.6
	8	0.68	1.53	2.53	3.70	5.05	6.53	8.17	9.91	11.8	13.7	15.7	17.7
	9	0.63	1.42	2.34	3.43	4.68	6.07	7.61	9.27	11.0	12.9	14.8	16.8
	10	0.58	1.31	2.17	3.19	4.36	5.66	7.12	8.69	10.4	12.2	14.0	16.0
	12	0.51	1.15	1.90	2.79	3.82	4.97	6.28	7.69	9.23	10.9	12.6	14.4
	14	0.45	1.02	1.69	2.48	3.40	4.43	5.61	6.88	8.29	9.79	11.4	13.1
	16	0.41	0.91	1.51	2.23	3.05	3.99	5.05	6.21	7.50	8.88	10.4	11.9
	18	0.37	0.83	1.37	2.02	2.77	3.63	4.60	5.66	6.84	8.11	9.48	11.0
	20	0.34	0.76	1.26	1.85	2.54	3.32	4.21	5.19	6.28	7.45	8.73	10.1
	24	0.29	0.65	1.07	1.58	2.16	2.84	3.60	4.45	5.39	6.40	7.52	8.71
	28	0.25	0.56	0.93	1.37	1.89	2.47	3.14	3.88	4.71	5.61	6.59	7.64
	32	0.23	0.50	0.83	1.22	1.67	2.19	2.78	3.44	4.18	4.98	5.86	6.80
	36	0.20	0.45	0.74	1.09	1.50	1.96	2.49	3.09	3.75	4.47	5.27	6.12
6	2	1.35	3.29	5.33	7.39	9.42	11.4	13.4	15.4	17.4	19.4	21.4	23.4
	3	1.16	2.94	4.93	6.99	9.05	11.1	13.1	15.2	17.2	19.2	21.2	23.2
	4	1.02	2.64	4.52	6.55	8.63	10.7	12.8	14.8	16.9	18.9	20.9	22.9
	5	0.90	2.38	4.15	6.12	8.18	10.3	12.4	14.4	16.5	18.5	20.6	22.6
	6	0.81	2.17	3.82	5.70	7.72	9.80	11.9	14.0	16.1	18.2	20.2	22.3
	7	0.74	1.99	3.52	5.31	7.28	9.33	11.4	13.5	15.6	17.7	19.8	21.9
	8	0.68	1.83	3.25	4.95	6.86	8.87	11.0	13.1	15.2	17.3	19.4	21.5
	9	0.63	1.69	3.02	4.63	6.46	8.43	10.5	12.6	14.7	16.8	18.9	21.0
	10	0.58	1.58	2.81	4.34	6.10	8.00	10.0	12.1	14.2	16.3	18.4	20.5
	12	0.51	1.38	2.47	3.84	5.45	7.23	9.15	11.2	13.2	15.3	17.4	19.6
	14	0.45	1.23	2.20	3.44	4.91	6.56	8.38	10.3	12.3	14.4	16.5	18.6
	16	0.41	1.10	1.98	3.11	4.46	5.99	7.69	9.52	11.5	13.5	15.5	17.6
	18	0.37	1.00	1.80	2.83	4.08	5.49	7.09	8.82	10.7	12.6	14.6	16.6
	20	0.34	0.92	1.65	2.60	3.75	5.06	6.56	8.20	9.96	11.8	13.8	15.7
	24	0.29	0.78	1.41	2.23	3.22	4.36	5.70	7.15	8.74	10.4	12.2	14.1
	28	0.25	0.68	1.23	1.95	2.82	3.83	5.02	6.32	7.76	9.31	11.0	12.7
	32	0.23	0.60	1.09	1.73	2.50	3.41	4.47	5.64	6.96	8.38	9.90	11.5
	36	0.20	0.54	0.97	1.55	2.25	3.07	4.03	5.09	6.30	7.60	9.01	10.5

Table 7-9 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 30°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

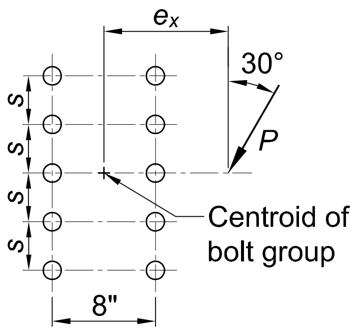
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.49	3.12	4.91	6.80	8.75	10.7	12.7	14.7	16.7	18.7	20.8	22.7
	3	1.29	2.74	4.39	6.16	8.04	9.98	12.0	14.0	16.0	18.0	20.0	22.1
	4	1.13	2.43	3.95	5.60	7.37	9.24	11.2	13.2	15.2	17.2	19.2	21.3
	5	1.00	2.18	3.58	5.10	6.77	8.55	10.4	12.4	14.3	16.3	18.4	20.4
	6	0.90	1.98	3.26	4.67	6.23	7.93	9.72	11.6	13.5	15.5	17.5	19.5
	7	0.82	1.81	2.99	4.30	5.76	7.37	9.08	10.9	12.8	14.7	16.7	18.7
	8	0.75	1.67	2.76	3.97	5.35	6.87	8.49	10.2	12.0	13.9	15.9	17.8
	9	0.70	1.55	2.56	3.69	4.98	6.42	7.96	9.62	11.4	13.2	15.1	17.0
	10	0.65	1.44	2.38	3.44	4.66	6.02	7.49	9.07	10.8	12.5	14.4	16.2
	12	0.57	1.26	2.09	3.03	4.13	5.34	6.66	8.12	9.67	11.3	13.0	14.8
	14	0.50	1.12	1.86	2.71	3.69	4.78	5.99	7.33	8.75	10.3	11.9	13.6
	16	0.45	1.01	1.67	2.44	3.33	4.33	5.44	6.66	7.98	9.39	10.9	12.5
	18	0.41	0.92	1.52	2.22	3.03	3.95	4.97	6.10	7.32	8.64	10.1	11.5
	20	0.38	0.84	1.39	2.03	2.78	3.62	4.57	5.62	6.75	7.98	9.30	10.7
	24	0.32	0.72	1.19	1.74	2.38	3.11	3.93	4.84	5.83	6.92	8.08	9.32
	28	0.28	0.63	1.04	1.52	2.08	2.72	3.44	4.24	5.13	6.09	7.12	8.24
	32	0.25	0.56	0.92	1.35	1.84	2.41	3.06	3.77	4.57	5.43	6.36	7.37
	36	0.23	0.50	0.83	1.21	1.66	2.17	2.75	3.40	4.11	4.89	5.74	6.66
6	2	1.49	3.36	5.36	7.37	9.38	11.4	13.4	15.4	17.4	19.3	21.3	23.3
	3	1.29	3.02	4.97	6.99	9.01	11.0	13.1	15.1	17.1	19.1	21.1	23.1
	4	1.13	2.73	4.60	6.58	8.61	10.7	12.7	14.7	16.7	18.8	20.8	22.8
	5	1.00	2.48	4.26	6.18	8.18	10.2	12.3	14.3	16.4	18.4	20.4	22.4
	6	0.90	2.27	3.96	5.80	7.76	9.79	11.8	13.9	15.9	18.0	20.0	22.1
	7	0.82	2.09	3.68	5.44	7.36	9.35	11.4	13.5	15.5	17.6	19.6	21.7
	8	0.75	1.93	3.43	5.11	6.97	8.93	11.0	13.0	15.1	17.1	19.2	21.2
	9	0.70	1.80	3.21	4.81	6.61	8.53	10.5	12.6	14.6	16.7	18.7	20.8
	10	0.65	1.68	3.01	4.53	6.27	8.14	10.1	12.1	14.2	16.2	18.3	20.4
	12	0.57	1.49	2.67	4.05	5.67	7.43	9.31	11.3	13.3	15.3	17.4	19.4
	14	0.50	1.33	2.39	3.65	5.15	6.81	8.60	10.5	12.4	14.4	16.5	18.5
	16	0.45	1.20	2.16	3.31	4.71	6.27	7.96	9.76	11.7	13.6	15.6	17.6
	18	0.41	1.09	1.97	3.03	4.34	5.79	7.39	9.12	10.9	12.8	14.8	16.8
	20	0.38	1.00	1.81	2.80	4.01	5.37	6.89	8.53	10.3	12.1	14.0	15.9
	24	0.32	0.86	1.55	2.41	3.48	4.68	6.04	7.53	9.14	10.8	12.6	14.5
	28	0.28	0.75	1.35	2.12	3.06	4.13	5.36	6.72	8.19	9.76	11.4	13.2
	32	0.25	0.67	1.20	1.89	2.73	3.69	4.81	6.05	7.40	8.86	10.4	12.0
	36	0.23	0.60	1.08	1.70	2.46	3.34	4.36	5.50	6.74	8.09	9.53	11.1

Table 7-9 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 45°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

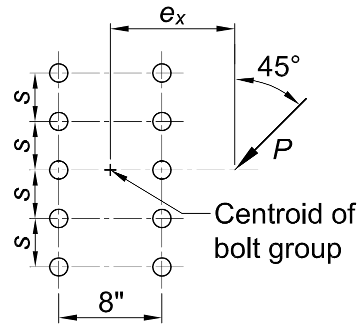
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.70	3.43	5.22	7.06	8.95	10.9	12.8	14.8	16.8	18.7	20.7	22.7
	3	1.51	3.09	4.76	6.52	8.35	10.2	12.2	14.1	16.1	18.0	20.0	22.0
	4	1.35	2.78	4.34	6.01	7.78	9.60	11.5	13.4	15.3	17.3	19.3	21.3
	5	1.21	2.52	3.97	5.57	7.25	9.01	10.8	12.7	14.6	16.6	18.5	20.5
	6	1.10	2.30	3.67	5.17	6.78	8.47	10.2	12.1	13.9	15.9	17.8	19.8
	7	1.00	2.12	3.40	4.82	6.35	7.97	9.67	11.5	13.3	15.2	17.1	19.0
	8	0.92	1.96	3.17	4.51	5.96	7.51	9.15	10.9	12.7	14.5	16.4	18.3
	9	0.85	1.82	2.96	4.23	5.60	7.08	8.68	10.4	12.1	13.9	15.7	17.6
	10	0.79	1.70	2.78	3.97	5.28	6.70	8.24	9.86	11.5	13.3	15.1	17.0
	12	0.69	1.50	2.46	3.54	4.73	6.04	7.46	8.97	10.6	12.2	14.0	15.7
	14	0.61	1.34	2.21	3.18	4.27	5.48	6.80	8.21	9.70	11.3	12.9	14.6
	16	0.55	1.21	2.00	2.88	3.89	5.01	6.23	7.54	8.95	10.4	12.0	13.6
	18	0.50	1.11	1.82	2.64	3.56	4.60	5.74	6.97	8.30	9.71	11.2	12.7
	20	0.46	1.02	1.67	2.42	3.29	4.25	5.31	6.47	7.73	9.06	10.5	11.9
	24	0.40	0.87	1.43	2.09	2.84	3.68	4.62	5.65	6.77	7.96	9.23	10.6
	28	0.35	0.76	1.26	1.83	2.49	3.24	4.07	5.00	6.00	7.08	8.24	9.47
	32	0.31	0.68	1.12	1.63	2.22	2.89	3.64	4.47	5.38	6.37	7.43	8.56
	36	0.28	0.61	1.00	1.46	2.00	2.60	3.29	4.04	4.87	5.78	6.75	7.79
6	2	1.70	3.52	5.44	7.40	9.37	11.4	13.3	15.3	17.3	19.3	21.3	23.2
	3	1.51	3.23	5.11	7.06	9.03	11.0	13.0	15.0	17.0	19.0	21.0	22.9
	4	1.35	2.96	4.79	6.70	8.67	10.7	12.7	14.6	16.6	18.6	20.6	22.6
	5	1.21	2.72	4.48	6.36	8.30	10.3	12.3	14.3	16.3	18.3	20.3	22.3
	6	1.10	2.51	4.20	6.03	7.94	9.90	11.9	13.9	15.9	17.9	19.9	21.9
	7	1.00	2.33	3.96	5.73	7.60	9.53	11.5	13.5	15.5	17.5	19.5	21.5
	8	0.92	2.18	3.73	5.45	7.27	9.17	11.1	13.1	15.1	17.1	19.1	21.1
	9	0.85	2.04	3.53	5.19	6.96	8.83	10.8	12.7	14.7	16.7	18.7	20.7
	10	0.79	1.92	3.35	4.94	6.67	8.50	10.4	12.4	14.3	16.3	18.3	20.3
	12	0.69	1.71	3.02	4.50	6.13	7.88	9.73	11.6	13.6	15.5	17.5	19.5
	14	0.61	1.55	2.75	4.12	5.65	7.33	9.11	11.0	12.9	14.8	16.8	18.8
	16	0.55	1.41	2.51	3.78	5.22	6.83	8.55	10.3	12.2	14.1	16.0	18.0
	18	0.50	1.29	2.31	3.49	4.85	6.39	8.04	9.77	11.6	13.4	15.3	17.3
	20	0.46	1.19	2.13	3.24	4.53	6.00	7.57	9.25	11.0	12.8	14.7	16.6
	24	0.40	1.03	1.84	2.82	3.99	5.32	6.76	8.32	9.97	11.7	13.5	15.3
	28	0.35	0.90	1.62	2.50	3.56	4.76	6.09	7.53	9.08	10.7	12.4	14.2
	32	0.31	0.80	1.44	2.24	3.20	4.30	5.52	6.86	8.32	9.85	11.5	13.1
	36	0.28	0.72	1.30	2.02	2.90	3.92	5.04	6.30	7.66	9.10	10.6	12.2

Table 7-9 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 60°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

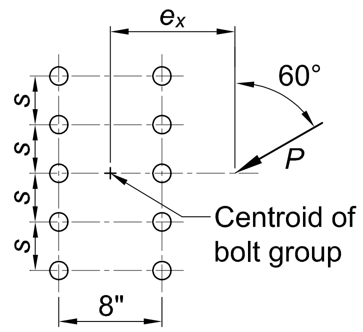
P = required force, P_u or P_a, kips

r_n = nominal strength per bolt, kips

e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.86	3.71	5.56	7.41	9.28	11.2	13.1	15.0	16.9	18.8	20.8	22.7
	3	1.77	3.52	5.29	7.07	8.88	10.7	12.6	14.5	16.4	18.3	20.2	22.1
	4	1.66	3.31	4.99	6.70	8.45	10.3	12.1	13.9	15.8	17.7	19.6	21.6
	5	1.54	3.10	4.70	6.34	8.04	9.79	11.6	13.4	15.3	17.1	19.0	21.0
	6	1.43	2.90	4.41	6.00	7.64	9.35	11.1	12.9	14.7	16.6	18.5	20.4
	7	1.33	2.71	4.15	5.68	7.27	8.94	10.7	12.4	14.2	16.1	17.9	19.8
	8	1.24	2.54	3.92	5.39	6.94	8.56	10.3	12.0	13.8	15.6	17.4	19.3
	9	1.16	2.38	3.70	5.12	6.63	8.22	9.86	11.6	13.3	15.1	16.9	18.7
	10	1.08	2.24	3.51	4.88	6.34	7.89	9.49	11.2	12.9	14.6	16.4	18.2
	12	0.96	2.00	3.17	4.44	5.82	7.28	8.81	10.4	12.1	13.8	15.5	17.3
	14	0.86	1.81	2.88	4.07	5.36	6.73	8.19	9.72	11.3	13.0	14.7	16.4
	16	0.77	1.64	2.64	3.74	4.95	6.25	7.64	9.11	10.7	12.2	13.9	15.6
	18	0.70	1.51	2.43	3.46	4.59	5.83	7.15	8.56	10.0	11.6	13.2	14.8
	20	0.65	1.39	2.25	3.21	4.28	5.45	6.71	8.06	9.48	11.0	12.5	14.1
	24	0.56	1.20	1.95	2.80	3.76	4.81	5.96	7.19	8.50	9.88	11.3	12.8
	28	0.49	1.06	1.72	2.48	3.34	4.29	5.34	6.47	7.68	8.97	10.3	11.7
	32	0.43	0.94	1.54	2.22	3.00	3.87	4.83	5.87	6.99	8.19	9.46	10.8
	36	0.39	0.85	1.39	2.01	2.72	3.52	4.40	5.36	6.41	7.53	8.71	9.96
6	2	1.86	3.72	5.59	7.50	9.43	11.4	13.3	15.3	17.3	19.2	21.2	23.2
	3	1.77	3.55	5.37	7.25	9.16	11.1	13.0	15.0	17.0	18.9	20.9	22.9
	4	1.66	3.36	5.14	6.98	8.88	10.8	12.7	14.7	16.7	18.6	20.6	22.6
	5	1.54	3.17	4.90	6.72	8.59	10.5	12.4	14.4	16.3	18.3	20.3	22.2
	6	1.43	2.99	4.67	6.46	8.31	10.2	12.1	14.1	16.0	18.0	19.9	21.9
	7	1.33	2.82	4.46	6.21	8.05	9.92	11.8	13.8	15.7	17.7	19.6	21.6
	8	1.24	2.67	4.26	5.98	7.79	9.65	11.5	13.5	15.4	17.3	19.3	21.3
	9	1.16	2.52	4.08	5.76	7.55	9.39	11.3	13.2	15.1	17.0	19.0	20.9
	10	1.08	2.40	3.91	5.56	7.32	9.14	11.0	12.9	14.8	16.7	18.7	20.6
	12	0.96	2.17	3.61	5.20	6.90	8.66	10.5	12.4	14.2	16.1	18.1	20.0
	14	0.86	1.98	3.35	4.87	6.51	8.23	10.0	11.8	13.7	15.6	17.5	19.4
	16	0.77	1.82	3.11	4.57	6.15	7.81	9.56	11.4	13.2	15.1	16.9	18.9
	18	0.70	1.69	2.91	4.30	5.81	7.43	9.13	10.9	12.7	14.5	16.4	18.3
	20	0.65	1.57	2.72	4.05	5.50	7.07	8.73	10.5	12.2	14.1	15.9	17.8
	24	0.56	1.37	2.41	3.61	4.96	6.43	8.00	9.67	11.4	13.2	15.0	16.8
	28	0.49	1.22	2.15	3.25	4.49	5.88	7.38	8.97	10.6	12.3	14.1	15.9
	32	0.43	1.09	1.94	2.94	4.10	5.41	6.83	8.34	9.92	11.6	13.3	15.0
	36	0.39	0.99	1.76	2.69	3.77	5.00	6.35	7.78	9.30	10.9	12.5	14.2

Table 7-9 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 75°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

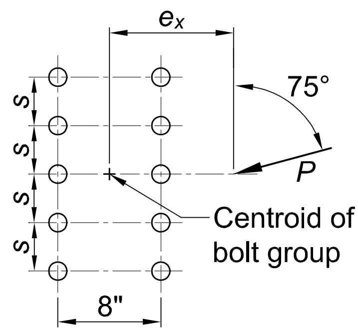
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.94	3.87	5.79	7.70	9.61	11.5	13.4	15.3	17.3	19.2	21.1	23.0
	3	1.92	3.82	5.70	7.58	9.45	11.3	13.2	15.1	17.0	18.9	20.8	22.7
	4	1.89	3.75	5.60	7.43	9.26	11.1	12.9	14.8	16.7	18.5	20.4	22.3
	5	1.85	3.67	5.48	7.28	9.07	10.9	12.7	14.5	16.4	18.2	20.1	22.0
	6	1.81	3.59	5.35	7.11	8.87	10.6	12.4	14.2	16.1	17.9	19.8	21.6
	7	1.76	3.50	5.22	6.94	8.67	10.4	12.2	14.0	15.8	17.6	19.4	21.3
	8	1.71	3.40	5.08	6.76	8.46	10.2	11.9	13.7	15.5	17.3	19.1	21.0
	9	1.66	3.30	4.94	6.59	8.26	9.96	11.7	13.4	15.2	17.0	18.8	20.6
	10	1.61	3.20	4.80	6.42	8.06	9.73	11.4	13.2	14.9	16.7	18.5	20.3
	12	1.51	3.01	4.53	6.08	7.67	9.30	11.0	12.7	14.4	16.2	17.9	19.7
	14	1.41	2.82	4.27	5.76	7.31	8.90	10.5	12.2	13.9	15.6	17.4	19.2
	16	1.31	2.65	4.03	5.47	6.96	8.52	10.1	11.8	13.4	15.2	16.9	18.6
	18	1.23	2.48	3.80	5.19	6.64	8.16	9.73	11.3	13.0	14.7	16.4	18.1
	20	1.15	2.34	3.60	4.93	6.34	7.82	9.36	10.9	12.6	14.2	15.9	17.7
	24	1.01	2.08	3.23	4.48	5.80	7.20	8.67	10.2	11.8	13.4	15.0	16.7
	28	0.90	1.87	2.93	4.08	5.33	6.65	8.06	9.52	11.0	12.6	14.2	15.9
	32	0.81	1.69	2.67	3.75	4.91	6.17	7.51	8.91	10.4	11.9	13.5	15.1
	36	0.73	1.54	2.45	3.45	4.55	5.74	7.01	8.36	9.77	11.2	12.8	14.3
6	2	1.94	3.86	5.77	7.68	9.60	11.5	13.5	15.4	17.6	19.6	21.5	23.5
	3	1.92	3.80	5.68	7.55	9.45	11.4	13.3	15.2	17.2	19.1	21.1	23.0
	4	1.89	3.74	5.57	7.42	9.29	11.2	13.1	15.0	16.9	18.9	20.8	22.8
	5	1.85	3.66	5.46	7.29	9.14	11.0	12.9	14.8	16.7	18.7	20.6	22.6
	6	1.81	3.58	5.35	7.15	8.98	10.8	12.7	14.6	16.5	18.5	20.4	22.3
	7	1.76	3.49	5.23	7.01	8.83	10.7	12.5	14.4	16.3	18.3	20.2	22.1
	8	1.71	3.40	5.12	6.88	8.68	10.5	12.4	14.3	16.2	18.1	20.0	21.9
	9	1.66	3.31	5.00	6.74	8.53	10.4	12.2	14.1	16.0	17.9	19.8	21.7
	10	1.61	3.22	4.89	6.61	8.38	10.2	12.0	13.9	15.8	17.7	19.6	21.5
	12	1.51	3.05	4.67	6.36	8.10	9.89	11.7	13.6	15.4	17.3	19.2	21.1
	14	1.41	2.88	4.46	6.12	7.84	9.61	11.4	13.3	15.1	17.0	18.9	20.8
	16	1.31	2.73	4.26	5.89	7.59	9.33	11.1	12.9	14.8	16.6	18.5	20.4
	18	1.23	2.58	4.08	5.68	7.35	9.08	10.8	12.7	14.5	16.3	18.2	20.1
	20	1.15	2.45	3.90	5.47	7.13	8.84	10.6	12.4	14.2	16.0	17.9	19.7
	24	1.01	2.21	3.59	5.10	6.71	8.38	10.1	11.9	13.6	15.5	17.3	19.1
	28	0.90	2.01	3.32	4.77	6.32	7.96	9.65	11.4	13.1	14.9	16.7	18.5
	32	0.81	1.84	3.08	4.47	5.97	7.56	9.21	10.9	12.7	14.4	16.2	18.0
	36	0.73	1.70	2.87	4.19	5.64	7.19	8.80	10.5	12.2	13.9	15.7	17.5

Table 7-10
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 0°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

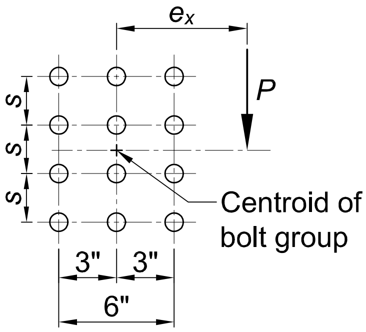
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.71	4.07	6.81	9.86	13.0	16.1	19.3	22.3	25.4	28.5	31.5	34.5
	3	1.42	3.40	5.79	8.61	11.7	14.8	18.0	21.1	24.3	27.4	30.5	33.6
	4	1.21	2.90	4.97	7.53	10.4	13.4	16.6	19.8	23.0	26.1	29.3	32.5
	5	1.05	2.51	4.34	6.64	9.24	12.1	15.2	18.3	21.5	24.7	27.9	31.1
	6	0.92	2.21	3.85	5.91	8.27	11.0	13.9	16.9	20.0	23.2	26.4	29.7
	7	0.81	1.96	3.44	5.31	7.46	9.95	12.7	15.6	18.6	21.8	25.0	28.2
	8	0.72	1.76	3.11	4.80	6.78	9.09	11.6	14.4	17.3	20.4	23.5	26.7
	9	0.64	1.60	2.83	4.38	6.20	8.34	10.7	13.3	16.1	19.1	22.1	25.2
	10	0.58	1.46	2.59	4.02	5.71	7.70	9.91	12.4	15.0	17.9	20.8	23.8
	12	0.49	1.24	2.21	3.44	4.91	6.65	8.59	10.8	13.2	15.7	18.5	21.3
	14	0.42	1.08	1.92	3.00	4.30	5.83	7.57	9.53	11.7	14.0	16.5	19.2
	16	0.37	0.95	1.70	2.66	3.82	5.19	6.75	8.51	10.5	12.6	14.9	17.3
	18	0.33	0.85	1.52	2.39	3.43	4.67	6.08	7.68	9.45	11.4	13.5	15.8
	20	0.29	0.77	1.37	2.16	3.11	4.24	5.53	6.99	8.61	10.4	12.3	14.4
	24	0.24	0.64	1.15	1.82	2.62	3.57	4.67	5.92	7.30	8.84	10.5	12.3
	28	0.21	0.55	0.99	1.57	2.26	3.08	4.04	5.12	6.33	7.67	9.13	10.7
	32	0.18	0.49	0.87	1.38	1.98	2.71	3.55	4.51	5.58	6.77	8.06	9.47
	36	0.16	0.43	0.77	1.23	1.77	2.42	3.17	4.03	4.99	6.05	7.21	8.48
	C', in.	5.89	15.8	28.0	44.7	64.3	88.5	116	148	183	223	267	315
6	2	1.71	4.85	8.04	11.2	14.2	17.3	20.3	23.2	26.2	29.2	32.2	35.1
	3	1.42	4.24	7.36	10.6	13.7	16.8	19.9	22.9	25.9	28.9	31.9	34.9
	4	1.21	3.72	6.66	9.86	13.1	16.2	19.4	22.4	25.5	28.5	31.6	34.6
	5	1.05	3.29	6.00	9.14	12.4	15.6	18.7	21.9	25.0	28.1	31.1	34.2
	6	0.92	2.93	5.41	8.44	11.6	14.9	18.1	21.2	24.4	27.5	30.6	33.7
	7	0.81	2.63	4.90	7.79	10.9	14.1	17.3	20.6	23.7	26.9	30.0	33.2
	8	0.72	2.38	4.46	7.20	10.2	13.4	16.6	19.8	23.0	26.2	29.4	32.6
	9	0.64	2.17	4.09	6.67	9.54	12.6	15.8	19.1	22.3	25.5	28.7	31.9
	10	0.58	2.00	3.78	6.20	8.94	12.0	15.1	18.3	21.6	24.8	28.0	31.2
	12	0.49	1.71	3.27	5.41	7.88	10.7	13.7	16.8	20.0	23.3	26.5	29.8
	14	0.42	1.49	2.87	4.78	7.01	9.61	12.4	15.4	18.6	21.8	25.0	28.2
	16	0.37	1.32	2.55	4.28	6.29	8.69	11.3	14.2	17.2	20.3	23.5	26.7
	18	0.33	1.19	2.30	3.86	5.70	7.91	10.4	13.1	15.9	18.9	22.0	25.2
	20	0.29	1.08	2.09	3.51	5.20	7.25	9.54	12.1	14.8	17.7	20.7	23.8
	24	0.24	0.91	1.76	2.97	4.42	6.19	8.19	10.4	12.9	15.5	18.3	21.2
	28	0.21	0.78	1.52	2.57	3.84	5.39	7.14	9.15	11.4	13.7	16.3	19.0
	32	0.18	0.69	1.33	2.27	3.39	4.77	6.33	8.13	10.1	12.3	14.6	17.1
	36	0.16	0.61	1.19	2.03	3.03	4.27	5.67	7.30	9.10	11.1	13.2	15.5
	C', in.	5.89	22.4	43.3	74.4	112	158	212	275	345	424	510	606

Table 7-10 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 15°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

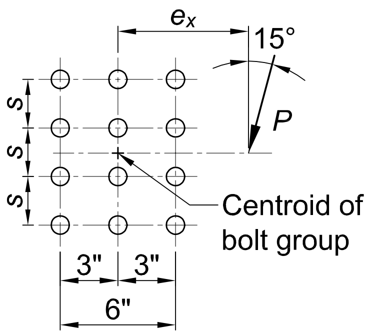
P = required force, P_u or P_a, kips

r_n = nominal strength per bolt, kips

e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.77	4.10	6.84	9.82	12.9	16.0	19.1	22.2	25.2	28.3	31.3	34.3
	3	1.47	3.45	5.86	8.61	11.6	14.7	17.8	20.9	24.1	27.2	30.3	33.3
	4	1.25	2.95	5.07	7.55	10.4	13.3	16.4	19.5	22.7	25.8	29.0	32.1
	5	1.08	2.57	4.44	6.67	9.26	12.1	15.1	18.1	21.3	24.4	27.6	30.7
	6	0.94	2.26	3.93	5.96	8.33	11.0	13.8	16.8	19.8	23.0	26.1	29.3
	7	0.83	2.01	3.52	5.37	7.55	9.97	12.7	15.5	18.5	21.5	24.7	27.8
	8	0.74	1.81	3.18	4.87	6.88	9.13	11.7	14.4	17.2	20.2	23.2	26.4
	9	0.66	1.64	2.90	4.45	6.31	8.40	10.8	13.3	16.1	18.9	21.9	25.0
	10	0.60	1.50	2.65	4.10	5.81	7.77	9.99	12.4	15.0	17.8	20.7	23.6
	12	0.50	1.28	2.27	3.52	5.01	6.74	8.71	10.9	13.2	15.8	18.4	21.2
	14	0.43	1.11	1.98	3.08	4.40	5.93	7.69	9.62	11.8	14.1	16.5	19.1
	16	0.38	0.98	1.75	2.73	3.91	5.29	6.87	8.62	10.6	12.7	15.0	17.4
	18	0.34	0.88	1.57	2.45	3.52	4.77	6.20	7.80	9.59	11.5	13.6	15.9
	20	0.30	0.79	1.42	2.22	3.19	4.33	5.65	7.12	8.76	10.5	12.5	14.6
	24	0.25	0.67	1.19	1.87	2.69	3.66	4.78	6.04	7.45	8.99	10.7	12.5
	28	0.22	0.57	1.02	1.61	2.32	3.17	4.14	5.24	6.47	7.82	9.31	10.9
	32	0.19	0.50	0.90	1.42	2.04	2.79	3.65	4.62	5.72	6.92	8.24	9.66
	36	0.17	0.45	0.80	1.26	1.82	2.49	3.26	4.13	5.11	6.20	7.38	8.66
6	2	1.77	4.83	7.98	11.1	14.1	17.2	20.2	23.2	26.1	29.1	32.1	35.0
	3	1.47	4.22	7.31	10.5	13.6	16.7	19.7	22.8	25.8	28.8	31.8	34.8
	4	1.25	3.71	6.64	9.77	12.9	16.1	19.2	22.3	25.3	28.3	31.4	34.4
	5	1.08	3.28	6.01	9.06	12.2	15.4	18.5	21.7	24.8	27.8	30.9	33.9
	6	0.94	2.94	5.45	8.38	11.5	14.7	17.8	21.0	24.1	27.2	30.3	33.4
	7	0.83	2.65	4.97	7.75	10.8	13.9	17.1	20.3	23.5	26.6	29.7	32.8
	8	0.74	2.40	4.55	7.17	10.1	13.2	16.4	19.6	22.7	25.9	29.1	32.2
	9	0.66	2.20	4.18	6.66	9.49	12.5	15.6	18.8	22.0	25.2	28.4	31.5
	10	0.60	2.02	3.86	6.20	8.92	11.9	14.9	18.1	21.3	24.5	27.6	30.8
	12	0.50	1.74	3.34	5.43	7.91	10.6	13.6	16.6	19.8	23.0	26.1	29.3
	14	0.43	1.52	2.94	4.82	7.07	9.60	12.4	15.3	18.4	21.5	24.6	27.8
	16	0.38	1.35	2.62	4.32	6.38	8.71	11.3	14.1	17.0	20.1	23.2	26.3
	18	0.34	1.22	2.36	3.91	5.79	7.95	10.4	13.0	15.8	18.8	21.8	24.9
	20	0.30	1.10	2.14	3.57	5.30	7.31	9.60	12.1	14.8	17.6	20.5	23.5
	24	0.25	0.93	1.81	3.03	4.52	6.26	8.28	10.5	12.9	15.5	18.2	21.1
	28	0.22	0.80	1.56	2.63	3.93	5.47	7.26	9.24	11.4	13.8	16.3	18.9
	32	0.19	0.71	1.37	2.32	3.47	4.85	6.45	8.23	10.2	12.4	14.7	17.1
	36	0.17	0.63	1.23	2.08	3.11	4.35	5.80	7.41	9.23	11.2	13.3	15.6

Table 7-10 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 30°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

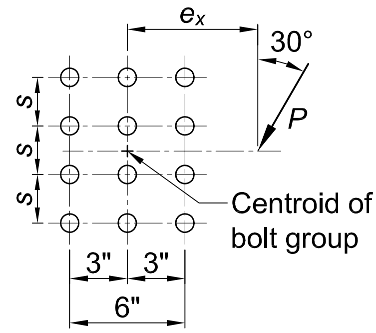
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	1	2	3	4	5	6	7	8	9	10	11	12
		Number of Bolts in One Vertical Row, *n*											
3	2	1.94	4.26	6.99	9.90	12.9	16.0	19.0	22.0	25.1	28.1	31.1	34.1
	3	1.61	3.63	6.09	8.80	11.7	14.7	17.7	20.8	23.9	27.0	30.0	33.1
	4	1.37	3.15	5.35	7.83	10.6	13.5	16.5	19.5	22.6	25.7	28.7	31.8
	5	1.19	2.77	4.74	7.00	9.54	12.3	15.2	18.2	21.2	24.3	27.4	30.5
	6	1.04	2.45	4.23	6.30	8.67	11.3	14.1	17.0	19.9	23.0	26.0	29.1
	7	0.92	2.19	3.81	5.71	7.92	10.4	13.0	15.8	18.7	21.7	24.7	27.8
	8	0.82	1.98	3.45	5.22	7.27	9.58	12.1	14.8	17.6	20.5	23.4	26.4
	9	0.74	1.80	3.16	4.79	6.71	8.88	11.2	13.8	16.5	19.3	22.2	25.2
	10	0.67	1.65	2.90	4.42	6.22	8.26	10.5	12.9	15.5	18.2	21.1	24.0
	12	0.56	1.41	2.49	3.82	5.41	7.22	9.23	11.5	13.8	16.4	19.0	21.8
	14	0.48	1.23	2.18	3.36	4.78	6.40	8.22	10.3	12.4	14.8	17.2	19.8
	16	0.42	1.08	1.93	2.99	4.26	5.73	7.40	9.25	11.3	13.4	15.7	18.2
	18	0.38	0.97	1.73	2.69	3.85	5.18	6.71	8.41	10.3	12.3	14.4	16.7
	20	0.34	0.88	1.57	2.44	3.50	4.73	6.14	7.70	9.42	11.3	13.3	15.4
	24	0.28	0.74	1.32	2.06	2.96	4.01	5.22	6.58	8.08	9.72	11.5	13.4
	28	0.24	0.64	1.14	1.78	2.56	3.48	4.54	5.73	7.05	8.51	10.1	11.8
	32	0.21	0.56	1.00	1.57	2.26	3.07	4.01	5.07	6.25	7.55	8.96	10.5
	36	0.19	0.50	0.89	1.40	2.02	2.75	3.59	4.54	5.61	6.78	8.06	9.44
6	2	1.94	4.86	7.96	11.0	14.1	17.1	20.1	23.1	26.0	29.0	32.0	35.0
	3	1.61	4.27	7.32	10.4	13.5	16.6	19.6	22.6	25.6	28.6	31.6	34.6
	4	1.37	3.78	6.70	9.75	12.9	15.9	19.0	22.1	25.1	28.1	31.1	34.2
	5	1.19	3.39	6.14	9.10	12.2	15.3	18.4	21.5	24.5	27.6	30.6	33.7
	6	1.04	3.06	5.64	8.48	11.5	14.6	17.7	20.8	23.9	27.0	30.1	33.1
	7	0.92	2.78	5.19	7.91	10.9	13.9	17.0	20.1	23.2	26.3	29.4	32.5
	8	0.82	2.54	4.80	7.38	10.3	13.3	16.3	19.4	22.6	25.7	28.8	31.9
	9	0.74	2.34	4.45	6.90	9.67	12.6	15.7	18.7	21.9	25.0	28.1	31.2
	10	0.67	2.16	4.14	6.46	9.14	12.0	15.0	18.1	21.2	24.3	27.4	30.5
	12	0.56	1.87	3.61	5.71	8.20	10.9	13.8	16.8	19.8	22.9	26.0	29.1
	14	0.48	1.65	3.20	5.10	7.41	9.95	12.7	15.6	18.5	21.5	24.6	27.7
	16	0.42	1.47	2.86	4.60	6.74	9.12	11.7	14.5	17.3	20.3	23.3	26.4
	18	0.38	1.33	2.58	4.19	6.17	8.39	10.8	13.5	16.2	19.1	22.0	25.0
	20	0.34	1.21	2.35	3.84	5.68	7.75	10.1	12.6	15.2	18.0	20.9	23.8
	24	0.28	1.02	2.00	3.29	4.89	6.71	8.78	11.1	13.5	16.1	18.8	21.6
	28	0.24	0.88	1.73	2.86	4.28	5.90	7.77	9.83	12.1	14.5	17.0	19.6
	32	0.21	0.78	1.52	2.54	3.80	5.25	6.95	8.83	10.9	13.1	15.4	17.9
	36	0.19	0.70	1.36	2.27	3.41	4.73	6.28	8.00	9.88	11.9	14.1	16.4

Table 7-10 (continued)
Coefficients C for Eccentrically Loaded Bolt Groups
Angle = 45°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

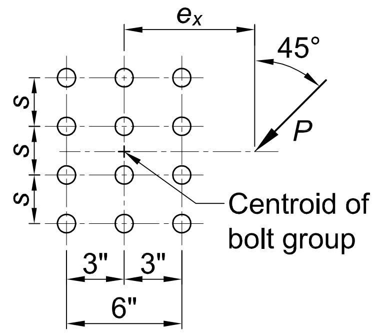
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	2.23	4.67	7.33	10.2	13.1	16.0	19.0	22.0	25.0	28.0	31.0	33.9
	3	1.89	4.06	6.50	9.19	12.0	14.9	17.9	20.9	23.9	26.9	29.9	32.9
	4	1.63	3.57	5.84	8.36	11.1	13.9	16.8	19.7	22.7	25.7	28.7	31.7
	5	1.42	3.17	5.27	7.63	10.2	12.9	15.7	18.6	21.5	24.5	27.5	30.5
	6	1.25	2.84	4.78	6.99	9.40	12.0	14.7	17.6	20.4	23.4	26.3	29.3
	7	1.11	2.57	4.36	6.42	8.70	11.2	13.8	16.6	19.4	22.3	25.2	28.2
	8	0.99	2.33	3.99	5.92	8.09	10.5	13.0	15.7	18.4	21.2	24.1	27.0
	9	0.90	2.13	3.68	5.49	7.54	9.80	12.2	14.8	17.5	20.3	23.1	26.0
	10	0.81	1.96	3.40	5.10	7.05	9.21	11.6	14.0	16.6	19.3	22.1	24.9
	12	0.68	1.68	2.95	4.46	6.22	8.19	10.4	12.7	15.1	17.7	20.3	23.0
	14	0.59	1.47	2.59	3.95	5.55	7.35	9.34	11.5	13.8	16.2	18.7	21.3
	16	0.52	1.31	2.31	3.54	4.99	6.65	8.49	10.5	12.7	14.9	17.3	19.8
	18	0.46	1.17	2.08	3.20	4.54	6.06	7.77	9.64	11.7	13.8	16.1	18.5
	20	0.41	1.06	1.89	2.92	4.15	5.56	7.15	8.90	10.8	12.8	15.0	17.2
	24	0.35	0.90	1.60	2.48	3.54	4.76	6.15	7.70	9.39	11.2	13.1	15.2
	28	0.30	0.77	1.38	2.15	3.08	4.16	5.39	6.77	8.28	9.91	11.7	13.5
	32	0.26	0.68	1.22	1.90	2.72	3.68	4.79	6.03	7.39	8.87	10.5	12.2
	36	0.23	0.61	1.08	1.69	2.44	3.30	4.30	5.42	6.66	8.02	9.49	11.1
6	2	2.23	5.02	8.01	11.0	14.0	17.0	20.0	23.0	25.9	28.9	31.9	34.8
	3	1.89	4.50	7.44	10.4	13.5	16.5	19.5	22.5	25.5	28.4	31.4	34.4
	4	1.63	4.05	6.89	9.86	12.9	15.9	18.9	21.9	24.9	27.9	30.9	33.9
	5	1.42	3.68	6.40	9.30	12.3	15.3	18.3	21.3	24.4	27.4	30.4	33.4
	6	1.25	3.36	5.96	8.78	11.7	14.7	17.7	20.7	23.8	26.8	29.8	32.8
	7	1.11	3.09	5.57	8.29	11.2	14.1	17.1	20.1	23.2	26.2	29.2	32.3
	8	0.99	2.86	5.22	7.84	10.6	13.6	16.5	19.5	22.6	25.6	28.6	31.7
	9	0.90	2.65	4.90	7.43	10.2	13.0	16.0	19.0	22.0	25.0	28.0	31.1
	10	0.81	2.47	4.61	7.04	9.69	12.5	15.4	18.4	21.4	24.4	27.4	30.4
	12	0.68	2.16	4.11	6.35	8.85	11.6	14.4	17.3	20.2	23.2	26.2	29.2
	14	0.59	1.92	3.69	5.76	8.11	10.7	13.4	16.2	19.1	22.1	25.0	28.0
	16	0.52	1.72	3.34	5.25	7.47	9.94	12.6	15.3	18.1	21.0	23.9	26.9
	18	0.46	1.56	3.04	4.82	6.91	9.26	11.8	14.4	17.2	20.0	22.9	25.8
	20	0.41	1.43	2.79	4.44	6.43	8.66	11.1	13.6	16.3	19.0	21.9	24.7
	24	0.35	1.22	2.38	3.84	5.62	7.64	9.84	12.2	14.7	17.3	20.0	22.8
	28	0.30	1.06	2.08	3.37	4.98	6.81	8.82	11.0	13.4	15.8	18.4	21.1
	32	0.26	0.94	1.84	3.00	4.46	6.12	7.97	10.0	12.2	14.6	17.0	19.5
	36	0.23	0.84	1.65	2.71	4.04	5.56	7.27	9.18	11.2	13.4	15.7	18.1

Table 7-10 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 60°

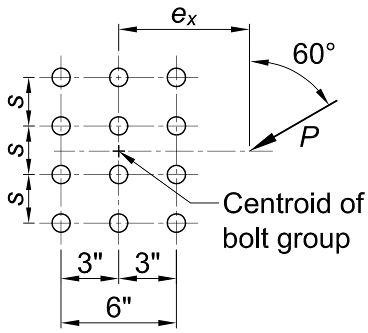
Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where
- P = required force, P_u or P_a, kips
- r_n = nominal strength per bolt, kips
- e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
- s = bolt spacing, in.
- C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	2.59	5.21	7.88	10.6	13.4	16.3	19.2	22.1	25.0	28.0	30.9	33.9
	3	2.32	4.73	7.27	9.91	12.7	15.5	18.3	21.2	24.1	27.0	30.0	32.9
	4	2.07	4.29	6.69	9.23	11.9	14.6	17.5	20.3	23.2	26.1	29.0	32.0
	5	1.84	3.90	6.18	8.63	11.2	13.9	16.6	19.5	22.3	25.2	28.1	31.0
	6	1.65	3.56	5.73	8.08	10.6	13.2	15.9	18.7	21.5	24.3	27.2	30.1
	7	1.49	3.27	5.32	7.59	10.0	12.6	15.2	17.9	20.7	23.5	26.3	29.2
	8	1.35	3.01	4.95	7.13	9.48	12.0	14.5	17.2	19.9	22.7	25.5	28.4
	9	1.23	2.78	4.63	6.71	8.98	11.4	13.9	16.5	19.2	22.0	24.7	27.6
	10	1.12	2.58	4.34	6.33	8.52	10.9	13.3	15.9	18.5	21.2	24.0	26.8
	12	0.95	2.25	3.84	5.67	7.70	9.91	12.3	14.7	17.3	19.9	22.6	25.3
	14	0.83	1.98	3.43	5.11	7.00	9.08	11.3	13.7	16.1	18.7	21.3	23.9
	16	0.73	1.77	3.09	4.64	6.40	8.36	10.5	12.7	15.1	17.5	20.1	22.6
	18	0.65	1.60	2.81	4.24	5.89	7.73	9.74	11.9	14.2	16.5	19.0	21.5
	20	0.59	1.46	2.57	3.90	5.44	7.19	9.09	11.1	13.3	15.6	17.9	20.4
	24	0.49	1.24	2.20	3.35	4.72	6.27	7.99	9.85	11.9	14.0	16.2	18.5
	28	0.42	1.07	1.91	2.93	4.15	5.55	7.10	8.81	10.7	12.6	14.7	16.8
	32	0.37	0.95	1.69	2.60	3.70	4.97	6.38	7.95	9.65	11.5	13.4	15.4
	36	0.33	0.85	1.51	2.34	3.34	4.49	5.79	7.23	8.81	10.5	12.3	14.2
6	2	2.59	5.32	8.17	11.1	14.0	17.0	19.9	22.9	25.8	28.8	31.8	34.7
	3	2.32	4.94	7.73	10.6	13.5	16.5	19.4	22.4	25.4	28.3	31.3	34.3
	4	2.07	4.57	7.31	10.2	13.1	16.0	19.0	21.9	24.9	27.8	30.8	33.8
	5	1.84	4.25	6.91	9.73	12.6	15.5	18.5	21.4	24.4	27.4	30.3	33.3
	6	1.65	3.95	6.55	9.32	12.2	15.1	18.0	20.9	23.9	26.9	29.8	32.8
	7	1.49	3.69	6.22	8.94	11.8	14.6	17.5	20.5	23.4	26.4	29.3	32.3
	8	1.35	3.46	5.92	8.58	11.4	14.2	17.1	20.0	22.9	25.9	28.8	31.8
	9	1.23	3.25	5.64	8.25	11.0	13.8	16.7	19.6	22.5	25.4	28.4	31.3
	10	1.12	3.06	5.39	7.94	10.6	13.4	16.3	19.1	22.0	24.9	27.9	30.8
	12	0.95	2.73	4.92	7.37	9.97	12.7	15.5	18.3	21.2	24.1	27.0	29.9
	14	0.83	2.46	4.52	6.85	9.36	12.0	14.7	17.5	20.3	23.2	26.1	29.0
	16	0.73	2.23	4.18	6.39	8.80	11.4	14.0	16.8	19.6	22.4	25.3	28.1
	18	0.65	2.04	3.87	5.97	8.28	10.8	13.4	16.1	18.8	21.6	24.4	27.3
	20	0.59	1.88	3.60	5.59	7.81	10.2	12.8	15.4	18.1	20.9	23.7	26.5
	24	0.49	1.63	3.15	4.94	6.99	9.25	11.7	14.2	16.8	19.5	22.2	25.0
	28	0.42	1.43	2.79	4.41	6.31	8.44	10.7	13.1	15.7	18.2	20.9	23.6
	32	0.37	1.27	2.49	3.97	5.74	7.74	9.90	12.2	14.6	17.1	19.7	22.3
	36	0.33	1.15	2.25	3.61	5.26	7.13	9.17	11.4	13.7	16.1	18.6	21.1

Table 7-10 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 75°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

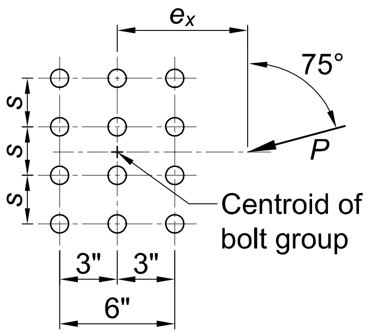
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	2.86	5.68	8.47	11.3	14.1	16.9	19.8	22.6	25.5	28.4	31.3	34.2
	3	2.77	5.49	8.19	10.9	13.7	16.4	19.2	22.1	24.9	27.8	30.7	33.6
	4	2.66	5.27	7.89	10.5	13.2	16.0	18.8	21.6	24.4	27.2	30.1	33.0
	5	2.53	5.04	7.58	10.2	12.8	15.5	18.3	21.0	23.9	26.7	29.5	32.4
	6	2.40	4.81	7.27	9.81	12.4	15.1	17.8	20.6	23.3	26.2	29.0	31.8
	7	2.26	4.57	6.97	9.47	12.0	14.7	17.4	20.1	22.9	25.6	28.4	31.3
	8	2.13	4.35	6.69	9.13	11.7	14.3	16.9	19.6	22.4	25.1	27.9	30.7
	9	2.00	4.13	6.41	8.82	11.3	13.9	16.5	19.2	21.9	24.7	27.4	30.2
	10	1.89	3.93	6.15	8.51	11.0	13.5	16.1	18.8	21.5	24.2	27.0	29.8
	12	1.67	3.57	5.67	7.95	10.4	12.9	15.4	18.0	20.7	23.4	26.1	28.8
	14	1.49	3.25	5.25	7.44	9.77	12.2	14.7	17.3	19.9	22.6	25.3	28.0
	16	1.34	2.97	4.87	6.98	9.23	11.6	14.1	16.6	19.2	21.8	24.5	27.2
	18	1.21	2.73	4.54	6.56	8.74	11.1	13.5	16.0	18.5	21.1	23.7	26.4
	20	1.10	2.53	4.24	6.18	8.28	10.5	12.9	15.3	17.8	20.4	23.0	25.6
	24	0.93	2.19	3.75	5.52	7.48	9.59	11.8	14.2	16.6	19.1	21.6	24.2
	28	0.80	1.93	3.34	4.97	6.79	8.78	10.9	13.2	15.5	17.9	20.4	22.9
	32	0.71	1.72	3.01	4.51	6.20	8.08	10.1	12.3	14.5	16.8	19.2	21.7
	36	0.63	1.55	2.74	4.12	5.70	7.47	9.40	11.5	13.6	15.9	18.2	20.6
6	2	2.86	5.66	8.48	11.3	14.2	17.1	20.1	23.0	26.4	29.3	32.3	35.2
	3	2.77	5.49	8.25	11.1	13.9	16.8	19.7	22.7	25.6	28.5	31.5	34.4
	4	2.66	5.30	8.02	10.8	13.6	16.5	19.4	22.3	25.2	28.2	31.1	34.0
	5	2.53	5.10	7.79	10.6	13.4	16.2	19.1	22.0	24.9	27.8	30.8	33.7
	6	2.40	4.91	7.56	10.3	13.1	15.9	18.8	21.7	24.6	27.5	30.4	33.3
	7	2.26	4.72	7.34	10.1	12.9	15.7	18.5	21.4	24.3	27.2	30.1	33.0
	8	2.13	4.54	7.14	9.83	12.6	15.4	18.3	21.1	24.0	26.9	29.8	32.7
	9	2.00	4.37	6.94	9.61	12.4	15.2	18.0	20.8	23.7	26.6	29.5	32.4
	10	1.89	4.21	6.75	9.40	12.1	14.9	17.7	20.6	23.4	26.3	29.2	32.1
	12	1.67	3.90	6.39	9.00	11.7	14.4	17.2	20.0	22.9	25.7	28.6	31.5
	14	1.49	3.63	6.06	8.63	11.3	14.0	16.8	19.6	22.4	25.2	28.1	30.9
	16	1.34	3.39	5.75	8.29	10.9	13.6	16.3	19.1	21.9	24.7	27.5	30.4
	18	1.21	3.17	5.47	7.96	10.6	13.2	15.9	18.7	21.4	24.2	27.0	29.9
	20	1.10	2.98	5.22	7.66	10.2	12.9	15.5	18.2	21.0	23.8	26.6	29.4
	24	0.93	2.65	4.76	7.10	9.57	12.2	14.8	17.5	20.2	22.9	25.7	28.5
	28	0.80	2.38	4.37	6.60	8.99	11.5	14.1	16.7	19.4	22.1	24.8	27.6
	32	0.71	2.16	4.03	6.15	8.45	10.9	13.4	16.0	18.7	21.3	24.0	26.8
	36	0.63	1.97	3.73	5.75	7.96	10.3	12.8	15.3	17.9	20.6	23.3	26.0

Table 7-11
Coefficients C for Eccentrically Loaded Bolt Groups
Angle = 0°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

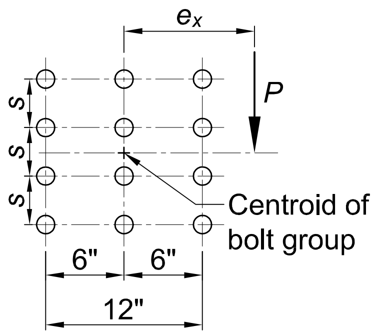
P = required force, P_u or P_a, kips

r_n = nominal strength per bolt, kips

e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
	2	2.15	4.55	7.17	10.0	13.0	16.0	19.1	22.2	25.3	28.3	31.4	34.4
	3	1.91	4.06	6.43	9.06	11.9	14.9	17.9	21.0	24.1	27.2	30.3	33.4
	4	1.71	3.65	5.80	8.23	10.9	13.7	16.7	19.8	22.9	26.0	29.1	32.3
	5	1.55	3.31	5.27	7.51	9.97	12.7	15.5	18.5	21.5	24.7	27.8	31.0
	6	1.42	3.02	4.82	6.88	9.16	11.7	14.4	17.3	20.3	23.3	26.4	29.6
	7	1.31	2.77	4.44	6.34	8.46	10.8	13.4	16.1	19.0	22.0	25.1	28.2
	8	1.21	2.56	4.10	5.87	7.85	10.1	12.5	15.1	17.9	20.7	23.7	26.8
	9	1.12	2.38	3.81	5.46	7.31	9.39	11.7	14.1	16.8	19.6	22.5	25.5
	10	1.05	2.21	3.55	5.09	6.84	8.79	10.9	13.3	15.8	18.5	21.3	24.2
3	12	0.92	1.94	3.12	4.48	6.03	7.78	9.70	11.8	14.1	16.6	19.1	21.9
	14	0.81	1.72	2.77	3.99	5.38	6.95	8.69	10.6	12.7	14.9	17.3	19.9
	16	0.72	1.53	2.48	3.58	4.84	6.27	7.85	9.60	11.5	13.6	15.8	18.1
	18	0.64	1.38	2.25	3.25	4.40	5.70	7.15	8.75	10.5	12.4	14.4	16.6
	20	0.58	1.26	2.05	2.96	4.02	5.21	6.55	8.03	9.65	11.4	13.3	15.3
	24	0.49	1.06	1.73	2.52	3.42	4.45	5.60	6.88	8.29	9.82	11.5	13.2
	28	0.42	0.92	1.50	2.19	2.97	3.87	4.88	6.00	7.24	8.59	10.1	11.6
	32	0.37	0.81	1.32	1.93	2.63	3.42	4.32	5.32	6.42	7.62	8.93	10.3
	36	0.33	0.72	1.18	1.72	2.35	3.06	3.87	4.77	5.76	6.84	8.02	9.29
	C', in.	11.8	26.5	43.3	63.7	86.8	114	144	178	216	257	302	352
	2	2.15	4.94	7.98	11.1	14.2	17.2	20.2	23.2	26.2	29.2	32.1	35.1
	3	1.91	4.48	7.39	10.5	13.6	16.7	19.8	22.8	25.8	28.9	31.9	34.8
	4	1.71	4.07	6.81	9.86	13.0	16.1	19.3	22.3	25.4	28.5	31.5	34.5
	5	1.55	3.71	6.27	9.22	12.3	15.5	18.6	21.8	24.9	28.0	31.0	34.1
	6	1.42	3.40	5.79	8.61	11.7	14.8	18.0	21.1	24.3	27.4	30.5	33.6
	7	1.31	3.13	5.35	8.05	11.0	14.1	17.3	20.5	23.6	26.8	29.9	33.1
	8	1.21	2.90	4.97	7.53	10.4	13.4	16.6	19.8	23.0	26.1	29.3	32.5
	9	1.12	2.69	4.64	7.07	9.78	12.8	15.9	19.0	22.2	25.4	28.6	31.8
	10	1.05	2.51	4.34	6.64	9.24	12.1	15.2	18.3	21.5	24.7	27.9	31.1
6	12	0.92	2.21	3.85	5.91	8.27	11.0	13.9	16.9	20.0	23.2	26.4	29.7
	14	0.81	1.96	3.44	5.31	7.46	9.95	12.7	15.6	18.6	21.8	25.0	28.2
	16	0.72	1.76	3.11	4.80	6.78	9.09	11.6	14.4	17.3	20.4	23.5	26.7
	18	0.64	1.60	2.83	4.38	6.20	8.34	10.7	13.3	16.1	19.1	22.1	25.2
	20	0.58	1.46	2.59	4.02	5.71	7.70	9.91	12.4	15.0	17.9	20.8	23.8
	24	0.49	1.24	2.21	3.44	4.91	6.65	8.59	10.8	13.2	15.7	18.5	21.3
	28	0.42	1.08	1.92	3.00	4.30	5.83	7.57	9.53	11.7	14.0	16.5	19.2
	32	0.37	0.95	1.70	2.66	3.82	5.19	6.75	8.51	10.5	12.6	14.9	17.3
	36	0.33	0.85	1.52	2.39	3.43	4.67	6.08	7.68	9.45	11.4	13.5	15.8
	C', in.	11.8	31.6	56.1	89.4	129	177	232	296	366	446	533	629

Table 7-11 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 15°

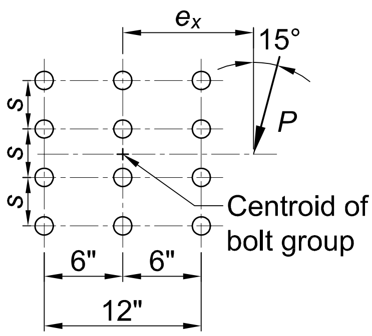
Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where
P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	2.22	4.62	7.25	10.1	13.0	16.0	19.0	22.1	25.1	28.2	31.2	34.2
	3	1.97	4.13	6.53	9.13	11.9	14.9	17.9	20.9	24.0	27.1	30.1	33.2
	4	1.77	3.72	5.91	8.31	10.9	13.7	16.7	19.7	22.7	25.8	28.9	32.0
	5	1.61	3.38	5.39	7.60	10.1	12.7	15.5	18.4	21.4	24.5	27.6	30.7
	6	1.47	3.10	4.93	6.98	9.28	11.8	14.4	17.2	20.2	23.2	26.2	29.3
	7	1.35	2.85	4.54	6.45	8.59	10.9	13.5	16.1	19.0	21.9	24.9	27.9
	8	1.25	2.63	4.21	5.98	7.98	10.2	12.6	15.1	17.8	20.7	23.6	26.6
	9	1.16	2.44	3.91	5.57	7.45	9.51	11.8	14.2	16.8	19.5	22.4	25.3
	10	1.08	2.28	3.65	5.21	6.97	8.92	11.1	13.4	15.9	18.5	21.2	24.1
	12	0.94	2.00	3.20	4.59	6.16	7.91	9.84	11.9	14.2	16.6	19.2	21.9
	14	0.83	1.77	2.85	4.09	5.50	7.08	8.84	10.8	12.8	15.0	17.4	19.9
	16	0.74	1.58	2.56	3.68	4.96	6.40	8.00	9.75	11.7	13.7	15.9	18.2
	18	0.66	1.43	2.31	3.34	4.51	5.83	7.30	8.91	10.7	12.6	14.6	16.8
	20	0.60	1.30	2.11	3.05	4.13	5.34	6.70	8.19	9.82	11.6	13.5	15.5
	24	0.50	1.10	1.79	2.59	3.52	4.56	5.74	7.03	8.45	10.0	11.7	13.4
	28	0.43	0.95	1.55	2.25	3.06	3.98	5.01	6.15	7.40	8.77	10.2	11.8
	32	0.38	0.84	1.37	1.99	2.70	3.52	4.43	5.45	6.57	7.79	9.12	10.5
	36	0.34	0.75	1.22	1.78	2.42	3.15	3.98	4.89	5.90	7.01	8.20	9.49
6	2	2.22	4.97	7.97	11.0	14.1	17.1	20.1	23.1	26.1	29.1	32.1	35.0
	3	1.97	4.50	7.40	10.5	13.5	16.6	19.7	22.7	25.7	28.7	31.7	34.7
	4	1.77	4.10	6.84	9.82	12.9	16.0	19.1	22.2	25.2	28.3	31.3	34.3
	5	1.61	3.75	6.32	9.20	12.3	15.4	18.5	21.6	24.7	27.8	30.8	33.9
	6	1.47	3.45	5.86	8.61	11.6	14.7	17.8	20.9	24.1	27.2	30.3	33.3
	7	1.35	3.18	5.44	8.06	11.0	14.0	17.1	20.3	23.4	26.5	29.6	32.7
	8	1.25	2.95	5.07	7.55	10.4	13.3	16.4	19.5	22.7	25.8	29.0	32.1
	9	1.16	2.75	4.73	7.09	9.78	12.7	15.7	18.8	22.0	25.1	28.3	31.4
	10	1.08	2.57	4.44	6.67	9.26	12.1	15.1	18.1	21.3	24.4	27.6	30.7
	12	0.94	2.26	3.93	5.96	8.33	11.0	13.8	16.8	19.8	23.0	26.1	29.3
	14	0.83	2.01	3.52	5.37	7.55	9.97	12.7	15.5	18.5	21.5	24.7	27.8
	16	0.74	1.81	3.18	4.87	6.88	9.13	11.7	14.4	17.2	20.2	23.2	26.4
	18	0.66	1.64	2.90	4.45	6.31	8.40	10.8	13.3	16.1	18.9	21.9	25.0
	20	0.60	1.50	2.65	4.10	5.81	7.77	9.99	12.4	15.0	17.8	20.7	23.6
	24	0.50	1.28	2.27	3.52	5.01	6.74	8.71	10.9	13.2	15.8	18.4	21.2
	28	0.43	1.11	1.98	3.08	4.40	5.93	7.69	9.62	11.8	14.1	16.5	19.1
	32	0.38	0.98	1.75	2.73	3.91	5.29	6.87	8.62	10.6	12.7	15.0	17.4
	36	0.34	0.88	1.57	2.45	3.52	4.77	6.20	7.80	9.59	11.5	13.6	15.9

Table 7-11 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 30°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

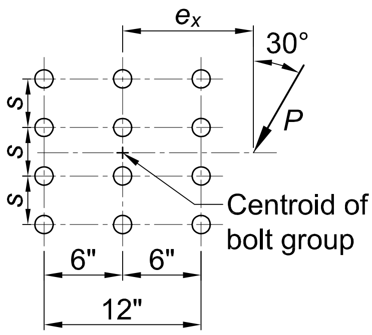
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	2.40	4.89	7.53	10.3	13.2	16.1	19.1	22.1	25.1	28.1	31.1	34.1
	3	2.15	4.40	6.84	9.45	12.2	15.1	18.0	21.0	24.0	27.0	30.0	33.0
	4	1.94	3.99	6.24	8.69	11.3	14.0	16.9	19.8	22.8	25.8	28.8	31.9
	5	1.76	3.65	5.74	8.02	10.5	13.1	15.8	18.7	21.6	24.6	27.6	30.6
	6	1.61	3.35	5.29	7.42	9.72	12.2	14.8	17.6	20.4	23.4	26.3	29.3
	7	1.49	3.10	4.90	6.89	9.06	11.4	13.9	16.6	19.3	22.2	25.1	28.1
	8	1.37	2.87	4.55	6.42	8.47	10.7	13.1	15.6	18.3	21.1	23.9	26.9
	9	1.28	2.67	4.24	6.00	7.94	10.1	12.4	14.8	17.4	20.0	22.8	25.7
	10	1.19	2.49	3.97	5.63	7.47	9.49	11.7	14.0	16.5	19.1	21.8	24.6
	12	1.04	2.19	3.50	4.98	6.64	8.48	10.5	12.6	14.9	17.3	19.9	22.5
	14	0.92	1.95	3.12	4.46	5.97	7.64	9.46	11.4	13.6	15.8	18.2	20.7
	16	0.82	1.75	2.81	4.03	5.40	6.93	8.61	10.4	12.4	14.5	16.7	19.1
	18	0.74	1.58	2.55	3.66	4.92	6.33	7.89	9.59	11.4	13.4	15.5	17.7
	20	0.67	1.44	2.33	3.35	4.52	5.82	7.27	8.85	10.6	12.4	14.4	16.4
	24	0.56	1.22	1.98	2.86	3.87	5.00	6.26	7.65	9.16	10.8	12.5	14.4
	28	0.48	1.06	1.72	2.49	3.37	4.37	5.48	6.71	8.06	9.51	11.1	12.8
	32	0.42	0.93	1.52	2.20	2.99	3.88	4.87	5.97	7.18	8.49	9.91	11.4
	36	0.38	0.83	1.36	1.97	2.68	3.48	4.38	5.38	6.47	7.66	8.95	10.3
6	2	2.40	5.11	8.05	11.1	14.1	17.1	20.1	23.0	26.0	29.0	32.0	34.9
	3	2.15	4.66	7.51	10.5	13.5	16.5	19.6	22.6	25.6	28.6	31.6	34.6
	4	1.94	4.26	6.99	9.90	12.9	16.0	19.0	22.0	25.1	28.1	31.1	34.1
	5	1.76	3.92	6.52	9.34	12.3	15.3	18.4	21.5	24.5	27.6	30.6	33.6
	6	1.61	3.63	6.09	8.80	11.7	14.7	17.7	20.8	23.9	27.0	30.0	33.1
	7	1.49	3.38	5.70	8.30	11.1	14.1	17.1	20.2	23.2	26.3	29.4	32.5
	8	1.37	3.15	5.35	7.83	10.6	13.5	16.5	19.5	22.6	25.7	28.7	31.8
	9	1.28	2.95	5.03	7.40	10.0	12.9	15.8	18.8	21.9	25.0	28.1	31.2
	10	1.19	2.77	4.74	7.00	9.54	12.3	15.2	18.2	21.2	24.3	27.4	30.5
	12	1.04	2.45	4.23	6.30	8.67	11.3	14.1	17.0	19.9	23.0	26.0	29.1
	14	0.92	2.19	3.81	5.71	7.92	10.4	13.0	15.8	18.7	21.7	24.7	27.8
	16	0.82	1.98	3.45	5.22	7.27	9.58	12.1	14.8	17.6	20.5	23.4	26.4
	18	0.74	1.80	3.16	4.79	6.71	8.88	11.2	13.8	16.5	19.3	22.2	25.2
	20	0.67	1.65	2.90	4.42	6.22	8.26	10.5	12.9	15.5	18.2	21.1	24.0
	24	0.56	1.41	2.49	3.82	5.41	7.22	9.23	11.5	13.8	16.4	19.0	21.8
	28	0.48	1.23	2.18	3.36	4.78	6.40	8.22	10.3	12.4	14.8	17.2	19.8
	32	0.42	1.08	1.93	2.99	4.26	5.73	7.40	9.25	11.3	13.4	15.7	18.2
	36	0.38	0.97	1.73	2.69	3.85	5.18	6.71	8.41	10.3	12.3	14.4	16.7

Table 7-11 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 45°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

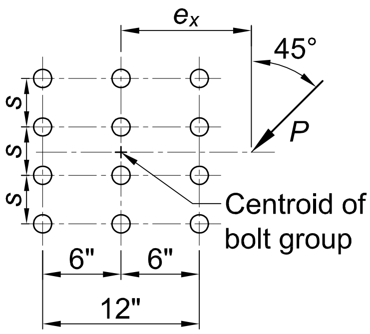
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	2.64	5.30	8.01	10.8	13.6	16.4	19.3	22.3	25.2	28.1	31.1	34.0
	3	2.43	4.90	7.44	10.1	12.8	15.6	18.4	21.3	24.2	27.1	30.1	33.1
	4	2.23	4.52	6.89	9.38	12.0	14.7	17.5	20.3	23.2	26.1	29.0	32.0
	5	2.05	4.17	6.40	8.75	11.2	13.9	16.6	19.3	22.2	25.0	27.9	30.9
	6	1.89	3.86	5.96	8.20	10.6	13.1	15.7	18.4	21.2	24.0	26.9	29.8
	7	1.75	3.59	5.57	7.70	9.99	12.4	14.9	17.5	20.2	23.0	25.8	28.7
	8	1.63	3.35	5.22	7.25	9.43	11.7	14.2	16.7	19.3	22.1	24.8	27.7
	9	1.52	3.13	4.90	6.83	8.91	11.1	13.5	15.9	18.5	21.2	23.9	26.7
	10	1.42	2.94	4.61	6.45	8.44	10.6	12.8	15.2	17.7	20.3	23.0	25.7
	12	1.25	2.60	4.11	5.78	7.60	9.58	11.7	14.0	16.3	18.8	21.3	23.9
	14	1.11	2.32	3.69	5.21	6.90	8.73	10.7	12.8	15.0	17.4	19.8	22.3
	16	0.99	2.09	3.34	4.74	6.29	8.00	9.85	11.8	13.9	16.1	18.5	20.9
	18	0.90	1.90	3.04	4.33	5.77	7.36	9.10	11.0	12.9	15.0	17.3	19.5
	20	0.81	1.73	2.79	3.98	5.33	6.81	8.44	10.2	12.1	14.1	16.2	18.4
	24	0.68	1.47	2.38	3.42	4.60	5.91	7.35	8.91	10.6	12.4	14.3	16.3
	28	0.59	1.28	2.08	2.99	4.03	5.20	6.49	7.90	9.42	11.1	12.8	14.6
	32	0.52	1.13	1.84	2.65	3.59	4.63	5.80	7.07	8.46	9.95	11.6	13.3
	36	0.46	1.01	1.65	2.38	3.23	4.17	5.23	6.40	7.67	9.04	10.5	12.1
6	2	2.64	5.38	8.22	11.1	14.1	17.0	20.0	23.0	25.9	28.9	31.9	34.8
	3	2.43	5.02	7.78	10.7	13.6	16.6	19.5	22.5	25.5	28.5	31.4	34.4
	4	2.23	4.67	7.33	10.2	13.1	16.0	19.0	22.0	25.0	28.0	31.0	33.9
	5	2.05	4.34	6.90	9.66	12.5	15.5	18.4	21.4	24.4	27.4	30.4	33.4
	6	1.89	4.06	6.50	9.19	12.0	14.9	17.9	20.9	23.9	26.9	29.9	32.9
	7	1.75	3.80	6.16	8.76	11.5	14.4	17.3	20.3	23.3	26.3	29.3	32.3
	8	1.63	3.57	5.84	8.36	11.1	13.9	16.8	19.7	22.7	25.7	28.7	31.7
	9	1.52	3.36	5.54	7.99	10.6	13.4	16.2	19.2	22.1	25.1	28.1	31.1
	10	1.42	3.17	5.27	7.63	10.2	12.9	15.7	18.6	21.5	24.5	27.5	30.5
	12	1.25	2.84	4.78	6.99	9.40	12.0	14.7	17.6	20.4	23.4	26.3	29.3
	14	1.11	2.57	4.36	6.42	8.70	11.2	13.8	16.6	19.4	22.3	25.2	28.2
	16	0.99	2.33	3.99	5.92	8.09	10.5	13.0	15.7	18.4	21.2	24.1	27.0
	18	0.90	2.13	3.68	5.49	7.54	9.80	12.2	14.8	17.5	20.3	23.1	26.0
	20	0.81	1.96	3.40	5.10	7.05	9.21	11.6	14.0	16.6	19.3	22.1	24.9
	24	0.68	1.68	2.95	4.46	6.22	8.19	10.4	12.7	15.1	17.7	20.3	23.0
	28	0.59	1.47	2.59	3.95	5.55	7.35	9.34	11.5	13.8	16.2	18.7	21.3
	32	0.52	1.31	2.31	3.54	4.99	6.65	8.49	10.5	12.7	14.9	17.3	19.8
	36	0.46	1.17	2.08	3.20	4.54	6.06	7.77	9.64	11.7	13.8	16.1	18.5

Table 7-11 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 60°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

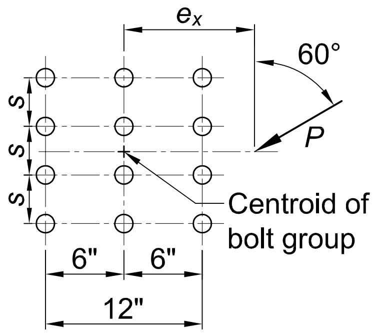
P = required force, P_u or P_a, kips

r_n = nominal strength per bolt, kips

e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	*ex*, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	2.83	5.64	8.45	11.3	14.1	16.9	19.8	22.6	25.5	28.4	31.3	34.2
	3	2.72	5.43	8.13	10.8	13.6	16.3	19.1	21.9	24.8	27.6	30.5	33.4
	4	2.59	5.18	7.77	10.4	13.0	15.7	18.5	21.2	24.0	26.8	29.7	32.5
	5	2.46	4.92	7.40	9.92	12.5	15.1	17.8	20.5	23.2	26.0	28.9	31.7
	6	2.32	4.66	7.03	9.46	12.0	14.5	17.1	19.8	22.5	25.2	28.0	30.8
	7	2.19	4.41	6.68	9.02	11.4	13.9	16.5	19.1	21.8	24.5	27.2	30.0
	8	2.07	4.17	6.35	8.61	11.0	13.4	15.9	18.4	21.1	23.7	26.5	29.2
	9	1.95	3.95	6.04	8.22	10.5	12.9	15.3	17.8	20.4	23.0	25.7	28.5
	10	1.84	3.74	5.75	7.86	10.1	12.4	14.8	17.3	19.8	22.4	25.0	27.7
	12	1.65	3.38	5.22	7.19	9.28	11.5	13.8	16.2	18.6	21.1	23.7	26.3
	14	1.49	3.06	4.76	6.61	8.58	10.7	12.9	15.2	17.5	20.0	22.5	25.0
	16	1.35	2.79	4.37	6.09	7.95	9.93	12.0	14.2	16.5	18.9	21.3	23.8
	18	1.23	2.55	4.02	5.64	7.39	9.28	11.3	13.4	15.6	17.9	20.3	22.7
	20	1.12	2.35	3.72	5.24	6.90	8.69	10.6	12.6	14.8	17.0	19.3	21.7
	24	0.95	2.02	3.22	4.57	6.06	7.68	9.43	11.3	13.3	15.4	17.5	19.8
	28	0.83	1.76	2.84	4.04	5.39	6.86	8.47	10.2	12.0	14.0	16.0	18.1
	32	0.73	1.56	2.53	3.61	4.84	6.19	7.66	9.26	11.0	12.8	14.7	16.7
	36	0.65	1.40	2.27	3.26	4.38	5.62	6.98	8.46	10.1	11.7	13.5	15.4
6	2	2.83	5.64	8.47	11.3	14.2	17.1	20.0	23.0	25.9	28.9	31.8	34.8
	3	2.72	5.44	8.19	11.0	13.8	16.7	19.6	22.6	25.5	28.4	31.4	34.3
	4	2.59	5.21	7.88	10.6	13.4	16.3	19.2	22.1	25.0	28.0	30.9	33.9
	5	2.46	4.97	7.57	10.3	13.1	15.9	18.8	21.7	24.6	27.5	30.4	33.4
	6	2.32	4.73	7.27	9.91	12.7	15.5	18.3	21.2	24.1	27.0	30.0	32.9
	7	2.19	4.51	6.97	9.56	12.3	15.0	17.9	20.8	23.7	26.6	29.5	32.4
	8	2.07	4.29	6.69	9.23	11.9	14.6	17.5	20.3	23.2	26.1	29.0	32.0
	9	1.95	4.09	6.43	8.92	11.5	14.3	17.0	19.9	22.8	25.6	28.6	31.5
	10	1.84	3.90	6.18	8.63	11.2	13.9	16.6	19.5	22.3	25.2	28.1	31.0
	12	1.65	3.56	5.73	8.08	10.6	13.2	15.9	18.7	21.5	24.3	27.2	30.1
	14	1.49	3.27	5.32	7.59	10.0	12.6	15.2	17.9	20.7	23.5	26.3	29.2
	16	1.35	3.01	4.95	7.13	9.48	12.0	14.5	17.2	19.9	22.7	25.5	28.4
	18	1.23	2.78	4.63	6.71	8.98	11.4	13.9	16.5	19.2	22.0	24.7	27.6
	20	1.12	2.58	4.34	6.33	8.52	10.9	13.3	15.9	18.5	21.2	24.0	26.8
	24	0.95	2.25	3.84	5.67	7.70	9.91	12.3	14.7	17.3	19.9	22.6	25.3
	28	0.83	1.98	3.43	5.11	7.00	9.08	11.3	13.7	16.1	18.7	21.3	23.9
	32	0.73	1.77	3.09	4.64	6.40	8.36	10.5	12.7	15.1	17.5	20.1	22.6
	36	0.65	1.60	2.81	4.24	5.89	7.73	9.74	11.9	14.2	16.5	19.0	21.5

Table 7-11 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 75°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

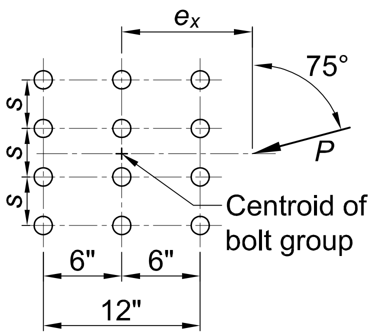
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	2.92	5.83	8.73	11.6	14.5	17.4	20.3	23.1	26.0	28.9	31.8	34.7
	3	2.89	5.77	8.63	11.5	14.3	17.2	20.0	22.8	25.7	28.5	31.4	34.2
	4	2.86	5.70	8.51	11.3	14.1	16.9	19.7	22.5	25.3	28.1	30.9	33.7
	5	2.82	5.61	8.38	11.1	13.9	16.6	19.4	22.1	24.9	27.7	30.5	33.3
	6	2.77	5.51	8.23	10.9	13.6	16.3	19.0	21.8	24.5	27.2	30.0	32.8
	7	2.72	5.40	8.06	10.7	13.4	16.0	18.7	21.4	24.1	26.8	29.6	32.3
	8	2.66	5.29	7.89	10.5	13.1	15.7	18.3	21.0	23.7	26.4	29.1	31.9
	9	2.60	5.16	7.71	10.3	12.8	15.4	18.0	20.6	23.3	26.0	28.7	31.4
	10	2.53	5.04	7.53	10.0	12.6	15.1	17.7	20.3	22.9	25.6	28.3	31.0
	12	2.40	4.78	7.16	9.57	12.0	14.5	17.0	19.6	22.1	24.8	27.4	30.1
	14	2.26	4.52	6.80	9.12	11.5	13.9	16.4	18.9	21.4	24.0	26.6	29.3
	16	2.13	4.27	6.45	8.68	11.0	13.3	15.8	18.2	20.7	23.3	25.9	28.5
	18	2.00	4.03	6.12	8.27	10.5	12.8	15.2	17.6	20.1	22.6	25.1	27.7
	20	1.89	3.81	5.80	7.88	10.1	12.3	14.6	17.0	19.4	21.9	24.4	27.0
	24	1.67	3.41	5.24	7.18	9.22	11.4	13.6	15.9	18.2	20.7	23.1	25.6
	28	1.49	3.06	4.75	6.56	8.49	10.5	12.6	14.9	17.1	19.5	21.9	24.3
	32	1.34	2.77	4.33	6.02	7.84	9.77	11.8	13.9	16.1	18.4	20.7	23.1
	36	1.21	2.52	3.97	5.56	7.27	9.10	11.1	13.1	15.2	17.4	19.7	22.0
6	2	2.92	5.82	8.71	11.6	14.5	17.4	20.3	23.5	26.4	29.3	32.3	35.2
	3	2.89	5.76	8.60	11.4	14.3	17.1	20.0	22.9	25.8	28.7	31.7	34.6
	4	2.86	5.68	8.47	11.3	14.1	16.9	19.8	22.6	25.5	28.4	31.3	34.2
	5	2.82	5.59	8.34	11.1	13.9	16.7	19.5	22.4	25.2	28.1	31.0	33.9
	6	2.77	5.49	8.19	10.9	13.7	16.4	19.2	22.1	24.9	27.8	30.7	33.6
	7	2.72	5.39	8.04	10.7	13.4	16.2	19.0	21.8	24.6	27.5	30.4	33.3
	8	2.66	5.27	7.89	10.5	13.2	16.0	18.8	21.6	24.4	27.2	30.1	33.0
	9	2.60	5.16	7.74	10.4	13.0	15.8	18.5	21.3	24.1	27.0	29.8	32.7
	10	2.53	5.04	7.58	10.2	12.8	15.5	18.3	21.0	23.9	26.7	29.5	32.4
	12	2.40	4.81	7.27	9.81	12.4	15.1	17.8	20.6	23.3	26.2	29.0	31.8
	14	2.26	4.57	6.97	9.47	12.0	14.7	17.4	20.1	22.9	25.6	28.4	31.3
	16	2.13	4.35	6.69	9.13	11.7	14.3	16.9	19.6	22.4	25.1	27.9	30.7
	18	2.00	4.13	6.41	8.82	11.3	13.9	16.5	19.2	21.9	24.7	27.4	30.2
	20	1.89	3.93	6.15	8.51	11.0	13.5	16.1	18.8	21.5	24.2	27.0	29.8
	24	1.67	3.57	5.67	7.95	10.4	12.9	15.4	18.0	20.7	23.4	26.1	28.8
	28	1.49	3.25	5.25	7.44	9.77	12.2	14.7	17.3	19.9	22.6	25.3	28.0
	32	1.34	2.97	4.87	6.98	9.23	11.6	14.1	16.6	19.2	21.8	24.5	27.2
	36	1.21	2.73	4.54	6.56	8.74	11.1	13.5	16.0	18.5	21.1	23.7	26.4

Table 7-12
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 0°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

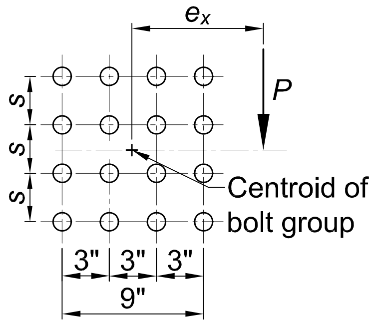
P = required force, P_u or P_a, kips

r_n = nominal strength per bolt, kips

e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	2.60	5.70	9.24	13.2	17.3	21.4	25.6	29.7	33.8	37.8	41.9	45.9
	3	2.23	4.92	8.05	11.7	15.6	19.7	23.9	28.1	32.3	36.4	40.6	44.7
	4	1.94	4.30	7.09	10.4	14.0	18.0	22.1	26.3	30.5	34.7	38.9	43.1
	5	1.69	3.79	6.30	9.29	12.6	16.4	20.3	24.4	28.6	32.9	37.1	41.4
	6	1.49	3.37	5.65	8.37	11.5	14.9	18.7	22.6	26.7	30.9	35.2	39.4
	7	1.32	3.03	5.10	7.59	10.4	13.7	17.2	21.0	24.9	29.0	33.2	37.5
	8	1.18	2.74	4.63	6.92	9.56	12.6	15.9	19.5	23.3	27.3	31.4	35.5
	9	1.07	2.50	4.24	6.35	8.81	11.6	14.7	18.1	21.7	25.6	29.6	33.7
	10	0.98	2.29	3.89	5.86	8.15	10.8	13.7	16.9	20.3	24.0	27.9	31.9
	12	0.83	1.96	3.34	5.06	7.06	9.37	12.0	14.8	17.9	21.3	24.9	28.6
	14	0.73	1.72	2.92	4.44	6.21	8.27	10.6	13.2	16.0	19.1	22.3	25.8
	16	0.65	1.52	2.59	3.95	5.54	7.39	9.48	11.8	14.4	17.2	20.2	23.4
	18	0.58	1.37	2.33	3.55	4.99	6.67	8.57	10.7	13.1	15.6	18.4	21.4
	20	0.53	1.24	2.11	3.23	4.53	6.07	7.81	9.77	11.9	14.3	16.9	19.6
	24	0.44	1.04	1.78	2.72	3.83	5.14	6.62	8.30	10.2	12.2	14.4	16.8
	28	0.38	0.90	1.54	2.35	3.31	4.45	5.73	7.20	8.82	10.6	12.6	14.7
	32	0.34	0.79	1.36	2.07	2.91	3.92	5.05	6.35	7.79	9.38	11.1	13.0
	36	0.30	0.71	1.21	1.85	2.60	3.50	4.51	5.68	6.96	8.39	9.95	11.6
	C′, in.	11.3	26.0	44.7	68.1	96.0	129	167	210	258	312	371	435
6	2	2.60	6.48	10.7	14.8	18.9	23.0	27.0	31.0	34.9	38.9	42.9	46.8
	3	2.23	5.75	9.79	14.0	18.2	22.3	26.4	30.5	34.5	38.5	42.5	46.5
	4	1.94	5.12	8.91	13.1	17.4	21.6	25.7	29.9	33.9	38.0	42.0	46.1
	5	1.69	4.58	8.10	12.2	16.4	20.7	24.9	29.1	33.2	37.4	41.4	45.5
	6	1.49	4.13	7.37	11.3	15.5	19.7	24.0	28.3	32.5	36.6	40.8	44.9
	7	1.32	3.74	6.74	10.5	14.5	18.8	23.1	27.3	31.6	35.8	40.0	44.1
	8	1.18	3.41	6.20	9.73	13.6	17.8	22.1	26.4	30.6	34.9	39.1	43.3
	9	1.07	3.13	5.73	9.05	12.8	16.9	21.1	25.4	29.7	34.0	38.2	42.5
	10	0.98	2.89	5.31	8.45	12.0	16.0	20.1	24.4	28.7	33.0	37.3	41.5
	12	0.83	2.50	4.63	7.43	10.7	14.3	18.3	22.4	26.7	31.0	35.3	39.6
	14	0.73	2.19	4.09	6.60	9.53	12.9	16.7	20.6	24.7	29.0	33.3	37.6
	16	0.65	1.95	3.65	5.93	8.59	11.7	15.2	19.0	22.9	27.1	31.3	35.5
	18	0.58	1.76	3.29	5.37	7.81	10.7	14.0	17.5	21.3	25.3	29.4	33.6
	20	0.53	1.60	2.99	4.90	7.15	9.85	12.9	16.2	19.8	23.6	27.6	31.7
	24	0.44	1.35	2.53	4.16	6.10	8.44	11.1	14.0	17.3	20.8	24.4	28.3
	28	0.38	1.17	2.19	3.61	5.31	7.37	9.69	12.3	15.2	18.4	21.8	25.3
	32	0.34	1.03	1.93	3.19	4.69	6.53	8.61	11.0	13.6	16.5	19.6	22.9
	36	0.30	0.92	1.72	2.85	4.20	5.85	7.73	9.89	12.3	14.9	17.7	20.8
	C′, in.	11.3	33.7	63.7	106	156	219	291	375	469	574	690	817

Table 7-12 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 15°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

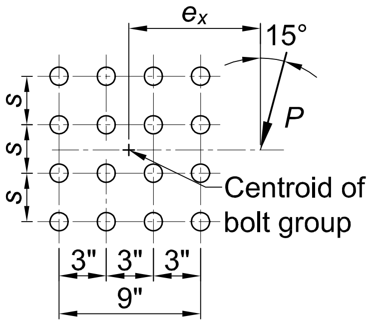
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	2.68	5.77	9.31	13.2	17.2	21.3	25.4	29.5	33.6	37.6	41.7	45.7
	3	2.30	5.00	8.17	11.7	15.6	19.6	23.7	27.8	32.0	36.1	40.2	44.3
	4	1.99	4.38	7.22	10.4	14.1	17.9	21.9	26.0	30.2	34.4	38.5	42.7
	5	1.74	3.88	6.43	9.37	12.7	16.4	20.2	24.2	28.3	32.5	36.7	40.9
	6	1.53	3.45	5.77	8.47	11.6	15.0	18.6	22.5	26.5	30.6	34.8	39.0
	7	1.36	3.10	5.21	7.71	10.6	13.7	17.2	20.9	24.8	28.8	32.9	37.1
	8	1.22	2.81	4.74	7.05	9.70	12.7	15.9	19.5	23.2	27.1	31.1	35.2
	9	1.11	2.57	4.34	6.48	8.95	11.7	14.8	18.1	21.7	25.5	29.4	33.4
	10	1.01	2.36	4.00	5.98	8.29	10.9	13.8	17.0	20.4	24.0	27.7	31.6
	12	0.86	2.02	3.44	5.18	7.21	9.52	12.1	15.0	18.1	21.4	24.9	28.5
	14	0.75	1.77	3.01	4.55	6.36	8.43	10.8	13.3	16.1	19.2	22.4	25.8
	16	0.67	1.57	2.68	4.05	5.67	7.54	9.66	12.0	14.6	17.3	20.3	23.5
	18	0.60	1.41	2.40	3.65	5.12	6.81	8.74	10.9	13.3	15.8	18.6	21.5
	20	0.54	1.28	2.18	3.32	4.66	6.21	7.98	9.95	12.1	14.5	17.1	19.8
	24	0.46	1.08	1.84	2.80	3.94	5.26	6.78	8.47	10.4	12.4	14.6	17.0
	28	0.40	0.93	1.59	2.43	3.41	4.56	5.89	7.37	9.02	10.8	12.8	14.9
	32	0.35	0.82	1.40	2.14	3.00	4.03	5.19	6.51	7.98	9.59	11.3	13.2
	36	0.31	0.73	1.25	1.91	2.68	3.60	4.65	5.83	7.15	8.59	10.2	11.9
6	2	2.68	6.48	10.6	14.7	18.8	22.9	26.9	30.9	34.8	38.8	42.8	46.7
	3	2.30	5.75	9.75	13.9	18.1	22.2	26.3	30.3	34.3	38.3	42.3	46.3
	4	1.99	5.13	8.91	13.0	17.2	21.4	25.5	29.6	33.7	37.7	41.8	45.8
	5	1.74	4.61	8.14	12.1	16.3	20.5	24.7	28.8	33.0	37.1	41.1	45.2
	6	1.53	4.17	7.45	11.2	15.3	19.5	23.7	27.9	32.1	36.3	40.4	44.5
	7	1.36	3.79	6.84	10.4	14.4	18.6	22.8	27.0	31.2	35.4	39.6	43.7
	8	1.22	3.46	6.30	9.71	13.6	17.6	21.8	26.0	30.3	34.5	38.7	42.9
	9	1.11	3.19	5.83	9.05	12.8	16.7	20.9	25.1	29.3	33.5	37.8	42.0
	10	1.01	2.94	5.42	8.47	12.0	15.9	19.9	24.1	28.3	32.6	36.8	41.0
	12	0.86	2.55	4.73	7.47	10.7	14.3	18.2	22.2	26.4	30.6	34.8	39.1
	14	0.75	2.24	4.18	6.66	9.62	12.9	16.6	20.5	24.5	28.6	32.8	37.1
	16	0.67	2.00	3.74	6.00	8.71	11.8	15.2	18.9	22.8	26.8	30.9	35.1
	18	0.60	1.80	3.38	5.45	7.94	10.8	14.0	17.5	21.2	25.1	29.1	33.2
	20	0.54	1.64	3.08	4.98	7.28	9.92	13.0	16.2	19.8	23.5	27.4	31.4
	24	0.46	1.39	2.60	4.25	6.23	8.54	11.2	14.1	17.3	20.8	24.4	28.1
	28	0.40	1.20	2.26	3.69	5.43	7.48	9.85	12.5	15.4	18.5	21.8	25.3
	32	0.35	1.06	1.99	3.26	4.81	6.65	8.77	11.1	13.8	16.6	19.7	22.9
	36	0.31	0.94	1.78	2.92	4.31	5.97	7.89	10.0	12.5	15.1	17.9	20.9

Table 7-12 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 30°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

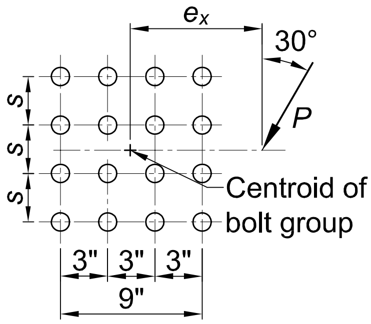
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	2.90	6.06	9.59	13.4	17.3	21.3	25.3	29.4	33.4	37.4	41.4	45.4
	3	2.50	5.31	8.52	12.1	15.8	19.7	23.7	27.8	31.8	35.9	40.0	44.0
	4	2.18	4.70	7.62	10.9	14.4	18.2	22.1	26.1	30.1	34.2	38.3	42.4
	5	1.91	4.18	6.85	9.86	13.2	16.8	20.5	24.4	28.4	32.5	36.6	40.7
	6	1.69	3.75	6.19	8.98	12.1	15.5	19.1	22.9	26.8	30.7	34.8	38.9
	7	1.51	3.38	5.63	8.21	11.1	14.3	17.8	21.4	25.2	29.1	33.1	37.1
	8	1.36	3.07	5.14	7.55	10.3	13.3	16.6	20.0	23.7	27.5	31.4	35.4
	9	1.23	2.81	4.73	6.97	9.54	12.4	15.5	18.8	22.3	26.0	29.8	33.7
	10	1.13	2.59	4.37	6.46	8.88	11.6	14.5	17.7	21.1	24.7	28.3	32.2
	12	0.96	2.23	3.78	5.62	7.78	10.2	12.9	15.8	18.9	22.2	25.7	29.3
	14	0.84	1.95	3.32	4.96	6.90	9.08	11.5	14.2	17.1	20.1	23.4	26.8
	16	0.74	1.73	2.96	4.43	6.19	8.17	10.4	12.9	15.5	18.4	21.4	24.6
	18	0.67	1.56	2.66	4.00	5.60	7.41	9.46	11.7	14.2	16.8	19.7	22.7
	20	0.61	1.42	2.42	3.65	5.11	6.77	8.67	10.8	13.1	15.5	18.2	21.0
	24	0.51	1.20	2.04	3.09	4.34	5.77	7.41	9.22	11.2	13.4	15.7	18.2
	28	0.44	1.03	1.77	2.68	3.77	5.01	6.46	8.05	9.83	11.8	13.9	16.1
	32	0.39	0.91	1.56	2.36	3.32	4.43	5.71	7.14	8.72	10.5	12.3	14.4
	36	0.35	0.81	1.39	2.11	2.97	3.97	5.12	6.40	7.84	9.41	11.1	13.0
6	2	2.90	6.59	10.6	14.7	18.7	22.7	26.7	30.7	34.7	38.7	42.6	46.6
	3	2.50	5.88	9.83	13.9	18.0	22.0	26.1	30.1	34.1	38.1	42.1	46.1
	4	2.18	5.30	9.05	13.0	17.1	21.2	25.3	29.4	33.5	37.5	41.5	45.5
	5	1.91	4.81	8.35	12.2	16.3	20.4	24.5	28.6	32.7	36.8	40.8	44.9
	6	1.69	4.38	7.72	11.4	15.4	19.5	23.6	27.7	31.8	35.9	40.0	44.1
	7	1.51	4.01	7.15	10.7	14.6	18.6	22.7	26.8	31.0	35.1	39.2	43.3
	8	1.36	3.69	6.64	10.0	13.8	17.7	21.8	25.9	30.0	34.2	38.3	42.4
	9	1.23	3.41	6.19	9.41	13.0	16.9	20.9	25.0	29.1	33.3	37.4	41.6
	10	1.13	3.16	5.79	8.85	12.4	16.1	20.1	24.1	28.2	32.4	36.5	40.6
	12	0.96	2.76	5.09	7.88	11.1	14.7	18.5	22.4	26.4	30.5	34.6	38.8
	14	0.84	2.44	4.54	7.08	10.1	13.4	17.0	20.8	24.7	28.8	32.8	36.9
	16	0.74	2.18	4.08	6.41	9.21	12.3	15.7	19.4	23.2	27.1	31.1	35.1
	18	0.67	1.97	3.70	5.85	8.45	11.4	14.6	18.1	21.7	25.5	29.4	33.4
	20	0.61	1.80	3.38	5.37	7.80	10.5	13.6	16.9	20.4	24.1	27.9	31.8
	24	0.51	1.53	2.87	4.61	6.74	9.16	11.9	14.9	18.1	21.5	25.1	28.8
	28	0.44	1.32	2.49	4.02	5.91	8.07	10.5	13.3	16.2	19.4	22.7	26.2
	32	0.39	1.17	2.20	3.57	5.26	7.20	9.45	11.9	14.6	17.6	20.7	23.9
	36	0.35	1.05	1.97	3.21	4.73	6.49	8.55	10.8	13.3	16.0	18.9	22.0

Table 7-12 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 45°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

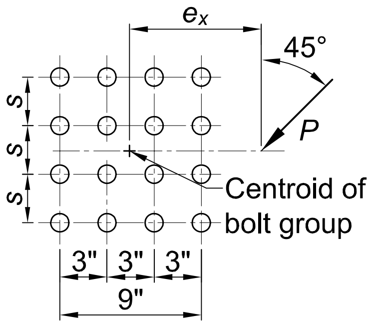
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	3.26	6.62	10.2	13.9	17.7	21.5	25.5	29.4	33.4	37.3	41.3	45.3
	3	2.87	5.92	9.19	12.7	16.4	20.2	24.0	28.0	31.9	35.9	39.9	43.9
	4	2.54	5.31	8.36	11.7	15.2	18.8	22.6	26.5	30.4	34.4	38.4	42.4
	5	2.25	4.78	7.63	10.8	14.1	17.6	21.3	25.1	29.0	32.9	36.8	40.8
	6	2.01	4.33	6.99	9.94	13.1	16.5	20.1	23.8	27.5	31.4	35.3	39.3
	7	1.81	3.93	6.42	9.20	12.2	15.5	18.9	22.5	26.2	30.0	33.8	37.7
	8	1.64	3.60	5.92	8.55	11.4	14.6	17.9	21.3	24.9	28.6	32.4	36.3
	9	1.49	3.31	5.49	7.96	10.7	13.7	16.9	20.3	23.8	27.4	31.1	34.9
	10	1.37	3.06	5.10	7.44	10.1	12.9	16.0	19.2	22.7	26.2	29.8	33.6
	12	1.17	2.65	4.46	6.55	8.93	11.6	14.4	17.5	20.7	24.0	27.5	31.1
	14	1.03	2.33	3.95	5.83	8.00	10.4	13.1	15.9	18.9	22.1	25.4	28.8
	16	0.91	2.08	3.54	5.24	7.23	9.47	11.9	14.6	17.4	20.4	23.6	26.8
	18	0.82	1.88	3.20	4.75	6.59	8.66	10.9	13.4	16.1	18.9	21.9	25.0
	20	0.74	1.71	2.92	4.35	6.04	7.96	10.1	12.4	15.0	17.6	20.5	23.5
	24	0.63	1.45	2.48	3.71	5.18	6.84	8.71	10.8	13.0	15.4	18.0	20.7
	28	0.54	1.26	2.15	3.23	4.52	5.99	7.65	9.50	11.5	13.7	16.0	18.5
	32	0.48	1.11	1.90	2.86	4.00	5.31	6.81	8.48	10.3	12.3	14.4	16.7
	36	0.43	0.99	1.69	2.56	3.59	4.77	6.13	7.64	9.30	11.1	13.1	15.2
6	2	3.26	6.89	10.8	14.7	18.7	22.7	26.6	30.6	34.6	38.5	42.5	46.5
	3	2.87	6.28	10.1	14.0	18.0	22.0	26.0	30.0	33.9	37.9	41.9	45.9
	4	2.54	5.74	9.38	13.3	17.2	21.2	25.2	29.2	33.2	37.2	41.2	45.2
	5	2.25	5.27	8.75	12.6	16.5	20.4	24.5	28.5	32.5	36.5	40.5	44.5
	6	2.01	4.85	8.20	11.9	15.7	19.7	23.7	27.7	31.7	35.7	39.7	43.8
	7	1.81	4.49	7.70	11.3	15.0	18.9	22.9	26.9	30.9	34.9	39.0	43.0
	8	1.64	4.16	7.25	10.7	14.4	18.2	22.1	26.1	30.1	34.1	38.2	42.2
	9	1.49	3.87	6.83	10.2	13.7	17.5	21.4	25.3	29.3	33.3	37.4	41.4
	10	1.37	3.62	6.45	9.65	13.1	16.8	20.7	24.6	28.5	32.5	36.6	40.6
	12	1.17	3.19	5.78	8.75	12.0	15.6	19.3	23.1	27.0	31.0	35.0	39.0
	14	1.03	2.84	5.21	7.97	11.1	14.5	18.1	21.8	25.6	29.5	33.4	37.4
	16	0.91	2.56	4.74	7.30	10.2	13.5	16.9	20.5	24.3	28.1	32.0	35.9
	18	0.82	2.33	4.33	6.72	9.48	12.6	15.9	19.4	23.0	26.7	30.6	34.4
	20	0.74	2.13	3.98	6.21	8.83	11.8	15.0	18.3	21.8	25.5	29.2	33.1
	24	0.63	1.82	3.42	5.38	7.74	10.4	13.3	16.5	19.8	23.2	26.8	30.5
	28	0.54	1.59	2.99	4.74	6.87	9.30	12.0	14.9	18.0	21.3	24.7	28.2
	32	0.48	1.41	2.65	4.22	6.17	8.38	10.8	13.6	16.5	19.5	22.8	26.1
	36	0.43	1.26	2.38	3.81	5.59	7.62	9.89	12.4	15.2	18.0	21.1	24.3

Table 7-12 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 60°

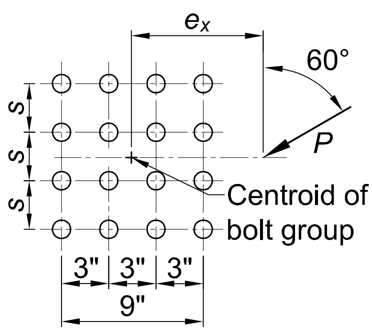
Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where
P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	3.63	7.25	10.9	14.6	18.3	22.1	25.9	29.7	33.6	37.5	41.4	45.3
	3	3.38	6.77	10.2	13.8	17.4	21.1	24.8	28.6	32.4	36.3	40.2	44.1
	4	3.10	6.27	9.55	13.0	16.5	20.1	23.7	27.5	31.3	35.1	38.9	42.8
	5	2.84	5.80	8.92	12.2	15.6	19.1	22.7	26.4	30.1	33.9	37.8	41.6
	6	2.60	5.36	8.33	11.5	14.8	18.2	21.7	25.4	29.1	32.8	36.6	40.4
	7	2.38	4.96	7.79	10.8	14.1	17.4	20.9	24.4	28.0	31.7	35.5	39.3
	8	2.19	4.60	7.30	10.2	13.4	16.6	20.0	23.5	27.1	30.7	34.4	38.2
	9	2.02	4.28	6.85	9.68	12.7	15.9	19.2	22.6	26.1	29.7	33.4	37.1
	10	1.87	3.99	6.45	9.17	12.1	15.2	18.4	21.8	25.3	28.8	32.4	36.1
	12	1.62	3.51	5.75	8.27	11.0	13.9	17.0	20.3	23.6	27.0	30.6	34.1
	14	1.43	3.12	5.18	7.50	10.1	12.8	15.8	18.9	22.1	25.4	28.9	32.4
	16	1.27	2.81	4.70	6.85	9.23	11.9	14.7	17.6	20.7	24.0	27.3	30.7
	18	1.15	2.56	4.29	6.28	8.52	11.0	13.7	16.5	19.5	22.6	25.8	29.1
	20	1.04	2.34	3.95	5.80	7.89	10.2	12.8	15.5	18.4	21.4	24.5	27.7
	24	0.88	2.00	3.39	5.01	6.87	8.98	11.3	13.8	16.4	19.2	22.1	25.2
	28	0.76	1.74	2.96	4.39	6.07	7.97	10.1	12.3	14.8	17.4	20.1	23.0
	32	0.67	1.54	2.63	3.91	5.43	7.15	9.06	11.2	13.4	15.8	18.4	21.1
	36	0.60	1.38	2.36	3.52	4.91	6.48	8.22	10.2	12.3	14.5	16.9	19.4
6	2	3.63	7.29	11.1	14.9	18.8	22.7	26.6	30.5	34.5	38.4	42.4	46.3
	3	3.38	6.88	10.6	14.3	18.2	22.1	26.0	29.9	33.9	37.8	41.8	45.7
	4	3.10	6.46	10.0	13.8	17.6	21.5	25.4	29.3	33.2	37.2	41.1	45.1
	5	2.84	6.06	9.55	13.2	17.0	20.9	24.7	28.7	32.6	36.5	40.4	44.4
	6	2.60	5.69	9.09	12.7	16.4	20.3	24.1	28.0	31.9	35.9	39.8	43.8
	7	2.38	5.34	8.66	12.2	15.9	19.7	23.5	27.4	31.3	35.2	39.2	43.1
	8	2.19	5.03	8.27	11.7	15.4	19.1	22.9	26.8	30.7	34.6	38.5	42.4
	9	2.02	4.74	7.90	11.3	14.9	18.6	22.4	26.2	30.1	34.0	37.9	41.8
	10	1.87	4.47	7.55	10.9	14.4	18.1	21.8	25.6	29.5	33.4	37.3	41.2
	12	1.62	4.01	6.93	10.1	13.6	17.1	20.8	24.5	28.3	32.2	36.0	39.9
	14	1.43	3.63	6.38	9.46	12.8	16.2	19.8	23.5	27.3	31.0	34.9	38.7
	16	1.27	3.31	5.91	8.84	12.0	15.4	18.9	22.5	26.2	30.0	33.8	37.6
	18	1.15	3.04	5.49	8.28	11.3	14.6	18.0	21.6	25.2	28.9	32.7	36.5
	20	1.04	2.81	5.12	7.77	10.7	13.9	17.2	20.7	24.3	28.0	31.7	35.4
	24	0.88	2.44	4.49	6.90	9.62	12.6	15.8	19.1	22.6	26.1	29.8	33.4
	28	0.76	2.15	3.99	6.18	8.70	11.5	14.5	17.7	21.1	24.5	28.0	31.6
	32	0.67	1.91	3.58	5.58	7.93	10.6	13.4	16.5	19.7	23.0	26.4	29.9
	36	0.60	1.73	3.24	5.08	7.27	9.76	12.5	15.4	18.4	21.6	24.9	28.3

Table 7-12 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 75°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

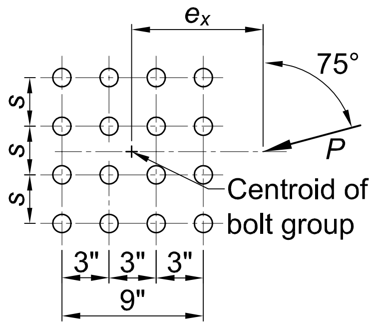
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	3.86	7.69	11.5	15.3	19.1	22.9	26.7	30.5	34.3	38.2	42.0	45.9
	3	3.79	7.53	11.2	14.9	18.6	22.4	26.1	29.9	33.6	37.4	41.3	45.1
	4	3.70	7.34	11.0	14.6	18.2	21.8	25.5	29.2	33.0	36.7	40.5	44.3
	5	3.59	7.13	10.6	14.2	17.7	21.3	24.9	28.6	32.3	36.1	39.8	43.6
	6	3.47	6.89	10.3	13.8	17.2	20.8	24.4	28.0	31.7	35.4	39.1	42.9
	7	3.34	6.65	9.98	13.4	16.8	20.3	23.8	27.4	31.1	34.7	38.4	42.2
	8	3.20	6.40	9.64	12.9	16.3	19.8	23.3	26.8	30.4	34.1	37.8	41.5
	9	3.07	6.16	9.31	12.6	15.9	19.3	22.8	26.3	29.9	33.5	37.1	40.8
	10	2.94	5.91	8.98	12.2	15.4	18.8	22.2	25.7	29.3	32.9	36.5	40.2
	12	2.68	5.45	8.36	11.4	14.6	17.9	21.3	24.7	28.2	31.8	35.4	39.0
	14	2.45	5.03	7.79	10.7	13.8	17.1	20.4	23.8	27.2	30.7	34.3	37.9
	16	2.24	4.65	7.28	10.1	13.1	16.3	19.5	22.9	26.3	29.7	33.2	36.8
	18	2.06	4.31	6.81	9.55	12.5	15.5	18.7	22.0	25.4	28.8	32.2	35.8
	20	1.90	4.01	6.40	9.03	11.9	14.8	18.0	21.2	24.5	27.9	31.3	34.8
	24	1.63	3.51	5.69	8.13	10.8	13.6	16.6	19.7	22.8	26.1	29.5	32.9
	28	1.43	3.11	5.11	7.36	9.83	12.5	15.3	18.3	21.4	24.6	27.8	31.1
	32	1.27	2.79	4.62	6.71	9.02	11.5	14.2	17.1	20.0	23.1	26.3	29.5
	36	1.14	2.53	4.22	6.15	8.31	10.7	13.3	16.0	18.8	21.8	24.9	28.0
6	2	3.86	7.67	11.5	15.3	19.1	23.0	26.9	30.8	35.2	39.1	43.0	47.0
	3	3.79	7.51	11.2	15.0	18.8	22.6	26.4	30.3	34.2	38.1	42.1	46.0
	4	3.70	7.32	11.0	14.7	18.4	22.2	26.0	29.9	33.8	37.7	41.6	45.5
	5	3.59	7.12	10.7	14.4	18.1	21.8	25.6	29.5	33.3	37.2	41.1	45.0
	6	3.47	6.92	10.4	14.1	17.7	21.5	25.3	29.1	32.9	36.8	40.7	44.6
	7	3.34	6.70	10.2	13.8	17.4	21.1	24.9	28.7	32.5	36.4	40.2	44.1
	8	3.20	6.49	9.92	13.5	17.1	20.8	24.5	28.3	32.1	36.0	39.8	43.7
	9	3.07	6.28	9.66	13.2	16.8	20.5	24.2	28.0	31.8	35.6	39.4	43.3
	10	2.94	6.08	9.42	12.9	16.5	20.2	23.9	27.6	31.4	35.2	39.0	42.9
	12	2.68	5.69	8.95	12.4	15.9	19.5	23.2	26.9	30.7	34.5	38.3	42.1
	14	2.45	5.33	8.51	11.9	15.4	19.0	22.6	26.3	30.0	33.8	37.6	41.4
	16	2.24	4.99	8.10	11.4	14.9	18.4	22.0	25.7	29.4	33.1	36.9	40.7
	18	2.06	4.69	7.72	11.0	14.4	17.9	21.5	25.1	28.8	32.5	36.2	40.0
	20	1.90	4.42	7.36	10.6	13.9	17.4	21.0	24.6	28.2	31.9	35.6	39.3
	24	1.63	3.95	6.74	9.83	13.1	16.5	20.0	23.5	27.1	30.7	34.4	38.1
	28	1.43	3.57	6.21	9.16	12.3	15.6	19.0	22.5	26.1	29.7	33.3	36.9
	32	1.27	3.25	5.74	8.56	11.6	14.8	18.2	21.6	25.1	28.6	32.2	35.9
	36	1.14	2.98	5.33	8.02	11.0	14.1	17.3	20.7	24.1	27.6	31.2	34.8

Table 7-13
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 0°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

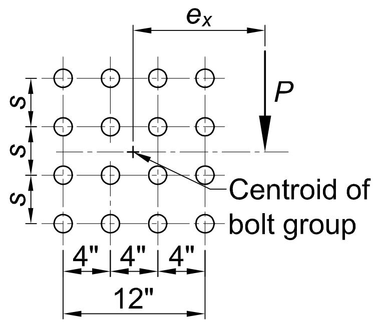
P = required force, P_u or P_a, kips

r_n = nominal strength per bolt, kips

e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
	2	2.82	5.98	9.46	13.3	17.3	21.3	25.5	29.6	33.7	37.7	41.8	45.8
	3	2.50	5.31	8.43	12.0	15.7	19.7	23.8	28.0	32.2	36.3	40.4	44.6
	4	2.23	4.74	7.58	10.8	14.3	18.2	22.2	26.3	30.4	34.6	38.8	43.0
	5	2.01	4.27	6.86	9.82	13.1	16.7	20.5	24.5	28.6	32.8	37.0	41.3
	6	1.81	3.86	6.24	8.96	12.0	15.4	19.0	22.9	26.9	31.0	35.2	39.4
	7	1.64	3.52	5.70	8.22	11.1	14.2	17.6	21.3	25.2	29.2	33.3	37.5
	8	1.49	3.22	5.24	7.57	10.2	13.2	16.4	19.9	23.6	27.5	31.5	35.6
	9	1.36	2.96	4.83	7.01	9.48	12.3	15.3	18.6	22.1	25.9	29.8	33.8
	10	1.25	2.73	4.47	6.51	8.83	11.4	14.3	17.5	20.8	24.4	28.2	32.1
3	12	1.07	2.37	3.89	5.68	7.74	10.1	12.6	15.5	18.5	21.8	25.3	29.0
	14	0.94	2.08	3.42	5.02	6.86	8.95	11.3	13.8	16.6	19.6	22.8	26.2
	16	0.83	1.86	3.05	4.49	6.15	8.04	10.2	12.5	15.0	17.8	20.7	23.9
	18	0.75	1.67	2.75	4.06	5.56	7.29	9.22	11.4	13.7	16.3	19.0	21.9
	20	0.68	1.52	2.50	3.70	5.07	6.65	8.43	10.4	12.6	14.9	17.5	20.2
	24	0.58	1.29	2.12	3.14	4.30	5.66	7.18	8.88	10.8	12.8	15.0	17.4
	28	0.50	1.12	1.84	2.72	3.73	4.92	6.24	7.73	9.37	11.2	13.1	15.2
	32	0.44	0.98	1.62	2.40	3.30	4.34	5.51	6.84	8.29	9.90	11.6	13.5
	36	0.40	0.88	1.45	2.15	2.95	3.89	4.94	6.13	7.43	8.88	10.4	12.1
	C′, in.	15.0	32.8	54.2	79.9	110	145	184	229	279	333	393	458
	2	2.82	6.54	10.6	14.8	18.9	22.9	26.9	30.9	34.9	38.9	42.8	46.8
	3	2.50	5.90	9.81	14.0	18.1	22.3	26.4	30.4	34.5	38.5	42.5	46.5
	4	2.23	5.33	9.01	13.1	17.3	21.5	25.7	29.8	33.9	37.9	42.0	46.0
	5	2.01	4.84	8.27	12.2	16.4	20.6	24.8	29.0	33.2	37.3	41.4	45.5
	6	1.81	4.42	7.60	11.4	15.5	19.7	24.0	28.2	32.4	36.6	40.7	44.8
	7	1.64	4.05	7.02	10.6	14.6	18.8	23.0	27.3	31.5	35.7	39.9	44.1
	8	1.49	3.73	6.51	9.94	13.7	17.8	22.0	26.3	30.6	34.8	39.1	43.3
	9	1.36	3.45	6.06	9.30	13.0	16.9	21.1	25.3	29.6	33.9	38.2	42.4
	10	1.25	3.20	5.66	8.72	12.2	16.1	20.2	24.4	28.6	32.9	37.2	41.5
6	12	1.07	2.80	4.98	7.73	10.9	14.5	18.4	22.5	26.7	30.9	35.2	39.5
	14	0.94	2.47	4.43	6.92	9.81	13.2	16.8	20.7	24.8	29.0	33.2	37.5
	16	0.83	2.21	3.98	6.25	8.90	12.0	15.4	19.1	23.0	27.1	31.3	35.5
	18	0.75	2.00	3.60	5.68	8.13	11.0	14.2	17.7	21.4	25.3	29.4	33.6
	20	0.68	1.82	3.29	5.21	7.47	10.1	13.1	16.4	20.0	23.7	27.7	31.7
	24	0.58	1.55	2.79	4.45	6.40	8.72	11.3	14.3	17.5	20.9	24.5	28.3
	28	0.50	1.34	2.42	3.87	5.59	7.64	9.96	12.6	15.5	18.6	21.9	25.5
	32	0.44	1.18	2.14	3.43	4.95	6.79	8.87	11.2	13.8	16.7	19.7	23.0
	36	0.40	1.06	1.92	3.07	4.44	6.10	7.98	10.1	12.5	15.1	17.9	20.9
	C′, in.	15.0	39.4	71.8	115	167	230	304	388	483	588	705	832

Table 7-13 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 15°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

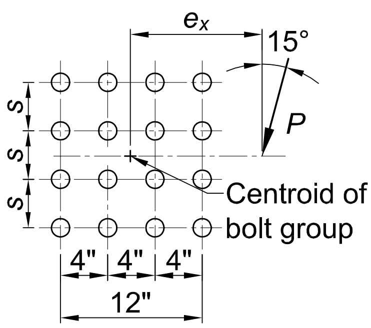
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	2.91	6.06	9.56	13.3	17.2	21.3	25.3	29.4	33.5	37.5	41.6	45.6
	3	2.57	5.40	8.57	12.0	15.8	19.7	23.7	27.8	31.9	36.1	40.2	44.3
	4	2.30	4.84	7.72	10.9	14.4	18.2	22.1	26.1	30.2	34.3	38.5	42.6
	5	2.06	4.37	6.99	9.93	13.2	16.7	20.5	24.4	28.5	32.6	36.7	40.9
	6	1.86	3.96	6.37	9.09	12.1	15.5	19.0	22.8	26.7	30.8	34.9	39.0
	7	1.69	3.61	5.83	8.36	11.2	14.3	17.7	21.3	25.1	29.0	33.1	37.2
	8	1.53	3.31	5.36	7.72	10.4	13.3	16.5	19.9	23.6	27.4	31.3	35.3
	9	1.40	3.04	4.95	7.15	9.64	12.4	15.4	18.7	22.2	25.8	29.7	33.6
	10	1.29	2.81	4.59	6.65	9.0	11.6	14.5	17.6	20.9	24.4	28.1	31.9
	12	1.11	2.44	4.00	5.82	7.9	10.2	12.8	15.6	18.7	21.9	25.3	28.9
	14	0.97	2.15	3.52	5.15	7.0	9.12	11.5	14.0	16.8	19.8	22.9	26.3
	16	0.86	1.92	3.15	4.61	6.3	8.21	10.3	12.7	15.2	18.0	20.9	24.0
	18	0.78	1.73	2.84	4.17	5.7	7.45	9.41	11.6	13.9	16.5	19.2	22.1
	20	0.71	1.57	2.59	3.80	5.2	6.81	8.61	10.6	12.8	15.2	17.7	20.4
	24	0.60	1.33	2.19	3.23	4.4	5.80	7.36	9.07	11.0	13.0	15.3	17.6
	28	0.52	1.15	1.90	2.80	3.9	5.05	6.41	7.91	9.59	11.4	13.4	15.5
	32	0.46	1.02	1.68	2.48	3.4	4.46	5.67	7.01	8.50	10.1	11.9	13.8
	36	0.41	0.91	1.50	2.22	3.0	4.00	5.08	6.29	7.63	9.09	10.7	12.4
6	2	2.91	6.57	10.6	14.7	18.8	22.8	26.8	30.8	34.8	38.8	42.7	46.7
	3	2.57	5.93	9.81	13.9	18.0	22.1	26.2	30.3	34.3	38.3	42.3	46.3
	4	2.30	5.37	9.04	13.0	17.2	21.3	25.5	29.6	33.6	37.7	41.7	45.8
	5	2.06	4.89	8.33	12.2	16.3	20.5	24.6	28.8	32.9	37.0	41.1	45.1
	6	1.86	4.48	7.70	11.4	15.4	19.5	23.7	27.9	32.1	36.2	40.3	44.4
	7	1.69	4.12	7.13	10.6	14.5	18.6	22.8	27.0	31.2	35.4	39.5	43.7
	8	1.53	3.80	6.62	9.95	13.7	17.7	21.8	26.0	30.2	34.4	38.6	42.8
	9	1.40	3.52	6.17	9.32	12.9	16.8	20.9	25.1	29.3	33.5	37.7	41.9
	10	1.29	3.27	5.77	8.76	12.2	16.0	20.0	24.1	28.3	32.5	36.8	41.0
	12	1.11	2.86	5.09	7.80	11.0	14.5	18.3	22.3	26.4	30.6	34.8	39.0
	14	0.97	2.54	4.53	7.00	9.92	13.2	16.8	20.6	24.6	28.7	32.8	37.1
	16	0.86	2.27	4.08	6.34	9.02	12.0	15.4	19.0	22.9	26.9	30.9	35.1
	18	0.78	2.06	3.70	5.78	8.26	11.1	14.2	17.7	21.3	25.2	29.1	33.2
	20	0.71	1.88	3.38	5.30	7.60	10.2	13.2	16.4	19.9	23.6	27.5	31.4
	24	0.60	1.59	2.88	4.54	6.54	8.84	11.5	14.4	17.5	20.9	24.5	28.2
	28	0.52	1.38	2.50	3.96	5.72	7.77	10.1	12.7	15.6	18.7	22.0	25.4
	32	0.46	1.22	2.21	3.51	5.08	6.92	9.03	11.4	14.0	16.8	19.9	23.1
	36	0.41	1.09	1.98	3.15	4.56	6.23	8.15	10.3	12.7	15.3	18.1	21.1

Table 7-13 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 30°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

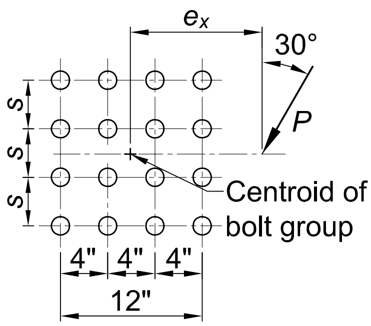
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	3.14	6.41	9.91	13.6	17.5	21.4	25.4	29.4	33.4	37.4	41.4	45.4
	3	2.79	5.75	8.95	12.4	16.1	20.0	23.9	27.9	31.9	35.9	40.0	44.0
	4	2.50	5.19	8.16	11.4	14.9	18.5	22.4	26.3	30.3	34.3	38.4	42.4
	5	2.25	4.71	7.45	10.5	13.7	17.2	20.9	24.7	28.6	32.6	36.7	40.7
	6	2.04	4.29	6.83	9.65	12.7	16.0	19.6	23.3	27.1	31.0	35.0	39.0
	7	1.85	3.93	6.28	8.92	11.8	15.0	18.3	21.9	25.6	29.4	33.3	37.3
	8	1.69	3.61	5.80	8.27	11.0	14.0	17.2	20.6	24.2	27.9	31.7	35.6
	9	1.55	3.33	5.38	7.70	10.3	13.1	16.2	19.4	22.9	26.5	30.2	34.0
	10	1.43	3.08	5.00	7.19	9.64	12.3	15.3	18.4	21.7	25.2	28.8	32.5
	12	1.23	2.68	4.37	6.32	8.52	11.0	13.6	16.5	19.6	22.8	26.2	29.8
	14	1.08	2.36	3.88	5.62	7.61	9.83	12.3	14.9	17.8	20.8	24.0	27.3
	16	0.96	2.11	3.47	5.05	6.86	8.89	11.1	13.6	16.2	19.0	22.0	25.2
	18	0.87	1.91	3.14	4.57	6.24	8.10	10.2	12.4	14.9	17.5	20.3	23.3
	20	0.79	1.74	2.86	4.18	5.71	7.43	9.35	11.5	13.8	16.2	18.9	21.6
	24	0.67	1.48	2.43	3.56	4.88	6.36	8.03	9.87	11.9	14.1	16.4	18.9
	28	0.58	1.28	2.11	3.10	4.25	5.55	7.02	8.65	10.4	12.4	14.5	16.7
	32	0.51	1.13	1.87	2.74	3.76	4.92	6.23	7.69	9.29	11.0	12.9	14.9
	36	0.46	1.01	1.67	2.45	3.37	4.41	5.60	6.91	8.36	9.95	11.7	13.5
6	2	3.14	6.75	10.7	14.7	18.7	22.7	26.7	30.7	34.7	38.6	42.6	46.6
	3	2.79	6.12	9.94	13.9	18.0	22.0	26.1	30.1	34.1	38.1	42.1	46.1
	4	2.50	5.58	9.23	13.1	17.2	21.2	25.3	29.4	33.4	37.5	41.5	45.5
	5	2.25	5.13	8.58	12.4	16.3	20.4	24.5	28.6	32.7	36.7	40.8	44.8
	6	2.04	4.73	8.00	11.6	15.5	19.5	23.6	27.7	31.8	35.9	40.0	44.1
	7	1.85	4.38	7.47	10.9	14.7	18.7	22.7	26.8	31.0	35.1	39.2	43.3
	8	1.69	4.06	6.98	10.3	14.0	17.9	21.9	25.9	30.1	34.2	38.3	42.4
	9	1.55	3.78	6.55	9.72	13.3	17.1	21.0	25.1	29.2	33.3	37.4	41.5
	10	1.43	3.53	6.15	9.18	12.6	16.3	20.2	24.2	28.3	32.4	36.5	40.6
	12	1.23	3.10	5.47	8.25	11.4	14.9	18.6	22.5	26.5	30.6	34.7	38.8
	14	1.08	2.76	4.90	7.46	10.4	13.7	17.2	21.0	24.9	28.8	32.9	37.0
	16	0.96	2.48	4.43	6.79	9.55	12.6	16.0	19.6	23.3	27.2	31.2	35.2
	18	0.87	2.25	4.04	6.22	8.79	11.7	14.9	18.3	21.9	25.7	29.5	33.5
	20	0.79	2.06	3.70	5.72	8.14	10.9	13.9	17.1	20.6	24.2	28.0	31.9
	24	0.67	1.76	3.17	4.93	7.06	9.48	12.2	15.2	18.3	21.7	25.3	28.9
	28	0.58	1.53	2.76	4.32	6.22	8.38	10.8	13.5	16.5	19.6	22.9	26.3
	32	0.51	1.35	2.45	3.84	5.54	7.50	9.73	12.2	14.9	17.8	20.9	24.1
	36	0.46	1.21	2.19	3.46	5.00	6.77	8.82	11.1	13.6	16.3	19.1	22.2

Table 7-13 (continued)
Coefficients C for Eccentrically Loaded Bolt Groups
Angle = 45°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

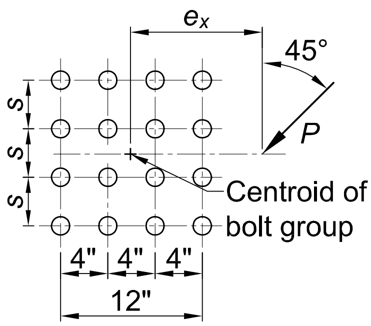
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	3.46	6.96	10.5	14.2	18.0	21.8	25.7	29.6	33.5	37.4	41.4	45.3
	3	3.15	6.38	9.73	13.2	16.8	20.6	24.4	28.2	32.1	36.1	40.0	44.0
	4	2.87	5.84	8.97	12.3	15.7	19.3	23.1	26.9	30.7	34.6	38.6	42.5
	5	2.61	5.36	8.30	11.4	14.7	18.2	21.8	25.5	29.3	33.2	37.1	41.0
	6	2.39	4.93	7.69	10.7	13.9	17.2	20.7	24.3	28.0	31.8	35.6	39.5
	7	2.19	4.55	7.15	9.98	13.0	16.2	19.6	23.1	26.7	30.4	34.2	38.1
	8	2.01	4.21	6.66	9.34	12.2	15.3	18.6	22.0	25.5	29.2	32.9	36.7
	9	1.86	3.90	6.21	8.76	11.5	14.5	17.7	21.0	24.4	27.9	31.6	35.3
	10	1.72	3.63	5.82	8.24	10.9	13.8	16.8	20.0	23.3	26.8	30.4	34.0
	12	1.49	3.18	5.14	7.33	9.76	12.4	15.2	18.3	21.4	24.7	28.1	31.6
	14	1.32	2.82	4.59	6.58	8.81	11.3	13.9	16.7	19.7	22.8	26.1	29.5
	16	1.17	2.53	4.14	5.95	8.00	10.3	12.7	15.4	18.2	21.2	24.3	27.5
	18	1.06	2.29	3.76	5.43	7.32	9.44	11.7	14.2	16.9	19.7	22.7	25.7
	20	0.96	2.10	3.44	4.98	6.74	8.71	10.9	13.2	15.7	18.4	21.2	24.2
	24	0.82	1.79	2.94	4.26	5.81	7.53	9.43	11.5	13.8	16.2	18.7	21.4
	28	0.71	1.56	2.56	3.73	5.09	6.61	8.31	10.2	12.2	14.4	16.7	19.2
	32	0.63	1.38	2.26	3.31	4.52	5.89	7.42	9.11	11.0	12.9	15.1	17.3
	36	0.56	1.23	2.03	2.97	4.06	5.30	6.69	8.23	9.91	11.7	13.7	15.8
6	2	3.46	7.09	10.9	14.8	18.7	22.7	26.7	30.6	34.6	38.5	42.5	46.5
	3	3.15	6.58	10.3	14.1	18.1	22.0	26.0	30.0	33.9	37.9	41.9	45.9
	4	2.87	6.09	9.65	13.4	17.3	21.3	25.3	29.3	33.3	37.3	41.2	45.2
	5	2.61	5.66	9.07	12.8	16.6	20.6	24.5	28.5	32.5	36.5	40.5	44.5
	6	2.39	5.26	8.54	12.1	15.9	19.8	23.8	27.8	31.8	35.8	39.8	43.8
	7	2.19	4.91	8.07	11.6	15.3	19.1	23.0	27.0	31.0	35.0	39.0	43.0
	8	2.01	4.59	7.63	11.0	14.6	18.4	22.3	26.2	30.2	34.2	38.2	42.2
	9	1.86	4.30	7.23	10.5	14.0	17.7	21.5	25.5	29.4	33.4	37.4	41.4
	10	1.72	4.04	6.85	10.0	13.4	17.1	20.8	24.7	28.6	32.6	36.6	40.6
	12	1.49	3.59	6.19	9.14	12.4	15.9	19.5	23.3	27.2	31.1	35.1	39.1
	14	1.32	3.22	5.62	8.38	11.4	14.8	18.3	22.0	25.8	29.6	33.5	37.5
	16	1.17	2.91	5.13	7.71	10.6	13.8	17.2	20.8	24.4	28.2	32.1	36.0
	18	1.06	2.66	4.71	7.12	9.87	12.9	16.2	19.6	23.2	26.9	30.7	34.6
	20	0.96	2.44	4.35	6.61	9.22	12.1	15.3	18.6	22.1	25.7	29.4	33.2
	24	0.82	2.10	3.76	5.76	8.11	10.8	13.7	16.7	20.0	23.4	27.0	30.6
	28	0.71	1.83	3.30	5.08	7.22	9.64	12.3	15.2	18.3	21.5	24.9	28.4
	32	0.63	1.63	2.94	4.54	6.50	8.71	11.2	13.9	16.7	19.8	23.0	26.3
	36	0.56	1.46	2.64	4.11	5.90	7.93	10.2	12.7	15.4	18.3	21.3	24.5

Table 7-13 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 60°

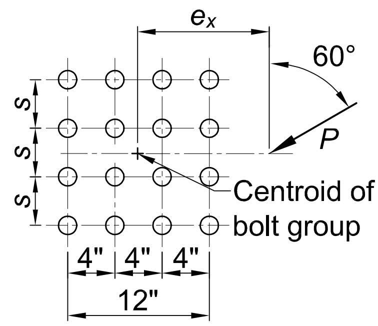
Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where
- P = required force, P_u or P_a, kips
- r_n = nominal strength per bolt, kips
- e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
- s = bolt spacing, in.
- C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	3.74	7.46	11.2	14.9	18.6	22.4	26.2	30.0	33.9	37.7	41.6	45.5
	3	3.57	7.12	10.7	14.3	17.9	21.6	25.3	29.0	32.8	36.7	40.5	44.4
	4	3.38	6.75	10.2	13.6	17.1	20.7	24.3	28.0	31.8	35.6	39.4	43.2
	5	3.17	6.36	9.61	12.9	16.4	19.8	23.4	27.0	30.7	34.5	38.2	42.0
	6	2.97	5.99	9.09	12.3	15.6	19.0	22.5	26.1	29.7	33.4	37.1	40.9
	7	2.78	5.63	8.59	11.7	14.9	18.2	21.6	25.1	28.7	32.3	36.0	39.8
	8	2.60	5.29	8.13	11.1	14.2	17.5	20.8	24.3	27.8	31.4	35.0	38.7
	9	2.44	4.98	7.69	10.6	13.6	16.8	20.1	23.4	26.9	30.4	34.0	37.7
	10	2.28	4.69	7.28	10.1	13.0	16.1	19.3	22.7	26.1	29.5	33.1	36.7
	12	2.02	4.18	6.56	9.16	11.9	14.9	18.0	21.2	24.5	27.8	31.3	34.8
	14	1.80	3.76	5.95	8.38	11.0	13.8	16.7	19.8	23.0	26.3	29.6	33.1
	16	1.62	3.40	5.43	7.70	10.2	12.8	15.6	18.6	21.6	24.8	28.1	31.4
	18	1.47	3.10	4.99	7.11	9.42	11.9	14.6	17.4	20.4	23.5	26.7	29.9
	20	1.34	2.85	4.61	6.59	8.76	11.1	13.7	16.4	19.3	22.2	25.3	28.5
	24	1.15	2.45	3.99	5.73	7.67	9.82	12.2	14.6	17.3	20.1	23.0	26.0
	28	1.00	2.15	3.51	5.06	6.80	8.76	10.9	13.2	15.6	18.2	20.9	23.8
	32	0.88	1.91	3.13	4.52	6.11	7.89	9.83	11.9	14.2	16.6	19.2	21.8
	36	0.79	1.72	2.81	4.08	5.53	7.16	8.95	10.9	13.0	15.3	17.7	20.2
6	2	3.74	7.47	11.2	15.0	18.9	22.8	26.7	30.6	34.5	38.5	42.4	46.4
	3	3.57	7.16	10.8	14.6	18.4	22.2	26.1	30.0	33.9	37.9	41.8	45.8
	4	3.38	6.82	10.4	14.1	17.8	21.7	25.5	29.4	33.3	37.3	41.2	45.1
	5	3.17	6.47	9.94	13.6	17.3	21.1	24.9	28.8	32.7	36.6	40.5	44.5
	6	2.97	6.14	9.52	13.1	16.7	20.5	24.3	28.2	32.1	36.0	39.9	43.8
	7	2.78	5.82	9.11	12.6	16.2	19.9	23.7	27.6	31.5	35.3	39.3	43.2
	8	2.60	5.52	8.73	12.1	15.7	19.4	23.2	27.0	30.8	34.7	38.6	42.5
	9	2.44	5.24	8.37	11.7	15.2	18.9	22.6	26.4	30.2	34.1	38.0	41.9
	10	2.28	4.98	8.03	11.3	14.8	18.4	22.1	25.8	29.7	33.5	37.4	41.3
	12	2.02	4.51	7.41	10.6	14.0	17.5	21.1	24.8	28.5	32.3	36.2	40.1
	14	1.80	4.10	6.86	9.91	13.2	16.6	20.1	23.8	27.5	31.2	35.0	38.9
	16	1.62	3.76	6.37	9.29	12.4	15.8	19.2	22.8	26.5	30.2	33.9	37.7
	18	1.47	3.46	5.94	8.74	11.8	15.0	18.4	21.9	25.5	29.2	32.9	36.6
	20	1.34	3.21	5.56	8.23	11.2	14.3	17.6	21.0	24.6	28.2	31.9	35.6
	24	1.15	2.79	4.91	7.34	10.1	13.0	16.2	19.5	22.9	26.4	30.0	33.6
	28	1.00	2.47	4.38	6.61	9.13	11.9	14.9	18.1	21.4	24.7	28.2	31.8
	32	0.88	2.21	3.95	5.99	8.33	11.0	13.8	16.8	20.0	23.2	26.6	30.1
	36	0.79	2.00	3.58	5.46	7.65	10.1	12.8	15.7	18.7	21.9	25.1	28.5

Table 7-13 (continued)
Coefficients C for Eccentrically Loaded Bolt Groups
Angle = 75°

Available strength of a bolt group, ϕR_n or R_n/Ω, is determined with

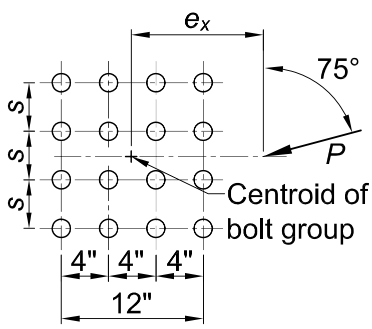
$$R_n = Cr_n$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

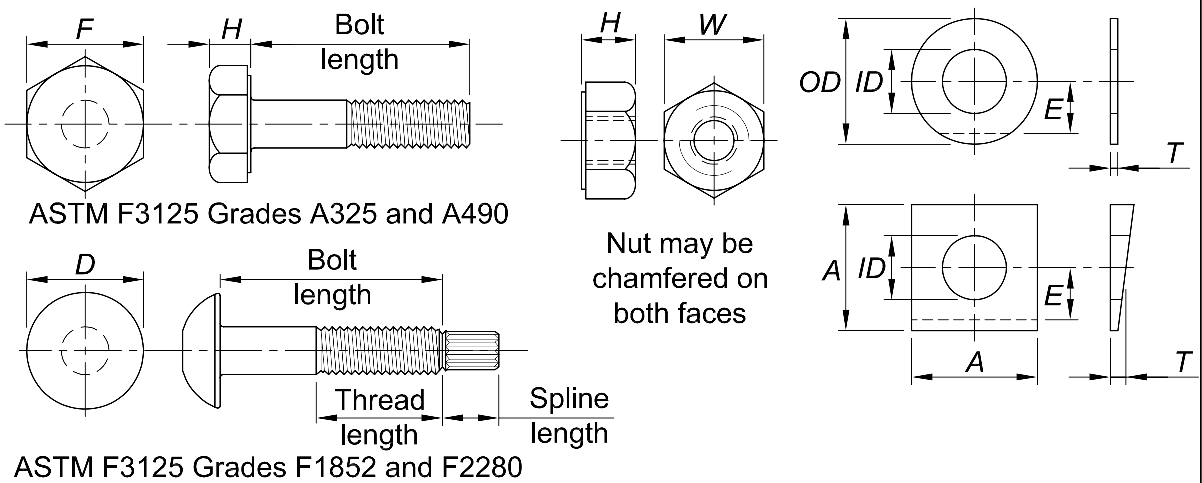
where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e_x = horizontal distance from the centroid of the bolt group to the line of action of P, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	3.89	7.75	11.6	15.5	19.3	23.1	26.9	30.8	34.6	38.5	42.3	46.2
	3	3.84	7.66	11.5	15.2	19.0	22.7	26.5	30.3	34.1	37.9	41.7	45.5
	4	3.79	7.54	11.3	15.0	18.7	22.4	26.1	29.8	33.5	37.3	41.0	44.8
	5	3.72	7.40	11.1	14.7	18.3	21.9	25.6	29.3	32.9	36.7	40.4	44.1
	6	3.65	7.25	10.8	14.4	17.9	21.5	25.1	28.7	32.4	36.1	39.8	43.5
	7	3.56	7.08	10.6	14.1	17.6	21.1	24.6	28.2	31.8	35.5	39.1	42.8
	8	3.47	6.90	10.3	13.7	17.2	20.6	24.1	27.7	31.3	34.9	38.5	42.2
	9	3.37	6.71	10.0	13.4	16.8	20.2	23.7	27.2	30.7	34.3	37.9	41.6
	10	3.27	6.52	9.77	13.1	16.4	19.8	23.2	26.7	30.2	33.7	37.3	41.0
	12	3.07	6.14	9.23	12.4	15.6	18.9	22.3	25.7	29.1	32.6	36.2	39.8
	14	2.87	5.76	8.71	11.8	14.9	18.1	21.4	24.7	28.1	31.6	35.1	38.7
	16	2.68	5.40	8.22	11.1	14.2	17.3	20.5	23.8	27.2	30.6	34.1	37.6
	18	2.50	5.07	7.76	10.6	13.5	16.6	19.7	23.0	26.3	29.7	33.1	36.6
	20	2.34	4.76	7.33	10.0	12.9	15.9	19.0	22.2	25.5	28.8	32.2	35.6
	24	2.06	4.23	6.57	9.10	11.8	14.7	17.6	20.7	23.9	27.1	30.4	33.8
	28	1.82	3.78	5.94	8.30	10.9	13.5	16.4	19.3	22.4	25.5	28.7	32.0
	32	1.63	3.41	5.41	7.61	10.0	12.6	15.3	18.1	21.0	24.1	27.2	30.4
	36	1.48	3.11	4.95	7.01	9.26	11.7	14.3	17.0	19.8	22.8	25.8	28.9
6	2	3.89	7.74	11.6	15.4	19.3	23.1	27.0	30.9	35.2	39.1	43.0	47.0
	3	3.84	7.64	11.4	15.2	19.0	22.8	26.6	30.5	34.4	38.3	42.2	46.1
	4	3.79	7.52	11.2	14.9	18.7	22.5	26.3	30.1	34.0	37.8	41.7	45.6
	5	3.72	7.38	11.0	14.7	18.4	22.1	25.9	29.7	33.6	37.4	41.3	45.2
	6	3.65	7.23	10.8	14.4	18.1	21.8	25.6	29.3	33.2	37.0	40.8	44.7
	7	3.56	7.07	10.6	14.2	17.8	21.5	25.2	29.0	32.8	36.6	40.4	44.3
	8	3.47	6.90	10.4	13.9	17.5	21.2	24.9	28.6	32.4	36.2	40.0	43.9
	9	3.37	6.73	10.1	13.6	17.2	20.8	24.5	28.3	32.0	35.8	39.6	43.5
	10	3.27	6.56	9.92	13.4	16.9	20.5	24.2	27.9	31.7	35.5	39.3	43.1
	12	3.07	6.21	9.48	12.9	16.4	19.9	23.6	27.3	31.0	34.7	38.5	42.3
	14	2.87	5.88	9.07	12.4	15.9	19.4	23.0	26.6	30.3	34.1	37.8	41.6
	16	2.68	5.57	8.67	11.9	15.4	18.8	22.4	26.0	29.7	33.4	37.1	40.9
	18	2.50	5.27	8.29	11.5	14.9	18.3	21.9	25.5	29.1	32.8	36.5	40.2
	20	2.34	4.99	7.94	11.1	14.4	17.8	21.3	24.9	28.5	32.2	35.8	39.6
	24	2.06	4.50	7.29	10.3	13.6	16.9	20.4	23.9	27.4	31.0	34.7	38.3
	28	1.82	4.08	6.73	9.67	12.8	16.1	19.4	22.9	26.4	30.0	33.6	37.2
	32	1.63	3.73	6.25	9.06	12.1	15.3	18.6	22.0	25.4	29.0	32.5	36.1
	36	1.48	3.43	5.82	8.51	11.4	14.5	17.8	21.1	24.5	28.0	31.5	35.1

Table 7-14
Dimensions of High-Strength Fasteners, in.



Measurement		Nominal Bolt Diameter, in								
		1/2	5/8	3/4	7/8	1	1 1/8	1 1/4	1 3/8	1 1/2
A325, F1852, A490, F2280 Bolts[a]	Width Across Flats, *F*	7/8	1 1/16	1 1/4	1 7/16	1 5/8	1 13/16	2	2 3/16	2 3/8
	Head Diameter, *D*[e]	1 1/8	1 5/16	1 9/16	1 7/8	2 3/16	2 3/8	–	–	–
	Height, *H*	5/16	25/64	15/32	35/64	39/64	11/16	25/32	27/32	15/16
	Thread Length	1	1 1/4	1 3/8	1 1/2	1 3/4	2	2	2 1/4	2 1/4
	Spline Length[e]	1/2	19/32	21/32	23/32	13/16	13/16	–	–	–
	Bolt Length = Grip + Washer Thickness + →	11/16	7/8	1	1 1/8	1 1/4	1 1/2	1 5/8	1 3/4	1 7/8
A563 Nuts[a]	Width Across Flats, *W*	7/8	1 1/16	1 1/4	1 7/16	1 5/8	1 13/16	2	2 3/16	2 3/8
	Height, *H*	31/64	39/64	47/64	55/64	63/64	1 7/64	1 7/32	1 11/32	1 15/32
F436 Circular Washers[b]	Nom. Outside Diameter, *OD*	1 1/16	1 5/16	1 15/32	1 3/4	2	2 1/4	2 1/2	2 3/4	3
	Nom. Inside Diameter, *ID*	17/32	11/16	13/16	15/16	1 1/8	1 1/4	1 3/8	1 1/2	1 5/8
	Thckns., *T* Min.	0.097	0.122	0.122	0.136	0.136	0.136	0.136	0.136	0.136
	Thckns., *T* Max.	0.177	0.177	0.177	0.177	0.177	0.177	0.177	0.177	0.177
	Min. Edge Distance, *E*[c]	7/16	9/16	21/32	25/32	7/8	1	1 3/32	1 7/32	1 5/16
F436 Square or Rect. Washers[b,d,e,f]	Min. Side Dimension, *A*	1 3/4	1 3/4	1 3/4	1 3/4	1 3/4	2 1/4	2 1/4	2 1/4	2 1/4
	Min. Edge Distance, *E*[c]	7/16	9/16	21/32	25/32	7/8	1	1 3/32	1 7/32	1 5/16

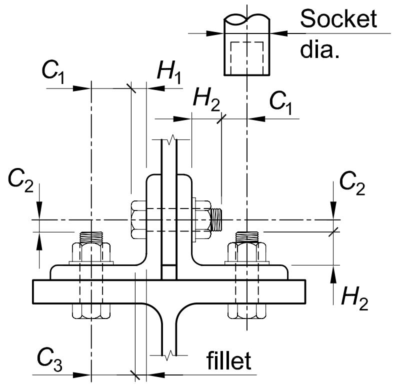
[a] Tolerances as specified in ASME B18.2.6
[b] ASTM F436 washer tolerances, in.:

Nominal outside diameter	–1/32; +1/32
Nominal diameter of hole	–0; +1/32
Flatness: max. deviation from straight-edge placed on cut side shall not exceed	0.010
Concentricity: center of hole to outside diameter (full indicator runout)	0.030
Burr shall not project above immediately adjacent washer surface more than	0.010

[c] For clipped washers only
[d] For use with American standard beams (S) and channels (C)
[e] For Grades F1852 and F2280 only
[f] For beveled washers mean thickness, $T = 5/16$ in.; taper in thickness = 2:12.

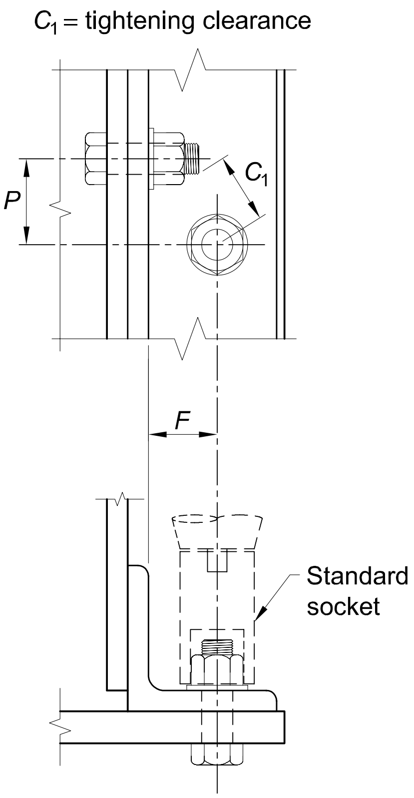
Table 7-15
Entering and Tightening Clearance, in.
ASTM F3125 Heavy Hex Bolts (A325 and A490)

Aligned Bolts



Nominal Bolt Dia.	Socket Dia.	H_1	H_2	C_1	C_2	C_3 Circular	C_3 Clipped
5/8	2 1/8	25/64	1 1/4	1 3/16	11/16	11/16	5/8
3/4	2 1/8	15/32	1 3/8	1 3/16	3/4	3/4	11/16
7/8	2 1/4	35/64	1 1/2	1 1/4	7/8	7/8	13/16
1	2 1/2	39/64	1 3/4	1 3/8	15/16	1	7/8
1 1/8	2 3/4	11/16	2	1 1/2	1 1/16	1 1/8	1
1 1/4	3 3/8	25/32	2	1 13/16	1 1/8	1 1/4	1 1/8
1 3/8	3 1/2	27/32	2 1/4	1 7/8	1 1/4	1 3/8	1 1/4
1 1/2	3 3/4	15/16	2 1/4	2	1 5/16	1 1/2	1 3/8

Staggered Bolts

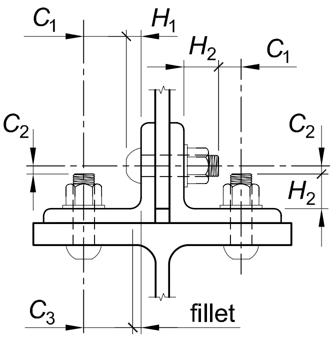


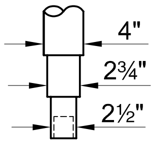
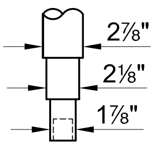
F	Stagger P, in. — Nominal Bolt Diameter, in. 5/8	3/4	7/8	1	1 1/8	1 1/4	1 3/8	1 1/2
1 1/4	1 5/8	1 13/16	2	–	–	–	–	–
1 3/8	1 1/2	1 3/4	1 15/16	2 1/4	–	–	–	–
1 1/2	1 1/2	1 9/16	1 7/8	2 3/16	2 1/2	–	–	–
1 5/8	1 7/16	1 9/16	1 11/16	2 1/8	2 7/16	–	–	–
1 3/4	1 3/8	1 1/2	1 11/16	2 1/16	2 3/8	–	–	–
1 7/8	1 5/16	1 7/16	1 5/8	1 7/8	2 5/16	2 7/8	3 1/16	–
2	1 1/4	1 3/8	1 9/16	1 7/8	2 1/4	2 13/16	3	3 5/16
2 1/8	1 1/8	1 5/16	1 1/2	1 13/16	2 1/16	2 7/16	2 15/16	3 1/4
2 1/4	15/16	1 3/16	1 7/16	1 3/4	2 1/16	2 7/16	2 7/8	3 3/16
2 3/8	11/16	1	1 5/16	1 3/4	2	2 3/8	2 9/16	2 3/4
2 1/2	–	3/4	1 3/16	1 5/8	2	2 3/8	2 9/16	2 3/4
2 5/8	–	–	1	1 9/16	1 15/16	2 5/16	2 1/2	2 11/16
2 3/4	–	–	7/16	1 7/16	1 7/8	2 1/4	2 1/2	2 11/16
2 7/8	–	–	–	1 5/16	1 3/4	2 3/16	2 7/16	2 5/8
3	–	–	–	1 1/16	1 11/16	2 1/8	2 3/8	2 5/8
3 1/8	–	–	–	1/2	1 9/16	2 1/16	2 3/8	2 9/16
3 1/4	–	–	–	–	1 3/8	1 15/16	2 1/4	2 1/2
3 3/8	–	–	–	–	1 3/16	1 13/16	2 3/16	2 3/8
3 1/2	–	–	–	–	9/16	1 5/8	2 1/16	2 5/16
3 5/8	–	–	–	–	–	1 7/16	1 15/16	2 3/16
3 3/4	–	–	–	–	–	1 1/8	1 13/16	2 1/16
3 7/8	–	–	–	–	–	–	1 5/8	1 15/16
4	–	–	–	–	–	–	1 3/8	1 11/16
4 1/8	–	–	–	–	–	–	11/16	1 7/16
4 1/4	–	–	–	–	–	–	–	3/4

Notes:
H_1 = height of head
H_2 = maximum shank extension*
C_1 = clearance for tightening
C_2 = clearance for entering
C_3 = clearance for fillet*
P = bolt stagger
F = clearance for tightening staggered bolts
* Based on the use of one ASTM F436 washer

Table 7-16
Entering and Tightening Clearance, in. ASTM F3125 Tension Control Bolts (F1852 and F2280)

Aligned Bolts

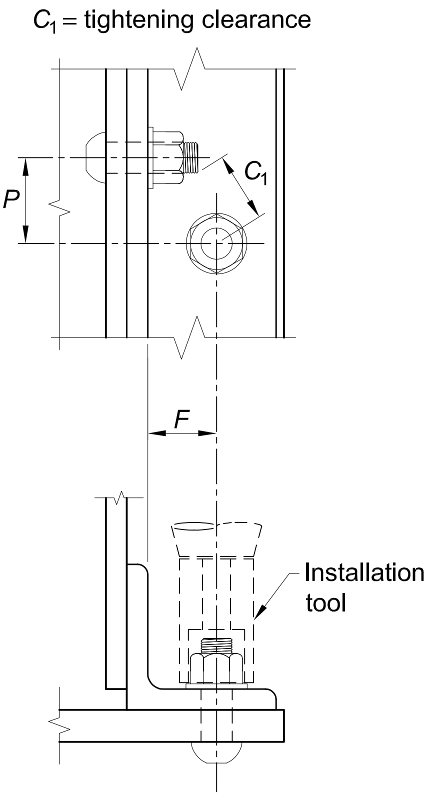


Tools	Nominal Bolt Dia.	H_1	H_2	C_1	C_2	C_3 Round	C_3 Clipped
Large Tools (S-110EZ) 	**4-in.-Diameter Critical**						
	7/8	35/64	1 1/2	2 1/8	1 1/8	7/8	–
	1	39/64	1 3/4	2 1/8	1 1/4	1	–
	1 1/8	11/16	2	2 1/8	1 5/16	1 1/8	–
	2 1/2-in.-Diameter Critical						
	7/8	35/64	1 1/2	1 3/8	1 1/8	7/8	–
	1	39/64	1 3/4	1 3/8	1 1/4	1	–
	1 1/8	11/16	2	1 3/8	1 5/16	1 1/8	–
Small Tools (S-60EZA) 	**2 7/8-in.-Diameter Critical**						
	5/8	25/64	1 1/4	1 9/16	13/16	11/16	–
	3/4	15/32	1 3/8	1 9/16	15/16	3/4	–
	7/8	35/64	1 1/2	1 9/16	1 1/8	7/8	–
	1 7/8-in.-Diameter Critical						
	5/8	25/64	1 1/4	1 1/16	13/16	11/16	–
	3/4	15/32	1 3/8	1 1/16	15/16	3/4	–
	7/8	35/64	1 1/2	1 1/16	1 1/8	7/8	–

Notes:
H_1 = height of head
H_2 = maximum shank extension*
C_1 = clearance for tightening
C_2 = clearance for entering
C_3 = clearance for fillet*
* Based on one standard hardened washer

Table 7-16 (continued)
Entering and Tightening Clearance, in. ASTM F3125 Tension Control Bolts (F1852 and F2280)

Staggered Bolts

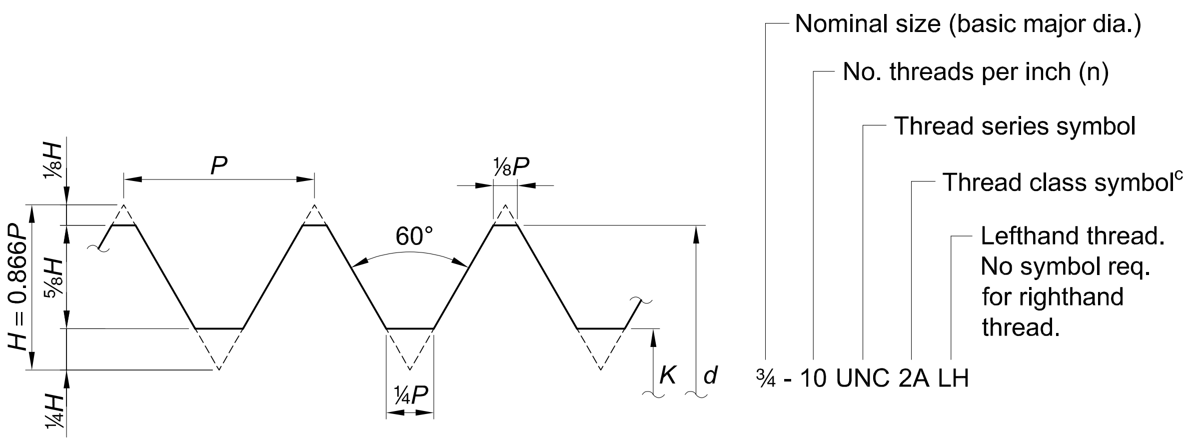


	Stagger P, in.				
	Nominal Bolt Diameter, in.				
F	**5/8**	**3/4**	**7/8**	**1**	**1 1/8**
1 5/8	1 13/16	1 15/16	2	–	–
1 3/4	1 13/16	1 7/8	2	–	–
1 7/8	1 3/4	1 7/8	1 15/16	–	–
2	1 11/16	1 13/16	1 15/16	–	–
2 1/8	1 5/8	1 3/4	1 7/8	2 9/16	2 11/16
2 1/4	1 1/2	1 11/16	1 13/16	2 9/16	2 11/16
2 3/8	1 3/8	1 9/16	1 3/4	2 1/2	2 5/8
2 1/2	1 1/4	1 7/16	1 5/8	2 1/2	2 5/8
2 5/8	1 1/16	1 5/16	1 1/2	2 7/16	2 9/16
2 3/4	3/4	1 1/8	1 3/8	2 3/8	2 9/16
2 7/8	–	13/16	1 3/16	2 5/16	2 1/2
3	–	–	7/8	2 3/16	2 7/16
3 3/8	–	–	–	1 7/8	2 3/16
3 1/2	–	–	–	1 11/16	2 1/16
3 5/8	–	–	–	1 1/2	1 15/16
3 3/4	–	–	–	1 3/16	1 3/4
3 7/8	–	–	–	1/2	1 9/16
4	–	–	–	–	1 1/4
4 1/8	–	–	–	–	9/16

Notes:
P = bolt stagger
F = clearance for tightening staggered bolts

Table 7-17
Threading Dimensions for High-Strength and Non-High-Strength Bolts

Screw Threads
Unified Standard Series-UNC/UNRC and 4UN/4UNR
ANSI/ASME B1.1



Thread Dimensions

Standard Designations

Diameter		Area			Threads per inch, n[b]
Bolt Diameter, d, in.	Min. Root, K, in.	Gross Bolt Area, in.^{2}	Min. Root Area, in.^{2}	Net Tensile Area[a], in.^{2}	
1/4	0.196	0.0490	0.0301	0.0320	20
3/8	0.307	0.110	0.0742	0.0780	16
1/2	0.417	0.196	0.136	0.142	13
5/8	0.527	0.307	0.218	0.226	11
3/4	0.642	0.442	0.323	0.334	10
7/8	0.755	0.601	0.447	0.462	9
1	0.865	0.785	0.587	0.606	8
1 1/8	0.970	0.994	0.740	0.763	7
1 1/4	1.10	1.23	0.942	0.969	7
1 3/8	1.19	1.49	1.12	1.16	6
1 1/2	1.32	1.77	1.37	1.41	6
1 3/4	1.53	2.41	1.85	1.90	5
2	1.76	3.14	2.43	2.50	4.5
2 1/4	2.01	3.98	3.17	3.25	4.5
2 1/2	2.23	4.91	3.90	4.00	4
2 3/4	2.48	5.94	4.83	4.93	4
3	2.73	7.07	5.85	5.97	4
3 1/4	2.98	8.30	6.97	7.10	4
3 1/2	3.23	9.62	8.19	8.33	4
3 3/4	3.48	11.0	9.51	9.66	4
4	3.73	12.6	10.9	11.1	4

[a] Net tensile area $= \frac{\pi}{4}\left(d - \frac{0.9743}{n}\right)^2$

[b] For diameters listed, thread series is UNC (coarse). For larger diameters, thread series is 4UN.

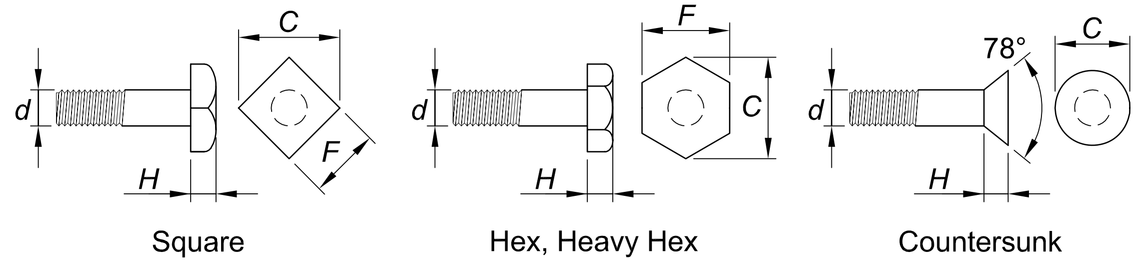
[c] 2A denotes Class 2A fit applicable to external threads; 2B denotes corresponding Class 2B fit for internal threads.

Table 7-18
Weights of High-Strength Fasteners, pounds per 100 count

	Bolt Length, in.	Nominal Bolt Diameter, in. 1/2	5/8	3/4	7/8	1	1 1/8	1 1/4	1 3/8	1 1/2
100 Conventional ASTM F3125 Gr. A325 or A490 Bolts with A563 Nuts	**1**	16.5	29.4	47.0	–	–	–	–	–	–
	1 1/4	17.8	31.1	49.6	74.4	104	–	–	–	–
	1 1/2	19.2	33.1	52.2	78.0	109	148	197	–	–
	1 3/4	20.5	35.3	55.3	81.9	114	154	205	261	333
	2	21.9	37.4	58.4	86.1	119	160	212	270	344
	2 1/4	23.3	39.8	61.6	90.3	124	167	220	279	355
	2 1/2	24.7	41.7	64.7	94.6	130	174	229	290	366
	2 3/4	26.1	43.9	67.8	98.8	135	181	237	300	379
	3	27.4	46.1	70.9	103	141	188	246	310	391
	3 1/4	28.8	48.2	74.0	107	146	195	255	321	403
	3 1/2	30.2	50.4	77.1	111	151	202	263	332	416
	3 3/4	31.6	52.5	80.2	116	157	209	272	342	428
	4	33.0	54.7	83.3	120	162	216	280	353	441
	4 1/4	34.3	56.9	86.4	124	168	223	289	363	453
	4 1/2	35.7	59.0	89.5	128	173	230	298	374	465
	4 3/4	37.1	61.2	92.7	133	179	237	306	384	478
	5	38.5	63.3	95.8	137	184	244	315	395	490
	5 1/4	39.9	65.5	98.9	141	190	251	324	405	503
	5 1/2	41.2	67.7	102	146	196	258	332	416	515
	5 3/4	42.6	69.8	105	150	201	265	341	426	527
	6	44.0	71.9	108	154	207	272	349	437	540
	6 1/4	–	74.1	111	158	212	279	358	447	552
	6 1/2	–	76.3	114	163	218	286	367	458	565
	6 3/4	–	78.5	118	167	223	293	375	468	577
	7	–	80.6	121	171	229	300	384	479	589
	7 1/4	–	82.8	124	175	234	307	392	489	602
	7 1/2	–	84.9	127	179	240	314	401	500	614
	7 3/4	–	87.1	130	183	246	321	410	510	626
	8	–	89.2	133	187	251	328	418	521	639
	8 1/4	–	–	–	192	257	335	427	531	651
	8 1/2	–	–	–	196	262	342	435	542	664
	8 3/4	–	–	–	–	–	–	444	552	676
	9	–	–	–	–	–	–	453	563	689
	Per inch add'tl. Add	5.50	8.60	12.4	16.9	22.1	28.0	34.4	42.5	49.7
100, F436 Circular Washers		2.10	3.60	4.80	7.00	9.40	11.3	13.8	16.8	20.0
100, F436 Square Washers		23.1	22.4	21.0	20.2	19.2	34.0	31.6	31.2	32.9

This table conforms to weight standards adopted by the Industrial Fasteners Institute (IFI), updated for washer weights.

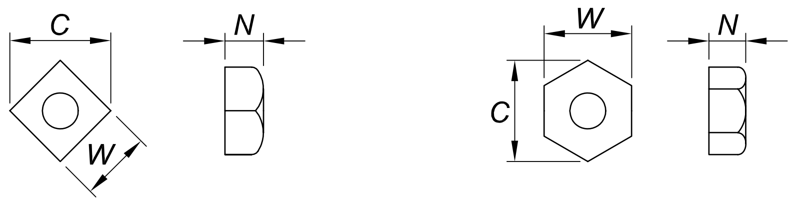
Table 7-19
Dimensions of Non-High-Strength Fasteners, in.



Bolt Dia., *d*, in.	Square			Hex			Heavy Hex			Countersunk		Min. Thrd. Length, in.	
	***F*, in.**	***C*, in.**	***H*, in.**	***F*, in.**	***C*, in.**	***H*, in.**	***F*, in.**	***C*, in.**	***H*, in.**	***C*, in.**	***H*, in.**	***L* ≤ 6 in.**	***L* > 6 in.**
1/4	3/8	1/2	3/16	7/16	1/2	3/16	–	–	–	1/2	1/8	3/4	1
3/8	9/16	13/16	1/4	9/16	5/8	1/4	–	–	–	11/16	3/16	1	1 1/4
1/2	3/4	1 1/16	5/16	3/4	7/8	3/8	7/8	1	3/8	7/8	1/4	1 1/4	1 1/2
5/8	15/16	1 5/16	7/16	15/16	1 1/16	7/16	1 1/16	1 1/4	7/16	1 1/8	5/16	1 1/2	1 3/4
3/4	1 1/8	1 9/16	1/2	1 1/8	1 5/16	1/2	1 1/4	1 7/16	1/2	1 3/8	3/8	1 3/4	2
7/8	1 5/16	1 7/8	5/8	1 5/16	1 1/2	9/16	1 7/16	1 11/16	9/16	1 9/16	7/16	2	2 1/4
1	1 1/2	2 1/8	11/16	1 1/2	1 3/4	11/16	1 5/8	1 7/8	11/16	1 13/16	1/2	2 1/4	2 1/2
1 1/8	1 11/16	2 3/8	3/4	1 11/16	1 15/16	3/4	1 13/16	2 1/16	3/4	2 1/16	9/16	2 1/2	2 3/4
1 1/4	1 7/8	2 5/8	7/8	1 7/8	2 3/16	7/8	2	2 5/16	7/8	2 1/4	5/8	2 3/4	3
1 3/8	2 1/16	2 15/16	15/16	2 1/16	2 3/8	15/16	2 3/16	2 1/2	15/16	2 1/2	11/16	3	3 1/4
1 1/2	2 1/4	3 3/16	1	2 1/4	2 5/8	1	2 3/8	2 3/4	1	2 11/16	3/4	3 1/4	3 1/2
1 3/4	–	–	–	2 5/8	3	1 3/16	2 3/4	3 3/16	1 3/16	–	–	3 3/4	4
2	–	–	–	3	3 7/16	1 3/8	3 1/8	3 5/8	1 3/8	–	–	4 1/4	4 1/2
2 1/4	–	–	–	3 3/8	3 7/8	1 1/2	3 1/2	4 1/16	1 1/2	–	–	4 3/4	5
2 1/2	–	–	–	3 3/4	4 5/16	1 11/16	3 7/8	4 1/2	1 11/16	–	–	5 1/4	5 1/2
2 3/4	–	–	–	4 1/8	4 3/4	1 13/16	4 1/4	4 15/16	1 13/16	–	–	5 3/4	6
3	–	–	–	4 1/2	5 3/16	2	4 5/8	5 5/16	2	–	–	6	6 1/2
3 1/4	–	–	–	4 7/8	5 5/8	2 3/16	–	–	–	–	–	6	7
3 1/2	–	–	–	5 1/4	6 1/16	2 5/16	–	–	–	–	–	6	7 1/2
3 3/4	–	–	–	5 5/8	6 1/2	2 1/2	–	–	–	–	–	6	8
4	–	–	–	6	6 15/16	2 11/16	–	–	–	–	–	6	8 1/2

Notes:
For high-strength bolt and nut dimensions, refer to Table 7-14.
Square, hex and heavy hex bolt dimensions, rounded to nearest 1/16 in., are in accordance with ASME B18.2.6.
Countersunk bolt dimensions, rounded to the nearest 1/16 in., are in accordance with ASME B18.5.
Minimum thread length = $2d + 1/4$ in. for bolts up to 6 in. long, and $2d + 1/2$ in. for bolts longer than 6 in.

Table 7-19 (continued)
Dimensions of Non-High-Strength Fasteners, in.



Square, Heavy Square   Hex, Heavy Hex

Nut Size, in.	Square			Hex			Heavy Square			Heavy Hex		
	W, in.	*C*, in.	*N*, in.	*W*, in.	*C*, in.	*N*, in.	*W*, in.	*C*, in.	*N*, in.	*W*, in.	*C*, in.	*N*, in.
1/4	7/16	5/8	1/4	7/16	1/2	3/16	1/2	11/16	1/4	1/2	9/16	1/4
3/8	5/8	7/8	5/16	9/16	5/8	1/4	11/16	1	3/8	11/16	13/16	3/8
1/2	4/5	1 1/8	7/16	3/4	7/8	3/8	7/8	1 1/4	1/2	7/8	1	1/2
5/8	1	1 7/16	9/16	15/16	1 1/16	7/16	1 1/16	1 1/2	5/8	1 1/16	1 1/4	5/8
3/4	1 1/8	1 9/16	11/16	1 1/8	1 5/16	1/2	1 1/4	1 3/4	3/4	1 1/4	1 7/16	3/4
7/8	1 5/16	1 7/8	3/4	1 5/16	1 1/2	9/16	1 7/16	2 1/16	7/8	1 7/16	1 11/16	7/8
1	1 1/2	2 1/8	7/8	1 1/2	1 3/4	11/16	1 5/8	2 5/16	1	1 5/8	1 7/8	1
1 1/8	1 11/16	2 3/8	1	1 11/16	1 15/16	3/4	1 13/16	2 9/16	1 1/8	1 13/16	2 1/16	1 1/8
1 1/4	1 7/8	2 5/8	1 1/8	1 7/8	2 3/16	7/8	2	2 13/16	1 1/4	2	2 5/16	1 1/4
1 3/8	2 1/16	2 15/16	1 1/4	2 1/16	2 3/8	15/16	2 3/16	3 1/8	1 3/8	2 3/16	2 1/2	1 3/8
1 1/2	2 1/4	3 3/16	1 5/16	2 1/4	2 5/8	1	2 3/8	3 3/8	1 1/2	2 3/8	2 3/4	1 1/2
1 3/4	–	–	–	–	–	–	–	–	–	2 3/4	3 3/16	1 3/4
2	–	–	–	–	–	–	–	–	–	3 1/8	3 5/8	2
2 1/4	–	–	–	–	–	–	–	–	–	3 1/2	4 1/16	2 3/16
2 1/2	–	–	–	–	–	–	–	–	–	3 7/8	4 1/2	2 7/16
2 3/4	–	–	–	–	–	–	–	–	–	4 1/4	4 15/16	2 11/16
3	–	–	–	–	–	–	–	–	–	4 5/8	5 5/16	2 15/16
3 1/4	–	–	–	–	–	–	–	–	–	5	5 3/4	3 3/16
3 1/2	–	–	–	–	–	–	–	–	–	5 3/8	6 3/16	3 7/16
3 3/4	–	–	–	–	–	–	–	–	–	5 3/4	6 5/8	3 11/16
4	–	–	–	–	–	–	–	–	–	6 1/8	7 1/16	3 15/16

Notes:
For high-strength bolt and nut dimensions, refer to Table 7-14.
Square, hex and heavy hex bolt dimensions, rounded to nearest 1/16 in., are in accordance with ASME B18.2.6.
Countersunk bolt dimensions, rounded to the nearest 1/16 in., are in accordance with ASME B18.5.
Minimum thread length = $2d + 1/4$ in. for bolts up to 6 in. long, and $2d + 1/2$ in. for bolts longer than 6 in.

Table 7-20
Weights of Non-High-Strength Fasteners, pounds

	Bolt Length, in.	Nominal Bolt Diameter, in.								
		1/4	3/8	1/2	5/8	3/4	7/8	1	1 1/8	1 1/4
100 Square Bolts with Hexagonal Nuts[a]	1	2.38	6.11	13.0	24.1	38.9	–	–	–	–
	1 1/4	2.71	6.71	14.0	25.8	41.5	–	–	–	–
	1 1/2	3.05	7.47	15.1	27.6	44.0	67.3	95.1	–	–
	1 3/4	3.39	8.23	16.5	29.3	46.5	70.8	99.7	–	–
	2	3.73	8.99	17.8	31.4	49.1	74.4	104	143	–
	2 1/4	4.06	9.75	19.1	33.5	52.1	77.9	109	149	–
	2 1/2	4.40	10.5	20.5	35.6	55.1	82.0	114	155	206
	2 3/4	4.74	11.3	21.8	37.7	58.2	86.1	119	161	213
	3	5.07	12.0	23.2	39.8	61.2	90.2	124	168	221
	3 1/4	5.41	12.8	24.5	41.9	64.2	94.4	129	174	229
	3 1/2	5.75	13.5	25.9	44.0	67.2	98.5	135	181	237
	3 3/4	6.09	14.3	27.2	46.1	70.2	103	140	188	246
	4	6.42	15.1	28.6	48.2	73.3	107	145	195	254
	4 1/4	6.76	15.8	29.9	50.3	76.3	111	151	202	262
	4 1/2	7.10	16.6	31.3	52.3	79.3	115	156	208	271
	4 3/4	7.43	17.3	32.6	54.4	82.3	119	162	215	279
	5	7.77	18.1	33.9	56.5	85.3	123	167	222	288
	5 1/4	8.11	18.9	35.3	58.6	88.4	127	172	229	296
	5 1/2	8.44	19.6	36.6	60.7	91.4	131	178	236	304
	5 3/4	8.78	20.4	38.0	62.8	94.4	136	183	242	313
	6	9.12	21.1	39.3	64.9	97.4	140	188	249	321
	6 1/4	9.37	21.7	40.4	66.7	100	143	193	255	329
	6 1/2	9.71	22.5	41.8	68.7	103	147	198	262	337
	6 3/4	10.1	23.3	43.1	70.8	106	151	204	269	345
	7	10.4	24.0	44.4	72.9	109	156	209	275	354
	7 1/4	10.7	24.8	45.8	75.0	112	160	214	282	362
	7 1/2	11.0	25.5	47.1	77.1	115	164	220	289	371
	7 3/4	11.4	26.3	48.5	79.2	118	168	225	296	379
	8	11.7	27.0	49.8	81.3	121	172	231	303	387
	8 1/2	–	28.6	52.5	85.5	127	180	241	316	404
	9	–	30.1	55.2	89.7	133	189	252	330	421
	9 1/2	–	31.6	57.9	93.9	139	197	263	343	438
	10	–	33.1	60.6	98.1	145	205	274	357	454
	10 1/2	–	34.6	63.3	102	151	213	284	371	471
	11	–	36.2	66.0	106	157	221	295	384	488
	11 1/2	–	37.7	68.7	110	163	230	306	398	505
	12	–	39.2	71.3	115	170	238	316	411	522
	12 1/2	–	–	74.0	119	176	246	327	425	538
	13	–	–	76.7	123	182	254	338	439	556
	13 1/2	–	–	79.4	127	188	263	349	452	572
	14	–	–	82.1	131	194	271	359	466	589
	14 1/2	–	–	84.8	135	200	279	370	479	605
	15	–	–	87.5	140	206	287	381	493	622
	15 1/2	–	–	90.2	144	212	296	392	507	639
	16	–	–	92.9	148	218	304	402	520	656
	Per inch add'tl. Add	1.3	3.0	5.4	8.4	12.1	16.5	21.4	27.2	33.6

Notes:
For weight of high-strength fasteners, see Table 7-18.
This table conforms to weight standards adopted by the Industrial Fasteners Institute (IFI).
[a] Square bolt per ASME B18.2.6, hexagonal nut per ASME B18.2.2. For other non-high-strength fasteners, refer to Tables 7-21 and 7-22.

Table 7-21
Weight Adjustments
for Combinations of Non-High-Strength Fasteners Other than Tabulated in Table 7-20[a], pounds

Combinations of 100		Add or Subtr.	Nominal Bolt Diameter, in.								
			1/4	3/8	1/2	5/8	3/4	7/8	1	1 1/8	1 1/4
Square Bolts With	Square Nuts	+	0.1	1.0	2.0	3.4	3.5	5.5	8.0	12.2	16.3
	Heavy Square Nuts	+	0.6	2.1	4.1	7.0	11.6	17.2	23.2	32.1	41.2
	Heavy Hex Nuts	+	0.4	1.5	2.8	4.6	7.6	10.7	14.2	18.9	24.3
100 Square Bolts with Hexagonal Nuts	Square Nuts	+	0.1	0.6	1.1	1.4	0.2	0.5	-0.2	-0.1	-1.7
	Hex Nuts	–	0.0	0.4	0.9	2.0	3.3	5.0	8.2	12.3	18.0
	Heavy Square Nuts	+	0.6	1.7	3.2	5.0	8.3	12.2	15.0	19.8	23.2
	Heavy Hex Nuts	+	0.4	1.1	1.9	2.6	4.3	5.7	6.0	6.6	6.3
100 Hex Bolts	Heavy Square Nuts	+	–	–	4.7	7.3	11.3	16.5	20.7	27.0	33.6
	Heavy Hex Nuts	+	–	–	3.4	4.9	7.3	10.0	11.7	13.8	16.7

Notes:
For weights of high-strength fasteners, see Table 7-18.
This table conforms to weight standards adopted by the Industrial Fasteners Institute (IFI).
[a] Add or subtract value in this table to or from the value in Table 7-20.

Table 7-22
Weights of Non-High-Strength Bolts
of Diameter Greater than $1^1/_4$ in., pounds

Weight of 100 Each		Nominal Bolt Diameter, in.											
		$1^3/_8$	$1^1/_2$	$1^3/_4$	2	$2^1/_4$	$2^1/_2$	$2^3/_4$	3	$3^1/_4$	$3^1/_2$	$3^3/_4$	4
Heads of:	Square Bolts	105	130	–	–	–	–	–	–	–	–	–	–
	Hex Bolts	84.0	112	178	259	369	508	680	900	1120	1390	1730	2130
	Heavy Hex Bolts	95.0	124	195	280	397	541	720	950	–	–	–	–
One Linear Inch, Unthreaded Shank		42.0	50.0	68.2	89.0	113	139	168	200	235	272	313	356
One Linear Inch, Threaded Shank		35.0	42.5	57.4	75.5	97.4	120	147	178	210	246	284	325
Square Nuts		94.5	122	–	–	–	–	–	–	–	–	–	–
Heavy Square Nuts		125	161	–	–	–	–	–	–	–	–	–	–
Heavy Hex Nuts		102	131	204	299	419	564	738	950	1190	1530	1810	2180

– Indicates that the bolt size is not available

PART 8

DESIGN CONSIDERATIONS FOR WELDS

SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of welded joints. For the design of connecting elements, see Part 9. For the design of simple shear, moment, bracing and other connections, see Parts 10 through 15.

GENERAL REQUIREMENTS FOR WELDED JOINTS

The requirements for welded construction are given in AISC *Specification* Section M2.4, which requires the use of AWS D1.1, except as modified in AISC *Specification* Section J2. For further information see also Blodgett et al. (1997).

Welding in structural steel is performed in compliance with written welding procedure specifications (WPS). WPS are qualified by test or prequalified in AWS D1.1. WPS are used to control base metal, consumables, joint geometry, electrical and other essential variables for welded joints.

Consumables

Requirements for welding consumables are given in AISC *Specification* Sections A3.5, J2.6 and J2.7. Permissible filler metal strengths are shown in Table J2.5, based on matching filler metals shown in AWS D1.1 Table 3.2. Filler metal notch-toughness requirements are given in AISC *Specification* Section J2.6. Low-hydrogen electrodes for shielded metal arc welding (SMAW) are required, as shown in AWS D1.1 Table 3.2. Low-hydrogen SMAW electrodes have a limited exposure time and rod ovens are necessary near the point of use for storage.

Requirements for the manufacture, classification and packing of consumables are given in AWS A5.x specifications. Consumables vary based upon their welding process. SMAW, or "stick" welding, is a manual process. Submerged arc welding (SAW) is a semiautomatic or automatic process. Consumables are classified as an electrode flux combination because the weld metal properties are dependant on both the electrode and the flux. SAW is suitable for long straight or circumferential welds but the work must be performed in horizontal or flat positions. Flux-cored arc welding (FCAW) uses wire electrode that contains flux in the center. FCAW electrodes are provided for use with a gas shield or self shield. Gas for shielding is argon, carbon dioxide or a combination of the two. Gas metal arc welding (GMAW) uses wire electrodes that are solid or have a metal core. GMAW is performed with gas shielding.

Thermal Cutting

Oxygen-fuel gas cutting can be used to cut almost any commercially available plate thickness. If the plate being cut contains large discontinuities or nonmetallic inclusions, turbulence may be created in the cutting stream, resulting in notches or gouges in the edge of the cut. Plasma-arc cutting is much faster and less susceptible to the effects of discontinuities or nonmetallic inclusions, but leaves a slight taper in the cut as it descends and can be used only up to about $1^1/_2$-in. thickness.

Air-Arc Gouging

In this method, a carbon arc is used to melt a nugget-shaped area of the base metal, which is blown away with a jet of compressed air. Air-arc gouging can be used to remove weld defects, gouge the weld root to sound weld metal, form a U groove on one side of a square butt joint, and for similar operations.

Inspection

The five most commonly used methods for welding inspection are discussed in the following and in the *Guide for the Nondestructive Examination of Welds* (AWS B1.10) (AWS, 2009). Chapter N of the AISC *Specification* contains requirements for nondestructive examination (NDE) of welds. The general contractor or owner must arrange for this. This work must be scheduled to minimize interruption of the fabricator and erector. The designer may specify in the contract documents the types of weld inspection required as well as the extent and application of each type of inspection differing from the requirements of Chapter N. In the absence of instructions for weld inspection, the fabricator or erector is only responsible for those weld discontinuities found by visual inspection (see AWS D1.1). Welds may have defects that cannot be rejected based on AWS criteria. Stipulation of various NDE methods has the effect of selecting acceptance criteria and therefore has a related effect on costs. Weld repairs which may be difficult to perform and which may potentially damage other aspects of the connection are best referred to the engineer of record to determine the necessity of the correction with due consideration of fitness for purpose.

Visual inspection is the most commonly required inspection process. The designer must realize that more stringent requirements for inspection can needlessly add significant cost to the project and should specify them only in those instances where they are essential to the integrity of the structure.

Visual Testing (VT)

Visual inspection provides the most economical way to check weld quality and is the most commonly used method. Joints are scrutinized prior to the commencement of welding to check fit-up, preparation bevels, gaps, alignment and other variables. After the joint is welded, it is then visually inspected in accordance with AWS D1.1. If a discontinuity is suspected, the weld is either repaired or other inspection methods are used to validate the integrity of the weld. In most cases, timely visual inspection by an experienced inspector is sufficient and offers the most practical and effective inspection alternative to other, more costly methods.

Penetrant Testing (PT)

This test uses a red dye penetrant applied to the work from a pressure spray can. The dye penetrates any crack or crevice open to the surface. Excess dye is removed and white developer is sprayed on. Dye seeps out of the crack, producing a red image on the white developer (See Figure 8-1).

Penetrant testing (PT) can be used to detect tight cracks as long as they are open to the surface. However, only surface cracks are detectable. Furthermore, deep weld ripples and scratches may give a false indication when PT is used.

Dye penetrant examination tends to be messy and slow, but can be helpful when determining the extent of a defect found by visual inspection. This is especially true when a defect is being removed by gouging or grinding for the repair of a weld to assure that the defect is completely removed.

Magnetic-Particle Testing (MT)

A magnetizing current is introduced with a yoke or contact prods into the weldment to be inspected, as sketched in Figure 8-2 (prods shown). This induces a magnetic field in the work, which will be distorted by any cracks, seams, inclusions, etc. located on or near (within approximately 0.1 in. of) the surface. A dry magnetic powder blown lightly on the surface by a rubber squirt bulb will be picked up at such discontinuities making a distinct mark. The magnetically held particles show the location, size, and shape of the discontinuity.

The method will indicate surface cracks that might be difficult for liquid penetrant to enter and subsurface cracks to about 0.1-in. depth, with proper magnetization. Records may be kept by picking up the powder pattern with clear plastic tape. Cleanup is easy, but demagnetizing, if necessary, may not be. If the magnetizing prod is lifted from the work while the current is still on, an arc strike may be produced, which could lead to cracking. If arc strikes occur, they should be ground out.

Magnetic particle examination can be useful when a defect is suspected from visual inspection or when the absence of cracking in areas of high restraint must be confirmed. Relatively smooth surfaces are required for MT and it is reasonably economical. Where delayed cracking is suspected, the nondestructive examination may have to be performed after a cooling time—typically 48 hours.

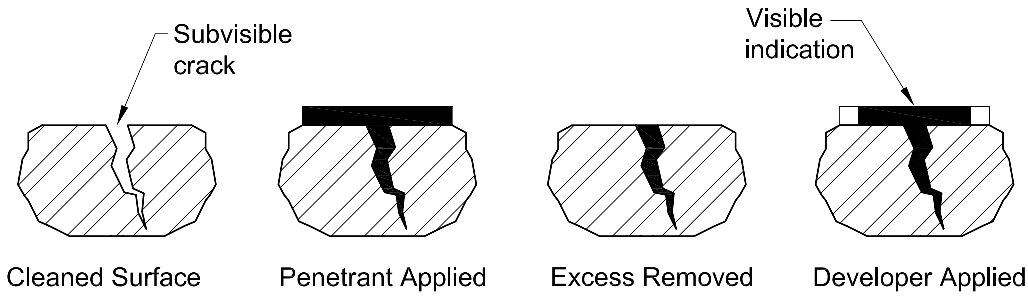


Fig. 8-1. Schematic illustration of penetrant testing (PT).

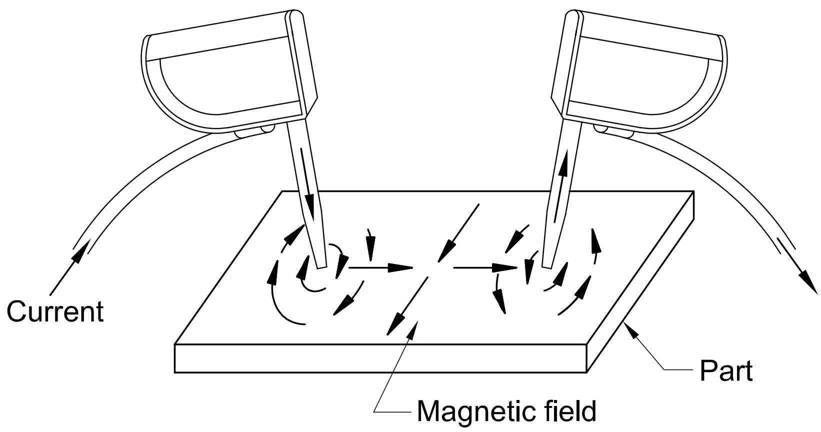


Fig. 8-2. Schematic illustration of magnetic particle testing (MT).

Ultrasonic Testing (UT)

The ultrasonic inspection process is analogous to sonar. A short pulse of high-frequency sound waves are broadcast from a crystal into a metal, after which the crystal waits to receive reflections from the far end of the metal member and from any voids encountered on the way through. The technique is called pulse echo. The sound beam is produced by a piezoelectric transducer energized by an electric current which causes the crystal to vibrate and transmit through a liquid couplant into the metal. Any reflections are displayed as pips on a cathode ray tube (CRT) grid whose horizontal scale represents distance through the metal. The vertical scale represents the strength (or area) of the reflecting surface. The system is shown schematically in Figure 8-3.

The accuracy of ultrasonic inspection is highly dependent upon the skill and training of the operator and frequent calibration of the instrument. There is a "dead" area beneath most transducers that makes it difficult to inspect members less than 5/16 in. in thickness. Austenitic stainless steels and extremely coarse-grained steels, e.g., electroslag welds, are difficult to inspect; but on structural carbon and low-alloy steels, the process can detect flat discontinuities (favorably oriented for reflection) smaller than 1/64 in. The crystal, which is 3/8 in. to 1 in. in size, can be readily moved about to check many orientations and can project the beam into the metal at angles of 90°, 70°, 60° and 45°. With the latter three angles,

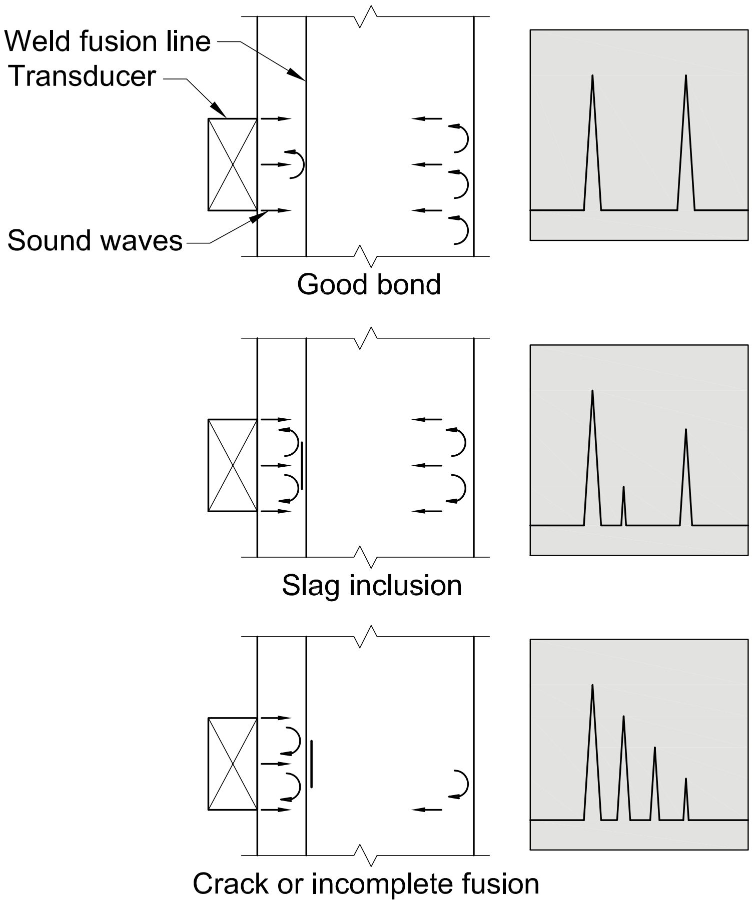


Fig. 8-3. Variations in UT reflections caused by defects at the boundary.

the beam can be bounced around inside the metal, producing echoes from any discontinuity on the way. For more information, see Krautkramer (1990) and Institute of Welding (1972).

Ultrasonic testing (UT) is a more versatile, rapid and economical inspection method than radiography, but it does not provide a permanent record like the X-ray negative. The operator, instead, makes a written record of discontinuity indications appearing on his CRT. Certain joint geometry limits the use of the ultrasonic method.

Ultrasonic examination has limited applicability in some applications, such as HSS fabrication. Relatively thin sections and variations in joint geometry can lead to difficulties in interpreting the signals, although technicians with specific experience on weldments similar to those to be examined may be able to decipher UT readings in some instances. Similarly, UT is usually not suitable for use with fillet welds and smaller partial-joint-penetration (PJP) groove welds. Complete-joint-penetration (CJP) groove welds with and without backing bars also give readings that are subject to differing interpretations. Ultrasonic examination may be specified to validate the integrity of CJP groove welds that are subject to tension. Ultrasonic examination has largely replaced radiographic examination for the inspection of critical CJP groove welds in building construction. New technology called phased array is in development and in use in some applications. Phased array is a computer controlled ultrasonic examination capable of providing an informative display. AWS D1.1 provisions for acceptance criteria have not been adopted for this method at this time.

Radiographic Testing (RT)

Radiographic testing (RT) is basically an X-ray film process. To be detected by radiography, a crack must be oriented roughly parallel to the impinging radiation beam, and occupy about $1\frac{1}{2}\%$ of the metal thickness along that beam. There are problems with radiographs of fillets, tee and corner joints, however, because the radiation beam must penetrate varying thicknesses.

Precautions for avoiding radiation hazards interfere with shop work, and equipment and film costs make it the most expensive inspection method. Ultrasonic systems have gradually supplemented and even supplanted radiography.

Radiographic examination has very limited applicability in some applications, such as for HSS fabrication, because of the irregular shape of common joints and the resulting variations in thickness of material as projected onto film. RT can be used successfully for butt splices, but can only provide limited information about the condition of fusion at backing bars near the root corners. The general inability to place either the radiation source or the film inside the HSS means that exposures must usually be taken through both the front and back faces of the section with the film attached to the outside of the back face. Several such shots progressing around the member are needed to examine the complete joint.

PROPER SPECIFICATION OF JOINT TYPE

Selection of Weld Type

The most common weld types are fillet and groove welds. Fillet welds are normally more economical than groove welds and generally should be used in applications for which groove welds are not required. Additionally, fillet welds around the inside of holes or slots require less weld metal than plug or slot welds of the same size, even though the diameters of holes and widths of slots for fillet welds must be larger to accommodate the necessary tilt of the electrode.

PJP groove welds are more economical than CJP groove welds. When groove welds are required, bevel and V groove welds, which can be flame-cut, are usually more economical than J and U groove welds, which must be air-arc gouged or planed. Also, double-bevel, double-V, double-J, and double-U groove welds are typically more economical than welds of the same type with single-sided preparation because they use less weld metal, particularly as the thickness of the connection element(s) being welded increases. The symmetry also results in less rotational distortion strain. However, in thinner connection elements, the savings in weld-metal volume may not offset the additional cost of double edge preparation, weld-root cleaning, and repositioning. As a general rule of thumb, double-sided joint preparation is normally less expensive than single-sided preparation above 1-in. thickness.

Welding Symbols

For guidance on the proper use of welding symbols, refer to Table 8-2. More extensive information on welding symbols may be found in AWS A2.4, *Standard Symbols for Welding, Brazing, and Nondestructive Examination* (AWS, 2007).

Available Strength

The available strength of a welded joint is determined in accordance with AISC *Specification* Section J2.4 and Table J2.5. Section 3.9.5 of AISC Design Guide 21, *Welded Connections—A Primer for Engineers* (Miller, 2006), includes a discussion of the strength of different weld types (groove, fillet, plug/slot) combined in a single joint.

The calculation of the available strength of a longitudinally loaded fillet weld can be simplified from that given in AISC *Specification* Table J2.5. For a fillet weld with length less than or equal to 100 times the weld size, the available shear strength, ϕR_n or R_n/Ω, may be calculated as follows:

$$R_n = 0.60F_{EXX}\left(\frac{\sqrt{2}}{2}\right)\left(\frac{D}{16}\right)l \qquad \text{(8-1)}$$

$$\phi = 0.75 \qquad \Omega = 2.00$$

where

D = weld size in sixteenths of an inch

l = length, in.

For F_{EXX} = 70 ksi:

LRFD	ASD
$\phi R_n = (1.392 \text{ kip/in.})\, Dl$ (8-2a)	$\dfrac{R_n}{\Omega} = (0.928 \text{ kip/in.})\, Dl$ (8-2b)

When the fillet weld is not longitudinally loaded, the alternative provisions in AISC *Specification* Section J2.4(b) may be used to take advantage of the increased strength due to load angle.

Effect of Load Angle

When designing fillet welds, the increased strength due to loading angle may be accounted for by multiplying the available strength of the weld by the following expression if strain compatibility of the various weld elements is considered, as given in AISC *Specification* Equation J2-5:

$$(1.0 + 0.50\sin^{1.5}\theta)$$

where

θ = angle between the line of action of the required force and the weld longitudinal axis, degrees

For transversely loaded welds, $\theta = 90°$. This accounts for a 50% increase in weld strength over a longitudinally loaded weld. However, this increased weld strength is accompanied by a decrease in ductility. For a single line weld, the decreased ductility is inconsequential for most applications. However, for weld groups composed of welds loaded at various angles, this change in ductility means that the designer must consider load-deformation compatibility.

CONCENTRICALLY LOADED WELD GROUPS

The load-deformation curves shown in Figure 8-5 highlight the need for consideration of deformation compatibility, since the transversely loaded weld will fracture before the longitudinally loaded weld obtains its full strength.

A simplified procedure for determining the available strength of concentrically loaded fillet weld groups is discussed later in Part 8 using Table 8-1. In lieu of using this procedure, it is permitted to sum the capacities of individual weld elements, neglecting load-deformation compatibility, when no increase in strength due to the loading angle is assumed.

ECCENTRICALLY LOADED WELD GROUPS

Eccentricity in the Plane of the Faying Surface

Eccentricity in the plane of the faying surface produces additional shear. The welds must be designed to resist the combined effect of the direct shear, P_u or P_a, and the additional shear from the induced moment, P_ue or P_ae. Two methods of analysis for this type of eccentricity are the instantaneous center of rotation method and the elastic method.

The instantaneous center of rotation method is more accurate, but generally requires the use of tabulated values or an iterative solution. The elastic method is simplified, but may be excessively conservative because it neglects the ductility of the weld group and the potential load increase.

Instantaneous Center of Rotation Method

Eccentricity produces both a rotation and a translation of one connection element with respect to the other. The combined effect of this rotation and translation is equivalent to a rotation about a point defined as the instantaneous center of rotation (IC) as illustrated in Figure 8-4(a). The location of the IC depends upon the geometry of the weld group as well as the direction and point of application of the load.

The load deformation relationship for a unit length segment of the weld, as illustrated in Figure 8-5, is an approximation of the equation by Lesik and Kennedy (1990). The nominal stress in the ith weld element, F_{nwi}, is limited by the deformation, Δ_{ui}, of the weld segment that first reaches its limit, where

$$F_{nwi} = 0.60F_{EXX}(1.0 + 0.50 \sin^{1.5}\theta_i)\ [p_i(1.9 - 0.9p_i)]^{0.3} \tag{8-3}$$

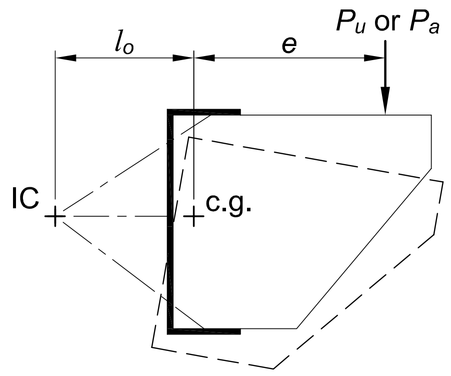
where

F_{EXX} = filler metal classification strength, ksi

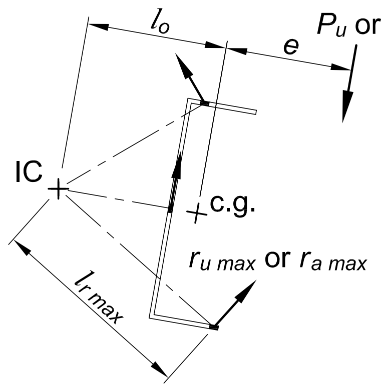
F_{nwi} = nominal stress in the ith weld element, ksi

θ_i = angle between the longitudinal axis of ith weld element and the direction of the resultant force acting on the element, degrees

p_i = Δ_i/Δ_{mi}
= ratio of element i deformation to its deformation at maximum stress



(a) Instantaneous center of rotation (IC)



(b) Forces on weld elements

Fig. 8-4. Instantaneous center of rotation method.

r_{cr} = distance from instantaneous center of rotation to weld element with minimum Δ_{ui}/r_i ratio, in.

w = weld leg size, in.

Δ_i = $r_i \Delta_{ucr}/r_{cr}$
= deformation of the *i*th weld element at an intermediate stress level, linearly proportioned to the critical deformation based on distance from the instantaneous center of rotation, r_i, in.

Δ_{ucr} = deformation of the weld element with minimum ratio Δ_{ui}/r_i at ultimate stress (rupture), usually in the element furthest from the instantaneous center of rotation, in.

Δ_{ui} = $1.087(\theta_i + 6)^{-0.65}\, w \le 0.17w$, in. (8-4)
= deformation of the *i*th weld element at ultimate stress (rupture), in.

Unlike the load-deformation relationship for bolts, the strength deformation of welds is dependent upon the angle, θ_i, that the resultant elemental force makes with the axis of the weld element. Load-deformation curves in Figure 8-5 for values of weld element shear strength, P, relative to $P_o = 0.60F_{EXX}$ for values of θ_i = 0º, 15º, 30º, 45º, 60º, 75º and 90º are shown. For further information, see AISC *Specification* Section J2.4 and its commentary.

The nominal strengths of the other unit-length weld segments in the joint can be determined by applying a deformation, Δ, that varies linearly with the distance from the IC. The nominal shear strength of the weld group is, then, the sum of the individual strengths of all weld segments. Because of the nonlinear nature of the requisite iterative solution, for sufficient accuracy, a minimum of 20 weld elements for the longest line segment is generally recommended.

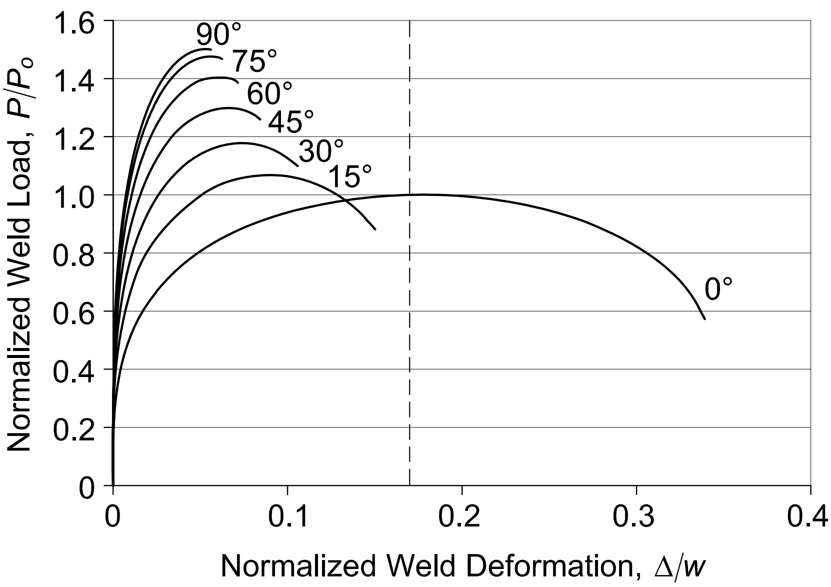


Fig. 8-5. Fillet weld strength versus deformation as a function of load angle, θ.

The individual resistance of each weld segment is assumed to act on a line perpendicular to a ray passing through the IC and the centroid of that weld segment, as illustrated in Figure 8-4(b). If the correct location of the instantaneous center has been selected, the three equations of in-plane static equilibrium, $\Sigma F_x A_{wei} = 0$, $\Sigma F_y A_{wei} = 0$, and $\Sigma M = 0$, will be satisfied, where A_{wei} is the effective weld area.

For further information, see Crawford and Kulak (1971) and Butler et al. (1972).

Elastic Method

For a force applied as illustrated in Figure 8-4, the eccentric force, P_u or P_a, is resolved into a force, P_u or P_a, acting through the center of gravity of the weld group and a moment, $P_u e$ or $P_a e$, where e is the eccentricity. Each weld element is then assumed to resist an equal share of the direct shear, P_u or P_a, and a share of the eccentric moment, $P_u e$ or $P_a e$, proportional to its distance from the center of gravity. The resultant vectorial sum of these forces, r_u or r_a, is the required strength for the weld.

The shear per linear inch of weld due to the concentric force, r_{pu} or r_{pa}, is determined as

LRFD	ASD
$r_{pu} = \dfrac{P_u}{l}$ (8-5a)	$r_{pa} = \dfrac{P_a}{l}$ (8-5b)

where

l = total length of the weld in the weld group, in.

To determine the resultant shear per linear inch of weld, r_{pu} or r_{pa} must be resolved into horizontal components, r_{pux} or r_{pax}, and vertical components, r_{puy} or r_{pay}, where

$$r_{pux} = r_{pu}\sin\theta \text{ (LRFD)} \quad (8\text{-}6a)$$

$$r_{pax} = r_{pa}\sin\theta \text{ (ASD)} \quad (8\text{-}6b)$$

$$r_{puy} = r_{pu}\cos\theta \text{ (LRFD)} \quad (8\text{-}7a)$$

$$r_{pay} = r_{pa}\cos\theta \text{ (ASD)} \quad (8\text{-}7b)$$

The shear per linear inch of weld due to the moment, $P_u e$ or $P_a e$, is r_{mu} or r_{ma}, where

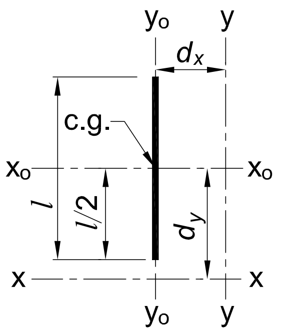
LRFD	ASD
$r_{mu} = \dfrac{P_u ec}{I_p}$ (8-8a)	$r_{ma} = \dfrac{P_a ec}{I_p}$ (8-8b)

where

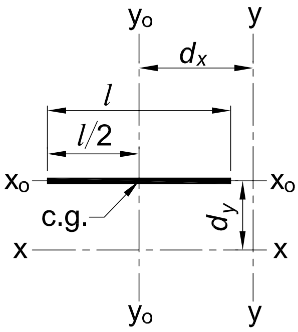
c = radial distance from the center of gravity to point in weld group most remote from the center of gravity, in.

$I_p = I_x + I_y$

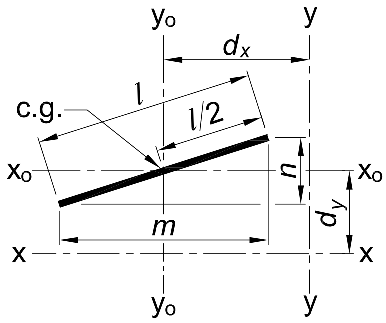
= polar moment of inertia of the weld group, in.^{4} per in. Refer to Figure 8-6. For section moduli and torsional constants of various welds treated as line elements, refer to Table 5 in Section 7.4 of Blodgett (1966).



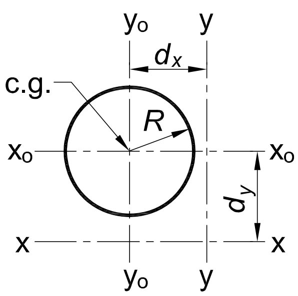
$$I_{xo} = \frac{l^3}{12}$$
$$I_x = \frac{l^3}{12} + l(d_y)^2$$
$$I_{yo} = 0$$
$$I_y = l(d_x)^2$$



$$I_{xo} = 0$$
$$I_x = l(d_y)^2$$
$$I_{yo} = \frac{l^3}{12}$$
$$I_y = \frac{l^3}{12} + l(d_x)^2$$

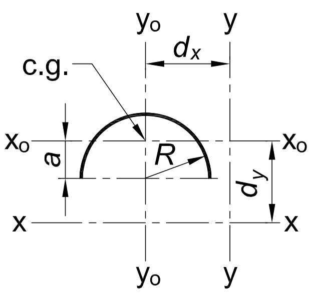


$$I_{xo} = \frac{ln^2}{12}$$
$$I_x = \frac{ln^2}{12} + l(d_y)^2$$
$$I_{yo} = \frac{lm^2}{12}$$
$$I_y = \frac{lm^2}{12} + l(d_x)^2$$



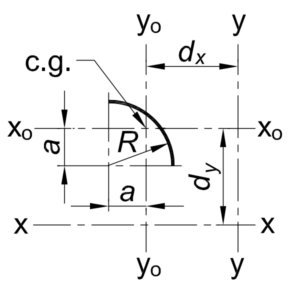
Length of weld, $l = 6.28R$

$$I_{xo} = \pi R^3$$
$$I_x = \pi R^3 + l(d_y)^2$$
$$I_{yo} = \pi R^3$$
$$I_y = \pi R^3 + l(d_x)^2$$



$a = 0.637R$
Length of weld, $l = 3.14R$

$$I_{yo} = \frac{\pi}{2} R^3$$
$$I_y = \frac{\pi}{2} R^3 + l(d_x)^2$$
$$I_{xo} = \left(\frac{\pi}{2} - \frac{4}{\pi}\right) R^3$$
$$I_x = \left(\frac{\pi}{2} - \frac{4}{\pi}\right) R^3 + l(d_y)^2$$



$a = 0.637R$
Length of weld, $l = 1.57R$

$$I_{xo} = \left(\frac{\pi}{4} - \frac{2}{\pi}\right) R^3$$
$$I_x = \left(\frac{\pi}{4} - \frac{2}{\pi}\right) R^3 + l\left(d_y\right)^2$$
$$I_{yo} = \left(\frac{\pi}{4} - \frac{2}{\pi}\right) R^3$$
$$I_y = \left(\frac{\pi}{4} - \frac{2}{\pi}\right) R^3 + l\left(d_x\right)^2$$

Fig. 8-6. Moments of inertia of various weld segments.

To determine the resultant force on the most highly stressed weld element, r_{mu} or r_{ma} must be resolved into horizontal component r_{mux} or r_{max} and vertical component r_{muy} or r_{may}, where

LRFD	ASD
$r_{mux} = \dfrac{P_u e c_y}{I_p}$ (8-9a)	$r_{max} = \dfrac{P_a e c_y}{I_p}$ (8-9b)
$r_{muy} = \dfrac{P_u e c_x}{I_p}$ (8-10a)	$r_{may} = \dfrac{P_a e c_x}{I_p}$ (8-10b)

In the above equations, c_x and c_y are the horizontal and vertical components of the radial distance c at the point where r_u or r_a is a maximum. The point in the weld group where the stress is highest will usually be at a corner, or a termination, or where the element is farthest from the center of gravity. Thus, the resultant force, r_u or r_a, is determined as

LRFD	ASD
$r_u = \sqrt{\left(r_{pux} + r_{mux}\right)^2 + \left(r_{puy} + r_{muy}\right)^2}$ (8-11a)	$r_a = \sqrt{\left(r_{pax} + r_{max}\right)^2 + \left(r_{pay} + r_{may}\right)^2}$ (8-11b)

which should be compared to the available strength, found in AISC *Specification* Table J2.5. For further information, see Higgins (1971).

Plastic Method

Table 8-4 provides coefficients that can be used to design pairs of linear welds subjected to an eccentric shear and a normal force, when k is taken equal to zero. These coefficients are calculated using the instantaneous center of rotation method. Given the prevalence with which these welds are encountered in design, simplified design methods have been developed and are presented in the following.

The simplest approach is to calculate the effects of the normal force and the moment independently, as shown in Figure 8-7, and combine them vectorially with the shear force. This approach produces:

$$f_v = \frac{V}{l_w} \tag{8-12}$$

$$f_a = \frac{N}{l_w} \tag{8-13}$$

$$f_b = \frac{4M}{{l_w}^2} \tag{8-14}$$

$$f_w = \sqrt{{f_v}^2 + \left(f_a + f_b\right)^2} \tag{8-15}$$

where

M = applied moment, kip-in.
N = applied normal force, kips
V = applied shear, kips
f_a = shear per linear inch of weld due to the applied normal force, kip/in.
f_b = shear per linear inch of weld due to the applied moment, kip/in.
f_v = shear per linear inch of weld due to the applied shear, kip/in.
f_w = total design stress, kip/in.
l_w = length of each weld, in.

A less conservative and more technically correct approach is to calculate the effects of the normal force and the moment based on a plastic normal stress distribution as shown in Figure 8-8, and then combine them vectorially with the shear. This approach produces:

$$f_v = \frac{V}{l_w} \tag{8-12}$$

$$l_a = \frac{\sqrt{4e_x^2 + l_w^2 \tan^2\theta} - 2e_x^2}{\tan\theta} \tag{8-16}$$

$$f_a = \frac{N}{l_a} = \frac{N\tan\theta}{\sqrt{4e_x^2 + l_w^2\tan^2\theta} - 2e_x^2} \tag{8-17}$$

$$f_b = \frac{M}{l_w^2 - l_a^2} \tag{8-18}$$

$$f_w = \sqrt{f_v^2 + f_b^2} \tag{8-19}$$

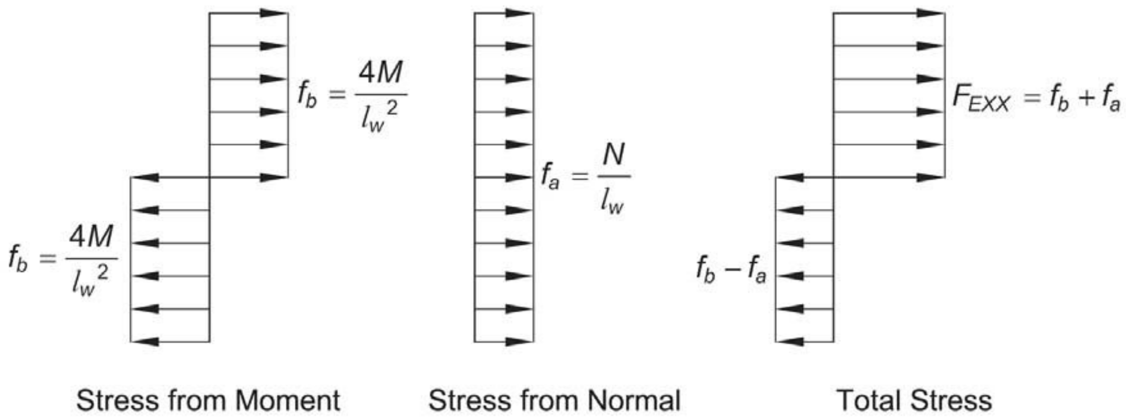


Fig. 8-7. Plastic method stress distribution.

where

M = applied moment, kip-in.
N = applied normal force, kips
V = applied shear, kips
f_a = shear per linear inch of weld due to the applied normal force, kip/in.
f_b = shear per linear inch of weld due to the applied moment, kip/in.
f_v = shear per linear inch of weld due to the applied shear, kip/in.
f_w = total design stress, kip/in.
l_a = length of weld over which the applied normal force is distributed, in.
l_w = length of each weld, in.

Eccentricity Normal to the Plane of the Faying Surface

Eccentricity normal to the plane of the faying surface, as illustrated in Figure 8-9 for a bracket connection, produces tension above and compression below the neutral axis. The eccentric force, P_u or P_a, is resolved into a direct shear, P_u or P_a, acting at the faying surface of the joint and a moment normal to the plane of the faying surface, P_ue or P_ae, where e is the eccentricity. Each unit-length segment of weld is then assumed to resist an equal share of the concentric force, P_u or P_a, and the moment is resisted by tension in the welds above the neutral axis and compression below the neutral axis.

In contrast to bolts, where the interaction of shear and tension must be considered, for welds, shear and tension can be combined vectorially into a resultant shear. Thus, the solution of a weld loaded eccentrically normal to the plane of the faying surface is similar to that discussed previously for welds loaded eccentrically in the plane of the faying surface.

OTHER SPECIFICATION REQUIREMENTS AND DESIGN CONSIDERATIONS

The following other specification requirements and design considerations apply to the design of welded joints.

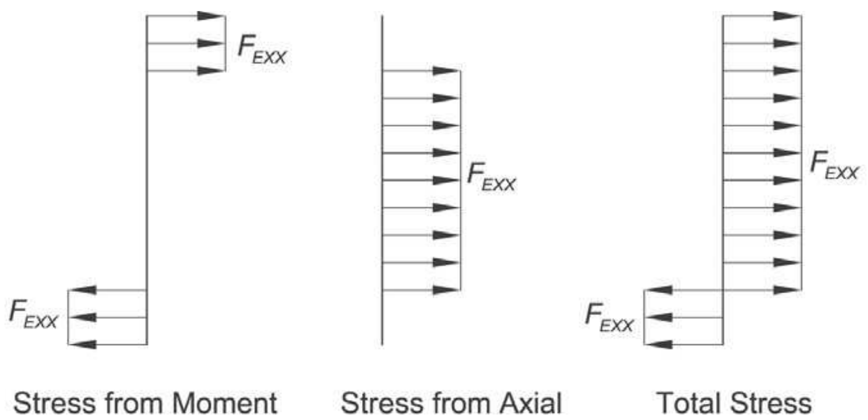


Fig. 8-8. Optimized plastic method stress distribution.

Special Requirements for Heavy Shapes and Plates

For CJP groove welded joints in heavy shapes with a flange thickness exceeding 2 in. or built-up sections consisting of plates with a thickness exceeding 2 in., see AISC *Specification* Sections A3.1c and A3.1d.

Placement of Weld Groups

For the required placement of weld groups at the ends of axially loaded members, see AISC *Specification* Section J1.7.

Welds in Combination with Bolts

For welds used in combination with bolts, see AISC *Specification* Section J1.8.

Fatigue

For applications involving fatigue, see AISC *Specification* Appendix 3.

One-Sided Fillet Welds

When lateral deformation is not otherwise prevented, a severe notch can result at locations of one-sided welds. For the fillet-welded joint illustrated in Figure 8-10, the unwelded side has no strength in tension and a notch may form from the unwelded side. Using one fillet weld on each side will eliminate this condition. This is also true with PJP groove welds.

Welding Considerations and Appurtenances

Clearance Requirements

Clearances are required to allow the welder to make proper welds. Ample room must be provided so that the welder or welding operator may manipulate the electrode and observe the weld as it is being deposited.

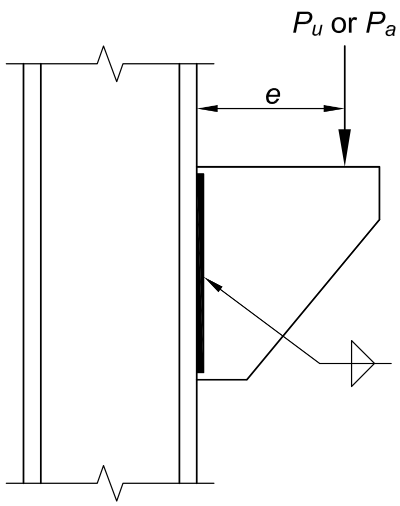


Fig. 8-9. Welds subject to eccentricity normal to the plane of the faying surface.

In the SMAW process, the preferred position of the electrode when welding in the horizontal position is in a plane forming 30° with the vertical side of the fillet weld being made. However, this angle, shown as angle *x* in Figure 8-11, may be varied somewhat to avoid contact with some projecting part of the work. A simple rule to provide adequate clearance for the electrode in horizontal fillet welding is that the clear distance to a projecting element should be at least one-half the distance *y* in Figure 8-11(b).

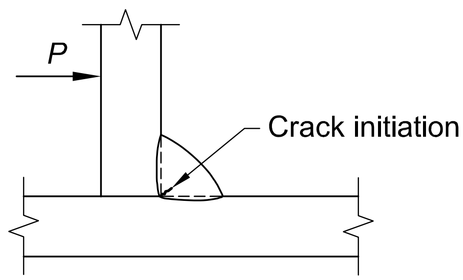


Fig. 8-10. Notch effect at one-sided weld.

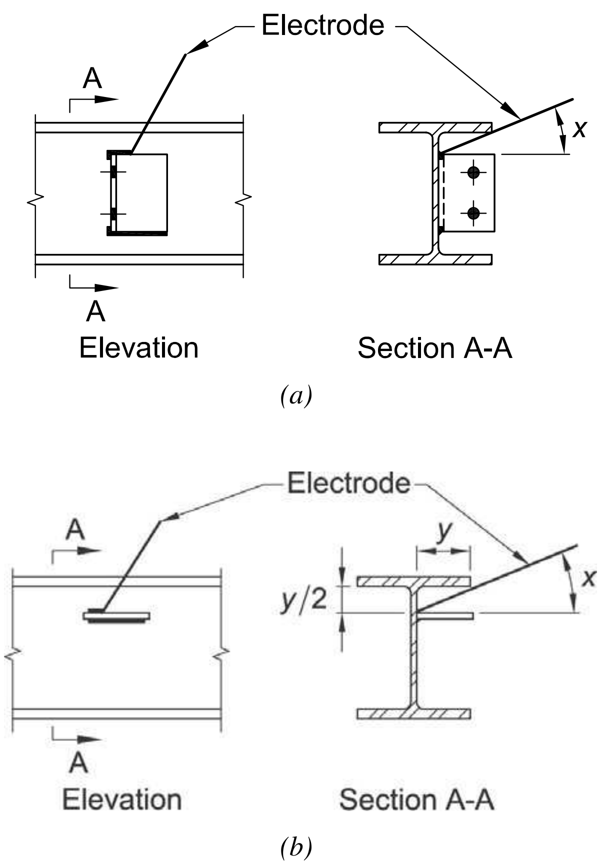


Fig. 8-11. Clearances for SMAW welding.

A special case of minimum clearance for welding with a straight electrode is illustrated in Figure 8-12. The 20° angle is the minimum that will allow satisfactory welding along the bottom of the angle and therefore governs the setback with respect to the end of the beam. If a $^1/_2$-in. setback and $^3/_8$-in. electrode diameter were used, the clearance between the angle and the beam flange could be no less than $1^1/_4$ in. for an angle with a leg dimension, w, of 3 in., nor less than $1^5/_8$ in. with a w of 4 in. When it is not possible to provide this clearance, the end of the angle may be cut as noted by the optional cut in Figure 8-12 to allow the necessary angle. However, this secondary cut will increase the cost of fabricating the connection.

Excessive Welding

The specification of over or excessive welding will increase the amount of heat input into the parts joined and thereby add to distortion in the joint. Distortion of the joint is caused by three fundamental dimensional changes that occur during and after welding:

1. Transverse shrinkage that occurs perpendicular to the weld line,
2. Longitudinal shrinkage that occurs parallel to the weld line, and
3. Angular change that consists of rotation around the weld line.

If these dimensional changes alter the joint so that it is no longer within fabrication tolerances, the joint may need to be repaired with additional heating to bring the joint back to within fabrication tolerances. This added work will result in expensive repair costs which could have been avoided with appropriately sized welds.

Over-specification of weld size also increases the cost of welding for no structural benefit.

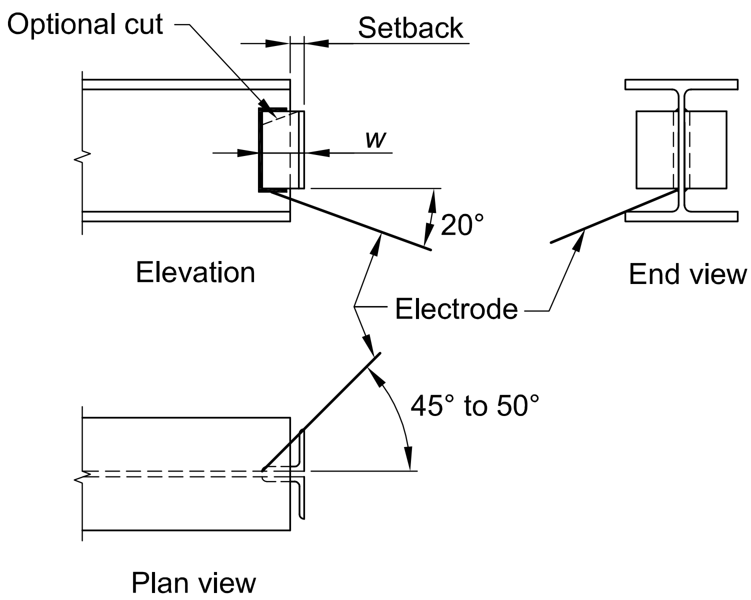


Fig. 8-12. Clearances for SMAW welding.

Minimum Shelf Dimensions for Fillet Welds

The recommended minimum shelf dimensions for normal size SMAW fillet welds are summarized in Figure 8-13. SAW fillet welds would require a greater shelf dimension to contain the flux, although auxiliary material can be clamped to the member to provide for this. The dimension *b* illustrated in Figure 8-14 must be sufficient to accommodate the combined dimensional variations of the angle length, cope depth, beam depth and weld size.

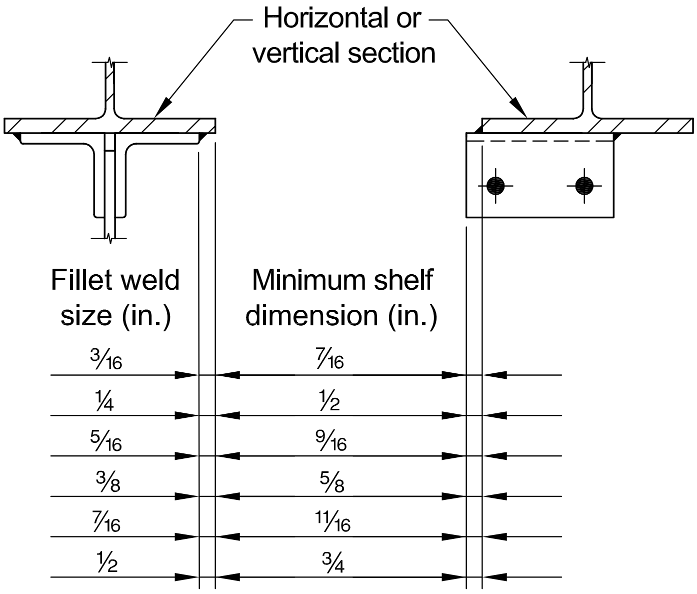


Fig. 8-13. Recommended minimum shelf dimensions for SMAW fillet welds.

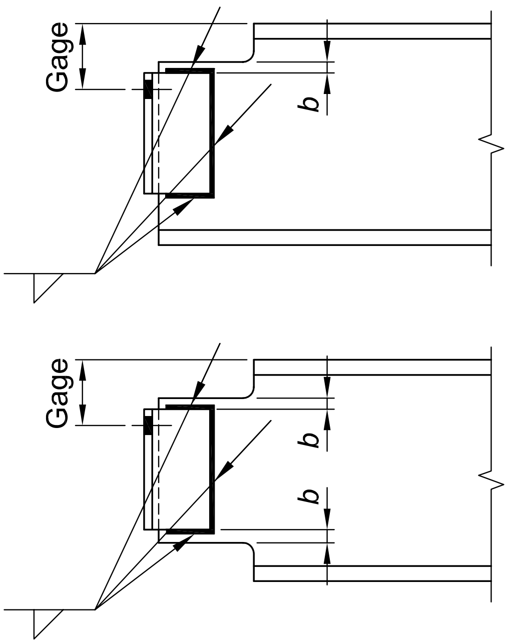


Fig. 8-14. Illustration of shelf dimensions for fillet welding.

Beam Copes and Weld Access Holes

Requirements for beam copes and weld access holes are given in AISC *Specification* Sections J1.6 and M2.2. Weld access holes, as illustrated in Figure 8-15, are used to permit down-hand welding to the beam bottom flange, as well as the placement of a continuous backing bar under the beam top flange. Weld access holes also help to mitigate the effects of weld shrinkage strains and prevent the intersection or close juncture of welds in orthogonal directions. Weld access holes should not be filled with weld metal because doing so may result in a state of triaxial stress under loading.

Corner Clips

Corners of stiffeners and similar elements that fit into a corner should be clipped generously to avoid the lack of fusion that would likely result in that corner. In general, a 3/4-in. clip will be adequate, although this dimension can be adjusted to suit conditions, such as when the

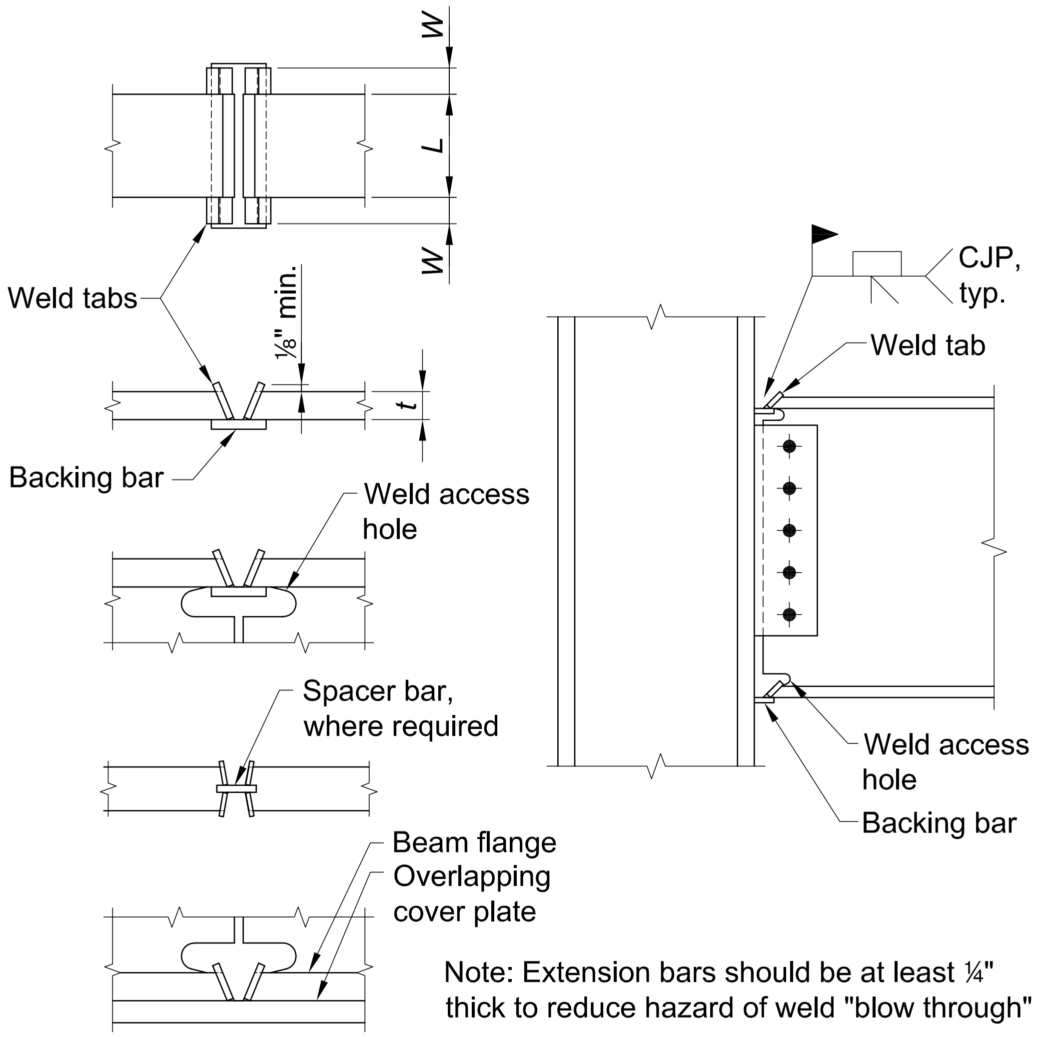


Fig. 8-15. Illustration of backing bars, spacer bars, weld tabs and other fittings for welding.

fillet radius is larger or smaller than that for which a 3/4-in. clip is appropriate. For further information, see Butler et al. (1972) and Blodgett (1980).

Corner clips of the sizes mentioned typically do not affect the available strength of gusset plates, except where these occur at or near a critical section. When this occurs, rupture or block shear limit states can be evaluated using the appropriate AISC *Specification* equations. However, corner clips of column stiffeners or continuity plates and similar stiffening elements should be included in the strength calculations because they can be of a significant size relative to the proportions of the plates.

Backing Bars

Backing bars, illustrated in Figure 8-15, should be of approved weldable material as specified in AWS D1.1 clause 5.2.2.2. Per AWS D1.1, backing bars on groove-welded joints are usually continuous or fully spliced to avoid stress concentrations or discontinuities and should be thoroughly fused with the weld metal.

Spacer Bars

Spacer bars, illustrated in Figure 8-15, must be of the same material specification as the base metal, per AWS D1.1 clause 5.2.2.3. This can create a procurement problem, since small tonnage requirements may make them difficult to obtain in the specified ASTM designation.

Weld Tabs

To obtain a fully welded cross section, the termination at either end of the joint must be of sound weld metal. Weld tabs, illustrated in Figure 8-15, should be of approved weldable material as specified in AWS D1.1 clause 5.2.2.1. Two configurations of weld tabs are illustrated in Figure 8-16, including flat-type weld tabs, which are normally used with bevel and V groove welds, and contour-type weld tabs, which are normally used with J and U groove welds. Weld-tab removal is addressed in AWS D1.1. Frequently, the backing bar can be extended to serve as the weld tab. Some welds performed in the horizontal position require shelf bars. Shelf bars will be left in place unless they are required to be removed by the engineer.

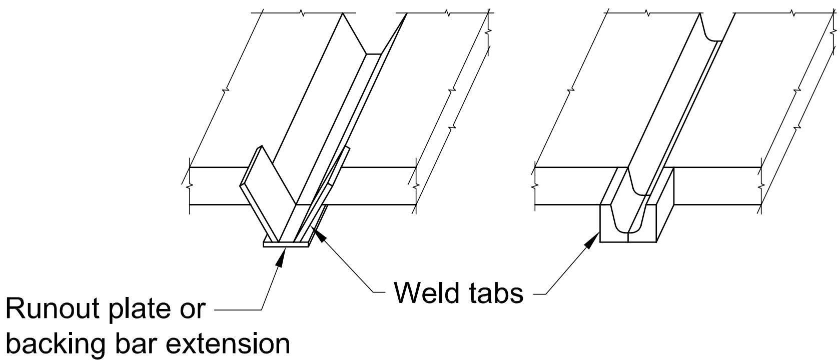


Fig. 8-16. Illustration of weld tabs.

Lamellar Tearing

Figures 8-17 and 8-18 illustrate preferred welded joint selection and connection configurations for avoiding susceptibility to lamellar tearing. Refer to the discussion "Avoiding Lamellar Tearing" in Part 2.

Prior Qualification of Welding Procedures

Evidence of prior qualification of welding procedures, welders, welding operators or tackers may be accepted at the discretion of the owner's designated representative for design, resulting in significant cost savings. Fabricators that participate in the AISC Quality Certification Program have the experience and documentation necessary to assure that such prior qualifications could be accepted. For more information about the AISC Quality Certification Program, visit **www.aisc.org**.

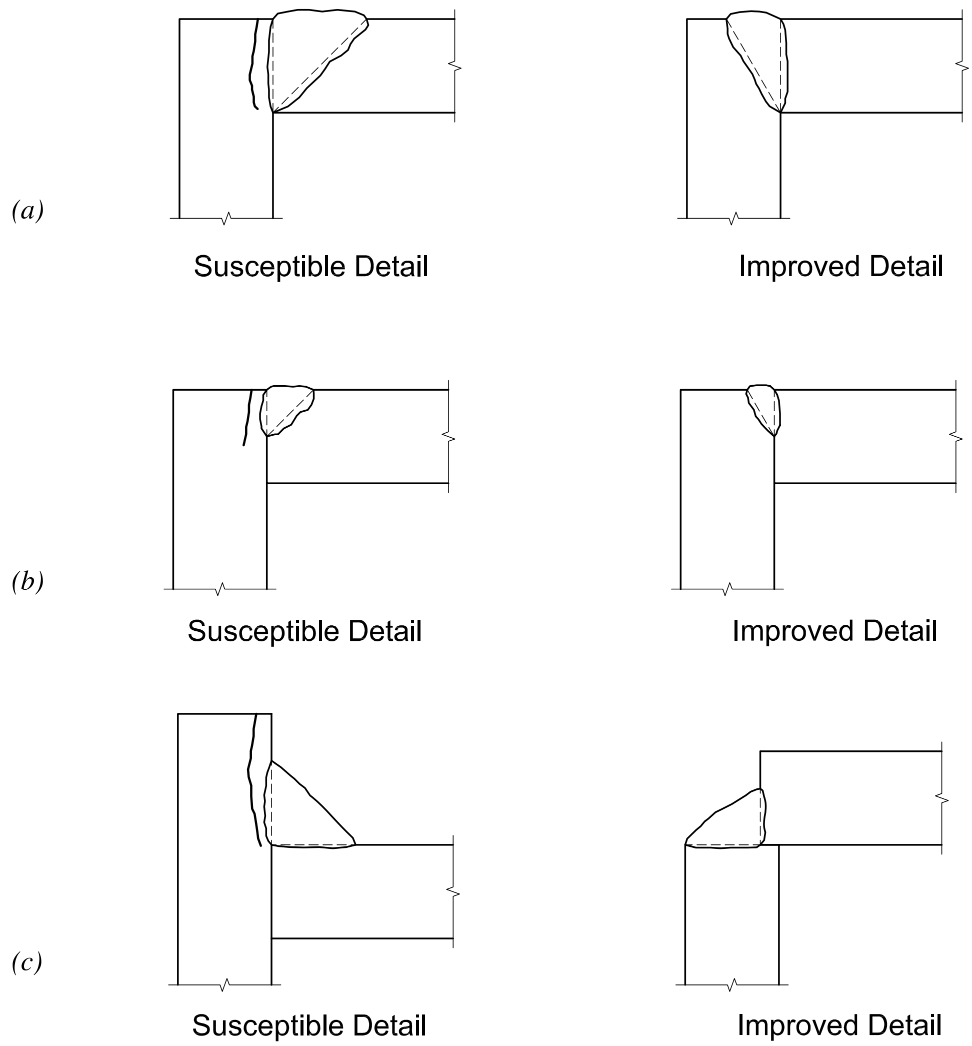


Fig. 8-17. Susceptible and improved details to reduce the incidence of lamellar tearing.

Painting Welded Connections

Paint is normally omitted in areas to be field-welded, per AISC *Specification* Section M3.5. Note that this requirement does not generally apply to shop-assembled connections, because painting is normally done after the welds are made. When required, the small paint-free areas can generally be identified with a general note (e.g., "no paint on OSL of connection angles," where OSL stands for outstanding leg).

WELDING CONSIDERATIONS FOR HSS

Flare welds are more common in HSS because of the increasing likelihood that the HSS corner is a part of the welded joint. A common flare bevel configuration that occurs when equal width sections are joined is illustrated in Figure 8-19. The easiest arrangement for welding occurs with equal wall thickness sections. However, when the corner radius increases due to wall thickness or manufacturing tolerances, the root gap may need to be adjusted by profile shaping, building out with weld metal, or by use of backing. See Figures 8-19 and 8-20.

HSS Welding Requirements in AWS D1.1

AWS uses the terminology "tubular" for all hollow members including pipe, hollow structural sections, and fabricated box sections. The following sections in AWS D1.1 apply to welded HSS-to-HSS connections:

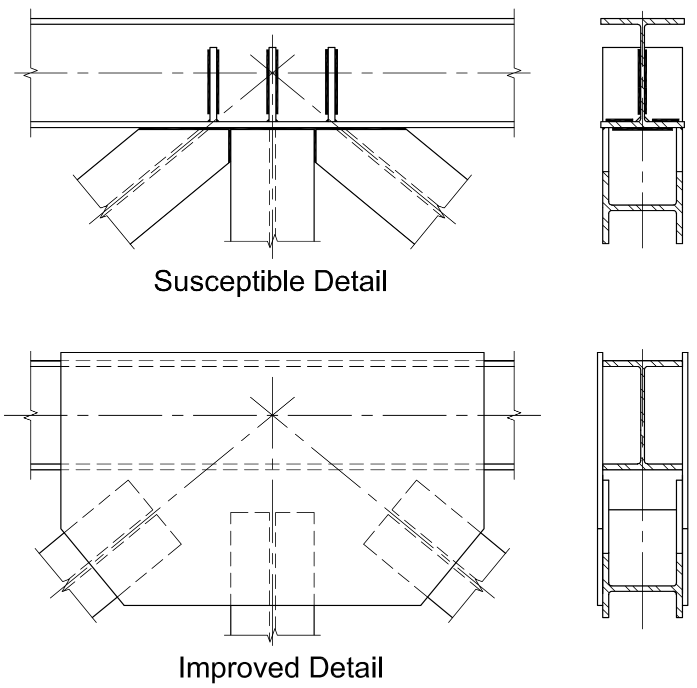


Fig. 8-18. Susceptible and improved details to avoid intersecting welds with high restraint.

Clause 9, Part A

As explained in AWS D1.1 Commentary Section C-9.2, "In commonly used types of tubular connections, the weld itself may not be the factor limiting the capacity of the joint. Such limitations as local failure (punching shear), general collapse of the main member, and lamellar tearing are discussed because they are not adequately covered in other codes." Because of these various failure modes, the design of HSS-to-HSS connections must be part of the member sizing process. The members selected must be capable of transmitting the required strength or adequate reinforcement must be shown on the design documents.

Differences in the relative stiffness across HSS walls loaded normal to their surface can make the load transfer highly nonuniform. To prevent progressive failure and to ensure ductile behavior of the joint, minimum welds must be provided in T-, Y- and K-connections to transmit the factored load in the branch or web member. For normal building applications, fillet welds and PJP welds can be used.

While clause 9, Part A, deals primarily with design of HSS-to-HSS connections, some of these provisions are applicable to welded attachments that deliver a load normal to the wall of a tubular member.

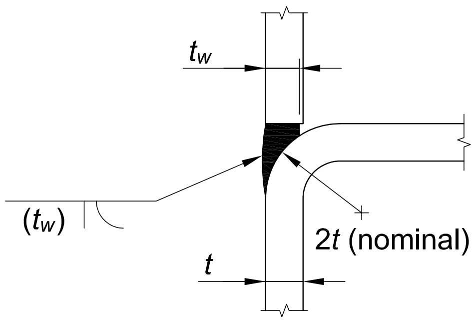


Fig. 8-19. Flare bevel weld, equal width HSS weld joint.

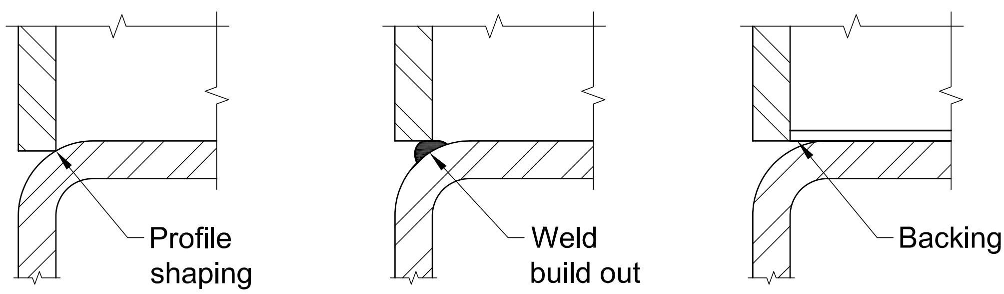


Fig. 8-20. Welding methods accounting for the HSS corner radius.

Clause 9, Part B

AWS D1.1 Figure 9.10 shows prequalified fillet weld details for tubular joints that differ from details for nontubular skewed T-joints. These details will provide the minimum weld strength needed to ensure ductile joint behavior.

AWS D1.1 Figure 3.2 shows the joint detail and the effective throat for a flare-bevel and flare-V PJP groove weld that is commonly used for welding connection material to the face of an HSS. Groove welded joint details for HSS are designed to accommodate both the geometry of the section and the lack of access to the back side of the joint.

AWS D1.1 Figure 9.11 shows various PJP groove welded HSS joint details and AWS D1.1 Figures 9.12, 9.14, 9.15 and 9.16 show CJP groove welded HSS joint details. The joint preparation and weld sizing are complex and critical to obtain a sound weld. These details also provide the weld strength needed to ensure ductile joint behavior.

Clause 9, Parts C and D

AWS D1.1 clause 9, Part C, WPS Qualification, covers the requirements for qualification testing of welding procedure specifications and Part D covers performance testing of the welder's ability to produce sound welds. HSS connections may not always meet the requirements for a prequalified WPS because of unique geometry, connection access or for other reasons. This section also gives the requirements for a procedure qualification record (PQR), which is the basis for qualifying a WPS.

The performance testing of welders and welding operators considers process, material thickness, position, nontubular or tubular joint access. AWS D1.1 Tables 4.10 through 4.12 and Tables 9.13 and 9.14 list the required qualifications needed for each type of joint. Most welders are qualified for a particular process and position-in-plate (nontubular) joints. These qualifications will allow the welder to make similar fillet, PJP groove and backed CJP welds in very large tubular members. However, certain types of tubular connections, such as unbacked T-, Y- and K-connections, require special welder certifications because the lack of access to the back of the joint, the position of the connection, and the access to the connection require special skill to produce a sound connection.

Clause 9, Part E

Clause 9, Part E, Fabrication, covers the requirements for the preparation, assembly and workmanship of welded hollow structural sections (HSS). AWS D1.1 Table 9.15, Tubular Root Opening Tolerances, gives the acceptable fitup for unbacked groove welds. AWS D1.1 Table 5.7, Minimum Fillet Weld Size, gives the minimum weld pass size based on material thickness and process.

Clause 9, Part F

Clause 9, Part F, Inspection, contains all of the requirements for the inspector's qualifications and responsibilities, acceptance criteria for discontinuities, and procedures for NDE. AWS D1.1 considers fabrication/erection inspection and testing a separate function from verification inspection and testing. Fabrication/erection inspection and testing is usually the responsibility of the contractor and is performed as appropriate prior to assembly, during assembly, during welding, and after welding to ensure the requirements of the contract documents are met. Verification inspection and testing are the prerogatives of the owner. The extent of NDE and verification inspection must be specified in the contract documents.

The inspection covers WPS qualification, equipment, welder qualification, joint preparation, joint fitup, welding techniques, and weld size length and location. It is especially important when inspecting HSS-to-HSS joints that joint preparation and fitup be checked prior to welding.

In addition to inspecting the above items, AWS requires all welds to be visually inspected for conformance to the standards in AWS D1.1 Table 9.16, Visual Inspection Acceptance Criteria.

Four types of nondestructive testing can be used to supplement visual inspection. They are penetrant testing, magnetic particle testing, radiographic testing, and ultrasonic testing.

The AWS ultrasonic testing (UT) acceptance criteria for non-HSS type groove welds starts at $^5/_{16}$-in.-thick material. The procedures for HSS T-, Y- and K- connections have a minimum applicable thickness of $^1/_2$ in., and diameter of $12^3/_4$ in. AWS does, however, make provision for qualifying UT procedures for smaller size applications. It is possible to UT portions of butt-type splices with backing bars using the non-HSS criteria, however, the corners of rectangular HSS cannot be inspected.

AWS D1.1 makes provision for using alternate acceptance criteria based upon an evaluation of suitability for service using past experience, experimental evidence or engineering analysis. This can be especially important when deciding if and how to make any repairs.

Weld Sizing for Uneven Distribution of Loads

The connection strength for a member welded normal to an HSS wall is a function of the geometric parameters of the connected members and is often less than the full strength of the member. When limited by geometry, the available strength cannot be increased by increasing the weld strength. Due to the varying relative flexibility of the HSS wall loaded normal to its surface and the axial stiffness of the connected member, the transfer of load along the weld line is highly nonuniform. To prevent progressive failure, or “unzipping” of the weld, it is important to provide adequate welds to maintain ductile behavior of the joint.

Welds that satisfy this ductility requirement can be proportioned for the required strength using an effective width criteria similar to that used for checking the axial strength of the branch member or plate. For effective weld length of HSS-to-HSS connections, refer to AISC *Specification* Section K5.

An alternative to the effective length procedure is the use of the prequalified fillet and PJP groove weld details in AWS D1.1 that are sized to ensure ductile behavior. In addition, fillet welds with an effective throat of 1.1 times the thickness of the branch member can be used. Either of these two alternatives will, in most cases, be conservative.

Detailing Considerations

1. Butt joints will require a groove weld detail. Where possible, the joint should be a prequalified PJP groove weld sized for actual load or a CJP groove weld with steel backing.
2. T-, Y- and K-connections should, where possible, use either fillet welds or PJP groove welds sized for the design forces and checked for the minimum size needed to ensure ductile joint behavior. Where CJP welds are required, joint details using steel backing should be used whenever possible. For a detailed discussion of various types of backing and the advantages of using backing, see Post (1990).

DESIGN TABLE DISCUSSION

Table 8-1. Coefficients, *C*, for Concentrically Loaded Weld Group Elements

Concentrically loaded fillet weld groups must consider the effect of loading angle and deformation compatibility on weld strength.

By multiplying the appropriate values of C from Table 8-1 by the available strength of each weld element, an effective strength is determined for each weld element. The available strength of the weld group can be determined by summing the effective strengths of all of the elements in a weld group. It should be noted that this table is to be entered at the largest load angle on any weld in the weld group. For the weld group shown in Figure 8-21, this is calculated as:

LRFD	ASD
$\phi R_w = 1.392D \times [1.50(1) + 1.29(1.41) + 0.825(1)] = 5.77D$ (8-20a)	$R_w/\Omega = 0.928D \times [1.50(1) + 1.29(1.41) + 0.825(1)] = 3.85D$ (8-20b)

Table 8-2. Prequalified Welded Joints

The prequalified welded joints details given in AWS D1.1 and Table 8-2 provide joint geometries, such as root openings, angles and clearances (see Figures 8-22 and 8-23) that will permit the deposition of sound weld material. Prequalified welded joints are not, in themselves, adequate consideration of welded design details and the other provisions in AWS D1.1 must be satisfied as they are referenced in AISC *Specification* Section J2. The design and detailing for successful welded construction requires consideration of factors which include, but are not limited to, the magnitude, type and distribution of forces to be transmitted, access, restraint against weld shrinkage, thickness of connected materials, residual stress, and distortion. AWS D1.1 has provisions for material that is thinner than is normally considered applicable for structural applications. See AWS D1.1 and D1.3 (AWS, 2008) for welding requirements and limits applicable to these materials in lieu of provisions such as AISC *Specification* Table J2.3.

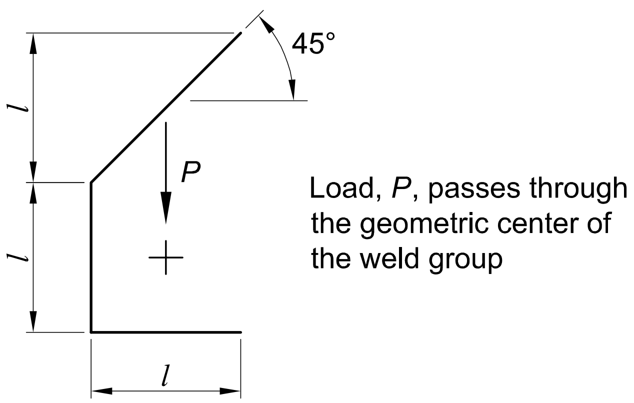


Fig. 8-21. Concentrically loaded weld group.

The designations such as B-L1a, B-U2 and B-P3 are those used in AWS D1.1. Note that lowercase letters (e.g., a, b, c, etc.) are often used to differentiate between joints that would otherwise have the same joint designation. These prequalified welded joints are limited to those made by the SMAW, SAW, GMAW (except short circuit transfer), and FCAW procedures. Small deviations from dimensions, angles of grooves, and variation in depth of groove joints are permissible within the tolerances given.

In general, all fillet welds are prequalified, provided they conform to the requirements in AWS D1.1. Groove welds are classified using the conventions indicated in the tables. Welded joints other than those prequalified by AWS may be qualified, provided they are tested and qualified in accordance with AWS D1.1.

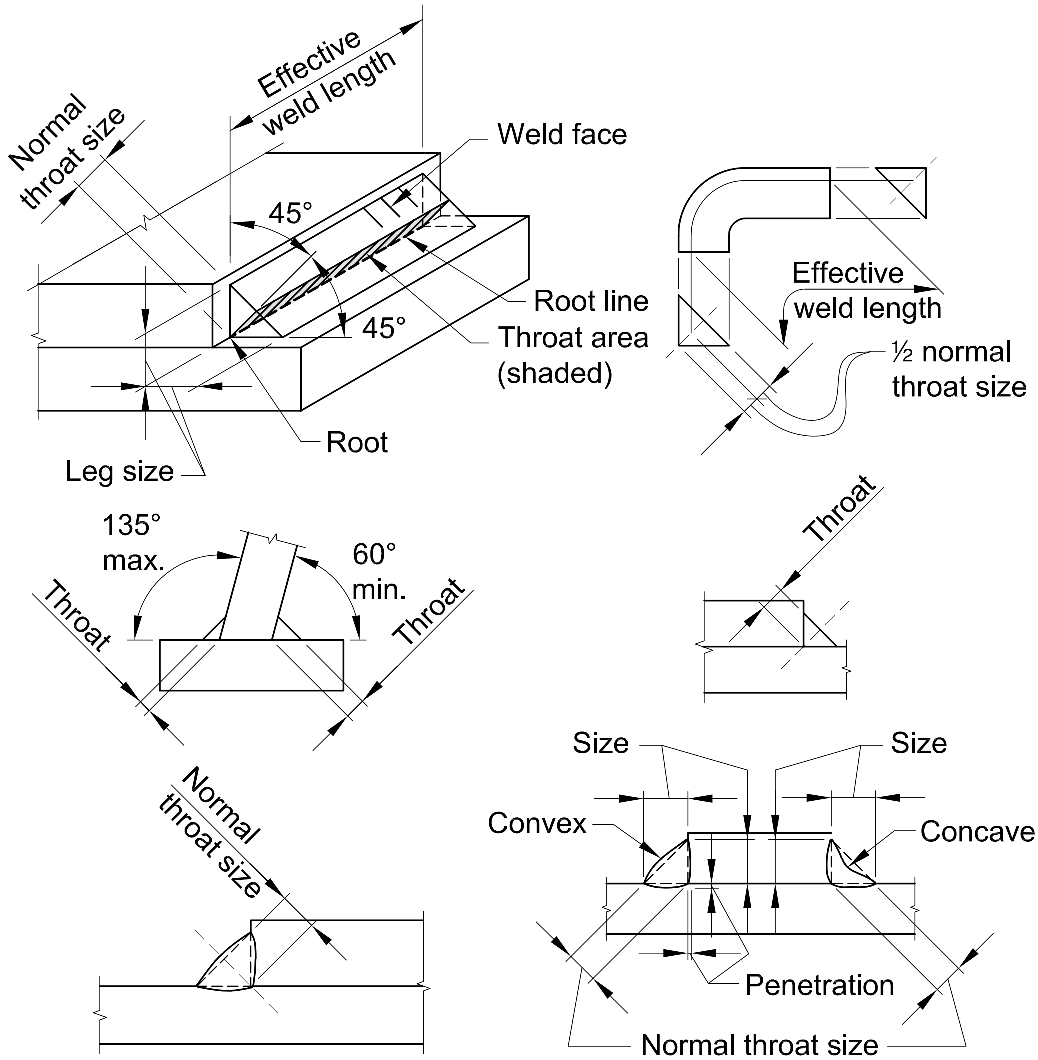


Fig. 8-22. Fillet weld nomenclature.

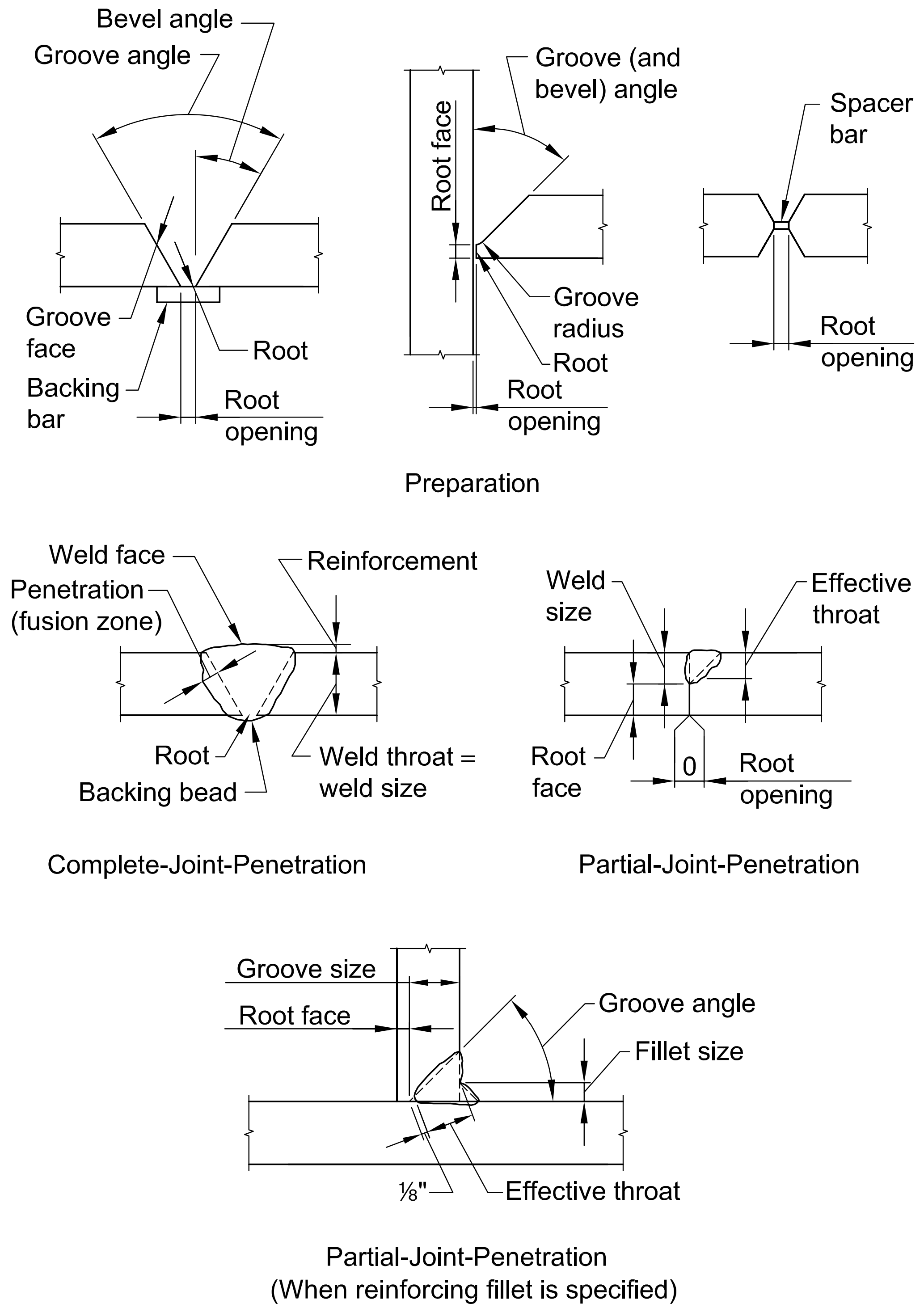


Fig. 8-23. Groove weld nomenclature.

Table 8-3. Electrode Strength Coefficient, C_1

Electrode strength coefficients, C_1, which can be used to adjust the tabulated values of Tables 8-4 through 8-11 for electrodes other than E70XX, are given in Table 8-3. Note that this coefficient includes an additional reduction factor of 0.90 for E80 and E90 electrodes and 0.85 for E100 and E110; this accounts for the uncertainty of extrapolation to these higher-strength electrodes.

Tables 8-4 through 8-11. Coefficients, *C*, for Eccentrically Loaded Weld Groups

Tables 8-4 through 8-11 employ the instantaneous center of rotation method, as discussed earlier in this Part, for the weld patterns and eccentric conditions indicated and inclined loads at 0°, 15°, 30°, 45°, 60° and 75°. The tabulated nondimensional coefficient, *C*, represents the effective strength of the weld group in resisting the eccentric shear force.

When Analyzing a Known Weld Group Geometry

For any of the weld group geometries shown, the available strength, ϕR_n or R_n/Ω, of the eccentrically loaded weld group is determined by

$$R_n = CC_1Dl \tag{8-21}$$

$$\phi = 0.75 \qquad \Omega = 2.00$$

where

C = tabular value

C_1 = electrode strength coefficient from Table 8-3

D = number of sixteenths-of-an-inch in the fillet weld size

l = length of the reference weld, in.

In developing these tables, the instantaneous center of rotation method was used, with a convergence criterion of less than $^1\!/_2$% and considering deformation compatibility of adjacent weld elements. The first row in each table (a = 0) gives the available strength of a concentrically loaded weld group in accordance with AISC *Specification* Section J2.4. Linear interpolation within a given table between adjacent a and k values is permitted.

Straight-line interpolation between values for loads at different angles may be significantly unconservative. Either a rational analysis should be performed or the values for the next lower angle increment in the tables should be used for design. For weld group patterns not treated in these tables, a rational analysis is required.

Table 8-12. Approximate Number of Passes for Welds

Table 8-12 lists the approximate number of passes required for various welds. The actual number of passes can vary depending on the welding position and process used. The table can be used as a guide in selecting economical welds because the labor required will be roughly proportional to the number of passes. Longer single-pass welds will generally be more economical than shorter multi-pass welds because the number of passes, and therefore the cost, required to deposit the larger multi-pass weld increases faster than the strength of the weld.

PART 8 REFERENCES

AWS (2008), *Structural Welding Code—Sheet Steel*, AWS D1.3/D1.3M:2008, American Welding Society, Miami, FL.

AWS (2009), *Guide for the Nondestructive Inspection of Welds*, AWS B1.10, American Welding Society, Miami, FL.

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Post, J.W. (1990), "Box-Tube Connections: Choices of Joint Details and Their Influence on Costs," *National Steel Construction Conference Proceedings*, AISC, Chicago, IL.

Table 8-1
Coefficients, *C*, for Concentrically Loaded Weld Group Elements

Load angle on weld element, degrees	Largest load angle on any weld group element, degrees						
	90	75	60	45	30	15	0
0	0.825	0.849	0.876	0.909	0.948	0.994	1.00
15	1.02	1.04	1.05	1.07	1.06	0.883	
30	1.16	1.17	1.18	1.17	1.10		
45	1.29	1.30	1.29	1.26			
60	1.40	1.40	1.39				
75	1.48	1.47					
90	1.50						

Table 8-2
Prequalified Welded Joints

Symbols for Joint Types

B	butt joint	BC	butt or corner joint
C	corner joint	TC	T- or corner joint
T	T-joint	BTC	butt, T- or corner joint

Symbols for Base Metal Thickness and Penetration

L	limited thickness, complete-joint-penetration
U	unlimited thickness, complete-joint-penetration
P	partial-joint-penetration

Symbols for Weld Types

1	square-groove	6	single-U-groove	11	flare-V-groove
2	single-V-groove	7	double-U-groove		
3	double-V-groove	8	single-J-groove		
4	single-bevel-groove	9	double-J-groove		
5	double-bevel-groove	10	flare-bevel-groove		

Symbols for Welding Processes if not Shielded Metal Arc Welding (SMAW)

S	submerged arc welding (SAW)
G	gas metal arc welding (GMAW)
F	flux cored arc welding (FCAW)

Symbols for Welding Positions

F	flat	V	vertical
H	horizontal	OH	overhead

Symbols for Joint Designation

The lower case letters (e.g., a, b, c, d, etc.) are used to differentiate between joints that would otherwise have the same joint designation.

Symbols for Dimensions

R	root opening	r	J- or U-groove radius
α, β	groove angles	S, S_1, S_2	PJP groove weld depth of groove
f	root face	E, E_1, E_2	PJP groove weld sizes corresponding to S, S_1, S_2, respectively

Notes to Prequalified Welded Joints

a Not prequalified for gas metal arc welding (GMAW) using short circuiting transfer nor GTAW.
b Joint is welded from one side only.
c Cyclic load application limits these joints to the horizontal welding position. Refer to AWS D1.1 clause 2.18.2.
d Backgouge root to sound metal before welding second side.
e SMAW joints may be used for prequalified GMAW (except GMAW-S) and FCAW.
f Minimum effective throat thickness (E) as shown in AISC *Specification* Table J2.3; S as specified on drawings.
g If fillet welds are used in statically loaded structures to reinforce groove welds in corner and T-joints, they shall be equal to $^1/_4$ T_1, but need not exceed $^3/_8$ in. Groove welds in corner and T-joints of cyclically loaded structures shall be reinforced with fillet welds equal to $^1/_4$ T_1, but need not exceed $^3/_8$ in.
h Double-groove welds may have grooves of unequal depth, but the depth of the shallower groove shall be no less than one-fourth of the thickness of the thinner part joined.
i Double-groove welds may have grooves of unequal depth, provided these conform to the limitations of Note f. Also, the effective throat thickness (E) applies individually to each groove.
j The orientation of the two members in the joints may vary from 135° to 180° for butt joints, or 45° to 135° for corner joints, or 45° to 90° for T-joints.
k For corner joints, the outside groove preparation may be in either or both members, provided the basic groove configuration is not changed and adequate edge distance is maintained to support the welding operations without excessive edge melting.
l Effective throat thickness (E) is based on joints welded flush.
m For flare-V-groove welds and flare-bevel-groove welds to rectangular tubular sections, r shall be taken as two times the wall thickness.
n For flare-V-groove welds to surfaces with different radii r, the smaller r shall be used.
o For corner and T-joints, the member orientation may vary from 90° to less than or equal to 170° provided the groove angle and root opening are maintained and the angle between the groove faces and the steel backing is at least 90°. See AWS D1.1 Figure 3.6.

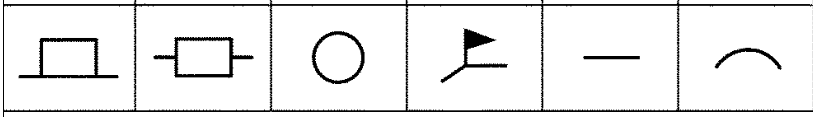
Table 8-2 (continued)
Prequalified Welded Joints

Basic Weld Symbols

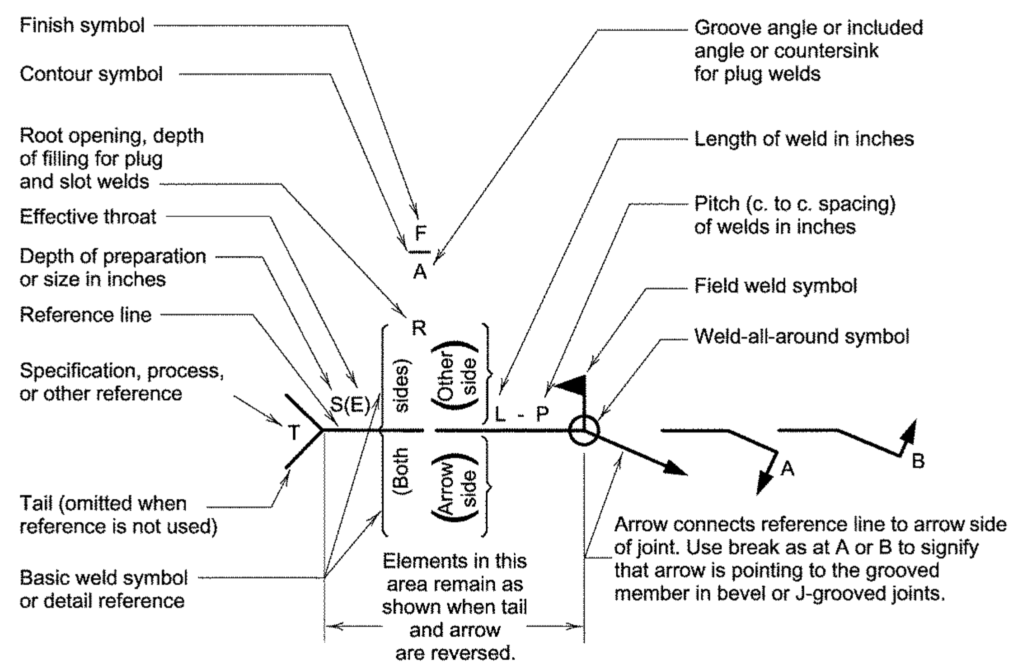
Back	Fillet	Plug or Slot	Groove or Butt						
			Square	V	Bevel	U	J	Flare V	Flare Bevel

Supplementary Weld Symbols

Backing	Spacer	Weld All Around	Field Weld	Contour		For other basic and supplementary weld symbols, see AWS A2.4 (2007).
				Flush	Convex	



Standard Location of Elements of a Welding Symbol



Note:

Size, weld symbol, length of weld, and spacing must read in that order, from left to right, along the reference line. Neither orientation of reference nor location of the arrow alters this rule.

The perpendicular leg of fillet, bevel, J and flare bevel weld symbols must be at left.

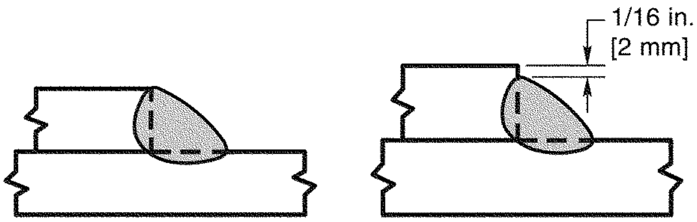
Dimensions of fillet welds must be shown on both the arrow side and the other side.

Symbols apply between abrupt changes in direction of welding unless governed by the "all around" symbol or otherwise dimensioned.

These symbols do not explicitly provide for the case that frequently occurs in structural work, where duplicate material (such as stiffeners) occurs on the far side of a web or gusset plate. The fabricating industry has adopted this convention: that when the billing of the detail material discloses the existence of a member on the far side as well as on the near side, the welding shown for the near side shall be duplicated on the far side.

Table 8-2 (continued)
Prequalified Welded Joints Fillet Welds

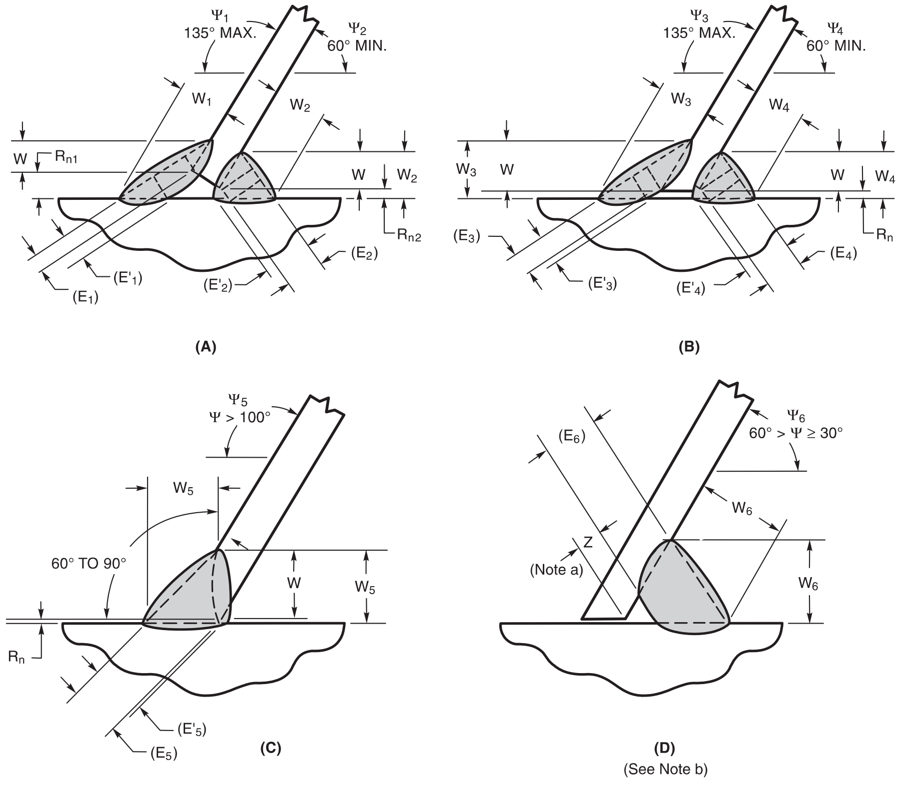
FILLET



BASE METAL LESS THAN 1/4 in. [6 mm] THICK
(A)

BASE METAL 1/4 in. [6 mm] OR MORE IN THICKNESS
(B)

MAXIMUM DETAILED SIZE OF FILLET WELD ALONG EDGES



[a] Detail (D). Apply Z loss dimension of Table 2.2 to determine effective throat.
[b] Detail (D) shall not be prequalified for under 30°. For welder qualifications, see Table 4.10.

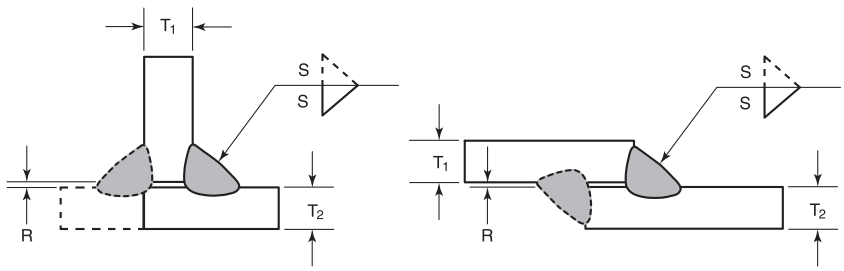
Notes:
1. (E_n), (E'_n) = Effective throats dependent on magnitude of root opening (R_n) (see 5.21.1). (n) represents 1 through 5.
2. t = thickness of thinner part.
3. Not prequalified for GMAW-S or GTAW

Note: Referenced clauses and tables in this figure are from AWS D1.1.

FILLET

Table 8-2 (continued)
Prequalified Welded Joints
Fillet Welds

Fillet weld (12)
T-joint (T)
Corner joint (C)
Lap joint (L)



ALL DIMENSIONS IN inches

Welding Process	Joint Designation	Base Metal Thickness	Joint Design/Geometry			Allowed Welding Positions	Notes
			Root Opening	Tolerances			
		T_1 or T_2		As Detailed	As Fit-Up		
SMAW	TC-F12	<3	R = 0	$+^{1}/_{16}, -0$	$^{3}/_{16}$ max.	All	a′, b′, d′
	TC-F12a	≥3			$^{5}/_{16}$ max.		a′, b′, d′
	L-F12	<3			$^{3}/_{16}$ max.		a′, b′, c′
	L-F12a	≥3			$^{5}/_{16}$ max.		a′, b′, c′
GMAW FCAW	TC-F12-GF	<3	R = 0	$+^{1}/_{16}, -0$	$^{3}/_{16}$ max.	All	a′, b′, d′
	TC-F12a-GF	≥3			$^{5}/_{16}$ max.		a′, b′, d′
	L-F12-GF	<3			$^{3}/_{16}$ max.		a′, b′, c′
	L-F12a-GF	≥3			$^{5}/_{16}$ max.		a′, b′, c′
SAW	TC-F12-S	<3	R = 0	$+^{1}/_{16}, -0$	$^{3}/_{16}$ max.	F, H	a′, b′, d′
	TC-F12a-S	≥3			$^{5}/_{16}$ max.		a′, b′, d′
	L-F12-S	<3			$^{3}/_{16}$ max.		a′, b′, c′
	L-F12a-S	≥3			$^{5}/_{16}$ max.		a′, b′, c′

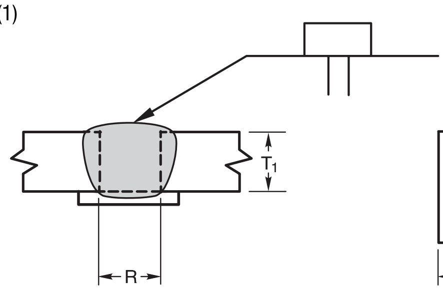
[a′] Fillet weld size (“S”). See 2.4.2.8 and Clause 5.13 for minimum fillet weld sizes. See Table 3.6 for maximum single pass size.
[b′] See 5.21.1 for additional fillet weld assembly requirements or exceptions.
[c′] See 2.4.2.9 for maximum weld size in lap joints.
[d′] Perpendicularity of the members shall be within ±10°.

Note: Referenced clauses and tables in this table are from AWS D1.1.

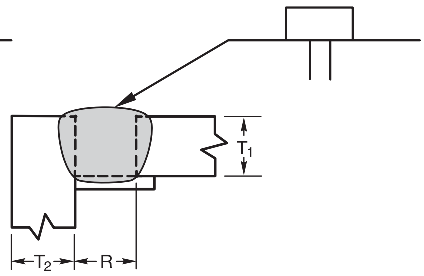
CJP

Table 8-2 (continued)
Prequalified Welded Joints
Complete-Joint-Penetration Groove Welds

Square-groove weld (1)
Butt joint (B)
Corner joint (C)



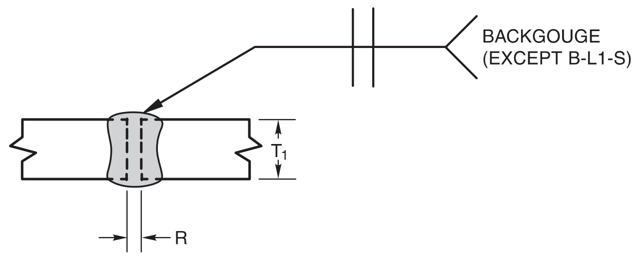
B-L1a



C-L1a

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited) T_1	Base Metal Thickness (U = unlimited) T_2	Groove Preparation: Root Opening	Groove Preparation: Tolerances, As Detailed (see 3.13.1)	Groove Preparation: Tolerances, As Fit-Up (see 3.13.1)	Allowed Welding Positions	Gas Shielding for FCAW	Notes
SMAW	B-L1a	1/4 max.	—	$R = T_1$	+1/16, −0	+1/4, −1/16	All	—	e, j
	C-L1a	1/4 max.	U	$R = T_1$	+1/16, −0	+1/4, −1/16	All	—	e, j
FCAW GMAW	B-L1a-GF	3/8 max.	—	$R = T_1$	+1/16, −0	+1/4, −1/16	All	Not Required	a, j

Square-groove weld (1)
Butt joint (B)



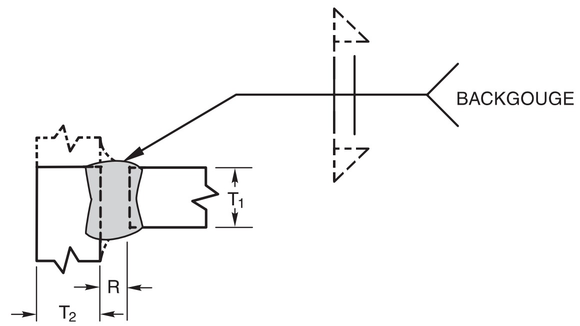
Welding Process	Joint Designation	Base Metal Thickness (U = unlimited) T_1	Base Metal Thickness (U = unlimited) T_2	Groove Preparation: Root Opening	Groove Preparation: Tolerances, As Detailed (see 3.13.1)	Groove Preparation: Tolerances, As Fit-Up (see 3.13.1)	Allowed Welding Positions	Gas Shielding for FCAW	Notes
SMAW	B-L1b	1/4 max.	—	$R = \frac{T_1}{2}$	+1/16, −0	+1/16, −1/8	All	—	d, e, j
GMAW FCAW	B-L1b-GF	3/8 max.	—	R = 0 to 1/8	+1/16, −0	+1/16, −1/8	All	Not Required	a, d, j
SAW	B-L1-S	3/8 max.	—	R = 0	±0	+1/16, −0	F	—	j
SAW	B-L1a-S	5/8 max.	—	R = 0	±0	+1/16, −0	F	—	d, j

Note: Referenced clauses in this table are from AWS D1.1. "Notes" on this page refer to Table 8-2, pg. 8-34, of this manual.

CJP

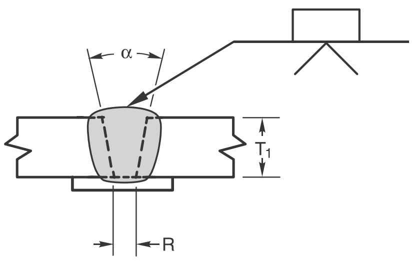
Table 8-2 (continued)
Prequalified Welded Joints
Complete-Joint-Penetration Groove Welds

Square-groove weld (1)
T-joint (T)
Corner joint (C)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
				Root Opening	Tolerances				
		T_1	T_2		As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)			
SMAW	TC-L1b	$^1/_4$ max.	U	$R = \frac{T_1}{2}$	$+^1/_{16}, -0$	$+^1/_{16}, -^1/_8$	All	—	d, e, g
GMAW FCAW	TC-L1-GF	$^3/_8$ max.	U	R = 0 to $^1/_8$	$+^1/_{16}, -0$	$+^1/_{16}, -^1/_8$	All	Not Required	a, d, g
SAW	TC-L1-S	$^3/_8$ max.	U	R = 0	±0	$+^1/_{16}, -0$	F	—	d, g

Single-V-groove weld (2)
Butt joint (B)



Tolerances	
As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
$R = +^1/_{16}, -0$	$+^1/_4, -^1/_{16}$
$\alpha = +10°, -0°$	$+10°, -5°$

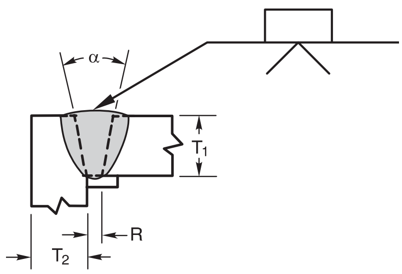
Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation		Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Groove Angle			
SMAW	B-U2a	U	—	$R = ^1/_4$	$\alpha = 45°$	All	—	e, j
				$R = ^3/_8$	$\alpha = 30°$	F, V, OH	—	e, j
				$R = ^1/_2$	$\alpha = 20°$	F, V, OH	—	e, j
GMAW FCAW	B-U2a-GF	U	—	$R = ^3/_{16}$	$\alpha = 30°$	F, V, OH	Required	a, j
				$R = ^3/_8$	$\alpha = 30°$	F, V, OH	Not req.	a, j
				$R = ^1/_4$	$\alpha = 45°$	F, V, OH	Not req.	a, j
SAW	B-L2a-S	2 max.	—	$R = ^1/_4$	$\alpha = 30°$	F	—	j
SAW	B-U2-S	U	—	$R = ^5/_8$	$\alpha = 20°$	F	—	j

Note: Referenced clauses in this table are from AWS D1.1. “Notes” on this page refer to Table 8-2, pg. 8-34, of this manual.

Table 8-2 (continued)
Prequalified Welded Joints
Complete-Joint-Penetration Groove Welds

CJP

Single-V-groove weld (2)
Corner joint (C)



Tolerances	
As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
R = +1/16, –0	+1/4, –1/16
α = +10°, –0°	+10°, –5°

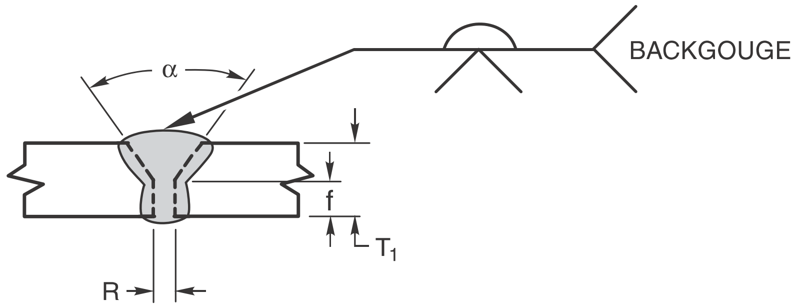
Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation		Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Groove Angle			
SMAW	C-U2a	U	U	R = 1/4	α = 45°	All	—	e, o
				R = 3/8	α = 30°	F, V, OH	—	e, o
				R = 1/2	α = 20°	F, V, OH	—	e, o
GMAW FCAW	C-U2a-GF	U	U	R = 3/16	α = 30°	F, V, OH	Required	a
				R = 3/8	α = 30°	F, V, OH	Not required	a, o
				R = 1/4	α = 45°	F, V, OH	Not required	a, o
SAW	C-L2a-S	2 max.	U	R = 1/4	α = 30°	F	—	o
SAW	C-U2-S	U	U	R = 5/8	α = 20°	F	—	o

Note: Referenced clauses in this table are from AWS D1.1. “Notes” on this page refer to Table 8-2, pg. 8-34, of this manual.

CJP

Table 8-2 (continued)
Prequalified Welded Joints
Complete-Joint-Penetration Groove Welds

Single-V-groove weld (2)
Butt joint (B)



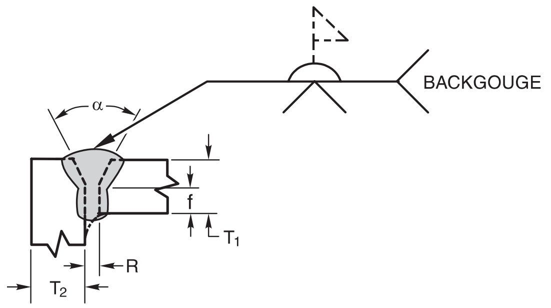
Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
				Root Opening Root Face Groove Angle	Tolerances				
		T_1	T_2		As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)			
SMAW	B-U2	U	—	R = 0 to 1/8 f = 0 to 1/8 α = 60°	+1/16, –0 +1/16, –0 +10°, –0°	+1/16, –1/8 Not Limited +10°, –5°	All	—	d, e, j
GMAW FCAW	B-U2-GF	U	—	R = 0 to 1/8 f = 0 to 1/8 α = 60°	+1/16, –0 +1/16, –0 +10°, –0°	+1/16, –1/8 Not Limited +10°, –5°	All	Not Required	a, d, j
SAW	B-L2c-S	Over 1/2 to 1	—	R = 0 f = 1/4 max. α = 60°	R = ±0 f = +0, –f α = +10°, –0°	+1/16, –0 ± 1/16 +10°, –5°	F	—	d, j
		Over 1 to 1 1/2	—	R = 0 f = 1/2 max. α = 60°					
		Over 1 1/2 to 2	—	R = 0 f = 5/8 max. α = 60°					

Note: Referenced clauses in this table are from AWS D1.1. “Notes” on this page refer to Table 8-2, pg. 8-34, of this manual.

CJP

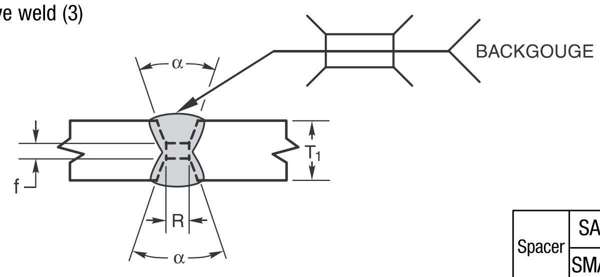
Table 8-2 (continued)
Prequalified Welded Joints
Complete-Joint-Penetration Groove Welds

Single-V-groove weld (2)
Corner joint (C)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
					Tolerances				
		T_1	T_2	Root Opening Root Face Groove Angle	As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)			
SMAW	C-U2	U	U	R = 0 to 1/8 f = 0 to 1/8 α = 60°	+1/16, –0 +1/16, –0 +10°, –0°	+1/16, –1/8 Not Limited +10°, –5°	All	—	d, e, g, j
GMAW FCAW	C-U2-GF	U	U	R = 0 to 1/8 f = 0 to 1/8 α = 60°	+1/16, –0 +1/16, –0 +10°, –0°	+1/16, –1/8 Not Limited +10°, –5°	All	Not Required	a, d, g, j
SAW	C-U2b-S	U	U	R = 0 to 1/8 f = 1/4 max. α = 60°	±0 +0, –1/4 +10°, –0°	+1/16, –0 ±1/16 +10°, –5°	F	—	d, g, j

Double-V-groove weld (3)
Butt joint (B)



		Tolerances	
		As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
		R = ±0	+1/4, –0
		f = ±0	+1/16, –0
		α = +10°, –0°	+10°, –5°
Spacer	SAW	±0	+1/16, –0
Spacer	SMAW	±0	+1/8, –0

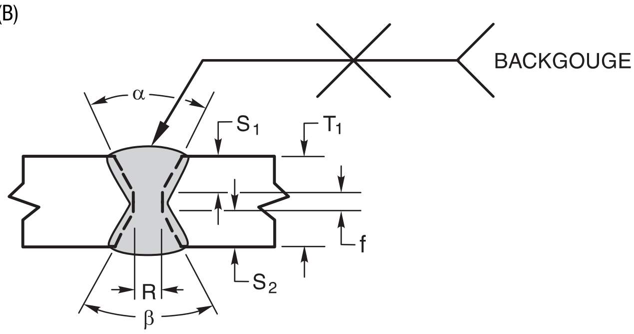
Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Root Face	Groove Angle			
SMAW	B-U3a	U Spacer = 1/8 × R	—	R = 1/4	f = 0 to 1/8	α = 45°	All	—	d, e, h, j
				R = 3/8	f = 0 to 1/8	α = 30°	F, V, OH	—	
				R = 1/2	f = 0 to 1/8	α = 20°	F, V, OH	—	
SAW	B-U3a-S	U Spacer = 1/4 × R	—	R = 5/8	f = 0 to 1/4	α = 20°	F	—	d, h, j

Note: Referenced clauses in this table are from AWS D1.1. "Notes" on this page refer to Table 8-2, pg. 8-34, of this manual.

CJP

Table 8-2 (continued)
Prequalified Welded Joints
Complete-Joint-Penetration Groove Welds

Double-V-groove weld (3)
Butt joint (B)



For B-U3c-S only		
T_1		S_1
Over	to	
2	$2^1/_2$	$1^3/_8$
$2^1/_2$	3	$1^3/_4$
3	$3^5/_8$	$2^1/_8$
$3^5/_8$	4	$2^3/_8$
4	$4^3/_4$	$2^3/_4$
$4^3/_4$	$5^1/_2$	$3^1/_4$
$5^1/_2$	$6^1/_4$	$3^3/_4$
For $T_1 > 6^1/_4$ or $T_1 \le 2$ $S_1 = {}^2/_3(T_1 - {}^1/_4)$		

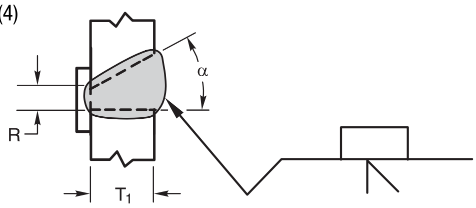
Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
					Tolerances				
		T_1	T_2	Root Opening Root Face Groove Angle	As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)			
SMAW	B-U3b	U	—	$R = 0$ to $^1/_8$ $f = 0$ to $^1/_8$ $\alpha = \beta = 60°$	$+^1/_{16}, -0$ $+^1/_{16}, -0$ $+10°, -0°$	$+^1/_{16}, -^1/_8$ Not limited $+10°, -5°$	All	—	d, e, h, j
GMAW FCAW	B-U3-GF						All	Not required	a, d, h, j
SAW	B-U3c-S	U	—	$R = 0$ $f = ^1/_4$ min. $\alpha = \beta = 60°$	$+^1/_{16}, -0$ $+^1/_4, -0$ $+10°, -0°$	$+^1/_{16}, -0$ $+^1/_4, -0$ $+10°, -5°$	F	—	d, h, j
				To find S_1 see table above: $S_2 = T_1 - (S_1+f)$					

Note: Referenced clauses in this table are from AWS D1.1. "Notes" on this page refer to Table 8-2, pg. 8-34, of this manual.

CJP

Table 8-2 (continued)
Prequalified Welded Joints
Complete-Joint-Penetration Groove Welds

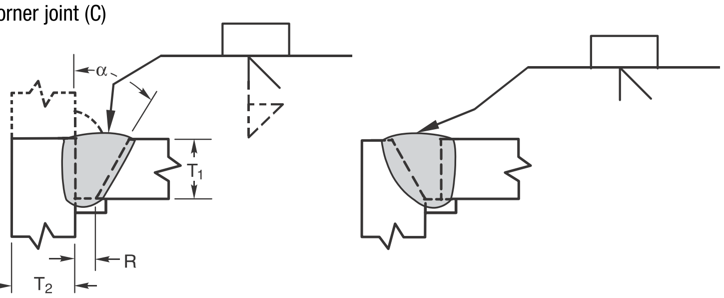
Single-bevel-groove weld (4)
Butt joint (B)



Tolerances	
As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
R = +1/16, −0	+1/4, −1/16
α = +10°, −0°	+10°, −5°

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation		Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Groove Angle			
SMAW	B-U4a	U	—	R = 1/4	α = 45°	All	—	c, e, j
				R = 3/8	α = 30°	All	—	c, e, j
GMAW FCAW	B-U4a-GF	U	—	R = 3/16	α = 30°	All	Required	a, c, j
				R = 1/4	α = 45°	All	Not required	a, c, j
				R = 3/8	α = 30°	F, H	Not required	a, c, j
SAW	B-U4a-S	U	—	R = 3/8	α = 30°	F	—	c, j
				R = 1/4	α = 45°			

Single-bevel-groove weld (4)
T-joint (T)
Corner joint (C)



Tolerances	
As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
R = +1/16, −0	+1/4, −1/16
α = +10°, −0°	+10°, −5°

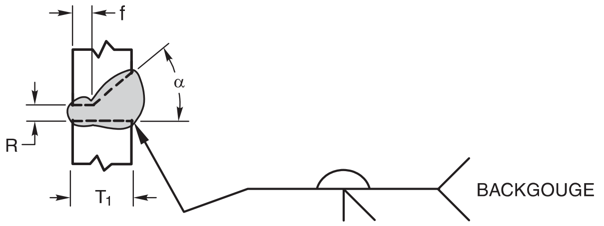
Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation		Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Groove Angle			
SMAW	TC-U4a	U	U	R = 1/4	α = 45°	All	—	e, g, k, o
				R = 3/8	α = 30°	F, V, OH	—	e, g, k, o
GMAW FCAW	TC-U4a-GF	U	U	R = 3/16	α = 30°	All	Required	a, g, k, o
				R = 3/8	α = 30°	F	Not required	a, g, k, o
				R = 1/4	α = 45°	All	Not required	a, g, k, o
SAW	TC-U4a-S	U	U	R = 3/8	α = 30°	F	—	g, k, o
				R = 1/4	α = 45°			

Note: Referenced clauses in this table are from AWS D1.1. “Notes” on this page refer to Table 8-2, pg. 8-34, of this manual.

Table 8-2 (continued)
Prequalified Welded Joints Complete-Joint-Penetration Groove Welds

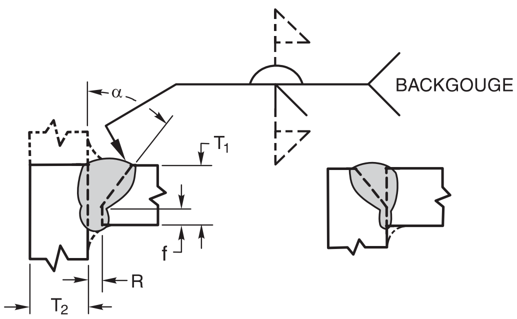
CJP

Single-bevel-groove weld (4)
Butt joint (B)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited) T1	T2	Groove Preparation: Root Opening Root Face Groove Angle	Tolerances As Detailed (see 3.13.1)	Tolerances As Fit-Up (see 3.13.1)	Allowed Welding Positions	Gas Shielding for FCAW	Notes
SMAW	B-U4b	U	—	R = 0 to 1/8 f = 0 to 1/8 α = 45°	+1/16, –0 +1/16, –0 +10°, –0°	+1/16, –1/8 Not Limited +10°, –5°	All	—	c, d, e, j
GMAW FCAW	B-U4b-GF	U	—				All	Not Required	a, c, d, j
SAW	B-U4b-S	U	—	R = 0 f = 1/4 max. α = 60°	±0 +0, –1/8 +10°, –0°	+1/4, –0 ±1/16 +10°, –5°	F	—	c, d, j

Single-bevel-groove weld (4)
T-joint (T)
Corner joint (C)



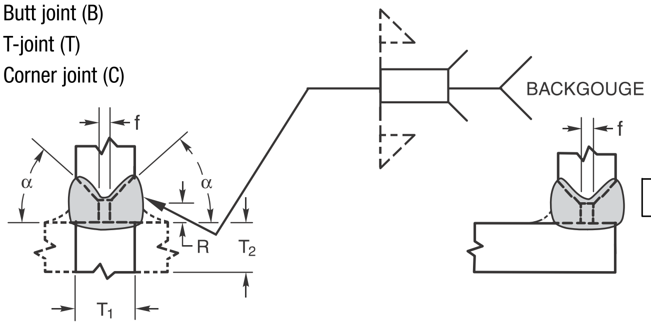
Welding Process	Joint Designation	Base Metal Thickness (U = unlimited) T1	T2	Groove Preparation: Root Opening Root Face Groove Angle	Tolerances As Detailed (see 3.13.1)	Tolerances As Fit-Up (see 3.13.1)	Allowed Welding Positions	Gas Shielding for FCAW	Notes
SMAW	TC-U4b	U	U	R = 0 to 1/8 f = 0 to 1/8 α = 45°	+1/16, –0 +1/16, –0 +10°, –0°	+1/16, –1/8 Not Limited +10°, –5°	All	—	d, e, g, j, k
GMAW FCAW	TC-U4b-GF	U	U				All	Not Required	a, d, g, j, k
SAW	TC-U4b-S	U	U	R = 0 f = 1/4 max. α = 60°	±0 +0, –1/8 +10°, –0°	+1/4, –0 ±1/16 +10°, –5°	F	—	d, g, j, k

Note: Referenced clauses in this table are from AWS D1.1. "Notes" on this page refer to Table 8-2, pg. 8-34, of this manual.

Table 8-2 (continued)
Prequalified Welded Joints Complete-Joint-Penetration Groove Welds

CJP

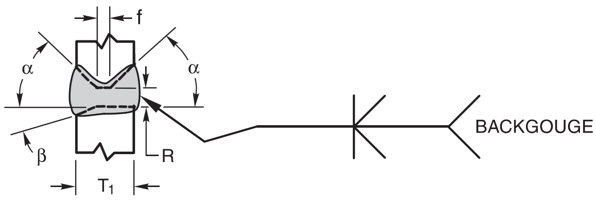
Double-bevel-groove weld (5)
Butt joint (B)
T-joint (T)
Corner joint (C)



	Tolerances	
	As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
	R = ±0	+1/4, −0
	f = +1/16, −0	±1/16
	α = +10°, −0°	+10°, −5°
Spacer	+1/16, −0	+1/8, −0

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Root Face	Groove Angle			
SMAW	B-U5b	U Spacer = 1/8 × R	—	R = 1/4	f = 0 to 1/8	α = 45°	All	—	c, d, e, h, j
SMAW	TC-U5a	U Spacer = 1/4 × R	U	R = 1/4	f = 0 to 1/8	α = 45°	All	—	d, e, g, h, j, k
				R = 3/8	f = 0 to 1/8	α = 30°	F, OH	—	d, e, g, h, j, k

Double-bevel-groove weld (5)
Butt joint (B)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
				Root Opening Root Face Groove Angle	Tolerances				
		T_1	T_2		As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)			
SMAW	B-U5a	U	—	R = 0 to 1/8 f = 0 to 1/8 α = 45° β = 0° to 15°	+1/16, −0 +1/16, −0 α + β = +10°, −0°	+1/16, −1/8 Not limited α + β = +10°, −5°	All	—	c, d, e, h, j
GMAW FCAW	B-U5-GF	U	—	R = 0 to 1/8 f = 0 to 1/8 α = 45° β = 0° to 15°	+1/16, −0 +1/16, −0 α + β = +10°, −0°	+1/16, −1/8 Not limited α + β = +10°, −5°	All	Not Required	a, c, d, h, j

Note: Referenced clauses in this table are from AWS D1.1. "Notes" on this page refer to Table 8-2, pg. 8-34, of this manual.

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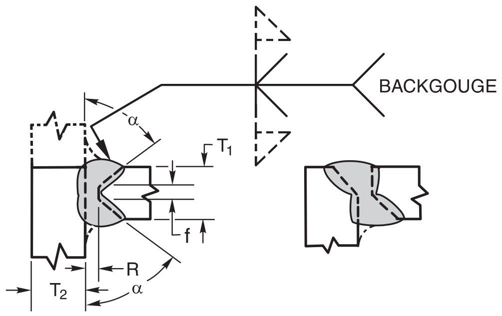
Table 8-2 (continued)
Prequalified Welded Joints
Complete-Joint-Penetration Groove Welds

Double-bevel-groove weld (5)
T-joint (T)
Corner joint (C)



<table>
<tr><th rowspan="3">Welding Process</th><th rowspan="3">Joint Designation</th><th colspan="2">Base Metal Thickness (U = unlimited)</th><th colspan="3">Groove Preparation</th><th rowspan="3">Allowed Welding Positions</th><th rowspan="3">Gas Shielding for FCAW</th><th rowspan="3">Notes</th></tr>
<tr><th rowspan="2">T_1</th><th rowspan="2">T_2</th><th rowspan="2">Root Opening Root Face Groove Angle</th><th colspan="2">Tolerances</th></tr>
<tr><th>As Detailed (see 3.13.1)</th><th>As Fit-Up (see 3.13.1)</th></tr>
<tr><td>SMAW</td><td>TC-U5b</td><td>U</td><td>U</td><td rowspan="2">R = 0 to 1/8
f = 0 to 1/8
α = 45°</td><td rowspan="2">+1/16, –0
+1/16, –0
+10°, –0</td><td rowspan="2">+1/16, –1/8
Not limited
+10°, –5°</td><td>All</td><td>—</td><td>d, e, g, h, j, k</td></tr>
<tr><td>GMAW FCAW</td><td>TC-U5-GF</td><td>U</td><td>U</td><td>All</td><td>Not Required</td><td>a, d, g, h, j, k</td></tr>
<tr><td>SAW</td><td>TC-U5-S</td><td>U</td><td>U</td><td>R = 0
f = 1/4 max.
α = 60°</td><td>± 0
+0, –3/16
+10°, –0°</td><td>+1/16, –0
±1/16
+10°, –5°</td><td>F</td><td>—</td><td>d, g, h, j, k</td></tr>
</table>

Note: Referenced clauses in this table are from AWS D1.1. “Notes” on this page refer to Table 8-2, pg. 8-34, of this manual.

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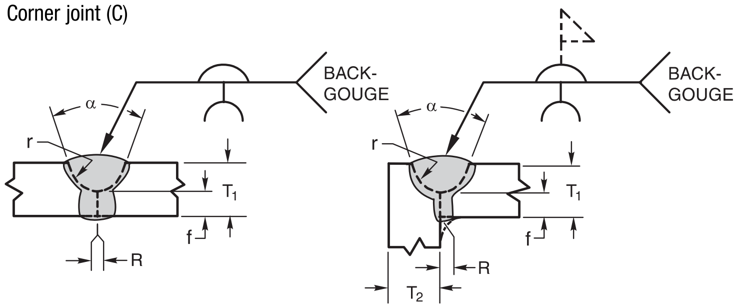
Table 8-2 (continued)
Prequalified Welded Joints
Complete-Joint-Penetration Groove Welds

Single-U-groove weld (6)
Butt joint (B)
Corner joint (C)



Tolerances	
As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
R = +1/16, –0	+1/16, –1/8
α = +10°, –0°	+10°, –5°
f = ±1/16	Not Limited
r = +1/8, –0	+1/8, –0

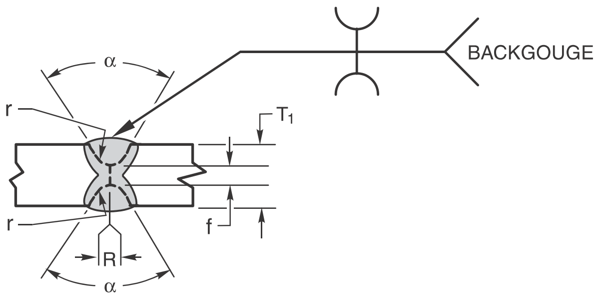
Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation				Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Groove Angle	Root Face	Bevel Radius			
SMAW	B-U6	U	—	R = 0 to 1/8	α = 45°	f = 1/8	r = 1/4	All	—	d, e, j
				R = 0 to 1/8	α = 20°	f = 1/8	r = 1/4	F, OH	—	d, e, j
	C-U6	U	U	R = 0 to 1/8	α = 45°	f = 1/8	r = 1/4	All	—	d, e, g, j
				R = 0 to 1/8	α = 20°	f = 1/8	r = 1/4	F, OH	—	d, e, g, j
GMAW FCAW	B-U6-GF	U	—	R = 0 to 1/8	α = 20°	f = 1/8	r = 1/4	All	Not required	a, d, j
	C-U6-GF	U	U	R = 0 to 1/8	α = 20°	f = 1/8	r = 1/4	All	Not required	a, d, g, j

Note: Referenced clauses in this table are from AWS D1.1. "Notes" on this page refer to Table 8-2, pg. 8-34, of this manual.

Table 8-2 (continued)
Prequalified Welded Joints
Complete-Joint-Penetration Groove Welds

CJP

Double-U-groove weld (7)
Butt joint (B)



Tolerances	
As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
For B-U7 and B-U7-GF	
R = +$^1/_{16}$, −0	$^1/_{16}$, −$^1/_8$
α = +10°, −0°	+10°, −5°
f = ±$^1/_{16}$, −0	Not Limited
r = +$^1/_4$, −0	±$^1/_{16}$
For B-U7-S	
R = ±0	+$^1/_{16}$, −0
α = +10°, −0°	+10°, −5°
f = +0, −$^1/_4$	±$^1/_{16}$
r = +$^1/_4$, −0	±$^1/_{16}$

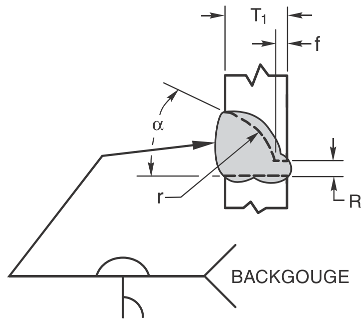
Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation				Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Groove Angle	Root Face	Bevel Radius			
SMAW	B-U7	U	—	R = 0 to $^1/_8$	α = 45°	f = $^1/_8$	r = $^1/_4$	All	—	d, e, h, j
				R = 0 to $^1/_8$	α = 20°	f = $^1/_8$	r = $^1/_4$	F, OH	—	d, e, h, j
GMAW FCAW	B-U7-GF	U	—	R = 0 to $^1/_8$	α = 20°	f = $^1/_8$	r = $^1/_4$	All	Not required	a, d, j, h
SAW	B-U7-S	U	—	R = 0	α = 20°	f = $^1/_4$ max.	r = $^1/_4$	F	—	d, h, j

Note: Referenced clauses in this table are from AWS D1.1. "Notes" on this page refer to Table 8-2, pg. 8-34, of this manual.

Table 8-2 (continued)
Prequalified Welded Joints
Complete-Joint-Penetration Groove Welds

CJP

Single-J-groove weld (8)
Butt joint (B)



Tolerances	
As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
B-U8 and B-U8-GF	
R = +1/16, −0	+1/16, −1/8
α = +10°, −0°	+10°, −5°
f = +1/8, −0	Not Limited
r = +1/4, −0	±1/16
B-U8-S	
R = ±0	+1/4, −0
α = +10°, −0°	+10°, −5°
f = +0, −1/8	±1/16
r = +1/4, −0	±1/16

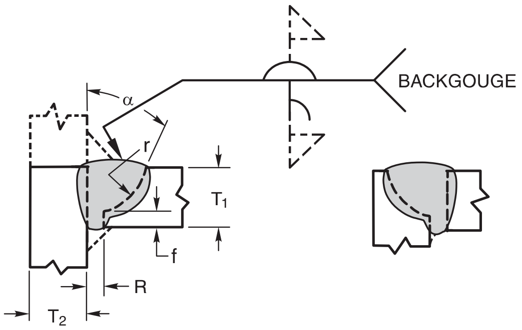
Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation				Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Groove Angle	Root Face	Bevel Radius			
SMAW	B-U8	U	—	R = 0 to 1/8	α = 45°	f = 1/8	r = 3/8	All	—	c, d, e, j
GMAW FCAW	B-U8-GF	U	—	R = 0 to 1/8	α = 30°	f = 1/8	r = 3/8	All	Not required	a, c, d, j
SAW	B-U8-S	U	—	R = 0	α = 45°	f = 1/4 max.	r = 3/8	F	—	c, d, j

Note: Referenced clauses in this table are from AWS D1.1. “Notes” on this page refer to Table 8-2, pg. 8-34, of this manual.

CJP

Table 8-2 (continued)
Prequalified Welded Joints
Complete-Joint-Penetration Groove Welds

Single-J-groove weld (8)
T-joint (T)
Corner joint (C)



Tolerances	
As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
TC-U8a and TC-U8a-GF	
R = +1/16, −0	+1/16, −1/8
α = +10°, −0°	+10°, −5°
f = +1/16, −0	Not Limited
r = +1/4, −0	±1/16
TC-U8a-S	
R = ±0	+1/4, −0
α = +10°, −0°	+10°, −5°
f = +0, −1/8	±1/16
r = +1/4, −0	±1/16

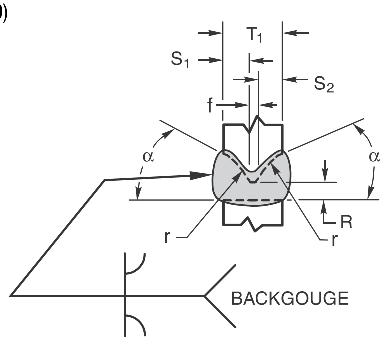
Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation				Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Groove Angle	Root Face	Bevel Radius			
SMAW	TC-U8a	U	U	R = 0 to 1/8	α = 45°	f = 1/8	r = 3/8	All	—	d, e, g, j, k
				R = 0 to 1/8	α = 30°	f = 1/8	r = 3/8	F, OH	—	d, e, g, j, k
GMAW FCAW	TC-U8a-GF	U	U	R = 0 to 1/8	α = 30°	f = 1/8	r = 3/8	All	Not required	a, d, g, j, k
SAW	TC-U8a-S	U	U	R = 0	α = 45°	f = 1/4 max.	r = 3/8	F	—	d, g, j, k

Note: Referenced clauses in this table are from AWS D1.1. "Notes" on this page refer to Table 8-2, pg. 8-34, of this manual.

Table 8-2 (continued)
Prequalified Welded Joints
Complete-Joint-Penetration Groove Welds

CJP

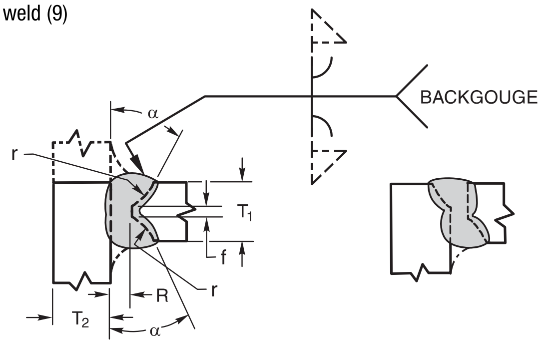
Double-J-groove weld (9)
Butt joint (B)



Tolerances	
As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
R = $+^1/_{16}$, −0	$+^1/_{16}$, $-^1/_8$
α = +10°, −0°	+10°, −5°
f = $+^1/_{16}$, −0	Not Limited
r = $+^1/_8$, −0	$\pm^1/_{16}$

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation				Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Groove Angle	Root Face	Bevel Radius			
SMAW	B-U9	U	—	R = 0 to $^1/_8$	α = 45°	f = $^1/_8$	r = $^3/_8$	All	—	c, d, e, h, j
GMAW FCAW	B-U9-GF	U	—	R = 0 to $^1/_8$	α = 30°	f = $^1/_8$	r = $^3/_8$	All	Not required	a, c, d, h, j

Double-J-groove weld (9)
T-joint (T)
Corner joint (C)



Tolerances	
As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
R = $+^1/_{16}$, −0	$+^1/_{16}$, $-^1/_8$
α = +10°, −0°	+10°, −5°
f = $+^1/_{16}$, −0	Not Limited
r = $+^1/_8$, −0	$\pm^1/_{16}$

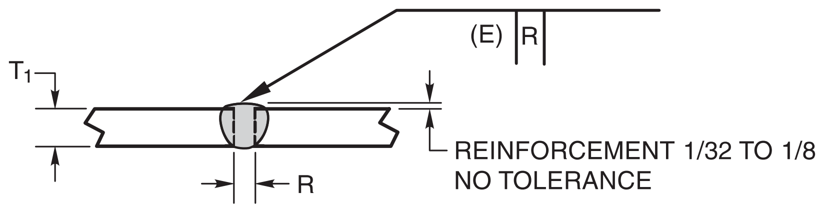
Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation				Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Groove Angle	Root Face	Bevel Radius			
SMAW	TC-U9a	U	U	R = 0 to $^1/_8$	α = 45°	f = $^1/_8$	r = $^3/_8$	All	—	d, e, g, h, j, k
				R = 0 to $^1/_8$	α = 30°	f = $^1/_8$	r = $^3/_8$	F, OH	—	d, e, g, h, k
GMAW FCAW	TC-U9a-GF	U	U	R = 0 to $^1/_8$	α = 30°	f = $^1/_8$	r = $^3/_8$	All	Not required	a, d, g, h, j, k

Note: Referenced clauses in this table are from AWS D1.1. “Notes” on this page refer to Table 8-2, pg. 8-34, of this manual.

PJP

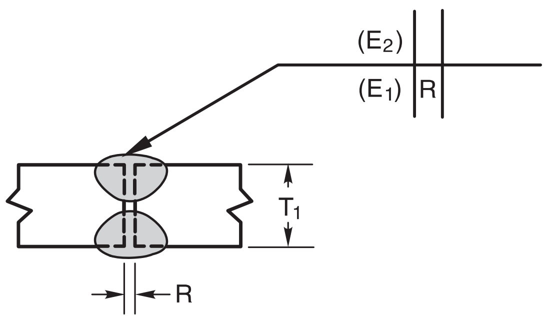
Table 8-2 (continued)
Prequalified Welded Joints Partial-Joint-Penetration Groove Welds

Square-groove weld (1)
Butt joint (B)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Weld Size (E)	Notes
		T_1	T_2	Root Opening	Tolerances: As Detailed (see 3.12.3)	Tolerances: As Fit-Up (see 3.12.3)			
SMAW	B-P1a	$^1/_8$	—	$R = 0$ to $^1/_{16}$	$+^1/_{16}, -0$	$\pm^1/_{16}$	All	$T_1 - ^1/_{32}$	b
SMAW	B-P1c	$^1/_4$ max.	—	$R = \frac{T_1}{2}$ min.	$+^1/_{16}, -0$	$\pm^1/_{16}$	All	$\frac{T_1}{2}$	b
GMAW FCAW	B-P1a-GF	$^1/_8$	—	$R = 0$ to $^1/_{16}$	$+^1/_{16}, -0$	$\pm^1/_{16}$	All	$T_1 - ^1/_{32}$	b, e
GMAW FCAW	B-P1c-GF	$^1/_4$ max.	—	$R = \frac{T_1}{2}$ min.	$+^1/_{16}, -0$	$\pm^1/_{16}$	All	$\frac{T_1}{2}$	b, e

Square-groove weld (1)
Butt joint (B)



$E_1 + E_2$ must not exceed $\frac{3T_1}{4}$

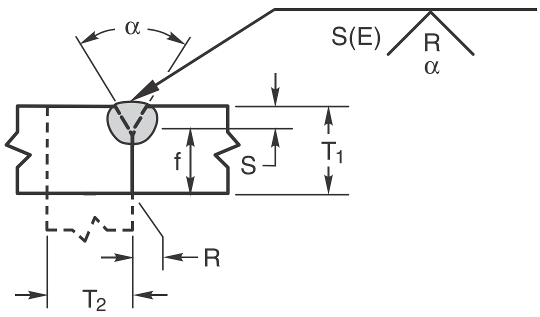
Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Total Weld Size ($E_1 + E_2$)	Notes
		T_1	T_2	Root Opening	Tolerances: As Detailed (see 3.12.3)	Tolerances: As Fit-Up (see 3.12.3)			
SMAW	B-P1b	$^1/_4$ max.	—	$R = \frac{T_1}{2}$	$+^1/_{16}, -0$	$\pm^1/_{16}$	All	$\frac{3T_1}{4}$	
GMAW FCAW	B-P1b-GF	$^1/_4$ max.	—	$R = \frac{T_1}{2}$	$+^1/_{16}, -0$	$\pm^1/_{16}$	All	$\frac{3T_1}{4}$	e

Note: Referenced clauses in this table are from AWS D1.1. "Notes" on this page refer to Table 8-2, pg. 8-34, of this manual.

Table 8-2 (continued)
Prequalified Welded Joints
Partial-Joint-Penetration Groove Welds

PJP

Single-V-groove weld (2)
Butt joint (B)
Corner joint (C)



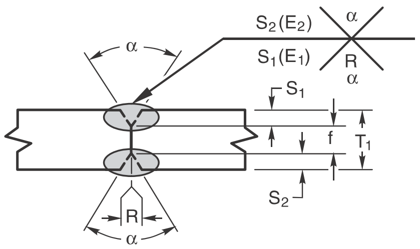
Welding Process	Joint Designation	Base Metal Thickness (U = unlimited) T_1	T_2	Groove Preparation: Root Opening Root Face Groove Angle	Tolerances: As Detailed (see 3.12.3)	Tolerances: As Fit-Up (see 3.12.3)	Allowed Welding Positions	Weld Size (E)	Notes
SMAW	BC-P2	1/4 min.	U	R = 0 f = 1/32 min. α = 60°	+1/16, −0 +U, −0 +10°, −0°	+1/8, −1/16 ±1/16 +10°, −5°	All	S	b, e, f, j
GMAW FCAW	BC-P2-GF	1/4 min.	U	R = 0 f = 1/8 min. α = 60°	+1/16, −0 +U, −0 +10°, −0°	+1/8, −1/16 ±1/16 +10°, −5°	All	S	a, b, f, j
SAW	BC-P2-S	7/16 min.	U	R = 0 f = 1/4 min. α = 60°	±0 +U, −0 +10°, −0°	+1/16, −0 ±1/16 +10°, −5°	F	S	b, f, j

Note: Referenced clauses in this table are from AWS D1.1. “Notes” on this page refer to Table 8-2, pg. 8-34, of this manual.

Table 8-2 (continued) Prequalified Welded Joints Partial-Joint-Penetration Groove Welds

PJP

Double-V-groove weld (3)
Butt joint (B)



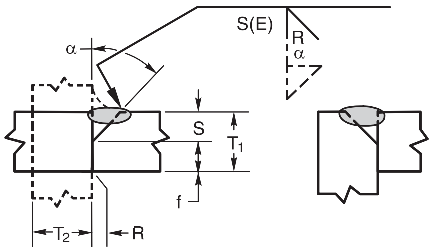
Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Total Weld Size ($E_1 + E_2$)	Notes
				Root Opening Root Face Groove Angle	Tolerances				
		T_1	T_2		As Detailed (see 3.12.3)	As Fit-Up (see 3.12.3)			
SMAW	B-P3	1/2 min.	—	R = 0 f = 1/8 min. α = 60°	+1/16, –0 +U, –0 +10°, –0°	+1/8, –1/16 ±1/16 +10°, –5°	All	$S_1 + S_2$	e, f, i, j
GMAW FCAW	B-P3-GF	1/2 min.	—	R = 0 f = 1/8 min. α = 60°	+1/16, –0 +U, –0 +10°, –0°	+1/8, –1/16 ±1/16 +10°, –5°	All	$S_1 + S_2$	a, f, i, j
SAW	B-P3-S	3/4 min.	—	R = 0 f = 1/4 min. α = 60°	+0 +U, –0 +10°, –0°	+1/16, –0 ±1/16 +10°, –5°	F	$S_1 + S_2$	f, i, j

Note: Referenced clauses in this table are from AWS D1.1. "Notes" on this page refer to Table 8-2, pg. 8-34, of this manual.

PJP

Table 8-2 (continued)
Prequalified Welded Joints
Partial-Joint-Penetration Groove Welds

Single-bevel-groove weld (4)
Butt joint (B)
T-joint (T)
Corner joint (C)



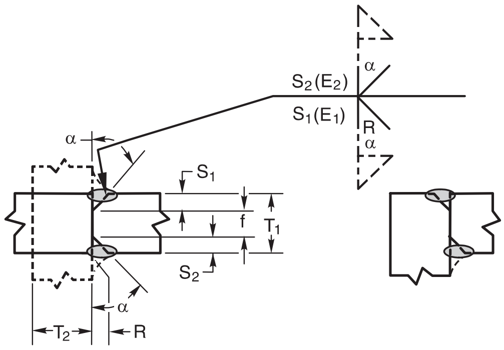
Welding Process	Joint Designation	Base Metal Thickness (U = unlimited) T_1	Base Metal Thickness (U = unlimited) T_2	Groove Preparation: Root Opening Root Face Groove Angle	Tolerances: As Detailed (see 3.12.3)	Tolerances: As Fit-Up (see 3.12.3)	Allowed Welding Positions	Total Weld Size (E)	Notes
SMAW	BTC-P4	U	U	R = 0 f = $^{1}/_{8}$ min. α = 45°	$+^{1}/_{16}, -0$ +U, –0 +10°, –0°	$+^{1}/_{8}, -^{1}/_{16}$ $\pm^{1}/_{16}$ +10°, –5°	All	S–$^{1}/_{8}$	b, e, f, g, j, k
GMAW FCAW	BTC-P4-GF	$^{1}/_{4}$ min.	U	R = 0 f = $^{1}/_{8}$ min. α = 45°	$+^{1}/_{16}, -0$ +U, –0 +10°, –0°	$+^{1}/_{8}, -^{1}/_{16}$ $\pm^{1}/_{16}$ +10°, –5°	F, H	S	a, b, f, g, j, k
							V, OH	S–$^{1}/_{8}$	
SAW	TC-P4-S	$^{7}/_{16}$ min.	U	R = 0 f = $^{1}/_{4}$ min. α = 60°	±0 +U, –0 +10°, –0°	$+^{1}/_{16}, -0$ $\pm^{1}/_{16}$ +10°, –5°	F	S	b, f, g, j, k

Note: Referenced clauses in this table are from AWS D1.1. "Notes" on this page refer to Table 8-2, pg. 8-34, of this manual.

Table 8-2 (continued)
Prequalified Welded Joints
Partial-Joint-Penetration Groove Welds

PJP

Double-bevel-groove weld (5)
Butt joint (B)
T-joint (T)
Corner joint (C)



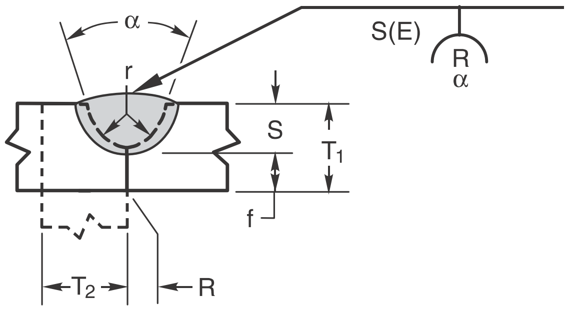
Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Total Weld Size ($E_1 + E_2$)	Notes
				Root Opening Root Face Groove Angle	Tolerances				
		T_1	T_2		As Detailed (see 3.12.3)	As Fit-Up (see 3.12.3)			
SMAW	BTC-P5	5/16 min.	U	R = 0 f = 1/8 min. α = 45°	+1/16, −0 +U, −0 +10°, −0°	+1/8, −1/16 ±1/16 +10°, −5°	All	$S_1 + S_2$ −1/4	e, f, g, i, j, k
GMAW FCAW	BTC-P5-GF	1/2 min.	U	R = 0 f = 1/8 min. α = 45°	+1/16, −0 +U, −0 +10°, −0°	+1/8, −1/16 ±1/16 +10°, −5°	F, H	$S_1 + S_2$	a, f, g, i, j, k
							V, OH	$S_1 + S_2$ −1/4	
SAW	TC-P5-S	3/4 min.	U	R = 0 f = 1/4 min. α = 60°	±0 +U, −0 +10°, −0°	+1/16, −0 ±1/16 +10°, −5°	F	$S_1 + S_2$	f, g, i, j, k

Note: Referenced clauses in this table are from AWS D1.1. "Notes" on this page refer to Table 8-2, pg. 8-34, of this manual.

Table 8-2 (continued)
Prequalified Welded Joints
Partial-Joint-Penetration Groove Welds

PJP

Single-U-groove weld (6)
Butt joint (B)
Corner joint (C)



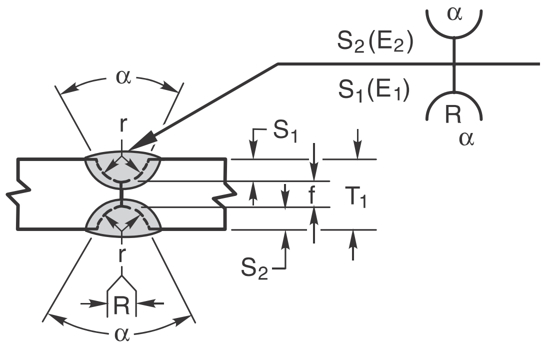
Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Total Weld Size (E)	Notes
		T_1	T_2	Root Opening Root Face Bevel Radius Groove Angle	Tolerances: As Detailed (see 3.12.3)	Tolerances: As Fit-Up (see 3.12.3)			
SMAW	BC-P6	1/4 min.	U	R = 0 f = 1/32 min. r = 1/4 α = 45°	+1/16, −0 +U, −0 +1/4, −0 +10°, −0°	+1/8, −1/16 ±1/16 ±1/16 +10°, −5°	All	S	b, e, f, j
GMAW FCAW	BC-P6-GF	1/4 min.	U	R = 0 f = 1/8 min. r = 1/4 α = 20°	+1/16, −0 +U, −0 +1/4, −0 +10°, −0°	+1/8, −1/16 ±1/16 ±1/16 +10°, −5°	All	S	a, b, f, j
SAW	BC-P6-S	7/16 min.	U	R = 0 f = 1/4 min. r = 1/4 α = 20°	±0 +U, −0 +1/4, −0 +10°, −0°	+1/16, −0 ±1/16 ±1/16 +10°, −5°	F	S	b, f, j

Note: Referenced clauses in this table are from AWS D1.1. "Notes" on this page refer to Table 8-2, pg. 8-34, of this manual.

PJP

Table 8-2 (continued)
Prequalified Welded Joints
Partial-Joint-Penetration Groove Welds

Double-U-groove weld (7)
Butt joint (B)



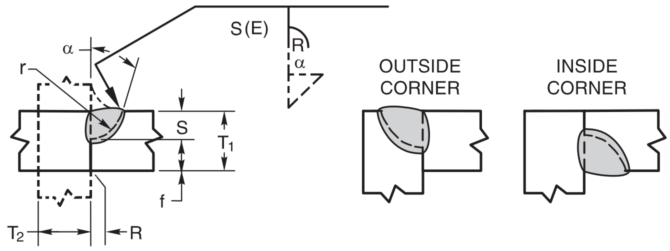
Welding Process	Joint Designation	Base Metal Thickness (U = unlimited) T1	Base Metal Thickness (U = unlimited) T2	Groove Preparation: Root Opening Root Face Bevel Radius Groove Angle	Groove Preparation: Tolerances As Detailed (see 3.12.3)	Groove Preparation: Tolerances As Fit-Up (see 3.12.3)	Allowed Welding Positions	Total Weld Size ($E_1 + E_2$)	Notes
SMAW	B-P7	1/2 min.	—	R = 0 f = 1/8 min. r = 1/4 α = 45°	+1/16, −0 +U, −0 +1/4, −0 +10°, −0°	+1/8, −1/16 ±1/16 ±1/16 +10°, −5°	All	$S_1 + S_2$	e, f, i, j
GMAW FCAW	B-P7-GF	1/2 min.	—	R = 0 f = 1/8 min. r = 1/4 α = 20°	+1/16, −0 +U, −0 +1/4, −0 +10°, −0°	+1/8, −1/16 ±1/16 ±1/16 +10°, −5°	All	$S_1 + S_2$	a, f, i, j
SAW	B-P7-S	3/4 min.	—	R = 0 f = 1/4 min. r = 1/4 α = 20°	±0 +U, −0 +1/4, −0 +10°, −0°	+1/16, −0 ±1/16 ±1/16 +10°, −5°	F	$S_1 + S_2$	f, i, j

Note: Referenced clauses in this table are from AWS D1.1. "Notes" on this page refer to Table 8-2, pg. 8-34, of this manual.

Table 8-2 (continued)
Prequalified Welded Joints
Partial-Joint-Penetration Groove Welds

PJP

Single-J-groove weld (8)
Butt joint (B)
T-joint (T)
Corner joint (C)



*α_{oc} = Outside corner groove angle.
**α_{ic} = Inside corner groove angle.

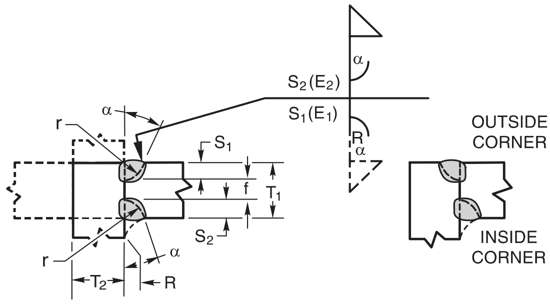
Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Total Weld Size (E)	Notes
		T_1	T_2	Root Opening Root Face Bevel Radius Groove Angle	Tolerances: As Detailed (see 3.12.3)	Tolerances: As Fit-Up (see 3.12.3)			
SMAW	B-P8	1/4 min.	—	R = 0 f = 1/8 min. r = 3/8 α = 30°	+1/16, –0 +U, –0 +1/4, –0 +10°, –0°	+1/8, –1/16 ±1/16 ±1/16 +10°, –5°	All	S	e, f, g, j, k
	TC-P8	1/4 min.	U	R = 0 f = 1/8 min. r = 3/8 α_{oc} = 30°* α_{ic} = 45°**	+1/16, –0 +U, –0 +1/4, –0 +10°, –0° +10°, –0°	+1/8, –1/16 ±1/16 ±1/16 +10°, –5° +10°, –5°	All	S	e, f, g, j, k
GMAW FCAW	B-P8-GF	1/4 min.	—	R = 0 f = 1/8 min. r = 3/8 α = 30°	+1/16, –0 +U, –0 +1/4, –0 +10°, –0°	+1/8, –1/16 ±1/16 ±1/16 +10°, –5°	All	S	a, f, g, j, k
	TC-P8-GF	1/4 min.	U	R = 0 f = 1/8 min. r = 3/8 α_{oc} = 30°* α_{ic} = 45°**	+1/16, –0 +U, –0 +1/4, –0 +10°, –0° +10°, –0°	+1/8, –1/16 ±1/16 ±1/16 +10°, –5° +10°, –5°	All	S	a, f, g, j, k
SAW	B-P8-S	7/16 min.	—	R = 0 f = 1/4 min. r = 1/2 α = 20°	±0 +U, –0 +1/4, –0 +10°, –0°	+1/16, –0 ±1/16 ±1/16 +10°, –5°	F	S	f, g, j, k
	TC-P8-S	7/16 min.	U	R = 0 f = 1/4 min. r = 1/2 α_{oc} = 20°* α_{ic} = 45°**	±0 +U, –0 +1/4, –0 +10°, –0° +10°, –0°	+1/16, –0 ±1/16 ±1/16 +10°, –5° +10°, –5°	F	S	f, g, j, k

Note: Referenced clauses in this table are from AWS D1.1. "Notes" on this page refer to Table 8-2, pg. 8-34, of this manual.

Table 8-2 (continued)
Prequalified Welded Joints
Partial-Joint-Penetration Groove Welds

PJP

Double-J-groove weld (9)
Butt joint (B)
T-joint (T)
Corner joint (C)



* α_{oc} = Outside corner groove angle.
** α_{ic} = Inside corner groove angle.

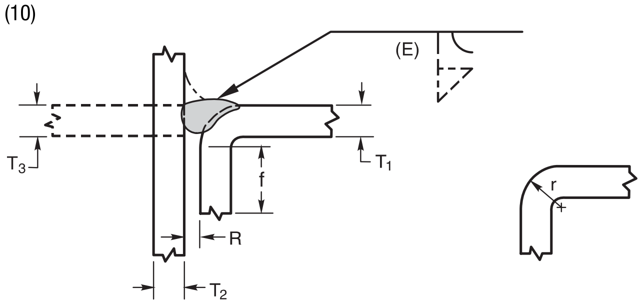
Welding Process	Joint Designation	Base Metal Thickness (U = unlimited) T1	T2	Groove Preparation: Root Opening Root Face Bevel Radius Groove Angle	Tolerances: As Detailed (see 3.12.3)	Tolerances: As Fit-Up (see 3.12.3)	Allowed Welding Positions	Total Weld Size ($E_1 + E_2$)	Notes
SMAW	B-P9	1/2 min.	—	R = 0 f = 1/8 min. r = 3/8 α = 30°	+1/16, −0 +U, −0 +1/4, −0 +10°, −0°	+1/8, −1/16 ±1/16 ±1/16 +10°, −5°	All	$S_1 + S_2$	e, f, g, i, j, k
SMAW	TC-P9	1/2 min.	U	R = 0 f = 1/8 min. r = 3/8 α_{oc} = 30°* α_{ic} = 45°**	+1/16, −0 +U, −0 +1/4, −0 +10°, −0° +10°, −0°	+1/8, −1/16 ±1/16 ±1/16 +10°, −5° +10°, −5°	All	$S_1 + S_2$	e, f, g, i, j, k
GMAW FCAW	B-P9-GF	1/2 min.	—	R = 0 f = 1/8 min. r = 3/8 α = 30°	+1/16, −0 +U, −0 +1/4, −0 +10°, −0°	+1/8, −1/16 ±1/16 ±1/16 +10°, −5°	All	$S_1 + S_2$	a, f, g, i, j, k
GMAW FCAW	TC-P9-GF	1/2 min.	U	R = 0 f = 1/8 min. r = 3/8 α_{oc} = 30°* α_{ic} = 45°**	±0 +U, −0 +1/4, −0 +10°, −0° +10°, −0°	+1/16, −0 ±1/16 ±1/16 +10°, −5° +10°, −5°	All	$S_1 + S_2$	a, f, g, i, j, k
SAW	B-P9-S	3/4 min.	—	R = 0 f = 1/4 min. r = 1/2 α = 20°	±0 +U, −0 +1/4, −0 +10°, −0°	+1/16, −0 ±1/16 ±1/16 +10°, −5°	F	$S_1 + S_2$	f, g, i, j, k
SAW	TC-P9-S	3/4 min.	U	R = 0 f = 1/4 min. r = 1/2 α_{oc} = 20°* α_{ic} = 45°**	±0 +U, −0 +1/4, −0 +10°, −0° +10°, −0°	+1/16, −0 ±1/16 ±1/16 +10°, −5° +10°, −5°	F	$S_1 + S_2$	f, g, i, j, k

Note: Referenced clauses in this table are from AWS D1.1. “Notes” on this page refer to Table 8-2, pg. 8-34, of this manual.

FLARE

Table 8-2 (continued)
Prequalified Welded Joints
Flare-Bevel Groove Welds

Flare-bevel-groove weld (10)
Butt joint (B)
T-joint (T)
Corner joint (C)



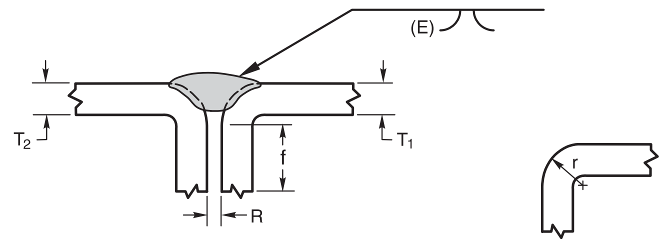
Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)			Groove Preparation			Allowed Welding Positions	Total Weld Size (E)	Notes
		T_1	T_2	T_3	Root Opening Root Face Bend Radius	Tolerances: As Detailed (see 3.12.3)	Tolerances: As Fit-Up (see 3.12.3)			
SMAW FCAW-S	BTC-P10	$^3/_{16}$ min.	U	T_1 min.	R = 0 f = $^3/_{16}$ min. $r = \frac{3T_1}{2}$ min.	$+^1/_{16}, -0$ +U, −0 +U, −0	$+^1/_8, -^1/_{16}$ $+U, -^1/_{16}$ +U, −0	All	$^5/_{16}$ r	e, g, j, i
GMAW FCAW-G	BTC-P10-GF	$^3/_{16}$ min.	U	T_1 min.	R = 0 f = $^3/_{16}$ min. $r = \frac{3T_1}{2}$ min.	$+^1/_{16}, -0$ +U, −0 +U, −0	$+^1/_8, -^1/_{16}$ $+U, -^1/_{16}$ +U, −0	All	$^5/_8$ r	a, g, j, l, m
SAW	B-P10-S	$^1/_2$ min.	N/A	$^1/_2$ min.	R = 0 f = $^1/_2$ min. $r = \frac{3T_1}{2}$ min.	±0 +U, −0 +U, −0	$+^1/_{16}, -0$ $+U, -^1/_{16}$ +U, −0	F	$^5/_{16}$ r	g, j, l, m

Note: Referenced clauses in this table are from AWS D1.1. “Notes” on this page refer to Table 8-2, pg. 8-34, of this manual.

FLARE

Table 8-2 (continued)
Prequalified Welded Joints
Flare-Bevel Groove Welds

Flare-V-groove weld (11)
Butt joint (B)

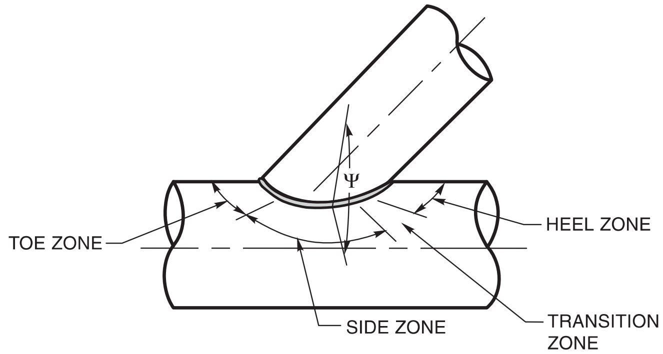


Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Total Weld Size (E)	Notes
		T_1	T_2	Root Opening Root Face Bend Radius	Tolerances: As Detailed (see 3.12.3)	Tolerances: As Fit-Up (see 3.12.3)			
SMAW FCAW-S	B-P11	$^3/_{16}$ min.	T_1 min.	R = 0 f = $^3/_{16}$ min. $r = \frac{3T_1}{2}$ min.	$+^1/_{16}, -0$ +U, –0 +U, –0	$+^1/_8, -^1/_{16}$ $+U, -^1/_{16}$ +U, –0	All	$^5/_8$ r	e, j, l, m, n
GMAW FCAW-G	B-P11-GF	$^3/_{16}$ min.	T_1 min.	R = 0 f = $^3/_{16}$ min. $r = \frac{3T_1}{2}$ min.	$+^1/_{16}, -0$ +U, –0 +U, –0	$+^1/_8, -^1/_{16}$ $+U, -^1/_{16}$ +U, –0	All	$^3/_4$ r	a, j, l, m, n
SAW	B-P11-S	$^1/_2$ min.	T_1 min.	R = 0 f = $^1/_2$ min. $r = \frac{3T_1}{2}$ min.	±0 +U, –0 +U, –0	$+^1/_{16}, -0$ $+U, -^1/_{16}$ +U, –0	F	$^1/_2$ r	j, l, m, n

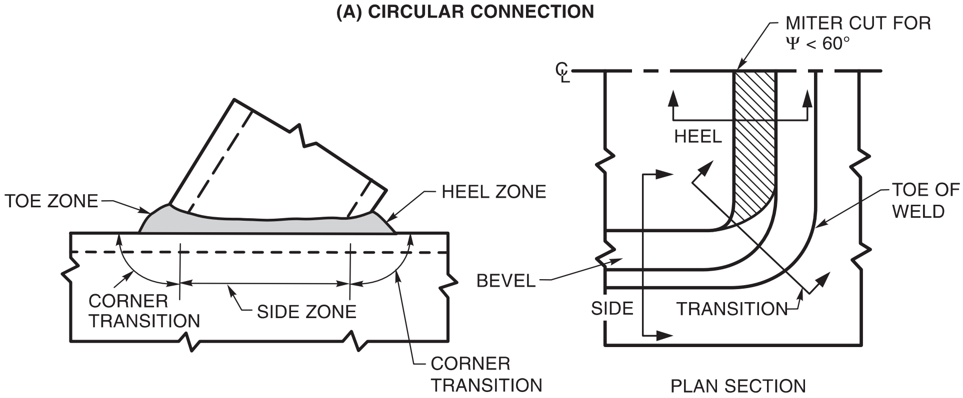
Note: Referenced clauses in this table are from AWS D1.1. “Notes” on this page refer to Table 8-2, pg. 8-34, of this manual.

TUBE

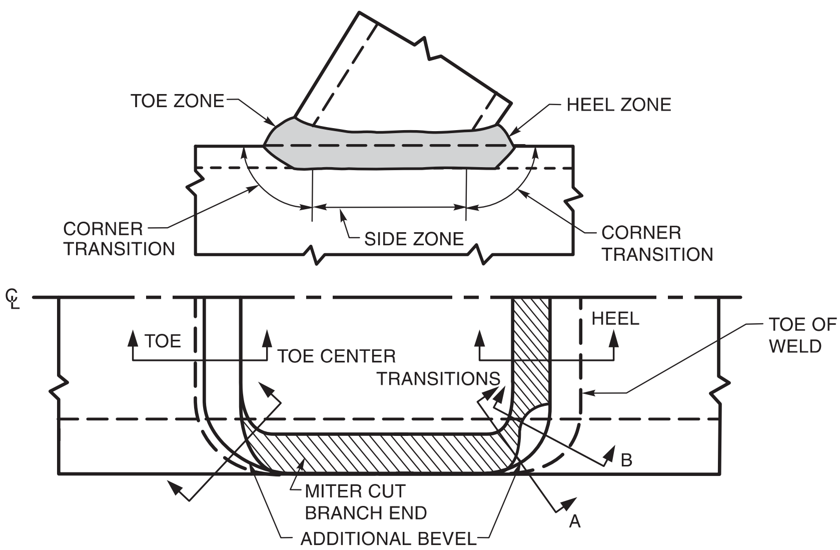
Table 8-2 (continued)
Prequalified Welded Joints
PJP T-, Y- and K-Tubular Connections



(A) CIRCULAR CONNECTION



(B) STEPPED BOX CONNECTION



(C) MATCHED BOX CONNECTION

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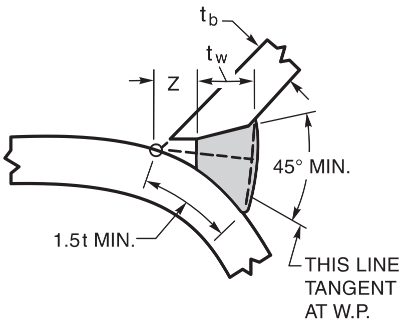
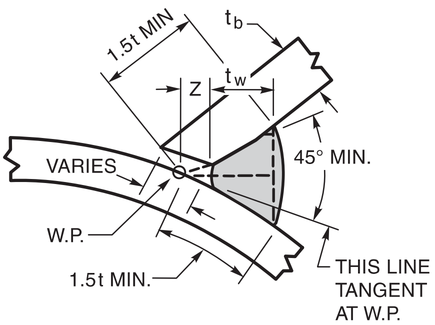
TUBE

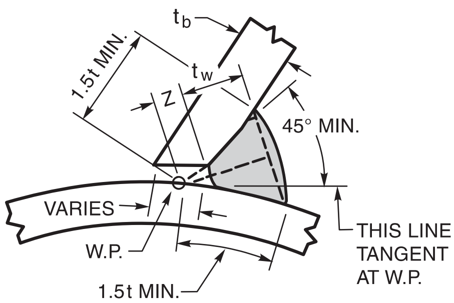
Table 8-2 (continued)
Prequalified Welded Joints
PJP T-, Y- and K-Tubular Connections



TRANSITION A

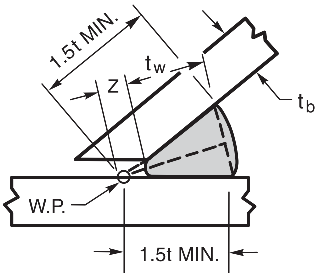


TRANSITION B



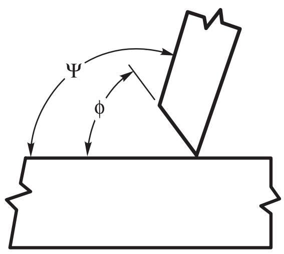
Ψ = 75°–60°

TRANSITION OR HEEL



Ψ = 60°–30°

HEEL



SKETCH FOR ANGULAR DEFINITION

$150° \geq \Psi \geq 30°$
$90° > \phi \geq 30°$

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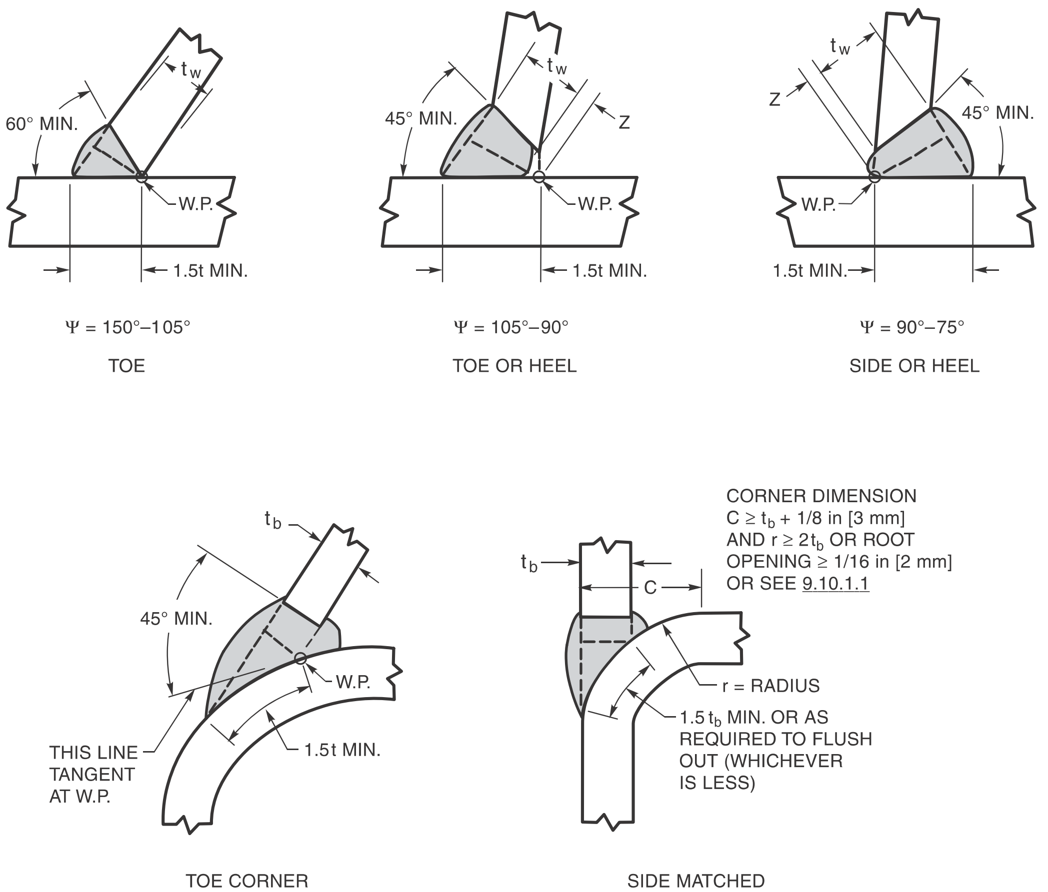
TUBE

Table 8-2 (continued)
Prequalified Welded Joints
PJP T-, Y- and K-Tubular Connections



Notes:
1. t = thickness of thinner section.
2. Bevel to feather edge except in transition and heel zones.
3. Root opening: 0 in to 3/16 in [5 mm].
4. Not prequalified for under 30°.
5. Weld size (effective throat) $t_w \geq t$; Z Loss Dimensions shown in Table 9.5.
6. Calculations per 9.6.1.3 shall be done for leg length less than 1.5t, as shown.
7. For Box Section, joint preparation for corner transitions shall provide a smooth transition from one detail to another. Welding shall be carried continuously around corners, with corners fully built up and all weld starts and stops within flat faces.
8. See Annex J for definition of local dihedral angle, Ψ.
9. W.P. = work point.

Note: Referenced clauses and tables in this figure are from AWS D1.1.

Table 8-3
Electrode Strength Coefficient, C_1

Electrode	F_{EXX} (ksi)	C_1
E60	60	0.857
E70	70	1.00
E80	80	1.03
E90	90	1.16
E100	100	1.21
E110	110	1.34

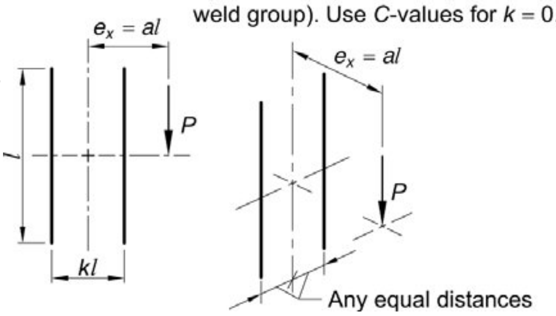
Table 8-4
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 0°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

Special case (load not in plane of weld group). Use C-values for $k = 0$.



a	*k* = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71
0.10	3.72	3.72	3.72	3.71	3.70	3.69	3.67	3.65	3.63	3.61	3.59	3.55	3.52	3.48	3.44	3.43
0.15	3.67	3.66	3.65	3.64	3.62	3.60	3.58	3.56	3.54	3.52	3.50	3.46	3.43	3.39	3.36	3.33
0.20	3.51	3.51	3.50	3.49	3.47	3.46	3.44	3.42	3.41	3.39	3.38	3.35	3.32	3.30	3.27	3.25
0.25	3.31	3.31	3.31	3.30	3.29	3.28	3.28	3.27	3.26	3.25	3.25	3.23	3.21	3.20	3.18	3.16
0.30	3.09	3.09	3.10	3.10	3.10	3.10	3.11	3.11	3.11	3.11	3.11	3.11	3.10	3.09	3.08	3.07
0.40	2.66	2.67	2.68	2.70	2.73	2.75	2.77	2.80	2.81	2.83	2.84	2.87	2.88	2.89	2.90	2.90
0.50	2.30	2.30	2.32	2.36	2.40	2.44	2.48	2.52	2.55	2.58	2.60	2.65	2.68	2.70	2.72	2.73
0.60	2.00	2.00	2.03	2.07	2.12	2.18	2.23	2.28	2.32	2.36	2.39	2.45	2.49	2.53	2.56	2.58
0.70	1.76	1.77	1.79	1.84	1.90	1.96	2.02	2.07	2.12	2.16	2.20	2.27	2.33	2.38	2.41	2.45
0.80	1.57	1.57	1.60	1.65	1.71	1.78	1.84	1.90	1.95	2.00	2.04	2.12	2.18	2.24	2.28	2.32
0.90	1.41	1.42	1.45	1.50	1.56	1.62	1.69	1.75	1.80	1.85	1.90	1.98	2.05	2.11	2.16	2.20
1.0	1.28	1.29	1.32	1.37	1.43	1.49	1.56	1.62	1.67	1.72	1.77	1.86	1.93	2.00	2.05	2.10
1.2	1.08	1.08	1.12	1.16	1.22	1.28	1.35	1.41	1.46	1.51	1.56	1.65	1.73	1.80	1.86	1.91
1.4	0.928	0.936	0.966	1.01	1.07	1.13	1.19	1.24	1.30	1.35	1.40	1.49	1.57	1.64	1.70	1.75
1.6	0.815	0.823	0.852	0.894	0.945	1.00	1.06	1.11	1.16	1.21	1.26	1.35	1.43	1.50	1.56	1.62
1.8	0.727	0.734	0.761	0.800	0.848	0.899	0.953	1.00	1.05	1.10	1.15	1.24	1.31	1.38	1.45	1.50
2.0	0.655	0.663	0.688	0.724	0.768	0.817	0.867	0.916	0.964	1.01	1.06	1.14	1.22	1.28	1.35	1.40
2.2	0.597	0.604	0.627	0.661	0.702	0.747	0.794	0.841	0.887	0.931	0.975	1.06	1.13	1.20	1.26	1.31
2.4	0.547	0.554	0.576	0.608	0.646	0.689	0.733	0.777	0.821	0.864	0.905	0.983	1.06	1.12	1.18	1.24
2.6	0.506	0.512	0.533	0.562	0.598	0.638	0.680	0.722	0.764	0.805	0.845	0.920	0.990	1.05	1.11	1.17
2.8	0.470	0.476	0.495	0.523	0.557	0.595	0.634	0.674	0.714	0.753	0.791	0.864	0.932	0.994	1.05	1.10
3.0	0.439	0.445	0.463	0.489	0.521	0.557	0.594	0.632	0.670	0.708	0.745	0.815	0.880	0.940	0.996	1.05

Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

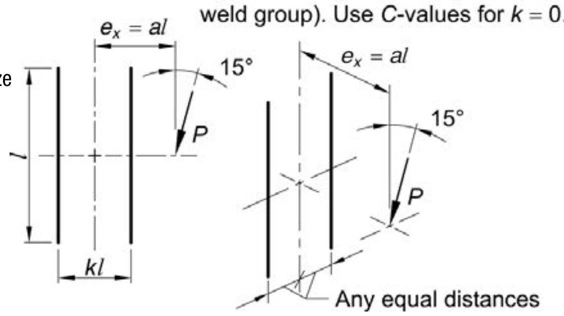
Table 8-4 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 15°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

Special case (load not in plane of weld group). Use C-values for $k = 0$.



a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	3.96	3.96	3.96	3.96	3.96	3.96	3.96	3.96	3.96	3.96	3.96	3.96	3.96	3.96	3.96	3.96
0.10	3.79	3.79	3.78	3.78	3.77	3.76	3.75	3.74	3.73	3.72	3.71	3.69	3.67	3.65	3.64	3.62
0.15	3.68	3.68	3.67	3.66	3.65	3.64	3.63	3.62	3.61	3.61	3.60	3.58	3.57	3.55	3.54	3.53
0.20	3.51	3.51	3.51	3.50	3.50	3.49	3.49	3.48	3.48	3.47	3.47	3.46	3.46	3.45	3.44	3.43
0.25	3.31	3.31	3.31	3.31	3.31	3.32	3.32	3.32	3.33	3.33	3.33	3.34	3.34	3.34	3.34	3.34
0.30	3.09	3.09	3.10	3.11	3.13	3.14	3.15	3.16	3.17	3.18	3.19	3.21	3.22	3.23	3.24	3.24
0.40	2.68	2.68	2.69	2.72	2.75	2.79	2.82	2.85	2.88	2.90	2.93	2.96	3.00	3.02	3.04	3.06
0.50	2.32	2.32	2.35	2.38	2.43	2.48	2.53	2.57	2.61	2.65	2.68	2.74	2.79	2.83	2.86	2.89
0.60	2.03	2.03	2.06	2.10	2.16	2.22	2.27	2.33	2.38	2.42	2.46	2.54	2.60	2.65	2.69	2.72
0.70	1.79	1.80	1.82	1.87	1.93	2.00	2.06	2.12	2.18	2.23	2.27	2.36	2.42	2.48	2.53	2.58
0.80	1.60	1.60	1.63	1.68	1.75	1.81	1.88	1.94	2.00	2.06	2.11	2.20	2.27	2.34	2.39	2.44
0.90	1.44	1.45	1.48	1.53	1.59	1.66	1.73	1.79	1.85	1.91	1.96	2.05	2.14	2.21	2.27	2.32
1.0	1.31	1.32	1.35	1.40	1.46	1.53	1.60	1.66	1.72	1.78	1.83	1.93	2.01	2.09	2.15	2.21
1.2	1.10	1.11	1.14	1.19	1.25	1.32	1.38	1.45	1.51	1.56	1.62	1.72	1.80	1.88	1.95	2.01
1.4	0.954	0.961	0.993	1.04	1.10	1.16	1.22	1.28	1.34	1.39	1.45	1.54	1.63	1.71	1.78	1.84
1.6	0.839	0.847	0.876	0.919	0.972	1.03	1.09	1.15	1.20	1.25	1.31	1.40	1.49	1.57	1.64	1.70
1.8	0.748	0.756	0.783	0.824	0.872	0.926	0.981	1.04	1.09	1.14	1.19	1.28	1.37	1.45	1.52	1.58
2.0	0.675	0.683	0.708	0.746	0.791	0.841	0.893	0.945	0.995	1.04	1.09	1.18	1.26	1.34	1.41	1.47
2.2	0.615	0.622	0.646	0.681	0.723	0.770	0.819	0.868	0.916	0.963	1.01	1.10	1.18	1.25	1.32	1.38
2.4	0.565	0.572	0.594	0.626	0.666	0.710	0.756	0.802	0.848	0.893	0.937	1.02	1.10	1.17	1.24	1.30
2.6	0.522	0.529	0.550	0.580	0.617	0.658	0.702	0.746	0.789	0.832	0.874	0.954	1.03	1.10	1.16	1.22
2.8	0.485	0.491	0.511	0.540	0.575	0.614	0.655	0.697	0.738	0.779	0.819	0.896	0.969	1.04	1.10	1.16
3.0	0.453	0.459	0.478	0.505	0.538	0.574	0.614	0.653	0.693	0.732	0.771	0.845	0.915	0.980	1.04	1.10

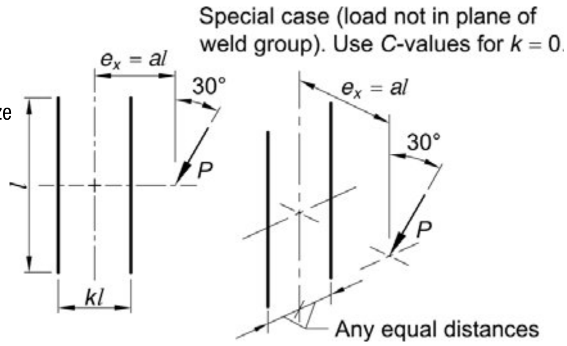
Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

Table 8-4 (continued)
Coefficients, C, for Eccentrically Loaded Weld Groups
Angle = 30°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



a	k 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	4.37	4.37	4.37	4.37	4.37	4.37	4.37	4.37	4.37	4.37	4.37	4.37	4.37	4.37	4.37	4.37
0.10	4.05	4.05	4.05	4.05	4.06	4.06	4.07	4.08	4.08	4.08	4.08	4.08	4.08	4.08	4.07	4.06
0.15	3.83	3.83	3.83	3.84	3.84	3.84	3.85	3.85	3.86	3.87	3.87	3.89	3.91	3.92	3.92	3.93
0.20	3.64	3.64	3.64	3.65	3.65	3.66	3.67	3.68	3.69	3.70	3.71	3.72	3.74	3.76	3.77	3.79
0.25	3.43	3.43	3.43	3.45	3.46	3.48	3.50	3.51	3.53	3.54	3.56	3.58	3.60	3.62	3.64	3.66
0.30	3.22	3.22	3.23	3.24	3.27	3.30	3.32	3.35	3.37	3.39	3.41	3.45	3.48	3.50	3.52	3.54
0.40	2.81	2.81	2.83	2.86	2.90	2.94	2.99	3.03	3.07	3.11	3.14	3.19	3.24	3.28	3.31	3.34
0.50	2.46	2.46	2.49	2.53	2.58	2.64	2.69	2.75	2.80	2.85	2.89	2.96	3.02	3.08	3.12	3.16
0.60	2.17	2.17	2.20	2.25	2.31	2.37	2.44	2.50	2.56	2.62	2.67	2.75	2.83	2.89	2.94	2.99
0.70	1.93	1.93	1.96	2.02	2.08	2.15	2.22	2.29	2.36	2.42	2.47	2.57	2.65	2.72	2.78	2.84
0.80	1.73	1.74	1.77	1.82	1.89	1.96	2.03	2.11	2.18	2.24	2.30	2.40	2.49	2.57	2.64	2.69
0.90	1.57	1.57	1.61	1.66	1.73	1.80	1.88	1.95	2.02	2.08	2.14	2.25	2.34	2.43	2.50	2.56
1.0	1.43	1.44	1.47	1.52	1.59	1.66	1.74	1.81	1.88	1.95	2.01	2.12	2.22	2.30	2.38	2.44
1.2	1.21	1.22	1.25	1.31	1.37	1.44	1.51	1.59	1.65	1.72	1.78	1.89	1.99	2.08	2.16	2.23
1.4	1.05	1.06	1.09	1.14	1.20	1.27	1.34	1.41	1.47	1.53	1.59	1.71	1.81	1.90	1.98	2.05
1.6	0.926	0.934	0.966	1.01	1.07	1.13	1.20	1.26	1.33	1.39	1.44	1.55	1.65	1.74	1.82	1.90
1.8	0.827	0.835	0.865	0.909	0.962	1.02	1.08	1.14	1.20	1.26	1.32	1.42	1.52	1.61	1.69	1.76
2.0	0.747	0.755	0.783	0.824	0.874	0.929	0.987	1.04	1.10	1.16	1.21	1.31	1.41	1.49	1.57	1.64
2.2	0.681	0.689	0.715	0.754	0.800	0.852	0.906	0.961	1.01	1.07	1.12	1.22	1.31	1.39	1.47	1.54
2.4	0.626	0.634	0.658	0.694	0.737	0.786	0.837	0.889	0.940	0.990	1.04	1.13	1.22	1.30	1.38	1.45
2.6	0.579	0.586	0.609	0.643	0.684	0.729	0.778	0.827	0.875	0.924	0.971	1.06	1.15	1.23	1.30	1.37
2.8	0.538	0.545	0.567	0.599	0.637	0.680	0.726	0.773	0.819	0.865	0.910	0.997	1.08	1.16	1.23	1.30
3.0	0.503	0.510	0.530	0.560	0.596	0.637	0.681	0.725	0.769	0.813	0.856	0.940	1.02	1.09	1.16	1.23

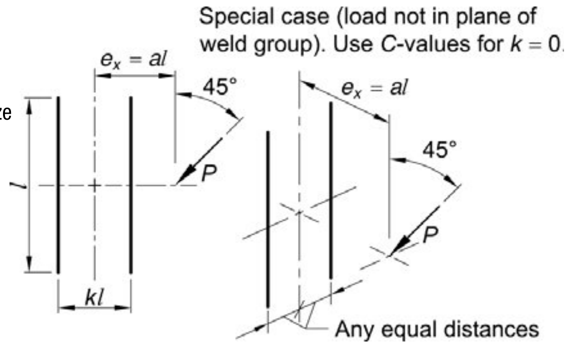
Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

Table 8-4 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 45°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	4.82	4.82	4.82	4.82	4.82	4.82	4.82	4.82	4.82	4.82	4.82	4.82	4.82	4.82	4.82	4.82
0.10	4.49	4.49	4.50	4.51	4.53	4.55	4.57	4.59	4.61	4.62	4.63	4.66	4.67	4.68	4.69	4.69
0.15	4.18	4.18	4.20	4.23	4.26	4.30	4.34	4.37	4.40	4.43	4.46	4.50	4.54	4.57	4.60	4.61
0.20	3.92	3.92	3.94	3.96	3.99	4.03	4.08	4.13	4.18	4.22	4.26	4.33	4.38	4.43	4.47	4.50
0.25	3.70	3.70	3.71	3.74	3.77	3.81	3.86	3.91	3.96	4.01	4.06	4.14	4.21	4.27	4.33	4.37
0.30	3.49	3.49	3.51	3.54	3.57	3.62	3.67	3.72	3.77	3.81	3.86	3.96	4.04	4.12	4.18	4.23
0.40	3.10	3.10	3.12	3.16	3.21	3.27	3.33	3.39	3.45	3.50	3.55	3.64	3.73	3.82	3.90	3.96
0.50	2.75	2.76	2.79	2.83	2.89	2.96	3.03	3.10	3.17	3.24	3.29	3.39	3.48	3.56	3.64	3.72
0.60	2.46	2.47	2.50	2.55	2.62	2.70	2.77	2.85	2.93	3.00	3.06	3.17	3.27	3.36	3.43	3.50
0.70	2.21	2.22	2.26	2.31	2.39	2.47	2.55	2.63	2.71	2.79	2.85	2.98	3.08	3.17	3.25	3.33
0.80	2.01	2.01	2.05	2.11	2.19	2.27	2.35	2.44	2.52	2.60	2.67	2.80	2.91	3.01	3.09	3.17
0.90	1.83	1.84	1.88	1.94	2.01	2.10	2.18	2.27	2.35	2.43	2.51	2.64	2.75	2.85	2.95	3.03
1.0	1.68	1.69	1.73	1.79	1.87	1.95	2.04	2.12	2.20	2.28	2.36	2.49	2.61	2.72	2.81	2.89
1.2	1.44	1.45	1.49	1.55	1.62	1.70	1.79	1.87	1.95	2.03	2.11	2.24	2.36	2.47	2.57	2.66
1.4	1.25	1.26	1.30	1.36	1.43	1.51	1.59	1.67	1.75	1.83	1.90	2.03	2.15	2.26	2.36	2.45
1.6	1.11	1.12	1.16	1.21	1.28	1.35	1.43	1.51	1.58	1.66	1.73	1.86	1.98	2.09	2.19	2.28
1.8	0.996	1.01	1.04	1.09	1.15	1.22	1.30	1.37	1.44	1.51	1.58	1.71	1.82	1.93	2.03	2.12
2.0	0.902	0.911	0.944	0.993	1.05	1.12	1.19	1.26	1.32	1.39	1.46	1.58	1.69	1.80	1.90	1.99
2.2	0.824	0.833	0.864	0.910	0.965	1.03	1.09	1.16	1.22	1.29	1.35	1.47	1.58	1.68	1.78	1.87
2.4	0.758	0.767	0.796	0.839	0.891	0.949	1.01	1.07	1.14	1.20	1.26	1.37	1.48	1.58	1.67	1.76
2.6	0.702	0.711	0.738	0.778	0.827	0.882	0.940	1.00	1.06	1.12	1.17	1.28	1.39	1.49	1.58	1.66
2.8	0.653	0.662	0.688	0.726	0.772	0.823	0.879	0.936	0.992	1.05	1.10	1.21	1.31	1.40	1.49	1.58
3.0	0.611	0.619	0.644	0.680	0.723	0.772	0.825	0.879	0.932	0.986	1.04	1.14	1.24	1.33	1.42	1.50

Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

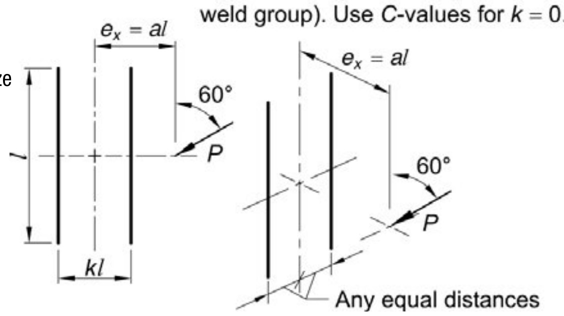
Table 8-4 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 60°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD			ASD		
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$	$D_{min} = \dfrac{P_u}{\phi CC_1 l}$	$l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$	$D_{min} = \dfrac{\Omega P_a}{CC_1 l}$	$l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

Special case (load not in plane of weld group). Use *C*-values for $k = 0$.



a	*k* = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	5.21	5.21	5.21	5.21	5.21	5.21	5.21	5.21	5.21	5.21	5.21	5.21	5.21	5.21	5.21	5.21
0.10	4.86	4.87	4.90	4.94	4.99	5.03	5.07	5.10	5.12	5.13	5.14	5.15	5.15	5.15	5.15	5.15
0.15	4.61	4.62	4.65	4.70	4.77	4.84	4.91	4.96	5.01	5.04	5.07	5.10	5.12	5.13	5.14	5.14
0.20	4.36	4.37	4.41	4.46	4.54	4.62	4.71	4.79	4.86	4.92	4.97	5.03	5.07	5.09	5.11	5.12
0.25	4.13	4.14	4.17	4.23	4.31	4.40	4.51	4.61	4.70	4.78	4.84	4.94	5.00	5.04	5.06	5.08
0.30	3.93	3.94	3.97	4.03	4.10	4.19	4.30	4.41	4.52	4.62	4.70	4.83	4.91	4.97	5.01	5.04
0.40	3.58	3.59	3.62	3.68	3.75	3.84	3.93	4.04	4.15	4.27	4.39	4.57	4.71	4.81	4.88	4.93
0.50	3.26	3.27	3.31	3.37	3.45	3.54	3.64	3.74	3.84	3.95	4.07	4.29	4.47	4.61	4.71	4.79
0.60	2.98	2.99	3.03	3.10	3.19	3.28	3.39	3.49	3.59	3.69	3.78	4.01	4.22	4.39	4.52	4.63
0.70	2.74	2.75	2.79	2.86	2.95	3.05	3.16	3.26	3.37	3.47	3.56	3.76	3.97	4.16	4.32	4.45
0.80	2.52	2.53	2.58	2.65	2.75	2.85	2.96	3.06	3.17	3.27	3.37	3.55	3.74	3.94	4.11	4.26
0.90	2.34	2.35	2.39	2.47	2.56	2.67	2.78	2.88	2.99	3.09	3.19	3.37	3.54	3.72	3.90	4.07
1.0	2.17	2.18	2.23	2.31	2.40	2.50	2.61	2.72	2.83	2.93	3.03	3.21	3.37	3.54	3.71	3.88
1.2	1.89	1.90	1.95	2.03	2.12	2.23	2.33	2.44	2.54	2.65	2.74	2.93	3.09	3.24	3.39	3.54
1.4	1.67	1.69	1.73	1.81	1.90	2.00	2.10	2.20	2.31	2.41	2.50	2.68	2.85	2.99	3.13	3.27
1.6	1.50	1.51	1.56	1.63	1.71	1.81	1.91	2.01	2.11	2.20	2.30	2.47	2.63	2.78	2.92	3.05
1.8	1.35	1.36	1.41	1.48	1.56	1.65	1.74	1.84	1.94	2.03	2.12	2.29	2.45	2.60	2.73	2.85
2.0	1.23	1.24	1.29	1.35	1.43	1.51	1.60	1.70	1.79	1.88	1.97	2.13	2.29	2.43	2.56	2.69
2.2	1.13	1.14	1.18	1.24	1.32	1.40	1.48	1.57	1.66	1.75	1.83	1.99	2.14	2.28	2.41	2.54
2.4	1.04	1.06	1.10	1.15	1.22	1.30	1.38	1.46	1.55	1.63	1.71	1.87	2.02	2.15	2.28	2.40
2.6	0.970	0.981	1.02	1.07	1.14	1.21	1.29	1.37	1.45	1.53	1.61	1.76	1.90	2.03	2.16	2.28
2.8	0.905	0.916	0.951	1.00	1.06	1.13	1.21	1.29	1.36	1.44	1.51	1.66	1.80	1.93	2.05	2.16
3.0	0.848	0.859	0.892	0.941	1.00	1.07	1.14	1.21	1.28	1.36	1.43	1.57	1.70	1.83	1.95	2.06

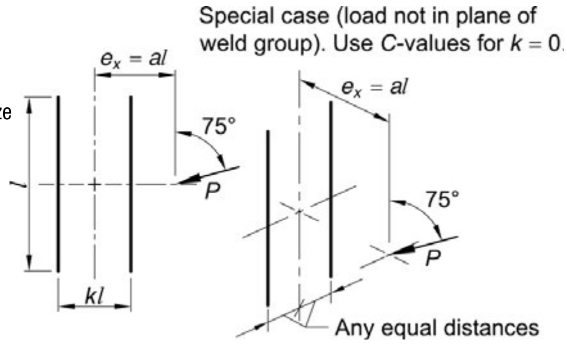
Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

Table 8-4 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 75°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD			ASD		
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$	$D_{min} = \dfrac{P_u}{\phi C C_1 l}$	$l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$	$D_{min} = \dfrac{\Omega P_a}{C C_1 l}$	$l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



a	*k* = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	5.47	5.47	5.47	5.47	5.47	5.47	5.47	5.47	5.47	5.47	5.47	5.47	5.47	5.47	5.47	5.47
0.10	5.17	5.19	5.25	5.32	5.38	5.42	5.44	5.45	5.45	5.46	5.46	5.46	5.46	5.46	5.45	5.45
0.15	5.00	5.03	5.10	5.19	5.28	5.34	5.38	5.41	5.43	5.44	5.45	5.45	5.45	5.45	5.45	5.45
0.20	4.85	4.87	4.95	5.06	5.16	5.25	5.32	5.36	5.39	5.41	5.42	5.44	5.45	5.45	5.45	5.45
0.25	4.71	4.73	4.80	4.92	5.04	5.15	5.24	5.30	5.34	5.37	5.39	5.42	5.43	5.44	5.44	5.45
0.30	4.57	4.59	4.65	4.78	4.92	5.04	5.15	5.23	5.28	5.33	5.36	5.40	5.42	5.43	5.44	5.44
0.40	4.32	4.33	4.39	4.51	4.67	4.82	4.95	5.06	5.15	5.22	5.27	5.33	5.37	5.40	5.41	5.42
0.50	4.09	4.11	4.17	4.27	4.43	4.60	4.76	4.89	5.00	5.09	5.16	5.25	5.32	5.35	5.38	5.40
0.60	3.88	3.90	3.96	4.07	4.21	4.38	4.56	4.71	4.84	4.95	5.04	5.16	5.25	5.30	5.34	5.36
0.70	3.69	3.71	3.77	3.87	4.01	4.18	4.36	4.53	4.68	4.80	4.91	5.06	5.17	5.24	5.29	5.33
0.80	3.51	3.53	3.59	3.70	3.83	3.99	4.17	4.35	4.51	4.65	4.77	4.96	5.08	5.17	5.24	5.28
0.90	3.34	3.36	3.42	3.53	3.66	3.81	3.99	4.18	4.35	4.50	4.64	4.84	4.99	5.10	5.17	5.23
1.0	3.18	3.20	3.27	3.37	3.50	3.65	3.83	4.01	4.19	4.35	4.49	4.73	4.90	5.02	5.11	5.18
1.2	2.90	2.92	2.99	3.09	3.22	3.37	3.53	3.70	3.88	4.06	4.22	4.49	4.69	4.85	4.97	5.06
1.4	2.65	2.67	2.74	2.85	2.97	3.11	3.27	3.43	3.61	3.78	3.95	4.24	4.48	4.67	4.81	4.92
1.6	2.44	2.46	2.53	2.63	2.75	2.89	3.04	3.19	3.36	3.53	3.70	4.01	4.27	4.48	4.65	4.78
1.8	2.26	2.27	2.34	2.44	2.56	2.69	2.84	2.99	3.14	3.30	3.47	3.78	4.06	4.29	4.48	4.63
2.0	2.09	2.11	2.18	2.27	2.39	2.52	2.66	2.80	2.95	3.10	3.26	3.57	3.86	4.10	4.31	4.48
2.2	1.95	1.97	2.03	2.13	2.24	2.36	2.50	2.63	2.78	2.92	3.07	3.38	3.66	3.92	4.14	4.32
2.4	1.82	1.84	1.90	1.99	2.10	2.22	2.35	2.48	2.62	2.76	2.90	3.20	3.48	3.74	3.97	4.16
2.6	1.71	1.73	1.79	1.88	1.98	2.10	2.22	2.35	2.48	2.62	2.75	3.04	3.31	3.57	3.80	4.01
2.8	1.61	1.63	1.69	1.77	1.87	1.98	2.10	2.23	2.36	2.49	2.62	2.88	3.16	3.41	3.64	3.85
3.0	1.52	1.54	1.60	1.68	1.77	1.88	2.00	2.12	2.24	2.37	2.49	2.75	3.01	3.26	3.49	3.71

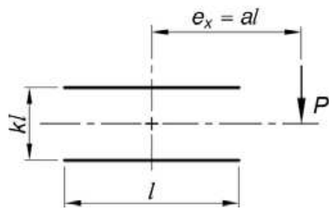
Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

Table 8-5
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 0°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



a	*k* = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57
0.10	4.32	4.36	4.48	4.65	4.82	4.97	5.11	5.21	5.29	5.35	5.39	5.45	5.48	5.50	5.52	5.53
0.15	3.90	3.94	4.04	4.20	4.39	4.58	4.75	4.90	5.02	5.12	5.20	5.31	5.38	5.42	5.45	5.48
0.20	3.54	3.57	3.67	3.81	3.99	4.20	4.40	4.57	4.73	4.86	4.97	5.13	5.24	5.32	5.37	5.41
0.25	3.22	3.25	3.34	3.47	3.64	3.85	4.06	4.26	4.43	4.59	4.72	4.93	5.08	5.19	5.26	5.32
0.30	2.94	2.97	3.06	3.19	3.34	3.53	3.74	3.95	4.14	4.32	4.47	4.72	4.91	5.04	5.14	5.22
0.40	2.48	2.51	2.60	2.71	2.85	3.01	3.19	3.40	3.61	3.81	3.99	4.29	4.54	4.72	4.87	4.99
0.50	2.14	2.17	2.24	2.34	2.47	2.62	2.78	2.95	3.15	3.35	3.54	3.88	4.16	4.39	4.58	4.73
0.60	1.87	1.89	1.96	2.06	2.17	2.31	2.45	2.61	2.78	2.96	3.15	3.50	3.81	4.06	4.28	4.46
0.70	1.65	1.68	1.74	1.83	1.93	2.06	2.19	2.33	2.48	2.64	2.81	3.17	3.48	3.75	3.99	4.19
0.80	1.48	1.50	1.56	1.64	1.74	1.85	1.97	2.10	2.24	2.38	2.54	2.87	3.18	3.46	3.71	3.92
0.90	1.34	1.36	1.41	1.49	1.58	1.68	1.79	1.91	2.04	2.17	2.31	2.61	2.92	3.20	3.45	3.68
1.0	1.22	1.24	1.29	1.36	1.44	1.54	1.64	1.75	1.87	1.99	2.12	2.39	2.69	2.97	3.22	3.45
1.2	1.04	1.05	1.10	1.16	1.23	1.31	1.41	1.50	1.60	1.71	1.82	2.05	2.30	2.56	2.81	3.03
1.4	0.900	0.914	0.952	1.00	1.07	1.14	1.23	1.31	1.40	1.49	1.59	1.79	2.00	2.24	2.47	2.69
1.6	0.794	0.807	0.840	0.888	0.946	1.01	1.08	1.16	1.24	1.33	1.41	1.59	1.78	1.98	2.19	2.40
1.8	0.710	0.722	0.752	0.795	0.848	0.907	0.973	1.04	1.12	1.19	1.27	1.43	1.60	1.77	1.96	2.16
2.0	0.643	0.653	0.680	0.719	0.767	0.822	0.881	0.945	1.01	1.08	1.15	1.30	1.45	1.61	1.77	1.95
2.2	0.586	0.596	0.621	0.657	0.701	0.751	0.805	0.864	0.925	0.988	1.05	1.19	1.33	1.47	1.62	1.78
2.4	0.539	0.548	0.571	0.604	0.644	0.691	0.741	0.795	0.852	0.910	0.970	1.09	1.22	1.35	1.49	1.64
2.6	0.498	0.507	0.528	0.559	0.597	0.640	0.687	0.737	0.789	0.844	0.899	1.01	1.13	1.26	1.38	1.51
2.8	0.464	0.472	0.491	0.520	0.555	0.595	0.639	0.686	0.735	0.786	0.838	0.946	1.06	1.17	1.29	1.41
3.0	0.434	0.441	0.459	0.486	0.519	0.557	0.598	0.642	0.688	0.736	0.785	0.886	0.990	1.10	1.21	1.32

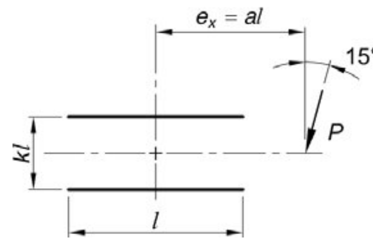
Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

Table 8-5 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 15°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $\quad D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $\quad l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $\quad D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $\quad l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



a	*k*															
	0	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	5.47	5.47	5.47	5.47	5.47	5.47	5.47	5.47	5.47	5.47	5.47	5.47	5.47	5.47	5.47	5.47
0.10	4.38	4.40	4.46	4.58	4.73	4.88	5.01	5.11	5.19	5.25	5.29	5.35	5.39	5.41	5.42	5.43
0.15	3.97	3.98	4.04	4.15	4.29	4.47	4.64	4.78	4.91	5.01	5.09	5.20	5.28	5.32	5.36	5.38
0.20	3.60	3.62	3.69	3.79	3.92	4.09	4.27	4.45	4.60	4.74	4.85	5.01	5.13	5.21	5.27	5.31
0.25	3.29	3.30	3.37	3.48	3.61	3.76	3.94	4.12	4.29	4.45	4.59	4.81	4.96	5.07	5.15	5.21
0.30	3.01	3.03	3.09	3.20	3.33	3.48	3.64	3.82	4.00	4.17	4.33	4.58	4.78	4.92	5.03	5.11
0.40	2.55	2.57	2.64	2.74	2.87	3.01	3.16	3.32	3.49	3.66	3.83	4.13	4.38	4.58	4.74	4.86
0.50	2.20	2.22	2.29	2.38	2.50	2.63	2.77	2.92	3.07	3.23	3.40	3.71	3.99	4.23	4.42	4.58
0.60	1.92	1.94	2.01	2.10	2.21	2.33	2.47	2.60	2.74	2.89	3.04	3.35	3.63	3.88	4.10	4.29
0.70	1.71	1.72	1.78	1.87	1.97	2.09	2.21	2.34	2.47	2.61	2.74	3.03	3.30	3.56	3.79	4.00
0.80	1.53	1.55	1.60	1.68	1.78	1.89	2.00	2.12	2.25	2.37	2.50	2.76	3.02	3.27	3.50	3.72
0.90	1.38	1.40	1.45	1.53	1.62	1.72	1.83	1.94	2.06	2.18	2.29	2.53	2.77	3.02	3.24	3.46
1.0	1.26	1.28	1.33	1.40	1.48	1.58	1.68	1.79	1.90	2.01	2.12	2.34	2.56	2.79	3.01	3.22
1.2	1.07	1.09	1.13	1.19	1.26	1.35	1.44	1.53	1.63	1.73	1.83	2.03	2.23	2.42	2.63	2.82
1.4	0.931	0.944	0.982	1.04	1.10	1.18	1.26	1.34	1.43	1.52	1.61	1.79	1.97	2.14	2.32	2.50
1.6	0.822	0.834	0.868	0.916	0.975	1.04	1.12	1.19	1.27	1.35	1.43	1.60	1.76	1.92	2.08	2.24
1.8	0.735	0.746	0.777	0.821	0.874	0.935	1.00	1.07	1.14	1.22	1.29	1.44	1.59	1.74	1.88	2.03
2.0	0.665	0.675	0.703	0.743	0.792	0.848	0.909	0.973	1.04	1.11	1.18	1.31	1.45	1.59	1.72	1.85
2.2	0.607	0.616	0.642	0.678	0.723	0.775	0.831	0.890	0.951	1.01	1.08	1.21	1.33	1.46	1.58	1.71
2.4	0.558	0.566	0.590	0.624	0.666	0.713	0.765	0.820	0.877	0.935	0.994	1.11	1.23	1.35	1.47	1.58
2.6	0.516	0.524	0.546	0.578	0.617	0.661	0.709	0.760	0.813	0.867	0.922	1.03	1.15	1.26	1.37	1.47
2.8	0.480	0.488	0.508	0.538	0.574	0.615	0.660	0.708	0.758	0.808	0.860	0.965	1.07	1.17	1.28	1.38
3.0	0.449	0.456	0.475	0.503	0.537	0.576	0.618	0.663	0.709	0.757	0.806	0.905	1.00	1.10	1.20	1.30

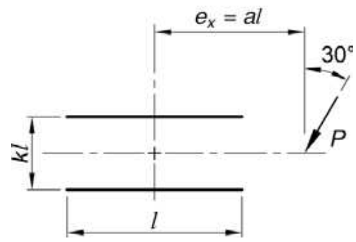
Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

Table 8-5 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 30°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	5.21	5.21	5.21	5.21	5.21	5.21	5.21	5.21	5.21	5.21	5.21	5.21	5.21	5.21	5.21	5.21
0.10	4.49	4.50	4.54	4.59	4.66	4.74	4.82	4.89	4.94	4.99	5.02	5.07	5.10	5.11	5.12	5.13
0.15	4.09	4.10	4.13	4.19	4.27	4.36	4.46	4.57	4.66	4.75	4.81	4.91	4.98	5.03	5.05	5.07
0.20	3.76	3.77	3.80	3.86	3.93	4.01	4.12	4.23	4.35	4.46	4.56	4.71	4.82	4.90	4.95	4.99
0.25	3.47	3.48	3.51	3.57	3.65	3.74	3.83	3.93	4.04	4.16	4.28	4.48	4.63	4.74	4.83	4.89
0.30	3.21	3.21	3.25	3.32	3.40	3.49	3.59	3.69	3.79	3.89	4.01	4.24	4.42	4.57	4.68	4.76
0.40	2.76	2.77	2.81	2.88	2.97	3.07	3.17	3.28	3.38	3.48	3.58	3.77	3.99	4.18	4.33	4.46
0.50	2.40	2.41	2.45	2.53	2.62	2.73	2.84	2.94	3.05	3.15	3.25	3.43	3.60	3.79	3.97	4.13
0.60	2.11	2.12	2.17	2.25	2.34	2.45	2.55	2.66	2.77	2.87	2.97	3.15	3.31	3.47	3.64	3.81
0.70	1.88	1.89	1.94	2.01	2.11	2.21	2.32	2.42	2.53	2.63	2.73	2.91	3.07	3.22	3.37	3.52
0.80	1.69	1.70	1.75	1.82	1.91	2.01	2.12	2.22	2.32	2.42	2.52	2.70	2.86	3.01	3.15	3.28
0.90	1.53	1.54	1.59	1.66	1.75	1.84	1.94	2.05	2.15	2.24	2.34	2.51	2.68	2.82	2.96	3.09
1.0	1.40	1.41	1.46	1.53	1.61	1.70	1.80	1.89	1.99	2.09	2.18	2.35	2.51	2.66	2.79	2.92
1.2	1.19	1.20	1.24	1.31	1.38	1.47	1.55	1.65	1.74	1.83	1.91	2.08	2.23	2.37	2.50	2.62
1.4	1.03	1.05	1.08	1.14	1.21	1.29	1.37	1.45	1.54	1.62	1.70	1.85	2.00	2.14	2.26	2.38
1.6	0.914	0.925	0.960	1.01	1.07	1.14	1.22	1.30	1.37	1.45	1.53	1.67	1.81	1.94	2.06	2.18
1.8	0.818	0.829	0.861	0.908	0.965	1.03	1.10	1.17	1.24	1.31	1.38	1.52	1.65	1.78	1.90	2.01
2.0	0.740	0.750	0.780	0.823	0.876	0.935	0.999	1.07	1.13	1.20	1.27	1.40	1.52	1.64	1.75	1.86
2.2	0.675	0.685	0.712	0.752	0.801	0.856	0.915	0.978	1.04	1.10	1.17	1.29	1.41	1.52	1.63	1.73
2.4	0.621	0.630	0.656	0.693	0.738	0.789	0.845	0.902	0.961	1.02	1.08	1.19	1.31	1.41	1.52	1.62
2.6	0.575	0.583	0.607	0.642	0.684	0.732	0.784	0.838	0.893	0.948	1.00	1.11	1.22	1.32	1.42	1.52
2.8	0.535	0.543	0.565	0.598	0.637	0.682	0.731	0.782	0.834	0.886	0.939	1.04	1.14	1.24	1.34	1.43
3.0	0.500	0.508	0.529	0.559	0.596	0.639	0.684	0.732	0.781	0.831	0.881	0.980	1.08	1.17	1.26	1.35

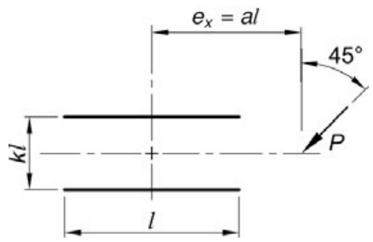
Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

Table 8-5 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 45°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



a	k: 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	4.82	4.82	4.82	4.82	4.82	4.82	4.82	4.82	4.82	4.82	4.82	4.82	4.82	4.82	4.82	4.82
0.10	4.49	4.49	4.50	4.51	4.53	4.55	4.57	4.59	4.61	4.62	4.63	4.66	4.67	4.68	4.69	4.69
0.15	4.18	4.18	4.20	4.23	4.26	4.30	4.34	4.37	4.40	4.43	4.46	4.50	4.54	4.57	4.60	4.61
0.20	3.92	3.92	3.94	3.96	3.99	4.03	4.08	4.13	4.18	4.22	4.26	4.33	4.38	4.43	4.47	4.50
0.25	3.70	3.70	3.71	3.74	3.77	3.81	3.86	3.91	3.96	4.01	4.06	4.14	4.21	4.27	4.33	4.37
0.30	3.49	3.49	3.51	3.54	3.57	3.62	3.67	3.72	3.77	3.81	3.86	3.96	4.04	4.12	4.18	4.23
0.40	3.10	3.10	3.12	3.16	3.21	3.27	3.33	3.39	3.45	3.50	3.55	3.64	3.73	3.82	3.90	3.96
0.50	2.75	2.76	2.79	2.83	2.89	2.96	3.03	3.10	3.17	3.24	3.29	3.39	3.48	3.56	3.64	3.72
0.60	2.46	2.47	2.50	2.55	2.62	2.70	2.77	2.85	2.93	3.00	3.06	3.17	3.27	3.36	3.43	3.50
0.70	2.21	2.22	2.26	2.31	2.39	2.47	2.55	2.63	2.71	2.79	2.85	2.98	3.08	3.17	3.25	3.33
0.80	2.01	2.01	2.05	2.11	2.19	2.27	2.35	2.44	2.52	2.60	2.67	2.80	2.91	3.01	3.09	3.17
0.90	1.83	1.84	1.88	1.94	2.01	2.10	2.18	2.27	2.35	2.43	2.51	2.64	2.75	2.85	2.95	3.03
1.0	1.68	1.69	1.73	1.79	1.87	1.95	2.04	2.12	2.20	2.28	2.36	2.49	2.61	2.72	2.81	2.89
1.2	1.44	1.45	1.49	1.55	1.62	1.70	1.79	1.87	1.95	2.03	2.11	2.24	2.36	2.47	2.57	2.66
1.4	1.25	1.26	1.30	1.36	1.43	1.51	1.59	1.67	1.75	1.83	1.90	2.03	2.15	2.26	2.36	2.45
1.6	1.11	1.12	1.16	1.21	1.28	1.35	1.43	1.51	1.58	1.66	1.73	1.86	1.98	2.09	2.19	2.28
1.8	0.996	1.01	1.04	1.09	1.15	1.22	1.30	1.37	1.44	1.51	1.58	1.71	1.82	1.93	2.03	2.12
2.0	0.902	0.911	0.944	0.993	1.05	1.12	1.19	1.26	1.32	1.39	1.46	1.58	1.69	1.80	1.90	1.99
2.2	0.824	0.833	0.864	0.910	0.965	1.03	1.09	1.16	1.22	1.29	1.35	1.47	1.58	1.68	1.78	1.87
2.4	0.758	0.767	0.796	0.839	0.891	0.949	1.01	1.07	1.14	1.20	1.26	1.37	1.48	1.58	1.67	1.76
2.6	0.702	0.711	0.738	0.778	0.827	0.882	0.940	1.00	1.06	1.12	1.17	1.28	1.39	1.49	1.58	1.66
2.8	0.653	0.662	0.688	0.726	0.772	0.823	0.879	0.936	0.992	1.05	1.10	1.21	1.31	1.40	1.49	1.58
3.0	0.611	0.619	0.644	0.680	0.723	0.772	0.825	0.879	0.932	0.986	1.04	1.14	1.24	1.33	1.42	1.50

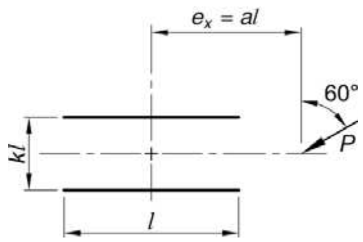
Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

Table 8-5 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 60°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



a	*k*															
	0	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	4.37	4.37	4.37	4.37	4.37	4.37	4.37	4.37	4.37	4.37	4.37	4.37	4.37	4.37	4.37	4.37
0.10	4.26	4.26	4.26	4.25	4.25	4.25	4.25	4.24	4.24	4.23	4.23	4.22	4.21	4.20	4.19	4.17
0.15	4.12	4.12	4.13	4.13	4.13	4.13	4.13	4.14	4.14	4.14	4.13	4.13	4.13	4.12	4.11	4.10
0.20	3.97	3.97	3.97	3.97	3.98	3.98	3.99	4.00	4.01	4.01	4.02	4.03	4.03	4.03	4.03	4.02
0.25	3.86	3.86	3.86	3.86	3.86	3.86	3.87	3.87	3.88	3.89	3.90	3.92	3.93	3.94	3.94	3.94
0.30	3.74	3.74	3.74	3.75	3.75	3.76	3.76	3.77	3.78	3.78	3.79	3.81	3.83	3.84	3.85	3.86
0.40	3.51	3.51	3.51	3.52	3.54	3.55	3.56	3.57	3.59	3.60	3.61	3.63	3.65	3.67	3.69	3.70
0.50	3.26	3.26	3.27	3.29	3.31	3.34	3.36	3.38	3.40	3.42	3.44	3.48	3.50	3.53	3.55	3.57
0.60	3.02	3.02	3.04	3.06	3.09	3.13	3.17	3.20	3.23	3.26	3.28	3.33	3.36	3.40	3.42	3.45
0.70	2.80	2.80	2.81	2.85	2.89	2.93	2.98	3.02	3.06	3.09	3.13	3.18	3.23	3.27	3.30	3.33
0.80	2.59	2.59	2.61	2.65	2.70	2.75	2.80	2.85	2.90	2.94	2.98	3.05	3.10	3.15	3.19	3.23
0.90	2.40	2.40	2.43	2.47	2.52	2.58	2.64	2.70	2.75	2.80	2.84	2.92	2.98	3.04	3.09	3.13
1.0	2.23	2.23	2.26	2.31	2.36	2.43	2.49	2.56	2.61	2.67	2.71	2.80	2.87	2.93	2.98	3.03
1.2	1.94	1.95	1.98	2.03	2.09	2.16	2.23	2.30	2.37	2.43	2.48	2.58	2.66	2.73	2.79	2.85
1.4	1.72	1.72	1.75	1.81	1.87	1.95	2.02	2.09	2.16	2.23	2.28	2.39	2.48	2.56	2.62	2.68
1.6	1.53	1.54	1.57	1.63	1.69	1.77	1.84	1.91	1.98	2.05	2.11	2.22	2.31	2.40	2.47	2.53
1.8	1.38	1.39	1.42	1.48	1.54	1.62	1.69	1.76	1.83	1.90	1.96	2.07	2.17	2.25	2.33	2.40
2.0	1.25	1.26	1.30	1.35	1.42	1.49	1.56	1.63	1.70	1.77	1.83	1.94	2.04	2.13	2.21	2.28
2.2	1.15	1.16	1.19	1.24	1.31	1.38	1.45	1.52	1.59	1.65	1.71	1.82	1.92	2.01	2.09	2.17
2.4	1.06	1.07	1.10	1.15	1.21	1.28	1.35	1.42	1.48	1.55	1.61	1.72	1.82	1.91	1.99	2.06
2.6	0.983	0.991	1.02	1.07	1.13	1.20	1.26	1.33	1.39	1.46	1.51	1.62	1.72	1.81	1.90	1.97
2.8	0.917	0.925	0.956	1.00	1.06	1.12	1.19	1.25	1.31	1.37	1.43	1.54	1.64	1.73	1.81	1.88
3.0	0.858	0.866	0.897	0.942	0.996	1.06	1.12	1.18	1.24	1.30	1.36	1.46	1.56	1.65	1.73	1.81

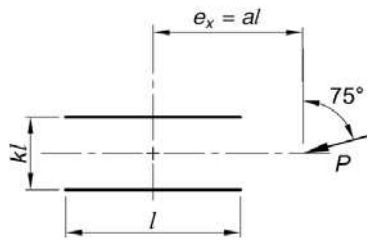
Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

Table 8-5 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 75°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD			ASD		
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$	$D_{min} = \dfrac{P_u}{\phi C C_1 l}$	$l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$	$D_{min} = \dfrac{\Omega P_a}{C C_1 l}$	$l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



a	*k*															
	0	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	3.96	3.96	3.96	3.96	3.96	3.96	3.96	3.96	3.96	3.96	3.96	3.96	3.96	3.96	3.96	3.96
0.10	3.82	3.83	3.84	3.84	3.85	3.85	3.85	3.85	3.85	3.85	3.85	3.84	3.82	3.80	3.78	3.76
0.15	3.85	3.86	3.86	3.86	3.86	3.85	3.85	3.85	3.84	3.83	3.83	3.81	3.79	3.77	3.75	3.73
0.20	3.84	3.84	3.84	3.84	3.83	3.83	3.82	3.82	3.81	3.80	3.80	3.78	3.76	3.74	3.72	3.71
0.25	3.83	3.83	3.83	3.82	3.82	3.81	3.80	3.80	3.79	3.78	3.77	3.75	3.73	3.72	3.70	3.68
0.30	3.82	3.82	3.81	3.81	3.80	3.79	3.78	3.77	3.76	3.76	3.75	3.73	3.71	3.69	3.67	3.66
0.40	3.78	3.78	3.77	3.76	3.75	3.74	3.73	3.72	3.71	3.70	3.69	3.67	3.66	3.64	3.62	3.61
0.50	3.72	3.72	3.71	3.70	3.69	3.68	3.67	3.66	3.65	3.64	3.64	3.62	3.60	3.59	3.57	3.56
0.60	3.65	3.64	3.64	3.63	3.62	3.61	3.60	3.60	3.59	3.58	3.57	3.56	3.54	3.53	3.52	3.51
0.70	3.56	3.55	3.55	3.54	3.54	3.53	3.52	3.52	3.51	3.51	3.50	3.49	3.48	3.47	3.47	3.46
0.80	3.46	3.45	3.45	3.45	3.45	3.44	3.44	3.44	3.44	3.43	3.43	3.43	3.42	3.42	3.41	3.41
0.90	3.35	3.35	3.35	3.35	3.35	3.35	3.35	3.35	3.35	3.36	3.36	3.36	3.36	3.36	3.36	3.35
1.0	3.23	3.23	3.24	3.24	3.25	3.25	3.26	3.27	3.27	3.28	3.28	3.29	3.30	3.30	3.30	3.30
1.2	3.00	3.00	3.01	3.02	3.04	3.06	3.08	3.09	3.11	3.12	3.14	3.16	3.17	3.19	3.20	3.20
1.4	2.78	2.78	2.79	2.81	2.84	2.87	2.90	2.93	2.95	2.97	2.99	3.02	3.05	3.07	3.09	3.10
1.6	2.57	2.57	2.59	2.62	2.65	2.69	2.73	2.77	2.80	2.83	2.85	2.90	2.93	2.96	2.99	3.01
1.8	2.38	2.38	2.40	2.44	2.48	2.53	2.58	2.62	2.66	2.69	2.72	2.78	2.82	2.86	2.89	2.91
2.0	2.21	2.21	2.24	2.27	2.32	2.38	2.43	2.48	2.52	2.56	2.60	2.66	2.72	2.76	2.80	2.83
2.2	2.05	2.06	2.09	2.13	2.18	2.24	2.30	2.35	2.40	2.44	2.48	2.56	2.61	2.66	2.71	2.74
2.4	1.92	1.92	1.95	2.00	2.05	2.12	2.18	2.24	2.29	2.33	2.38	2.45	2.52	2.57	2.62	2.66
2.6	1.80	1.80	1.83	1.88	1.94	2.00	2.07	2.13	2.18	2.23	2.28	2.36	2.43	2.49	2.54	2.58
2.8	1.69	1.69	1.72	1.77	1.83	1.90	1.97	2.03	2.09	2.14	2.19	2.27	2.35	2.41	2.46	2.51
3.0	1.59	1.60	1.63	1.68	1.74	1.81	1.87	1.94	2.00	2.05	2.10	2.19	2.27	2.33	2.39	2.44

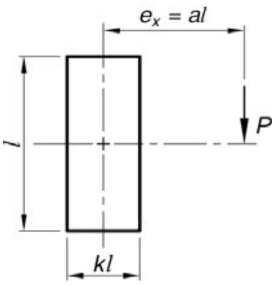
Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

Table 8-6
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 0°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $\quad D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $\quad l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $\quad D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $\quad l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



a	*k* = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	3.71	4.08	4.45	4.83	5.38	5.94	6.50	7.05	7.61	8.17	8.72	9.84	10.9	12.1	13.2	14.3
0.10	3.72	4.09	4.55	5.04	5.54	6.04	6.55	7.07	7.58	8.10	8.62	9.66	10.7	11.8	12.8	13.9
0.15	3.67	4.06	4.49	4.94	5.41	5.89	6.38	6.87	7.36	7.86	8.36	9.36	10.4	11.4	12.4	13.4
0.20	3.51	3.93	4.34	4.77	5.21	5.66	6.13	6.59	7.07	7.54	8.03	9.00	9.98	11.0	12.0	13.0
0.25	3.31	3.72	4.13	4.54	4.96	5.39	5.84	6.29	6.74	7.20	7.67	8.61	9.57	10.5	11.5	12.5
0.30	3.09	3.48	3.89	4.29	4.69	5.11	5.53	5.97	6.41	6.86	7.31	8.23	9.17	10.1	11.1	12.1
0.40	2.66	3.01	3.39	3.77	4.16	4.55	4.94	5.35	5.76	6.19	6.62	7.50	8.40	9.33	10.3	11.2
0.50	2.30	2.60	2.94	3.30	3.67	4.04	4.41	4.79	5.19	5.59	6.00	6.84	7.71	8.61	9.52	10.5
0.60	2.00	2.27	2.57	2.90	3.25	3.60	3.96	4.32	4.69	5.07	5.46	6.27	7.11	7.97	8.86	9.77
0.70	1.76	2.00	2.27	2.57	2.90	3.24	3.57	3.91	4.26	4.63	5.00	5.77	6.58	7.41	8.27	9.15
0.80	1.57	1.78	2.02	2.30	2.61	2.93	3.25	3.57	3.90	4.24	4.60	5.34	6.11	6.91	7.74	8.59
0.90	1.41	1.60	1.82	2.08	2.36	2.67	2.97	3.27	3.59	3.91	4.25	4.95	5.69	6.45	7.25	8.07
1.0	1.28	1.45	1.66	1.90	2.16	2.45	2.73	3.02	3.32	3.62	3.94	4.61	5.31	6.04	6.81	7.60
1.2	1.08	1.22	1.40	1.61	1.84	2.09	2.35	2.61	2.87	3.15	3.43	4.03	4.67	5.34	6.04	6.77
1.4	0.928	1.05	1.21	1.40	1.60	1.83	2.06	2.29	2.53	2.78	3.03	3.58	4.16	4.77	5.42	6.09
1.6	0.815	0.927	1.07	1.23	1.42	1.62	1.83	2.04	2.25	2.48	2.71	3.21	3.74	4.30	4.90	5.53
1.8	0.727	0.827	0.954	1.10	1.27	1.45	1.64	1.83	2.03	2.24	2.45	2.90	3.39	3.92	4.47	5.05
2.0	0.655	0.746	0.861	0.996	1.15	1.31	1.49	1.66	1.85	2.04	2.23	2.65	3.10	3.59	4.10	4.65
2.2	0.597	0.679	0.785	0.908	1.05	1.20	1.36	1.52	1.69	1.87	2.05	2.44	2.86	3.31	3.79	4.30
2.4	0.547	0.623	0.721	0.835	0.963	1.10	1.25	1.41	1.56	1.72	1.89	2.26	2.65	3.07	3.52	4.00
2.6	0.506	0.576	0.666	0.772	0.891	1.02	1.16	1.30	1.45	1.60	1.76	2.10	2.47	2.86	3.29	3.74
2.8	0.470	0.536	0.620	0.718	0.829	0.950	1.08	1.21	1.35	1.49	1.64	1.96	2.31	2.68	3.08	3.50
3.0	0.439	0.500	0.579	0.671	0.775	0.888	1.01	1.14	1.27	1.40	1.54	1.84	2.17	2.52	2.90	3.30

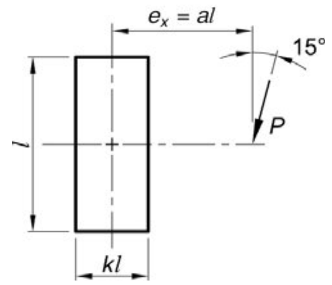
Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

Table 8-6 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 15°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $\quad D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $\quad l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $\quad D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $\quad l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



a	*k* 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	3.96	4.39	4.94	5.48	6.03	6.57	7.12	7.66	8.21	8.75	9.30	10.4	11.5	12.6	13.7	14.7
0.10	3.79	4.22	4.70	5.19	5.70	6.21	6.73	7.25	7.77	8.29	8.82	9.87	10.9	12.0	13.0	14.1
0.15	3.68	4.14	4.59	5.05	5.53	6.01	6.49	6.98	7.48	7.97	8.47	9.47	10.5	11.5	12.5	13.6
0.20	3.51	3.95	4.40	4.85	5.31	5.76	6.23	6.69	7.17	7.64	8.12	9.09	10.1	11.1	12.1	13.1
0.25	3.31	3.72	4.16	4.61	5.04	5.49	5.93	6.38	6.84	7.30	7.76	8.71	9.66	10.6	11.6	12.6
0.30	3.09	3.48	3.90	4.33	4.76	5.19	5.62	6.06	6.50	6.95	7.40	8.32	9.26	10.2	11.2	12.2
0.40	2.68	3.02	3.39	3.79	4.20	4.62	5.02	5.44	5.86	6.29	6.72	7.60	8.51	9.43	10.4	11.3
0.50	2.32	2.62	2.95	3.31	3.70	4.10	4.49	4.88	5.29	5.69	6.10	6.95	7.83	8.73	9.65	10.6
0.60	2.03	2.29	2.59	2.92	3.28	3.65	4.03	4.41	4.79	5.17	5.57	6.39	7.23	8.10	8.99	9.91
0.70	1.79	2.03	2.30	2.60	2.93	3.28	3.64	4.00	4.36	4.73	5.11	5.89	6.70	7.55	8.41	9.30
0.80	1.60	1.81	2.05	2.33	2.64	2.97	3.31	3.65	4.00	4.35	4.71	5.45	6.23	7.04	7.88	8.73
0.90	1.44	1.63	1.86	2.11	2.40	2.71	3.03	3.36	3.68	4.01	4.35	5.07	5.81	6.59	7.39	8.22
1.0	1.31	1.48	1.69	1.93	2.20	2.49	2.80	3.10	3.40	3.72	4.05	4.72	5.43	6.18	6.95	7.75
1.2	1.10	1.25	1.43	1.64	1.88	2.14	2.41	2.68	2.95	3.24	3.53	4.14	4.79	5.47	6.19	6.93
1.4	0.954	1.08	1.24	1.43	1.64	1.87	2.11	2.36	2.60	2.86	3.12	3.68	4.27	4.90	5.56	6.25
1.6	0.839	0.953	1.10	1.26	1.45	1.66	1.87	2.10	2.32	2.55	2.79	3.30	3.85	4.43	5.04	5.68
1.8	0.748	0.850	0.980	1.13	1.30	1.49	1.68	1.89	2.09	2.31	2.53	3.00	3.50	4.03	4.60	5.19
2.0	0.675	0.768	0.885	1.02	1.18	1.35	1.53	1.72	1.90	2.10	2.30	2.74	3.20	3.70	4.23	4.78
2.2	0.615	0.700	0.808	0.934	1.08	1.23	1.40	1.57	1.75	1.93	2.12	2.52	2.95	3.41	3.91	4.43
2.4	0.565	0.642	0.742	0.859	0.990	1.13	1.29	1.45	1.61	1.78	1.96	2.33	2.74	3.17	3.63	4.12
2.6	0.522	0.594	0.687	0.795	0.916	1.05	1.19	1.34	1.50	1.65	1.82	2.17	2.55	2.96	3.39	3.85
2.8	0.485	0.552	0.639	0.739	0.853	0.977	1.11	1.25	1.40	1.54	1.70	2.03	2.38	2.77	3.18	3.61
3.0	0.453	0.516	0.597	0.691	0.798	0.914	1.04	1.17	1.31	1.45	1.59	1.90	2.24	2.60	2.99	3.40

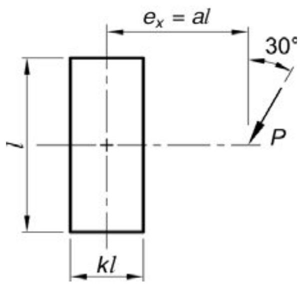
Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

Table 8-6 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 30°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD			ASD		
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$	$D_{min} = \dfrac{P_u}{\phi CC_1 l}$	$l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$	$D_{min} = \dfrac{\Omega P_a}{CC_1 l}$	$l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



a	***k***															
	0	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	4.37	4.89	5.40	5.91	6.43	6.94	7.46	7.97	8.48	9.00	9.51	10.5	11.6	12.6	13.6	14.7
0.10	4.05	4.60	5.13	5.65	6.16	6.67	7.17	7.68	8.18	8.69	9.20	10.2	11.2	12.3	13.3	14.4
0.15	3.83	4.33	4.85	5.36	5.86	6.36	6.86	7.35	7.85	8.35	8.85	9.85	10.9	11.9	12.9	14.0
0.20	3.64	4.09	4.57	5.06	5.55	6.04	6.52	7.00	7.48	7.97	8.46	9.45	10.4	11.5	12.5	13.5
0.25	3.43	3.85	4.30	4.77	5.24	5.72	6.20	6.66	7.12	7.59	8.06	9.03	10.0	11.0	12.1	13.1
0.30	3.22	3.61	4.03	4.47	4.93	5.40	5.87	6.33	6.78	7.24	7.70	8.64	9.61	10.6	11.6	12.6
0.40	2.81	3.15	3.53	3.93	4.36	4.80	5.25	5.71	6.15	6.59	7.03	7.94	8.86	9.81	10.8	11.8
0.50	2.46	2.77	3.10	3.47	3.86	4.28	4.71	5.15	5.58	6.01	6.44	7.31	8.21	9.14	10.1	11.0
0.60	2.17	2.44	2.75	3.08	3.45	3.84	4.25	4.67	5.09	5.50	5.91	6.76	7.64	8.54	9.45	10.4
0.70	1.93	2.17	2.45	2.76	3.11	3.47	3.86	4.26	4.67	5.06	5.46	6.27	7.12	7.99	8.88	9.79
0.80	1.73	1.95	2.21	2.50	2.82	3.16	3.53	3.91	4.30	4.67	5.05	5.84	6.65	7.49	8.35	9.24
0.90	1.57	1.77	2.00	2.28	2.58	2.90	3.25	3.61	3.97	4.33	4.70	5.44	6.23	7.04	7.87	8.74
1.0	1.43	1.61	1.83	2.09	2.37	2.68	3.00	3.35	3.69	4.03	4.38	5.09	5.84	6.63	7.44	8.27
1.2	1.21	1.37	1.56	1.79	2.04	2.31	2.61	2.91	3.22	3.53	3.85	4.50	5.19	5.92	6.67	7.46
1.4	1.05	1.19	1.36	1.56	1.79	2.03	2.29	2.57	2.85	3.13	3.42	4.02	4.66	5.33	6.03	6.76
1.6	0.926	1.05	1.20	1.38	1.59	1.81	2.05	2.29	2.55	2.80	3.07	3.62	4.21	4.84	5.49	6.18
1.8	0.827	0.938	1.08	1.24	1.43	1.63	1.84	2.07	2.30	2.54	2.78	3.29	3.84	4.42	5.03	5.67
2.0	0.747	0.848	0.977	1.13	1.29	1.48	1.68	1.89	2.10	2.32	2.54	3.02	3.52	4.07	4.64	5.24
2.2	0.681	0.774	0.892	1.03	1.18	1.35	1.54	1.73	1.93	2.13	2.34	2.78	3.26	3.76	4.30	4.86
2.4	0.626	0.711	0.821	0.948	1.09	1.25	1.42	1.60	1.78	1.97	2.16	2.58	3.02	3.50	4.00	4.53
2.6	0.579	0.658	0.760	0.878	1.01	1.16	1.31	1.48	1.65	1.83	2.01	2.40	2.82	3.27	3.74	4.24
2.8	0.538	0.612	0.707	0.818	0.942	1.08	1.23	1.38	1.54	1.71	1.88	2.25	2.64	3.06	3.51	3.99
3.0	0.503	0.572	0.661	0.765	0.882	1.01	1.15	1.29	1.45	1.60	1.77	2.11	2.48	2.88	3.31	3.76

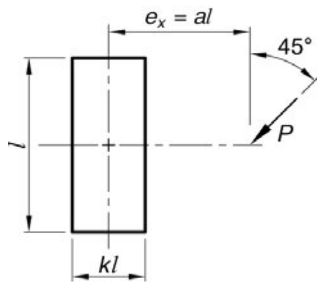
Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

Table 8-6 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 45°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	4.82	5.14	5.61	6.08	6.54	7.01	7.48	7.95	8.41	8.88	9.35	10.3	11.2	12.2	13.1	14.0
0.10	4.49	4.99	5.48	5.96	6.45	6.94	7.43	7.92	8.41	8.90	9.39	10.4	11.4	12.3	13.3	14.3
0.15	4.18	4.69	5.19	5.67	6.16	6.65	7.15	7.65	8.15	8.65	9.14	10.1	11.1	12.1	13.1	14.1
0.20	3.92	4.39	4.87	5.36	5.84	6.33	6.83	7.33	7.84	8.34	8.85	9.86	10.9	11.9	12.9	13.9
0.25	3.70	4.13	4.58	5.05	5.52	6.01	6.50	7.00	7.50	8.02	8.53	9.54	10.6	11.6	12.6	13.6
0.30	3.49	3.89	4.32	4.76	5.22	5.70	6.18	6.67	7.18	7.69	8.20	9.21	10.2	11.3	12.3	13.3
0.40	3.10	3.45	3.84	4.25	4.68	5.13	5.60	6.07	6.56	7.06	7.57	8.56	9.57	10.6	11.6	12.7
0.50	2.75	3.07	3.42	3.81	4.22	4.65	5.10	5.56	6.03	6.52	7.01	7.96	8.94	9.96	11.0	12.0
0.60	2.46	2.75	3.08	3.44	3.83	4.24	4.67	5.11	5.58	6.05	6.52	7.43	8.38	9.37	10.4	11.4
0.70	2.21	2.48	2.78	3.12	3.49	3.88	4.30	4.73	5.17	5.62	6.08	6.96	7.87	8.83	9.81	10.8
0.80	2.01	2.25	2.53	2.85	3.20	3.57	3.97	4.39	4.81	5.25	5.69	6.54	7.42	8.34	9.29	10.3
0.90	1.83	2.06	2.32	2.62	2.95	3.31	3.69	4.08	4.49	4.91	5.33	6.16	7.01	7.89	8.81	9.76
1.0	1.68	1.89	2.13	2.42	2.73	3.08	3.44	3.81	4.20	4.60	5.01	5.81	6.63	7.48	8.38	9.30
1.2	1.44	1.62	1.84	2.10	2.38	2.69	3.02	3.36	3.71	4.08	4.46	5.20	5.97	6.77	7.60	8.47
1.4	1.25	1.41	1.61	1.84	2.10	2.38	2.68	2.99	3.32	3.65	4.00	4.69	5.41	6.17	6.95	7.76
1.6	1.11	1.25	1.43	1.64	1.88	2.13	2.40	2.69	2.99	3.30	3.62	4.27	4.94	5.65	6.38	7.15
1.8	0.996	1.13	1.29	1.48	1.70	1.93	2.18	2.44	2.72	3.00	3.30	3.90	4.53	5.20	5.89	6.62
2.0	0.902	1.02	1.17	1.35	1.55	1.76	1.99	2.23	2.49	2.75	3.03	3.59	4.18	4.81	5.46	6.15
2.2	0.824	0.934	1.07	1.24	1.42	1.62	1.83	2.06	2.29	2.54	2.80	3.32	3.88	4.47	5.09	5.74
2.4	0.758	0.860	0.990	1.14	1.31	1.49	1.69	1.90	2.12	2.36	2.60	3.09	3.62	4.17	4.76	5.37
2.6	0.702	0.797	0.918	1.06	1.22	1.39	1.57	1.77	1.98	2.19	2.42	2.89	3.38	3.91	4.46	5.05
2.8	0.653	0.742	0.855	0.987	1.14	1.30	1.47	1.66	1.85	2.05	2.27	2.71	3.18	3.67	4.20	4.76
3.0	0.611	0.694	0.801	0.925	1.06	1.22	1.38	1.55	1.74	1.93	2.13	2.55	2.99	3.47	3.97	4.50

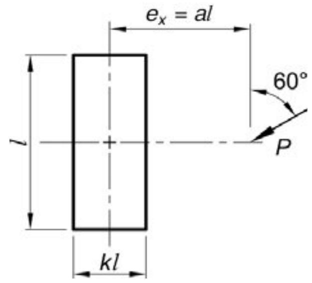
Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

Table 8-6 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 60°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $\quad D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $\quad l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $\quad D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $\quad l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	5.21	5.58	6.01	6.45	6.89	7.33	7.76	8.20	8.64	9.07	9.51	10.4	11.3	12.1	13.0	13.9
0.10	4.86	5.29	5.73	6.19	6.65	7.12	7.59	8.06	8.52	8.98	9.43	10.3	11.2	12.1	13.0	13.9
0.15	4.61	5.04	5.48	5.93	6.40	6.88	7.37	7.86	8.34	8.81	9.28	10.2	11.1	12.0	12.9	13.8
0.20	4.36	4.80	5.23	5.67	6.14	6.63	7.13	7.62	8.12	8.61	9.10	10.1	11.0	11.9	12.8	13.7
0.25	4.13	4.56	4.99	5.43	5.89	6.37	6.87	7.38	7.89	8.39	8.89	9.87	10.8	11.8	12.7	13.6
0.30	3.93	4.34	4.76	5.19	5.64	6.12	6.62	7.13	7.65	8.16	8.67	9.67	10.6	11.6	12.5	13.5
0.40	3.58	3.95	4.35	4.77	5.20	5.66	6.15	6.66	7.17	7.69	8.21	9.24	10.2	11.2	12.2	13.2
0.50	3.26	3.60	3.98	4.39	4.82	5.27	5.74	6.24	6.75	7.27	7.78	8.81	9.83	10.8	11.8	12.8
0.60	2.98	3.30	3.66	4.05	4.47	4.92	5.39	5.86	6.36	6.87	7.38	8.41	9.44	10.4	11.4	12.4
0.70	2.74	3.04	3.38	3.75	4.17	4.60	5.06	5.52	6.00	6.50	7.01	8.03	9.05	10.1	11.1	12.1
0.80	2.52	2.81	3.13	3.49	3.89	4.31	4.75	5.21	5.68	6.16	6.65	7.66	8.68	9.70	10.7	11.7
0.90	2.34	2.60	2.91	3.26	3.64	4.05	4.48	4.92	5.38	5.85	6.33	7.32	8.32	9.32	10.3	11.3
1.0	2.17	2.42	2.71	3.05	3.42	3.82	4.23	4.66	5.11	5.56	6.03	6.99	7.98	8.96	9.95	10.9
1.2	1.89	2.12	2.39	2.70	3.04	3.41	3.79	4.20	4.61	5.05	5.49	6.40	7.33	8.28	9.24	10.2
1.4	1.67	1.88	2.12	2.41	2.73	3.07	3.43	3.80	4.20	4.60	5.02	5.89	6.76	7.66	8.60	9.55
1.6	1.50	1.68	1.91	2.18	2.47	2.78	3.12	3.47	3.84	4.22	4.62	5.44	6.26	7.12	8.01	8.93
1.8	1.35	1.52	1.73	1.98	2.25	2.54	2.86	3.19	3.53	3.89	4.26	5.04	5.82	6.63	7.49	8.37
2.0	1.23	1.39	1.59	1.81	2.07	2.34	2.63	2.94	3.26	3.60	3.96	4.69	5.44	6.20	7.02	7.86
2.2	1.13	1.28	1.46	1.67	1.91	2.16	2.44	2.73	3.03	3.35	3.68	4.38	5.10	5.82	6.59	7.41
2.4	1.04	1.18	1.35	1.55	1.77	2.01	2.27	2.54	2.83	3.13	3.44	4.10	4.79	5.49	6.22	6.99
2.6	0.970	1.10	1.26	1.45	1.65	1.88	2.12	2.38	2.65	2.94	3.23	3.86	4.51	5.18	5.88	6.62
2.8	0.905	1.02	1.18	1.35	1.55	1.76	1.99	2.23	2.49	2.76	3.04	3.64	4.26	4.90	5.57	6.28
3.0	0.848	0.961	1.10	1.27	1.46	1.66	1.88	2.11	2.35	2.61	2.87	3.44	4.04	4.65	5.29	5.97

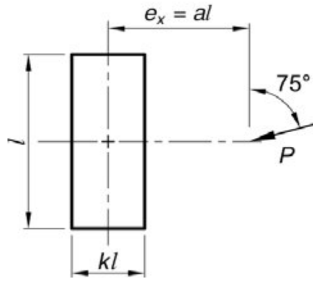
Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

Table 8-6 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 75°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



a	*k*															
	0	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	5.47	5.83	6.22	6.60	6.99	7.37	7.76	8.14	8.53	8.91	9.30	10.1	10.8	11.6	12.4	13.1
0.10	5.17	5.55	5.97	6.41	6.84	7.26	7.67	8.07	8.47	8.86	9.25	10.0	10.8	11.6	12.3	13.1
0.15	5.00	5.38	5.80	6.25	6.70	7.14	7.57	7.99	8.40	8.80	9.20	9.98	10.8	11.5	12.3	13.1
0.20	4.85	5.22	5.64	6.09	6.56	7.01	7.46	7.89	8.31	8.72	9.13	9.93	10.7	11.5	12.3	13.1
0.25	4.71	5.07	5.48	5.94	6.41	6.87	7.33	7.78	8.21	8.63	9.05	9.87	10.7	11.5	12.2	13.0
0.30	4.57	4.94	5.34	5.79	6.26	6.73	7.20	7.66	8.10	8.54	8.96	9.79	10.6	11.4	12.2	13.0
0.40	4.32	4.68	5.07	5.52	5.99	6.48	6.95	7.42	7.88	8.32	8.76	9.62	10.5	11.3	12.1	12.9
0.50	4.09	4.45	4.84	5.27	5.74	6.23	6.72	7.20	7.67	8.13	8.58	9.44	10.3	11.1	11.9	12.7
0.60	3.88	4.23	4.62	5.05	5.51	5.99	6.49	6.98	7.46	7.94	8.40	9.28	10.1	11.0	11.8	12.6
0.70	3.69	4.03	4.41	4.84	5.29	5.77	6.26	6.76	7.25	7.74	8.21	9.12	10.0	10.8	11.7	12.5
0.80	3.51	3.84	4.22	4.64	5.09	5.56	6.05	6.55	7.04	7.54	8.02	8.96	9.85	10.7	11.5	12.4
0.90	3.34	3.66	4.03	4.45	4.90	5.36	5.84	6.34	6.84	7.34	7.83	8.78	9.70	10.6	11.4	12.3
1.0	3.18	3.49	3.86	4.27	4.72	5.17	5.64	6.14	6.64	7.14	7.63	8.60	9.54	10.4	11.3	12.2
1.2	2.90	3.19	3.55	3.95	4.38	4.82	5.28	5.76	6.25	6.75	7.25	8.24	9.21	10.1	11.0	11.9
1.4	2.65	2.93	3.27	3.65	4.07	4.51	4.95	5.41	5.89	6.38	6.88	7.88	8.86	9.82	10.8	11.7
1.6	2.44	2.71	3.03	3.40	3.79	4.22	4.65	5.10	5.56	6.04	6.53	7.52	8.51	9.49	10.4	11.4
1.8	2.26	2.51	2.82	3.17	3.55	3.96	4.38	4.82	5.26	5.73	6.21	7.19	8.17	9.16	10.1	11.1
2.0	2.09	2.33	2.63	2.96	3.33	3.72	4.13	4.55	4.99	5.44	5.90	6.86	7.84	8.83	9.80	10.8
2.2	1.95	2.18	2.46	2.78	3.13	3.50	3.90	4.31	4.74	5.17	5.62	6.56	7.53	8.50	9.47	10.4
2.4	1.82	2.04	2.31	2.61	2.95	3.31	3.69	4.09	4.50	4.93	5.36	6.28	7.22	8.19	9.16	10.1
2.6	1.71	1.92	2.18	2.47	2.79	3.13	3.50	3.89	4.29	4.70	5.12	6.01	6.93	7.88	8.85	9.81
2.8	1.61	1.81	2.06	2.34	2.64	2.97	3.33	3.70	4.09	4.49	4.90	5.76	6.66	7.60	8.55	9.51
3.0	1.52	1.71	1.95	2.21	2.51	2.83	3.17	3.53	3.90	4.29	4.69	5.53	6.41	7.32	8.26	9.21

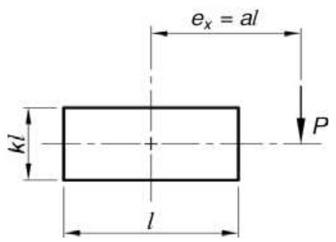
Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

Table 8-7
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 0°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



a	k: 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	5.57	5.88	6.20	6.51	6.83	7.15	7.46	7.78	8.09	8.41	8.72	9.35	9.98	10.6	11.2	11.9
0.10	4.32	4.68	5.08	5.54	6.02	6.49	6.95	7.40	7.82	8.23	8.62	9.37	10.1	10.8	11.5	12.1
0.15	3.90	4.24	4.65	5.08	5.55	6.04	6.52	7.00	7.47	7.92	8.36	9.18	9.96	10.7	11.4	12.1
0.20	3.54	3.86	4.26	4.69	5.14	5.61	6.10	6.60	7.08	7.56	8.03	8.92	9.76	10.6	11.3	12.1
0.25	3.22	3.53	3.91	4.34	4.77	5.23	5.71	6.20	6.69	7.19	7.67	8.61	9.50	10.3	11.2	12.0
0.30	2.94	3.24	3.60	4.01	4.44	4.88	5.35	5.83	6.32	6.82	7.31	8.27	9.20	10.1	11.0	11.8
0.40	2.48	2.76	3.09	3.46	3.87	4.30	4.73	5.18	5.65	6.13	6.62	7.60	8.57	9.52	10.4	11.3
0.50	2.14	2.38	2.69	3.03	3.40	3.80	4.21	4.64	5.07	5.53	6.00	6.96	7.93	8.90	9.85	10.8
0.60	1.87	2.09	2.37	2.68	3.02	3.39	3.78	4.18	4.59	5.02	5.46	6.38	7.34	8.30	9.26	10.2
0.70	1.65	1.86	2.11	2.40	2.71	3.05	3.41	3.79	4.18	4.58	5.00	5.87	6.79	7.73	8.69	9.64
0.80	1.48	1.67	1.90	2.16	2.45	2.77	3.10	3.46	3.82	4.20	4.60	5.42	6.30	7.21	8.14	9.09
0.90	1.34	1.51	1.73	1.97	2.24	2.53	2.84	3.17	3.52	3.88	4.25	5.03	5.86	6.73	7.64	8.56
1.0	1.22	1.38	1.58	1.81	2.06	2.33	2.62	2.92	3.25	3.59	3.94	4.68	5.47	6.31	7.18	8.07
1.2	1.04	1.17	1.35	1.55	1.76	2.00	2.26	2.53	2.82	3.12	3.43	4.10	4.81	5.57	6.37	7.21
1.4	0.900	1.02	1.17	1.35	1.54	1.75	1.98	2.22	2.48	2.75	3.03	3.64	4.29	4.98	5.71	6.48
1.6	0.794	0.902	1.04	1.19	1.37	1.56	1.76	1.98	2.21	2.45	2.71	3.26	3.85	4.48	5.16	5.85
1.8	0.710	0.807	0.930	1.07	1.23	1.40	1.59	1.78	1.99	2.22	2.45	2.95	3.49	4.08	4.69	5.33
2.0	0.643	0.731	0.842	0.972	1.12	1.27	1.44	1.62	1.81	2.02	2.23	2.69	3.19	3.73	4.30	4.89
2.2	0.586	0.667	0.770	0.888	1.02	1.17	1.32	1.49	1.66	1.85	2.05	2.48	2.94	3.44	3.97	4.51
2.4	0.539	0.613	0.708	0.818	0.941	1.07	1.22	1.37	1.54	1.71	1.89	2.29	2.72	3.18	3.68	4.19
2.6	0.498	0.568	0.656	0.758	0.872	0.996	1.13	1.27	1.43	1.59	1.76	2.13	2.53	2.97	3.42	3.90
2.8	0.464	0.528	0.611	0.706	0.812	0.929	1.05	1.19	1.33	1.48	1.64	1.99	2.37	2.77	3.20	3.65
3.0	0.434	0.494	0.571	0.661	0.760	0.870	0.988	1.11	1.25	1.39	1.54	1.87	2.22	2.60	3.01	3.43

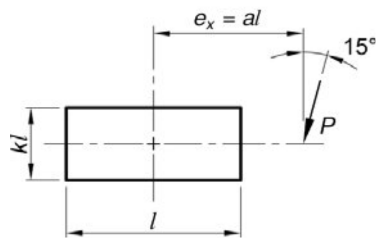
Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

Table 8-7 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 15°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



a	*k* = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	5.47	5.83	6.22	6.60	6.99	7.37	7.76	8.14	8.53	8.91	9.30	10.1	10.8	11.6	12.4	13.1
0.10	4.38	4.75	5.14	5.59	6.06	6.54	7.02	7.48	7.93	8.38	8.82	9.67	10.5	11.3	12.1	12.9
0.15	3.97	4.32	4.71	5.13	5.60	6.09	6.58	7.07	7.55	8.01	8.47	9.35	10.2	11.0	11.8	12.7
0.20	3.60	3.94	4.32	4.75	5.19	5.67	6.16	6.66	7.16	7.64	8.12	9.05	9.93	10.8	11.6	12.4
0.25	3.29	3.60	3.98	4.39	4.84	5.29	5.77	6.27	6.77	7.27	7.76	8.72	9.65	10.5	11.4	12.2
0.30	3.01	3.31	3.67	4.07	4.51	4.95	5.42	5.91	6.40	6.90	7.40	8.39	9.34	10.3	11.2	12.0
0.40	2.55	2.82	3.16	3.53	3.94	4.37	4.81	5.26	5.74	6.22	6.72	7.71	8.70	9.67	10.6	11.5
0.50	2.20	2.45	2.75	3.10	3.47	3.87	4.30	4.73	5.17	5.63	6.10	7.08	8.06	9.05	10.0	11.0
0.60	1.92	2.15	2.43	2.75	3.09	3.46	3.86	4.27	4.69	5.12	5.57	6.50	7.46	8.44	9.41	10.4
0.70	1.71	1.91	2.17	2.46	2.78	3.12	3.49	3.88	4.28	4.69	5.11	5.99	6.92	7.87	8.83	9.79
0.80	1.53	1.72	1.95	2.22	2.52	2.84	3.18	3.54	3.92	4.31	4.71	5.54	6.42	7.34	8.28	9.23
0.90	1.38	1.56	1.78	2.03	2.30	2.60	2.92	3.25	3.61	3.98	4.35	5.15	5.99	6.86	7.77	8.70
1.0	1.26	1.42	1.63	1.86	2.12	2.39	2.69	3.01	3.34	3.69	4.05	4.80	5.59	6.44	7.31	8.20
1.2	1.07	1.21	1.39	1.59	1.82	2.06	2.32	2.60	2.90	3.21	3.53	4.21	4.93	5.70	6.48	7.30
1.4	0.931	1.05	1.21	1.39	1.59	1.81	2.04	2.29	2.55	2.83	3.12	3.74	4.40	5.09	5.80	6.56
1.6	0.822	0.932	1.07	1.23	1.41	1.61	1.82	2.04	2.28	2.53	2.79	3.36	3.96	4.60	5.25	5.93
1.8	0.735	0.834	0.961	1.11	1.27	1.45	1.64	1.84	2.06	2.29	2.53	3.04	3.59	4.18	4.78	5.42
2.0	0.665	0.755	0.870	1.00	1.15	1.31	1.49	1.68	1.87	2.08	2.30	2.78	3.29	3.83	4.39	4.98
2.2	0.607	0.690	0.795	0.918	1.05	1.20	1.37	1.54	1.72	1.91	2.12	2.55	3.03	3.53	4.05	4.60
2.4	0.558	0.634	0.732	0.845	0.972	1.11	1.26	1.42	1.59	1.77	1.96	2.36	2.80	3.27	3.76	4.27
2.6	0.516	0.587	0.678	0.783	0.901	1.03	1.17	1.32	1.47	1.64	1.82	2.20	2.61	3.05	3.50	3.99
2.8	0.480	0.546	0.631	0.730	0.840	0.960	1.09	1.23	1.38	1.53	1.70	2.05	2.44	2.85	3.28	3.73
3.0	0.449	0.511	0.591	0.683	0.786	0.899	1.02	1.15	1.29	1.44	1.59	1.93	2.29	2.67	3.08	3.51

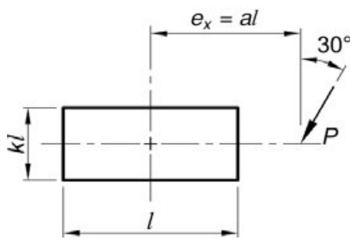
Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

Table 8-7 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 30°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD			ASD		
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$	$D_{min} = \dfrac{P_u}{\phi CC_1 l}$	$l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$	$D_{min} = \dfrac{\Omega P_a}{CC_1 l}$	$l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



a	*k* = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	5.21	5.58	6.01	6.45	6.89	7.33	7.76	8.20	8.64	9.07	9.51	10.4	11.3	12.1	13.0	13.9
0.10	4.49	4.93	5.36	5.81	6.28	6.77	7.26	7.75	8.24	8.72	9.20	10.1	11.1	12.0	12.9	13.8
0.15	4.09	4.51	4.94	5.38	5.84	6.32	6.82	7.33	7.84	8.35	8.85	9.83	10.8	11.7	12.7	13.6
0.20	3.76	4.15	4.56	4.99	5.43	5.90	6.40	6.91	7.42	7.94	8.46	9.47	10.5	11.4	12.4	13.3
0.25	3.47	3.83	4.22	4.64	5.07	5.52	6.01	6.51	7.03	7.55	8.06	9.09	10.1	11.1	12.1	13.0
0.30	3.21	3.54	3.92	4.32	4.75	5.20	5.67	6.16	6.67	7.19	7.70	8.73	9.75	10.8	11.7	12.7
0.40	2.76	3.06	3.40	3.77	4.19	4.62	5.08	5.55	6.03	6.53	7.03	8.06	9.08	10.1	11.1	12.1
0.50	2.40	2.67	2.98	3.33	3.72	4.14	4.57	5.02	5.48	5.95	6.44	7.43	8.44	9.45	10.4	11.4
0.60	2.11	2.35	2.64	2.98	3.34	3.73	4.14	4.56	5.00	5.45	5.91	6.87	7.85	8.82	9.81	10.8
0.70	1.88	2.10	2.37	2.68	3.02	3.38	3.77	4.17	4.59	5.02	5.46	6.37	7.29	8.24	9.20	10.2
0.80	1.69	1.89	2.14	2.43	2.75	3.09	3.45	3.83	4.22	4.63	5.05	5.92	6.80	7.71	8.64	9.59
0.90	1.53	1.72	1.95	2.22	2.52	2.84	3.18	3.53	3.91	4.30	4.70	5.52	6.36	7.23	8.13	9.05
1.0	1.40	1.57	1.79	2.04	2.32	2.62	2.94	3.28	3.63	4.00	4.38	5.17	5.96	6.79	7.66	8.56
1.2	1.19	1.34	1.53	1.76	2.00	2.27	2.55	2.85	3.17	3.50	3.85	4.56	5.30	6.05	6.84	7.68
1.4	1.03	1.17	1.34	1.54	1.76	2.00	2.25	2.52	2.81	3.11	3.42	4.07	4.75	5.45	6.17	6.94
1.6	0.914	1.03	1.19	1.37	1.56	1.78	2.01	2.25	2.51	2.79	3.07	3.67	4.30	4.94	5.61	6.32
1.8	0.818	0.927	1.07	1.23	1.41	1.60	1.81	2.04	2.27	2.52	2.78	3.33	3.92	4.51	5.14	5.80
2.0	0.740	0.840	0.966	1.11	1.28	1.46	1.65	1.86	2.07	2.30	2.54	3.05	3.59	4.15	4.74	5.35
2.2	0.675	0.767	0.884	1.02	1.17	1.34	1.51	1.70	1.90	2.12	2.34	2.81	3.31	3.83	4.39	4.97
2.4	0.621	0.706	0.814	0.939	1.08	1.23	1.40	1.57	1.76	1.96	2.16	2.61	3.07	3.56	4.08	4.63
2.6	0.575	0.653	0.754	0.871	1.00	1.14	1.30	1.46	1.64	1.82	2.01	2.43	2.87	3.32	3.81	4.33
2.8	0.535	0.608	0.702	0.812	0.934	1.07	1.21	1.37	1.53	1.70	1.88	2.27	2.68	3.11	3.57	4.06
3.0	0.500	0.569	0.657	0.760	0.874	1.00	1.14	1.28	1.43	1.59	1.77	2.13	2.52	2.93	3.36	3.83

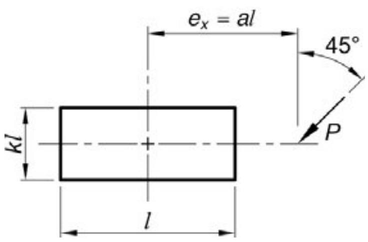
Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

Table 8-7 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 45°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD			ASD		
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$	$D_{min} = \dfrac{P_u}{\phi C C_1 l}$	$l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$	$D_{min} = \dfrac{\Omega P_a}{C C_1 l}$	$l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	4.82	5.14	5.61	6.08	6.54	7.01	7.48	7.95	8.41	8.88	9.35	10.3	11.2	12.2	13.1	14.0
0.10	4.49	4.99	5.48	5.96	6.45	6.94	7.43	7.92	8.41	8.90	9.39	10.4	11.4	12.3	13.3	14.3
0.15	4.18	4.69	5.19	5.67	6.16	6.65	7.15	7.65	8.15	8.65	9.14	10.1	11.1	12.1	13.1	14.1
0.20	3.92	4.39	4.87	5.36	5.84	6.33	6.83	7.33	7.84	8.34	8.85	9.86	10.9	11.9	12.9	13.9
0.25	3.70	4.13	4.58	5.05	5.52	6.01	6.50	7.00	7.50	8.02	8.53	9.54	10.6	11.6	12.6	13.6
0.30	3.49	3.89	4.32	4.76	5.22	5.70	6.18	6.67	7.18	7.69	8.20	9.21	10.2	11.3	12.3	13.3
0.40	3.10	3.45	3.84	4.25	4.68	5.13	5.60	6.07	6.56	7.06	7.57	8.56	9.57	10.6	11.6	12.7
0.50	2.75	3.07	3.42	3.81	4.22	4.65	5.10	5.56	6.03	6.52	7.01	7.96	8.94	9.96	11.0	12.0
0.60	2.46	2.75	3.08	3.44	3.83	4.24	4.67	5.11	5.58	6.05	6.52	7.43	8.38	9.37	10.4	11.4
0.70	2.21	2.48	2.78	3.12	3.49	3.88	4.30	4.73	5.17	5.62	6.08	6.96	7.87	8.83	9.81	10.8
0.80	2.01	2.25	2.53	2.85	3.20	3.57	3.97	4.39	4.81	5.25	5.69	6.54	7.42	8.34	9.29	10.3
0.90	1.83	2.06	2.32	2.62	2.95	3.31	3.69	4.08	4.49	4.91	5.33	6.16	7.01	7.89	8.81	9.76
1.0	1.68	1.89	2.13	2.42	2.73	3.08	3.44	3.81	4.20	4.60	5.01	5.81	6.63	7.48	8.38	9.30
1.2	1.44	1.62	1.84	2.10	2.38	2.69	3.02	3.36	3.71	4.08	4.46	5.20	5.97	6.77	7.60	8.47
1.4	1.25	1.41	1.61	1.84	2.10	2.38	2.68	2.99	3.32	3.65	4.00	4.69	5.41	6.17	6.95	7.76
1.6	1.11	1.25	1.43	1.64	1.88	2.13	2.40	2.69	2.99	3.30	3.62	4.27	4.94	5.65	6.38	7.15
1.8	0.996	1.13	1.29	1.48	1.70	1.93	2.18	2.44	2.72	3.00	3.30	3.90	4.53	5.20	5.89	6.62
2.0	0.902	1.02	1.17	1.35	1.55	1.76	1.99	2.23	2.49	2.75	3.03	3.59	4.18	4.81	5.46	6.15
2.2	0.824	0.934	1.07	1.24	1.42	1.62	1.83	2.06	2.29	2.54	2.80	3.32	3.88	4.47	5.09	5.74
2.4	0.758	0.860	0.990	1.14	1.31	1.49	1.69	1.90	2.12	2.36	2.60	3.09	3.62	4.17	4.76	5.37
2.6	0.702	0.797	0.918	1.06	1.22	1.39	1.57	1.77	1.98	2.19	2.42	2.89	3.38	3.91	4.46	5.05
2.8	0.653	0.742	0.855	0.987	1.14	1.30	1.47	1.66	1.85	2.05	2.27	2.71	3.18	3.67	4.20	4.76
3.0	0.611	0.694	0.801	0.925	1.06	1.22	1.38	1.55	1.74	1.93	2.13	2.55	2.99	3.47	3.97	4.50

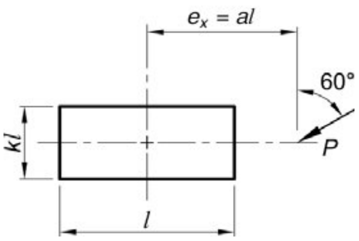
Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

Table 8-7 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 60°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



a	*k*															
	0	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	4.37	4.89	5.40	5.91	6.43	6.94	7.46	7.97	8.48	9.00	9.51	10.5	11.6	12.6	13.6	14.7
0.10	4.26	4.79	5.31	5.82	6.34	6.85	7.37	7.88	8.40	8.91	9.43	10.5	11.5	12.5	13.6	14.6
0.15	4.12	4.67	5.19	5.71	6.22	6.73	7.24	7.75	8.26	8.77	9.28	10.3	11.3	12.4	13.4	14.5
0.20	3.97	4.51	5.05	5.57	6.07	6.58	7.08	7.58	8.09	8.59	9.10	10.1	11.1	12.2	13.2	14.2
0.25	3.86	4.36	4.88	5.39	5.90	6.40	6.90	7.39	7.89	8.39	8.89	9.90	10.9	11.9	13.0	14.0
0.30	3.74	4.22	4.72	5.22	5.72	6.21	6.70	7.19	7.68	8.17	8.67	9.67	10.7	11.7	12.7	13.8
0.40	3.51	3.94	4.40	4.88	5.36	5.84	6.32	6.79	7.25	7.73	8.21	9.19	10.2	11.2	12.2	13.3
0.50	3.26	3.66	4.09	4.54	5.00	5.47	5.94	6.40	6.86	7.32	7.78	8.73	9.70	10.7	11.7	12.7
0.60	3.02	3.39	3.79	4.21	4.66	5.11	5.57	6.03	6.48	6.93	7.38	8.30	9.25	10.2	11.2	12.2
0.70	2.80	3.14	3.51	3.91	4.33	4.77	5.23	5.68	6.12	6.56	7.01	7.91	8.84	9.78	10.8	11.8
0.80	2.59	2.91	3.26	3.64	4.04	4.47	4.90	5.35	5.79	6.22	6.65	7.54	8.45	9.38	10.3	11.3
0.90	2.40	2.70	3.03	3.39	3.78	4.19	4.61	5.05	5.48	5.90	6.33	7.20	8.09	9.01	9.95	10.9
1.0	2.23	2.51	2.82	3.17	3.54	3.93	4.34	4.77	5.20	5.61	6.03	6.88	7.76	8.67	9.59	10.5
1.2	1.94	2.19	2.47	2.79	3.13	3.50	3.88	4.28	4.69	5.09	5.49	6.31	7.15	8.02	8.92	9.84
1.4	1.72	1.94	2.19	2.48	2.80	3.14	3.50	3.88	4.27	4.64	5.02	5.80	6.61	7.45	8.31	9.20
1.6	1.53	1.73	1.96	2.23	2.52	2.85	3.19	3.54	3.90	4.26	4.62	5.36	6.13	6.94	7.77	8.62
1.8	1.38	1.56	1.77	2.02	2.30	2.60	2.92	3.25	3.59	3.92	4.26	4.97	5.71	6.48	7.28	8.10
2.0	1.25	1.42	1.62	1.85	2.11	2.39	2.69	3.00	3.32	3.63	3.96	4.62	5.33	6.07	6.83	7.63
2.2	1.15	1.30	1.49	1.70	1.94	2.21	2.49	2.78	3.08	3.38	3.68	4.32	4.99	5.70	6.43	7.20
2.4	1.06	1.20	1.37	1.58	1.80	2.05	2.31	2.59	2.87	3.15	3.44	4.05	4.69	5.37	6.07	6.81
2.6	0.983	1.11	1.28	1.47	1.68	1.91	2.16	2.42	2.69	2.96	3.23	3.81	4.42	5.07	5.75	6.45
2.8	0.917	1.04	1.19	1.37	1.57	1.79	2.03	2.27	2.53	2.78	3.04	3.59	4.18	4.80	5.45	6.13
3.0	0.858	0.973	1.12	1.29	1.48	1.69	1.91	2.14	2.38	2.62	2.87	3.40	3.96	4.55	5.18	5.84

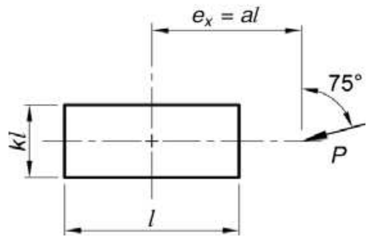
Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

Table 8-7 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 75°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



a	k: 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	3.96	4.39	4.94	5.48	6.03	6.57	7.12	7.66	8.21	8.75	9.30	10.4	11.5	12.6	13.7	14.7
0.10	3.82	4.36	4.90	5.44	5.99	6.53	7.07	7.62	8.16	8.70	9.25	10.3	11.4	12.5	13.6	14.7
0.15	3.85	4.32	4.86	5.41	5.95	6.49	7.03	7.57	8.11	8.65	9.20	10.3	11.4	12.4	13.5	14.6
0.20	3.84	4.26	4.81	5.36	5.90	6.44	6.98	7.52	8.05	8.59	9.13	10.2	11.3	12.4	13.4	14.5
0.25	3.83	4.23	4.75	5.30	5.84	6.38	6.91	7.45	7.98	8.52	9.05	10.1	11.2	12.3	13.3	14.4
0.30	3.82	4.22	4.72	5.24	5.77	6.30	6.84	7.37	7.90	8.43	8.96	10.0	11.1	12.1	13.2	14.3
0.40	3.78	4.21	4.68	5.18	5.68	6.18	6.69	7.21	7.72	8.24	8.76	9.81	10.9	11.9	13.0	14.0
0.50	3.72	4.17	4.63	5.11	5.59	6.08	6.57	7.07	7.57	8.07	8.58	9.59	10.6	11.7	12.7	13.7
0.60	3.65	4.10	4.56	5.02	5.49	5.96	6.44	6.92	7.41	7.90	8.40	9.39	10.4	11.4	12.4	13.5
0.70	3.56	4.00	4.46	4.91	5.37	5.83	6.30	6.77	7.25	7.73	8.21	9.19	10.2	11.2	12.2	13.2
0.80	3.46	3.89	4.34	4.78	5.23	5.69	6.14	6.61	7.07	7.54	8.02	8.98	9.96	10.9	11.9	12.9
0.90	3.35	3.76	4.20	4.65	5.09	5.54	5.98	6.44	6.90	7.36	7.83	8.77	9.74	10.7	11.7	12.7
1.0	3.23	3.64	4.06	4.51	4.94	5.38	5.82	6.27	6.72	7.17	7.63	8.57	9.52	10.5	11.5	12.5
1.2	3.00	3.38	3.79	4.21	4.64	5.06	5.49	5.92	6.36	6.80	7.25	8.16	9.10	10.0	11.0	12.0
1.4	2.78	3.13	3.51	3.92	4.34	4.75	5.17	5.59	6.01	6.44	6.88	7.77	8.69	9.62	10.6	11.5
1.6	2.57	2.90	3.26	3.64	4.05	4.46	4.86	5.27	5.69	6.11	6.53	7.41	8.30	9.22	10.2	11.1
1.8	2.38	2.69	3.02	3.39	3.78	4.19	4.58	4.98	5.38	5.79	6.21	7.06	7.94	8.85	9.77	10.7
2.0	2.21	2.50	2.81	3.16	3.54	3.93	4.32	4.70	5.10	5.50	5.90	6.74	7.61	8.49	9.40	10.3
2.2	2.05	2.32	2.63	2.96	3.32	3.70	4.08	4.45	4.84	5.23	5.62	6.44	7.29	8.16	9.06	9.97
2.4	1.92	2.17	2.46	2.77	3.12	3.48	3.86	4.22	4.59	4.97	5.36	6.16	7.00	7.85	8.74	9.64
2.6	1.80	2.03	2.30	2.61	2.94	3.29	3.66	4.01	4.37	4.74	5.12	5.91	6.72	7.56	8.43	9.32
2.8	1.69	1.91	2.17	2.46	2.78	3.12	3.47	3.82	4.17	4.53	4.90	5.66	6.46	7.29	8.14	9.01
3.0	1.59	1.80	2.05	2.32	2.63	2.96	3.30	3.64	3.98	4.33	4.69	5.44	6.22	7.02	7.86	8.71

Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

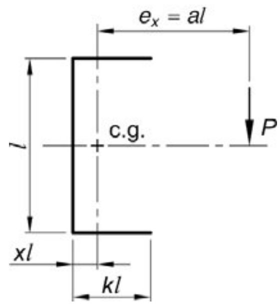
Table 8-8
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 0°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.



a	k: 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	1.86	2.23	2.69	3.25	3.80	4.36	4.92	5.47	6.03	6.59	7.15	8.26	9.37	10.5	11.6	12.7
0.10	1.86	2.28	2.78	3.30	3.83	4.37	4.92	5.46	6.01	6.56	7.11	8.22	9.32	10.4	11.5	12.7
0.15	1.83	2.25	2.73	3.23	3.75	4.27	4.80	5.33	5.87	6.41	6.94	8.02	9.11	10.2	11.3	12.4
0.20	1.76	2.18	2.63	3.11	3.60	4.11	4.61	5.13	5.64	6.16	6.68	7.72	8.77	9.83	10.9	12.0
0.25	1.66	2.07	2.51	2.96	3.42	3.90	4.38	4.87	5.37	5.86	6.36	7.37	8.39	9.42	10.5	11.5
0.30	1.55	1.95	2.36	2.79	3.23	3.68	4.14	4.60	5.08	5.55	6.03	7.01	8.00	9.00	10.0	11.0
0.40	1.33	1.69	2.07	2.45	2.84	3.24	3.65	4.07	4.50	4.94	5.39	6.30	7.24	8.19	9.16	10.1
0.50	1.15	1.46	1.79	2.14	2.49	2.85	3.22	3.60	4.00	4.40	4.82	5.67	6.56	7.47	8.40	9.35
0.60	0.999	1.27	1.57	1.88	2.19	2.52	2.85	3.20	3.57	3.94	4.33	5.13	5.97	6.84	7.74	8.65
0.70	0.879	1.12	1.38	1.66	1.95	2.24	2.55	2.87	3.20	3.55	3.91	4.66	5.46	6.29	7.15	8.04
0.80	0.783	0.996	1.23	1.48	1.75	2.02	2.30	2.59	2.90	3.22	3.56	4.27	5.02	5.82	6.64	7.50
0.90	0.704	0.896	1.11	1.34	1.58	1.83	2.09	2.36	2.65	2.95	3.26	3.93	4.65	5.40	6.19	7.01
1.0	0.639	0.813	1.00	1.21	1.44	1.67	1.91	2.16	2.43	2.71	3.01	3.64	4.31	5.03	5.78	6.56
1.2	0.538	0.684	0.845	1.02	1.21	1.42	1.63	1.85	2.08	2.33	2.59	3.15	3.75	4.39	5.07	5.79
1.4	0.464	0.589	0.729	0.883	1.05	1.23	1.42	1.61	1.82	2.04	2.27	2.77	3.31	3.89	4.50	5.15
1.6	0.408	0.517	0.640	0.775	0.924	1.09	1.25	1.43	1.61	1.81	2.02	2.46	2.95	3.48	4.04	4.64
1.8	0.363	0.461	0.570	0.691	0.825	0.970	1.12	1.28	1.45	1.62	1.81	2.22	2.66	3.14	3.66	4.21
2.0	0.328	0.415	0.514	0.623	0.744	0.877	1.01	1.16	1.31	1.47	1.64	2.01	2.42	2.86	3.34	3.85
2.2	0.298	0.378	0.468	0.567	0.678	0.800	0.926	1.06	1.20	1.35	1.50	1.84	2.22	2.62	3.07	3.54
2.4	0.274	0.347	0.429	0.521	0.623	0.735	0.852	0.973	1.10	1.24	1.38	1.70	2.04	2.42	2.84	3.28
2.6	0.253	0.320	0.396	0.481	0.576	0.680	0.788	0.901	1.02	1.15	1.28	1.57	1.90	2.25	2.64	3.05
2.8	0.235	0.297	0.368	0.447	0.535	0.632	0.734	0.839	0.950	1.07	1.19	1.47	1.77	2.10	2.46	2.85
3.0	0.219	0.278	0.343	0.417	0.500	0.591	0.686	0.784	0.889	1.00	1.12	1.37	1.66	1.97	2.31	2.68
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

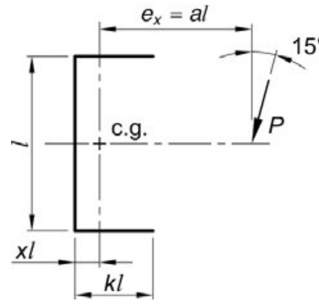
Table 8-8 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 15°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.



a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	1.98	2.47	3.01	3.56	4.10	4.65	5.19	5.74	6.28	6.83	7.37	8.46	9.55	10.6	11.7	12.8
0.10	1.90	2.35	2.87	3.41	3.95	4.50	5.05	5.60	6.15	6.70	7.24	8.34	9.43	10.5	11.6	12.7
0.15	1.84	2.30	2.79	3.30	3.81	4.33	4.86	5.39	5.92	6.45	6.98	8.06	9.13	10.2	11.3	12.4
0.20	1.76	2.21	2.68	3.16	3.65	4.15	4.65	5.16	5.67	6.18	6.69	7.72	8.76	9.80	10.9	11.9
0.25	1.65	2.08	2.54	3.00	3.47	3.94	4.42	4.91	5.39	5.89	6.38	7.38	8.39	9.40	10.4	11.5
0.30	1.55	1.95	2.39	2.82	3.27	3.72	4.18	4.64	5.11	5.58	6.06	7.03	8.01	9.00	10.0	11.0
0.40	1.34	1.69	2.07	2.47	2.88	3.28	3.70	4.12	4.55	4.99	5.43	6.34	7.27	8.23	9.19	10.2
0.50	1.16	1.47	1.80	2.16	2.53	2.89	3.27	3.66	4.05	4.46	4.87	5.73	6.62	7.53	8.46	9.41
0.60	1.01	1.28	1.58	1.89	2.23	2.56	2.91	3.26	3.63	4.00	4.39	5.20	6.04	6.91	7.81	8.73
0.70	0.895	1.13	1.40	1.68	1.98	2.29	2.60	2.93	3.27	3.62	3.98	4.74	5.54	6.38	7.24	8.13
0.80	0.799	1.01	1.25	1.50	1.77	2.06	2.35	2.65	2.96	3.29	3.63	4.35	5.11	5.91	6.74	7.60
0.90	0.720	0.912	1.12	1.35	1.60	1.87	2.14	2.42	2.71	3.01	3.33	4.01	4.74	5.50	6.29	7.11
1.0	0.654	0.829	1.02	1.23	1.46	1.70	1.96	2.22	2.49	2.78	3.08	3.72	4.40	5.12	5.88	6.67
1.2	0.552	0.700	0.863	1.04	1.24	1.45	1.67	1.90	2.14	2.40	2.66	3.23	3.84	4.49	5.18	5.90
1.4	0.477	0.604	0.746	0.902	1.07	1.26	1.46	1.66	1.87	2.10	2.34	2.84	3.39	3.98	4.61	5.27
1.6	0.420	0.531	0.656	0.794	0.946	1.11	1.29	1.47	1.66	1.86	2.08	2.53	3.03	3.57	4.14	4.75
1.8	0.374	0.474	0.585	0.709	0.845	0.995	1.16	1.32	1.49	1.68	1.87	2.28	2.74	3.23	3.75	4.32
2.0	0.338	0.427	0.528	0.640	0.764	0.900	1.05	1.19	1.35	1.52	1.70	2.08	2.49	2.94	3.43	3.95
2.2	0.308	0.389	0.481	0.583	0.696	0.822	0.956	1.09	1.24	1.39	1.55	1.90	2.28	2.70	3.16	3.64
2.4	0.282	0.357	0.441	0.535	0.640	0.756	0.880	1.00	1.14	1.28	1.43	1.75	2.11	2.50	2.92	3.37
2.6	0.261	0.330	0.408	0.495	0.592	0.699	0.814	0.931	1.05	1.19	1.32	1.63	1.96	2.32	2.72	3.14
2.8	0.242	0.307	0.379	0.460	0.551	0.651	0.758	0.866	0.982	1.10	1.23	1.51	1.83	2.17	2.54	2.94
3.0	0.226	0.286	0.354	0.430	0.515	0.609	0.709	0.810	0.918	1.03	1.15	1.42	1.71	2.03	2.38	2.76
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

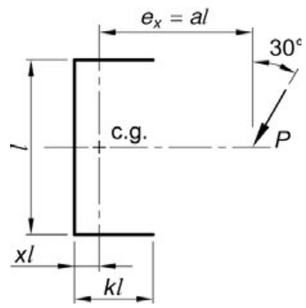
Table 8-8 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 30°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD			ASD		
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$	$D_{min} = \dfrac{P_u}{\phi C C_1 l}$	$l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$	$D_{min} = \dfrac{\Omega P_a}{C C_1 l}$	$l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.



a	*k*															
	0	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	2.18	2.70	3.21	3.73	4.24	4.76	5.27	5.78	6.30	6.81	7.33	8.35	9.38	10.4	11.4	12.5
0.10	2.02	2.57	3.10	3.62	4.14	4.67	5.19	5.71	6.23	6.75	7.28	8.32	9.37	10.4	11.5	12.5
0.15	1.92	2.43	2.95	3.47	3.98	4.49	5.00	5.52	6.03	6.54	7.05	8.09	9.12	10.2	11.2	12.2
0.20	1.82	2.29	2.79	3.29	3.78	4.28	4.77	5.27	5.77	6.27	6.77	7.78	8.80	9.83	10.9	11.9
0.25	1.71	2.15	2.62	3.10	3.58	4.06	4.53	5.01	5.49	5.97	6.46	7.45	8.45	9.47	10.5	11.5
0.30	1.61	2.01	2.45	2.91	3.37	3.83	4.29	4.75	5.21	5.68	6.15	7.11	8.09	9.10	10.1	11.1
0.40	1.41	1.76	2.15	2.55	2.97	3.40	3.83	4.26	4.69	5.13	5.57	6.49	7.42	8.38	9.36	10.4
0.50	1.23	1.54	1.88	2.24	2.62	3.01	3.41	3.81	4.22	4.63	5.05	5.92	6.82	7.74	8.68	9.65
0.60	1.08	1.36	1.66	1.99	2.33	2.68	3.06	3.43	3.81	4.20	4.60	5.42	6.28	7.17	8.09	9.03
0.70	0.964	1.21	1.48	1.77	2.08	2.41	2.75	3.11	3.46	3.83	4.20	4.99	5.81	6.67	7.56	8.47
0.80	0.865	1.09	1.33	1.60	1.88	2.18	2.50	2.83	3.16	3.51	3.86	4.61	5.40	6.22	7.07	7.95
0.90	0.783	0.986	1.21	1.45	1.71	1.99	2.29	2.60	2.91	3.23	3.57	4.28	5.03	5.81	6.63	7.47
1.0	0.714	0.900	1.10	1.33	1.57	1.83	2.10	2.39	2.69	3.00	3.31	3.98	4.70	5.45	6.23	7.04
1.2	0.606	0.764	0.939	1.13	1.34	1.57	1.81	2.07	2.33	2.60	2.89	3.49	4.13	4.81	5.53	6.29
1.4	0.525	0.663	0.815	0.983	1.17	1.37	1.58	1.81	2.05	2.29	2.55	3.09	3.67	4.30	4.96	5.66
1.6	0.463	0.584	0.719	0.868	1.03	1.21	1.41	1.61	1.82	2.04	2.27	2.77	3.30	3.87	4.49	5.13
1.8	0.414	0.522	0.643	0.777	0.925	1.09	1.27	1.45	1.64	1.84	2.05	2.50	2.99	3.52	4.09	4.69
2.0	0.374	0.472	0.581	0.703	0.838	0.988	1.15	1.32	1.49	1.67	1.87	2.28	2.73	3.22	3.75	4.31
2.2	0.341	0.430	0.530	0.642	0.766	0.903	1.05	1.21	1.37	1.53	1.71	2.09	2.51	2.97	3.46	3.98
2.4	0.313	0.395	0.487	0.590	0.705	0.832	0.970	1.11	1.26	1.41	1.58	1.93	2.32	2.75	3.21	3.70
2.6	0.289	0.365	0.451	0.546	0.653	0.771	0.899	1.03	1.17	1.31	1.47	1.80	2.16	2.56	2.99	3.45
2.8	0.269	0.340	0.419	0.508	0.608	0.718	0.838	0.960	1.09	1.22	1.37	1.68	2.02	2.39	2.80	3.24
3.0	0.251	0.317	0.392	0.475	0.569	0.672	0.784	0.899	1.02	1.15	1.28	1.57	1.89	2.25	2.63	3.04
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

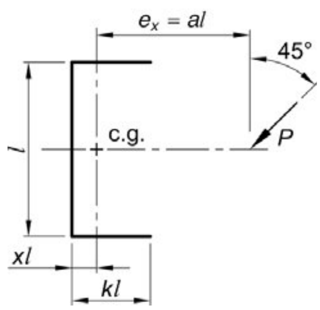
Table 8-8 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 45°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.



a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.41	2.80	3.27	3.74	4.21	4.67	5.14	5.61	6.08	6.54	7.01	7.95	8.88	9.82	10.8	11.7
0.10	2.24	2.74	3.24	3.73	4.23	4.73	5.22	5.72	6.21	6.71	7.20	8.19	9.17	10.1	11.1	12.1
0.15	2.09	2.60	3.09	3.58	4.07	4.57	5.06	5.56	6.06	6.55	7.05	8.04	9.03	10.0	11.0	12.0
0.20	1.96	2.44	2.92	3.40	3.88	4.37	4.86	5.36	5.85	6.35	6.84	7.83	8.83	9.82	10.8	11.8
0.25	1.85	2.29	2.75	3.21	3.68	4.16	4.64	5.13	5.62	6.11	6.60	7.58	8.58	9.58	10.6	11.6
0.30	1.74	2.16	2.59	3.03	3.48	3.94	4.42	4.89	5.38	5.86	6.34	7.32	8.31	9.32	10.3	11.3
0.40	1.55	1.91	2.30	2.70	3.12	3.55	3.99	4.44	4.91	5.37	5.83	6.77	7.75	8.76	9.77	10.8
0.50	1.38	1.70	2.05	2.42	2.80	3.20	3.62	4.04	4.48	4.93	5.37	6.27	7.22	8.20	9.20	10.2
0.60	1.23	1.52	1.84	2.18	2.53	2.90	3.29	3.70	4.11	4.54	4.96	5.83	6.73	7.68	8.65	9.65
0.70	1.11	1.38	1.66	1.97	2.30	2.65	3.01	3.40	3.79	4.20	4.61	5.44	6.30	7.21	8.15	9.12
0.80	1.00	1.25	1.51	1.80	2.11	2.43	2.77	3.13	3.51	3.91	4.29	5.08	5.91	6.78	7.69	8.64
0.90	0.915	1.14	1.39	1.65	1.94	2.24	2.56	2.91	3.27	3.64	4.01	4.76	5.56	6.39	7.27	8.19
1.0	0.839	1.05	1.28	1.52	1.79	2.08	2.38	2.71	3.05	3.40	3.75	4.47	5.24	6.04	6.89	7.77
1.2	0.719	0.900	1.10	1.31	1.55	1.80	2.08	2.37	2.68	3.00	3.31	3.98	4.68	5.43	6.22	7.04
1.4	0.627	0.786	0.961	1.15	1.36	1.59	1.84	2.11	2.39	2.67	2.96	3.57	4.22	4.91	5.65	6.42
1.6	0.555	0.697	0.854	1.03	1.22	1.42	1.65	1.89	2.15	2.40	2.67	3.23	3.83	4.48	5.16	5.88
1.8	0.498	0.625	0.767	0.923	1.10	1.29	1.49	1.72	1.95	2.18	2.42	2.94	3.50	4.10	4.74	5.42
2.0	0.451	0.567	0.696	0.839	0.997	1.17	1.36	1.57	1.78	1.99	2.22	2.70	3.22	3.78	4.38	5.02
2.2	0.412	0.518	0.636	0.768	0.914	1.08	1.25	1.44	1.63	1.83	2.04	2.49	2.97	3.50	4.07	4.67
2.4	0.379	0.477	0.586	0.708	0.844	0.995	1.16	1.33	1.51	1.70	1.89	2.31	2.76	3.26	3.79	4.36
2.6	0.351	0.442	0.543	0.657	0.784	0.924	1.08	1.24	1.40	1.58	1.76	2.15	2.58	3.05	3.55	4.09
2.8	0.327	0.411	0.506	0.612	0.731	0.863	1.01	1.16	1.31	1.47	1.64	2.01	2.42	2.86	3.33	3.84
3.0	0.306	0.385	0.474	0.573	0.685	0.809	0.943	1.09	1.23	1.38	1.54	1.89	2.27	2.69	3.14	3.63
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

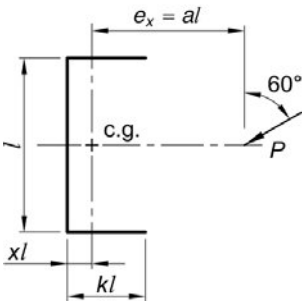
Table 8-8 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 60°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.



a	*k* 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.60	3.01	3.44	3.88	4.32	4.76	5.19	5.63	6.07	6.50	6.94	7.82	8.69	9.56	10.4	11.3
0.10	2.43	2.86	3.30	3.75	4.21	4.68	5.14	5.61	6.07	6.53	6.99	7.89	8.79	9.67	10.5	11.4
0.15	2.31	2.74	3.17	3.62	4.07	4.54	5.01	5.49	5.96	6.44	6.90	7.83	8.74	9.64	10.5	11.4
0.20	2.18	2.61	3.04	3.47	3.93	4.39	4.86	5.34	5.83	6.31	6.79	7.73	8.66	9.57	10.5	11.4
0.25	2.07	2.49	2.91	3.33	3.77	4.23	4.70	5.18	5.67	6.16	6.64	7.61	8.55	9.48	10.4	11.3
0.30	1.97	2.37	2.78	3.20	3.63	4.07	4.54	5.02	5.51	6.00	6.49	7.46	8.42	9.36	10.3	11.2
0.40	1.79	2.16	2.55	2.94	3.35	3.77	4.22	4.69	5.17	5.66	6.15	7.14	8.12	9.09	10.0	11.0
0.50	1.63	1.98	2.34	2.71	3.10	3.50	3.93	4.38	4.85	5.33	5.82	6.80	7.79	8.77	9.73	10.7
0.60	1.49	1.81	2.15	2.50	2.87	3.26	3.67	4.10	4.55	5.02	5.50	6.48	7.46	8.42	9.38	10.3
0.70	1.37	1.67	1.99	2.32	2.67	3.05	3.44	3.86	4.29	4.74	5.21	6.16	7.11	8.07	9.04	10.0
0.80	1.26	1.54	1.84	2.16	2.50	2.85	3.23	3.63	4.05	4.48	4.94	5.85	6.78	7.73	8.69	9.65
0.90	1.17	1.43	1.71	2.02	2.34	2.68	3.04	3.43	3.83	4.25	4.68	5.57	6.47	7.40	8.35	9.31
1.0	1.08	1.33	1.60	1.89	2.19	2.52	2.87	3.24	3.63	4.03	4.45	5.30	6.18	7.09	8.03	8.98
1.2	0.946	1.17	1.41	1.67	1.95	2.25	2.58	2.92	3.28	3.65	4.04	4.82	5.65	6.52	7.42	8.35
1.4	0.837	1.04	1.25	1.49	1.75	2.03	2.33	2.65	2.98	3.33	3.69	4.42	5.19	6.01	6.87	7.77
1.6	0.748	0.930	1.13	1.34	1.58	1.84	2.12	2.42	2.73	3.05	3.38	4.07	4.79	5.56	6.38	7.24
1.8	0.676	0.842	1.02	1.22	1.44	1.68	1.94	2.22	2.51	2.81	3.12	3.76	4.45	5.17	5.95	6.77
2.0	0.616	0.768	0.936	1.12	1.32	1.55	1.79	2.05	2.32	2.60	2.90	3.50	4.14	4.83	5.56	6.34
2.2	0.565	0.706	0.861	1.03	1.22	1.43	1.66	1.90	2.15	2.42	2.69	3.26	3.87	4.53	5.22	5.96
2.4	0.522	0.653	0.797	0.958	1.14	1.33	1.55	1.77	2.01	2.26	2.52	3.05	3.63	4.25	4.91	5.62
2.6	0.485	0.607	0.742	0.893	1.06	1.25	1.44	1.66	1.88	2.12	2.36	2.87	3.42	4.01	4.64	5.31
2.8	0.453	0.567	0.694	0.835	0.994	1.17	1.36	1.56	1.77	1.99	2.22	2.70	3.22	3.79	4.39	5.03
3.0	0.424	0.531	0.651	0.785	0.934	1.10	1.28	1.47	1.67	1.88	2.09	2.55	3.05	3.59	4.17	4.78
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

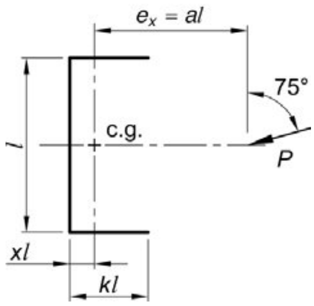
Table 8-8 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 75°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.



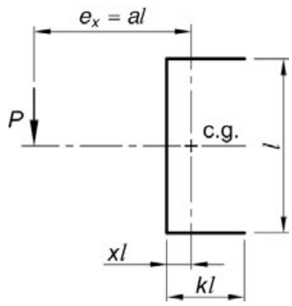
a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.74	3.11	3.49	3.88	4.26	4.65	5.03	5.42	5.80	6.19	6.57	7.34	8.11	8.88	9.65	10.4
0.10	2.59	2.95	3.34	3.75	4.16	4.58	4.99	5.40	5.80	6.20	6.59	7.37	8.15	8.92	9.69	10.5
0.15	2.50	2.87	3.26	3.67	4.09	4.51	4.94	5.35	5.76	6.17	6.57	7.36	8.14	8.91	9.69	10.5
0.20	2.43	2.79	3.18	3.59	4.01	4.44	4.87	5.29	5.71	6.13	6.53	7.33	8.12	8.90	9.67	10.4
0.25	2.35	2.72	3.10	3.51	3.93	4.36	4.80	5.23	5.66	6.08	6.49	7.30	8.09	8.88	9.66	10.4
0.30	2.28	2.65	3.03	3.43	3.85	4.28	4.72	5.16	5.59	6.02	6.44	7.26	8.06	8.85	9.63	10.4
0.40	2.16	2.52	2.88	3.27	3.69	4.12	4.57	5.01	5.45	5.88	6.31	7.15	7.97	8.78	9.57	10.4
0.50	2.05	2.40	2.75	3.13	3.54	3.97	4.41	4.86	5.30	5.75	6.18	7.04	7.86	8.68	9.48	10.3
0.60	1.94	2.28	2.63	3.00	3.40	3.82	4.26	4.71	5.16	5.61	6.06	6.93	7.77	8.59	9.39	10.2
0.70	1.85	2.18	2.52	2.88	3.26	3.68	4.11	4.56	5.02	5.47	5.92	6.81	7.67	8.51	9.32	10.1
0.80	1.75	2.08	2.41	2.76	3.14	3.54	3.97	4.42	4.87	5.33	5.79	6.69	7.57	8.42	9.25	10.1
0.90	1.67	1.98	2.31	2.65	3.02	3.42	3.84	4.28	4.73	5.19	5.64	6.56	7.45	8.32	9.16	9.98
1.0	1.59	1.90	2.21	2.55	2.91	3.30	3.71	4.14	4.59	5.04	5.50	6.42	7.33	8.21	9.07	9.91
1.2	1.45	1.74	2.04	2.36	2.71	3.08	3.47	3.89	4.32	4.77	5.22	6.15	7.07	7.97	8.86	9.72
1.4	1.33	1.60	1.89	2.20	2.53	2.88	3.26	3.66	4.07	4.51	4.95	5.87	6.79	7.71	8.62	9.51
1.6	1.22	1.48	1.75	2.05	2.37	2.71	3.06	3.44	3.85	4.27	4.70	5.60	6.52	7.44	8.36	9.27
1.8	1.13	1.37	1.63	1.91	2.22	2.54	2.89	3.25	3.64	4.04	4.46	5.34	6.25	7.17	8.10	9.01
2.0	1.05	1.28	1.52	1.79	2.09	2.40	2.73	3.08	3.45	3.84	4.24	5.10	5.99	6.90	7.81	8.73
2.2	0.975	1.19	1.43	1.69	1.97	2.27	2.58	2.92	3.27	3.65	4.04	4.87	5.74	6.62	7.53	8.44
2.4	0.912	1.12	1.34	1.59	1.86	2.15	2.45	2.77	3.11	3.47	3.85	4.65	5.50	6.36	7.25	8.15
2.6	0.856	1.05	1.27	1.50	1.76	2.04	2.33	2.64	2.97	3.31	3.68	4.45	5.26	6.10	6.98	7.87
2.8	0.806	0.993	1.20	1.42	1.67	1.94	2.22	2.52	2.83	3.17	3.52	4.27	5.05	5.86	6.72	7.60
3.0	0.762	0.940	1.14	1.35	1.59	1.84	2.12	2.40	2.71	3.03	3.37	4.09	4.84	5.64	6.47	7.34
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Table 8-9
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 0°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

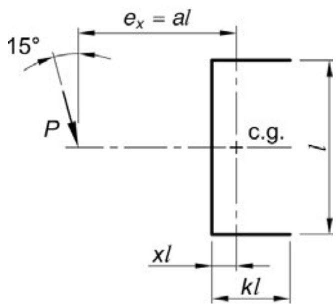
a	*k*															
	0	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	1.86	2.23	2.69	3.25	3.80	4.36	4.92	5.47	6.03	6.59	7.15	8.26	9.37	10.5	11.6	12.7
0.10	1.86	2.30	2.80	3.30	3.82	4.32	4.83	5.34	5.84	6.34	6.84	7.84	8.84	9.83	10.8	11.8
0.15	1.83	2.26	2.73	3.21	3.69	4.18	4.66	5.14	5.62	6.10	6.58	7.54	8.51	9.48	10.4	11.4
0.20	1.76	2.18	2.62	3.07	3.53	3.99	4.45	4.91	5.37	5.83	6.30	7.22	8.16	9.11	10.1	11.0
0.25	1.66	2.06	2.48	2.91	3.35	3.79	4.23	4.67	5.11	5.55	6.00	6.90	7.81	8.73	9.67	10.6
0.30	1.55	1.93	2.33	2.74	3.15	3.57	3.99	4.41	4.84	5.27	5.70	6.57	7.46	8.37	9.29	10.2
0.40	1.33	1.67	2.03	2.39	2.77	3.15	3.53	3.92	4.32	4.72	5.12	5.95	6.79	7.66	8.54	9.44
0.50	1.15	1.45	1.75	2.07	2.41	2.76	3.12	3.47	3.84	4.21	4.59	5.37	6.17	7.00	7.86	8.73
0.60	0.999	1.26	1.52	1.81	2.11	2.43	2.77	3.10	3.44	3.79	4.14	4.88	5.65	6.45	7.27	8.11
0.70	0.879	1.11	1.34	1.60	1.88	2.18	2.48	2.80	3.12	3.44	3.78	4.47	5.20	5.96	6.75	7.56
0.80	0.783	0.982	1.20	1.43	1.69	1.96	2.25	2.55	2.84	3.15	3.47	4.12	4.81	5.54	6.29	7.07
0.90	0.704	0.882	1.08	1.30	1.53	1.78	2.05	2.33	2.61	2.90	3.20	3.82	4.47	5.16	5.89	6.64
1.0	0.639	0.800	0.980	1.18	1.40	1.64	1.88	2.14	2.41	2.69	2.97	3.55	4.17	4.83	5.52	6.24
1.2	0.538	0.674	0.829	1.00	1.19	1.40	1.61	1.84	2.08	2.33	2.58	3.11	3.67	4.27	4.90	5.57
1.4	0.464	0.582	0.717	0.869	1.04	1.22	1.41	1.61	1.83	2.05	2.28	2.76	3.27	3.82	4.40	5.01
1.6	0.408	0.511	0.631	0.766	0.915	1.08	1.25	1.43	1.63	1.83	2.04	2.48	2.95	3.45	3.98	4.55
1.8	0.363	0.456	0.563	0.684	0.818	0.964	1.12	1.29	1.46	1.65	1.84	2.24	2.67	3.14	3.63	4.16
2.0	0.328	0.411	0.508	0.618	0.740	0.872	1.01	1.17	1.33	1.49	1.67	2.05	2.45	2.88	3.34	3.82
2.2	0.298	0.375	0.463	0.563	0.675	0.796	0.926	1.06	1.21	1.37	1.53	1.88	2.25	2.65	3.08	3.54
2.4	0.274	0.344	0.425	0.518	0.620	0.732	0.852	0.980	1.11	1.26	1.41	1.73	2.09	2.46	2.86	3.29
2.6	0.253	0.318	0.393	0.479	0.574	0.678	0.789	0.908	1.03	1.16	1.30	1.61	1.94	2.29	2.67	3.07
2.8	0.235	0.295	0.365	0.445	0.534	0.630	0.735	0.845	0.960	1.08	1.21	1.50	1.81	2.15	2.50	2.88
3.0	0.219	0.276	0.341	0.416	0.499	0.589	0.687	0.791	0.897	1.01	1.13	1.40	1.70	2.02	2.35	2.71
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Table 8-9 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 15°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD			ASD		
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$	$D_{min} = \dfrac{P_u}{\phi C C_1 l}$	$l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$	$D_{min} = \dfrac{\Omega P_a}{C C_1 l}$	$l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	1.98	2.47	3.01	3.56	4.10	4.65	5.19	5.74	6.28	6.83	7.37	8.46	9.55	10.6	11.7	12.8
0.10	1.90	2.36	2.87	3.38	3.88	4.38	4.88	5.38	5.87	6.37	6.86	7.85	8.84	9.84	10.8	11.9
0.15	1.84	2.30	2.78	3.26	3.74	4.21	4.69	5.16	5.63	6.10	6.57	7.52	8.47	9.43	10.4	11.4
0.20	1.76	2.20	2.65	3.11	3.56	4.02	4.47	4.92	5.37	5.82	6.27	7.18	8.10	9.04	9.98	10.9
0.25	1.65	2.07	2.49	2.93	3.37	3.80	4.23	4.66	5.09	5.53	5.96	6.84	7.74	8.65	9.58	10.5
0.30	1.55	1.93	2.33	2.74	3.16	3.58	3.99	4.41	4.82	5.24	5.66	6.52	7.39	8.28	9.19	10.1
0.40	1.34	1.67	2.02	2.38	2.75	3.13	3.52	3.92	4.31	4.70	5.10	5.90	6.74	7.59	8.47	9.37
0.50	1.16	1.45	1.75	2.06	2.39	2.74	3.10	3.47	3.85	4.22	4.60	5.38	6.18	7.00	7.85	8.73
0.60	1.01	1.27	1.53	1.80	2.10	2.42	2.75	3.10	3.46	3.82	4.19	4.92	5.69	6.48	7.30	8.15
0.70	0.895	1.12	1.35	1.60	1.88	2.17	2.48	2.80	3.14	3.48	3.83	4.53	5.26	6.02	6.81	7.62
0.80	0.799	0.997	1.21	1.44	1.69	1.96	2.25	2.55	2.86	3.19	3.52	4.19	4.89	5.61	6.37	7.15
0.90	0.720	0.898	1.09	1.31	1.54	1.79	2.05	2.33	2.63	2.94	3.25	3.89	4.56	5.25	5.97	6.73
1.0	0.654	0.816	0.996	1.20	1.41	1.64	1.89	2.15	2.43	2.72	3.02	3.63	4.26	4.92	5.62	6.34
1.2	0.552	0.689	0.845	1.02	1.21	1.41	1.63	1.86	2.10	2.36	2.63	3.18	3.76	4.37	5.01	5.68
1.4	0.477	0.596	0.733	0.886	1.05	1.23	1.43	1.63	1.85	2.08	2.32	2.83	3.36	3.91	4.50	5.12
1.6	0.420	0.525	0.646	0.782	0.933	1.10	1.27	1.45	1.65	1.86	2.08	2.54	3.03	3.54	4.08	4.66
1.8	0.374	0.468	0.577	0.700	0.836	0.983	1.14	1.31	1.49	1.68	1.88	2.30	2.75	3.23	3.73	4.27
2.0	0.338	0.423	0.522	0.633	0.757	0.891	1.04	1.19	1.35	1.53	1.71	2.10	2.52	2.96	3.43	3.93
2.2	0.308	0.385	0.476	0.578	0.692	0.815	0.948	1.09	1.24	1.40	1.57	1.93	2.32	2.73	3.17	3.64
2.4	0.282	0.354	0.437	0.532	0.636	0.750	0.873	1.00	1.14	1.29	1.45	1.79	2.15	2.54	2.95	3.39
2.6	0.261	0.327	0.404	0.492	0.589	0.695	0.809	0.931	1.06	1.20	1.34	1.66	2.00	2.36	2.75	3.17
2.8	0.242	0.304	0.376	0.458	0.548	0.647	0.754	0.868	0.989	1.12	1.25	1.54	1.87	2.21	2.58	2.97
3.0	0.226	0.284	0.352	0.428	0.513	0.606	0.706	0.812	0.926	1.04	1.17	1.45	1.75	2.08	2.43	2.80
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

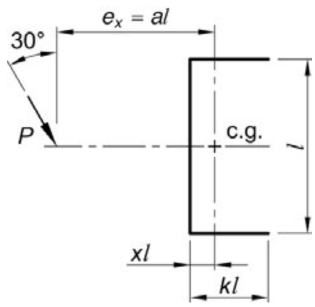
Table 8-9 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 30°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.



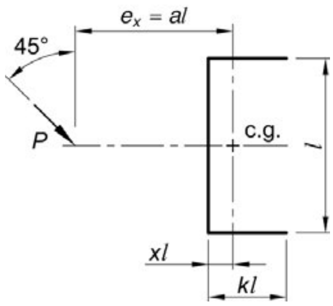
a	*k*															
	0	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	2.18	2.70	3.21	3.73	4.24	4.76	5.27	5.78	6.30	6.81	7.33	8.35	9.38	10.4	11.4	12.5
0.10	2.02	2.56	3.06	3.54	4.02	4.50	4.98	5.46	5.94	6.43	6.92	7.90	8.89	9.89	10.9	11.9
0.15	1.92	2.41	2.90	3.37	3.83	4.28	4.73	5.19	5.65	6.12	6.58	7.54	8.51	9.50	10.5	11.5
0.20	1.82	2.27	2.72	3.16	3.60	4.03	4.46	4.89	5.34	5.78	6.23	7.16	8.11	9.08	10.1	11.1
0.25	1.71	2.13	2.55	2.97	3.37	3.78	4.19	4.60	5.02	5.46	5.90	6.79	7.72	8.68	9.66	10.7
0.30	1.61	1.99	2.38	2.77	3.16	3.55	3.94	4.34	4.75	5.18	5.61	6.48	7.38	8.31	9.27	10.2
0.40	1.41	1.74	2.08	2.43	2.78	3.14	3.50	3.89	4.29	4.70	5.12	5.95	6.81	7.69	8.61	9.54
0.50	1.23	1.52	1.82	2.13	2.45	2.79	3.14	3.51	3.89	4.28	4.69	5.50	6.31	7.16	8.04	8.94
0.60	1.08	1.34	1.60	1.88	2.18	2.50	2.83	3.18	3.54	3.92	4.30	5.09	5.88	6.69	7.53	8.40
0.70	0.964	1.20	1.43	1.69	1.96	2.26	2.57	2.90	3.25	3.60	3.97	4.73	5.48	6.26	7.07	7.91
0.80	0.865	1.07	1.29	1.53	1.79	2.06	2.35	2.66	2.99	3.32	3.67	4.40	5.13	5.88	6.66	7.47
0.90	0.783	0.970	1.17	1.40	1.64	1.89	2.16	2.45	2.76	3.08	3.41	4.11	4.81	5.53	6.29	7.07
1.0	0.714	0.885	1.07	1.28	1.51	1.75	2.00	2.28	2.56	2.87	3.18	3.85	4.53	5.22	5.94	6.70
1.2	0.606	0.753	0.918	1.10	1.30	1.51	1.74	1.98	2.24	2.51	2.80	3.40	4.03	4.67	5.34	6.05
1.4	0.525	0.653	0.800	0.963	1.14	1.33	1.53	1.75	1.98	2.23	2.49	3.04	3.63	4.22	4.84	5.50
1.6	0.463	0.577	0.708	0.854	1.01	1.19	1.37	1.57	1.78	2.00	2.24	2.74	3.29	3.84	4.42	5.03
1.8	0.414	0.516	0.634	0.767	0.913	1.07	1.24	1.42	1.61	1.81	2.03	2.49	3.00	3.51	4.05	4.63
2.0	0.374	0.467	0.574	0.695	0.829	0.974	1.13	1.29	1.47	1.66	1.85	2.28	2.75	3.23	3.74	4.28
2.2	0.341	0.426	0.525	0.636	0.759	0.893	1.04	1.19	1.35	1.52	1.71	2.11	2.54	2.99	3.47	3.97
2.4	0.313	0.392	0.483	0.586	0.699	0.823	0.956	1.10	1.25	1.41	1.58	1.95	2.36	2.78	3.23	3.71
2.6	0.289	0.362	0.447	0.542	0.649	0.764	0.888	1.02	1.16	1.31	1.47	1.82	2.20	2.60	3.02	3.47
2.8	0.269	0.337	0.416	0.505	0.604	0.713	0.829	0.953	1.09	1.23	1.38	1.70	2.06	2.44	2.84	3.27
3.0	0.251	0.315	0.389	0.473	0.566	0.667	0.777	0.894	1.02	1.15	1.29	1.60	1.93	2.29	2.68	3.08
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Table 8-9 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 45°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

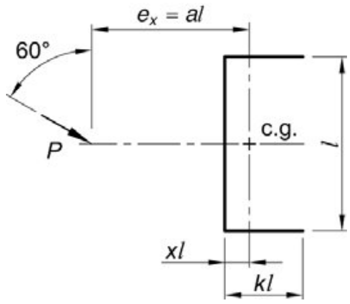
a	*k* 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.41	2.80	3.27	3.74	4.21	4.67	5.14	5.61	6.08	6.54	7.01	7.95	8.88	9.82	10.8	11.7
0.10	2.24	2.72	3.17	3.61	4.05	4.49	4.94	5.41	5.88	6.35	6.82	7.78	8.74	9.71	10.7	11.7
0.15	2.09	2.57	3.00	3.41	3.82	4.24	4.67	5.13	5.59	6.06	6.54	7.51	8.48	9.46	10.4	11.4
0.20	1.96	2.41	2.83	3.21	3.59	3.99	4.41	4.85	5.30	5.77	6.24	7.21	8.19	9.17	10.2	11.2
0.25	1.85	2.27	2.66	3.02	3.38	3.76	4.16	4.59	5.03	5.49	5.95	6.91	7.88	8.87	9.87	10.9
0.30	1.74	2.13	2.50	2.86	3.20	3.57	3.96	4.38	4.81	5.25	5.70	6.64	7.59	8.56	9.55	10.6
0.40	1.55	1.89	2.22	2.55	2.89	3.24	3.62	4.01	4.42	4.84	5.28	6.18	7.11	8.05	8.99	9.95
0.50	1.38	1.68	1.98	2.29	2.61	2.96	3.32	3.69	4.08	4.49	4.91	5.78	6.69	7.60	8.52	9.45
0.60	1.23	1.50	1.77	2.06	2.37	2.71	3.05	3.41	3.78	4.17	4.58	5.42	6.30	7.18	8.08	8.99
0.70	1.11	1.36	1.60	1.88	2.17	2.48	2.81	3.16	3.52	3.89	4.28	5.09	5.94	6.79	7.67	8.57
0.80	1.00	1.23	1.46	1.72	2.00	2.29	2.61	2.93	3.28	3.63	4.01	4.79	5.61	6.43	7.29	8.16
0.90	0.915	1.12	1.34	1.59	1.84	2.12	2.42	2.73	3.06	3.41	3.76	4.51	5.31	6.10	6.93	7.78
1.0	0.839	1.03	1.24	1.47	1.71	1.98	2.26	2.56	2.87	3.20	3.54	4.26	5.03	5.80	6.60	7.43
1.2	0.719	0.886	1.07	1.28	1.50	1.73	1.99	2.26	2.54	2.84	3.16	3.83	4.54	5.26	6.01	6.79
1.4	0.627	0.775	0.943	1.13	1.33	1.54	1.77	2.02	2.28	2.55	2.84	3.46	4.12	4.81	5.50	6.23
1.6	0.555	0.688	0.840	1.01	1.19	1.39	1.60	1.82	2.06	2.31	2.58	3.15	3.77	4.41	5.07	5.75
1.8	0.498	0.618	0.756	0.910	1.08	1.26	1.45	1.66	1.87	2.11	2.36	2.89	3.47	4.07	4.69	5.34
2.0	0.451	0.561	0.687	0.829	0.984	1.15	1.33	1.52	1.72	1.94	2.17	2.66	3.20	3.78	4.36	4.97
2.2	0.412	0.513	0.630	0.760	0.904	1.06	1.22	1.40	1.59	1.79	2.01	2.47	2.97	3.52	4.07	4.65
2.4	0.379	0.473	0.581	0.702	0.836	0.981	1.14	1.30	1.48	1.66	1.86	2.30	2.77	3.29	3.81	4.36
2.6	0.351	0.438	0.539	0.652	0.777	0.913	1.06	1.21	1.38	1.55	1.74	2.15	2.60	3.08	3.58	4.10
2.8	0.327	0.408	0.502	0.608	0.726	0.853	0.990	1.14	1.29	1.46	1.63	2.02	2.44	2.90	3.37	3.87
3.0	0.306	0.382	0.470	0.570	0.680	0.801	0.930	1.07	1.21	1.37	1.54	1.90	2.30	2.74	3.19	3.66
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Table 8-9 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 60°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

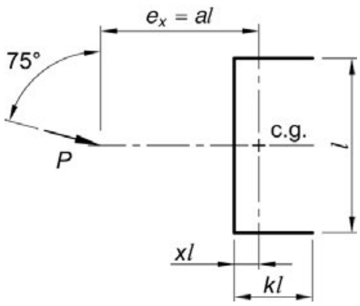
a	k: 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.60	3.01	3.44	3.88	4.32	4.76	5.19	5.63	6.07	6.50	6.94	7.82	8.69	9.56	10.4	11.3
0.10	2.43	2.84	3.23	3.62	4.04	4.47	4.91	5.36	5.81	6.26	6.71	7.61	8.51	9.40	10.3	11.2
0.15	2.31	2.70	3.07	3.44	3.84	4.26	4.69	5.14	5.59	6.05	6.51	7.43	8.34	9.25	10.2	11.1
0.20	2.18	2.58	2.92	3.27	3.65	4.06	4.48	4.92	5.37	5.83	6.30	7.23	8.16	9.08	9.99	10.9
0.25	2.07	2.46	2.79	3.12	3.49	3.89	4.30	4.73	5.17	5.62	6.08	7.01	7.95	8.89	9.81	10.7
0.30	1.97	2.34	2.67	3.00	3.36	3.75	4.15	4.58	5.01	5.45	5.90	6.81	7.73	8.68	9.62	10.6
0.40	1.79	2.13	2.45	2.78	3.12	3.49	3.89	4.30	4.72	5.16	5.60	6.49	7.39	8.30	9.22	10.1
0.50	1.63	1.95	2.25	2.57	2.91	3.27	3.65	4.05	4.46	4.89	5.33	6.21	7.11	8.01	8.92	9.82
0.60	1.49	1.79	2.08	2.39	2.72	3.06	3.43	3.82	4.22	4.64	5.07	5.95	6.85	7.75	8.65	9.56
0.70	1.37	1.64	1.92	2.22	2.54	2.88	3.23	3.60	4.00	4.40	4.83	5.70	6.59	7.49	8.40	9.30
0.80	1.26	1.52	1.78	2.07	2.38	2.71	3.05	3.41	3.79	4.19	4.60	5.45	6.33	7.23	8.14	9.05
0.90	1.17	1.41	1.66	1.94	2.24	2.55	2.88	3.23	3.60	3.98	4.38	5.22	6.09	6.98	7.89	8.80
1.0	1.08	1.31	1.56	1.82	2.11	2.41	2.73	3.07	3.42	3.79	4.18	5.00	5.85	6.74	7.64	8.54
1.2	0.946	1.15	1.38	1.62	1.88	2.16	2.46	2.78	3.11	3.46	3.82	4.59	5.41	6.27	7.15	8.04
1.4	0.837	1.02	1.23	1.46	1.70	1.96	2.23	2.53	2.84	3.17	3.51	4.24	5.02	5.84	6.69	7.56
1.6	0.748	0.919	1.11	1.32	1.54	1.78	2.04	2.32	2.61	2.91	3.24	3.92	4.66	5.45	6.27	7.10
1.8	0.676	0.832	1.01	1.20	1.41	1.64	1.88	2.13	2.41	2.69	3.00	3.65	4.35	5.09	5.88	6.67
2.0	0.616	0.760	0.924	1.11	1.30	1.51	1.73	1.97	2.23	2.50	2.79	3.40	4.07	4.78	5.52	6.28
2.2	0.565	0.699	0.852	1.02	1.21	1.40	1.61	1.84	2.08	2.33	2.60	3.19	3.82	4.49	5.20	5.93
2.4	0.522	0.647	0.790	0.948	1.12	1.31	1.51	1.72	1.94	2.19	2.44	2.99	3.59	4.24	4.91	5.61
2.6	0.485	0.602	0.735	0.885	1.05	1.22	1.41	1.61	1.82	2.05	2.30	2.82	3.39	4.00	4.65	5.32
2.8	0.453	0.562	0.688	0.829	0.983	1.15	1.33	1.52	1.72	1.94	2.17	2.66	3.21	3.79	4.42	5.05
3.0	0.424	0.528	0.646	0.779	0.926	1.08	1.25	1.43	1.62	1.83	2.05	2.52	3.04	3.60	4.20	4.81
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Table 8-9 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 75°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD			ASD		
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$	$D_{min} = \dfrac{P_u}{\phi C C_1 l}$	$l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$	$D_{min} = \dfrac{\Omega P_a}{C C_1 l}$	$l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

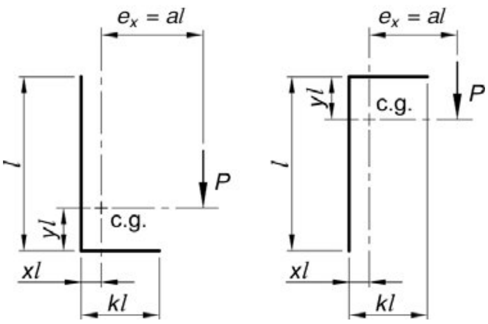
a	*k* = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.74	3.11	3.49	3.88	4.26	4.65	5.03	5.42	5.80	6.19	6.57	7.34	8.11	8.88	9.65	10.4
0.10	2.59	2.94	3.30	3.68	4.07	4.47	4.88	5.28	5.69	6.08	6.48	7.27	8.05	8.83	9.61	10.4
0.15	2.50	2.84	3.19	3.56	3.94	4.34	4.75	5.16	5.57	5.98	6.39	7.19	7.98	8.77	9.55	10.3
0.20	2.43	2.76	3.09	3.46	3.84	4.24	4.63	5.04	5.45	5.86	6.28	7.10	7.90	8.70	9.49	10.3
0.25	2.35	2.68	3.01	3.37	3.76	4.15	4.55	4.95	5.35	5.75	6.16	6.99	7.81	8.62	9.42	10.2
0.30	2.28	2.61	2.93	3.29	3.68	4.07	4.47	4.88	5.28	5.68	6.07	6.88	7.71	8.53	9.34	10.1
0.40	2.16	2.48	2.80	3.15	3.53	3.93	4.33	4.74	5.15	5.55	5.95	6.75	7.54	8.33	9.14	9.97
0.50	2.05	2.37	2.68	3.02	3.40	3.79	4.20	4.61	5.02	5.43	5.84	6.64	7.44	8.22	9.01	9.80
0.60	1.94	2.25	2.57	2.90	3.27	3.66	4.06	4.48	4.89	5.31	5.73	6.55	7.35	8.14	8.92	9.70
0.70	1.85	2.15	2.46	2.79	3.15	3.53	3.93	4.35	4.77	5.19	5.61	6.44	7.26	8.06	8.85	9.63
0.80	1.75	2.05	2.36	2.69	3.03	3.41	3.81	4.22	4.64	5.06	5.49	6.33	7.16	7.98	8.78	9.57
0.90	1.67	1.96	2.26	2.59	2.93	3.29	3.69	4.09	4.51	4.93	5.36	6.22	7.06	7.89	8.70	9.50
1.0	1.59	1.87	2.17	2.49	2.83	3.18	3.57	3.97	4.38	4.81	5.24	6.10	6.95	7.79	8.62	9.43
1.2	1.45	1.72	2.00	2.31	2.64	2.98	3.35	3.74	4.14	4.56	4.99	5.85	6.72	7.59	8.43	9.27
1.4	1.33	1.58	1.86	2.15	2.47	2.80	3.15	3.53	3.92	4.33	4.75	5.61	6.48	7.36	8.23	9.08
1.6	1.22	1.46	1.73	2.01	2.31	2.63	2.97	3.33	3.71	4.11	4.52	5.37	6.24	7.12	8.00	8.87
1.8	1.13	1.36	1.61	1.88	2.17	2.48	2.81	3.15	3.52	3.90	4.30	5.14	6.00	6.88	7.77	8.65
2.0	1.05	1.27	1.51	1.77	2.04	2.34	2.66	2.99	3.34	3.71	4.10	4.92	5.77	6.65	7.53	8.42
2.2	0.975	1.18	1.41	1.66	1.93	2.21	2.52	2.84	3.18	3.54	3.91	4.71	5.54	6.41	7.30	8.19
2.4	0.912	1.11	1.33	1.57	1.82	2.10	2.39	2.70	3.03	3.38	3.74	4.51	5.33	6.18	7.06	7.95
2.6	0.856	1.04	1.26	1.48	1.73	1.99	2.27	2.57	2.89	3.23	3.58	4.32	5.12	5.96	6.83	7.71
2.8	0.806	0.986	1.19	1.41	1.65	1.90	2.17	2.46	2.76	3.09	3.43	4.15	4.93	5.75	6.61	7.48
3.0	0.762	0.933	1.13	1.34	1.57	1.81	2.07	2.35	2.64	2.96	3.29	3.99	4.75	5.55	6.39	7.26
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Table 8-10
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 0°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

a	*k*															
	0	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	1.86	2.04	2.23	2.41	2.69	2.97	3.25	3.53	3.80	4.08	4.36	4.92	5.47	6.03	6.59	7.15
0.10	1.86	2.04	2.28	2.53	2.78	3.04	3.31	3.57	3.84	4.11	4.38	4.93	5.48	6.00	6.55	7.10
0.15	1.83	2.03	2.25	2.49	2.74	2.99	3.24	3.50	3.75	4.01	4.28	4.81	5.34	5.89	6.44	7.00
0.20	1.76	1.97	2.18	2.40	2.64	2.87	3.11	3.36	3.60	3.85	4.11	4.62	5.14	5.66	6.20	6.73
0.25	1.66	1.86	2.07	2.29	2.50	2.73	2.95	3.19	3.42	3.66	3.90	4.40	4.90	5.42	5.94	6.47
0.30	1.55	1.74	1.94	2.15	2.36	2.57	2.78	3.00	3.22	3.45	3.69	4.17	4.66	5.17	5.68	6.20
0.40	1.33	1.49	1.67	1.85	2.05	2.24	2.44	2.63	2.84	3.05	3.27	3.73	4.20	4.69	5.19	5.70
0.50	1.15	1.29	1.44	1.60	1.77	1.95	2.13	2.31	2.50	2.70	2.90	3.33	3.78	4.25	4.74	5.23
0.60	0.999	1.12	1.25	1.39	1.54	1.70	1.87	2.04	2.21	2.40	2.59	2.99	3.42	3.87	4.34	4.82
0.70	0.879	0.987	1.10	1.22	1.35	1.50	1.66	1.82	1.98	2.15	2.32	2.71	3.11	3.55	4.00	4.47
0.80	0.783	0.878	0.978	1.09	1.20	1.34	1.48	1.63	1.78	1.94	2.11	2.46	2.85	3.27	3.70	4.15
0.90	0.704	0.790	0.879	0.976	1.08	1.20	1.33	1.48	1.62	1.77	1.92	2.26	2.63	3.02	3.43	3.86
1.0	0.639	0.717	0.797	0.885	0.983	1.09	1.21	1.35	1.48	1.62	1.76	2.08	2.43	2.80	3.20	3.61
1.2	0.538	0.603	0.671	0.745	0.828	0.922	1.03	1.14	1.26	1.38	1.51	1.79	2.10	2.44	2.80	3.18
1.4	0.464	0.520	0.579	0.643	0.715	0.796	0.888	0.991	1.10	1.21	1.32	1.57	1.85	2.15	2.48	2.83
1.6	0.408	0.457	0.508	0.564	0.628	0.700	0.783	0.874	0.972	1.07	1.17	1.40	1.65	1.93	2.22	2.54
1.8	0.363	0.407	0.453	0.503	0.560	0.625	0.699	0.782	0.871	0.957	1.05	1.26	1.49	1.74	2.01	2.31
2.0	0.328	0.367	0.408	0.454	0.505	0.564	0.632	0.706	0.788	0.867	0.952	1.14	1.35	1.58	1.84	2.11
2.2	0.298	0.334	0.372	0.413	0.460	0.514	0.576	0.644	0.719	0.792	0.870	1.04	1.24	1.45	1.69	1.94
2.4	0.274	0.306	0.341	0.379	0.422	0.472	0.529	0.592	0.661	0.728	0.801	0.960	1.14	1.34	1.56	1.79
2.6	0.253	0.283	0.315	0.350	0.390	0.437	0.489	0.547	0.611	0.674	0.741	0.890	1.06	1.24	1.45	1.67
2.8	0.235	0.263	0.293	0.325	0.363	0.406	0.455	0.509	0.568	0.628	0.690	0.829	0.986	1.16	1.35	1.56
3.0	0.219	0.246	0.273	0.304	0.339	0.379	0.425	0.475	0.531	0.587	0.645	0.776	0.924	1.09	1.27	1.46
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

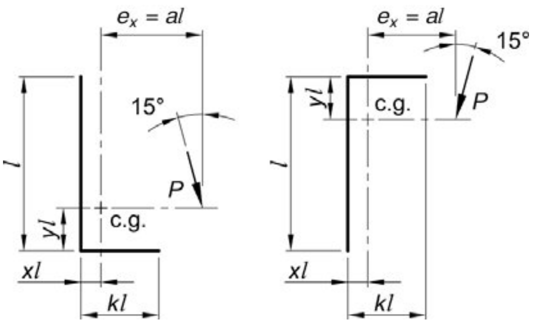
Table 8-10 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 15°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.



a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	1.98	2.20	2.47	2.74	3.01	3.29	3.56	3.83	4.10	4.38	4.65	5.19	5.74	6.28	6.83	7.37
0.10	1.90	2.13	2.41	2.68	2.97	3.25	3.53	3.81	4.09	4.36	4.64	5.18	5.73	6.28	6.83	7.37
0.15	1.84	2.10	2.35	2.62	2.88	3.15	3.42	3.69	3.96	4.23	4.50	5.04	5.58	6.12	6.66	7.20
0.20	1.76	1.99	2.26	2.52	2.77	3.02	3.28	3.53	3.79	4.05	4.31	4.84	5.37	5.90	6.44	6.98
0.25	1.65	1.87	2.11	2.37	2.63	2.87	3.11	3.36	3.60	3.85	4.10	4.61	5.13	5.66	6.19	6.72
0.30	1.55	1.75	1.97	2.20	2.45	2.69	2.93	3.16	3.40	3.64	3.88	4.38	4.89	5.41	5.93	6.46
0.40	1.34	1.51	1.69	1.89	2.10	2.33	2.56	2.77	2.99	3.21	3.44	3.91	4.41	4.91	5.42	5.94
0.50	1.16	1.31	1.46	1.63	1.81	2.01	2.21	2.42	2.63	2.83	3.05	3.50	3.97	4.45	4.95	5.46
0.60	1.01	1.14	1.27	1.42	1.58	1.75	1.93	2.13	2.32	2.51	2.71	3.14	3.59	4.06	4.54	5.04
0.70	0.895	1.01	1.12	1.25	1.39	1.54	1.71	1.89	2.07	2.25	2.44	2.84	3.26	3.71	4.18	4.66
0.80	0.799	0.898	1.00	1.11	1.24	1.38	1.53	1.69	1.86	2.03	2.21	2.58	2.99	3.41	3.86	4.32
0.90	0.720	0.809	0.901	1.00	1.11	1.24	1.38	1.53	1.69	1.85	2.01	2.36	2.75	3.15	3.58	4.03
1.0	0.654	0.735	0.818	0.910	1.01	1.13	1.25	1.39	1.54	1.69	1.85	2.18	2.54	2.92	3.33	3.76
1.2	0.552	0.620	0.690	0.767	0.854	0.951	1.06	1.18	1.31	1.45	1.58	1.87	2.20	2.54	2.92	3.31
1.4	0.477	0.535	0.596	0.662	0.737	0.822	0.918	1.03	1.14	1.26	1.38	1.64	1.93	2.25	2.58	2.94
1.6	0.420	0.471	0.524	0.582	0.648	0.724	0.809	0.905	1.01	1.11	1.22	1.46	1.72	2.01	2.32	2.65
1.8	0.374	0.420	0.467	0.519	0.578	0.646	0.723	0.809	0.902	0.997	1.09	1.31	1.55	1.81	2.09	2.40
2.0	0.338	0.378	0.421	0.468	0.522	0.583	0.653	0.731	0.816	0.902	0.991	1.19	1.41	1.65	1.91	2.19
2.2	0.308	0.345	0.383	0.426	0.475	0.532	0.596	0.666	0.744	0.824	0.905	1.08	1.29	1.51	1.75	2.01
2.4	0.282	0.316	0.352	0.391	0.436	0.488	0.547	0.612	0.684	0.757	0.833	0.999	1.19	1.39	1.62	1.86
2.6	0.261	0.292	0.325	0.362	0.403	0.451	0.506	0.566	0.632	0.701	0.771	0.925	1.10	1.29	1.50	1.73
2.8	0.242	0.272	0.302	0.336	0.375	0.420	0.470	0.526	0.588	0.652	0.717	0.862	1.03	1.21	1.40	1.62
3.0	0.226	0.254	0.282	0.314	0.350	0.392	0.439	0.492	0.549	0.610	0.671	0.806	0.960	1.13	1.32	1.52
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

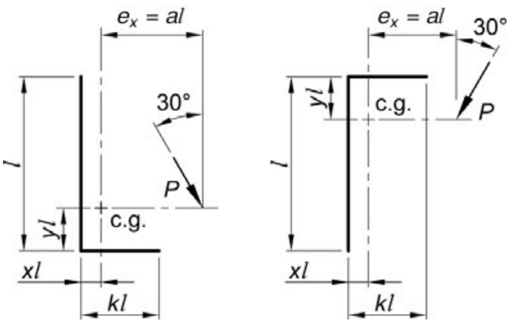
Table 8-10 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 30°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.



a	*k* = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.18	2.44	2.70	2.96	3.21	3.47	3.73	3.98	4.24	4.50	4.76	5.27	5.78	6.30	6.81	7.33
0.10	2.02	2.35	2.66	2.96	3.24	3.52	3.79	4.06	4.33	4.59	4.86	5.38	5.90	6.43	6.95	7.47
0.15	1.92	2.22	2.53	2.84	3.13	3.41	3.69	3.96	4.23	4.49	4.76	5.29	5.81	6.34	6.86	7.38
0.20	1.82	2.09	2.38	2.67	2.97	3.26	3.53	3.80	4.07	4.33	4.60	5.13	5.65	6.18	6.71	7.23
0.25	1.71	1.96	2.22	2.50	2.78	3.06	3.34	3.60	3.87	4.13	4.39	4.92	5.45	5.98	6.51	7.05
0.30	1.61	1.83	2.07	2.32	2.59	2.86	3.13	3.40	3.65	3.91	4.17	4.70	5.23	5.76	6.30	6.83
0.40	1.41	1.59	1.79	2.01	2.23	2.47	2.72	2.98	3.24	3.48	3.72	4.23	4.75	5.28	5.82	6.37
0.50	1.23	1.39	1.56	1.74	1.94	2.15	2.37	2.61	2.85	3.09	3.32	3.80	4.30	4.83	5.36	5.90
0.60	1.08	1.22	1.37	1.53	1.70	1.89	2.09	2.30	2.53	2.75	2.97	3.43	3.91	4.41	4.94	5.47
0.70	0.964	1.09	1.21	1.35	1.51	1.67	1.86	2.05	2.26	2.48	2.68	3.12	3.58	4.06	4.56	5.07
0.80	0.865	0.974	1.09	1.21	1.35	1.50	1.66	1.84	2.04	2.24	2.44	2.84	3.28	3.74	4.22	4.72
0.90	0.783	0.881	0.983	1.09	1.22	1.36	1.51	1.67	1.85	2.04	2.23	2.61	3.03	3.47	3.93	4.40
1.0	0.714	0.803	0.896	0.997	1.11	1.24	1.38	1.53	1.70	1.88	2.05	2.41	2.80	3.22	3.66	4.12
1.2	0.606	0.681	0.759	0.844	0.940	1.05	1.17	1.30	1.45	1.61	1.76	2.08	2.43	2.81	3.22	3.64
1.4	0.525	0.590	0.657	0.731	0.814	0.908	1.02	1.13	1.26	1.40	1.53	1.82	2.14	2.49	2.86	3.25
1.6	0.463	0.520	0.579	0.644	0.717	0.801	0.897	1.00	1.12	1.24	1.36	1.62	1.91	2.23	2.57	2.92
1.8	0.414	0.464	0.517	0.575	0.641	0.716	0.802	0.897	1.00	1.11	1.22	1.46	1.72	2.01	2.32	2.66
2.0	0.374	0.419	0.467	0.519	0.579	0.647	0.725	0.811	0.905	1.01	1.11	1.32	1.57	1.83	2.12	2.43
2.2	0.341	0.382	0.425	0.473	0.528	0.590	0.661	0.740	0.826	0.919	1.01	1.21	1.44	1.68	1.95	2.24
2.4	0.313	0.351	0.391	0.434	0.485	0.543	0.608	0.680	0.760	0.845	0.930	1.12	1.32	1.55	1.80	2.07
2.6	0.289	0.324	0.361	0.402	0.448	0.502	0.562	0.629	0.703	0.782	0.861	1.03	1.23	1.44	1.67	1.92
2.8	0.269	0.302	0.336	0.373	0.417	0.467	0.523	0.585	0.654	0.727	0.801	0.963	1.15	1.35	1.56	1.80
3.0	0.251	0.282	0.314	0.349	0.389	0.436	0.489	0.547	0.611	0.680	0.749	0.901	1.07	1.26	1.47	1.69
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

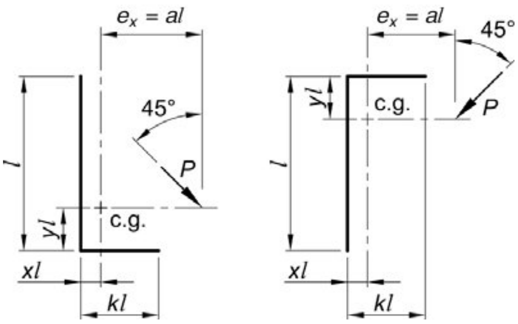
Table 8-10 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 45°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.



a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.41	2.57	2.80	3.04	3.27	3.51	3.74	3.97	4.21	4.44	4.67	5.14	5.61	6.08	6.54	7.01
0.10	2.24	2.54	2.83	3.12	3.40	3.67	3.94	4.20	4.45	4.71	4.95	5.44	5.93	6.41	6.89	7.36
0.15	2.09	2.41	2.71	3.00	3.28	3.57	3.85	4.13	4.40	4.67	4.93	5.43	5.92	6.41	6.90	7.38
0.20	1.96	2.26	2.56	2.84	3.13	3.42	3.71	4.00	4.28	4.56	4.84	5.35	5.85	6.35	6.85	7.34
0.25	1.85	2.12	2.40	2.68	2.96	3.25	3.54	3.83	4.12	4.41	4.69	5.22	5.74	6.25	6.75	7.25
0.30	1.74	1.99	2.25	2.51	2.79	3.07	3.35	3.64	3.93	4.23	4.52	5.06	5.59	6.11	6.63	7.14
0.40	1.55	1.76	1.98	2.21	2.46	2.71	2.98	3.26	3.54	3.83	4.13	4.68	5.23	5.77	6.31	6.84
0.50	1.38	1.56	1.75	1.95	2.17	2.40	2.64	2.90	3.17	3.45	3.74	4.29	4.84	5.40	5.95	6.49
0.60	1.23	1.39	1.56	1.74	1.93	2.14	2.36	2.60	2.85	3.12	3.39	3.92	4.46	5.02	5.58	6.13
0.70	1.11	1.25	1.40	1.56	1.73	1.92	2.13	2.35	2.59	2.84	3.10	3.59	4.12	4.66	5.21	5.77
0.80	1.00	1.13	1.26	1.41	1.57	1.74	1.93	2.14	2.36	2.59	2.84	3.30	3.80	4.33	4.87	5.42
0.90	0.915	1.03	1.15	1.28	1.43	1.59	1.76	1.96	2.16	2.38	2.61	3.06	3.53	4.04	4.56	5.10
1.0	0.839	0.945	1.06	1.18	1.31	1.46	1.62	1.80	2.00	2.20	2.42	2.84	3.29	3.77	4.28	4.80
1.2	0.719	0.809	0.902	1.00	1.12	1.25	1.39	1.55	1.72	1.90	2.10	2.48	2.88	3.32	3.79	4.28
1.4	0.627	0.705	0.786	0.875	0.975	1.09	1.22	1.36	1.51	1.67	1.84	2.19	2.56	2.96	3.39	3.85
1.6	0.555	0.624	0.695	0.774	0.863	0.964	1.08	1.20	1.34	1.49	1.64	1.96	2.30	2.66	3.06	3.48
1.8	0.498	0.559	0.623	0.693	0.773	0.865	0.968	1.08	1.20	1.34	1.48	1.76	2.08	2.42	2.78	3.17
2.0	0.451	0.506	0.564	0.628	0.700	0.783	0.877	0.980	1.09	1.21	1.34	1.61	1.89	2.21	2.55	2.91
2.2	0.412	0.462	0.515	0.573	0.639	0.716	0.801	0.896	0.999	1.11	1.23	1.47	1.74	2.03	2.35	2.69
2.4	0.379	0.425	0.474	0.527	0.588	0.659	0.738	0.825	0.920	1.02	1.13	1.36	1.61	1.88	2.17	2.49
2.6	0.351	0.394	0.438	0.488	0.545	0.610	0.683	0.764	0.853	0.948	1.05	1.26	1.49	1.75	2.03	2.32
2.8	0.327	0.366	0.408	0.454	0.507	0.568	0.636	0.712	0.794	0.883	0.979	1.18	1.39	1.63	1.89	2.18
3.0	0.306	0.343	0.381	0.424	0.474	0.531	0.595	0.666	0.743	0.826	0.916	1.10	1.31	1.53	1.78	2.04
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

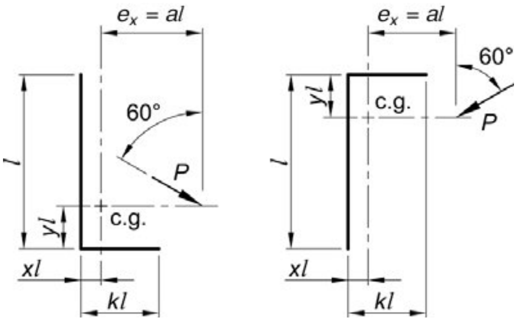
Table 8-10 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 60°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.



a	*k* = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.60	2.79	3.01	3.23	3.44	3.66	3.88	4.10	4.32	4.54	4.76	5.19	5.63	6.07	6.50	6.94
0.10	2.43	2.70	2.97	3.23	3.48	3.72	3.96	4.19	4.42	4.64	4.87	5.31	5.75	6.18	6.62	7.05
0.15	2.31	2.59	2.86	3.13	3.40	3.66	3.91	4.16	4.40	4.64	4.87	5.32	5.77	6.21	6.64	7.08
0.20	2.18	2.47	2.74	3.01	3.29	3.56	3.83	4.09	4.35	4.59	4.84	5.30	5.76	6.21	6.65	7.09
0.25	2.07	2.35	2.62	2.89	3.16	3.44	3.72	3.99	4.26	4.52	4.77	5.26	5.73	6.19	6.64	7.08
0.30	1.97	2.24	2.50	2.76	3.03	3.31	3.59	3.88	4.16	4.43	4.69	5.20	5.68	6.15	6.61	7.06
0.40	1.79	2.03	2.27	2.52	2.77	3.04	3.32	3.61	3.90	4.19	4.48	5.02	5.54	6.04	6.52	6.98
0.50	1.63	1.84	2.06	2.29	2.53	2.78	3.05	3.34	3.63	3.93	4.22	4.79	5.34	5.87	6.37	6.86
0.60	1.49	1.68	1.88	2.09	2.31	2.55	2.81	3.08	3.37	3.66	3.96	4.55	5.11	5.66	6.19	6.69
0.70	1.37	1.54	1.73	1.92	2.12	2.35	2.59	2.85	3.12	3.41	3.71	4.30	4.87	5.43	5.97	6.50
0.80	1.26	1.42	1.59	1.77	1.96	2.17	2.40	2.64	2.90	3.18	3.47	4.05	4.64	5.20	5.74	6.28
0.90	1.17	1.32	1.47	1.63	1.81	2.01	2.23	2.46	2.71	2.97	3.25	3.82	4.39	4.95	5.50	6.04
1.0	1.08	1.22	1.36	1.52	1.69	1.87	2.08	2.30	2.53	2.78	3.05	3.60	4.15	4.71	5.26	5.80
1.2	0.946	1.07	1.19	1.32	1.47	1.64	1.82	2.02	2.23	2.46	2.70	3.21	3.72	4.26	4.80	5.34
1.4	0.837	0.942	1.05	1.17	1.30	1.45	1.62	1.80	1.99	2.20	2.42	2.88	3.36	3.86	4.38	4.92
1.6	0.748	0.842	0.939	1.04	1.16	1.30	1.45	1.61	1.79	1.98	2.18	2.60	3.04	3.52	4.02	4.53
1.8	0.676	0.760	0.847	0.943	1.05	1.17	1.31	1.46	1.62	1.80	1.98	2.37	2.78	3.23	3.70	4.19
2.0	0.616	0.692	0.772	0.859	0.958	1.07	1.20	1.33	1.48	1.64	1.82	2.18	2.55	2.97	3.42	3.88
2.2	0.565	0.635	0.708	0.788	0.879	0.983	1.10	1.23	1.36	1.51	1.67	2.01	2.36	2.75	3.17	3.61
2.4	0.522	0.586	0.653	0.728	0.812	0.908	1.02	1.13	1.26	1.40	1.55	1.86	2.19	2.56	2.95	3.37
2.6	0.485	0.544	0.607	0.675	0.754	0.844	0.944	1.05	1.17	1.30	1.44	1.74	2.05	2.39	2.76	3.16
2.8	0.453	0.508	0.566	0.630	0.704	0.787	0.881	0.984	1.10	1.22	1.35	1.63	1.92	2.24	2.59	2.97
3.0	0.424	0.476	0.530	0.590	0.659	0.738	0.826	0.923	1.03	1.14	1.26	1.53	1.80	2.11	2.44	2.80
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

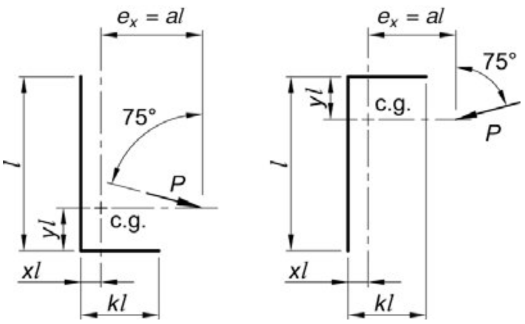
Table 8-10 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 75°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.



a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.74	2.92	3.11	3.30	3.49	3.69	3.88	4.07	4.26	4.46	4.65	5.03	5.42	5.80	6.19	6.57
0.10	2.59	2.86	3.11	3.31	3.50	3.69	3.88	4.07	4.27	4.46	4.65	5.04	5.42	5.81	6.20	6.58
0.15	2.50	2.78	3.04	3.28	3.50	3.70	3.90	4.09	4.28	4.47	4.67	5.05	5.44	5.83	6.21	6.60
0.20	2.43	2.69	2.96	3.22	3.46	3.68	3.89	4.09	4.29	4.48	4.68	5.06	5.45	5.84	6.22	6.61
0.25	2.35	2.62	2.88	3.14	3.40	3.63	3.86	4.07	4.28	4.48	4.68	5.07	5.46	5.84	6.23	6.61
0.30	2.28	2.55	2.80	3.07	3.33	3.58	3.82	4.04	4.26	4.46	4.67	5.06	5.46	5.84	6.23	6.62
0.40	2.16	2.41	2.66	2.92	3.19	3.45	3.71	3.95	4.18	4.41	4.62	5.04	5.44	5.83	6.23	6.61
0.50	2.05	2.29	2.53	2.78	3.05	3.32	3.58	3.84	4.09	4.32	4.55	4.99	5.40	5.81	6.21	6.60
0.60	1.94	2.18	2.41	2.64	2.90	3.18	3.45	3.72	3.97	4.22	4.46	4.92	5.35	5.77	6.17	6.57
0.70	1.85	2.07	2.29	2.52	2.77	3.04	3.31	3.58	3.85	4.11	4.36	4.83	5.28	5.71	6.12	6.53
0.80	1.75	1.97	2.18	2.40	2.64	2.90	3.18	3.45	3.73	3.99	4.25	4.74	5.20	5.64	6.06	6.48
0.90	1.67	1.87	2.08	2.29	2.52	2.77	3.04	3.32	3.60	3.87	4.14	4.65	5.12	5.57	6.00	6.42
1.0	1.59	1.79	1.98	2.19	2.41	2.65	2.92	3.19	3.47	3.75	4.02	4.55	5.04	5.50	5.94	6.37
1.2	1.45	1.63	1.81	2.00	2.21	2.44	2.68	2.95	3.22	3.50	3.78	4.33	4.85	5.34	5.81	6.25
1.4	1.33	1.49	1.66	1.84	2.03	2.24	2.47	2.72	2.99	3.27	3.55	4.11	4.65	5.16	5.65	6.12
1.6	1.22	1.37	1.53	1.69	1.88	2.07	2.29	2.53	2.78	3.05	3.32	3.88	4.43	4.97	5.48	5.96
1.8	1.13	1.27	1.41	1.57	1.74	1.93	2.13	2.35	2.59	2.85	3.11	3.66	4.22	4.76	5.29	5.79
2.0	1.05	1.18	1.31	1.46	1.62	1.79	1.99	2.20	2.42	2.67	2.92	3.46	4.01	4.56	5.09	5.61
2.2	0.975	1.10	1.22	1.36	1.51	1.68	1.86	2.06	2.27	2.50	2.75	3.27	3.81	4.36	4.90	5.42
2.4	0.912	1.03	1.14	1.27	1.41	1.57	1.74	1.93	2.14	2.36	2.59	3.09	3.62	4.16	4.70	5.23
2.6	0.856	0.963	1.07	1.19	1.33	1.48	1.64	1.82	2.02	2.22	2.45	2.93	3.44	3.97	4.50	5.03
2.8	0.806	0.906	1.01	1.12	1.25	1.39	1.55	1.72	1.90	2.10	2.32	2.78	3.28	3.79	4.30	4.83
3.0	0.762	0.856	0.954	1.06	1.18	1.32	1.47	1.63	1.80	2.00	2.20	2.64	3.12	3.61	4.12	4.64
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

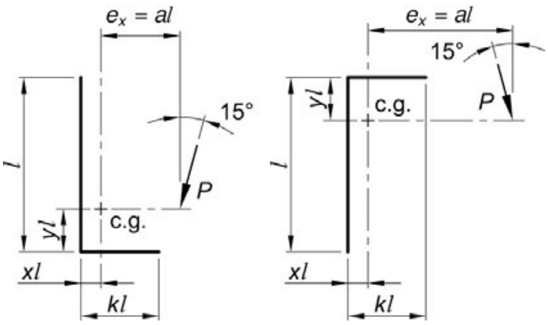
Table 8-10a
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 15°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.



a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	1.98	2.20	2.47	2.74	3.01	3.29	3.56	3.83	4.10	4.38	4.65	5.19	5.74	6.28	6.83	7.37
0.10	1.90	2.08	2.30	2.54	2.79	3.04	3.30	3.57	3.84	4.12	4.41	4.99	5.57	6.15	6.72	7.29
0.15	1.84	2.04	2.25	2.47	2.70	2.94	3.18	3.43	3.68	3.94	4.19	4.72	5.26	5.82	6.38	6.95
0.20	1.76	1.97	2.17	2.38	2.59	2.82	3.04	3.28	3.52	3.76	4.00	4.51	5.02	5.55	6.08	6.63
0.25	1.65	1.86	2.07	2.26	2.46	2.67	2.89	3.11	3.33	3.57	3.80	4.29	4.79	5.30	5.82	6.35
0.30	1.55	1.74	1.95	2.13	2.32	2.52	2.72	2.93	3.15	3.37	3.60	4.07	4.56	5.06	5.57	6.09
0.40	1.34	1.51	1.70	1.87	2.04	2.22	2.40	2.59	2.79	3.00	3.21	3.66	4.12	4.61	5.10	5.61
0.50	1.16	1.31	1.47	1.63	1.79	1.95	2.12	2.29	2.47	2.67	2.87	3.29	3.74	4.20	4.69	5.18
0.60	1.01	1.15	1.29	1.42	1.57	1.72	1.88	2.04	2.21	2.38	2.57	2.97	3.40	3.85	4.32	4.80
0.70	0.895	1.01	1.13	1.25	1.39	1.53	1.68	1.82	1.98	2.15	2.32	2.70	3.11	3.54	3.99	4.46
0.80	0.799	0.906	1.01	1.12	1.24	1.37	1.51	1.65	1.79	1.95	2.11	2.47	2.86	3.27	3.71	4.16
0.90	0.720	0.816	0.909	1.01	1.12	1.24	1.37	1.50	1.63	1.78	1.94	2.27	2.64	3.04	3.45	3.89
1.0	0.654	0.742	0.825	0.915	1.01	1.12	1.25	1.37	1.50	1.64	1.78	2.10	2.45	2.83	3.22	3.64
1.2	0.552	0.626	0.695	0.771	0.856	0.950	1.06	1.17	1.29	1.41	1.54	1.82	2.13	2.47	2.84	3.22
1.4	0.477	0.540	0.600	0.665	0.739	0.822	0.916	1.02	1.12	1.23	1.35	1.60	1.88	2.19	2.52	2.87
1.6	0.420	0.474	0.527	0.585	0.650	0.724	0.808	0.901	0.995	1.09	1.20	1.43	1.68	1.96	2.27	2.59
1.8	0.374	0.422	0.469	0.521	0.580	0.646	0.722	0.806	0.892	0.981	1.08	1.28	1.52	1.78	2.05	2.35
2.0	0.338	0.381	0.423	0.470	0.523	0.584	0.653	0.729	0.809	0.889	0.976	1.17	1.38	1.62	1.88	2.16
2.2	0.308	0.346	0.385	0.428	0.476	0.532	0.595	0.665	0.739	0.813	0.893	1.07	1.27	1.49	1.73	1.99
2.4	0.282	0.318	0.353	0.393	0.437	0.489	0.547	0.612	0.680	0.749	0.822	0.986	1.17	1.37	1.60	1.84
2.6	0.261	0.294	0.326	0.363	0.404	0.452	0.506	0.566	0.630	0.694	0.762	0.914	1.09	1.28	1.49	1.71
2.8	0.242	0.273	0.303	0.337	0.376	0.420	0.470	0.526	0.586	0.646	0.710	0.852	1.01	1.19	1.39	1.60
3.0	0.226	0.255	0.283	0.315	0.351	0.392	0.439	0.492	0.549	0.604	0.664	0.798	0.949	1.12	1.30	1.50
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

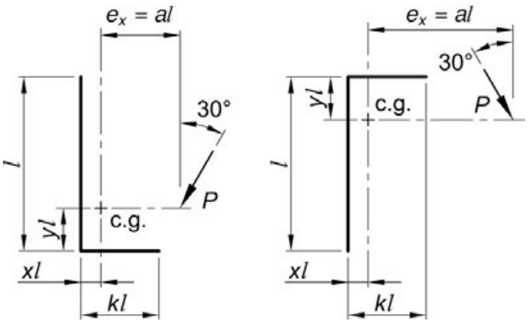
Table 8-10a (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 30°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.



a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.18	2.44	2.70	2.96	3.21	3.47	3.73	3.98	4.24	4.50	4.76	5.27	5.78	6.30	6.81	7.33
0.10	2.02	2.24	2.47	2.70	2.94	3.18	3.43	3.69	3.95	4.21	4.48	5.01	5.56	6.11	6.65	7.20
0.15	1.92	2.13	2.34	2.55	2.77	3.00	3.23	3.47	3.71	3.96	4.21	4.73	5.27	5.82	6.37	6.93
0.20	1.82	2.02	2.23	2.43	2.64	2.85	3.07	3.29	3.52	3.76	4.00	4.50	5.01	5.55	6.09	6.64
0.25	1.71	1.91	2.11	2.31	2.50	2.70	2.91	3.12	3.34	3.57	3.80	4.28	4.78	5.30	5.83	6.37
0.30	1.61	1.79	1.98	2.18	2.37	2.56	2.75	2.96	3.17	3.39	3.61	4.08	4.57	5.08	5.60	6.13
0.40	1.41	1.57	1.74	1.92	2.10	2.28	2.45	2.64	2.84	3.04	3.26	3.71	4.18	4.67	5.18	5.69
0.50	1.23	1.38	1.53	1.70	1.87	2.03	2.19	2.36	2.55	2.74	2.94	3.37	3.83	4.30	4.80	5.30
0.60	1.08	1.22	1.36	1.51	1.66	1.81	1.96	2.13	2.30	2.48	2.67	3.08	3.52	3.98	4.46	4.95
0.70	0.964	1.08	1.21	1.35	1.49	1.63	1.77	1.92	2.08	2.26	2.44	2.83	3.25	3.69	4.15	4.64
0.80	0.865	0.974	1.09	1.22	1.34	1.48	1.61	1.75	1.90	2.06	2.23	2.60	3.01	3.44	3.89	4.35
0.90	0.783	0.882	0.989	1.10	1.22	1.34	1.47	1.60	1.74	1.90	2.06	2.41	2.80	3.21	3.64	4.09
1.0	0.714	0.805	0.904	1.01	1.11	1.23	1.35	1.48	1.61	1.75	1.91	2.24	2.61	3.00	3.42	3.86
1.2	0.606	0.684	0.769	0.852	0.944	1.05	1.16	1.27	1.39	1.52	1.66	1.96	2.29	2.65	3.04	3.44
1.4	0.525	0.593	0.665	0.737	0.818	0.908	1.01	1.11	1.22	1.34	1.46	1.73	2.04	2.37	2.72	3.09
1.6	0.463	0.523	0.585	0.649	0.720	0.801	0.892	0.990	1.09	1.19	1.30	1.55	1.83	2.13	2.46	2.80
1.8	0.414	0.468	0.522	0.579	0.644	0.717	0.799	0.890	0.978	1.07	1.18	1.40	1.66	1.93	2.23	2.56
2.0	0.374	0.423	0.471	0.523	0.581	0.648	0.724	0.807	0.889	0.977	1.07	1.28	1.51	1.77	2.05	2.35
2.2	0.341	0.386	0.429	0.476	0.530	0.591	0.661	0.738	0.814	0.895	0.982	1.17	1.39	1.63	1.89	2.17
2.4	0.313	0.354	0.394	0.437	0.487	0.543	0.608	0.679	0.750	0.825	0.906	1.08	1.29	1.51	1.75	2.02
2.6	0.289	0.327	0.364	0.404	0.450	0.503	0.562	0.628	0.696	0.766	0.841	1.01	1.20	1.40	1.63	1.88
2.8	0.269	0.304	0.338	0.376	0.418	0.467	0.523	0.585	0.648	0.714	0.784	0.940	1.12	1.31	1.53	1.76
3.0	0.251	0.284	0.316	0.351	0.391	0.437	0.489	0.547	0.607	0.668	0.734	0.881	1.05	1.23	1.44	1.66
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

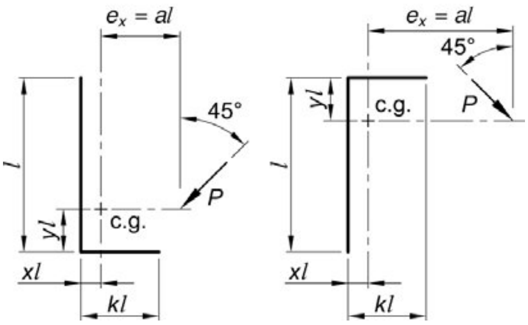
Table 8-10a (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 45°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.



a	*k* = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.41	2.57	2.80	3.04	3.27	3.51	3.74	3.97	4.21	4.44	4.67	5.14	5.61	6.08	6.54	7.01
0.10	2.24	2.44	2.65	2.86	3.07	3.29	3.52	3.76	4.00	4.24	4.49	5.01	5.53	6.06	6.59	7.12
0.15	2.09	2.28	2.48	2.68	2.89	3.11	3.33	3.56	3.79	4.03	4.28	4.79	5.32	5.85	6.40	6.94
0.20	1.96	2.14	2.33	2.54	2.74	2.95	3.16	3.38	3.61	3.84	4.08	4.58	5.10	5.64	6.19	6.74
0.25	1.85	2.02	2.21	2.40	2.61	2.81	3.01	3.22	3.44	3.67	3.90	4.39	4.90	5.43	5.98	6.53
0.30	1.74	1.91	2.09	2.28	2.47	2.67	2.87	3.07	3.29	3.51	3.73	4.21	4.72	5.24	5.78	6.33
0.40	1.55	1.70	1.87	2.04	2.23	2.42	2.60	2.80	3.00	3.21	3.43	3.89	4.38	4.89	5.41	5.95
0.50	1.38	1.52	1.67	1.84	2.01	2.19	2.36	2.55	2.74	2.94	3.15	3.60	4.07	4.57	5.09	5.62
0.60	1.23	1.36	1.50	1.66	1.82	1.99	2.16	2.33	2.51	2.70	2.90	3.33	3.80	4.28	4.79	5.31
0.70	1.11	1.23	1.36	1.50	1.66	1.82	1.97	2.13	2.31	2.49	2.68	3.10	3.55	4.02	4.52	5.03
0.80	1.00	1.12	1.24	1.37	1.52	1.67	1.81	1.97	2.13	2.31	2.49	2.89	3.33	3.79	4.27	4.77
0.90	0.915	1.02	1.13	1.26	1.39	1.54	1.67	1.82	1.98	2.14	2.32	2.71	3.12	3.57	4.04	4.53
1.0	0.839	0.938	1.04	1.16	1.29	1.42	1.55	1.69	1.84	2.00	2.17	2.54	2.94	3.37	3.83	4.30
1.2	0.719	0.805	0.900	1.00	1.12	1.24	1.35	1.48	1.61	1.76	1.91	2.25	2.62	3.02	3.45	3.90
1.4	0.627	0.704	0.788	0.880	0.979	1.08	1.19	1.31	1.43	1.56	1.70	2.01	2.36	2.73	3.13	3.54
1.6	0.555	0.624	0.700	0.783	0.868	0.962	1.07	1.17	1.28	1.40	1.53	1.82	2.14	2.48	2.85	3.24
1.8	0.498	0.560	0.629	0.701	0.778	0.864	0.961	1.06	1.16	1.27	1.39	1.66	1.95	2.27	2.61	2.98
2.0	0.451	0.508	0.571	0.635	0.704	0.784	0.873	0.964	1.06	1.16	1.27	1.52	1.79	2.09	2.41	2.75
2.2	0.412	0.464	0.522	0.579	0.643	0.716	0.799	0.885	0.974	1.07	1.17	1.40	1.65	1.93	2.23	2.55
2.4	0.379	0.428	0.480	0.532	0.592	0.660	0.736	0.818	0.900	0.989	1.08	1.30	1.53	1.79	2.08	2.38
2.6	0.351	0.396	0.444	0.493	0.548	0.611	0.683	0.760	0.837	0.920	1.01	1.21	1.43	1.67	1.94	2.23
2.8	0.327	0.369	0.413	0.458	0.510	0.569	0.636	0.709	0.781	0.859	0.943	1.13	1.34	1.57	1.82	2.10
3.0	0.306	0.345	0.386	0.428	0.477	0.532	0.595	0.665	0.733	0.806	0.885	1.06	1.26	1.48	1.72	1.98
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

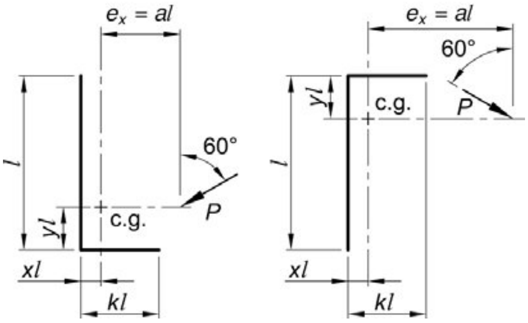
Table 8-10a (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 60°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.



a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.60	2.79	3.01	3.23	3.44	3.66	3.88	4.10	4.32	4.54	4.76	5.19	5.63	6.07	6.50	6.94
0.10	2.43	2.59	2.76	2.94	3.14	3.35	3.57	3.80	4.03	4.28	4.52	5.04	5.56	6.07	6.56	7.03
0.15	2.31	2.45	2.62	2.80	3.00	3.21	3.43	3.66	3.89	4.13	4.38	4.89	5.43	5.96	6.48	6.97
0.20	2.18	2.32	2.49	2.67	2.87	3.08	3.30	3.52	3.75	3.99	4.23	4.75	5.28	5.83	6.37	6.88
0.25	2.07	2.21	2.38	2.56	2.75	2.96	3.17	3.40	3.62	3.86	4.10	4.60	5.14	5.69	6.24	6.78
0.30	1.97	2.11	2.27	2.45	2.64	2.84	3.06	3.28	3.50	3.73	3.97	4.47	5.00	5.55	6.11	6.66
0.40	1.79	1.93	2.08	2.25	2.44	2.64	2.84	3.06	3.27	3.50	3.73	4.22	4.74	5.28	5.84	6.41
0.50	1.63	1.76	1.91	2.08	2.26	2.45	2.65	2.86	3.06	3.28	3.51	3.99	4.50	5.03	5.58	6.15
0.60	1.49	1.62	1.76	1.92	2.09	2.28	2.47	2.67	2.87	3.08	3.30	3.77	4.28	4.80	5.35	5.91
0.70	1.37	1.49	1.63	1.78	1.95	2.12	2.31	2.50	2.70	2.90	3.12	3.58	4.07	4.59	5.13	5.68
0.80	1.26	1.38	1.51	1.66	1.82	1.99	2.17	2.35	2.54	2.74	2.95	3.39	3.88	4.39	4.92	5.46
0.90	1.17	1.28	1.41	1.55	1.70	1.86	2.04	2.21	2.39	2.58	2.79	3.23	3.70	4.20	4.72	5.25
1.0	1.08	1.19	1.31	1.45	1.59	1.75	1.92	2.08	2.26	2.45	2.64	3.07	3.53	4.02	4.53	5.05
1.2	0.946	1.05	1.16	1.28	1.41	1.56	1.71	1.87	2.03	2.20	2.39	2.79	3.22	3.69	4.17	4.68
1.4	0.837	0.928	1.03	1.14	1.27	1.40	1.54	1.68	1.83	2.00	2.17	2.54	2.96	3.40	3.86	4.34
1.6	0.748	0.832	0.926	1.03	1.14	1.27	1.40	1.53	1.67	1.82	1.98	2.33	2.72	3.14	3.58	4.04
1.8	0.676	0.754	0.840	0.936	1.04	1.16	1.28	1.40	1.53	1.67	1.82	2.15	2.52	2.91	3.33	3.77
2.0	0.616	0.688	0.768	0.857	0.957	1.07	1.17	1.29	1.41	1.54	1.68	1.99	2.34	2.71	3.11	3.53
2.2	0.565	0.632	0.707	0.790	0.883	0.981	1.08	1.19	1.30	1.43	1.56	1.85	2.18	2.53	2.91	3.31
2.4	0.522	0.585	0.655	0.733	0.818	0.909	1.01	1.11	1.21	1.33	1.46	1.73	2.04	2.37	2.73	3.11
2.6	0.485	0.544	0.609	0.682	0.760	0.845	0.940	1.03	1.13	1.24	1.36	1.62	1.91	2.23	2.57	2.93
2.8	0.453	0.508	0.570	0.638	0.709	0.789	0.879	0.969	1.06	1.17	1.28	1.53	1.80	2.10	2.43	2.78
3.0	0.424	0.476	0.535	0.598	0.665	0.740	0.825	0.911	1.00	1.10	1.21	1.44	1.70	1.99	2.30	2.63
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

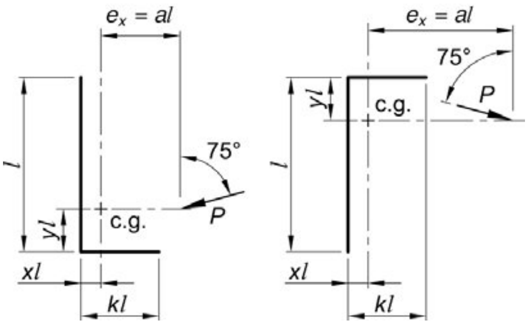
Table 8-10a (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 75°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.



a	*k* = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.74	2.92	3.11	3.30	3.49	3.69	3.88	4.07	4.26	4.46	4.65	5.03	5.42	5.80	6.19	6.57
0.10	2.59	2.68	2.81	2.97	3.16	3.36	3.58	3.82	4.07	4.33	4.58	5.06	5.50	5.91	6.31	6.69
0.15	2.50	2.60	2.74	2.90	3.08	3.29	3.51	3.75	4.00	4.26	4.52	5.02	5.48	5.90	6.30	6.69
0.20	2.43	2.53	2.66	2.83	3.01	3.22	3.44	3.68	3.93	4.19	4.46	4.98	5.45	5.88	6.29	6.69
0.25	2.35	2.46	2.60	2.76	2.94	3.15	3.37	3.61	3.86	4.12	4.39	4.92	5.42	5.86	6.28	6.68
0.30	2.28	2.39	2.53	2.69	2.88	3.09	3.31	3.54	3.79	4.05	4.32	4.86	5.37	5.84	6.26	6.67
0.40	2.16	2.27	2.41	2.57	2.76	2.96	3.18	3.42	3.66	3.92	4.19	4.72	5.27	5.77	6.22	6.64
0.50	2.05	2.16	2.30	2.46	2.64	2.85	3.06	3.30	3.54	3.80	4.06	4.59	5.13	5.66	6.15	6.59
0.60	1.94	2.05	2.19	2.35	2.54	2.73	2.95	3.18	3.42	3.68	3.93	4.46	5.00	5.54	6.06	6.54
0.70	1.85	1.96	2.10	2.25	2.43	2.63	2.84	3.07	3.31	3.56	3.81	4.33	4.87	5.42	5.95	6.45
0.80	1.75	1.87	2.00	2.16	2.34	2.53	2.74	2.97	3.20	3.45	3.69	4.21	4.75	5.30	5.84	6.35
0.90	1.67	1.78	1.92	2.07	2.25	2.44	2.65	2.87	3.10	3.34	3.58	4.09	4.62	5.17	5.72	6.24
1.0	1.59	1.70	1.84	1.99	2.16	2.35	2.55	2.77	3.00	3.24	3.47	3.97	4.50	5.05	5.60	6.13
1.2	1.45	1.56	1.69	1.84	2.00	2.18	2.38	2.60	2.82	3.04	3.27	3.76	4.27	4.81	5.37	5.91
1.4	1.33	1.43	1.56	1.70	1.86	2.04	2.23	2.44	2.65	2.86	3.08	3.55	4.06	4.59	5.13	5.68
1.6	1.22	1.32	1.45	1.58	1.74	1.91	2.09	2.29	2.49	2.69	2.91	3.37	3.86	4.37	4.91	5.46
1.8	1.13	1.23	1.35	1.48	1.63	1.79	1.97	2.16	2.34	2.54	2.75	3.19	3.67	4.17	4.70	5.24
2.0	1.05	1.14	1.26	1.38	1.52	1.68	1.85	2.03	2.21	2.40	2.60	3.03	3.50	3.99	4.50	5.03
2.2	0.975	1.07	1.18	1.30	1.44	1.59	1.75	1.92	2.09	2.27	2.47	2.88	3.33	3.81	4.31	4.83
2.4	0.912	1.00	1.11	1.22	1.35	1.50	1.66	1.82	1.98	2.16	2.34	2.74	3.18	3.64	4.13	4.64
2.6	0.856	0.943	1.04	1.15	1.28	1.42	1.57	1.72	1.88	2.05	2.23	2.62	3.04	3.49	3.96	4.46
2.8	0.806	0.890	0.986	1.09	1.21	1.35	1.49	1.64	1.79	1.95	2.12	2.50	2.91	3.35	3.81	4.29
3.0	0.762	0.842	0.934	1.04	1.15	1.28	1.42	1.56	1.70	1.86	2.03	2.39	2.78	3.21	3.66	4.13
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

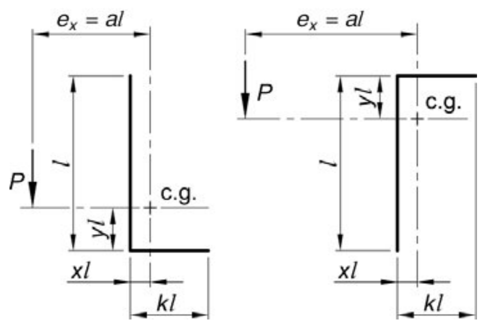
Table 8-11
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 0°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.



a	*k* = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	1.86	2.04	2.23	2.41	2.69	2.97	3.25	3.53	3.80	4.08	4.36	4.92	5.47	6.03	6.59	7.15
0.10	1.86	2.06	2.32	2.57	2.83	3.08	3.32	3.55	3.77	3.98	4.19	4.60	5.02	5.45	5.89	6.35
0.15	1.83	2.04	2.27	2.51	2.74	2.97	3.18	3.39	3.58	3.78	3.97	4.37	4.79	5.22	5.66	6.11
0.20	1.76	1.96	2.17	2.38	2.59	2.78	2.98	3.17	3.36	3.56	3.76	4.16	4.57	5.00	5.44	5.89
0.25	1.66	1.85	2.03	2.22	2.40	2.58	2.76	2.95	3.14	3.34	3.55	3.95	4.36	4.78	5.22	5.67
0.30	1.55	1.72	1.89	2.06	2.22	2.39	2.56	2.74	2.94	3.14	3.35	3.76	4.16	4.58	5.02	5.46
0.40	1.33	1.48	1.63	1.76	1.90	2.05	2.22	2.40	2.59	2.78	2.99	3.40	3.80	4.21	4.64	5.08
0.50	1.15	1.28	1.40	1.52	1.65	1.79	1.94	2.11	2.29	2.48	2.68	3.08	3.48	3.88	4.30	4.73
0.60	0.999	1.11	1.22	1.33	1.45	1.58	1.72	1.88	2.05	2.23	2.41	2.81	3.20	3.59	3.99	4.41
0.70	0.879	0.979	1.08	1.18	1.29	1.41	1.54	1.69	1.85	2.01	2.19	2.56	2.95	3.33	3.72	4.12
0.80	0.783	0.871	0.960	1.06	1.16	1.27	1.39	1.53	1.67	1.83	2.00	2.35	2.73	3.10	3.48	3.87
0.90	0.704	0.783	0.865	0.954	1.05	1.15	1.27	1.39	1.53	1.68	1.84	2.17	2.53	2.89	3.26	3.63
1.0	0.639	0.711	0.786	0.869	0.959	1.06	1.16	1.28	1.41	1.55	1.69	2.01	2.36	2.71	3.06	3.42
1.2	0.538	0.599	0.664	0.735	0.814	0.900	0.993	1.09	1.21	1.33	1.46	1.75	2.07	2.40	2.72	3.06
1.4	0.464	0.517	0.574	0.636	0.706	0.782	0.865	0.956	1.06	1.17	1.28	1.54	1.83	2.14	2.44	2.76
1.6	0.408	0.454	0.505	0.560	0.622	0.691	0.766	0.847	0.937	1.04	1.14	1.38	1.64	1.92	2.21	2.51
1.8	0.363	0.405	0.450	0.500	0.556	0.618	0.686	0.760	0.841	0.931	1.03	1.24	1.48	1.75	2.02	2.29
2.0	0.328	0.365	0.406	0.451	0.502	0.559	0.621	0.689	0.763	0.845	0.935	1.13	1.35	1.60	1.85	2.11
2.2	0.298	0.333	0.370	0.411	0.458	0.510	0.567	0.630	0.698	0.773	0.856	1.04	1.24	1.47	1.71	1.95
2.4	0.274	0.305	0.340	0.378	0.421	0.469	0.522	0.580	0.643	0.713	0.789	0.959	1.15	1.36	1.58	1.82
2.6	0.253	0.282	0.314	0.349	0.389	0.434	0.483	0.537	0.596	0.661	0.731	0.890	1.07	1.26	1.47	1.69
2.8	0.235	0.262	0.292	0.324	0.362	0.403	0.450	0.500	0.555	0.615	0.682	0.830	0.997	1.18	1.37	1.58
3.0	0.219	0.245	0.272	0.303	0.338	0.377	0.420	0.468	0.519	0.576	0.638	0.777	0.934	1.10	1.28	1.48
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

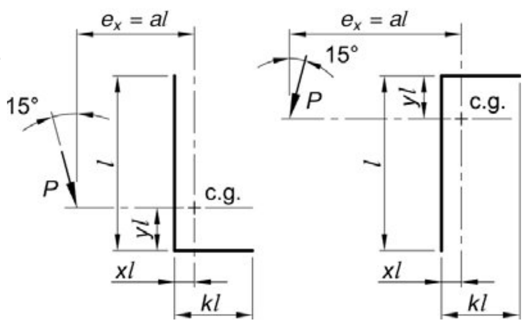
Table 8-11 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 15°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.



a	*k* = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	1.98	2.20	2.47	2.74	3.01	3.29	3.56	3.83	4.10	4.38	4.65	5.19	5.74	6.28	6.83	7.37
0.10	1.90	2.09	2.32	2.55	2.79	3.02	3.26	3.49	3.71	3.94	4.16	4.60	5.04	5.49	5.95	6.42
0.15	1.84	2.05	2.26	2.48	2.70	2.92	3.13	3.35	3.56	3.77	3.98	4.40	4.83	5.27	5.72	6.18
0.20	1.76	1.96	2.17	2.38	2.58	2.78	2.99	3.19	3.38	3.58	3.78	4.19	4.61	5.05	5.49	5.95
0.25	1.65	1.85	2.05	2.25	2.44	2.63	2.82	3.01	3.20	3.39	3.58	3.99	4.41	4.84	5.28	5.74
0.30	1.55	1.74	1.92	2.10	2.28	2.46	2.64	2.82	3.01	3.21	3.40	3.81	4.22	4.65	5.09	5.54
0.40	1.34	1.51	1.67	1.82	1.97	2.12	2.29	2.48	2.67	2.87	3.08	3.47	3.88	4.29	4.73	5.17
0.50	1.16	1.31	1.44	1.58	1.71	1.86	2.02	2.19	2.37	2.56	2.77	3.17	3.57	3.97	4.39	4.83
0.60	1.01	1.14	1.26	1.38	1.51	1.65	1.79	1.95	2.12	2.31	2.50	2.91	3.29	3.69	4.09	4.51
0.70	0.895	1.01	1.12	1.23	1.34	1.47	1.61	1.75	1.91	2.09	2.27	2.66	3.04	3.43	3.82	4.23
0.80	0.799	0.897	0.995	1.10	1.21	1.32	1.45	1.59	1.74	1.90	2.07	2.44	2.83	3.19	3.58	3.97
0.90	0.720	0.809	0.897	0.991	1.09	1.20	1.32	1.45	1.59	1.74	1.90	2.26	2.63	2.99	3.36	3.74
1.0	0.654	0.735	0.816	0.902	0.996	1.10	1.21	1.33	1.46	1.60	1.76	2.09	2.45	2.80	3.16	3.53
1.2	0.552	0.621	0.689	0.763	0.845	0.936	1.03	1.14	1.25	1.38	1.52	1.82	2.15	2.48	2.81	3.16
1.4	0.477	0.536	0.595	0.660	0.733	0.813	0.900	0.994	1.10	1.21	1.33	1.60	1.90	2.22	2.53	2.85
1.6	0.420	0.471	0.523	0.581	0.646	0.718	0.796	0.881	0.974	1.08	1.19	1.43	1.70	2.00	2.29	2.59
1.8	0.374	0.420	0.467	0.519	0.577	0.642	0.713	0.790	0.874	0.967	1.07	1.29	1.54	1.81	2.09	2.37
2.0	0.338	0.379	0.421	0.468	0.521	0.580	0.645	0.716	0.793	0.877	0.969	1.18	1.41	1.66	1.92	2.19
2.2	0.308	0.345	0.384	0.426	0.475	0.529	0.589	0.654	0.725	0.803	0.888	1.08	1.29	1.52	1.77	2.02
2.4	0.282	0.317	0.352	0.391	0.436	0.486	0.542	0.602	0.668	0.739	0.818	0.994	1.19	1.41	1.64	1.88
2.6	0.261	0.292	0.325	0.362	0.403	0.450	0.501	0.557	0.619	0.685	0.758	0.923	1.11	1.31	1.52	1.75
2.8	0.242	0.272	0.302	0.336	0.375	0.418	0.466	0.519	0.576	0.638	0.707	0.860	1.03	1.22	1.42	1.64
3.0	0.226	0.254	0.282	0.314	0.350	0.391	0.436	0.485	0.539	0.598	0.662	0.806	0.967	1.14	1.33	1.53
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

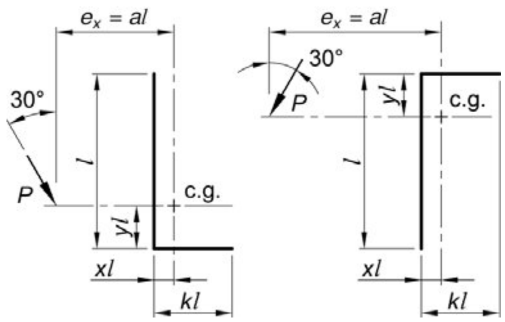
Table 8-11 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 30°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $\quad D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $\quad l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $\quad D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $\quad l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.



a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.18	2.44	2.70	2.96	3.21	3.47	3.73	3.98	4.24	4.50	4.76	5.27	5.78	6.30	6.81	7.33
0.10	2.02	2.24	2.47	2.70	2.93	3.17	3.40	3.63	3.87	4.10	4.34	4.82	5.31	5.80	6.30	6.81
0.15	1.92	2.12	2.33	2.54	2.76	2.98	3.20	3.42	3.64	3.86	4.09	4.55	5.02	5.51	6.01	6.53
0.20	1.82	2.01	2.21	2.41	2.62	2.83	3.03	3.24	3.46	3.67	3.89	4.33	4.79	5.26	5.74	6.24
0.25	1.71	1.90	2.08	2.28	2.47	2.67	2.88	3.08	3.28	3.49	3.70	4.13	4.57	5.03	5.50	5.99
0.30	1.61	1.78	1.96	2.14	2.32	2.52	2.72	2.91	3.11	3.31	3.51	3.93	4.37	4.82	5.29	5.76
0.40	1.41	1.56	1.72	1.87	2.05	2.24	2.43	2.63	2.82	3.01	3.21	3.62	4.04	4.48	4.93	5.39
0.50	1.23	1.37	1.50	1.66	1.82	2.00	2.19	2.37	2.56	2.75	2.95	3.34	3.75	4.18	4.62	5.06
0.60	1.08	1.21	1.33	1.48	1.63	1.80	1.96	2.13	2.31	2.51	2.71	3.10	3.50	3.91	4.33	4.77
0.70	0.964	1.07	1.19	1.33	1.47	1.62	1.77	1.93	2.10	2.28	2.48	2.87	3.26	3.66	4.08	4.50
0.80	0.865	0.965	1.07	1.20	1.33	1.46	1.60	1.75	1.92	2.09	2.27	2.67	3.05	3.44	3.84	4.25
0.90	0.783	0.874	0.976	1.09	1.21	1.33	1.46	1.60	1.76	1.92	2.10	2.47	2.85	3.23	3.62	4.03
1.0	0.714	0.798	0.893	0.997	1.10	1.22	1.34	1.48	1.62	1.77	1.94	2.30	2.68	3.04	3.42	3.81
1.2	0.606	0.678	0.761	0.847	0.938	1.04	1.15	1.27	1.39	1.53	1.68	2.01	2.37	2.71	3.07	3.44
1.4	0.525	0.589	0.661	0.734	0.815	0.904	1.00	1.11	1.22	1.35	1.48	1.78	2.10	2.44	2.77	3.12
1.6	0.463	0.520	0.582	0.647	0.719	0.799	0.887	0.982	1.09	1.20	1.32	1.59	1.89	2.21	2.52	2.85
1.8	0.414	0.465	0.520	0.577	0.642	0.715	0.795	0.882	0.975	1.08	1.19	1.43	1.71	2.01	2.31	2.61
2.0	0.374	0.421	0.469	0.521	0.580	0.647	0.720	0.799	0.885	0.978	1.08	1.31	1.56	1.84	2.12	2.41
2.2	0.341	0.384	0.427	0.475	0.529	0.590	0.657	0.730	0.809	0.895	0.989	1.20	1.44	1.69	1.96	2.24
2.4	0.313	0.353	0.392	0.436	0.486	0.542	0.604	0.672	0.745	0.825	0.912	1.11	1.32	1.56	1.81	2.08
2.6	0.289	0.326	0.363	0.403	0.450	0.502	0.559	0.622	0.690	0.765	0.845	1.03	1.23	1.45	1.68	1.94
2.8	0.269	0.303	0.337	0.375	0.418	0.467	0.520	0.579	0.643	0.712	0.788	0.958	1.15	1.35	1.57	1.81
3.0	0.251	0.283	0.315	0.350	0.391	0.436	0.487	0.542	0.602	0.667	0.738	0.898	1.07	1.26	1.47	1.70
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

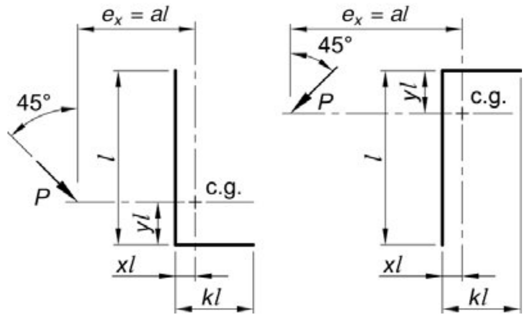
Table 8-11 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 45°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.



a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.41	2.57	2.80	3.04	3.27	3.51	3.74	3.97	4.21	4.44	4.67	5.14	5.61	6.08	6.54	7.01
0.10	2.24	2.44	2.65	2.87	3.09	3.32	3.56	3.79	4.03	4.26	4.50	4.99	5.47	5.96	6.45	6.94
0.15	2.09	2.28	2.48	2.69	2.91	3.14	3.38	3.62	3.85	4.09	4.33	4.83	5.32	5.82	6.31	6.81
0.20	1.96	2.14	2.32	2.51	2.72	2.94	3.17	3.42	3.66	3.90	4.15	4.65	5.15	5.65	6.15	6.65
0.25	1.85	2.02	2.19	2.37	2.56	2.76	2.98	3.21	3.45	3.70	3.95	4.45	4.95	5.46	5.97	6.47
0.30	1.74	1.90	2.06	2.23	2.41	2.61	2.82	3.04	3.26	3.50	3.74	4.24	4.75	5.26	5.77	6.28
0.40	1.55	1.69	1.84	1.99	2.17	2.36	2.56	2.77	2.99	3.22	3.44	3.89	4.36	4.86	5.37	5.88
0.50	1.38	1.51	1.64	1.80	1.97	2.15	2.35	2.56	2.77	2.98	3.20	3.63	4.07	4.54	5.02	5.52
0.60	1.23	1.35	1.48	1.63	1.79	1.97	2.16	2.36	2.57	2.78	2.99	3.41	3.84	4.28	4.74	5.21
0.70	1.11	1.22	1.34	1.48	1.64	1.81	1.99	2.19	2.38	2.59	2.80	3.20	3.62	4.05	4.50	4.95
0.80	1.00	1.11	1.22	1.36	1.51	1.67	1.84	2.03	2.22	2.42	2.62	3.01	3.42	3.84	4.28	4.72
0.90	0.915	1.01	1.12	1.25	1.39	1.54	1.71	1.88	2.07	2.25	2.44	2.84	3.24	3.65	4.07	4.51
1.0	0.839	0.929	1.03	1.15	1.29	1.43	1.59	1.75	1.92	2.10	2.28	2.68	3.07	3.47	3.88	4.31
1.2	0.719	0.799	0.891	0.997	1.12	1.25	1.38	1.52	1.67	1.83	2.00	2.37	2.76	3.14	3.53	3.94
1.4	0.627	0.699	0.782	0.877	0.981	1.09	1.21	1.34	1.47	1.62	1.78	2.11	2.49	2.86	3.23	3.62
1.6	0.555	0.620	0.695	0.781	0.870	0.967	1.07	1.19	1.31	1.45	1.59	1.90	2.24	2.61	2.97	3.34
1.8	0.498	0.557	0.625	0.701	0.780	0.868	0.965	1.07	1.18	1.31	1.44	1.72	2.04	2.38	2.73	3.09
2.0	0.451	0.505	0.568	0.634	0.706	0.786	0.875	0.972	1.08	1.19	1.31	1.57	1.86	2.18	2.53	2.86
2.2	0.412	0.462	0.520	0.579	0.644	0.718	0.800	0.889	0.986	1.09	1.20	1.44	1.72	2.01	2.33	2.67
2.4	0.379	0.426	0.479	0.532	0.593	0.661	0.737	0.819	0.909	1.01	1.11	1.33	1.59	1.87	2.17	2.49
2.6	0.351	0.394	0.443	0.492	0.549	0.612	0.682	0.760	0.843	0.933	1.03	1.24	1.48	1.74	2.02	2.32
2.8	0.327	0.367	0.412	0.458	0.510	0.570	0.635	0.707	0.786	0.870	0.961	1.16	1.38	1.63	1.89	2.18
3.0	0.306	0.344	0.385	0.428	0.477	0.533	0.594	0.662	0.735	0.814	0.900	1.09	1.29	1.53	1.78	2.05
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

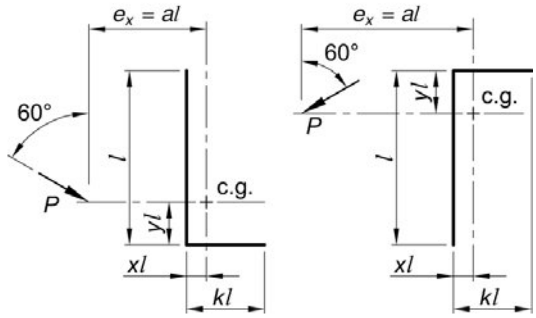
Table 8-11 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 60°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $\quad D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $\quad l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $\quad D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $\quad l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.



a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.60	2.79	3.01	3.23	3.44	3.66	3.88	4.10	4.32	4.54	4.76	5.19	5.63	6.07	6.50	6.94
0.10	2.43	2.59	2.76	2.94	3.14	3.36	3.59	3.83	4.07	4.30	4.54	5.00	5.46	5.92	6.37	6.82
0.15	2.31	2.45	2.61	2.79	2.98	3.20	3.42	3.67	3.91	4.16	4.41	4.89	5.36	5.82	6.28	6.74
0.20	2.18	2.32	2.48	2.64	2.83	3.04	3.27	3.51	3.75	4.00	4.25	4.76	5.24	5.72	6.19	6.65
0.25	2.07	2.21	2.35	2.51	2.70	2.91	3.14	3.38	3.62	3.87	4.11	4.61	5.11	5.60	6.08	6.55
0.30	1.97	2.10	2.24	2.40	2.59	2.79	3.01	3.25	3.50	3.75	3.99	4.48	4.97	5.47	5.96	6.44
0.40	1.79	1.92	2.05	2.21	2.39	2.59	2.81	3.03	3.27	3.52	3.77	4.26	4.75	5.23	5.71	6.20
0.50	1.63	1.75	1.88	2.04	2.22	2.42	2.63	2.85	3.07	3.31	3.55	4.06	4.55	5.04	5.52	5.99
0.60	1.49	1.61	1.74	1.89	2.07	2.26	2.47	2.68	2.90	3.13	3.36	3.85	4.36	4.85	5.34	5.81
0.70	1.37	1.48	1.61	1.76	1.93	2.12	2.32	2.53	2.75	2.97	3.20	3.67	4.16	4.67	5.16	5.64
0.80	1.26	1.37	1.49	1.64	1.81	1.99	2.18	2.39	2.60	2.82	3.04	3.51	3.98	4.48	4.98	5.47
0.90	1.17	1.27	1.39	1.53	1.69	1.87	2.06	2.26	2.46	2.68	2.90	3.35	3.82	4.30	4.79	5.29
1.0	1.08	1.18	1.30	1.44	1.59	1.76	1.94	2.14	2.34	2.55	2.76	3.21	3.67	4.13	4.61	5.11
1.2	0.946	1.04	1.15	1.27	1.41	1.57	1.74	1.92	2.11	2.30	2.49	2.92	3.37	3.83	4.29	4.75
1.4	0.837	0.921	1.02	1.14	1.27	1.41	1.57	1.74	1.90	2.07	2.26	2.66	3.09	3.55	4.00	4.45
1.6	0.748	0.827	0.920	1.03	1.15	1.28	1.43	1.58	1.73	1.89	2.06	2.43	2.84	3.28	3.74	4.17
1.8	0.676	0.749	0.836	0.935	1.05	1.17	1.31	1.44	1.58	1.72	1.88	2.23	2.62	3.04	3.49	3.92
2.0	0.616	0.684	0.765	0.856	0.961	1.08	1.20	1.32	1.45	1.59	1.73	2.06	2.43	2.83	3.25	3.69
2.2	0.565	0.629	0.704	0.790	0.887	0.991	1.10	1.22	1.34	1.47	1.61	1.91	2.26	2.64	3.04	3.46
2.4	0.522	0.582	0.652	0.732	0.822	0.916	1.02	1.13	1.24	1.36	1.49	1.78	2.11	2.47	2.85	3.26
2.6	0.485	0.541	0.607	0.682	0.763	0.851	0.948	1.05	1.16	1.27	1.39	1.67	1.98	2.32	2.68	3.07
2.8	0.453	0.506	0.568	0.638	0.712	0.794	0.885	0.984	1.08	1.19	1.31	1.57	1.86	2.18	2.53	2.90
3.0	0.424	0.475	0.533	0.599	0.667	0.744	0.830	0.923	1.02	1.12	1.23	1.47	1.75	2.06	2.39	2.74
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

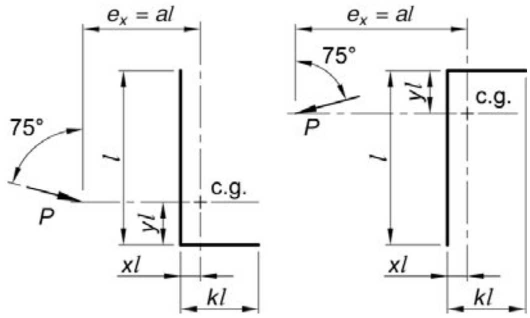
Table 8-11 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 75°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.



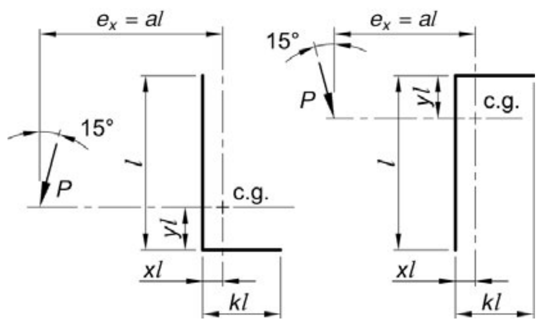
a	*k* = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.74	2.92	3.11	3.30	3.49	3.69	3.88	4.07	4.26	4.46	4.65	5.03	5.42	5.80	6.19	6.57
0.10	2.59	2.67	2.78	2.93	3.12	3.32	3.53	3.75	3.96	4.17	4.38	4.78	5.22	5.64	6.06	6.46
0.15	2.50	2.59	2.70	2.86	3.05	3.26	3.48	3.70	3.92	4.13	4.34	4.74	5.15	5.58	6.01	6.42
0.20	2.43	2.52	2.63	2.79	2.98	3.19	3.42	3.64	3.87	4.09	4.30	4.71	5.11	5.52	5.95	6.37
0.25	2.35	2.44	2.56	2.73	2.92	3.13	3.36	3.59	3.82	4.04	4.26	4.68	5.08	5.48	5.89	6.31
0.30	2.28	2.38	2.50	2.66	2.85	3.07	3.30	3.53	3.77	4.00	4.22	4.65	5.06	5.45	5.85	6.26
0.40	2.16	2.25	2.38	2.55	2.74	2.95	3.17	3.41	3.66	3.90	4.13	4.58	5.00	5.41	5.80	6.19
0.50	2.05	2.14	2.27	2.44	2.63	2.83	3.06	3.30	3.55	3.79	4.04	4.50	4.94	5.35	5.76	6.15
0.60	1.94	2.04	2.17	2.34	2.52	2.73	2.95	3.19	3.43	3.69	3.94	4.42	4.87	5.30	5.71	6.11
0.70	1.85	1.94	2.08	2.24	2.42	2.63	2.85	3.08	3.32	3.58	3.83	4.33	4.80	5.24	5.66	6.07
0.80	1.75	1.85	1.99	2.15	2.33	2.53	2.75	2.98	3.22	3.47	3.73	4.23	4.72	5.17	5.60	6.02
0.90	1.67	1.77	1.90	2.06	2.24	2.44	2.66	2.89	3.12	3.37	3.62	4.14	4.63	5.10	5.54	5.97
1.0	1.59	1.69	1.82	1.98	2.16	2.36	2.57	2.80	3.03	3.27	3.52	4.04	4.54	5.02	5.47	5.91
1.2	1.45	1.55	1.68	1.83	2.00	2.20	2.40	2.62	2.85	3.09	3.33	3.83	4.35	4.85	5.33	5.78
1.4	1.33	1.43	1.55	1.70	1.86	2.05	2.25	2.47	2.69	2.92	3.15	3.64	4.15	4.67	5.16	5.64
1.6	1.22	1.32	1.44	1.58	1.74	1.92	2.11	2.32	2.54	2.76	2.98	3.45	3.96	4.48	4.99	5.48
1.8	1.13	1.22	1.34	1.47	1.63	1.80	1.99	2.19	2.40	2.61	2.82	3.27	3.76	4.28	4.81	5.31
2.0	1.05	1.14	1.25	1.38	1.53	1.69	1.87	2.07	2.27	2.46	2.67	3.11	3.58	4.09	4.61	5.14
2.2	0.975	1.06	1.17	1.30	1.44	1.60	1.77	1.95	2.14	2.33	2.53	2.95	3.41	3.90	4.42	4.95
2.4	0.912	0.998	1.10	1.22	1.36	1.51	1.68	1.85	2.03	2.21	2.40	2.81	3.25	3.73	4.23	4.75
2.6	0.856	0.940	1.04	1.15	1.29	1.43	1.59	1.76	1.92	2.09	2.28	2.67	3.11	3.57	4.06	4.57
2.8	0.806	0.887	0.983	1.09	1.22	1.36	1.51	1.67	1.83	1.99	2.17	2.55	2.97	3.42	3.90	4.40
3.0	0.762	0.839	0.932	1.04	1.16	1.29	1.44	1.59	1.74	1.90	2.07	2.44	2.84	3.28	3.75	4.24
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

Table 8-11a
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 15°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $\quad D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $\quad l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $\quad D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $\quad l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.

a	*k*: 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	1.98	2.20	2.47	2.74	3.01	3.29	3.56	3.83	4.10	4.38	4.65	5.19	5.74	6.28	6.83	7.37
0.10	1.90	2.15	2.44	2.71	2.97	3.20	3.42	3.62	3.82	4.02	4.22	4.64	5.07	5.52	5.99	6.47
0.15	1.84	2.09	2.34	2.58	2.80	3.00	3.19	3.38	3.57	3.77	3.98	4.40	4.83	5.28	5.75	6.22
0.20	1.76	1.98	2.20	2.41	2.61	2.79	2.97	3.15	3.35	3.55	3.76	4.18	4.61	5.05	5.51	5.99
0.25	1.65	1.85	2.05	2.24	2.42	2.59	2.76	2.94	3.14	3.34	3.55	3.97	4.40	4.84	5.30	5.77
0.30	1.55	1.73	1.90	2.07	2.24	2.40	2.57	2.75	2.94	3.14	3.35	3.78	4.20	4.64	5.09	5.56
0.40	1.34	1.49	1.64	1.77	1.92	2.07	2.24	2.42	2.60	2.80	3.00	3.42	3.84	4.27	4.71	5.17
0.50	1.16	1.29	1.41	1.54	1.67	1.81	1.97	2.14	2.32	2.50	2.70	3.11	3.52	3.94	4.37	4.81
0.60	1.01	1.13	1.24	1.35	1.47	1.60	1.75	1.91	2.08	2.26	2.44	2.83	3.25	3.65	4.06	4.50
0.70	0.895	0.998	1.10	1.20	1.31	1.43	1.57	1.72	1.88	2.05	2.22	2.60	3.00	3.39	3.79	4.21
0.80	0.799	0.889	0.980	1.08	1.18	1.29	1.42	1.56	1.71	1.87	2.03	2.39	2.77	3.16	3.55	3.95
0.90	0.720	0.801	0.885	0.975	1.07	1.18	1.29	1.42	1.56	1.71	1.87	2.21	2.58	2.95	3.33	3.71
1.0	0.654	0.728	0.806	0.889	0.980	1.08	1.19	1.31	1.44	1.58	1.73	2.05	2.40	2.77	3.13	3.50
1.2	0.552	0.615	0.682	0.755	0.835	0.921	1.02	1.12	1.24	1.36	1.50	1.79	2.11	2.45	2.79	3.14
1.4	0.477	0.532	0.590	0.654	0.725	0.803	0.887	0.980	1.08	1.20	1.32	1.58	1.87	2.19	2.51	2.83
1.6	0.420	0.468	0.520	0.577	0.640	0.710	0.786	0.870	0.963	1.07	1.17	1.42	1.68	1.97	2.27	2.58
1.8	0.374	0.417	0.464	0.515	0.573	0.636	0.706	0.781	0.866	0.958	1.06	1.28	1.52	1.79	2.07	2.36
2.0	0.338	0.377	0.419	0.465	0.518	0.576	0.639	0.709	0.786	0.870	0.962	1.16	1.39	1.64	1.91	2.17
2.2	0.308	0.343	0.382	0.424	0.472	0.526	0.584	0.648	0.719	0.797	0.882	1.07	1.28	1.51	1.76	2.01
2.4	0.282	0.315	0.350	0.390	0.434	0.483	0.538	0.597	0.662	0.735	0.813	0.987	1.18	1.40	1.63	1.87
2.6	0.261	0.291	0.324	0.360	0.401	0.447	0.498	0.553	0.614	0.681	0.754	0.917	1.10	1.30	1.51	1.74
2.8	0.242	0.271	0.301	0.335	0.373	0.416	0.464	0.516	0.572	0.635	0.703	0.856	1.03	1.21	1.41	1.63
3.0	0.226	0.253	0.281	0.313	0.349	0.389	0.434	0.483	0.536	0.594	0.659	0.802	0.963	1.14	1.32	1.53
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

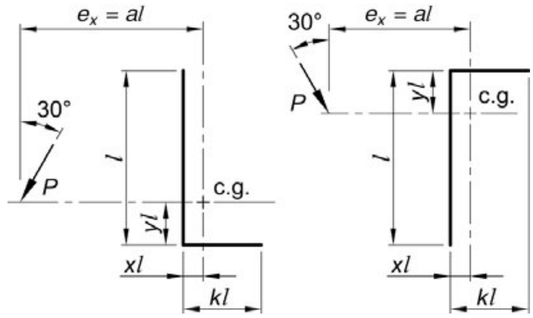
Table 8-11a (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 30°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.



a	*k* = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.18	2.44	2.70	2.96	3.21	3.47	3.73	3.98	4.24	4.50	4.76	5.27	5.78	6.30	6.81	7.33
0.10	2.02	2.34	2.61	2.86	3.08	3.27	3.46	3.66	3.86	4.08	4.31	4.78	5.27	5.76	6.26	6.77
0.15	1.92	2.20	2.46	2.68	2.88	3.05	3.23	3.42	3.63	3.85	4.07	4.54	5.03	5.52	6.02	6.52
0.20	1.82	2.07	2.30	2.50	2.68	2.85	3.02	3.21	3.41	3.63	3.86	4.33	4.81	5.30	5.79	6.29
0.25	1.71	1.93	2.14	2.33	2.50	2.66	2.83	3.02	3.22	3.43	3.65	4.12	4.61	5.09	5.58	6.07
0.30	1.61	1.81	1.99	2.16	2.32	2.49	2.66	2.84	3.04	3.25	3.47	3.93	4.41	4.89	5.38	5.87
0.40	1.41	1.57	1.72	1.87	2.02	2.18	2.35	2.53	2.72	2.92	3.13	3.58	4.05	4.53	5.01	5.49
0.50	1.23	1.37	1.50	1.63	1.78	1.93	2.09	2.26	2.45	2.64	2.84	3.27	3.73	4.20	4.67	5.14
0.60	1.08	1.21	1.33	1.45	1.57	1.72	1.88	2.04	2.21	2.40	2.59	3.00	3.45	3.91	4.36	4.82
0.70	0.964	1.08	1.18	1.29	1.41	1.54	1.69	1.85	2.01	2.19	2.37	2.77	3.19	3.64	4.08	4.53
0.80	0.865	0.965	1.06	1.17	1.28	1.40	1.54	1.68	1.84	2.01	2.18	2.56	2.97	3.40	3.83	4.27
0.90	0.783	0.873	0.964	1.06	1.16	1.28	1.40	1.54	1.69	1.85	2.02	2.38	2.77	3.19	3.60	4.03
1.0	0.714	0.796	0.881	0.971	1.07	1.17	1.29	1.42	1.56	1.71	1.87	2.22	2.59	2.99	3.39	3.81
1.2	0.606	0.676	0.749	0.828	0.914	1.01	1.11	1.23	1.35	1.49	1.63	1.95	2.29	2.66	3.04	3.42
1.4	0.525	0.586	0.650	0.720	0.797	0.881	0.974	1.08	1.19	1.31	1.44	1.73	2.04	2.38	2.74	3.10
1.6	0.463	0.516	0.574	0.636	0.706	0.782	0.865	0.958	1.06	1.17	1.29	1.55	1.84	2.15	2.49	2.82
1.8	0.414	0.462	0.513	0.570	0.633	0.702	0.778	0.862	0.955	1.06	1.17	1.41	1.67	1.96	2.27	2.59
2.0	0.374	0.417	0.464	0.515	0.573	0.637	0.706	0.783	0.868	0.961	1.06	1.28	1.53	1.80	2.09	2.39
2.2	0.341	0.380	0.423	0.470	0.523	0.582	0.646	0.717	0.795	0.881	0.974	1.18	1.41	1.66	1.93	2.21
2.4	0.313	0.349	0.389	0.432	0.481	0.536	0.595	0.661	0.733	0.813	0.900	1.09	1.30	1.54	1.79	2.06
2.6	0.289	0.323	0.360	0.400	0.445	0.496	0.552	0.613	0.680	0.755	0.836	1.01	1.21	1.44	1.67	1.92
2.8	0.269	0.300	0.334	0.372	0.415	0.462	0.514	0.571	0.634	0.704	0.780	0.947	1.14	1.34	1.56	1.80
3.0	0.251	0.281	0.313	0.348	0.388	0.432	0.481	0.535	0.594	0.659	0.731	0.889	1.07	1.26	1.47	1.69
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

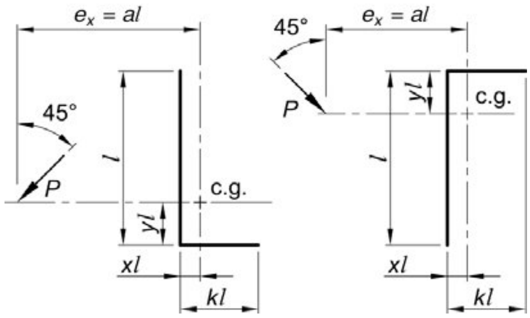
Table 8-11a (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 45°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD			ASD		
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$	$D_{min} = \dfrac{P_u}{\phi C C_1 l}$	$l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$	$D_{min} = \dfrac{\Omega P_a}{C C_1 l}$	$l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.



a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.41	2.57	2.80	3.04	3.27	3.51	3.74	3.97	4.21	4.44	4.67	5.14	5.61	6.08	6.54	7.01
0.10	2.24	2.52	2.76	2.97	3.17	3.37	3.58	3.80	4.02	4.25	4.49	4.98	5.47	5.96	6.45	6.94
0.15	2.09	2.38	2.61	2.80	2.98	3.17	3.37	3.58	3.81	4.04	4.28	4.77	5.27	5.77	6.27	6.77
0.20	1.96	2.23	2.45	2.63	2.80	2.98	3.18	3.39	3.61	3.84	4.08	4.57	5.06	5.57	6.08	6.59
0.25	1.85	2.10	2.30	2.47	2.63	2.81	3.00	3.21	3.44	3.67	3.90	4.39	4.88	5.38	5.88	6.39
0.30	1.74	1.97	2.16	2.33	2.49	2.65	2.84	3.05	3.27	3.50	3.73	4.22	4.71	5.21	5.71	6.22
0.40	1.55	1.73	1.90	2.06	2.22	2.38	2.56	2.76	2.97	3.19	3.43	3.91	4.40	4.90	5.41	5.91
0.50	1.38	1.54	1.68	1.83	1.99	2.15	2.32	2.51	2.71	2.92	3.15	3.62	4.12	4.62	5.12	5.62
0.60	1.23	1.38	1.51	1.64	1.79	1.95	2.11	2.29	2.48	2.69	2.90	3.36	3.85	4.34	4.84	5.35
0.70	1.11	1.24	1.36	1.48	1.62	1.77	1.93	2.10	2.28	2.48	2.68	3.13	3.60	4.09	4.59	5.09
0.80	1.00	1.12	1.23	1.35	1.48	1.62	1.77	1.94	2.11	2.29	2.49	2.92	3.38	3.85	4.34	4.84
0.90	0.915	1.02	1.13	1.24	1.36	1.49	1.64	1.79	1.96	2.13	2.32	2.73	3.17	3.64	4.12	4.60
1.0	0.839	0.937	1.04	1.14	1.25	1.38	1.51	1.66	1.82	1.99	2.17	2.56	2.99	3.44	3.90	4.38
1.2	0.719	0.802	0.889	0.982	1.08	1.19	1.31	1.45	1.59	1.75	1.91	2.27	2.66	3.09	3.53	3.98
1.4	0.627	0.700	0.777	0.860	0.950	1.05	1.16	1.28	1.41	1.55	1.70	2.03	2.40	2.79	3.20	3.64
1.6	0.555	0.620	0.689	0.764	0.846	0.935	1.03	1.14	1.27	1.40	1.53	1.84	2.17	2.54	2.93	3.34
1.8	0.498	0.556	0.618	0.686	0.761	0.843	0.933	1.03	1.14	1.26	1.39	1.67	1.98	2.32	2.69	3.08
2.0	0.451	0.504	0.560	0.622	0.691	0.766	0.849	0.942	1.04	1.15	1.27	1.53	1.82	2.14	2.48	2.85
2.2	0.412	0.460	0.512	0.569	0.632	0.702	0.779	0.864	0.959	1.06	1.17	1.41	1.69	1.98	2.30	2.65
2.4	0.379	0.423	0.471	0.524	0.583	0.648	0.719	0.798	0.886	0.982	1.08	1.31	1.57	1.84	2.15	2.47
2.6	0.351	0.392	0.436	0.485	0.540	0.601	0.668	0.741	0.823	0.913	1.01	1.22	1.46	1.72	2.01	2.31
2.8	0.327	0.365	0.406	0.452	0.503	0.560	0.623	0.692	0.768	0.852	0.943	1.14	1.37	1.62	1.88	2.17
3.0	0.306	0.341	0.380	0.423	0.471	0.525	0.584	0.649	0.720	0.799	0.885	1.07	1.29	1.52	1.77	2.04
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

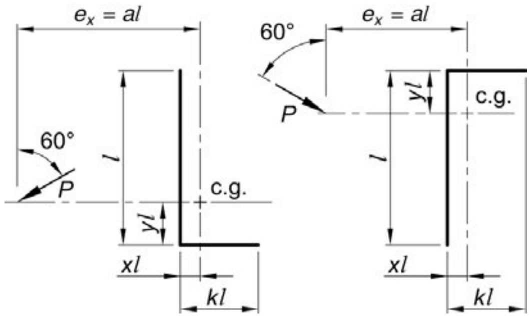
Table 8-11a (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 60°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.



a	*k* = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.60	2.79	3.01	3.23	3.44	3.66	3.88	4.10	4.32	4.54	4.76	5.19	5.63	6.07	6.50	6.94
0.10	2.43	2.68	2.91	3.12	3.34	3.56	3.79	4.01	4.24	4.46	4.69	5.14	5.58	6.02	6.47	6.91
0.15	2.31	2.56	2.77	2.97	3.18	3.40	3.62	3.85	4.08	4.32	4.55	5.01	5.47	5.93	6.38	6.83
0.20	2.18	2.44	2.63	2.82	3.02	3.24	3.46	3.69	3.92	4.15	4.39	4.87	5.34	5.81	6.27	6.73
0.25	2.07	2.32	2.50	2.68	2.88	3.09	3.31	3.54	3.77	4.01	4.24	4.71	5.19	5.67	6.15	6.61
0.30	1.97	2.21	2.39	2.56	2.75	2.96	3.18	3.41	3.64	3.88	4.11	4.58	5.05	5.53	6.01	6.49
0.40	1.79	2.00	2.19	2.35	2.53	2.72	2.94	3.16	3.40	3.64	3.88	4.35	4.83	5.29	5.76	6.23
0.50	1.63	1.82	1.99	2.16	2.33	2.51	2.72	2.94	3.17	3.41	3.65	4.14	4.62	5.10	5.57	6.03
0.60	1.49	1.67	1.82	1.99	2.15	2.33	2.52	2.74	2.97	3.20	3.44	3.94	4.43	4.91	5.39	5.86
0.70	1.37	1.53	1.68	1.83	2.00	2.17	2.35	2.55	2.77	3.01	3.24	3.74	4.23	4.73	5.21	5.69
0.80	1.26	1.41	1.55	1.70	1.85	2.02	2.20	2.39	2.60	2.83	3.06	3.55	4.04	4.54	5.03	5.52
0.90	1.17	1.30	1.44	1.57	1.73	1.89	2.06	2.24	2.44	2.66	2.89	3.37	3.86	4.36	4.86	5.35
1.0	1.08	1.21	1.34	1.47	1.61	1.77	1.93	2.11	2.30	2.51	2.73	3.20	3.69	4.18	4.68	5.18
1.2	0.946	1.06	1.17	1.29	1.42	1.56	1.71	1.88	2.06	2.25	2.45	2.89	3.36	3.85	4.35	4.84
1.4	0.837	0.935	1.04	1.15	1.26	1.39	1.53	1.69	1.85	2.03	2.22	2.63	3.08	3.55	4.04	4.53
1.6	0.748	0.837	0.929	1.03	1.13	1.25	1.38	1.53	1.68	1.85	2.02	2.41	2.83	3.28	3.75	4.23
1.8	0.676	0.756	0.840	0.931	1.03	1.14	1.26	1.39	1.54	1.69	1.85	2.21	2.61	3.04	3.49	3.96
2.0	0.616	0.689	0.766	0.850	0.941	1.04	1.15	1.28	1.41	1.56	1.71	2.05	2.42	2.83	3.26	3.71
2.2	0.565	0.632	0.703	0.781	0.866	0.960	1.06	1.18	1.30	1.44	1.58	1.90	2.26	2.64	3.05	3.49
2.4	0.522	0.584	0.650	0.722	0.802	0.889	0.986	1.09	1.21	1.34	1.47	1.77	2.11	2.48	2.87	3.28
2.6	0.485	0.542	0.604	0.671	0.746	0.828	0.918	1.02	1.13	1.25	1.38	1.66	1.98	2.33	2.70	3.10
2.8	0.453	0.506	0.563	0.626	0.697	0.774	0.859	0.954	1.06	1.17	1.29	1.56	1.86	2.19	2.55	2.93
3.0	0.424	0.474	0.528	0.587	0.653	0.727	0.807	0.896	0.995	1.10	1.22	1.47	1.76	2.07	2.41	2.77
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

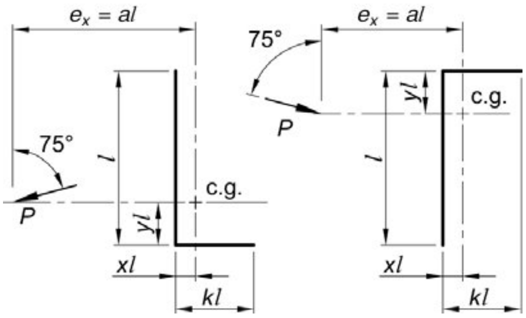
Table 8-11a (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups
Angle = 75°

Available strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

Note: Shaded values indicate the value is based on the greatest available strength permitted by AISC *Specification* Section J2.4.



a	k: 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.74	2.92	3.11	3.30	3.49	3.69	3.88	4.07	4.26	4.46	4.65	5.03	5.42	5.80	6.19	6.57
0.10	2.59	2.86	3.11	3.30	3.49	3.69	3.88	4.07	4.26	4.46	4.65	5.03	5.42	5.80	6.19	6.57
0.15	2.50	2.76	3.02	3.26	3.48	3.68	3.88	4.07	4.26	4.45	4.65	5.03	5.42	5.80	6.19	6.57
0.20	2.43	2.67	2.92	3.17	3.40	3.63	3.84	4.05	4.25	4.45	4.64	5.03	5.41	5.80	6.18	6.57
0.25	2.35	2.59	2.83	3.07	3.30	3.53	3.76	3.97	4.19	4.39	4.59	4.99	5.39	5.78	6.17	6.56
0.30	2.28	2.52	2.74	2.97	3.21	3.44	3.67	3.89	4.10	4.32	4.53	4.93	5.34	5.73	6.13	6.52
0.40	2.16	2.39	2.59	2.81	3.04	3.27	3.50	3.73	3.95	4.16	4.37	4.79	5.21	5.62	6.02	6.42
0.50	2.05	2.27	2.46	2.67	2.89	3.13	3.36	3.60	3.82	4.04	4.25	4.67	5.07	5.48	5.90	6.31
0.60	1.94	2.16	2.35	2.54	2.76	2.99	3.23	3.47	3.70	3.93	4.15	4.58	4.99	5.38	5.78	6.18
0.70	1.85	2.05	2.24	2.43	2.64	2.86	3.10	3.34	3.58	3.82	4.05	4.49	4.91	5.32	5.71	6.11
0.80	1.75	1.95	2.14	2.32	2.52	2.74	2.98	3.22	3.46	3.71	3.95	4.40	4.84	5.25	5.66	6.05
0.90	1.67	1.86	2.04	2.22	2.41	2.63	2.86	3.10	3.35	3.59	3.84	4.31	4.76	5.19	5.60	6.00
1.0	1.59	1.77	1.95	2.13	2.31	2.52	2.75	2.98	3.23	3.48	3.73	4.21	4.67	5.11	5.54	5.95
1.2	1.45	1.62	1.78	1.95	2.13	2.32	2.54	2.77	3.01	3.26	3.51	4.01	4.49	4.96	5.40	5.83
1.4	1.33	1.48	1.64	1.80	1.97	2.15	2.35	2.57	2.81	3.05	3.30	3.81	4.31	4.79	5.25	5.70
1.6	1.22	1.36	1.51	1.66	1.82	2.00	2.19	2.40	2.62	2.86	3.11	3.61	4.12	4.61	5.09	5.55
1.8	1.13	1.26	1.40	1.54	1.69	1.86	2.04	2.24	2.45	2.68	2.92	3.42	3.93	4.43	4.92	5.40
2.0	1.05	1.17	1.30	1.43	1.58	1.74	1.91	2.10	2.30	2.52	2.75	3.24	3.75	4.25	4.75	5.23
2.2	0.975	1.09	1.21	1.34	1.48	1.63	1.80	1.97	2.17	2.38	2.60	3.07	3.57	4.07	4.58	5.07
2.4	0.912	1.02	1.13	1.26	1.39	1.53	1.69	1.86	2.05	2.25	2.46	2.92	3.41	3.91	4.41	4.90
2.6	0.856	0.959	1.07	1.18	1.31	1.44	1.59	1.76	1.94	2.13	2.33	2.78	3.25	3.74	4.24	4.74
2.8	0.806	0.903	1.00	1.11	1.23	1.36	1.51	1.67	1.84	2.02	2.21	2.64	3.11	3.59	4.08	4.58
3.0	0.762	0.853	0.949	1.05	1.17	1.29	1.43	1.58	1.74	1.92	2.11	2.52	2.97	3.44	3.93	4.42
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

Table 8-12
Approximate Number of Passes for Welds

Weld Size* in.	Fillet Welds	Single-Bevel Groove Welds (Back-Up Weld not Included)		Single-V Groove Welds (Back-Up Weld not Included)		
		30º Bevel	45º Bevel	30º Groove Angle	60º Groove Angle	90º Groove Angle
$^{3}/_{16}$	1	–	–	–	–	–
$^{1}/_{4}$	1	1	1	2	3	3
$^{5}/_{16}$	1	1	1	2	3	3
$^{3}/_{8}$	3	2	2	3	4	6
$^{7}/_{16}$	4	2	2	3	4	6
$^{1}/_{2}$	4	2	2	4	5	7
$^{5}/_{8}$	6	3	3	4	6	8
$^{3}/_{4}$	8	4	5	4	7	9
$^{7}/_{8}$	–	5	8	5	10	10
1	–	5	11	5	13	22
$1^{1}/_{8}$	–	7	11	9	15	27
$1^{1}/_{4}$	–	8	11	12	16	32
$1^{3}/_{8}$	–	9	15	13	21	36
$1^{1}/_{2}$	–	9	18	13	25	40
$1^{3}/_{4}$	–	11	21	13	25	40

*Indicates plate thickness for groove welds.

PART 9

DESIGN OF CONNECTING ELEMENTS

SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of connecting elements (angles, plates, tees, gussets, etc.) used to transfer load from one structural member to another, as well as the affected elements of the connected members (beam webs, beam flanges, column webs, column flanges, etc.). For design considerations for bolts and welds, see Parts 7 and 8, respectively. For design provisions specific to particular connection configurations, see Parts 10 through 15.

GROSS AREA, EFFECTIVE NET AREA, AND WHITMORE SECTION

In the determination of the available axial strength of connecting elements, the gross area, A_g, is used for the yielding limit states, and the effective net area, A_e, is used for the rupture limit states. In either case, the Whitmore section may limit the effective width to less than the overall dimension of a connecting element. See Thornton and Lini (2011) for further information.

Gross Area

The gross area, A_g, is determined as specified in AISC *Specification* Section B4.3, subject to the limitations given in the following for the Whitmore section.

Effective Net Area

The effective net area, A_e, is determined as specified in AISC *Specification* Section J4.1, subject to the limitations given in the following for the Whitmore section. The reduction in area for bolt holes can be determined using Table 9-1.

Whitmore Section (Effective Width)

When connecting elements are large in comparison to the bolted or welded joints within them, the Whitmore section may limit the gross and net areas of the connecting element to less than the full area (Whitmore, 1952). As illustrated in Figure 9-1, the width of the Whitmore section, l_w, is determined at the end of the joint by spreading the force from the start of the joint 30° to each side in the connecting element along the line of force. The Whitmore section may spread across the joint between connecting elements, but cannot spread beyond an unconnected edge.

CONNECTING ELEMENTS SUBJECT TO COMBINED LOADING

Connection design has traditionally been based on simple shear, axial and flexural stresses calculated using beam theory and other models using a first yield criterion. Usually, combinations of these stresses were not required because the maximum stress for each type of loading occurred at different locations on the cross section. Designs using beam theory, and other models that are based on a first yield criterion, underestimate the strength of connection elements. Because the AISC *Specification* is based on strength design, the combination of connection design loads based on a plastic strength approach is appropriate.

Many connection elements can be modeled as rectangular members under various combinations of shear, flexural, torsional and axial loads. For rectangular connection elements with in-plane and out-of-plane loads, a plastic interaction equation for any possible load combination was developed by Dowswell (2015). For the more common case of in-plane loading only, the solution reduces to Equation 9-1, which was originally developed by Neal (1961) and later simplified by Astaneh (1998):

$$\frac{M_r}{M_c} + \left(\frac{P_r}{P_c}\right)^2 + \left(\frac{V_r}{V_c}\right)^4 \le 1.0 \tag{9-1}$$

When the required shear load is low, the shear term in Equation 9-1 makes up only a small portion of the total interaction ratio. For $V_r/V_c \le 0.40$, it is acceptable to neglect the shear term in Equation 9-1,

where

M_c = available flexural strength, determined in accordance with AISC *Specification* Chapter F, kip-in.

M_r = required flexural strength, determined in accordance with AISC *Specification* Chapter C, using LRFD or ASD load combinations, kip-in.

P_c = available axial strength, kips

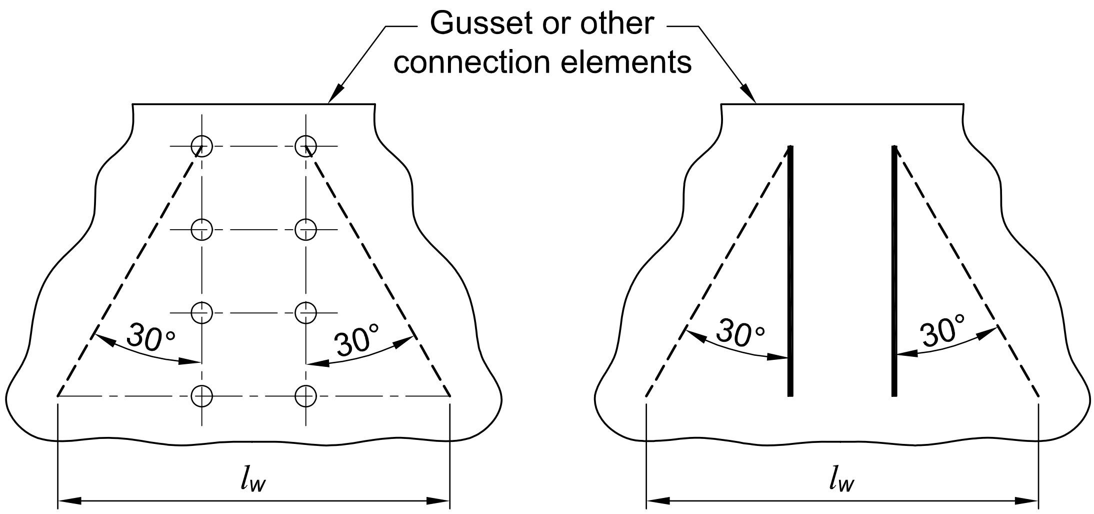
P_r = required axial strength, determined in accordance with AISC *Specification* Chapter C, using LRFD or ASD load combinations, kips

V_c = available shear strength, determined in accordance with AISC *Specification* Chapter G, kips

V_r = required shear strength, determined in accordance with AISC *Specification* Chapter C, using LRFD or ASD load combinations, kips

CONNECTING ELEMENTS SUBJECT TO TENSION

The available strength due to tension yielding and tension rupture, ϕR_n or R_n/Ω, which must equal or exceed the required tensile strength, R_u or R_a, respectively, is determined in accordance with AISC *Specification* Section J4.1.



(a) Bolted joint *(b) Welded joint*

Fig. 9-1. Illustration of the width of the Whitmore section.

CONNECTING ELEMENTS SUBJECT TO SHEAR

The available strength due to shear yielding and shear rupture, ϕR_n or R_n/Ω, which must equal or exceed the required shear strength, R_u or R_a, respectively, are determined in accordance with AISC *Specification* Section J4.2. If a wide-flange beam is uncoped it does not need to be checked for shear rupture.

CONNECTING ELEMENTS SUBJECT TO BLOCK SHEAR RUPTURE

The available strength due to block shear rupture, ϕR_n or R_n/Ω, which must equal or exceed the required strength, R_u or R_a, respectively, is determined in accordance with AISC *Specification* Section J4.3. The values tabulated in Table 9-3 are used to calculate the available block shear rupture strength.

CONNECTING ELEMENT RUPTURE STRENGTH AT WELDS

In many cases, the load path from a weld to the connecting element is such that the strength of the connecting element can be evaluated directly, for example, the wall of an HSS subject to tensile force. However, in some cases, the available strength of the connecting element is not directly calculable. For example, while the strength of the beam-web welds for a double-angle connection can be directly calculated, the strength of the beam web at this weld cannot. In cases such as these, it is often convenient to calculate the minimum base metal thickness that will match the available shear rupture strength of the base metal to the available shear rupture strength of the weld(s).

For fillet welds with F_{EXX} = 70 ksi only on one side of an element, the minimum base metal thickness required to match the shear rupture strength of the weld is

$$t_{min} = \frac{0.60 F_{EXX}\left(\frac{\sqrt{2}}{2}\right)\left(\frac{D}{16}\right)}{0.6F_u} = \frac{3.09D}{F_u} \tag{9-2}$$

where

D = required number of sixteenths of an inch in the weld size on each side of the connecting element

F_u = specified minimum tensile strength of the base metal, ksi

For fillet welds with F_{EXX} = 70 ksi on both sides of an element, the minimum base metal thickness required to match the shear rupture strength of the weld is

$$t_{min} = \frac{6.19D}{F_u} \tag{9-3}$$

There is no limit on the fillet weld size where one of the elements is subject to a tensile force and that element is not in the plane of the element being connected. Examples are the wall of an HSS to a base plate or a built-up tee-hanger web to a flange plate. For such cases, the fillet welds need only be sized to resist the tensile force and a base metal check is not required.

CONNECTING ELEMENTS SUBJECT TO COMPRESSION YIELDING AND BUCKLING

When connecting elements are subject to compression, the available compressive strength, ϕP_n or P_n/Ω, which must equal or exceed the required compressive strength, P_u or P_a, respectively, is determined in accordance with AISC *Specification* Section J4.4.

AFFECTED AND CONNECTING ELEMENTS SUBJECT TO FLEXURE

Affected and connecting elements are normally short enough and thick enough that flexural effects, if present at all, do not impact the design. When such elements are long enough and thin enough that flexural effects must be considered, the following provisions are used for determining the available strength.

Yielding, Lateral-Torsional Buckling, and Local Buckling

Generally, the available flexural strength, ϕM_n or M_n/Ω, which must equal or exceed the required flexural strength of affected and connecting elements, M_u or M_a, respectively, is determined in accordance with AISC *Specification* Section J4.5 and Chapter F. The Table User Note in AISC *Specification* Section F1.1 provides guidance based upon cross-section shape for the applicable Chapter F section.

Treatment of coped beams is provided in the following.

Rupture

For rolled or built-up shapes with bolt holes in the tension flange, see AISC *Specification* Section F13.1. For affected and connecting elements, the available flexural rupture strength, $\phi_b M_n$ or M_n/Ω_b, is

$$M_n = F_u Z_{net} \tag{9-4}$$

$$\phi_b = 0.75 \qquad \Omega_b = 2.00$$

where

Z_{net} = net plastic section modulus of the affected or connecting element, in.^{3}

Coped Beam Strength

For beam ends with short copes no greater than the length of the connection angle(s), plate (except extended single-plate connections), or tee, flexural local web buckling will generally not occur. Otherwise, the end reaction for a coped beam may be limited by the flexural limit states of yielding, rupture, flexural local buckling, or lateral-torsional buckling. The strength of coped beams with bolted shear connections as shown in Part 10 will rarely be governed by flexural rupture. For a coped beam laterally braced at the end of the uncoped section, the required flexural strength is

LRFD	ASD
$M_u = R_u e$ (9-5a)	$M_a = R_a e$ (9-5b)

where

R_u or R_a = beam end reaction (LRFD or ASD), kips

e = distance from the face of the supporting member to the face of the cope, unless a lower value can be justified, in.

The available flexural local buckling strength of a beam coped at the top flange or both the top and bottom flanges must equal or exceed the required strength. The available strength, $\phi_b M_n$ or M_n/Ω_b, is determined as follows.

1. For beams coped at the top flange only as shown in Figure 9-2, the connection element should be located near the coped edge. The minimum length of the connection elements is one-half of the reduced beam depth, h_o. The flexural strength at the coped section is as follows.

   When $\lambda \le \lambda_p$

$$M_n = M_p \tag{9-6}$$

   When $\lambda_p < \lambda \le 2\lambda_p$

$$M_n = M_p - (M_p - M_y)\left(\frac{\lambda}{\lambda_p} - 1\right) \tag{9-7}$$

   When $\lambda > 2\lambda_p$

$$M_n = F_{cr} S_{net} \tag{9-8}$$

   where

   F_{cr} = critical stress, ksi

$$= \frac{0.903Ek_1}{\lambda^2} \tag{9-9}$$

   E = modulus of elasticity of steel
   = 29,000 ksi

   F_y = specified minimum yield stress, ksi

   M_p = plastic bending moment, kip-in.
   = $F_y Z_{net}$

   M_y = flexural yield moment, kip-in.
   = $F_y S_{net}$

   S_{net} = elastic section modulus at the cope, in.^{3}

   Z_{net} = plastic section modulus at the cope, in.^{3}

   k_1 = modified plate buckling coefficient

$$= fk \ge 1.61 \tag{9-10}$$

   λ = web slenderness

$$= \frac{h_o}{t_w} \tag{9-11}$$

λ_p = limiting slenderness for a compact web

$$= 0.475\sqrt{\frac{k_1 E}{F_y}} \tag{9-12}$$

ϕ_b = 0.90
Ω_b = 1.67

The plate buckling coefficient, k, is determined as follows.

When $\dfrac{c}{h_o} \le 1.0$

$$k = 2.2\left(\frac{h_o}{c}\right)^{1.65} \tag{9-13a}$$

When $\dfrac{c}{h_o} > 1.0$

$$k = 2.2\left(\frac{h_o}{c}\right) \tag{9-13b}$$

The buckling adjustment factor, f, is determined as follows.

When $\dfrac{c}{d} \le 1.0$

$$f = 2\left(\frac{c}{d}\right) \tag{9-14a}$$

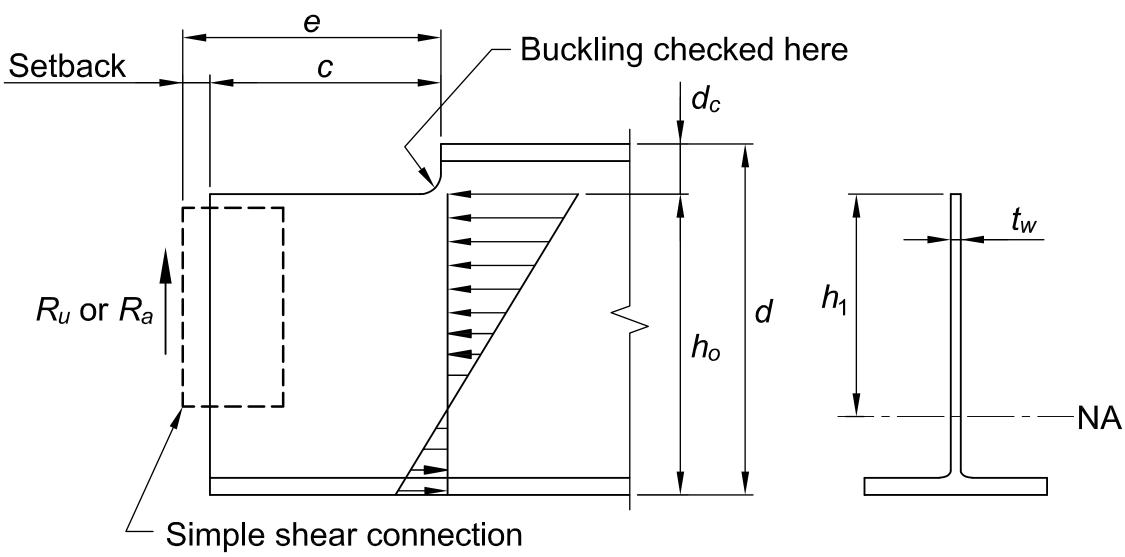


Fig. 9-2. Flexural local buckling of beam web coped at top flange only.

When $\dfrac{c}{d} > 1.0$

$$f = 1 + \frac{c}{d} \le 3 \tag{9-14b}$$

c = cope length, in.
d = beam depth, in.
h_o = depth of coped section, in.
t_w = web thickness, in.

2. For a beam that is coped at both flanges, the local flexural strength is determined in accordance with AISC *Specification* Section F11 (Dowswell and Whyte, 2014). Refer to Figure 9-3.

   (a) When the bottom (tension) cope is equal to or longer than the length of the top cope

$$C_b = \left[3 + \ln\left(\frac{L_b}{d}\right)\right]\left(1 - \frac{d_{ct}}{d}\right) \le 1.84 \tag{9-15}$$

   where
   C_b = lateral-torsional buckling modification factor
   L_b = c_t, in.
   c_t = length of top cope, in.
   d = depth of beam, in.
   d_{ct} = cope depth at top flange as illustrated in Figure 9-3, in.

   Yielding should also be checked at the end of the bottom cope.

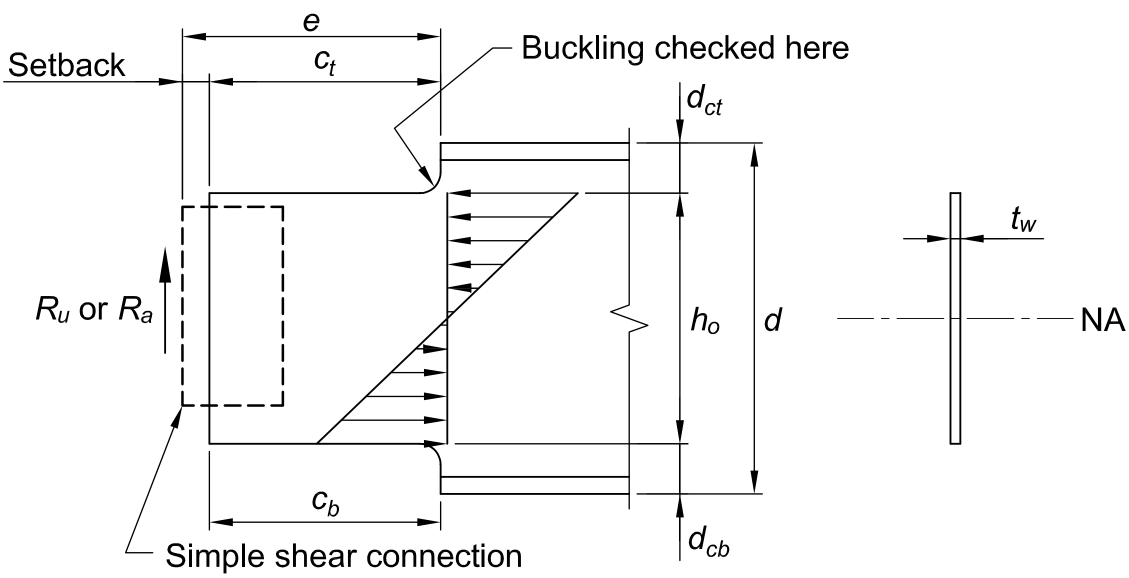


Fig. 9-3. Flexural local buckling of beam web coped at both flanges.

(b) When the top cope is longer than the bottom cope

$$C_b = \left(\frac{c_b}{c_t}\right)\left[3 + \ln\left(\frac{L_b}{d}\right)\right]\left(1 - \frac{d_{ct}}{d}\right) \le 1.84 \tag{9-16}$$

where

$L_b = \dfrac{c_t + c_b}{2}$, in.

c_b = length of bottom cope, in.

BEARING LIMIT STATES

Bearing Strength and Tearout at Bolt Holes

For available bearing and tearout strength at bolt holes, see Part 7.

Steel-on-Steel Bearing Strength (Other Than at Bolt Holes)

Bearing strength for applications other than at bolt holes is determined in accordance with AISC *Specification* Section J7. The fabrication and erection requirements in AISC *Specification* Sections M2.6, M2.8 and M4.4 are applicable to connecting elements that transfer load by contact bearing on steel.

Bearing Strength on Concrete or Masonry

The bearing strength of concrete is determined in accordance with AISC *Specification* Section J8. For bearing on masonry, see *Building Code Requirements for Masonry Structures*, ACI 530/ASCE 5/TMS 402 (ACI/ASCE/TMS, 2013a) and *Specification for Masonry Structures,* ACI 530.1/ASCE 6/TMS 602 (ACI/ASCE/TMS, 2013b). The fabrication and erection requirements in AISC *Specification* Sections M2.8 and M4.1 are applicable to connecting elements that transfer load by contact bearing on concrete or masonry.

OTHER SPECIFICATION REQUIREMENTS AND DESIGN CONSIDERATIONS

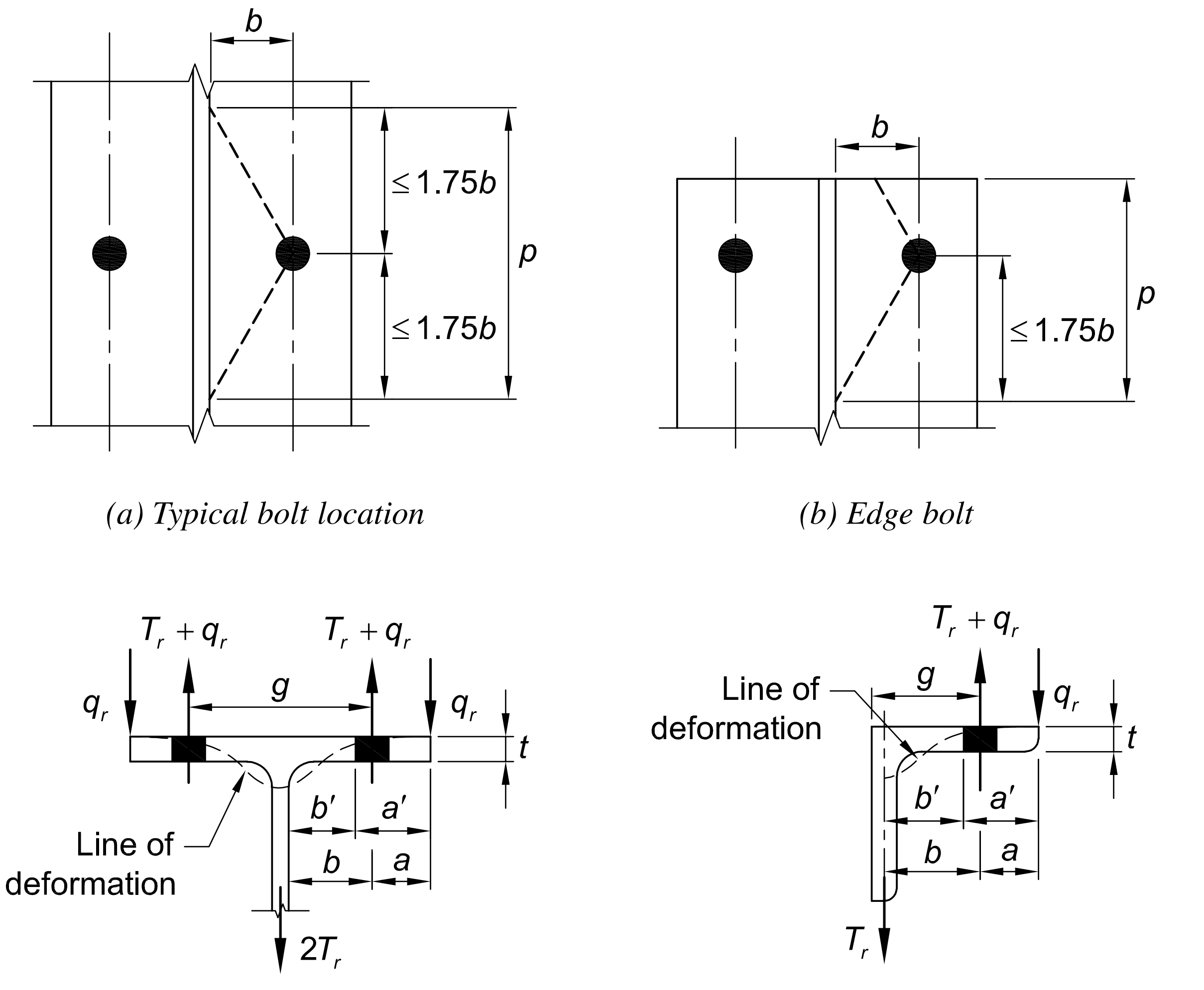
Other design considerations may apply and may be required by the AISC *Specification*. Some considerations are described in the following using concepts that have proven effective. Other rational methods may also be used.

Prying Action

Prying action is a phenomenon (in bolted construction only, and only in connections with tensile bolt forces) whereby the deformation of a connecting element under a tensile force increases the tensile force in the bolt above that due to the direct tensile force alone. Proper design for prying action includes the selection of bolt diameter and fitting thickness, t, such that there is sufficient stiffness and strength in the connecting element and strength in the bolt. The following discussion of prying action is similar to what has been considered in the past, except that the design basis has been changed to calculate strength in terms of F_u, which provides better correlation with available test data than previous design methods. For the development of the prying action equations presented here, see Thornton (1992) and Swanson (2002).

The dimensions b and b', in Figure 9-4, are measured from the face of the tee stem or the center of the angle leg. These values are valid for tees and for angles if the load, $2T_r$ (T_u for LRFD and T_a for ASD), is delivered symmetrically and the angle shown represents one of a pair of back-to-back angles. When the angles are not back-to-back and connected to a relatively flexible support, the effective eccentricity may be increased and a distance measured to the heel of the angle or possibly somewhat larger might be warranted. It is common to assume there is no moment transfer between the flange and the element to which it is attached; that is, all of the moment required for equilibrium of the flange is assumed to be taken at the bolt line. This discussion is not intended to be applied to asymmetrical conditions. When the load is delivered asymmetrically, an even greater moment will result. For instance, if the angle is attached to only one flange of a wide-flange member used as a hanger and the hanger is not restrained from rotating about the bolt line, then the eccentricity would be measured from the centerline of the hanger or to the point of application of the load.

Consider the tee or angle used in a hanger connection as shown in Figure 9-4. The deformation of the connected tee flange or angle leg is assumed to be in double curvature if prying forces exist. In Figure 9-4, T_r is the required tension force per bolt using LRFD or ASD load combinations and q_r is the corresponding prying force.



(a) Typical bolt location *(b) Edge bolt*

(c) Prying forces in tee *(d) Prying forces in angle*

Fig. 9-4. Illustration of variables in prying action calculations.

The required thickness to eliminate prying action, t_{np} is:

LRFD	ASD
$$t_{np} = \sqrt{\frac{4T_u b'}{\phi p F_u}} \qquad \text{(9-17a)}$$	$$t_{np} = \sqrt{\frac{\Omega 4T_a b'}{p F_u}} \qquad \text{(9-17b)}$$

where

F_u = specified minimum tensile strength of connecting element, ksi
T_a = required tension force per bolt using ASD load combinations, kips
T_u = required tension force per bolt using LRFD load combinations, kips

$$b' = \left(b - \frac{d_b}{2}\right) \qquad \text{(9-18)}$$

b = for a tee-type connecting element, the distance from bolt centerline to the face of the tee stem, in.; for an angle-type connecting element, the distance from bolt centerline to centerline of angle leg, in.
d_b = bolt diameter, in.
p = tributary length, based on yield line theory [see Dowswell (2011) and Wheeler et al. (1998)] or conservatively taken as 3.5b, but ≤ s, in.
s = bolt spacing, in.
ϕ = 0.90
Ω = 1.67

When the resulting fitting thickness is reasonable, no further check of prying action is necessary as long as the required bolt force, T_r, does not exceed the available bolt strength, T_c. In this solution, the additional force in the bolt due to prying action, q_r, is essentially zero and the flange or angle leg is in single curvature.

Alternatively, it is usually possible to determine a lesser required thickness by designing the connecting element and bolted joint for the actual effects of prying action with q_r greater than zero; however, a larger required bolt diameter may result.

The thickness required to ensure an acceptable combination of fitting strength and stiffness and bolt strength, t_{min}, is:

LRFD	ASD
$$t_{min} = \sqrt{\frac{4T_u b'}{\phi p F_u (1 + \delta\alpha')}} \qquad \text{(9-19a)}$$	$$t_{min} = \sqrt{\frac{\Omega 4T_a b'}{p F_u (1 + \delta\alpha')}} \qquad \text{(9-19b)}$$

where

$$\delta = 1 - \frac{d'}{p} \qquad \text{(9-20)}$$

= 1 − ratio of the net length at bolt line to gross length at the face of the stem or leg of angle

d' = width of the hole along the length of the fitting, in.

α' = 1.0 if $\beta \geq 1$

= lesser of 1 and $\frac{1}{\delta}\left(\frac{\beta}{1-\beta}\right)$ if $\beta < 1$

$$\beta = \frac{1}{\rho}\left(\frac{B_c}{T_r} - 1\right) \tag{9-21}$$

$$\rho = \frac{b'}{a'} \tag{9-22}$$

$$a' = \left(a + \frac{d_b}{2}\right) \le \left(1.25b + \frac{d_b}{2}\right) \tag{9-23}$$

a = distance from the bolt centerline to the edge of the fitting, in.
B_c = available tension per bolt based on the limit state of tension only or the combined limit states of tension and shear rupture, ϕr_n or r_n/Ω, kips
ϕ = 0.90
Ω = 1.67

If $t_{min} \le t$, the preliminary fitting thickness is satisfactory. Otherwise, a fitting with a thicker flange, or a change in geometry (i.e., b and p) is required.

Although it is not necessary to do so, if desired, the prying force per bolt, q_r, can be determined as

$$q_r = B_c\left[\delta\alpha\rho\left(\frac{t}{t_c}\right)^2\right] \tag{9-24}$$

where

$$\alpha = \frac{1}{\delta}\left[\frac{T_r}{B_c}\left(\frac{t_c}{t}\right)^2 - 1\right] \text{ with } 0 \le \alpha \le 1.0 \tag{9-25}$$

The flange or angle thickness, t_c, required to develop the available strength of the bolt, B_c, with no prying action is:

LRFD	ASD
$t_c = \sqrt{\dfrac{4B_c b'}{\phi p F_u}}$ (9-26a)	$t_c = \sqrt{\dfrac{\Omega 4B_c b'}{p F_u}}$ (9-26b)

The parameter α is the ratio of the moment at the bolt line to the moment at the face of the tee stem, or at the center of the unconnected angle leg thickness. When $\alpha = 0$, the connection is strong enough to prevent prying action. When $\alpha > 1$, the connection is not adequate. The total force per bolt including the effects of prying action is then $T_r + q_r$.

Alternatively, when the fitting geometry is known, the available tensile strength per bolt, B_c, determined per AISC *Specification* Sections J3.6 or J3.7, is multiplied by Q to determine the available tensile strength including the effects of prying action, T_c, as follows:

$$T_c = B_c Q \tag{9-27}$$

where

Q = 1 if $\alpha' < 0$, which means that the fitting has sufficient strength and stiffness to develop the full available tensile strength of the bolt.

$= \left(\dfrac{t}{t_c}\right)^2 (1 + \delta\alpha')$ if $0 \le \alpha' \le 1$, which means that the fitting has sufficient strength to develop the full bolt available tensile strength, but insufficient strength to prevent prying action.

$= \left(\dfrac{t}{t_c}\right)^2 (1 + \delta)$ if $\alpha' > 1$, which means that the fitting has insufficient strength to develop the full bolt available tensile strength.

$$\alpha' = \frac{1}{\delta(1+\rho)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right] \tag{9-28}$$

= value of α that either maximizes the bolt available tensile strength for a given thickness or minimizes the thickness required for a given bolt available tensile strength.

Plate Elements Subjected to Out-of-Plane Loads

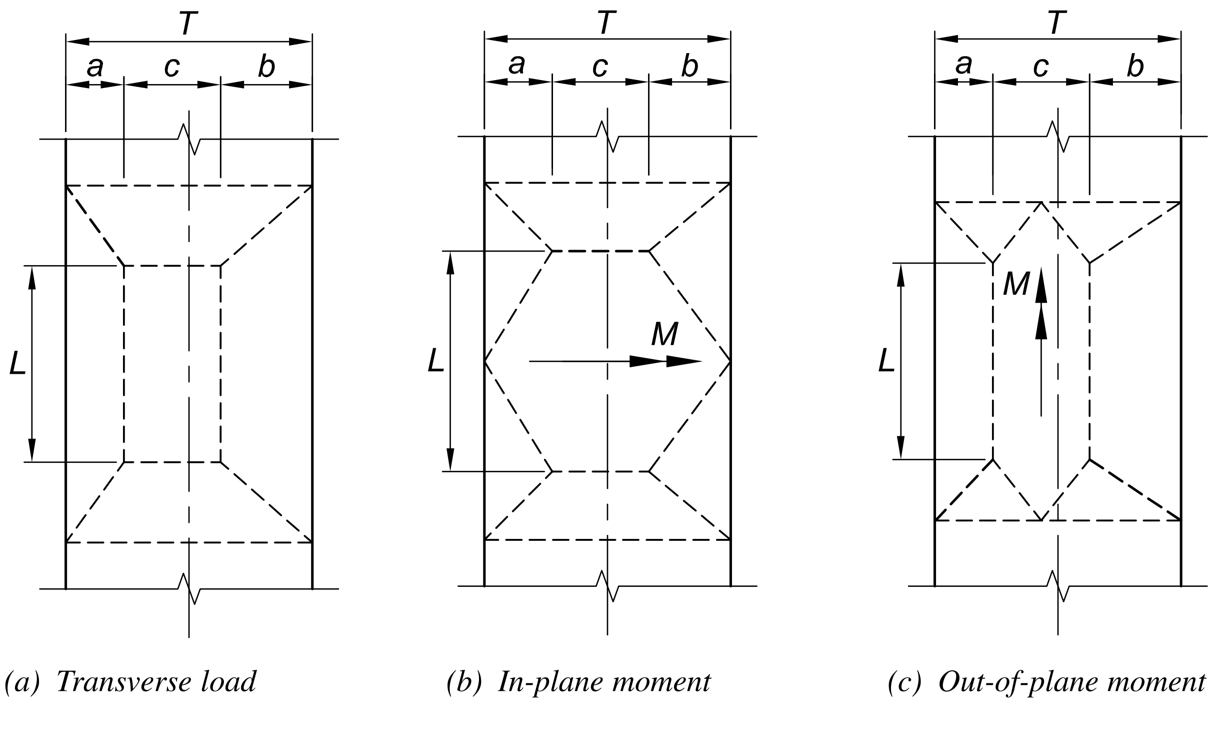
Generally out-of-plane loading of elements, such as the webs of wide-flange members, is avoided. However, when such loading is unavoidable or the loads are relatively small, the strength of the plate element can be determined based on shear and weak-axis bending of the element per AISC *Specification* Section J10.10. The available shear strength is ϕR_n or R_n/Ω, and the available flexural strength is ϕM_n or M_n/Ω, where $\phi = 1.00$ and $\Omega = 1.50$.

A punching shear approach is typically used to determine the shear strength of the element. The nominal punching shear strength can be obtained from the product of the shear stress, the element thickness, and the punching perimeter. Using the definitions provided in Figure 9-5:

$$R_n = 0.6F_y t_p (2c_{eff} + 2L) \tag{9-29}$$

where

F_y = specified minimum yield stress of element, ksi
L = length over which the load is delivered, measured parallel to the supported edges, in.
c_{eff} = effective width of the attached element accounting for uneven stress distribution, in.
t_p = thickness of element, in.



(a) Transverse load *(b) In-plane moment* *(c) Out-of-plane moment*

Fig. 9-5. Yield-line analysis.

The effective width, c_{eff}, for an element attaching to the face of a rectangular HSS is determined in accordance with AISC *Specification* Equation K1-1:

$$c_{eff} = B_e = \left(\frac{10t}{B}\right)\left(\frac{F_y t}{F_{yb} t_b}\right) B_b \le B_b \qquad (Spec.\ \text{Eq. K1-1})$$

This same approach should also be applicable to a wide-flange member where both the flanges are restrained against rotation. If the branch is welded to the web of a wide-flange and the flanges are not restrained, then the effective width, c_{eff}, is the T-dimension of the wide-flange.

A yield-line approach is typically used to determine the weak-axis flexural strength of the plate element. The yield line analysis is a work-energy method in which assumed patterns of flexural mechanisms (the yield lines) are used to determine the strength of the element. The AISC *Engineering Journal* has published several articles describing the yield line approach (Abolitz and Warner, 1965; Kapp, 1974; Stockwell, 1974; Dranger, 1977). Several solutions dependent on loadings and boundary conditions are presented in the text to follow.

The yield-line approach to establishing the weak-axis flexural strength of elements serves to limit connection deformations and is known to be well below the ultimate connection strength. When the load is applied over a width that exceeds 85% of the plate element width, this yield-line failure mechanism will result in a noncritical design load. The effective width approach, per AISC *Specification* Section K1.2a, is used to determine the effective punching shear perimeter, with the total perimeter being an upper limit on this length.

Note the factor, Q_f, reduces the local strength at the connection due to the global forces in the member. This factor has historically not been applied to wide-flange members, perhaps because a greater portion of the area in a wide-flange member is generally located at the flanges and because the local reduction does not induce any eccentricity.

For rectangular HSS of the material specified in AISC *Specification* Section A3.1, where the wall slenderness, b/t, is less than or equal to 30, Q_f can be determined as:

Q_f = 1 for HSS (connecting surface) in tension
= $1.0 - 0.3U(1 + U)$ for HSS (connecting surface) in compression (*Spec*. Eq. K2-3)

where

$$U = \left| \frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S} \right| \qquad (Spec.\ \text{Eq. K2-4})$$

A_g = gross cross-sectional area of the member, in.^{2}
F_c = available stress in main member, ksi
= F_y for LRFD; $0.60F_y$ for ASD
M_{ro} = required moment of the HSS, determined in accordance with AISC *Specification* Chapter C, based on the LRFD (M_u) or ASD (M_a) load combinations, kip-in.
P_{ro} = required strength of the HSS, determined in accordance with AISC *Specification* Chapter C, based on the LRFD (P_u) or ASD (P_a) load combinations, kips
S = elastic section modulus about the axis of bending, in.^{3}

P_{ro} and M_{ro} are determined on the side of the joint that has the lower compressive stress.

For out-of-plane transverse loads

For rectangular HSS, the edges of the HSS wall are generally assumed to be clamped:

$$R_n = \frac{t^2 F_y}{2}\left[\frac{(a+b)\left(4\sqrt{\dfrac{Tab}{a+b}}+L\right)}{ab}\right]Q_f \tag{9-30}$$

When the concentrated force is applied at a distance from the member end that is less than $2\sqrt{\dfrac{Tab}{a+b}}$, R_n is reduced by 50%.

For wide-flange sections, the edges of the column web are generally assumed to be pinned:

$$R_n = \frac{t_w{}^2 F_y}{4}\left[\frac{4\sqrt{2Tab(a+b)}+L(a+b)}{ab}\right] \tag{9-31}$$

When the concentrated force is applied at a distance from the member end that is less than $\sqrt{\dfrac{8Tab}{a+b}}$, R_n is reduced by 50%,

where

T = width of element, in. (width of HSS, depth of wide-flange minus k-dimension)

a = distance measured along width of element from one edge of connected element to nearest support, in.

b = distance measured along width of plate element from one edge of connected element to farthest support, in.

For in-plane moments

Plastification need not be checked when the rotation is self-limiting, such as at framed, simple beam end connections.

For rectangular HSS, the edges of the HSS wall are generally assumed to be clamped:

$$M_n = \frac{t^2 F_y}{4}\left(\frac{2T}{L}+\frac{4L}{T-c}+8\sqrt{\frac{T}{T-c}}\right)LQ_f \tag{9-32}$$

For wide-flange sections, the edges of the web are generally assumed to be pinned:

$$M_n = \frac{t^2 F_y}{4}\left(\frac{2T}{L}+\frac{2L}{T-c}+4\sqrt{\frac{2T}{T-c}}\right)L \tag{9-33}$$

For out-of-plane moments

Plastification need not be checked when the rotation is self-limiting, such as at framed, simple-beam end connections.

For rectangular HSS, the edges of the HSS wall are generally assumed to be clamped:

$$M_n = \frac{t^2 F_y}{4}\left(\frac{4\sqrt{abcT\rho}+L\rho}{ab}\right)Q_f \tag{9-34}$$

For wide-flange sections, the edges of the web are generally assumed to be pinned:

$$M_n = \frac{t^2 F_y}{4}\left(\frac{4\sqrt{2abcT\rho} + L\rho}{2ab}\right)Q_f \tag{9-35}$$

where

L = distance over which the load is delivered, measured along the longer dimension of the plate element, in.

a = distance measured along width of the plate element from one edge of connected element to nearest support, in.

b = distance measured along width of the plate element from one edge of connected element to farthest support, in.

c = distance over which the load is delivered, measured along the shorter dimension of the plate element, in.

$$\rho = 2ab + ac + bc \tag{9-36}$$

Rotational Ductility

Simple shear connections provide for the rotational ductility required by AISC *Specification* Section J1.2 as follows:

1. For double-angle, shear end-plate, single-angle, and tee shear connections, the geometry and thickness of the connecting elements attached to the support (angle legs, plate, or tee flange) are configured so that flexing of those connecting elements accommodates the simple-beam end rotation.
2. For unstiffened and stiffened seated connections, the geometry and thickness of the top or side stability angle is configured so that flexing of that connecting element accommodates the simple-beam end rotation.
3. For single-plate connections, the geometry and thickness of the plate are configured so that the connecting material or beam web will yield, bolt group will rotate, and/or the bolt holes in the connecting material or beam web will elongate at failure prior to the failure of the welds or bolts.

For each of the simple-shear connections in Part 10, except tee shear connections, prescriptive guidance is provided to ensure adequate rotational ductility. Rotational ductility can be ensured for tee shear connections as follows. Note that this approach can also be used to demonstrate adequate rotational ductility in other simple shear connections that flex to accommodate the simple-beam end rotation, but with configurations that differ from those prescribed in Part 10.

When the flanges of the tee stub are welded to the support and bolted to the supported beam, weld size, w, with F_{EXX} = 70 ksi, must be such that the minimum weld size, w_{min}, is

$$w_{min} = 0.0155\,\frac{F_y t_f^2}{b}\left(\frac{b^2}{l^2} + 2\right) \tag{9-37}$$

but need not exceed $(5/8)t_{sw}$ (Thornton, 1996),

where

b = flexible width in connecting element as illustrated in Figure 9-6, in.

l = length of connecting element as illustrated in Figure 9-6, in.

t_f = thickness of the tee flange, in.
t_{sw} = thickness of the tee stem, or supported beam web, in.

For a tee bolted to the support and bolted or welded to the supported beam, the minimum diameter for bolts through the tee flange for ductility is

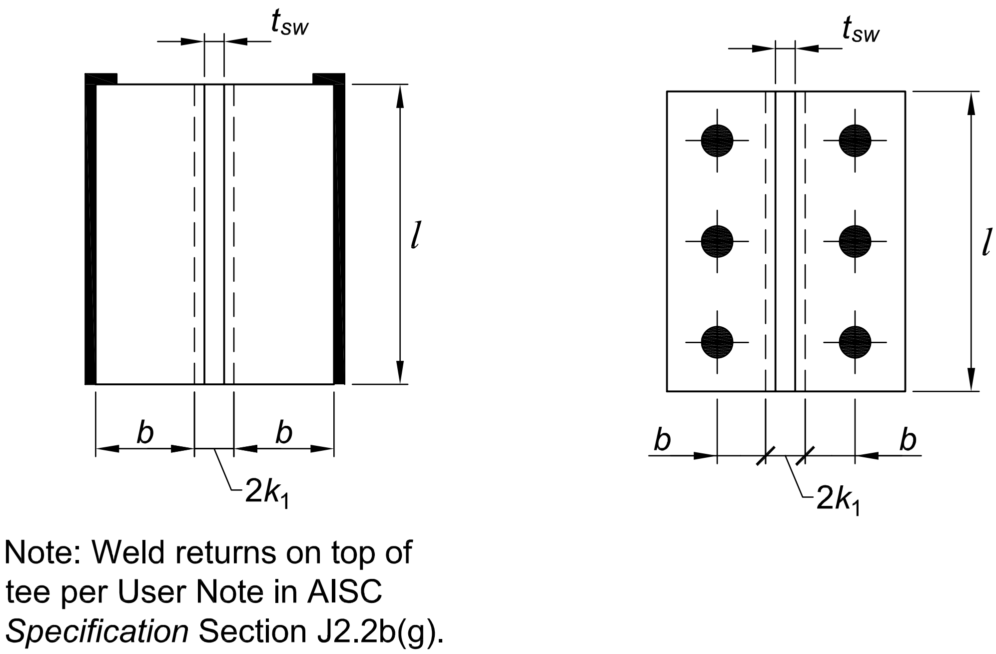
$$d_{min} = 0.163 t_f \sqrt{\frac{F_y}{b}\left(\frac{b^2}{l^2} + 2\right)} \tag{9-38}$$

but need not exceed $0.69\sqrt{t_{sw}}$. Alternatively, to provide for rotational ductility when the tee stem is bolted to the supported beam, the maximum tee stem thickness or beam web is

$$t_{sw} = \frac{d}{2} + {}^{1}/_{16} \text{ in.} \tag{9-39}$$

where
d = bolt diameter, in.

When the tee stem is welded to the supported beam, there is no perceived ductility problem for this weld. Connections satisfying the parameters discussed in the foregoing can be expected to accommodate rotations in the range of 0.03 rad. The checks are intended for use with connections between 6 in. and 36 in. deep and configured similarly to the connections shown in Part 10. The use of deeper connections, smaller offset distances between the supported and supporting members, or smaller edge distances can affect the ability of connections to accommodate large rotations in a ductile manner. Connections satisfying these parameters satisfy the intent of AISC *Specification* Section B3.4a for simple connections.



(a) Welded flange *(b) Bolted flange*

Fig. 9-6. Illustration of variables in shear connection ductility checks.

Concentrated Forces

If the connecting element delivers a concentrated force to a member or other connecting element, see AISC *Specification* Section J10 or Section K2, as appropriate. See also AISC Design Guide 13, *Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications* (Carter, 1999).

Shims and Fillers

Shims are furnished to the erector for use in filling the spaces allowed for field clearance that might be present at connections, such as simple shear connections, PR and FR moment connections, column base plates, and column splices. These shims, illustrated in Figure 9-7, may be either strip shims with round punched holes or finger shims with slots cut through the edge. Whereas strip shims are less expensive to fabricate, finger shims may be laterally inserted and eliminate the need to remove erection bolts or pins already in place.

Finger shims, when inserted fully against the bolt shank, are acceptable for slip-critical connections and are not to be considered as an internal ply with the slotted hole determining the available strength of the connection. This is because less than 25% of the contact surface is lost, which is not enough to affect the performance of the joint.

A filler is furnished to occupy spaces that will be present because of dimensional separations between elements of a connection across which load transfer occurs. Examples where fillers might be used are beams framing off center on a column and raised beams.

For the effect of fillers and shims on available joint strength, see AISC *Specification* Sections J3.8 and J5.2.

Copes, Blocks and Cuts

When structural members frame together, a minimum clearance of $^1/_2$ in. should be provided, when possible. In cases where material removal is necessary to provide such a clearance, material may be removed by coping, blocking or cutting, as illustrated in Figure 9-8.

Material removal is costly and should be avoided when possible. In some cases, it may be feasible to do so by setting the elevations of the tops of infill beams a sufficient distance below the tops of girders to clear the girder fillet radius. Alternatively, a connection such as that illustrated in Figure 9-9 could be used.

When material removal is necessary, coping is usually the most economical method to remove material. The recommended practices for coping are illustrated in Figure 9-10. The potential notch left by the first cut will occur in waste material and will subsequently be removed by the second cut. All re-entrant corners must be shaped notch-free, per AWS D1.1, to a radius. An approximate minimum radius to which this corner must be shaped is $^1/_2$ in.

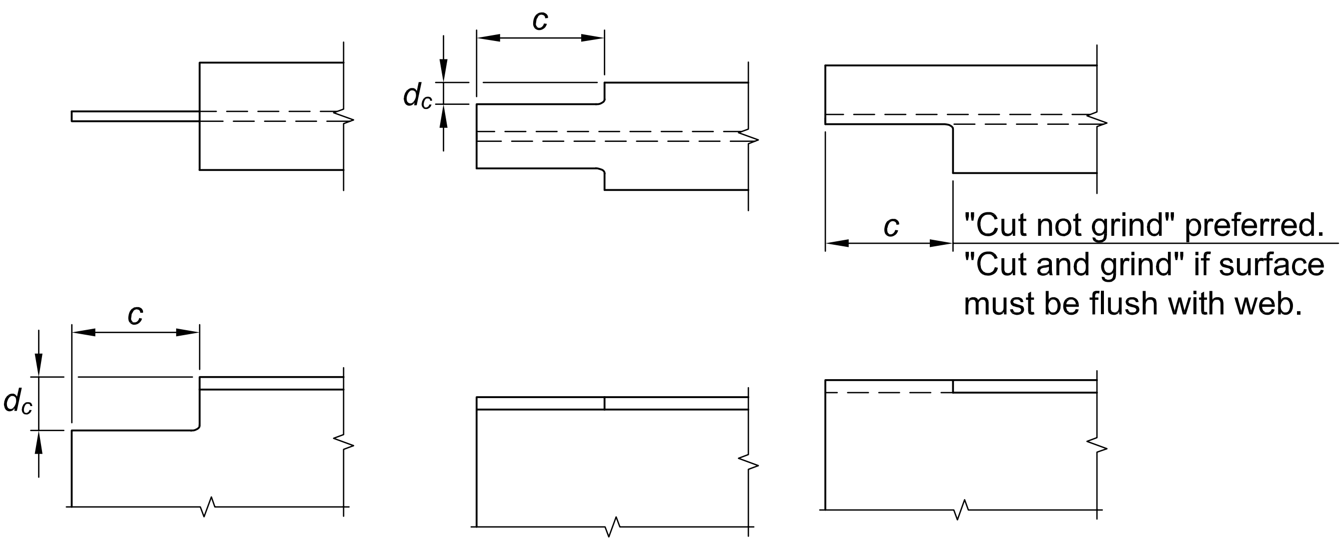
(a) Strip *(b) Finger*

Fig. 9-7. Shims.

Copes, blocks and cuts can significantly reduce the available strengths of members and may require web reinforcement; it may be more economical to use a heavier member than to provide such reinforcement.

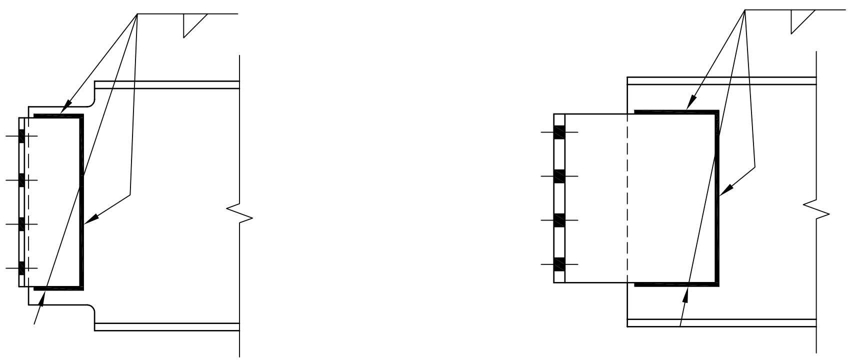
Web Reinforcement of Coped Beams

When the strength of a coped beam is inadequate, either a different beam with a thicker web can be selected to eliminate the need for reinforcement, or reinforcement can be provided to increase the strength. In spite of the increase in material cost, the former solution may be the most economical option due to the appreciable labor cost associated with adding stiffeners and/or doubler plates. When the latter solution is required, some typical reinforcing details are illustrated in Figure 9-11.



(a) Cope *(b) Blocks* *(c) Cut*

Fig. 9-8. Copes, blocks and cuts.



(a) Coping required *(b) Coping eliminated*

Fig. 9-9. Eliminating coping requirements.

The doubler plate illustrated in Figure 9-11(a) and the longitudinal stiffener illustrated in Figure 9-11(b) are used with rolled sections where $h/t_w \leq 60$. When a doubler plate is used, the required doubler-plate thickness, $t_{d\ req}$, is determined by substituting the quantity $(t_w + t_{d\ req})$ for t_w in the available strength calculations for flexural yielding and web local buckling. To prevent local crippling of the beam web, the doubler plate must be extended at least a distance d_c (depth of cope) beyond the cope as illustrated in Figure 9-11(a). When longitudinal stiffening is used, the stiffening elements must be proportioned to meet the width-to-thickness ratios specified in AISC *Specification* Table B4.1b. The stiffened cross section must then be checked for flexural yielding, but web local buckling need not be checked. To prevent local crippling of the beam web, the longitudinal stiffening must be extended a minimum distance of d_c beyond the cope as illustrated in Figure 9-11(b).

The combination of longitudinal and transverse stiffeners shown in Figure 9-11(c) may be required for thin-web plate girders, where $h/t_w > 60$. When longitudinal and transverse stiffening is used, the stiffening elements must be proportioned to meet the width-to-thickness ratios specified in AISC *Specification* Table B4.1b. The stiffened cross section must then be checked for flexural yielding, but web local buckling need not be checked. To prevent local crippling of the beam web, longitudinal stiffeners must be extended a minimum distance of $c/3$ beyond the cope, as illustrated in Figure 9-11(c).

DESIGN TABLE DISCUSSION

Table 9-1. Reduction in Area for Holes

Area reduction for standard, oversized, short-slotted and long-slotted holes in material thicknesses from 3/16 in. to 1 in. are given in Table 9-1. For material thicknesses not listed, the tabular value for 1-in. thickness can be multiplied by the actual thickness. The table is based on a net area using a width that is 1/16 in. greater than the actual hole width.

Table 9-2. Elastic Section Modulus for Coped W-Shapes

Values are given for the gross and net elastic section modulus for coped W-shapes, as illustrated in the table header.

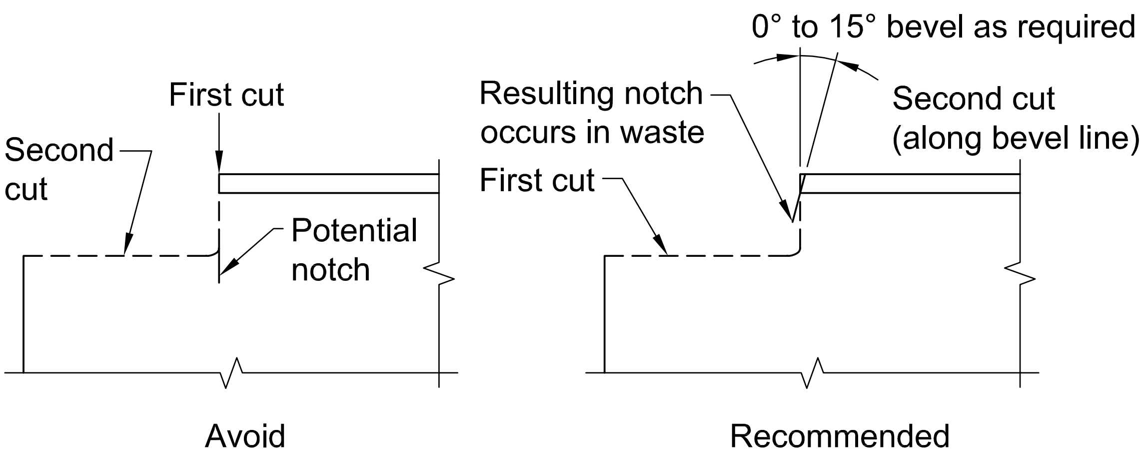


Fig. 9-10. Recommended coping practices.

Tables 9-3. Block Shear Rupture

The terms in AISC *Specification* Equation J4-5 are tabulated in Tables 9-3a, 9-3b and 9-3c. The indicated values are given per inch of material thickness.

Table 9-4. Beam Bearing Constants

At beam ends and at any location on beams or columns where concentrated loads occur, the available strength for web local yielding and web local crippling, ϕR_n or R_n/Ω, at concentrated loads is determined per AISC *Specification* Sections J10.2 and J10.3. Values of R_n are given for a bearing length, $l_b = 3^1/_4$ in. The equations for web local yielding (AISC *Specification* Equations J10-2 and J10-3) and web local crippling (AISC *Specification* Equations J10-4, J10-5a and J10-5b) can be simplified using the bearing length, l_b, and the constants R_1 through R_6 as follows.

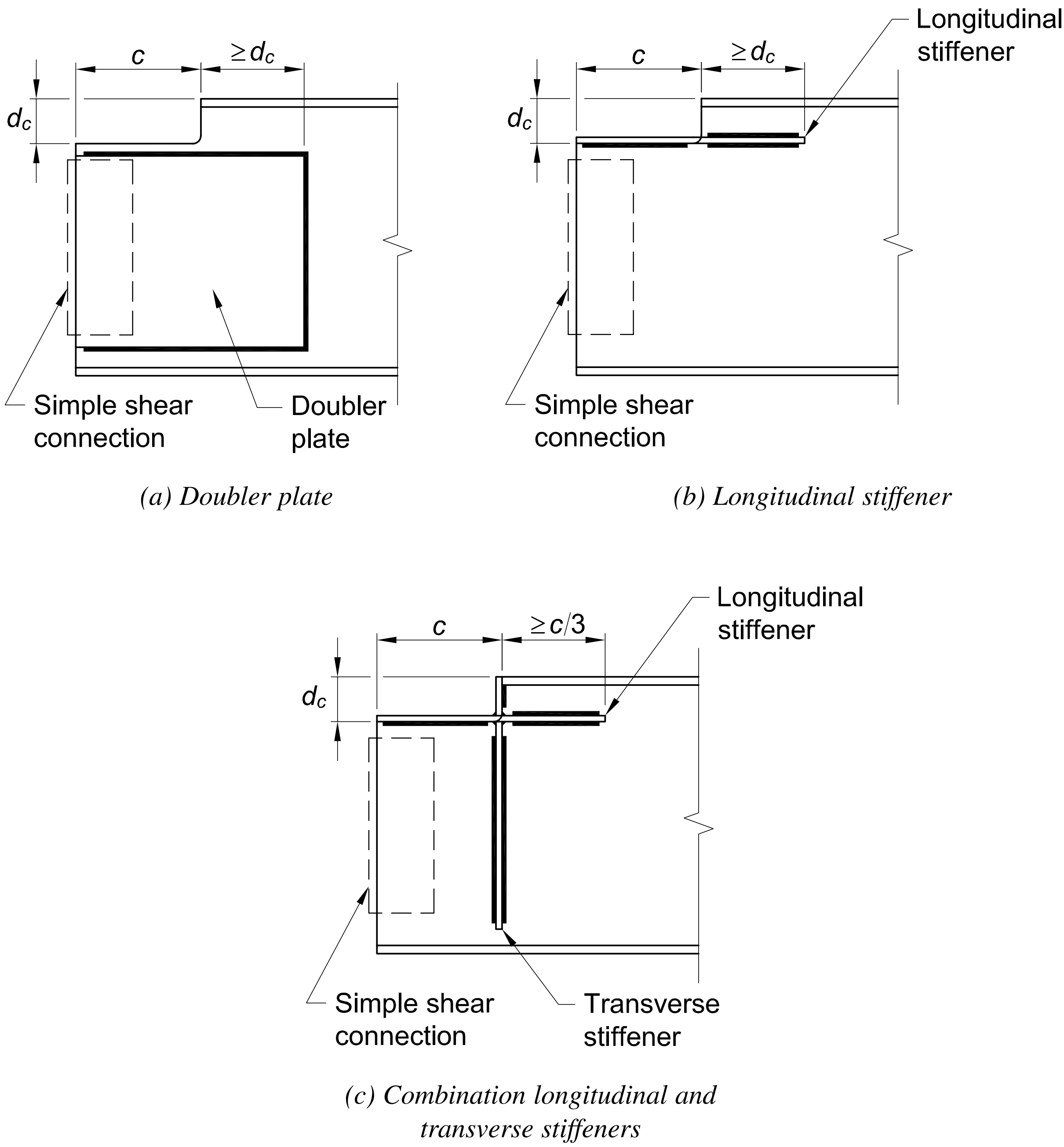


Fig. 9-11. Web reinforcement of coped beams.

$$R_1 = 2.5kF_{yw}t_w \tag{9-40}$$

$$R_2 = F_{yw}t_w \tag{9-41}$$

$$R_3 = 0.40t_w^2\sqrt{\frac{EF_{yw}t_f}{t_w}} \tag{9-42}$$

$$R_4 = 0.40t_w^2\left(\frac{3}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\sqrt{\frac{EF_{yw}t_f}{t_w}} \tag{9-43}$$

$$R_5 = 0.40t_w^2\left[1-0.2\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}} \tag{9-44}$$

$$R_6 = 0.40t_w^2\left(\frac{4}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\sqrt{\frac{EF_{yw}t_f}{t_w}} \tag{9-45}$$

Web Local Yielding

The available strength for web local yielding, ϕR_n or R_n/Ω, is determined per AISC *Specification* Section J10.2 using Equations J10-2 or J10-3, which can be simplified using the constants R_1 and R_2 from Table 9-4 as follows, where $\phi = 1.00$ and $\Omega = 1.50$.

When the compressive force to be resisted is applied at a distance, x, from the member end that is less than or equal to the depth of the member ($x \le d$):

LRFD	ASD
$\phi R_n = \phi R_1 + l_b(\phi R_2)$ (9-46a)	$R_n/\Omega = R_1/\Omega + l_b(R_2/\Omega)$ (9-46b)

When the compressive force to be resisted is applied at a distance, x, from the member end that is greater than the depth of the member ($x > d$):

LRFD	ASD
$\phi R_n = 2(\phi R_1) + l_b(\phi R_2)$ (9-47a)	$R_n/\Omega = 2(R_1/\Omega) + l_b(R_2/\Omega)$ (9-47b)

Note that the minimum length of bearing, l_b, is k, per AISC *Specification* Section J10.2 for end beam reactions, where $k = k_{des}$ for W-shapes.

Web Local Crippling

The available strength for web local crippling, ϕR_n or R_n/Ω, is determined per AISC *Specification* Section J10.3 using Equations J10-4, J10-5a or J10-5b, which can be simplified using constants R_3, R_4, R_5 and R_6 from Table 9-4 as follows, where $\phi = 0.75$ and $\Omega = 2.00$.

When the compressive force to be resisted is applied at a distance, x, from the member end that is less than one-half of the depth of the member ($x < d/2$):

For $l_b/d \le 0.2$

LRFD	ASD
$\phi R_n = \phi R_3 + l_b(\phi R_4)$ (9-48a)	$R_n/\Omega = R_3/\Omega + l_b(R_4/\Omega)$ (9-48b)

For $l_b/d > 0.2$

LRFD	ASD
$\phi R_n = \phi R_5 + l_b(\phi R_6)$ (9-49a)	$R_n/\Omega = R_5/\Omega + l_b(R_6/\Omega)$ (9-49b)

When the compressive force to be resisted is applied at a distance, x, from the member end that is greater than or equal to one-half of the depth of the member ($x \ge d/2$):

LRFD	ASD
$\phi R_n = 2\left[\phi R_3 + l_b(\phi R_4)\right]$ (9-50a)	$R_n/\Omega = 2\left[R_3/\Omega + l_b(R_4/\Omega)\right]$ (9-50b)

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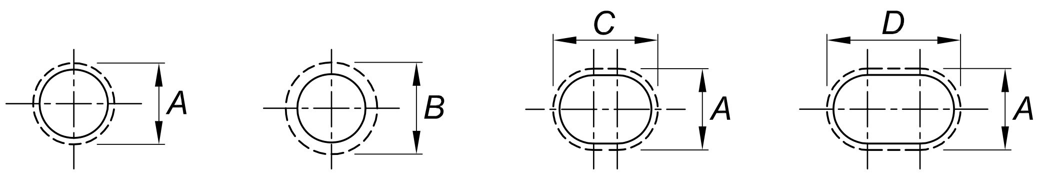
Table 9-1
Reduction in Area for Holes, in.^{2}

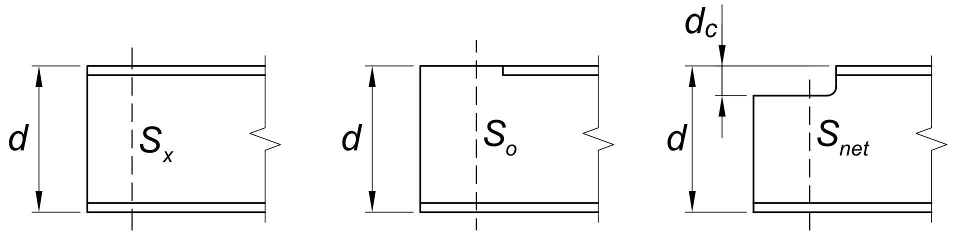


STD Standard hole | OVS Oversized hole | SSL Short-Slotted hole | LSL Long-Slotted hole

Thickness, t, in.	$A \times t$							$B \times t$						
	Bolt Diameter, d, in.							Bolt Diameter, d, in.						
	3/4	7/8	1	1 1/8	1 1/4	1 3/8	1 1/2	3/4	7/8	1	1 1/8	1 1/4	1 3/8	1 1/2
3/16	0.164	0.188	0.223	0.246	0.270	0.293	0.316	0.188	0.211	0.246	0.281	0.305	0.328	0.352
1/4	0.219	0.250	0.297	0.328	0.359	0.391	0.422	0.250	0.281	0.328	0.375	0.406	0.438	0.469
5/16	0.273	0.313	0.371	0.410	0.449	0.488	0.527	0.313	0.352	0.410	0.469	0.508	0.547	0.586
3/8	0.328	0.375	0.445	0.492	0.539	0.586	0.633	0.375	0.422	0.492	0.563	0.609	0.656	0.703
7/16	0.383	0.438	0.520	0.574	0.629	0.684	0.738	0.438	0.492	0.574	0.656	0.711	0.766	0.820
1/2	0.438	0.500	0.594	0.656	0.719	0.781	0.844	0.500	0.563	0.656	0.750	0.813	0.875	0.938
9/16	0.492	0.563	0.668	0.738	0.809	0.879	0.949	0.563	0.633	0.738	0.844	0.914	0.984	1.05
5/8	0.547	0.625	0.742	0.820	0.898	0.977	1.05	0.625	0.703	0.820	0.938	1.02	1.09	1.17
11/16	0.602	0.688	0.816	0.902	0.988	1.07	1.16	0.688	0.773	0.902	1.03	1.12	1.20	1.29
3/4	0.656	0.750	0.891	0.984	1.08	1.17	1.27	0.750	0.844	0.984	1.13	1.22	1.31	1.41
13/16	0.711	0.813	0.965	1.07	1.17	1.27	1.37	0.813	0.914	1.07	1.22	1.32	1.42	1.52
7/8	0.766	0.875	1.04	1.15	1.26	1.37	1.48	0.875	0.984	1.15	1.31	1.42	1.53	1.64
15/16	0.820	0.938	1.11	1.23	1.35	1.46	1.58	0.938	1.05	1.23	1.41	1.52	1.64	1.76
1	0.875	1.00	1.19	1.31	1.44	1.56	1.69	1.00	1.13	1.31	1.50	1.63	1.75	1.88

Thickness, t, in.	$C \times t$							$D \times t$						
	Bolt Diameter, d, in.							Bolt Diameter, d, in.						
	3/4	7/8	1	1 1/8	1 1/4	1 3/8	1 1/2	3/4	7/8	1	1 1/8	1 1/4	1 3/8	1 1/2
3/16	0.199	0.223	0.258	0.293	0.316	0.340	0.363	0.363	0.422	0.480	0.539	0.598	0.656	0.715
1/4	0.266	0.297	0.344	0.391	0.422	0.453	0.484	0.484	0.563	0.641	0.719	0.797	0.875	0.953
5/16	0.332	0.371	0.430	0.488	0.527	0.566	0.605	0.605	0.703	0.801	0.898	0.996	1.09	1.19
3/8	0.398	0.445	0.516	0.586	0.633	0.680	0.727	0.727	0.844	0.961	1.08	1.20	1.31	1.43
7/16	0.465	0.520	0.602	0.684	0.738	0.793	0.848	0.848	0.984	1.12	1.26	1.39	1.53	1.67
1/2	0.531	0.594	0.688	0.781	0.844	0.906	0.969	0.969	1.13	1.28	1.44	1.59	1.75	1.91
9/16	0.598	0.668	0.773	0.879	0.949	1.02	1.09	1.09	1.27	1.44	1.62	1.79	1.97	2.14
5/8	0.664	0.742	0.859	0.977	1.05	1.13	1.21	1.21	1.41	1.60	1.80	1.99	2.19	2.38
11/16	0.730	0.816	0.945	1.07	1.16	1.25	1.33	1.33	1.55	1.76	1.98	2.19	2.41	2.62
3/4	0.797	0.891	1.03	1.17	1.27	1.36	1.45	1.45	1.69	1.92	2.16	2.39	2.63	2.86
13/16	0.863	0.965	1.12	1.27	1.37	1.47	1.57	1.57	1.83	2.08	2.34	2.59	2.84	3.10
7/8	0.930	1.04	1.20	1.37	1.48	1.59	1.70	1.70	1.97	2.24	2.52	2.79	3.06	3.34
15/16	0.996	1.11	1.29	1.46	1.58	1.70	1.82	1.82	2.11	2.40	2.70	2.99	3.28	3.57
1	1.06	1.19	1.38	1.56	1.69	1.81	1.94	1.94	2.25	2.56	2.88	3.19	3.50	3.81

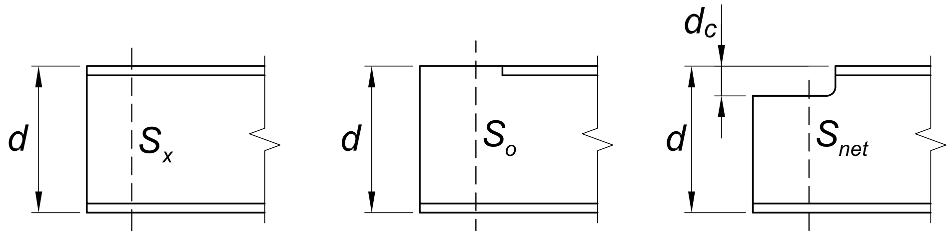
Table 9-2
Elastic Section Modulus for Coped W-Shapes



Shape	d, in.	t_f, in.	S_x, in.^{3}	S_o, in.^{3}	S_{net}, in.^{3}								
					d_c, in.								
					2	3	4	5	6	7	8	9	10
W44×335	44.0	1.77	1410	494	453	433	413	394	375	357	339	321	304
×290	43.6	1.58	1240	415	380	363	346	330	314	298	283	268	254
×262	43.3	1.42	1110	372	340	325	310	295	281	267	253	240	227
×230	42.9	1.22	971	330	301	288	274	261	249	236	224	212	200
W40×655	43.6	3.54	2590	910	–	–	757	720	685	650	616	583	550
×593	43.0	3.23	2340	810	–	–	671	639	607	575	545	515	486
×503	42.1	2.76	1980	671	–	582	554	527	500	473	448	423	398
×431	41.3	2.36	1690	567	–	491	467	444	421	398	376	355	334
×397	41.0	2.20	1560	512	–	444	422	400	379	359	339	319	300
×372	40.6	2.05	1460	480	–	415	394	374	354	335	316	298	280
×362	40.6	2.01	1420	463	–	400	380	361	342	323	305	287	270
×324	40.2	1.81	1280	408	371	352	335	317	300	284	268	252	237
×297	39.8	1.65	1170	374	339	323	306	290	275	259	245	230	216
×277	39.7	1.58	1100	335	304	289	274	260	246	232	219	206	193
×249	39.4	1.42	993	299	271	258	245	232	219	207	195	183	172
×215	39.0	1.22	859	256	231	220	208	197	186	176	166	156	146
×199	38.7	1.07	770	247	224	213	202	191	180	170	160	150	141
W40×392	41.6	2.52	1440	579	–	503	478	454	431	408	386	364	343
×331	40.8	2.13	1210	483	–	419	398	378	358	339	320	302	284
×327	40.8	2.13	1200	470	–	407	387	367	348	329	311	293	276
×294	40.4	1.93	1080	417	379	360	342	325	308	291	275	259	243
×278	40.2	1.81	1020	397	361	344	326	310	293	277	262	246	232
×264	40.0	1.73	971	371	337	321	305	289	274	259	244	230	216
×235	39.7	1.58	875	320	291	276	262	249	235	222	210	197	185
×211	39.4	1.42	786	286	259	246	234	221	209	198	186	175	165
×183	39.0	1.20	675	243	221	210	199	188	178	168	158	149	140
×167	38.6	1.03	600	234	212	201	191	181	171	161	152	143	134
×149	38.2	0.830	513	217	196	186	177	167	158	149	140	132	123

– Indicates that cope depth is less than flange thickness.

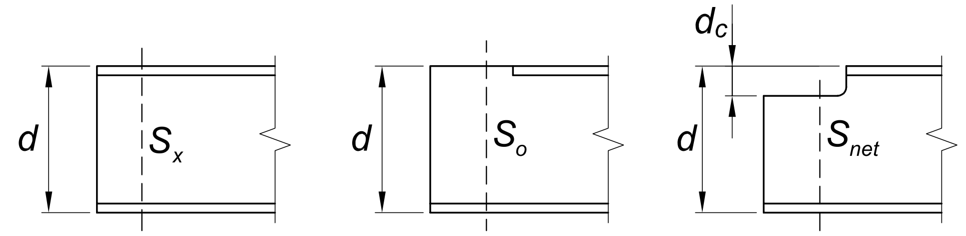
Table 9-2 (continued)
Elastic Section Modulus for Coped W-Shapes



Shape	d, in.	t_f, in.	S_x, in.^{3}	S_o, in.^{3}	S_{net}, in.^{3}								
					d_c, in.								
					2	3	4	5	6	7	8	9	10
W36×925	43.1	4.53	3390	1320	–	–	–	1040	984	933	883	835	788
×853	43.1	4.53	3250	1130	–	–	–	887	842	799	756	714	673
×802	42.6	4.29	3040	1050	–	–	–	820	778	737	697	658	620
×723	41.8	3.90	2740	925	–	–	761	723	685	648	612	577	543
×652	41.1	3.54	2460	816	–	–	669	635	601	568	536	505	475
×529	39.8	2.91	1990	636	–	547	519	491	464	438	413	388	364
×487	39.3	2.68	1830	581	–	499	473	448	423	399	375	352	330
×441	38.9	2.44	1650	518	–	444	420	398	375	354	332	312	292
×395	38.4	2.20	1490	457	–	391	370	350	330	311	292	274	256
×361	38.0	2.01	1350	412	–	352	333	315	297	279	262	246	230
×330	37.7	1.85	1240	371	335	317	300	283	267	251	235	220	206
×302	37.3	1.68	1130	338	305	289	273	258	243	228	214	200	187
×282	37.1	1.57	1050	314	283	268	253	239	225	211	198	185	173
×262	36.9	1.44	972	294	264	250	236	223	210	197	185	172	161
×247	36.7	1.35	913	277	249	236	223	210	198	185	174	162	151
×231	36.5	1.26	854	260	234	222	209	197	186	174	163	152	142
W36×256	37.4	1.73	895	329	297	281	266	251	237	223	209	196	183
×232	37.1	1.57	809	295	266	251	238	224	211	199	186	174	163
×210	36.7	1.36	719	272	245	232	219	207	195	183	172	161	150
×194	36.5	1.26	664	249	224	212	201	189	178	167	157	146	137
×182	36.3	1.18	623	234	211	199	188	178	167	157	147	137	128
×170	36.2	1.10	581	218	196	185	175	165	155	146	137	128	119
×160	36.0	1.02	542	206	185	175	165	156	147	138	129	120	112
×150	35.9	0.940	504	195	176	166	157	148	139	130	122	114	106
×135	35.6	0.790	439	181	163	154	145	137	129	121	113	105	98.1
W33×387	36.0	2.28	1350	413	–	349	329	310	291	272	254	237	220
×354	35.6	2.09	1240	373	–	315	297	279	262	245	229	213	198
×318	35.2	1.89	1110	330	295	278	262	246	230	216	201	187	173
×291	34.8	1.73	1020	300	268	253	238	223	209	195	182	169	157
×263	34.5	1.57	919	268	239	226	212	199	186	174	162	151	139
×241	34.2	1.40	831	250	223	210	197	185	173	162	150	140	129
×221	33.9	1.28	759	230	205	193	181	170	159	148	138	128	118
×201	33.7	1.15	686	209	186	175	165	154	144	135	125	116	107

– Indicates that cope depth is less than flange thickness.

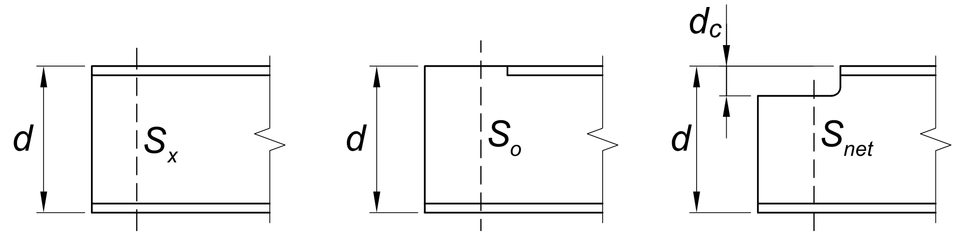
Table 9-2 (continued)
Elastic Section Modulus for Coped W-Shapes



Shape	d, in.	t_f, in.	S_x, in.^{3}	S_o, in.^{3}	S_{net}, in.^{3}								
					d_c, in.								
					2	3	4	5	6	7	8	9	10
W33×169	33.8	1.22	549	191	170	161	151	141	132	124	115	107	98.6
×152	33.5	1.06	487	176	157	148	139	130	122	114	106	97.9	90.5
×141	33.3	0.960	448	165	147	139	130	122	114	106	98.8	91.6	84.6
×130	33.1	0.855	406	155	138	130	122	114	107	99.6	92.5	85.7	79.2
×118	32.9	0.740	359	143	128	120	113	106	98.6	91.9	85.4	79.1	73.0
W30×391	33.2	2.44	1250	378	–	315	295	276	257	239	222	205	188
×357	32.8	2.24	1140	339	–	282	264	246	230	213	197	182	167
×326	32.4	2.05	1040	305	–	254	237	221	206	191	177	163	150
×292	32.0	1.85	930	269	238	223	208	194	180	167	155	142	130
×261	31.6	1.65	829	240	212	198	185	172	160	148	137	126	115
×235	31.3	1.50	748	211	186	174	163	152	141	130	120	110	101
×211	30.9	1.32	665	192	170	159	148	138	128	118	109	99.8	91.2
×191	30.7	1.19	600	174	153	143	133	124	115	106	97.7	89.6	81.8
×173	30.4	1.07	541	158	139	130	121	112	104	96.1	88.4	81.0	73.9
W30×148	30.7	1.18	436	152	134	125	117	109	101	93.3	86.0	78.9	72.1
×132	30.3	1.00	380	139	123	115	107	99.3	92.1	85.1	78.3	71.8	65.5
×124	30.2	0.930	355	131	115	108	100	93.4	86.5	79.9	73.6	67.4	61.5
×116	30.0	0.850	329	124	109	102	95.3	88.6	82.1	75.8	69.7	63.9	58.2
×108	29.8	0.760	299	118	103	96.5	89.9	83.6	77.4	71.4	65.7	60.1	54.8
×99	29.7	0.670	269	110	96.4	90.0	83.9	77.9	72.1	66.5	61.1	56.0	51.0
×90	29.5	0.610	245	98.7	86.7	80.9	75.4	70.0	64.8	59.7	54.9	50.2	45.7
W27×539	32.5	3.54	1570	509	–	–	394	367	341	316	292	269	247
×368	30.4	2.48	1060	321	–	262	244	226	209	193	177	162	147
×336	30.0	2.28	972	287	–	234	218	202	186	172	157	143	130
×307	29.6	2.09	887	259	–	211	196	181	167	154	141	128	116
×281	29.3	1.93	814	233	203	189	176	162	150	137	126	114	104
×258	29.0	1.77	745	212	185	172	159	147	136	124	114	103	93.3
×235	28.7	1.61	677	193	168	156	145	134	123	113	103	93.2	84.2
×217	28.4	1.50	627	174	152	141	130	120	111	101	92.3	83.7	75.5
×194	28.1	1.34	559	155	134	125	115	106	97.6	89.3	81.3	73.6	66.3
×178	27.8	1.19	505	145	126	117	108	99.7	91.5	83.6	76.1	68.8	61.9
×161	27.6	1.08	458	131	113	105	97.2	89.5	82.0	74.9	68.1	61.5	55.3
×146	27.4	0.975	414	118	102	95.0	87.7	80.7	74.0	67.5	61.3	55.3	49.7

– Indicates that cope depth is less than flange thickness.

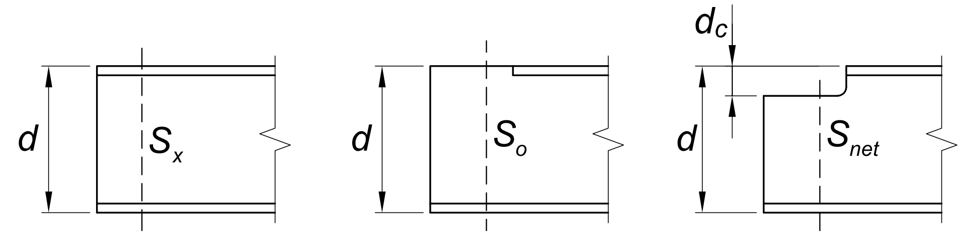
Table 9-2 (continued)
Elastic Section Modulus for Coped W-Shapes



Shape	d, in.	t_f, in.	S_x, in.^{3}	S_o, in.^{3}	S_{net}, in.^{3}								
					d_c, in.								
					2	3	4	5	6	7	8	9	10
W27×129	27.6	1.10	345	117	101	94.0	86.9	80.1	73.5	67.2	61.1	55.3	49.7
×114	27.3	0.930	299	106	91.6	84.9	78.4	72.2	66.2	60.5	54.9	49.6	44.6
×102	27.1	0.830	267	94.2	81.6	75.6	69.8	64.2	58.9	53.7	48.8	44.0	39.5
×94	26.9	0.745	243	88.0	76.2	70.6	65.1	59.9	54.9	50.1	45.4	41.0	36.8
×84	26.7	0.640	213	80.5	69.7	64.5	59.5	54.7	50.1	45.7	41.4	37.4	33.5
W24×370	28.0	2.72	957	295	–	237	219	201	184	168	153	138	124
×335	27.5	2.48	864	261	–	209	193	177	162	147	133	120	108
×306	27.1	2.28	789	234	–	186	172	157	144	131	118	106	94.9
×279	26.7	2.09	718	210	–	167	154	141	128	116	105	94.3	84.0
×250	26.3	1.89	644	184	158	146	134	123	112	101	91.2	81.7	72.6
×229	26.0	1.73	588	167	143	132	121	111	101	91.0	81.8	73.1	64.9
×207	25.7	1.57	531	149	127	117	107	98.0	89.0	80.4	72.2	64.4	57.0
×192	25.5	1.46	491	136	117	107	98.2	89.5	81.2	73.3	65.8	58.6	51.8
×176	25.2	1.34	450	124	106	97.6	89.4	81.4	73.8	66.5	59.6	53.0	46.8
×162	25.0	1.22	414	115	98.0	90.0	82.3	74.9	67.9	61.1	54.7	48.6	42.8
×146	24.7	1.09	371	104	88.5	81.2	74.2	67.5	61.1	54.9	49.1	43.6	38.3
×131	24.5	0.960	329	94.4	80.3	73.7	67.3	61.1	55.3	49.7	44.3	39.3	34.5
×117	24.3	0.850	291	84.4	71.7	65.7	60.0	54.5	49.2	44.2	39.4	34.8	30.5
×104	24.1	0.750	258	75.4	64.1	58.7	53.5	48.6	43.8	39.3	35.0	30.9	27.1
W24×103	24.5	0.980	245	82.9	70.7	64.9	59.3	53.9	48.8	43.9	39.2	34.8	30.6
×94	24.3	0.875	222	76.2	64.9	59.5	54.3	49.4	44.6	40.1	35.8	31.7	27.9
×84	24.1	0.770	196	68.3	58.0	53.2	48.6	44.1	39.8	35.8	31.9	28.2	24.8
×76	23.9	0.680	176	62.6	53.2	48.7	44.5	40.4	36.4	32.7	29.1	25.8	22.6
×68	23.7	0.585	154	57.5	48.8	44.7	40.8	37.0	33.4	29.9	26.6	23.5	20.6
W24×62	23.7	0.590	131	56.9	48.3	44.3	40.4	36.7	33.1	29.7	26.5	23.4	20.5
×55	23.6	0.505	114	51.1	43.4	39.7	36.2	32.9	29.7	26.6	23.7	20.9	18.3
W21×275	24.1	2.19	638	179	–	138	126	114	102	91.4	81.1	71.4	62.2
×248	23.7	1.99	576	158	133	121	110	99.3	89.1	79.5	70.3	61.6	53.5
×223	23.4	1.79	520	141	118	108	97.7	88.1	79.0	70.3	62.0	54.3	47.0
×201	23.0	1.63	461	125	105	95.2	86.2	77.6	69.4	61.6	54.2	47.3	40.8
×182	22.7	1.48	417	111	93.3	84.8	76.6	68.8	61.4	54.4	47.8	41.6	35.8
×166	22.5	1.36	380	99.3	83.0	75.3	68.0	61.0	54.4	48.1	42.2	36.6	31.4
×147	22.1	1.15	329	91.2	76.1	68.9	62.1	55.7	49.5	43.7	38.2	33.1	28.2
×132	21.8	1.04	295	81.0	67.5	61.1	55.0	49.2	43.7	38.5	33.6	29.0	24.7
×122	21.7	0.960	273	74.1	61.6	55.7	50.2	44.8	39.8	35.0	30.5	26.3	22.4
×111	21.5	0.875	249	67.1	55.7	50.4	45.3	40.4	35.9	31.5	27.4	23.6	20.1
×101	21.4	0.800	227	60.4	50.1	45.3	40.7	36.3	32.1	28.2	24.5	21.1	17.9

– Indicates that cope depth is less than flange thickness.

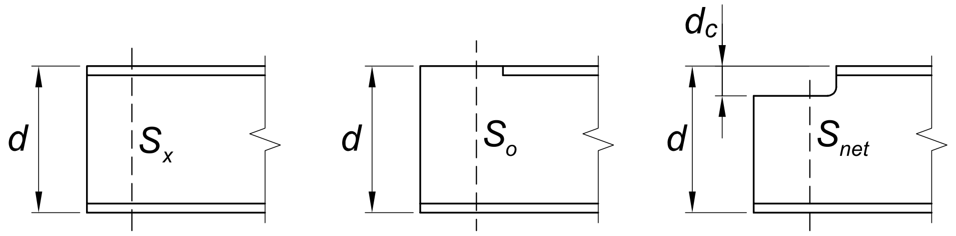
Table 9-2 (continued)
Elastic Section Modulus for Coped W-Shapes



Shape	d, in.	t_f, in.	S_x, in.^{3}	S_o, in.^{3}	S_{net}, in.^{3}								
					d_c, in.								
					2	3	4	5	6	7	8	9	10
W21×93	21.6	0.930	192	67.2	56.0	50.7	45.7	40.9	36.3	32.0	27.9	24.1	20.5
×83	21.4	0.835	171	59.0	49.1	44.4	40.0	35.7	31.7	27.9	24.3	20.9	17.8
×73	21.2	0.740	151	51.5	42.7	38.7	34.8	31.0	27.5	24.2	21.0	18.1	15.3
×68	21.1	0.685	140	48.1	39.9	36.1	32.4	29.0	25.6	22.5	19.6	16.8	14.2
×62	21.0	0.615	127	44.1	36.5	33.0	29.7	26.5	23.4	20.5	17.8	15.3	12.9
×55	20.8	0.522	110	40.1	33.2	30.0	26.9	24.0	21.2	18.6	16.1	13.8	11.7
×48	20.6	0.430	93.0	36.2	30.0	27.0	24.2	21.6	19.1	16.7	14.5	12.4	10.4
W21×57	21.1	0.650	111	43.4	36.1	32.6	29.3	26.2	23.2	20.4	17.7	15.2	12.9
×50	20.8	0.535	94.5	39.2	32.5	29.4	26.4	23.6	20.8	18.3	15.9	13.6	11.5
×44	20.7	0.450	81.6	35.2	29.1	26.3	23.6	21.0	18.6	16.3	14.1	12.1	10.2
W18×311	22.3	2.74	624	186	–	140	126	113	100	88.2	77.0	66.5	56.8
×283	21.9	2.50	565	166	–	124	111	99.3	87.8	77.1	67.0	57.6	48.9
×258	21.5	2.30	514	148	–	110	98.3	87.4	77.2	67.5	58.5	50.0	42.3
×234	21.1	2.11	466	130	–	96.1	85.9	76.2	67.1	58.5	50.4	43.0	36.1
×211	20.7	1.91	419	115	94.5	84.8	75.6	66.9	58.7	51.0	43.8	37.1	31.0
×192	20.4	1.75	380	102	83.4	74.7	66.5	58.7	51.4	44.5	38.1	32.1	26.7
×175	20.0	1.59	344	92.1	75.1	67.2	59.7	52.6	45.9	39.6	33.8	28.4	23.5
×158	19.7	1.44	310	81.7	66.4	59.3	52.6	46.2	40.2	34.6	29.4	24.6	
×143	19.5	1.32	282	72.5	58.8	52.4	46.4	40.7	35.4	30.4	25.7	21.5	
×130	19.3	1.20	256	65.2	52.8	47.0	41.5	36.4	31.5	27.0	22.8	19.0	
×119	19.0	1.06	231	61.7	49.8	44.3	39.1	34.2	29.5	25.2	21.2	17.6	
×106	18.7	0.940	204	54.4	43.8	38.9	34.3	29.9	25.8	22.0	18.5	15.2	
×97	18.6	0.870	188	48.9	39.3	34.9	30.7	26.8	23.1	19.6	16.4	13.5	
×86	18.4	0.770	166	43.1	34.6	30.6	26.9	23.4	20.2	17.1	14.3	11.7	
×76	18.2	0.680	146	37.6	30.1	26.7	23.4	20.3	17.5	14.8	12.3	10.1	
W18×71	18.5	0.810	127	42.4	34.1	30.3	26.7	23.3	20.1	17.1	14.3	11.8	
×65	18.4	0.750	117	38.3	30.8	27.3	24.0	20.9	18.0	15.3	12.8	10.5	
×60	18.2	0.695	108	35.0	28.1	24.9	21.9	19.1	16.4	13.9	11.6	9.53	
×55	18.1	0.630	98.3	32.4	26.0	23.0	20.2	17.6	15.1	12.8	10.7	8.72	
×50	18.0	0.570	88.9	29.1	23.4	20.7	18.2	15.8	13.5	11.5	9.54		
W18×46	18.1	0.605	78.8	28.9	23.2	20.6	18.1	15.7	13.5	11.5	9.56	7.81	
×40	17.9	0.525	68.4	24.9	20.0	17.7	15.5	13.5	11.6	9.80	8.16		
×35	17.7	0.425	57.6	22.7	18.2	16.1	14.1	12.3	10.5	8.88	7.37		

– Indicates that cope depth is less than flange thickness.
Note: Values are omitted when cope depth exceeds d/2.

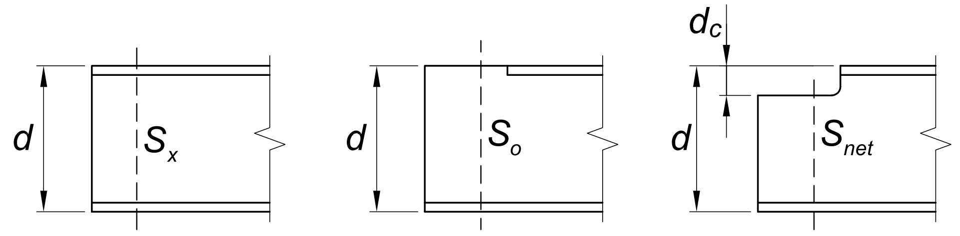
Table 9-2 (continued)
Elastic Section Modulus for Coped W-Shapes



Shape	d, in.	t_f, in.	S_x, in.^{3}	S_o, in.^{3}	S_{net}, in.^{3}								
					d_c, in.								
					2	3	4	5	6	7	8	9	10
W16×100	17.0	0.985	175	44.4	34.9	30.5	26.4	22.6	19.0	15.7	12.8		
×89	16.8	0.875	155	39.0	30.6	26.7	23.1	19.7	16.5	13.6	11.0		
×77	16.5	0.760	134	33.1	25.9	22.6	19.4	16.5	13.8	11.4	9.13		
×67	16.3	0.665	117	28.3	22.1	19.2	16.5	14.0	11.7	9.58	7.66		
W16×57	16.4	0.715	92.2	29.4	23.0	20.1	17.3	14.8	12.4	10.2	8.17		
×50	16.3	0.630	81.0	25.6	20.0	17.4	15.0	12.7	10.7	8.74	6.99		
×45	16.1	0.565	72.7	22.9	17.9	15.5	13.4	11.3	9.47	7.75	6.19		
×40	16.0	0.505	64.7	20.1	15.6	13.6	11.7	9.89	8.24	6.73	5.35		
×36	15.9	0.430	56.5	18.8	14.6	12.7	10.9	9.21	7.67	6.25			
W16×31	15.9	0.440	47.2	17.1	13.3	11.6	9.96	8.44	7.03	5.73			
×26	15.7	0.345	38.4	14.9	11.6	10.1	8.64	7.31	6.08	4.95			
W14×873	23.6	5.51	1530	504	–	–	–	–	279	248	220	193	169
×808	22.8	5.12	1390	450	–	–	–	–	243	215	189	165	143
×730	22.4	4.91	1280	365	–	–	–	220	195	172	151	132	76.2
×665	21.6	4.52	1150	317	–	–	–	187	165	144	126	109	93.3
×605	20.9	4.16	1040	275	–	–	–	158	139	121	105	89.6	76.2
×550	20.2	3.82	931	238	–	–	153	134	117	101	86.9	73.8	62.1
×500	19.6	3.50	838	208	–	–	131	115	99.4	85.3	72.5	60.9	
×455	19.0	3.21	756	182	–	–	113	98.2	84.6	72.1	60.7	50.6	
×426	18.7	3.04	706	164	–	–	101	87.6	75.2	63.8	53.4	44.2	
×398	18.3	2.85	656	150	–	104	91.1	78.7	67.2	56.7	47.2	38.7	
×370	17.9	2.66	607	135	–	93.7	81.4	70.1	59.6	50.0	41.3		
×342	17.5	2.47	558	122	–	83.4	72.3	61.9	52.3	43.6	35.8		
×311	17.1	2.26	506	107	–	72.7	62.7	53.5	44.9	37.2	30.2		
×283	16.7	2.07	459	94.4	–	63.6	54.6	46.3	38.7	31.8	25.6		
×257	16.4	1.89	415	83.1	64.1	55.5	47.4	40.0	33.3	27.1	21.6		
×233	16.0	1.72	375	73.2	56.1	48.4	41.3	34.6	28.6	23.2	18.3		
×211	15.7	1.56	338	64.9	49.5	42.6	36.1	30.2	24.8	19.9			
×193	15.5	1.44	310	57.6	43.8	37.5	31.7	26.4	21.6	17.3			
×176	15.2	1.31	281	52.2	39.5	33.8	28.5	23.6	19.2	15.2			
×159	15.0	1.19	254	45.7	34.5	29.4	24.7	20.4	16.5	13.0			
×145	14.8	1.09	232	40.9	30.7	26.1	21.9	18.0	14.5	11.4			

– Indicates that cope depth is less than flange thickness.
Note: Values are omitted when cope depth exceeds d/2.

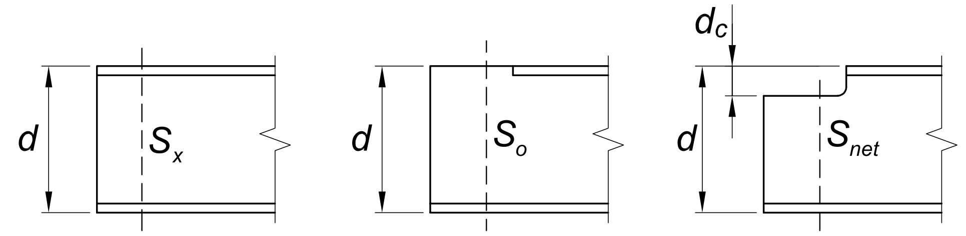
Table 9-2 (continued)
Elastic Section Modulus for Coped W-Shapes



Shape	d, in.	t_f, in.	S_x, in.^{3}	S_o, in.^{3}	S_{net}, in.^{3}								
					d_c, in.								
					2	3	4	5	6	7	8	9	10
W14×132	14.7	1.03	209	38.1	28.6	24.3	20.3	16.7	13.4	10.5			
×120	14.5	0.940	190	34.2	25.5	21.7	18.1	14.8	11.8	9.20			
×109	14.3	0.860	173	30.0	22.3	18.9	15.7	12.8	10.2	7.91			
×99	14.2	0.780	157	27.2	20.2	17.0	14.2	11.5	9.15	7.04			
×90	14.0	0.710	143	24.3	18.0	15.2	12.6	10.2	8.07	6.18			
W14×82	14.3	0.855	123	28.0	20.9	17.7	14.8	12.1	9.64	7.46			
×74	14.2	0.785	112	24.4	18.2	15.4	12.8	10.4	8.31	6.40			
×68	14.0	0.720	103	22.2	16.5	13.9	11.6	9.41	7.46	5.72			
×61	13.9	0.645	92.1	19.7	14.6	12.3	10.2	8.28	6.54				
W14×53	13.9	0.660	77.8	19.1	14.2	12.0	9.93	8.07	6.39				
×48	13.8	0.595	70.2	17.3	12.8	10.8	8.93	7.23	5.71				
×43	13.7	0.530	62.6	15.3	11.3	9.49	7.84	6.34	4.99				
W14×38	14.1	0.515	54.6	16.0	12.0	10.2	8.48	6.94	5.54	4.28			
×34	14.0	0.455	48.6	14.4	10.8	9.14	7.62	6.22	4.95				
×30	13.8	0.385	42.0	13.2	9.88	8.37	6.96	5.68	4.51				
W14×26	13.9	0.420	35.3	12.3	9.20	7.80	6.50	5.31	4.23				
×22	13.7	0.335	29.0	10.7	7.97	6.75	5.62	4.58	3.64				
W12×336	16.8	2.96	483	123	–	83.1	71.4	60.6	50.8	41.9	34.1		
×305	16.3	2.71	435	108	–	71.4	61.0	51.4	42.7	34.9	28.0		
×279	15.9	2.47	393	96.1	–	63.1	53.5	44.8	36.9	29.8			
×252	15.4	2.25	353	83.7	–	54.2	45.7	38.0	31.0	24.8			
×230	15.1	2.07	321	74.2	–	47.5	39.9	32.9	26.7	21.1			
×210	14.7	1.90	292	65.6	49.0	41.6	34.7	28.5	22.9	17.9			
×190	14.4	1.74	263	57.0	42.3	35.7	29.7	24.2	19.3	14.9			
×170	14.0	1.56	235	49.6	36.5	30.7	25.3	20.5	16.2	12.4			
×152	13.7	1.40	209	43.3	31.6	26.5	21.7	17.5	13.7				
×136	13.4	1.25	186	37.9	27.5	22.9	18.7	14.9	11.6				
×120	13.1	1.11	163	32.8	23.7	19.7	16.0	12.6	9.70				
×106	12.9	0.990	145	27.6	19.8	16.3	13.2	10.4	7.91				
×96	12.7	0.900	131	24.3	17.4	14.3	11.5	9.03	6.83				
×87	12.5	0.810	118	22.2	15.8	13.0	10.4	8.11	6.09				
×79	12.4	0.735	107	19.9	14.1	11.5	9.23	7.16	5.35				
×72	12.3	0.670	97.4	17.9	12.6	10.3	8.24	6.37	4.73				
×65	12.1	0.605	87.9	16.0	11.2	9.16	7.28	5.61	4.14				

– Indicates that cope depth is less than flange thickness.
Note: Values are omitted when cope depth exceeds $d/2$.

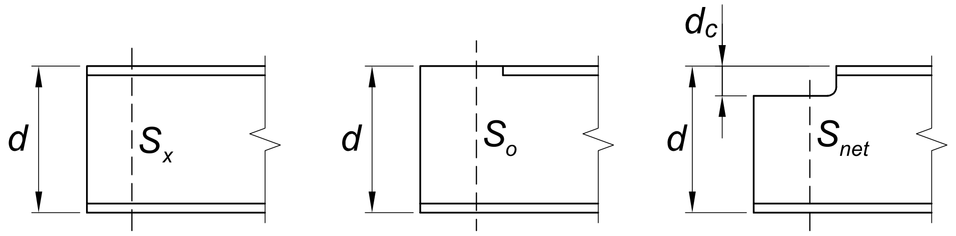
Table 9-2 (continued)
Elastic Section Modulus for Coped W-Shapes



Shape	d, in.	t_f, in.	S_x, in.^{3}	S_o, in.^{3}	S_{net}, in.^{3}								
					d_c, in.								
					2	3	4	5	6	7	8	9	10
W12×58	12.2	0.640	78.0	14.8	10.4	8.52	6.79	5.24	3.88				
×53	12.1	0.575	70.6	13.9	9.75	7.94	6.31	4.85	3.58				
W12×50	12.2	0.640	64.2	14.8	10.4	8.54	6.82	5.27	3.91				
×45	12.1	0.575	57.7	13.1	9.27	7.56	6.02	4.63	3.42				
×40	11.9	0.515	51.5	11.4	8.03	6.54	5.19	3.98					
W12×35	12.5	0.520	45.6	12.3	8.85	7.30	5.89	4.61	3.48				
×30	12.3	0.440	38.6	10.5	7.47	6.15	4.94	3.86	2.90				
×26	12.2	0.380	33.4	9.08	6.47	5.32	4.27	3.32	2.48				
W12×22	12.3	0.425	25.4	9.60	6.89	5.69	4.59	3.59	2.71				
×19	12.2	0.350	21.3	8.39	6.01	4.95	3.98	3.11	2.33				
×16	12.0	0.265	17.1	7.43	5.30	4.36	3.50	2.72					
×14	11.9	0.225	14.9	6.61	4.71	3.86	3.10	2.41					
W10×112	11.4	1.25	126	25.7	17.5	13.9	10.8	8.02					
×100	11.1	1.12	112	22.3	15.0	11.9	9.12	6.72					
×88	10.8	0.990	98.5	19.1	12.8	10.0	7.62	5.54					
×77	10.6	0.870	85.9	16.2	10.7	8.35	6.29	4.52					
×68	10.4	0.770	75.7	13.9	9.13	7.10	5.30	3.77					
×60	10.2	0.680	66.7	12.1	7.88	6.09	4.52	3.18					
×54	10.1	0.615	60.0	10.5	6.78	5.22	3.85	2.69					
×49	10.0	0.560	54.6	9.49	6.13	4.71	3.46	2.40					
W10×45	10.1	0.620	49.1	9.75	6.33	4.88	3.61	2.52					
×39	9.92	0.530	42.1	8.49	5.48	4.20	3.08						
×33	9.73	0.435	35.0	7.49	4.80	3.67	2.67						
W10×30	10.5	0.510	32.4	8.64	5.75	4.51	3.41	2.45					
×26	10.3	0.440	27.9	7.33	4.86	3.80	2.85	2.04					
×22	10.2	0.360	23.2	6.51	4.29	3.34	2.50	1.77					
W10×19	10.2	0.395	18.8	6.52	4.33	3.39	2.55	1.82					
×17	10.1	0.330	16.2	6.01	3.98	3.10	2.33	1.65					
×15	9.99	0.270	13.8	5.53	3.65	2.84	2.12	1.50					
×12	9.87	0.210	10.9	4.43	2.91	2.26	1.68						

Note: Values are omitted when cope depth exceeds $d/2$.

Table 9-2 (continued)
Elastic Section Modulus for Coped W-Shapes

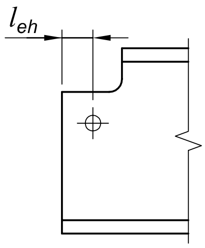


Shape	d, in.	t_f, in.	S_x, in.^{3}	S_o, in.^{3}	S_{net}, in.^{3}								
					d_c, in.								
					2	3	4	5	6	7	8	9	10
W8×67	9.00	0.935	60.4	12.2	7.42	5.44	3.77						
×58	8.75	0.810	52.0	10.4	6.24	4.52	3.08						
×48	8.50	0.685	43.2	7.89	4.63	3.32	2.21						
×40	8.25	0.560	35.5	6.71	3.89	2.74	1.80						
×35	8.12	0.495	31.2	5.66	3.24	2.28	1.47						
×31	8.00	0.435	27.5	5.06	2.88	2.01	1.28						
W8×28	8.06	0.465	24.3	5.04	2.89	2.02	1.30						
×24	7.93	0.400	20.9	4.23	2.40	1.67							
W8×21	8.28	0.400	18.2	4.55	2.67	1.91	1.26						
×18	8.14	0.330	15.2	4.02	2.35	1.66	1.09						
W8×15	8.11	0.315	11.8	4.03	2.36	1.68	1.10						
×13	7.99	0.255	9.91	3.61	2.10	1.49							
×10	7.89	0.205	7.81	2.65	1.54	1.08							

Note: Values are omitted when cope depth exceeds d/2.

U_{bs} = 1.0

Table 9-3a
Block Shear Tension Rupture Component
per inch of thickness, kip/in.

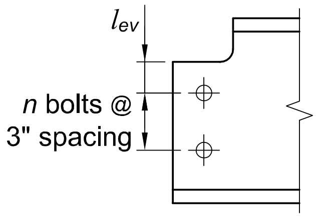


F_u	58 ksi					
l_{eh}, in.	Bolt diameter, d, in.[a]					
	3/4		7/8		1	
	$\frac{F_uA_{nt}}{\Omega t}$	$\frac{\phi F_uA_{nt}}{t}$	$\frac{F_uA_{nt}}{\Omega t}$	$\frac{\phi F_uA_{nt}}{t}$	$\frac{F_uA_{nt}}{\Omega t}$	$\frac{\phi F_uA_{nt}}{t}$
	ASD	LRFD	ASD	LRFD	ASD	LRFD
1	16.3	24.5	14.5	21.8	11.8	17.7
1⅛	19.9	29.9	18.1	27.2	15.4	23.1
1¼	23.6	35.3	21.8	32.6	19.0	28.5
1⅜	27.2	40.8	25.4	38.1	22.7	34.0
1½	30.8	46.2	29.0	43.5	26.3	39.4
1⅝	34.4	51.7	32.6	48.9	29.9	44.9
1¾	38.1	57.1	36.3	54.4	33.5	50.3
1⅞	41.7	62.5	39.9	59.8	37.2	55.7
2	45.3	68.0	43.5	65.3	40.8	61.2
2¼	52.6	78.8	50.7	76.1	48.0	72.0
2½	59.8	89.7	58.0	87.0	55.3	82.9
2¾	67.1	101	65.3	97.9	62.5	93.8
3	74.3	111	72.5	109	69.8	105
F_u	65 ksi					
l_{eh}, in.	Bolt diameter, d, in.[a]					
	3/4		7/8		1	
	$\frac{F_uA_{nt}}{\Omega t}$	$\frac{\phi F_uA_{nt}}{t}$	$\frac{F_uA_{nt}}{\Omega t}$	$\frac{\phi F_uA_{nt}}{t}$	$\frac{F_uA_{nt}}{\Omega t}$	$\frac{\phi F_uA_{nt}}{t}$
	ASD	LRFD	ASD	LRFD	ASD	LRFD
1	18.3	27.4	16.3	24.4	13.2	19.8
1⅛	22.3	33.5	20.3	30.5	17.3	25.9
1¼	26.4	39.6	24.4	36.6	21.3	32.0
1⅜	30.5	45.7	28.4	42.7	25.4	38.1
1½	34.5	51.8	32.5	48.8	29.5	44.2
1⅝	38.6	57.9	36.6	54.8	33.5	50.3
1¾	42.7	64.0	40.6	60.9	37.6	56.4
1⅞	46.7	70.1	44.7	67.0	41.6	62.5
2	50.8	76.2	48.8	73.1	45.7	68.6
2¼	58.9	88.4	56.9	85.3	53.8	80.7
2½	67.0	101	65.0	97.5	62.0	92.9
2¾	75.2	113	73.1	110	70.1	105
3	83.3	125	81.3	122	78.2	117

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

[a] Values are for standard hole types.

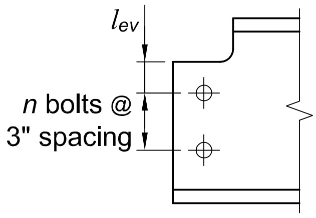
Table 9-3b
Block Shear
Shear Yielding Component
per inch of thickness, kip/in.



l_{ev}, in.	n	F_y = 36 ksi, $\frac{0.6F_yA_{gv}}{\Omega t}$ ASD	F_y = 36 ksi, $\frac{\phi 0.6F_yA_{gv}}{t}$ LRFD	F_y = 50 ksi, $\frac{0.6F_yA_{gv}}{\Omega t}$ ASD	F_y = 50 ksi, $\frac{\phi 0.6F_yA_{gv}}{t}$ LRFD	n	F_y = 36 ksi, $\frac{0.6F_yA_{gv}}{\Omega t}$ ASD	F_y = 36 ksi, $\frac{\phi 0.6F_yA_{gv}}{t}$ LRFD	F_y = 50 ksi, $\frac{0.6F_yA_{gv}}{\Omega t}$ ASD	F_y = 50 ksi, $\frac{\phi 0.6F_yA_{gv}}{t}$ LRFD
1¼		370	555	514	771		273	409	379	568
1⅜		371	557	516	773		274	411	381	571
1½		373	559	518	776		275	413	383	574
1⅝		374	561	519	779		277	415	384	577
1¾		375	563	521	782		278	417	386	579
1⅞	12	377	565	523	785	9	279	419	388	582
2		378	567	525	788		281	421	390	585
2¼		381	571	529	793		284	425	394	591
2½		383	575	533	799		286	429	398	596
2¾		386	579	536	804		289	433	401	602
3		389	583	540	810		292	437	405	608
1¼		337	506	469	703		240	360	334	501
1⅜		339	508	471	706		242	362	336	503
1½		340	510	473	709		243	364	338	506
1⅝		342	512	474	712		244	367	339	509
1¾		343	514	476	714		246	369	341	512
1⅞	11	344	516	478	717	8	247	371	343	515
2		346	518	480	720		248	373	345	518
2¼		348	522	484	726		251	377	349	523
2½		351	526	488	731		254	381	353	529
2¾		354	531	491	737		257	385	356	534
3		356	535	495	743		259	389	360	540
1¼		305	458	424	636		208	312	289	433
1⅜		306	460	426	638		209	314	291	436
1½		308	462	428	641		211	316	293	439
1⅝		309	464	429	644		212	318	294	442
1¾		310	466	431	647		213	320	296	444
1⅞	10	312	468	433	650	7	215	322	298	447
2		313	470	435	653		216	324	300	450
2¼		316	474	439	658		219	328	304	456
2½		319	478	443	664		221	332	308	461
2¾		321	482	446	669		224	336	311	467
3		324	486	450	675		227	340	315	473

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

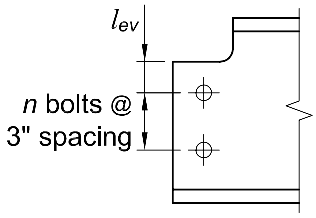
Table 9-3b (continued)
Block Shear
Shear Yielding
Component
per inch of thickness, kip/in.



l_{ev}, in.	n	F_y = 36 ksi, $\frac{0.6F_yA_{gv}}{\Omega t}$ ASD	F_y = 36 ksi, $\frac{\phi 0.6F_yA_{gv}}{t}$ LRFD	F_y = 50 ksi, $\frac{0.6F_yA_{gv}}{\Omega t}$ ASD	F_y = 50 ksi, $\frac{\phi 0.6F_yA_{gv}}{t}$ LRFD	n	F_y = 36 ksi, $\frac{0.6F_yA_{gv}}{\Omega t}$ ASD	F_y = 36 ksi, $\frac{\phi 0.6F_yA_{gv}}{t}$ LRFD	F_y = 50 ksi, $\frac{0.6F_yA_{gv}}{\Omega t}$ ASD	F_y = 50 ksi, $\frac{\phi 0.6F_yA_{gv}}{t}$ LRFD
1¼		175	263	244	366		78.3	117	109	163
1⅜		177	265	246	368		79.6	119	111	166
1½		178	267	248	371		81.0	121	113	169
1⅝		180	269	249	374		82.3	124	114	172
1¾		181	271	251	377		83.7	126	116	174
1⅞	6	182	273	253	380	3	85.0	128	118	177
2		184	275	255	383		86.4	130	120	180
2¼		186	279	259	388		89.1	134	124	186
2½		189	283	263	394		91.8	138	128	191
2¾		192	288	266	399		94.5	142	131	197
3		194	292	270	405		97.2	146	135	203
1¼		143	215	199	298		45.9	68.8	63.8	95.6
1⅜		144	217	201	301		47.2	70.9	65.6	98.4
1½		146	219	203	304		48.6	72.9	67.5	101
1⅝		147	221	204	307		49.9	74.9	69.4	104
1¾		148	223	206	309		51.3	76.9	71.3	107
1⅞	5	150	225	208	312	2	52.7	79.0	73.1	110
2		151	227	210	315		54.0	81.0	75.0	113
2¼		154	231	214	321		56.7	85.0	78.8	118
2½		157	235	218	326		59.4	89.1	82.5	124
2¾		159	239	221	332		62.1	93.1	86.3	129
3		162	243	225	338		64.8	97.2	90.0	135
1¼		111	166	154	231					
1⅜		112	168	156	233					
1½		113	170	158	236					
1⅝		115	172	159	239					
1¾		116	174	161	242					
1⅞	4	117	176	163	245					
2		119	178	165	248					
2¼		121	182	169	253					
2½		124	186	173	259					
2¾		127	190	176	264					
3		130	194	180	270					

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

Table 9-3c
Block Shear
Shear Rupture Component
per inch of thickness, kip/in.

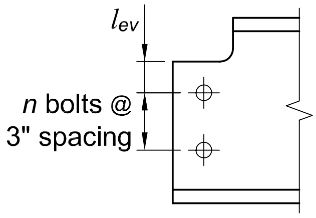


F_u, ksi		58						65					
n	l_{ev}, in.	Bolt diameter, *d*, in.[a]											
		3/4		7/8		1		3/4		7/8		1	
		$\frac{0.6F_uA_{nv}}{\Omega t}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{\Omega t}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{\Omega t}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{\Omega t}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{\Omega t}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{\Omega t}$	$\frac{\phi 0.6F_uA_{nv}}{t}$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	1 1/4	421	631	396	594	358	537	472	707	444	665	402	602
	1 3/8	423	635	398	597	361	541	474	711	446	669	404	606
	1 1/2	425	638	400	600	363	544	477	715	449	673	406	610
	1 5/8	427	641	402	604	365	547	479	718	451	676	409	613
	1 3/4	430	644	405	607	367	551	481	722	453	680	411	617
12	1 7/8	432	648	407	610	369	554	484	726	456	684	414	621
	2	434	651	409	613	371	557	486	729	458	687	416	624
	2 1/4	438	657	413	620	376	564	491	737	463	695	421	632
	2 1/2	443	664	418	626	380	570	496	744	468	702	426	639
	2 3/4	447	670	422	633	384	577	501	751	473	709	431	646
	3	451	677	426	639	389	583	506	759	478	717	436	654
	1 1/4	384	576	361	542	327	490	430	645	405	607	366	549
	1 3/8	386	579	363	545	329	493	433	649	407	611	369	553
	1 1/2	388	582	365	548	331	497	435	653	410	614	371	557
	1 5/8	390	586	368	551	333	500	438	656	412	618	374	560
	1 3/4	393	589	370	555	335	503	440	660	414	622	376	564
11	1 7/8	395	592	372	558	338	507	442	664	417	625	378	568
	2	397	595	374	561	340	510	445	667	419	629	381	571
	2 1/4	401	602	378	568	344	516	450	675	424	636	386	579
	2 1/2	406	608	383	574	349	523	455	682	429	644	391	586
	2 3/4	410	615	387	581	353	529	459	689	434	651	395	593
	3	414	622	391	587	357	536	464	697	439	658	400	601
	1 1/4	347	520	326	489	295	443	389	583	366	548	331	496
	1 3/8	349	524	328	493	297	446	391	587	368	552	333	500
	1 1/2	351	527	331	496	300	449	394	590	371	556	336	504
	1 5/8	353	530	333	499	302	453	396	594	373	559	338	507
	1 3/4	356	533	335	502	304	456	399	598	375	563	341	511
10	1 7/8	358	537	337	506	306	459	401	601	378	567	343	515
	2	360	540	339	509	308	462	403	605	380	570	346	518
	2 1/4	364	546	344	515	313	469	408	612	385	578	350	526
	2 1/2	369	553	348	522	317	476	413	620	390	585	355	533
	2 3/4	373	560	352	529	321	482	418	627	395	592	360	540
	3	377	566	357	535	326	489	423	634	400	600	365	548

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

[a] Values are for standard hole types.

Table 9-3c (continued)
Block Shear
Shear Rupture Component
per inch of thickness, kip/in.

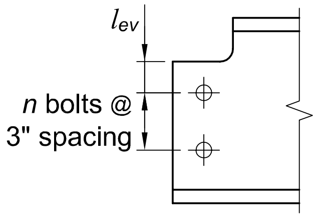


F_u, ksi		58						65					
		Bolt diameter, d, in.[a]											
n	l_{ev}, in.	3/4		7/8		1		3/4		7/8		1	
		$\frac{0.6F_uA_{nv}}{\Omega t}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{\Omega t}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{\Omega t}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{\Omega t}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{\Omega t}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{\Omega t}$	$\frac{\phi 0.6F_uA_{nv}}{t}$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
9	1¼	310	465	291	437	264	396	347	521	327	490	296	443
	1⅜	312	468	294	440	266	399	350	525	329	494	298	447
	1½	314	471	296	444	268	402	352	528	332	497	300	451
	1⅝	316	475	298	447	270	405	355	532	334	501	303	454
	1¾	319	478	300	450	272	409	357	536	336	505	305	458
	1⅞	321	481	302	453	275	412	360	539	339	508	308	462
	2	323	484	305	457	277	415	362	543	341	512	310	465
	2¼	327	491	309	463	281	422	367	550	346	519	315	473
	2½	332	498	313	470	285	428	372	558	351	527	320	480
	2¾	336	504	318	476	290	435	377	565	356	534	325	487
	3	340	511	322	483	294	441	381	572	361	541	330	495
8	1¼	273	409	257	385	232	348	306	459	288	431	260	390
	1⅜	275	413	259	388	234	352	308	463	290	435	263	394
	1½	277	416	261	392	237	355	311	466	293	439	265	398
	1⅝	279	419	263	395	239	358	313	470	295	442	268	401
	1¾	282	422	265	398	241	361	316	473	297	446	270	405
	1⅞	284	426	268	401	243	365	318	477	300	450	272	409
	2	286	429	270	405	245	368	321	481	302	453	275	412
	2¼	290	436	274	411	250	374	325	488	307	461	280	420
	2½	295	442	278	418	254	381	330	495	312	468	285	427
	2¾	299	449	283	424	258	387	335	503	317	475	289	434
	3	303	455	287	431	263	394	340	510	322	483	294	441
7	1¼	236	354	222	333	201	301	264	397	249	373	225	337
	1⅜	238	357	224	336	203	304	267	400	251	377	227	341
	1½	240	361	226	339	205	307	269	404	254	380	230	345
	1⅝	243	364	228	343	207	311	272	408	256	384	232	348
	1¾	245	367	231	346	209	314	274	411	258	388	235	352
	1⅞	247	370	233	349	212	317	277	415	261	391	237	356
	2	249	374	235	352	214	321	279	419	263	395	239	359
	2¼	253	380	239	359	218	327	284	426	268	402	244	367
	2½	258	387	244	365	222	334	289	433	273	410	249	374
	2¾	262	393	248	372	227	340	294	441	278	417	254	381
	3	266	400	252	378	231	347	299	448	283	424	259	388

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

[a] Values are for standard hole types.

Table 9-3c (continued)
Block Shear
Shear Rupture Component
per inch of thickness, kip/in.

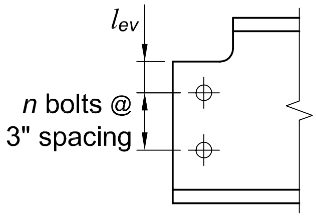


F_u, ksi		58						65					
		Bolt diameter, d, in.[a]											
		$^3/_4$		$^7/_8$		1		$^3/_4$		$^7/_8$		1	
n	l_{ev}, in.	$\frac{0.6F_uA_{nv}}{\Omega t}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{\Omega t}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{\Omega t}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{\Omega t}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{\Omega t}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{\Omega t}$	$\frac{\phi 0.6F_uA_{nv}}{t}$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
6	1¼	199	299	187	281	169	254	223	335	210	314	190	284
	1⅜	201	302	189	284	171	257	225	338	212	318	192	288
	1½	203	305	191	287	173	260	228	342	215	322	194	292
	1⅝	206	308	194	290	176	263	230	346	217	325	197	295
	1¾	208	312	196	294	178	267	233	349	219	329	199	299
	1⅞	210	315	198	297	180	270	235	353	222	333	202	303
	2	212	318	200	300	182	273	238	356	224	336	204	306
	2¼	216	325	204	307	187	280	243	364	229	344	209	314
	2½	221	331	209	313	191	286	247	371	234	351	214	321
	2¾	225	338	213	320	195	293	252	378	239	358	219	328
	3	229	344	217	326	200	299	257	386	244	366	224	335
5	1¼	162	243	152	228	138	206	182	272	171	256	154	231
	1⅜	164	246	154	232	140	210	184	276	173	260	157	235
	1½	166	250	157	235	142	213	186	280	176	263	159	239
	1⅝	169	253	159	238	144	216	189	283	178	267	161	242
	1¾	171	256	161	241	146	219	191	287	180	271	164	246
	1⅞	173	259	163	245	148	223	194	291	183	274	166	250
	2	175	263	165	248	151	226	196	294	185	278	169	253
	2¼	179	269	170	254	155	232	201	302	190	285	174	261
	2½	184	276	174	261	159	239	206	309	195	293	179	268
	2¾	188	282	178	268	164	246	211	316	200	300	183	275
	3	192	289	183	274	168	252	216	324	205	307	188	282
4	1¼	125	188	117	176	106	159	140	210	132	197	119	178
	1⅜	127	191	120	179	108	162	143	214	134	201	121	182
	1½	129	194	122	183	110	166	145	218	137	205	124	186
	1⅝	132	197	124	186	113	169	147	221	139	208	126	189
	1¾	134	201	126	189	115	172	150	225	141	212	129	193
	1⅞	136	204	128	192	117	175	152	229	144	216	131	197
	2	138	207	131	196	119	179	155	232	146	219	133	200
	2¼	142	214	135	202	123	185	160	239	151	227	138	207
	2½	147	220	139	209	128	192	165	247	156	234	143	215
	2¾	151	227	144	215	132	198	169	254	161	241	148	222
	3	156	233	148	222	136	205	174	261	166	249	153	229

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

[a] Values are for standard hole types.

Table 9-3c (continued)
Block Shear
Shear Rupture Component
per inch of thickness, kip/in.



F_u, ksi		58						65					
		Bolt diameter, d, in.[a]											
		3/4		7/8		1		3/4		7/8		1	
n	l_{ev}, in.	$\frac{0.6F_uA_{nv}}{\Omega t}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{\Omega t}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{\Omega t}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{\Omega t}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{\Omega t}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{\Omega t}$	$\frac{\phi 0.6F_uA_{nv}}{t}$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
3	1¼	88.1	132	82.6	124	74.5	112	98.7	148	92.6	139	83.5	125
	1⅜	90.3	135	84.8	127	76.7	115	101	152	95.1	143	85.9	129
	1½	92.4	139	87.0	131	78.8	118	104	155	97.5	146	88.4	133
	1⅝	94.6	142	89.2	134	81.0	122	106	159	99.9	150	90.8	136
	1¾	96.8	145	91.4	137	83.2	125	108	163	102	154	93.2	140
	1⅞	99.0	148	93.5	140	85.4	128	111	166	105	157	95.7	144
	2	101	152	95.7	144	87.5	131	113	170	107	161	98.1	147
	2¼	105	158	100	150	91.9	138	118	177	112	168	103	154
	2½	110	165	104	157	96.2	144	123	185	117	176	108	162
	2¾	114	171	109	163	101	151	128	192	122	183	113	169
	3	119	178	113	170	105	157	133	199	127	190	118	176
2	1¼	51.1	76.7	47.8	71.8	43.0	64.4	57.3	85.9	53.6	80.4	48.1	72.2
	1⅜	53.3	79.9	50.0	75.0	45.1	67.7	59.7	89.6	56.1	84.1	50.6	75.9
	1½	55.5	83.2	52.2	78.3	47.3	71.0	62.2	93.2	58.5	87.8	53.0	79.5
	1⅝	57.6	86.5	54.4	81.6	49.5	74.2	64.6	96.9	60.9	91.4	55.5	83.2
	1¾	59.8	89.7	56.6	84.8	51.7	77.5	67.0	101	63.4	95.1	57.9	86.8
	1⅞	62.0	93.0	58.7	88.1	53.8	80.7	69.5	104	65.8	98.7	60.3	90.5
	2	64.2	96.2	60.9	91.4	56.0	84.0	71.9	108	68.3	102	62.8	94.1
	2¼	68.5	103	65.3	97.9	60.4	90.5	76.8	115	73.1	110	67.6	101
	2½	72.9	109	69.6	104	64.7	97.1	81.7	122	78.0	117	72.5	109
	2¾	77.2	116	73.9	111	69.1	104	86.5	130	82.9	124	77.4	116
	3	81.6	122	78.3	117	73.4	110	91.4	137	87.8	132	82.3	123

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

[a] Values are for standard hole types.

F_y = 50 ksi

Table 9-4
Beam Bearing Constants

Shape	R_1/Ω kips ASD	ϕR_1 kips LRFD	R_2/Ω kip/in. ASD	ϕR_2 kip/in. LRFD	R_3/Ω kips ASD	ϕR_3 kips LRFD	R_4/Ω kip/in. ASD	ϕR_4 kip/in. LRFD
W44×335	220	330	34.3	51.5	335	502	10.1	15.2
×290	170	255	28.8	43.3	244	365	6.79	10.2
×262	144	216	26.2	39.3	200	299	5.68	8.53
×230[v]	119	178	23.7	35.5	159	239	4.94	7.41
W40×655	775	1160	65.7	98.5	1250	1880	35.8	53.7
×593	658	987	59.7	89.5	1040	1550	29.8	44.8
×503	506	758	51.3	77.0	765	1150	22.7	34.1
×431	395	593	44.7	67.0	574	861	17.8	26.8
×397	344	515	40.7	61.0	481	722	14.5	21.8
×372	312	468	38.7	58.0	431	646	13.5	20.3
×362	298	447	37.3	56.0	405	607	12.4	18.7
×324	249	374	33.3	50.0	324	486	9.93	14.9
×297	219	329	31.0	46.5	277	416	8.85	13.3
×277	191	286	27.7	41.5	229	343	6.59	9.88
×249	163	244	25.0	37.5	186	280	5.45	8.17
×215	130	195	21.7	32.5	139	209	4.17	6.26
×199	122	183	21.7	32.5	131	196	4.79	7.19
W40×392	438	657	47.3	71.0	647	970	19.7	29.6
×331	337	505	40.7	61.0	474	710	15.1	22.6
×327	325	488	39.3	59.0	451	676	13.7	20.5
×294	275	412	35.3	53.0	365	548	11.0	16.6
×278	257	385	34.3	51.5	339	508	10.9	16.3
×264	233	349	32.0	48.0	298	447	9.24	13.9
×235	191	286	27.7	41.5	229	343	6.59	9.88
×211	163	244	25.0	37.5	186	280	5.45	8.17
×183	129	193	21.7	32.5	138	207	4.24	6.36
×167	120	180	21.7	32.5	128	192	4.99	7.49
×149[v]	106	158	21.0	31.5	110	165	5.70	8.55

For R_1 and R_2		For R_3, R_4, R_5 and R_6		For V_{nx}	
ASD	LRFD	ASD	LRFD	ASD	LRFD
$\Omega = 1.50$	$\phi = 1.00$	$\Omega = 2.00$	$\phi = 0.75$	$\Omega_v = 1.50$	$\phi_v = 1.00$

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.

Table 9-4 (continued)
Beam Bearing Constants

F_y = 50 ksi

Nominal Wt.	R_5/Ω	ϕR_5	R_6/Ω	ϕR_6	$l_b = 3^1/_4$ in. $x < d/2$ R_n/Ω	ϕR_n	$d/2 \le x \le d$ R_n/Ω	ϕR_n	$x > d$ R_n/Ω	ϕR_n	V_{nx}/Ω_v	$\phi_v V_{nx}$
	kips	kips	kip/in.	kip/in.	kips	kips	kips	kips	kips	kips	kips	kips
lb/ft	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
335	305	458	13.5	20.3	331	497	331	497	551	827	906	1360
290	224	336	9.05	13.6	264	396	264	396	434	651	754	1130
262	183	275	7.58	11.4	218	327	229	344	373	560	680	1020
230	145	218	6.59	9.88	175	263	196	293	315	471	547	822
655	1150	1720	47.7	71.6	–	–	–	–	1760	2640	1720	2580
593	951	1430	39.8	59.7	–	–	–	–	1510	2260	1540	2310
503	701	1050	30.3	45.4	–	–	–	–	1180	1770	1300	1950
431	525	787	23.8	35.7	–	–	–	–	935	1400	1110	1660
397	442	662	19.4	29.1	–	–	–	–	820	1230	1000	1500
372	394	591	18.1	27.1	438	657	438	657	750	1120	942	1410
362	371	557	16.6	24.9	419	629	419	629	717	1080	909	1360
324	297	446	13.2	19.9	356	534	357	537	606	911	804	1210
297	254	381	11.8	17.7	306	459	320	480	539	809	740	1110
277	211	317	8.78	13.2	250	375	281	421	472	707	659	989
249	172	258	7.26	10.9	204	307	244	366	407	610	591	887
215	129	193	5.56	8.34	153	229	201	301	305	459	507	761
199	118	177	6.39	9.58	147	219	193	289	293	439	503	755
392	592	888	26.3	39.5	–	–	–	–	1030	1540	1180	1770
331	433	649	20.1	30.2	–	–	–	–	806	1210	996	1490
327	413	620	18.2	27.3	–	–	–	–	778	1170	963	1440
294	335	503	14.7	22.1	390	584	390	584	665	996	856	1280
278	310	464	14.5	21.7	368	552	368	552	625	937	828	1240
264	273	410	12.3	18.5	328	492	337	505	570	854	768	1150
235	211	317	8.78	13.2	250	375	281	421	472	707	659	989
211	172	258	7.26	10.9	204	307	244	366	407	610	591	887
183	127	191	5.65	8.48	152	228	200	299	304	455	507	761
167	115	173	6.65	9.98	144	216	191	286	288	433	502	753
149	95.2	143	7.60	11.4	129	193	174	260	257	386	432	650

– Indicates that $3^1/_4$-in. bearing length is insufficient for end beam reactions since $l_b < k$.
l_b = length of bearing, in.
x = location of concentrated force with respect to the member end, in.

F_y = 50 ksi

Table 9-4 (continued)
Beam Bearing Constants

Shape	R_1/Ω	ϕR_1	R_2/Ω	ϕR_2	R_3/Ω	ϕR_3	R_4/Ω	ϕR_4
	kips	kips	kip/in.	kip/in.	kips	kips	kip/in.	kip/in.
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W36×925	1330	1990	101	151	2690	4040	102	153
×853	1110	1660	84.0	126	2050	3080	59.2	88.8
×802	1000	1500	79.3	119	1830	2750	53.3	79.9
×723	841	1260	72.3	109	1520	2280	45.3	67.9
×652	737	1110	65.7	98.5	1250	1880	38.0	56.9
×529	518	777	53.7	80.5	839	1260	26.0	39.1
×487	454	681	50.0	75.0	724	1090	23.2	34.7
×441	384	576	45.3	68.0	597	895	19.1	28.7
×395	320	480	40.7	61.0	481	722	15.5	23.3
×361	276	414	37.3	56.0	405	607	13.3	19.9
×330	238	357	34.0	51.0	337	506	11.0	16.5
×302	207	311	31.5	47.3	287	430	9.73	14.6
×282	186	279	29.5	44.3	251	377	8.60	12.9
×262	167	251	28.0	42.0	222	334	8.06	12.1
×247	153	230	26.7	40.0	200	300	7.47	11.2
×231	140	210	25.3	38.0	179	269	6.90	10.3
W36×256	198	298	32.0	48.0	298	447	9.88	14.8
×232	168	252	29.0	43.5	245	367	8.17	12.3
×210	146	219	27.7	41.5	212	319	8.28	12.4
×194	128	192	25.5	38.3	181	271	7.03	10.5
×182	117	175	24.2	36.3	161	242	6.43	9.64
×170	105	157	22.7	34.0	142	212	5.71	8.56
×160	95.9	144	21.7	32.5	127	191	5.40	8.11
×150	88.0	132	20.8	31.3	115	173	5.23	7.84
×135[v]	77.0	116	20.0	30.0	99.5	149	5.55	8.32
W33×387	322	484	42.0	63.0	514	771	17.6	26.4
×354	278	418	38.7	58.0	435	652	15.2	22.7
×318	232	348	34.7	52.0	351	527	12.2	18.3
×291	202	302	32.0	48.0	298	447	10.6	15.9
×263	171	257	29.0	43.5	245	367	8.78	13.2
×241	151	227	27.7	41.5	215	323	8.63	12.9
×221	133	200	25.8	38.8	186	279	7.75	11.6
×201	116	173	23.8	35.8	156	234	6.81	10.2
W33×169	107	161	22.3	33.5	146	219	5.27	7.90
×152	93.1	140	21.2	31.8	125	188	5.21	7.81
×141	83.7	126	20.2	30.3	111	167	5.00	7.51
×130	75.4	113	19.3	29.0	98.4	148	4.98	7.47
×118[v]	66.0	99.0	18.3	27.5	84.5	127	4.94	7.41

For R_1 and R_2		For R_3, R_4, R_5 and R_6		For V_{nx}	
ASD	LRFD	ASD	LRFD	ASD	LRFD
$\Omega = 1.50$	$\phi = 1.00$	$\Omega = 2.00$	$\phi = 0.75$	$\Omega_v = 1.50$	$\phi_v = 1.00$

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.

Table 9-4 (continued)
Beam Bearing Constants

F_y = 50 ksi

Nominal Wt.	R_5/Ω	ϕR_5	R_6/Ω	ϕR_6	$l_b = 3^1/_4$ in.						V_{nx}/Ω_v	$\phi_v V_{nx}$
					$x < d/2$		$d/2 \le x \le d$		$x > d$			
					R_n/Ω	ϕR_n	R_n/Ω	ϕR_n	R_n/Ω	ϕR_n		
	kips	kips	kip/in.	kip/in.	kips	kips	kips	kips	kips	kips	kips	kips
lb/ft	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
925	2400	3600	136	204	–	–	–	–	2990	4470	2600	3900
853	1880	2820	79.0	118	–	–	–	–	2490	3730	2170	3260
802	1680	2520	71.1	107	–	–	–	–	2260	3390	2030	3040
723	1390	2090	60.4	90.6	–	–	–	–	1920	2870	1810	2720
652	1150	1720	50.6	75.9	–	–	–	–	1690	2540	1620	2430
529	770	1160	34.7	52.1	–	–	–	–	1210	1820	1280	1920
487	664	995	30.9	46.3	–	–	–	–	1070	1610	1180	1770
441	547	820	25.5	38.3	–	–	–	–	915	1370	1060	1590
395	442	662	20.7	31.1	452	678	452	678	772	1160	937	1410
361	371	557	17.7	26.6	397	596	397	596	673	1010	851	1280
330	310	465	14.7	22.0	349	523	349	523	587	880	769	1150
302	263	394	13.0	19.5	309	465	309	465	516	776	705	1060
282	230	345	11.5	17.2	279	419	282	423	468	702	657	985
262	203	304	10.7	16.1	248	373	258	388	425	639	620	930
247	182	273	9.96	14.9	224	336	240	360	393	590	587	881
231	162	243	9.19	13.8	201	302	222	334	362	544	555	832
256	273	410	13.2	19.8	302	454	302	454	500	752	718	1080
232	225	337	10.9	16.3	262	393	262	393	430	645	646	968
210	192	288	11.0	16.6	236	354	236	354	382	573	609	914
194	164	246	9.38	14.1	204	305	211	316	339	508	558	838
182	146	219	8.57	12.9	182	273	196	293	313	468	526	790
170	128	192	7.61	11.4	161	240	179	268	284	425	492	738
160	114	172	7.20	10.8	145	217	166	250	262	394	468	702
150	103	154	6.97	10.5	132	198	156	234	244	366	449	673
135	86.3	129	7.40	11.1	118	176	142	214	219	330	384	577
387	472	708	23.5	35.2	459	689	459	689	781	1170	907	1360
354	399	599	20.2	30.3	404	607	404	607	682	1020	826	1240
318	322	484	16.3	24.4	345	517	345	517	577	865	732	1100
291	273	410	14.2	21.2	306	458	306	458	508	760	668	1000
263	225	337	11.7	17.6	265	398	265	398	436	655	600	900
241	196	294	11.5	17.3	241	362	241	362	392	589	568	852
221	168	253	10.3	15.5	211	317	217	326	350	526	525	788
201	141	211	9.09	13.6	178	267	193	289	309	462	482	723
169	134	201	7.03	10.5	163	245	179	270	286	431	453	679
152	114	171	6.95	10.4	142	213	162	243	255	383	425	638
141	99.9	150	6.67	10.0	127	191	149	224	233	350	403	604
130	87.4	131	6.64	9.96	115	172	138	207	214	320	384	576
118	73.7	111	6.58	9.87	101	151	125	188	191	287	325	489

– Indicates that $3^1/_4$-in. bearing length is insufficient for end beam reactions since $l_b < k$.
l_b = length of bearing, in.
x = location of concentrated force with respect to the member end, in.

F_y = 50 ksi

Table 9-4 (continued)
Beam Bearing Constants

Shape	R_1/Ω kips ASD	ϕR_1 kips LRFD	R_2/Ω kip/in. ASD	ϕR_2 kip/in. LRFD	R_3/Ω kips ASD	ϕR_3 kips LRFD	R_4/Ω kip/in. ASD	ϕR_4 kip/in. LRFD
W30×391	366	549	45.3	68.0	597	895	22.4	33.7
×357	313	470	41.3	62.0	498	747	18.7	28.1
×326	270	405	38.0	57.0	420	630	16.1	24.2
×292	224	337	34.0	51.0	337	506	13.0	19.4
×261	189	284	31.0	46.5	277	416	11.1	16.7
×235	158	238	27.7	41.5	223	335	8.80	13.2
×211	136	203	25.8	38.8	189	283	8.25	12.4
×191	117	175	23.7	35.5	157	236	7.08	10.6
×173	101	151	21.8	32.8	132	198	6.24	9.36
W30×148	99.1	149	21.7	32.5	137	206	5.48	8.22
×132	84.6	127	20.5	30.8	116	174	5.55	8.32
×124	77.0	116	19.5	29.3	104	156	5.15	7.73
×116	70.6	106	18.8	28.3	94.3	141	5.11	7.67
×108	64.0	96.1	18.2	27.3	84.5	127	5.16	7.75
×99	57.2	85.8	17.3	26.0	73.9	111	5.11	7.66
×90[v]	49.4	74.0	15.7	23.5	60.6	90.9	4.17	6.25
W27×539	711	1070	65.7	98.5	1250	1880	48.0	72.0
×368	376	564	46.0	69.0	615	922	25.2	37.8
×336	322	484	42.0	63.0	514	771	21.1	31.7
×307	278	418	38.7	58.0	435	652	18.2	27.3
×281	240	360	35.3	53.0	365	548	15.2	22.8
×258	209	314	32.7	49.0	311	466	13.2	19.9
×235	182	273	30.3	45.5	265	398	11.8	17.7
×217	158	238	27.7	41.5	223	335	9.70	14.5
×194	133	200	25.0	37.5	181	272	8.09	12.1
×178	120	179	24.2	36.3	162	243	8.32	12.5
×161	103	154	22.0	33.0	134	201	6.97	10.5
×146	88.7	133	20.2	30.3	112	168	5.99	8.98
W27×129	86.4	130	20.3	30.5	120	181	5.40	8.10
×114	72.7	109	19.0	28.5	99.9	150	5.27	7.91
×102	61.4	92.1	17.2	25.8	81.1	122	4.39	6.58
×94	54.7	82.1	16.3	24.5	71.3	107	4.24	6.36
×84	47.5	71.3	15.3	23.0	60.1	90.2	4.12	6.17

For R_1 and R_2		For R_3, R_4, R_5 and R_6		For V_{nx}	
ASD	LRFD	ASD	LRFD	ASD	LRFD
$\Omega = 1.50$	$\phi = 1.00$	$\Omega = 2.00$	$\phi = 0.75$	$\Omega_v = 1.50$	$\phi_v = 1.00$

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.

Table 9-4 (continued)
Beam Bearing Constants

F_y = 50 ksi

Nominal Wt.	R_5/Ω	ϕR_5	R_6/Ω	ϕR_6	$l_b = 3^1/_4$ in. — $x < d/2$ — R_n/Ω	$x < d/2$ — ϕR_n	$d/2 \le x \le d$ — R_n/Ω	$d/2 \le x \le d$ — ϕR_n	$x > d$ — R_n/Ω	$x > d$ — ϕR_n	V_{nx}/Ω_v	$\phi_v V_{nx}$
	kips	kips	kip/in.	kip/in.	kips	kips	kips	kips	kips	kips	kips	kips
lb/ft	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
391	547	820	29.9	44.9	513	770	513	770	879	1320	903	1350
357	457	685	25.0	37.5	447	672	447	672	760	1140	813	1220
326	385	577	21.5	32.2	394	590	394	590	664	995	739	1110
292	310	465	17.3	25.9	335	503	335	503	559	840	653	979
261	254	381	14.9	22.3	290	435	290	435	479	719	588	882
235	205	307	11.7	17.6	248	373	248	373	406	611	520	779
211	172	258	11.0	16.5	216	323	220	329	356	532	479	718
191	143	214	9.44	14.2	180	270	194	290	311	465	436	654
173	119	179	8.32	12.5	152	228	172	258	273	409	398	597
148	126	189	7.30	11.0	155	233	170	255	269	404	399	599
132	105	157	7.40	11.1	134	201	151	227	236	354	373	559
124	93.5	140	6.87	10.3	121	181	140	211	217	327	353	530
116	84.1	126	6.81	10.2	111	166	132	198	202	304	339	509
108	74.2	111	6.89	10.3	101	152	123	185	187	281	325	487
99	63.8	95.7	6.81	10.2	90.5	136	113	170	171	256	309	463
90	52.4	78.6	5.56	8.34	74.2	111	100	150	148	222	249	374
539	1150	1720	64.0	96.0	–	–	–	–	1640	2460	1280	1920
368	564	846	33.6	50.4	–	–	–	–	902	1350	839	1260
336	472	708	28.2	42.3	459	689	459	689	781	1170	756	1130
307	399	599	24.3	36.5	404	607	404	607	682	1020	687	1030
281	335	503	20.3	30.4	355	532	355	532	595	892	621	932
258	285	428	17.7	26.5	315	473	315	473	524	787	568	853
235	243	364	15.7	23.6	280	421	280	421	462	694	522	784
217	205	307	12.9	19.4	248	373	248	373	406	611	471	707
194	166	249	10.8	16.2	207	311	214	322	347	522	422	632
178	147	220	11.1	16.6	189	284	199	297	319	476	403	605
161	121	182	9.29	13.9	157	235	175	261	278	415	364	546
146	101	151	7.99	12.0	131	197	154	231	243	364	332	497
129	110	166	7.20	10.8	138	207	152	229	239	359	337	505
114	90.4	136	7.03	10.5	117	176	134	202	207	311	311	467
102	73.2	110	5.85	8.77	95.4	143	117	176	179	268	279	419
94	63.7	95.5	5.66	8.48	85.1	128	108	162	162	244	264	395
84	52.8	79.2	5.49	8.23	73.5	110	97.2	146	145	217	246	368

– Indicates that $3^1/_4$-in. bearing length is insufficient for end beam reactions since $l_b < k$.
l_b = length of bearing, in.
x = location of concentrated force with respect to the member end, in.

F_y = 50 ksi

Table 9-4 (continued)
Beam Bearing Constants

Shape	R_1/Ω kips ASD	ϕR_1 kips LRFD	R_2/Ω kip/in. ASD	ϕR_2 kip/in. LRFD	R_3/Ω kips ASD	ϕR_3 kips LRFD	R_4/Ω kip/in. ASD	ϕR_4 kip/in. LRFD
W24×370	408	612	50.7	76.0	744	1120	33.3	50.0
×335	343	514	46.0	69.0	615	922	27.8	41.8
×306	292	438	42.0	63.0	514	771	23.4	35.1
×279	250	376	38.7	58.0	435	652	20.2	30.3
×250	207	311	34.7	52.0	351	527	16.3	24.5
×229	178	268	32.0	48.0	298	447	14.2	21.3
×207	150	225	29.0	43.5	245	367	11.8	17.7
×192	132	198	27.0	40.5	212	318	10.3	15.5
×176	115	173	25.0	37.5	181	272	9.03	13.5
×162	101	152	23.5	35.3	157	236	8.30	12.5
×146	86.1	129	21.7	32.5	132	198	7.37	11.1
×131	73.6	110	20.2	30.3	111	167	6.80	10.2
×117	61.9	92.8	18.3	27.5	90.6	136	5.82	8.73
×104	52.1	78.1	16.7	25.0	73.7	111	5.00	7.49
W24×103	67.8	102	18.3	27.5	97.2	146	5.01	7.51
×94	59.2	88.8	17.2	25.8	83.3	125	4.64	6.96
×84	49.7	74.6	15.7	23.5	68.1	102	4.04	6.06
×76	43.3	64.9	14.7	22.0	58.0	86.9	3.79	5.68
×68	37.7	56.5	13.8	20.8	49.2	73.9	3.72	5.59
×62	39.1	58.6	14.3	21.5	52.2	78.2	4.11	6.16
×55[v]	33.2	49.9	13.2	19.8	42.5	63.7	3.74	5.60
W21×275	343	514	40.7	61.0	480	720	24.9	37.3
×248	291	436	36.7	55.0	392	588	20.4	30.6
×223	248	371	33.3	50.0	322	483	17.2	25.9
×201	162	242	30.3	45.5	267	400	14.5	21.8
×182	137	205	27.7	41.5	222	332	12.3	18.4
×166	116	174	25.0	37.5	182	274	9.96	14.9
×147	99.0	149	24.0	36.0	158	237	10.6	15.9
×132	83.4	125	21.7	32.5	129	193	8.75	13.1
×122	73.0	110	20.0	30.0	110	165	7.49	11.2
×111	63.3	94.9	18.3	27.5	91.9	138	6.39	9.58
×101	54.2	81.3	16.7	25.0	76.2	114	5.28	7.91

For R_1 and R_2		For R_3, R_4, R_5 and R_6		For V_{nx}	
ASD	LRFD	ASD	LRFD	ASD	LRFD
$\Omega = 1.50$	$\phi = 1.00$	$\Omega = 2.00$	$\phi = 0.75$	$\Omega_v = 1.50$	$\phi_v = 1.00$

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.

Table 9-4 (continued)
Beam Bearing Constants

F_y = 50 ksi

Nominal Wt.	R_5/Ω	ϕR_5	R_6/Ω	ϕR_6	$l_b = 3^1/_4$ in., $x < d/2$, R_n/Ω	$l_b = 3^1/_4$ in., $x < d/2$, ϕR_n	$l_b = 3^1/_4$ in., $d/2 \le x \le d$, R_n/Ω	$l_b = 3^1/_4$ in., $d/2 \le x \le d$, ϕR_n	$l_b = 3^1/_4$ in., $x > d$, R_n/Ω	$l_b = 3^1/_4$ in., $x > d$, ϕR_n	V_{nx}/Ω_v	$\phi_v V_{nx}$
	kips	kips	kip/in.	kip/in.	kips	kips	kips	kips	kips	kips	kips	kips
lb/ft	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
370	682	1020	44.4	66.6	573	859	573	859	981	1470	851	1280
335	564	846	37.1	55.7	493	738	493	738	836	1250	759	1140
306	472	708	31.2	46.8	429	643	429	643	721	1080	683	1020
279	399	599	26.9	40.4	376	565	376	565	626	941	619	929
250	322	484	21.8	32.7	320	480	320	480	527	791	547	821
229	273	410	18.9	28.4	282	424	282	424	460	692	499	749
207	225	337	15.7	23.6	244	366	244	366	394	591	447	671
192	195	292	13.8	20.6	220	330	220	330	352	528	413	620
176	166	249	12.0	18.1	196	295	196	295	311	468	378	567
162	144	215	11.1	16.6	177	267	177	267	278	419	353	529
146	120	179	9.83	14.7	156	234	157	235	243	364	321	482
131	99.9	150	9.07	13.6	133	200	139	208	213	318	296	445
117	81.1	122	7.76	11.6	110	164	121	182	183	275	267	401
104	65.7	98.6	6.66	9.99	90.0	135	106	159	158	237	241	362
103	89.1	134	6.68	10.0	113	170	127	191	195	293	270	404
94	75.7	114	6.19	9.28	98.4	148	115	173	174	261	250	375
84	61.6	92.4	5.39	8.08	81.2	122	101	151	150	226	227	340
76	51.9	77.9	5.05	7.57	70.3	105	91.1	136	134	201	210	315
68	43.4	65.0	4.97	7.45	61.3	92.1	82.6	124	120	181	197	295
62	45.7	68.5	5.48	8.22	65.6	98.2	85.6	128	125	187	204	306
55	36.6	54.9	4.98	7.47	54.7	81.9	76.1	114	109	164	167	252
275	440	660	33.1	49.7	–	–	–	–	818	1230	588	882
248	360	540	27.2	40.8	410	615	410	615	701	1050	521	782
223	295	443	23.0	34.5	356	534	356	534	604	905	468	702
201	245	367	19.4	29.0	260	390	260	390	422	632	419	628
182	203	304	16.4	24.6	227	340	227	340	364	545	377	565
166	167	251	13.3	19.9	197	296	197	296	313	470	338	506
147	142	213	14.1	21.2	177	266	177	266	276	415	318	477
132	116	174	11.7	17.5	154	231	154	231	237	356	283	425
122	98.8	148	9.99	15.0	134	201	138	208	211	318	260	391
111	82.7	124	8.52	12.8	113	169	123	184	186	279	237	355
101	68.6	103	7.03	10.6	93.4	140	108	163	163	244	214	321

– Indicates that $3^1/_4$-in. bearing length is insufficient for end beam reactions since $l_b < k$.
l_b = length of bearing, in.
x = location of concentrated force with respect to the member end, in.

F_y = 50 ksi

Table 9-4 (continued)
Beam Bearing Constants

Shape	R_1/Ω kips ASD	ϕR_1 kips LRFD	R_2/Ω kip/in. ASD	ϕR_2 kip/in. LRFD	R_3/Ω kips ASD	ϕR_3 kips LRFD	R_4/Ω kip/in. ASD	ϕR_4 kip/in. LRFD
W21×93	69.1	104	19.3	29.0	103	154	7.02	10.5
×83	57.5	86.3	17.2	25.8	81.3	122	5.52	8.28
×73	47.0	70.5	15.2	22.8	63.6	95.4	4.34	6.51
×68	42.6	64.0	14.3	21.5	56.2	84.3	3.97	5.96
×62	37.3	56.0	13.3	20.0	47.8	71.7	3.58	5.37
×55	31.9	47.8	12.5	18.8	40.0	59.9	3.51	5.26
×48	27.1	40.7	11.7	17.5	32.7	49.1	3.50	5.25
W21×57	38.8	58.2	13.5	20.3	50.0	75.1	3.50	5.25
×50	32.9	49.4	12.7	19.0	41.3	61.9	3.56	5.34
×44	27.7	41.6	11.7	17.5	33.5	50.2	3.33	4.99
W18×311	410	616	50.7	76.0	747	1120	41.5	62.3
×283	350	525	46.7	70.0	631	946	36.2	54.3
×258	288	432	42.7	64.0	529	793	30.6	46.0
×234	243	364	38.7	58.0	437	656	25.3	38.0
×211	204	306	35.3	53.0	363	545	21.8	32.6
×192	172	258	32.0	48.0	300	450	17.9	26.9
×175	148	221	29.7	44.5	255	382	16.0	24.0
×158	124	186	27.0	40.5	211	316	13.5	20.3
×143	105	157	24.3	36.5	173	259	10.9	16.4
×130	89.3	134	22.3	33.5	145	217	9.38	14.1
×119	79.7	120	21.8	32.8	131	197	10.1	15.1
×106	65.9	98.8	19.7	29.5	106	159	8.44	12.7
×97	56.6	84.9	17.8	26.8	87.9	132	6.84	10.3
×86	46.8	70.2	16.0	24.0	70.3	105	5.64	8.46
×76	38.3	57.4	14.2	21.3	55.0	82.5	4.48	6.72
W18×71	49.9	74.9	16.5	24.8	75.5	113	5.85	8.77
×65	43.1	64.7	15.0	22.5	63.0	94.4	4.77	7.16
×60	38.0	57.1	13.8	20.8	53.7	80.5	4.08	6.12
×55	33.5	50.2	13.0	19.5	46.6	69.8	3.76	5.64
×50	28.8	43.1	11.8	17.8	38.5	57.7	3.15	4.73
W18×46	30.3	45.5	12.0	18.0	40.5	60.7	3.08	4.62
×40	24.3	36.5	10.5	15.8	30.9	46.3	2.40	3.60
×35	20.7	31.0	10.0	15.0	25.8	38.7	2.59	3.89
W16×100	67.8	102	19.5	29.3	107	160	8.64	13.0
×89	56.0	84.0	17.5	26.3	85.7	129	7.11	10.7
×77	44.0	66.0	15.2	22.8	64.4	96.7	5.43	8.14
×67	35.2	52.8	13.2	19.8	48.8	73.1	4.11	6.16

For R_1 and R_2		For R_3, R_4, R_5 and R_6		For V_{nx}	
ASD	LRFD	ASD	LRFD	ASD	LRFD
$\Omega = 1.50$	$\phi = 1.00$	$\Omega = 2.00$	$\phi = 0.75$	$\Omega_v = 1.50$	$\phi_v = 1.00$

Table 9-4 (continued)
Beam Bearing Constants

F_y = 50 ksi

Nominal Wt.	R_5/Ω	ϕR_5	R_6/Ω	ϕR_6	$l_b = 3^1/_4$ in., $x < d/2$, R_n/Ω	$x < d/2$, ϕR_n	$d/2 \le x \le d$, R_n/Ω	$d/2 \le x \le d$, ϕR_n	$x > d$, R_n/Ω	$x > d$, ϕR_n	V_{nx}/Ω_v	$\phi_v V_{nx}$
	kips	kips	kip/in.	kip/in.	kips	kips	kips	kips	kips	kips	kips	kips
lb/ft	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
93	92.5	139	9.36	14.0	126	188	132	198	201	302	251	376
83	73.5	110	7.36	11.0	99.2	149	113	170	171	256	220	331
73	57.5	86.2	5.78	8.68	77.7	117	96.4	145	143	215	193	289
68	50.6	75.9	5.30	7.95	69.1	104	89.1	134	132	198	181	272
62	42.8	64.2	4.77	7.16	59.4	89.2	80.5	121	118	177	168	252
55	35.1	52.6	4.68	7.02	51.4	77.0	72.5	109	103	154	156	234
48	27.9	41.8	4.66	6.99	44.1	66.2	65.1	97.6	88.2	132	144	216
57	45.1	67.7	4.67	7.00	61.4	92.2	82.7	124	121	182	171	256
50	36.3	54.5	4.75	7.13	52.9	79.3	74.2	111	106	159	158	237
44	28.9	43.3	4.43	6.65	44.3	66.4	65.7	98.5	88.6	133	145	217
311	685	1030	55.4	83.1	575	863	575	863	985	1480	678	1020
283	578	867	48.3	72.4	502	753	502	753	852	1280	613	920
258	485	728	40.9	61.3	427	640	427	640	715	1070	550	826
234	401	602	33.8	50.7	369	553	369	553	612	917	490	734
211	333	500	29.0	43.5	319	478	319	478	523	784	439	658
192	275	413	23.9	35.8	276	414	276	414	448	672	392	588
175	234	350	21.4	32.0	245	366	245	366	393	587	356	534
158	193	289	18.0	27.1	212	318	212	318	336	504	319	479
143	158	238	14.6	21.8	184	276	184	276	289	433	285	427
130	133	199	12.5	18.8	162	243	162	243	251	377	259	388
119	119	178	13.4	20.2	151	227	151	227	230	347	249	373
106	95.3	143	11.3	16.9	130	195	130	195	196	293	221	331
97	79.4	119	9.12	13.7	110	165	114	172	171	257	199	299
86	63.4	95.0	7.52	11.3	88.6	132	98.8	148	146	218	177	265
76	49.6	74.4	5.98	8.96	69.6	104	84.5	127	123	184	155	232
71	68.3	102	7.80	11.7	94.5	142	104	156	153	230	183	275
65	57.1	85.7	6.36	9.54	78.5	118	91.9	138	135	203	166	248
60	48.7	73.1	5.44	8.16	67.0	100	82.9	125	121	182	151	227
55	42.0	63.0	5.01	7.52	58.8	88.1	75.8	114	109	164	141	212
50	34.7	52.0	4.20	6.30	48.7	73.1	67.2	101	96.0	144	128	192
46	36.7	55.1	4.10	6.16	50.5	75.7	69.3	104	99.6	150	130	195
40	28.0	42.0	3.20	4.81	38.7	58.0	58.4	87.9	77.4	116	113	169
35	22.7	34.1	3.46	5.19	34.2	51.3	53.2	79.8	68.4	103	106	159
100	97.2	146	11.5	17.3	131	197	131	197	199	299	199	298
89	77.7	117	9.48	14.2	109	164	113	169	169	253	176	265
77	58.5	87.7	7.24	10.9	82.0	123	93.4	140	137	206	150	225
67	44.3	66.4	5.48	8.22	62.2	93.1	78.1	117	113	170	129	193

l_b = length of bearing, in.
x = location of concentrated force with respect to the member end, in.

F_y = 50 ksi

Table 9-4 (continued) Beam Bearing Constants

Shape	R_1/Ω	ϕR_1	R_2/Ω	ϕR_2	R_3/Ω	ϕR_3	R_4/Ω	ϕR_4
	kips	kips	kip/in.	kip/in.	kips	kips	kip/in.	kip/in.
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W16×57	40.1	60.2	14.3	21.5	57.4	86.1	4.90	7.35
×50	32.6	48.9	12.7	19.0	44.8	67.2	3.86	5.79
×45	27.8	41.7	11.5	17.3	36.7	55.0	3.26	4.89
×40	23.1	34.6	10.2	15.3	28.8	43.2	2.54	3.81
×36	20.5	30.7	9.83	14.8	25.3	38.0	2.71	4.07
W16×31	19.3	28.9	9.17	13.8	23.0	34.6	2.15	3.22
×26[v]	15.6	23.3	8.33	12.5	17.7	26.5	2.08	3.13
W14×873	2000	3000	131	197	4420	6630	340	510
×808	1780	2670	125	187	3940	5910	324	486
×730	1410	2110	102	154	2870	4310	190	285
×665	1210	1810	94.3	142	2440	3660	168	252
×605	1030	1550	86.7	130	2060	3090	146	219
×550	877	1310	79.3	119	1730	2590	126	189
×500	748	1120	73.0	110	1460	2190	111	166
×455	641	962	67.3	101	1240	1860	97.6	146
×426	569	853	62.7	94.0	1080	1620	84.4	127
×398	507	761	59.0	88.5	957	1440	76.8	115
×370	451	676	55.3	83.0	840	1260	69.4	104
×342	394	591	51.3	77.0	723	1090	61.0	91.6
×311	336	504	47.0	70.5	606	909	52.4	78.6
×283	287	431	43.0	64.5	508	762	44.9	67.3
×257	245	367	39.3	59.0	424	637	38.3	57.4
×233	207	310	35.7	53.5	350	524	32.2	48.2
×211	176	265	32.7	49.0	292	438	27.8	41.6
×193	151	227	29.7	44.5	243	364	22.8	34.2
×176	132	198	27.7	41.5	208	313	20.7	31.1
×159	111	167	24.8	37.3	169	253	16.7	25.1
×145	95.8	144	22.7	34.0	141	211	14.1	21.1
W14×132	87.6	131	21.5	32.3	127	190	12.8	19.2
×120	75.7	114	19.7	29.5	106	159	10.9	16.3
×109	63.9	95.8	17.5	26.3	85.0	127	8.50	12.8
×99	55.8	83.7	16.2	24.3	71.8	108	7.44	11.2
×90	48.0	72.1	14.7	22.0	59.2	88.8	6.19	9.29
W14×82	61.6	92.4	17.0	25.5	81.1	122	7.84	11.8
×74	51.8	77.6	15.0	22.5	64.4	96.6	5.91	8.86
×68	45.3	68.0	13.8	20.8	54.6	81.9	5.12	7.68
×61	38.8	58.1	12.5	18.8	44.4	66.6	4.25	6.37

For R_1 and R_2		For R_3, R_4, R_5 and R_6		For V_{nx}	
ASD	LRFD	ASD	LRFD	ASD	LRFD
$\Omega = 1.50$	$\phi = 1.00$	$\Omega = 2.00$	$\phi = 0.75$	$\Omega_v = 1.50$	$\phi_v = 1.00$

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.

Table 9-4 (continued)
Beam Bearing Constants

F_y = 50 ksi

Nominal Wt.	R_5/Ω	ϕR_5	R_6/Ω	ϕR_6	$l_b = 3^1/_4$ in.						V_{nx}/Ω_v	$\phi_v V_{nx}$
					$x < d/2$		$d/2 \le x \le d$		$x > d$			
					R_n/Ω	ϕR_n	R_n/Ω	ϕR_n	R_n/Ω	ϕR_n		
	kips	kips	kip/in.	kip/in.	kips	kips	kips	kips	kips	kips	kips	kips
lb/ft	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
57	52.1	78.1	6.53	9.80	73.3	110	86.6	130	127	190	141	212
50	40.6	60.9	5.15	7.72	57.3	86.0	73.9	111	106	160	124	186
45	33.2	49.8	4.35	6.52	47.3	71.0	65.2	97.9	93.0	140	111	167
40	26.1	39.2	3.38	5.07	37.1	55.7	56.3	84.3	74.1	111	97.6	146
36	22.4	33.6	3.62	5.43	34.2	51.2	52.4	78.8	68.2	102	93.8	141
31	20.8	31.1	2.86	4.30	30.1	45.1	49.1	73.8	60.0	90.1	87.5	131
26	15.5	23.3	2.78	4.17	24.5	36.9	42.7	63.9	48.9	73.3	70.5	106
873	3890	5830	453	680	–	–	–	–	4430	6640	1860	2790
808	3450	5170	432	648	–	–	–	–	3970	5950	1710	2560
730	2590	3880	253	380	–	–	–	–	3150	4720	1380	2060
665	2200	3290	224	335	–	–	–	–	2730	4080	1220	1830
605	1860	2780	195	292	–	–	–	–	2340	3520	1090	1630
550	1560	2340	168	252	–	–	–	–	2010	3010	962	1440
500	1320	1970	147	221	–	–	–	–	1730	2600	858	1290
455	1120	1670	130	195	–	–	–	–	1500	2250	768	1150
426	977	1470	113	169	–	–	–	–	1340	2010	703	1050
398	864	1300	102	154	–	–	–	–	1210	1810	648	972
370	757	1140	92.5	139	–	–	–	–	1080	1620	594	891
342	652	978	81.4	122	561	841	561	841	955	1430	539	809
311	546	820	69.9	105	489	733	489	733	825	1240	482	723
283	458	687	59.8	89.7	427	641	427	641	714	1070	431	646
257	383	574	51.1	76.6	373	559	373	559	618	926	387	581
233	315	473	42.9	64.3	323	484	323	484	530	794	342	514
211	263	394	37.0	55.5	282	424	282	424	458	689	308	462
193	219	329	30.4	45.6	248	372	248	372	399	599	276	414
176	187	281	27.7	41.5	222	333	222	333	354	531	252	378
159	152	228	22.3	33.5	192	288	192	288	303	455	224	335
145	127	191	18.8	28.2	170	255	170	255	265	399	201	302
132	114	171	17.1	25.6	157	236	157	236	245	367	190	284
120	95.3	143	14.5	21.8	140	210	140	210	215	324	171	257
109	76.9	115	11.3	17.0	114	170	121	181	185	277	150	225
99	64.8	97.2	9.92	14.9	97.0	146	108	163	164	246	138	207
90	53.4	80.2	8.26	12.4	80.2	121	95.8	144	144	216	123	185
82	73.6	110	10.5	15.7	108	161	117	175	178	268	146	219
74	58.8	88.2	7.88	11.8	84.4	127	101	151	152	228	128	192
68	49.9	74.8	6.83	10.2	72.1	108	90.2	136	135	204	116	174
61	40.5	60.7	5.67	8.50	58.9	88.3	79.4	119	116	175	104	156

– Indicates that $3^1/_4$-in. bearing length is insufficient for end beam reactions since $l_b < k$.
l_b = length of bearing, in.
x = location of concentrated force with respect to the member end, in.

F_y = 50 ksi

Table 9-4 (continued)
Beam Bearing Constants

Shape	R_1/Ω kips ASD	ϕR_1 kips LRFD	R_2/Ω kip/in. ASD	ϕR_2 kip/in. LRFD	R_3/Ω kips ASD	ϕR_3 kips LRFD	R_4/Ω kip/in. ASD	ϕR_4 kip/in. LRFD
W14×53	38.5	57.8	12.3	18.5	44.0	66.1	3.99	5.98
×48	33.7	50.6	11.3	17.0	36.8	55.2	3.46	5.19
×43	28.5	42.7	10.2	15.3	29.5	44.3	2.82	4.23
W14×38	23.6	35.5	10.3	15.5	29.8	44.7	2.96	4.45
×34	20.3	30.5	9.50	14.3	24.7	37.1	2.63	3.94
×30	17.7	26.5	9.00	13.5	21.0	31.4	2.68	4.01
W14×26	17.4	26.1	8.50	12.8	20.1	30.1	2.05	3.08
×22	14.1	21.1	7.67	11.5	15.4	23.1	1.92	2.87
W12×336	527	790	59.3	89.0	984	1480	81.9	123
×305	448	672	54.3	81.5	825	1240	70.8	106
×279	391	587	51.0	76.5	716	1070	65.9	98.8
×252	333	499	46.7	70.0	598	898	57.2	85.8
×230	287	431	43.0	64.5	508	762	49.6	74.4
×210	246	369	39.3	59.0	426	638	42.5	63.8
×190	206	309	35.3	53.0	347	520	34.3	51.5
×170	173	259	32.0	48.0	283	424	29.3	43.9
×152	145	218	29.0	43.5	231	347	24.8	37.2
×136	122	183	26.3	39.5	189	284	21.3	31.9
×120	101	151	23.7	35.5	152	228	17.8	26.7
×106	80.8	121	20.3	30.5	114	171	12.8	19.3
×96	68.8	103	18.3	27.5	93.2	140	10.5	15.8
×87	60.5	90.8	17.2	25.8	80.1	120	9.75	14.6
×79	52.1	78.1	15.7	23.5	66.5	99.8	8.23	12.3
×72	45.5	68.3	14.3	21.5	55.6	83.4	6.97	10.5
×65	39.0	58.5	13.0	19.5	45.6	68.4	5.85	8.78
W12×58	37.2	55.8	12.0	18.0	41.6	62.4	4.32	6.48
×53	33.9	50.9	11.5	17.3	37.0	55.5	4.26	6.40
W12×50	35.2	52.7	12.3	18.5	43.4	65.0	4.69	7.03
×45	30.2	45.2	11.2	16.8	35.4	53.1	3.90	5.86
×40	25.1	37.6	9.83	14.8	27.7	41.5	3.03	4.54
W12×35	20.5	30.8	10.0	15.0	28.5	42.8	3.00	4.50
×30	16.0	24.1	8.67	13.0	21.2	31.8	2.35	3.52
×26	13.0	19.6	7.67	11.5	16.4	24.6	1.90	2.84
W12×22	15.7	23.6	8.67	13.0	20.8	31.2	2.43	3.64
×19	12.7	19.1	7.83	11.8	16.2	24.3	2.20	3.29
×16	10.4	15.5	7.33	11.0	12.8	19.2	2.42	3.63
×14[v]	8.75	13.1	6.67	10.0	10.2	15.3	2.16	3.24

For R_1 and R_2		For R_3, R_4, R_5 and R_6		For V_{nx}	
ASD	LRFD	ASD	LRFD	ASD	LRFD
$\Omega = 1.50$	$\phi = 1.00$	$\Omega = 2.00$	$\phi = 0.75$	$\Omega_v = 1.50$	$\phi_v = 1.00$

[v] Shape does not meet the h/t_w limit for shear in AISC *Specification* Section G2.1(a) with $F_y = 50$ ksi; therefore, $\phi_v = 0.90$ and $\Omega_v = 1.67$.

Table 9-4 (continued)
Beam Bearing Constants

F_y = 50 ksi

Nom-inal Wt.	R_5/Ω	ϕR_5	R_6/Ω	ϕR_6	$l_b = 3^1/_4$ in. $x < d/2$ R_n/Ω	$x < d/2$ ϕR_n	$d/2 \le x \le d$ R_n/Ω	$d/2 \le x \le d$ ϕR_n	$x > d$ R_n/Ω	$x > d$ ϕR_n	V_{nx}/Ω_v	$\phi_v V_{nx}$
	kips	kips	kip/in.	kip/in.	kips	kips	kips	kips	kips	kips	kips	kips
lb/ft	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
53	40.3	60.5	5.32	7.98	57.6	86.4	78.5	118	114	171	103	154
48	33.6	50.5	4.61	6.92	48.6	73.0	70.4	106	96.1	144	93.8	141
43	27.0	40.4	3.76	5.65	39.2	58.8	61.7	92.4	77.3	116	83.6	125
38	27.0	40.6	3.95	5.93	39.8	59.9	57.1	85.9	78.8	118	87.4	131
34	22.3	33.4	3.50	5.25	33.7	50.5	51.2	77.0	66.5	99.8	79.8	120
30	18.5	27.8	3.57	5.35	30.1	45.2	47.0	70.4	59.4	88.9	74.5	112
26	18.2	27.3	2.74	4.10	27.1	40.6	45.0	67.7	53.5	80.2	70.9	106
22	13.6	20.4	2.55	3.83	21.9	32.8	39.0	58.5	43.3	64.9	63.0	94.5
336	892	1340	109	164	–	–	–	–	1250	1870	598	897
305	748	1120	94.4	142	–	–	–	–	1070	1610	531	797
279	646	970	87.9	132	557	836	557	836	948	1420	487	730
252	540	809	76.3	114	485	727	485	727	818	1230	431	647
230	458	687	66.2	99.2	427	641	427	641	714	1070	390	584
210	384	576	56.7	85.0	374	561	374	561	620	930	347	520
190	314	471	45.8	68.7	321	481	321	481	527	790	305	458
170	256	383	39.0	58.5	277	415	277	415	450	674	269	403
152	209	313	33.1	49.6	239	359	239	359	384	577	238	358
136	170	255	28.4	42.5	207	311	207	311	329	494	212	318
120	136	204	23.7	35.6	178	266	178	266	279	417	186	279
106	103	155	17.1	25.7	147	220	147	220	228	341	157	236
96	84.3	126	14.0	21.0	128	192	128	192	197	295	140	210
87	72.0	108	13.0	19.5	114	171	116	175	177	265	129	193
79	59.7	89.6	11.0	16.5	95.5	143	103	154	155	233	117	175
72	49.9	74.8	9.29	13.9	80.1	120	92.0	138	137	206	106	159
65	40.9	61.4	7.81	11.7	66.3	99.4	81.3	122	120	180	94.4	142
58	38.1	57.2	5.76	8.63	56.8	85.2	76.2	114	111	167	87.8	132
53	33.6	50.3	5.69	8.53	52.1	78.0	71.3	107	102	153	83.5	125
50	39.5	59.3	6.25	9.37	59.8	89.8	75.2	113	110	166	90.3	135
45	32.3	48.4	5.21	7.81	49.2	73.8	66.6	99.8	96.2	144	81.1	122
40	25.3	37.9	4.04	6.05	38.4	57.6	57.0	85.7	75.1	113	70.2	105
35	26.0	39.1	4.00	6.00	39.0	58.6	53.0	79.6	73.5	110	75.0	113
30	19.3	28.9	3.13	4.69	29.5	44.1	44.2	66.4	57.7	86.5	64.0	95.9
26	14.8	22.3	2.53	3.79	23.0	34.6	37.9	57.0	45.2	67.7	56.1	84.2
22	18.8	28.2	3.24	4.86	29.3	44.0	43.9	65.9	57.4	86.1	64.0	95.9
19	14.4	21.7	2.93	4.39	23.9	36.0	38.1	57.5	46.7	70.0	57.3	86.0
16	10.9	16.3	3.23	4.84	21.4	32.0	34.2	51.3	41.3	62.0	52.8	79.2
14	8.51	12.8	2.88	4.32	17.9	26.8	30.4	45.6	34.4	51.7	42.8	64.3

– Indicates that $3^1/_4$-in. bearing length is insufficient for end beam reactions since $l_b < k$.
l_b = length of bearing, in.
x = location of concentrated force with respect to the member end, in.

Table 9-4 (continued)
Beam Bearing Constants

F_y = 50 ksi

Shape	R_1/Ω kips ASD	ϕR_1 kips LRFD	R_2/Ω kip/in. ASD	ϕR_2 kip/in. LRFD	R_3/Ω kips ASD	ϕR_3 kips LRFD	R_4/Ω kip/in. ASD	ϕR_4 kip/in. LRFD
W10×112	110	165	25.2	37.8	177	265	21.8	32.7
×100	91.8	138	22.7	34.0	143	214	18.3	27.4
×88	75.1	113	20.2	30.3	113	169	15.0	22.4
×77	60.5	90.8	17.7	26.5	86.7	130	11.7	17.5
×68	49.7	74.6	15.7	23.5	68.1	102	9.37	14.1
×60	41.3	62.0	14.0	21.0	54.1	81.1	7.72	11.6
×54	34.5	51.8	12.3	18.5	42.5	63.8	5.89	8.84
×49	30.0	45.1	11.3	17.0	35.7	53.6	5.07	7.61
W10×45	32.7	49.0	11.7	17.5	39.3	58.9	4.95	7.42
×39	27.0	40.6	10.5	15.8	31.0	46.5	4.30	6.44
×33	22.6	33.9	9.67	14.5	24.8	37.2	4.16	6.24
W10×30	20.3	30.4	10.0	15.0	28.3	42.4	3.64	5.46
×26	16.0	24.1	8.67	13.0	21.2	31.8	2.80	4.20
×22	13.2	19.8	8.00	12.0	17.0	25.5	2.72	4.08
W10×19	14.5	21.7	8.33	12.5	18.9	28.4	2.80	4.20
×17	12.6	18.9	8.00	12.0	16.3	24.4	3.00	4.49
×15	10.9	16.4	7.67	11.5	13.8	20.7	3.26	4.89
×12	8.08	12.1	6.33	9.50	9.14	13.7	2.39	3.59
W8×67	63.2	94.8	19.0	28.5	100	150	15.9	23.9
×58	51.0	76.5	17.0	25.5	78.9	118	13.5	20.3
×48	36.0	54.0	13.3	20.0	50.4	75.6	7.94	11.9
×40	28.6	42.9	12.0	18.0	38.9	58.4	7.30	10.9
×35	23.0	34.4	10.3	15.5	29.2	43.9	5.35	8.03
×31	19.7	29.5	9.50	14.3	24.2	36.3	4.81	7.21
W8×28	20.4	30.6	9.50	14.3	25.0	37.5	4.46	6.69
×24	16.2	24.3	8.17	12.3	18.5	27.7	3.35	5.02
W8×21	14.6	21.9	8.33	12.5	19.0	28.6	3.41	5.11
×18	12.1	18.1	7.67	11.5	15.3	22.9	3.27	4.91
W8×15	12.6	18.8	8.17	12.3	16.4	24.6	4.16	6.24
×13	10.6	16.0	7.67	11.5	13.4	20.1	4.31	6.47
×10	7.15	10.7	5.67	8.50	7.64	11.5	2.19	3.29

For R_1 and R_2		For R_3, R_4, R_5 and R_6		For V_{nx}	
ASD	LRFD	ASD	LRFD	ASD	LRFD
$\Omega = 1.50$	$\phi = 1.00$	$\Omega = 2.00$	$\phi = 0.75$	$\Omega_v = 1.50$	$\phi_v = 1.00$

Table 9-4 (continued)
Beam Bearing Constants

F_y = 50 ksi

Nom-inal Wt.	R_5/Ω	ϕR_5	R_6/Ω	ϕR_6	$l_b = 3^1/_4$ in. $x < d/2$ R_n/Ω	ϕR_n	$d/2 \le x \le d$ R_n/Ω	ϕR_n	$x > d$ R_n/Ω	ϕR_n	V_{nx}/Ω_v	$\phi_v V_{nx}$
	kips	kips	kip/in.	kip/in.	kips	kips	kips	kips	kips	kips	kips	kips
lb/ft	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
112	160	240	29.1	43.6	192	288	192	288	302	453	172	258
100	129	194	24.4	36.5	166	249	166	249	257	387	151	226
88	102	153	20.0	29.9	141	211	141	211	216	324	131	196
77	78.4	118	15.6	23.3	118	177	118	177	179	268	112	169
68	61.6	92.4	12.5	18.7	101	151	101	151	150	226	97.8	147
60	48.8	73.2	10.3	15.4	82.3	123	86.8	130	128	192	85.7	129
54	38.5	57.8	7.86	11.8	64.0	96.2	74.5	112	109	164	74.7	112
49	32.3	48.5	6.76	10.1	54.3	81.3	66.7	100	96.7	145	68.0	102
45	35.9	53.9	6.60	9.89	57.4	86.0	70.7	106	103	155	70.7	106
39	28.2	42.2	5.73	8.59	46.8	70.1	61.1	92.0	88.1	133	62.5	93.7
33	22.1	33.2	5.55	8.33	40.1	60.3	54.0	81.0	76.6	115	56.4	84.7
30	25.7	38.6	4.86	7.29	41.5	62.3	52.8	79.2	73.1	110	63.0	94.5
26	19.3	28.9	3.74	5.60	31.5	47.1	44.2	66.4	60.2	90.5	53.6	80.3
22	15.1	22.7	3.63	5.44	26.9	40.4	39.2	58.8	51.7	77.5	49.0	73.4
19	17.0	25.5	3.74	5.60	29.2	43.7	41.6	62.3	56.0	84.0	51.0	76.5
17	14.2	21.4	4.00	5.99	27.2	40.9	38.6	57.9	51.2	76.8	48.5	72.7
15	11.6	17.4	4.35	6.52	25.7	38.6	35.8	53.8	46.7	70.2	46.0	68.9
12	7.57	11.4	3.19	4.78	17.9	26.9	28.7	43.0	33.8	50.7	37.5	56.3
67	90.7	136	21.2	31.8	125	187	125	187	188	282	103	154
58	71.1	107	18.0	27.0	106	159	106	159	157	236	89.3	134
48	45.9	68.9	10.6	15.9	79.2	119	79.2	119	115	173	68.0	102
40	34.9	52.4	9.73	14.6	66.5	99.9	67.6	101	96.2	144	59.4	89.1
35	26.3	39.5	7.14	10.7	49.5	74.3	56.5	84.8	79.5	119	50.3	75.5
31	21.6	32.4	6.41	9.61	42.4	63.6	50.6	76.0	70.3	105	45.6	68.4
28	22.6	33.9	5.95	8.93	41.9	62.9	51.3	77.1	71.7	108	45.9	68.9
24	16.7	25.1	4.47	6.70	31.2	46.9	42.8	64.3	58.8	88.0	38.9	58.3
21	17.2	25.7	4.54	6.82	32.0	47.9	41.7	62.5	56.3	84.4	41.4	62.1
18	13.5	20.2	4.36	6.55	27.7	41.5	37.0	55.5	49.1	73.6	37.4	56.2
15	14.1	21.2	5.55	8.32	32.1	48.2	39.2	58.8	51.8	77.6	39.7	59.6
13	11.1	16.7	5.75	8.63	29.8	44.7	35.5	53.4	46.1	69.4	36.8	55.1
10	6.49	9.73	2.93	4.39	16.0	24.0	25.6	38.3	29.5	44.4	26.8	40.2

l_b = length of bearing, in.
x = location of concentrated force with respect to the member end, in.

PART 10

DESIGN OF SIMPLE SHEAR CONNECTIONS

SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of simple shear connections. For the design of partially restrained moment connections, see Part 11. For the design of fully restrained (FR) moment connections, see Part 12.

FORCE TRANSFER

The required strength (end reaction), R_u or R_a, is determined by analysis as indicated in AISC *Specification* Section B3. Per AISC *Specification* Section J1.2, the ends of members with simple shear connections are normally assumed to be free to rotate under load. While simple shear connections do actually possess some rotational restraint (see curve A in Figure 10-1), this small amount can be neglected and the connection idealized as completely flexible. The simple shear connections shown in this Manual are suitable to accommodate the end rotations required per AISC *Specification* Section J1.2.

Support rotation is acceptably limited for most framing details involving simple shear connections without explicit consideration. The case of a bare spandrel girder supporting infill beams, however, may require consideration to verify that an acceptable level of support rotational stiffness is present. Sumner (2003) showed that a nominal interconnection between the top flange of the girder and the top flange of the framing beam is sufficient to limit support rotation.

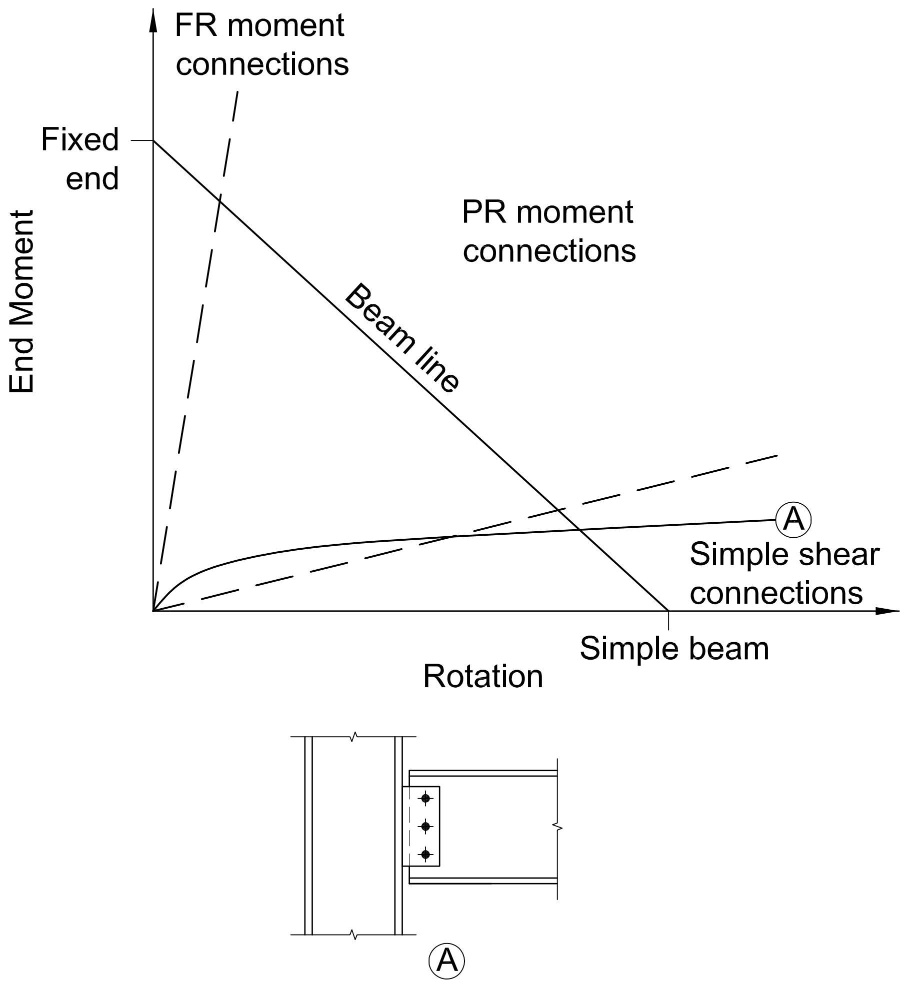


Fig. 10-1. Illustration of typical moment rotation curve for simple shear connections.

COMPARING CONNECTION ALTERNATIVES

Two-Sided Connections

Two-sided connections, such as double-angle and shear end-plate connections, offer the following advantages:

1. Suitability for use when the end reaction is large
2. Compact connections (usually, the entire connection is contained within the flanges of the supported beam)
3. Eccentricity perpendicular to the beam axis need not be considered for workable gages (see Table 1-7A)

Note that two-sided connections may require additional consideration for erectability, as discussed in the following section, “Constructability Considerations”.

Seated Connections

Unstiffened and stiffened seated connections offer the following advantages:

1. Seats can be shop attached to the support, simplifying erection
2. Ample erection clearance is provided
3. Excellent safety during erection since double connections often can be eliminated
4. The bay length of the structure is easily maintained (seated connections may be preferable when maintaining bay length is a concern for repetitive bays of framing)

One-Sided Connections

One-sided connections such as single-plate, single-angle and tee connections offer the following advantages:

1. Shop attachment of connection elements to the support, simplifying shop fabrication and erection
2. Reduced material and shop labor requirements
3. Ample erection clearance is provided
4. Excellent safety during erection since double connections often can be eliminated

CONSTRUCTABILITY CONSIDERATIONS

Double Connections

A double connection occurs in field-bolted construction when beams or girders frame opposite each other. Double connections are a safety concern when they occur in the web of a column (see Figure 10-2) or the web of a beam that frames continuously over the top of a column and all field bolts take the same open holes. A positive connection must be made and maintained for the first member to be erected while the second member to be erected is brought into its final position[1]. OSHA requirements prohibit the condition where one beam is temporarily hung on a partially inserted bolt or drift pin.

[1]This requirement applies only at the location of the column, not at locations away from the column.

Framing details can be configured using staggered angles or other similar details to provide a means to make a positive connection for the first member while the second member is brought into its final position. Alternatively, a temporary erection seat, as shown in Figure 10-2, can be provided. The erection seat, usually an angle, is sized and attached to the column web to support the dead weight of the member, unless additional loading is indicated in the contract documents. The clearance shown in Figure 10-2 is located to clear the bottom flange of the supported member by approximately $^{3}/_{8}$ in. to accommodate mill, fabrication and erection tolerances.

The sequence of erection is most important in determining the need for erection seats. If the erection sequence is known, the erection seat is provided on the side needing the support. If the erection sequence is not known, a seat can be provided on both sides of the column web. Temporary erection seats may be reused at other locations after the connection(s) are made, but need not be removed unless they create an interference or removal is required in the contract documents.

See also the discussion under “Special Considerations for Simple Shear Connections.”

Accessibility in Column Webs

Because of bolting and welding clearances, double-angle, shear end-plate, single-plate, single-angle, and tee shear connections may not be suitable for connections to the webs of W-shape and similar columns, particularly for W8 columns, unless gages are reduced. Such connections may be impossible for W6, W5 and W4 columns.

There is also an accessibility concern for entering and tightening the field bolts when the connection material is shop-attached to the supporting column web and contained within the column flanges.

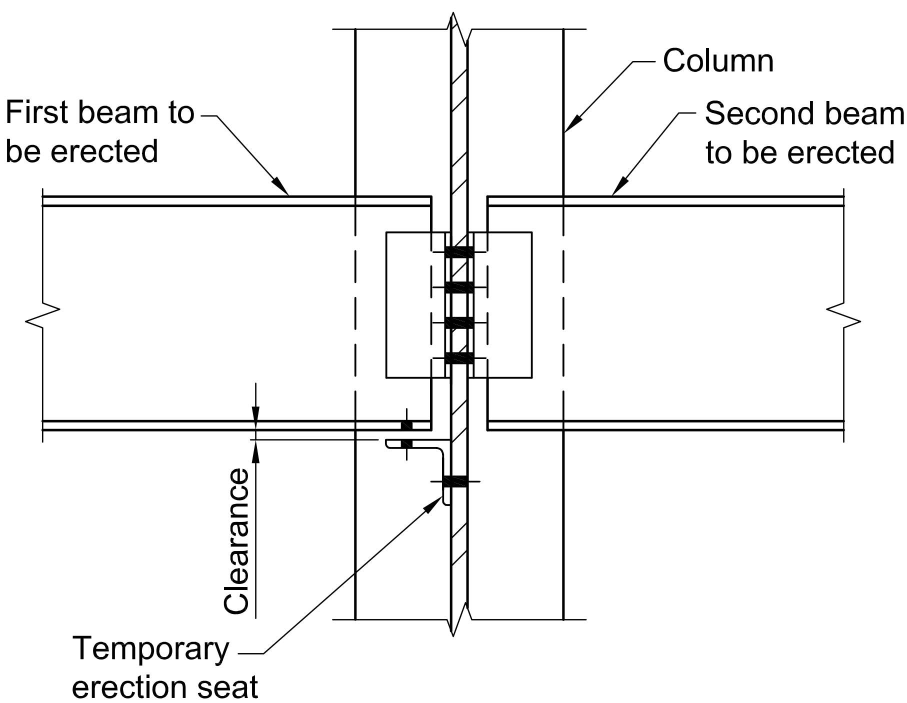


Fig. 10-2. Erection seat.

Field-Welded Connections

In field-welded connections, temporary erection bolts are usually provided to support the member until final welding is performed. A minimum of two bolts (one bolt in bracing members) must be placed for erection safety per OSHA requirements. Additional erection bolts may be required for loads during erection, to assist in pulling the connection angles up tightly against the web of the supporting beam prior to welding or for other reasons. Temporary erection bolts may be reused at other locations after final welding, but need not be removed unless they create an interference or removal is required in the contract documents.

Recommended Connection Length (Riding the Fillet)

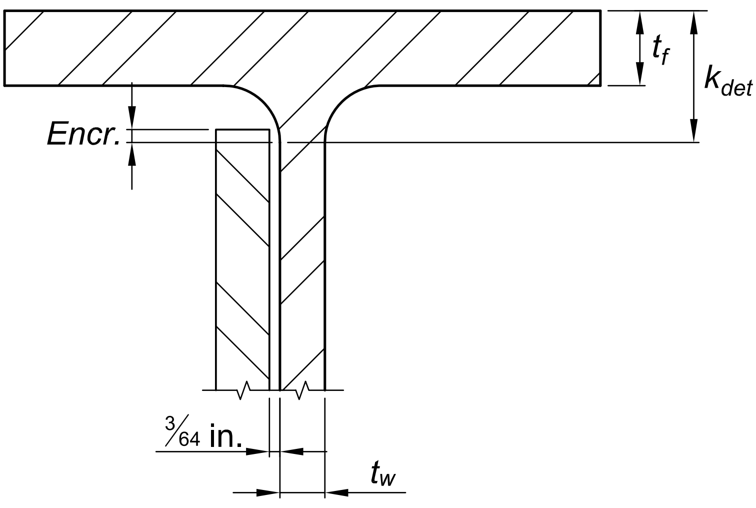
It is recommended that the minimum length of simple shear framed connections be one-half the *T*-dimension of the beam to be supported. This provides for beam end stability during erection. When a beam is otherwise restrained against rotation about its longitudinal axis, such as is the case for a composite beam, the torsional end restraint is not critical.

The detailed dimensions of connection elements must be compatible with the *T*-dimension of an uncoped beam and the remaining web depth of a coped beam. Note that the element may encroach upon the fillet(s), as given in Figure 10-3.

DOUBLE-ANGLE CONNECTIONS

A double-angle connection is made with two angles, one on each side of the web of the beam to be supported, as illustrated in Figure 10-4. These angles may be bolted or welded to the supported beam as well as to the supporting member.

When the angles are welded to the support, adequate flexibility must be provided in the connection. As illustrated in Figure 10-4(c), line welds are placed along the toes of the angles with a return at the top limited by AISC *Specification* Section J2.2b. Note that welding across the entire top of the angles must be avoided as it inhibits the flexibility and, therefore, the necessary end rotation of the connection. The performance of the resulting connection would not be as intended for simple shear connections.

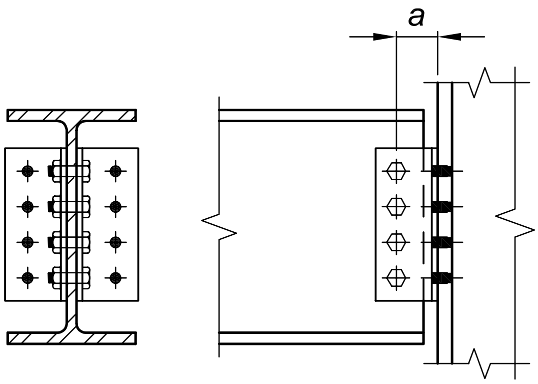


$k_{det} - t_f$, in.	*Encr.*, in.
5⁄16	1⁄8
3⁄8 to 1⁄2	3⁄16
9⁄16 to 13⁄16	1⁄4
7⁄8 to 1 1⁄4	5⁄16
1 5⁄16 to 1 3⁄8	3⁄8

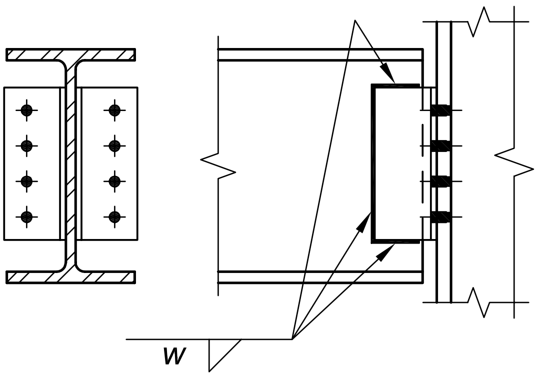
Fig. 10-3. Fillet encroachment (riding the fillet).

Available Strength and Flexibility

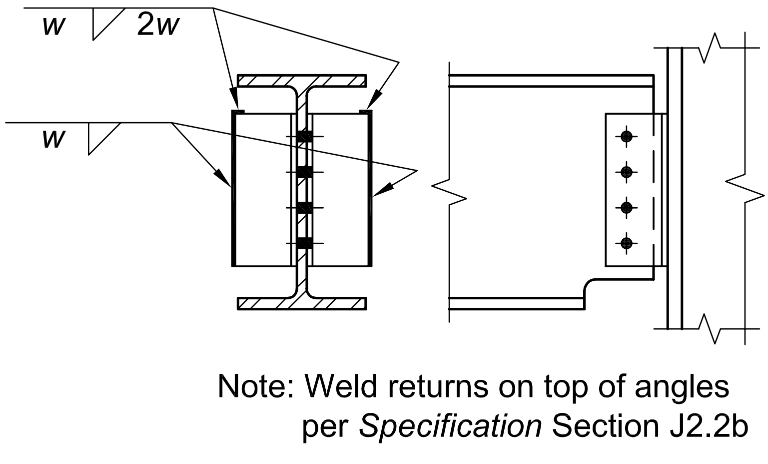
The available strength of a double-angle connection is determined from the applicable limit states for bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a.



(a) All-bolted



(b) Bolted/welded, angles welded to support beam



(c) Bolted/welded, angles welded to support

Fig. 10-4. Double-angle connections.

The eccentricity on the supported side of double-angle connections may be neglected for connections with a single vertical row of bolts through standard or short-slotted holes with dimension *a* [see Figure 10-4(a)] not exceeding 3 in. The eccentricity should be considered for the design of double-angle connections with two or more vertical rows of bolts on the supported side of the connection and for the design of double-angle connections welded to the supported member.

To provide for flexibility, the maximum angle thickness for use with workable gages should be limited to $^5/_8$ in. Alternatively, the shear-connection ductility checks illustrated in Part 9 can be used to justify other combinations of gage and angle thickness.

Shop and Field Practices

When framing to a girder web, both angles are usually shop-attached to the web of the supported beam. When framing to a column web, both angles should be shop-attached to the supported beam, when possible, and the associated constructability considerations should be addressed (see the preceding discussion under "Constructability Considerations").

When framing to a column flange, both angles can be shop-attached to the column flange or the supported beam. In the former case, as illustrated in Figure 10-4(c), this is a knifed connection, which requires coping the bottom flange of the supported beam and an erection clearance as shown in Figure 10-5(a). Also, provision must be made for possible mill variation in the depth of the columns, particularly in fairly long runs (i.e., six or more bays of framing). If both angles are shop-attached to the beam web, the beam length can be shortened to provide for mill overrun with shims furnished at the appropriate intervals to fill the resulting gaps or to provide for mill underrun. If both angles are shop-attached to the column flange, the erected beam is knifed into place and play in the open holes is normally sufficient to provide for the necessary adjustment. Alternatively, short-slotted holes can also be used.

When special requirements preclude the use of any of the foregoing practices, one angle could be shop-attached to the support and the other shipped loose. In this case, the spread between the outstanding legs should equal the decimal beam web thickness plus a clearance that will produce an opening to the next higher $^1/_{16}$-in. increment, as illustrated in Figure 10-5(b). Alternatively, short-slotted holes in the support leg of the angle eliminate the need to provide for variations in web thickness and also allow for minor adjustment during erection. Note that the practice of shipping one angle loose is not desirable because it requires additional material handling as well as added erection costs and complexity.

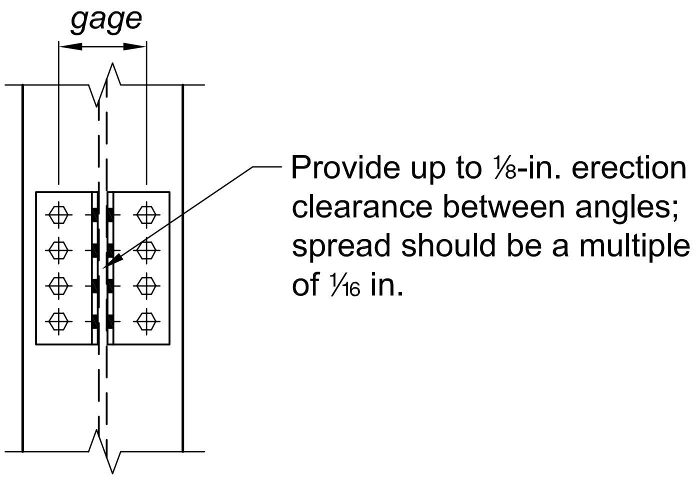
DESIGN TABLE DISCUSSION (TABLES 10-1, 10-2 AND 10-3)

Table 10-1. All-Bolted Double-Angle Connections

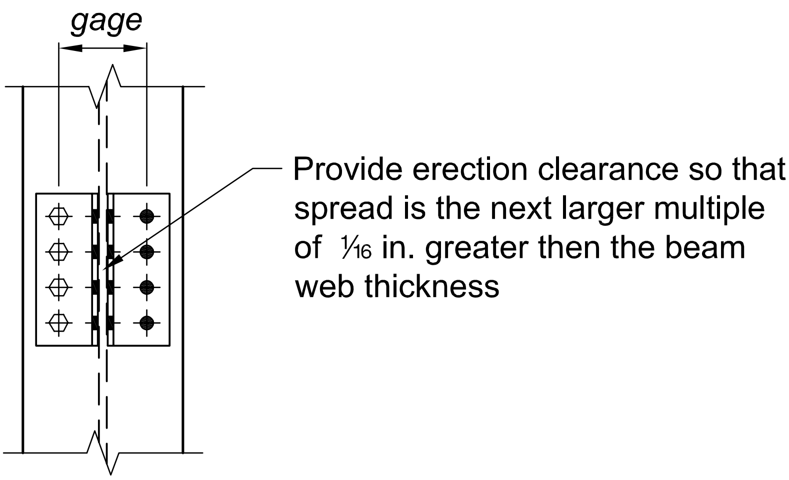
Table 10-1 is a design aid for all-bolted double-angle connections. Available strengths are tabulated for supporting angle material with $F_y = 36$ ksi and $F_u = 58$ ksi. All values, including slip-critical bolt available strengths, are for comparison with the governing LRFD or ASD load combination.

Tabulated bolt and angle available strengths consider the limit states of bolt shear, slip resistance for slip-critical bolts, bolt bearing and tearout on the angles, shear yielding of the angles, shear rupture of the angles, and block shear rupture of the angles. Values are tabulated for 2 through 12 rows of $^3/_4$-in.-, $^7/_8$-in.- and 1-in.-diameter Group A and Group B bolts (as defined in AISC *Specification* Section J3.1) at 3-in. spacing. For calculation purposes, angle vertical edge distance, l_{ev}, is assumed to be 1¼ in. and horizontal edge distance, l_{eh}, is assumed to be 1⅜ in. For bearing-type bolts, tabulated strengths in Table 10-1 are based on short-slotted holes transverse to the direction of load in the support angle leg. Table 10-1 can be conservatively used when standard holes are employed in the support angle leg.

Available beam web strength can be determined as the lesser of the limit states of block shear rupture, shear yielding, shear rupture, and the sum of the effective strengths of the individual fasteners. The effective strength of an individual fastener is the lesser of the fastener shear strength per AISC *Specification* Section J3.6 (or slip resistance for slip-critical bolts per Section J3.8) and fastener bearing and tearout strength at the hole per AISC *Specification* Section J3.10. For coped members, the limit states of flexural yielding



(a) Both angles shop attached to the column flange (beam knifed into place)



(b) One angle shop attached to the column flange, other angle shipped loose

Fig. 10-5. Erection clearances for double-angle connections.

and local buckling must be checked independently per Part 9. When required, web reinforcement of coped members is treated in Part 9.

Note that resistance and safety factors are not noted in these tables, as they vary by limit state.

Table 10-2. Available Weld Strength of Bolted/Welded Double-Angle Connections

Table 10-2 is a design aid arranged to permit substitution of welds for bolts in connections designed with Table 10-1. Electrode strength is assumed to be 70 ksi. Holes for erection bolts may be placed as required in angle legs that are to be field-welded.

Welds A may be used in place of bolts through the supported-beam web legs of the double angles or welds B may be used in place of bolts through the support legs of the double angles. Although it is permissible to use welds A and B from Table 10-2 in combination to obtain all-welded connections, it is recommended that such connections be selected from Table 10-3. This table will allow increased flexibility in the selection of angle lengths and connection strengths because Table 10-2 conforms to the bolt spacing and edge distance requirements for the all-bolted double-angle connections of Table 10-1.

Weld available strengths are tabulated for the limit state of weld shear. Available strengths for welds A are determined by the instantaneous center of rotation method using Table 8-8 with $\theta = 0°$. Available strengths for welds B are determined by the elastic method. With the neutral axis assumed at one-sixth the depth of the angles measured downward and the tops of the angles in compression against each other through the beam web, the available strength, ϕR_n or R_n/Ω, of these welds is determined by

LRFD	ASD
$\phi R_n = 2\left(\dfrac{1.392Dl}{\sqrt{1+\dfrac{12.96e^2}{l^2}}}\right)$ (10-1a)	$\dfrac{R_n}{\Omega} = 2\left(\dfrac{0.928Dl}{\sqrt{1+\dfrac{12.96e^2}{l^2}}}\right)$ (10-1b)

where

D = number of sixteenths-of-an-inch in the weld size
e = width of the leg of the connection angle attached to the support, in.
l = length of the connection angles, in.

Note that $\phi = 0.75$ is included in the right hand side of Equation 10-1a and $\Omega = 2.00$ is included in the right hand side of Equation 10-1b.

The tabulated minimum thicknesses of the supported beam web for welds A and the support for welds B match the shear rupture strength of these elements with the strength of the weld metal. As derived in Part 9, the minimum supported beam web thickness for welds A (two lines of weld) is

$$t_{min} = \frac{6.19D}{F_u} \quad (9\text{-}3)$$

and the minimum supporting flange or web thickness for welds B (one line of weld) is

$$t_{min} = \frac{3.09D}{F_u} \tag{9-2}$$

When welds B line up on opposite sides of the support, the minimum thickness is the sum of the thicknesses required for each weld. In either case, when less than the minimum material thickness is present, the tabulated weld available strength must be reduced by the ratio of the thickness provided to the minimum thickness.

When Table 10-2 is used, the minimum angle thickness is the weld size plus $^{1}/_{16}$ in., but not less than the angle thickness determined from Table 10-1. The angle length, *l*, must be as tabulated in Table 10-2. The width of outstanding legs in Case II (web legs bolted and outstanding legs welded) may be optionally reduced from 4 in. to 3 in. for values of *l* from $5^{1}/_{2}$ through $17^{1}/_{2}$ in.

Interpolation between values in this table may produce an incorrect result.

Table 10-3. Available Weld Strength of All-Welded Double-Angle Connections

Table 10-3 is a design aid for all-welded double-angle connections. Electrode strength is assumed to be 70 ksi. Holes for erection bolts may be placed as required in angle legs that are to be field-welded.

Weld available strengths are tabulated for the limit state of weld shear. Available strengths for welds A are determined by the instantaneous center of rotation method using Table 8-8 with θ = 0°. Available strengths for welds B are determined by the elastic method as discussed previously for bolted/welded double-angle connections.

The tabulated minimum thicknesses of the supported beam web for welds A and the support for welds B match the shear rupture strength of these elements with the strength of the weld metal and are determined as discussed previously for Table 10-2. When welds B line up on opposite sides of the support, the minimum thickness is the sum of the thicknesses required for each weld. When less than the minimum material thickness is present, the tabulated weld available strength must be reduced by the ratio of the thickness provided to the minimum thickness.

When Table 10-3 is used, the minimum angle thickness must be equal to the weld size plus $^{1}/_{16}$ in. The angle length, *l*, must be as tabulated in Table 10-3. $2L4\times3^{1}/_{2}$ should be used for angle lengths equal to or greater than 18 in. For angle length less than 18 in., the 4-in. leg can be reduced to 3 in.

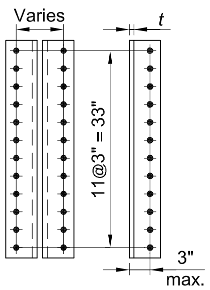
F_y = 36 ksi
Angles

Table 10-1
All-Bolted Double-Angle Connections

$^3/_4$-in.
Bolts

Bolt and Angle Available Strength, kips

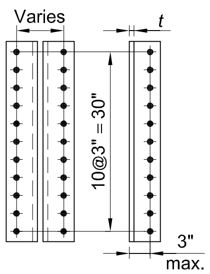
12 Rows
W44



Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
Group A	N	STD/SSLT	197	296	246	370	284	427	286	429
	X	STD/SSLT	197	296	246	370	296	444	360	540
	SC Class A	STD	152	228	152	228	152	228	152	228
		OVS	129	194	129	194	129	194	129	194
		SSLT	152	228	152	228	152	228	152	228
	SC Class B	STD	197	296	246	370	253	380	253	380
		OVS	197	296	215	321	216	323	216	323
		SSLT	197	296	246	370	253	380	253	380
Group B	N	STD/SSLT	197	296	246	370	296	444	360	540
	X	STD/SSLT	197	296	246	370	296	444	394	592
	SC Class A	STD	189	283	190	285	190	285	190	285
		OVS	162	242	162	242	162	242	162	242
		SSLT	189	283	190	285	190	285	190	285
	SC Class B	STD	197	296	246	370	296	444	316	475
		OVS	197	296	246	370	268	400	270	403
		SSLT	197	296	246	370	296	444	316	475

Bolt and Angle Available Strength, kips

11 Rows
W44, 40



Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
Group A	N	STD/SSLT	181	271	226	339	261	391	262	394
	X	STD/SSLT	181	271	226	339	271	407	330	495
	SC Class A	STD	139	209	139	209	139	209	139	209
		OVS	119	178	119	178	119	178	119	178
		SSLT	139	209	139	209	139	209	139	209
	SC Class B	STD	181	271	226	339	232	348	232	348
		OVS	181	271	197	294	198	296	198	296
		SSLT	181	271	226	339	232	348	232	348
Group B	N	STD/SSLT	181	271	226	339	271	407	330	495
	X	STD/SSLT	181	271	226	339	271	407	362	543
	SC Class A	STD	173	259	174	261	174	261	174	261
		OVS	148	222	148	222	148	222	148	222
		SSLT	173	259	174	261	174	261	174	261
	SC Class B	STD	181	271	226	339	271	407	290	435
		OVS	181	271	226	339	245	367	247	370
		SSLT	181	271	226	339	271	407	290	435

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

Slip-critical bolt values assume no more than one filler has been provided.

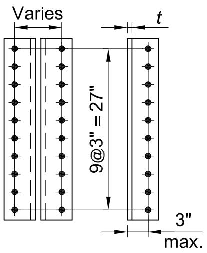
Table 10-1 (continued) All-Bolted Double-Angle Connections

F_y = 36 ksi Angles

3/4-in. Bolts

Bolt and Angle Available Strength, kips

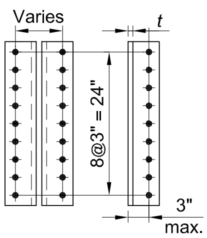
10 Rows — W44, 40, 36



Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
Group A	N	STD/SSLT	165	247	206	309	237	355	239	358
	X	STD/SSLT	165	247	206	309	247	371	300	450
	SC Class A	STD	127	190	127	190	127	190	127	190
		OVS	108	161	108	161	108	161	108	161
		SSLT	127	190	127	190	127	190	127	190
	SC Class B	STD	165	247	206	309	211	316	211	316
		OVS	165	247	179	268	180	269	180	269
		SSLT	165	247	206	309	211	316	211	316
Group B	N	STD/SSLT	165	247	206	309	247	371	300	450
	X	STD/SSLT	165	247	206	309	247	371	330	494
	SC Class A	STD	157	236	158	237	158	237	158	237
		OVS	135	202	135	202	135	202	135	202
		SSLT	157	236	158	237	158	237	158	237
	SC Class B	STD	165	247	206	309	247	371	264	396
		OVS	165	247	206	309	223	333	225	336
		SSLT	165	247	206	309	247	371	264	396

Bolt and Angle Available Strength, kips

9 Rows — W44, 40, 36, 33



Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
Group A	N	STD/SSLT	149	223	186	279	213	319	215	322
	X	STD/SSLT	149	223	186	279	223	334	270	405
	SC Class A	STD	114	171	114	171	114	171	114	171
		OVS	97.1	145	97.1	145	97.1	145	97.1	145
		SSLT	114	171	114	171	114	171	114	171
	SC Class B	STD	149	223	186	279	190	285	190	285
		OVS	149	223	161	241	162	242	162	242
		SSLT	149	223	186	279	190	285	190	285
Group B	N	STD/SSLT	149	223	186	279	223	334	270	405
	X	STD/SSLT	149	223	186	279	223	334	297	446
	SC Class A	STD	141	212	142	214	142	214	142	214
		OVS	121	182	121	182	121	182	121	182
		SSLT	141	212	142	214	142	214	142	214
	SC Class B	STD	149	223	186	279	223	334	237	356
		OVS	149	223	186	279	200	300	202	303
		SSLT	149	223	186	279	223	334	237	356

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

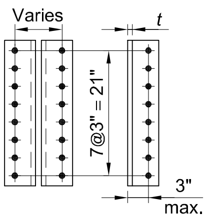
Slip-critical bolt values assume no more than one filler has been provided.

F_y = 36 ksi
Angles

Table 10-1 (continued)
All-Bolted Double-Angle Connections

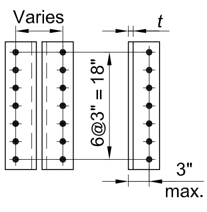
$^3/_4$-in.
Bolts

Bolt and Angle Available Strength, kips



8 Rows W44, 40, 36, 33, 30	Bolt Group	Thread Cond.	Hole Type	Angle Thickness, in. $^1/_4$ ASD	$^1/_4$ LRFD	$^5/_{16}$ ASD	$^5/_{16}$ LRFD	$^3/_8$ ASD	$^3/_8$ LRFD	$^1/_2$ ASD	$^1/_2$ LRFD
	Group A	N	STD/SSLT	132	199	165	248	189	284	191	286
		X	STD/SSLT	132	199	165	248	199	298	240	359
		SC Class A	STD	101	152	101	152	101	152	101	152
			OVS	86.3	129	86.3	129	86.3	129	86.3	129
			SSLT	101	152	101	152	101	152	101	152
		SC Class B	STD	132	199	165	248	169	253	169	253
			OVS	132	199	143	214	144	215	144	215
			SSLT	132	199	165	248	169	253	169	253
	Group B	N	STD/SSLT	132	199	165	248	199	298	240	359
		X	STD/SSLT	132	199	165	248	199	298	265	397
		SC Class A	STD	125	188	127	190	127	190	127	190
			OVS	108	161	108	161	108	161	108	161
			SSLT	125	188	127	190	127	190	127	190
		SC Class B	STD	132	199	165	248	199	298	211	316
			OVS	132	199	165	248	178	266	180	269
			SSLT	132	199	165	248	199	298	211	316

Bolt and Angle Available Strength, kips



7 Rows W44, 40, 36, 33, 30, 27, 24	Bolt Group	Thread Cond.	Hole Type	Angle Thickness, in. $^1/_4$ ASD	$^1/_4$ LRFD	$^5/_{16}$ ASD	$^5/_{16}$ LRFD	$^3/_8$ ASD	$^3/_8$ LRFD	$^1/_2$ ASD	$^1/_2$ LRFD
	Group A	N	STD/SSLT	116	174	145	218	165	248	167	250
		X	STD/SSLT	116	174	145	218	174	261	210	314
		SC Class A	STD	88.6	133	88.6	133	88.6	133	88.6	133
			OVS	75.5	113	75.5	113	75.5	113	75.5	113
			SSLT	88.6	133	88.6	133	88.6	133	88.6	133
		SC Class B	STD	116	174	145	217	148	221	148	221
			OVS	116	174	125	187	126	188	126	188
			SSLT	116	174	145	217	148	221	148	221
	Group B	N	STD/SSLT	116	174	145	218	174	261	210	314
		X	STD/SSLT	116	174	145	218	174	261	232	349
		SC Class A	STD	110	164	111	166	111	166	111	166
			OVS	94.4	141	94.4	141	94.4	141	94.4	141
			SSLT	110	164	111	166	111	166	111	166
		SC Class B	STD	116	174	145	218	174	261	185	277
			OVS	116	174	145	218	155	232	157	235
			SSLT	116	174	145	218	174	261	185	277

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

Slip-critical bolt values assume no more than one filler has been provided.

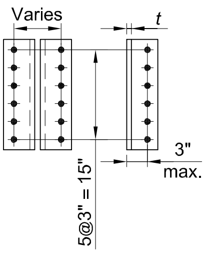
F_y = 36 ksi
Angles

Table 10-1 (continued)
All-Bolted Double-Angle Connections

3/4-in. Bolts

Bolt and Angle Available Strength, kips

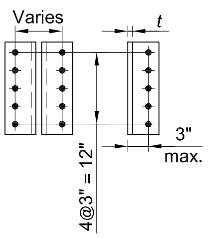
6 Rows: W40, 36, 33, 30, 27, 24, 21



Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
Group A	N	STD/SSLT	100	150	125	187	141	212	143	215
	X	STD/SSLT	100	150	125	187	150	225	180	269
	SC Class A	STD	75.9	114	75.9	114	75.9	114	75.9	114
		OVS	64.7	96.8	64.7	96.8	64.7	96.8	64.7	96.8
		SSLT	75.9	114	75.9	114	75.9	114	75.9	114
	SC Class B	STD	100	150	124	186	127	190	127	190
		OVS	100	150	107	160	108	161	108	161
		SSLT	100	150	124	186	127	190	127	190
Group B	N	STD/SSLT	100	150	125	187	150	225	180	269
	X	STD/SSLT	100	150	125	187	150	225	200	300
	SC Class A	STD	93.8	141	94.9	142	94.9	142	94.9	142
		OVS	80.9	121	80.9	121	80.9	121	80.9	121
		SSLT	93.8	141	94.9	142	94.9	142	94.9	142
	SC Class B	STD	100	150	125	187	150	225	158	237
		OVS	100	150	125	187	133	199	135	202
		SSLT	100	150	125	187	150	225	158	237

Bolt and Angle Available Strength, kips

5 Rows: W30, 27, 24, 21, 18



Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
Group A	N	STD/SSLT	83.8	126	105	157	117	176	119	179
	X	STD/SSLT	83.8	126	105	157	126	189	150	224
	SC Class A	STD	63.3	94.9	63.3	94.9	63.3	94.9	63.3	94.9
		OVS	53.9	80.7	53.9	80.7	53.9	80.7	53.9	80.7
		SSLT	63.3	94.9	63.3	94.9	63.3	94.9	63.3	94.9
	SC Class B	STD	83.8	126	103	154	105	158	105	158
		OVS	82.7	124	88.9	133	89.9	134	89.9	134
		SSLT	83.8	126	103	154	105	158	105	158
Group B	N	STD/SSLT	83.8	126	105	157	126	189	150	224
	X	STD/SSLT	83.8	126	105	157	126	189	168	251
	SC Class A	STD	78.0	117	79.1	119	79.1	119	79.1	119
		OVS	67.4	101	67.4	101	67.4	101	67.4	101
		SSLT	78.0	117	79.1	119	79.1	119	79.1	119
	SC Class B	STD	83.8	126	105	157	126	189	132	198
		OVS	82.7	124	103	155	110	165	112	168
		SSLT	83.8	126	105	157	126	189	132	198

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

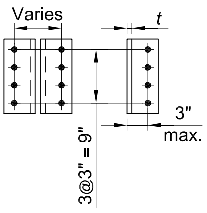
Slip-critical bolt values assume no more than one filler has been provided.

F_y = 36 ksi
Angles

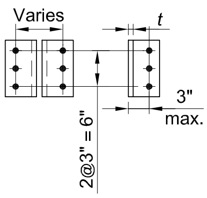
Table 10-1 (continued)
All-Bolted Double-Angle Connections

3/4-in. Bolts

Bolt and Angle Available Strength, kips

4 Rows W24, 21, 18, 16	Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	Group A	N	STD/SSLT	67.6	101	84.5	127	93.6	140	95.4	143
		X	STD/SSLT	67.6	101	84.5	127	101	152	119	179
		SC Class A	STD	50.6	75.9	50.6	75.9	50.6	75.9	50.6	75.9
			OVS	43.1	64.5	43.1	64.5	43.1	64.5	43.1	64.5
			SSLT	50.6	75.9	50.6	75.9	50.6	75.9	50.6	75.9
		SC Class B	STD	67.6	101	81.6	122	84.4	127	84.4	127
			OVS	65.3	97.9	70.9	106	71.9	108	71.9	108
			SSLT	67.6	101	81.6	122	84.4	127	84.4	127
	Group B	N	STD/SSLT	67.6	101	84.5	127	101	152	119	179
		X	STD/SSLT	67.6	101	84.5	127	101	152	135	203
		SC Class A	STD	62.1	93.2	63.3	94.9	63.3	94.9	63.3	94.9
			OVS	53.9	80.7	53.9	80.7	53.9	80.7	53.9	80.7
			SSLT	62.1	93.2	63.3	94.9	63.3	94.9	63.3	94.9
		SC Class B	STD	67.6	101	84.5	127	101	152	105	158
			OVS	65.3	97.9	81.6	122	87.8	131	89.9	134
			SSLT	67.6	101	84.5	127	101	152	105	158

Bolt and Angle Available Strength, kips

3 Rows W18, 16, 14, 12, 10+ +Ltd. to W10x12, 15, 17, 19, 22, 26, 30	Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	Group A	N	STD/SSLT	51.1	76.7	63.9	95.8	69.7	105	71.6	107
		X	STD/SSLT	51.1	76.7	63.9	95.8	76.7	115	89.4	134
		SC Class A	STD	38.0	57.0	38.0	57.0	38.0	57.0	38.0	57.0
			OVS	32.4	48.4	32.4	48.4	32.4	48.4	32.4	48.4
			SSLT	38.0	57.0	38.0	57.0	38.0	57.0	38.0	57.0
		SC Class B	STD	51.1	76.7	60.5	90.8	63.3	94.9	63.3	94.9
			OVS	47.9	71.8	52.9	79.3	53.9	80.7	53.9	80.7
			SSLT	51.1	76.7	60.5	90.8	63.3	94.9	63.3	94.9
	Group B	N	STD/SSLT	51.1	76.7	63.9	95.8	76.7	115	89.4	134
		X	STD/SSLT	51.1	76.7	63.9	95.8	76.7	115	102	153
		SC Class A	STD	46.3	69.5	47.5	71.2	47.5	71.2	47.5	71.2
			OVS	40.4	60.5	40.4	60.5	40.4	60.5	40.4	60.5
			SSLT	46.3	69.5	47.5	71.2	47.5	71.2	47.5	71.2
		SC Class B	STD	51.1	76.7	63.9	95.8	74.8	112	79.1	119
			OVS	47.9	71.8	59.8	89.7	65.3	97.8	67.4	101
			SSLT	51.1	76.7	63.9	95.8	74.8	112	79.1	119

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

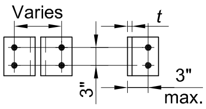
Slip-critical bolt values assume no more than one filler has been provided.

F_y = 36 ksi
Angles

Table 10-1 (continued)
All-Bolted Double-Angle Connections

3/4-in. Bolts

Bolt and Angle Available Strength, kips

2 Rows / W12, 10, 8	Bolt Group	Thread Cond.	Hole Type	Angle Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	Group A	N	STD/SSLT	32.6	48.9	40.8	61.2	45.9	68.8	47.7	71.6
		X	STD/SSLT	32.6	48.9	40.8	61.2	48.9	73.4	59.4	89.1
		SC Class A	STD	25.3	38.0	25.3	38.0	25.3	38.0	25.3	38.0
			OVS	21.6	32.3	21.6	32.3	21.6	32.3	21.6	32.3
			SSLT	25.3	38.0	25.3	38.0	25.3	38.0	25.3	38.0
		SC Class B	STD	32.6	48.9	39.4	59.2	42.2	63.3	42.2	63.3
			OVS	30.5	45.7	35.0	52.4	36.0	53.8	36.0	53.8
			SSLT	32.6	48.9	39.4	59.2	42.2	63.3	42.2	63.3
	Group B	N	STD/SSLT	32.6	48.9	40.8	61.2	48.9	73.4	59.4	89.1
		X	STD/SSLT	32.6	48.9	40.8	61.2	48.9	73.4	65.3	97.9
		SC Class A	STD	30.5	45.8	31.6	47.5	31.6	47.5	31.6	47.5
			OVS	27.0	40.3	27.0	40.3	27.0	40.3	27.0	40.3
			SSLT	30.5	45.8	31.6	47.5	31.6	47.5	31.6	47.5
		SC Class B	STD	32.6	48.9	40.8	61.2	48.4	72.6	52.7	79.1
			OVS	30.5	45.7	38.1	57.1	42.9	64.2	44.9	67.2
			SSLT	32.6	48.9	40.8	61.2	48.4	72.6	52.7	79.1

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

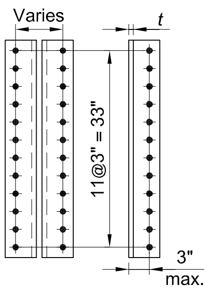
Slip-critical bolt values assume no more than one filler has been provided.

F_y = 36 ksi
Angles

Table 10-1 (continued)
All-Bolted Double-Angle Connections

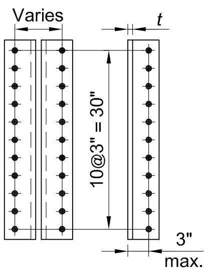
7/8-in.
Bolts

Bolt and Angle Available Strength, kips



12 Rows W44	Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	Group A	N	STD/SSLT	196	294	245	368	294	442	384	577
		X	STD/SSLT	196	294	245	368	294	442	393	589
		SC Class A	STD	196	294	211	316	212	317	212	317
			OVS	178	266	180	270	180	270	180	270
			SSLT	196	294	211	316	212	317	212	317
		SC Class B	STD	196	294	245	368	294	442	350	526
			OVS	191	287	239	359	287	431	300	450
			SSLT	196	294	245	368	294	442	350	526
	Group B	N	STD/SSLT	196	294	245	368	294	442	393	589
		X	STD/SSLT	196	294	245	368	294	442	393	589
		SC Class A	STD	196	294	245	368	264	396	266	399
			OVS	191	287	223	334	226	339	227	339
			SSLT	196	294	245	368	264	396	266	399
		SC Class B	STD	196	294	245	368	294	442	393	589
			OVS	191	287	239	359	287	431	371	555
			SSLT	196	294	245	368	294	442	393	589

Bolt and Angle Available Strength, kips



11 Rows W44, 40	Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	Group A	N	STD/SSLT	180	270	225	338	270	405	352	528
		X	STD/SSLT	180	270	225	338	270	405	360	540
		SC Class A	STD	180	270	193	290	194	291	194	291
			OVS	163	244	165	247	165	247	165	247
			SSLT	180	270	193	290	194	291	194	291
		SC Class B	STD	180	270	225	338	270	405	321	481
			OVS	175	263	219	328	263	394	275	412
			SSLT	180	270	225	338	270	405	321	481
	Group B	N	STD/SSLT	180	270	225	338	270	405	360	540
		X	STD/SSLT	180	270	225	338	270	405	360	540
		SC Class A	STD	180	270	225	338	242	363	244	365
			OVS	175	263	204	306	208	311	208	311
			SSLT	180	270	225	338	242	363	244	365
		SC Class B	STD	180	270	225	338	270	405	360	540
			OVS	175	263	219	328	263	394	340	508
			SSLT	180	270	225	338	270	405	360	540

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

Slip-critical bolt values assume no more than one filler has been provided.

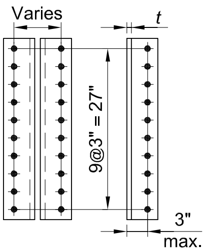
F_y = 36 ksi
Angles

Table 10-1 (continued)
All-Bolted Double-Angle Connections

7/8-in. Bolts

Bolt and Angle Available Strength, kips

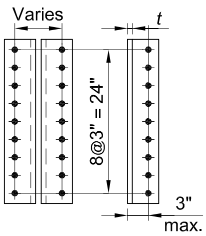
10 Rows — W44, 40, 36



Bolt Group	Thread Cond.	Hole Type	Angle Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
Group A	N	STD/SSLT	164	246	205	307	246	369	319	479
	X	STD/SSLT	164	246	205	307	246	369	328	492
	SC Class A	STD	164	246	176	263	176	264	176	264
		OVS	148	221	150	225	150	225	150	225
		SSLT	164	246	176	263	176	264	176	264
	SC Class B	STD	164	246	205	307	246	369	292	437
		OVS	159	238	198	298	238	357	250	375
		SSLT	164	246	205	307	246	369	292	437
Group B	N	STD/SSLT	164	246	205	307	246	369	328	492
	X	STD/SSLT	164	246	205	307	246	369	328	492
	SC Class A	STD	164	246	205	307	220	330	221	332
		OVS	159	238	186	278	189	282	189	282
		SSLT	164	246	205	307	220	330	221	332
	SC Class B	STD	164	246	205	307	246	369	328	492
		OVS	159	238	198	298	238	357	308	461
		SSLT	164	246	205	307	246	369	328	492

Bolt and Angle Available Strength, kips

9 Rows — W44, 40, 36, 33



Bolt Group	Thread Cond.	Hole Type	Angle Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
Group A	N	STD/SSLT	148	222	185	277	222	332	287	430
	X	STD/SSLT	148	222	185	277	222	332	295	443
	SC Class A	STD	148	222	158	237	159	238	159	238
		OVS	133	199	135	202	135	202	135	202
		SSLT	148	222	158	237	159	238	159	238
	SC Class B	STD	148	222	185	277	222	332	262	393
		OVS	142	214	178	267	214	321	225	337
		SSLT	148	222	185	277	222	332	262	393
Group B	N	STD/SSLT	148	222	185	277	222	332	295	443
	X	STD/SSLT	148	222	185	277	222	332	295	443
	SC Class A	STD	148	222	185	277	198	296	199	299
		OVS	142	214	167	249	170	254	170	254
		SSLT	148	222	185	277	198	296	199	299
	SC Class B	STD	148	222	185	277	222	332	295	443
		OVS	142	214	178	267	214	321	277	414
		SSLT	148	222	185	277	222	332	295	443

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

Slip-critical bolt values assume no more than one filler has been provided.

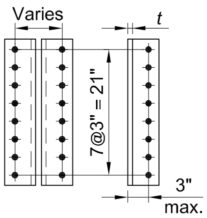
Table 10-1 (continued)
All-Bolted Double-Angle Connections

F_y = 36 ksi Angles

$^7/_8$-in. Bolts

Bolt and Angle Available Strength, kips

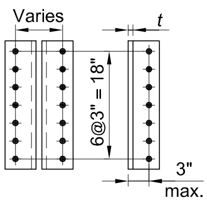
8 Rows — W44, 40, 36, 33, 30



Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
Group A	N	STD/SSLT	131	197	164	247	197	296	254	382
	X	STD/SSLT	131	197	164	247	197	296	263	394
	SC Class A	STD	131	197	140	211	141	212	141	212
		OVS	118	176	120	180	120	180	120	180
		SSLT	131	197	140	211	141	212	141	212
	SC Class B	STD	131	197	164	247	197	296	233	349
		OVS	126	189	158	237	189	284	200	300
		SSLT	131	197	164	247	197	296	233	349
Group B	N	STD/SSLT	131	197	164	247	197	296	263	394
	X	STD/SSLT	131	197	164	247	197	296	263	394
	SC Class A	STD	131	197	164	247	175	263	177	266
		OVS	126	189	148	221	151	226	151	226
		SSLT	131	197	164	247	175	263	177	266
	SC Class B	STD	131	197	164	247	197	296	263	394
		OVS	126	189	158	237	189	284	245	367
		SSLT	131	197	164	247	197	296	263	394

Bolt and Angle Available Strength, kips

7 Rows — W44, 40, 36, 33, 30, 27, 24



Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
Group A	N	STD/SSLT	115	173	144	216	173	259	222	333
	X	STD/SSLT	115	173	144	216	173	259	231	346
	SC Class A	STD	115	173	123	184	123	185	123	185
		OVS	103	154	105	157	105	157	105	157
		SSLT	115	173	123	184	123	185	123	185
	SC Class B	STD	115	173	144	216	173	259	203	305
		OVS	110	165	137	206	165	247	175	262
		SSLT	115	173	144	216	173	259	203	305
Group B	N	STD/SSLT	115	173	144	216	173	259	231	346
	X	STD/SSLT	115	173	144	216	173	259	231	346
	SC Class A	STD	115	173	144	216	153	230	155	233
		OVS	110	165	129	193	132	198	132	198
		SSLT	115	173	144	216	153	230	155	233
	SC Class B	STD	115	173	144	216	173	259	231	346
		OVS	110	165	137	206	165	247	214	320
		SSLT	115	173	144	216	173	259	231	346

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

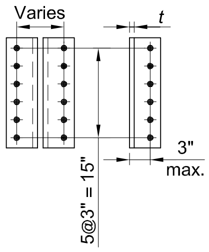
Slip-critical bolt values assume no more than one filler has been provided.

F_y = 36 ksi Angles

Table 10-1 (continued)
All-Bolted Double-Angle Connections

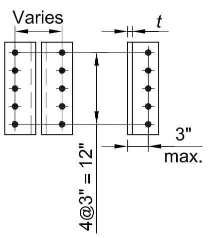
7/8-in. Bolts

Bolt and Angle Available Strength, kips



6 Rows	Bolt Group	Thread Cond.	Hole Type	Angle Thickness, in.							
W40, 36, 33, 30, 27, 24, 21				1/4		5/16		3/8		1/2	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	Group A	N	STD/SSLT	99.1	149	124	186	149	223	190	284
		X	STD/SSLT	99.1	149	124	186	149	223	198	297
		SC Class A	STD	99.1	149	105	158	106	159	106	159
			OVS	87.6	131	90.1	135	90.1	135	90.1	135
			SSLT	99.1	149	105	158	106	159	106	159
		SC Class B	STD	99.1	149	124	186	149	223	174	261
			OVS	93.5	140	117	175	140	210	150	225
			SSLT	99.1	149	124	186	149	223	174	261
	Group B	N	STD/SSLT	99.1	149	124	186	149	223	198	297
		X	STD/SSLT	99.1	149	124	186	149	223	198	297
		SC Class A	STD	99.1	149	124	186	131	197	133	199
			OVS	93.5	140	110	165	113	169	113	169
			SSLT	99.1	149	124	186	131	197	133	199
		SC Class B	STD	99.1	149	124	186	149	223	198	297
			OVS	93.5	140	117	175	140	210	182	273
			SSLT	99.1	149	124	186	149	223	198	297

Bolt and Angle Available Strength, kips



5 Rows	Bolt Group	Thread Cond.	Hole Type	Angle Thickness, in.							
W30, 27, 24, 21, 18				1/4		5/16		3/8		1/2	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	Group A	N	STD/SSLT	82.7	124	103	155	124	186	157	236
		X	STD/SSLT	82.7	124	103	155	124	186	165	248
		SC Class A	STD	82.7	124	87.5	131	88.1	132	88.1	132
			OVS	72.6	109	75.1	112	75.1	112	75.1	112
			SSLT	82.7	124	87.5	131	88.1	132	88.1	132
		SC Class B	STD	82.7	124	103	155	124	186	145	217
			OVS	77.2	116	96.5	145	116	174	125	187
			SSLT	82.7	124	103	155	124	186	145	217
	Group B	N	STD/SSLT	82.7	124	103	155	124	186	165	248
		X	STD/SSLT	82.7	124	103	155	124	186	165	248
		SC Class A	STD	82.7	124	103	155	109	163	111	166
			OVS	77.2	116	91.1	136	94.3	141	94.4	141
			SSLT	82.7	124	103	155	109	163	111	166
		SC Class B	STD	82.7	124	103	155	124	186	165	248
			OVS	77.2	116	96.5	145	116	174	151	226
			SSLT	82.7	124	103	155	124	186	165	248

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

Slip-critical bolt values assume no more than one filler has been provided.

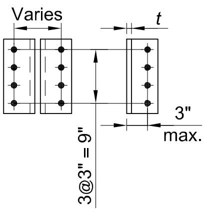
Table 10-1 (continued)
All-Bolted Double-Angle Connections

F_y = 36 ksi Angles

7/8-in. Bolts

Bolt and Angle Available Strength, kips

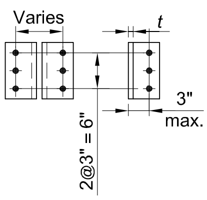
4 Rows — W24, 21, 18, 16



Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
Group A	N	STD/SSLT	65.3	97.9	81.6	122	97.9	147	125	187
	X	STD/SSLT	65.3	97.9	81.6	122	97.9	147	131	196
	SC Class A	STD	65.3	97.9	69.9	105	70.5	106	70.5	106
		OVS	57.6	86.2	60.1	89.9	60.1	89.9	60.1	89.9
		SSLT	65.3	97.9	69.9	105	70.5	106	70.5	106
	SC Class B	STD	65.3	97.9	81.6	122	97.9	147	115	173
		OVS	60.9	91.4	76.1	114	91.4	137	100	150
		SSLT	65.3	97.9	81.6	122	97.9	147	115	173
Group B	N	STD/SSLT	65.3	97.9	81.6	122	97.9	147	131	196
	X	STD/SSLT	65.3	97.9	81.6	122	97.9	147	131	196
	SC Class A	STD	65.3	97.9	81.6	122	86.8	130	88.6	133
		OVS	60.9	91.4	72.3	108	75.4	113	75.5	113
		SSLT	65.3	97.9	81.6	122	86.8	130	88.6	133
	SC Class B	STD	65.3	97.9	81.6	122	97.9	147	131	196
		OVS	60.9	91.4	76.1	114	91.4	137	119	179
		SSLT	65.3	97.9	81.6	122	97.9	147	131	196

Bolt and Angle Available Strength, kips

3 Rows — W18, 16, 14, 12, 10+
+Ltd. to W10x12, 15, 17, 19, 22, 26, 30



Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
Group A	N	STD/SSLT	47.9	71.8	59.8	89.7	71.8	108	92.1	138
	X	STD/SSLT	47.9	71.8	59.8	89.7	71.8	108	95.7	144
	SC Class A	STD	47.9	71.8	52.2	78.4	52.9	79.3	52.9	79.3
		OVS	42.6	63.7	45.1	67.4	45.1	67.4	45.1	67.4
		SSLT	47.9	71.8	52.2	78.4	52.9	79.3	52.9	79.3
	SC Class B	STD	47.9	71.8	59.8	89.7	71.8	108	85.9	129
		OVS	44.6	66.9	55.7	83.6	66.9	100	75.1	112
		SSLT	47.9	71.8	59.8	89.7	71.8	108	85.9	129
Group B	N	STD/SSLT	47.9	71.8	59.8	89.7	71.8	108	95.7	144
	X	STD/SSLT	47.9	71.8	59.8	89.7	71.8	108	95.7	144
	SC Class A	STD	47.9	71.8	59.8	89.7	64.7	97.0	66.4	99.7
		OVS	44.6	66.9	53.4	79.9	56.5	84.6	56.6	84.7
		SSLT	47.9	71.8	59.8	89.7	64.7	97.0	66.4	99.7
	SC Class B	STD	47.9	71.8	59.8	89.7	71.8	108	95.7	144
		OVS	44.6	66.9	55.7	83.6	66.9	100	87.9	132
		SSLT	47.9	71.8	59.8	89.7	71.8	108	95.7	144

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

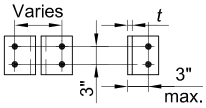
Slip-critical bolt values assume no more than one filler has been provided.

F_y = 36 ksi
Angles

Table 10-1 (continued)
All-Bolted Double-Angle Connections

7/8-in.
Bolts

Bolt and Angle Available Strength, kips



2 Rows W12, 10, 8	Bolt Group	Thread Cond.	Hole Type	Angle Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	Group A	N	STD/SSLT	30.5	45.7	38.1	57.1	45.7	68.5	59.7	89.5
		X	STD/SSLT	30.5	45.7	38.1	57.1	45.7	68.5	60.9	91.4
		SC Class A	STD	30.5	45.7	34.6	51.9	35.3	52.9	35.3	52.9
			OVS	27.5	41.2	30.0	45.0	30.0	45.0	30.0	45.0
			SSLT	30.5	45.7	34.6	51.9	35.3	52.9	35.3	52.9
		SC Class B	STD	30.5	45.7	38.1	57.1	45.7	68.5	56.6	84.9
			OVS	28.3	42.4	35.3	53.0	42.4	63.6	50.1	74.9
			SSLT	30.5	45.7	38.1	57.1	45.7	68.5	56.6	84.9
	Group B	N	STD/SSLT	30.5	45.7	38.1	57.1	45.7	68.5	60.9	91.4
		X	STD/SSLT	30.5	45.7	38.1	57.1	45.7	68.5	60.9	91.4
		SC Class A	STD	30.5	45.7	38.1	57.1	42.5	63.8	44.3	66.4
			OVS	28.3	42.4	34.5	51.7	37.6	56.4	37.8	56.5
			SSLT	30.5	45.7	38.1	57.1	42.5	63.8	44.3	66.4
		SC Class B	STD	30.5	45.7	38.1	57.1	45.7	68.5	60.9	91.4
			OVS	28.3	42.4	35.3	53.0	42.4	63.6	56.5	84.6
			SSLT	30.5	45.7	38.1	57.1	45.7	68.5	60.9	91.4

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

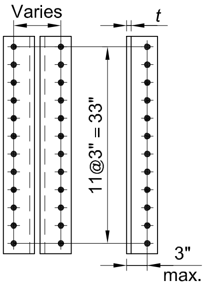
Note: Slip-critical bolt values assume no more than one filler has been provided.

F_y = 36 ksi
Angles

Table 10-1 (continued)
All-Bolted Double-Angle Connections

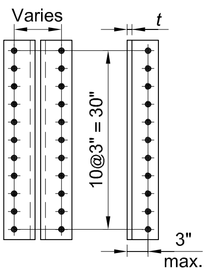
1-in.
Bolts

Bolt and Angle Available Strength, kips



12 Rows W44	Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	Group A	N	STD/SSLT	185	277	231	347	277	416	370	555
		X	STD/SSLT	185	277	231	347	277	416	370	555
		SC Class A	STD	185	277	231	347	272	407	277	415
			OVS	172	258	215	322	232	348	236	353
			SSLT	185	277	231	347	272	407	277	415
		SC Class B	STD	185	277	231	347	277	416	370	555
			OVS	172	258	215	322	258	387	344	515
			SSLT	185	277	231	347	277	416	370	555
	Group B	N	STD/SSLT	185	277	231	347	277	416	370	555
		X	STD/SSLT	185	277	231	347	277	416	370	555
		SC Class A	STD	185	277	231	347	277	416	342	513
			OVS	172	258	215	322	258	387	293	438
			SSLT	185	277	231	347	277	416	342	513
		SC Class B	STD	185	277	231	347	277	416	370	555
			OVS	172	258	215	322	258	387	344	515
			SSLT	185	277	231	347	277	416	370	555

Bolt and Angle Available Strength, kips



11 Rows W44, 40	Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	Group A	N	STD/SSLT	169	254	211	317	254	380	338	507
		X	STD/SSLT	169	254	211	317	254	380	338	507
		SC Class A	STD	169	254	211	317	248	373	254	380
			OVS	157	236	196	295	213	318	216	323
			SSLT	169	254	211	317	248	373	254	380
		SC Class B	STD	169	254	211	317	254	380	338	507
			OVS	157	236	196	295	236	354	314	471
			SSLT	169	254	211	317	254	380	338	507
	Group B	N	STD/SSLT	169	254	211	317	254	380	338	507
		X	STD/SSLT	169	254	211	317	254	380	338	507
		SC Class A	STD	169	254	211	317	254	380	313	470
			OVS	157	236	196	295	236	354	268	401
			SSLT	169	254	211	317	254	380	313	470
		SC Class B	STD	169	254	211	317	254	380	338	507
			OVS	157	236	196	295	236	354	314	471
			SSLT	169	254	211	317	254	380	338	507

Angle Thickness, in.

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

Slip-critical bolt values assume no more than one filler has been provided.

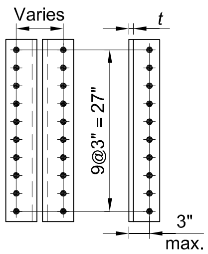
F_y = 36 ksi
Angles

Table 10-1 (continued)
All-Bolted Double-Angle Connections

1-in. Bolts

Bolt and Angle Available Strength, kips

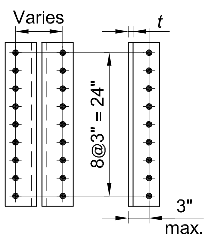
10 Rows — W44, 40, 36



Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
Group A	N	STD/SSLT	153	230	192	288	230	345	307	460
	X	STD/SSLT	153	230	192	288	230	345	307	460
	SC Class A	STD	153	230	192	288	225	338	231	346
		OVS	142	214	178	267	193	289	196	294
		SSLT	153	230	192	288	225	338	231	346
	SC Class B	STD	153	230	192	288	230	345	307	460
		OVS	142	214	178	267	214	321	285	427
		SSLT	153	230	192	288	230	345	307	460
Group B	N	STD/SSLT	153	230	192	288	230	345	307	460
	X	STD/SSLT	153	230	192	288	230	345	307	460
	SC Class A	STD	153	230	192	288	230	345	284	426
		OVS	142	214	178	267	214	321	244	365
		SSLT	153	230	192	288	230	345	284	426
	SC Class B	STD	153	230	192	288	230	345	307	460
		OVS	142	214	178	267	214	321	285	427
		SSLT	153	230	192	288	230	345	307	460

Bolt and Angle Available Strength, kips

9 Rows — W44, 40, 36, 33



Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
Group A	N	STD/SSLT	138	206	172	258	206	310	275	413
	X	STD/SSLT	138	206	172	258	206	310	275	413
	SC Class A	STD	138	206	172	258	202	304	207	311
		OVS	128	192	160	240	173	260	177	265
		SSLT	138	206	172	258	202	304	207	311
	SC Class B	STD	138	206	172	258	206	310	275	413
		OVS	128	192	160	240	192	288	256	383
		SSLT	138	206	172	258	206	310	275	413
Group B	N	STD/SSLT	138	206	172	258	206	310	275	413
	X	STD/SSLT	138	206	172	258	206	310	275	413
	SC Class A	STD	138	206	172	258	206	310	255	383
		OVS	128	192	160	240	192	288	219	328
		SSLT	138	206	172	258	206	310	255	383
	SC Class B	STD	138	206	172	258	206	310	275	413
		OVS	128	192	160	240	192	288	256	383
		SSLT	138	206	172	258	206	310	275	413

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

Slip-critical bolt values assume no more than one filler has been provided.

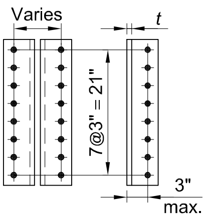
F_y = 36 ksi
Angles

Table 10-1 (continued)
All-Bolted Double-Angle Connections

1-in.
Bolts

Bolt and Angle Available Strength, kips

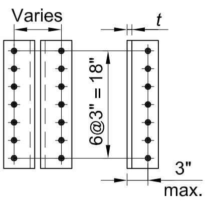
8 Rows
W44, 40, 36, 33, 30



Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
Group A	N	STD/SSLT	122	183	152	228	183	274	244	365
	X	STD/SSLT	122	183	152	228	183	274	244	365
	SC Class A	STD	122	183	152	228	179	269	184	277
		OVS	113	170	141	212	154	230	157	235
		SSLT	122	183	152	228	179	269	184	277
	SC Class B	STD	122	183	152	228	183	274	244	365
		OVS	113	170	141	212	170	254	226	339
		SSLT	122	183	152	228	183	274	244	365
Group B	N	STD/SSLT	122	183	152	228	183	274	244	365
	X	STD/SSLT	122	183	152	228	183	274	244	365
	SC Class A	STD	122	183	152	228	183	274	226	340
		OVS	113	170	141	212	170	254	194	291
		SSLT	122	183	152	228	183	274	226	340
	SC Class B	STD	122	183	152	228	183	274	244	365
		OVS	113	170	141	212	170	254	226	339
		SSLT	122	183	152	228	183	274	244	365

Bolt and Angle Available Strength, kips

7 Rows
W44, 40, 36, 33, 30, 27, 24



Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
Group A	N	STD/SSLT	106	159	133	199	159	239	212	318
	X	STD/SSLT	106	159	133	199	159	239	212	318
	SC Class A	STD	106	159	133	199	156	234	161	242
		OVS	98.4	148	123	185	134	201	138	206
		SSLT	106	159	133	199	156	234	161	242
	SC Class B	STD	106	159	133	199	159	239	212	318
		OVS	98.4	148	123	185	148	221	197	295
		SSLT	106	159	133	199	159	239	212	318
Group B	N	STD/SSLT	106	159	133	199	159	239	212	318
	X	STD/SSLT	106	159	133	199	159	239	212	318
	SC Class A	STD	106	159	133	199	159	239	197	296
		OVS	98.4	148	123	185	148	221	170	254
		SSLT	106	159	133	199	159	239	197	296
	SC Class B	STD	106	159	133	199	159	239	212	318
		OVS	98.4	148	123	185	148	221	197	295
		SSLT	106	159	133	199	159	239	212	318

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

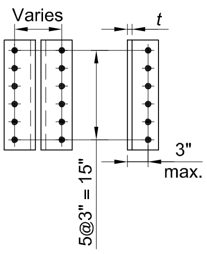
Slip-critical bolt values assume no more than one filler has been provided.

F_y = 36 ksi
Angles

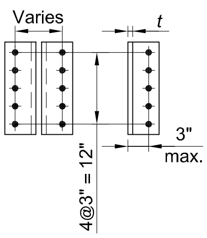
Table 10-1 (continued)
All-Bolted Double-Angle Connections

1-in.
Bolts

Bolt and Angle Available Strength, kips

6 Rows W40, 36, 33, 30, 27, 24, 21	Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	Group A	N	STD/SSLT	90.3	135	113	169	135	203	181	271
		X	STD/SSLT	90.3	135	113	169	135	203	181	271
		SC Class A	STD	90.3	135	113	169	133	200	138	207
			OVS	83.7	126	105	157	115	171	118	176
			SSLT	90.3	135	113	169	133	200	138	207
		SC Class B	STD	90.3	135	113	169	135	203	181	271
			OVS	83.7	126	105	157	126	188	167	251
			SSLT	90.3	135	113	169	135	203	181	271
	Group B	N	STD/SSLT	90.3	135	113	169	135	203	181	271
		X	STD/SSLT	90.3	135	113	169	135	203	181	271
		SC Class A	STD	90.3	135	113	169	135	203	169	253
			OVS	83.7	126	105	157	126	188	145	217
			SSLT	90.3	135	113	169	135	203	169	253
		SC Class B	STD	90.3	135	113	169	135	203	181	271
			OVS	83.7	126	105	157	126	188	167	251
			SSLT	90.3	135	113	169	135	203	181	271

Bolt and Angle Available Strength, kips

5 Rows W30, 27, 24, 21, 18	Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	Group A	N	STD/SSLT	74.5	112	93.1	140	112	168	149	223
		X	STD/SSLT	74.5	112	93.1	140	112	168	149	223
		SC Class A	STD	74.5	112	93.1	140	110	165	115	173
			OVS	69.1	104	86.3	129	94.9	142	98.2	147
			SSLT	74.5	112	93.1	140	110	165	115	173
		SC Class B	STD	74.5	112	93.1	140	112	168	149	223
			OVS	69.1	104	86.3	129	104	155	138	207
			SSLT	74.5	112	93.1	140	112	168	149	223
	Group B	N	STD/SSLT	74.5	112	93.1	140	112	168	149	223
		X	STD/SSLT	74.5	112	93.1	140	112	168	149	223
		SC Class A	STD	74.5	112	93.1	140	112	168	140	209
			OVS	69.1	104	86.3	129	104	155	120	180
			SSLT	74.5	112	93.1	140	112	168	140	209
		SC Class B	STD	74.5	112	93.1	140	112	168	149	223
			OVS	69.1	104	86.3	129	104	155	138	207
			SSLT	74.5	112	93.1	140	112	168	149	223

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

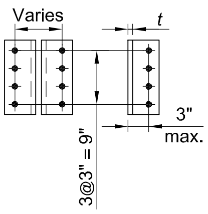
Note: Slip-critical bolt values assume no more than one filler has been provided.

F_y = 36 ksi Angles

Table 10-1 (continued) All-Bolted Double-Angle Connections

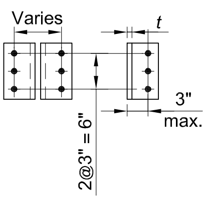
1-in. Bolts

Bolt and Angle Available Strength, kips



4 Rows W24, 21, 18, 16	Bolt Group	Thread Cond.	Hole Type	Angle Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	Group A	N	STD/SSLT	58.7	88.1	73.4	110	88.1	132	117	176
		X	STD/SSLT	58.7	88.1	73.4	110	88.1	132	117	176
		SC Class A	STD	58.7	88.1	73.4	110	87.1	131	92.2	138
			OVS	54.4	81.6	68.0	102	75.3	113	78.6	118
			SSLT	58.7	88.1	73.4	110	87.1	131	92.2	138
		SC Class B	STD	58.7	88.1	73.4	110	88.1	132	117	176
			OVS	54.4	81.6	68.0	102	81.6	122	109	163
			SSLT	58.7	88.1	73.4	110	88.1	132	117	176
	Group B	N	STD/SSLT	58.7	88.1	73.4	110	88.1	132	117	176
		X	STD/SSLT	58.7	88.1	73.4	110	88.1	132	117	176
		SC Class A	STD	58.7	88.1	73.4	110	88.1	132	111	166
			OVS	54.4	81.6	68.0	102	81.6	122	95.7	143
			SSLT	58.7	88.1	73.4	110	88.1	132	111	166
		SC Class B	STD	58.7	88.1	73.4	110	88.1	132	117	176
			OVS	54.4	81.6	68.0	102	81.6	122	109	163
			SSLT	58.7	88.1	73.4	110	88.1	132	117	176

Bolt and Angle Available Strength, kips



3 Rows W18, 16, 14, 12, 10+ +Ltd. to W10x12, 15, 17, 19, 22, 26, 30	Bolt Group	Thread Cond.	Hole Type	Angle Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	Group A	N	STD/SSLT	43.0	64.4	53.7	80.5	64.4	96.7	85.9	129
		X	STD/SSLT	43.0	64.4	53.7	80.5	64.4	96.7	85.9	129
		SC Class A	STD	43.0	64.4	53.7	80.5	64.0	96.1	69.2	104
			OVS	39.7	59.5	49.6	74.4	55.6	83.3	58.9	88.2
			SSLT	43.0	64.4	53.7	80.5	64.0	96.1	69.2	104
		SC Class B	STD	43.0	64.4	53.7	80.5	64.4	96.7	85.9	129
			OVS	39.7	59.5	49.6	74.4	59.5	89.3	79.4	119
			SSLT	43.0	64.4	53.7	80.5	64.4	96.7	85.9	129
	Group B	N	STD/SSLT	43.0	64.4	53.7	80.5	64.4	96.7	85.9	129
		X	STD/SSLT	43.0	64.4	53.7	80.5	64.4	96.7	85.9	129
		SC Class A	STD	43.0	64.4	53.7	80.5	64.4	96.7	81.8	123
			OVS	39.7	59.5	49.6	74.4	59.5	89.3	71.1	106
			SSLT	43.0	64.4	53.7	80.5	64.4	96.7	81.8	123
		SC Class B	STD	43.0	64.4	53.7	80.5	64.4	96.7	85.9	129
			OVS	39.7	59.5	49.6	74.4	59.5	89.3	79.4	119
			SSLT	43.0	64.4	53.7	80.5	64.4	96.7	85.9	129

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

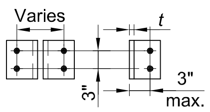
Slip-critical bolt values assume no more than one filler has been provided.

F_y = 36 ksi
Angles

Table 10-1 (continued)
All-Bolted Double-Angle Connections

1-in. Bolts

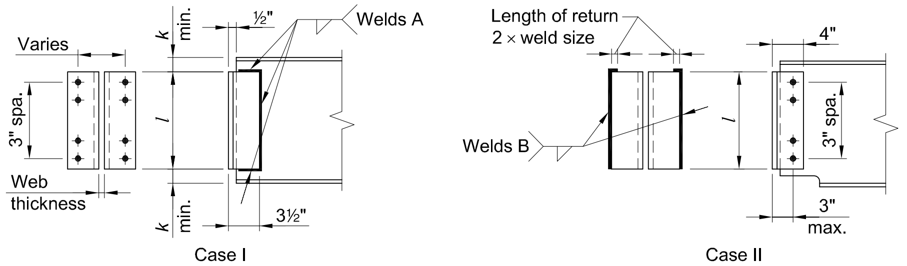
Bolt and Angle Available Strength, kips

2 Rows	Bolt Group	Thread Cond.	Hole Type	Angle Thickness, in.							
W12, 10, 8				1/4		5/16		3/8		1/2	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	Group A	N	STD/SSLT	27.2	40.8	34.0	51.0	40.8	61.2	54.4	81.6
		X	STD/SSLT	27.2	40.8	34.0	51.0	40.8	61.2	54.4	81.6
		SC Class A	STD	27.2	40.8	34.0	51.0	40.8	61.2	46.1	69.2
			OVS	25.0	37.5	31.3	46.9	36.0	53.9	39.3	58.8
			SSLT	27.2	40.8	34.0	51.0	40.8	61.2	46.1	69.2
		SC Class B	STD	27.2	40.8	34.0	51.0	40.8	61.2	54.4	81.6
			OVS	25.0	37.5	31.3	46.9	37.5	56.3	50.0	75.0
			SSLT	27.2	40.8	34.0	51.0	40.8	61.2	54.4	81.6
	Group B	N	STD/SSLT	27.2	40.8	34.0	51.0	40.8	61.2	54.4	81.6
		X	STD/SSLT	27.2	40.8	34.0	51.0	40.8	61.2	54.4	81.6
		SC Class A	STD	27.2	40.8	34.0	51.0	40.8	61.2	52.9	79.3
			OVS	25.0	37.5	31.3	46.9	37.5	56.3	46.4	69.5
			SSLT	27.2	40.8	34.0	51.0	40.8	61.2	52.9	79.3
		SC Class B	STD	27.2	40.8	34.0	51.0	40.8	61.2	54.4	81.6
			OVS	25.0	37.5	31.3	46.9	37.5	56.3	50.0	75.0
			SSLT	27.2	40.8	34.0	51.0	40.8	61.2	54.4	81.6

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

Slip-critical bolt values assume no more than one filler has been provided.

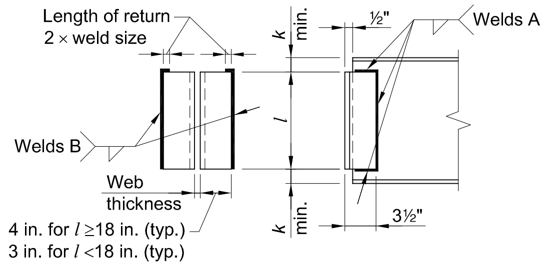
Table 10-2
Available Weld Strength of Bolted/Welded Double-Angle Connections



n	l, in.	Welds A (70 ksi) Weld Size, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	Minimum Web Thickness, in.	Welds B (70 ksi) Weld Size, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	Minimum Support Thickness, in.
12	35½	5/16	393	589	0.476	3/8	366	550	0.286
		1/4	314	471	0.381	5/16	305	458	0.238
		3/16	236	353	0.286	1/4	244	366	0.190
11	32½	5/16	365	548	0.476	3/8	331	496	0.286
		1/4	292	438	0.381	5/16	276	414	0.238
		3/16	219	329	0.286	1/4	221	331	0.190
10	29½	5/16	337	505	0.476	3/8	295	443	0.286
		1/4	269	404	0.381	5/16	246	369	0.238
		3/16	202	303	0.286	1/4	197	295	0.190
9	26½	5/16	309	463	0.476	3/8	259	389	0.286
		1/4	247	371	0.381	5/16	216	324	0.238
		3/16	185	278	0.286	1/4	173	259	0.190
8	23½	5/16	281	422	0.476	3/8	223	335	0.286
		1/4	225	337	0.381	5/16	186	279	0.238
		3/16	169	253	0.286	1/4	149	223	0.190
7	20½	5/16	253	379	0.476	3/8	187	280	0.286
		1/4	202	303	0.381	5/16	156	234	0.238
		3/16	152	227	0.286	1/4	125	187	0.190
6	17½	5/16	222	334	0.476	3/8	150	226	0.286
		1/4	178	267	0.381	5/16	125	188	0.238
		3/16	133	200	0.286	1/4	100	150	0.190
5	14½	5/16	191	287	0.476	3/8	115	172	0.286
		1/4	153	229	0.381	5/16	95.5	143	0.238
		3/16	115	172	0.286	1/4	76.4	115	0.190
4	11½	5/16	158	237	0.476	3/8	79.9	120	0.286
		1/4	127	190	0.381	5/16	66.6	99.9	0.238
		3/16	95.0	142	0.286	1/4	53.3	79.9	0.190
3	8½	5/16	122	184	0.476	3/8	48.1	72.2	0.286
		1/4	98.0	147	0.381	5/16	40.1	60.2	0.238
		3/16	73.5	110	0.286	1/4	32.1	48.1	0.190
2	5½	5/16	83.7	125	0.476	3/8	21.9	32.8	0.286
		1/4	66.9	100	0.381	5/16	18.2	27.3	0.238
		3/16	50.2	75.3	0.286	1/4	14.6	21.9	0.190

ASD	LRFD	Beam	
$\Omega = 2.00$	$\phi = 0.75$	$F_y = 50$ ksi	$F_u = 65$ ksi

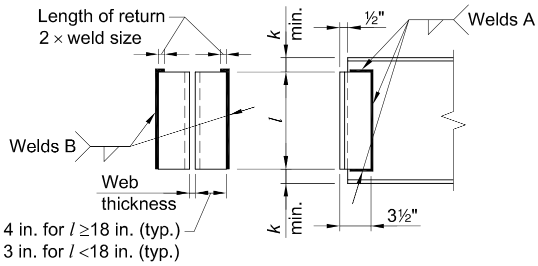
Table 10-3
Available Weld Strength of All-Welded Double-Angle Connections



l, in.	Welds A (70 ksi)				Welds B (70 ksi)			
	Weld Size, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	Minimum Web Thickness, in.	Weld Size, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	Minimum Support Thickness, in.
36	5/16	397	596	0.476	3/8	372	558	0.286
	1/4	318	477	0.381	5/16	310	465	0.238
	3/16	238	357	0.286	1/4	248	372	0.190
34	5/16	379	568	0.476	3/8	349	523	0.286
	1/4	303	455	0.381	5/16	291	436	0.238
	3/16	227	341	0.286	1/4	232	349	0.190
32	5/16	360	541	0.476	3/8	325	487	0.286
	1/4	288	432	0.381	5/16	271	406	0.238
	3/16	216	324	0.286	1/4	217	325	0.190
30	5/16	341	512	0.476	3/8	301	452	0.286
	1/4	273	410	0.381	5/16	251	377	0.238
	3/16	205	307	0.286	1/4	201	301	0.190
28	5/16	323	484	0.476	3/8	277	416	0.286
	1/4	258	387	0.381	5/16	231	347	0.238
	3/16	194	291	0.286	1/4	185	277	0.190
26	5/16	304	457	0.476	3/8	253	380	0.286
	1/4	243	365	0.381	5/16	211	317	0.238
	3/16	183	274	0.286	1/4	169	253	0.190
24	5/16	286	429	0.476	3/8	229	344	0.286
	1/4	229	343	0.381	5/16	191	286	0.238
	3/16	171	257	0.286	1/4	153	229	0.190
22	5/16	267	401	0.476	3/8	205	308	0.286
	1/4	214	321	0.381	5/16	171	256	0.238
	3/16	160	240	0.286	1/4	137	205	0.190
20	5/16	248	372	0.476	3/8	181	271	0.286
	1/4	198	297	0.381	5/16	151	226	0.238
	3/16	149	223	0.286	1/4	121	181	0.190
18	5/16	227	341	0.476	3/8	157	235	0.286
	1/4	182	273	0.381	5/16	130	196	0.238
	3/16	136	205	0.286	1/4	104	157	0.190
16	5/16	207	310	0.476	3/8	148	222	0.286
	1/4	166	248	0.381	5/16	123	185	0.238
	3/16	124	186	0.286	1/4	98.5	148	0.190

ASD	LRFD	Beam	
$\Omega = 2.00$	$\phi = 0.75$	$F_y = 50$ ksi	$F_u = 65$ ksi

Table 10-3 (continued)
Available Weld Strength of All-Welded Double-Angle Connections



l, in.	Welds A (70 ksi) Weld Size, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	Minimum Web Thickness, in.	Welds B (70 ksi) Weld Size, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	Minimum Support Thickness, in.
14	5/16	186	279	0.476	3/8	123	185	0.286
	1/4	149	223	0.381	5/16	103	154	0.238
	3/16	111	167	0.286	1/4	82.3	123	0.190
12	5/16	164	246	0.476	3/8	99.3	149	0.286
	1/4	131	197	0.381	5/16	82.8	124	0.238
	3/16	98.5	148	0.286	1/4	66.2	99.3	0.190
10	5/16	141	211	0.476	3/8	75.7	113	0.286
	1/4	112	169	0.381	5/16	63.1	94.6	0.238
	3/16	84.3	127	0.286	1/4	50.4	75.7	0.190
9	5/16	129	193	0.476	3/8	64.2	96.3	0.286
	1/4	103	154	0.381	5/16	53.5	80.2	0.238
	3/16	77.2	116	0.286	1/4	42.8	64.2	0.190
8	5/16	116	174	0.476	3/8	53.0	79.5	0.286
	1/4	92.9	139	0.381	5/16	44.2	66.3	0.238
	3/16	69.7	105	0.286	1/4	35.4	53.0	0.190
7	5/16	103	155	0.476	3/8	42.4	63.6	0.286
	1/4	82.6	124	0.381	5/16	35.3	53.0	0.238
	3/16	62.0	92.9	0.286	1/4	28.3	42.4	0.190
6	5/16	90.4	136	0.476	3/8	32.5	48.7	0.286
	1/4	72.3	108	0.381	5/16	27.0	40.6	0.238
	3/16	54.2	81.3	0.286	1/4	21.6	32.5	0.190
5	5/16	77.1	116	0.476	3/8	23.4	35.1	0.286
	1/4	61.7	92.6	0.381	5/16	19.5	29.2	0.238
	3/16	46.3	69.4	0.286	1/4	15.6	23.4	0.190
4	5/16	64.2	96.3	0.476	3/8	15.5	23.2	0.286
	1/4	51.4	77.0	0.381	5/16	12.9	19.3	0.238
	3/16	38.5	57.8	0.286	1/4	10.3	15.5	0.190

ASD	LRFD	Beam	
$\Omega = 2.00$	$\phi = 0.75$	$F_y = 50$ ksi	$F_u = 65$ ksi

SHEAR END-PLATE CONNECTIONS

A shear end-plate connection is made with a plate length less than the supported beam depth, as illustrated in Figure 10-6. The end plate is always shop-welded to the beam web with fillet welds on each side and usually field-bolted to the supporting member. Welds connecting the end plate to the beam web should not be returned across the thickness of the beam web at the top or bottom of the end plate because of the danger of creating a notch in the beam web.

If the end plate is field-welded to the support, adequate flexibility must be provided in the connection. Line welds are placed along the vertical edges of the plate with a return at the top per AISC *Specification* Section J2.2b. Note that welding across the entire top of the plate must be avoided as it would inhibit the flexibility and, therefore, the necessary end rotation of the connection. The performance of the resulting connection would not be as intended for simple shear connections.

Design Checks

The available strength of a shear end-plate connection is determined from the applicable limit states for bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). Note that the limit state of shear rupture of the beam web must be checked along the length of weld connecting the end plate to the beam web. In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a.

Recommended End-Plate Thickness

To provide for flexibility, the combination of plate thickness and gage should be consistent with the recommendations given previously for a double-angle connection of similar thickness and gage.

Shop and Field Practices

When framing to a column web, the associated constructability considerations should be addressed (see the preceding discussion under "Constructability Considerations").

When framing to a column flange, provision must be made for possible mill variation in the depth of the columns, tolerance in column/foundation placement, particularly in fairly

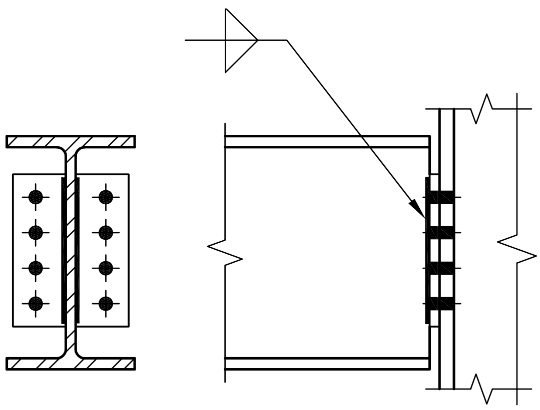


Fig. 10-6. Shear end-plate connections.

long runs (i.e., six or more bays of framing). The beam length can be shortened to provide for mill overrun with shims furnished at the appropriate intervals to fill the resulting gaps or to provide for mill underrun. Shear end-plate connections require close control in cutting the beam to the proper length and in squaring the beam ends such that both end plates are parallel, particularly when beams are cambered.

DESIGN TABLE DISCUSSION (TABLE 10-4)

Table 10-4. Bolted/Welded Shear End-Plate Connections

Table 10-4 is a design aid for shear end-plate connections bolted to the supporting member and welded to the supported beam. Available strengths are tabulated for supported and supporting member material with F_y = 50 ksi and F_u = 65 ksi, and end-plate material with F_y = 36 ksi and F_u = 58 ksi. Electrode strength is assumed to be 70 ksi. All values, including slip-critical bolt available strengths, are for comparison with the governing LRFD or ASD load combination.

Tabulated bolt and end-plate available strengths consider the limit states of bolt shear, slip resistance for slip-critical bolts, bolt bearing and tearout on the end plate, shear yielding of the end plate, shear rupture of the end plate, and block shear rupture of the end plate. Values are included for 2 through 12 rows of 3/4-in.-, 7/8-in.- and 1-in.-diameter Group A and Group B bolts at 3-in. spacing. End-plate edge distances, l_{ev} and l_{eh}, are assumed to be 1 1/4 in. The total end plate length, l, is based on this bolt spacing and edge distance, l_{ev}.

Tabulated weld available strengths consider the limit state of weld shear assuming an effective weld length equal to the end-plate length minus twice the weld size. The tabulated minimum beam web thickness matches the shear rupture strength of the web material to the strength of the weld metal. As derived in Part 9, the minimum supported beam web thickness for two lines of weld is

$$t_{min} = \frac{6.19D}{F_u} \tag{9-3}$$

where D is the number of sixteenths-of-an-inch in the weld size. When less than the minimum material thickness is present, the tabulated weld available strength must be reduced by the ratio of the thickness provided to the minimum thickness.

Tabulated supporting member available strengths, per in. of flange or web thickness, consider the limit state of bolt bearing only.

$^3/_4$-in. Bolts
12 Rows
$l = 35^1/_2$ in.

Table 10-4
Bolted/Welded Shear End-Plate Connections

W44

Bolt and End-Plate Available Strength, kips

Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD
Group A	N	STD	197	295	246	369	284	427
	X	STD	197	295	246	369	295	443
	SC Class A	STD	152	228	152	228	152	228
		OVS	129	194	129	194	129	194
		SSLT	152	228	152	228	152	228
	SC Class B	STD	197	295	246	369	253	380
		OVS	196	294	215	321	216	323
		SSLT	195	293	244	366	253	380
Group B	N	STD	197	295	246	369	295	443
	X	STD	197	295	246	369	295	443
	SC Class A	STD	189	283	190	285	190	285
		OVS	162	242	162	242	162	242
		SSLT	189	283	190	285	190	285
	SC Class B	STD	197	295	246	369	295	443
		OVS	196	294	245	367	268	400
		SSLT	195	293	244	366	293	440

Weld and Beam Web Available Strength, kips

70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD
3/16	0.286	196	293
1/4	0.381	260	390
5/16	0.476	324	486
3/8	0.571	387	581

Support Available Strength per Inch Thickness, kip/in.

ASD	LRFD
1400	2110

End-Plate	Beam
$F_y = 36$ ksi $F_u = 58$ ksi	$F_y = 50$ ksi $F_u = 65$ ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

Note: Slip-critical bolt values assume no more than one filler has been provided.

W44, 40

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

3/4-in. Bolts
11 Rows
$l = 32^1/_2$ in.

Bolt and End-Plate Available Strength, kips

Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD
Group A	N	STD	181	271	226	338	261	391
	X	STD	181	271	226	338	271	406
	SC Class A	STD	139	209	139	209	139	209
		OVS	119	178	119	178	119	178
		SSLT	139	209	139	209	139	209
	SC Class B	STD	181	271	226	338	232	348
		OVS	180	269	197	294	198	296
		SSLT	179	269	224	336	232	348
Group B	N	STD	181	271	226	338	271	406
	X	STD	181	271	226	338	271	406
	SC Class A	STD	173	259	174	261	174	261
		OVS	148	222	148	222	148	222
		SSLT	173	259	174	261	174	261
	SC Class B	STD	181	271	226	338	271	406
		OVS	180	269	225	337	245	367
		SSLT	179	269	224	336	269	403

Weld and Beam Web Available Strength, kips

70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD
3/16	0.286	179	268
1/4	0.381	238	356
5/16	0.476	296	444
3/8	0.571	354	530

Support Available Strength per Inch Thickness, kip/in.

ASD	LRFD
1290	1930

End-Plate	Beam
$F_y = 36$ ksi $F_u = 58$ ksi	$F_y = 50$ ksi $F_u = 65$ ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

Note: Slip-critical bolt values assume no more than one filler has been provided.

$^3/_4$-in. Bolts

10 Rows

$l = 29^1/_2$ in.

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

W44, 40, 36

Bolt and End-Plate Available Strength, kips

Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			$^1/_4$		$^5/_{16}$		$^3/_8$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
Group A	N	STD	164	246	205	308	237	355
	X	STD	164	246	205	308	246	370
	SC Class A	STD	127	190	127	190	127	190
		OVS	108	161	108	161	108	161
		SSLT	127	190	127	190	127	190
	SC Class B	STD	164	246	205	308	211	316
		OVS	163	245	179	268	180	269
		SSLT	163	244	204	306	211	316
Group B	N	STD	164	246	205	308	246	370
	X	STD	164	246	205	308	246	370
	SC Class A	STD	157	236	158	237	158	237
		OVS	135	202	135	202	135	202
		SSLT	157	236	158	237	158	237
	SC Class B	STD	164	246	205	308	246	370
		OVS	163	245	204	306	223	333
		SSLT	163	244	204	306	244	367

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kip/in.	
70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
$^3/_{16}$	0.286	162	243	1170	1760
$^1/_4$	0.381	215	323		
$^5/_{16}$	0.476	268	402		
$^3/_8$	0.571	320	480		

End-Plate	Beam
$F_y = 36$ ksi $F_u = 58$ ksi	$F_y = 50$ ksi $F_u = 65$ ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

Note: Slip-critical bolt values assume no more than one filler has been provided.

W44, 40, 36, 33

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

3/4-in. Bolts
9 Rows
l = 26 1/2 in.

Bolt and End-Plate Available Strength, kips								
Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
Group A	N	STD	148	222	185	278	213	319
	X	STD	148	222	185	278	222	333
	SC Class A	STD	114	171	114	171	114	171
		OVS	97.1	145	97.1	145	97.1	145
		SSLT	114	171	114	171	114	171
	SC Class B	STD	148	222	185	278	190	285
		OVS	147	221	161	241	162	242
		SSLT	147	220	183	275	190	285
Group B	N	STD	148	222	185	278	222	333
	X	STD	148	222	185	278	222	333
	SC Class A	STD	141	212	142	214	142	214
		OVS	121	182	121	182	121	182
		SSLT	141	212	142	214	142	214
	SC Class B	STD	148	222	185	278	222	333
		OVS	147	221	184	276	200	300
		SSLT	147	220	183	275	220	330

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kip/in.	
70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
3/16	0.286	145	218	1050	1580
1/4	0.381	193	290		
5/16	0.476	240	360		
3/8	0.571	287	430		

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

Note: Slip-critical bolt values assume no more than one filler has been provided.

3/4-in. Bolts
8 Rows
l = 23 1/2 in.

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

W44, 40, 36, 33, 30

Bolt and End-Plate Available Strength, kips								
Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
Group A	N	STD	132	198	165	247	189	284
	X	STD	132	198	165	247	198	297
	SC Class A	STD	101	152	101	152	101	152
		OVS	86.3	129	86.3	129	86.3	129
		SSLT	101	152	101	152	101	152
	SC Class B	STD	132	198	165	247	169	253
		OVS	131	197	143	214	144	215
		SSLT	131	196	163	245	169	253
Group B	N	STD	132	198	165	247	198	297
	X	STD	132	198	165	247	198	297
	SC Class A	STD	125	188	127	190	127	190
		OVS	108	161	108	161	108	161
		SSLT	125	188	127	190	127	190
	SC Class B	STD	132	198	165	247	198	297
		OVS	131	197	164	246	178	266
		SSLT	131	196	163	245	196	294

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kip/in.	
70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
3/16	0.286	129	193	936	1400
1/4	0.381	171	256		
5/16	0.476	212	318		
3/8	0.571	253	380		
				End-Plate	Beam
				F_y = 36 ksi, F_u = 58 ksi	F_y = 50 ksi, F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

Note: Slip-critical bolt values assume no more than one filler has been provided.

W44, 40, 36, 33, 30, 27, 24

Table 10-4 (continued) Bolted/Welded Shear End-Plate Connections

3/4-in. Bolts
7 Rows
$l = 20^1/_2$ in.

Bolt and End-Plate Available Strength, kips								
Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
Group A	N	STD	116	174	145	217	165	248
	X	STD	116	174	145	217	174	260
	SC Class A	STD	88.6	133	88.6	133	88.6	133
		OVS	75.5	113	75.5	113	75.5	113
		SSLT	88.6	133	88.6	133	88.6	133
	SC Class B	STD	116	174	145	217	148	221
		OVS	115	172	125	187	126	188
		SSLT	114	172	143	214	148	221
Group B	N	STD	116	174	145	217	174	260
	X	STD	116	174	145	217	174	260
	SC Class A	STD	110	164	111	166	111	166
		OVS	94.4	141	94.4	141	94.4	141
		SSLT	110	164	111	166	111	166
	SC Class B	STD	116	174	145	217	174	260
		OVS	115	172	144	215	155	232
		SSLT	114	172	143	214	172	257

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kip/in.	
70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
3/16	0.286	112	168	819	1230
1/4	0.381	148	223		
5/16	0.476	184	277		
3/8	0.571	220	330		
				End-Plate	Beam
				$F_y = 36$ ksi, $F_u = 58$ ksi	$F_y = 50$ ksi, $F_u = 65$ ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

Note: Slip-critical bolt values assume no more than one filler has been provided.

3/4-in. Bolts
6 Rows
l = 17 1/2 in.

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

W44, 40, 36, 33, 30, 27, 24, 21

Bolt and End-Plate Available Strength, kips

Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
Group A	N	STD	99.5	149	124	187	141	212
	X	STD	99.5	149	124	187	149	224
	SC Class A	STD	75.9	114	75.9	114	75.9	114
		OVS	64.7	96.8	64.7	96.8	64.7	96.8
		SSLT	75.9	114	75.9	114	75.9	114
	SC Class B	STD	99.5	149	124	186	127	190
		OVS	98.6	148	107	160	108	161
		SSLT	98.2	147	123	184	127	190
Group B	N	STD	99.5	149	124	187	149	224
	X	STD	99.5	149	124	187	149	224
	SC Class A	STD	93.8	141	94.9	142	94.9	142
		OVS	80.9	121	80.9	121	80.9	121
		SSLT	93.8	141	94.9	142	94.9	142
	SC Class B	STD	99.5	149	124	187	149	224
		OVS	98.6	148	123	185	133	199
		SSLT	98.2	147	123	184	147	221

Weld and Beam Web Available Strength, kips

70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD
3/16	0.286	95.4	143
1/4	0.381	126	189
5/16	0.476	157	235
3/8	0.571	187	280

Support Available Strength per Inch Thickness, kip/in.

ASD	LRFD
702	1050

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

Note: Slip-critical bolt values assume no more than one filler has been provided.

W30, 27, 24, 21, 18

Table 10-4 (continued) Bolted/Welded Shear End-Plate Connections

$^3/_4$-in. Bolts
5 Rows
$l = 14^1/_2$ in.

Bolt and End-Plate Available Strength, kips

Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in. $^1/_4$ ASD	$^1/_4$ LRFD	$^5/_{16}$ ASD	$^5/_{16}$ LRFD	$^3/_8$ ASD	$^3/_8$ LRFD
Group A	N	STD	83.3	125	104	156	117	176
	X	STD	83.3	125	104	156	125	187
	SC Class A	STD	63.3	94.9	63.3	94.9	63.3	94.9
		OVS	53.9	80.7	53.9	80.7	53.9	80.7
		SSLT	63.3	94.9	63.3	94.9	63.3	94.9
	SC Class B	STD	83.3	125	103	154	105	158
		OVS	82.4	124	88.9	133	89.9	134
		SSLT	82.0	123	102	154	105	158
Group B	N	STD	83.3	125	104	156	125	187
	X	STD	83.3	125	104	156	125	187
	SC Class A	STD	78.0	117	79.1	119	79.1	119
		OVS	67.4	101	67.4	101	67.4	101
		SSLT	78.0	117	79.1	119	79.1	119
	SC Class B	STD	83.3	125	104	156	125	187
		OVS	82.4	124	103	155	110	165
		SSLT	82.0	123	102	154	123	184

Weld and Beam Web Available Strength, kips

70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD
$^3/_{16}$	0.286	78.7	118
$^1/_4$	0.381	104	156
$^5/_{16}$	0.476	129	193
$^3/_8$	0.571	153	230

Support Available Strength per Inch Thickness, kip/in.

ASD	LRFD
585	878

End-Plate	Beam
$F_y = 36$ ksi, $F_u = 58$ ksi	$F_y = 50$ ksi, $F_u = 65$ ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

Note: Slip-critical bolt values assume no more than one filler has been provided.

3/4-in. Bolts
4 Rows
$l = 11^1/_2$ in.

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

W24, 21, 18, 16

Bolt and End-Plate Available Strength, kips

Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD
Group A	N	STD	67.1	101	83.9	126	93.6	140
	X	STD	67.1	101	83.9	126	101	151
	SC Class A	STD	50.6	75.9	50.6	75.9	50.6	75.9
		OVS	43.1	64.5	43.1	64.5	43.1	64.5
		SSLT	50.6	75.9	50.6	75.9	50.6	75.9
	SC Class B	STD	67.1	101	81.6	122	84.4	127
		OVS	65.3	97.9	70.9	106	71.9	108
		SSLT	65.8	98.7	81.6	122	84.4	127
Group B	N	STD	67.1	101	83.9	126	101	151
	X	STD	67.1	101	83.9	126	101	151
	SC Class A	STD	62.1	93.2	63.3	94.9	63.3	94.9
		OVS	53.9	80.7	53.9	80.7	53.9	80.7
		SSLT	62.1	93.2	63.3	94.9	63.3	94.9
	SC Class B	STD	67.1	101	83.9	126	101	151
		OVS	65.3	97.9	81.6	122	87.8	131
		SSLT	65.8	98.7	82.2	123	98.7	148

Weld and Beam Web Available Strength, kips

70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD
3/16	0.286	61.9	92.9
1/4	0.381	81.7	123
5/16	0.476	101	151
3/8	0.571	120	180

Support Available Strength per Inch Thickness, kip/in.

ASD	LRFD
468	702

End-Plate	Beam
$F_y = 36$ ksi, $F_u = 58$ ksi	$F_y = 50$ ksi, $F_u = 65$ ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

Note: Slip-critical bolt values assume no more than one filler has been provided.

W18, 16, 14, 12, 10*

Table 10-4 (continued) Bolted/Welded Shear End-Plate Connections

¾-in. Bolts
3 Rows
$l = 8\frac{1}{2}$ in.

Bolt and End-Plate Available Strength, kips

Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in. ¼ ASD	¼ LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD
Group A	N	STD	50.9	76.4	63.7	95.5	69.7	105
	X	STD	50.9	76.4	63.7	95.5	76.4	115
	SC Class A	STD	38.0	57.0	38.0	57.0	38.0	57.0
		OVS	32.4	48.4	32.4	48.4	32.4	48.4
		SSLT	38.0	57.0	38.0	57.0	38.0	57.0
	SC Class B	STD	50.9	76.4	60.5	90.8	63.3	94.9
		OVS	47.9	71.8	52.9	79.3	53.9	80.7
		SSLT	49.6	74.4	60.5	90.8	63.3	94.9
Group B	N	STD	50.9	76.4	63.7	95.5	76.4	115
	X	STD	50.9	76.4	63.7	95.5	76.4	115
	SC Class A	STD	46.3	69.5	47.5	71.2	47.5	71.2
		OVS	40.4	60.5	40.4	60.5	40.4	60.5
		SSLT	46.3	69.5	47.5	71.2	47.5	71.2
	SC Class B	STD	50.9	76.4	63.7	95.5	74.8	112
		OVS	47.9	71.8	59.8	89.7	65.3	97.8
		SSLT	49.6	74.4	62.0	92.9	74.4	112

Weld and Beam Web Available Strength, kips / **Support Available Strength per Inch Thickness, kip/in.**

70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	Support ASD	Support LRFD
3/16	0.286	45.2	67.9	351	527
1/4	0.381	59.4	89.1		
5/16	0.476	73.1	110		
3/8	0.571	86.3	129		

	End-Plate	Beam
	$F_y = 36$ ksi, $F_u = 58$ ksi	$F_y = 50$ ksi, $F_u = 65$ ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

*Limited to W10×12, 15, 17, 19, 22, 26, 30
Note: Slip-critical bolt values assume no more than one filler has been provided.

$^3/_4$-in. Bolts
2 Rows
$l = 5^1/_2$ in.

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

W12, 10, 8

Bolt and End-Plate Available Strength, kips

Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in. $^1/_4$ ASD	$^1/_4$ LRFD	$^5/_{16}$ ASD	$^5/_{16}$ LRFD	$^3/_8$ ASD	$^3/_8$ LRFD
Group A	N	STD	32.6	48.9	40.8	61.2	45.9	68.8
	X	STD	32.6	48.9	40.8	61.2	48.9	73.4
	SC Class A	STD	25.3	38.0	25.3	38.0	25.3	38.0
		OVS	21.6	32.3	21.6	32.3	21.6	32.3
		SSLT	25.3	38.0	25.3	38.0	25.3	38.0
	SC Class B	STD	32.6	48.9	39.4	59.2	42.2	63.3
		OVS	30.5	45.7	35.0	52.4	36.0	53.8
		SSLT	32.6	48.9	39.4	59.2	42.2	63.3
Group B	N	STD	32.6	48.9	40.8	61.2	48.9	73.4
	X	STD	32.6	48.9	40.8	61.2	48.9	73.4
	SC Class A	STD	30.5	45.8	31.6	47.5	31.6	47.5
		OVS	27.0	40.3	27.0	40.3	27.0	40.3
		SSLT	30.5	45.8	31.6	47.5	31.6	47.5
	SC Class B	STD	32.6	48.9	40.8	61.2	48.4	72.6
		OVS	30.5	45.7	38.1	57.1	42.9	64.2
		SSLT	32.6	48.9	40.8	61.2	48.4	72.6

Weld and Beam Web Available Strength, kips

70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	Support Available Strength per Inch Thickness, kip/in. ASD	Support Available Strength per Inch Thickness, kip/in. LRFD
$^3/_{16}$	0.286	28.5	42.8	234	351
$^1/_4$	0.381	37.1	55.7		
$^5/_{16}$	0.476	45.2	67.9		
$^3/_8$	0.571	52.9	79.4		

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

Note: Slip-critical bolt values assume no more than one filler has been provided.

W44

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

7/8-in. Bolts
12 Rows
l = 35 1/2 in.

Bolt and End-Plate Available Strength, kips

Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD
Group A	N	STD	196	294	245	367	294	441
	X	STD	196	294	245	367	294	441
	SC Class A	STD	196	294	211	316	212	317
		OVS	178	266	180	270	180	270
		SSLT	194	292	211	316	212	317
	SC Class B	STD	196	294	245	367	294	441
		OVS	191	287	239	359	287	431
		SSLT	194	292	243	365	292	438
Group B	N	STD	196	294	245	367	294	441
	X	STD	196	294	245	367	294	441
	SC Class A	STD	196	294	245	367	264	396
		OVS	191	287	223	334	226	339
		SSLT	194	292	243	365	264	396
	SC Class B	STD	196	294	245	367	294	441
		OVS	191	287	239	359	287	431
		SSLT	194	292	243	365	292	438

Weld and Beam Web Available Strength, kips

70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD
3/16	0.286	196	293
1/4	0.381	260	390
5/16	0.476	324	486
3/8	0.571	387	581

Support Available Strength per Inch Thickness, kip/in.

ASD	LRFD
1640	2460

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

Note: Slip-critical bolt values assume no more than one filler has been provided.

$^7/_8$-in. Bolts
11 Rows
$l = 32^1/_2$ in.

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

W44, 40

Bolt and End-Plate Available Strength, kips

Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in. $^1/_4$ ASD	$^1/_4$ LRFD	$^5/_{16}$ ASD	$^5/_{16}$ LRFD	$^3/_8$ ASD	$^3/_8$ LRFD
Group A	N	STD	180	269	225	337	269	404
	X	STD	180	269	225	337	269	404
	SC Class A	STD	180	269	193	290	194	291
		OVS	163	244	165	247	165	247
		SSLT	178	267	193	290	194	291
	SC Class B	STD	180	269	225	337	269	404
		OVS	175	263	219	328	263	394
		SSLT	178	267	223	334	267	401
Group B	N	STD	180	269	225	337	269	404
	X	STD	180	269	225	337	269	404
	SC Class A	STD	180	269	225	337	242	363
		OVS	175	263	204	306	208	311
		SSLT	178	267	223	334	242	363
	SC Class B	STD	180	269	225	337	269	404
		OVS	175	263	219	328	263	394
		SSLT	178	267	223	334	267	401

Weld and Beam Web Available Strength, kips

70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	Support Available Strength per Inch Thickness, kip/in. ASD	Support Available Strength per Inch Thickness, kip/in. LRFD
$^3/_{16}$	0.286	179	268	1500	2250
$^1/_4$	0.381	238	356		
$^5/_{16}$	0.476	296	444		
$^3/_8$	0.571	354	530		

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

End-Plate	Beam
$F_y = 36$ ksi $F_u = 58$ ksi	$F_y = 50$ ksi $F_u = 65$ ksi

Note: Slip-critical bolt values assume no more than one filler has been provided.

W44, 40, 36

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

7/8-in. Bolts
10 Rows
$l = 29^1/_2$ in.

Bolt and End-Plate Available Strength, kips

Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD
Group A	N	STD	163	245	204	306	245	368
	X	STD	163	245	204	306	245	368
	SC Class A	STD	163	245	176	263	176	264
		OVS	148	221	150	225	150	225
		SSLT	162	243	176	263	176	264
	SC Class B	STD	163	245	204	306	245	368
		OVS	159	238	198	298	238	357
		SSLT	162	243	203	304	243	365
Group B	N	STD	163	245	204	306	245	368
	X	STD	163	245	204	306	245	368
	SC Class A	STD	163	245	204	306	220	330
		OVS	159	238	186	278	189	282
		SSLT	162	243	203	304	220	330
	SC Class B	STD	163	245	204	306	245	368
		OVS	159	238	198	298	238	357
		SSLT	162	243	203	304	243	365

Weld and Beam Web Available Strength, kips

70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD
3/16	0.286	162	243
1/4	0.381	215	323
5/16	0.476	268	402
3/8	0.571	320	480

Support Available Strength per Inch Thickness, kip/in.

ASD	LRFD
1370	2050

End-Plate	Beam
$F_y = 36$ ksi, $F_u = 58$ ksi	$F_y = 50$ ksi, $F_u = 65$ ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

Note: Slip-critical bolt values assume no more than one filler has been provided.

7/8-in. Bolts
9 Rows
l = 26 1/2 in.

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

W44, 40, 36, 33

Bolt and End-Plate Available Strength, kips

Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD
Group A	N	STD	147	221	184	276	221	331
	X	STD	147	221	184	276	221	331
	SC Class A	STD	147	221	158	237	159	238
		OVS	133	199	135	202	135	202
		SSLT	146	219	158	237	159	238
	SC Class B	STD	147	221	184	276	221	331
		OVS	142	214	178	267	214	321
		SSLT	146	219	182	273	219	328
Group B	N	STD	147	221	184	276	221	331
	X	STD	147	221	184	276	221	331
	SC Class A	STD	147	221	184	276	198	296
		OVS	142	214	167	249	170	254
		SSLT	146	219	182	273	198	296
	SC Class B	STD	147	221	184	276	221	331
		OVS	142	214	178	267	214	321
		SSLT	146	219	182	273	219	328

Weld and Beam Web Available Strength, kips

70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD
3/16	0.286	145	218
1/4	0.381	193	290
5/16	0.476	240	360
3/8	0.571	287	430

Support Available Strength per Inch Thickness, kip/in.

ASD	LRFD
1230	1840

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

Note: Slip-critical bolt values assume no more than one filler has been provided.

W44, 40, 36, 33, 30

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

7/8-in. Bolts
8 Rows
l = 23 1/2 in.

Bolt and End-Plate Available Strength, kips

Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD
Group A	N	STD	131	197	164	246	197	295
	X	STD	131	197	164	246	197	295
	SC Class A	STD	131	197	140	211	141	212
		OVS	118	176	120	180	120	180
		SSLT	130	194	140	211	141	212
	SC Class B	STD	131	197	164	246	197	295
		OVS	126	189	158	237	189	284
		SSLT	130	194	162	243	194	292
Group B	N	STD	131	197	164	246	197	295
	X	STD	131	197	164	246	197	295
	SC Class A	STD	131	197	164	246	175	263
		OVS	126	189	148	221	151	226
		SSLT	130	194	162	243	175	263
	SC Class B	STD	131	197	164	246	197	295
		OVS	126	189	158	237	189	284
		SSLT	130	194	162	243	194	292

Weld and Beam Web Available Strength, kips

70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD
3/16	0.286	129	193
1/4	0.381	171	256
5/16	0.476	212	318
3/8	0.571	253	380

Support Available Strength per Inch Thickness, kip/in.

ASD	LRFD
1090	1640

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

Note: Slip-critical bolt values assume no more than one filler has been provided.

7/8-in. Bolts
7 Rows
l = 20 1/2 in.

Table 10-4 (continued) Bolted/Welded Shear End-Plate Connections

W44, 40, 36, 33, 30, 27, 24

Bolt and End-Plate Available Strength, kips								
Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
Group A	N	STD	115	172	144	215	172	258
	X	STD	115	172	144	215	172	258
	SC Class A	STD	115	172	123	184	123	185
		OVS	103	154	105	157	105	157
		SSLT	113	170	123	184	123	185
	SC Class B	STD	115	172	144	215	172	258
		OVS	110	165	137	206	165	247
		SSLT	113	170	142	213	170	255
Group B	N	STD	115	172	144	215	172	258
	X	STD	115	172	144	215	172	258
	SC Class A	STD	115	172	144	215	153	230
		OVS	110	165	129	193	132	198
		SSLT	113	170	142	213	153	230
	SC Class B	STD	115	172	144	215	172	258
		OVS	110	165	137	206	165	247
		SSLT	113	170	142	213	170	255

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kip/in.	
70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
3/16	0.286	112	168	956	1430
1/4	0.381	148	223		
5/16	0.476	184	277		
3/8	0.571	220	330		

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

End-Plate	Beam
F_y = 36 ksi, F_u = 58 ksi	F_y = 50 ksi, F_u = 65 ksi

Note: Slip-critical bolt values assume no more than one filler has been provided.

W44, 40, 36, 33, 30, 27, 24, 21

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

7/8-in. Bolts
6 Rows
$l = 17\frac{1}{2}$ in.

Bolt and End-Plate Available Strength, kips

Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD
Group A	N	STD	98.6	148	123	185	148	222
	X	STD	98.6	148	123	185	148	222
	SC Class A	STD	98.6	148	105	158	106	159
		OVS	87.6	131	90.1	135	90.1	135
		SSLT	97.3	146	105	158	106	159
	SC Class B	STD	98.6	148	123	185	148	222
		OVS	93.5	140	117	175	140	210
		SSLT	97.3	146	122	182	146	219
Group B	N	STD	98.6	148	123	185	148	222
	X	STD	98.6	148	123	185	148	222
	SC Class A	STD	98.6	148	123	185	131	197
		OVS	93.5	140	110	165	113	169
		SSLT	97.3	146	122	182	131	197
	SC Class B	STD	98.6	148	123	185	148	222
		OVS	93.5	140	117	175	140	210
		SSLT	97.3	146	122	182	146	219

Weld and Beam Web Available Strength, kips / **Support Available Strength per Inch Thickness, kip/in.**

70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	Support ASD	Support LRFD
3/16	0.286	95.4	143	819	1230
1/4	0.381	126	189		
5/16	0.476	157	235		
3/8	0.571	187	280		

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

End-Plate	Beam
$F_y = 36$ ksi $F_u = 58$ ksi	$F_y = 50$ ksi $F_u = 65$ ksi

Note: Slip-critical bolt values assume no more than one filler has been provided.

$^7/_8$-in. Bolts

5 Rows

$l = 14^1/_2$ in.

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

W30, 27, 24, 21, 18

Bolt and End-Plate Available Strength, kips

Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in. $^1/_4$ ASD	$^1/_4$ LRFD	$^5/_{16}$ ASD	$^5/_{16}$ LRFD	$^3/_8$ ASD	$^3/_8$ LRFD
Group A	N	STD	82.4	124	103	155	124	185
	X	STD	82.4	124	103	155	124	185
	SC Class A	STD	82.4	124	87.5	131	88.1	132
		OVS	72.6	109	75.1	112	75.1	112
		SSLT	81.1	122	87.5	131	88.1	132
	SC Class B	STD	82.4	124	103	155	124	185
		OVS	77.2	116	96.5	145	116	174
		SSLT	81.1	122	101	152	122	182
Group B	N	STD	82.4	124	103	155	124	185
	X	STD	82.4	124	103	155	124	185
	SC Class A	STD	82.4	124	103	155	109	163
		OVS	77.2	116	91.1	136	94.3	141
		SSLT	81.1	122	101	152	109	163
	SC Class B	STD	82.4	124	103	155	124	185
		OVS	77.2	116	96.5	145	116	174
		SSLT	81.1	122	101	152	122	182

Weld and Beam Web Available Strength, kips

70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD
$^3/_{16}$	0.286	78.7	118
$^1/_4$	0.381	104	156
$^5/_{16}$	0.476	129	193
$^3/_8$	0.571	153	230

Support Available Strength per Inch Thickness, kip/in.

ASD	LRFD
683	1020

End-Plate	Beam
$F_y = 36$ ksi $F_u = 58$ ksi	$F_y = 50$ ksi $F_u = 65$ ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

Note: Slip-critical bolt values assume no more than one filler has been provided.

W24, 21, 18, 16

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

7/8-in. Bolts
4 Rows
l = 11 1/2 in.

Bolt and End-Plate Available Strength, kips								
Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
Group A	N	STD	65.3	97.9	81.6	122	97.9	147
	X	STD	65.3	97.9	81.6	122	97.9	147
	SC Class A	STD	65.3	97.9	69.9	105	70.5	106
		OVS	57.6	86.2	60.1	89.9	60.1	89.9
		SSLT	64.9	97.3	69.9	105	70.5	106
	SC Class B	STD	65.3	97.9	81.6	122	97.9	147
		OVS	60.9	91.4	76.1	114	91.4	137
		SSLT	64.9	97.3	81.1	122	97.3	146
Group B	N	STD	65.3	97.9	81.6	122	97.9	147
	X	STD	65.3	97.9	81.6	122	97.9	147
	SC Class A	STD	65.3	97.9	81.6	122	86.8	130
		OVS	60.9	91.4	72.3	108	75.4	113
		SSLT	64.9	97.3	81.1	122	86.8	130
	SC Class B	STD	65.3	97.9	81.6	122	97.9	147
		OVS	60.9	91.4	76.1	114	91.4	137
		SSLT	64.9	97.3	81.1	122	97.3	146

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kip/in.	
70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
3/16	0.286	61.9	92.9	546	819
1/4	0.381	81.7	123		
5/16	0.476	101	151		
3/8	0.571	120	180		

		End-Plate	Beam
STD = Standard holes OVS = Oversized holes SSLT = Short-slotted holes transverse to direction of load	N = Threads included X = Threads excluded SC = Slip critical	F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

Note: Slip-critical bolt values assume no more than one filler has been provided.

$^7/_8$-in. Bolts
3 Rows
$l = 8^1/_2$ in.

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

W18, 16, 14, 12, 10*

Bolt and End-Plate Available Strength, kips

Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in. $^1/_4$ ASD	$^1/_4$ LRFD	$^5/_{16}$ ASD	$^5/_{16}$ LRFD	$^3/_8$ ASD	$^3/_8$ LRFD
Group A	N	STD	47.9	71.8	59.8	89.7	71.8	108
	X	STD	47.9	71.8	59.8	89.7	71.8	108
	SC Class A	STD	47.9	71.8	52.2	78.4	52.9	79.3
		OVS	42.6	63.7	45.1	67.4	45.1	67.4
		SSLT	47.9	71.8	52.2	78.4	52.9	79.3
	SC Class B	STD	47.9	71.8	59.8	89.7	71.8	108
		OVS	44.6	66.9	55.7	83.6	66.9	100
		SSLT	47.9	71.8	59.8	89.7	71.8	108
Group B	N	STD	47.9	71.8	59.8	89.7	71.8	108
	X	STD	47.9	71.8	59.8	89.7	71.8	108
	SC Class A	STD	47.9	71.8	59.8	89.7	64.7	97.0
		OVS	44.6	66.9	53.4	79.9	56.5	84.6
		SSLT	47.9	71.8	59.8	89.7	64.7	97.0
	SC Class B	STD	47.9	71.8	59.8	89.7	71.8	108
		OVS	44.6	66.9	55.7	83.6	66.9	100
		SSLT	47.9	71.8	59.8	89.7	71.8	108

Weld and Beam Web Available Strength, kips

70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	Support Available Strength per Inch Thickness, kip/in. ASD	LRFD
$^3/_{16}$	0.286	45.2	67.9	409	614
$^1/_4$	0.381	59.4	89.1		
$^5/_{16}$	0.476	73.1	110		
$^3/_8$	0.571	86.3	129		

End-Plate	Beam
$F_y = 36$ ksi, $F_u = 58$ ksi	$F_y = 50$ ksi, $F_u = 65$ ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

*Limited to W10×12, 15, 17, 19, 22, 26, 30
Note: Slip-critical bolt values assume no more than one filler has been provided.

W12, 10, 8

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

$\mathbf{^{7}/_{8}}$-in. Bolts
2 Rows
$l = 5^{1}/_{2}$ in.

Bolt and End-Plate Available Strength, kips

Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			$^{1}/_{4}$		$^{5}/_{16}$		$^{3}/_{8}$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
Group A	N	STD	30.5	45.7	38.1	57.1	45.7	68.5
	X	STD	30.5	45.7	38.1	57.1	45.7	68.5
	SC Class A	STD	30.5	45.7	34.6	51.9	35.3	52.9
		OVS	27.5	41.2	30.0	45.0	30.0	45.0
		SSLT	30.5	45.7	34.6	51.9	35.3	52.9
	SC Class B	STD	30.5	45.7	38.1	57.1	45.7	68.5
		OVS	28.3	42.4	35.3	53.0	42.4	63.6
		SSLT	30.5	45.7	38.1	57.1	45.7	68.5
Group B	N	STD	30.5	45.7	38.1	57.1	45.7	68.5
	X	STD	30.5	45.7	38.1	57.1	45.7	68.5
	SC Class A	STD	30.5	45.7	38.1	57.1	42.5	63.8
		OVS	28.3	42.4	34.5	51.7	37.6	56.4
		SSLT	30.5	45.7	38.1	57.1	42.5	63.8
	SC Class B	STD	30.5	45.7	38.1	57.1	45.7	68.5
		OVS	28.3	42.4	35.3	53.0	42.4	63.6
		SSLT	30.5	45.7	38.1	57.1	45.7	68.5

Weld and Beam Web Available Strength, kips

70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips
		ASD	LRFD
$^{3}/_{16}$	0.286	28.5	42.8
$^{1}/_{4}$	0.381	37.1	55.7
$^{5}/_{16}$	0.476	45.2	67.9
$^{3}/_{8}$	0.571	52.9	79.4

Support Available Strength per Inch Thickness, kip/in.

ASD	LRFD
273	410

End-Plate	Beam
$F_y = 36$ ksi $F_u = 58$ ksi	$F_y = 50$ ksi $F_u = 65$ ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

Note: Slip-critical bolt values assume no more than one filler has been provided.

1-in. Bolts
12 Rows
l = 35½ in.

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

W44

Bolt and End-Plate Available Strength, kips

Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in. ¼ ASD	¼ LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD
Group A	N	STD	185	277	231	347	277	416
	X	STD	185	277	231	347	277	416
	SC Class A	STD	185	277	231	347	272	407
		OVS	172	258	215	322	232	348
		SSLT	185	277	231	347	272	407
	SC Class B	STD	185	277	231	347	277	416
		OVS	172	258	215	322	258	387
		SSLT	185	277	231	347	277	416
Group B	N	STD	185	277	231	347	277	416
	X	STD	185	277	231	347	277	416
	SC Class A	STD	185	277	231	347	277	416
		OVS	172	258	215	322	258	387
		SSLT	185	277	231	347	277	416
	SC Class B	STD	185	277	231	347	277	416
		OVS	172	258	215	322	258	387
		SSLT	185	277	231	347	277	416

Weld and Beam Web Available Strength, kips

70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD
3/16	0.286	196	293
1/4	0.381	260	390
5/16	0.476	324	486
3/8	0.571	387	581

Support Available Strength per Inch Thickness, kip/in.

Hole Type	ASD	LRFD
STD/SSLT	1760	2650
OVS	1660	2490

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

Note: Slip-critical bolt values assume no more than one filler has been provided.

W44, 40

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

1-in. Bolts
11 Rows
$l = 32^1/_2$ in.

Bolt and End-Plate Available Strength, kips

Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
Group A	N	STD	169	254	211	317	254	380
	X	STD	169	254	211	317	254	380
	SC Class A	STD	169	254	211	317	248	373
		OVS	157	236	196	295	213	318
		SSLT	169	254	211	317	248	373
	SC Class B	STD	169	254	211	317	254	380
		OVS	157	236	196	295	236	354
		SSLT	169	254	211	317	254	380
Group B	N	STD	169	254	211	317	254	380
	X	STD	169	254	211	317	254	380
	SC Class A	STD	169	254	211	317	254	380
		OVS	157	236	196	295	236	354
		SSLT	169	254	211	317	254	380
	SC Class B	STD	169	254	211	317	254	380
		OVS	157	236	196	295	236	354
		SSLT	169	254	211	317	254	380

Weld and Beam Web Available Strength, kips

70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips
		ASD	LRFD
3/16	0.286	179	268
1/4	0.381	238	356
5/16	0.476	296	444
3/8	0.571	354	530

Support Available Strength per Inch Thickness, kip/in.

ASD		LRFD	
1620	STD/SSLT	2430	STD/SSLT
1520	OVS	2280	OVS

End-Plate	Beam
$F_y = 36$ ksi $F_u = 58$ ksi	$F_y = 50$ ksi $F_u = 65$ ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

Note: Slip-critical bolt values assume no more than one filler has been provided.

1-in. Bolts
10 Rows
$l = 29^1/_2$ in.

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

W44, 40, 36

Bolt and End-Plate Available Strength, kips								
Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
Group A	N	STD	153	230	192	288	230	345
	X	STD	153	230	192	288	230	345
	SC Class A	STD	153	230	192	288	225	338
		OVS	142	214	178	267	193	289
		SSLT	153	230	192	288	225	338
	SC Class B	STD	153	230	192	288	230	345
		OVS	142	214	178	267	214	321
		SSLT	153	230	192	288	230	345
Group B	N	STD	153	230	192	288	230	345
	X	STD	153	230	192	288	230	345
	SC Class A	STD	153	230	192	288	230	345
		OVS	142	214	178	267	214	321
		SSLT	153	230	192	288	230	345
	SC Class B	STD	153	230	192	288	230	345
		OVS	142	214	178	267	214	321
		SSLT	153	230	192	288	230	345

Weld and Beam Web Available Strength, kips			
70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips
		ASD	LRFD
3/16	0.286	162	243
1/4	0.381	215	323
5/16	0.476	268	402
3/8	0.571	320	480

Support Available Strength per Inch Thickness, kip/in.	
ASD	LRFD
1470 STD/SSLT	2210 STD/SSLT
1380 OVS	2080 OVS

End-Plate	Beam
$F_y = 36$ ksi $F_u = 58$ ksi	$F_y = 50$ ksi $F_u = 65$ ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

Note: Slip-critical bolt values assume no more than one filler has been provided.

W44, 40, 36, 33

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

1-in. Bolts
9 Rows
l = 26 1/2 in.

Bolt and End-Plate Available Strength, kips								
Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
Group A	N	STD	138	206	172	258	206	310
	X	STD	138	206	172	258	206	310
	SC Class A	STD	138	206	172	258	202	304
		OVS	128	192	160	240	173	260
		SSLT	138	206	172	258	202	304
	SC Class B	STD	138	206	172	258	206	310
		OVS	128	192	160	240	192	288
		SSLT	138	206	172	258	206	310
Group B	N	STD	138	206	172	258	206	310
	X	STD	138	206	172	258	206	310
	SC Class A	STD	138	206	172	258	206	310
		OVS	128	192	160	240	192	288
		SSLT	138	206	172	258	206	310
	SC Class B	STD	138	206	172	258	206	310
		OVS	128	192	160	240	192	288
		SSLT	138	206	172	258	206	310

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kip/in.	
70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
3/16	0.286	145	218	1330 STD/SSLT	1990 STD/SSLT
1/4	0.381	193	290		
5/16	0.476	240	360	1250 OVS	1870 OVS
3/8	0.571	287	430		

	End-Plate	Beam
STD = Standard holes OVS = Oversized holes SSLT = Short-slotted holes transverse to direction of load N = Threads included X = Threads excluded SC = Slip critical	F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

Note: Slip-critical bolt values assume no more than one filler has been provided.

1-in. Bolts
8 Rows
$l = 23^1/_2$ in.

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

W44, 40, 36, 33, 30

Bolt and End-Plate Available Strength, kips								
Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
Group A	N	STD	122	183	152	228	183	274
	X	STD	122	183	152	228	183	274
	SC Class A	STD	122	183	152	228	179	269
		OVS	113	170	141	212	154	230
		SSLT	122	183	152	228	179	269
	SC Class B	STD	122	183	152	228	183	274
		OVS	113	170	141	212	170	254
		SSLT	122	183	152	228	183	274
Group B	N	STD	122	183	152	228	183	274
	X	STD	122	183	152	228	183	274
	SC Class A	STD	122	183	152	228	183	274
		OVS	113	170	141	212	170	254
		SSLT	122	183	152	228	183	274
	SC Class B	STD	122	183	152	228	183	274
		OVS	113	170	141	212	170	254
		SSLT	122	183	152	228	183	274

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kip/in.	
70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
3/16	0.286	129	193	1180 STD/SSLT	1770 STD/SSLT
1/4	0.381	171	256		
5/16	0.476	212	318	1110 OVS	1670 OVS
3/8	0.571	253	380		

		End-Plate	Beam
STD = Standard holes OVS = Oversized holes SSLT = Short-slotted holes transverse to direction of load	N = Threads included X = Threads excluded SC = Slip critical	$F_y = 36$ ksi $F_u = 58$ ksi	$F_y = 50$ ksi $F_u = 65$ ksi

Note: Slip-critical bolt values assume no more than one filler has been provided.

W44, 40, 36, 33, 30, 27, 24

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

1-in. Bolts
7 Rows
l = 20$^1/_2$ in.

Bolt and End-Plate Available Strength, kips								
Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
Group A	N	STD	106	159	133	199	159	239
	X	STD	106	159	133	199	159	239
	SC Class A	STD	106	159	133	199	156	234
		OVS	98.4	148	123	185	134	201
		SSLT	106	159	133	199	156	234
	SC Class B	STD	106	159	133	199	159	239
		OVS	98.4	148	123	185	148	221
		SSLT	106	159	133	199	159	239
Group B	N	STD	106	159	133	199	159	239
	X	STD	106	159	133	199	159	239
	SC Class A	STD	106	159	133	199	159	239
		OVS	98.4	148	123	185	148	221
		SSLT	106	159	133	199	159	239
	SC Class B	STD	106	159	133	199	159	239
		OVS	98.4	148	123	185	148	221
		SSLT	106	159	133	199	159	239

Weld and Beam Web Available Strength, kips			
70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips
		ASD	LRFD
3/16	0.286	112	168
1/4	0.381	148	223
5/16	0.476	184	277
3/8	0.571	220	330

Support Available Strength per Inch Thickness, kip/in.	
ASD	LRFD
1030 STD/SSLT	1550 STD/SSLT
975 OVS	1460 OVS

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

Note: Slip-critical bolt values assume no more than one filler has been provided.

1-in. Bolts
6 Rows
$l = 17\frac{1}{2}$ in.

Table 10-4 (continued) Bolted/Welded Shear End-Plate Connections

W44, 40, 36, 33, 30, 27, 24, 21

Bolt and End-Plate Available Strength, kips

Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD
Group A	N	STD	90.3	135	113	169	135	203
	X	STD	90.3	135	113	169	135	203
	SC Class A	STD	90.3	135	113	169	133	200
		OVS	83.7	126	105	157	115	171
		SSLT	90.3	135	113	169	133	200
	SC Class B	STD	90.3	135	113	169	135	203
		OVS	83.7	126	105	157	126	188
		SSLT	90.3	135	113	169	135	203
Group B	N	STD	90.3	135	113	169	135	203
	X	STD	90.3	135	113	169	135	203
	SC Class A	STD	90.3	135	113	169	135	203
		OVS	83.7	126	105	157	126	188
		SSLT	90.3	135	113	169	135	203
	SC Class B	STD	90.3	135	113	169	135	203
		OVS	83.7	126	105	157	126	188
		SSLT	90.3	135	113	169	135	203

Weld and Beam Web Available Strength, kips

70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD
3/16	0.286	95.4	143
1/4	0.381	126	189
5/16	0.476	157	235
3/8	0.571	187	280

Support Available Strength per Inch Thickness, kip/in.

Hole Type	ASD	LRFD
STD/SSLT	887	1330
OVS	839	1260

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

End-Plate	Beam
$F_y = 36$ ksi $F_u = 58$ ksi	$F_y = 50$ ksi $F_u = 65$ ksi

Note: Slip-critical bolt values assume no more than one filler has been provided.

W30, 27, 24, 21, 18

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

1-in. Bolts
5 Rows
l = 14 1/2 in.

Bolt and End-Plate Available Strength, kips								
Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
Group A	N	STD	74.5	112	93.1	140	112	168
	X	STD	74.5	112	93.1	140	112	168
	SC Class A	STD	74.5	112	93.1	140	110	165
		OVS	69.1	104	86.3	129	94.9	142
		SSLT	74.5	112	93.1	140	110	165
	SC Class B	STD	74.5	112	93.1	140	112	168
		OVS	69.1	104	86.3	129	104	155
		SSLT	74.5	112	93.1	140	112	168
Group B	N	STD	74.5	112	93.1	140	112	168
	X	STD	74.5	112	93.1	140	112	168
	SC Class A	STD	74.5	112	93.1	140	112	168
		OVS	69.1	104	86.3	129	104	155
		SSLT	74.5	112	93.1	140	112	168
	SC Class B	STD	74.5	112	93.1	140	112	168
		OVS	69.1	104	86.3	129	104	155
		SSLT	74.5	112	93.1	140	112	168

Weld and Beam Web Available Strength, kips			
70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips
		ASD	LRFD
3/16	0.286	78.7	118
1/4	0.381	104	156
5/16	0.476	129	193
3/8	0.571	153	230

Support Available Strength per Inch Thickness, kip/in.			
ASD		LRFD	
741	STD/SSLT	1110	STD/SSLT
702	OVS	1050	OVS

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

Note: Slip-critical bolt values assume no more than one filler has been provided.

1-in. Bolts
4 Rows
l = 11 1/2 in.

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

W24, 21, 18, 16

Bolt and End-Plate Available Strength, kips

Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD
Group A	N	STD	58.7	88.1	73.4	110	88.1	132
	X	STD	58.7	88.1	73.4	110	88.1	132
	SC Class A	STD	58.7	88.1	73.4	110	87.1	131
		OVS	54.4	81.6	68.0	102	75.3	113
		SSLT	58.7	88.1	73.4	110	87.1	131
	SC Class B	STD	58.7	88.1	73.4	110	88.1	132
		OVS	54.4	81.6	68.0	102	81.6	122
		SSLT	58.7	88.1	73.4	110	88.1	132
Group B	N	STD	58.7	88.1	73.4	110	88.1	132
	X	STD	58.7	88.1	73.4	110	88.1	132
	SC Class A	STD	58.7	88.1	73.4	110	88.1	132
		OVS	54.4	81.6	68.0	102	81.6	122
		SSLT	58.7	88.1	73.4	110	88.1	132
	SC Class B	STD	58.7	88.1	73.4	110	88.1	132
		OVS	54.4	81.6	68.0	102	81.6	122
		SSLT	58.7	88.1	73.4	110	88.1	132

Weld and Beam Web Available Strength, kips

70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD
3/16	0.286	61.9	92.9
1/4	0.381	81.7	123
5/16	0.476	101	151
3/8	0.571	120	180

Support Available Strength per Inch Thickness, kip/in.

ASD		LRFD	
595	STD/SSLT	892	STD/SSLT
566	OVS	848	OVS

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

Note: Slip-critical bolt values assume no more than one filler has been provided.

W18, 16, 14, 12, 10*

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

1-in. Bolts
3 Rows
$l = 8^1/_2$ in.

Bolt and End-Plate Available Strength, kips

Bolt Group	Thread Cond.	Hole Type	End-Plate Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD
Group A	N	STD	43.0	64.4	53.7	80.5	64.4	96.7
	X	STD	43.0	64.4	53.7	80.5	64.4	96.7
	SC Class A	STD	43.0	64.4	53.7	80.5	64.0	96.1
		OVS	39.7	59.5	49.6	74.4	55.6	83.3
		SSLT	43.0	64.4	53.7	80.5	64.0	96.1
	SC Class B	STD	43.0	64.4	53.7	80.5	64.4	96.7
		OVS	39.7	59.5	49.6	74.4	59.5	89.3
		SSLT	43.0	64.4	53.7	80.5	64.4	96.7
Group B	N	STD	43.0	64.4	53.7	80.5	64.4	96.7
	X	STD	43.0	64.4	53.7	80.5	64.4	96.7
	SC Class A	STD	43.0	64.4	53.7	80.5	64.4	96.7
		OVS	39.7	59.5	49.6	74.4	59.5	89.3
		SSLT	43.0	64.4	53.7	80.5	64.4	96.7
	SC Class B	STD	43.0	64.4	53.7	80.5	64.4	96.7
		OVS	39.7	59.5	49.6	74.4	59.5	89.3
		SSLT	43.0	64.4	53.7	80.5	64.4	96.7

Weld and Beam Web Available Strength, kips

70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD
3/16	0.286	45.2	67.9
1/4	0.381	59.4	89.1
5/16	0.476	73.1	110
3/8	0.571	86.3	129

Support Available Strength per Inch Thickness, kip/in.

ASD	LRFD
449 STD/SSLT	673 STD/SSLT
429 OVS	644 OVS

End-Plate	Beam
$F_y = 36$ ksi $F_u = 58$ ksi	$F_y = 50$ ksi $F_u = 65$ ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

*Limited to W10×12, 15, 17, 19, 22, 26, 30
Note: Slip-critical bolt values assume no more than one filler has been provided.

1-in. Bolts
2 Rows
l = 5½ in.

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

W12, 10, 8

Bolt and End-Plate Available Strength, kips								
			End-Plate Thickness, in.					
			1/4		5/16		3/8	
Bolt Group	Thread Cond.	Hole Type	ASD	LRFD	ASD	LRFD	ASD	LRFD
Group A	N	STD	27.2	40.8	34.0	51.0	40.8	61.2
	X	STD	27.2	40.8	34.0	51.0	40.8	61.2
	SC Class A	STD	27.2	40.8	34.0	51.0	40.8	61.2
		OVS	25.0	37.5	31.3	46.9	36.0	53.9
		SSLT	27.2	40.8	34.0	51.0	40.8	61.2
	SC Class B	STD	27.2	40.8	34.0	51.0	40.8	61.2
		OVS	25.0	37.5	31.3	46.9	37.5	56.3
		SSLT	27.2	40.8	34.0	51.0	40.8	61.2
Group B	N	STD	27.2	40.8	34.0	51.0	40.8	61.2
	X	STD	27.2	40.8	34.0	51.0	40.8	61.2
	SC Class A	STD	27.2	40.8	34.0	51.0	40.8	61.2
		OVS	25.0	37.5	31.3	46.9	37.5	56.3
		SSLT	27.2	40.8	34.0	51.0	40.8	61.2
	SC Class B	STD	27.2	40.8	34.0	51.0	40.8	61.2
		OVS	25.0	37.5	31.3	46.9	37.5	56.3
		SSLT	27.2	40.8	34.0	51.0	40.8	61.2

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kip/in.	
70-ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
3/16	0.286	28.5	42.8	302 STD/SSLT	453 STD/SSLT
1/4	0.381	37.1	55.7		
5/16	0.476	45.2	67.9	293 OVS	439 OVS
3/8	0.571	52.9	79.4		

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

Note: Slip-critical bolt values assume no more than one filler has been provided.

UNSTIFFENED SEATED CONNECTIONS

An unstiffened seated connection is made with a seat angle and a top angle, as illustrated in Figure 10-7. These angles may be bolted or welded to the supported beam as well as to the supporting member.

While the seat angle is assumed to carry the entire end reaction of the supported beam, the top angle must be placed as shown or in the optional side location for satisfactory performance and stability (Roeder and Dailey, 1989). The top angle and its connections are not usually sized for any calculated strength requirement. A 1/4-in.-thick angle with a 4-in. vertical leg dimension will generally be adequate. It may be bolted with two bolts

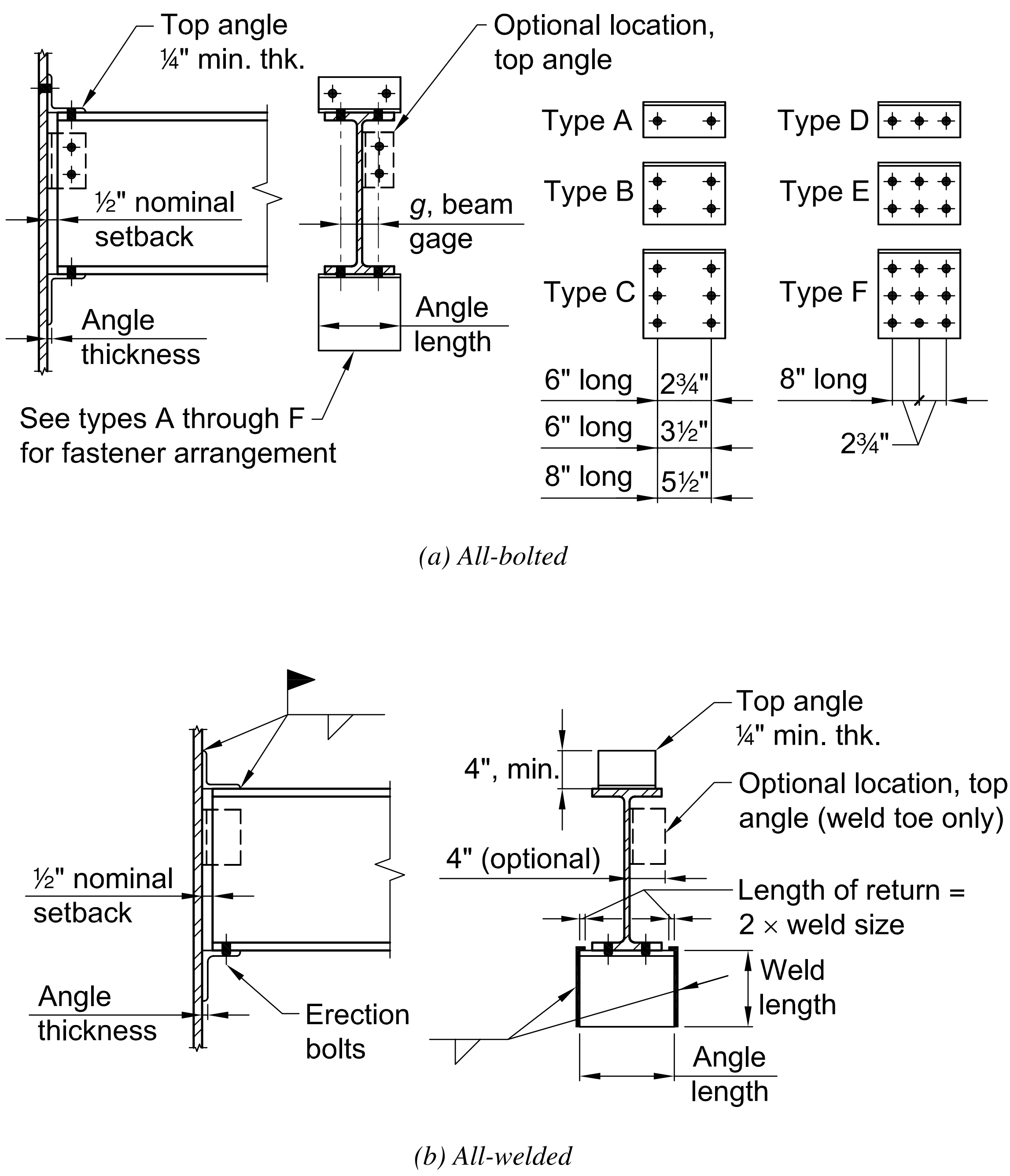


Fig. 10-7. Unstiffened seated connections.

through each leg or welded with minimum size welds to either the supported or the supporting members.

When the top angle is welded to the support and/or the supported beam, adequate flexibility must be provided in the connection. As illustrated in Figure 10-7(b), line welds are placed along the toe of each angle leg. Note that welding along the sides of the vertical angle leg must be avoided as it would inhibit the flexibility and, therefore, the necessary end rotation of the connection. The performance of such a connection would not be as intended for unstiffened seated connections.

Design Checks

The available strength of an unstiffened seated connection is determined from the applicable limit states for bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). Additionally, the strength of the supported beam web must be checked for the limit states of web local yielding and web local crippling. In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a. The available strength for web local yielding and web local crippling, ϕR_n or R_n/Ω, is determined per AISC *Specification* Sections J10.2 and J10.3, respectively, which is simplified using the constants in Table 9-4. For further information, see Carter et al. (1997).

Shop and Field Practices

Unstiffened seated connections may be made to the webs and flanges of supporting columns. If adequate clearance exists, unstiffened seated connections may also be made to the webs of supporting girders.

To provide for overrun in beam length, the nominal setback for the beam end is $^1/_2$ in. To provide for underrun in beam length, this setback is assumed to be $^3/_4$ in. for calculation purposes.

The seat angle is preferably shop-attached to the support. Since the bottom flange typically establishes the plane of reference for seated connections, mill variation in beam depth may result in variation in the elevation of the top flange. Such variation is usually of no consequence with concrete slab and metal deck floors, but may be a concern when a grating or steel-plate floor is used. Unless special care is required, the usual mill tolerances for member depth of $^1/_8$ in. to $^1/_4$ in. are ignored. However, when the top angle is shop-attached to the supported beam and field bolted to the support, mill variation in beam depth must be considered. Slotted holes, as illustrated in Figure 10-8(a), will accommodate both overrun and underrun in the beam depth and are the preferred method for economy and convenience to both the fabricator and erector. Alternatively, the angle could be shipped loose with clearance provided, as shown in Figure 10-8(b). When the top angle is to be field-welded to the support, no provision for mill variation in the beam depth is necessary.

When the top angle is shop-attached to the support, an appropriate erection clearance is provided, as illustrated in Figure 10-8(c).

Bolted/Welded Unstiffened Seated Connections

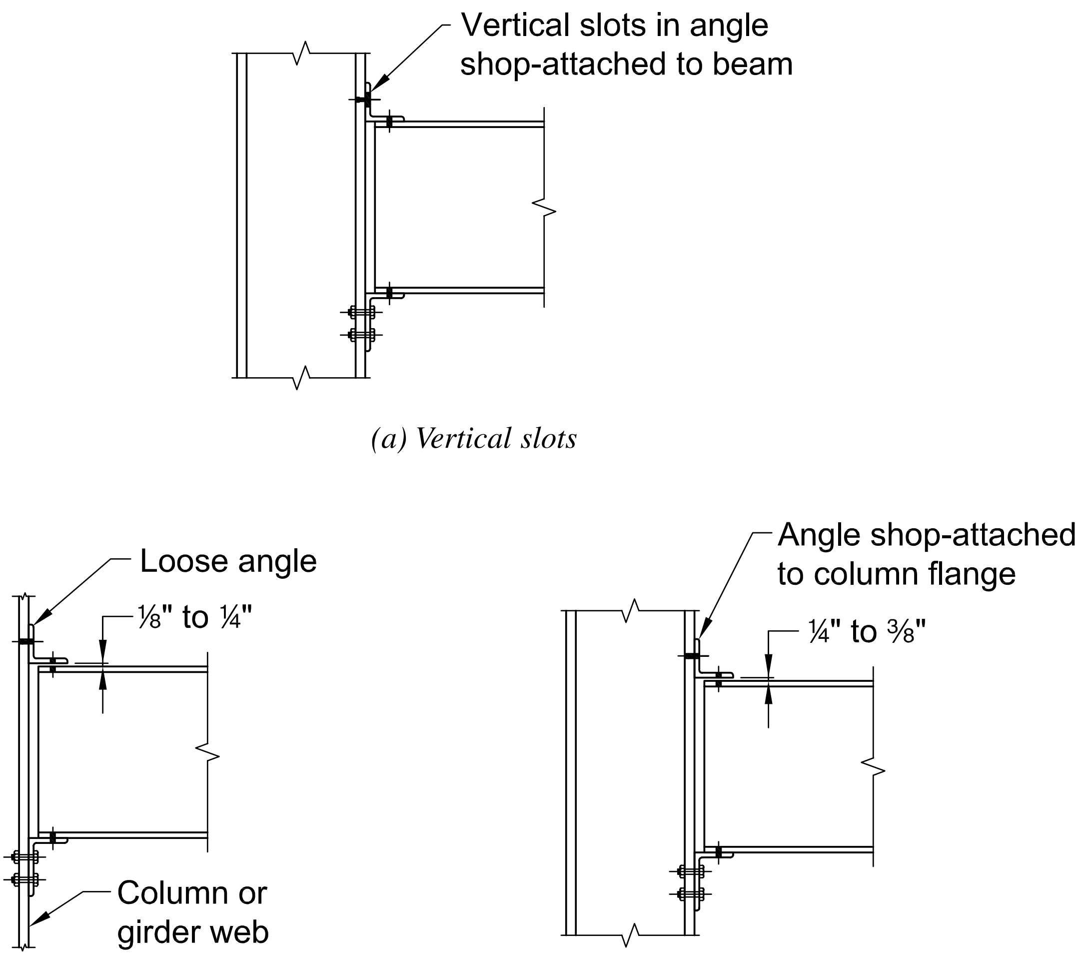
Tables 10-5 and 10-6 may be used in combination to design unstiffened seated connections that are welded to the supporting member and bolted to the supported beam.

DESIGN TABLE DISCUSSION (TABLES 10-5 AND 10-6)

Table 10-5. All-Bolted Unstiffened Seated Connections

Table 10-5 is a design aid for all-bolted unstiffened seats. Seat available strengths are tabulated, assuming a 4-in. outstanding leg, for angle material with F_y = 36 ksi and F_u = 58 ksi and beam material with F_y = 50 ksi and F_u = 65 ksi. All values are for comparison with the governing LRFD or ASD load combination.

Tabulated seat available strengths consider the limit states of shear yielding and flexural yielding of the outstanding angle leg. The required bearing length, $l_{b,req}$, is determined by the designer as the larger value of l_b required for the limit states of local yielding and crippling of the beam web. As noted in AISC *Specification* Section J10.2, the length of bearing must not be less than k for end beam reactions. A nominal beam setback of $^1/_2$ in. is assumed in these tables. However, this setback is increased to $^3/_4$ in. for calculation purposes in determining the tabulated values to account for the possibility of underrun in beam length.



(a) Vertical slots

(b) Loose angle with clearance as shown

(c) Shop-attached to column flange with clearance as shown

Fig. 10-8. Providing for variation in beam depth with seated connections.

Bolt available strengths are tabulated for the seat types illustrated in Figure 10-7(a) with $^3/_4$-in.-, $^7/_8$-in.- and 1-in.-diameter Group A and Group B bolts. Vertical spacing of bolts and gages in seat angles may be arranged to suit conditions, provided the edge distance and spacing requirements in AISC *Specification* Section J3 are met. Where thick angles are used, larger entering and tightening clearances may be required in the outstanding angle leg. The suitability of angle sizes and thicknesses for the seat types illustrated in Figure 10-7(a) is also listed in Table 10-5.

Table 10-6. All-Welded Unstiffened Seated Connections

Table 10-6 is a design aid for all-welded unstiffened seats (exception: the beam is bolted to the seat). Seat available strengths are tabulated, assuming either a $3^1/_2$-in. or 4-in. outstanding leg (as indicated in the table), for angle material with F_y = 36 ksi and F_u = 58 ksi and beam material with F_y = 50 ksi and F_u = 65 ksi. Electrode strength is assumed to be 70 ksi.

Tabulated seat available strengths consider the limit states of shear yielding and flexural yielding of the outstanding angle leg. The required bearing length, $l_{b,req}$, is to be determined by the designer as the larger value of l_b required for the limit states of local yielding and crippling of the beam web. As noted in AISC *Specification* Section J10.2, the length of bearing must not be less than k for end beam reactions. A nominal beam setback of $^1/_2$ in. is assumed in these tables. However, this setback is increased to $^3/_4$ in. for calculation purposes in determining the tabulated values to account for the possibility of underrun in beam length.

Tabulated weld available strengths are determined using the elastic method. The minimum and maximum angle thickness for each case is also tabulated. While these tabular values are based upon 70-ksi electrodes, they may be used for other electrodes, provided the tabular values are adjusted for the electrodes used (e.g., for 60-ksi electrodes, the tabular values are to be multiplied by 60/70 = 0.857, etc.) and the welds and base metal meet the available strength provisions of AISC *Specification* Table J2.5. Should combinations of material thickness and weld size selected from Table 10-6 exceed the limits in AISC *Specification* Section J2.2, the weld size or material thickness should be increased as required. Table 8-4 is not applicable to the design of these welds in this type of connection.

As can be seen from the following, reduction of the tabulated weld strength is not normally required when unstiffened seats line up on opposite sides of the supporting web. From Salmon et al. (2009), the available strength, ϕR_n or R_n/Ω, of the welds to the support is

LRFD	ASD
$\phi R_n = 2\left(\dfrac{1.392Dl}{\sqrt{1+\dfrac{20.25e^2}{l^2}}}\right)$ (10-2a)	$\dfrac{R_n}{\Omega} = 2\left(\dfrac{0.928Dl}{\sqrt{1+\dfrac{20.25e^2}{l^2}}}\right)$ (10-2b)

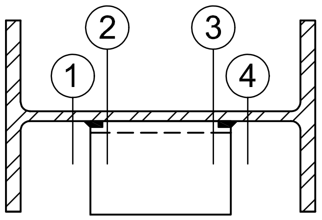
where

D = number of sixteenths-of-an-inch in the weld size
e = eccentricity of the beam end reaction with respect to the weld lines, in.
l = vertical leg dimension of the seat angle, in.

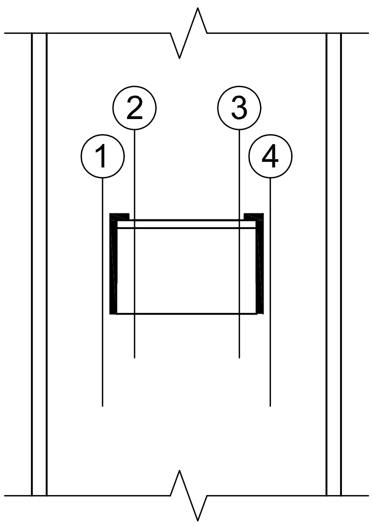
The term in the denominator that accounts for the eccentricity, e, increases the weld size far beyond what is required for shear alone, but with seats on both sides of the supporting member web, the forces due to eccentricity react against each other and have no effect on the web. Furthermore, as illustrated in Figure 10-9, there are actually two shear planes per weld; one at each weld toe and heel for a total of four shear planes. Thus, for an 8-in.-long L7×4×1 seat angle supporting an LRFD required strength of 70 kips or an equivalent ASD required strength of 46.7 kips, the minimum support thickness is determined as follows:

LRFD	ASD
$\dfrac{70 \text{ kips}}{0.75(0.60)(65 \text{ ksi})(7 \text{ in.})(4 \text{ planes})} = 0.0855 \text{ in.}$	$\dfrac{2.00(46.7 \text{ kips})}{0.60(65 \text{ ksi})(7 \text{ in.})(4 \text{ planes})} = 0.0855 \text{ in.}$

For the identical connection on both sides of the support, the minimum support thickness is less than $^3/_{16}$ in. Thus, the supporting web thickness is generally not a concern.



(a) Plan view



(b) Elevation

Fig. 10-9. Shear planes in column web for unstiffened seated connections.

Angle
F_y = 36 ksi

Table 10-5
All-Bolted Unstiffened Seated Connections

L6

Outstanding Angle Leg Length Strength, kips

Required Bearing Length $l_{b,req}$, in.	Angle Length, in. 6 — Angle Thickness, in. 3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD	5/8 ASD	5/8 LRFD	3/4 ASD	3/4 LRFD	1 ASD	1 LRFD	Min. Angle Leg, in.
1/2	18.2	27.3									
9/16	16.2	24.3	43.2	64.8							
5/8	14.6	21.9	43.1	64.8							
11/16	13.2	19.9	37.0	55.5							
3/4	12.1	18.2	32.3	48.6							
13/16	11.2	16.8	28.7	43.2							
7/8	10.4	15.6	25.9	38.9							
15/16	9.70	14.6	23.5	35.3	54.0	81.0					
1	9.09	13.7	21.6	32.4	50.5	75.9					
1 1/16	8.56	12.9	19.9	29.9	44.9	67.5					
1 1/8	8.08	12.2	18.5	27.8	40.4	60.8					
1 3/16	7.66	11.5	17.2	25.9	36.7	55.2					
1 1/4	7.28	10.9	16.2	24.3	33.7	50.6	64.8	97.2			
1 5/16	6.93	10.4	15.2	22.9	31.1	46.7	64.7	97.2			3 1/2
1 3/8	6.61	9.94	14.4	21.6	28.9	43.4	58.2	87.5			
1 7/16	6.33	9.51	13.6	20.5	26.9	40.5	52.9	79.5			
1 1/2	6.06	9.11	12.9	19.4	25.3	38.0	48.5	72.9			
1 5/8	5.60	8.41	11.8	17.7	22.5	33.8	41.6	62.5			
1 3/4	5.20	7.81	10.8	16.2	20.2	30.4	36.4	54.7			
1 7/8	4.85	7.29	10.0	15.0	18.4	27.6	32.3	48.6	86.4	130	
2	4.55	6.83	9.24	13.9	16.8	25.3	29.1	43.7	86.2	130	
2 1/8	4.28	6.43	8.62	13.0	15.5	23.4	26.5	39.8	73.9	111	
2 1/4	4.04	6.08	8.08	12.2	14.4	21.7	24.3	36.5	64.7	97.2	
2 3/8	3.83	5.76	7.61	11.4	13.5	20.3	22.4	33.6	57.5	86.4	
2 1/2	3.64	5.47	7.19	10.8	12.6	19.0	20.8	31.2	51.7	77.8	
2 5/8	3.46	5.21	6.81	10.2	11.9	17.9	19.4	29.2	47.0	70.7	
2 3/4	3.31	4.97	6.47	9.72	11.2	16.9	18.2	27.3	43.1	64.8	
2 7/8	3.16	4.75	6.16	9.26	10.6	16.0	17.1	25.7	39.8	59.8	
3	3.03	4.56	5.88	8.84	10.1	15.2	16.2	24.3	37.0	55.5	4
3 1/8	2.91	4.37	5.62	8.45	9.62	14.5	15.3	23.0	34.5	51.8	
3 1/4	2.80	4.21	5.39	8.10	9.19	13.8	14.6	21.9	32.3	48.6	

Bolt Available Strength, kips

Bolt Dia., in.	Bolt Group	Thread Cond.	Connection Type* A ASD	A LRFD	B ASD	B LRFD	C ASD	C LRFD
3/4	Group A	N	23.9	35.8	47.7	71.6	71.6	107
		X	30.1	45.1	60.1	90.2	90.2	135
	Group B	N	30.1	45.1	60.1	90.2	90.2	135
		X	37.1	55.7	74.3	111	111	167
7/8	Group A	N	32.5	48.7	64.9	97.4	97.4	146
		X	40.9	61.3	81.7	123	123	184
	Group B	N	40.9	61.3	81.7	123	123	184
		X	50.5	75.7	101	151	151	227
1	Group A	N	42.4	63.6	84.8	127	a	a
		X	53.4	80.1	107	160	a	a
	Group B	N	53.4	80.1	107	160	a	a
		X	65.9	98.9	132	198	a	a

Available Angles

Connection Type*	Angle Size	t, in.
A	4×3	3/8 – 1/2
	4×3 1/2	3/8 – 1/2
	4×4	3/8 – 3/4
B	6×4	3/8 – 3/4
	7×4	3/8 – 3/4
	8×4	1/2 – 1
C[a]	8×4	1/2 – 1

[a]Not suitable for use with 1-in.-diameter bolts.

ASD	LRFD
Ω = 2.00	ϕ = 0.75

For tabulated values above the heavy line, shear yielding of the angle leg controls the available strength.
*Connection type shown in Figure 10-7(a).

Table 10-5 (continued) All-Bolted Unstiffened Seated Connections

Angle F_y = 36 ksi

L8

Outstanding Angle Leg Length Strength, kips

Required Bearing Length $l_{b,req}$, in.	Angle Length, in. 8 — Angle Thickness, in. 3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD	5/8 ASD	5/8 LRFD	3/4 ASD	3/4 LRFD	1 ASD	1 LRFD	Min. Angle Leg, in.
1/2	24.3	36.5									
9/16	21.6	32.4	57.6	86.4							
5/8	19.4	29.2	57.5	86.4							
11/16	17.6	26.5	49.3	74.1							
3/4	16.2	24.3	43.1	64.8							
13/16	14.9	22.4	38.3	57.6							
7/8	13.9	20.8	34.5	51.8							
15/16	12.9	19.4	31.4	47.1	72.0	108					
1	12.1	18.2	28.7	43.2	67.4	101					
1 1/16	11.4	17.2	26.5	39.9	59.9	90.0					
1 1/8	10.8	16.2	24.6	37.0	53.9	81.0					
1 3/16	10.2	15.3	23.0	34.6	49.0	73.6					
1 1/4	9.70	14.6	21.6	32.4	44.9	67.5	86.4	130			
1 5/16	9.24	13.9	20.3	30.5	41.5	62.3	86.2	130			3 1/2
1 3/8	8.82	13.3	19.2	28.8	38.5	57.9	77.6	117			
1 7/16	8.44	12.7	18.2	27.3	35.9	54.0	70.5	106			
1 1/2	8.08	12.2	17.2	25.9	33.7	50.6	64.7	97.2			
1 5/8	7.46	11.2	15.7	23.6	29.9	45.0	55.4	83.3			
1 3/4	6.93	10.4	14.4	21.6	26.9	40.5	48.5	72.9			
1 7/8	6.47	9.72	13.3	19.9	24.5	36.8	43.1	64.8			
2	6.06	9.11	12.3	18.5	22.5	33.8	38.8	58.3	115	173	
2 1/8	5.71	8.58	11.5	17.3	20.7	31.2	35.3	53.0	98.5	148	
2 1/4	5.39	8.10	10.8	16.2	19.2	28.9	32.3	48.6	86.2	130	
2 3/8	5.11	7.67	10.1	15.2	18.0	27.0	29.8	44.9	76.6	115	
2 1/2	4.85	7.29	9.58	14.4	16.8	25.3	27.7	41.7	69.0	104	
2 5/8	4.62	6.94	9.08	13.6	15.9	23.8	25.9	38.9	62.7	94.3	
2 3/4	4.41	6.63	8.62	13.0	15.0	22.5	24.3	36.5	57.5	86.4	
2 7/8	4.22	6.34	8.21	12.3	14.2	21.3	22.8	34.3	53.1	79.8	
3	4.04	6.08	7.84	11.8	13.5	20.3	21.6	32.4	49.3	74.1	4
3 1/8	3.88	5.83	7.50	11.3	12.8	19.3	20.4	30.7	46.0	69.1	
3 1/4	3.73	5.61	7.19	10.8	12.2	18.4	19.4	29.2	43.1	64.8	

Bolt Available Strength, kips

Bolt Dia., in.	Bolt Group	Thread Cond.	Connection Type* D ASD	D LRFD	E ASD	E LRFD	F ASD	F LRFD
3/4	Group A	N	35.8	53.7	71.6	107	107	161
		X	45.1	67.6	90.2	135	135	203
	Group B	N	45.1	67.6	90.2	135	135	203
		X	55.7	83.5	111	167	167	251
7/8	Group A	N	48.7	73.0	97.4	146	146	219
		X	61.3	92.0	123	184	184	276
	Group B	N	61.3	92.0	123	184	184	276
		X	75.7	114	151	227	227	341
1	Group A	N	63.6	95.4	127	191	a	a
		X	80.1	120	160	240	a	a
	Group B	N	80.1	120	160	240	a	a
		X	98.9	148	198	297	a	a

Available Angles

Connection Type*	Angle Size	t, in.
A, D	4×3	3/8 – 1/2
	4×3 1/2	3/8 – 1/2
	4×4	3/8 – 3/4
B, E	6×4	3/8 – 3/4
	7×4	3/8 – 3/4
	8×4	1/2 – 1
C[a], F[a]	8×4	1/2 – 1

[a]Not suitable for use with 1-in.-diameter bolts.

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

For tabulated values above the heavy line, shear yielding of the angle leg controls the available strength.

*Connection type shown in Figure 10-7(a).

Angle
F_y = 36 ksi

Table 10-6
All-Welded Unstiffened Seated Connections

L6

Outstanding Angle Leg Length Strength, kips

Required Bearing Length $l_{b,req}$, in.	Angle Length, in. 6 — Angle Thickness, in. 3/8		1/2		5/8		3/4		1		Min. Angle Leg
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	in.
1/2	18.2	27.3									
9/16	16.2	24.3									
5/8	14.6	21.9	43.1	64.8							
11/16	13.2	19.9	37.0	55.5							
3/4	12.1	18.2	32.3	48.6							
13/16	11.2	16.8	28.7	43.2							
7/8	10.4	15.6	25.9	38.9							
15/16	9.70	14.6	23.5	35.3	54.0	81.0					
1	9.09	13.7	21.6	32.4	50.5	75.9					
1 1/16	8.56	12.9	19.9	29.9	44.9	67.5					
1 1/8	8.08	12.2	18.5	27.8	40.4	60.8					
1 3/16	7.66	11.5	17.2	25.9	36.7	55.2					
1 1/4	7.28	10.9	16.2	24.3	33.7	50.6					
1 5/16	6.93	10.4	15.2	22.9	31.1	46.7	64.7	97.2			3 1/2
1 3/8	6.61	9.94	14.4	21.6	28.9	43.4	58.2	87.5			
1 7/16	6.33	9.51	13.6	20.5	26.9	40.5	52.9	79.5			
1 1/2	6.06	9.11	12.9	19.4	25.3	38.0	48.5	72.9			
1 5/8	5.60	8.41	11.8	17.7	22.5	33.8	41.6	62.5			
1 3/4	5.20	7.81	10.8	16.2	20.2	30.4	36.4	54.7			
1 7/8	4.85	7.29	9.95	15.0	18.4	27.6	32.3	48.6			
2	4.55	6.83	9.24	13.9	16.8	25.3	29.1	43.7	86.2	130	
2 1/8	4.28	6.43	8.62	13.0	15.5	23.4	26.5	39.8	73.9	111	
2 1/4	4.04	6.08	8.08	12.2	14.4	21.7	24.3	36.5	64.7	97.2	
2 3/8	3.83	5.76	7.61	11.4	13.5	20.3	22.4	33.6	57.5	86.4	
2 1/2	3.64	5.47	7.19	10.8	12.6	19.0	20.8	31.2	51.7	77.8	
2 5/8	3.46	5.21	6.81	10.2	11.9	17.9	19.4	29.2	47.0	70.7	
2 3/4	3.31	4.97	6.47	9.72	11.2	16.9	18.2	27.3	43.1	64.8	
2 7/8	3.16	4.75	6.16	9.26	10.6	16.0	17.1	25.7	39.8	59.8	
3	3.03	4.56	5.88	8.84	10.1	15.2	16.2	24.3	37.0	55.5	4
3 1/8	2.91	4.37	5.62	8.45	9.62	14.5	15.3	23.0	34.5	51.8	
3 1/4	2.80	4.21	5.39	8.10	9.19	13.8	14.6	21.9	32.3	48.6	

Weld Available Strength, kips

70-ksi Weld Size, in.	Seat Angle Size (long leg vertical) 4×3 1/2		5×3 1/2	
Design	ASD	LRFD	ASD	LRFD
1/4	11.5	17.2	17.2	25.8
5/16	14.3	21.5	21.5	32.2
3/8	17.2	25.8	25.8	38.7
7/16	20.1	30.1	30.1	45.2
1/2	–	–	34.4	51.6
9/16	–	–	38.7	58.1
5/8	–	–	43.0	64.5
11/16	–	–	47.3	71.0

Available Angle Thickness, in.

	4×3 1/2	5×3 1/2
Minimum	3/8	3/8
Maximum	1/2	3/4

ASD	LRFD
Ω = 2.00	ϕ = 0.75

For tabulated values above the heavy line, shear yielding of the angle leg controls the available strength.

– Indicates weld size exceeds that permitted for maximum angle thickness of 1/2 in.

Angle
F_y = 36 ksi

Table 10-6 (continued)
All-Welded Unstiffened Seated Connections

L8

Outstanding Angle Leg Length Strength, kips

Required Bearing Length $l_{b,req}$, in.	Angle Length, in. 8 — Angle Thickness, in. 3/8		1/2		5/8		3/4		1		Min. Angle Leg
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	in.
1/2	24.3	36.5									
9/16	21.6	32.4									
5/8	19.4	29.2	57.5	86.4							
11/16	17.6	26.5	49.3	74.1							
3/4	16.2	24.3	43.1	64.8							
13/16	14.9	22.4	38.3	57.6							
7/8	13.9	20.8	34.5	51.8							
15/16	12.9	19.4	31.4	47.1	72.0	108					
1	12.1	18.2	28.7	43.2	67.4	101					
1 1/16	11.4	17.2	26.5	39.9	59.9	90.0					
1 1/8	10.8	16.2	24.6	37.0	53.9	81.0					
1 3/16	10.2	15.3	23.0	34.6	49.0	73.6					
1 1/4	9.70	14.6	21.6	32.4	44.9	67.5					
1 5/16	9.24	13.9	20.3	30.5	41.5	62.3	86.2	130			3 1/2
1 3/8	8.82	13.3	19.2	28.8	38.5	57.9	77.6	117			
1 7/16	8.44	12.7	18.2	27.3	35.9	54.0	70.5	106			
1 1/2	8.08	12.2	17.2	25.9	33.7	50.6	64.7	97.2			
1 5/8	7.46	11.2	15.7	23.6	29.9	45.0	55.4	83.3			
1 3/4	6.93	10.4	14.4	21.6	26.9	40.5	48.5	72.9			
1 7/8	6.47	9.72	13.3	19.9	24.5	36.8	43.1	64.8			
2	6.06	9.11	12.3	18.5	22.5	33.8	38.8	58.3	115	173	
2 1/8	5.71	8.58	11.5	17.3	20.7	31.2	35.3	53.0	98.5	148	
2 1/4	5.39	8.10	10.8	16.2	19.2	28.9	32.3	48.6	86.2	130	
2 3/8	5.11	7.67	10.1	15.2	18.0	27.0	29.8	44.9	76.6	115	
2 1/2	4.85	7.29	9.58	14.4	16.8	25.3	27.7	41.7	69.0	104	
2 5/8	4.62	6.94	9.08	13.6	15.9	23.8	25.9	38.9	62.7	94.3	
2 3/4	4.41	6.63	8.62	13.0	15.0	22.5	24.3	36.5	57.5	86.4	
2 7/8	4.22	6.34	8.21	12.3	14.2	21.3	22.8	34.3	53.1	79.8	
3	4.04	6.08	7.84	11.8	13.5	20.3	21.6	32.4	49.3	74.1	4
3 1/8	3.88	5.83	7.50	11.3	12.8	19.3	20.4	30.7	46.0	69.1	
3 1/4	3.73	5.61	7.19	10.8	12.2	18.4	19.4	29.2	43.1	64.8	

Weld Available Strength, kips

70-ksi Weld Size, in.	Seat Angle Size (long leg vertical) 6×4		7×4		8×4	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD
1/4	21.8	32.7	28.5	42.7	35.6	53.4
5/16	27.3	40.9	35.6	53.4	44.5	66.7
3/8	32.7	49.1	42.7	64.1	53.4	80.1
7/16	38.2	57.2	49.8	74.7	62.3	93.4
1/2	43.6	65.4	57.0	85.4	71.2	107
9/16	49.1	73.6	64.1	96.1	80.1	120
5/8	54.5	81.8	71.2	107	89.0	133
11/16	60.0	90.0	78.3	117	97.9	147

Available Angle Thickness, in.

	6×4	7×4	8×4
Minimum	3/8	3/8	1/2
Maximum	3/4	3/4	1

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

For tabulated values above the heavy line, shear yielding of the angle leg controls the available strength.

STIFFENED SEATED CONNECTIONS

A stiffened seated connection is made with a seat plate and stiffening element (e.g., a plate, structural tee, or pair of angles) and a top angle, as illustrated in Figure 10-10. The top angle may be bolted or welded to the supported beam as well as to the supporting member and the stiffening element may be bolted or welded to the support. The supported beam is bolted to the seat plate.

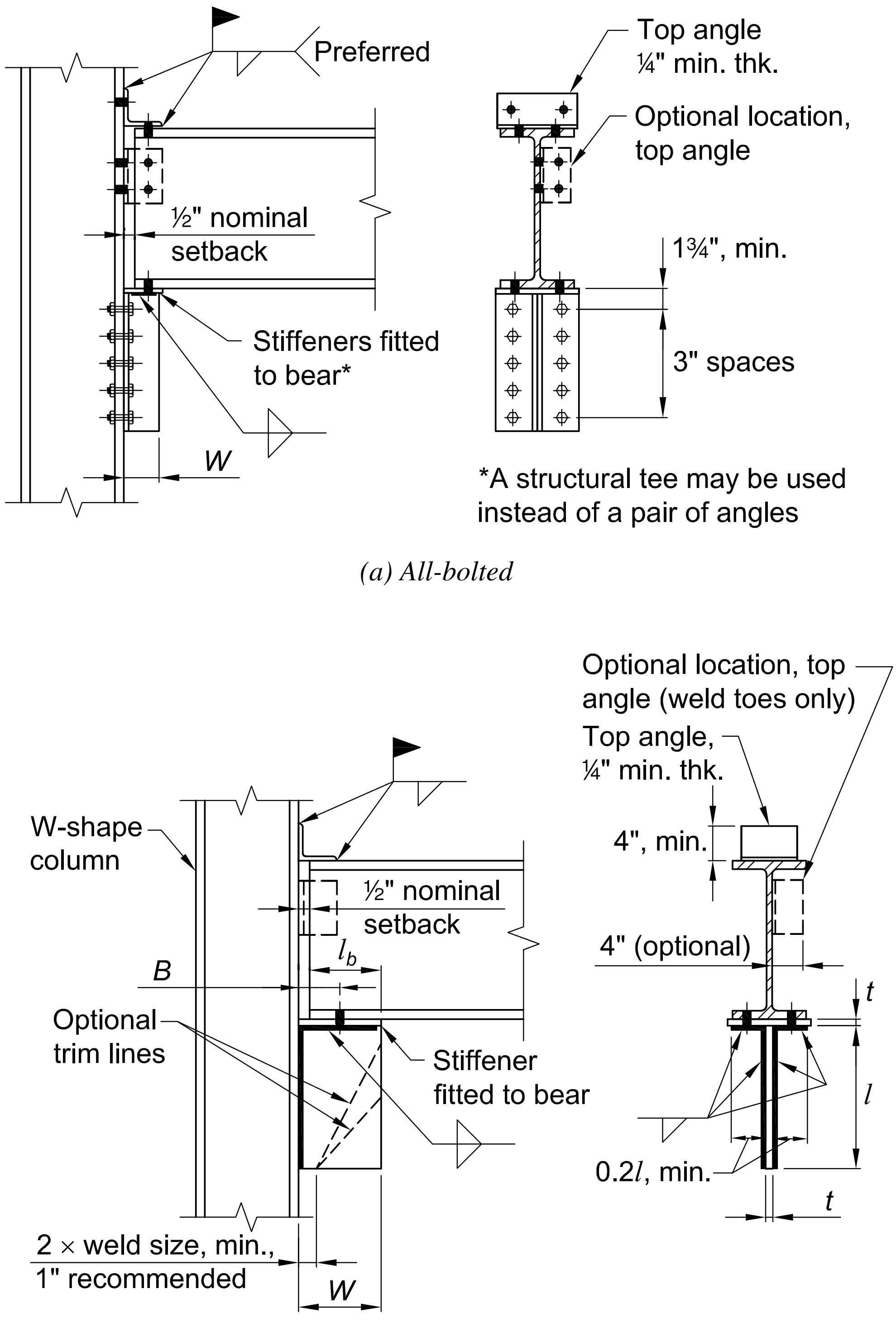


Fig. 10-10. Stiffened seated connections.

The stiffening element is assumed to carry the entire end reaction of the supported beam applied at a distance equal to $0.8W$, where W is the dimension of the stiffening element parallel to the beam web. The top angle must be placed as shown or in the optional side location for satisfactory performance and stability (Roeder and Dailey, 1989). The top angle and its connections are not usually sized for any calculated strength requirement. A $^1/_4$-in.-thick angle with a 4-in. vertical leg dimension will generally be adequate. It may be fastened with two bolts through each leg or welded with minimum size welds to either the supported or the supporting members.

When the top angle is welded to the support and/or the supported beam, adequate flexibility must be provided in the connection. As illustrated in Figure 10-10(b), line welds are placed along the toe of each angle leg. Note that welding along the sides of the vertical angle leg must be avoided as it would inhibit the flexibility and, therefore, the necessary end rotation of the connection. The performance of such a connection would not be as intended for simple shear connections.

Design Checks

The available strength of a stiffened seated connection is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). Additionally, the strength of the supported beam web must be checked for the limit states of web local yielding and web local crippling. In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a. The available strength for web local yielding and web local crippling, ϕR_n or R_n/Ω, is determined per AISC *Specification* Sections J10.2 and J10.3, respectively, which is simplified using the constants in Table 9-4.

When stiffened seated connections, such as the one shown in Figure 10-10(b), are made to one side of a supporting column web, the column web may also need to be investigated for resistance to punching shear. In lieu of a more detailed analysis, Sputo and Ellifritt (1991) showed that punching shear will not be critical if the design parameters following and those summarized graphically in Figure 10-10(b) are met.

1. This simplified approach is applicable to the following column sections:

W14×43 to 873	W12×40 to 336	W10×33 to 112
W8×24 to 67	W6×20 and 25	W5×16 and 19

2. The supported beam must be bolted to the seat plate with high-strength bolts to account for the prying action caused by rotation of the connection. Welding the beam to the seat plate is not recommended because welds may lack the required strength and ductility. The centerline of the bolts should be located no more than the greater of $W/2$ or $2^5/_8$ in. from the column web face.
3. For seated connections where W = 8 in. or 9 in. and $3^1/_2$ in. < $B \le W/2$, or where W = 7 in. and 3 in. < $B \le W/2$ for a W14×43 column, refer to Sputo and Ellifritt (1991).
4. The top angle may be bolted or welded, but must have a minimum $^1/_4$-in. thickness.
5. The seat plate should not be welded to the beam flange.

See also Ellifritt and Sputo (1999).

Shop and Field Practices

The comments for unstiffened seated connections are equally applicable to stiffened seated connections.

DESIGN TABLE DISCUSSION (TABLES 10-7 AND 10-8)

Table 10-7. All-Bolted Stiffened Seated Connections

Table 10-7 is a design aid for all-bolted stiffened seats. Stiffener available strengths are tabulated for stiffener material with $F_y = 36$ ksi and $F_u = 58$ ksi and with $F_y = 50$ ksi and $F_u = 65$ ksi.

Tabulated values consider the limit state of bearing on the stiffening material. The designer must independently check the available strength of the beam web based upon the limit states of web local yielding and web local crippling. A nominal beam setback of $^1/_2$ in. is assumed in these tables. However, this setback is increased to $^3/_4$ in. for calculation purposes in determining the tabulated values to account for the possibility of underrun in beam length.

Bolt available strengths are tabulated for two vertical rows of from three to seven $^3/_4$-in.-, $^7/_8$-in.- and 1-in.-diameter Group A and Group B bolts based upon the limit state of bolt shear. Vertical spacing of bolts and gages in seat angles may be arranged to suit conditions, provided the edge distance and spacing requirements in AISC *Specification* Section J3 are met.

Table 10-8. Bolted/Welded Stiffened Seated Connections

Table 10-8 is a design aid for stiffened seated connections welded to the support and bolted to the supported beam. Electrode strength is assumed to be 70 ksi.

Weld available strengths are tabulated using the elastic method. While these tabular values are based upon 70-ksi electrodes, they may be used for other electrodes, provided the tabular values are adjusted for the electrodes used (e.g., for 60-ksi electrodes, the tabular values are multiplied by 60/70 = 0.857, etc.) and the weld and base metal meet the required strength provisions of AISC *Specification* Table J2.5.

The thickness of the horizontal seat plate or tee flange should not be less than $^3/_8$ in. If the seat and stiffener are built up from separate plates, the stiffener should be finished to bear under the seat. In order to take advantage of the T-shaped weld to the column, the connection between the seat plate and stiffener must have a strength equal to or greater than that of the horizontal portion of the T-shaped weld.

The designer must independently check the beam web for web local yielding and web local crippling. The nominal beam setback of $^1/_2$ in. should be assumed to be $^3/_4$ in. for calculation purposes to account for possible underrun in beam length.

The stiffener thickness is conservatively determined as follows. The minimum stiffener plate thickness, t, for supported beams with unstiffened webs is the supported beam web thickness, t_w, multiplied by the ratio of F_y of the beam material to F_y of the stiffener material (e.g., $F_{y,beam} = 50$ ksi, $F_{y,stiffener} = 36$ ksi, $t = t_w \times 50/36$ minimum). Additionally, the minimum stiffener plate thickness, t, should be at least $2w$ for stiffener material with

F_y = 36 ksi or 1.5w for stiffener material with F_y = 50 ksi, where w is the weld size for 70-ksi electrodes.

For 70-ksi electrodes, the minimum column web thickness is

$$t_{min} = \frac{3.09D}{F_u} \tag{9-2}$$

where

D = weld size in sixteenths of an inch
F_u = specified minimum tensile strength of the connecting element, ksi

When welds line up on opposite sides of the support, the minimum thickness is the sum of the thicknesses required for each weld. In either case, when less than the minimum material thickness is present, the weld available strength must be reduced by the ratio of the thickness provided to the minimum thickness. As with unstiffened seated connections, the contribution of eccentricity to the required shear yielding strength is negligible. Should combinations of material thickness and weld size selected from Table 10-8 exceed the limits of AISC *Specification* Section J2.2, the weld size or material thickness must be increased.

Table 10-7
All-Bolted Stiffened Seated Connections

Stiffener Material		Outstanding Angle Leg Available Strength, kips[a]											
		F_y = 36 ksi						F_y = 50 ksi					
Stiffener Outstanding Leg, W, in.[b]		$3^1/_2$		4		5		$3^1/_2$		4		5	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Thickness of Stiffener Outstanding Legs, in.	$^5/_{16}$	55.7	83.5	65.8	98.7	86.1	129	77.3	116	91.4	137	120	179
	$^3/_8$	66.8	100	79.0	118	103	155	92.8	139	110	165	143	215
	$^1/_2$	89.1	134	105	158	138	207	124	186	146	219	191	287
	$^5/_8$	111	167	132	197	172	258	155	232	183	274	239	359
	$^3/_4$	134	200	158	237	207	310	186	278	219	329	287	430

Use minimum $^3/_8$-in.-thick seat plate wide enough to extend beyond outstanding legs of stiffener.

[a] See AISC *Specification* Section J7.

[b] Beam bearing length assumed $^3/_4$ in. less for calculation purposes.

Bolt Available Strength, kips												
Bolt Diameter, in.	Bolt Group	Thread Cond.	Number of Bolts in One Vertical Row									
			3		4		5		6		7	
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
$^3/_4$	Group A	N	71.6	107	95.5	143	119	179	143	215	167	251
		X	90.2	135	120	180	150	225	180	271	210	316
	Group B	N	90.2	135	120	180	150	225	180	271	210	316
		X	111	167	149	223	186	278	223	334	260	390
$^7/_8$	Group A	N	97.4	146	130	195	162	243	195	292	227	341
		X	123	184	163	245	204	307	245	368	286	429
	Group B	N	123	184	163	245	204	307	245	368	286	429
		X	151	227	202	303	252	379	303	454	353	530
1	Group A	N	127	191	170	254	212	318	254	382	297	445
		X	160	240	214	320	267	400	320	480	374	560
	Group B	N	160	240	214	320	267	400	320	480	374	560
		X	198	297	264	396	330	495	396	593	462	692

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$
$\dfrac{R_n}{\Omega} = \dfrac{1.8F_yA_{pb}}{\Omega}$	$\phi R_n = \phi\left(1.8F_yA_{pb}\right)$

Table 10-8
Bolted/Welded Stiffened Seated Connections
Weld Available Strength, kips

l, in.	Width of Seat, W, in.											
	4								5			
	70-ksi Weld Size, in.								70-ksi Weld Size, in.			
	1/4		5/16		3/8		7/16		5/16		3/8	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
6	22.7	34.0	28.4	42.5	34.0	51.1	39.7	59.6	23.5	35.2	28.2	42.2
7	29.9	44.9	37.4	56.1	44.9	67.3	52.4	78.6	31.2	46.9	37.5	56.2
8	37.8	56.7	47.2	70.8	56.7	85.0	66.1	99.2	39.8	59.8	47.8	71.7
9	46.1	69.2	57.7	86.5	69.2	104	80.7	121	49.1	73.7	59.0	88.5
10	54.9	82.3	68.6	103	82.3	123	96.0	144	59.0	88.5	70.8	106
11	63.9	95.8	79.8	120	95.8	144	112	168	69.4	104	83.3	125
12	73.1	110	91.4	137	110	165	128	192	80.2	120	96.2	144
13	82.5	124	103	155	124	186	144	217	91.3	137	110	164
14	92.1	138	115	173	138	207	161	242	103	154	123	185
15	102	152	127	191	152	229	178	267	114	171	137	206
16	111	167	139	209	167	250	195	292	126	189	151	227
17	121	181	151	227	181	272	212	318	138	207	165	248
18	131	196	163	245	196	294	229	343	150	225	180	270
19	140	211	175	263	211	316	246	369	162	243	194	291
20	150	225	188	281	225	338	263	394	174	261	209	313
21	160	240	200	300	240	359	280	419	186	279	223	335
22	169	254	212	318	254	381	296	445	198	297	238	357
23	179	269	224	336	269	403	313	470	210	315	252	378
24	189	283	236	354	283	425	330	495	222	334	267	400
25	198	297	248	372	297	446	347	520	235	352	281	422
26	208	312	260	390	312	468	364	546	247	370	296	444
27	217	326	272	408	326	489	380	571	259	388	310	466

Limitations for Connections to Column Webs

B = 2 5/8 in. max	B = 2 5/8 in. max
W12×40, W14×43 for $l \geq 9$ in. limit weld ≤ 1/4 in.	None

Notes:
1. Values shown assume 70-ksi electrodes. For 60-ksi electrodes, multiply tabular values by 0.857, or enter table with 1.17 times the required strength, R_u or R_a. For 80-ksi electrodes, multiply tabular values by 1.14, or enter table with 0.875 times the required strength.
2. Tabulated values are valid for stiffeners with minimum thickness of

$$t_{min} = \left(\frac{F_{y,\,beam}}{F_{y,\,stiffener}}\right) t_w$$

but not less than $2w$ for stiffeners with $F_y = 36$ ksi nor $1.5w$ for stiffeners with $F_y = 50$ ksi. In the above, t_w is the thickness of the unstiffened supported beam web and w is the nominal weld size.
3. Tabulated values may be limited by shear yielding of the stiffener, or bearing on the stiffener; refer to AISC *Specification* Sections J4.2 and J7, respectively.

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

Table 10-8 (continued)
Bolted/Welded Stiffened Seated Connections
Weld Available Strength, kips

l, in.	Width of Seat, W, in.											
	5				6							
	70-ksi Weld Size, in.				70-ksi Weld Size, in.							
	7/16		1/2		5/16		3/8		7/16		1/2	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
6	32.8	49.3	37.5	56.3	19.9	29.9	23.9	35.9	27.9	41.9	31.9	47.8
7	43.7	65.6	50.0	75.0	26.7	40.1	32.0	48.1	37.4	56.1	42.7	64.1
8	55.8	83.7	63.8	95.6	34.3	51.4	41.1	61.7	48.0	72.0	54.8	82.2
9	68.8	103	78.6	118	42.5	63.8	51.1	76.6	59.6	89.3	68.1	102
10	82.6	124	94.4	142	51.4	77.2	61.7	92.6	72.0	108	82.3	123
11	97.2	146	111	167	60.9	91.3	73.1	110	85.3	128	97.4	146
12	112	168	128	192	70.8	106	85.0	127	99.2	149	113	170
13	128	192	146	219	81.2	122	97.4	146	114	170	130	195
14	144	216	164	246	91.9	138	110	165	129	193	147	220
15	160	240	183	274	103	154	123	185	144	216	165	247
16	176	265	202	302	114	171	137	205	160	240	183	274
17	193	290	221	331	126	188	151	226	176	264	201	301
18	210	315	240	360	137	206	165	247	192	288	219	329
19	227	340	259	388	149	223	179	268	208	313	238	357
20	244	365	278	417	161	241	193	289	225	337	257	386
21	260	391	298	446	173	259	207	311	242	362	276	414
22	277	416	317	476	185	277	222	332	258	388	295	443
23	294	442	336	505	197	295	236	354	275	413	315	472
24	311	467	356	534	209	313	250	376	292	438	334	501
25	328	492	375	563	221	331	265	397	309	464	353	530
26	345	518	395	592	233	349	280	419	326	489	373	559
27	362	543	414	621	245	368	294	441	343	515	392	588

Limitations for Connections to Column Webs	
$B = 2^5/_8$ in. max	$B = 3$ in. max
None	None

Notes:
1. Values shown assume 70-ksi electrodes. For 60-ksi electrodes, multiply tabular values by 0.857, or enter table with 1.17 times the required strength, R_u or R_a. For 80-ksi electrodes, multiply tabular values by 1.14, or enter table with 0.875 times the required strength.
2. Tabulated values are valid for stiffeners with minimum thickness of

$$t_{min} = \left(\frac{F_{y,\,beam}}{F_{y,\,stiffener}}\right) t_w$$

but not less than $2w$ for stiffeners with $F_y = 36$ ksi nor $1.5w$ for stiffeners with $F_y = 50$ ksi. In the above, t_w is the thickness of the unstiffened supported beam web and w is the nominal weld size.
3. Tabulated values may be limited by shear yielding of the stiffener, or bearing on the stiffener; refer to AISC *Specification* Sections J4.2 and J7, respectively.

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

Table 10-8 (continued)
Bolted/Welded Stiffened Seated Connections
Weld Available Strength, kips

l, in.	Width of Seat, *W*, in.											
	7								8			
	70-ksi Weld Size, in.								70-ksi Weld Size, in.			
	5/16		3/8		7/16		1/2		5/16		3/8	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
11	54.0	81.0	64.8	97.2	75.6	113	86.4	130	48.4	72.5	58.0	87.1
12	63.1	94.7	75.7	114	88.4	133	101	151	56.7	85.1	68.1	102
13	72.7	109	87.2	131	102	153	116	174	65.6	98.3	78.7	118
14	82.6	124	99.2	149	116	174	132	198	74.8	112	89.8	135
15	93.0	139	112	167	130	195	149	223	84.5	127	101	152
16	104	155	124	186	145	217	166	249	94.4	142	113	170
17	114	172	137	206	160	240	183	275	105	157	126	189
18	126	188	151	226	176	264	201	301	115	173	138	208
19	137	205	164	246	192	287	219	329	126	189	151	227
20	148	223	178	267	208	312	237	356	137	206	165	247
21	160	240	192	288	224	336	256	384	148	222	178	267
22	172	258	206	309	240	361	275	412	160	240	192	287
23	184	275	220	330	257	385	294	440	171	257	205	308
24	195	293	234	352	274	410	313	469	183	274	219	329
25	207	311	249	373	290	435	332	498	195	292	233	350
26	219	329	263	395	307	461	351	526	206	309	248	371
27	231	347	278	417	324	486	370	555	218	327	262	393
28	244	365	292	438	341	511	390	584	230	345	276	414
29	256	383	307	460	358	537	409	613	242	363	291	436
30	268	402	321	482	375	562	428	643	254	381	305	457
31	280	420	336	504	392	588	448	672	266	399	319	479
32	292	438	350	526	409	613	467	701	278	417	334	501

Limitations for Connections to Column Webs	
$B = 3^1/_2$ in. max	$B = 3^1/_2$ in. max
W14×43, limit $B \leq 3$ in. See item 3 in preceding discussion "Design Checks"	See item 3 in preceding discussion "Design Checks"

Notes:
1. Values shown assume 70-ksi electrodes. For 60-ksi electrodes, multiply tabular values by 0.857, or enter table with 1.17 times the required strength, R_u or R_a. For 80-ksi electrodes, multiply tabular values by 1.14, or enter table with 0.875 times the required strength.
2. Tabulated values are valid for stiffeners with minimum thickness of

$$t_{min} = \left(\frac{F_{y,\,beam}}{F_{y,\,stiffener}}\right) t_w$$

but not less than $2w$ for stiffeners with $F_y = 36$ ksi nor $1.5w$ for stiffeners with $F_y = 50$ ksi. In the above, t_w is the thickness of the unstiffened supported beam web and w is the nominal weld size.
3. Tabulated values may be limited by shear yielding of the stiffener, or bearing on the stiffener; refer to AISC *Specification* Sections J4.2 and J7, respectively.

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

Table 10-8 (continued)
Bolted/Welded Stiffened Seated Connections
Weld Available Strength, kips

l, in.	Width of Seat, *W*, in.											
	8				9							
	70-ksi Weld Size, in.				70-ksi Weld Size, in.							
	1/2		5/8		5/16		3/8		1/2		5/8	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
11	77.4	116	96.7	145	43.7	65.6	52.5	78.7	69.9	105	87.4	131
12	90.8	136	113	170	51.4	77.1	61.7	92.5	82.2	123	103	154
13	105	157	131	197	59.6	89.3	71.5	107	95.3	143	119	179
14	120	180	150	224	68.2	102	81.8	123	109	164	136	204
15	135	203	169	253	77.2	116	92.6	139	123	185	154	232
16	151	227	189	283	86.5	130	104	156	138	208	173	260
17	168	251	209	314	96.2	144	115	173	154	231	192	289
18	184	277	231	346	106	159	127	191	170	255	212	319
19	202	303	252	378	117	175	140	210	186	280	233	350
20	219	329	274	411	127	191	152	229	203	305	254	381
21	237	356	297	445	138	207	165	248	220	331	276	413
22	256	383	319	479	149	223	178	268	238	357	297	446
23	274	411	342	514	160	240	192	288	256	384	320	480
24	292	439	366	548	171	257	205	308	274	411	342	513
25	311	467	389	584	183	274	219	329	292	438	365	548
26	330	495	413	619	194	291	233	349	310	466	388	582
27	349	524	436	655	206	308	247	370	329	494	411	617
28	368	552	460	690	217	326	261	391	348	522	435	652
29	387	581	484	726	229	344	275	412	367	550	458	687
30	407	610	508	762	241	362	289	434	386	578	482	723
31	426	639	532	799	253	379	304	455	405	607	506	759
32	445	668	557	835	265	397	318	477	424	636	530	795

Limitations for Connections to Column Webs	
B **= 3 1/2 in. max**	***B*** **= 3 1/2 in. max**
See item 3 in preceding discussion "Design Checks"	See item 3 in preceeding discussion "Design Checks"

Notes:

1. Values shown assume 70-ksi electrodes. For 60-ksi electrodes, multiply tabular values by 0.857, or enter table with 1.17 times the required strength, R_u or R_a. For 80-ksi electrodes, multiply tabular values by 1.14, or enter table with 0.875 times the required strength.
2. Tabulated values are valid for stiffeners with minimum thickness of

$$t_{min} = \left(\frac{F_{y,\,beam}}{F_{y,\,stiffener}}\right) t_w$$

but not less than $2w$ for stiffeners with $F_y = 36$ ksi nor $1.5w$ for stiffeners with $F_y = 50$ ksi. In the above, t_w is the thickness of the unstiffened supported beam web and w is the nominal weld size.

3. Tabulated values may be limited by shear yielding of the stiffener, or bearing on the stiffener; refer to AISC *Specification* Sections J4.2 and J7, respectively.

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

SINGLE-PLATE CONNECTIONS

A single-plate connection is made with a plate, as illustrated in Figures 10-11 and 10-12. The plate must be welded to the support on both sides of the plate and bolted to the supported member.

Design Checks

The available strength of a single-plate connection is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a, respectively.

Single-plate shear connections that satisfy the corresponding dimensional limitations can be designed using the simplified design procedure for the "conventional" configuration. Other single-plate shear connections can be designed using the procedure for the "extended" configuration, which is applicable to any configuration of single-plate shear connections, regardless of connection geometry.

Both the conventional and extended configurations permit the use of Group A or Group B bolts. The procedure is valid for bolts that are snug-tightened, pretensioned or slip-critical. In both the conventional and extended configuration, the design recommendations are equally applicable to plate and beam web material with F_y = 36 ksi or 50 ksi. In both cases, the weld between the single plate and the support should be sized as $(5/8)t_p$, which will develop the strength of either a 36-ksi or 50-ksi plate.

Conventional Configuration

The following method may be used when the dimensional and other limitations upon which it is based are satisfied. See Muir and Thornton (2011).

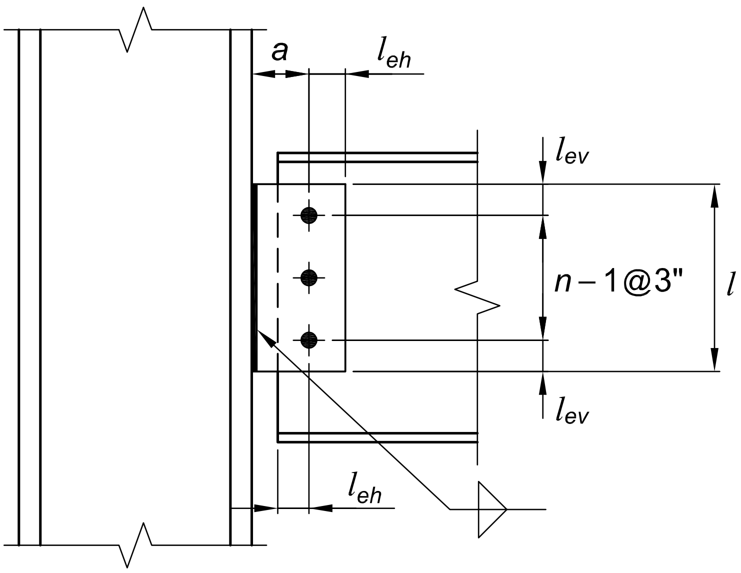


Fig. 10-11. Single-plate connection—Conventional Configuration.

Table 10-9
Design Values for Conventional Single-Plate Shear Connections

n	Hole Type	*e*, in.	Maximum t_p or t_w, in.
2 to 5	SSLT	$a/2$	None
	STD	$a/2$	$d/2 + 1/16$
6 to 12	SSLT	$a/2$	$d/2 + 1/16$
	STD	a	$d/2 - 1/16$

Dimensional Limitations

1. Only a single vertical row of bolts is permitted. The number of bolts in the connection, n, must be between 2 and 12.
2. The distance from the bolt line to the weld line, a, must be equal to or less than $3^1/_2$ in.
3. Standard holes (STD) or short-slotted holes transverse to the direction of the supported member reaction (SSLT) are permitted to be used as noted in Table 10-9.
4. The vertical edge distance, l_{ev}, must satisfy AISC *Specification* Table J3.4 requirements. The horizontal edge distance, l_{eh}, should be greater than or equal to $2d$ for both the plate and the beam web, where d is the bolt diameter.
5. Either the plate thickness, t_p, or the beam web thickness, t_w, must satisfy the maximum thickness requirement given in Table 10-9.

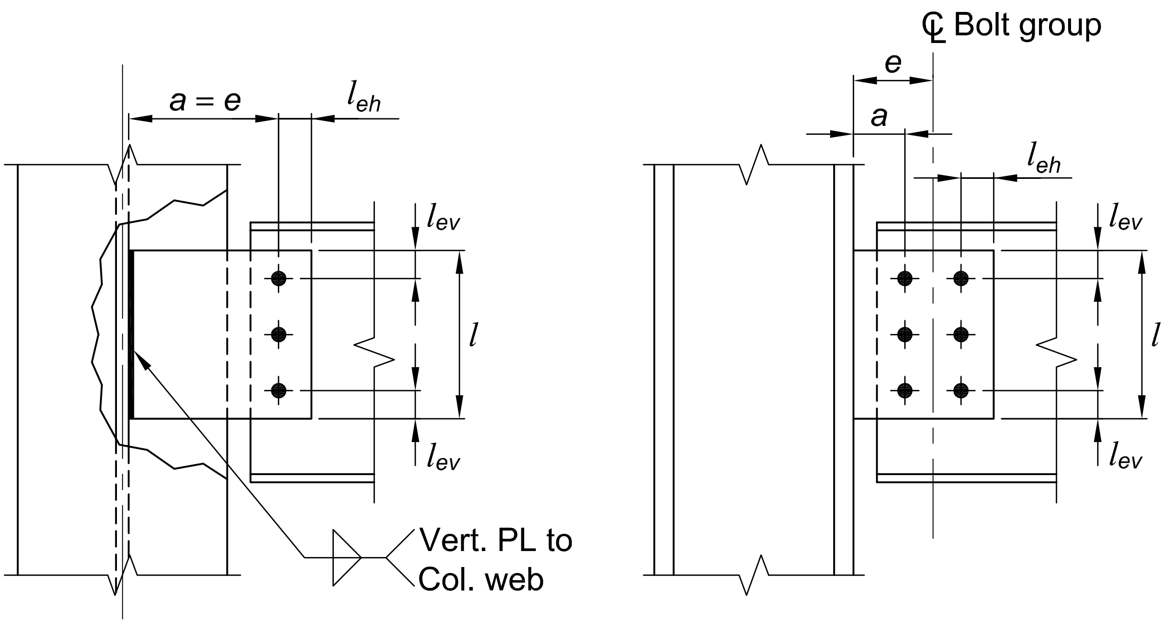


Fig. 10-12. Single-plate connection—Extended Configuration.

Design Checks

1. Bolt shear is checked in accordance with AISC *Specification* Section J3.6 assuming the eccentricity, e, shown in Table 10-9 and the effective number of bolts from Table 7-6.
2. Plate bearing and tearout are checked in accordance with AISC *Specification* Section J3.10 assuming the reaction is applied concentrically.
3. Plate buckling will not control for the conventional configuration.

Extended Configuration

The following method can be used when the dimensional and other limitations of the conventional method are not satisfied. This procedure can be used to determine the strength of single-plate shear connections with multiple vertical rows or in the extended configuration, as shown in Figure 10-12.

Dimensional Limitations

1. The number of bolts, n, is not limited.
2. The distance from the weld line to the bolt line closest to the support, a, is not limited.
3. The use of holes must satisfy AISC *Specification* Section J3.2 requirements.
4. The horizontal and vertical edge distances, l_{eh} and l_{ev}, must satisfy AISC *Specification* Table J3.4 requirements.

Design Checks

1. Determine the bolt group required for bolt shear, bearing and tearout, with eccentricity e, where e is defined as the distance from the support to the centroid of the bolt group. Exception: Alternative considerations of the design eccentricity are acceptable when justified by rational analysis. For example, see Sherman and Ghorbanpoor (2002).
2. Determine the maximum plate thickness permitted such that the plate moment strength does not exceed the moment strength of the bolt group in shear, as follows:

$$t_{max} = \frac{6M_{max}}{F_y l^2} \qquad (10\text{-}3)$$

where

F_y = specified minimum yield stress of plate, ksi

$$M_{max} = \frac{F_{nv}}{0.90}\left(A_b C'\right) \qquad (10\text{-}4)$$

$\dfrac{F_{nv}}{0.90}$ = shear strength of an individual bolt from AISC *Specification* Table J3.2, ksi, divided by a factor of 0.90 to remove the 10% reduction for uneven force distribution in end-loaded bolt groups (Kulak, 2002). The joint in question is not end-loaded.

A_b = area of an individual bolt, in.^{2}

C' = coefficient from Part 7 for the moment-only case (instantaneous center of rotation at the centroid of the bolt group)

l = depth of plate, in.

The foregoing check is made at the nominal strength level, since the check is to ensure ductility, not strength.

Exceptions:

a. For a single vertical row of bolts only, the foregoing criterion need not be satisfied if either the beam web or the plate satisfies the thickness requirements of Table 10-9 and both satisfy $l_{eh} \geq 2d_b$.

b. For a double vertical row of bolts only, the foregoing criterion need not be satisfied if both the beam web and the plate satisfy the thickness requirements of Table 10-9 and $l_{eh} \geq 2d_b$.

3. Check the plate for the limit states of shear yielding, shear rupture, block shear rupture, and flexural rupture. Check the beam web for the same limit states, as applicable.
4. Check the plate for the limit states of shear yielding, shear buckling, and yielding due to flexure as follows:

$$\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{M_r}{M_c}\right)^2 \leq 1.0 \tag{10-5}$$

where

$M_c = \phi_b M_n$ (LRFD) or M_n/Ω_b (ASD), kip-in.
$M_n = F_y Z_{pl}$, kip-in.
$M_r = M_u$ (LRFD) or M_a (ASD)
$\quad = V_r a$, kip-in.
$V_c = \phi_v V_n$ (LRFD) or V_n/Ω_v (ASD), kips
$V_n = 0.6F_y A_g$, kips
A_g = gross cross-sectional area of the shear plate, in.^{2}
$V_r = V_u$ (LRFD) or V_a (ASD), kips
Z_{pl} = plastic section modulus of the shear plate, in.^{3}
a = distance from the support to the first line of bolts, in.
$\phi_b = 0.90$
$\phi_v = 1.00$
$\Omega_b = 1.67$
$\Omega_v = 1.50$

5. Check the plate for the limit state of buckling using the double-coped beam procedure given in Part 9. This check assumes that beam is supported near the end of the plate as indicated in Step 6. For other conditions, see Thornton and Fortney (2011).
6. Ensure that the supported beam is braced at points of support.

The design procedure for extended single-plate shear connections permits the column to be designed for an axial force without eccentricity. In some cases, economy may be gained by considering alternative design procedures that allow the transfer of some moment into the column. A percentage of the column's weak-axis flexural strength, such as 5%, may be used as a mechanism to reduce the required eccentricity on the bolt group, provided that this moment is also considered in the design of the column. Larger percentages of the column's weak-axis flexural strength may be justified at the roof level.

Short-slotted holes can be used with the extended configuration with the bolts designed as bearing. Any slip of the bolts is a serviceability issue and does not affect the connection strength (Muir and Hewitt, 2009).

Shop and Field Practices

Conventional and extended single-plate connections may be made to the webs of supporting girders and to the flanges of supporting columns. Extended single-plate connections are suitable for connections to the webs of supporting columns when the bolt line is located a sufficient distance beyond the column flanges.

With the plate shop-attached to the support, side erection of the beam is permitted. Play in the open holes usually compensates for mill variation in column flange supports and other field adjustments.

DESIGN TABLE DISCUSSION (TABLE 10-10)

Table 10-10. Single-Plate Connections

Table 10-10 is a design aid for single-plate connections welded to the support and bolted to the supported beam. Available strengths are tabulated in Table 10-10a for plate material with $F_y = 36$ ksi and Table 10-10b for plate material with $F_y = 50$ ksi.

Tabulated bolt and plate available strengths consider the limit states of bolt shear, bolt bearing and tearout on the plate, shear yielding of the plate, shear rupture of the plate, block shear rupture of the plate, and weld shear. Values are tabulated for two through twelve rows of $^3/_4$-in.-, $^7/_8$-in.-, 1-in.- and $1^1/_8$-in.-diameter Group A and Group B bolts at 3-in. spacing. For calculation purposes, plate edge distance, l_{ev}, is consistent with conventional field practices and exceeds the requirements given in AISC *Specification* Section J3.10 and Table J3.4. Edge distance, l_{eh}, is provided as 2 times the diameter of the bolt, to match tested connections. Weld sizes are tabulated equal to $(^5/_8)t_p$.

While the tabular values are based on $a = 3$ in., they may conservatively be used when the distance from the support to the bolt line, a, is between $2^1/_2$ in. and 3 in. The tabulated values are valid for laterally supported beams in steel and composite construction, all types of loading, snug-tightened or pretensioned bolts, and for supported and supporting members of all grades of steel.

Table 10-10a
Single-Plate Connections
Bolt, Weld and Single-Plate Available Strengths, kips

F_y = 36 ksi Plate

$^3/_4$-in. Bolts

n	Bolt Group	Thread Cond.	Hole Type	Plate Thickness, in.											
				$^1/_4$		$^5/_{16}$		$^3/_8$		$^7/_{16}$		$^1/_2$		$^9/_{16}$	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
12 (l = 35$^1/_2$)	Group A	N	STD	100	150	125	188	–	–	–	–	–	–	–	–
			SSLT	99.5	149	124	187	138	208	138	208	–	–	–	–
		X	STD	100	150	125	188	–	–	–	–	–	–	–	–
			SSLT	99.5	149	124	187	149	224	174	261	–	–	–	–
	Group B	N	STD	100	150	125	188	–	–	–	–	–	–	–	–
			SSLT	99.5	149	124	187	149	224	174	261	–	–	–	–
		X	STD	100	150	125	188	–	–	–	–	–	–	–	–
			SSLT	99.5	149	124	187	149	224	174	261	–	–	–	–
11 (l = 32$^1/_2$)	Group A	N	STD	92.1	138	115	173	–	–	–	–	–	–	–	–
			SSLT	91.4	137	114	171	126	190	126	190	–	–	–	–
		X	STD	92.1	138	115	173	–	–	–	–	–	–	–	–
			SSLT	91.4	137	114	171	137	206	159	239	–	–	–	–
	Group B	N	STD	92.1	138	115	173	–	–	–	–	–	–	–	–
			SSLT	91.4	137	114	171	137	206	159	239	–	–	–	–
		X	STD	92.1	138	115	173	–	–	–	–	–	–	–	–
			SSLT	91.4	137	114	171	137	206	160	240	–	–	–	–
10 (l = 29$^1/_2$)	Group A	N	STD	84.0	126	105	157	–	–	–	–	–	–	–	–
			SSLT	83.3	125	104	156	115	172	115	172	–	–	–	–
		X	STD	84.0	126	105	157	–	–	–	–	–	–	–	–
			SSLT	83.3	125	104	156	125	187	145	217	–	–	–	–
	Group B	N	STD	84.0	126	105	157	–	–	–	–	–	–	–	–
			SSLT	83.3	125	104	156	125	187	145	217	–	–	–	–
		X	STD	84.0	126	105	157	–	–	–	–	–	–	–	–
			SSLT	83.3	125	104	156	125	187	146	219	–	–	–	–
9 (l = 26$^1/_2$)	Group A	N	STD	75.9	114	94.8	142	–	–	–	–	–	–	–	–
			SSLT	75.2	113	94.0	141	103	155	103	155	–	–	–	–
		X	STD	75.9	114	94.8	142	–	–	–	–	–	–	–	–
			SSLT	75.2	113	94.0	141	113	169	130	194	–	–	–	–
	Group B	N	STD	75.9	114	94.8	142	–	–	–	–	–	–	–	–
			SSLT	75.2	113	94.0	141	113	169	130	194	–	–	–	–
		X	STD	75.9	114	94.8	142	–	–	–	–	–	–	–	–
			SSLT	75.2	113	94.0	141	113	169	132	197	–	–	–	–
Weld Size, in.				$^3/_{16}$		$^1/_4$		$^1/_4$		$^5/_{16}$		$^5/_{16}$		$^3/_8$	

STD = Standard holes
SSLT = Short-slotted holes transverse to direction of load
– Indicates that the plate thickness is greater than the maximum given in Table 10-9.

N = Threads included
X = Threads excluded

Table 10-10a (continued)
Single-Plate Connections
Bolt, Weld and Single-Plate Available Strengths, kips

F_y = 36 ksi Plate

3/4-in. Bolts

n	Bolt Group	Thread Cond.	Hole Type	Plate Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	7/16 ASD	7/16 LRFD	1/2 ASD	1/2 LRFD	9/16 ASD	9/16 LRFD
8 (l = 23 1/2)	Group A	N	STD	67.8	102	84.7	127	–	–	–	–	–	–	–	–
			SSLT	67.1	101	83.9	126	90.9	137	90.9	137	–	–	–	–
		X	STD	67.8	102	84.7	127	–	–	–	–	–	–	–	–
			SSLT	67.1	101	83.9	126	101	151	115	172	–	–	–	–
	Group B	N	STD	67.8	102	84.7	127	–	–	–	–	–	–	–	–
			SSLT	67.1	101	83.9	126	101	151	115	172	–	–	–	–
		X	STD	67.8	102	84.7	127	–	–	–	–	–	–	–	–
			SSLT	67.1	101	83.9	126	101	151	117	176	–	–	–	–
7 (l = 20 1/2)	Group A	N	STD	59.7	89.5	72.1	108	–	–	–	–	–	–	–	–
			SSLT	59.0	88.5	73.7	111	78.8	119	78.8	119	–	–	–	–
		X	STD	59.7	89.5	74.6	112	–	–	–	–	–	–	–	–
			SSLT	59.0	88.5	73.7	111	88.5	133	99.4	149	–	–	–	–
	Group B	N	STD	59.7	89.5	74.6	112	–	–	–	–	–	–	–	–
			SSLT	59.0	88.5	73.7	111	88.5	133	99.4	149	–	–	–	–
		X	STD	59.7	89.5	74.6	112	–	–	–	–	–	–	–	–
			SSLT	59.0	88.5	73.7	111	88.5	133	103	155	–	–	–	–
6 (l = 17 1/2)	Group A	N	STD	51.6	77.4	59.3	89.1	–	–	–	–	–	–	–	–
			SSLT	50.9	76.3	63.6	95.4	66.8	100	66.8	100	–	–	–	–
		X	STD	51.6	77.4	64.5	96.7	–	–	–	–	–	–	–	–
			SSLT	50.9	76.3	63.6	95.4	76.3	115	84.2	126	–	–	–	–
	Group B	N	STD	51.6	77.4	64.5	96.7	–	–	–	–	–	–	–	–
			SSLT	50.9	76.3	63.6	95.4	76.3	115	84.2	126	–	–	–	–
		X	STD	51.6	77.4	64.5	96.7	–	–	–	–	–	–	–	–
			SSLT	50.9	76.3	63.6	95.4	76.3	115	89.1	134	–	–	–	–
5 (l = 14 1/2)	Group A	N	STD	43.5	65.2	54.3	81.5	54.5	82.0	54.5	82.0	–	–	–	–
			SSLT	42.8	64.2	53.5	80.2	54.5	82.0	54.5	82.0	54.5	82.0	54.5	82.0
		X	STD	43.5	65.2	54.3	81.5	65.2	97.8	68.7	103	–	–	–	–
			SSLT	42.8	64.2	53.5	80.2	64.2	96.3	68.7	103	68.7	103	68.7	103
	Group B	N	STD	43.5	65.2	54.3	81.5	65.2	97.8	68.7	103	–	–	–	–
			SSLT	42.8	64.2	53.5	80.2	64.2	96.3	68.7	103	68.7	103	68.7	103
		X	STD	43.5	65.2	54.3	81.5	65.2	97.8	76.1	114	–	–	–	–
			SSLT	42.8	64.2	53.5	80.2	64.2	96.3	74.9	112	85.2	127	85.2	127
Weld Size, in.				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard holes
SSLT = Short-slotted holes transverse to direction of load
– Indicates that the plate thickness is greater than the maximum given in Table 10-9.

N = Threads included
X = Threads excluded

Table 10-10a (continued)

F_y = 36 ksi Plate

Single-Plate Connections

Bolt, Weld and Single-Plate Available Strengths, kips

$^3/_4$-in. Bolts

n	Bolt Group	Thread Cond.	Hole Type	Plate Thickness, in.											
				$^1/_4$		$^5/_{16}$		$^3/_8$		$^7/_{16}$		$^1/_2$		$^9/_{16}$	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
4 ($l = 11^1/_2$)	Group A	N	STD	34.8	52.2	42.1	63.3	42.1	63.3	42.1	63.3	–	–	–	–
			SSLT	34.7	52.0	42.1	63.3	42.1	63.3	42.1	63.3	42.1	63.3	42.1	63.3
		X	STD	34.8	52.2	43.5	65.3	52.2	78.3	53.0	79.5	–	–	–	–
			SSLT	34.7	52.0	43.4	65.1	52.0	78.1	53.0	79.5	53.0	79.5	53.0	79.5
	Group B	N	STD	34.8	52.2	43.5	65.3	52.2	78.3	53.0	79.5	–	–	–	–
			SSLT	34.7	52.0	43.4	65.1	52.0	78.1	53.0	79.5	53.0	79.5	53.0	79.5
		X	STD	34.8	52.2	43.5	65.3	52.2	78.3	60.9	91.4	–	–	–	–
			SSLT	34.7	52.0	43.4	65.1	52.0	78.1	60.7	91.1	65.8	98.3	65.8	98.3
3 ($l = 8^1/_2$)	Group A	N	STD	25.6	38.3	29.4	44.2	29.4	44.2	29.4	44.2	–	–	–	–
			SSLT	25.6	38.3	29.4	44.2	29.4	44.2	29.4	44.2	29.4	44.2	29.4	44.2
		X	STD	25.6	38.3	31.9	47.9	37.1	55.6	37.1	55.6	–	–	–	–
			SSLT	25.6	38.3	31.9	47.9	37.1	55.6	37.1	55.6	37.1	55.6	37.1	55.6
	Group B	N	STD	25.6	38.3	31.9	47.9	37.1	55.6	37.1	55.6	–	–	–	–
			SSLT	25.6	38.3	31.9	47.9	37.1	55.6	37.1	55.6	37.1	55.6	37.1	55.6
		X	STD	25.6	38.3	31.9	47.9	38.3	57.5	44.7	67.1	–	–	–	–
			SSLT	25.6	38.3	31.9	47.9	38.3	57.5	44.7	67.1	45.9	68.7	45.9	68.7
2 ($l = 5^1/_2$)	Group A	N	STD	16.3	24.5	16.7	25.1	16.7	25.1	16.7	25.1	–	–	–	–
			SSLT	16.3	24.5	16.7	25.1	16.7	25.1	16.7	25.1	16.7	25.1	16.7	25.1
		X	STD	16.3	24.5	20.4	30.6	21.1	31.6	21.1	31.6	–	–	–	–
			SSLT	16.3	24.5	20.4	30.6	21.1	31.6	21.1	31.6	21.1	31.6	21.1	31.6
	Group B	N	STD	16.3	24.5	20.4	30.6	21.1	31.6	21.1	31.6	–	–	–	–
			SSLT	16.3	24.5	20.4	30.6	21.1	31.6	21.1	31.6	21.1	31.6	21.1	31.6
		X	STD	16.3	24.5	20.4	30.6	24.5	36.7	26.1	39.1	–	–	–	–
			SSLT	16.3	24.5	20.4	30.6	24.5	36.7	26.1	39.1	26.1	39.1	26.1	39.1
Weld Size, in.				$^3/_{16}$		$^1/_4$		$^1/_4$		$^5/_{16}$		$^5/_{16}$		$^3/_8$	

STD = Standard holes
SSLT = Short-slotted holes transverse to direction of load
– Indicates that the plate thickness is greater than the maximum given in Table 10-9.

N = Threads included
X = Threads excluded

Table 10-10a (continued)
Single-Plate Connections
Bolt, Weld and Single-Plate Available Strengths, kips

F_y = 36 ksi Plate

7/8-in. Bolts

n	Bolt Group	Thread Cond.	Hole Type	Plate Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	7/16 ASD	7/16 LRFD	1/2 ASD	1/2 LRFD	9/16 ASD	9/16 LRFD
12 (l = 36)	Group A	N	STD	102	153	128	192	153	230	–	–	–	–	–	–
			SSLT	102	152	127	190	152	228	178	267	188	282	–	–
		X	STD	102	153	128	192	153	230	–	–	–	–	–	–
			SSLT	102	152	127	190	152	228	178	267	203	305	–	–
	Group B	N	STD	102	153	128	192	153	230	–	–	–	–	–	–
			SSLT	102	152	127	190	152	228	178	267	203	305	–	–
		X	STD	102	153	128	192	153	230	–	–	–	–	–	–
			SSLT	102	152	127	190	152	228	178	267	203	305	–	–
11 (l = 33)	Group A	N	STD	94.1	141	118	176	141	212	–	–	–	–	–	–
			SSLT	93.4	140	117	175	140	210	164	245	172	258	–	–
		X	STD	94.1	141	118	176	141	212	–	–	–	–	–	–
			SSLT	93.4	140	117	175	140	210	164	245	187	280	–	–
	Group B	N	STD	94.1	141	118	176	141	212	–	–	–	–	–	–
			SSLT	93.4	140	117	175	140	210	164	245	187	280	–	–
		X	STD	94.1	141	118	176	141	212	–	–	–	–	–	–
			SSLT	93.4	140	117	175	140	210	164	245	187	280	–	–
10 (l = 30)	Group A	N	STD	86.0	129	108	161	129	194	–	–	–	–	–	–
			SSLT	85.3	128	107	160	128	192	149	224	156	234	–	–
		X	STD	86.0	129	108	161	129	194	–	–	–	–	–	–
			SSLT	85.3	128	107	160	128	192	149	224	171	256	–	–
	Group B	N	STD	86.0	129	108	161	129	194	–	–	–	–	–	–
			SSLT	85.3	128	107	160	128	192	149	224	171	256	–	–
		X	STD	86.0	129	108	161	129	194	–	–	–	–	–	–
			SSLT	85.3	128	107	160	128	192	149	224	171	256	–	–
9 (l = 27)	Group A	N	STD	77.9	117	97.4	146	117	175	–	–	–	–	–	–
			SSLT	77.2	116	96.5	145	116	174	135	203	140	210	–	–
		X	STD	77.9	117	97.4	146	117	175	–	–	–	–	–	–
			SSLT	77.2	116	96.5	145	116	174	135	203	154	232	–	–
	Group B	N	STD	77.9	117	97.4	146	117	175	–	–	–	–	–	–
			SSLT	77.2	116	96.5	145	116	174	135	203	154	232	–	–
		X	STD	77.9	117	97.4	146	117	175	–	–	–	–	–	–
			SSLT	77.2	116	96.5	145	116	174	135	203	154	232	–	–
Weld Size, in.				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard holes
SSLT = Short-slotted holes transverse to direction of load
– Indicates that the plate thickness is greater than the maximum given in Table 10-9.

N = Threads included
X = Threads excluded

Table 10-10a (continued)
Single-Plate Connections
Bolt, Weld and Single-Plate Available Strengths, kips

F_y = 36 ksi Plate

7/8-in. Bolts

n	Bolt Group	Thread Cond.	Hole Type	Plate Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	7/16 ASD	7/16 LRFD	1/2 ASD	1/2 LRFD	9/16 ASD	9/16 LRFD
8 (l = 24)	Group A	N	STD	69.6	104	87.0	131	104	157	–	–	–	–	–	–
			SSLT	69.1	104	86.4	130	104	156	121	181	124	186	–	–
		X	STD	69.6	104	87.0	131	104	157	–	–	–	–	–	–
			SSLT	69.1	104	86.4	130	104	156	121	181	138	207	–	–
	Group B	N	STD	69.6	104	87.0	131	104	157	–	–	–	–	–	–
			SSLT	69.1	104	86.4	130	104	156	121	181	138	207	–	–
		X	STD	69.6	104	87.0	131	104	157	–	–	–	–	–	–
			SSLT	69.1	104	86.4	130	104	156	121	181	138	207	–	–
7 (l = 21)	Group A	N	STD	60.9	91.4	76.1	114	91.4	137	–	–	–	–	–	–
			SSLT	60.9	91.4	76.1	114	91.4	137	107	160	107	161	–	–
		X	STD	60.9	91.4	76.1	114	91.4	137	–	–	–	–	–	–
			SSLT	60.9	91.4	76.1	114	91.4	137	107	160	122	183	–	–
	Group B	N	STD	60.9	91.4	76.1	114	91.4	137	–	–	–	–	–	–
			SSLT	60.9	91.4	76.1	114	91.4	137	107	160	122	183	–	–
		X	STD	60.9	91.4	76.1	114	91.4	137	–	–	–	–	–	–
			SSLT	60.9	91.4	76.1	114	91.4	137	107	160	122	183	–	–
6 (l = 18)	Group A	N	STD	52.2	78.3	65.3	97.9	78.3	117	–	–	–	–	–	–
			SSLT	52.2	78.3	65.3	97.9	78.3	117	90.9	136	90.9	136	–	–
		X	STD	52.2	78.3	65.3	97.9	78.3	117	–	–	–	–	–	–
			SSLT	52.2	78.3	65.3	97.9	78.3	117	91.4	137	104	157	–	–
	Group B	N	STD	52.2	78.3	65.3	97.9	78.3	117	–	–	–	–	–	–
			SSLT	52.2	78.3	65.3	97.9	78.3	117	91.4	137	104	157	–	–
		X	STD	52.2	78.3	65.3	97.9	78.3	117	–	–	–	–	–	–
			SSLT	52.2	78.3	65.3	97.9	78.3	117	91.4	137	104	157	–	–
5 (l = 15)	Group A	N	STD	43.5	65.3	54.4	81.6	65.3	97.9	74.2	111	74.2	111	–	–
			SSLT	43.5	65.3	54.4	81.6	65.3	97.9	74.2	111	74.2	111	74.2	111
		X	STD	43.5	65.3	54.4	81.6	65.3	97.9	76.1	114	87.0	131	–	–
			SSLT	43.5	65.3	54.4	81.6	65.3	97.9	76.1	114	87.0	131	93.4	141
	Group B	N	STD	43.5	65.3	54.4	81.6	65.3	97.9	76.1	114	87.0	131	–	–
			SSLT	43.5	65.3	54.4	81.6	65.3	97.9	76.1	114	87.0	131	93.4	141
		X	STD	43.5	65.3	54.4	81.6	65.3	97.9	76.1	114	87.0	131	–	–
			SSLT	43.5	65.3	54.4	81.6	65.3	97.9	76.1	114	87.0	131	97.9	147
Weld Size, in.				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard holes
SSLT = Short-slotted holes transverse to direction of load
– Indicates that the plate thickness is greater than the maximum given in Table 10-9.

N = Threads included
X = hreads excluded

Table 10-10a (continued)
Single-Plate Connections
Bolt, Weld and Single-Plate Available Strengths, kips

F_y = 36 ksi Plate

7/8-in. Bolts

n	Bolt Group	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
4 (l = 12)	Group A	N	STD	34.8	52.2	43.5	65.3	52.2	78.3	57.3	85.9	57.3	85.9	–	–
			SSLT	34.8	52.2	43.5	65.3	52.2	78.3	57.3	85.9	57.3	85.9	57.3	85.9
		X	STD	34.8	52.2	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	–	–
			SSLT	34.8	52.2	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	72.1	109
	Group B	N	STD	34.8	52.2	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	–	–
			SSLT	34.8	52.2	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	72.1	109
		X	STD	34.8	52.2	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	–	–
			SSLT	34.8	52.2	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	78.3	117
3 (l = 9)	Group A	N	STD	26.1	39.2	32.6	48.9	39.2	58.7	40.0	60.0	40.0	60.0	–	–
			SSLT	26.1	39.2	32.6	48.9	39.2	58.7	40.0	60.0	40.0	60.0	40.0	60.0
		X	STD	26.1	39.2	32.6	48.9	39.2	58.7	45.7	68.5	50.4	75.8	–	–
			SSLT	26.1	39.2	32.6	48.9	39.2	58.7	45.7	68.5	50.4	75.8	50.4	75.8
	Group B	N	STD	26.1	39.2	32.6	48.9	39.2	58.7	45.7	68.5	50.4	75.8	–	–
			SSLT	26.1	39.2	32.6	48.9	39.2	58.7	45.7	68.5	50.4	75.8	50.4	75.8
		X	STD	26.1	39.2	32.6	48.9	39.2	58.7	45.7	68.5	52.2	78.3	–	–
			SSLT	26.1	39.2	32.6	48.9	39.2	58.7	45.7	68.5	52.2	78.3	58.7	88.1
2 (l = 6)	Group A	N	STD	17.4	26.1	21.8	32.6	22.8	34.1	22.8	34.1	22.8	34.1	–	–
			SSLT	17.4	26.1	21.8	32.6	22.8	34.1	22.8	34.1	22.8	34.1	22.8	34.1
		X	STD	17.4	26.1	21.8	32.6	26.1	39.2	28.7	43.1	28.7	43.1	–	–
			SSLT	17.4	26.1	21.8	32.6	26.1	39.2	28.7	43.1	28.7	43.1	28.7	43.1
	Group B	N	STD	17.4	26.1	21.8	32.6	26.1	39.2	28.7	43.1	28.7	43.1	–	–
			SSLT	17.4	26.1	21.8	32.6	26.1	39.2	28.7	43.1	28.7	43.1	28.7	43.1
		X	STD	17.4	26.1	21.8	32.6	26.1	39.2	30.5	45.7	34.8	52.2	–	–
			SSLT	17.4	26.1	21.8	32.6	26.1	39.2	30.5	45.7	34.8	52.2	35.4	53.2
Weld Size, in.				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard holes
SSLT = Short-slotted holes transverse to direction of load
– Indicates that the plate thickness is greater than the maximum given in Table 10-9.

N = Threads included
X = Threads excluded

Table 10-10a (continued)
Single-Plate Connections
Bolt, Weld and Single-Plate Available Strengths, kips

F_y = 36 ksi Plate

1-in. Bolts

n	Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	7/16 ASD	7/16 LRFD	1/2 ASD	1/2 LRFD	9/16 ASD	9/16 LRFD
12 (l = 36 1/2)	Group A	N	STD	96.8	145	121	181	145	218	169	254	–	–	–	–
			SSLT	96.8	145	121	181	145	218	169	254	194	290	218	327
		X	STD	96.8	145	121	181	145	218	169	254	–	–	–	–
			SSLT	96.8	145	121	181	145	218	169	254	194	290	218	327
	Group B	N	STD	96.8	145	121	181	145	218	169	254	–	–	–	–
			SSLT	96.8	145	121	181	145	218	169	254	194	290	218	327
		X	STD	96.8	145	121	181	145	218	169	254	–	–	–	–
			SSLT	96.8	145	121	181	145	218	169	254	194	290	218	327
11 (l = 33 1/2)	Group A	N	STD	88.9	133	111	167	133	200	156	233	–	–	–	–
			SSLT	88.9	133	111	167	133	200	156	233	178	267	200	300
		X	STD	88.9	133	111	167	133	200	156	233	–	–	–	–
			SSLT	88.9	133	111	167	133	200	156	233	178	267	200	300
	Group B	N	STD	88.9	133	111	167	133	200	156	233	–	–	–	–
			SSLT	88.9	133	111	167	133	200	156	233	178	267	200	300
		X	STD	88.9	133	111	167	133	200	156	233	–	–	–	–
			SSLT	88.9	133	111	167	133	200	156	233	178	267	200	300
10 (l = 30 1/2)	Group A	N	STD	81.0	122	101	152	122	182	142	213	–	–	–	–
			SSLT	81.0	122	101	152	122	182	142	213	162	243	182	273
		X	STD	81.0	122	101	152	122	182	142	213	–	–	–	–
			SSLT	81.0	122	101	152	122	182	142	213	162	243	182	273
	Group B	N	STD	81.0	122	101	152	122	182	142	213	–	–	–	–
			SSLT	81.0	122	101	152	122	182	142	213	162	243	182	273
		X	STD	81.0	122	101	152	122	182	142	213	–	–	–	–
			SSLT	81.0	122	101	152	122	182	142	213	162	243	182	273
9 (l = 27 1/2)	Group A	N	STD	73.1	110	91.4	137	110	165	128	192	–	–	–	–
			SSLT	73.1	110	91.4	137	110	165	128	192	146	219	165	247
		X	STD	73.1	110	91.4	137	110	165	128	192	–	–	–	–
			SSLT	73.1	110	91.4	137	110	165	128	192	146	219	165	247
	Group B	N	STD	73.1	110	91.4	137	110	165	128	192	–	–	–	–
			SSLT	73.1	110	91.4	137	110	165	128	192	146	219	165	247
		X	STD	73.1	110	91.4	137	110	165	128	192	–	–	–	–
			SSLT	73.1	110	91.4	137	110	165	128	192	146	219	165	247
Weld Size, in.				3/16		1/4		1/4		5/16		5/16		3/8	

(Plate Thickness, in. heads the column groups 1/4 through 9/16.)

STD = Standard holes
SSLT = Short-slotted holes transverse to direction of load
– Indicates that the plate thickness is greater than the maximum given in Table 10-9.
N = Threads included
X = Threads excluded

Table 10-10a (continued)
Single-Plate Connections
Bolt, Weld and Single-Plate Available Strengths, kips

F_y = 36 ksi Plate

1-in. Bolts

n	Bolt Group	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
8 (l = 24 1/2)	Group A	N	STD	65.3	97.9	81.6	122	97.9	147	114	171	–	–	–	–
			SSLT	65.3	97.9	81.6	122	97.9	147	114	171	131	196	147	220
		X	STD	65.3	97.9	81.6	122	97.9	147	114	171	–	–	–	–
			SSLT	65.3	97.9	81.6	122	97.9	147	114	171	131	196	147	220
	Group B	N	STD	65.3	97.9	81.6	122	97.9	147	114	171	–	–	–	–
			SSLT	65.3	97.9	81.6	122	97.9	147	114	171	131	196	147	220
		X	STD	65.3	97.9	81.6	122	97.9	147	114	171	–	–	–	–
			SSLT	65.3	97.9	81.6	122	97.9	147	114	171	131	196	147	220
7 (l = 21 1/2)	Group A	N	STD	57.4	86.0	71.7	108	86.0	129	100	151	–	–	–	–
			SSLT	57.4	86.0	71.7	108	86.0	129	100	151	115	172	129	194
		X	STD	57.4	86.0	71.7	108	86.0	129	100	151	–	–	–	–
			SSLT	57.4	86.0	71.7	108	86.0	129	100	151	115	172	129	194
	Group B	N	STD	57.4	86.0	71.7	108	86.0	129	100	151	–	–	–	–
			SSLT	57.4	86.0	71.7	108	86.0	129	100	151	115	172	129	194
		X	STD	57.4	86.0	71.7	108	86.0	129	100	151	–	–	–	–
			SSLT	57.4	86.0	71.7	108	86.0	129	100	151	115	172	129	194
6 (l = 18 1/2)	Group A	N	STD	49.5	74.2	61.9	92.8	74.2	111	86.6	130	–	–	–	–
			SSLT	49.5	74.2	61.9	92.8	74.2	111	86.6	130	99.0	148	111	167
		X	STD	49.5	74.2	61.9	92.8	74.2	111	86.6	130	–	–	–	–
			SSLT	49.5	74.2	61.9	92.8	74.2	111	86.6	130	99.0	148	111	167
	Group B	N	STD	49.5	74.2	61.9	92.8	74.2	111	86.6	130	–	–	–	–
			SSLT	49.5	74.2	61.9	92.8	74.2	111	86.6	130	99.0	148	111	167
		X	STD	49.5	74.2	61.9	92.8	74.2	111	86.6	130	–	–	–	–
			SSLT	49.5	74.2	61.9	92.8	74.2	111	86.6	130	99.0	148	111	167
5 (l = 15 1/2)	Group A	N	STD/SSLT	41.6	62.4	52.0	78.0	62.4	93.6	72.8	109	83.2	125	93.6	140
		X		41.6	62.4	52.0	78.0	62.4	93.6	72.8	109	83.2	125	93.6	140
	Group B	N		41.6	62.4	52.0	78.0	62.4	93.6	72.8	109	83.2	125	93.6	140
		X		41.6	62.4	52.0	78.0	62.4	93.6	72.8	109	83.2	125	93.6	140
4 (l = 12 1/2)	Group A	N	STD/SSLT	33.7	50.6	42.1	63.2	50.6	75.9	59.0	88.5	67.4	101	74.9	112
		X		33.7	50.6	42.1	63.2	50.6	75.9	59.0	88.5	67.4	101	75.9	114
	Group B	N		33.7	50.6	42.1	63.2	50.6	75.9	59.0	88.5	67.4	101	75.9	114
		X		33.7	50.6	42.1	63.2	50.6	75.9	59.0	88.5	67.4	101	75.9	114
Weld Size, in.				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
– Indicates that the plate thickness is greater than the maximum given in Table 10-9.
Tabulated values are grouped when available strength is independent of hole type.

N = Threads included
X = Threads excluded

F_y = 36 ksi Plate

Table 10-10a (continued)
Single-Plate Connections
Bolt, Weld and Single-Plate Available Strengths, kips

1-in. Bolts

n	Bolt Group	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
3 ($l = 9^1/_2$)	Group A	N	STD/ SSLT	25.8	38.7	32.3	48.4	38.7	58.1	45.2	67.8	51.7	77.5	52.4	78.5
		X		25.8	38.7	32.3	48.4	38.7	58.1	45.2	67.8	51.7	77.5	58.1	87.2
	Group B	N		25.8	38.7	32.3	48.4	38.7	58.1	45.2	67.8	51.7	77.5	58.1	87.2
		X		25.8	38.7	32.3	48.4	38.7	58.1	45.2	67.8	51.7	77.5	58.1	87.2
2 ($l = 6^1/_2$)	Group A	N	STD/ SSLT	17.9	26.9	22.4	33.6	26.9	40.4	29.8	44.7	29.8	44.7	29.8	44.7
		X		17.9	26.9	22.4	33.6	26.9	40.4	31.4	47.1	35.9	53.8	37.5	56.2
	Group B	N		17.9	26.9	22.4	33.6	26.9	40.4	31.4	47.1	35.9	53.8	37.5	56.2
		X		17.9	26.9	22.4	33.6	26.9	40.4	31.4	47.1	35.9	53.8	40.4	60.6
Weld Size, in.				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
– Indicates that the plate thickness is greater than the maximum given in Table 10-9.
Tabulated values are grouped when available strength is independent of hole type.

N = Threads included
X = Threads excluded

F_y = 36 ksi Plate

Table 10-10a (continued)
Single-Plate Connections
Bolt, Weld and Single-Plate Available Strengths, kips

$1^1/_8$-in. Bolts

n	Bolt Group	Thread Cond.	Hole Type	Plate Thickness, in.											
				$^5/_{16}$		$^3/_8$		$^7/_{16}$		$^1/_2$		$^9/_{16}$		$^5/_8$	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
12 (l = 37)	Group A	N	STD	116	173	139	208	162	243	185	277	–	–	–	–
			SSLT	116	173	139	208	162	243	185	277	208	312	231	347
		X	STD	116	173	139	208	162	243	185	277	–	–	–	–
			SSLT	116	173	139	208	162	243	185	277	208	312	231	347
	Group B	N	STD	116	173	139	208	162	243	185	277	–	–	–	–
			SSLT	116	173	139	208	162	243	185	277	208	312	231	347
		X	STD	116	173	139	208	162	243	185	277	–	–	–	–
			SSLT	116	173	139	208	162	243	185	277	208	312	231	347
11 (l = 34)	Group A	N	STD	106	160	128	191	149	223	170	255	–	–	–	–
			SSLT	106	160	128	191	149	223	170	255	191	287	213	319
		X	STD	106	160	128	191	149	223	170	255	–	–	–	–
			SSLT	106	160	128	191	149	223	170	255	191	287	213	319
	Group B	N	STD	106	160	128	191	149	223	170	255	–	–	–	–
			SSLT	106	160	128	191	149	223	170	255	191	287	213	319
		X	STD	106	160	128	191	149	223	170	255	–	–	–	–
			SSLT	106	160	128	191	149	223	170	255	191	287	213	319
10 (l = 31)	Group A	N	STD	97.2	146	117	175	136	204	156	233	–	–	–	–
			SSLT	97.2	146	117	175	136	204	156	233	175	262	194	292
		X	STD	97.2	146	117	175	136	204	156	233	–	–	–	–
			SSLT	97.2	146	117	175	136	204	156	233	175	262	194	292
	Group B	N	STD	97.2	146	117	175	136	204	156	233	–	–	–	–
			SSLT	97.2	146	117	175	136	204	156	233	175	262	194	292
		X	STD	97.2	146	117	175	136	204	156	233	–	–	–	–
			SSLT	97.2	146	117	175	136	204	156	233	175	262	194	292
9 (l = 28)	Group A	N	STD	88.0	132	106	158	123	185	141	211	–	–	–	–
			SSLT	88.0	132	106	158	123	185	141	211	158	238	176	264
		X	STD	88.0	132	106	158	123	185	141	211	–	–	–	–
			SSLT	88.0	132	106	158	123	185	141	211	158	238	176	264
	Group B	N	STD	88.0	132	106	158	123	185	141	211	–	–	–	–
			SSLT	88.0	132	106	158	123	185	141	211	158	238	176	264
		X	STD	88.0	132	106	158	123	185	141	211	–	–	–	–
			SSLT	88.0	132	106	158	123	185	141	211	158	238	176	264
Weld Size, in.				$^1/_4$		$^1/_4$		$^5/_{16}$		$^5/_{16}$		$^3/_8$		$^7/_{16}$	

STD = Standard holes
SSLT = Short-slotted holes transverse to direction of load
– Indicates that the plate thickness is greater than the maximum given in Table 10-9.

N = Threads included
X = Threads excluded

Table 10-10a (continued)

Single-Plate Connections

Bolt, Weld and Single-Plate Available Strengths, kips

F_y = 36 ksi Plate

$1\frac{1}{8}$-in. Bolts

n	Bolt Group	Thread Cond.	Hole Type	Plate Thickness, in.											
				5/16		3/8		7/16		1/2		9/16		5/8	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
8 (l = 25)	Group A	N	STD	78.8	118	94.6	142	110	166	126	189	–	–	–	–
			SSLT	78.8	118	94.6	142	110	166	126	189	142	213	158	237
		X	STD	78.8	118	94.6	142	110	166	126	189	–	–	–	–
			SSLT	78.8	118	94.6	142	110	166	126	189	142	213	158	237
	Group B	N	STD	78.8	118	94.6	142	110	166	126	189	–	–	–	–
			SSLT	78.8	118	94.6	142	110	166	126	189	142	213	158	237
		X	STD	78.8	118	94.6	142	110	166	126	189	–	–	–	–
			SSLT	78.8	118	94.6	142	110	166	126	189	142	213	158	237
7 (l = 22)	Group A	N	STD	69.7	105	83.6	125	97.5	146	111	167	–	–	–	–
			SSLT	69.7	105	83.6	125	97.5	146	111	167	125	188	139	209
		X	STD	69.7	105	83.6	125	97.5	146	111	167	–	–	–	–
			SSLT	69.7	105	83.6	125	97.5	146	111	167	125	188	139	209
	Group B	N	STD	69.7	105	83.6	125	97.5	146	111	167	–	–	–	–
			SSLT	69.7	105	83.6	125	97.5	146	111	167	125	188	139	209
		X	STD	69.7	105	83.6	125	97.5	146	111	167	–	–	–	–
			SSLT	69.7	105	83.6	125	97.5	146	111	167	125	188	139	209
6 (l = 19)	Group A	N	STD	60.5	90.7	72.6	109	84.7	127	96.8	145	–	–	–	–
			SSLT	60.5	90.7	72.6	109	84.7	127	96.8	145	109	163	121	181
		X	STD	60.5	90.7	72.6	109	84.7	127	96.8	145	–	–	–	–
			SSLT	60.5	90.7	72.6	109	84.7	127	96.8	145	109	163	121	181
	Group B	N	STD	60.5	90.7	72.6	109	84.7	127	96.8	145	–	–	–	–
			SSLT	60.5	90.7	72.6	109	84.7	127	96.8	145	109	163	121	181
		X	STD	60.5	90.7	72.6	109	84.7	127	96.8	145	–	–	–	–
			SSLT	60.5	90.7	72.6	109	84.7	127	96.8	145	109	163	121	181
5 (l = 16)	Group A	N	STD/ SSLT	51.3	77.0	61.6	92.4	71.8	108	82.1	123	92.4	139	103	154
		X		51.3	77.0	61.6	92.4	71.8	108	82.1	123	92.4	139	103	154
	Group B	N		51.3	77.0	61.6	92.4	71.8	108	82.1	123	92.4	139	103	154
		X		51.3	77.0	61.6	92.4	71.8	108	82.1	123	92.4	139	103	154
4 (l = 13)	Group A	N	STD/ SSLT	42.1	63.2	50.6	75.9	59.0	88.5	67.4	101	75.9	114	84.3	126
		X		42.1	63.2	50.6	75.9	59.0	88.5	67.4	101	75.9	114	84.3	126
	Group B	N		42.1	63.2	50.6	75.9	59.0	88.5	67.4	101	75.9	114	84.3	126
		X		42.1	63.2	50.6	75.9	59.0	88.5	67.4	101	75.9	114	84.3	126
Weld Size, in.				1/4		1/4		5/16		5/16		3/8		7/16	

STD = Standard holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
– Indicates that the plate thickness is greater than the maximum given in Table 10-9.
Tabulated values are grouped when available strength is independent of hole type.

N = Threads included
X = Threads excluded

Table 10-10a (continued)
Single-Plate Connections
Bolt, Weld and Single-Plate Available Strengths, kips

F_y = 36 ksi Plate

1⅛-in. Bolts

n	Bolt Group	Thread Cond.	Hole Type	Plate Thickness, in. 5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	7/16 ASD	7/16 LRFD	1/2 ASD	1/2 LRFD	9/16 ASD	9/16 LRFD	5/8 ASD	5/8 LRFD
3 (l = 10)	Group A	N	STD/ SSLT	33.0	49.4	39.6	59.3	46.2	69.2	52.7	79.1	59.3	89.0	65.9	98.9
		X		33.0	49.4	39.6	59.3	46.2	69.2	52.7	79.1	59.3	89.0	65.9	98.9
	Group B	N		33.0	49.4	39.6	59.3	46.2	69.2	52.7	79.1	59.3	89.0	65.9	98.9
		X		33.0	49.4	39.6	59.3	46.2	69.2	52.7	79.1	59.3	89.0	65.9	98.9
2 (l = 7)	Group A	N	STD/ SSLT	23.8	35.7	28.5	42.8	33.3	50.0	37.7	56.6	37.7	56.6	37.7	56.6
		X		23.8	35.7	28.5	42.8	33.3	50.0	38.1	57.1	42.8	64.2	47.5	71.2
	Group B	N		23.8	35.7	28.5	42.8	33.3	50.0	38.1	57.1	42.8	64.2	47.5	71.2
		X		23.8	35.7	28.5	42.8	33.3	50.0	38.1	57.1	42.8	64.2	47.6	71.4
Weld Size, in.				1/4		1/4		5/16		5/16		3/8		7/16	

STD = Standard holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
– Indicates that the plate thickness is greater than the maximum given in Table 10-9.
Tabulated values are grouped when available strength is independent of hole type.

N = Threads included
X = Threads excluded

Table 10-10b
Single-Plate Connections
Bolt, Weld and Single-Plate Available Strengths, kips

F_y = 50 ksi Plate

3/4-in. Bolts

n	Bolt Group	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
12 (l = 35 1/2)	Group A	N	STD	122	183	134	202	–	–	–	–	–	–	–	–
			SSLT	122	183	138	208	138	208	138	208	–	–	–	–
		X	STD	122	183	152	229	–	–	–	–	–	–	–	–
			SSLT	122	183	152	229	174	261	174	261	–	–	–	–
	Group B	N	STD	122	183	152	229	–	–	–	–	–	–	–	–
			SSLT	122	183	152	229	174	261	174	261	–	–	–	–
		X	STD	122	183	152	229	–	–	–	–	–	–	–	–
			SSLT	122	183	152	229	183	274	213	320	–	–	–	–
11 (l = 32 1/2)	Group A	N	STD	112	167	121	183	–	–	–	–	–	–	–	–
			SSLT	112	167	126	190	126	190	126	190	–	–	–	–
		X	STD	112	167	139	209	–	–	–	–	–	–	–	–
			SSLT	112	167	139	209	159	239	159	239	–	–	–	–
	Group B	N	STD	112	167	139	209	–	–	–	–	–	–	–	–
			SSLT	112	167	139	209	159	239	159	239	–	–	–	–
		X	STD	112	167	139	209	–	–	–	–	–	–	–	–
			SSLT	112	167	139	209	167	251	195	293	–	–	–	–
10 (l = 29 1/2)	Group A	N	STD	101	152	110	165	–	–	–	–	–	–	–	–
			SSLT	101	152	115	172	115	172	115	172	–	–	–	–
		X	STD	101	152	126	190	–	–	–	–	–	–	–	–
			SSLT	101	152	126	190	145	217	145	217	–	–	–	–
	Group B	N	STD	101	152	126	190	–	–	–	–	–	–	–	–
			SSLT	101	152	126	190	145	217	145	217	–	–	–	–
		X	STD	101	152	126	190	–	–	–	–	–	–	–	–
			SSLT	101	152	126	190	152	228	177	266	–	–	–	–
9 (l = 26 1/2)	Group A	N	STD	90.8	136	97.2	146	–	–	–	–	–	–	–	–
			SSLT	90.8	136	103	155	103	155	103	155	–	–	–	–
		X	STD	90.8	136	113	170	–	–	–	–	–	–	–	–
			SSLT	90.8	136	113	170	130	194	130	194	–	–	–	–
	Group B	N	STD	90.8	136	113	170	–	–	–	–	–	–	–	–
			SSLT	90.8	136	113	170	130	194	130	194	–	–	–	–
		X	STD	90.8	136	113	170	–	–	–	–	–	–	–	–
			SSLT	90.8	136	113	170	136	204	159	238	–	–	–	–
Weld Size, in.				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard holes
SSLT = Short-slotted holes transverse to direction of load
– Indicates that the plate thickness is greater than the maximum given in Table 10-9.

N = Threads included
X = Threads excluded

Table 10-10b (continued)
Single-Plate Connections
Bolt, Weld and Single-Plate Available Strengths, kips

F_y = 50 ksi Plate

3/4-in. Bolts

n	Bolt Group	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
8 (l = 23 1/2)	Group A	N	STD	80.4	121	84.7	127	–	–	–	–	–	–	–	–
			SSLT	80.4	121	90.9	137	90.9	137	90.9	137	–	–	–	–
		X	STD	80.4	121	101	151	–	–	–	–	–	–	–	–
			SSLT	80.4	121	101	151	115	172	115	172	–	–	–	–
	Group B	N	STD	80.4	121	101	151	–	–	–	–	–	–	–	–
			SSLT	80.4	121	101	151	115	172	115	172	–	–	–	–
		X	STD	80.4	121	101	151	–	–	–	–	–	–	–	–
			SSLT	80.4	121	101	151	121	181	141	211	–	–	–	–
7 (l = 20 1/2)	Group A	N	STD	70.1	105	72.1	108	–	–	–	–	–	–	–	–
			SSLT	70.1	105	78.8	119	78.8	119	78.8	119	–	–	–	–
		X	STD	70.1	105	87.6	131	–	–	–	–	–	–	–	–
			SSLT	70.1	105	87.6	131	99.4	149	99.4	149	–	–	–	–
	Group B	N	STD	70.1	105	87.6	131	–	–	–	–	–	–	–	–
			SSLT	70.1	105	87.6	131	99.4	149	99.4	149	–	–	–	–
		X	STD	70.1	105	87.6	131	–	–	–	–	–	–	–	–
			SSLT	70.1	105	87.6	131	105	158	123	184	–	–	–	–
6 (l = 17 1/2)	Group A	N	STD	59.3	89.1	59.3	89.1	–	–	–	–	–	–	–	–
			SSLT	59.7	89.6	66.5	100	66.8	100	66.8	100	–	–	–	–
		X	STD	59.7	89.6	74.6	112	–	–	–	–	–	–	–	–
			SSLT	59.7	89.6	74.6	112	84.2	126	84.2	126	–	–	–	–
	Group B	N	STD	59.7	89.6	74.6	112	–	–	–	–	–	–	–	–
			SSLT	59.7	89.6	74.6	112	84.2	126	84.2	126	–	–	–	–
		X	STD	59.7	89.6	74.6	112	–	–	–	–	–	–	–	–
			SSLT	59.7	89.6	74.6	112	89.6	134	104	156	–	–	–	–
5 (l = 14 1/2)	Group A	N	STD	49.4	74.0	54.5	82.0	54.5	82.0	54.5	82.0	–	–	–	–
			SSLT	49.4	74.0	54.5	82.0	54.5	82.0	54.5	82.0	54.5	82.0	54.5	82.0
		X	STD	49.4	74.0	61.7	92.5	68.7	103	68.7	103	–	–	–	–
			SSLT	49.4	74.0	61.7	92.5	68.7	103	68.7	103	68.7	103	68.7	103
	Group B	N	STD	49.4	74.0	61.7	92.5	68.7	103	68.7	103	–	–	–	–
			SSLT	49.4	74.0	61.7	92.5	68.7	103	68.7	103	68.7	103	68.7	103
		X	STD	49.4	74.0	61.7	92.5	74.0	111	85.2	127	–	–	–	–
			SSLT	49.4	74.0	61.7	92.5	74.0	111	85.2	127	85.2	127	85.2	127
Weld Size, in.				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard holes
SSLT = Short-slotted holes transverse to direction of load
– Indicates that the plate thickness is greater than the maximum given in Table 10-9.

N = Threads included
X = Threads excluded

Table 10-10b (continued)
Single-Plate Connections
Bolt, Weld and Single-Plate Available Strengths, kips

F_y = 50 ksi Plate

3/4-in. Bolts

n	Bolt Group	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
4 (l = 11 1/2)	Group A	N	STD	39.0	58.5	42.1	63.3	42.1	63.3	42.1	63.3	–	–	–	–
			SSLT	39.0	58.5	42.1	63.3	42.1	63.3	42.1	63.3	42.1	63.3	42.1	63.3
		X	STD	39.0	58.5	48.8	73.1	53.0	79.5	53.0	79.5	–	–	–	–
			SSLT	39.0	58.5	48.8	73.1	53.0	79.5	53.0	79.5	53.0	79.5	53.0	79.5
	Group B	N	STD	39.0	58.5	48.8	73.1	53.0	79.5	53.0	79.5	–	–	–	–
			SSLT	39.0	58.5	48.8	73.1	53.0	79.5	53.0	79.5	53.0	79.5	53.0	79.5
		X	STD	39.0	58.5	48.8	73.1	58.5	87.8	65.8	98.3	–	–	–	–
			SSLT	39.0	58.5	48.8	73.1	58.5	87.8	65.8	98.3	65.8	98.3	65.8	98.3
3 (l = 8 1/2)	Group A	N	STD	28.6	43.0	29.4	44.2	29.4	44.2	29.4	44.2	–	–	–	–
			SSLT	28.6	43.0	29.4	44.2	29.4	44.2	29.4	44.2	29.4	44.2	29.4	44.2
		X	STD	28.6	43.0	35.8	53.7	37.1	55.6	37.1	55.6	–	–	–	–
			SSLT	28.6	43.0	35.8	53.7	37.1	55.6	37.1	55.6	37.1	55.6	37.1	55.6
	Group B	N	STD	28.6	43.0	35.8	53.7	37.1	55.6	37.1	55.6	–	–	–	–
			SSLT	28.6	43.0	35.8	53.7	37.1	55.6	37.1	55.6	37.1	55.6	37.1	55.6
		X	STD	28.6	43.0	35.8	53.7	43.0	64.4	45.9	68.7	–	–	–	–
			SSLT	28.6	43.0	35.8	53.7	43.0	64.4	45.9	68.7	45.9	68.7	45.9	68.7
2 (l = 5 1/2)	Group A	N	STD	16.7	25.1	16.7	25.1	16.7	25.1	16.7	25.1	–	–	–	–
			SSLT	16.7	25.1	16.7	25.1	16.7	25.1	16.7	25.1	16.7	25.1	16.7	25.1
		X	STD	18.3	27.4	21.1	31.6	21.1	31.6	21.1	31.6	–	–	–	–
			SSLT	18.3	27.4	21.1	31.6	21.1	31.6	21.1	31.6	21.1	31.6	21.1	31.6
	Group B	N	STD	18.3	27.4	21.1	31.6	21.1	31.6	21.1	31.6	–	–	–	–
			SSLT	18.3	27.4	21.1	31.6	21.1	31.6	21.1	31.6	21.1	31.6	21.1	31.6
		X	STD	18.3	27.4	22.9	34.3	26.1	39.1	26.1	39.1	–	–	–	–
			SSLT	18.3	27.4	22.9	34.3	26.1	39.1	26.1	39.1	26.1	39.1	26.1	39.1
Weld Size, in.				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard holes
SSLT = Short-slotted holes transverse to direction of load
– Indicates that the plate thickness is greater than the maximum given in Table 10-9.

N = Threads included
X = Threads excluded

Table 10-10b (continued)
Single-Plate Connections
Bolt, Weld and Single-Plate Available Strengths, kips

F_y = 50 ksi Plate

7/8-in. Bolts

n	Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	7/16 ASD	7/16 LRFD	1/2 ASD	1/2 LRFD	9/16 ASD	9/16 LRFD
				Plate Thickness, in.											
12 (*l* = 36)	Group A	N	STD	117	176	146	219	176	263	–	–	–	–	–	–
			SSLT	117	176	146	219	176	263	188	282	188	282	–	–
		X	STD	117	176	146	219	176	263	–	–	–	–	–	–
			SSLT	117	176	146	219	176	263	205	307	234	351	–	–
	Group B	N	STD	117	176	146	219	176	263	–	–	–	–	–	–
			SSLT	117	176	146	219	176	263	205	307	234	351	–	–
		X	STD	117	176	146	219	176	263	–	–	–	–	–	–
			SSLT	117	176	146	219	176	263	205	307	234	351	–	–
11 (*l* = 33)	Group A	N	STD	107	161	134	201	161	241	–	–	–	–	–	–
			SSLT	107	161	134	201	161	241	172	258	172	258	–	–
		X	STD	107	161	134	201	161	241	–	–	–	–	–	–
			SSLT	107	161	134	201	161	241	188	282	215	322	–	–
	Group B	N	STD	107	161	134	201	161	241	–	–	–	–	–	–
			SSLT	107	161	134	201	161	241	188	282	215	322	–	–
		X	STD	107	161	134	201	161	241	–	–	–	–	–	–
			SSLT	107	161	134	201	161	241	188	282	215	322	–	–
10 (*l* = 30)	Group A	N	STD	97.5	146	122	183	146	219	–	–	–	–	–	–
			SSLT	97.5	146	122	183	146	219	156	234	156	234	–	–
		X	STD	97.5	146	122	183	146	219	–	–	–	–	–	–
			SSLT	97.5	146	122	183	146	219	171	256	195	293	–	–
	Group B	N	STD	97.5	146	122	183	146	219	–	–	–	–	–	–
			SSLT	97.5	146	122	183	146	219	171	256	195	293	–	–
		X	STD	97.5	146	122	183	146	219	–	–	–	–	–	–
			SSLT	97.5	146	122	183	146	219	171	256	195	293	–	–
9 (*l* = 27)	Group A	N	STD	87.8	132	110	165	132	197	–	–	–	–	–	–
			SSLT	87.8	132	110	165	132	197	140	210	140	210	–	–
		X	STD	87.8	132	110	165	132	197	–	–	–	–	–	–
			SSLT	87.8	132	110	165	132	197	154	230	176	263	–	–
	Group B	N	STD	87.8	132	110	165	132	197	–	–	–	–	–	–
			SSLT	87.8	132	110	165	132	197	154	230	176	263	–	–
		X	STD	87.8	132	110	165	132	197	–	–	–	–	–	–
			SSLT	87.8	132	110	165	132	197	154	230	176	263	–	–
Weld Size, in.				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard holes
SSLT = Short-slotted holes transverse to direction of load
– Indicates that the plate thickness is greater than the maximum given in Table 10-9.

N = Threads included
X = Threads excluded

Table 10-10b (continued)
Single-Plate Connections
Bolt, Weld and Single-Plate Available Strengths, kips

F_y = 50 ksi Plate

7/8-in. Bolts

n	Bolt Group	Thread Cond.	Hole Type	Plate Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	7/16 ASD	7/16 LRFD	1/2 ASD	1/2 LRFD	9/16 ASD	9/16 LRFD
8 (l = 24)	Group A	N	STD	78.0	117	97.5	146	115	173	–	–	–	–	–	–
			SSLT	78.0	117	97.5	146	117	176	124	186	124	186	–	–
		X	STD	78.0	117	97.5	146	117	176	–	–	–	–	–	–
			SSLT	78.0	117	97.5	146	117	176	137	205	156	234	–	–
	Group B	N	STD	78.0	117	97.5	146	117	176	–	–	–	–	–	–
			SSLT	78.0	117	97.5	146	117	176	137	205	156	234	–	–
		X	STD	78.0	117	97.5	146	117	176	–	–	–	–	–	–
			SSLT	78.0	117	97.5	146	117	176	137	205	156	234	–	–
7 (l = 21)	Group A	N	STD	68.3	102	85.3	128	98.2	147	–	–	–	–	–	–
			SSLT	68.3	102	85.3	128	102	154	107	161	107	161	–	–
		X	STD	68.3	102	85.3	128	102	154	–	–	–	–	–	–
			SSLT	68.3	102	85.3	128	102	154	119	179	135	203	–	–
	Group B	N	STD	68.3	102	85.3	128	102	154	–	–	–	–	–	–
			SSLT	68.3	102	85.3	128	102	154	119	179	135	203	–	–
		X	STD	68.3	102	85.3	128	102	154	–	–	–	–	–	–
			SSLT	68.3	102	85.3	128	102	154	119	179	137	205	–	–
6 (l = 18)	Group A	N	STD	58.5	87.8	73.1	110	80.7	121	–	–	–	–	–	–
			SSLT	58.5	87.8	73.1	110	87.8	132	90.9	136	90.9	136	–	–
		X	STD	58.5	87.8	73.1	110	87.8	132	–	–	–	–	–	–
			SSLT	58.5	87.8	73.1	110	87.8	132	102	154	114	172	–	–
	Group B	N	STD	58.5	87.8	73.1	110	87.8	132	–	–	–	–	–	–
			SSLT	58.5	87.8	73.1	110	87.8	132	102	154	114	172	–	–
		X	STD	58.5	87.8	73.1	110	87.8	132	–	–	–	–	–	–
			SSLT	58.5	87.8	73.1	110	87.8	132	102	154	117	176	–	–
5 (l = 15)	Group A	N	STD	48.8	73.1	60.9	91.4	73.1	110	74.2	111	74.2	111	–	–
			SSLT	48.8	73.1	60.9	91.4	73.1	110	74.2	111	74.2	111	74.2	111
		X	STD	48.8	73.1	60.9	91.4	73.1	110	85.3	128	93.4	141	–	–
			SSLT	48.8	73.1	60.9	91.4	73.1	110	85.3	128	93.4	141	93.4	141
	Group B	N	STD	48.8	73.1	60.9	91.4	73.1	110	85.3	128	93.4	141	–	–
			SSLT	48.8	73.1	60.9	91.4	73.1	110	85.3	128	93.4	141	93.4	141
		X	STD	48.8	73.1	60.9	91.4	73.1	110	85.3	128	97.5	146	–	–
			SSLT	48.8	73.1	60.9	91.4	73.1	110	85.3	128	97.5	146	110	165
Weld Size, in.				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard holes
SSLT = Short-slotted holes transverse to direction of load
– Indicates that the plate thickness is greater than the maximum given in Table 10-9.
N = Threads included
X = Threads excluded

F_y = 50 ksi Plate

Table 10-10b (continued)
Single-Plate Connections
Bolt, Weld and Single-Plate Available Strengths, kips

7/8-in. Bolts

n	Bolt Group	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
4 (l = 12)	Group A	N	STD	39.0	58.5	48.8	73.1	57.3	85.9	57.3	85.9	57.3	85.9	–	–
			SSLT	39.0	58.5	48.8	73.1	57.3	85.9	57.3	85.9	57.3	85.9	57.3	85.9
		X	STD	39.0	58.5	48.8	73.1	58.5	87.8	68.3	102	72.1	109	–	–
			SSLT	39.0	58.5	48.8	73.1	58.5	87.8	68.3	102	72.1	109	72.1	109
	Group B	N	STD	39.0	58.5	48.8	73.1	58.5	87.8	68.3	102	72.1	109	–	–
			SSLT	39.0	58.5	48.8	73.1	58.5	87.8	68.3	102	72.1	109	72.1	109
		X	STD	39.0	58.5	48.8	73.1	58.5	87.8	68.3	102	78.0	117	–	–
			SSLT	39.0	58.5	48.8	73.1	58.5	87.8	68.3	102	78.0	117	87.8	132
3 (l = 9)	Group A	N	STD	29.3	43.9	36.6	54.8	40.0	60.0	40.0	60.0	40.0	60.0	–	–
			SSLT	29.3	43.9	36.6	54.8	40.0	60.0	40.0	60.0	40.0	60.0	40.0	60.0
		X	STD	29.3	43.9	36.6	54.8	43.9	65.8	50.4	75.8	50.4	75.8	–	–
			SSLT	29.3	43.9	36.6	54.8	43.9	65.8	50.4	75.8	50.4	75.8	50.4	75.8
	Group B	N	STD	29.3	43.9	36.6	54.8	43.9	65.8	50.4	75.8	50.4	75.8	–	–
			SSLT	29.3	43.9	36.6	54.8	43.9	65.8	50.4	75.8	50.4	75.8	50.4	75.8
		X	STD	29.3	43.9	36.6	54.8	43.9	65.8	51.2	76.8	58.5	87.8	–	–
			SSLT	29.3	43.9	36.6	54.8	43.9	65.8	51.2	76.8	58.5	87.8	62.2	93.6
2 (l = 6)	Group A	N	STD	19.5	29.3	22.8	34.1	22.8	34.1	22.8	34.1	22.8	34.1	–	–
			SSLT	19.5	29.3	22.8	34.1	22.8	34.1	22.8	34.1	22.8	34.1	22.8	34.1
		X	STD	19.5	29.3	24.4	36.6	28.7	43.1	28.7	43.1	28.7	43.1	–	–
			SSLT	19.5	29.3	24.4	36.6	28.7	43.1	28.7	43.1	28.7	43.1	28.7	43.1
	Group B	N	STD	19.5	29.3	24.4	36.6	28.7	43.1	28.7	43.1	28.7	43.1	–	–
			SSLT	19.5	29.3	24.4	36.6	28.7	43.1	28.7	43.1	28.7	43.1	28.7	43.1
		X	STD	19.5	29.3	24.4	36.6	29.3	43.9	34.1	51.2	35.4	53.2	–	–
			SSLT	19.5	29.3	24.4	36.6	29.3	43.9	34.1	51.2	35.4	53.2	35.4	53.2
Weld Size, in.				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard holes
SSLT = Short-slotted holes transverse to direction of load
– Indicates that the plate thickness is greater than the maximum given in Table 10-9.

N = Threads included
X = Threads excluded

Table 10-10b (continued)

F_y = 50 ksi Plate

Single-Plate Connections

Bolt, Weld and Single-Plate Available Strengths, kips

1-in. Bolts

n	Bolt Group	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
12 (l = 36 1/2)	Group A	N	STD	108	163	136	203	163	244	190	285	–	–	–	–
			SSLT	108	163	136	203	163	244	190	285	217	325	244	366
		X	STD	108	163	136	203	163	244	190	285	–	–	–	–
			SSLT	108	163	136	203	163	244	190	285	217	325	244	366
	Group B	N	STD	108	163	136	203	163	244	190	285	–	–	–	–
			SSLT	108	163	136	203	163	244	190	285	217	325	244	366
		X	STD	108	163	136	203	163	244	190	285	–	–	–	–
			SSLT	108	163	136	203	163	244	190	285	217	325	244	366
11 (l = 33 1/2)	Group A	N	STD	99.6	149	125	187	149	224	174	262	–	–	–	–
			SSLT	99.6	149	125	187	149	224	174	262	199	299	224	336
		X	STD	99.6	149	125	187	149	224	174	262	–	–	–	–
			SSLT	99.6	149	125	187	149	224	174	262	199	299	224	336
	Group B	N	STD	99.6	149	125	187	149	224	174	262	–	–	–	–
			SSLT	99.6	149	125	187	149	224	174	262	199	299	224	336
		X	STD	99.6	149	125	187	149	224	174	262	–	–	–	–
			SSLT	99.6	149	125	187	149	224	174	262	199	299	224	336
10 (l = 30 1/2)	Group A	N	STD	90.8	136	113	170	136	204	159	238	–	–	–	–
			SSLT	90.8	136	113	170	136	204	159	238	182	272	204	306
		X	STD	90.8	136	113	170	136	204	159	238	–	–	–	–
			SSLT	90.8	136	113	170	136	204	159	238	182	272	204	306
	Group B	N	STD	90.8	136	113	170	136	204	159	238	–	–	–	–
			SSLT	90.8	136	113	170	136	204	159	238	182	272	204	306
		X	STD	90.8	136	113	170	136	204	159	238	–	–	–	–
			SSLT	90.8	136	113	170	136	204	159	238	182	272	204	306
9 (l = 27 1/2)	Group A	N	STD	82.0	123	102	154	123	184	143	215	–	–	–	–
			SSLT	82.0	123	102	154	123	184	143	215	164	246	183	275
		X	STD	82.0	123	102	154	123	184	143	215	–	–	–	–
			SSLT	82.0	123	102	154	123	184	143	215	164	246	184	277
	Group B	N	STD	82.0	123	102	154	123	184	143	215	–	–	–	–
			SSLT	82.0	123	102	154	123	184	143	215	164	246	184	277
		X	STD	82.0	123	102	154	123	184	143	215	–	–	–	–
			SSLT	82.0	123	102	154	123	184	143	215	164	246	184	277
Weld Size, in.				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard holes
SSLT = Short-slotted holes transverse to direction of load
– Indicates that the plate thickness is greater than the maximum given in Table 10-9.

N = Threads included
X = Threads excluded

Table 10-10b (continued)
Single-Plate Connections
Bolt, Weld and Single-Plate Available Strengths, kips

F_y = 50 ksi Plate

1-in. Bolts

n	Bolt Group	Thread Cond.	Hole Type	Plate Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	7/16 ASD	7/16 LRFD	1/2 ASD	1/2 LRFD	9/16 ASD	9/16 LRFD
8 (l = 24 1/2)	Group A	N	STD	73.1	110	91.4	137	110	165	128	192	–	–	–	–
			SSLT	73.1	110	91.4	137	110	165	128	192	146	219	162	243
		X	STD	73.1	110	91.4	137	110	165	128	192	–	–	–	–
			SSLT	73.1	110	91.4	137	110	165	128	192	146	219	165	247
	Group B	N	STD	73.1	110	91.4	137	110	165	128	192	–	–	–	–
			SSLT	73.1	110	91.4	137	110	165	128	192	146	219	165	247
		X	STD	73.1	110	91.4	137	110	165	128	192	–	–	–	–
			SSLT	73.1	110	91.4	137	110	165	128	192	146	219	165	247
7 (l = 21 1/2)	Group A	N	STD	64.3	96.4	80.4	121	96.4	145	113	169	–	–	–	–
			SSLT	64.3	96.4	80.4	121	96.4	145	113	169	129	193	140	211
		X	STD	64.3	96.4	80.4	121	96.4	145	113	169	–	–	–	–
			SSLT	64.3	96.4	80.4	121	96.4	145	113	169	129	193	145	217
	Group B	N	STD	64.3	96.4	80.4	121	96.4	145	113	169	–	–	–	–
			SSLT	64.3	96.4	80.4	121	96.4	145	113	169	129	193	145	217
		X	STD	64.3	96.4	80.4	121	96.4	145	113	169	–	–	–	–
			SSLT	64.3	96.4	80.4	121	96.4	145	113	169	129	193	145	217
6 (l = 18 1/2)	Group A	N	STD	55.5	83.2	69.3	104	83.2	125	97.0	146	–	–	–	–
			SSLT	55.5	83.2	69.3	104	83.2	125	97.0	146	111	166	119	178
		X	STD	55.5	83.2	69.3	104	83.2	125	97.0	146	–	–	–	–
			SSLT	55.5	83.2	69.3	104	83.2	125	97.0	146	111	166	125	187
	Group B	N	STD	55.5	83.2	69.3	104	83.2	125	97.0	146	–	–	–	–
			SSLT	55.5	83.2	69.3	104	83.2	125	97.0	146	111	166	125	187
		X	STD	55.5	83.2	69.3	104	83.2	125	97.0	146	–	–	–	–
			SSLT	55.5	83.2	69.3	104	83.2	125	97.0	146	111	166	125	187
5 (l = 15 1/2)	Group A	N	STD/SSLT	46.6	69.9	58.3	87.4	69.9	105	81.6	122	93.2	140	97.1	146
		X		46.6	69.9	58.3	87.4	69.9	105	81.6	122	93.2	140	105	157
	Group B	N		46.6	69.9	58.3	87.4	69.9	105	81.6	122	93.2	140	105	157
		X		46.6	69.9	58.3	87.4	69.9	105	81.6	122	93.2	140	105	157
4 (l = 12 1/2)	Group A	N	STD/SSLT	37.8	56.7	47.2	70.8	56.7	85.0	66.1	99.2	74.9	112	74.9	112
		X		37.8	56.7	47.2	70.8	56.7	85.0	66.1	99.2	75.6	113	85.0	128
	Group B	N		37.8	56.7	47.2	70.8	56.7	85.0	66.1	99.2	75.6	113	85.0	128
		X		37.8	56.7	47.2	70.8	56.7	85.0	66.1	99.2	75.6	113	85.0	128
Weld Size, in.				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
– Indicates that the plate thickness is greater than the maximum given in Table 10-9.
Tabulated values are grouped when available strength is independent of hole type.

N = Threads included
X = Threads excluded

Table 10-10b (continued)
Single-Plate Connections
Bolt, Weld and Single-Plate Available Strengths, kips

F_y = 50 ksi Plate

1-in. Bolts

n	Bolt Group	Thread Cond.	Hole Type	Plate Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	7/16 ASD	7/16 LRFD	1/2 ASD	1/2 LRFD	9/16 ASD	9/16 LRFD
3 ($l = 9^1/_2$)	Group A	N	STD/SSLT	28.9	43.4	36.2	54.3	43.4	65.1	50.7	76.0	52.4	78.5	52.4	78.5
		X		28.9	43.4	36.2	54.3	43.4	65.1	50.7	76.0	57.9	86.8	65.1	97.7
	Group B	N		28.9	43.4	36.2	54.3	43.4	65.1	50.7	76.0	57.9	86.8	65.1	97.7
		X		28.9	43.4	36.2	54.3	43.4	65.1	50.7	76.0	57.9	86.8	65.1	97.7
2 ($l = 6^1/_2$)	Group A	N	STD/SSLT	20.1	30.2	25.1	37.7	29.8	44.7	29.8	44.7	29.8	44.7	29.8	44.7
		X		20.1	30.2	25.1	37.7	30.2	45.2	35.2	52.8	37.5	56.2	37.5	56.2
	Group B	N		20.1	30.2	25.1	37.7	30.2	45.2	35.2	52.8	37.5	56.2	37.5	56.2
		X		20.1	30.2	25.1	37.7	30.2	45.2	35.2	52.8	40.2	60.3	45.2	67.9
Weld Size, in.				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
– Indicates that the plate thickness is greater than the maximum given in Table 10-9.
Tabulated values are grouped when available strength is independent of hole type.

N = Threads included
X = Threads excluded

Table 10-10b (continued)
Single-Plate Connections
Bolt, Weld and Single-Plate Available Strengths, kips

F_y = 50 ksi Plate

$1^1/_8$-in. Bolts

n	Bolt Group	Thread Cond.	Hole Type	Plate Thickness, in.											
				5/16		3/8		7/16		1/2		9/16		5/8	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
12 (l = 37)	Group A	N	STD	129	194	155	233	181	272	207	311	–	–	–	–
			SSLT	129	194	155	233	181	272	207	311	233	350	259	388
		X	STD	129	194	155	233	181	272	207	311	–	–	–	–
			SSLT	129	194	155	233	181	272	207	311	233	350	259	388
	Group B	N	STD	129	194	155	233	181	272	207	311	–	–	–	–
			SSLT	129	194	155	233	181	272	207	311	233	350	259	388
		X	STD	129	194	155	233	181	272	207	311	–	–	–	–
			SSLT	129	194	155	233	181	272	207	311	233	350	259	388
11 (l = 34)	Group A	N	STD	119	179	143	215	167	250	191	286	–	–	–	–
			SSLT	119	179	143	215	167	250	191	286	215	322	238	358
		X	STD	119	179	143	215	167	250	191	286	–	–	–	–
			SSLT	119	179	143	215	167	250	191	286	215	322	238	358
	Group B	N	STD	119	179	143	215	167	250	191	286	–	–	–	–
			SSLT	119	179	143	215	167	250	191	286	215	322	238	358
		X	STD	119	179	143	215	167	250	191	286	–	–	–	–
			SSLT	119	179	143	215	167	250	191	286	215	322	238	358
10 (l = 31)	Group A	N	STD	109	163	131	196	152	229	174	261	–	–	–	–
			SSLT	109	163	131	196	152	229	174	261	196	294	218	327
		X	STD	109	163	131	196	152	229	174	261	–	–	–	–
			SSLT	109	163	131	196	152	229	174	261	196	294	218	327
	Group B	N	STD	109	163	131	196	152	229	174	261	–	–	–	–
			SSLT	109	163	131	196	152	229	174	261	196	294	218	327
		X	STD	109	163	131	196	152	229	174	261	–	–	–	–
			SSLT	109	163	131	196	152	229	174	261	196	294	218	327
9 (l = 28)	Group A	N	STD	98.6	148	118	178	138	207	158	237	–	–	–	–
			SSLT	98.6	148	118	178	138	207	158	237	178	266	197	296
		X	STD	98.6	148	118	178	138	207	158	237	–	–	–	–
			SSLT	98.6	148	118	178	138	207	158	237	178	266	197	296
	Group B	N	STD	98.6	148	118	178	138	207	158	237	–	–	–	–
			SSLT	98.6	148	118	178	138	207	158	237	178	266	197	296
		X	STD	98.6	148	118	178	138	207	158	237	–	–	–	–
			SSLT	98.6	148	118	178	138	207	158	237	178	266	197	296
Weld Size, in.				1/4		1/4		5/16		5/16		3/8		7/16	

STD = Standard holes
SSLT = Short-slotted holes transverse to direction of load
– Indicates that the plate thickness is greater than the maximum given in Table 10-9.
Tabulated values are grouped when available strength is independent of hole type.

N = Threads included
X = Threads excluded

Table 10-10b (continued)
Single-Plate Connections
Bolt, Weld and Single-Plate Available Strengths, kips

F_y = 50 ksi Plate

$1\frac{1}{8}$-in. Bolts

n	Bolt Group	Thread Cond.	Hole Type	Plate Thickness, in. 5/16		3/8		7/16		1/2		9/16		5/8	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
8 (l = 25)	Group A	N	STD	88.4	133	106	159	124	186	141	212	–	–	–	–
			SSLT	88.4	133	106	159	124	186	141	212	159	239	177	265
		X	STD	88.4	133	106	159	124	186	141	212	–	–	–	–
			SSLT	88.4	133	106	159	124	186	141	212	159	239	177	265
	Group B	N	STD	88.4	133	106	159	124	186	141	212	–	–	–	–
			SSLT	88.4	133	106	159	124	186	141	212	159	239	177	265
		X	STD	88.4	133	106	159	124	186	141	212	–	–	–	–
			SSLT	88.4	133	106	159	124	186	141	212	159	239	177	265
7 (l = 22)	Group A	N	STD	78.1	117	93.7	141	109	164	125	187	–	–	–	–
			SSLT	78.1	117	93.7	141	109	164	125	187	141	211	156	234
		X	STD	78.1	117	93.7	141	109	164	125	187	–	–	–	–
			SSLT	78.1	117	93.7	141	109	164	125	187	141	211	156	234
	Group B	N	STD	78.1	117	93.7	141	109	164	125	187	–	–	–	–
			SSLT	78.1	117	93.7	141	109	164	125	187	141	211	156	234
		X	STD	78.1	117	93.7	141	109	164	125	187	–	–	–	–
			SSLT	78.1	117	93.7	141	109	164	125	187	141	211	156	234
6 (l = 19)	Group A	N	STD	67.8	102	81.4	122	94.9	142	108	163	–	–	–	–
			SSLT	67.8	102	81.4	122	94.9	142	108	163	122	183	136	203
		X	STD	67.8	102	81.4	122	94.9	142	108	163	–	–	–	–
			SSLT	67.8	102	81.4	122	94.9	142	108	163	122	183	136	203
	Group B	N	STD	67.8	102	81.4	122	94.9	142	108	163	–	–	–	–
			SSLT	67.8	102	81.4	122	94.9	142	108	163	122	183	136	203
		X	STD	67.8	102	81.4	122	94.9	142	108	163	–	–	–	–
			SSLT	67.8	102	81.4	122	94.9	142	108	163	122	183	136	203
5 (l = 16)	Group A	N	STD/ SSLT	57.5	86.3	69.0	104	80.5	121	92.0	138	104	155	115	173
		X		57.5	86.3	69.0	104	80.5	121	92.0	138	104	155	115	173
	Group B	N		57.5	86.3	69.0	104	80.5	121	92.0	138	104	155	115	173
		X		57.5	86.3	69.0	104	80.5	121	92.0	138	104	155	115	173
4 (l = 13)	Group A	N	STD/ SSLT	47.2	70.8	56.7	85.0	66.1	99.2	75.6	113	85.0	128	94.5	142
		X		47.2	70.8	56.7	85.0	66.1	99.2	75.6	113	85.0	128	94.5	142
	Group B	N		47.2	70.8	56.7	85.0	66.1	99.2	75.6	113	85.0	128	94.5	142
		X		47.2	70.8	56.7	85.0	66.1	99.2	75.6	113	85.0	128	94.5	142
Weld Size, in.				1/4		1/4		5/16		5/16		3/8		7/16	

STD = Standard holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
– Indicates that the plate thickness is greater than the maximum given in Table 10-9.
Tabulated values are grouped when available strength is independent of hole type.

N = Threads included
X = Threads excluded

Table 10-10b (continued)
Single-Plate Connections
Bolt, Weld and Single-Plate Available Strengths, kips

F_y = 50 ksi Plate

$1\frac{1}{8}$-in. Bolts

n	Bolt Group	Thread Cond.	Hole Type	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	7/16 ASD	7/16 LRFD	1/2 ASD	1/2 LRFD	9/16 ASD	9/16 LRFD	5/8 ASD	5/8 LRFD
3 (l = 10)	Group A	N	STD/SSLT	36.9	55.4	44.3	66.5	51.7	77.6	59.1	88.7	66.2	99.5	66.2	99.5
		X		36.9	55.4	44.3	66.5	51.7	77.6	59.1	88.7	66.5	99.7	73.9	111
	Group B	N		36.9	55.4	44.3	66.5	51.7	77.6	59.1	88.7	66.5	99.7	73.9	111
		X		36.9	55.4	44.3	66.5	51.7	77.6	59.1	88.7	66.5	99.7	73.9	111
2 (l = 7)	Group A	N	STD/SSLT	26.7	40.0	32.0	48.0	37.3	56.0	37.7	56.6	37.7	56.6	37.7	56.6
		X		26.7	40.0	32.0	48.0	37.3	56.0	42.7	64.0	47.5	71.2	47.5	71.2
	Group B	N		26.7	40.0	32.0	48.0	37.3	56.0	42.7	64.0	47.5	71.2	47.5	71.2
		X		26.7	40.0	32.0	48.0	37.3	56.0	42.7	64.0	48.0	72.0	53.3	80.0
Weld Size, in.				1/4		1/4		5/16		5/16		3/8		7/16	

Plate Thickness, in. (column groups 5/16, 3/8, 7/16, 1/2, 9/16, 5/8)

STD = Standard holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
– Indicates that the plate thickness is greater than the maximum given in Table 10-9.
Tabulated values are grouped when available strength is independent of hole type.

N = Threads included
X = Threads excluded

SINGLE-ANGLE CONNECTIONS

A single-angle connection is made with an angle on one side of the web of the beam to be supported, as illustrated in Figure 10-13. This angle is preferably shop-bolted or welded to the supporting member and field-bolted to the supported beam.

When the angle is welded to the support, adequate flexibility must be provided in the connection. As illustrated in Figure 10-13(c), the weld is placed along the toe and across the bottom of the angle with a return at the top limited by AISC *Specification* Section J2.2b. Note that welding across the entire top of the angle must be avoided as it would inhibit the flexibility and, therefore, the necessary end rotation of the connection. The performance of the resulting connection would not be as intended for simple shear connections.

Design Checks

The available strength of a single-angle connection is determined from the applicable limit states for bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a.

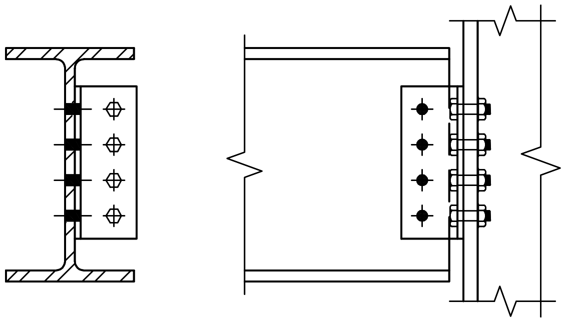
As illustrated in Figure 10-14, the effect of eccentricity must be considered in the angle leg attached to the supporting member. Additionally, eccentricity must be considered if the eccentricity exceeds 3 in. (to the face of the supporting member) or if a double vertical row of bolts through the web of the supported member is used. Eccentricity must be considered in the design of welds for single-angle connections. Holes in the angle leg to the supporting member must be standard holes to facilitate erection and provide torsional resistance due to the nonconcentric loading in the connection. Holes in the angle leg to the supported member can be standard holes or horizontal short slots.

Recommended Angle Thickness

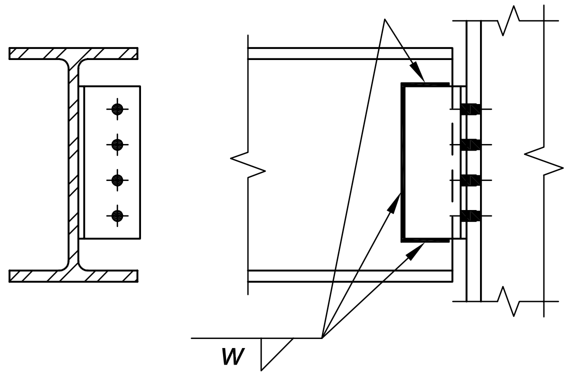
A minimum angle thickness of $^3/_8$-in. for $^3/_4$-in.- and $^7/_8$-in.-diameter bolts, and $^1/_2$-in. for 1-in.-diameter bolts should be used. A 4×3 angle is normally selected for a single angle welded to the support with the 3-in. leg being the welded leg.

Shop and Field Practices

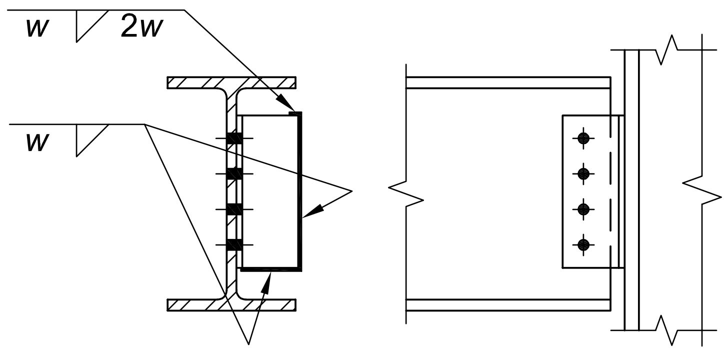
Single-angle connections may be easily erected to the webs of supporting girders and to the flanges of supporting columns. When framing to a column flange, provision must be made for possible mill variation in the depth of the column. Because the angle is usually shop-attached to the column flange, horizontal short slots in the supported angle leg may be used to provide the necessary adjustment for any mill variations. Attaching the angle to the column flange offers the advantage of side erection of the beam. The same is true for a girder web or truss support. Additionally, proper bay dimensions may be maintained without the need for shims. This advantage is lost when the angle is shop-attached (bolted or welded) to the supported beam web.



(a) All-bolted



(b) Bolted/welded, angle welded to supported beam



Note: Weld return on top of angle
per *Specification* Section J2.2b.

(c) Bolted/welded, angle welded to support

Fig. 10-13. Single-angle connections.

DESIGN TABLE DISCUSSION (TABLES 10-11 AND 10-12)

Table 10-11. All-Bolted Single-Angle Connections

Table 10-11 is a design aid for all-bolted single-angle connections. The tabulated eccentrically loaded bolt group coefficients, C, are used to determine the available strength of the bolt group, ϕR_n or R_n/Ω, where

$$R_n = Cr_n \tag{10-6}$$

$$\phi = 0.75 \qquad \Omega = 2.00$$

where

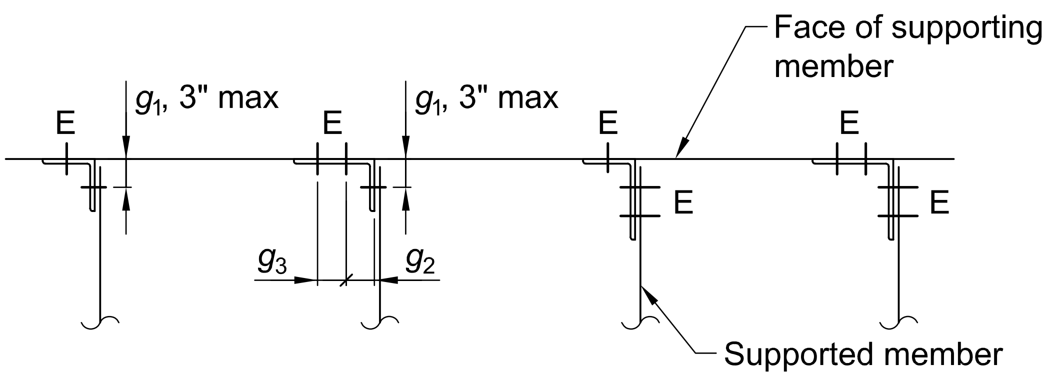
C = coefficient from Table 10-11

r_n = nominal strength of one bolt in shear or bearing, kips

Case I single-angle connection coefficients are for a single vertical row of bolts assuming $2^1/_2$ in. eccentricity. Case II single-angle connection coefficients are for a double vertical row of bolts assuming $4^1/_4$ in. eccentricity. The eccentricities shown in the table include the supported beam half-web thickness, $t_w/2$. If a greater eccentricity is required, the coefficient C must be recalculated from Part 7. If a lesser eccentricity exists, use of the table values will produce conservative results. Interpolation between values in this table may produce an incorrect result.

Table 10-12. Bolted/Welded Single-Angle Connections

Table 10-12 is a design aid for bolted/welded single-angle connections. Tabulated bolt and angle available strengths consider the limit states of bolt shear, bolt bearing and tearout on the angle, shear yielding of the angle, shear rupture of the angle, and block shear rupture of the angle. Values are tabulated for 2 through 12 rows of $^3/_4$-in.- and $^7/_8$-in.-diameter Group



Notes: E indicates that eccentricity must be considered in this leg.
Gages g_1, g_2 and g_3 are workable gages as shown in Table 1-7A.

Fig. 10-14. Eccentricity in angles.

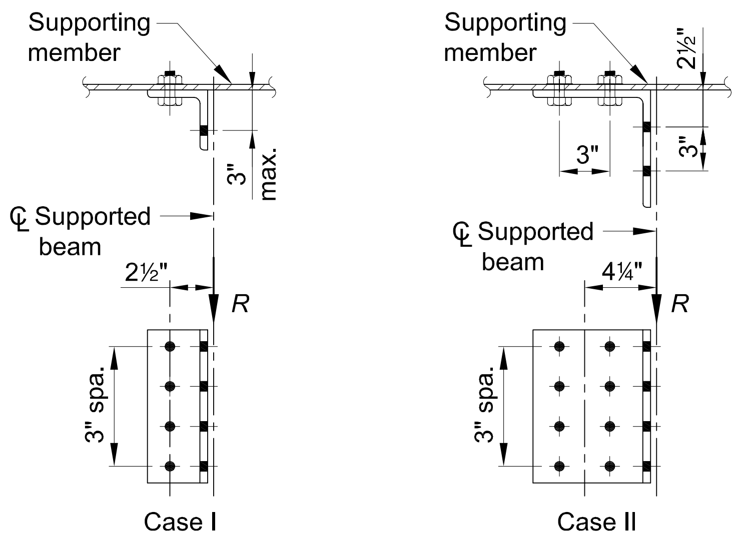
A bolts (as defined in AISC *Specification* Section J3.1) at 3-in. spacing. For calculation purposes, angle edge distances, l_{ev} and l_{eh}, are assumed to be 1¼ in. Electrode strength is assumed to be 70 ksi. Listed strengths are based on angle material with F_y = 36 ksi and F_u = 58 ksi. In cases where a single-angle connection must be field-welded, erection bolts may be placed in the field-welded leg.

Weld available strengths are determined by the instantaneous center of rotation method using Table 8-10 with $\theta = 0°$. The tabulated values assume a half-web thickness of ¼ in. and may be used conservatively for lesser half-web thicknesses. For half-web thicknesses greater than ¼ in., the tabulated values should be reduced proportionally by an amount up to 8% at a half-web thickness of ½ in. The tabulated minimum supporting flange or web thickness is the thickness that matches the strength of the support material to the strength of the weld material. In a manner similar to that illustrated previously for Table 10-2, the minimum material thickness (for one line of weld) is:

$$t_{min} = \frac{3.09D}{F_u} \tag{9-2}$$

where D is the number of sixteenths in the weld size. When welds line up on opposite sides of the support, the minimum thickness is the sum of the thicknesses required for each weld. In either case, when less than the minimum material thickness is present, the tabulated weld available strength should be multiplied by the ratio of the thickness provided to the minimum thickness. Interpolation between values in this table may produce an incorrect result.

Table 10-11
All-Bolted Single-Angle Connections



Note: Standard holes in support leg of angle.

Eccentrically Loaded Bolt Group Coefficients, *C*

Number of Bolts in One Vertical Row, *n*	Case I	Case II
12	11.4	21.5
11	10.4	19.4
10	9.37	17.3
9	8.34	15.2
8	7.31	13.0
7	6.27	10.9
6	5.22	8.70
5	4.15	6.63
4	3.07	4.70
3	1.99	2.94
2	1.03	1.61
1	—	0.518

$\phi R_n = C(\phi r_n)$ or $R_n/\Omega = C(r_n/\Omega)$

where

C = coefficient from Table 10-11 for eccentrically loaded bolt group

ϕr_n = design strength of one bolt in shear, bearing or tearout, kips/bolt

r_n/Ω = allowable strength of one bolt in shear, bearing or tearout, kips/bolt

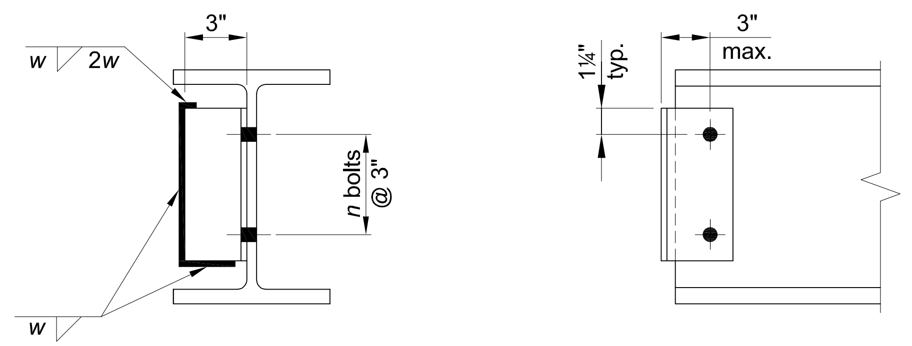
Notes:

For eccentricities less than or equal to those shown above, tabulated values may be used.

For greater eccentricities, coefficient C should be recalculated from Part 7.

Connection may be bearing-type or slip-critical.

Table 10-12
Bolted/Welded Single-Angle Connections



Number of Bolts in One Vertical Row	Bolt and Angle Strength, kips, Group A Bolts, 3/4 in. ASD	3/4 in. LRFD	7/8 in. ASD	7/8 in. LRFD	Angle Size (F_y = 36 ksi)	Angle Length, in.	Weld (70 ksi) Size, *w*, in.	Weld (70 ksi) Available Strength, kips ASD	Available Strength, kips LRFD	Minimum t_w of Supporting Member with Angles Both Sides of Web, in.
12	143	215	144	216	L4×3×3/8	35 1/2	5/16	179	268	0.475
							1/4	143	214	0.380
							3/16	107	161	0.285
11	131	197	132	198		32 1/2	5/16	165	247	0.475
							1/4	132	198	0.380
							3/16	98.8	148	0.285
10	119	179	120	180		29 1/2	5/16	151	226	0.475
							1/4	121	181	0.380
							3/16	90.4	136	0.285
9	107	161	108	162		26 1/2	5/16	137	205	0.475
							1/4	110	164	0.380
							3/16	82.2	123	0.285
8	95.5	143	95.6	143		23 1/2	5/16	123	185	0.475
							1/4	98.5	148	0.380
							3/16	73.9	111	0.285
7	83.5	125	83.4	125		20 1/2	5/16	109	164	0.475
							1/4	87.4	131	0.380
							3/16	65.6	98.4	0.285

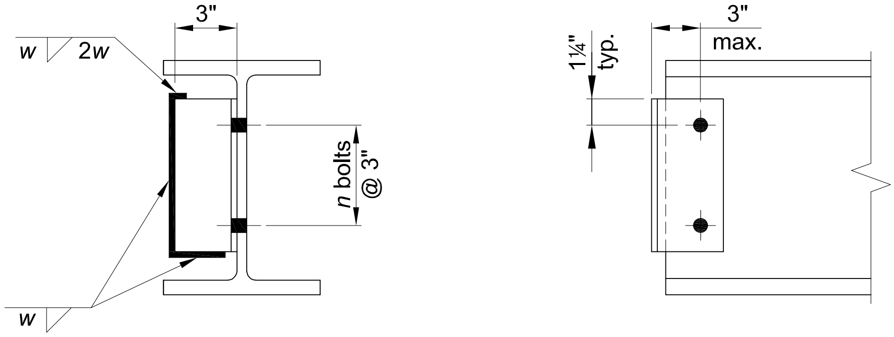
Notes:

Gage in angle leg attached to beam web as well as leg width may be decreased. 3-in. welded leg may not be increased or decreased.

Tabulated weld available strengths are based on a 1/4-in. half web for the supported member. Smaller half webs will result in these values being conservative. For half webs over 1/4 in., weld values must be reduced proportionally by an amount up to 8% for a 1/2-in. half web or recalculated.

When the beam web thickness of the supporting member is less than the minimum and single-angle connections are back to back, either stagger the angles, or multiply the weld design strength by the ratio of the actual web thickness to the tabulated minimum thickness to determine the reduced weld design strength.

Table 10-12 (continued)
Bolted/Welded Single-Angle Connections



Number of Bolts in One Vertical Row	Bolt and Angle Strength, kips Group A Bolts				Angle Size (F_y = 36 ksi)	Angle Length, in.	Weld (70 ksi)			Minimum t_w of Supporting Member with Angles Both Sides of Web, in.
	3/4 in.		7/8 in.				Size, w, in.	Available Strength, kips		
	ASD	LRFD	ASD	LRFD				ASD	LRFD	
6	71.6	107	71.3	107	L4×3×3/8	17 1/2	5/16	94.3	141	0.475
							1/4	75.5	113	0.380
							3/16	56.6	84.9	0.285
5	59.7	89.5	59.1	88.7		14 1/2	5/16	79.1	119	0.475
							1/4	63.3	94.9	0.380
							3/16	47.4	71.2	0.285
4	47.6	71.4	47.0	70.4		11 1/2	5/16	62.9	94.4	0.475
							1/4	50.3	75.5	0.380
							3/16	37.8	56.6	0.285
3	35.5	53.2	34.8	52.2		8 1/2	5/16	45.7	68.5	0.475
							1/4	36.6	54.8	0.380
							3/16	27.4	41.1	0.285
2	23.3	35.0	22.7	34.0		5 1/2	5/16	28.2	42.2	0.475
							1/4	22.5	33.8	0.380
							3/16	16.9	25.3	0.285

Notes:

Gage in angle leg attached to beam web as well as leg width may be decreased. 3-in. welded leg may not be increased or decreased.

Tabulated weld available strengths are based on a 1/4-in. half web for the supported member. Smaller half webs will result in these values being conservative. For half webs over 1/4 in., weld values must be reduced proportionally by an amount up to 8% for a 1/2-in. half web or recalculated.

When the beam web thickness of the supporting member is less than the minimum and single-angle connections are back to back, either stagger the angles, or multiply the weld design strength by the ratio of the actual web thickness to the tabulated minimum thickness to determine the reduced weld design strength.

TEE CONNECTIONS

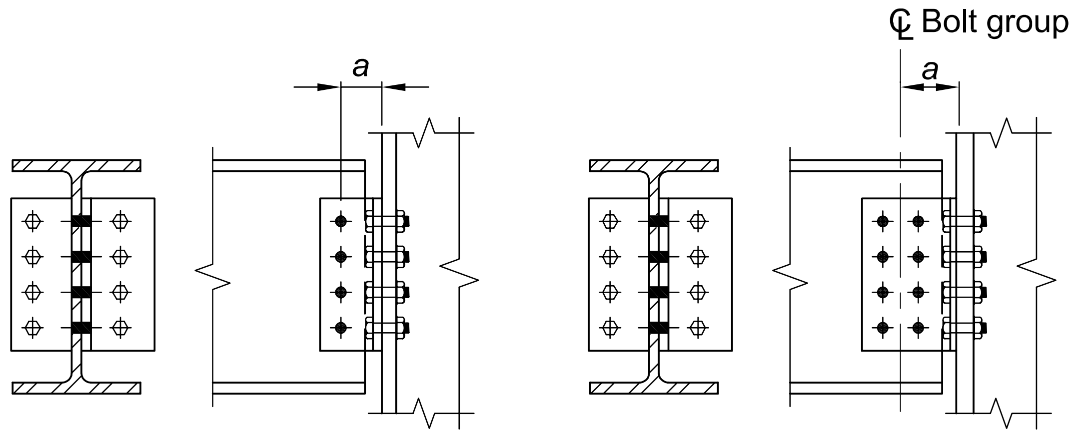
A tee connection is made with a structural tee, as illustrated in Figure 10-15. The tee is preferably shop-bolted or welded to the supporting member and field-bolted to the supported beam.

When the tee is welded to the support, adequate flexibility must be provided in the connection. As illustrated in Figure 10-15(b), line welds are placed along the toes of the tee flange with a return at the top per AISC *Specification* Section J2.2b. Note that welding across the entire top of the tee must be avoided as it would inhibit the flexibility and, therefore, the necessary end rotation of the connection. The performance of the resulting connection would not be as intended for simple shear connections.

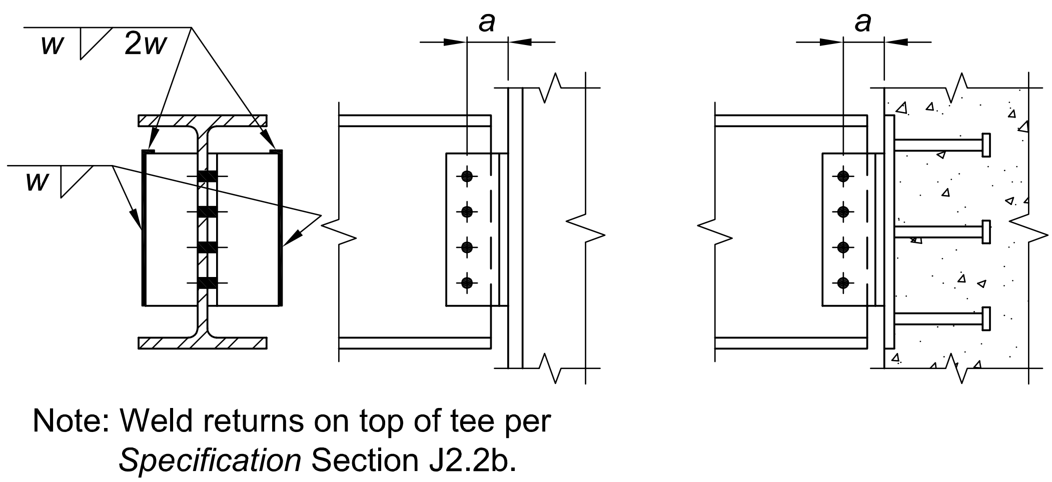
Design Checks

The available strength of a tee connection is determined from the applicable limit states for bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a.

Eccentricity must be considered when determining the available strength of tee connections. For a flexible support, the bolts or welds attaching the tee flange to the support



(a) All-bolted



(b) Bolted/welded

Fig. 10-15. Tee connections.

must be designed for the shear, R_u or R_a. Also, the bolts through the tee stem must be designed for the shear and the eccentric moment, $R_u a$ or $R_a a$, where a is the distance from the face of the support to the centroid of the bolt group through the tee stem.

For a rigid support, the bolts or welds attaching the tee flange to the support must be designed for the shear and the eccentric moment; the bolts through the tee stem must be designed for the shear.

Recommended Tee Length and Flange and Web Thicknesses

To provide for stability during erection, it is recommended that the mimimum tee length be one-half the T-dimension of the beam to be supported. The maximum length of the tee must be compatible with the T-dimension of an uncoped beam and the remaining web depth, exclusive of fillets, of a coped beam. Note that the tee may encroach upon the fillet(s) as given in Figure 10-3.

To provide for flexibility, the tee selected should meet the ductility checks illustrated in Part 9. The flange thickness of tees used in simple shear connections should be held to a minimum to permit the flexure necessary to accommodate the end rotation of the beam, unless the tee stem connection is proportioned to meet the geometric requirements for single-plate connections.

Shop and Field Practices

When framing to a column flange, provision must be made for possible mill variation in the depth of the columns. If the tee is shop-attached to the column flange, play in the open holes usually furnishes the necessary adjustment to compensate for the mill variation. This approach offers the advantage of side erection of the beam. Alternatively, if the tee is shop-attached to the supported beam web, the beam length could be shortened to provide for mill overrun and shims could be furnished at the appropriate intervals to fill the resulting gaps or to provide for mill underrun.

When a single vertical row of bolts is used in a tee stem, a 4-in. or 5-in. stem is required to accommodate the end distance of the supported beam and possible overrun/underrun in beam length. A double vertical row of bolts will require a 7-in. or 8-in. tee stem. There is no maximum limit on l_{eh} for the tee stem.

SHEAR SPLICES

Shear splices are usually made with a single plate, as shown in Figure 10-16(a), or two plates, as shown in Figures 10-16(b) and 10-16(c). Although the rotational flexibility required at a shear splice is usually much less than that required at the end of a simple-span beam, when a highly flexible splice is desired, the splice utilizing four framing angles, shown in Figure 10-17, is especially useful. These shear splices may be bolted and/or welded.

The available strength of a shear splice is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a.

Eccentricity must be considered in the design of shear splices, with the exception of all-bolted shear splices utilizing four framing angles, as illustrated in Figure 10-17. When the splice is symmetrical, as shown for the bolted splice in Figure 10-16(a), each side of the splice is equally restrained regardless of the relative flexibility of the spliced members.

Accordingly, as illustrated in Figure 10-18, the eccentricity of the shear to the center of gravity of either bolt group is equal to half the distance between the centroids of the bolt groups. Therefore, each bolt group can be designed for the shear, R_u or R_a, and one-half the eccentric moment, $R_u e$ or $R_a e$ (Kulak and Green, 1990). This approach is also applicable to symmetrical welded splices.

When the splice is not symmetrical, as shown in Figures 10-16(b) and 10-16(c), one side of the splice will possess a higher degree of rigidity. For the splice shown in Figure 10-16(b), the right side is more rigid because the stiffness of the weld group exceeds the stiffness of the bolt group, even if the bolts are pretensioned or slip-critical. Also, for the splice shown in

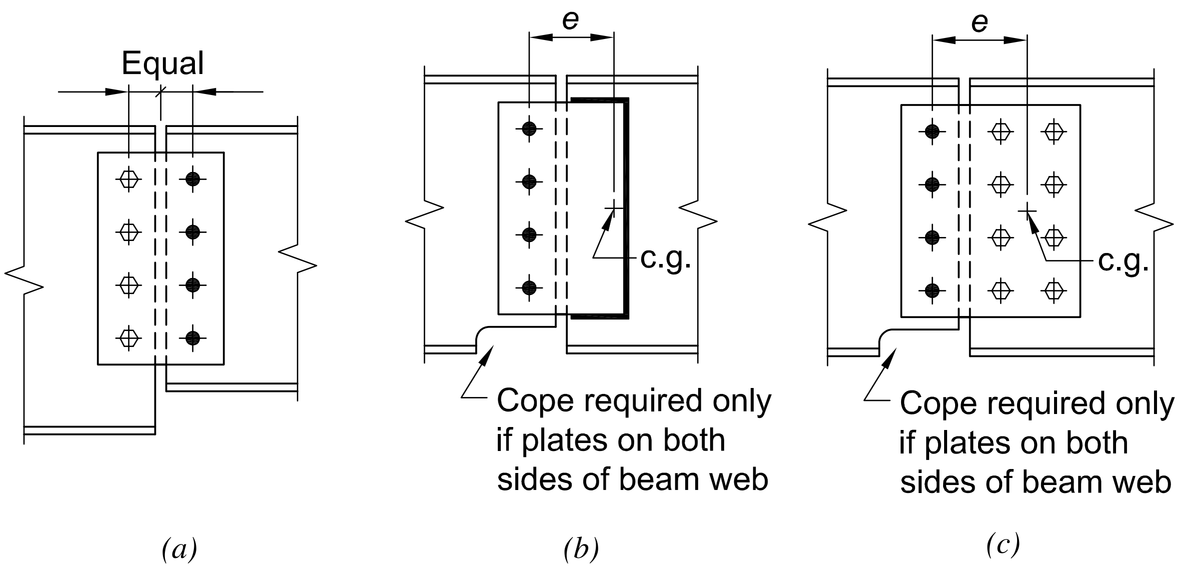


Fig. 10-16. Plate-type shear splices.

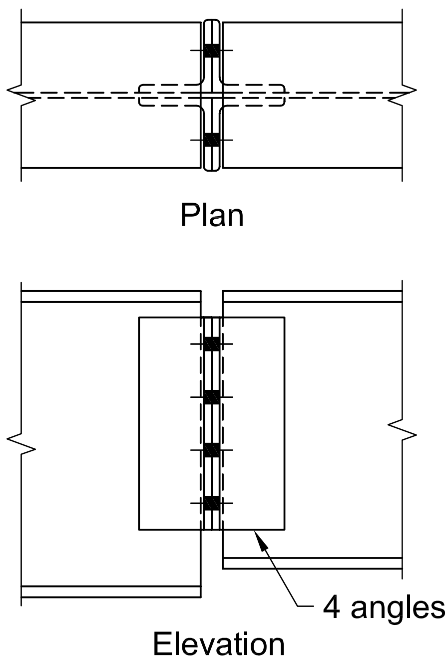


Fig. 10-17. Angle-type shear splice.

Figure 10-16(c), the right side is more rigid since there are two vertical rows of bolts while the left side has only one. In these cases, it is conservative to design the side with the higher rigidity for the shear, R_u or R_a, and the full eccentric moment, R_ue or R_ae. The side with the lower rigidity can then be designed for the shear only. This approach is applicable regardless of the relative flexibility of the spliced members.

Some splices, such as those that occur at expansion joints, require special attention and are beyond the scope of this Manual.

SPECIAL CONSIDERATIONS FOR SIMPLE SHEAR CONNECTIONS

Simple Shear Connections Subject to Axial Forces

When simple shear connections are subjected to axial loading in addition to shear, additional limit states and connection behavior must be evaluated to provide proper performance of the connections. Additional applicable limit states and performance criteria may include prying action and plate/outstanding leg angle bending for end-plate and double-angle connections, which may require the plate or angle thickness to increase or gage to decrease (or both). These strength requirements may compromise the ability of the connection to remain flexible enough to accommodate the simple beam end rotation. The shear connection rotational ductility checks derived in Part 9 can be used to ensure that adequate ductility exists. There are also interaction checks required due to the orthogonal loading in the connection that must be evaluated in addition to the individual shear and axial loading limit states.

The AISC *Design Examples* companion to the Manual provides several connection design examples subject to axial loading in addition to shear. Double-angle knife connections, knife-plate connections, or single-angle connections are not well suited for axial loads in tension.

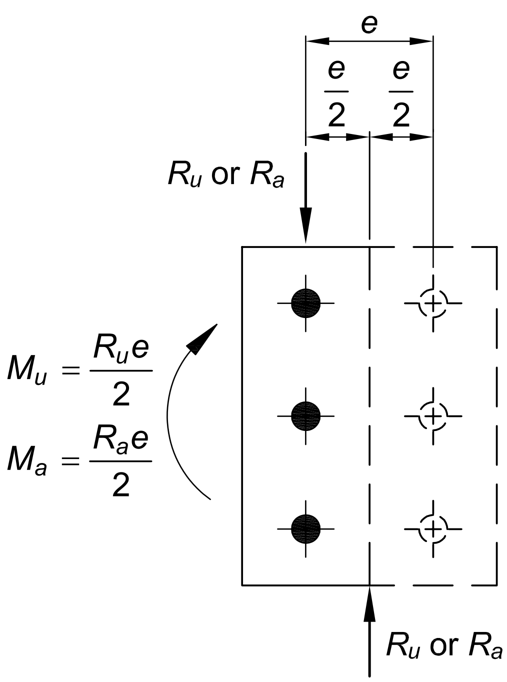


Fig. 10-18. Eccentricity in a symmetrical shear splice.

Simple Shear Connections at Stiffened Column-Web Locations

Stiffeners are obstacles for direct connections to the column web. Figure 10-19 illustrates three examples of various means for connections. Figure 10-19(a) illustrates a plate extended beyond the column flanges with a shear tab welded to the plate for the connection. Figure 10-19(b) illustrates a seat angle extended beyond the column flange. Figure 10-19(c) is an extended shear tab and is also used quite frequently.

When applying a beam end reaction to the column flange toes, eccentricity must still be considered to the column centerline to avoid introducing weak-axis bending to the column. The eccentricity can be taken at the column flange toes if the column has been designed for the weak-axis bending due to the beam end reaction or the weak-axis bending applied to the column is less than 5% of the weak-axis available flexural strength of the column.

Eccentric Effect of Extended Gages

Consider a simple shear connection to the web of a column that requires transverse stiffeners for two concurrent beam-to-column-flange moment connections. If it were not possible to eliminate the stiffeners by selection of a heavier column section, the field connection would have to be located clear of the column flanges, as shown in Figure 10-20, to provide for access and erectability.

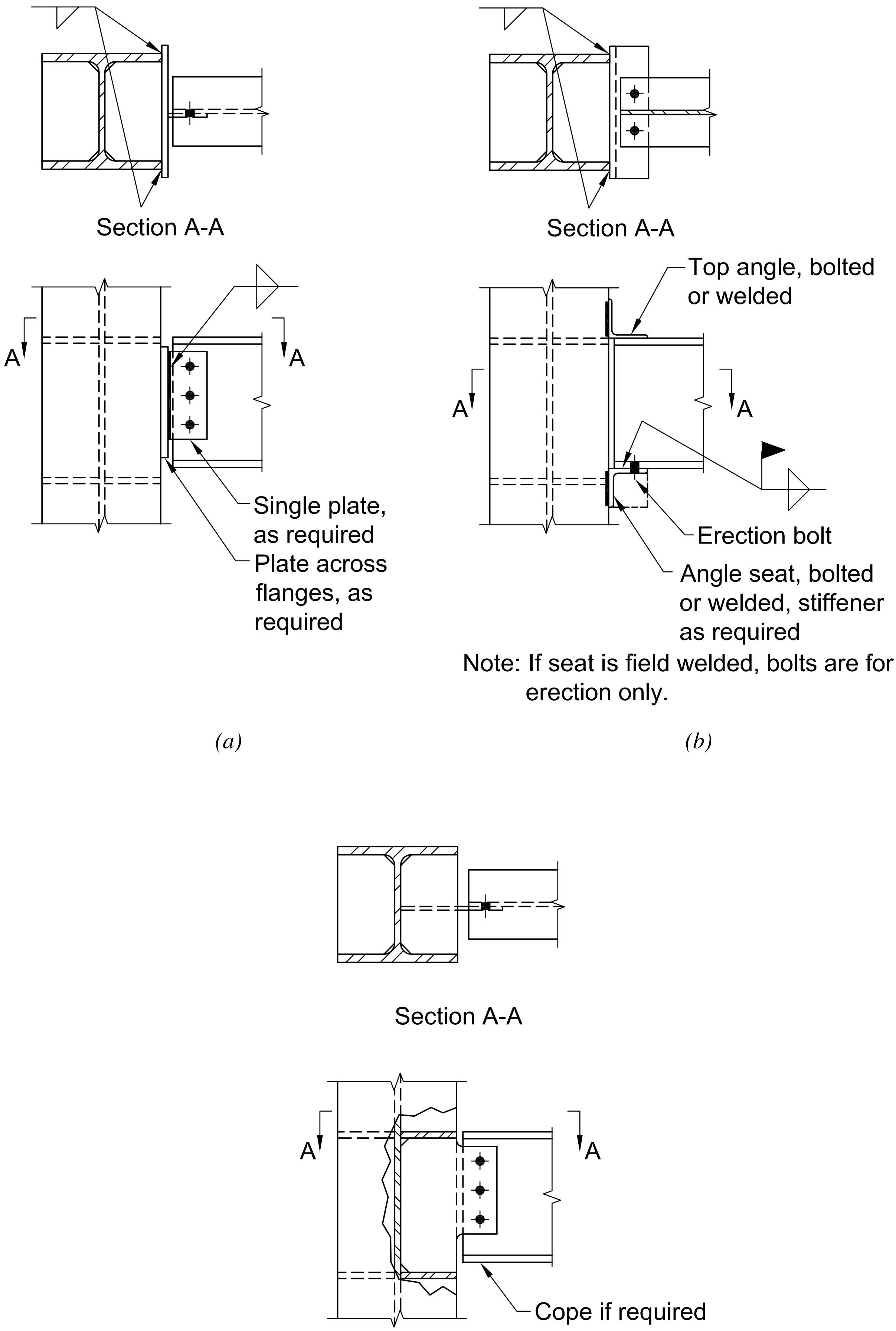
The extension of the connection beyond normal gage lines results in an eccentric moment. While this eccentric moment is usually neglected in a connection framing to a column flange, the resistance of the column to weak-axis bending is typically only 20% to 50% of that in the strong axis. Thus the eccentric moment should be considered in this column-web connection, especially if the eccentricity, e, is large. Similarly, eccentricities larger than normal gages may also be a concern in connections to girder webs.

Column-Web Supports

There are two components contributing to the total eccentric moment: (1) the eccentricity of the beam end reaction, Re; and (2) M_{pr}, the partial restraint of the connection. To determine what eccentric moment must be considered in the design, first assume that the column is part of a braced frame for weak-axis bending, is pinned-ended with $K = 1$, and will be concentrically loaded, as illustrated in Figure 10-21. The beam is loaded before the column and will deflect under load as shown in Figure 10-22. Because of the partial restraint of the connection, a couple, M_{pr}, develops between the beam and column and adds to the eccentric couple, Re. Thus, $M_{con} = Re + M_{pr}$.

As the loading of the column begins, the assembly will deflect further in the same direction under load, as indicated in Figure 10-23, until the column load reaches some magnitude, P_{sbr}, when the rotation of the column will equal the simply supported beam end rotation. At this load, the rotation of the column negates M_{pr} since it also relieves the partial restraint effect of the connection, and $M_{con} = Re$. As the column load is increased above P_{sbr}, the column rotation exceeds the simply supported beam end rotation and a moment M'_{pr} results such that $M_{con} = Re - M'_{pr}$.

Note that the partial restraint of the connection now actually stabilizes the column and reduces its effective length factor, K, below the originally assumed value of 1. Thus, since M'_{pr} must be greater than zero, it must also be true that $Re > M_{con}$. It is therefore conservative to design the connection for the shear, R, and the eccentric moment, Re.



(a) *(b)*

(c)

Fig. 10-19. Simple shear connections at stiffened column-web locations.

The welds connecting the plate to the supporting column web should be designed to resist the full shear, R, only; the top and bottom plate-to-stiffener welds have minimal strength normal to their length, are not assumed to carry any calculated force, and may be of minimum size in accordance with AISC *Specification* Section J2.

If simple shear connections frame to both sides of the column web, as illustrated in Figure 10-21, each connection should be designed for its respective shear, R_1 and R_2, and the eccentric moment $|R_2e_2 - R_1e_1|$ may be apportioned between the two simple shear connections as the designer sees fit. The total eccentric moment may be assumed to act on the larger connection, the moment may be divided proportionally among the connections according to the polar moments of inertia of the bolt groups (relative stiffness), or the moment may be divided proportionally between the connections according to the section moduli of the bolt groups (relative moment strength). If provision is made for ductility and

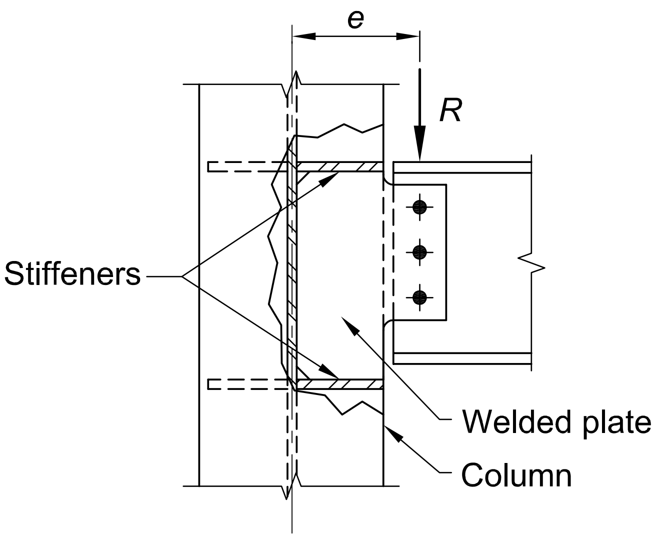


Fig. 10-20. Eccentric effect of extended gages.

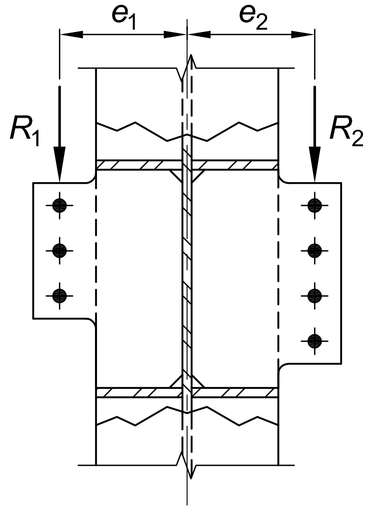


Fig. 10-21. Column subject to dual eccentric moments.

stability, it follows from the lower bound theorem of limit states analysis that the distribution which yields the greatest strength is closest to the true strength. Note that the possibility exists that one of the beams may be devoid of live load at the same time that the opposite beam is fully loaded. This condition must be considered by the designer when apportioning the moment.

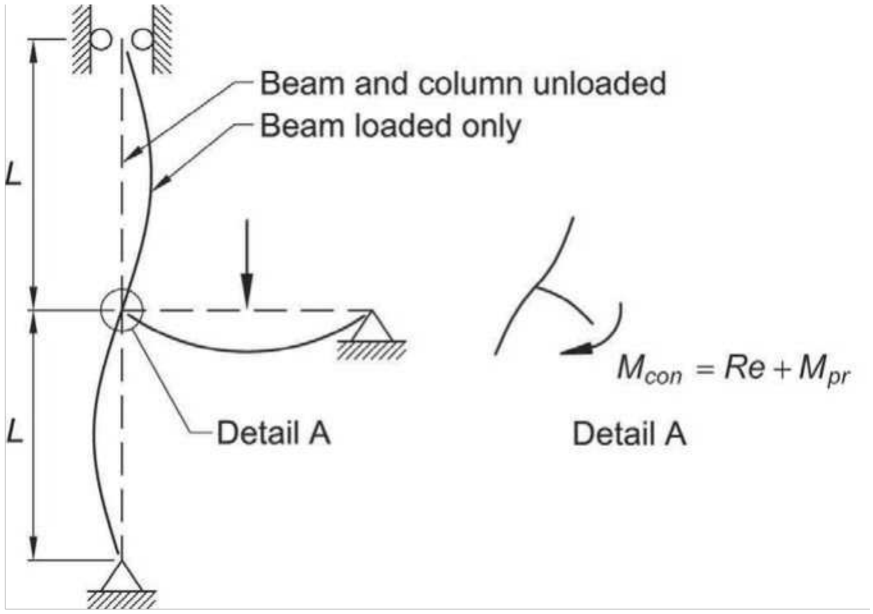


Fig. 10-22. Illustration of beam, column and connection behavior under loading of beam only.

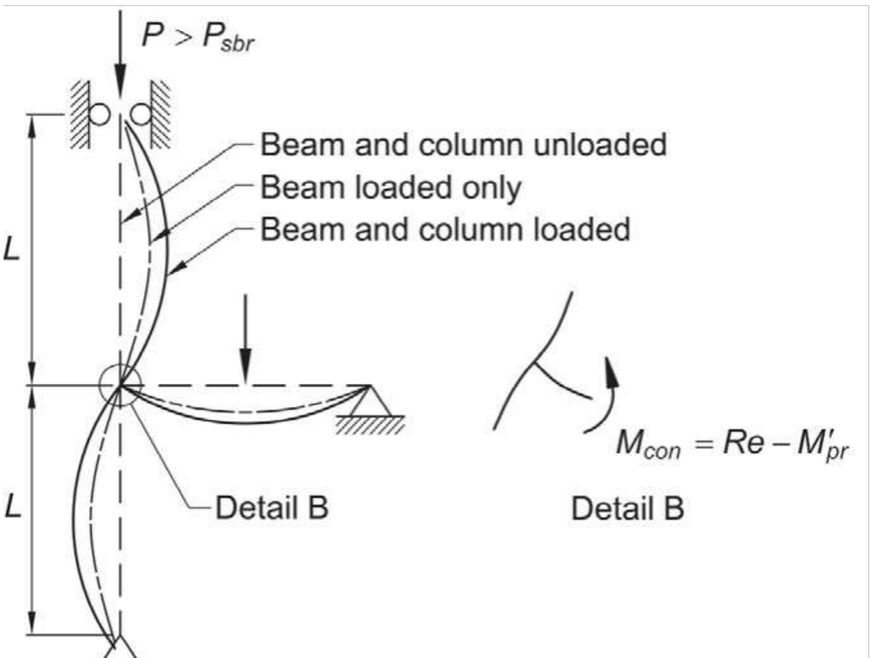


Fig. 10-23. Illustration of beam, column and connection behavior under loading of beam and column.

Girder-Web Supports

The girder-web support of Figure 10-24 usually provides only minimal torsional stiffness or strength. When larger-than-normal gages are used, the end rotation of the supported beam will usually be accommodated through rotation of the girder support. It follows that the bolt group should be designed to resist both the shear, R, and the eccentric moment, Re. The beam end reaction will then be carried through to the center of the supporting girder web.

The welds connecting the plate to the supporting girder web should be designed to resist the shear, R, only; the top and bottom plate-to-girder-flange welds have minimal strength normal to their length, are not assumed to carry any calculated force, and may be of minimum size in accordance with AISC *Specification* Section J2.

Similarly, for the girder illustrated in Figure 10-25 supporting two eccentric reactions, each connection should be designed for its respective shear, R_1 and R_2, and the eccentric moment, $|R_2e_2 - R_1e_1|$, may be apportioned between the two simple shear connections as the designer sees fit.

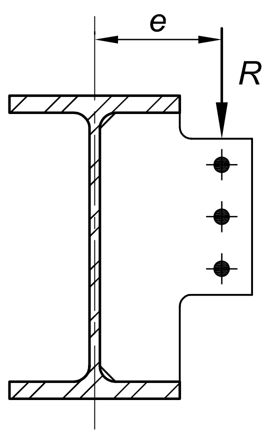


Fig. 10-24. Eccentric moment on girder-web support.

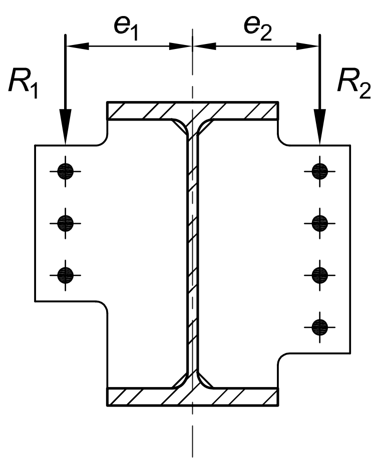


Fig. 10-25. Girder-web support subject to dual eccentric moments.

Alternative Treatment of Eccentric Moment

In the foregoing treatment of eccentric moments with column- and girder-web supports, it is possible to design the support (instead of the connection) for the eccentric moment, *Re*. The engineer of record may choose to use a rational approach based on engineering judgment and taking into consideration member strength and stiffness, composite slab interaction, and alternate load paths to resist the eccentric moment, *Re*. In these cases, the connection may be designed for the shear, *R,* only or the shear and a reduced eccentric moment.

Double Connections

When beams frame opposite each other and are welded to the web of the supporting girder or column, there are usually no dimensional constraints imposed on one connection by the presence of the other connection unless erection bolts are common to each connection. When the connections are bolted to the web of the supporting column or girder, however, the close proximity of the connections requires that some or all fasteners be common to both connections. This is known as a double connection. See also the discussion under "Constructability Considerations" in an earlier section in this Part.

Supported Beams of Different Nominal Depths

When beams of different nominal depths frame into a double connection, care must be taken to avoid interference from the bottom flange of the shallower beam with the entering and tightening clearances for the bolts of the connection for the deeper beam. Access to the bolts that will support the deeper beam may be provided by coping or blocking the bottom flange of the shallower beam. Alternatively, stagger may be used to favorably position the bolts around the bottom flange of the shallower beam.

Supported Beams Offset Laterally

Frequently, beams do not frame exactly opposite each other, but are offset slightly, as illustrated in Figure 10-26. Several connection configurations are possible, depending on the offset dimension.

If the offset were equal to the gage on the support, the connection could be designed with all bolts on the same gage lines, as shown in Figure 10-26(b), and the angles arranged, as shown in Figure 10-26(d). If the offset were less than the gage on the support, staggering the bolts, as shown in Figure 10-26(c), would reduce the required gage and the angles could be arranged, as shown in Figure 10-26(c). In any case, each bolt transmits an equal share of its beam reaction(s) to the supporting member, with the bolts that are loaded in double shear ultimately carrying twice as much force as those loaded in single shear. Once the geometry of the connection has been determined, the distribution of the forces is patterned after that in the design of a typical connection. For normal gages, eccentricity may be ignored in this type of connection.

Beams Offset From Column Centerline

Framing to the Column Flange from the Strong Axis

As illustrated in Figure 10-27, beam-to-column-flange connections offset from the column centerline may be supported on a typical welded seat, stiffened or unstiffened, provided the welds for the seat can be spaced approximately equal on either side of the beam centerline.

Two such seats offset from the W12×65 column centerline by 2¼ in. and 3½ in. are shown in Figures 10-27(a) and 10-27(b), respectively. While not shown, top angles should be used with this connection.

Since the entire seat fits within the flange width of the column, the connection of Figure 10-27(a) is readily selected from the design aids presented previously. However, the larger beam offsets in Figures 10-27(b) and 10-27(c) require that one of the welds be made along the edge of the column flange against the back side of the seat angle. Note that the end return is omitted because weld returns should not be carried around such a corner.

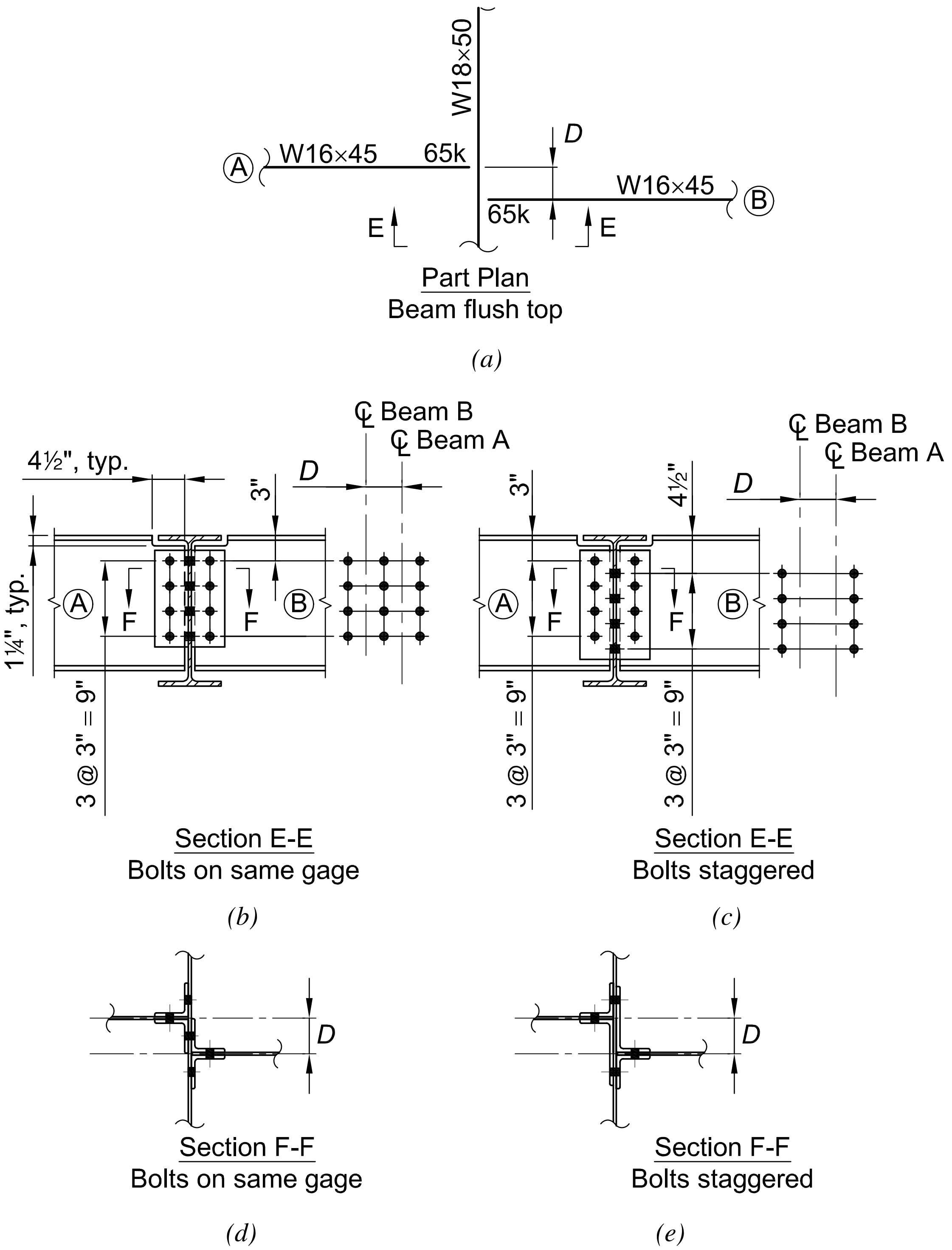
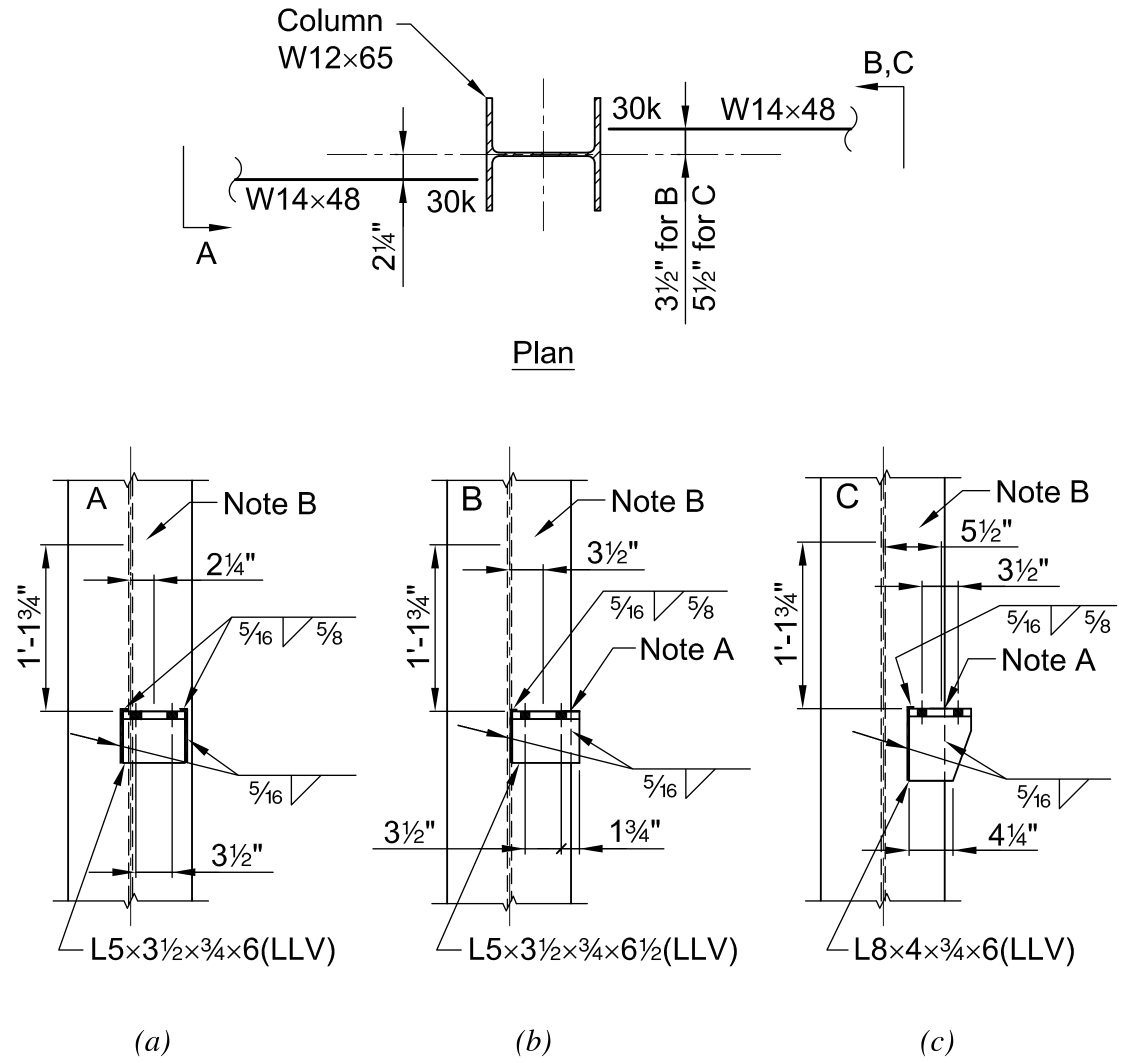


Fig. 10-26. Offset beams connected to girder.

For the beam offset of $5^1/_2$ in. shown in Figure 10-27(c), the seat angle overhangs the edge of the beam and the horizontal distance between the vertical welds is reduced to $3^1/_2$ in.; the center of gravity of the weld group is located $1^1/_4$ in. to the left of the beam centerline. The force on each weld may be determined by statics. In this case, the larger force is in the right-hand weld and may be determined by summing moments about the lefthand weld. Once the larger force has been determined, each weld should be designed to share the force in the more highly loaded weld.



Note A: End return is omitted because the AWS Code does not permit weld returns to be carried around the corner formed by the column flange toe and seat angle heel.

Note B: Beam and top angle not shown for clarity.

Fig. 10-27. Offset beams connected to column flanges.

Framing to the Column Flange from the Weak Axis

Spandrel beams X and Y in the partial plan shown in Figure 10-28 are offset $4^1/_8$ in. from the centerline of column C1, permitting the beam web to be connected directly to the column flange. At column B2, spandrel beam X is offset $4^5/_8$ in. and requires a $^1/_2$-in. filler between the beam web and the column flange. Beams X and Y are both plain-punched beams, with flanges coped top and bottom, as noted in Figure 10-28(a), Section F-F.

In establishing gages, the requirements of other connections to the column at adjacent locations must be considered. The workable flange gage is 4 in. for the W8×28 columns supporting the spandrel beams, for beams Z, the combination of a 4-in. column gage and $1^1/_2$-in. stagger of fasteners is used to provide entering and tightening clearance for the field bolts and sufficient edge distance on the column flange, as illustrated in Figure 10-28(b). The 4-in. column gage also permits a $1^1/_2$-in. edge distance at the ends of the spandrel beams, which will accommodate the normal length tolerance of $\pm^1/_4$ in. as specified in "Standard Mill Practice" in Part 1.

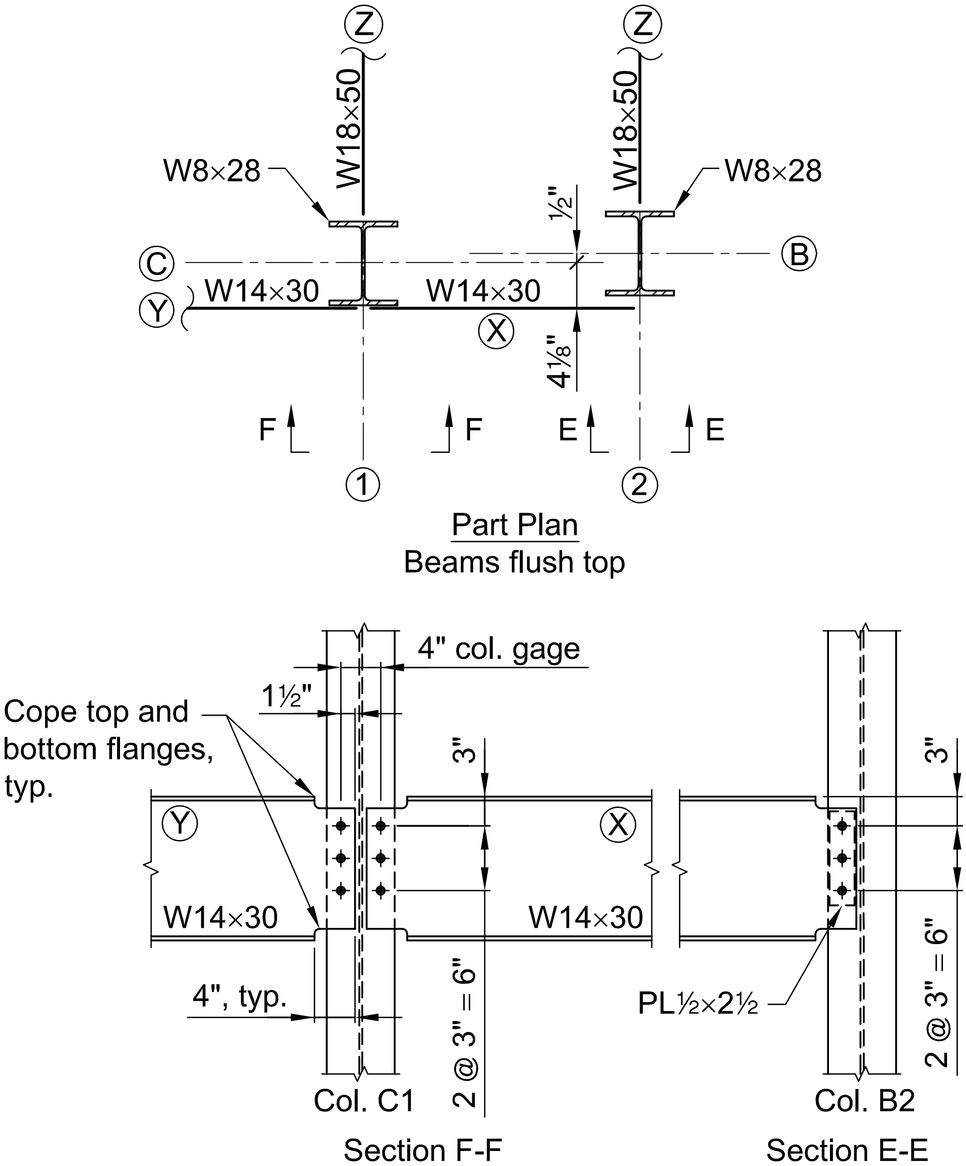
The notation, "Cope top and bottom flanges," is applicable to the spandrel beams shown in Sections E-E and F-F. The copes permit the beam web to lie flush against the column flange. The $2^1/_2\times^1/_2$-in. filler is required between the spandrel beam web and the flange of column B2 because of the $^1/_2$-in. offset. Accordingly, the filler provisions of AISC *Specification* Section J5 must be satisfied.

In the part plan in Figure 10-29(a), the W16×40 beam is offset $6^1/_4$ in. from the centerline of column D1. This prevents the web of the W16×40 from being placed flush against the side of the column flange. A plate and filler are used to connect the beam to the column flange, as shown in Figure 10-29(b). Such a connection is eccentric and one group of fasteners must be designed for the eccentricity. Lack of space on the inner flange face of the column requires development of the moment induced by the eccentricity in the beam web fasteners.

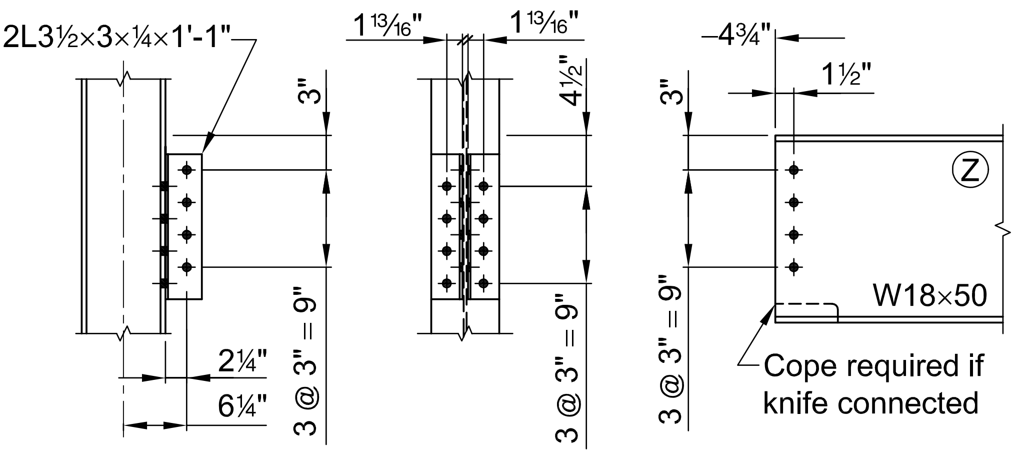
To minimize the number of field fasteners, the plate in this case is shop-bolted to the beam and field-bolted to the column. A careful check must be made to ensure that the beam can be erected without interference from fittings on the column web. Some fabricators would elect to shop-attach the plate to the column to eliminate possible interference and permit use of plain-punched beams. Additionally, if the column were a heavy section, the fabricator may elect to shop-weld the plate to the column to avoid drilling the thick flanges. The welding of this plate to the column creates a much stiffer connection and the design should be modified to recognize the increased rigidity.

If the centerline of the W16 were offset $6^1/_{16}$ in. from line 1, it would be possible to cope or cut the flanges flush top and bottom and frame the web directly to the column flange with details similar to those shown in Figure 10-29. This type of framing also provides a connection with more rigidity than normally contemplated in simple construction. A coped connection of this type would create a bending moment at the root of the cope that might require reinforcement of the beam web.

One method frequently adopted to avoid moment transfer to the column because of beam connection rigidity is to use slotted holes and a bearing connection to provide some flexibility. The slotted holes would be provided in the connection plate only and would be in the field connection only. These slotted connections also would accommodate fabrication and erection tolerances.

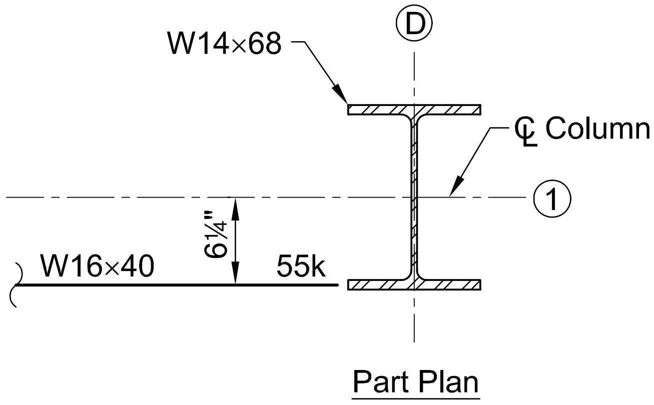


(a)

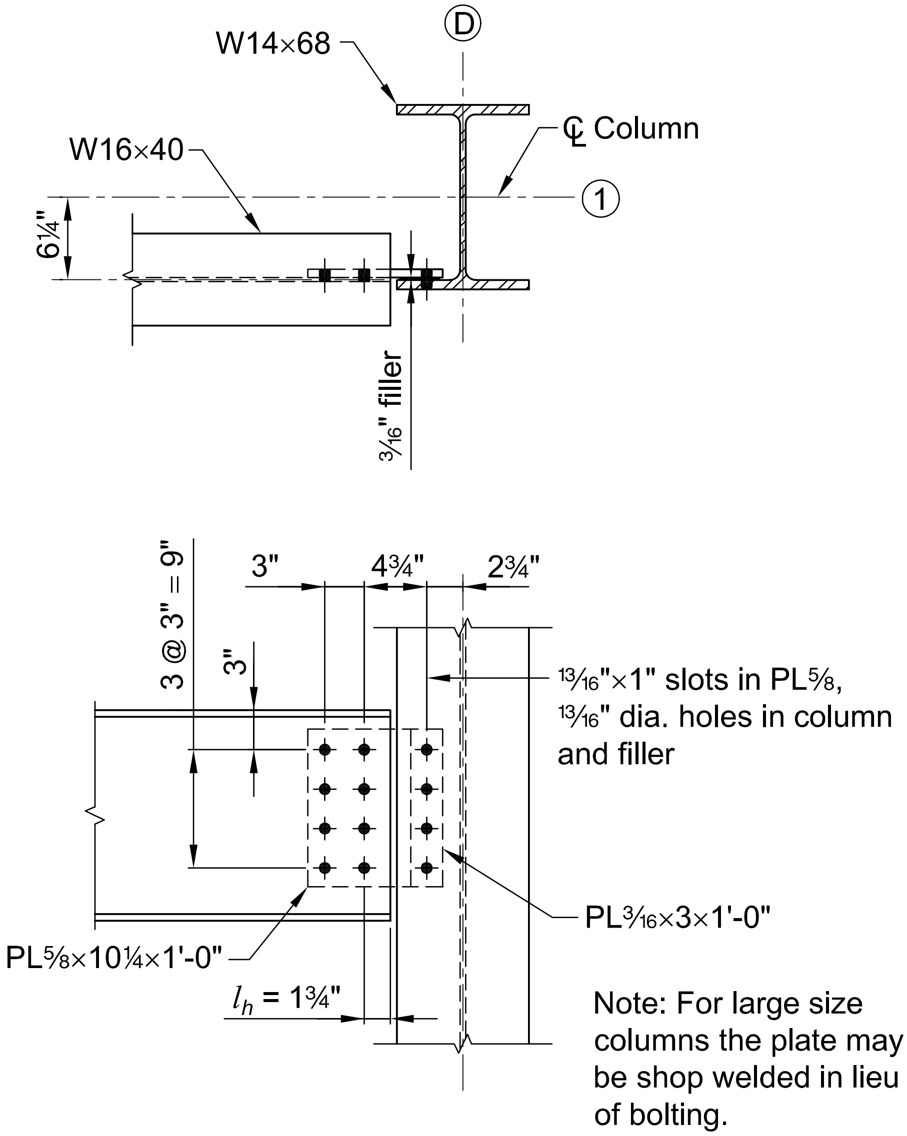


(b)

Fig. 10-28. Offset beams connected to column.



(a)



(b)

Fig. 10-29. Offset beam connected to column.

The type of connection detailed in Figure 10-29 is similar to a coped beam and should be checked for buckling, as illustrated in Part 9. The following differences are apparent and should be recognized in the analysis:

1. The effective length of equivalent “cope” is longer by the amount of end distance to the first bolt gage line.
2. There is an inherent eccentricity due to the beam web and plate thickness. The ordinary web and plate thicknesses normally will not require an analysis for this condition, since the inelastic rotation allowed by the AISC *Specification* will relieve this secondary moment effect. Two plates may sometimes be required to counter this eccentricity when dimensions are significant.
3. The connection plate can be made of sufficient thickness as required for bending or buckling stresses with a minimum thickness of 3/8 in.

Framing to the Column Web

If the offset of the beam from the centerline of the column web is small enough that the connection may still be centered on or under the supported beam, no special considerations need be made. However, when the offset of the beam is too large to permit the centering of the connection under the beam, as in Figure 10-30, it may be necessary to consider the effect of eccentricity in the fastener group.

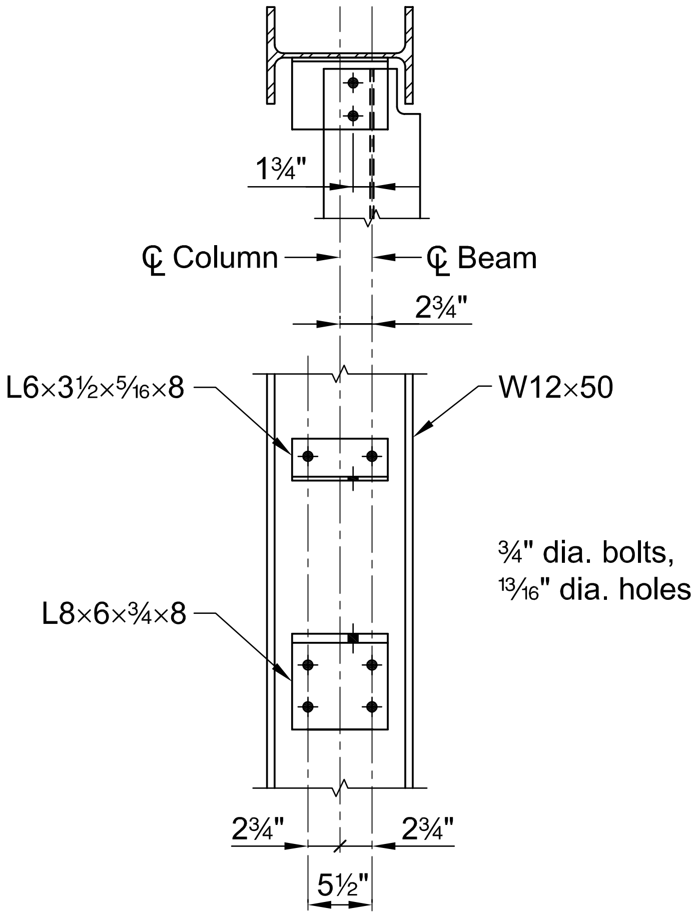


Fig. 10-30. Offset beam connected to column web.

The offset of the beam in Figure 10-30 requires that the top and bottom flanges be blocked to provide erection clearance at the column flange. Since only half of each flange, then, remains in which to punch holes, a 6-in. outstanding leg is used for both the seat and top angles of these connections; this permits the use of two field bolts to each of the seat and top angles, which are required by OSHA.

Connections for Raised Beams

When raised beams are connected to column flanges or webs, there is usually no special consideration required. However, when the support is a girder, the differing tops of steel may preclude the use of typical connections. Figure 10-31 shows several typical details commonly used for such cases in bolted construction. Figure 10-32 shows several typical details commonly used in welded construction.

In Figure 10-31(a), since the top of the W12×35 is located somewhat less than 12 in. above the top of the W18 supporting beam, a double-angle connection is used. This connection would be designed for the beam reaction and the shop bolts would be governed by double shear or bearing, just as if they were located in a vertical position. However, the field bolts are not required to carry any calculated force under gravity loading.

The maximum permissible distance, *m*, depends on the beam reaction, since the web remaining after the bottom cope must provide sufficient area to resist the vertical shear as well as the bending moment which would be critical at the end of the cope. The beam can be reinforced by extending the angles beyond the cope and adding additional shop bolts for development. The angle size and/or thickness can be increased to gain shear area or section modulus, if required. The effect of any eccentricity would be a matter of judgment, but could be neglected for small dimensions.

When this connection is used for flexure or for dynamic or cyclical loading, the web is subjected to high stress concentrations at the end of the cope, and it is good practice to extend the angles, as shown in Figure 10-31(a), to add at least two additional web fasteners.

Figure 10-31(b) covers the case where the bottom flange of the W12×35 is located a few inches above the top of the W18. The beam bears directly upon fillers and is connected to the W18 by four field bolts which are not required to transmit a calculated gravity load. If the distance *m* exceeds the thickest plate which can be punched, two or more plates may be used. Even though the fillers in this case need only be $6^1/_2$-in. square, the amount of material required increases rapidly as *m* increases. If *m* exceeds 2 or 3 in., another type of detail may be more economical.

The detail shown in Figure 10-31(c) is used frequently when *m* is up to 6 or 7 in. The load on the shop bolts in this case is no greater than that in Figure 10-31(a). However, to provide more lateral stiffness, the fittings are cut from a 15-in. channel and are detailed to overlap the beam web sufficiently to permit four shop bolts on two gage lines.

A stool or pedestal, cut from a rolled shape, can be used with or without fillers to provide for the necessary *m* distance, as in Figure 10-31(d). A pair of connection angles and a tee will also serve a similar purpose, as shown in Figure 10-31(e). To provide adequate strength to carry the beam end reaction and to provide lateral stiffness, the web thickness of the pedestal in each of these cases should be at least as thick as the member being supported.

In Figure 10-32(a), welded framing angles are substituted for the bolted angles of Figure 10-31(a). In Figure 10-32(b), a single horizontal plate is shown replacing the pair of framing angles; this results in a savings in material and the amount of shop-welding. In this case, particular care must be taken in cutting the beam web and positioning the plate at right

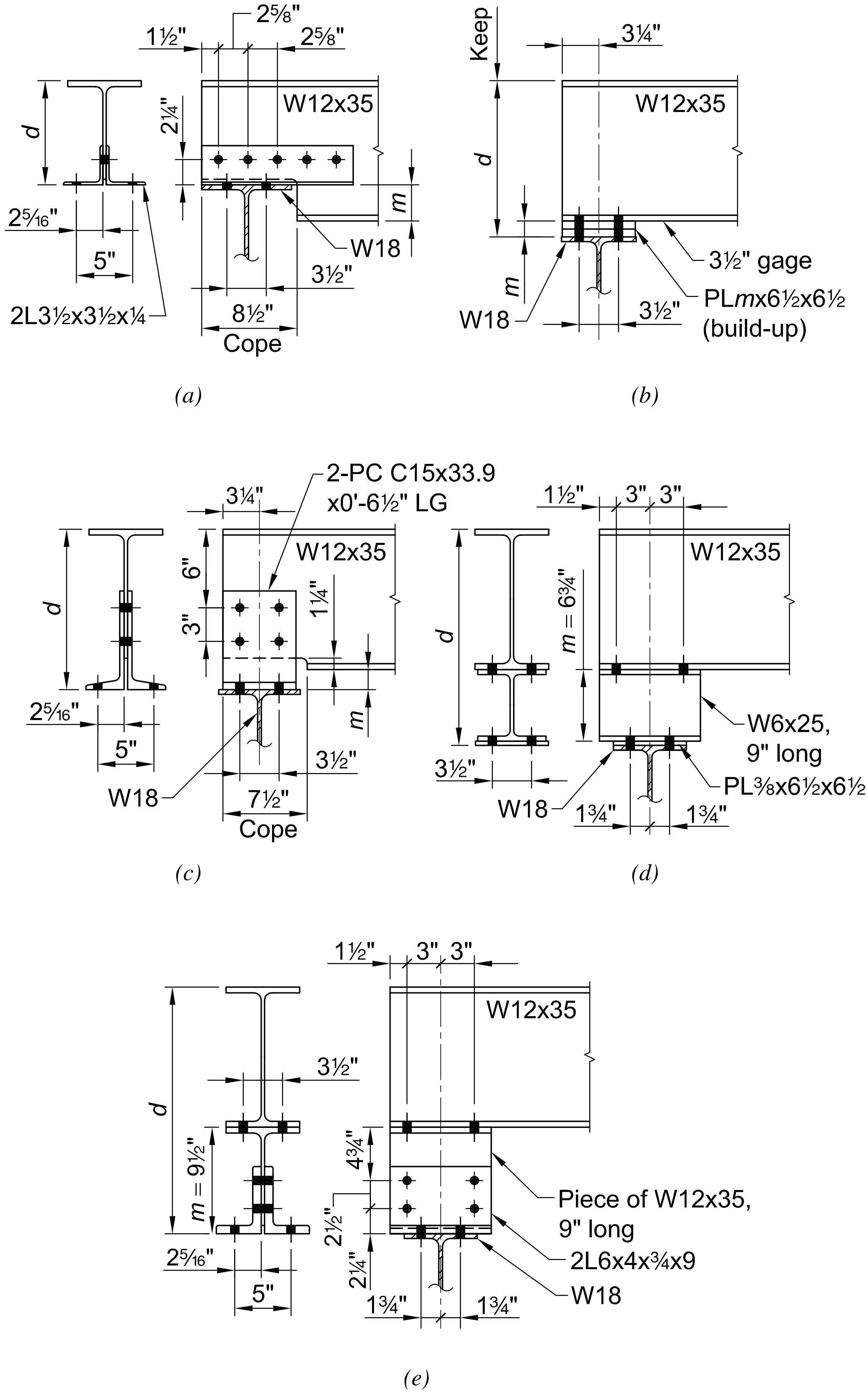


Fig. 10-31. Bolted raised-beam connections.

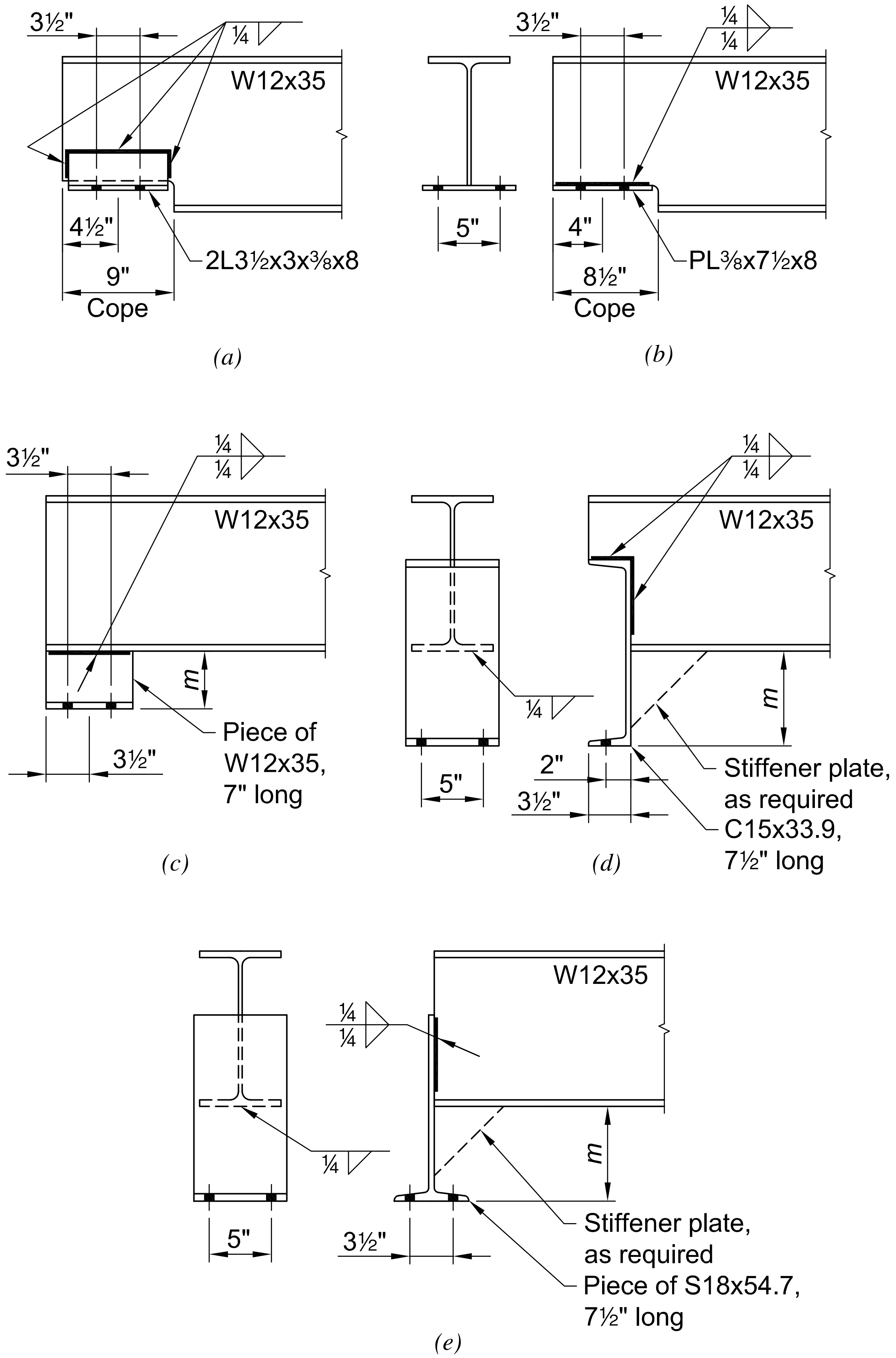


Fig. 10-32. Welded raised-beam connections.

angles to the beam web. For this reason, if only a few connections of this type are to be made, some fabricators prefer to use the angles, as in Figure 10-32(a). If sufficient duplication were available to warrant making a simple jig to position the plate during welding, the solution of Figure 10-32(b) may be economical.

Figure 10-32(c) shows a tee centered on the beam web and welded to the bottom flange of the beam. The tee stem thickness should not be less than the beam web thickness. The welded solutions shown in Figures 10-32(d) and 10-32(e) are capable of providing good lateral stiffness. The latter two types also permit end rotation as the beam deflects under load. However, if the *m* distance exceeds 3 or 4 in., it is advisable to shop-weld a triangular bracket plate at one end of the beam, as indicated by the dashed lines, to prevent the beam from deflecting along its longitudinal axis.

Other equally satisfactory details may be devised to meet the needs of connections for raised beams. They will vary depending on the size of the supported beam and the distance *m*. When using this type of connection where the load is transmitted through bearing, the provisions of AISC *Specification* Sections J10.2 and J10.3 must be satisfied for both the supported and supporting members. For the detail of Figure 10-32(b), since the rolled fillet has been removed by the cut, the value of *k* would be taken as the thickness of the plate plus the fillet weld size.

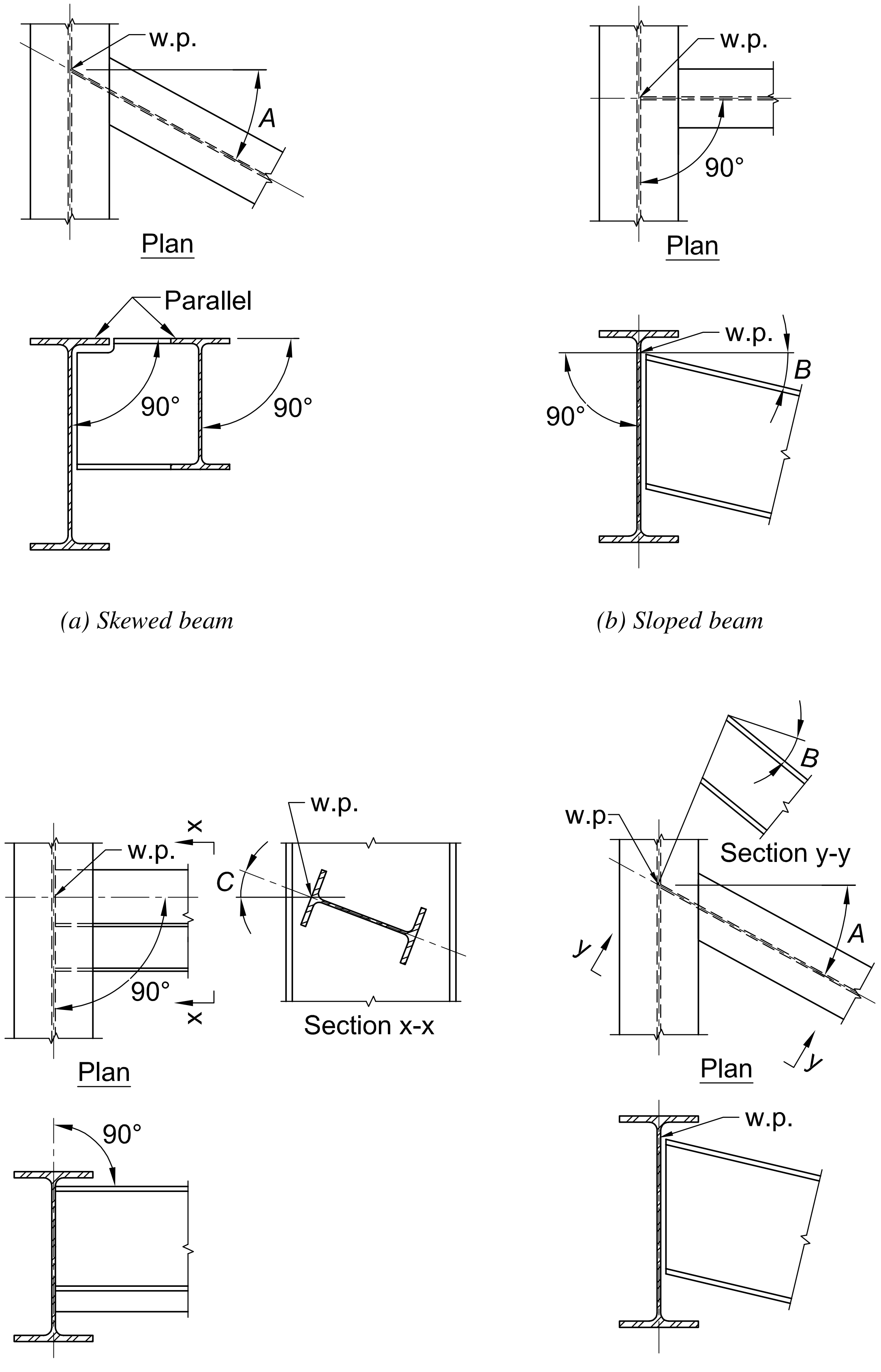
AISC *Specification* Appendix 6 requires stability and restraint against rotation about the beam's longitudinal axis. This provision is most easily accomplished with a floor on top of the supported beam. In the absence of a floor, the top flange may be supported by a strut or bracket attached to the supporting member. When the beam is encased in a wall, this stability may also be provided with wall anchors.

This discussion has considered that the field bolts which attach the beam to the pedestal or support beam are subject to no calculated load. It is important, however, to recognize that when the beam deflects about its neutral axis, a tensile force can be exerted on the outside bolts. The intensity of this tensile force is a function of the dimension *d,* indicated in Figure 10-31, the span length of the supported member, and the beam stiffness. If these forces are large, high-strength bolts should be used and the connection analyzed for the effects of prying action.

Raised-beam connections such as these are used frequently as equipment or machinery supports where it is important to maintain a true and level surface or elevation. When this tolerance becomes important, the dimension *d* should be noted "keep" to advise the fabricator of this importance, as shown in Figure 10-31(b). Since the supporting beam is subject to certain camber/deflection tolerances, it also may be appropriate to furnish shim packs between the connection and the supporting member.

Non-Rectangular Simple Shear Connections

It is often necessary to design connections for beams that do not frame into a support orthogonally. Such a beam may be inclined with respect to the supporting member in various directions. Depending upon the relative angular position that a beam assumes, the connection may be classified among three categories: skewed, sloped or canted. These conditions are illustrated in Figure 10-33 for beam-to-girder web connections; the same descriptions apply to beam-to-column-flange and web connections. Additionally, beams may be oriented in a combination of any or all of these conditions. For any condition of skewed, sloped or canted framing, the single-plate connection is generally the simplest and most economical of those illustrated in this text.



(a) Skewed beam

(b) Sloped beam

(c) Canted beam

(d) Skewed and sloped beam

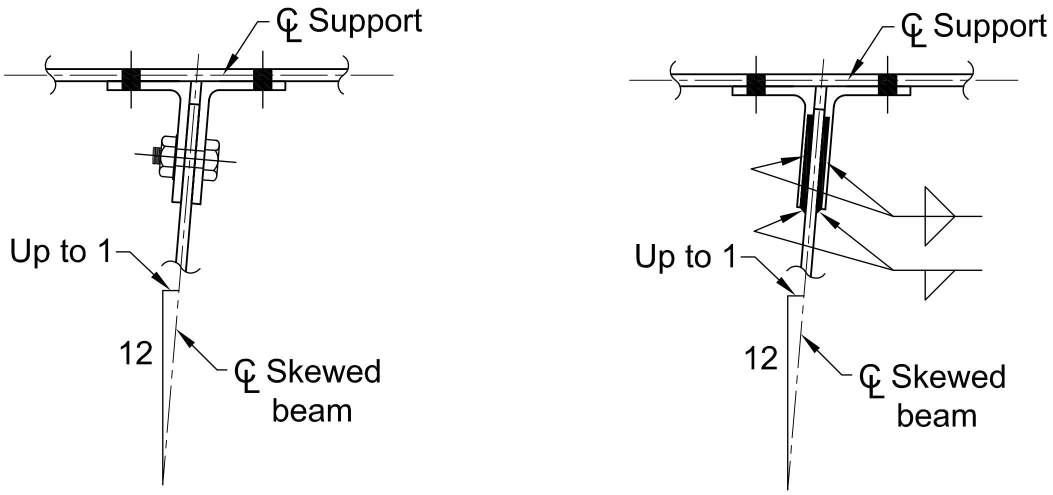
Fig. 10-33. Non-rectangular connections.

Skewed Connections

A beam is said to be skewed when its flanges lie in a plane perpendicular to the plane of the face of the supporting member, but its web inclined to the face of the supporting member. The angle of skew, *A*, appears in Figure 10-33(a) and represents the horizontal bevel to which the fittings must be bent or set, or the direction of gage lines on a seated connection.

When the skew angle is less than 5° (1-in-12 slope), a pair of double angles can be bent inward or outward to make the connection, as shown in Figure 10-34. While bent angle sections are usually drawn as bending in a straight line from the heel, rolled angles will tend to bend about the root of the fillet (dimension *k* in Manual Part 1). This produces a significant jog in the leg alignment, which is magnified by the amount of bend. Above this angle of skew, it becomes impractical to bend rolled angles.

For skews approximately greater than 5° (1-in-12 slope), a pair of bent plates, shown in Figure 10-35, may be a more practical solution. Bent plates are not subject to the deformation problem described for bent angles, but the radius and direction of the bend must be considered to avoid cracking during the cold-bending operation.



(a) All-bolted *(b) Bolted/welded*

Fig. 10-34. Skewed beam connections with bent double angles.

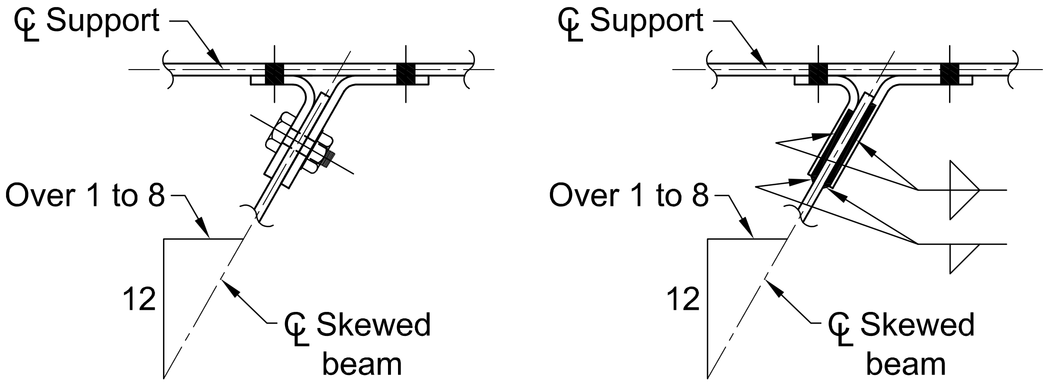


Fig. 10-35. Skewed beam connections with double-bent plates.

Bent plates exhibit better ductility when bent perpendicular to the rolling direction and are, therefore, less likely to crack. Whenever possible, bent connection plates should be billed with the width dimension parallel to the bend line. The length of the plate is measured on its mid-thickness, without regard to the radius of the bend. While this will provide a plate that is slightly longer than necessary, this will be corrected when the bend is laid out to the proper radius prior to fabrication.

Before bending, special attention should be given to the condition of plate edges transverse to the bend lines. Flame-cut edges of hardenable steels should be machined or softened by heat treatment. Nicks should be ground out and sharp corners should be rounded.

The strength of bent angles and bent plate connections may be calculated in the same manner as for square framed beams, making due allowances for eccentricity. The load is assumed to be applied at the point where the skewed beam center line intersects the face of the supporting member.

As the angle of skew increases, entering and tightening clearances on the acutely angled side of the connection will require a larger gage on the support. If the gage were to become objectionable, a single bent plate, illustrated in Figure 10-36, may provide a better solution. Note that the single-bent plate may be of the conventional type, or a more compact connection may be developed by "wrapping" the single bent plate, as illustrated in Figure 10-36(c).

In all-bolted construction, both the shop and field bolts should be designed for shear and the eccentric moment. A C-shaped weld is preferable to avoid turning the beam during shop fabrication. Single bent plates should be checked for flexural strength.

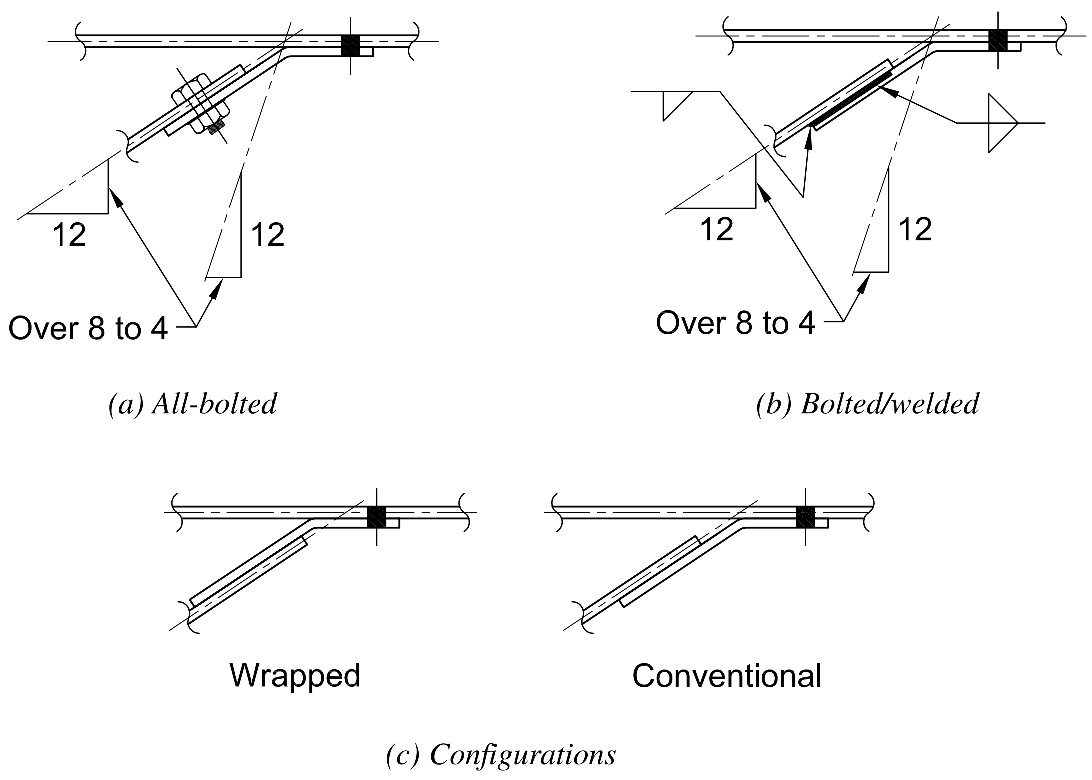
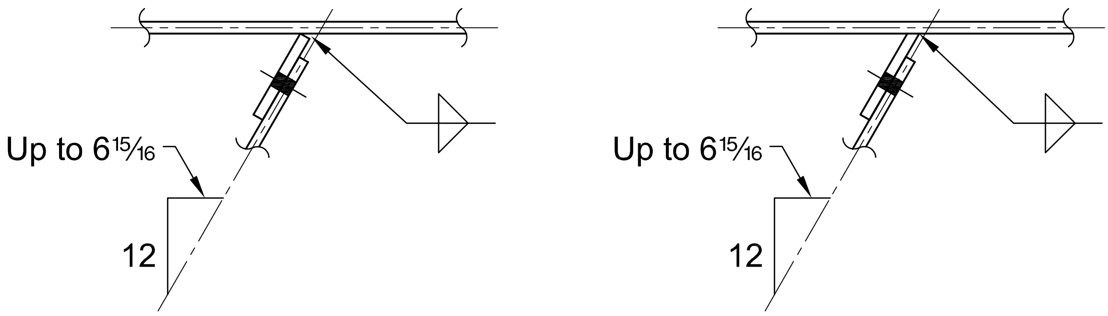


Fig. 10-36. Skewed-beam connections with single-bent plates.

Skewed single-plate and skewed end-plate connections, shown in Figures 10-37 and 10-38, provide a simple, direct connection with a minimum of fittings and multiple punching requirements. When fillet-welded, these connections may be used for skews up to 30° (or a slope of $6^5/_{16}$-in-12) provided the root opening formed does not exceed $^3/_{16}$ in. For skew angles greater than 30°, see AWS D1.1 clause 2.4.2.6.

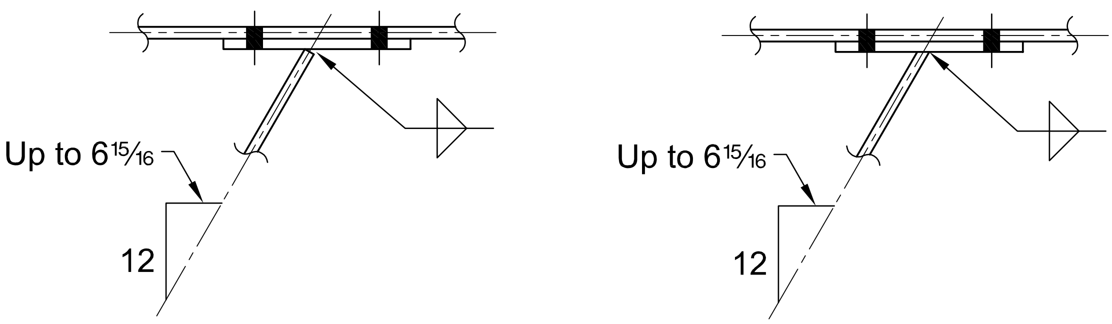
The maximum beam-web thickness that may be supported is a function of the maximum root opening and the angle of skew. If the thickness of the beam web were such that a larger root opening were encountered, the skewed single plate or the web connecting to the skewed end plate may be beveled, as shown in Figures 10-37(b) and 10-38(b). Since no root opening occurs with the bevel, there is no limitation on the thickness of the beam web. However, beveling, especially of the beam web, requires careful finishing and is an expensive procedure that may outweigh its advantages.

The design of skewed end-plate connections is similar to that discussed previously in "Shear End-Plate Connections" in this Part. However, when the gage of the bolts is not centered on the beam web, this eccentric loading should be considered. The design of skewed single-plate connections is similar to that discussed previously in "Single-Plate Connections" in this Part.



(a) Square edge (preferred) *(b) Beveled edge (alternative)*

Fig. 10-37. Skewed single-plate connections.



(a) Square edge (preferred) *(b) Beveled edge (alternative)*

Fig. 10-38. Skewed shear end-plate connections.

When skewed, stiffened seated connections are used, the stiffening element should be located so as to cross the skewed beam centerline well out on the seat. This can be accomplished by shifting the stiffener to the left or right of center to support beams which skew to the left or to the right, respectively. Alternatively, it may be possible to skew the stiffening element.

Sloped Connections

A beam is said to be sloped if the plane of its web is perpendicular to the plane of the face of the supporting member, but its flanges are not perpendicular to this face. The angle of slope, *B*, is shown in Figure 10-33(b) and represents the vertical angle to which the fittings must be set to the web of the sloped beam, or the amount that seat and top angles must be bent.

The design of sloped connections usually can be adapted directly from the rectangular connections covered earlier in this part, with consideration of the geometry of the connection to establish the location of fittings and fasteners. Note that sloped beams often require copes to clear supporting girders, as illustrated in Figure 10-39.

Figure 10-40 shows a sloped beam with double-angle connections, welded to the beam and bolted to the support. The design of this connection is essentially similar to that for rectangular double-angle connections. Alternatively, shear end-plate, tee, single-angle, single-plate, or seated connections could be used. Selection of a particular connection type may be influenced by fabrication economy, erectability, and/or by the types of connections used elsewhere in the structure.

Sloped seated beam connections may utilize either bent angles or plates, depending on the angle of slope. Dimensioning and entering and clearance requirements for sloped seated connections are generally similar to those for skewed connections. The bent seat and top plate shown in Figure 10-41 may be used for smaller bevels.

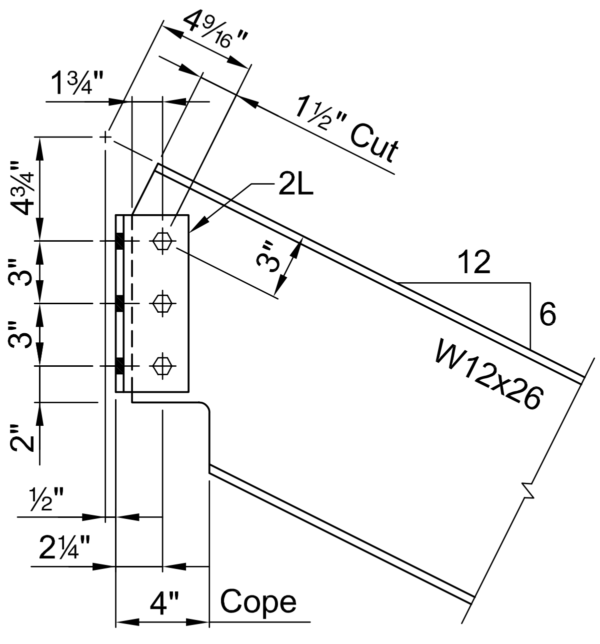


Fig. 10-39. Sloped all-bolted double-angle connection.

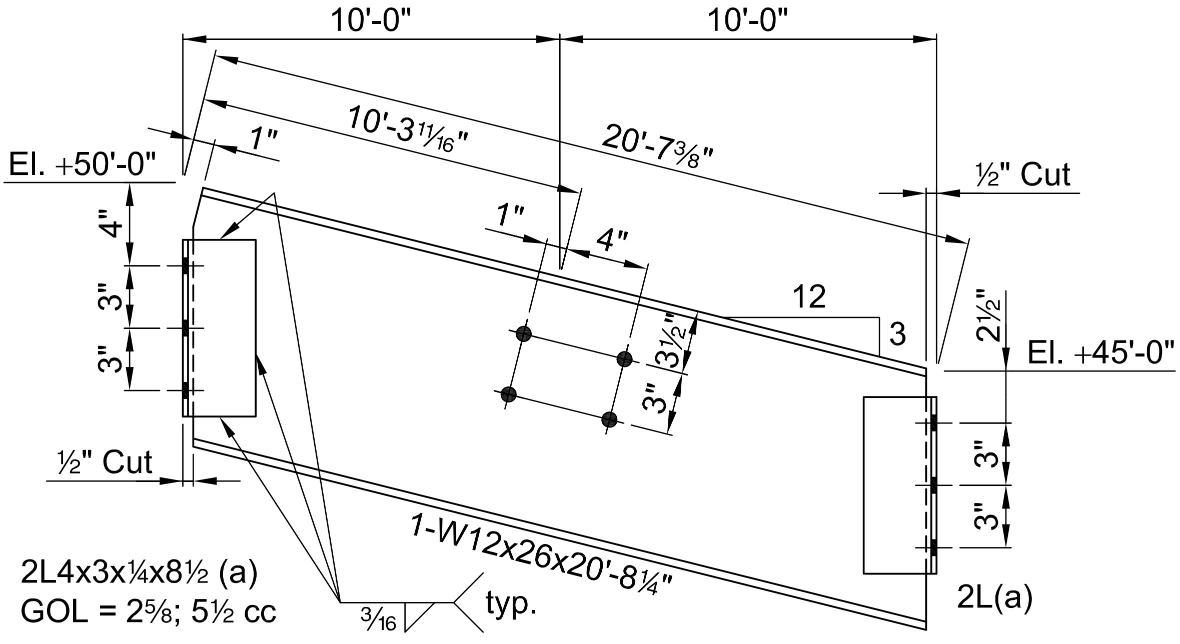


Fig. 10-40. Sloped bolted/welded double-angle connection.

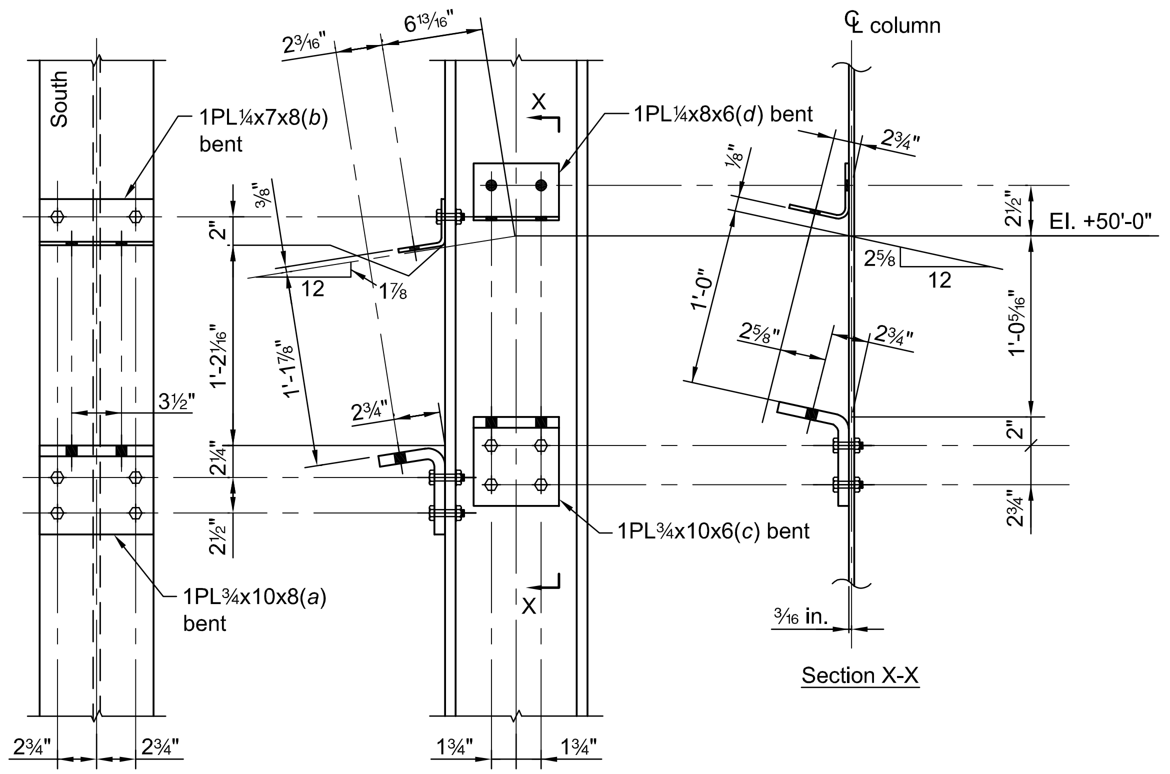


Fig. 10-41. Sloped seated connections.

When the angle of slope is small, it is economical to place transverse holes in the beam web on lines perpendicular to the beam flange; this requires only one stroke of a multiple punch per line. Since non-standard hole arrangements, then, usually occur in the connecting materials (which are single-punched), this requires that sufficient dimensions be provided for the connecting material to contain fasteners with adequate edges and gages, and at the same time fit the angle to the web without encroaching on the flange fillets of the beam. For the end connection of the beam, this was accomplished by using a 6-in. angle leg; a 4-in. or even a 5-in. leg would not have furnished sufficient edge distance at the extreme fastener.

As the angle of slope increases, however, bolts for the end connections cannot conveniently be lined up to permit simultaneous punching of all holes in a transverse row. In this case, the fabricator may choose to disregard beam gage lines and arrange the hole-punching so that ordinary square-framed connection material can be used throughout, as shown in Figure 10-42.

Canted Connections

A beam perpendicular to the face of a supporting member, but rotated so that its flanges are tilted with respect to those of the support, is said to be canted. The angle of cant, C, is shown in Figure 10-33(c).

The design of canted connections usually can be adapted directly from the rectangular connections covered earlier in this part. In Figure 10-43, a double-angle connection is used.

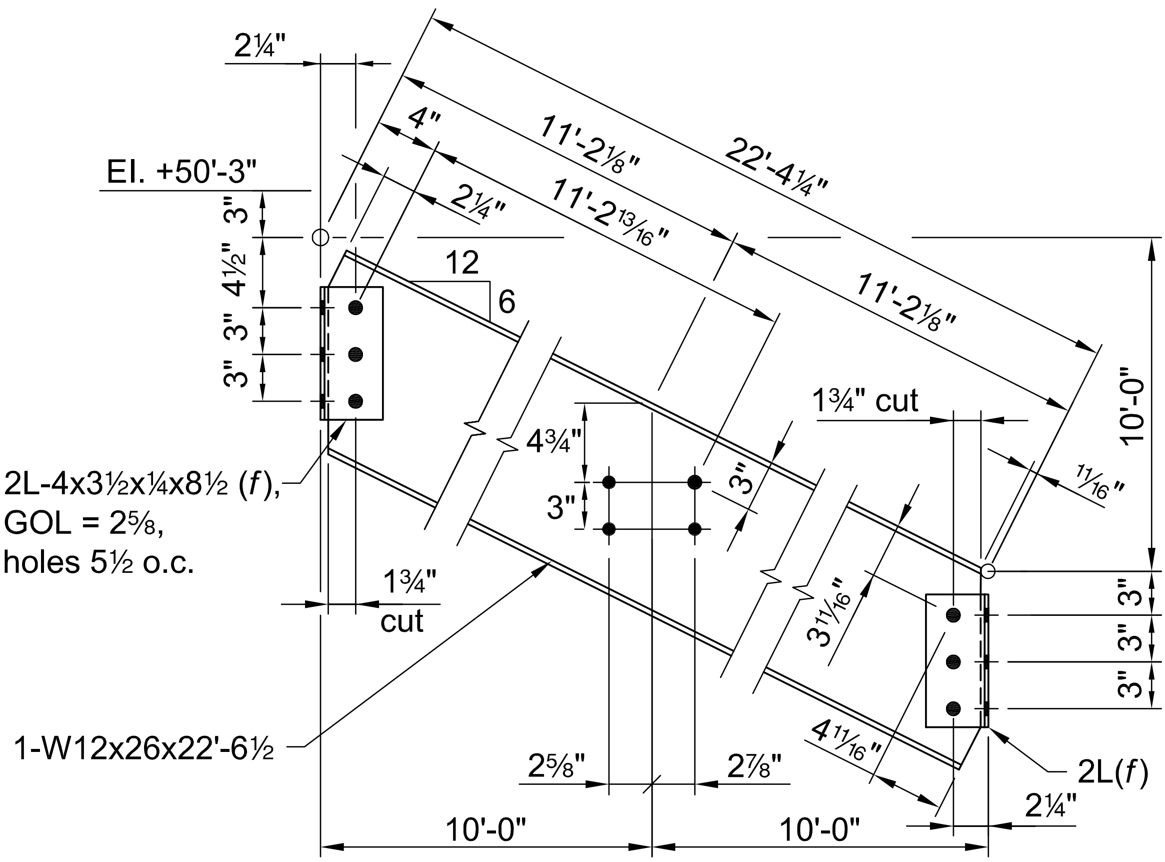


Fig. 10-42. Sloped beam with rectangular connections.

Alternatively, shear end-plate, seated, single-angle, single-plate, and tee connections may also be used.

For the channel in Figure 10-44, which is supported by a sloping member (not shown), to match the hole pattern in the supporting member, the holes in the connecting materials must be canted. As shown in Figure 10-44, the top flange of the channel and the connection angles, d^R and d^L, are cut to clear the flanges of the supporting beam. In this detail, with a 3-in-12 angle of cant, 4-in. legs were wide enough to contain the pattern of hole-punching.

Since the multiple punching or drilling of column flanges requires strict adherence to column gage lines, punching is generally skewed in the fittings. When, for some reason, this is not possible, as in Figure 10-45, skewed reference lines are shown on the column to aid in matching connections.

When canted connecting materials are assembled on the beam, particular care must be used in determining the direction of skew for punching the connection angles. An error reversing this skew may permit matching of holes in both members, but the beam will be canted opposite to the intended direction.

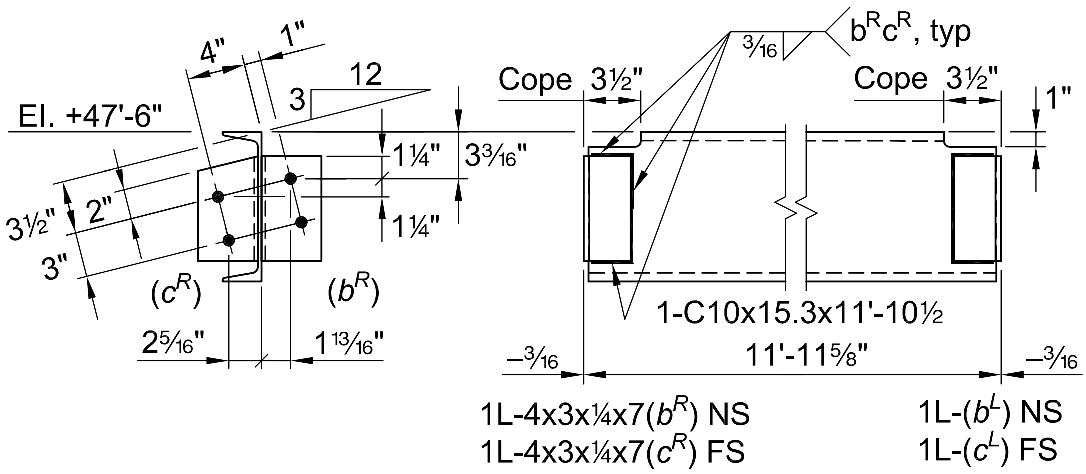


Fig. 10-43. Canted double-angle connections.

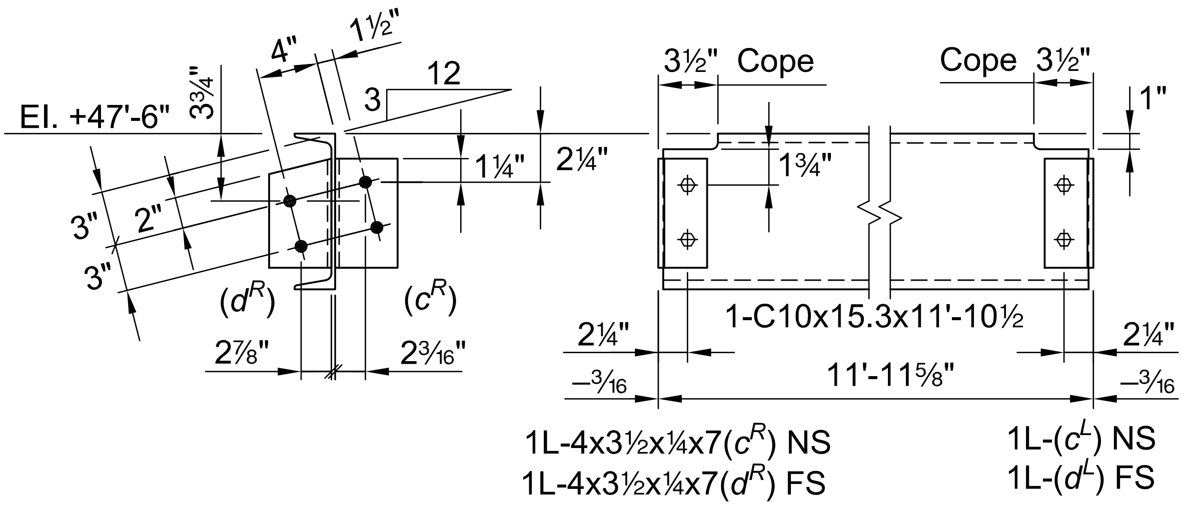


Fig. 10-44. Canted connections to a sloping support.

Note the connection angles in Figure 10-45 are shown shop-welded to the beam. This was done to provide tightening clearance for 3/4-in. high-strength field bolts in the opposite leg. Had the shop fasteners been bolts, it would have been necessary to stagger the field and shop fasteners and provide longer angles for the increased spacing.

Canted seated beams, shown in Figure 10-46, present few problems other than those in ordinary square-end seated beams. Sufficient width and length of angle leg must be provided to contain the gage line punching or drilling in the column face, as well as the off-center location of the holes matching the punching in the beam flange. The elevation of the top flange centerline and the bevel of the beam flange may be given for reference on the beam detail, although the bevel shown will not affect the fabrication.

Inclines in Two or More Directions (Hip and Valley Framing)

When a beam inclines in two or more directions with respect to the axis of its supporting member, it can be classified as a combination of those inclination directions. For example, the beam of Figure 10-33(d) is both skewed and sloped. Angle A shows the skew and angle B shows the slope. Note that, since the inclined beam is foreshortened in the elevation, the true angle B appears only in the auxiliary projection, Section X-X. The development of these details is quite complicated and graphical solutions to this compound angle work can be found in any textbook on descriptive geometry. Accurate dimensions may then be determined with basic trigonometry.

DESIGN CONSIDERATIONS FOR SIMPLE SHEAR CONNECTIONS TO HSS COLUMNS

Many of the familiar simple shear connections that are used to connect to wide-flange columns can be used with HSS columns. These include double and single angles, unstiffened and stiffened seats, single plates, and tee connections. One additional connection that is unique for HSS columns is the through-plate; note that this alternative is seldom required structurally and presents a significant economic penalty when a single-plate

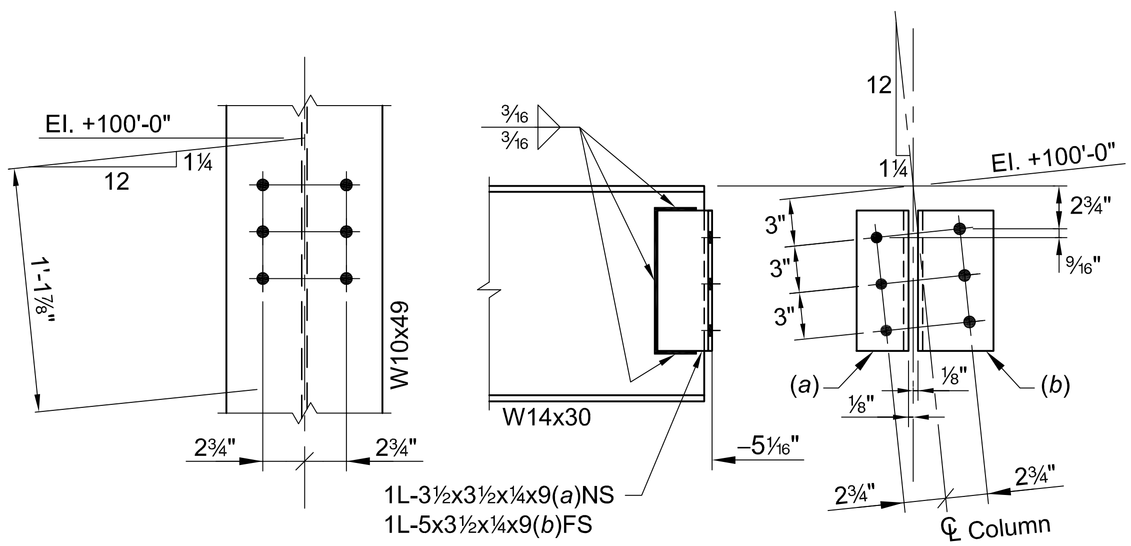


Fig. 10-45. Canted connection to column flange.

connection would otherwise suffice. Variations in attachments are more limited with HSS columns since the connecting element will typically be shop-welded to the HSS and bolted to the supported beam. Except for seated connections, the bolting will be to the web of a wide-flange or other open profile section. Coping is not required except for bottom-flange copes that facilitate knifed erection with double-angle connections.

Double-Angle Connections to HSS

Table 10-1 is a design aid for double-angle connections. The table shows the compatible sizes of W-shapes for the various connection configurations. Based on maximum beam web thickness, maximum weld size, maximum HSS corner radius, and 4-in. outstanding angle legs, double-angle connections may be used with any HSS having a width greater than or equal to 12 in. If 3-in. outstanding angle legs are used for connections with six bolts or less, HSS with widths of 10 in. are acceptable for obtaining welds on the flat of the side. For smaller web thicknesses, welds and corner radii, it may be possible to fit the connection on widths of 10 in. if the outstanding angle legs are 4 in. and on widths of 8 in. for outstanding angle legs of 3 in. However, these dimensions must be verified for a particular case. See the tabulated workable flat dimensions for HSS in Part 1.

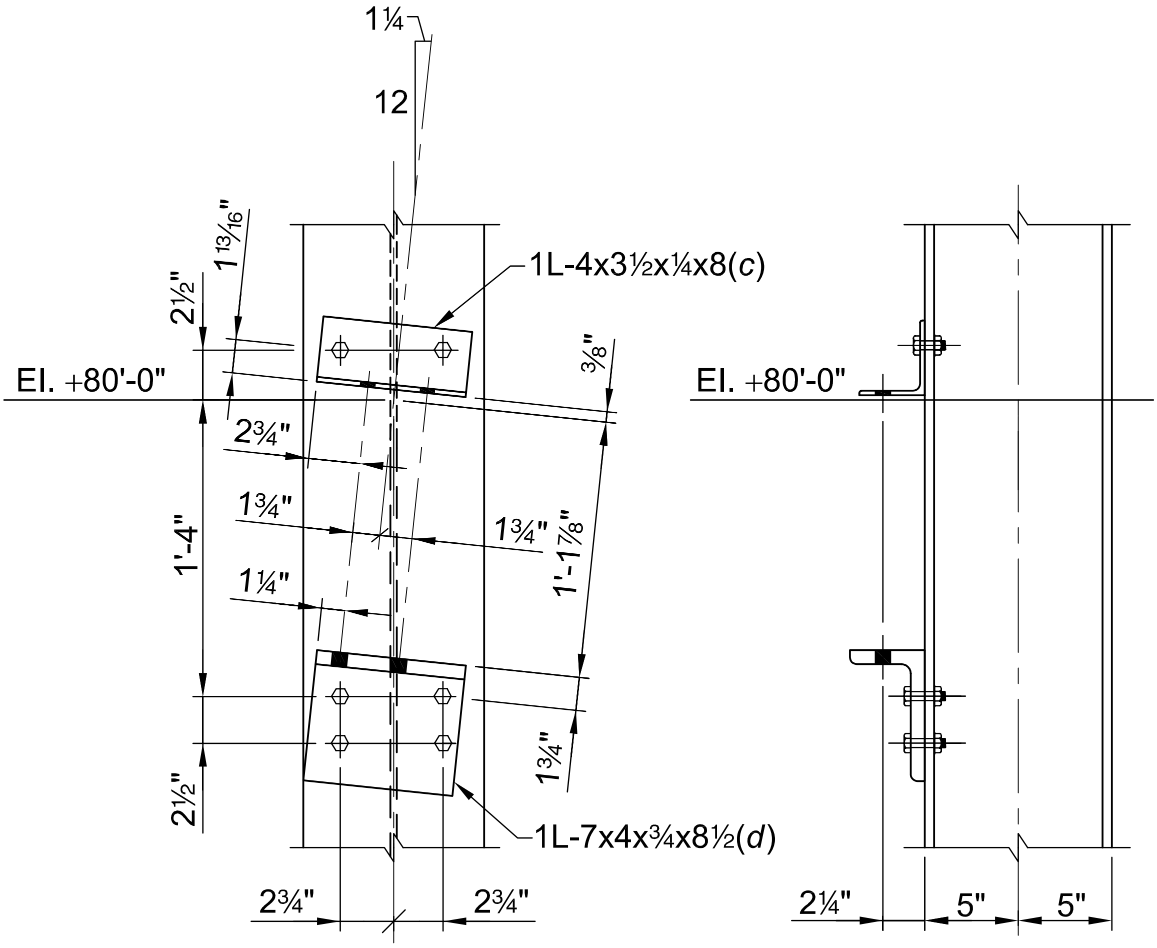


Fig. 10-46. Canted seated connections.

Single-Plate Connections to HSS

As long as the HSS wall is not classified as a slender element, the local distortion caused by the single-plate connection will be insignificant in reducing the column strength of the HSS (Sherman, 1996). Therefore, single-plate connections may be used with rectangular HSS when $b/t \le 1.40(E/F_y)^{0.5}$ or 33.7 for $F_y = 50$ ksi. Single-plate connections may also be used with round HSS as long as they are nonslender under axial load ($D/t \le 0.11E/F_y$).

Yielding (plastification) of the HSS face has not been a governing limit state in physical tests. Punching shear (shear rupture), however, should be checked as follows:

LRFD	ASD
$R_u e \le \dfrac{\phi F_u t l_p^{\,2}}{5}$ (10-7a)	$R_a e \le \dfrac{F_u t l_p^{\,2}}{5\Omega}$ (10-7b)

where

F_u = specified minimum tensile strength of the HSS member, ksi
R_a = required shear strength (ASD), kips
R_u = required shear strength (LRFD), kips
e = eccentricity, taken as the distance from the HSS wall to the center of gravity of the bolt group, in.
l_p = length of the single-plate shear connection, in.
t = design wall thickness of HSS member, in.
ϕ = 0.75
Ω = 2.00

Unstiffened Seated Connections to HSS

In order to properly attach seat angles to the flat of the HSS, the workable flat must be large enough to accommodate both the width of the seat angle and the welds. Seat widths are usually 6 in. or 8 in., but other widths may also be used. See the tabulated workable flat dimensions for HSS in Part 1.

Table 10-6 may be used for unstiffened seated connections to HSS. The minimum HSS thicknesses are established based on the weld strength. If the HSS thickness is less than the minimum value, the weld strength must be reduced proportionally.

Stiffened Seated Connections to HSS

Tables 10-8 and 10-15 are design aids for stiffened seated connections (refer to Figure 10-47). Table 10-8 is applicable to all member types and Table 10-14 presents specific limits for HSS based on the yield-line mechanism limit state for HSS. Some values for small connection lengths, l, and large HSS widths, B, have been reduced to meet the limit state for a line load with a width of $0.4l$ across the HSS, per AISC *Specification* Section K1.

The design procedure for stiffened seated connections to W-shape column webs (Sputo and Ellifritt, 1991) includes a yield line limit state based on an analysis by Abolitz and Warner (1965). This has been applied to the HSS wall which is also supported on two edges.

However, since the HSS side supports are the same thickness rather than much heavier as in the case of W-shape flanges, the equation (Abolitz and Warner, 1965) for rotationally free edge supports has been used instead of fixed edge supports.

The strength of the connection is obtained by multiplying the tabulated value for a particular HSS width and stiffener length by the square of the HSS thickness and dividing by the width of the seat. For combinations of B and l that are not listed in Table 10-14, the HSS does not have sufficient flat width to accommodate a weld to the seat that is $0.2l$ on each side of the stiffener. Because the required width also depends on the stiffener thickness and the HSS corner radius, the HSS width must be checked even when the values are tabulated. See the tabulated workable flat dimensions for HSS in Part 1.

The minimum HSS thicknesses associated with the weld strengths of Table 10-8 are given in Table 10-14. If the HSS thickness is less than the minimum tabulated value, the weld strength must be reduced proportionally.

Through-Plate Connections

In the through-plate connection shown in Figure 10-48, the front and rear faces of the HSS are slotted so that the plate can be passed completely through the HSS and welded to both faces. Through-plate connections should be used when the HSS wall is classified as a slender element [$b/t > 1.40(E/F_y)^{0.5}$ or 33.7 for $F_y = 50$ ksi for rectangular HSS; $D/t > 0.11E/F_y$ for round HSS and Pipe] or does not satisfy the punching shear limit state. A single-plate connection is more economical and should be used if the HSS is neither slender nor inadequate for the punching shear rupture limit state.

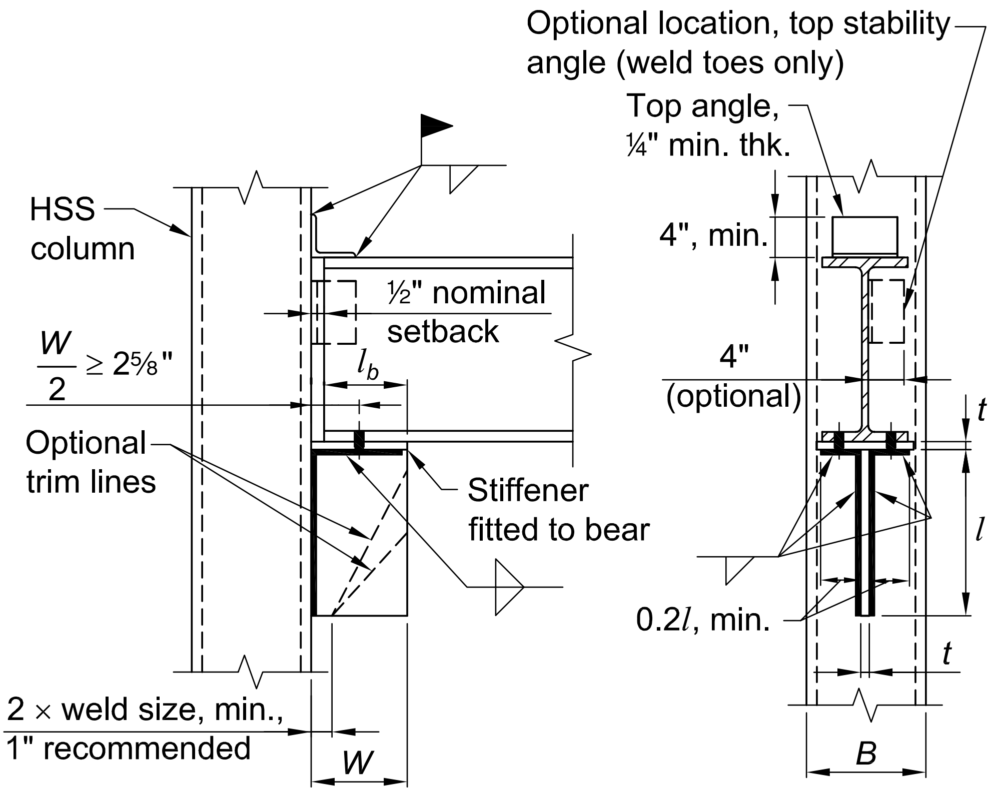


Fig. 10-47. Stiffened seated connection to HSS column.

Through-plate connections have the same limit states as single-plate connections and Table 10-10 may be used to determine the size and number of bolts and the plate thickness. The welds, however, are subject to direct shear and may not have to be as large as those for single-plate connections. For equilibrium of the forces in Figure 10-48, the shear in the welds on the front face should not exceed the strength of the pair of welds. The HSS wall strength can be matched to the weld shear strength to determine the minimum thickness, as illustrated in Part 9. If the thickness of the HSS is less than the minimum, the weld strength must be reduced proportionally. Conservatively, the welds on the rear face may be the same size.

When a connection is made on both sides of the HSS with an extended through-plate, the portion of the plate inside the HSS is subject to a uniform bending moment. For long connections, this portion of the plate may buckle in a lateral-torsional mode prior to yielding, unless H is very small. Using a thicker plate to prevent lateral-torsional buckling would restrict the rotational flexibility of the connection. Therefore, it must be recognized that the plate may buckle and that the moment will be shared with the HSS wall in a complex manner. However, if the HSS would be satisfactory for a single-plate connection, the lateral-torsional buckling limit state is not a critical concern involving loss of strength.

Single-Angle Connections

For fillet welding on the flat of the HSS side, while keeping the center of the beam web in line with the center of the HSS, single-angle connections must be compatible with one-half the workable flat dimension provided in Part 1. Generally, the following HSS widths and thicknesses will work:

b = 8 in. and $t \leq 1/4$ in.
b = 9 in. and $t \leq 3/8$ in.
$b \geq 10$ in. and any nominal thickness

Alternatively, single angles can be welded to narrow HSS with a flare-bevel weld.

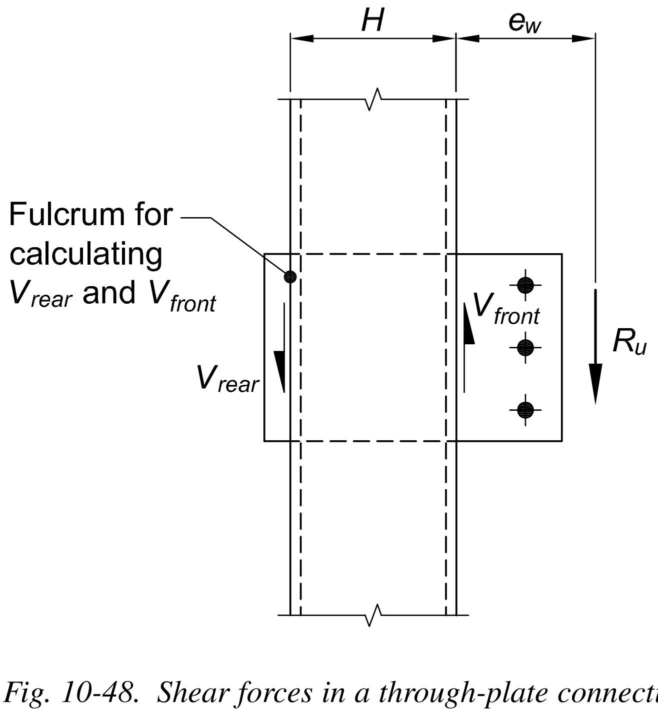


Fig. 10-48. Shear forces in a through-plate connection.

DESIGN TABLE DISCUSSION (TABLES 10-13, 10-14A, 10-14B, 10-14C AND 10-15)

Table 10-13. Minimum Inside Radius for Cold-Bending

Table 10-13 is a design aid providing generally accepted minimum inside-bending radius for a given plate thickness, *t*, for various grades of steel. Values are for bend lines transverse to the direction of final rolling (Brockenbrough, 2006). When bend lines are parallel to the direction of final rolling, the tabular values should be increased by 50%. When bend lines are longer than 36 in., all radii may have to be increased if problems in bending are encountered.

Table 10-14A. Clearances for All-Bolted Skewed Connections

Table 10-14A is a design aid providing clearance dimensions for skewed bent double-angle connections and double and single-bent plate all-bolted connections, and specifies beam setbacks and gages. Since these dimensions are based on the maximum material thicknesses and fastener sizes indicated, it is suggested that in cases where many duplicate connections with less than maximum material or fasteners are required, savings can be realized if these dimensions are developed from specific bevels, beam sizes and fitting thicknesses.

Table 10-14B. Clearances for Bolted/Welded Skewed Connections

Table 10-14B is a design aid providing clearance dimensions, beam setbacks and gages for skewed bent double-angle connections and double and single-bent plate bolted/welded connections. Table 10-13B also specifies the dimension *A* which is added to the fillet weld size, *S*, to compensate for the root opening for skewed end-plate connections. This table is based conservatively on a gap of $^1/_8$ in. For beam webs beveled to the appropriate skew, values of H_1 for the entire table are valid and $A = 0$.

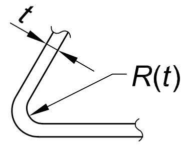
Table 10-14C. Welding Details for Skewed Single-Plate Connections

Table 10-14C is one acceptable design aid providing weld information for skewed single-plate shear connections. Additionally, this table provides clearances and dimensions for groove-welded single-plate connections without backing bars for skews greater than 30°; refer to AWS D1.1/D1.1M for prequalified welds for both types of joints. The weld between the single plate and the support will develop the strength of either 36-ksi or 50-ksi plate.

Table 10-15. Required Length and Thickness for Stiffened Seated Connections to HSS

Table 10-15 is a design aid for stiffened seated connections to HSS. Specific limits are based on the yield-line mechanism limit state of the HSS wall. Some values for small connection lengths, *l*, and large HSS widths, *B*, have been reduced to meet the limit state for a line load with a width of 0.4*l* across the HSS, per AISC *Specification* Section K1.

Table 10-13
Minimum Inside Radius for Cold-Bending[1]



Inside radius as a function of plate thickness

ASTM Designation[2]	Thickness, *t*, in.			
	Up to 3/4	Over 3/4 to 1	Over 1 to 2	Over 2
A36, A572-42	$1\frac{1}{2}t$	$1\frac{1}{2}t$	$1\frac{1}{2}t$	$2t$
A242, A529-50, A529-55, A572-50, A588, A992	$1\frac{1}{2}t$	$1\frac{1}{2}t$	$2t$	$2\frac{1}{2}t$
A572-55, A852	$1\frac{1}{2}t$	$1\frac{1}{2}t$	$2\frac{1}{2}t$	$3t$
A572-60, A572-65	$1\frac{1}{2}t$	$1\frac{1}{2}t$	$3t$	$3\frac{1}{2}t$
A514	$1\frac{3}{4}t$	$2\frac{1}{4}t$	$4\frac{1}{2}t$	$5\frac{1}{2}t$

[1] Values are for bend lines perpendicular to direction of final rolling. If bend lines are parallel to final rolling direction, multiply values by 1.5.
[2] The grade designation follows the dash; where no grade is shown, all grades and/or classes are included.

The design procedure for stiffened seated connections to W-shape column webs (Sputo and Ellifritt, 1991) includes a yield limit state based on an analysis by Abolitz and Warner (1965). This has been applied to the HSS wall which is also supported on two edges. However, since the HSS side supports are the same thickness rather than much heavier, as in the case of W-shape column flanges compared to the column web, the equation for rotationally free edge supports has been used instead of fixed edge supports (Abolitz and Warner, 1965).

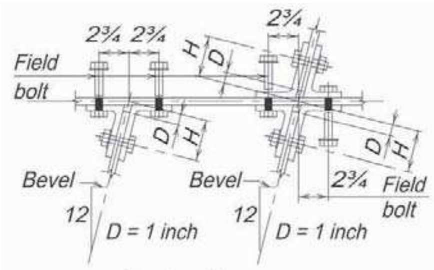
The strength of the connection is obtained by multiplying the tabulated value for a particular HSS width and stiffener length by the square of the HSS thickness and dividing by the width of the seat. For combinations of B and l that are not listed in Table 10-15, the HSS does not have sufficient flat width to accommodate a weld to the seat that is $0.2l$ on each side of the stiffener. Since the required width also depends on the stiffener thickness and the HSS corner radius, the HSS width must be checked even when the values are tabulated. See the tabulated workable flat dimensions for HSS in Part 1.

Table 10-8 is applicable to all member types for stiffened seated connections. The minimum HSS thicknesses associated with the weld strengths of Table 10-8 are given in Table 10-15. If the HSS thickness is less than the minimum tabulated value, the weld strength must be reduced proportionally.

Interpolation between values in this table may produce an incorrect result.

Table 10-14A
Clearances for All-Bolted Skewed Connections

Values given are for webs up to 3/4 in. thick, angles up to 5/8 in. thick, and bent plates up to 1/2 in. thick. Bolts are either 7/8-in. diameter or 1 in. diameter, as noted. Values will be conservative for material thinner than the maximums listed, or for work with smaller bolts, and may be reduced to suit conditions by calculation or layout. For thicker material or larger bolts, check entering, driving, and tightening clearances and increase *D* and bolt gages as necessary. All dimensions are in inches. Enter bolts as shown.



Bent angles

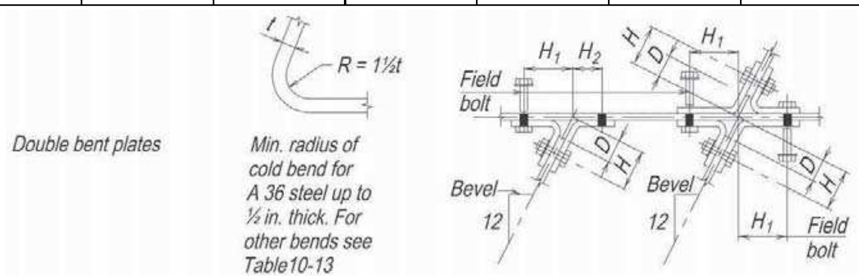
Values of *H* for Various Fastener Combinations

Field Bolts		7/8	1
Shop Bolts		7/8	1
Bevel	Up to 1	4*	4 1/4*
	Over 1 to 2	4 1/8	4 3/8
	Over 2 to 3	4 3/8	4 3/4

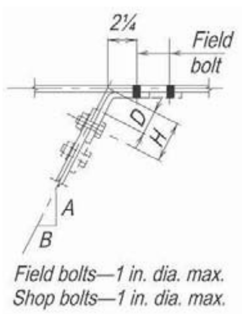
*For back-to-back connections, stagger shop and field bolts or increase the 2 3/4-in. field bolt dimension to 3 1/4.

Values of *H*, H_1, H_2 and *D* for Various Bolt Combinations

Field Fastener		7/8			1			
Shop Fastener		7/8			1			
Dimension		*H*	H_1	H_2	*H*	H_1	H_2	*D*
Bevel	Over 3 to 4	3 3/4	3 1/4	2 1/2	4 1/4	3 1/4	2 3/4	1 1/4
	Over 4 to 5	3 3/4	3 1/2	2 1/4	4 1/2	3 1/2	2 1/2	1 1/4
	Over 5 to 6	4	3 3/4	2 1/4	4 3/4	3 3/4	2 1/4	1 1/2
	Over 6 to 7	4 1/2	4	2 1/4	5	4	2 1/4	1 1/2
	Over 7 to 8	4 3/4	4 1/4	2 1/4	5 1/4	4 1/4	2 1/4	1 1/2



Double bent plates

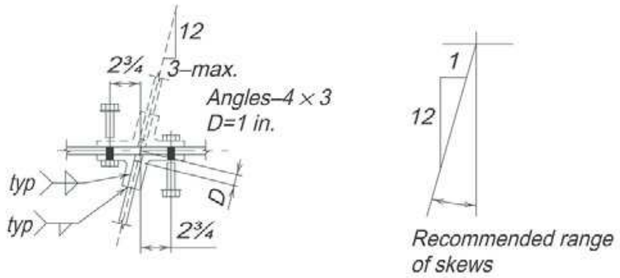


Single bent plates

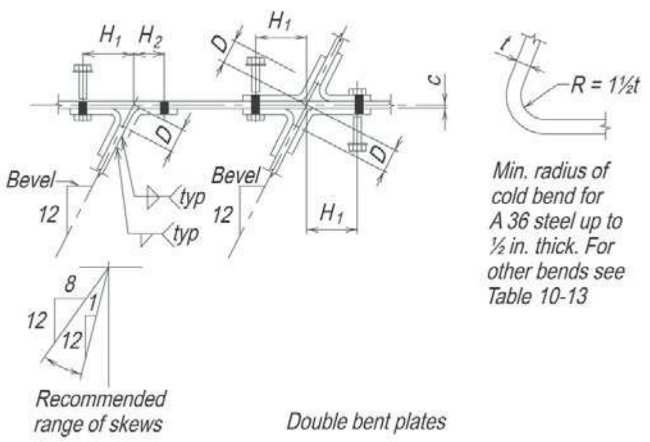
A	*B*	Shop Bolts *D*	Shop Bolts *H*
12	Over 8 to 9	1 1/2	3
12	Over 9 to 10	1 5/8	3 1/8
12	Over 10 to 11	1 3/4	3 1/4
12	Over 11 to 12	1 7/8	3 3/8
Under 12 to 11	12	2 1/8	3 5/8
Under 11 to 10	12	2 1/4	3 3/4
Under 10 to 9	12	2 1/2	4
Under 9 to 8	12	2 3/4	4 1/4
Under 8 to 7	12	3 1/4	4 3/4
Under 7 to 6	12	3 3/4	5 1/4
Under 6 to 5	12	4 1/2	6
Under 5 to 4	12	5 5/8	7 1/8

Table 10-14B
Clearances for Bolted/Welded Skewed Connections

Values given are for webs up to ¾ in. thick, angles up to ⅝ in. thick, and bent plates up to ½ in. thick, with bolts 1 in. diameter maximum. Values will be conservative for thinner material and for work with smaller bolts, and may be reduced to suit conditions by calculation or layout. For thicker material or larger bolts, check entering and tightening clearances and increase beam set-back *D* and bolt gages as necessary. Enter bolts as shown. All dimensions are in inches.



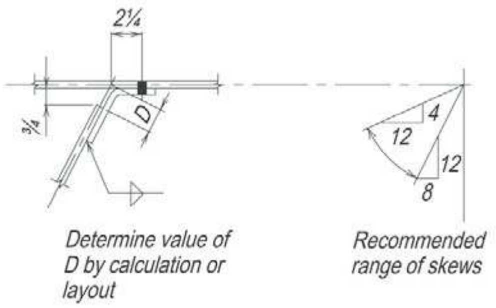
Bent angles



Double bent plates

Bevel	D	H_1	H_2
Over 3 to 4	c + ⅝	3¼	2¾
Over 4 to 5	c + 1¹⁄₁₆	3½	2½
Over 5 to 6	c + ¾	3¾	2¼
Over 6 to 7	c + ¹³⁄₁₆	4	2¼
Over 7 to 8	c + ⅞	4¼	2¼

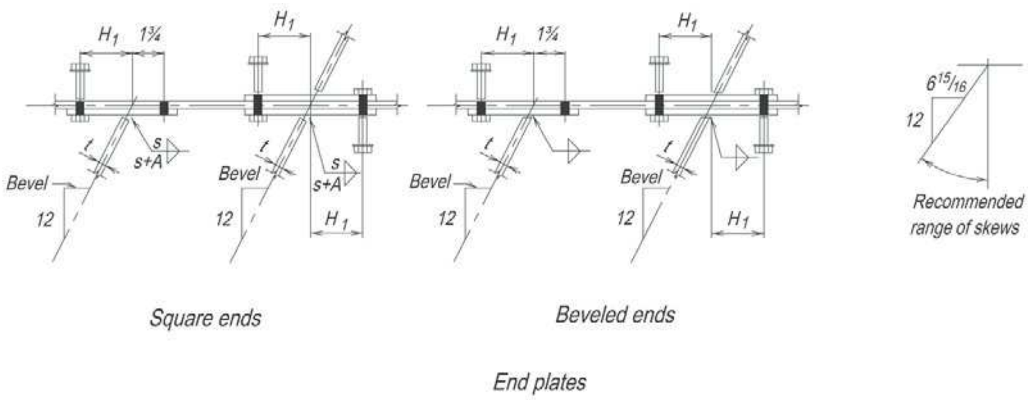
$$C = \frac{t_w}{2} + \frac{1}{16}''$$



Single bent plates

Table 10-14B (continued)
Clearances for Bolted/Welded Skewed Connections

Values given are for material and bolt sizes noted below. See "Shear End-Plate Connections" in Part 10 for proportioning these connections. *S* indicates weld size required for strength, or a size suitable to the thickness of material. When the beam web is cut square, only that portion of the table above the heavy lines is applicable. Dimension *A* is added to the weld size to compensate for the root opening caused by the skew. When the beam web is beveled to the required skew, values of H_1 for the entire table are valid, and A = 0. In either case, where weld strength is critical, increase the weld size to obtain the required throat dimension. Enter bolts as shown. All dimensions are in inches.



Bevel	t = 1/4		t = 5/16		t = 3/8		t = 7/16		t = 1/2		t = 5/8		t = 3/4	
	H_1	A	H_1	A	H_1	A	H_1	A	H_1	A	H_1	A	H_1	A
Up to 1 5/8	1 3/4	0	1 3/4	0	1 3/4	1/16	1 3/4	1/16	1 3/4	1/16	1 7/8	1/8	1 7/8	1/8
Over 1 5/8 to 2 1/8	1 3/4	0	1 3/4	1/16	1 7/8	1/16	1 7/8	1/16	1 7/8	1/8	2	1/8	2	1/8
Over 2 1/8 to 3 1/4	1 7/8	1/16	1 7/8	1/8	2	1/8	2	1/8	2	1/8	2 1/8	0	2 1/8	0
Over 3 1/4 to 4 3/8	2 1/8	1/8	2 1/8	1/8	2 1/8	1/8	2 1/8	0	2 1/4	0	2 1/4	0	2 3/8	0
Over 4 3/8 to 5 5/8	2 1/4	1/8	2 1/4	1/8	2 3/8	0	2 3/8	0	2 3/8	0	2 1/2	0	2 1/2	0
Over 5 5/8 to 6 15/16	2 1/2	1/8	2 1/2	0	2 1/2	0	2 1/2	0	2 5/8	0	2 5/8	0	2 3/4	0

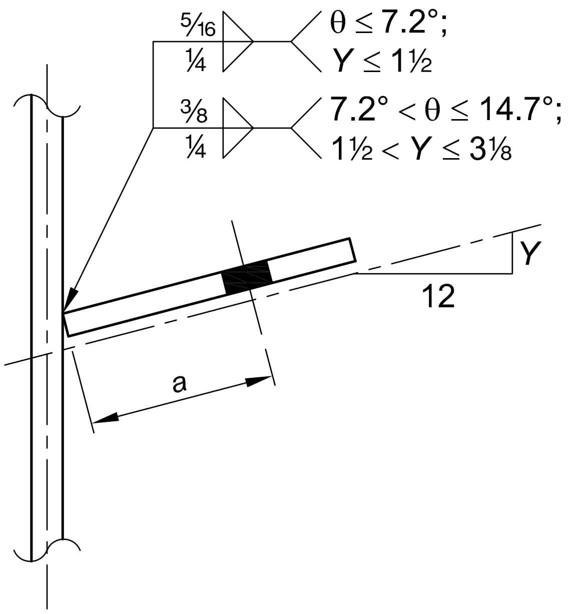
Bolts: 7/8-in.-diameter maximum
End plate thickness: 3/8-in. maximum
Supporting web thickness: 3/4-in. maximum

Use of fillet welds is limited to connections with bevels of 6 15/16 in 12 and less.
For greater bevels consider use of double or single bent plates.

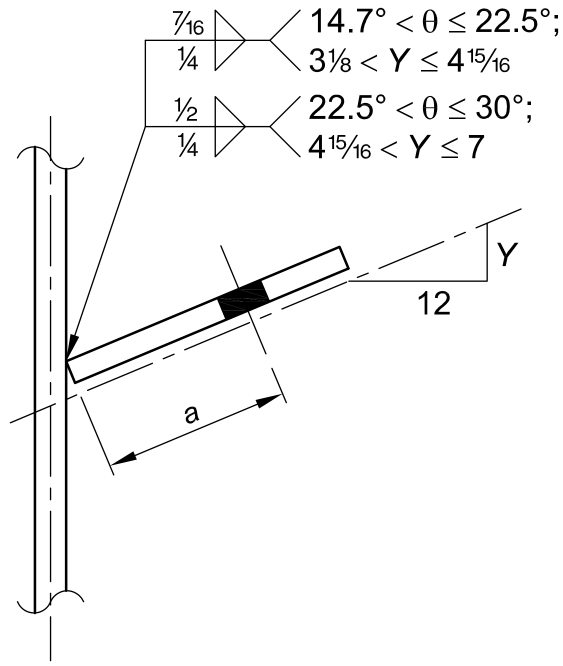
Table 10-14C
Weld Details for Skewed Single-Plate Connections

5/16- and 3/8-in. Plate Thickness*

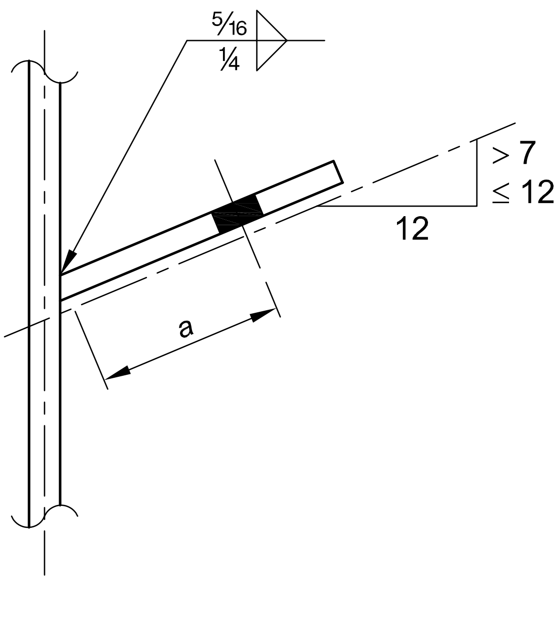
For θ ≤ 14.7° from Perpendicular



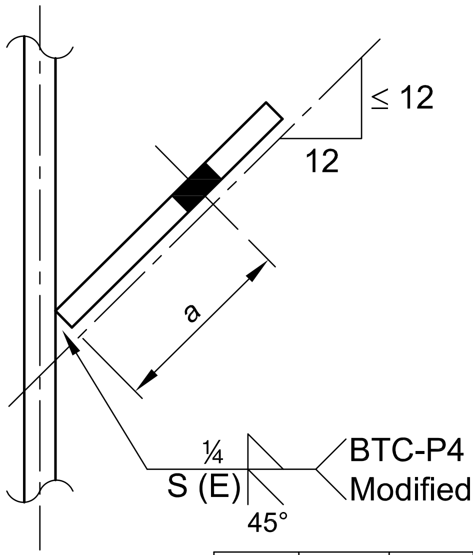
For 14.7° < θ ≤ 30° from Perpendicular



For 30° < θ < 45° from Perpendicular



Alternative for θ ≤ 45° from Perpendicular



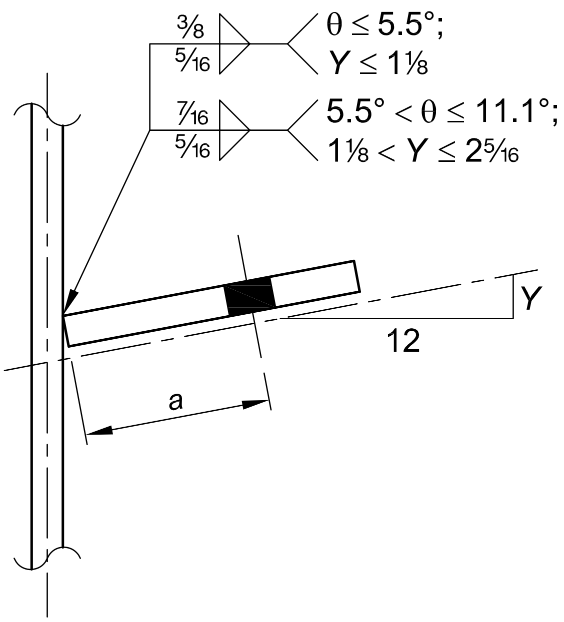
t_p	S	E
5⁄16	5⁄16	3⁄16
3⁄8	3⁄8	¼

*Satisfies single-plate weld requirements for these thicknesses.

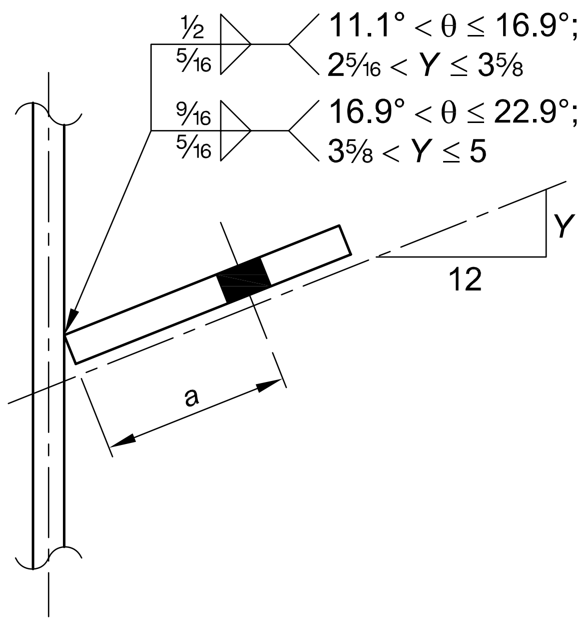
Table 10-14C (continued)
Weld Details for Skewed Single-Plate Connections

1/2-in. Plate Thickness*

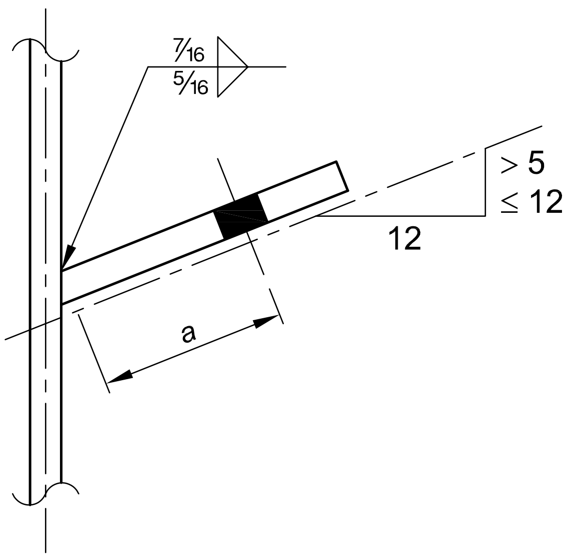
For θ ≤ 11.1° from Perpendicular



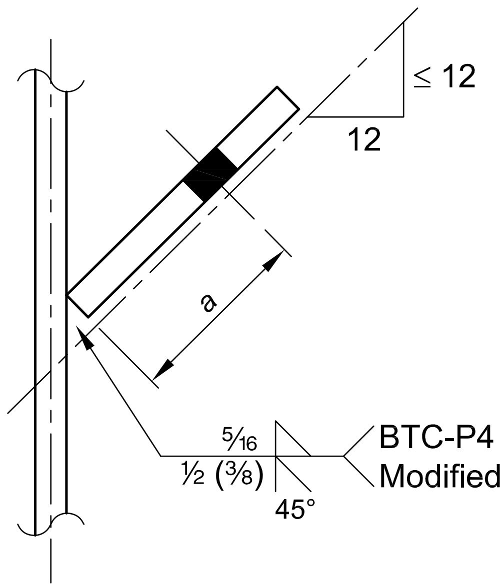
For 11.1° < θ ≤ 22.9° from Perpendicular



For 22.9° < θ ≤ 45° from Perpendicular



Alternative for θ ≤ 45° from Perpendicular

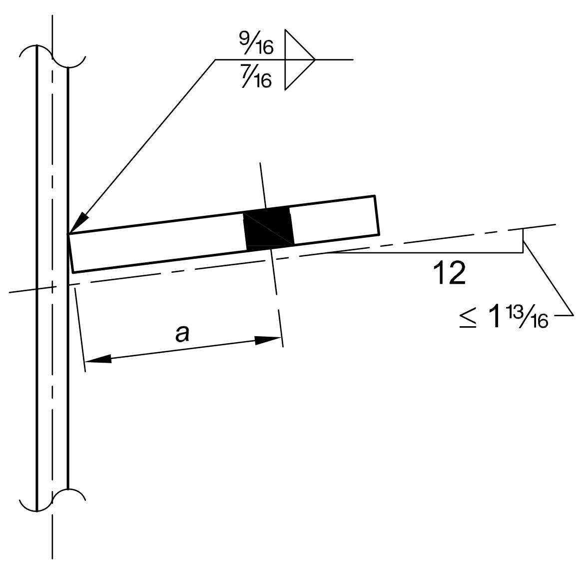


*Satisfies single-plate weld requirements for this thickness.

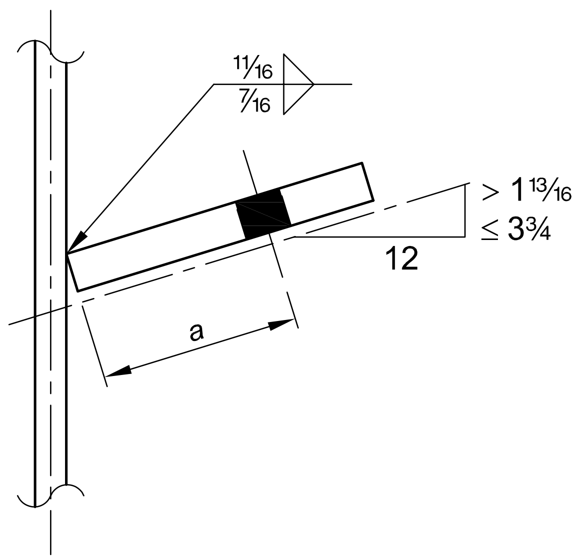
Table 10-14C (continued)
Weld Details for Skewed Single-Plate Connections

5/8-in. Plate Thickness*

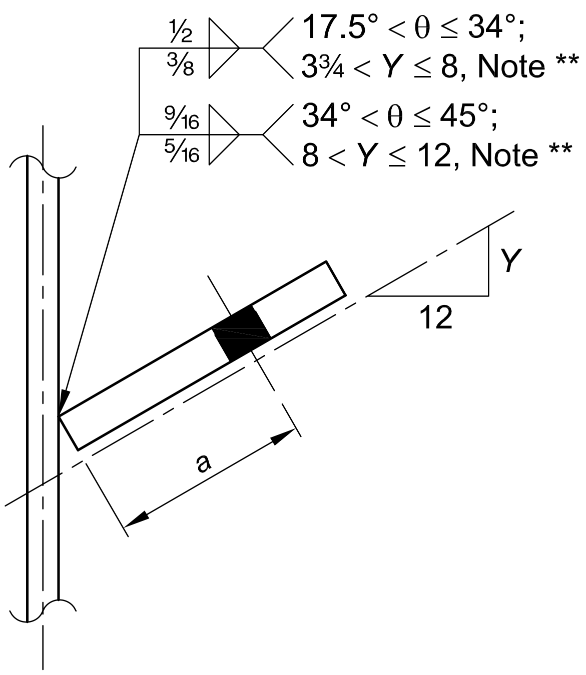
For θ ≤ 8.6° from Perpendicular



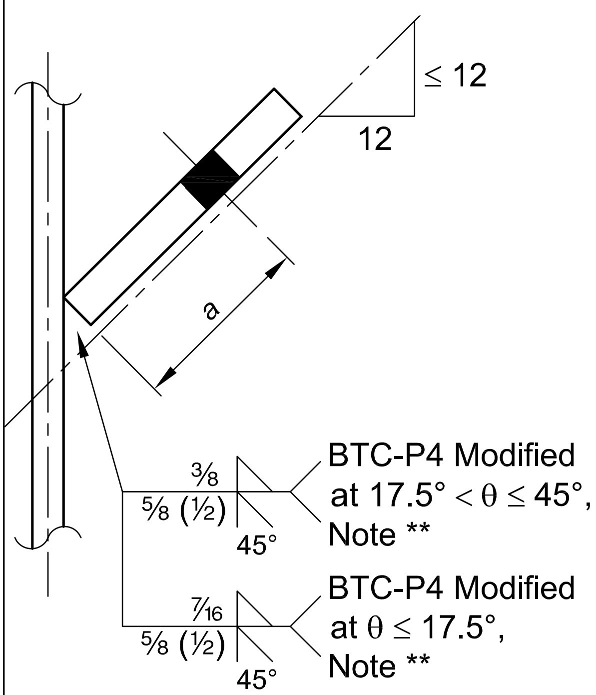
For 8.6° < θ ≤ 17.5° from Perpendicular



For 17.5° < θ ≤ 45° from Perpendicular



Alternative for θ ≤ 45° from Perpendicular



*Satisfies single-plate weld requirements for this thickness.

**Satisfies single-plate weld requirements per AWS dihedral angle "a" reduction factors (AWS D1.1/D1.1M Annex B, Table B.1).

Table 10-15
Required Length and Thickness for Stiffened Seated Connections to HSS

HSS Wall Strength Factor, R_uW/t^2 or R_aW/t^2, kip/in.

l, in.	HSS Width, *B*, in.											
	5		5.5		6		7		8		9	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
6	558	839	545	819	536	805	526	791	525	789	528	793
7	687	1030	664	997	646	971	625	940	615	925	612	920
8			798	1200	771	1160	735	1100	714	1070	704	1060
9					911	1370	856	1290	823	1240	804	1210
10					1070	1600	990	1490	942	1420	912	1370
11							1140	1710	1070	1610	1030	1550
12							1300	1960	1210	1820	1160	1740
13									1370	2060	1290	1940
14									1540	2310	1440	2170
15									1720	2580	1600	2410
16											1700	2660
17											1960	2940

Required HSS Thickness

Weld Size, in.	Min. HSS Thickness, in.
1/4	0.224
5/16	0.280
3/8	0.336
7/16	0.392
1/2	0.448
5/8	0.560

Table 10-15 (continued)
Required Length and Thickness for Stiffened Seated Connections to HSS

l, in.	HSS Wall Strength Factor, R_uW/t^2 or R_aW/t^2, kip/in.											
	HSS Width, B, in.											
	10		12		14		16		18		20	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
6	534	802	552	830	561	843	491	737	437	656	393	590
7	614	922	625	940	644	968	667	1000	594	892	535	803
8	700	1050	704	1060	717	1080	736	1110	759	1140	699	1050
9	793	1190	787	1180	794	1190	809	1220	828	1240	851	1280
10	893	1340	876	1320	876	1320	885	1330	901	1350	920	1380
11	1000	1500	971	1460	962	1450	965	1450	976	1470	993	1490
12	1120	1680	1070	1610	1050	1580	1050	1580	1060	1590	1070	1600
13	1240	1870	1180	1770	1150	1730	1140	1710	1140	1710	1150	1720
14	1370	2070	1290	1940	1250	1880	1230	1850	1220	1840	1230	1840
15	1520	2280	1410	2120	1360	2040	1330	1990	1310	1980	1310	1970
16	1670	2510	1540	2320	1470	2210	1430	2150	1410	2120	1400	2100
17	1830	2760	1680	2520	1590	2390	1540	2310	1510	2260	1490	2240
18	2010	3020	1820	2740	1710	2570	1650	2470	1610	2420	1590	2380
19	2190	3300	1970	2970	1840	2770	1760	2650	1710	2580	1680	2530
20	2390	3600	2130	3210	1980	2980	1880	2830	1820	2740	1790	2680
21			2300	3460	2120	3190	2010	3020	1940	2910	1890	2840
22			2480	3730	2280	3420	2140	3220	2060	3090	2000	3010
23			2670	4020	2440	3660	2280	3430	2180	3280	2120	3180
24			2870	4310	2600	3910	2430	3650	2310	3480	2230	3360
25			3080	4630	2780	4170	2580	3880	2450	3680	2360	3540
26					2960	4450	2740	4110	2590	3890	2480	3730
27					3150	4730	2900	4360	2730	4110	2610	3930
28					3350	5030	3070	4620	2880	4330	2750	4130
29					3560	5340	3250	4890	3040	4570	2890	4340
30					3770	5660	3440	5160	3200	4810	3040	4560
31							3630	5450	3370	5070	3190	4790
32							3830	5750	3540	5330	3340	5020

Required HSS Thickness	
Weld Size, in.	Min. HSS Thickness, in.
1/4	0.224
5/16	0.280
3/8	0.336
7/16	0.392
1/2	0.448
5/8	0.560

Table 10-15 (continued)
Required Length and Thickness for Stiffened Seated Connections to HSS

HSS Wall Strength Factor, $R_u W/t^2$ or $R_a W/t^2$, kip/in.

l, in.	HSS Width, B, in.											
	22		24		26		28		30		32	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
6	357	536	328	492	302	454	281	421	262	393	246	369
7	486	730	446	669	412	618	382	574	357	535	334	502
8	635	953	582	874	537	807	499	749	466	699	437	656
9	804	1210	737	1110	680	1020	632	948	590	885	553	830
10	943	1420	910	1370	840	1260	780	1170	728	1090	682	1020
11	1010	1520	1030	1560	1020	1530	944	1420	881	1320	826	1240
12	1080	1630	1100	1660	1130	1690	1120	1690	1050	1570	983	1470
13	1160	1740	1180	1770	1200	1800	1220	1830	1230	1850	1150	1730
14	1240	1860	1250	1880	1270	1910	1290	1940	1310	1970	1330	2010
15	1320	1980	1330	2000	1340	2020	1360	2040	1380	2070	1400	2110
16	1400	2100	1410	2120	1420	2130	1430	2160	1450	2180	1470	2210
17	1490	2230	1490	2240	1500	2250	1510	2270	1530	2290	1540	2320
18	1580	2370	1570	2370	1580	2370	1590	2390	1600	2410	1620	2430
19	1670	2510	1660	2500	1660	2500	1670	2510	1680	2520	1690	2540
20	1760	2650	1750	2630	1750	2630	1750	2630	1760	2640	1770	2660
21	1860	2800	1850	2770	1840	2760	1840	2760	1840	2770	1850	2780
22	1960	2950	1940	2920	1930	2900	1920	2890	1920	2890	1930	2900
23	2070	3110	2040	3070	2020	3040	2010	3030	2010	3020	2010	3030
24	2180	3280	2140	3220	2120	3190	2110	3170	2100	3160	2100	3150
25	2290	3450	2250	3380	2220	3340	2200	3310	2190	3290	2190	3290
26	2410	3620	2360	3540	2320	3490	2300	3450	2280	3430	2280	3420
27	2530	3800	2470	3710	2430	3650	2400	3600	2380	3570	2370	3560
28	2650	3990	2590	3890	2540	3810	2500	3760	2480	3720	2460	3700
29	2780	4180	2700	4060	2650	3980	2610	3920	2580	3870	2560	3840
30	2920	4380	2830	4250	2760	4150	2710	4080	2680	4030	2650	3990
31	3050	4590	2950	4440	2880	4330	2820	4250	2780	4180	2760	4140
32	3190	4800	3080	4630	3000	4510	2940	4420	2890	4350	2860	4300

Required HSS Thickness

Weld Size, in.	Min. HSS Thickness, in.
1/4	0.224
5/16	0.280
3/8	0.336
7/16	0.392
1/2	0.448
5/8	0.560

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PART 11

DESIGN OF PARTIALLY RESTRAINED MOMENT CONNECTIONS

SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of partially restrained moment connections. For the design of simple shear connections, see Part 10. For the design of fully restrained moment connections, see Part 12.

LOAD DETERMINATION

The behavior of partially restrained (PR) moment connections is intermediate in degree between the flexibility of simple shear connections and the full rigidity of fully restrained (FR) moment connections. AISC *Specification* Section B3.4b(b), Partially Restrained (PR) Moment Connections, defines PR connections as ones that transfer moment but for which the rotation between connected members is not negligible. When used, the analytical model of the PR connection must include the force-deformation characteristics of the specific connection. For further information on the use of PR moment connections, see Geschwindner (1991), Nethercot and Chen (1988), Gerstle and Ackroyd (1989), Deierlein et al. (1990), Goverdhan (1983), and Kishi and Chen (1986).

As an alternative, flexible moment connections (FMC) may be used as a simplified approach to PR moment connection design (Geschwindner and Disque, 2005), particularly for preliminary design. Using FMC, any end restraint that the connection may provide to the girder is assumed zero for gravity load because of the uncertainty of that restraint after repeated loading. The beam and its web connections are thus designed as simple, considering only the gravity loads. For lateral loads, the connection is assumed to behave as an FR moment connection for analysis and the full lateral load is carried by the assigned lateral frames. The resulting flexible moment connections are then designed as "fully restrained" for the calculated required strength due to lateral loads only.

Strength

With PR moment connections, the full strength of the connection is accompanied by some definite amount of rotation between the connected members. The AISC *Specification* requires that the structural engineer have a reliable moment-rotation, M-θ, curve before a design can proceed. These M-θ curves are generally taken directly from the results of multiple connection tests as found in compilations such as those presented by Goverdhan (1983) and Kishi and Chen (1986) or from normalized curves developed from these tests. For information on PR composite connections, see AISC Design Guide 8, *Partially Restrained Composite Connections* (Leon et al., 1996).

Although the M-θ curves are generally quite nonlinear in nature, as the connections undergo alternating cycles of loading and unloading, the connection "shakes down" so that its behavior may be modeled essentially as a linear relationship. This "shakedown" process is fully described in Rex and Goverdhan (2002) and Geschwindner and Disque (2005). Both the nonlinear behavior and the shakedown behavior of the connection must be included in the determination of the connection strength and stiffness for design.

PR moment connections deliver concentrated forces to the flanges of columns that must be accounted for in the design of the column and column panel-zone per AISC *Specification* Section J10. Either the column size can be selected with adequate flange and web thicknesses to eliminate the need for column stiffening, or transverse stiffeners and/or web doubler plates can be provided. For further information, refer to AISC Design Guide 13, *Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications* (Carter, 1999).

Stability

Stability and second-order effects for frames that include PR moment connections are evaluated by the same methods as provided in the AISC *Specification* for frames with simple pin connections and FR moment connections. These are the direct analysis method of Chapter C and the effective length and the first-order analysis methods of Appendix 7. Although the analysis and design of frames with PR moment connections may be more complex than frames with simple or FR moment connections, there may be situations where using the exact behavior of the connection will be advantageous to the designer.

For additional information on designing PR moment frames for stability, see the work of Chen and Lui (1991) and Chen et al. (1996).

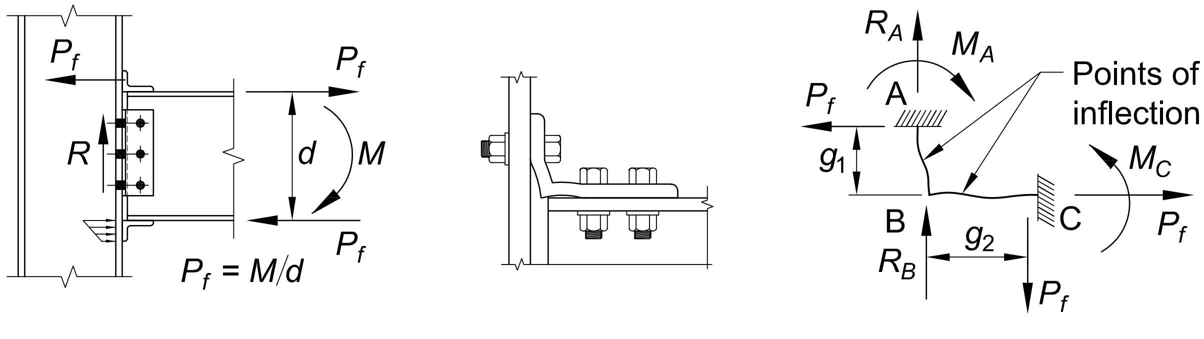
FLANGE-ANGLE PR MOMENT CONNECTIONS

Flange-angle PR moment connections are made with top and bottom angles and a simple shear connection.

The available strength of a flange-angle PR moment connection is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a.

The tensile force is carried to the angle by the flange bolts, with the angle assumed to deform as illustrated in Figure 11-1. A point of inflection is assumed between the bolt gage line and the face of the connection angle, for use in calculating the local bending moment and the corresponding required angle thickness. The effect of prying action must also be considered.

The strength of this type of connection is often limited by the available angle thickness and the maximum number of fasteners that can be placed on a single gage line of the vertical



(a) *(b)* *(c)*

Fig. 11-1. Partially restrained moment connection behavior.

leg of the connection angle at the tension flange. Figure 11-2 illustrates the column flange deformation and shows that only the fasteners closest to the column web are fully effective in transferring forces.

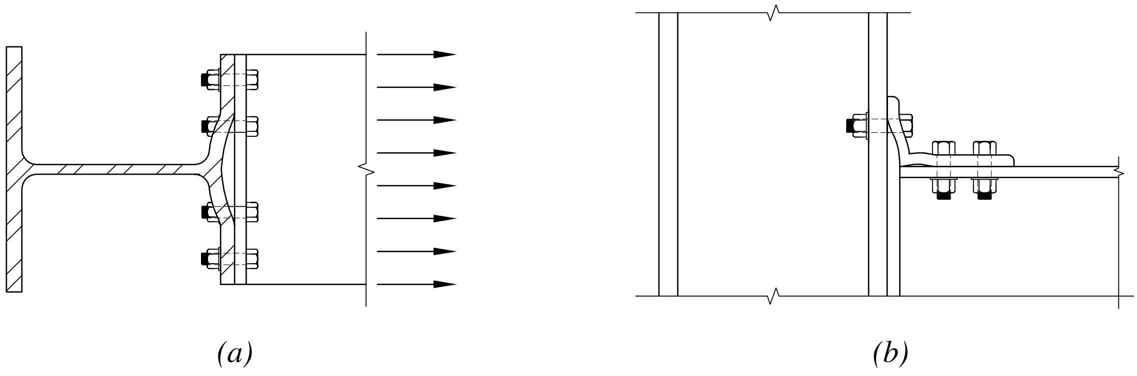


Fig. 11-2. Illustration of deformations in partially restrained moment connections.

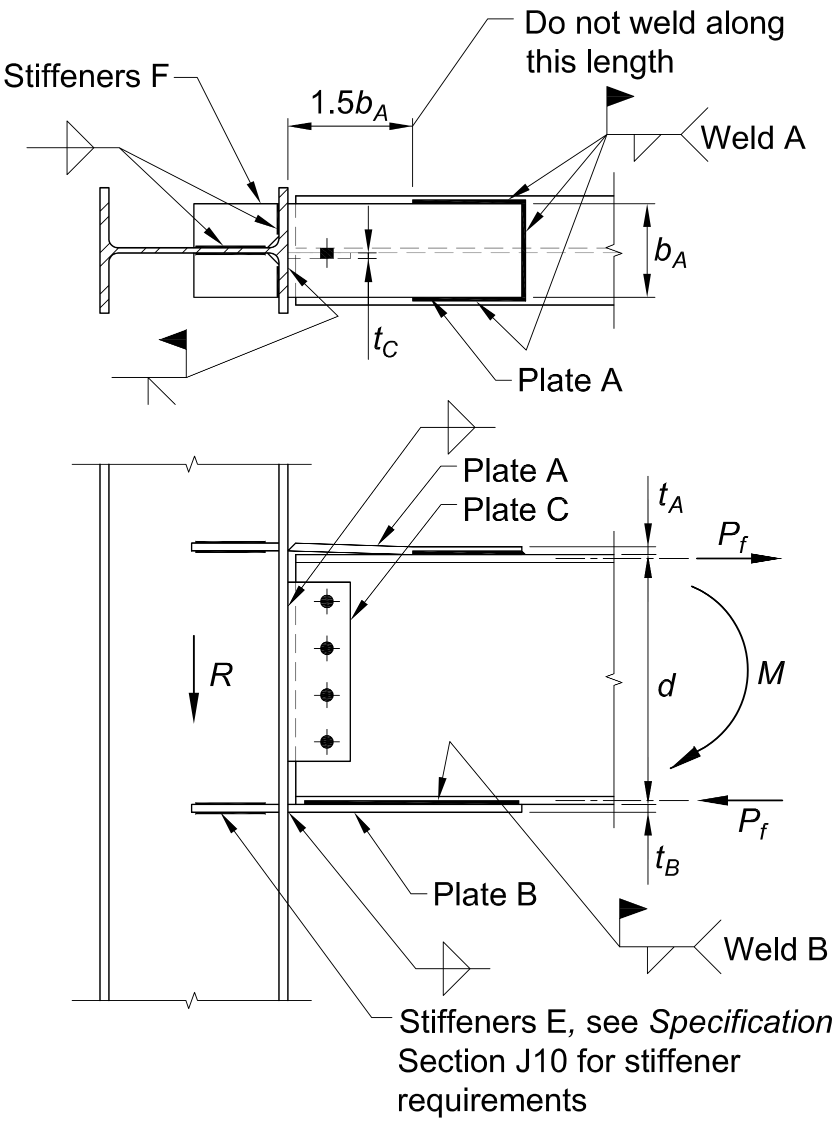


Fig. 11-3. Flange-plated partially restrained moment connections.

FLANGE-PLATED PR MOMENT CONNECTIONS

Originally proposed by Blodgett (1966), and illustrated in Figure 11-3, a flange-plated PR moment connection consists of a simple shear connection and top and bottom flange plates that connect the flanges of the supported beam to the supporting column. These flange plates are welded to the supporting column and may be bolted or welded to the flanges of the supported beam. An unwelded length of $1^1/_2$ times the flange-plate width, b_A, is normally assumed to permit the elongation of the plate necessary for PR moment connection behavior. Other flange-plated details are illustrated in Figures 11-4(a) and 11-4(b).

The available strength of a flange plated PR moment connection is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8), and connecting elements

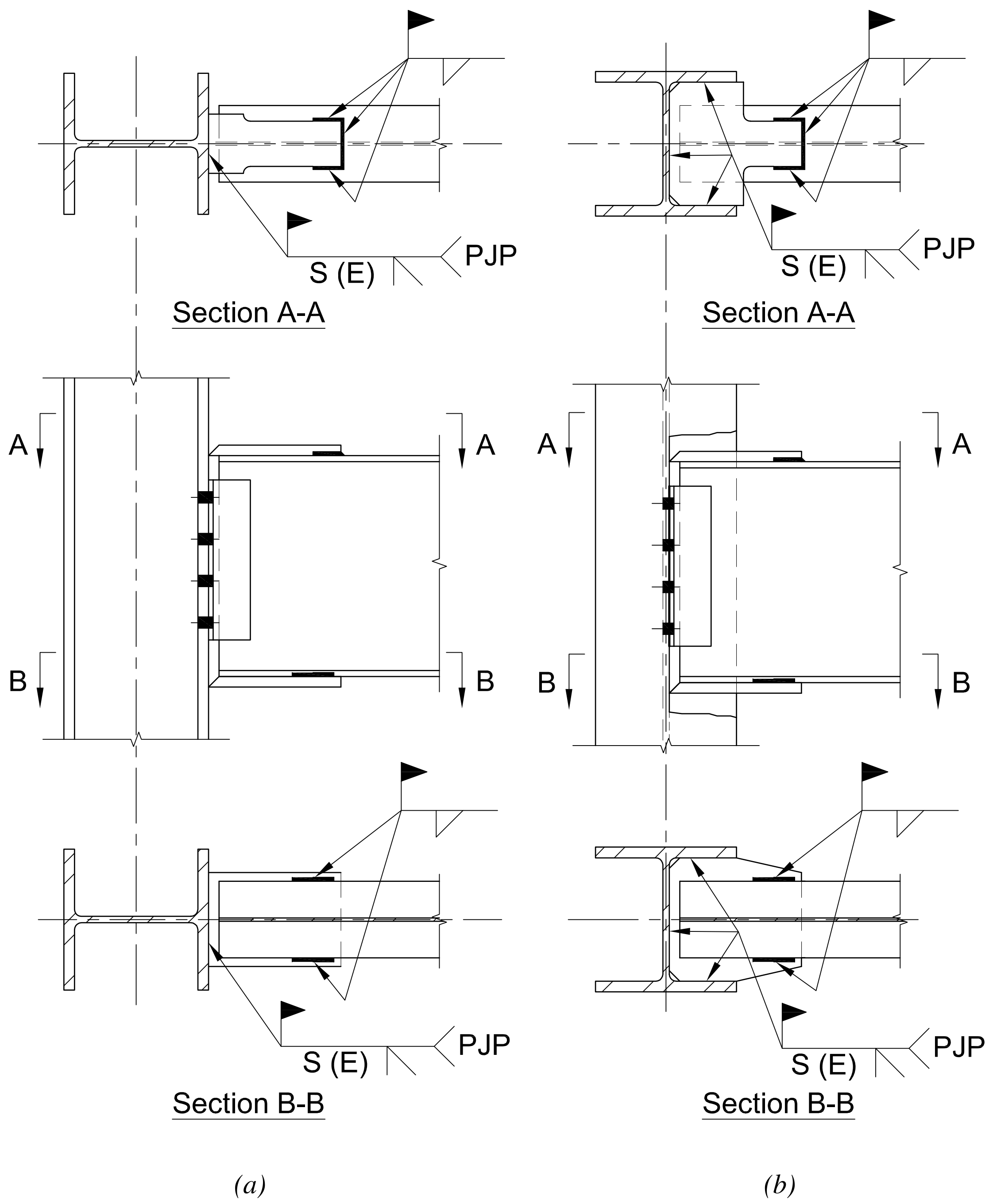


Fig. 11-4. Typical flange-plated partially restrained moment connections.

(see Part 9). In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a.

The shop and field practices for flange-plated FR moment connections (see Part 12) are equally applicable to flange-plated PR moment connections.

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PART 12

DESIGN OF FULLY RESTRAINED MOMENT CONNECTIONS

SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of fully restrained (FR) moment connections. For the design of simple shear connections, see Part 10. For the design of partially restrained moment connections, see Part 11.

FR MOMENT CONNECTIONS

Load Determination

As defined in AISC *Specification* Section B3.6b, FR moment connections possess sufficient rigidity to maintain the angles between connected members at the strength limit states, as illustrated in Figure 12-1. While connections considered to be fully restrained seldom actually provide for zero rotation between members, the small amount of rotation present is usually neglected and the connection is idealized as one exhibiting zero end rotation.

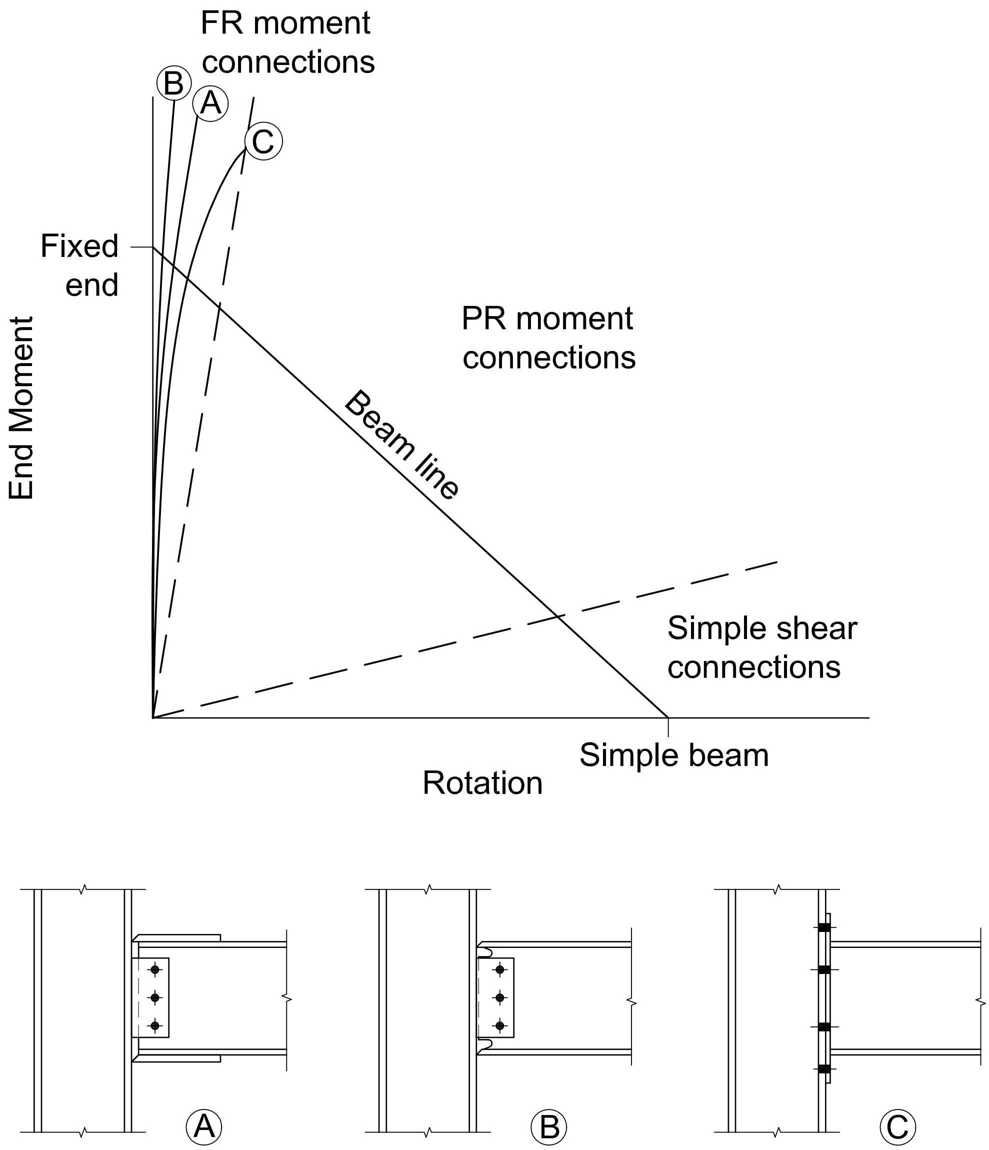


Fig. 12-1. FR moment connection behavior.

End connections in FR construction are designed to carry the required forces and moments, except that some inelastic but self-limiting deformation of a part of the connection is permitted. Huang et al. (1973) showed that the moment can be resolved into an effective tension-compression couple acting as axial forces at the beam flanges. The flange force, P_{uf} or P_{af}, *is* determined as:

LRFD	ASD
$P_{uf} = \dfrac{M_u}{d_m}$ (12-1a)	$P_{af} = \dfrac{M_a}{d_m}$ (12-1b)

where

M_u or M_a = required beam end moment, kip-in.
d_m = moment arm between the flange forces, in. (varies for all FR connections and for stiffener design)

Shear is transferred through the beam-web shear connection. Since, by definition, the angle between the beam and column in an FR moment connection remains unchanged under loading, eccentricity can be neglected entirely in the shear connection. Additionally, it is permissible to use bolts in bearing in either standard or slotted holes perpendicular to the line of force. Axial forces, if present, are normally assumed to be distributed uniformly across the beam flange cross-sectional area. However, if the beam-web connection has sufficient stiffness, it can also be assumed to participate in the transfer of beam axial force.

Moment connections deliver concentrated forces to the flanges of columns that must be accounted for in the design of the column and column panel-zone per AISC *Specification* Section J10. Either the column size can be selected with adequate flange and web thickness to eliminate the need for column stiffening, or transverse stiffeners and/or web doubler plates can be provided. For further information, refer to AISC Design Guide 13, *Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications* (Carter, 1999).

Design Checks

The available strength of an FR moment connection is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). The effect of eccentricity in the shear connection can be neglected. Additionally, the strength of the supporting column (and thus the need for stiffening) must be checked. In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a.

Temporary Support During Erection

Bolted construction provides a ready means to erect and temporarily connect members by use of the bolt holes. In contrast, FR moment connections in welded construction must be given special attention so that all pieces affecting the alignment of the welded joint may be erected, fitted and supported until the necessary welds are made. Temporary support can be provided in welded construction by furnishing holes for erection bolts, temporary seats, special lugs or by other means.

The effects of temporary erection aids on the finished structure should be considered, particularly on members subjected to tension loading or fatigue. They should be permitted to remain in place whenever possible since they seldom are reusable and the cost to remove them can be significant. If left in place, erection aids should be located so as not to cause a stress concentration. If, however, erection aids are to be removed, care should be taken so that the base metal is not damaged.

Temporary supports should be sufficient to carry any loads imposed by the erection process, such as the dead weight of the member, additional construction equipment, or material storage. Additionally, they must be flexible enough to allow plumbing of the structure, particularly in tier buildings.

Welding Considerations for Fully Restrained Moment Connections

Field welding should be arranged for welding in the flat or horizontal position and preference should be given to fillet welds over groove welds, whenever possible. Additionally, the joint detail and welding procedure should be constructed to minimize distortion and the possibility of lamellar tearing.

The typical complete-joint-penetration (CJP) groove weld in a directly welded flange connection for a rolled beam can be expected to shrink about $^1/_{16}$ in. in the length dimension of the beam when it cools and contracts. Thicker welds, such as for welded plate-girder flanges, will shrink even more—up to $^1/_8$ in. or $^3/_{16}$ in. This amount of shrinkage can cause erection problems in locating and plumbing the columns along lines of continuous beams. A method of calculating weld shrinkage can be found in Lincoln Electric Company (1973). Unnecessarily thick stiffeners with CJP groove welds should be avoided since the accompanying weld shrinkage may contribute to lamellar tearing and distortion.

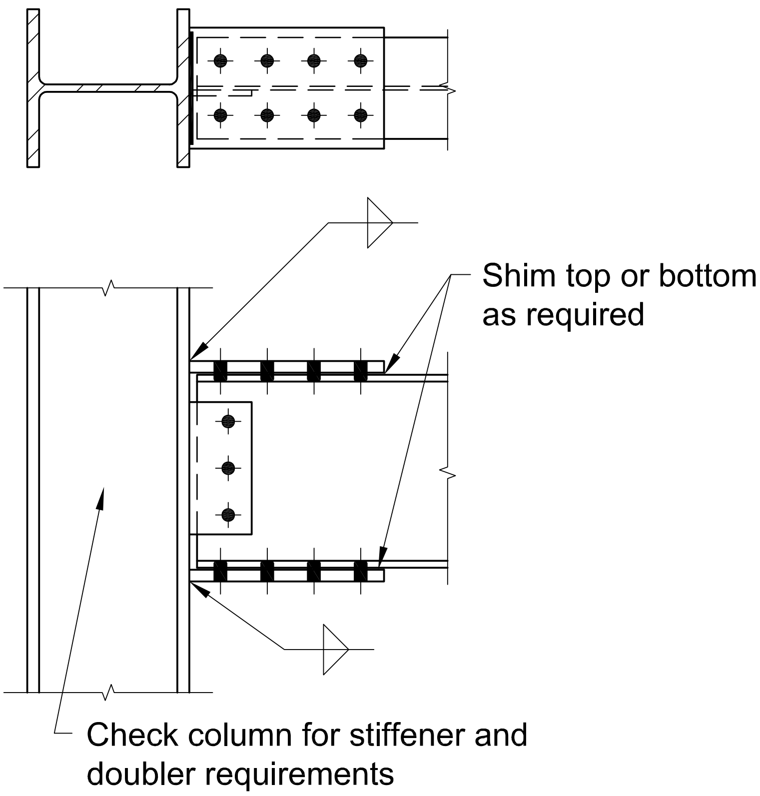
Weld shrinkage can best be controlled by fabricating the beam longer than required by the amount of the anticipated weld shrinkage. Alternatively, the weld-joint root opening can be increased. For further information, refer to AWS D1.1.

FR CONNECTIONS WITH WIDE-FLANGE COLUMNS

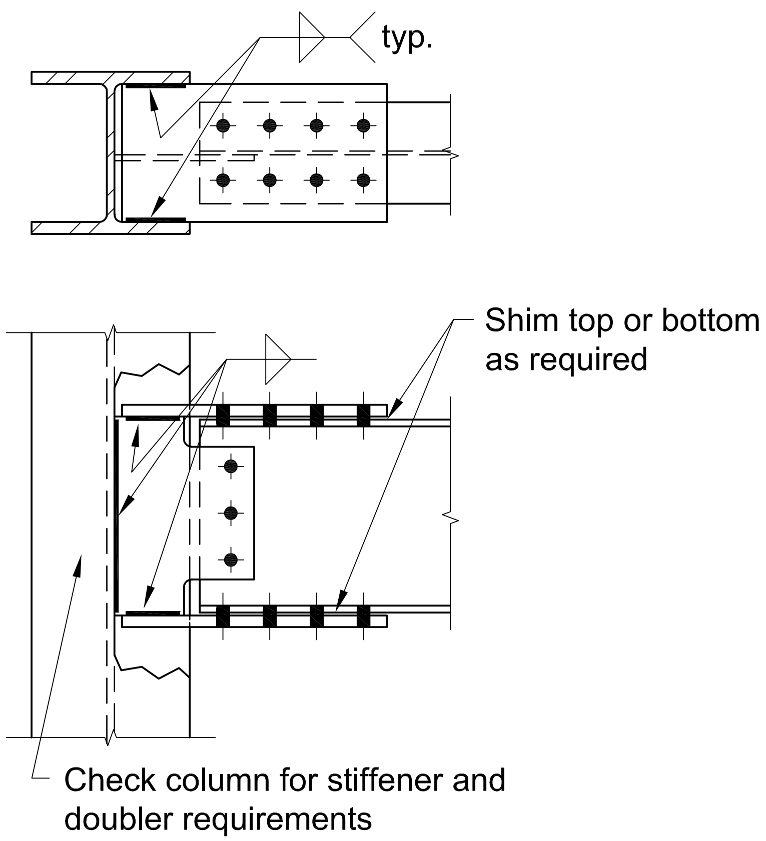
Flange-Plated FR Moment Connections

As illustrated in Figure 12-2, a flange-plated FR moment connection consists of a shear connection and top and bottom flange plates that connect the flanges of the supported beam to the supporting column. These flange plates are welded to the supporting column and may be bolted or welded to the flanges of the supported beam.

In a column-flange connection, the flange plates are usually located with respect to the column web centerline. Because of the column-flange mill tolerance on out-of-squareness with the web, it is desirable to shop-fit long flange plates from the theoretical column-web centerline to assure good field fit-up with the beam. Misalignment on short connections, as illustrated in Figure 12-3, can be accommodated by providing oversized holes in the plates. Since mill tolerances in both the beam and the column may cause significant shop and/or field assembly problems, it may be desirable to ship the flange plates loose for field attachment to the column.

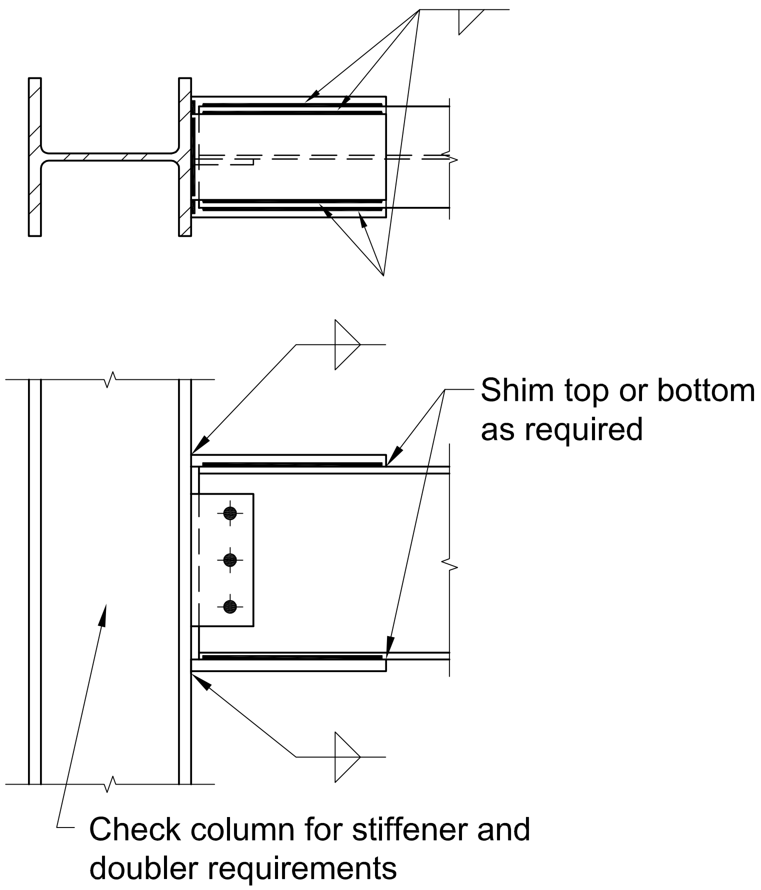


(a) Column flange support, bolted flange plates



(b) Column web support, bolted flange plates

Fig. 12-2. Flange-plated FR moment connections.



(c) Column flange support, welded flange plates

Fig. 12-2. (continued) Flange-plated FR moment connections.

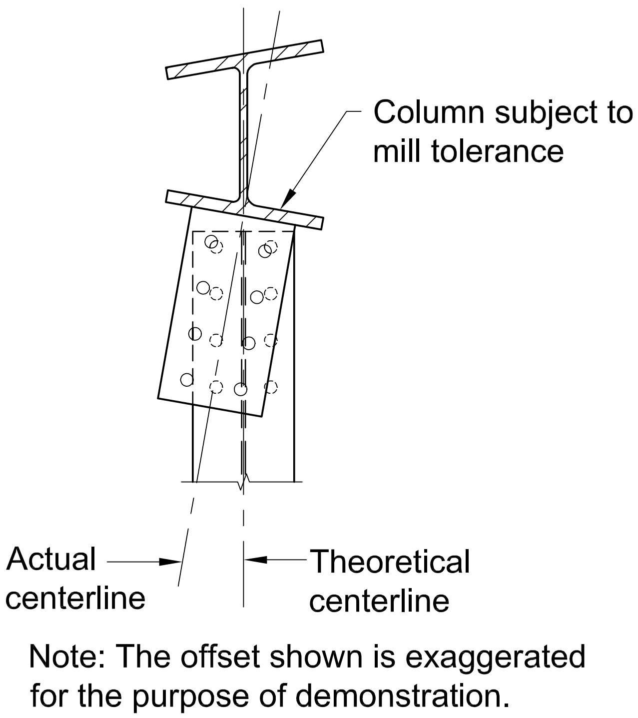


Fig. 12-3. Effect of mill tolerances on flange-plated connections.

Directly Welded Flange FR Moment Connections

As illustrated in Figure 12-4, a directly welded flange FR moment connection consists of a shear connection and CJP groove welds, which directly connect the top and bottom flanges of the supported beam to the supporting column. Tests have shown that connections with beam flanges welded to column flanges and bearing bolts in horizontal short slots, as shown in Figure 12-4(a), can resist moments greater than the plastic bending moment of the beam, even when combined with shear loads approaching the shear yield strength of the beam (Dowswell and Muir, 2012). Note, in Figure 12-4(b), the stiffener extends beyond the toe of the column flange to eliminate the effects of triaxial stresses.

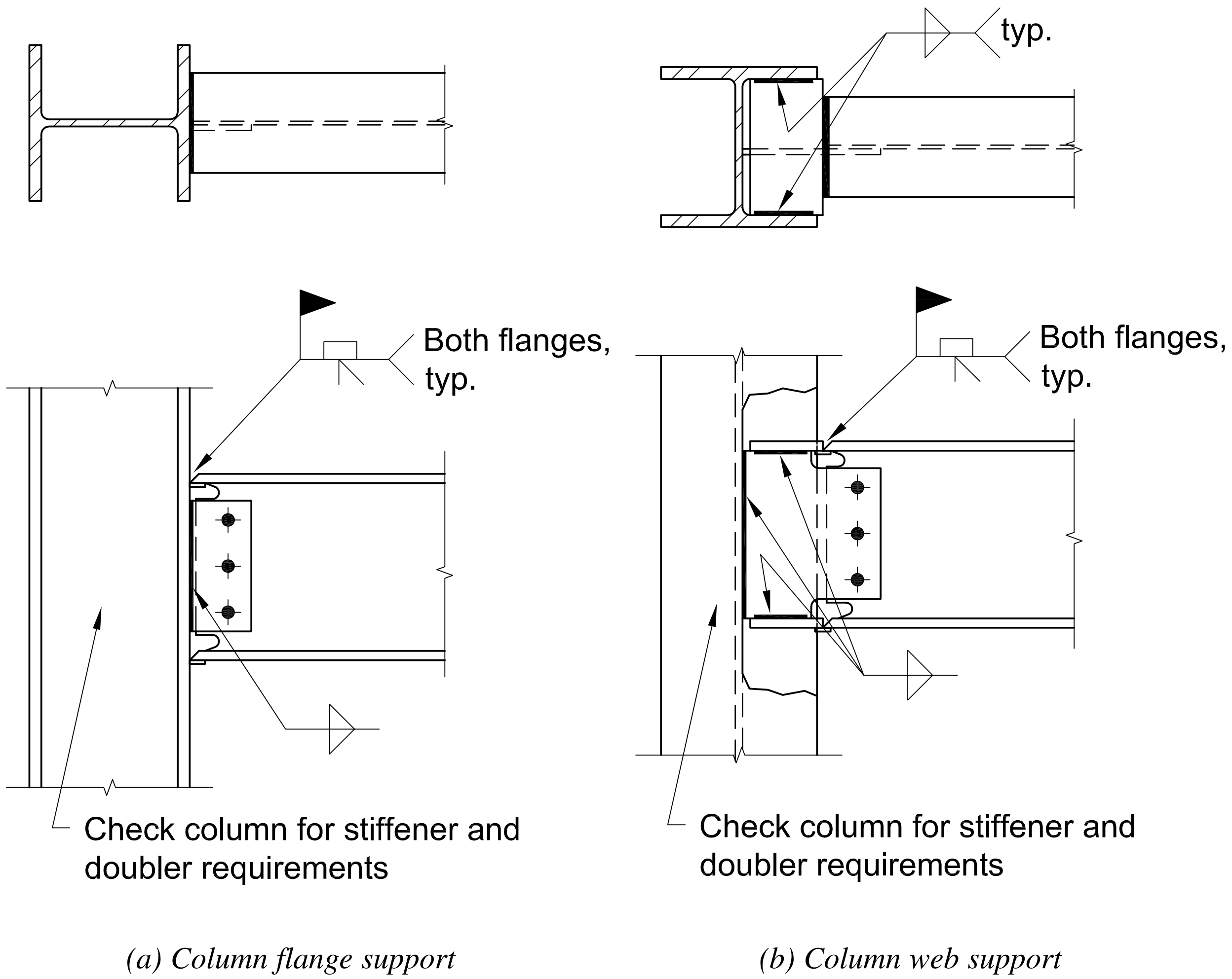


Fig. 12-4. Directly welded flange FR moment connections.

Extended End-Plate FR Moment Connections

As illustrated in Figure 12-5, an extended end-plate moment connection consists of a plate of length greater than the beam depth, perpendicular to the longitudinal axis of the supported beam. The end plate is always welded to the web and flanges of the supported beam and bolted to the supporting member. The principal advantage of extended end-plate moment connections is that all welding is done in the shop; thus, the erection process is relatively fast and economical.

Figure 12-6 illustrates three commonly used extended end-plate connections. The connections are classified by the number of bolts at the tension flange and by the presence of end-plate to beam flange stiffeners. The four-bolt unstiffened and stiffened extended end-plate connections, 4E and 4ES, of Figure 12-6(a) and 12-6(b) are generally limited by bolt strength and can be designed to develop the flexural design strength of nearly one-half of the available beam sections. Alternatively, the eight-bolt stiffened extended end-plate connection, 8ES, shown in Figure 12-6(c) can generally be designed to develop the flexural design strength of approximately 90% of the available beam sections.

A complete discussion of the design procedures, along with design examples for the 4E, 4ES and 8ES connections, are found in AISC Design Guide 4, *Extended End-Plate Moment Connections—Seismic and Wind Applications* (Murray and Sumner, 2003). Design procedures and example calculations for the 4E, 4ES and seven other end-plate connections, which are commonly used in the metal building industry, are found in AISC Design Guide 16, *Flush and Extended Multiple-Row Moment End-Plate Connections* (Murray and Shoemaker, 2002). The design procedures in both AISC Design Guides 4 and 16 are based on

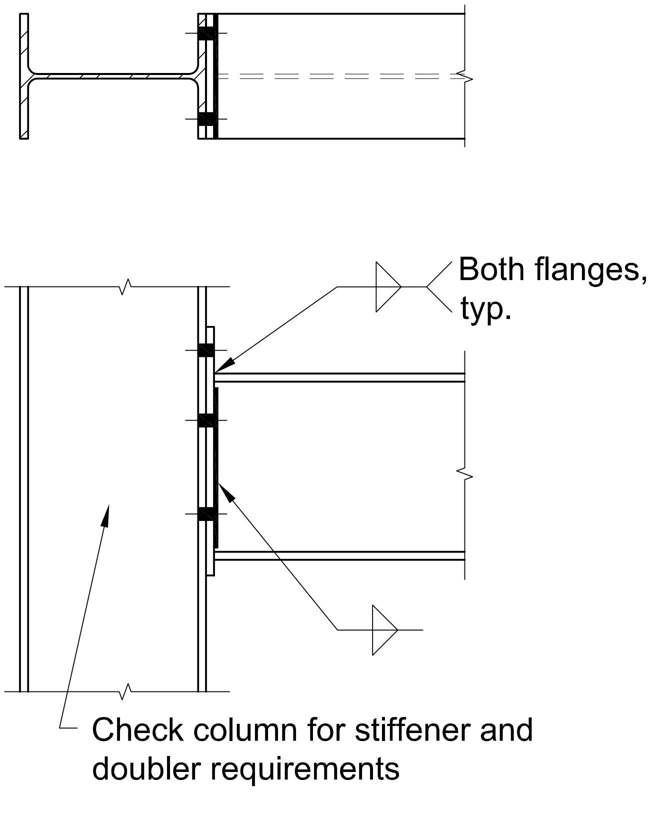


Fig. 12-5. Extended end-plate FR moment connection.

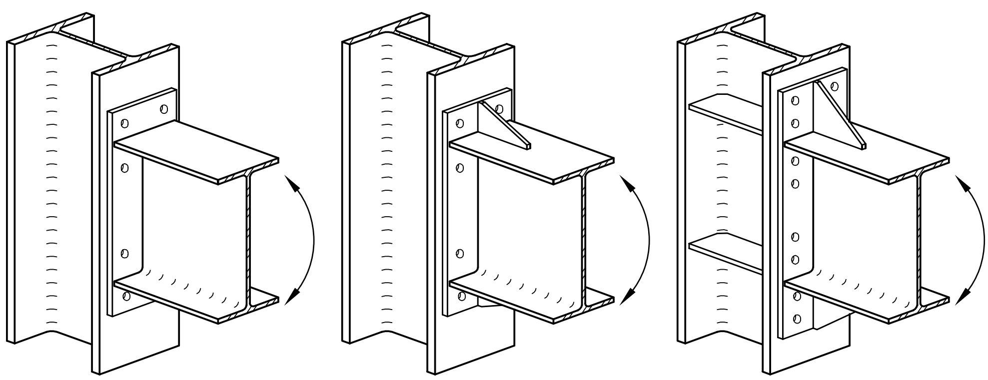
yield-line analysis for determining end-plate thickness and modified tee-hanger analysis to determine required bolt strength. The procedures in AISC Design Guide 4 are for pretensioned bolts and "thick plates" and result in connections with the smallest possible bolt diameter. For these connections, prying forces are zero. The procedures in AISC Design Guide 16 allow for both "thick plate" and "thin plate" designs. A thin plate design results in the smallest possible end-plate thickness and the maximum bolt prying force. These connections can be designed using either pretensioned or snug-tight bolts, if Group A bolts are used. Group B bolts must be pretensioned. Column side design procedures are included in AISC Design Guide 4. Recommended shop and field erection practices and basic design assumptions follow.

Shop and Field Practices

End-plate moment connections require extra care in shop fabrication and field erection. The fit-up of extended end-plate connections is sensitive to the column flange conditions and may be affected by column flange-to-web squareness, beam camber, or squareness of the beam end. The beam is frequently fabricated short to accommodate the column overrun tolerances with shims furnished to fill any gaps which might result.

As reported by Meng and Murray (1997), use of weld access holes can result in beam flange cracking, especially in high-seismic applications. If CJP groove welds are used, the weld cannot be inspected over the web; however, because this location is a relative "soft" spot in the connection, the weld can be considered to be an uninspected partial-joint-penetration (PJP) groove weld.

The heat from welding can cause the end plates to distort. Finger shims are an option to address gaps, and tests have shown that the use of finger shims between the end plate and the column flange do not affect the performance of the connection (Sumner et al., 2000). The erector should exercise judgment and may elect to pull the plies together when they are bolted. Using the bolts to pull the plies together in this manner will not reduce the strength of the bolts relative to applied shear or tension.



(a) 4E *(b) 4ES* *(c) 8ES*

Fig. 12-6. Configurations of extended end-plate FR moment connections.

Design Assumptions

A summary of the assumptions made in AISC Design Guides 4 and 16 procedures follows:

1. Group A or Group B high-strength bolts of a diameter not greater than 1½ in. must be used.
2. The specified minimum yield stress of the end-plate material must be 50 ksi or less.
3. The procedures in AISC Design Guide 4 are applicable to static loads and the design of ordinary moment frames under the AISC *Seismic Provisions*. (Static loadings are considered to be wind, snow, temperature and low-seismic loadings.) For high-seismic loading, the procedures in ANSI/AISC 358 supersede those in Design Guide 4. When the procedures in AISC Design Guide 16 are used, only static loading is permitted.
4. The recommended minimum distance from the face of the beam flange to the nearest bolt centerline (the vertical bolt pitch) is the bolt diameter, d_b, plus ½ in. if the bolt diameter is not greater than 1 in., and plus ¾ in. for larger diameter bolts. However, many fabricators prefer to use a standard pitch dimension of 2 in. or 2½ in. for all bolt diameters.
5. All of the shear force at a connection is assumed to be resisted by the compression side bolts. End-plate connections need not be designed as slip-critical connections and it is noted that shear is rarely a major concern in the design of moment end-plate connections.
6. The end-plate width effective in resisting the applied moment must be taken as not greater than the beam flange width, b_f, plus 1 in., or the end-plate thickness, whichever is greater.
7. The gage of the tension bolts (horizontal distance between vertical bolt lines) must not exceed the beam tension flange width.
8. When CJP groove welds are used, weld access holes should not be used, and the weld between beam flange-to-web fillets should be treated as a PJP groove weld relative to fabrication.
9. For static and low-seismic loadings, normally the flange to end-plate weld is designed to develop the yield strength of the connected beam flange. This is generally done with CJP groove welds but, alternatively, fillet welds or any combination of groove and fillet welds may be used. When the required moment is less than the available flexural strength of the beam, the beam flange to end-plate connections can be designed for the required moment, but it is recommended that the connections be designed for not less than 60% of the available flexural strength of the beam. This minimum demand is intended to account for uneven stress distributions that can occur across the flange at end-plate welds. Beam web to end-plate welds in the vicinity of the tension bolts should be designed using the same strength requirements as for the design of the flange to end-plate welds.
10. Only the web to end-plate weld between the mid-depth of the beam and the inside face of the beam compression flange, or the weld between the inner row of tension bolts plus two times the bolt diameter, $2d_b$, and the inside face of the beam compression flange, whichever is smaller, is considered effective in resisting the beam end shear.

FR MOMENT SPLICES

Beams and girders sometimes are spliced in locations where both shear and moment must be transferred across the splice. Per AISC *Specification* Section J6, the nominal strength of the smaller section being spliced must be developed in groove-welded butt splices. Other types of beam or girder splices must develop the strength required by the actual forces at the point of the splice.

Location of Moment Splices

A careful analysis is particularly important in continuous structures where a splice may be located at or near the point of inflection. Since this inflection point can and does migrate under service loading, actual forces and moments may differ significantly from those assumed. Furthermore, since loading application and frequency can change in the lifetime of the structure, it is prudent for the designer to specify some minimum strength requirement at the splice. Hart and Milek (1965) propose that splices in fixed-ended beams be located at the one-sixth point of the span and be adequate to resist a moment equal to one-sixth of the flexural strength of the member, as a minimum.

Force Transfer in Moment Splices

Force transfer in moment splices can be assumed to occur in a manner similar to that developed for FR moment connections. That is, the required shear, R_u or R_a, is primarily transferred through the beam-web connection and the moment can be resolved into an effective tension-compression couple where the required force at each flange, P_{uf} or P_{af}, is determined by:

LRFD	ASD
$$P_{uf} = \frac{M_u}{d_m} \quad (12\text{-}2a)$$	$$P_{af} = \frac{M_a}{d_m} \quad (12\text{-}2b)$$

where

M_u or M_a = required moment in the beam at the splice, kip-in.

d_m = moment arm, in. (varies based upon actual connection geometry)

Axial forces, if present, are normally assumed to be distributed uniformly across the beam flange cross-sectional area. However, if the beam-web connection has sufficient stiffness, it can also be assumed to participate in the transfer of beam axial force.

Flange-Plated FR Moment Splices

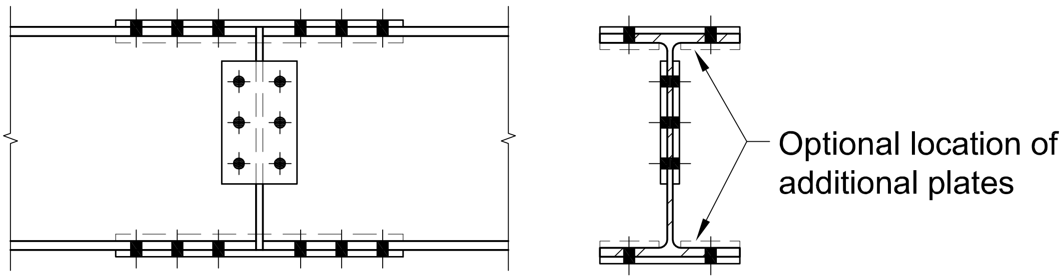
Moment splices can be designed as shown in Figure 12-7, to utilize flange plates and a web connection. The flange plates and web connection may be bolted or welded.

The splice and spliced beams should be checked in a manner similar to that described previously under "Flange-Plated FR Moment Connections," except that the web connection should be designed as illustrated previously for shear splices in Part 10 without consideration of eccentricity.

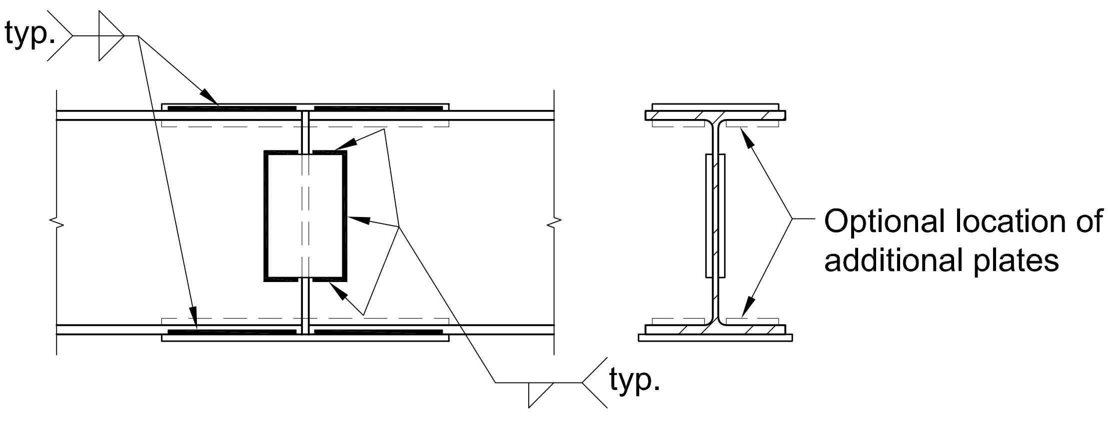
Figure 12-7 illustrates two types of splices—bolted and welded. Figure 12-7(a) illustrates the detail of a bolted flange-plated moment splice. For this case, the flange plates are normally made approximately the same width as the beam flange as shown in Figure 12-7(a).

Alternatively, Figure 12-7(b) illustrates the detail of a welded splice. As shown in Figure 12-7(b), the top plate is narrower and the bottom plate is wider than the beam flange, permitting the deposition of weld metal in the downhand or horizontal position without inverting the beam. While this is a benefit in shop fabrication (the beam does not have to be turned over), it is of extreme importance in the field where the weld can be made in the horizontal instead of the overhead position, since the beam cannot be turned over. This detail also provides tolerance for field alignment, since the joint gap can be opened or closed. When splices are field-welded, some means for temporary support must be provided as discussed previously in "Temporary Support During Erection."

If the beam or girder flange is thick and the flange forces are large, it may be desirable to place additional plates on the insides of the flanges. In a bolted splice [Figure 12-7(a)], the bolts are then loaded in double shear and a more compact joint may result. Note that these additional plates must have sufficient area to develop their share of the double-shear bolt load.



(a) Bolted



(b) Welded

Fig. 12-7. Flange-plated moment splice.

In a welded splice [Figure 12-7(b)], these additional plates must have sufficient area to match the strength of the welds that connect them. Additionally, these plates must be set away from the beam web a distance sufficient to permit deposition of weld metal as shown in Figure 12-8(a). This distance is a function of the beam depth and flange width, as well as the welding equipment to be used. A distance of 2 to $2^1/_2$ in. or more may be required for this access. One alternative is to bevel the bottom edge of the plate to clear the beam fillet and place the plate tight to the beam web with a fillet weld as illustrated in Figure 12-8(a). The effects of this bevel on the area of the plate must be considered in determining the required plate width and thickness. Another alternative would be to use unbeveled inclined plates as shown in Figure 12-8(b).

Directly Welded Flange FR Moment Splices

Moment splices can be designed, as shown in Figure 12-9, to utilize a CJP groove weld connecting the flanges of the members being spliced. The web connection may then be bolted or welded. The splice and spliced beams should be checked in a manner similar to that described previously under "Directly Welded Flange FR Moment Connections," except that the web connection should be designed as illustrated previously for shear splices in Part 10.

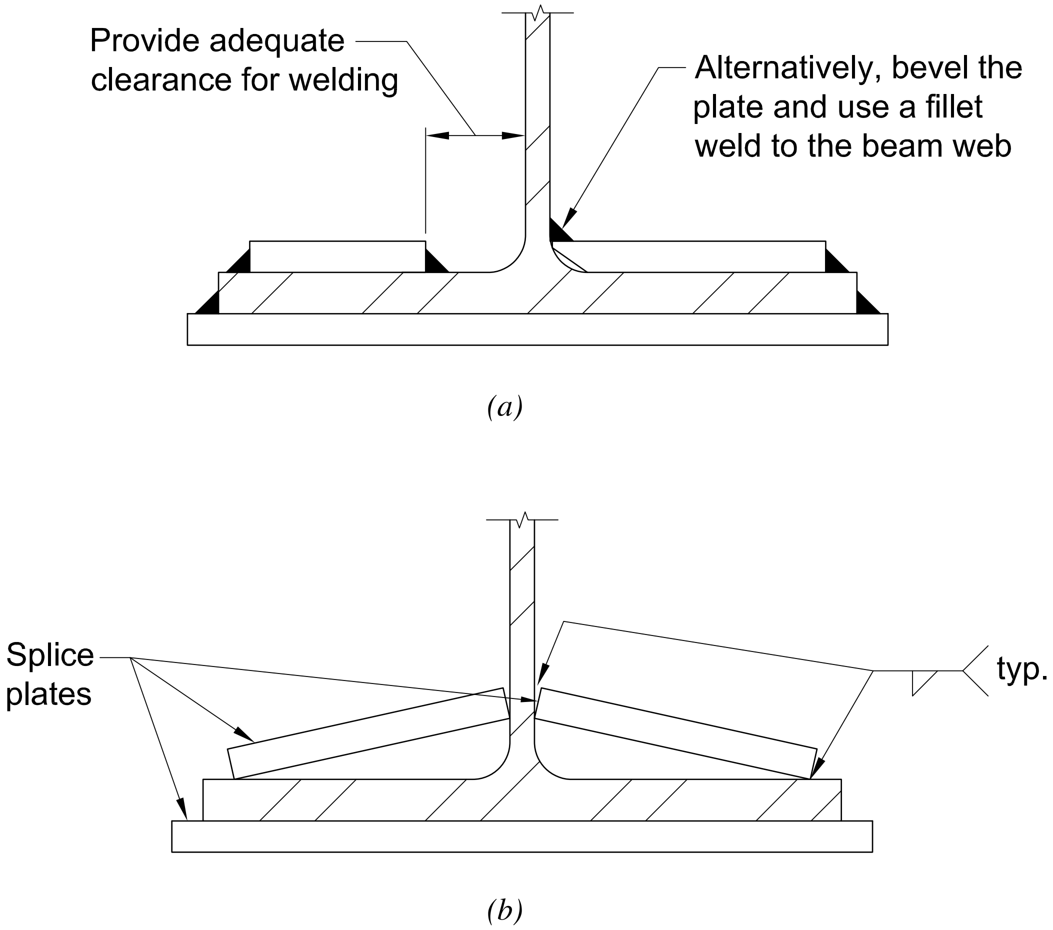


Fig. 12-8. Welding clearances for flange-plated moment splices.

Although rare in occurrence, some spliced members must be level on top. Where the depths of these spliced members differ, consideration should be given to the use of a flange plate of uniform thickness for the full length of the shallower member. This avoids the fabrication problems created by an inverted transition.

Frequently, the spliced shapes are different sizes, but of the same shape depth grouping. Because rolled shapes from the same shape depth grouping have the same dimension between the flanges, aligning the inside flange surfaces avoids a more difficult offset transition. Eccentricity resulting from differing flange thicknesses is usually ignored in the design. The web plates normally are aligned to their centerlines.

The groove- (butt-) welded splice preparation shown in Figure 12-9 may be used for either shop or field welding. Alternatively, for shop welding where the beam may be turned over, the joint preparation of the bottom flange could be inverted.

Sloped transitions as shown in Figure 12-10 are only required for splices subject to seismic and dynamic loads. In splices subjected to dynamic or fatigue loading, the backing bar should be removed and the weld should be ground flush when it is normal to the applied stress (AISC, 1977). The access holes should be free of notches and should provide a smooth transition at the juncture of the web and flange.

Extended End-Plate FR Moment Splices

Moment splices loaded in one direction can be designed as shown in Figure 12-11 where a four-bolt unstiffened end-plate configuration is utilized to connect the tension flanges. It is usually possible to design this type of connection to reach the full plastic moment capacity of the beam, $\phi_b M_p$ or M_p / Ω_b.

The splice and spliced beams should be checked in a manner similar to that described previously under “Extended End-Plate FR Moment Connections.” The comments in that section are equally applicable to end-plate moment splices.

SPECIAL CONSIDERATIONS

FR Moment Connections to Column Webs

It is frequently required that FR moment connections be made to column webs. While the mechanics of analysis and design do not differ from FR moment connections to column

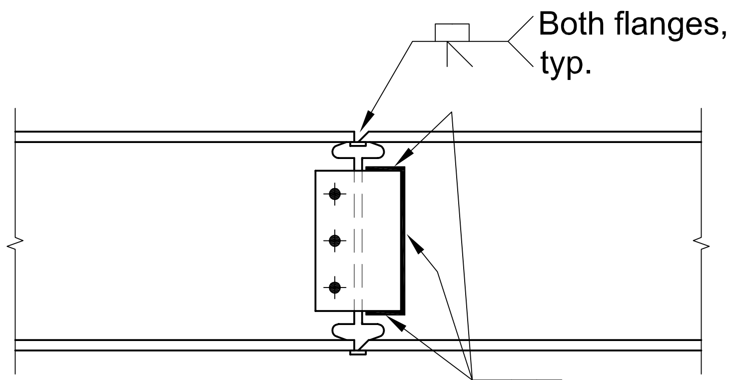


Fig. 12-9. Directly welded flange moment splice.

flanges, the details of the connection design as well as the ductility considerations required are significantly different.

Recommended Details

When an FR moment connection is made to a column web, it is normal practice to stop the beam short and locate all bolts outside of the column flanges as illustrated in Figure 12-2(b). This simplifies the erection of the beam and permits the use of an impact wrench to tighten all bolts. It is also preferable to locate welds outside the column flanges to provide adequate clearance.

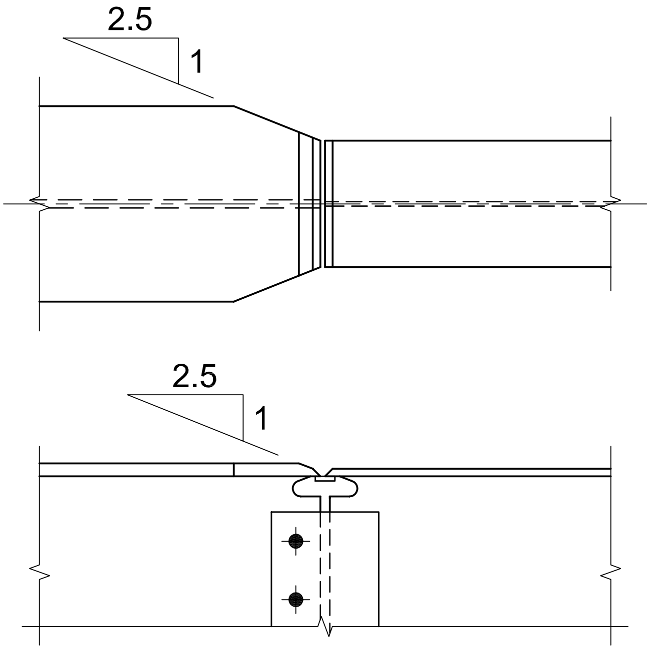


Fig. 12-10. Transitions at tension flange for directly welded flange moment splices, for seismically and dynamically loaded splices.

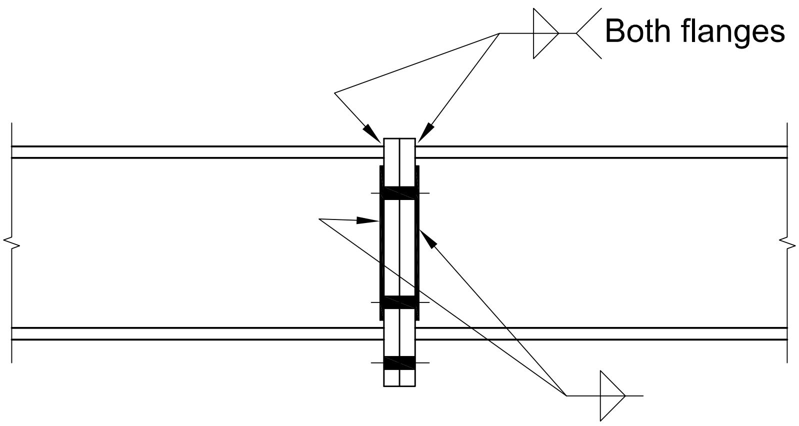


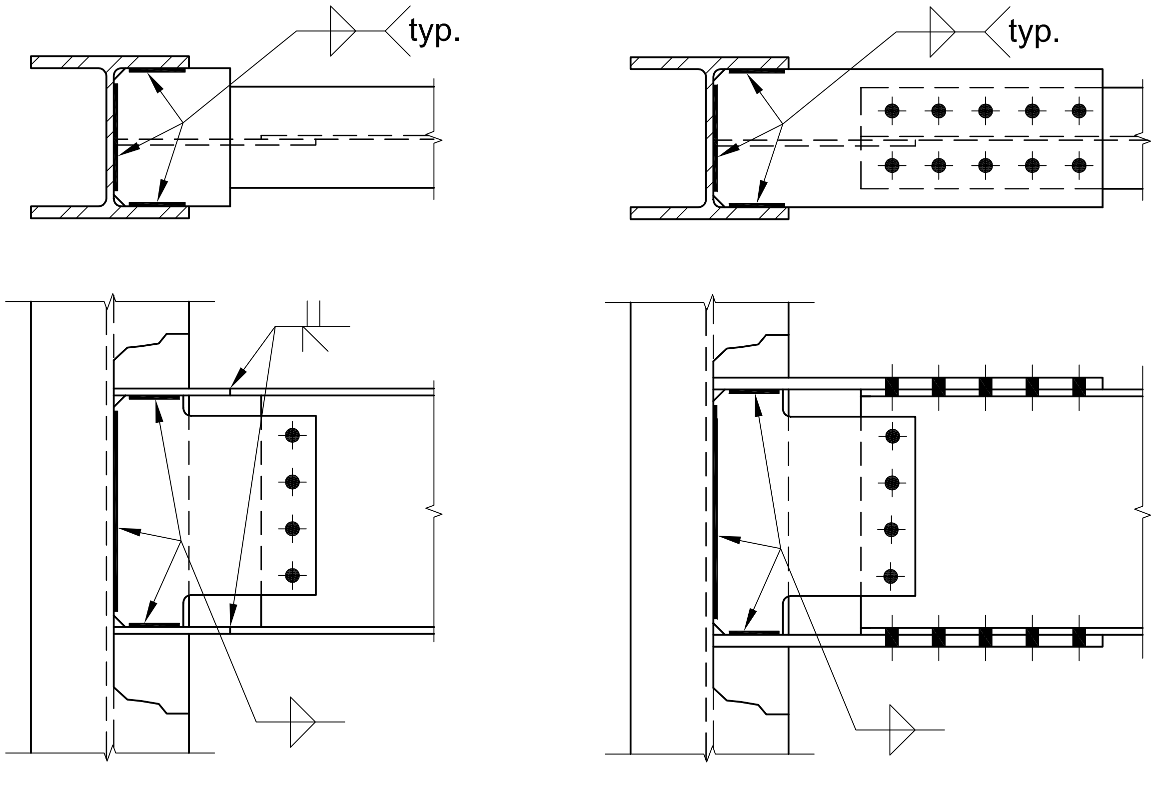
Fig. 12-11. Extended end-plate moment splice.

Ductility Considerations

Driscoll and Beedle (1982) discuss the testing and failure of two FR moment connections to column webs: a directly welded flange connection and a bolted flange-plated connection, shown respectively in Figures 12-12(a) and 12-12(b). Although the connections in these tests were proportioned to be critical, they were expected to provide inelastic rotations at full plastic load. Failure occurred unexpectedly, however, on the first cycle of loading; brittle fracture occurred in the tension connection plate at the load corresponding to the plastic moment before significant inelastic rotation had occurred.

Examination and testing after the unexpected failure revealed that the welds were of proper size and quality and that the plate had normal strength and ductility. The following is quoted, with minor editorial changes relative to figure numbers, directly from Driscoll and Beedle (1982).

> Calculations indicate that the failures occurred due to high strain concentrations. These concentrations are: (1) at the junction of the connection plate and the column flange tip and (2) at the edge of the butt weld joining the beam flange and the connection plate.
>
> Figure 4 (Figure 12-13 here) illustrates the distribution of longitudinal stress across the width of the connection plate and the concentration of stress in the plate at the column flange tips. It also illustrates the uniform longitudinal stress distribution in the connection plate at some distance away from the connection.



(a) Directly welded flange FR connection

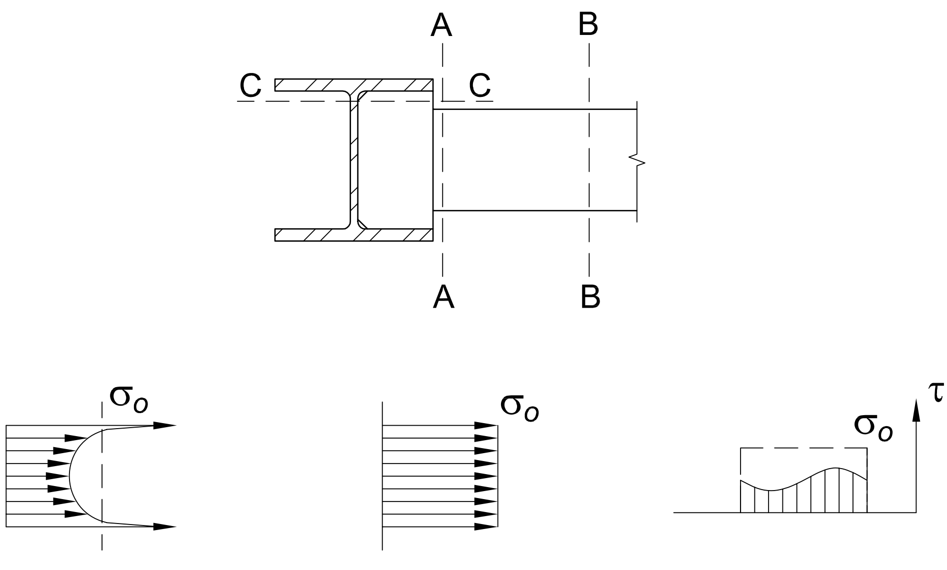
(b) Bolted flange-plated FR connection

Fig. 12-12. Test specimens used by Driscoll and Beedle (1982).

The stress distribution shown represents schematically the values measured during the load tests and those obtained from finite element analysis. (σ_o is a nominal stress in the elastic range.) The results of the analyses are valid up to the loading that causes the combined stress to equal the yield point. Furthermore, the analyses indicate that localized yielding could begin when the applied uniform stress is less than one-third of the yield point. Another contribution of the non-uniformity is the fact that there is no back-up stiffener. This means that the welds to the web near its center are not fully effective.

The longitudinal stresses in the moment connection plate introduce strains in the transverse and the through-thickness directions (the Poisson effect). Because of the attachment of the connection plate to the column flanges, restraint is introduced; this causes tensile stresses in the transverse and the through-thickness directions. Thus, referring to Figure 12-13, tri-axial tensile stresses are present along Section A-A and they are at their maximum values at the intersections of Sections A-A and C-C. In such a situation, and when the magnitudes of the stresses are sufficiently high, materials that are otherwise ductile may fail by premature brittle fracture.

The results of nine simulated weak-axis FR moment connection tests performed by Driscoll et al. (1983) are summarized in Figure 12-14. In these tests, the beam flange was simulated by a plate measuring either 1 in. × 10 in. or $1^1/_8$ in. × 9 in. The fracture strength exceeds the yield strength in every case, and sufficient ductility is provided in all cases except for that of Specimen D. Also, if the rolling direction in the first five specimens (A, B, C, D and E) were parallel to the loading direction, which would more closely approximate an actual beam flange, the ductility ratios for these would be higher. The connections with extended connection plates (i.e., projection of 3 in.), with extensions either rectangular or tapered, appeared equally suitable for the static loads of the tests.



(a) Longitudinal stress distribution on Section A-A

(b) Longitudinal stress distribution on Section B-B

(c) Shear stress distribution on Section C-C

Fig. 12-13. Stress distributions in test specimens used by Driscoll and Beedle (1982).

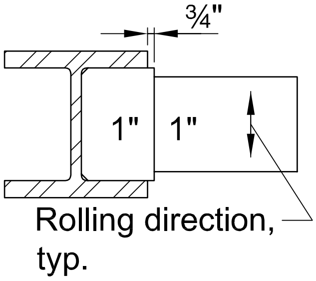
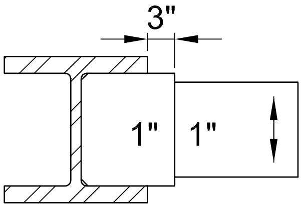
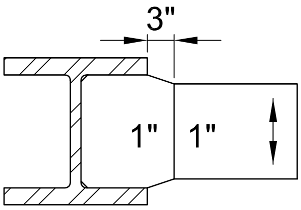
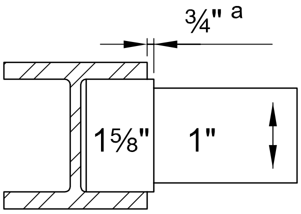
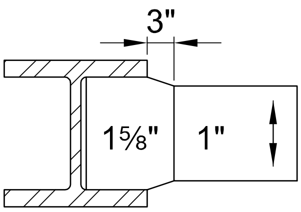
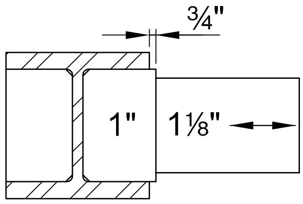
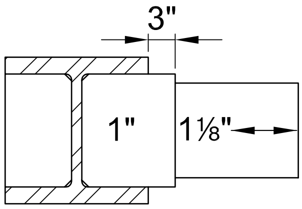
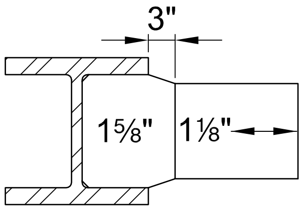
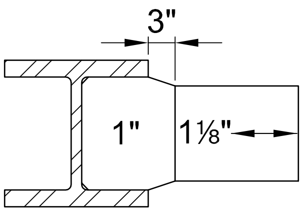
Specimen No.	Sketch W14x257 (typical)	Fracture Load (kips)	Fracture Load / Yield Load	Ductility Ratio
A		730	1.38	6.3
B		824	1.55	5.3
C		756	1.43	5.43
D		570	1.11	1.71
E		802	1.51	6.81

Fig. 12-14. Results of weak-axis FR moment connection ductility tests performed by Driscoll et al. (1983).

Specimen No.	Sketch W14x257 (typical)	Fracture Load (kips)	Fracture Load / Yield Load	Ductility Ratio
A2		762	1.40	17.7
B2		795	1.46	16.5
E2		814	1.49	16.4[b]
C2		813	1.49	29.6

Notes: [a] 3/4” dimension is estimated—no dimension provided in Driscoll et al. (1983).
[b] Ductility ratio estimated. Actual value not known due to malfunction in deflection gauge.

Fig. 12-14 (continued). Results of weak-axis FR moment connection ductility tests performed by Driscoll et al. (1983).

Based on the tests, Driscoll et al. (1983) report that those specimens with extended connection plates have better toughness and ductility and are preferred in design for seismic loads, even though the other connection types (except D) may be deemed adequate to meet the requirements of many design situations.

In accordance with the preceding discussion, the following suggestions are made regarding the design of this type of connection:

1. For directly welded (butt) flange-to-plate connections, the connection plate should be thicker than the beam flange. This greater area accounts for shear lag and also provides for misalignment tolerances.

   AWS D1.1 clause 5.21.3 restricts the misalignment of abutting parts such as this to 10% of the thickness, with 1/8-in. maximum for a part restrained against bending due to eccentricity of alignment. Considering the various tolerances in mill rolling (± 1/8 in. for W-shapes), fabrication and erection, it is prudent design to call for the connection plate thickness to be increased to accommodate these tolerances and avoid the subsequent problems encountered at erection. An increase of 1/8 in. to 1/4 in. generally is used.

   Frequently, this connection plate also serves as the stiffener for a strong-axis FR or PR moment connection. The welds that attach the plate/stiffener to the column flange may then be subjected to combined tensile and shearing, or compression and shearing forces. Vector analysis is commonly used to determine weld size and stress.

   It is good practice to use fillet welds whenever possible. Welds should not be made in the column *k*-area.
2. The connection plate should extend at least 3/4 in. beyond the column flange to avoid intersecting welds and to provide for strain elongation of the plate. The extension should also provide adequate room for runoff tabs when required.
3. Tapering an extended connection plate is only necessary when the connection plate is not welded to the column web (Specimen E, Figure 12-14). Tapering is not necessary if the flange force is always compressive (e.g., at the bottom flange of a cantilevered beam).
4. To provide for increased ductility under seismic loading, a tapered connection plate should extend 3 in. Alternatively, a backup stiffener and an untapered connection plate with 3-in. extension could be used.

Normal and acceptable quality of workmanship for connections involving gravity and wind loading in building construction would tolerate the following:

1. Runoff tabs and backing bars may be left for beams with flange thicknesses greater than 2 in. (subject to tensile stress only) where they are welded to columns or used as tension members in a truss.
2. Welds need not be ground, except as required for nondestructive testing.
3. Connection plates that are made thicker or wider for control of tolerances, tensile stress and shear lag need not be ground or cut to a transition thickness or width to match the beam flange to which they connect.
4. Connection plate edges may be sheared, or plasma- or gas-cut.

5. Intersections and transitions may be made without fillets or radii.
6. Flame-cut edges may have reasonable roughness and notches within AWS tolerances.

If a structure is subjected to loads other than gravity and wind loads, such as seismic, dynamic or fatigue loading, more stringent control of the quality of fabrication and erection with regard to stress risers, notches, transition geometry, welding and testing may be necessary; refer to the AISC *Seismic Provisions*.

FR Moment Connections Across Girder Supports

Frequently, beam-to-girder-web connections must be made continuous across a girder-web support, as with continuous beams and with cantilevered beams at wall, roof-canopy or building lines. While the same principles of force transfer discussed previously for FR moment connections may be applied, the designer must carefully investigate the relative stiffness of the assembled members being subjected to moment or torsion and provide the fabricator and erector with reliable camber ordinates.

Additionally, the design should still provide some means for final field adjustment to accommodate the accumulated tolerances of mill production, fabrication and erection; it is very desirable that the details of field connections provide for some adjustment during erection. Figure 12-15 illustrates several details that have been used in this type of connection and the designer may select the desirable components of one or more of the sketches to suit a particular application. Therefore, these components are discussed here as a top flange, bottom flange and web connection.

Top Flange Connection

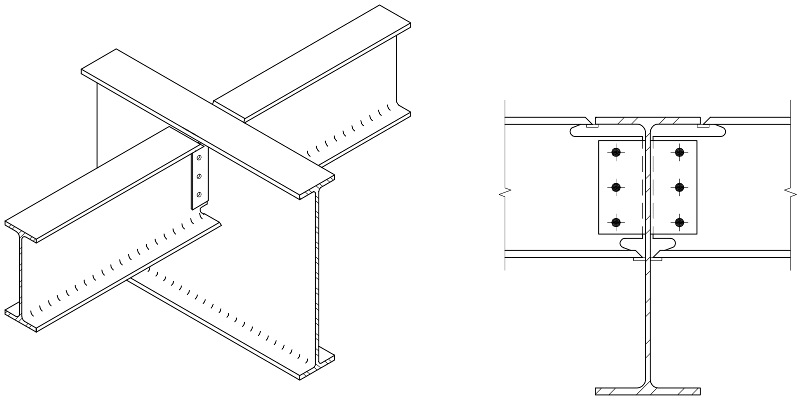
As shown in Figure 12-15(a), the top flange connection may be directly welded to the top flange of the supporting girder. Figures 12-15(b) and 12-15(c) illustrate an independent splice plate that ties the two beams together by use of a longitudinal fillet weld or bolts. This tie plate does not require attachment to the girder flange, although it is sometimes so connected to control noise if the connection is subjected to vibration.

Bottom Flange Connection

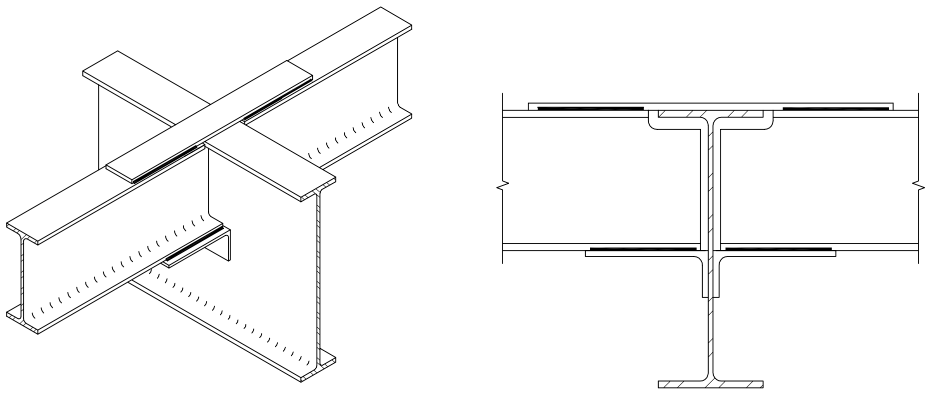
When the bottom flanges deliver a compressive force only, the flange forces are frequently developed by directly welding these flanges to the girder web as illustrated in Figure 12-15(a). Figure 12-15(b) illustrates the use of an angle or channel below the beam flange to provide for a horizontal fillet weld. The angle or channel should be wider than the beam flange to allow for downhand welding. Figure 12-15(c) is similar, but uses bolts instead of welds to develop the flange force.

Web Connection

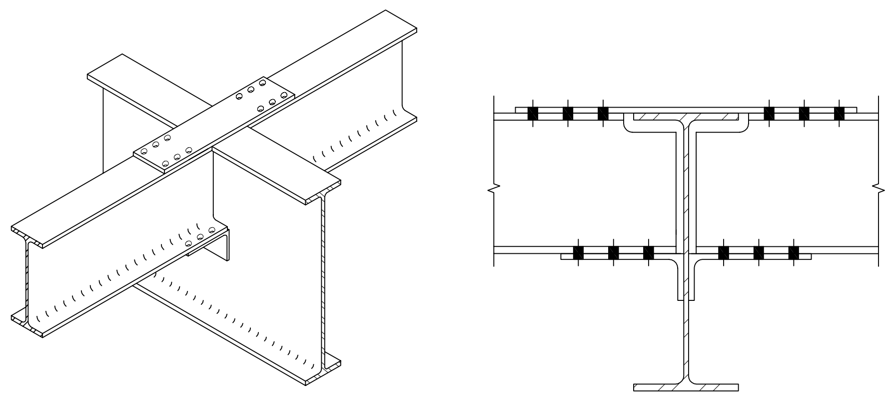
While a single-plate connection is shown in Figure 12-15(a) and unstiffened seated connections are shown in Figures 12-15(b) and 12-15(c), any of the shear connections in Part 10 may be used. Note that the effect of eccentricity in the shear connection may be neglected.



(a)



(b)



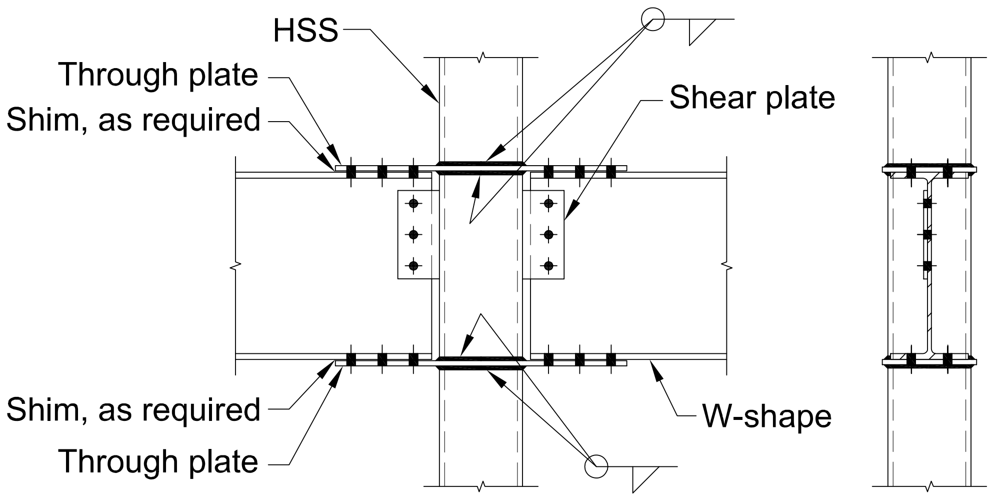
(c)

Fig. 12-15. FR moment connections across girder-web supports.

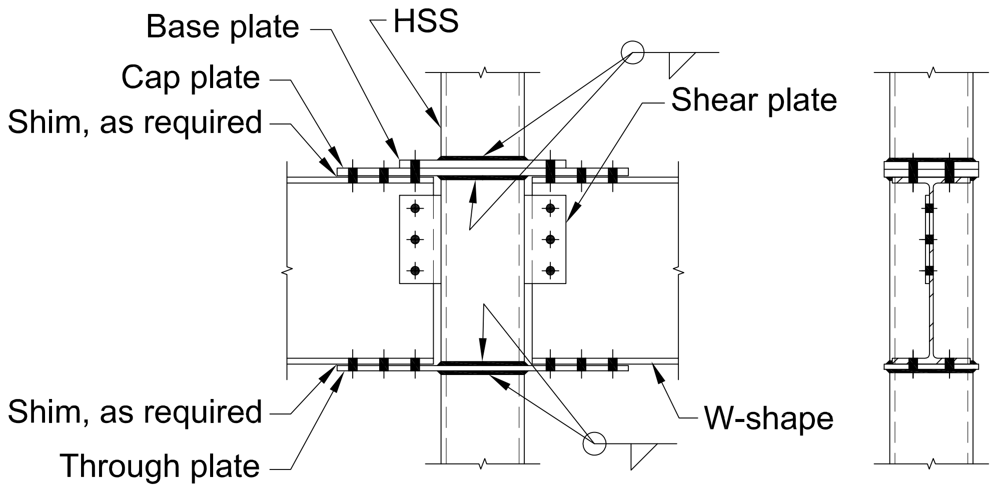
FR CONNECTIONS WITH HSS

HSS Through-Plate Flange-Plated FR Moment Connections

If the required moment transfer to the column is larger than can be provided by the bolted base plate or cap plate, or if the hollow structural section (HSS) width is larger than that of the wide-flange beam, a through-plate moment connection can be used as illustrated in Figure 12-16. It should be noted that through-plate connections are more difficult to erect than the continuous beam connected framing.



(a) Between column splices



(b) At column splice

Fig. 12-16. Through-plate moment connection.

When moment connections are made using through-plates, such as is shown in Figure 12-16, the fabricator must allow adequate clearance between the through-plates and the structural section W-shape so as to allow for the combined effects of mill, fabrication and erection tolerances. The permissible mill tolerances for W-shape variations in depth and squareness are shown in Table 1-22. Shimming in the field during erection with conventional shims or finger shims is the most commonly used method to fill the gap between the W-shape and the through-plate.

Specific design considerations for through-plate moment connections are as follows:

1. In Figures 12-16(a) and 12-16(b), the column moment transfer into the joint is limited by the fillet weld of the HSS to the through-plates. If necessary, a PJP groove weld can be used to improve the connection strength or a CJP groove weld with backing bars can be used.
2. In Figure 12-16, an end plate (base plate) is employed to create a splice in the column. Bolt tension with prying on the base plate will determine its thickness and thus limit the moment that can be transferred to the upper HSS.
3. The cap plate, which is also a flange splice plate, should be at least the same thickness as the base plate so that moment transfer between the HSS columns need not rely on load transfer through the beam flanges. The cap plate may need to be thicker than the HSS base plate due to the combined effect of plate bending from the bolted base plate and plate tension or compression from the wide-flange moment transfer.
4. The welding of the HSS to the cap and through-plate must be examined for both the HSS normal forces and the shear produced from the moment transfer from the W-shape.

HSS Cut-Out Plate Flange-Plated FR Moment Connections

An alternative to interrupting the HSS for the cover or through-plate is to use a wider plate with a cut-out to slip around the HSS, as illustrated in Figure 12-17. A shear plate can be placed on the front and rear of the HSS faces to provide simple connections for perpendicular beams. The cut-out plate can easily be extended on the near and far sides so that a moment splice is created about both horizontal axes through the joint. The perpendicular framing should ideally be of the same depth for this detail to work well or, in the case of the simple connections, the perpendicular beams could be shallower than the space between the horizontal plates. The cut-out plates are shown as shop-welded; however, they could be field-welded.

For cut-out plate connections, the erection of the beams is more difficult than for continuous beam connections. The beams must be slipped between the two plates and against the single-plate connection with shimming being required, unless the upper plate is field-welded in place.

Design Considerations for HSS Directly Welded FR Moment Connections

It may be possible to accomplish the moment transfer to the HSS without having to use a WT splice plate, end-plates, or diaphragm plates. Significant moment transfer can be achieved by attaching the W-shape directly to the face of the HSS, either by welding or by bolting. These connections are capable of developing the available flexural strength of the HSS. The available flexural strength of the W-shape, however, is seldom achieved because of the flexibility of the HSS wall.

The flexural strength for the welded W-shape is based on the strength of the respective flanges in tension and compression acting against the face of the HSS. This flange force can be considered to be the same as that of a plate with the dimensions of the flange.

Several design limit states exist for the plate length (flange width) oriented perpendicular to the length of the HSS (Packer and Henderson, 1997; Packer et al., 2010).

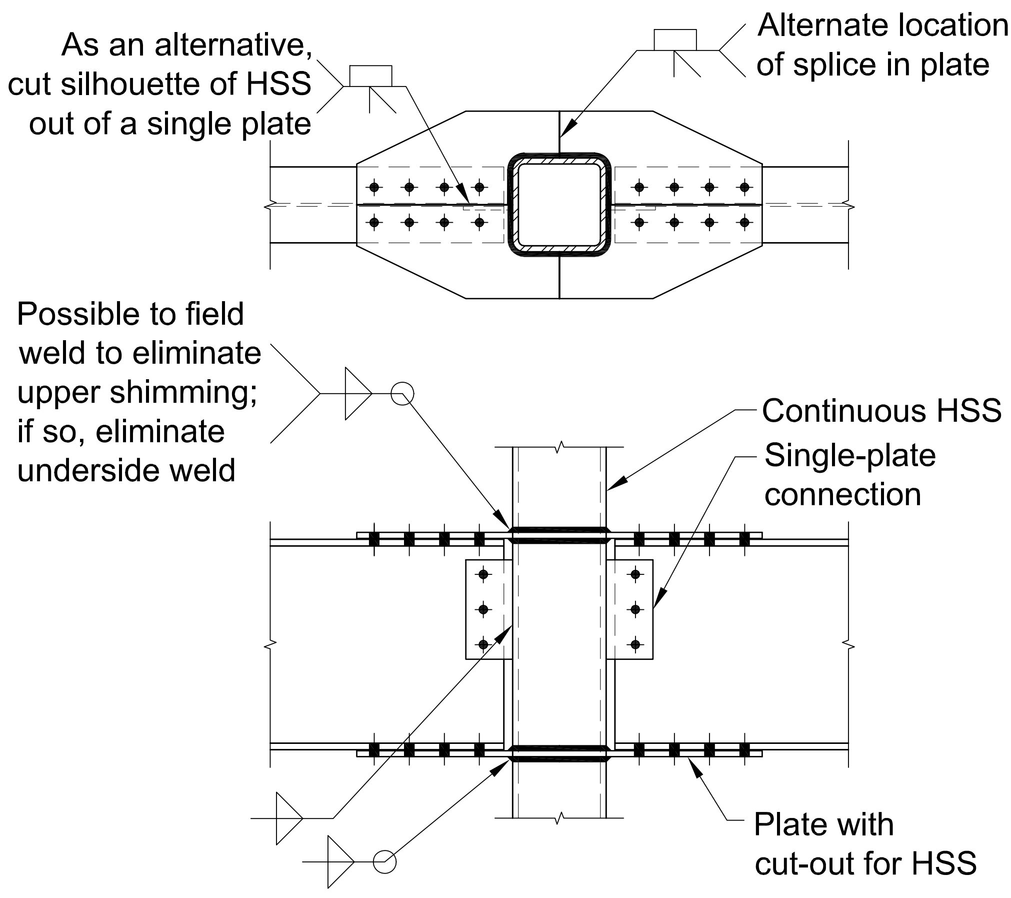


Fig. 12-17. Exterior plate moment connection.

HSS Columns Above and Below Continuous Beams

Field connection to the flanges of the beam and of continuous beams can be used at joints where there is an HSS above and below a continuous beam. This situation is illustrated in Figures 12-18 and 12-19. If the column load is not high, stiffener plates may be used to transfer the axial load across the beam as shown in Figure 12-18(a). If the axial load is higher, it may be necessary to use a split HSS instead of plate stiffeners, as shown in Figure 12-18(b). The width of the W-shape must be at least as wide as the HSS and should preferably be wider than the HSS for this detail to be used as shown. It may be necessary to use a rectangular HSS column in order to fit the HSS base plate on the beam flange. The moment transfer to the HSS is limited by the strength of the four bolts, the W-shape flange thickness, and the base and cap plate thicknesses.

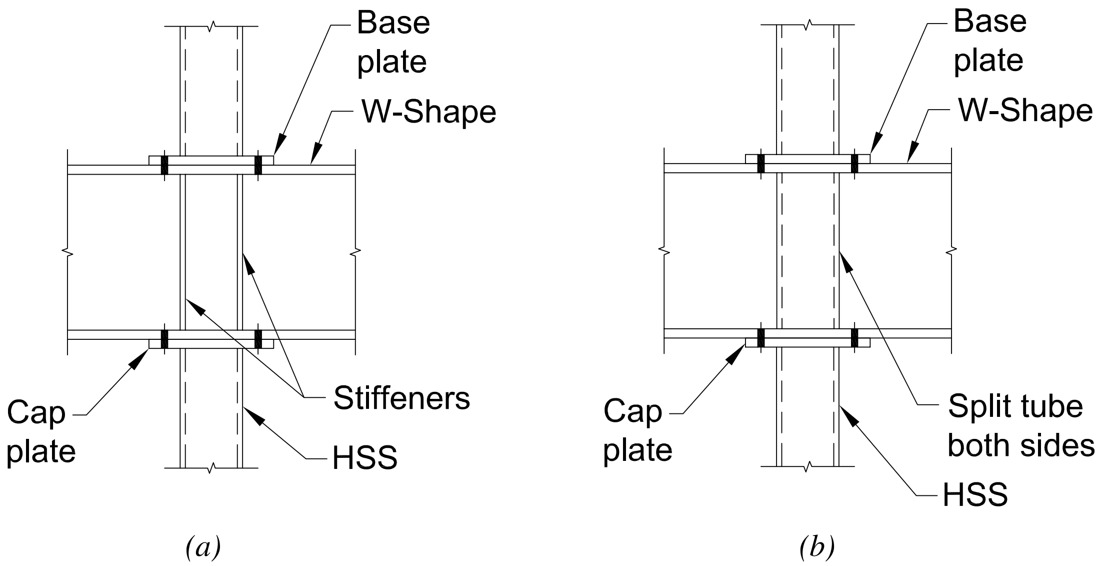


Fig. 12-18. HSS columns spliced to continuous beams.

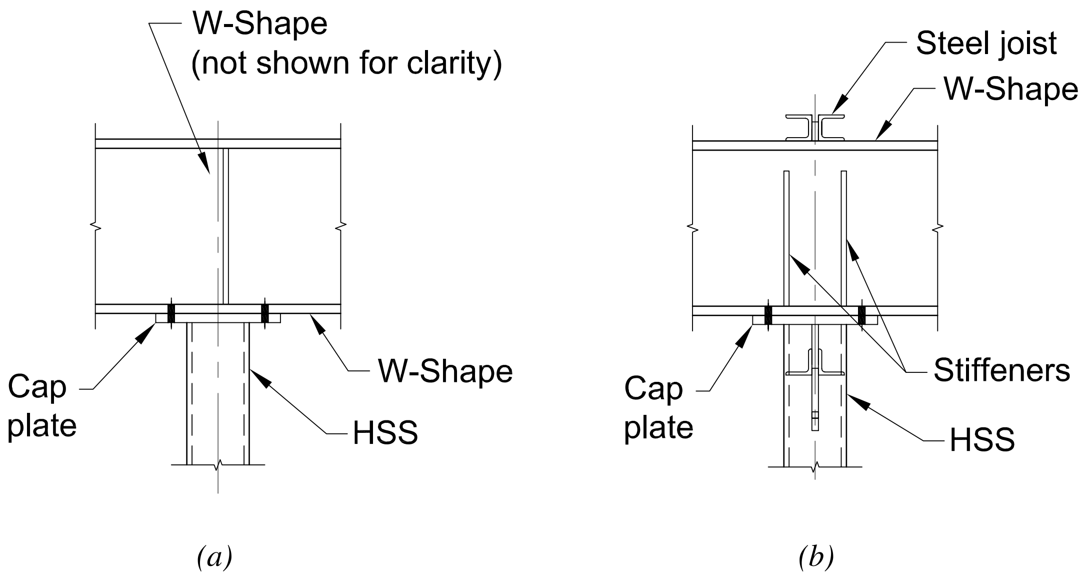


Fig. 12-19. Roof beam continuous over HSS column.

HSS Welded Tee Flange Connections

If the primary moment transfer is from a wide flange to an HSS, rather than through the HSS to another wide flange, a number of other connection concepts will work well. One of these is to use structural tee sections to transfer the force from the flanges of the wide flange to the walls of the HSS, as is illustrated in Figure 12-20. The tees should be long enough so that a flare bevel-groove (or single J-groove) weld with weld reinforcement can be used to connect the tee to the HSS. An alternative to using the vertical tee stiffener to transfer the beam shear would be to use a single-plate connection, if a deep enough plate can be fitted between the flanges of the tees.

HSS Diaphragm Plate Connections

If the moment delivered by the W-shape to the HSS cannot be transmitted by other means, then use of diaphragm plates that transfer the flange loads to the sides of the HSS is appropriate. This is illustrated in Figure 12-21. For this moment connection, the limit states are those indicated for the cut-out plate connection plus a check of the weld transferring shear from the flange plate to the HSS wall.

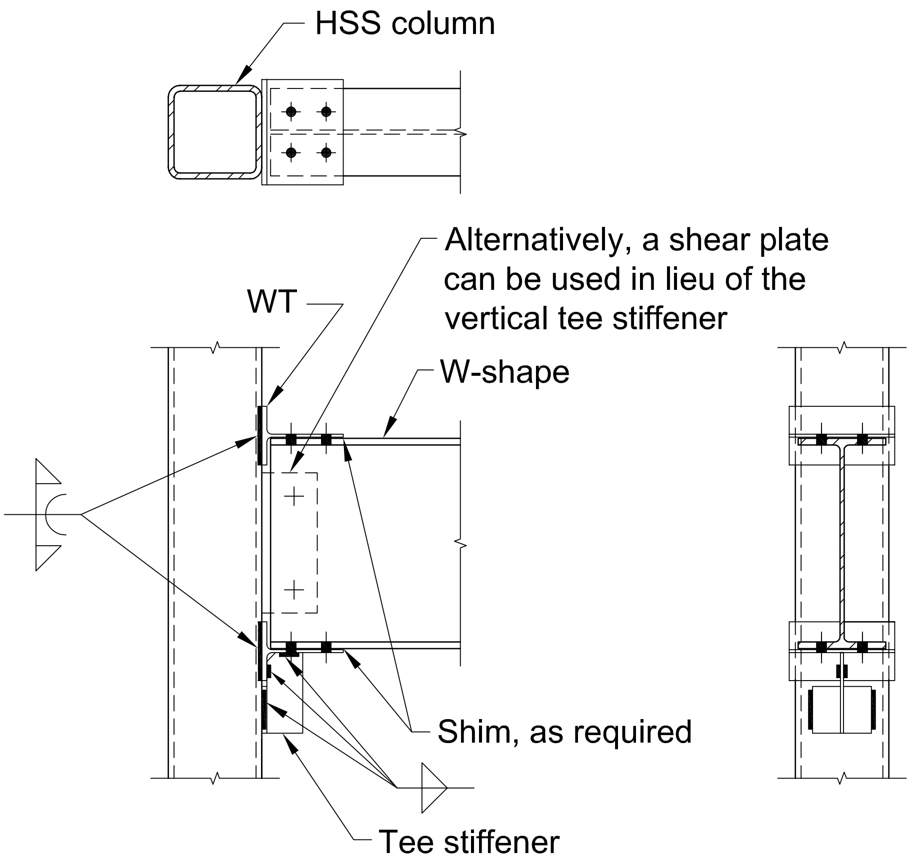


Fig. 12-20. Tee splice plates to HSS column.

Additional Suggested Details for HSS to Wide-Flange Moment Connections

The details shown in Figures 12-22 and 12-23 are suggested details only and are not intended to prohibit the use of other connection details.

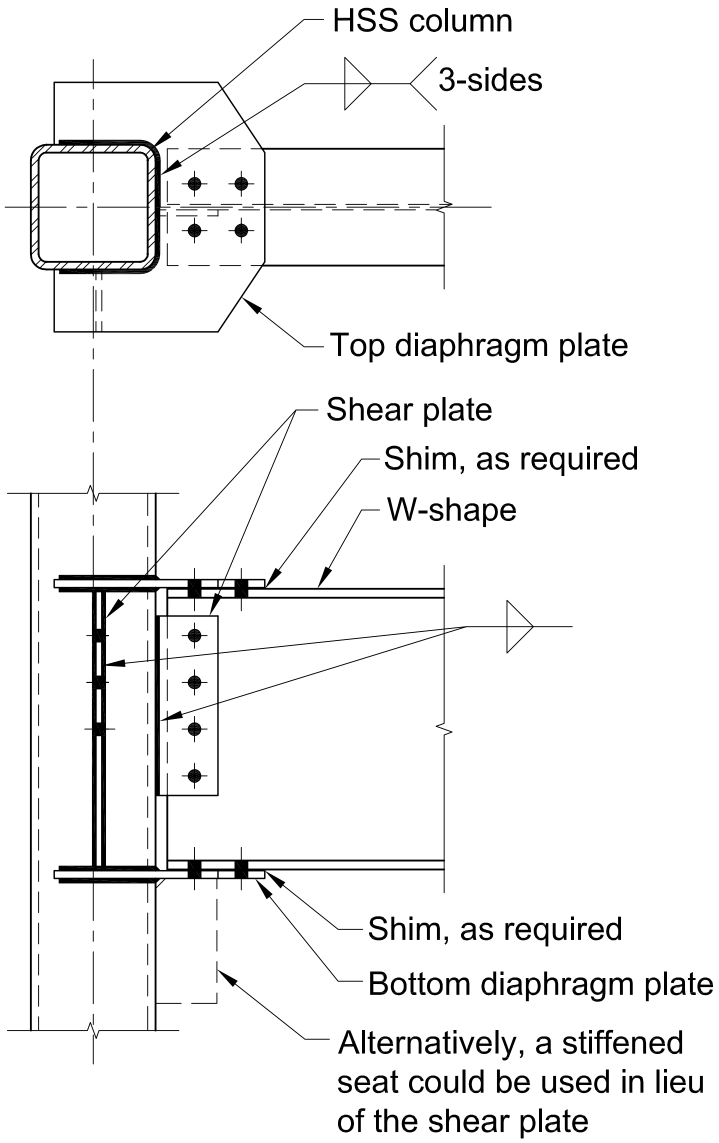


Fig. 12-21. Diaphragm plate splice to exterior HSS column.

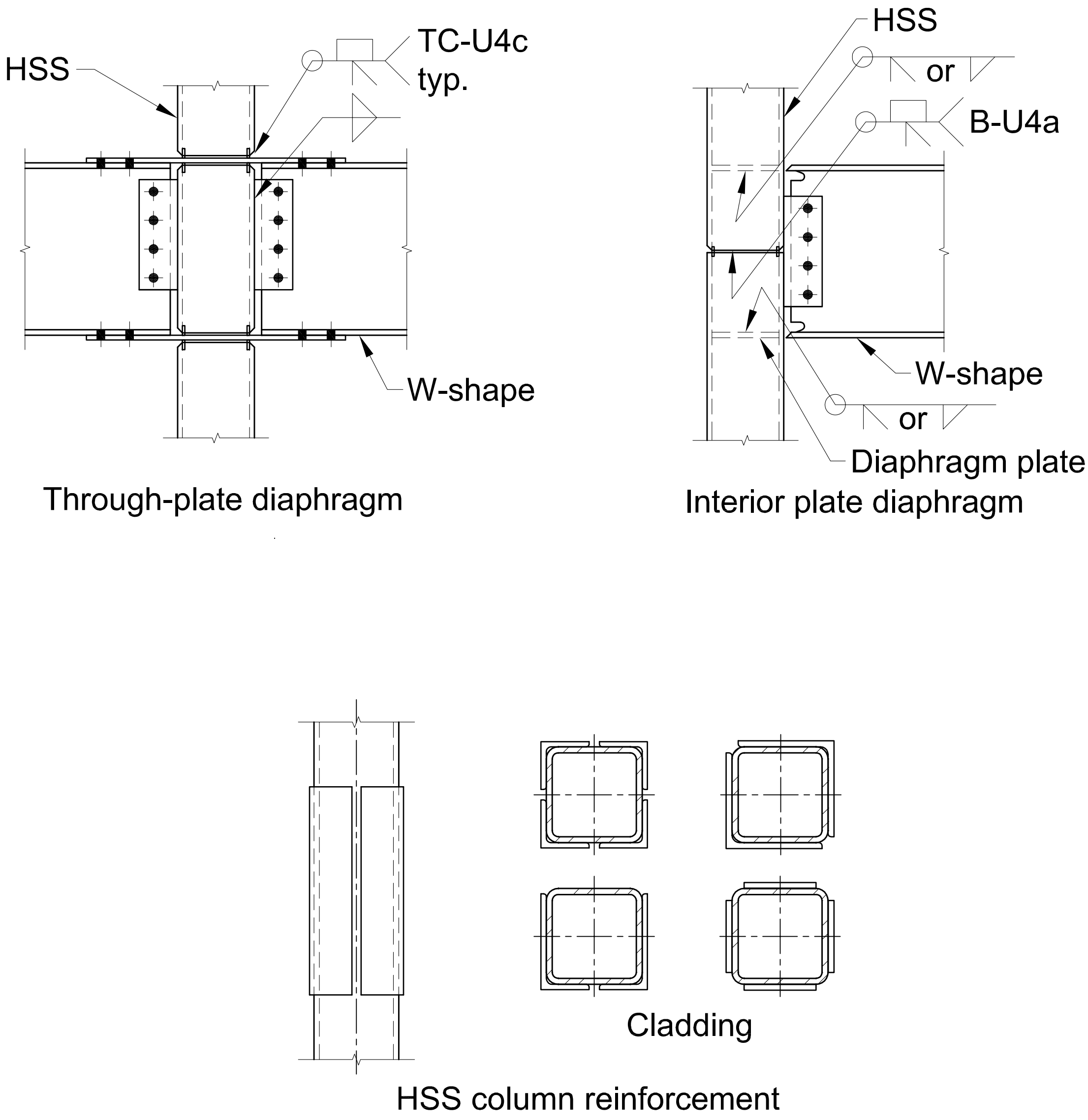


Fig. 12-22. Suggested details.

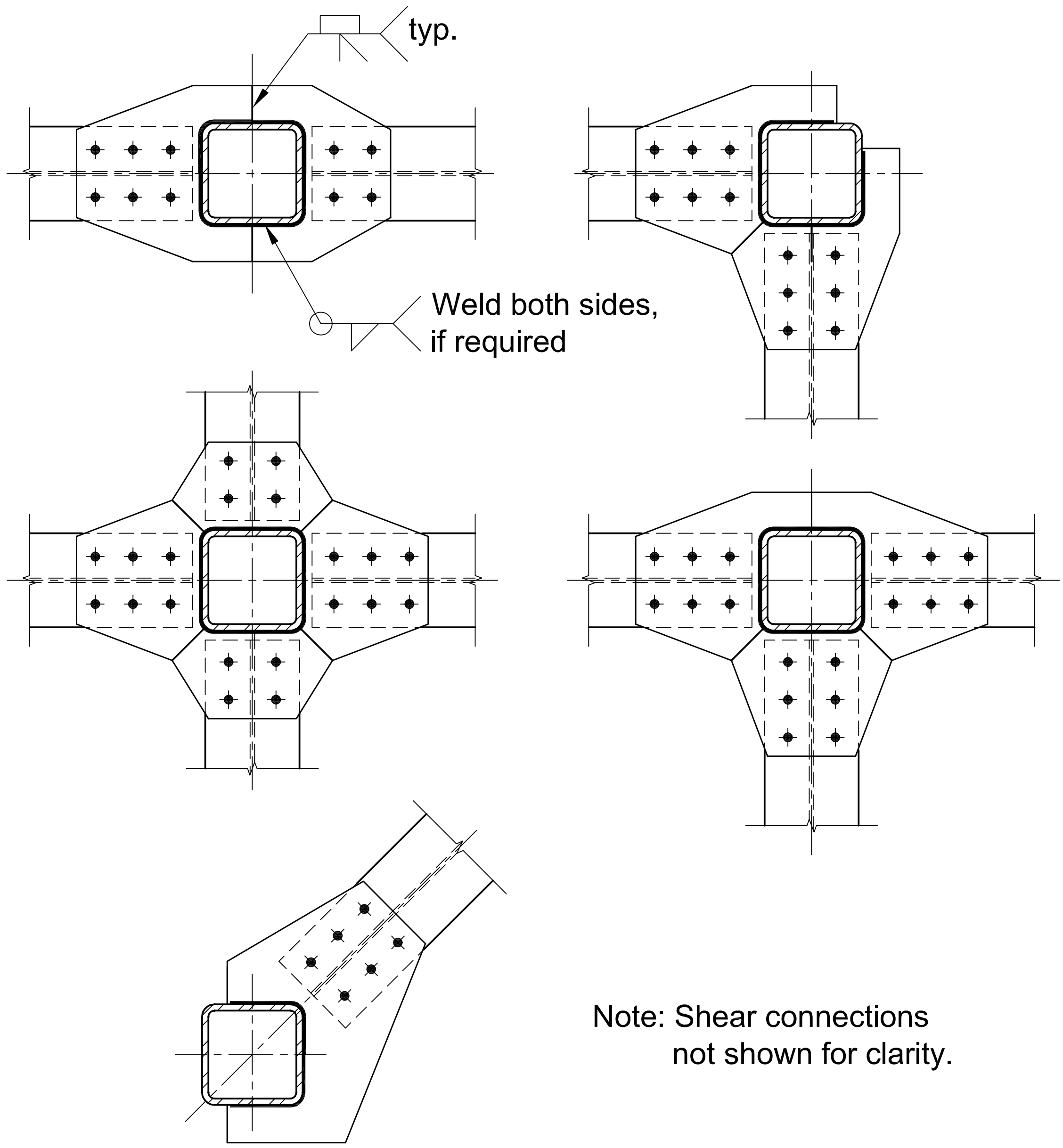


Fig. 12-23. Suggested details.

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PART 13

DESIGN OF BRACING CONNECTIONS AND TRUSS CONNECTIONS

SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of concentric bracing connections and truss connections. For additional information on this topic, refer to AISC Design Guide 29, *Vertical Bracing Connections—Analysis and Design* (Muir and Thornton, 2014).

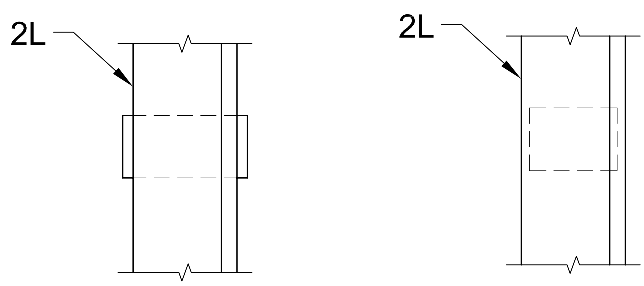
BRACING CONNECTIONS

Diagonal Bracing Members

Diagonal bracing members can be rods, single angles, channels, double angles, tees, W-shapes, or hollow structural sections (HSS) as required by the loads. Slender diagonal bracing members are relatively flexible and, thus, vibration and sag may be considerations. In slender tension-only bracing composed of light angles, these problems can be minimized with "draw" or pretension created by shortening the fabricated length of the diagonal brace from the theoretical length, L, between member working points. In general, the following deductions will be sufficient to accomplish the required draw: no deduction for $L \leq 10$ ft; deduct $^1/_{16}$ in. for 10 ft $< L \leq 20$ ft; deduct $^1/_8$ in. for 20 ft $< L \leq 35$ ft; and, deduct $^3/_{16}$ in. for $L > 35$ ft. This approach is not applicable to heavier diagonal bracing members, since it is difficult to stretch these members; vibration and sag are not usually design considerations in heavier diagonal bracing members. In any diagonal bracing member, however, it is permissible to deduct an additional $^1/_{32}$ in. when necessary to avoid dimensioning to thirty-seconds of an inch.

When double-angle diagonal bracing members are separated, as at "sandwiched" end connections to gussets, intermittent connections should be provided if the unsupported length of the diagonal brace exceeds the limits specified in the User Note in AISC *Specification* Section D4 for tension members. For compression members, the provisions of AISC *Specification* Section E6 must be satisfied. Either bolted or welded stitch-fillers may be provided as stipulated in AISC *Specification* Section E6. Many fabricators prefer ring or rectangular bolted stitch-fillers when the angles require other punching, as at the end connections. In welded construction, a stitch-filler with protruding ends, as shown in Figure 13-1(a), is preferred because it is easy to fit and weld. The short stitch-filler shown in Figure 13-1(b) is used if a smooth appearance is desired.

When a full-length filler is provided, as in corrosive environments, the maximum spacing of stitch bolts should be as specified in AISC *Specification* Section J3.5. Alternatively, the edges of the filler may be seal welded.



a) Protruding *b) Short*

Fig. 13-1. Welded stitch-fillers.

Force Transfer in Diagonal Bracing Connections

There has been some discussion as to which of several available analysis methods provides the best means for the safe and economical design and analysis of diagonal bracing connections. To better understand the technical issues, starting in 1981, AISC sponsored extensive computer studies of this connection by Richard (1986). Associated with Richard's work, full-scale tests were performed by Bjorhovde and Chakrabarti (1985), Gross and Cheok (1988), and Gross (1990). Also, AISC and ASCE formed a task group to recommend a design method for this connection. In 1990, this task group recommended three methods for further study; refer to Appendix A of Thornton (1991).

Using the results of the aforementioned full scale tests, Thornton (1991) showed that these three methods yield safe designs, and that of the three methods, the Uniform Force Method [see model 3 of Thornton (1991)] best predicts both the available strength and critical limit state of the connection. Furthermore, Thornton (1992) showed that the Uniform Force Method yields the most economical design through comparison of actual designs by the different methods and through consideration of the efficiency of force transmission. For the above reasons, and also because it is the most versatile method, the Uniform Force Method has been adopted for use in this manual.

The Uniform Force Method

The essence of the Uniform Force Method is to select the geometry of the connection so that moments do not exist on the three connection interfaces; i.e., gusset-to-beam, gusset-to-column, and beam-to-column. In the absence of moment, these connections may then be designed for shear and/or tension only, hence the origin of the name Uniform Force Method.

Required Strength

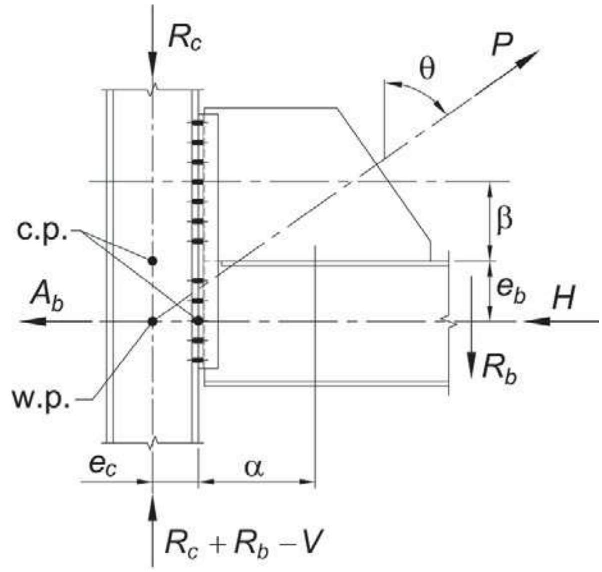
With the control points (c.p.) as illustrated in Figure 13-2 and the working point (w.p.) chosen at the intersection of the centerlines of the beam, column and diagonal brace as shown in Figure 13-2(a), four geometric parameters e_b, e_c, α and β can be identified, where

e_b = one-half the depth of the beam, in.
e_c = one-half the depth of the column, in. Note that, for a column web support, $e_c \approx 0$.
α = distance from the face of the column flange or web to the centroid of the gusset-to-beam connection, in.
β = distance from the face of the beam flange to the centroid of the gusset-to-column connection, in.

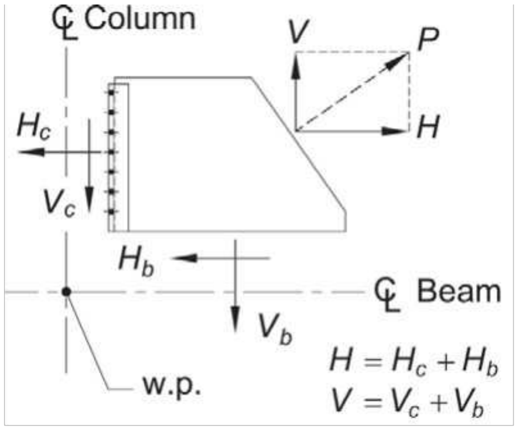
For the force distribution shown in the free-body diagrams of Figures 13-2(b), 13-2(c) and 13-2(d) to remain free of moments on the connection interfaces, the following expression must be satisfied:

$$\alpha - \beta \tan\theta = e_b \tan\theta - e_c \qquad (13\text{-}1)$$

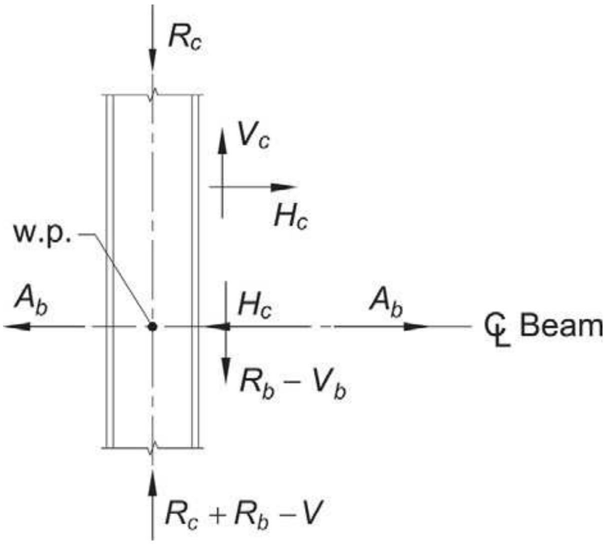
Since the variables on the right of the equal sign (e_b, e_c and θ) are all defined by the members being connected and the geometry of the structure, the designer may select values of α and β for which the equation is true, thereby locating the centroids of the gusset-to-beam and gusset-to-column connections.



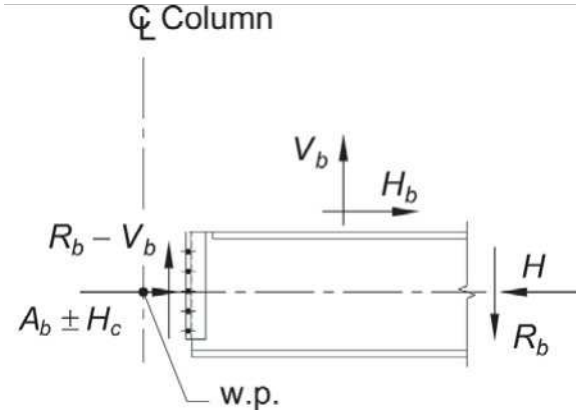
(a) Diagonal bracing connection and external forces



(b) Gusset free-body diagram



(c) Column free-body diagram



(d) Beam free-body diagram

R_b = R_{ub} or R_{ab}, required end reaction of the beam
R_c = R_{uc} or R_{ac}, required column axial load above the connection
A_b = A_{ub} or A_{ab}, required transverse force from adjacent bay
H = horizontal component of the required axial force
H_b = H_{ub} or H_{ab}, required shear force on the gusset-to-beam connection
H_c = H_{uc} or H_{ac}, required axial force on the gusset-to-column connection
V_b = V_{ub} or V_{ab}, required axial force on the gusset-to-beam connection
V_c = V_{uc} or V_{ac}, required shear force on the gusset-to-column connection
P = P_u or P_a, required axial force
V = vertical component of the required axial force

Fig. 13-2. Force transfer by the Uniform Force Method, work point (w.p.) and control points (c.p.) as indicated.

Once α and β have been determined, the required axial and shear forces for which these connections must be designed can be determined from the following equations:

$$V_c = \frac{\beta}{r}P \tag{13-2}$$

$$H_c = \frac{e_c}{r}P \tag{13-3}$$

$$V_b = \frac{e_b}{r}P \tag{13-4}$$

$$H_b = \frac{\alpha}{r}P \tag{13-5}$$

where

$$r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2} \tag{13-6}$$

The gusset-to-beam connection must be designed for the required shear force, H_b, and the required axial force, V_b, the gusset-to-column connection must be designed for the required shear force, V_c, and the required axial force, H_c, and the beam-to-column connection must be designed for the required shear:

$$R_b - V_b$$

and the required axial force:

$$A_b \pm (H - H_b)$$

Note that while the axial force, P_u or P_a, is shown as a tensile force, it may also be a compressive force; were this the case, the signs of the resulting gusset forces would change.

Special Case 1, Modified Working Point Location

As illustrated in Figure 13-3(a), the working point in Special Case 1 of the Uniform Force Method is chosen at the corner of the gusset; this may be done to simplify layout or for a column web connection. With this assumption, the terms in the gusset force equations involving e_b and e_c drop out and the interface forces, as shown in Figures 13-3(b), 13-3(c) and 13-3(d), are:

$$V_c = P\cos\theta = V \tag{13-7}$$

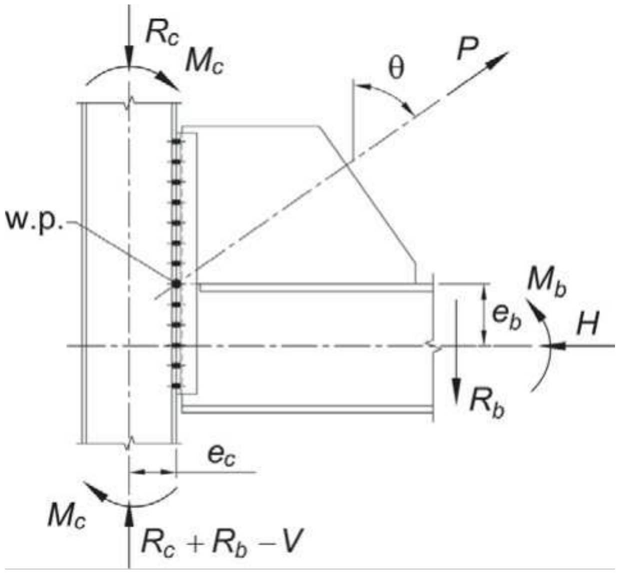
$$V_b = 0 \tag{13-8}$$

$$H_b = P\sin\theta = H \tag{13-9}$$

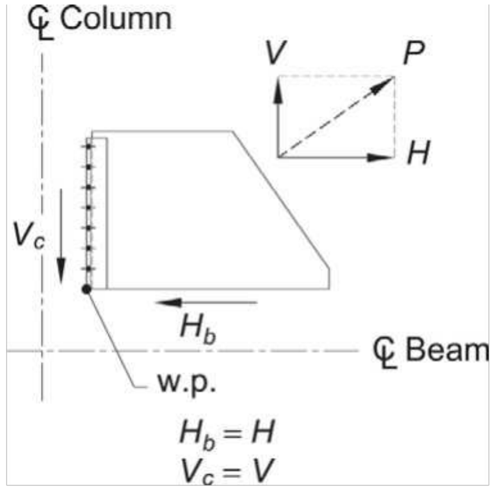
$$H_c = 0 \tag{13-10}$$

The gusset-to-beam connection must be designed for the required shear force, H_b, and the gusset-to-column connection must be designed for the required shear force, V_c. Note, however, that the change in working point requires that the beam be designed for the required moment, M_b, where

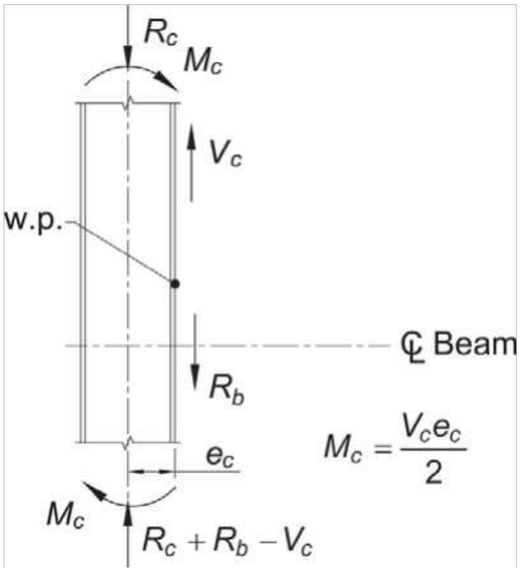
$$M_b = H_b e_b \tag{13-11}$$



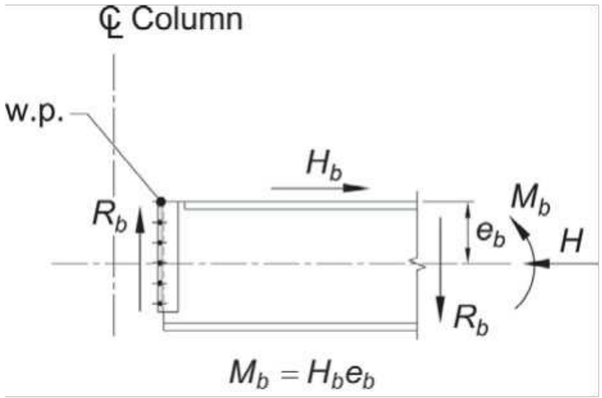
(a) Diagonal bracing connection



(b) Gusset free-body diagram



(c) Column free-body diagram



(d) Beam free-body diagram

R_b = R_{ub} or R_{ab}, required end reaction of the beam
R_c = R_{uc} or R_{ac}, required column axial load above the connection
A_b = A_{ub} or A_{ab}, required transverse force from adjacent bay
H = horizontal component of the required axial force
H_b = H_{ub} or H_{ab}, required shear force on the gusset-to-beam connection
V_c = V_{uc} or V_{ac}, required shear force on the gusset-to-column connection
P = P_u or P_a, required axial force
V = vertical component of the required axial force

Fig. 13-3. Force transfer, Uniform Force Method Special Case 1.

and the column must be designed for the required moment, M_c. For an intermediate floor, this is determined as:

$$M_c = \frac{V_c e_c}{2} \tag{13-12}$$

An example demonstrating this eccentric special case is presented in AISC (1984). This eccentric case was endorsed by the AISC/ASCE task group (Thornton, 1991) as a reduction of the three recommended methods when the work point is located at the gusset corner. While calculations are somewhat simplified, it should be noted that resolution of the required force, P, into the shears, V_c and H_b, may not result in the most economical connection.

Special Case 2, Minimizing Shear in the Beam-to-Column Connection

If the brace force, as illustrated in Figure 13-4(a), were compressive instead of tensile and the required beam reaction, R_b, were high, the addition of the extra shear force, V_b, into the beam might exceed the available strength of the beam and require doubler plates or a haunched connection. Alternatively, the vertical force in the gusset-to-beam connection, V_b, can be limited in a manner that is somewhat analogous to using the gusset itself as a haunch.

As illustrated in Figure 13-4(b), assume that V_b is reduced by an arbitrary amount, ΔV_b. By statics, the vertical force at the gusset-to-column interface will be increased to $V_c + \Delta V_b$, and a moment M_b will result on the gusset-to-beam connection, where

$$M_b = (\Delta V_b)\alpha \tag{13-13}$$

If ΔV_b is taken equal to V_b, none of the vertical component of the brace force is transmitted to the beam; the resulting procedure is that presented by AISC (1984) for concentric gravity axes, extended to connections to column flanges. This method was also recommended by the AISC/ASCE task group (Thornton, 1991).

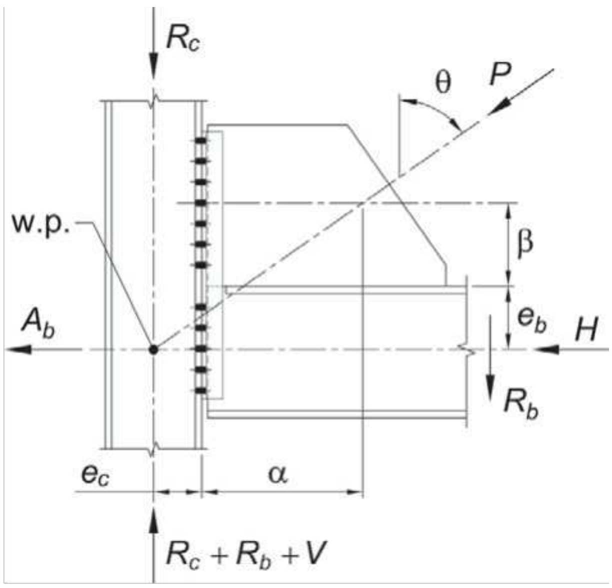
Design by this method may be uneconomical. It is very punishing to the gusset and beam because of the moment, M_b, induced on the gusset-to-beam connection. This moment will require a larger connection and a thicker gusset. Additionally, the limit state of local web yielding may limit the strength of the beam. This special case interrupts the natural flow of forces assumed in the Uniform Force Method and thus is best used when the beam-to-column interface is already highly loaded, independently of the brace, by a high shear, R_b, in the beam-to-column connection.

Special Case 3, No Gusset-to-Column Web Connection

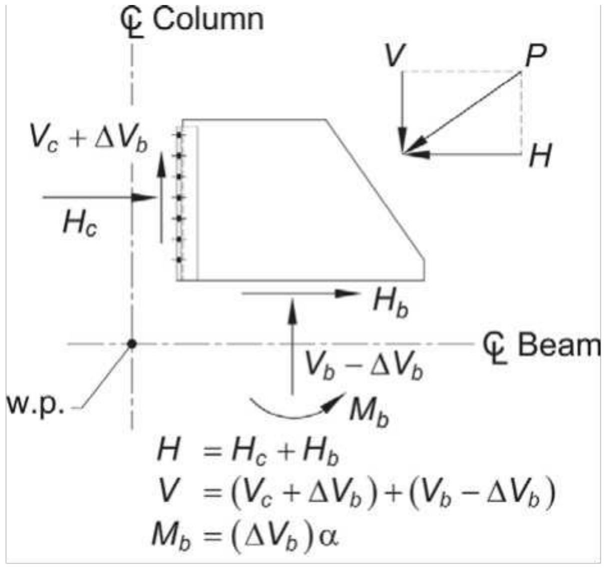
When the connection is to a column web and the brace is shallow (as for large θ) or the beam is deep, it may be more economical to eliminate the gusset-to-column connection entirely and connect the gusset to the beam only. The Uniform Force Method can be applied to this situation by setting β and e_c equal to zero, as illustrated in Figure 13-5. Since there is to be no gusset-to-column connection, V_c and H_c also equal zero. Thus, $V_b = V$ and $H_b = H$.

If $\bar{\alpha} = \alpha = e_b \tan\theta$, there is no moment on the gusset-to-beam interface and the gusset-to-beam connection can be designed for the required shear force, H_b, and the required axial force, V_b. If $\bar{\alpha} \neq \alpha = e_b \tan\theta$, the gusset-to-beam interface must be designed for the moment, M_b, in addition to H_b and V_b, where

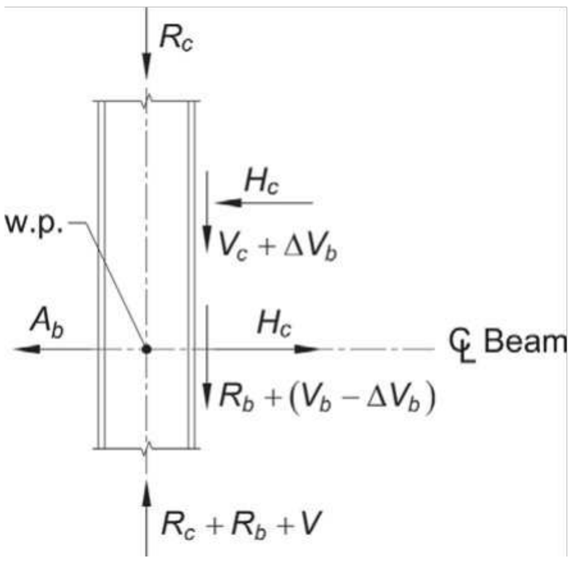
$$M_b = V_b\,(\alpha - \bar{\alpha}) \tag{13-14}$$



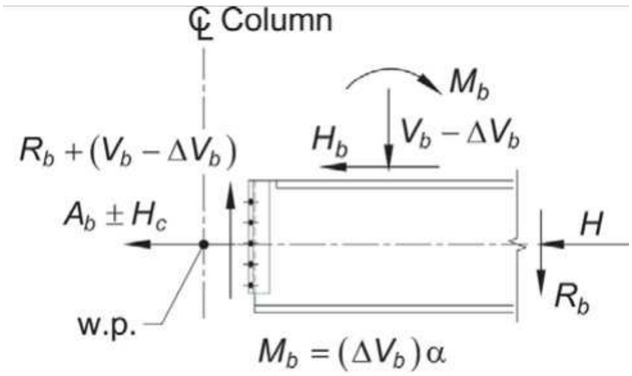
(a) Diagonal bracing connection



(b) Gusset free-body diagram



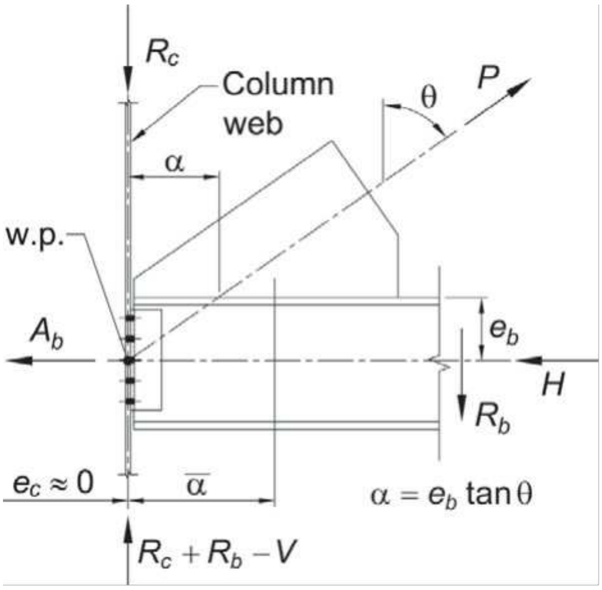
(c) Column free-body diagram



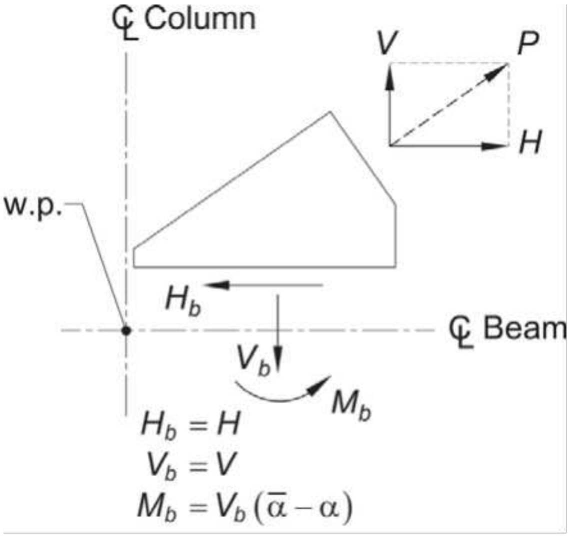
(d) Beam free-body diagram

R_b = R_{ub} or R_{ua}, required end reaction of the beam
R_c = R_{uc} or R_{ac}, required column axial load above the connection
A_b = A_{ub} or A_{ab}, required transverse force from adjacent bay
H = horizontal component of the required axial force
H_b = H_{ub} or H_{ab}, required shear force on the gusset-to-beam connection
H_c = H_{uc} or H_{ac}, required axial force on the gusset-to-column connection
V_b = V_{ub} or V_{ab}, required axial force on the gusset-to-beam connection
V_c = V_{uc} or V_{ac}, required shear force on the gusset-to-column connection
P = P_u or P_a, required axial force
V = vertical component of the required axial force

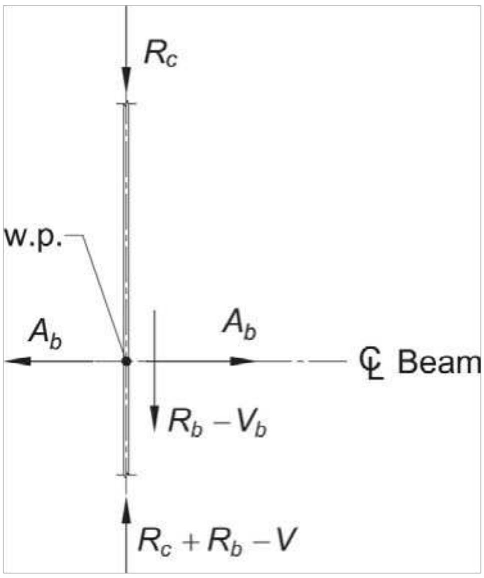
Fig. 13-4. Force transfer, Uniform Force Method Special Case 2.



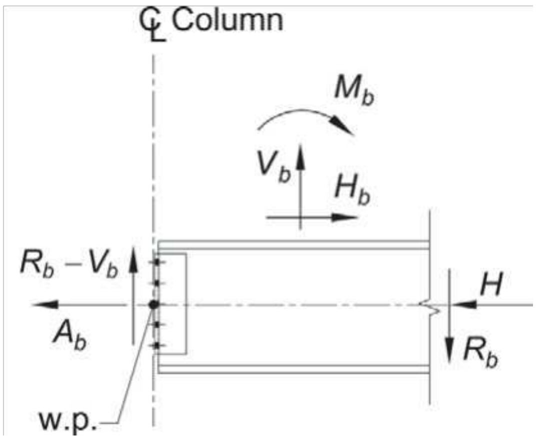
(a) Diagonal bracing connection



(b) Gusset free-body diagram



(c) Column free-body diagram



(d) Beam free-body diagram

R_b = R_{ub} or R_{ua}, required end reaction of the beam
R_c = R_{uc} or R_{ac}, required column axial load above the connection
A_b = A_{ub} or A_{ab}, required transverse force from adjacent bay
H = horizontal component of the required axial force
H_b = H_{ub} or H_{ab}, required shear force on the gusset-to-beam connection
V_b = V_{ub} or V_{ab}, required axial force on the gusset-to-beam connection
P = P_u or P_a, required axial force
V = vertical component of the required axial force

Fig. 13-5. Force transfer, Uniform Force Method Special Case 3.

The beam-to-column connection must be designed for the required shear force, $R_b + V_b$.

Note that, since the connection is to a column web, e_c is zero and hence H_c is also zero. For a connection to a column flange, if the gusset-to-column-flange connection is eliminated, the beam-to-column connection must be a moment connection designed for the moment, Ve_c, in addition to the shear, V. Thus, uniform forces on all interfaces are no longer possible.

Analysis of Existing Diagonal Bracing Connections

A combination of α and β which provides for no moments on the three interfaces can usually be achieved when a connection is being designed. However, when analyzing an existing connection or when other constraints exist on gusset dimensions, the values of α and β may not satisfy the basic relationship

$$\alpha - \beta \tan\theta = e_b \tan\theta - e_c \tag{13-1}$$

When this happens, uniform interface forces will not satisfy equilibrium and moments will exist on one or both gusset edges or at the beam-to-column interface.

To illustrate this point, consider an existing design where the actual centroids of the gusset-to-beam and gusset-to-column connections are at $\bar{\alpha}$ and $\bar{\beta}$, respectively. If the connection at one edge of the gusset is more rigid than the other, it is logical to assume that the more rigid edge takes all of the moment necessary for equilibrium. For instance, the gusset of Figure 13-2 is shown welded to the beam and bolted with double angles to the column. For this configuration, the gusset-to-beam connection will be much more rigid than the gusset-to-column connection.

Take α and β as the ideal centroids of the gusset-to-beam and gusset-to-column connections, respectively. Setting $\beta = \bar{\beta}$, the α required for no moment on the gusset-to-beam connection may be calculated as

$$\alpha = K + \bar{\beta} \tan\theta \tag{13-15}$$

where

$$K = e_b \tan\theta - e_c \tag{13-16}$$

If $\alpha \neq \bar{\alpha}$, a moment, M_b, will exist on the gusset-to-beam connection where

$$M_b = V_b (\alpha - \bar{\alpha}) \tag{13-17}$$

Conversely, suppose the gusset-to-column connection were judged to be more rigid. Setting $\alpha = \bar{\alpha}$, the β required for no moment on the gusset-to-column connection may be calculated as

$$\beta = \frac{\bar{\alpha} - K}{\tan\theta} \tag{13-18}$$

If $\beta \neq \bar{\beta}$, a moment, M_c, will exist on the gusset-to-column connection where

$$M_c = H_c(\beta - \bar{\beta}) \tag{13-19}$$

If both connections were equally rigid and no obvious allocation of moment could be made, the moment could be distributed based on minimized eccentricities $\alpha - \bar{\alpha}$ and $\beta - \bar{\beta}$ by minimizing the objective function, ξ, where

$$\xi = \left(\frac{\alpha - \bar{\alpha}}{\bar{\alpha}}\right)^2 + \left(\frac{\beta - \bar{\beta}}{\bar{\beta}}\right)^2 - \lambda\left(\alpha - \beta \tan\theta - K\right) \tag{13-20}$$

In the preceding equation, λ is a Lagrange multiplier.

The values of α and β that minimize ξ are

$$\alpha = \frac{K' \tan\theta + K\left(\frac{\bar{\alpha}}{\bar{\beta}}\right)^2}{D} \tag{13-21}$$

and

$$\beta = \frac{K' - K \tan\theta}{D} \tag{13-22}$$

where

$$K' = \bar{\alpha}\left(\tan\theta + \frac{\bar{\alpha}}{\bar{\beta}}\right) \tag{13-23}$$

$$D = \tan^2\theta + \left(\frac{\bar{\alpha}}{\bar{\beta}}\right)^2 \tag{13-24}$$

Available Strength

The available strength of a diagonal bracing connection is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a. Note that when the gusset is directly welded to the beam or column, the connection should be designed for the larger of the peak stress and 1.25 times the average stress, but the weld size need not be larger than that required to develop the strength of the gusset. This 25% increase is recommended to allow adequate redistribution of transverse stresses in the weld group. This adjustment should not be applied to welds that resist only shear forces (Hewitt and Thornton, 2004).

TRUSS CONNECTIONS

Members in Trusses

For light loads, trusses are commonly composed of tees for the top and bottom chords with single-angle or double-angle web members. In welded construction, the single-angle and double-angle web members may, in many cases, be welded to the stem of the tee, thus, eliminating the need for gussets. When single-angle web members are used, all web members should be placed on the same side of the chord; staggering the web members causes a torque on the chord, as illustrated in Figure 13-6. Also see “Design Considerations for HSS-to-HSS Truss Connections” at the end of this Part.

Double-angle truss members are usually designed to act as a unit. When unequal-leg angles are used, long legs are normally assembled back-to-back. A simple notation for the angle assembly is LLBB (long legs back-to-back) and SLBB (short legs back-to-back). Alternatively, the notation might be graphical in nature as and . For large loads, W-shapes may be used with the web vertical and gussets welded to the flange for the truss connections. Web members may be single angles or double angles, although W-shapes are sometimes used for both chord and web members as shown in Figure 13-7. Heavy shapes in trusses must meet the design and fabrication restrictions and special requirements in AISC *Specification* Sections A3.1c and A3.1d. With member orientation as shown for the field-welded truss joint in Figure 13-7(a), connections usually are made by groove welding flanges to flanges and fillet welding webs directly or indirectly by the use of gussets. Fit-up of joints in this type of construction are very sensitive to dimensional variations in the rolled shapes; fabricators sometimes prefer to use built-up shapes in these cases.

The web connection plate in Figure 13-7(a) is a typical detail. While the diagonal member could theoretically be cut so that the diagonal web would be extended into the web of the chord for a direct connection, such a detail is difficult to fabricate. Additionally, welding access becomes very limited. Note the obvious difficulty of welding the gusset or diagonal directly to the chord web; therefore, this weld is usually omitted.

When stiffeners and doubler plates are required for concentrated flange forces, the designer should consider selecting a heavier section to eliminate the need for stiffening. Although this will increase the material cost of the member, the heavier section will likely provide a more economical solution due to the reduction in labor cost associated with the elimination of stiffening (Ricker, 1992; Thornton, 1992).

Minimum Connection Strength

In the absence of defined design loads, a minimum required strength of 10 kips for LRFD or 6 kips for ASD should be considered, as noted in AISC *Specification* Commentary Section J1.1. For smaller elements, a required strength more appropriate to the size and

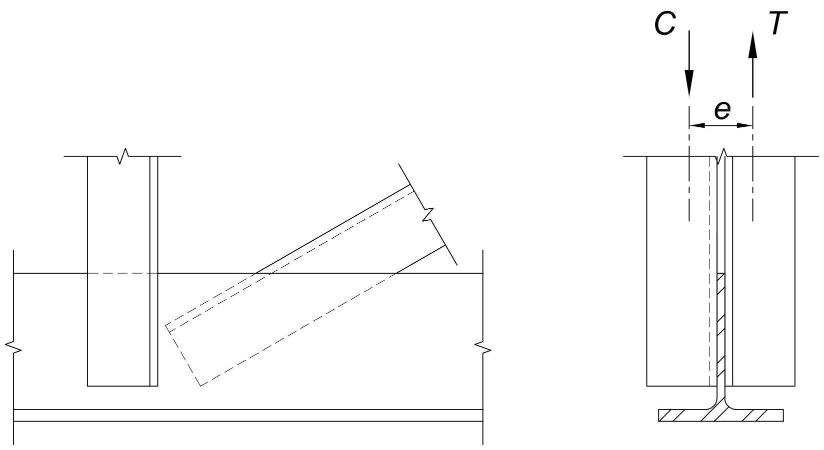
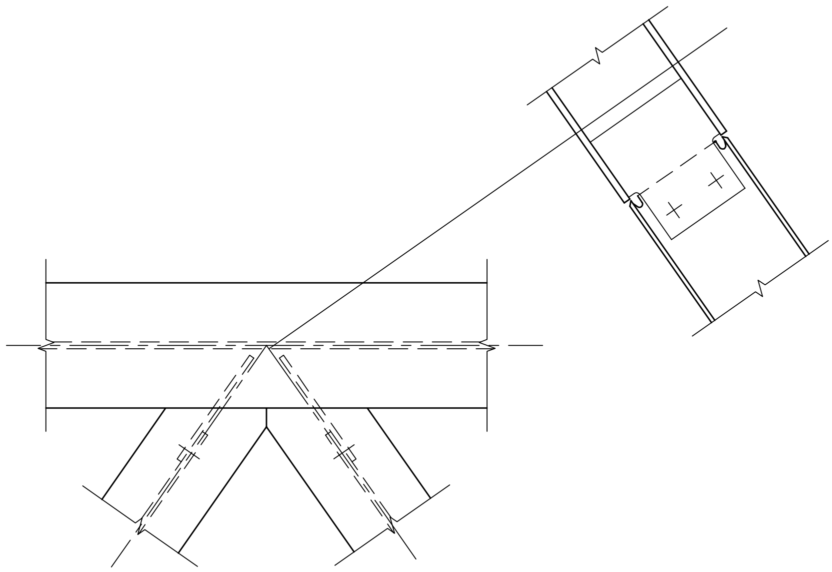


Fig. 13-6. Staggered web members result in a torque on the truss chord.

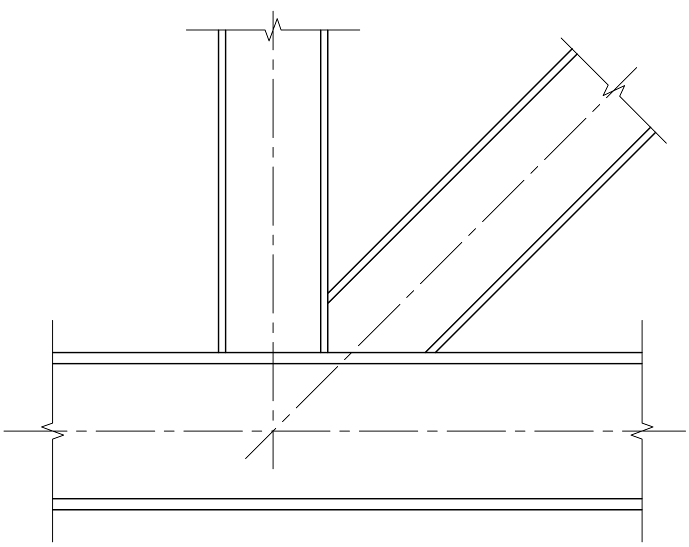
use of the part should be used. Additionally, when trusses are shop-assembled or field-assembled on the ground for subsequent erection, consideration should be given to loads induced during handling, shipping and erection.

Panel-Point Connections

A panel-point connection connects diagonal and/or vertical web members to the chord member of a truss. These web members deliver axial forces, tensile or compressive, to the truss chord. In bolted construction, a gusset is usually required because of bolt spacing and edge distance requirements. In welded construction, it is sometimes possible to eliminate the need for a gusset.



(a) Shop and field welding



Note: Check vertical and chord for reinforcing requirements.

(b) Shop welding

Fig. 13-7. Truss panel-point connections for W-shape truss members.

Design Checks

The available strength of a panel-point connection is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). In all cases, the available strength, ϕR_n or R_n/Ω, must exceed the required strength, R_u or R_a.

In the panel-point connection of Figure 13-8, the neutral axes of the vertical and diagonal truss members intersect on the neutral axis of the truss chord. As a result, the forces in all members of the truss are axial. It is common practice, however, to modify working lines slightly from the gravity axes to establish repetitive panels and avoid fractional dimensions less than $^1/_8$ in. or to accommodate a larger panel-point connection or a connection for bottom-chord lateral bracing, a purlin, or a sway-frame. This eccentricity and the resulting moment should be considered in the design of the truss chord.

In contrast, for the design of end connections of truss web members consisting of single or double angles or similar members, the center of gravity of the connection need not coincide with the gravity axis of the connected members, as permitted in AISC *Specification* Section J1.7. This is because tests have shown that there is no appreciable difference in the available strength between balanced and unbalanced connections subjected to static loading. Accordingly, the truss web members and their end connections may be designed for the axial load, neglecting the effect of this minor eccentricity.

Shop and Field Practices

In bolted construction, it is convenient to use standard gage lines of the angles (see Table 1-7A) as truss working lines; where wider angles with two gage lines are used, the gage line nearest the heel of the angle is the one which is substituted for the gravity axis.

As shown in Figure 13-8, to provide for stiffness in the finished truss, the web members of the truss are extended to near the edge of the fillet of the tee chord (*k*-dimension). If welded, the required welds are then applied along the heel and toe of each angle, beginning at their ends rather than at the edge of the tee stem.

Support Connections

A truss support connection connects the ends of trusses to supporting members.

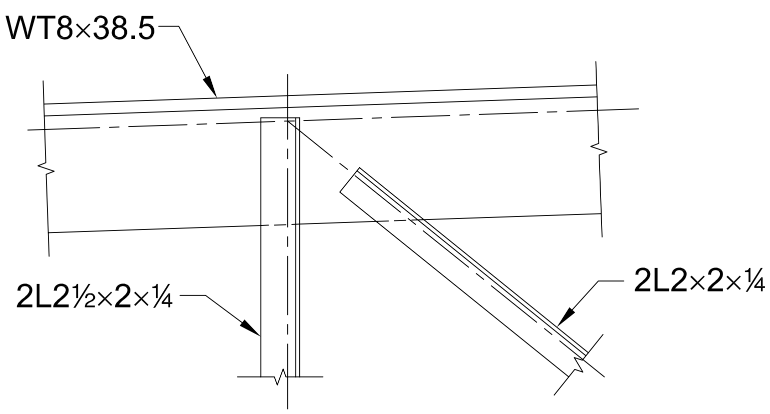


Fig. 13-8. Truss panel-point connection.

Design Checks

The available strength of a support connection is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). Additionally, truss support connections produce tensile or compressive single concentrated forces at the beam end; the limit states of the available flange strength in local bending and the limit states of the available web strength in local yielding, crippling and compression buckling may have to be checked. In all cases, the available strength, ϕR_n or R_n/Ω, must exceed the required strength, R_u or R_a.

At the end of a truss supported by a column, all member axes may not intersect at a common point. When this is the case, an eccentricity results. Typically, it is the neutral axis of the column that does not meet at the working point.

If trusses with similar reactions line up on opposite sides of the column, consideration of eccentricity would not be required since any moment would be transferred through the column and into the other truss. However, if there is little or no load on the opposite side of the column, the resulting eccentricity must be considered.

In Figure 13-9, the truss chord and diagonal intersect at a common working point on the face of the column flange. In this detail, there is no eccentricity in the gusset, gusset-to-column connection, truss chord, or diagonal. However, the column must be designed for the moment due to the eccentricity of the truss reaction from the neutral axis of the column.

For the truss support connection illustrated in Figure 13-10, this eccentricity results in a moment. Assuming the connection between the members is adequate, joint rotation is resisted by the combined flexural strength of the column, the truss top chord, and the truss diagonal. However, the distribution of moment between these members will be proportional to the stiffness of the members. Thus, when the stiffness of the column is much greater than the stiffness of the other elements of the truss support connection, it is good practice to design the column and gusset-to-column connection for the full eccentricity.

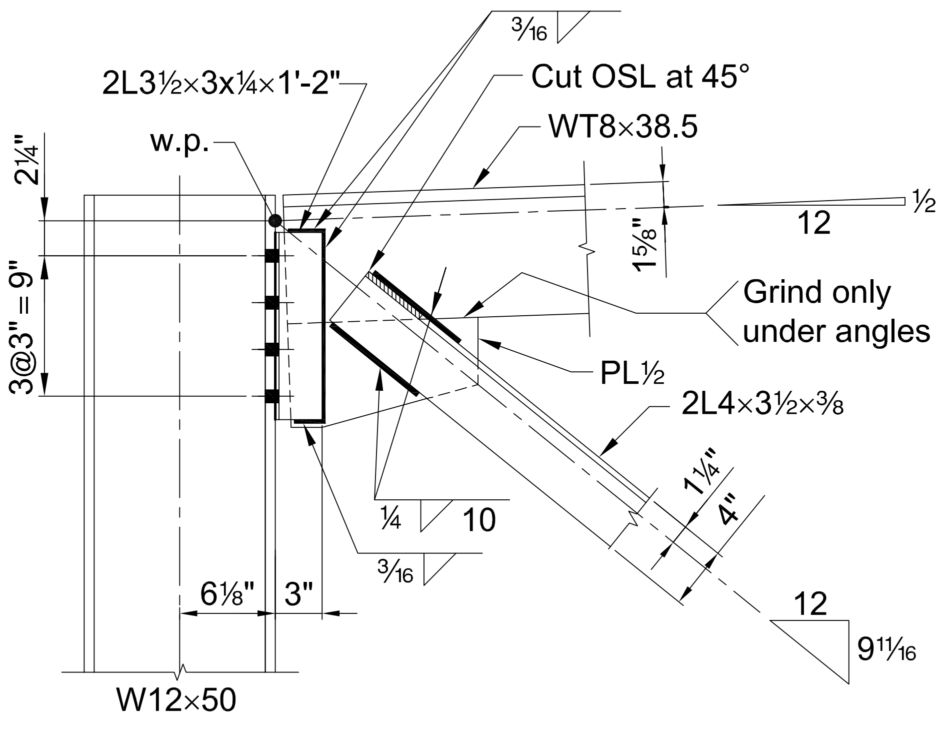


Fig. 13-9. Truss support connection, working point (w.p.) on column face.

Due to its importance, the truss support connection is frequently shown in detail on the design drawing.

Shop and Field Practices

When a truss is erected in place and loaded, truss members in tension will lengthen and truss members in compression will shorten. At the support connection, this may cause the tension chord of a "square-ended" truss to encroach on its connection to the supporting column. When the connection is shop-attached to the truss, erection clearance must be provided with shims to fill out whatever space remains after the truss is erected and loaded. In field erected connections, however, provision must be made for the necessary adjustment in the connection.

When the tension chord delivers no calculated force to the connection, adjustment can usually be provided with slotted holes. For short spans with relatively light loads, the comparatively small deflections can be absorbed by the normal hole clearances provided for bolted construction. Slightly greater misalignment can be corrected in the field by reaming the holes. If appreciable deflection is expected, the connection may be welded. Alternatively, bolt holes may be field-drilled, but this is an expensive operation which should be avoided if at all possible.

An approximation of the elongation, Δ, can be determined as

$$\Delta = \frac{Pl}{AE} \tag{13-25}$$

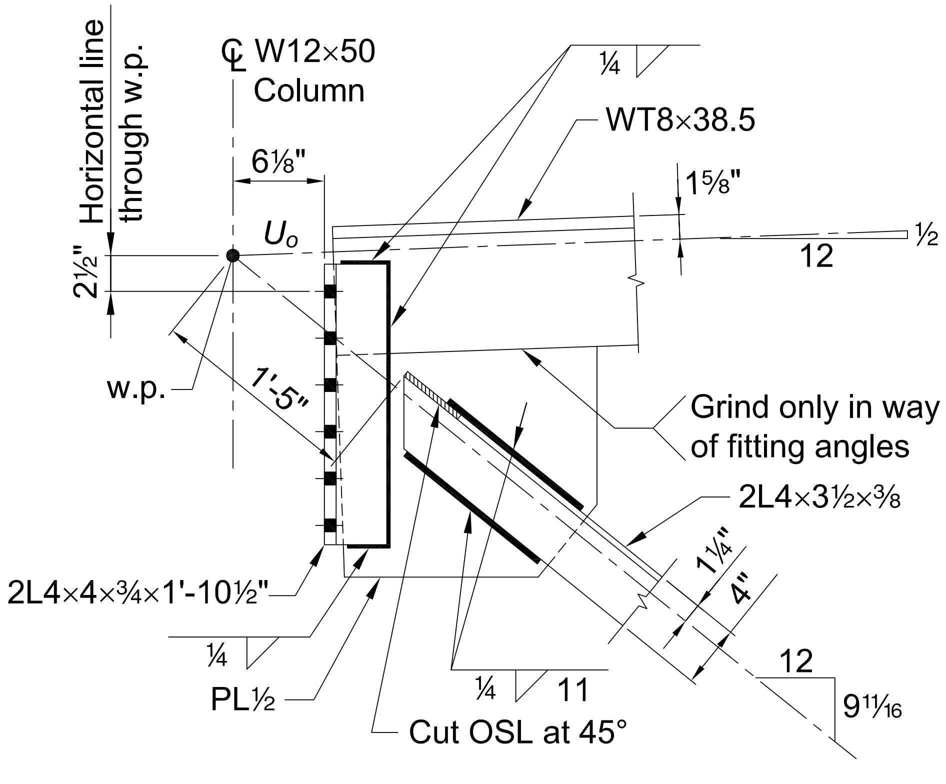


Fig. 13-10. Truss-support connection, working point (w.p.) at column centerline.

where
A = gross area of the truss chord, in.^{2}
P = axial force due to service loads, kips
l = length, in.^{2}
Δ = elongation, in.

The total change in length of the truss chord is $\Sigma\Delta_i$, the sum of the changes in the lengths of the individual panel segments of the truss chord. The misalignment at each support connection of the tension chord is one-half the total elongation.

Truss Chord Splices

Truss chord splices are expensive to fabricate and should be avoided whenever possible. In general, chord splices in ordinary building trusses are confined to cases where

1. The finished truss is too large to be shipped in one piece;
2. The truss chord exceeds the available material length;
3. The reduction in member size of the chord justifies the added cost of a splice; or
4. A sharp change in direction occurs in the working line of the chord and bending does not provide a satisfactory alternative.

Splices at truss chord ends that are finished to bear should be designed in accordance with AISC *Specification* Section J1.4.

Design Considerations for HSS-to-HSS Truss Connections

The connection types covered in Chapter K of the AISC *Specification* and in AISC Design Guide 24, *Hollow Structural Section Connections* (Packer et al., 2010a), are only some of the potential configurations of HSS-to-HSS connections.

The structural analysis of HSS trusses should assume either pin-jointed analysis or analysis using web members pin-connected to continuous chord members such that only axial forces exist in the web members. The centerlines of the web members and the chord members should lie in a common plane, and rectangular HSS trusses should have all members oriented with walls parallel to the plane. Angles between the web member(s) and the chord less than 30° should be avoided. In accordance with AISC *Specification* Section K3, eccentricities, measured from the intersection between the web member centerlines to the centerline of the chord, can be neglected if the eccentricity is less than or equal to 25% of the chord depth from the centerline of the chord away from the web members or less than or equal to 55% of the chord depth from the centerline of the chord toward the web members. Additionally, AISC *Specification* Chapter K is predicated on HSS truss members having a specified minimum yield strength of less than or equal to 52 ksi and F_y / F_u of less than or equal to 0.8.

HSS member sizes are often critical in connection design. Connection design, including weld requirements in AISC *Specification* Section K5, should be considered during main member selection as the connection limit states may force an increase in the member wall thickness over the main member design thickness. Compression chords should be sized such that the demand-to-capacity ratio is considerably less than one, such that the effects of web members do not cause the face of the chord to be overstressed. At initial design, Packer et al. (2010b) recommends that chords have thick walls rather than thin walls; web members have thin walls rather than thick walls; web members be wide relative to the chord members, but still able to sit on the "flat" face of the chord section if possible; and gap connections (for K and N situations) are preferred to overlap connections because the members are easier to prepare, fit and weld. Where a gap is provided between the web members, the gap should be equal to or greater than the sum of the thicknesses of the web members to facilitate welding. Where web members are overlapped, the thicker web member should run through to the chord, and the overlap length (measured along the connecting face of the chord beneath the two web members) should be between 25% and 100% (inclusive) of the projected length of the overlapping web member on the chord. Members should be sized to satisfy the limits of applicability shown in Tables K3.1A and K3.2A of the AISC *Specification*.

For reinforced connections and connections not covered in the AISC *Specification*, refer to CIDECT Design Guide 3, *Design Guide for Rectangular Hollow Section (RHS) Joints under Predominantly Static Loading* (Packer et al., 2010b).

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PART 14

DESIGN OF BEAM BEARING PLATES, COLUMN BASE PLATES, ANCHOR RODS, AND COLUMN SPLICES

SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of beam bearing plates, column base plates, anchor rods, and column splices. For complete coverage of column base plate connections, see AISC Design Guide 1, *Base Plate and Anchor Rod Design* (Fisher and Kloiber, 2006).

BEAM BEARING PLATES

A beam bearing plate is made with a plate as illustrated in Figure 14-1.

Force Transfer

The required strength (beam end reaction), R_u or R_a, is distributed from the beam bottom flange to the bearing plate over an area equal to l_b times $2k$, where l_b is the bearing length (length of contact between the beam bottom flange and the bearing plate), in. The bearing plate is then assumed to distribute the beam end reaction to the concrete or masonry as a uniform bearing pressure by cantilevered bending of the plate. The bearing plate cantilever dimension is taken as

$$n = \frac{B}{2} - k \qquad (14\text{-}1)$$

where B is the bearing plate width, in.

In the rare case where a bearing plate is not required, the beam end reaction, R_u or R_a, is assumed to be uniformly distributed from the beam bottom flange to the concrete or masonry as a uniform bearing pressure by cantilevered bending of the beam flanges. The beam-flange cantilever dimension is calculated as for a bearing plate, but using the beam flange width, b_f, in place of B.

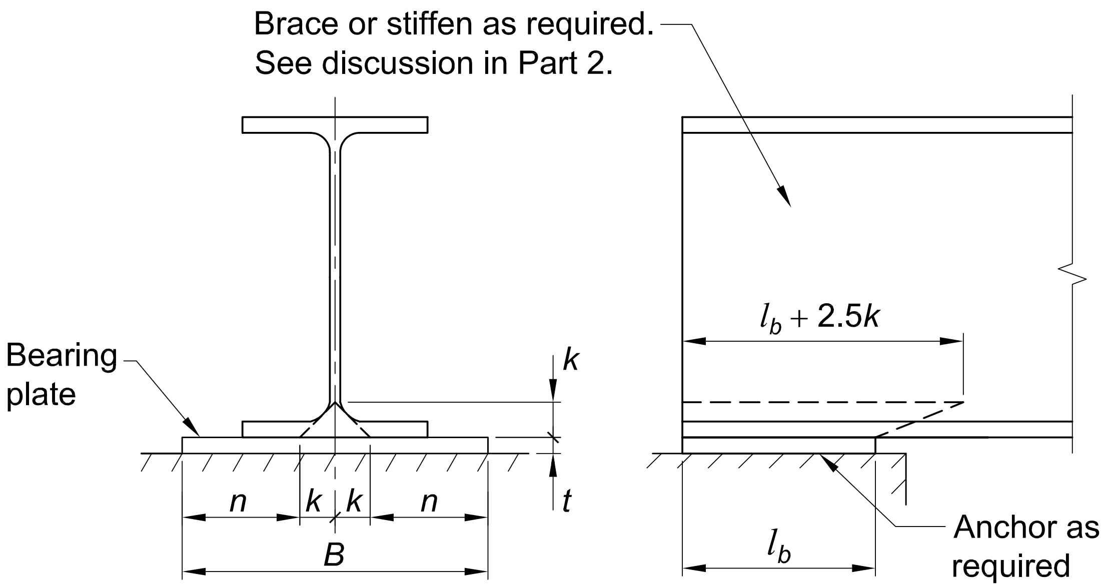


Fig. 14-1. Beam bearing plate variables.

Recommended Bearing Plate Dimensions and Thickness

The length of bearing, l_b, may be established by available wall thickness, clearance requirements, or by the minimum requirements based on local web yielding or web crippling. The selected dimensions of the bearing plate, B and l_b, should preferably be in full inches. Bearing plate thickness should be specified in multiples of $^1/_8$ in. up to $1^1/_4$-in. thickness and in multiples of $^1/_4$ in. thereafter.

Available Strength

The available strength of a beam bearing plate is determined from the applicable limit states for connecting elements (see Part 9). In all cases, the available strength, ϕR_n or R_n/Ω, must exceed the required strength, R_u or R_a. The stability of the beam end must also be addressed as discussed in "Stability Bracing" in Part 2.

COLUMN BASE PLATES FOR AXIAL COMPRESSION

A column base plate is made with a plate and a minimum of four anchor rods as illustrated in Figure 14-2. Base plates for posts as defined by OSHA (see Part 2) may be supported with two anchor rods. The base plate is often attached to the base of the column in the shop. Large heavy columns can be difficult to handle and set plumb with the base plate attached in the shop. When the column is over a certain weight, it may be better to detail the base plate loose for setting and leveling before the column is set. When the column-to-base-plate assembly weighs more than 4 tons, loose base plates should be considered.

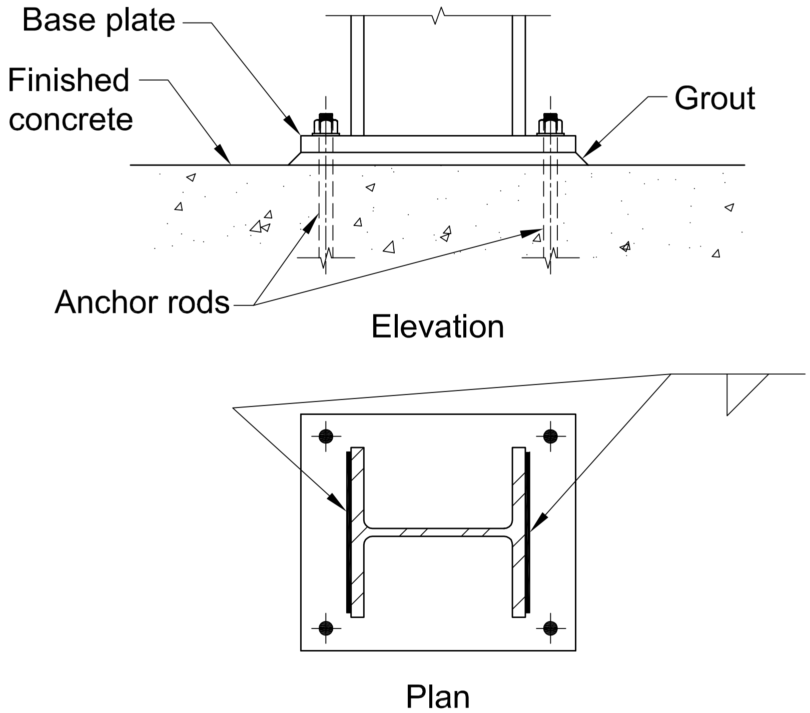


Fig. 14-2. Typical column base for axial compressive loads.

Force Transfer

In Figure 14-3, the required strength (column axial force), P_u or P_a, is distributed from the column end to the column base plate in direct bearing. The column base plate is then assumed to distribute the column axial force to the concrete or masonry as a uniform bearing pressure by cantilevered bending of the plate. The critical base plate cantilever dimension, l, is determined as the larger of m, n and $\lambda n'$ where

$$m = \frac{N - 0.95d}{2} \tag{14-2}$$

$$n = \frac{B - 0.8b_f}{2} \tag{14-3}$$

$$n' = \frac{\sqrt{db_f}}{4} \tag{14-4}$$

$$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} \le 1 \tag{14-5}$$

LRFD	ASD
$X = \left[\frac{4db_f}{\left(d + b_f\right)^2}\right]\frac{P_u}{\phi_c P_p}$ (14-6a)	$X = \left[\frac{4db_f}{\left(d + b_f\right)^2}\right]\frac{P_a}{P_p/\Omega_c}$ (14-6b)

Note that, because both the term in brackets and the ratio of the required strength, P_u or P_a, to the available strength, $\phi_c P_p$ or P_p/Ω_c, are always less than or equal to 1, the value of X will always be less than or equal to 1. Note also that λ can always be taken conservatively as 1. For further information, see Thornton (1990a, 1990b), and AISC Design Guide 1, *Base Plate and Anchor Rod Design* (Fisher and Kloiber, 2006).

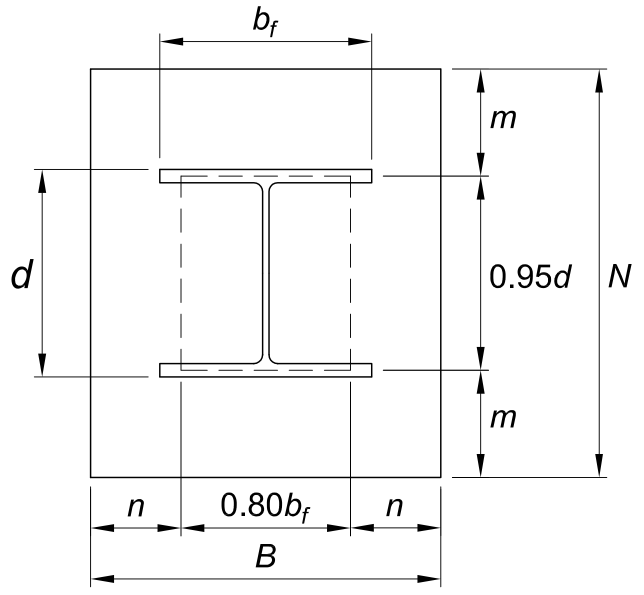


Fig. 14-3. Column base plate design variables.

Recommended Base Plate Dimensions and Thickness

The selected dimensions of the base plate, B and N, should preferably be in full inches. Base plate thickness should be specified in multiples of 1/8 in. up to 1 1/4-in. thickness and in multiples of 1/4 in. thereafter.

Available Strength

The available strength of an axially loaded column base plate is determined from the applicable limit states for connecting elements (see Part 9). From Thornton (1990a), the minimum base plate thickness can be calculated as

LRFD	ASD
$t_{min} = l\sqrt{\dfrac{2P_u}{0.90F_yBN}}$ (14-7a)	$t_{min} = l\sqrt{\dfrac{1.67(2P_a)}{F_yBN}}$ (14-7b)

The length, l, the critical base plate cantilever dimension, is determined as the larger of m, n and $\lambda n'$.

In all cases, the available strength, ϕR_n or R_n/Ω, must exceed the required strength, R_u or R_a.

Finishing Requirements

Base plate finishing requirements are given in AISC *Specification* Section M2.8. When finishing is required, the plate material must be ordered thicker than the specified base plate thickness to allow for the material removed in finishing. Finishing allowances are given in Table 14-1 per ASTM A6 flatness tolerances for steel base plates with F_u equal to or less than 60 ksi based upon the width, thickness, and whether one or both sides are to be finished. Finishing allowances for steel base plates with F_u greater than 60 ksi should be increased by 50%.

The criteria for fit-up of column splices given in AISC *Specification* Section M4.4 are also applicable to column base plates.

Holes for Anchor Rods and Grouting

Recommended anchor rod hole sizes are given in Table 14-2. These hole sizes will accommodate reasonable misalignments in the setting of the anchor rods and allow better adjustment of the column base to the correct centerlines. It is normally unnecessary to deduct the area of holes when determining the required base plate area. An adequate washer should be provided for each anchor rod.

When base plates with large areas are used, at least one grout hole should be provided near the center of the base plate through which grout may be placed. This will provide for a more even distribution of the grout and also prevent air pockets. Note that a grout hole may not be required when the grout is dry-packed. Grout holes do not require the same accuracy for size and location as anchor rod holes.

Holes in base plates for anchor rods and grouting often must be flame-cut, because drill sizes and punching capabilities may be limited to smaller diameters. Flame-cut holes may have a slight taper and should be inspected to assure proper clearances for anchor rods.

Grouting and Leveling

High-strength, non-shrink grout is placed between the column base plate and the supporting foundation. When base plates are shipped attached to the column, three methods of column support are:

1. The use of leveling nuts and, in some cases, washers on the anchor rods beneath the base plate, as illustrated in Figure 14-4.
2. The use of shim stacks between the base plate and the supporting foundation.
3. The use of a steel leveling plate (normally $^1/_4$ in. thick), set to elevation and grouted prior to the setting of the column. The leveling plate should meet the flatness tolerances specified in ASTM A6. It may be larger than the base plate to accommodate anchor rod placement tolerances and can be used as a setting template for the anchor rods.

Temporary support of a column by means of leveling nuts and shims induces forces on permanent elements of the structure, such as anchor rods and foundations. When leveling nuts and/or shims are used, the determination of required loads and associated strengths is the responsibility of the erector.

For further information on grouting and leveling of column base plates, see AISC Design Guide 10, *Erection Bracing of Low-Rise Structural Steel Frames* (Fisher and West, 1997).

When base plates are shipped loose, the base plates are usually grouted after the base plate has been aligned and leveled with one of the preceding methods. For heavy loose base plates, three-point leveling bolts, illustrated in Figure 14-5, are commonly used. These threaded attachments may consist of a nut or an angle and nut welded to the base plate. Leveling bolts must be of sufficient length to compensate for the space provided for grouting. Rounding the point of the leveling bolt will prevent it from "walking" or moving laterally as it is turned. Additionally, a small steel pad under the point reduces friction and prevents damage to the concrete.

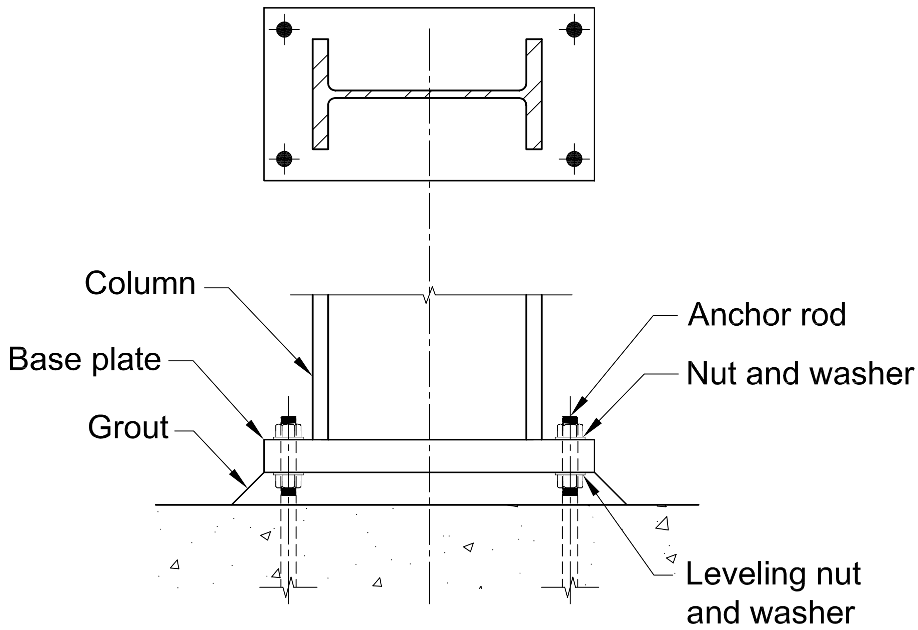


Fig. 14-4. Leveling nuts and washers.

Heavy loose base plates should be provided with some means of handling at the erection site. Lifting holes can be provided in the vertical legs of shop-attached connection angles. Lifting lugs can also be used and can remain in place after erection, unless they create an interference or removal is required in the contract documents.

Leveling bolts or nuts should not be used to support the column during erection. If grouting is delayed until after steel erection, the base plate must be shimmed to properly distribute loads to the foundation without overstressing either the base plate or the concrete. This difficulty of supporting columns while leveling and grouting their bases makes it advisable that footings be finished to near the proper elevation (Ricker, 1989). The top of the rough footing should be set approximately 1 to 2 in. below the bottom of the base plate to provide for adjustment. Alternatively, an angle frame as illustrated in Figure 14-6 could be constructed to the proper elevation and filled with grout prior to erection.

COLUMN BASE PLATES FOR AXIAL TENSION, SHEAR OR MOMENT

For anchor rod diameters not greater than $1^1/_4$ in., angles bolted or welded to the column as shown in Figure 14-7(a) are generally adequate to transfer uplift forces resulting from axial loads and moments. When greater resistance is required, stiffeners may be used with horizontal plates or angles as illustrated in Figure 14-7(b). These stiffeners are not usually considered to be part of the column area in bearing on the base plate. The angles preferably should be set back from the column end about $^1/_8$ in. Stiffeners preferably should be set back

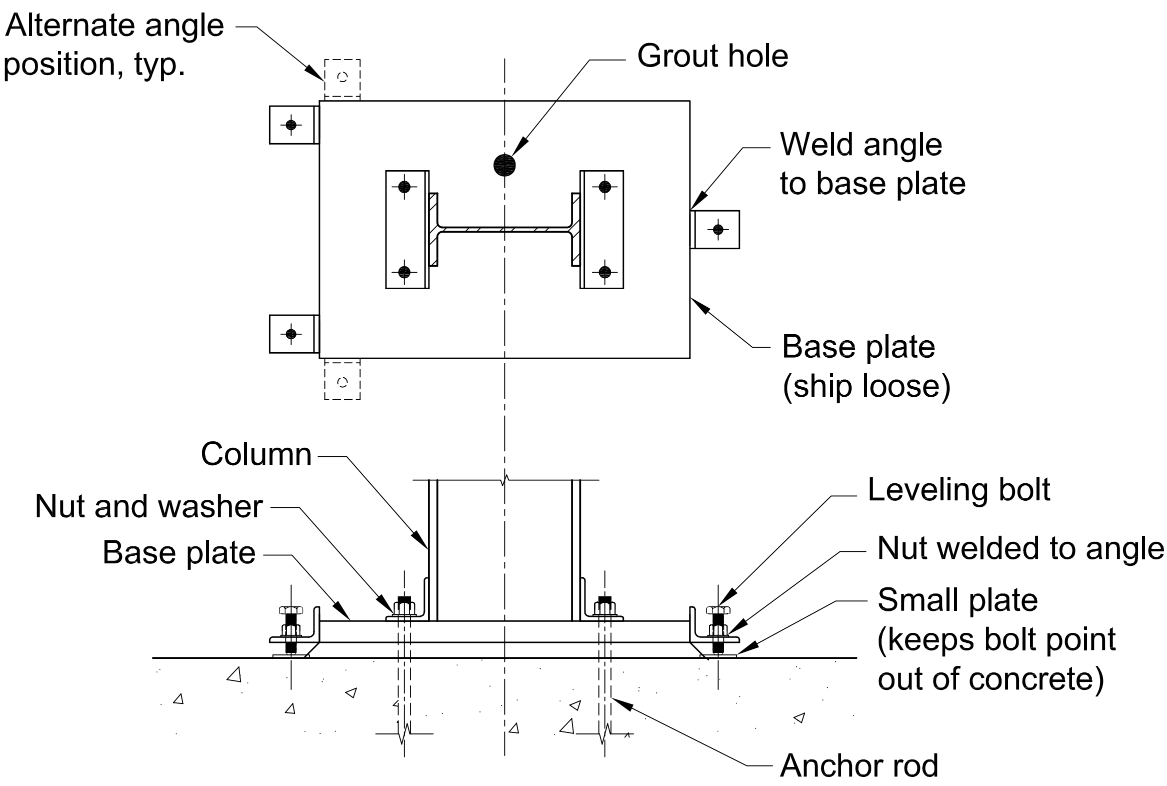


Fig. 14-5. Three-point leveling.

about 1 in. from the base plate to eliminate a pocket that might prevent drainage and, thus, protect the column and column base plate from corrosion.

For further information, see AISC Design Guide 1, *Base Plate and Anchor Rod Design* (Fisher and Kloiber, 2006).

ANCHOR RODS

Cast-in-place anchor rods, illustrated in Figure 14-8, are generally made from unheaded rod material or headed bolt material. Drilled-in (post-installed) anchors can be used for corrective work or in new work as determined by the owner's designated representative for design

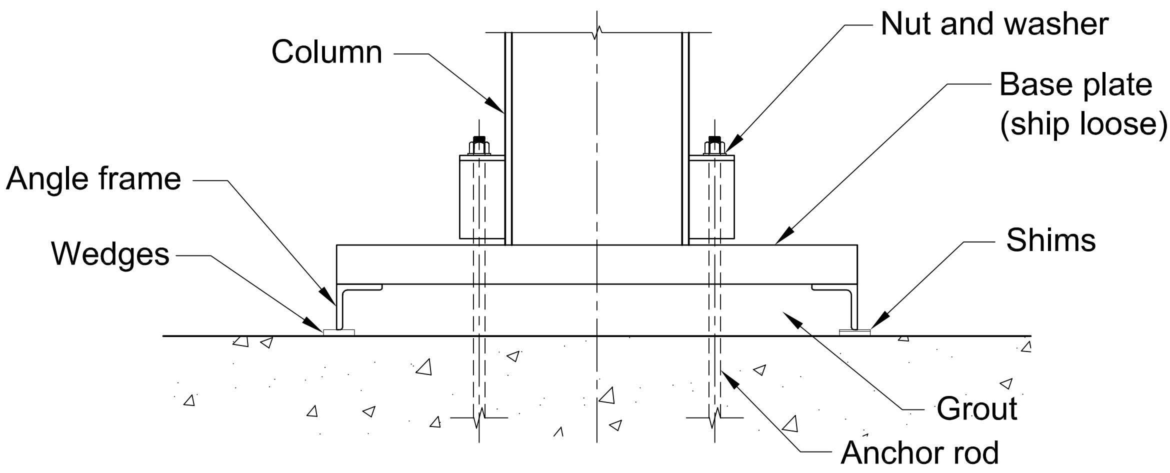


Fig. 14-6. Angle-frame leveling.

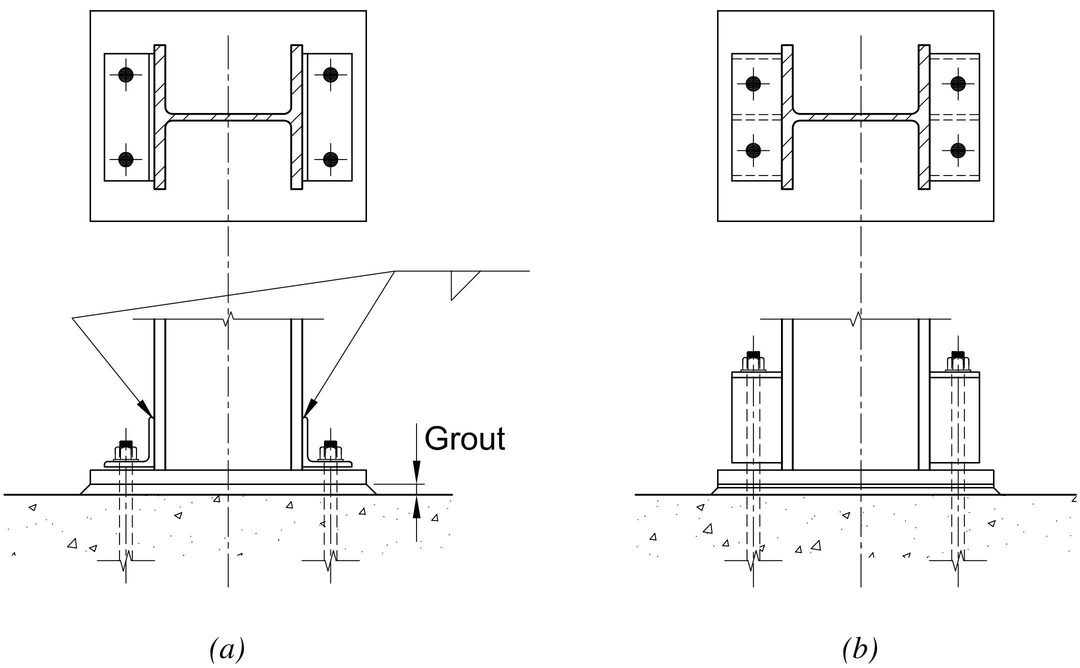


Fig. 14-7. Typical column bases for uplift.

and as permitted in the applicable building code. The design of post-installed anchors is governed by manufacturers' specifications; see also ACI 318 Chapter 17 (ACI, 2014). Post-installed anchors that rely upon torque or tension to develop anchorage by wedging action should not be used unless the stability of the column during erection is provided by means other than the post-installed anchors.

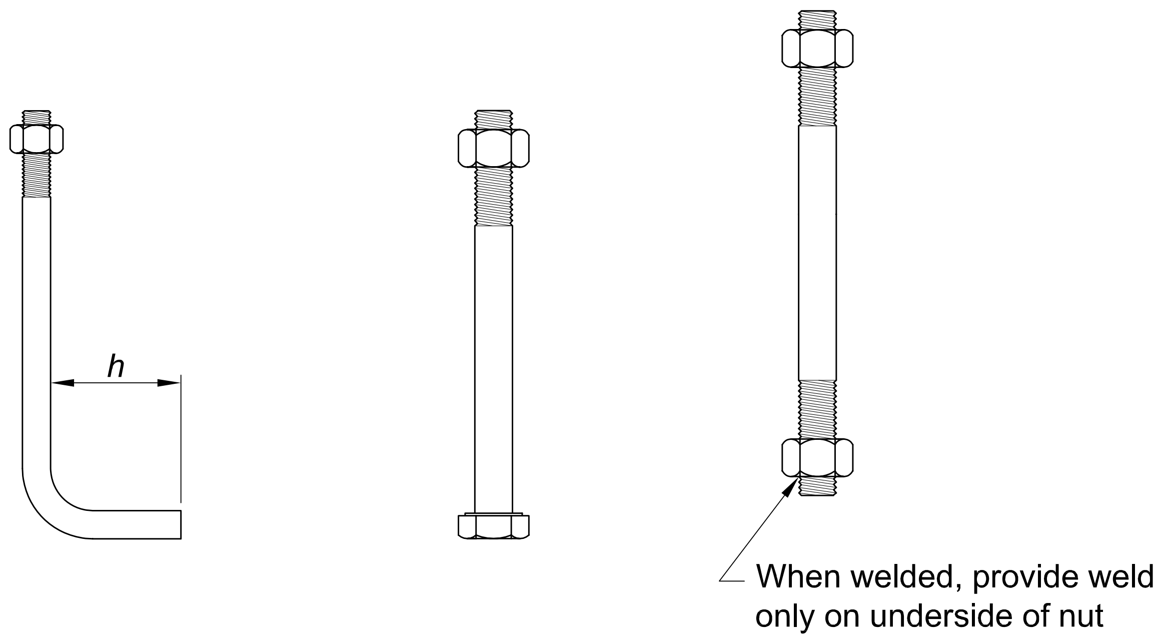
Minimum Edge Distance and Embedment Length

In general, minimum edge distances, embedment lengths, and the design of anchorages into concrete are covered by ACI 318 (ACI, 2014). These provisions include methods to account for edge distance and group action, as does ACI 349. AISC Design Guides 1, 7 and 10 provide additional material on the design of anchor rods in concrete (Fisher and Kloiber, 2006; Fisher, 2004; Fisher and West, 1997).

In addition to providing the recommended minimum embedment length, anchor rods must extend a distance above the foundation that is sufficient to permit adequate thread engagement of the nut. Adequate thread engagement for anchor rods is identical to the condition described in the RCSC *Specification* as adequate for steel-to-steel structural joints using high-strength bolts: having the end of the (anchor rod) flush with or outside the face of the nut.

Washer Requirements

Because base plates typically have holes larger than oversized holes to allow for tolerances on the location of the anchor rod, washers are usually furnished from ASTM A36 steel plate. They may be round, square or rectangular, and generally have holes that are $^{1}/_{16}$ in. larger than the anchor rod diameter. The thickness must be suitable for the forces to be transferred. Recommended washer sizes and minimum thicknesses are given in Table 14-2.



(a) Hooked *(b) Headed* *(c) Threaded with nut*

Fig. 14-8. Cast-in-place anchor rods.

Hooked Anchor Rods

Hooked anchor rods, as illustrated in Figure 14-8(a), should be used only for axially loaded members subject to compression only to locate and prevent the displacement or overturning of columns due to erection loads or accidental collisions during erection. Additionally, high-strength steels are not recommended for use in hooked rods since bending with heat may materially affect their strength.

Headed or Threaded and Nutted Anchor Rods

When anchor rods are required for a calculated tensile force, *T*, a more positive anchorage is formed when headed anchor rods, illustrated in Figure 14-8(b), are used. With adequate embedment and edge distance, the limit state is either a tensile failure of the anchor rod or the breakout of a truncated pyramid of concrete radiating outward from the head as illustrated in Figure 14-9. Marsh and Burdette (1985a, 1985b) showed that the head of the anchor rod usually provides sufficient anchorage and the use of an additional washer or plate does not add significantly to the anchorage. The nut and threading shown in Figure 14-8(c) is acceptable in lieu of a bolt head. The nut should be welded to the rod on the underside of the nut to prevent the rod from turning out when the top nut is tightened. Alternatively, the nut can be secured by means of a jammed double nut, or deformed threads above and below the nut.

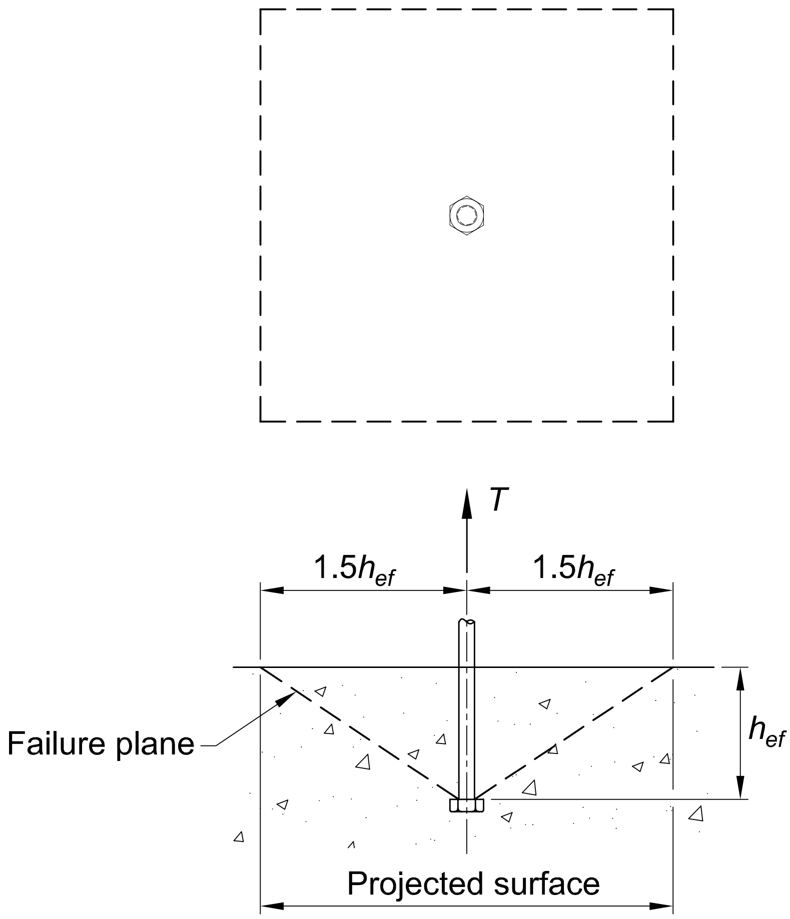


Fig. 14-9. Concrete truncated pyramid subject to breakout.

Anchor Rod Nut Installation

The majority of anchorage applications in buildings do not require special anchor rod nut installation procedures or pretension in the anchor rod. The anchor rod nuts should be "drawn down tight" as columns and bases are erected, per ANSI/ASSE A10.13 Section 9.6 (ASSE, 2011). This condition can be achieved by following the same practices as recommended for snug-tightened installation in steel-to-steel bolted joints in the RCSC *Specification*. Snug-tight is the condition that exists when all plies in a connection have been pulled into firm contact by the bolts in the joint and all the bolts in the joint have been tightened sufficiently to prevent the removal of the nuts without the use of a wrench.

When, in the judgment of the owner's designated representative for design, the performance of the structure will be compromised by excessive elongation of the anchor rods under tensile loads, pretension may be required. Some examples of applications that may require pretension include structures that cantilever from concrete foundations, moment-resisting column bases with significant tensile forces in the anchor rods, or where load reversal might result in the progressive loosening of the nuts on the anchor rods.

When pretensioning of anchor rods is specified, care must be taken in the design of the column base and the embedment of the anchor rod. The shaft of the anchor rod must be free of bond to the encasing concrete so that the rod is free to elongate as it is pretensioned. Also, loss of pretension due to creep in the concrete must be taken into account. Although the design of pretensioned anchorage devices is beyond the scope of this Manual, it should be noted that pretension should not be specified for anchorage devices that have not been properly designed and configured to be pretensioned.

COLUMN SPLICES

When the height of a building exceeds the available length of column sections, or when it is economically advantageous to change the column size at a given floor level, it becomes necessary to splice two columns together. Column splices at the final exterior and interior perimeter and at interior openings must be located a minimum of 48 in. above the finished floor to accommodate the attachment of safety cables, except when constructability does not allow. For simplicity and uniformity, other column splices should be located at the same height. Note that column splices placed significantly higher than this are impractical in terms of field assembly.

Fit-Up of Column Splices

From AISC *Specification* Section M2.6, the ends of columns in a column splice which depend upon contact bearing for the transfer of axial forces must be finished to a common plane by milling, sawing, or other equivalent means. In theory, if this were done and the pieces were erected truly plumb, there would be full contact bearing across the entire surface; this is true in most cases. However, AISC *Specification* Section M4.4 recognizes that a perfect fit on the entire available surface will not exist in all cases.

A $^1/_{16}$-in. gap is permissible with no requirements for repair or shimming. During erection, at the time of tightening the bolts or depositing the welds, columns will usually be subjected to loads that are significantly less than the design loads. Full-scale tests (Popov and Stephen, 1977) that progressed to column failure have demonstrated that subsequent loading to the design loads does not result in distress in the bolts or welds of the splice.

If the gap exceeds 1/16 in. but is equal to or less than 1/4 in., and if an engineering investigation shows that sufficient contact area does not exist, nontapered steel shims are required. Mild steel shims are acceptable regardless of the steel grade of the column or bearing material. If required, these shims must be contained, usually with a tack weld, so that they cannot be worked out of the joint.

There is no provision in the AISC *Specification* for gaps larger than 1/4 in. When such a gap exists, an engineering evaluation should be made of this condition based upon the type of loading transferred by the column splice. Tightly driven tapered shims may be required or the required strength may be developed through flange and web splice plates. Alternatively, the gap may be ground or gouged to a suitable profile and filled with weld metal.

Lifting Devices

As illustrated in Figure 14-10, lifting devices are typically used to facilitate the handling and erection of columns. When flange-plated or web-plated column splices are used for W-shape columns, it is convenient to place lifting holes in these flange plates as illustrated in Figure 14-10(a). When butt-plated column splices are used, additional temporary plates with lifting holes may be required as illustrated in Figure 14-10(b). W-shape column splices which do not utilize web-plated or butt-plated column splices (i.e., groove-welded column splices) may be provided with a lifting hole in the column web as illustrated in Figure 14-10(c). While a hole in the column web reduces the cross-sectional area of the column, this reduction will seldom be critical since the column is sized for the loads at the floor below and the splice is located above the floor. Alternatively, auxiliary plates with lifting holes may be connected to the column so that they do not interfere with the welding. Typical column splices for HSS and box-section columns are illustrated in Figure 14-10(d). Holes in lifting devices may be drilled, reamed or flame-cut with a mechanically guided torch. In the latter case, the bearing surface of the hole in the direction of the lift must be smooth.

The lifting device and its attachment to the column must be of sufficient strength to support the weight of the column as it is brought from the horizontal position (as delivered) to the vertical position (as erected); the lifting device and its attachment to the column must be adequate for the tensile forces, shear forces and moments induced during handling and erection.

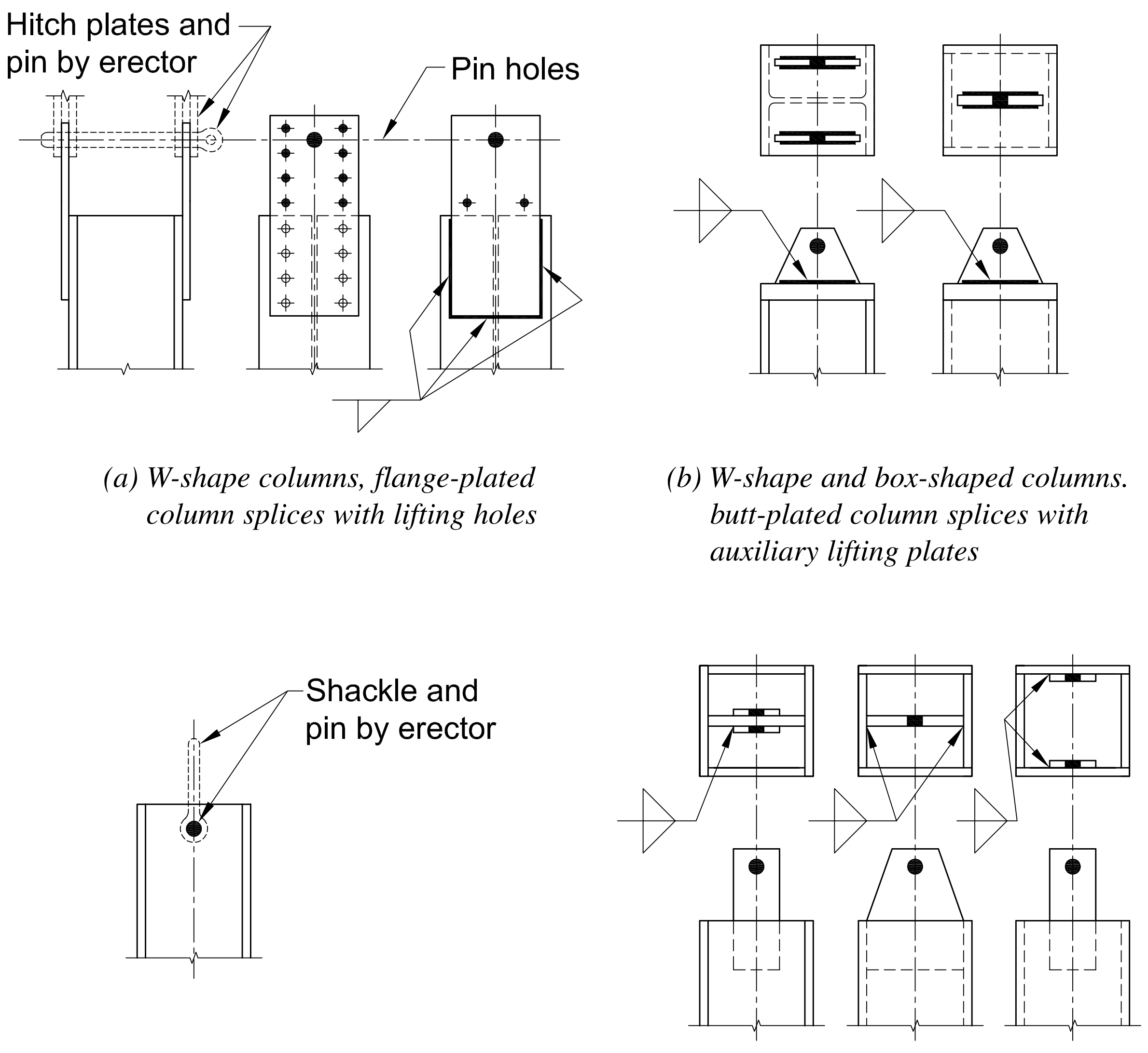
A suitable shackle and pin are connected to the lifting device while the column is on the ground. The steel erector usually establishes the size and type of shackle and pin to be used in erection and this information must be transmitted to the fabricator prior to detailing. Except for excessively heavy lifting pieces, it is customary to select a single pin and pinhole diameter to accommodate the majority of structural steel members, whether they are columns or other heavy structural steel members. The pin is attached to the lifting hook and a lanyard trails to the ground or floor level. After the column is erected and connected, the pin is removed from the device by means of the lanyard, eliminating the need for an ironworker to climb the column. The shackle pin, as assembled with the column, must be free and clear, so that it may be withdrawn laterally after the column has been landed and stabilized.

The safety of the structure, equipment and personnel is of utmost importance during the erection period. It is recommended that all welds that are used on the lifting devices and stability devices be inspected very carefully, both in the shop and later in the field, for any damage that may have occurred in handling and shipping. Groove welds frequently are inspected with ultrasonic methods (UT) and fillet welds are inspected with magnetic particle (MT) or liquid dye penetrant (PT) methods.

Column Alignment and Stability During Erection

Column splices should provide for safety and stability during erection when the columns might be subjected to wind, construction, and/or accidental loading prior to the placing of the floor system. The nominal flange-plated, web-plated, and butt-plated column splices developed here consider this type of loading.

In other splices, column alignment and stability during erection are achieved by the addition of temporary lugs for field bolting as illustrated in Figure 14-11. The material thickness, weld size and bolt diameter required are a function of the loading. A conservative resisting moment arm is normally taken as the distance from the compressive toe or flange face to the gage line of the temporary lug. The overturning moment should be checked about both axes



(a) W-shape columns, flange-plated column splices with lifting holes

(b) W-shape and box-shaped columns. butt-plated column splices with auxiliary lifting plates

(c) W-shape columns, no splice plates, lifting hole in column web

(d) HSS and box-section columns, auxiliary lifting plates

Fig. 14-10. Lifting devices for columns.

of the column. The recommended minimum plate or angle thickness is 1/2 in.; the recommended minimum weld size is 5/16 in.; additionally, high-strength bolts are normally used as stability devices.

Temporary lugs are not normally used as lifting devices. Unless required to be removed in the contract documents, these temporary lugs may remain.

Column alignment is provided with centerpunch marks that are useful in centering the columns in two directions.

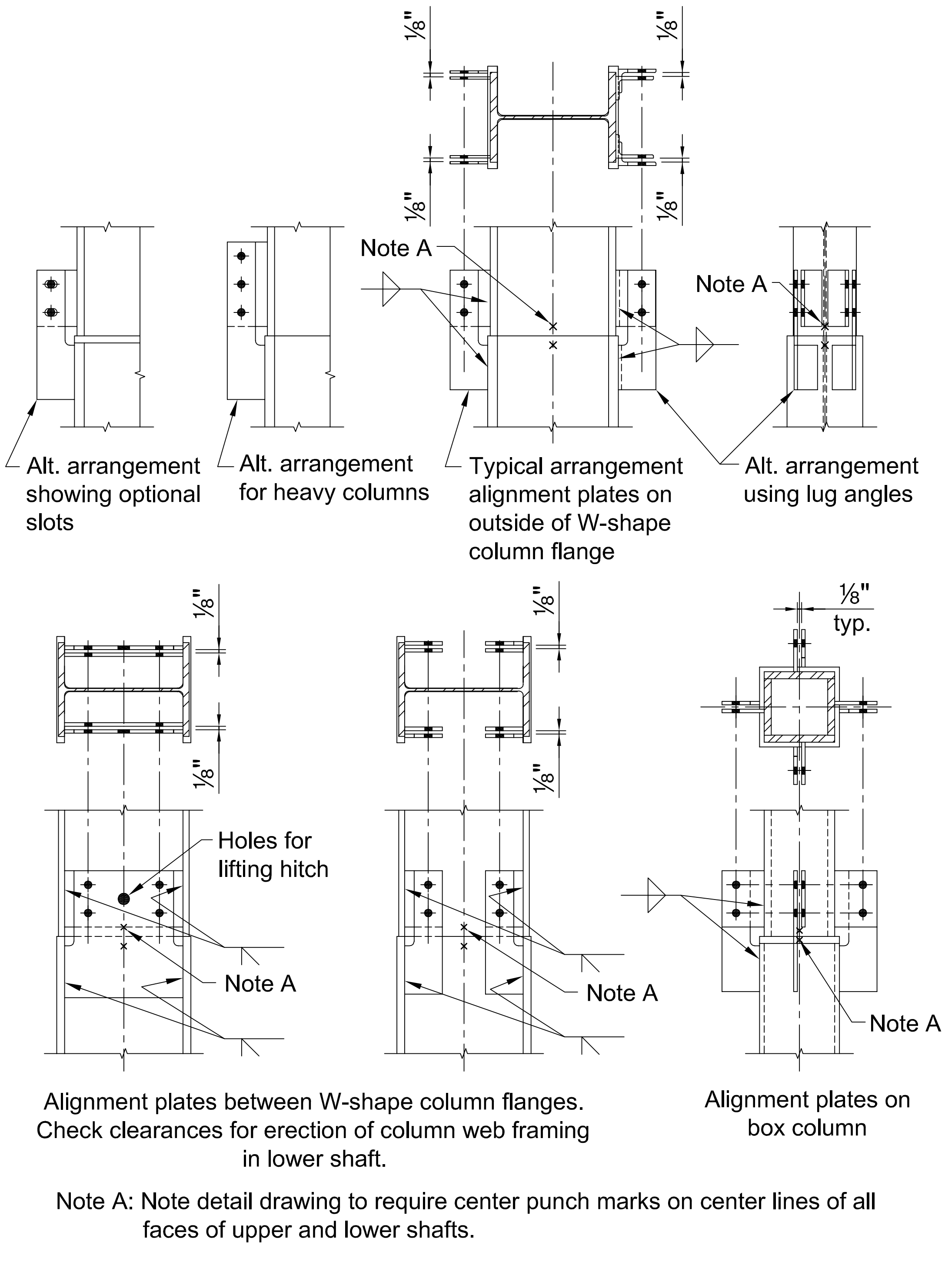


Fig. 14-11. Column stability and alignment devices.

Force Transfer in Column Splices

As illustrated in Figure 14-12, for the W-shapes most frequently used as columns, the distance between the inner faces of the flanges is constant throughout any given nominal depth group; as the nominal weight per foot increases for each nominal depth, the flange and web thicknesses increase. The available bearing strength, ϕR_n or R_n/Ω, of the contact area of a finished surface is determined using AISC *Specification* Equation J7-1:

$$R_n = 1.8F_y A_{pb} \qquad (Spec.\text{ Eq. J7-1})$$

$$\phi = 0.75 \qquad \Omega = 2.00$$

where

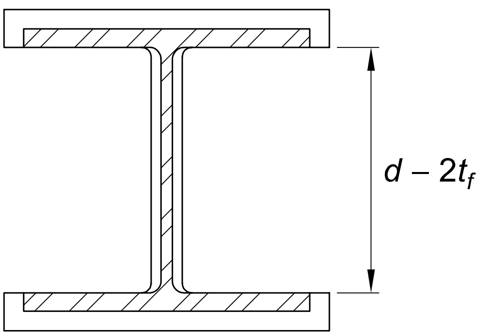
A_{pb} = projected area in bearing, in.^{2}
F_y = specified minimum yield stress of the column, ksi

This bearing strength is much greater than the axial strength of the column and will seldom prove critical in the member design. For column splices transferring only axial forces, complete axial force transfer may be achieved through bearing on finished surfaces; bolts or welds are required by AISC *Specification* Section J1.4 to be sufficient to hold all parts securely in place.

In addition to axial compressive forces, from AISC *Specification* Section J1.4, column splices must be proportioned to achieve the required strength in tension, due to the combination of dead load and lateral loads. Note that it is not permissible to use forces due to live load to offset the tensile forces from wind or seismic loads. Additional column splice requirements are provided in the AISC *Seismic Provisions*.

For dead and wind loads, if the required strength due to the effect of the dead load is greater than the required strength due to the wind load, the splice is not subjected to tension and a nominal splice may be selected from those in Table 14-3. When the required strength due to dead load is less than the required strength due to the wind load, the splice will be subjected to tension and the nominal splices from Table 14-3 are acceptable if the available tensile strength of the splice is greater than or equal to the required strength. Otherwise, a splice must be designed with sufficient area and attachment.

When shear from lateral loads is divided among several columns, the force on any single column is relatively small and can usually be resisted by friction on the contact bearing surfaces and/or by the flange plates, web plates or butt plates. If the required shear strength



Column Size	**$d - 2t_f$ (in.)**
W8×24–67	7.13
W10×33–112	8.86
W12×40–336	10.9
W14×43–873	12.6

Fig. 14-12. Distance between flanges for typical W-shape columns.

exceeds the available shear strength of the column splice selected from Table 14-3, a column splice must be designed with sufficient area and attachment.

The column splices shown in Table 14-3 meet the OSHA requirement for 300 lb located 18 in. from the column face.

Flange-Plated Column Splices

Table 14-3 gives typical flange-plated column splice details for W-shape columns. These details are not splice requirements, but rather, typical column splices in accordance with the AISC *Specification* and typical erection requirements. Other splice designs may also be developed. It is assumed in all cases that the lower shaft will be the heavier, although not necessarily the deeper, section.

Full-contact bearing is always achieved when lighter sections are centered over heavier sections of the same nominal depth group. If the upper column is not centered on the lower column, or if columns of different nominal depths must bear on each other, some areas of the upper column will not be in contact with the lower column. These areas are hatched in Figure 14-13.

When additional bearing area is not required, unfinished fillers may be used. These fillers are intended for "pack-out" of thickness and are usually set back 1/4 in. or more from the finished column end. Since no force is transferred by these fillers, only nominal attachment to the column is required.

When additional bearing area is required, fillers finished to bear on the larger column may be provided. Such fillers are proportioned to carry bearing loads at the bearing strength calculated from AISC *Specification* Section J7 and must be connected to the column to transfer this calculated force.

In Table 14-3, Cases I and II are for all-bolted flange-plated column splices for W-shape columns. Bolts in column splices are usually the same size and type as for other bolts on the column. Bolt spacing, end distance and edge distances resulting from the plate sizes shown permit the use of 3/4-in.- and 7/8-in.-diameter bolts in the splice details shown. Larger diameter bolts may require an increase in edge or end distances. Refer to AISC *Specification* Chapter J. The use of high-strength bolts in bearing-type connections is assumed in all field

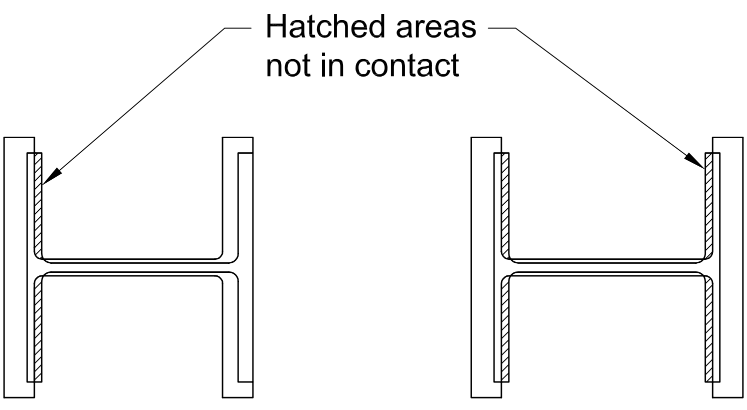


Fig. 14-13. Columns not centered or of different nominal depth.

and shop splices. However, when slotted or oversized holes are utilized, a slip-critical connection is required. For ease of erection, field clearances for lap splices in Table 14-3 fastened by bolts range from $^1/_8$ in. to $^3/_{16}$ in. under each plate.

Cases IV and V are for all-welded flange-plated column splices for W-shape columns. Splice welds are assumed to be made with E70XX electrodes and are proportioned as required by the AISC *Specification*. The GMAW and FCAW equivalents to E70XX electrodes may be substituted if desired. Field clearance for welded splices are limited to $^1/_{16}$ in. to control the expense of building up welds to close openings. Note that the fillet weld lengths, *Y,* as compared to the lengths $l/2$, provide 2-in. unwelded distance below and above the column shaft finish line. This provides a degree of flexibility in the splice plates to assist the erector.

Cases VI and VII apply to combination bolted and welded column splices. Since the available strength of the welds will, in most cases, exceed the strength of the bolts, the weld and splice lengths shown may be reduced, if desired, to balance the strength of the fasteners to the upper or lower column, provided that the available strength of the splice is still greater than the required strength of the splice, including erection loading.

Directly Welded Flange Column Splices

Table 14-3 also includes typical directly welded flange column splice details for W-shape and HSS or box-shaped columns. These details are not splice requirements, but rather, typical column splices in accordance with AISC *Specification* provisions and typical erection requirements. Other splice designs may also be developed. It is assumed in all cases that the lower shaft will be the heavier, although not necessarily the deeper, section.

Case VIII applies to W-shape columns spliced with either partial-joint-penetration or complete-joint-penetration groove welds. Case X applies to HSS or box-section columns spliced with partial-joint-penetration or complete-joint-penetration groove welds.

Butt-Plated Column Splices

Table 14-3 includes typical butt-plated column splice details for W-shape and HSS or box-section columns. These details are not splice requirements, but rather, present typical column splices in accordance with AISC *Specification* provisions and typical erection requirements. Other splice designs may also be developed. It is assumed in all cases that the lower shaft will be the heavier, although not necessarily the deeper, section.

Butt plates are used frequently on welded splices where the upper and lower columns are of different nominal depths, but may not be economical for bolted splices since fillers cannot be eliminated. Typical butt plates are $1^1/_2$ in. thick for a W8 over W10 splice, and 2 in. thick for other W-shape combinations such as W10 over W12 and W12 over W14. Butt plates that are subjected to substantial bending stresses, such as required on box-section columns, will require a more careful review and analysis. One common method is to assume forces are transferred through the butt plate on a 45° angle and check the thickness obtained for shear and bearing strength. Finishing requirements for butt plates are specified in AISC *Specification* Section M2.8.

Case III is a combination flange-plated and butt-plated column splice for W-shape columns. Case IX applies to welded butt-plated column splices for W-shape columns. Case XI applies to welded butt-plated column splices for HSS or box-section columns. Case XII applies to welded butt-plated column splices between W-shape and HSS or box-section columns.

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Table 14-1
Finish Allowances

Size	Thickness, in.	Add to Finish One Side, in.	Add to Finish Two Sides, in.
Maximum dimension 24 in. or less	$1^1/_4$ or less	$^1/_{16}$	$^1/_8$
	over $1^1/_4$ to 2, incl.	$^1/_8$	$^1/_4$
Maximum dimension over 24 in.	$1^1/_4$ or less	$^1/_8$	$^1/_4$
	over $1^1/_4$ to 2, incl.	$^3/_{16}$	$^3/_8$
56 in. wide or less	over 2 to $7^1/_2$, incl.	$^1/_4$	$^3/_8$
	over $7^1/_2$ to 10, incl.	$^1/_2$	$^5/_8$
	over 10 to 15, incl.	$^3/_4$	$^7/_8$
Over 56 in. wide to 72 in. wide	over 2 to 6, incl.	$^1/_4$	$^3/_8$
	over 6 to 10, incl.	$^1/_2$	$^5/_8$
	over 10 to 15, incl.	$^3/_4$	$^7/_8$

Note: These allowances apply for material with $F_u \leq 60$ ksi.

Table 14-2
Recommended Sizes for Washers and Anchor Rod Holes in Base Plates

Anchor Rod Diameter	Hole Diameter	Washer Size	Min. Washer Thickness	Anchor Rod Diameter	Hole Diameter	Washer Size	Min. Washer Thickness
in.	in.	in.	in.	in.	in.	in.	in.
$^3/_4$	$1^5/_{16}$	2	$^1/_4$	$1^1/_2$	$2^3/_8$	4	$^1/_2$
$^7/_8$	$1^9/_{16}$	$2^1/_2$	$^5/_{16}$	$1^3/_4$	$2^7/_8$	$4^1/_2$	$^5/_8$
1	$1^7/_8$	3	$^3/_8$	2	$3^1/_4$	5	$^3/_4$
$1^1/_4$	$2^1/_8$	$3^1/_2$	$^1/_2$	$2^1/_2$	$3^3/_4$	$5^1/_2$	$^7/_8$

Notes:
1. Hole sizes provided are based on anchor rod size and correlate with ACI 117 (ACI, 2010).
2. Circular or square washers meeting the washer size are acceptable.
3. Clearance must be considered when choosing an appropriate anchor rod hole location, noting effects such as the position of the rod in the hole with respect to the column, weld size, and other interferences.
4. ASTM F844 washers are permitted instead of plate washers when hole clearances are limited to $^5/_{16}$ in. for rod diameters up to 1 in., $^1/_2$ in. for rod diameters over 1 in. up to 2 in., and 1 in. for rod diameters over 2 in. This exception should not be used unless the general contractor has agreed to meet smaller tolerances for anchor rod placement than those permitted in ACI 117.

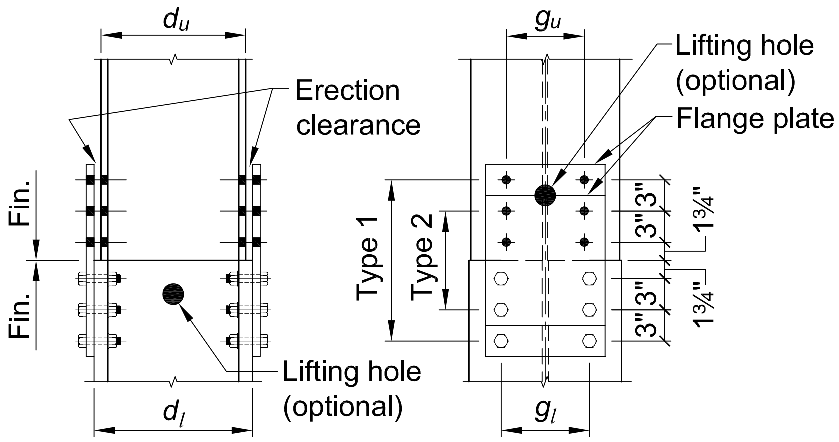
Table 14-3
Typical Column Splices
Case I:
All-bolted flange-plated column splices between columns with depth d_u and d_l nominally the same

Column Size	Gage[a], g_u or g_l	Flange Plates			
		Type	Width	Thick.	Length
	in.		in.	in.	
W14×455 to 873	13 1/2	1	16	3/4	1′-6 1/2″
×257 to 426	11 1/2	1	14	5/8	1′-6 1/2″
×145 to 233	11 1/2	1	14	1/2	1′-6 1/2″
×90 to 132	11 1/2	2	14	3/8	1′-0 1/2″
×43 to 82	5 1/2	2	8	3/8	1′-0 1/2″
W12×120 to 336	5 1/2	2	8	5/8	1′-0 1/2″
×40 to 106	5 1/2	2	8	3/8	1′-0 1/2″
W10×33 to 112	5 1/2	2	8	3/8	1′-0 1/2″
W8×31 to 67	5 1/2	2	8	3/8	1′-0 1/2″
×24 & 28	4	2	6	3/8	1′-0 1/2″

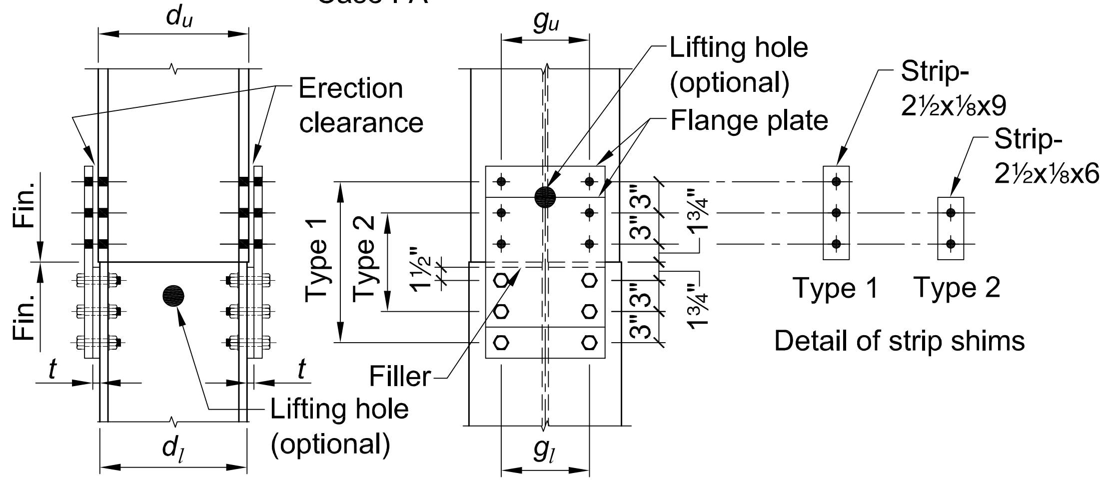
Case I-A: $d_l = (d_u + 1/4$ in.) to $(d_u + 5/8$ in.)	Flange plates: Select g_u for upper column; select g_l and flange plate dimensions for lower column. Fillers: None. Shims: Furnish sufficient strip shims 2 1/2×1/8 to provide 0 to 1/16-in. clearance each side.
Case I-B: $d_l = (d_u - 1/4$ in.) to $(d_u + 1/8$ in.)	Flange plates: Same as Case I-A. Fillers (shop bolted under flange plates): Select thickness as 1/8 in. for $d_l = d_u$ and $d_l = (d_u + 1/8$ in.) or as 1/4 in. for $d_l = (d_u - 1/8$ in.) and $d_l = (d_u - 1/4$ in.). Select width to match flange plate and length as 0′-9″ for Type 1 or 0′-6″ for Type 2. Shims: Same as Case I-A.
Case I-C: $d_l = (d_u + 3/4$ in.) and over	Flange plates: Same as Case I-A. Fillers (shop bolted to upper column): Select thickness as $(d_l - d_u)/2$ minus 1/8 in. or 3/16 in., whichever results in 1/8 in. multiples of filler thickness. Select width to match flange plate, but not greater than upper column flange width. Select length as 1′-0″ for Type 1 or 0′-9″ for Type 2. Shims: Same as Case I-A.

[a] Gages shown may be modified if necessary to accommodate fittings elsewhere on the column.
Note: For lifting devices, see Figure 14-10.

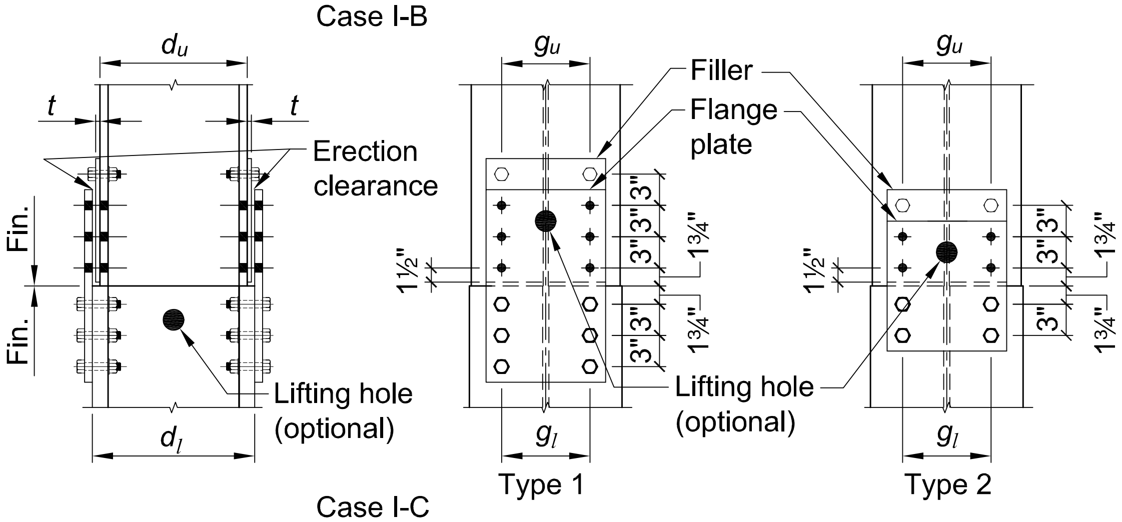
Table 14-3 (continued)
Typical Column Splices
Case I:
All-bolted flange-plated column splices between columns with depth d_u and d_l nominally the same



Case I-A



Case I-B



Case I-C

Table 14-3 (continued)
Typical Column Splices
Case II:
All-bolted flange-plated column splices between columns with depth d_u nominally 2 in. less than depth d_l

Fillers on upper column developed for bearing on lower column.	Flange plates: Same as Case I-A. Fillers (shop bolted to upper column): Select thickness, t, as $(d_l - d_u)/2 - 1/8$ in. or $3/16$ in., whichever results in $1/8$-in. multiples of filler thickness. Select bolts through fillers (including bolts through flange plates) on each side to develop bearing strength of the filler. Select width to match flange plate, but not greater than upper column flange width unless required for bearing strength. Select length as required to accommodate required number of bolts. Shims: Same as Case I-A.

Table 14-3 (continued)
Typical Column Splices
Case III:
All-bolted flange-plated and butt-plated column splices between columns with depth d_u nominally 2 in. less than depth d_l

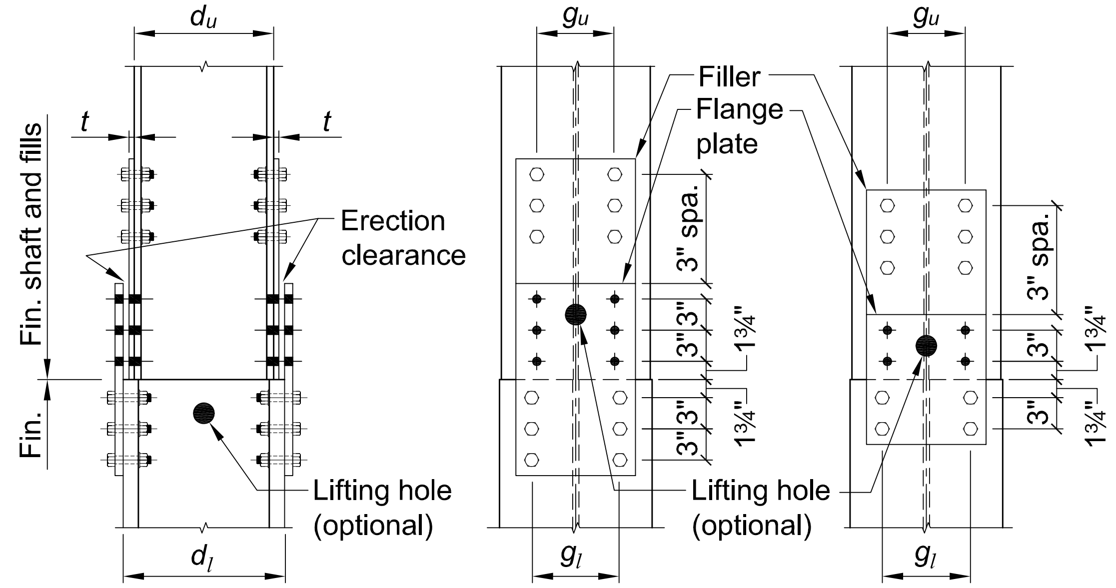
Fillers on upper column developed for bearing on lower column.

Column Size	Gage[a], g_u or g_l	Flange Plates			Length
		Type	Width	Thick.	
	in.		in.	in.	
W14×455 to 873	13 1/2	1	16	3/4	1′-8 1/2″
×257 to 426	11 1/2	1	14	5/8	1′-8 1/2″
×145 to 233	11 1/2	1	14	1/2	1′-8 1/2″
×90 to 132	11 1/2	2	14	3/8	1′-2 1/2″
×43 to 82	5 1/2	2	8	3/8	1′-2 1/2″
W12×120 to 336	5 1/2	2	8	5/8	1′-2 1/2″
×40 to 106	5 1/2	2	8	3/8	1′-2 1/2″
W10×33 to 112	5 1/2	2	8	3/8	1′-2 1/2″
W8×31 to 67	5 1/2	2	8	3/8	1′-2″
×24 & 28	3 1/2	2	8	3/8	1′-2″

Flange plates: Select g_u for upper column, select g_l and flange plate dimensions for lower column.
Fillers (shop bolted to upper column): Same as Case I-C.
Shims: Same as Case I-A.
Butt plate: Select thickness as 1 1/2 in. for W8 upper column or 2 in. for others. Select width the same as upper column and length as $d_l - 1/4$ in.

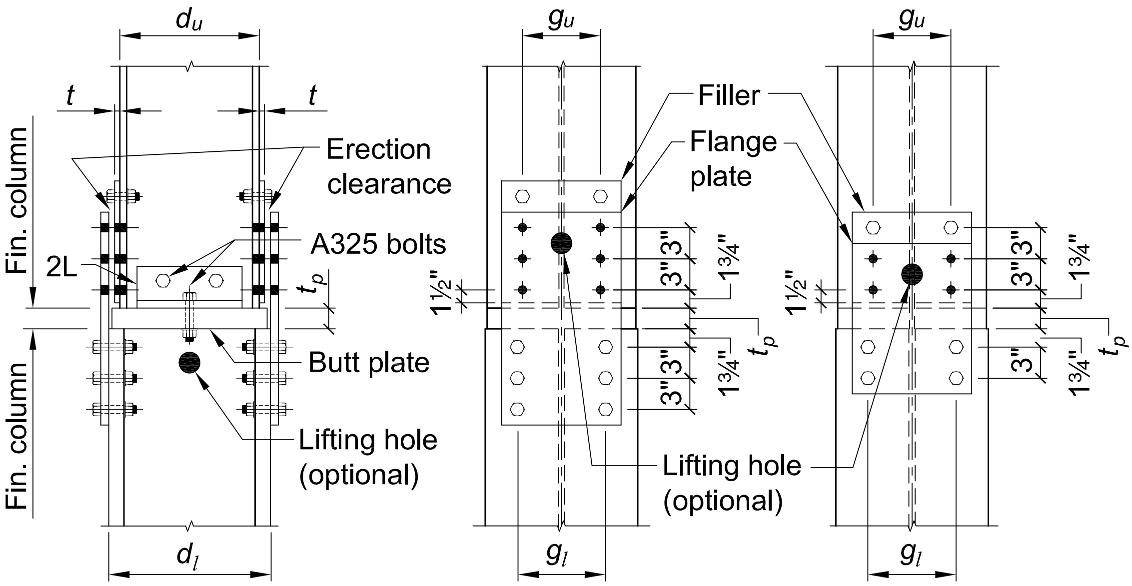
[a] Gages shown may be modified if necessary to accommodate fittings elsewhere on the column.
Note: For lifting devices, see Figure 14-10.

Table 14-3 (continued)
Typical Column Splices
Case II and III:
All-bolted flange-plated column splices between columns with depth d_u nominally 2 in. less than depth d_l



Type 1 Type 2

Case II



Type 1 Type 2

Case III

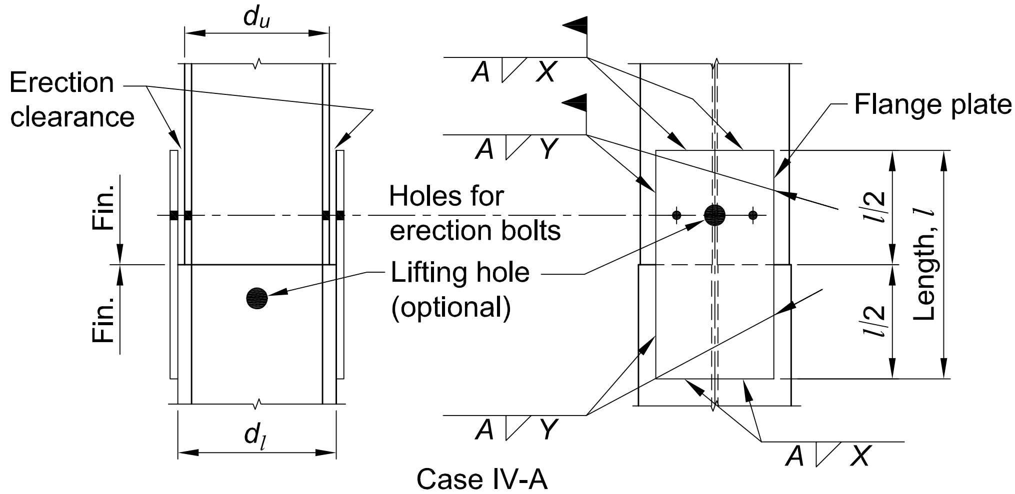
Table 14-3 (continued)
Typical Column Splices
Case IV:
All-welded flange-plated column splices between columns with depth d_u and d_l nominally the same

Column Size	Flange Plate			Welds			Minimum Space for Welding	
	Width	Thick.	Length, l	Size, A	Length			
					X	Y	M	N
	in.	in.		in.	in.	in.	in.	in.
W14×455 & over	14	5/8	1′-6″	1/2	5	7	13/16	11/16
×311 to 426	12	5/8	1′-4″	1/2	4	6	13/16	11/16
×211 to 283	12	1/2	1′-4″	3/8	4	6	11/16	9/16
×90 to 193	12	3/8	1′-4″	5/16	4	6	5/8	1/2
×61 to 82	8	3/8	1′-4″	5/16	3	6	5/8	1/2
×43 to 53	6	5/16	1′-2″	1/4	2	5	9/16	7/16
W12×120 to 336	8	1/2	1′-4″	3/8	3	6	11/16	9/16
×53 to 106	8	3/8	1′-4″	5/16	3	6	5/8	1/2
×40 to 50	6	5/16	1′-2″	1/4	2	5	9/16	7/16
W10×49 to 112	8	3/8	1′-4″	5/16	3	6	5/8	1/2
×33 to 45	6	5/16	1′-2″	1/4	2	5	9/16	7/16
W8×31 to 67	6	3/8	1′-2″	5/16	2	5	5/8	1/2
×24 & 28	5	5/16	1′-0″	1/4	2	4	9/16	7/16

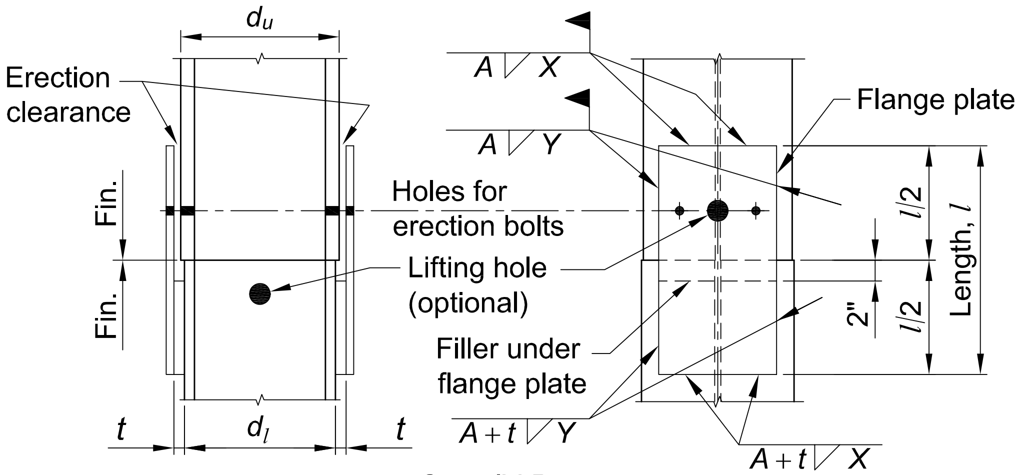
Case IV-A: $d_l = (d_u + 1/8$ in.)	Flange plates: Select flange-plate width and length and weld lengths for upper (lighter) column; select flange plate thickness and weld size for lower (heavier) column. Fillers: None.
Case IV-B: $d_l = (d_u - 1/4$ in.) to d_u	Flange plates: Same as Case IV-A, except use weld size, $A + t$, on lower column. Fillers (underdeveloped on lower column, shop welded under flange plates): Select thickness, t, as $[(d_l - d_u)/2] + 1/16$ in. Select width to match flange plate and length as $(l/2) - 2$ in.
Case IV-C: $d_l = (d_u + 1/4$ in.) to $(d_u + 1/2$ in.)	Flange plates: Same as Case IV-A, except use weld size, $A + t$, on upper column. Fillers (underdeveloped on upper column, shipped loose): Select thickness, t, as $[(d_l - d_u)/2] - 1/16$ in. Select width to match flange plate and length as $(l/2) - 2$ in.

Note: For lifting devices, see Figure 14-10.

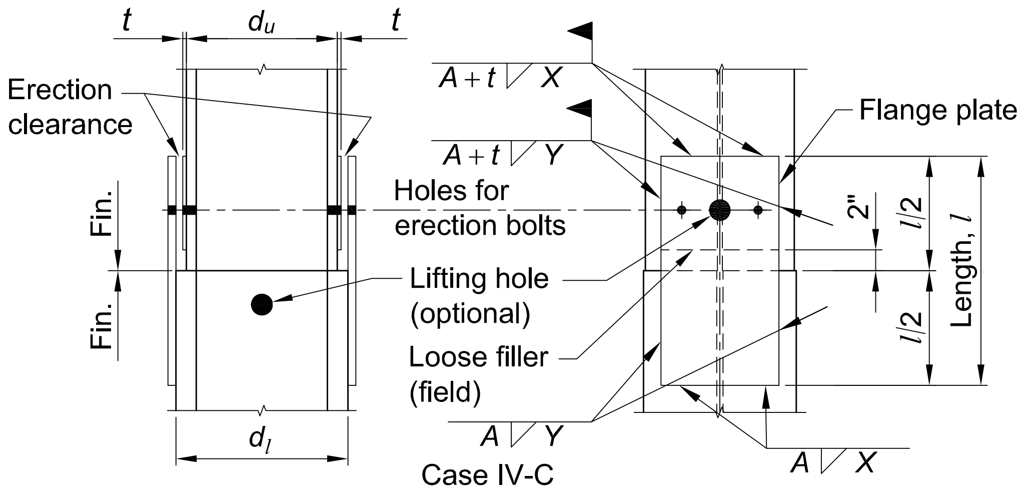
Table 14-3 (continued)
Typical Column Splices
Case IV:
All-welded flange-plated column splices between columns with depth d_u nominally 2 in. less than depth d_l



Case IV-A



Case IV-B



Case IV-C

Table 14-3 (continued)
Typical Column Splices
Case IV:
All-welded flange-plated column splices between columns with depths d_u and d_l nominally the same

Case IV-D: $d_l = (d_u + 5/8$ in.) and over Filler width less than upper column flange width.	Flange plates: Same as Case IV-A, except see Note 1. Fillers (developed on upper column, shop welded to upper column): Select thickness, t, as $[(d_l - d_u)/2] - 1/16$ in. Select weld size, B, from the AISC *Specification* Section J2; 5/16 in. or less preferred. Select weld length, l_B, such that $l_B \geq [A(X + Y)/B] \geq (l/2 + 1$ in.). Select filler width greater than flange plate width plus 2N, but less than upper column flange width minus 2M. Select filler length, l_B, subject to Note 2.
Case IV-E: $d_l = (d_u + 5/8$ in.) and over Filler width greater than upper column flange width. Use this case only when M or N in Case IV-D are inadequate for welds B and A.	Flange plates: Same as Case IV-A, except see Note 1. Fillers (developed on upper column, shop welded to upper column): Select thickness, t, as $[(d_l - d_u)/2] - 1/16$ in. Select weld size, B, from the AISC *Specification* Section J2; 5/16 in. or less preferred. Select weld length, l_B, such that $l_B \geq [A(X + Y)/B] \geq (l/2 + 1$ in.). Select filler width as the larger of the flange plate width plus 2N and the upper column flange width plus 2M, rounded to the next higher 1/4-in. increment. Select filler length, l_B, subject to Note 2.

Note 1: Where welds fasten flange plates to developed fillers, or developed fillers to column flanges (Cases IV-E and V-B), use Table 14-3A to check minimum fill thickness for balanced fill and weld shear strength.

Assume that an E70XX weld with $A = 1/2$ in., $X = 4$ in., and $Y = 6$ in. is to be used at full strength on a 1/4-in.-thick fill (A36). Since this table shows that the minimum fill thickness to develop this 1/2-in. weld is 0.51 in., the 1/4-in. fill will be overstressed. A balanced condition is obtained by multiplying the length $(X + Y)$ by the ratio of the minimum to the actual thickness of fill, thus:

$$(4\text{ in.} + 6\text{ in.})\left(\frac{0.51\text{ in.}}{0.25\text{ in.}}\right) = 20.4\text{ in.}$$

Use $(X + Y) = 20\tfrac{1}{2}$ in.

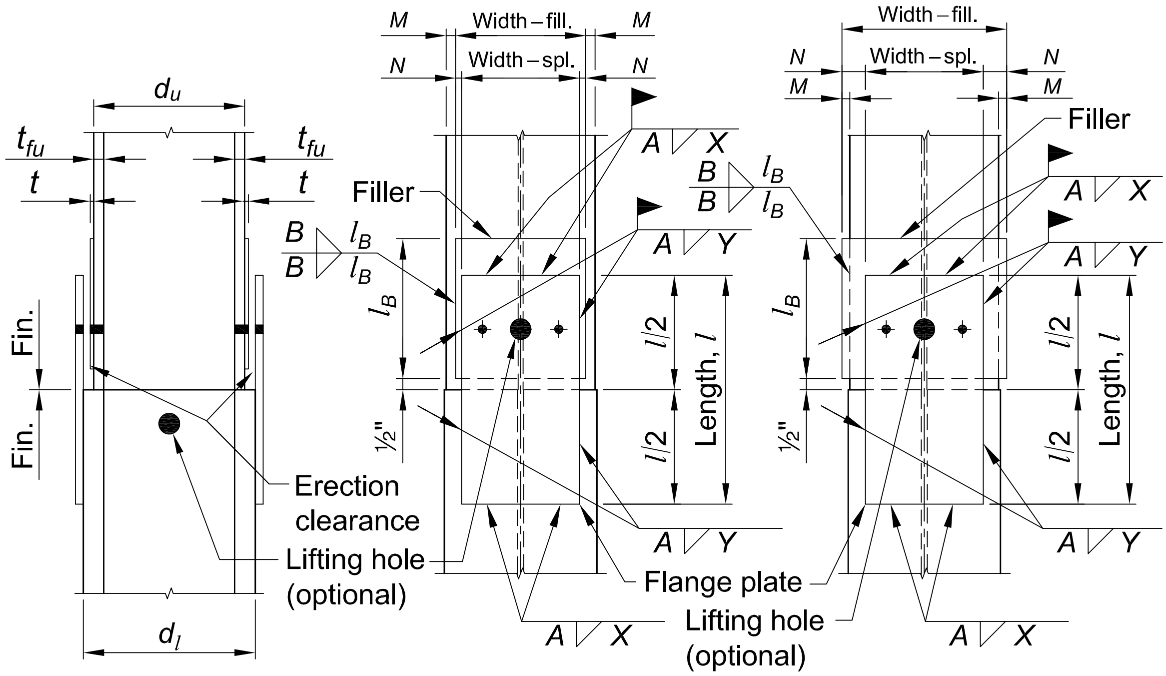
Table 14-3A
Minimum Fill Thickness for Balanced Weld and Plate Shear, in.

Weld A E70XX, in.	F_y, ksi	
	36	50
1/4	0.26	0.19
5/16	0.32	0.23
3/8	0.38	0.28
7/16	0.45	0.33
1/2	0.51	0.37

Placing this additional increment of $(X + Y)$ can be done by making weld lengths, X, continuous across the end of the splice plate and by increasing Y (and therefore the plate length), if required.

Note 2: If fill length, l_B, is excessive, place weld of size B across one or both ends of fill and reduce l_B accordingly, but not less than $(l/2 + 1$ in.). Omit return welds in Cases IV-E and V-B.

Table 14-3 (continued)
Typical Column Splices
Case IV:
All-welded flange-plated column splices between columns with depths d_u and d_l nominally the same



Case IV-D

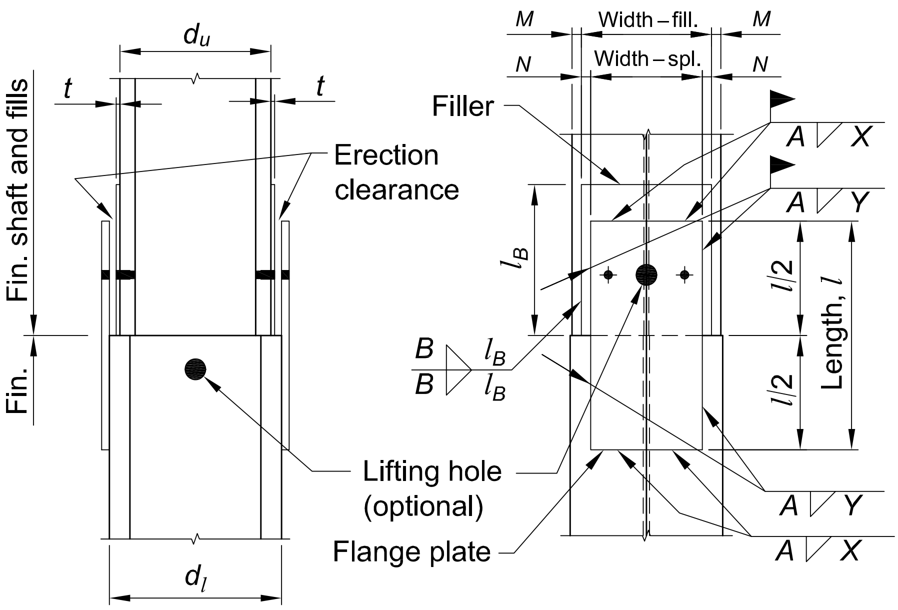
Case IV-E

Table 14-3 (continued)
Typical Column Splices
Case V:
All-welded flange-plated column splices between columns with depth d_u nominally 2 in. less than depth d_l

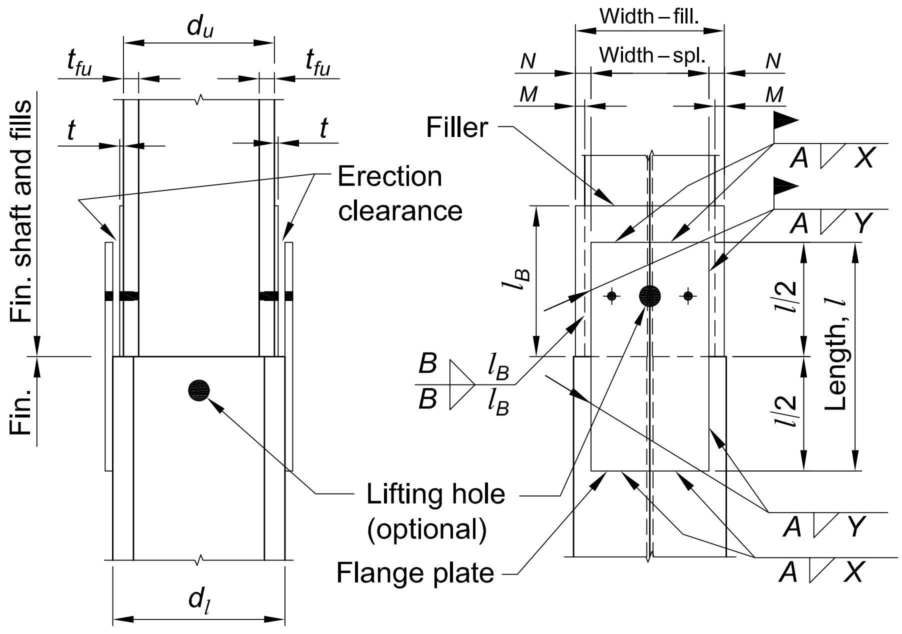
Case V-A: Filler on upper column developed for bearing on lower column. Filler width less than upper column flange width.	Flange plates: Same as Case IV-A, except see Note 1 for Case IV. Fillers (shop welded to upper column): Select thickness as $[(d_l - d_u)/2] - 1/16$ in. Select weld size B from AISC *Specification* Section J2; $5/16$ in. or less preferred. Select weld length, l_B, to develop bearing strength of the filler but not less than $(l/2 + 1\,1/2$ in.). Select filler width greater than flange plate width plus $2N$ but less than the upper column flange width minus $2M$. See Case IV for M and N.
Case V-B: Same as Case V-A except filler width is greater than upper column flange width. Use this case only when M or N in Case V-A are inadequate for weld A, or when additional filler bearing area is required.	Flange plates: Same as Case IV-A, except see Note 1. Fillers (shop welded to upper column): Select thickness as $[(d_l - d_u)/2] - 1/16$ in. Select weld size B from the AISC *Specification* Section J2; $5/16$ in. or less preferred. Select weld length, l_B, to develop bearing strength of the filler but not less than $(l/2 + 1\,1/2$ in.). Select filler width as the larger of the flange plate width plus $2N$ and the upper column flange width plus $2M$, rounded to the next higher $1/4$-in. increment. Select filler length, l_B, subject to Note 3.

Note 3: If fill length, based on l_B, is excessive, place weld of size B across end of fill and reduce l_B by one-half of the additional weld length, but not less than $(l/2 + 1\,1/2$ in.). Omit return welds in Case V-B.

Table 14-3 (continued)
Typical Column Splices
Case V:
All-welded flange-plated column splices between columns with depth d_u nominally 2 in. less than depth d_l



Case V-A



Case V-B

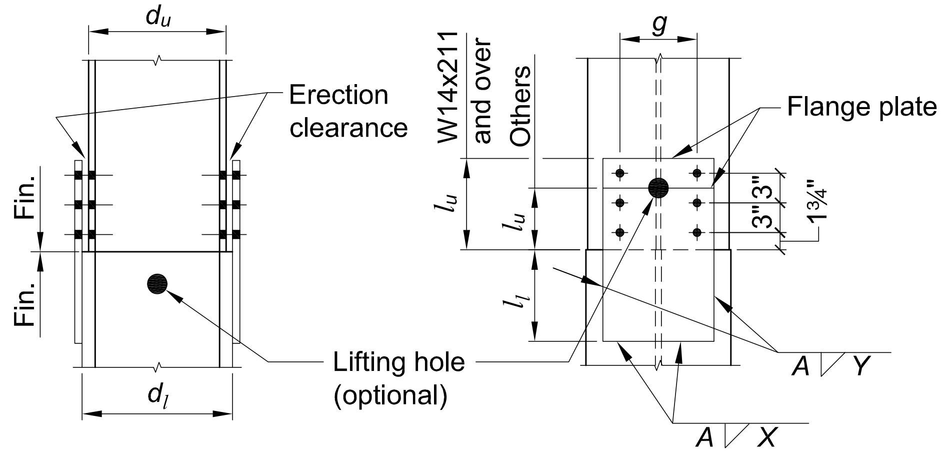
Table 14-3 (continued)
Typical Column Splices
Case VI:
Combination bolted and welded column splices between columns with depths d_u and d_l nominally the same

Column Size	Flange Plate				Bolts		Welds		
	Width	Thick.	Length		No. of Rows	Gage[a] g	Size A	Length	
			l_u	l_l				X	Y
	in.	in.	in.	in.		in.	in.	in.	in.
W14×455 & over	14	5/8	9 1/4	9	3	11 1/2	1/2	5	7
×311 to 426	12	5/8	9 1/4	8	3	9 1/2	1/2	4	6
×211 to 283	12	1/2	9 1/4	8	3	9 1/2	3/8	4	6
×90 to 193	12	3/8	6 1/4	8	2	9 1/2	5/16	4	6
×61 to 82	8	3/8	6 1/4	8	2	5 1/2	5/16	3	6
×43 to 53	6	5/16	6 1/4	7	2	3 1/2	1/4	2	5
W12×120 to 336	8	1/2	6 1/4	8	2	5 1/2	3/8	3	6
×53 to 106	8	3/8	6 1/4	8	2	5 1/2	5/16	3	6
×40 to 50	6	5/16	6 1/4	7	2	3 1/2	1/4	2	5
W10×49 to 112	8	3/8	6 1/4	8	2	5 1/2	5/16	3	6
×33 to 45	6	5/16	6 1/4	7	2	3 1/2	1/4	2	5
W8×31 to 67	6	3/8	6 1/4	7	2	3 1/2	5/16	2	5
×24 & 28	5	5/16	6 1/4	6	2	3 1/2	1/4	2	4

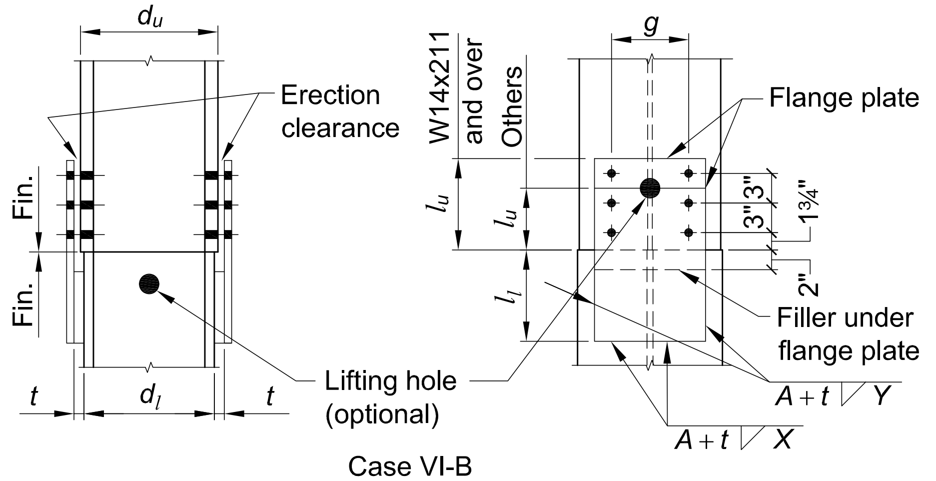
Case VI-A: $d_l = (d_u + 1/4$ in.) to $(d_u + 5/8$ in.)	Flange plates: Select flange plate width, bolts, gage and length l_u for upper column; select flange plate thickness, weld size A, weld lengths X and Y, and length l_l for lower column. Total flange plate length is $l_u + l_l$. Fillers: None. Shims: Furnish sufficient strip shims 2 1/2 in. × 1/8 in. to obtain 0 to 1/16-in. clearance on each side.
Case VI-B: $d_l = (d_u - 1/4$ in.) to $(d_u + 1/8$ in.)	Flange plates: Same as Case VI-A, except use weld size, $A + t$, on lower column. Fillers (shop welded to lower column under flange plate): Select thickness, t, as 1/8 in. for $d_l = d_u$ and $d_l = (d_u + 1/8$ in.) or as 3/16 in. for $d_l = (d_u - 1/8$ in.) and $d_l = (d_u - 1/4$ in.). Select width to match flange plate and length as $l_l - 2$ in. Shims: Same as Case VI-A.
Case VI-C: $d_l = (d_u + 3/4$ in.) and over	Flange plates: Same as Case VI-A. Fillers (shop welded to upper column): Select thickness, t, as $[(d_l - d_u)/2] - 1/8$ in. or 3/16 in., whichever results in 1/8-in. multiples of fill thickness. Select weld size B as minimum size from AISC *Specification* Section J2. Select weld length as $l_u - 1/4$ in. Select filler width as flange plate width and filler length as $l_u - 1/4$ in. Shims: Same as Case VI-A.

[a] Gages shown may be modified if necessary to accommodate fittings elsewhere on the columns.

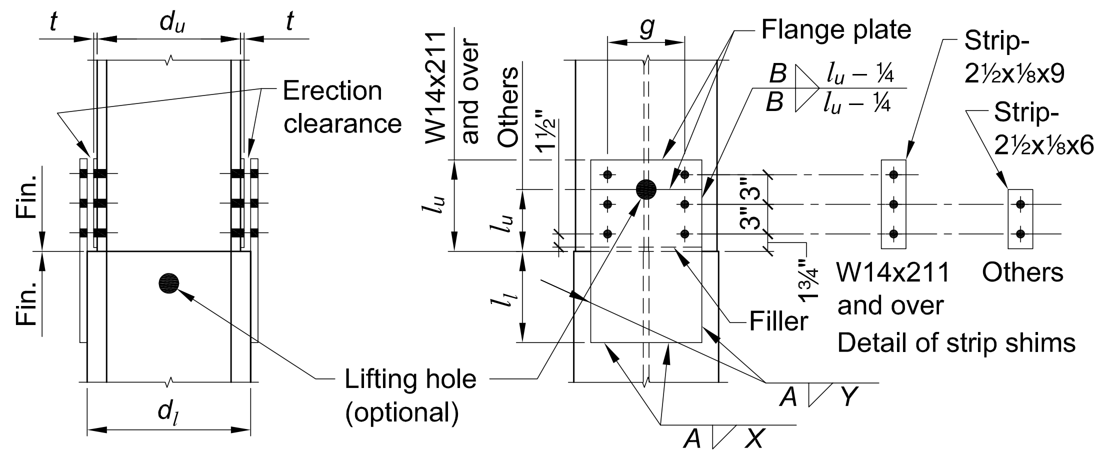
Table 14-3 (continued)
Typical Column Splices
Case VI:
Combination bolted and welded column splices between columns with depths d_u and d_l nominally the same



Case VI-A



Case VI-B



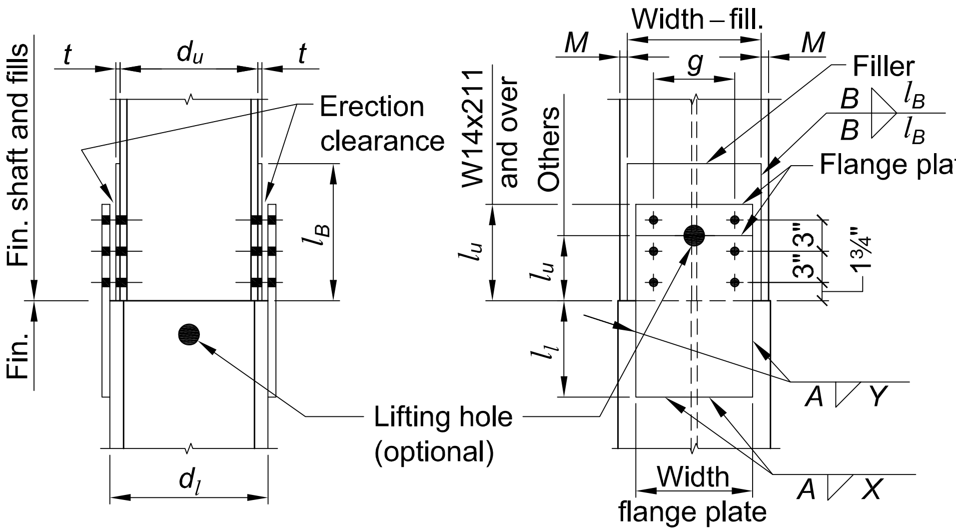
Case VI-C

Table 14-3 (continued)
Typical Column Splices
Case VII:
Combination bolted and welded flange-plated column splices between columns with depth d_u nominally 2 in. less than depth d_l, fillers developed for bearing

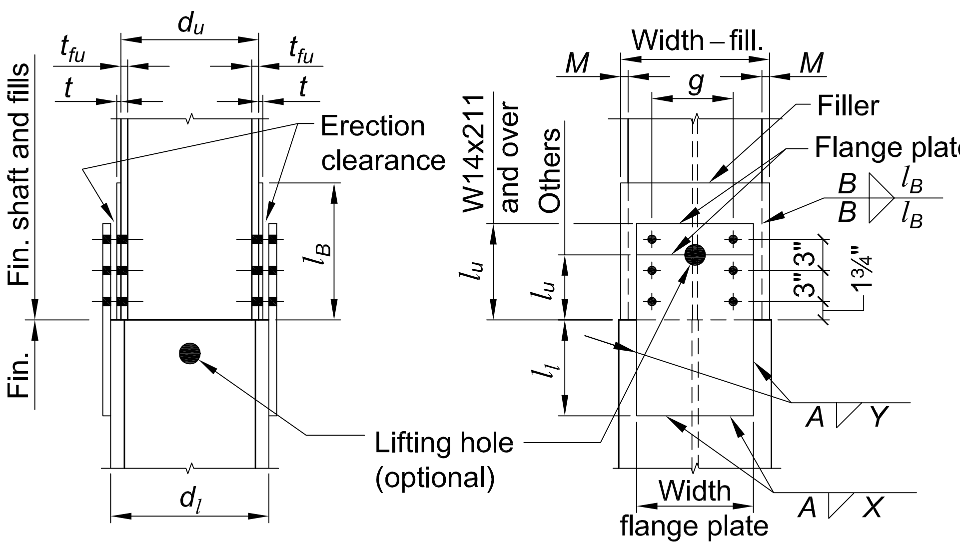
Case VII-A: Filler of width less than upper column flange width.	Flange plates: Same as Case VI-A. Fillers (shop welded to upper column): Select filler thickness, *t*, as $[(d_l - d_u)/2] - 1/8$ in. or $3/16$ in., whichever results in $1/8$-in. multiples of filler thickness. Select weld size *B* from the AISC *Specification* Section J2; $5/16$ in. or less preferred. Select weld length l_B to develop bearing strength of filler. Select filler width not less than flange plate width but not greater than upper column flange width minus 2*M* (see Case IV). Select filler length, l_B, subject to Note 4.
Case VII-B: Filler of width greater than upper column flange width. Use Case VII-B only when fillers must be widened to provide additional bearing area.	Flange plates: Same as Case VI-A. Fillers (shop welded to upper columns): Same as Case VII-A, except select filler width as upper column flange width plus 2*M* (see Case IV) rounded to the next larger $1/2$-in. increment.

Note 4: If fill length based on l_B is excessive, place weld of size *B* across end of fill and reduce l_B by one-half of the additional weld length, but not less than l_u. Omit return welds in Case VII-B.

Table 14-3 (continued)
Typical Column Splices
Case VII:
Combination bolted and welded flange-plated column splices between columns with depth d_u nominally 2 in. less than depth d_l, fillers developed for bearing



Case VII-A



Case VII-B

Table 14-3 (continued)
Typical Column Splices
Case VIII:
Directly welded flange column splices between columns with depths d_u and d_l nominally the same

These types of splices exhibit versatility. The flanges may be partial-joint-penetration groove welded as in Cases VIII-A and VIII-B, or complete-joint-penetration groove welded as in Cases VIII-C, VIII-D, and VIII-E. The webs may be spliced using the channel(s) as shown in Cases VIII-A, VIII-B, VIII-C, and VIII-D, or complete-joint-penetration groove welded as shown in Case VIII-E. The use of a channel or channels at the web splice provides a higher degree of restraint during the erection phase than does a plate or plates. The use of partial-joint-penetration groove flange welds provide greater stability during the erection phase than do complete-joint-penetration groove welds.

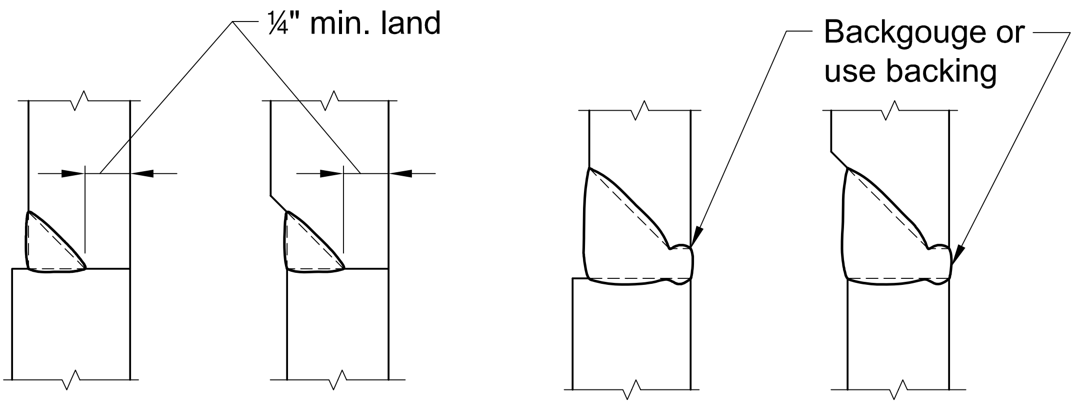
The adequacy of any splice arrangement must be confirmed by the user. This is especially true in regions where high winds are prevalent or when the concentrated weight of the fabricated column is significantly off its center-line. When using partial-joint-penetration groove flange welds, a land width of 1/4 in. or greater should be used. The weld sizes are based on the thickness of the thinner column flange, regardless of whether it is the upper or lower column.

When column flange thicknesses are less than 1/2 in., it may be more efficient to use flange splice plates as shown in previous cases.

See the table below for minimum effective weld sizes for partial-joint-penetration groove welds.

Partial-Joint-Penetration Groove Width	
Thickness of Column Material,[a] T_u in.	**Minimum Effective Weld Size, E in.**
Over 1/2 to 3/4, incl.[b]	1/4
Over 3/4 to 1 1/2, incl.	5/16
Over 1 1/2 to 2 1/4, incl.	3/8
Over 2 1/4 to 6, incl.	1/2
Over 6	5/8

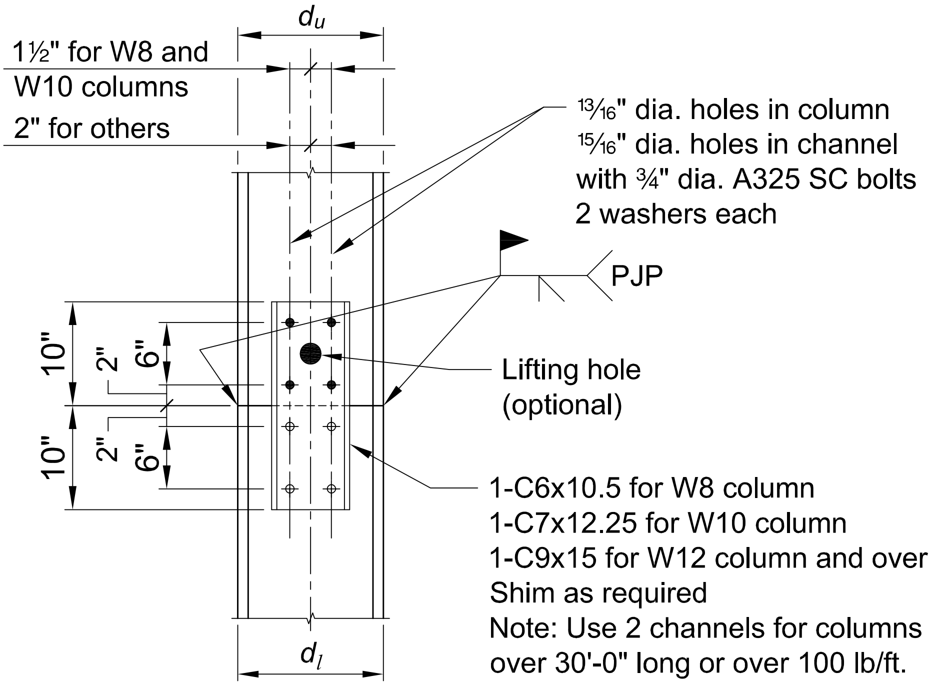
[a] Thickness of thinner part jointed.
[b] For less than 1/2 in., use splice plates.



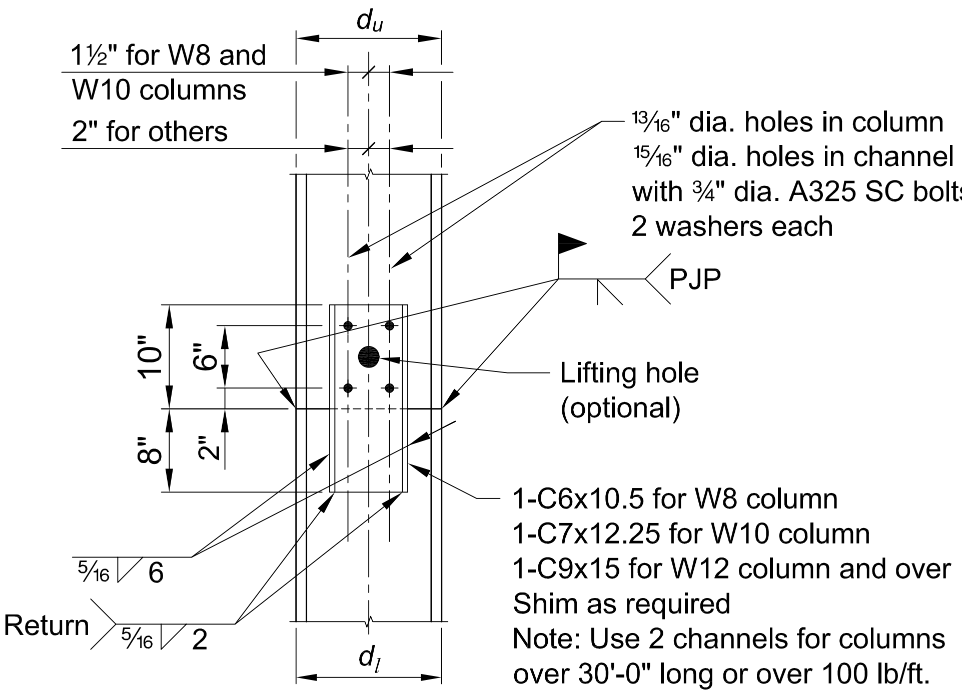
(a) Partial-joint-penetration groove welds

(b) Complete-joint-penetration groove welds

Table 14-3 (continued)
Typical Column Splices
Case VIII:
Directly welded flange column splices between columns with depths d_u and d_l nominally the same



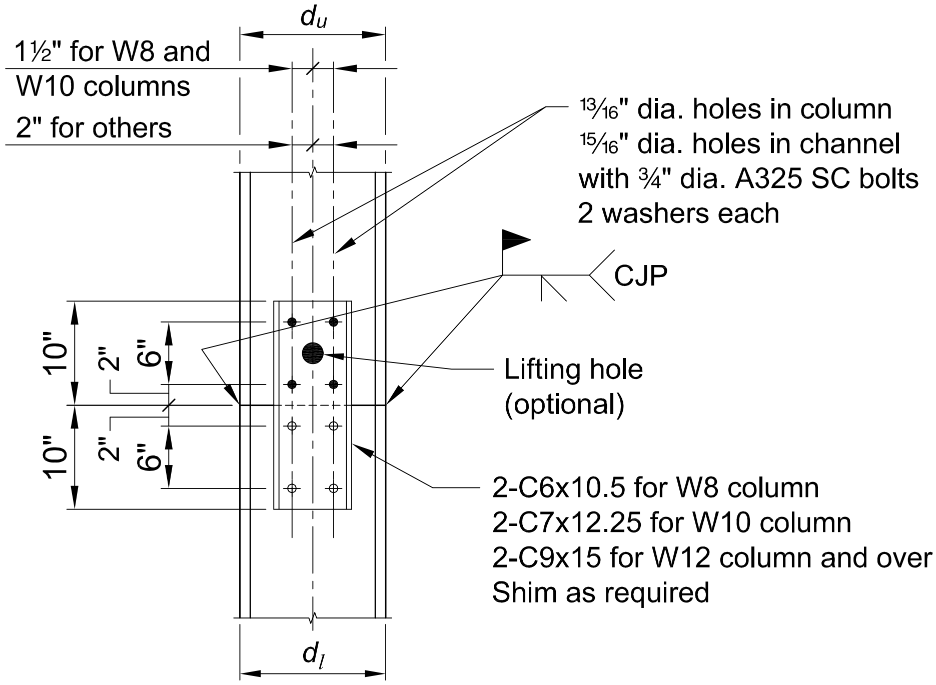
Case VIII-A
All-bolted web splice, partial-joint-penetration groove flange welds



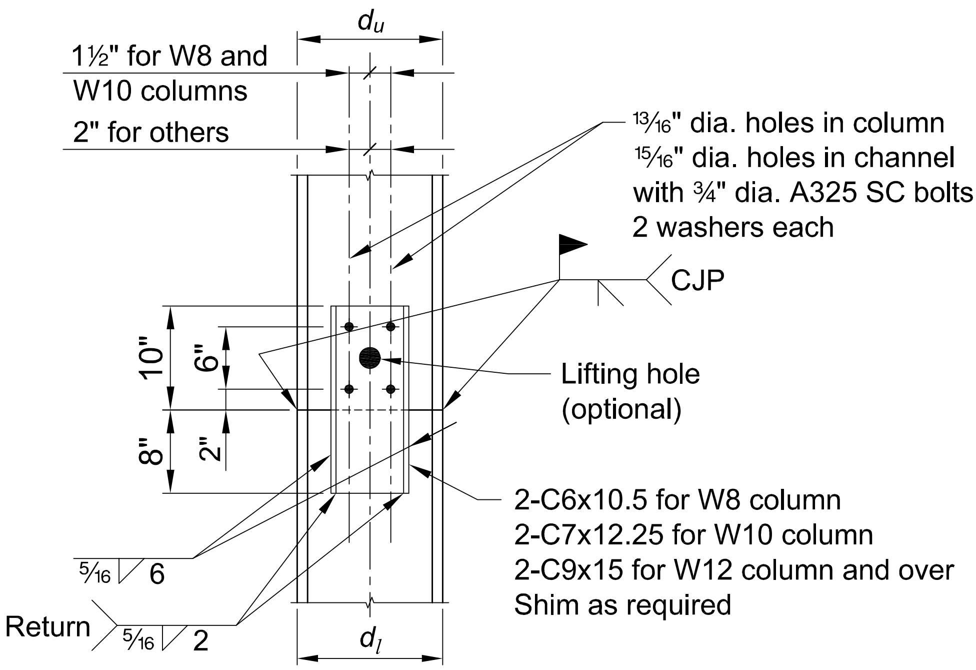
Note: User to verify weld accessibility of channel to lower column shaft, or consider the use of a bolted-bolted connection.

Case VIII-B
Combination bolted and welded web splice, partial-joint-penetration groove flange welds

Table 14-3 (continued)
Typical Column Splices
Case VIII:
Directly welded flange column splices between columns with depths d_u and d_l nominally the same



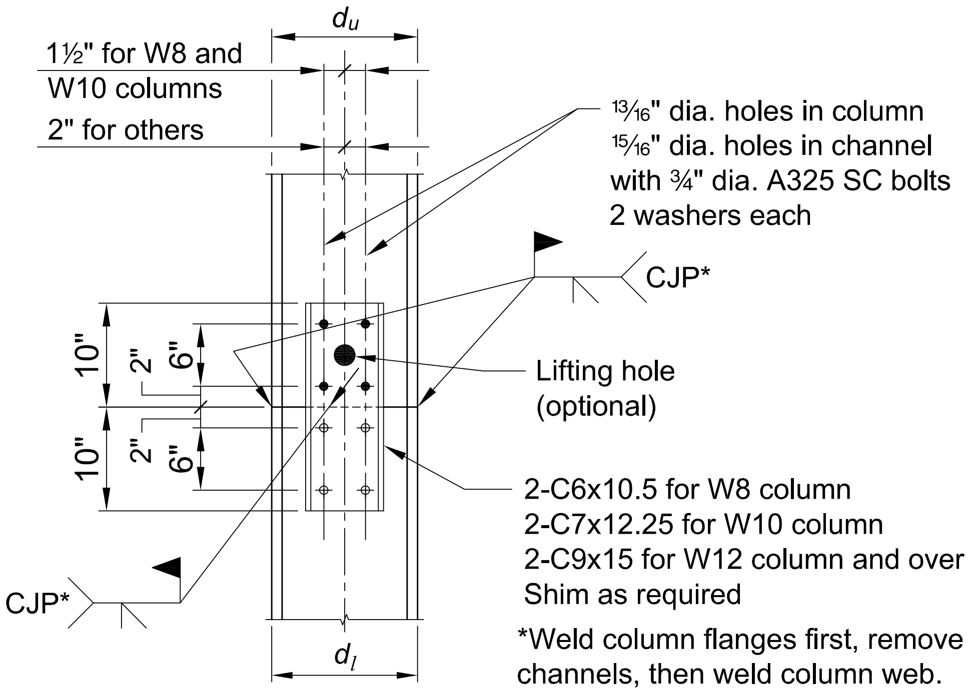
Case VIII-C
All-bolted web splice, complete-joint-penetration groove flange welds



Note: User to verify weld accessibility of channel to lower column shaft, or consider the use of a bolted-bolted connection.

Case VIII-D
Combination bolted and welded web splice, complete-joint-penetration groove flange welds

Table 14-3 (continued)
Typical Column Splices
Case VIII:
Directly welded flange column splices between columns with depths d_u and d_l nominally the same



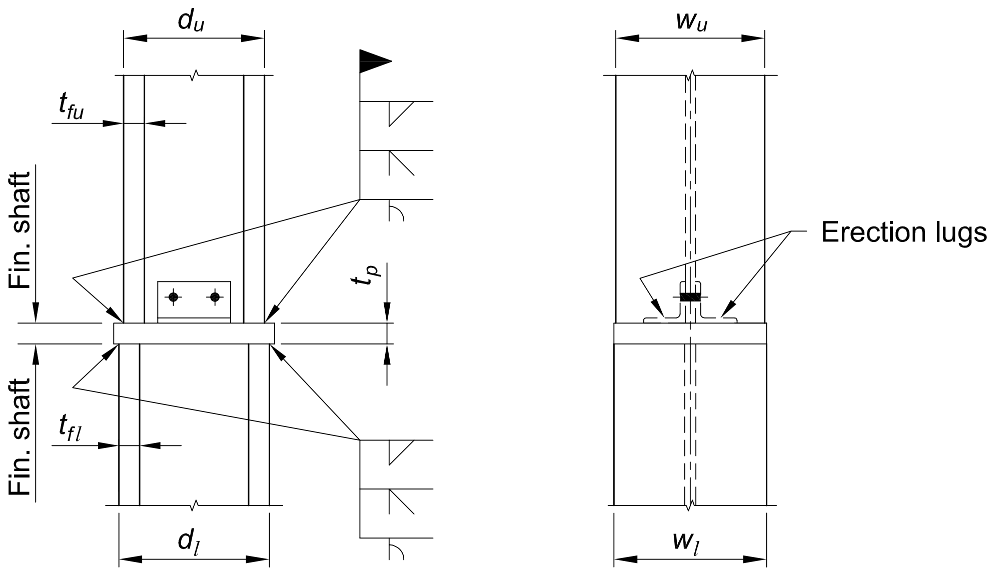
Case VIII-E
Complete-joint-penetration groove flange and web welds

Table 14-3 (continued)
Typical Column Splices
Case IX:
Butt-plated column splices between columns with depth d_u nominally 2 in. less than depth d_l

Butt-plate: Select a butt-plate thickness of 1$^1/_2$ in. for W8 over W10 columns and 2 in. for all other combinations. Select butt-plate width and length not less than w_l and d_l assuming the lower member is the larger column shaft.

Weld: Select weld to upper column based on the thicker of t_{fu} and t_p. Select weld to lower column based on the thicker of t_{fl} and t_p. The edge preparation required by the groove weld is usually performed on the column shafts. However, special cases such as when the butt plate must be field welded to the lower column require special consideration.

Erection: Clip angles, such as those shown for Case IX, help to locate and stabilize the upper column during the erection phase.

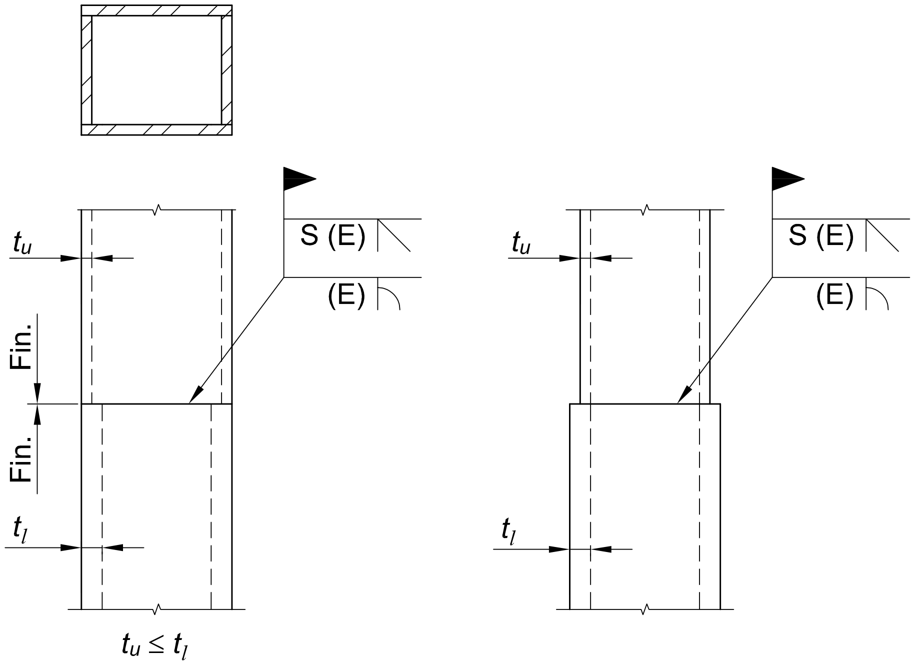


Case IX

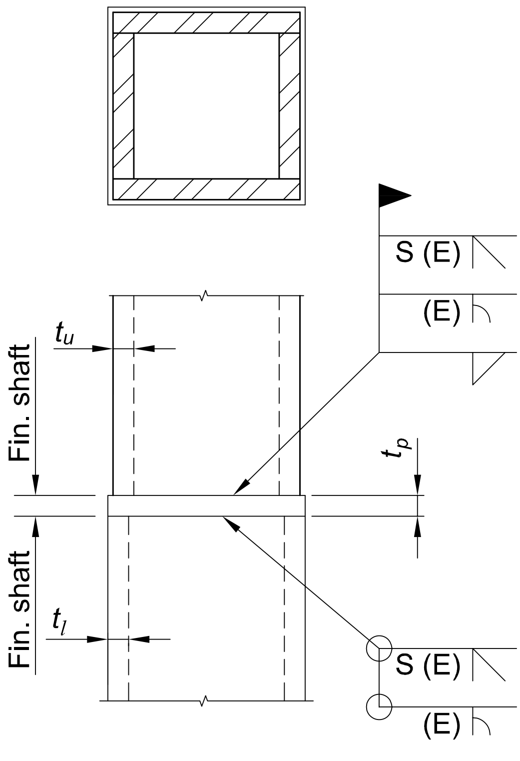
Table 14-3 (continued)
Typical Column Splices
Cases X, XI, XII

Case X: Directly welded splice between HSS and/or box-section columns.	Welds may be either partial- or complete-joint-penetration groove welds. The strength of partial-joint-penetration groove welds is a function of the column wall thickness and appropriate guidelines for minimum land width and effective weld size must be observed. This type of splice usually requires lifting and alignment devices. For lifting devices, see Figure 14-10. For alignment devices, see Figure 14-11.
Case XI: Butt-plated splices between HSS and/or box-section columns.	The butt-plate thickness is selected based on the AISC *Specification*. Welds may be either partial- or complete-joint-penetration groove welds, or, if adequate space is provided, fillet welds may be used. Weld strength is based on the thickness of connected material. See comments related to Case X regarding lifting and alignment devices.
Case XII: Butt-plated column splices between W-shape columns and HSS or box-section columns.	See comments related to Case XI.

Table 14-3 (continued)
Typical Column Splices
Cases X, XI, XII

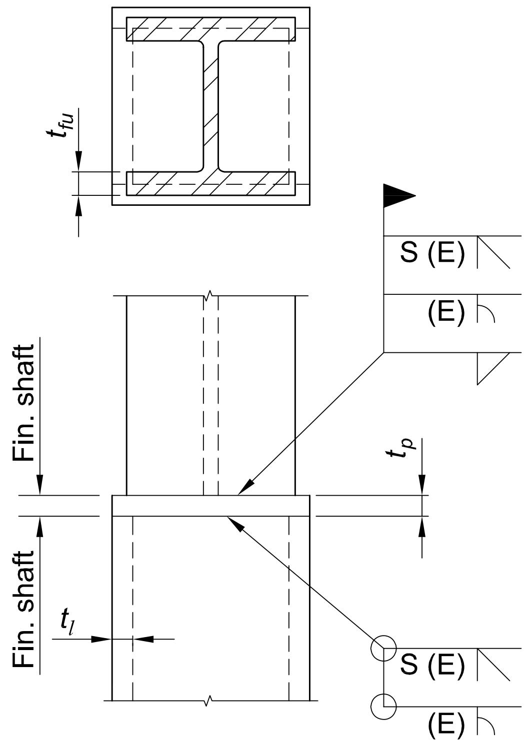


CASE X



CASE XI

Table 14-3 (continued)
Typical Column Splices
Cases X, XI, XII



CASE XII

PART 15

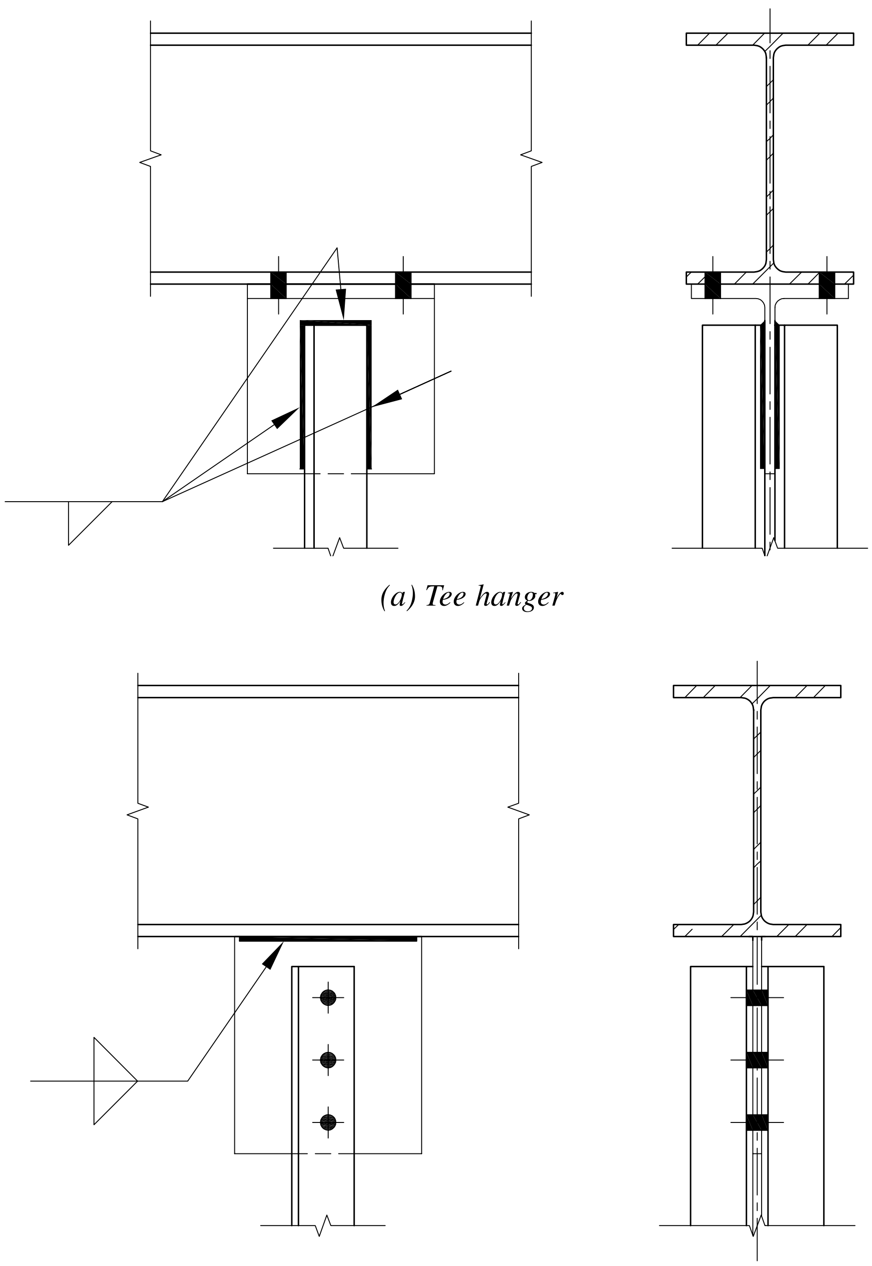
DESIGN OF HANGER CONNECTIONS, BRACKET PLATES, AND CRANE-RAIL CONNECTIONS

SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of hanger connections, bracket plates, and crane-rail connections. For the design of similar connections for rectangular and round HSS, see AISC *Specification* Chapter K.

HANGER CONNECTIONS

Hanger connections, illustrated in Figure 15-1, are usually made with a plate, tee, angle, or pair of angles. The available strength of a hanger connection is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). In all cases, the available strength, ϕR_n or R_n/Ω, must exceed the required strength, R_u or R_a.



(a) Tee hanger

(b) Plate hanger

Fig. 15-1. Typical hanger connections.

BRACKET PLATES

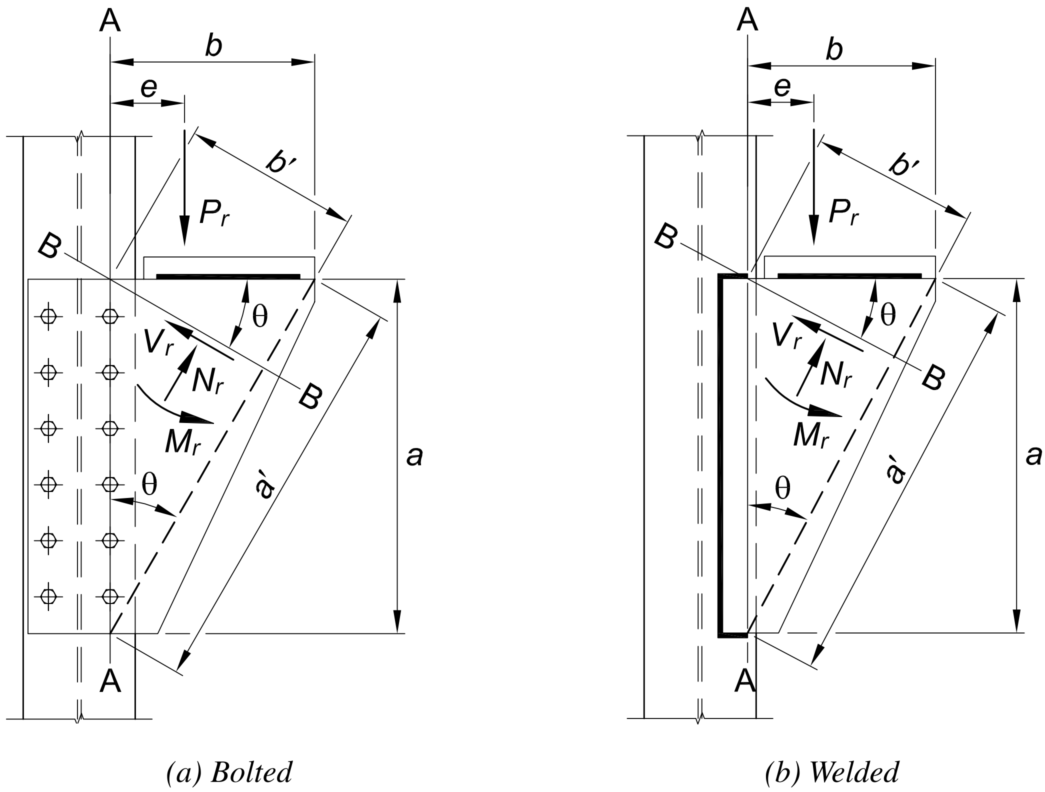
A bracket plate, illustrated in Figure 15-2, acts as a cantilevered beam. The available strength of a bracket plate is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). Additionally the following checks must be considered: flexural yielding at Sections A-A in Figure 15-2; flexural rupture through Sections A-A in Figure 15-2; and shear yielding, local yielding and local buckling through Sections B-B in Figure 15-2 (Muir and Thornton, 2004). The following procedures are for a single bracket plate with the applied load P_r, where P_r is the required strength using LRFD load combinations, P_u, or the required strength using ASD load combinations, P_a. In all cases, the available strength must equal or exceed the required strength. The seat plate shown in Figure 15-2 should be attached to the bracket plate(s) with a minimum continuous single-sided fillet weld per AISC *Specification* Table J2.4.

The required flexural strength at Sections A-A in Figure 15-2 is

LRFD	ASD
$M_u = P_u e$ (15-1a)	$M_a = P_a e$ (15-1b)

where

e = distance shown in Figure 15-2, in.



$N_r = P_r \cos\theta$

$V_r = P_r \sin\theta$

$M_r = P_r e - N_r\,(b'/2)$

Fig. 15-2. Bracket-plate connections.

For flexural yielding, the available strength, ϕM_n or M_n/Ω, of the bracket plate is

$$M_n = F_y Z \tag{15-2}$$

$$\phi = 0.90 \qquad \Omega = 1.67$$

where
Z = gross plastic section modulus of the bracket plate at Sections A-A in Figure 15-2, in.^{3}

For flexural rupture, the available strength, ϕM_n or M_n/Ω, of the bracket plate is

$$M_n = F_u Z_{net} \tag{15-3}$$

$$\phi = 0.75 \qquad \Omega = 2.00$$

where
Z_{net} = net plastic section modulus of the bracket plate at Sections A-A in Figure 15-2, in.^{3} See Table 15-3 for the determination of Z_{net} for brackets with standard holes. General equations for determination of Z_{net} follow (Mohr and Murray, 2008).

For an odd number of bolt rows

$$Z_{net} = \frac{1}{4}t(s - d'_h)(n^2 s + d'_h) \tag{15-4}$$

For an even number of bolt rows

$$Z_{net} = \frac{1}{4}t(s - d'_h)n^2 s \tag{15-5}$$

where
d'_h = hole diameter + $^1/_{16}$, in.
n = number of bolt rows
s = vertical bolt row spacing, in.

In both cases, the vertical edge distances are assumed to be $s/2$ with plate depth of $a = ns$.

The required shear strength at Sections B-B in Figure 15-2 is

LRFD	ASD
$V_u = P_u \sin\theta$ (15-6a)	$V_a = P_a \sin\theta$ (15-6b)

For shear yielding, the available strength, ϕV_n or V_n/Ω, of the bracket plate is

$$V_n = 0.6 F_y t b' \tag{15-7}$$

$$\phi = 1.00 \qquad \Omega = 1.50$$

where
a = depth of bracket plate, in.
b' = $a \sin\theta$, in.
t = thickness of bracket plate, in.
θ = angle shown in Figure 15-2, degrees

The required normal and flexural strength at Sections B-B in Figure 15-2 is

LRFD	ASD
$M_u = P_u e - N_u\left(\frac{b'}{2}\right)$ (15-8a)	$M_a = P_a e - N_a\left(\frac{b'}{2}\right)$ (15-8b)
$N_u = P_u \cos\theta$ (15-9a)	$N_a = P_a \cos\theta$ (15-9b)

For interaction of normal and flexural strengths, the following interaction equation must be satisfied:

$$\frac{N_r}{N_c} + \frac{M_r}{M_c} \le 1.0 \tag{15-10}$$

The nominal normal strength of the bracket plate for the limit states of local yielding and local buckling is

$$N_n = F_{cr} t b', \text{ kips} \tag{15-11}$$

and the nominal flexural strength of the bracket plate for the limit states of local yielding and local buckling is

$$M_n = \frac{F_{cr} t b'^2}{4}, \text{ kip-in.} \tag{15-12}$$

For design by LRFD

$M_c = \phi M_n$

$M_r = M_u$

$N_c = \phi N_n$

$N_r = N_u$

$\phi \;= 0.90$

For design by ASD

$M_c = \dfrac{M_n}{\Omega}$

$M_r = M_a$

$N_c = \dfrac{N_n}{\Omega}$

$N_r = N_a$

$\Omega \;= 1.67$

For the limit state of local yielding of the bracket plate

$$F_{cr} = F_y \tag{15-13}$$

For the limit state of local buckling of the bracket plate

$$F_{cr} = QF_y \tag{15-14}$$

When $\lambda \le 0.70$, the limit state of local buckling need not be considered (that is, $Q = 1$).
When $0.70 < \lambda \le 1.41$

$$Q = 1.34 - 0.486\lambda \tag{15-15}$$

When $1.41 < \lambda$

$$Q = \frac{1.30}{\lambda^2} \tag{15-16}$$

where

$$a' = \frac{a}{\cos\theta} \tag{15-17}$$

= length of free edge, in.

$$\lambda = \frac{\left(\dfrac{b'}{t}\right)\sqrt{F_y}}{5\sqrt{475 + 1{,}120\left(\dfrac{b'}{a'}\right)^2}} \tag{15-18}$$

CRANE-RAIL CONNECTIONS

Bolted Splices

It is desirable to use properly installed and maintained bolted splice bars in crane-rail connections rather than welded splice bars, which are frequently subject to failure in service.

Standard rail drilling and joint-bar punching, as furnished by manufacturers of light standard rails for track work, include round holes in rail ends and slotted holes in joint bars to receive standard oval-neck track bolts. Holes in rails are oversized and punching in joint bars is spaced to allow $^1/_{16}$-in. to $^1/_8$-in. clearance between rail ends (see manufacturers' catalogs for spacing and dimensions of holes and slots). Although this construction is satisfactory for track and light crane service, its use in general crane service may lead to high maintenance and joint failure. Welded splices are therefore preferable.

For best service in bolted splices, it is recommended that tight joints be required for all rails for crane service. This will require rail ends to be finished, and the special rail drilling and joint-bar punching tabulated in Table 15-1 and shown in Figure 15-3. Special rail drilling is accepted by some mills, or rails may be ordered blank for shop drilling. End finishing of standard rails can be done at the mill. However, light rails often must be end-finished in the shop or ground at the site prior to erection. In the crane rail range from 104 to 175 lb per yard, rails and joint bars are manufactured to obtain a tight fit and no further special end finishing, drilling or punching is required. Because of cumulative tolerance variations in holes, bolt diameters and rail ends, a slight gap may sometimes occur. It may sometimes be necessary to ream holes through joined bar and rail to permit entry of bolts.

Joint bars for crane service are provided in various sections to match the rails. Joint bars for light and standard rails can be purchased blank for special shop punching to obtain tight joints. See manufacturer data for dimensions, material specifications, and the identification necessary to match the crane-rail section.

Joint-bar bolts, as distinguished from oval-neck track bolts, have straight shanks to the head and are manufactured to ASTM A449 specifications. Nuts are manufactured to ASTM A563 Grade B specifications. Alternatively, ASTM F3125 Grade A325 bolts and compatible ASTM A563 nuts can be used. Bolt assembly includes an alloy steel spring washer, furnished to American Railway Engineering and Maintenance of Way Association (AREMA) specifications. After installation, bolts should be retightened within 30 days and every three months thereafter.

Table 15-1
Crane Rail Splices

Rail						Joint Bar						Bolt				Washer		Wt. 2 Bars Bolts, Nuts, Washers	
	Drilling					Punching													
Wt. per Yard	*g*	Hole Dia.	*A*	*B*	*C*	Hole Dia.	*D*	*B*	*C*	*L*	*G*	Dia.	Grip	*l*	*H*	In-side Dia.	Thick-ness and Width	With Ftg.	W/O Ftg.
lb	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.	lb	lb
40	$1\frac{71}{128}$	$\frac{13}{16}$*	$2\frac{1}{2}$	5	–	$\frac{13}{16}$*	$4\frac{15}{16}$*	5	–	20	$2\frac{3}{16}$	$\frac{3}{4}$	$1\frac{15}{16}$	$3\frac{1}{2}$	$2\frac{1}{2}$	$\frac{13}{16}$	$\frac{7}{16} \times \frac{3}{8}$	20.0	16.5
60	$1\frac{115}{128}$	$\frac{13}{16}$*	$2\frac{1}{2}$	5	–	$\frac{13}{16}$*	$4\frac{15}{16}$*	5	–	24	$2\frac{11}{16}$	$\frac{3}{4}$	$2\frac{19}{32}$	4	$2\frac{11}{16}$	$\frac{13}{16}$	$\frac{7}{16} \times \frac{3}{8}$	36.5	29.6
85	$2\frac{17}{64}$	$\frac{15}{16}$*	$2\frac{1}{2}$	5	–	$\frac{15}{16}$*	$4\frac{15}{16}$*	5	–	24	$3\frac{11}{32}$	$\frac{7}{8}$	$3\frac{5}{32}$	$4\frac{3}{4}$	$3\frac{3}{16}$	$\frac{15}{16}$	$\frac{7}{16} \times \frac{3}{8}$	56.6	45.3
104	$2\frac{7}{16}$	$1\frac{1}{16}$	4	5	6	$1\frac{1}{16}$	$7\frac{15}{16}$	5	6	34	$3\frac{1}{2}$	1	$3\frac{1}{2}$	$5\frac{1}{4}$	$3\frac{1}{2}$	$1\frac{1}{16}$	$\frac{7}{16} \times \frac{1}{2}$	73.5	55.4
135	$2\frac{15}{32}$	$1\frac{3}{16}$	4	5	6	$1\frac{3}{16}$	$7\frac{15}{16}$	5	6	34	–	$1\frac{1}{8}$	$3\frac{5}{8}$	$5\frac{1}{2}$	$3\frac{11}{16}$	$1\frac{3}{16}$	$\frac{7}{16} \times \frac{1}{2}$	–	75.3
171	$2\frac{5}{8}$	$1\frac{3}{16}$	4	5	6	$1\frac{3}{16}$	$7\frac{15}{16}$	5	6	34	–	$1\frac{1}{8}$	$4\frac{7}{16}$	$6\frac{1}{4}$	$4\frac{1}{16}$	$1\frac{3}{16}$	$\frac{7}{16} \times \frac{1}{2}$	–	90.8
175	$2\frac{21}{32}$	$1\frac{3}{16}$	4	5	6	$1\frac{3}{16}$	$7\frac{15}{16}$	5	6	34	–	$1\frac{1}{8}$	$4\frac{1}{8}$	$6\frac{1}{4}$	$3\frac{15}{16}$	$1\frac{3}{16}$	$\frac{7}{16} \times \frac{1}{2}$	–	87.7

*Special rail drilling and joint bar punching.
Ftg. = fitting

Welded Splices

When welded splices are specified, consult the manufacturer for recommended rail-end preparation, welding procedure, and method of ordering. Although the joint continuity made possible by this method of splicing is desirable, the careful control required in all stages of the welding operation may be difficult to meet during crane-rail installation. Rails should not be attached to structural supports by welding. Rails with holes for joint bar bolts should not be used in making welded splices.

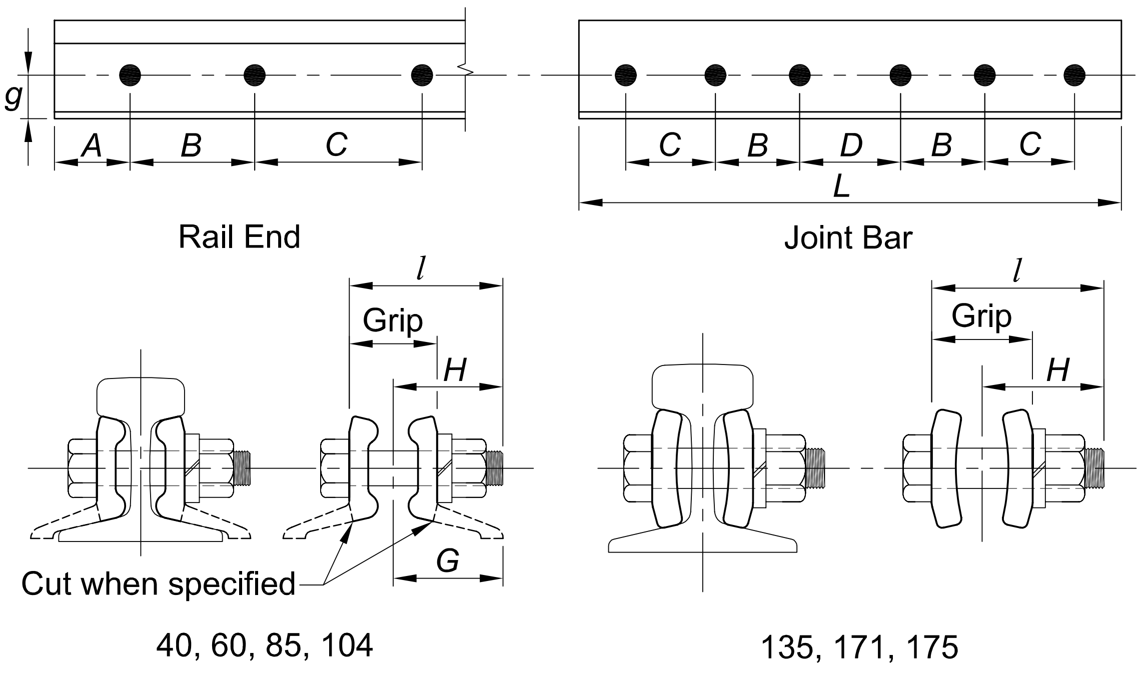


Fig. 15-3. Special rail drilling and joint-bar punching.

Hook Bolt Fastenings

Hook bolts (Figure 15-4) are used primarily with light rails when attached to beams that are too narrow for clamps. Rail adjustment to $\pm^1/_2$ in. is inherent in the threaded shank. Hook bolts are paired alternately 3 to 4 in. apart, spaced at about 24 in. on center. The special rail drilling required must be done in the fabricator's shop. Hook bolts are not recommended for use with heavy-duty cycle cranes [Crane Manufacturers Association of America (CMAA) Classes D, E and F]. It is generally recommended that hook bolts should not be used in runway systems that are longer than 500 ft because the bolts do not allow for longitudinal movement of the rail.

Rail Clip Fastenings

Rail clips are forged or cast devices that are shaped to match specific rail profiles. They are usually bolted to the runway girder flange with one bolt or are sometimes welded. Rail clips have been used satisfactorily with all classes of cranes. However, one drawback is that when a single bolt is used, the clip can rotate in response to rail longitudinal movement. This clip rotation can cause cam action that might force the rail out of alignment. Because of this limitation, rail clips should only be used in crane systems subject to infrequent use, and for runways less than 500 ft in length.

Rail Clamp Fastenings

Rail clamps are a common method of attachment for heavy-duty cycle cranes. Rail clamps are detailed to provide two types: tight and floating (see Figure 15-5). Each clamp consists of two plates: an upper clamp plate and a lower filler plate. Dimensions shown are suggested. See manufacturers' catalogs for recommended gages, bolt sizes and detail dimensions not shown.

The lower plate is flat and nominally matches the height of the toe of the rail flange. The upper plate covers the lower plate and extends over the top of the lower rail flange. In the tight clamp, the upper plate is detailed to fit tightly to the lower tail flange top, thus "clamping" it tightly in place when the fasteners are tightened. The tight clamp is illustrated with the filler plates fitted tightly against the rail flange toe. This tight fit-up is rarely achieved in practice and is not considered to be necessary to achieve a tight type clamp. In the floating type clamp, the pieces are detailed to provide a clearance both alongside the rail flange toe and below the upper plate. The floating type does not, in

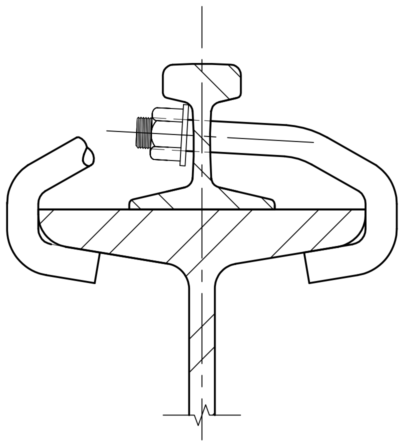


Fig. 15-4. Hook bolts.

reality, clamp the rail but merely holds the rail within the limits of the clamp clearances. High-strength bolts are recommended for both clamp types. Both types should be spaced 3 ft or less apart.

Patented Rail Clip Fastenings

Each manufacturer's literature presents in detail the desirable aspects of the various designs. In general, patented rail clips are easy to install due to their range of adjustment and provide both limitation of lateral movement and allowance for longitudinal movement. Patented rail clips should be considered as a viable alternative to conventional hook bolts, clips or clamps. Because of their desirable characteristics, patented rail clips can be used without restriction except as limited by the specific manufacturer's recommendations. Installations using patented rail clips sometimes incorporate pads beneath the rail. When this is done, the lateral float of the rail should be limited as in the case of the tight rail clamps.

DESIGN TABLE DISCUSSION

Table 15-2. Preliminary Hanger Connection Selection Table

Values are given for the available tensile strength per in. of fitting length in bending of a tee fitting flange or angle leg with F_u = 58 ksi and F_u = 65 ksi. The bending strength is calculated in terms of F_u, which provides good correlation with available test data (Thornton, 1992; Swanson, 2002). Table 15-2 can be used to select a trial fitting once the number and

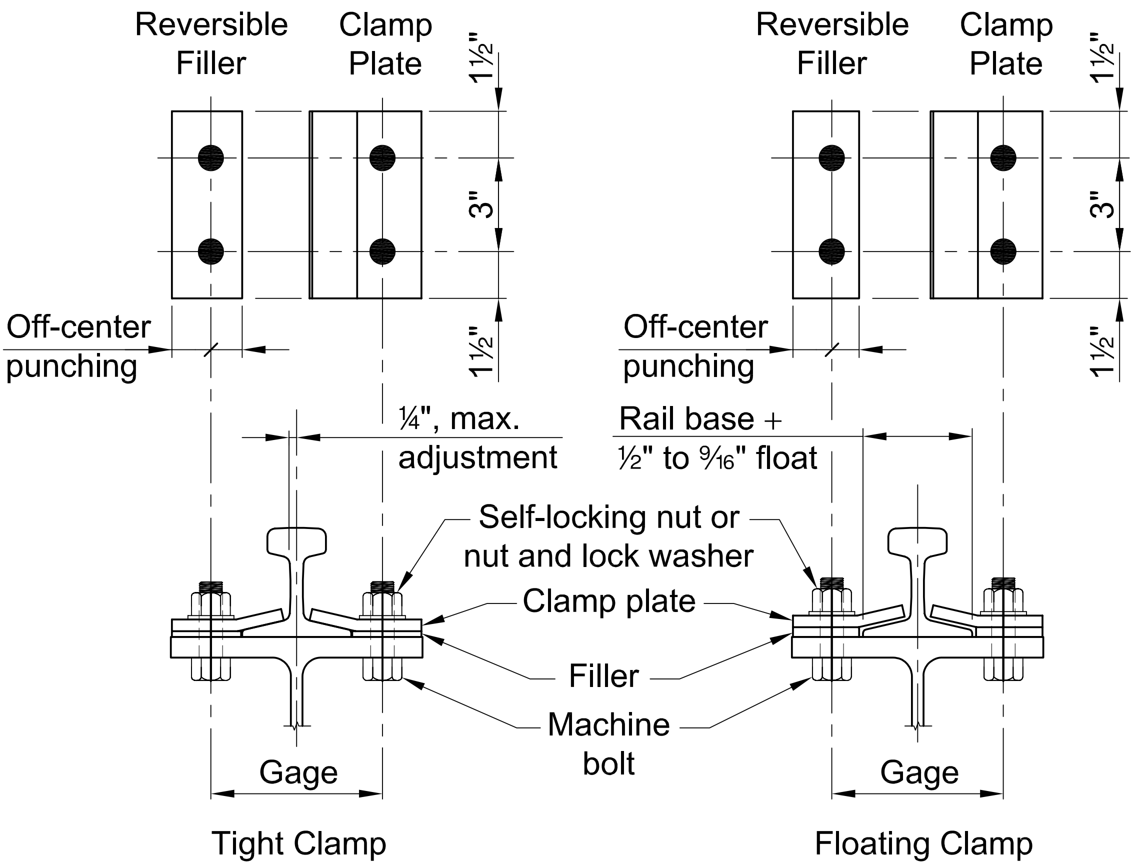


Fig. 15-5. Rail clamps.

size of bolts required is known. The number of bolts required must be selected such that the available tensile strength of one bolt, ϕr_n or r_n/Ω, exceeds the required tensile force per bolt, r_{ut} or r_{at}.

In this table, it is assumed that equal moments exist at the face of the tee stem or angle leg and at the bolt line. The available flexural strength of the tee flange or angle leg, $\phi_b M_n$ or M_n/Ω_b, is determined with

$$M_n = M_p = F_u Z \tag{15-19}$$

$$\phi_b = 0.90 \qquad \Omega_b = 1.67$$

In the above equation, the plastic section modulus, Z, per unit length of the angle or tee flange is

$$Z = \frac{t^2}{4} \tag{15-20}$$

where t is the thickness of the angle or tee flange, in. Thus, for a unit length of the angle or tee flange the available flexural strength, $\phi_b M_n$ or M_n/Ω_b, is determined with

$$M_n = \frac{F_u t^2}{4} \tag{15-21}$$

$$\phi_b = 0.90 \qquad \Omega_b = 1.67$$

The tensile force on the fitting per bolt row, $2r_{ut}$ or $2r_{at}$, must be less than the appropriate (LRFD or ASD) value shown in Table 15-2 times the tributary length per pair of bolts, p (length perpendicular to the elevation shown in Table 15-2).

Table 15-3. Net Plastic Section Modulus, Z_{net}

Values of the net plastic section modulus, Z_{net}, are given in Table 15-3 for brackets with standard holes and numbers of fasteners spaced 3 in. on center, the usual spacing for these connections. The values are determined using Equations 15-4 and 15-5.

Forged Steel Structural Hardware

Table 15-4. Dimensions and Weights of Clevises

Dimensions, weights and available strengths of clevises are listed in Table 15-4.

Table 15-5. Clevis Numbers Compatible with Various Rods and Pins

Compatibility of clevises with various rods and pins is given in Table 15-5.

Table 15-6. Dimensions and Weights of Turnbuckles

Dimensions, weights and available strengths of turnbuckles are listed in Table 15-6.

PART 15 REFERENCES

Mohr, B.A. and Murray, T.M. (2008), "Bending Strength of Steel Bracket and Splice Plates," *Engineering Journal*, AISC, Vol. 45, No. 2, pp. 97–106.

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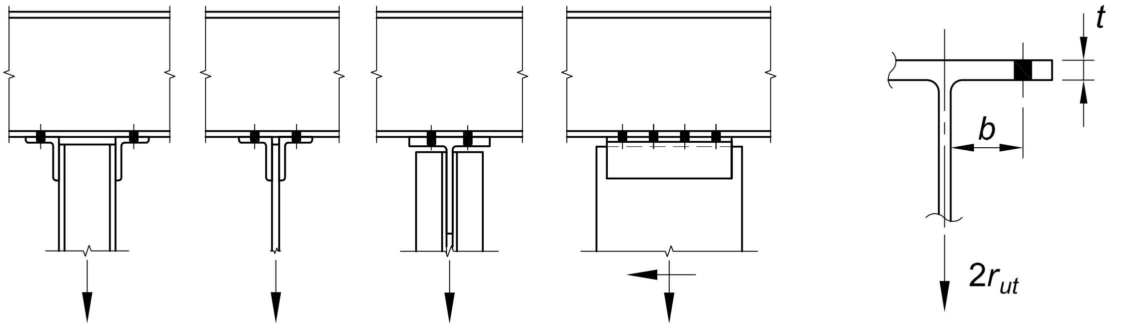
Table 15-2a
Preliminary Hanger Connection Selection Table
Available tensile strength, kips per linear in., limited by bending of the flange

F_u = 58 ksi

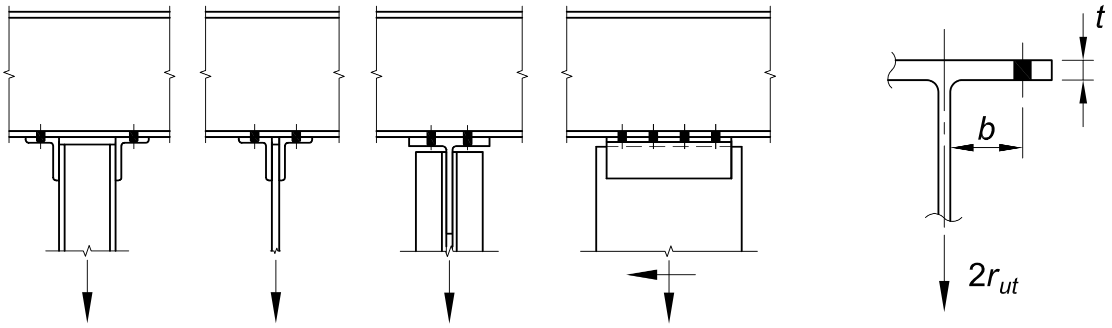


t, in.	*b*, in. 1 ASD	1 LRFD	1 1/4 ASD	1 1/4 LRFD	1 1/2 ASD	1 1/2 LRFD	1 3/4 ASD	1 3/4 LRFD	2 ASD	2 LRFD
5/16	3.39	5.10	2.71	4.08	2.26	3.40	1.94	2.91	1.70	2.55
3/8	4.88	7.34	3.91	5.87	3.26	4.89	2.79	4.19	2.44	3.67
7/16	6.65	9.99	5.32	7.99	4.43	6.66	3.80	5.71	3.32	5.00
1/2	8.68	13.1	6.95	10.4	5.79	8.70	4.96	7.46	4.34	6.53
9/16	11.0	16.5	8.79	13.2	7.33	11.0	6.28	9.44	5.49	8.26
5/8	13.6	20.4	10.9	16.3	9.04	13.6	7.75	11.7	6.78	10.2
11/16	16.4	24.7	13.1	19.7	10.9	16.4	9.38	14.1	8.21	12.3
3/4	19.5	29.4	15.6	23.5	13.0	19.6	11.2	16.8	9.77	14.7
13/16	22.9	34.5	18.3	27.6	15.3	23.0	13.1	19.7	11.5	17.2
7/8	26.6	40.0	21.3	32.0	17.7	26.6	15.2	22.8	13.3	20.0
15/16	30.5	45.9	24.4	36.7	20.3	30.6	17.4	26.2	15.3	22.9
1	34.7	52.2	27.8	41.8	23.2	34.8	19.8	29.8	17.4	26.1
1 1/16	39.2	58.9	31.4	47.1	26.1	39.3	22.4	33.7	19.6	29.5
1 1/8	44.0	66.1	35.2	52.9	29.3	44.0	25.1	37.8	22.0	33.0
1 3/16	49.0	73.6	39.2	58.9	32.6	49.1	28.0	42.1	24.5	36.8
1 1/4	54.3	81.6	43.4	65.3	36.2	54.4	31.0	46.6	27.1	40.8

t, in.	2 1/4 ASD	2 1/4 LRFD	2 1/2 ASD	2 1/2 LRFD	2 3/4 ASD	2 3/4 LRFD	3 ASD	3 LRFD	3 1/4 ASD	3 1/4 LRFD
5/16	1.51	2.27	1.36	2.04	1.23	1.85	1.13	1.70	1.04	1.57
3/8	2.17	3.26	1.95	2.94	1.78	2.67	1.63	2.45	1.50	2.26
7/16	2.95	4.44	2.66	4.00	2.42	3.63	2.22	3.33	2.05	3.07
1/2	3.86	5.80	3.47	5.22	3.16	4.75	2.89	4.35	2.67	4.02
9/16	4.88	7.34	4.40	6.61	4.00	6.01	3.66	5.51	3.38	5.08
5/8	6.03	9.06	5.43	8.16	4.93	7.41	4.52	6.80	4.17	6.27
11/16	7.30	11.0	6.57	9.87	5.97	8.97	5.47	8.22	5.05	7.59
3/4	8.68	13.1	7.81	11.7	7.10	10.7	6.51	9.79	6.01	9.03
13/16	10.2	15.3	9.17	13.8	8.34	12.5	7.64	11.5	7.05	10.6
7/8	11.8	17.8	10.6	16.0	9.67	14.5	8.86	13.3	8.18	12.3
15/16	13.6	20.4	12.2	18.4	11.1	16.7	10.2	15.3	9.39	14.1
1	15.4	23.2	13.9	20.9	12.6	19.0	11.6	17.4	10.7	16.1
1 1/16	17.4	26.2	15.7	23.6	14.3	21.4	13.1	19.6	12.1	18.1
1 1/8	19.5	29.4	17.6	26.4	16.0	24.0	14.7	22.0	13.5	20.3
1 3/16	21.8	32.7	19.6	29.4	17.8	26.8	16.3	24.5	15.1	22.6
1 1/4	24.1	36.3	21.7	32.6	19.7	29.7	18.1	27.2	16.7	25.1

F_u = 65 ksi

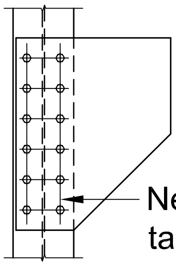
Table 15-2b
Preliminary Hanger Connection Selection Table
Available tensile strength, kips per linear in., limited by bending of the flange



t, in.	*b*, in. 1 ASD	1 LRFD	1 1/4 ASD	1 1/4 LRFD	1 1/2 ASD	1 1/2 LRFD	1 3/4 ASD	1 3/4 LRFD	2 ASD	2 LRFD
5/16	3.80	5.71	3.04	4.57	2.53	3.81	2.17	3.26	1.90	2.86
3/8	5.47	8.23	4.38	6.58	3.65	5.48	3.13	4.70	2.74	4.11
7/16	7.45	11.2	5.96	8.96	4.97	7.46	4.26	6.40	3.72	5.60
1/2	9.73	14.6	7.78	11.7	6.49	9.75	5.56	8.36	4.87	7.31
9/16	12.3	18.5	9.85	14.8	8.21	12.3	7.04	10.6	6.16	9.25
5/8	15.2	22.9	12.2	18.3	10.1	15.2	8.69	13.1	7.60	11.4
11/16	18.4	27.7	14.7	22.1	12.3	18.4	10.5	15.8	9.20	13.8
3/4	21.9	32.9	17.5	26.3	14.6	21.9	12.5	18.8	10.9	16.5
13/16	25.7	38.6	20.6	30.9	17.1	25.7	14.7	22.1	12.8	19.3
7/8	29.8	44.8	23.8	35.8	19.9	29.9	17.0	25.6	14.9	22.4
15/16	34.2	51.4	27.4	41.1	22.8	34.3	19.5	29.4	17.1	25.7
1	38.9	58.5	31.1	46.8	25.9	39.0	22.2	33.4	19.5	29.3
1 1/16	43.9	66.0	35.2	52.8	29.3	44.0	25.1	37.7	22.0	33.0
1 1/8	49.3	74.0	39.4	59.2	32.8	49.4	28.1	42.3	24.6	37.0
1 3/16	54.9	82.5	43.9	66.0	36.6	55.0	31.4	47.1	27.4	41.2
1 1/4	60.8	91.4	48.7	73.1	40.5	60.9	34.8	52.2	30.4	45.7

t, in.	2 1/4 ASD	2 1/4 LRFD	2 1/2 ASD	2 1/2 LRFD	2 3/4 ASD	2 3/4 LRFD	3 ASD	3 LRFD	3 1/4 ASD	3 1/4 LRFD
5/16	1.69	2.54	1.52	2.29	1.38	2.08	1.27	1.90	1.17	1.76
3/8	2.43	3.66	2.19	3.29	1.99	2.99	1.82	2.74	1.68	2.53
7/16	3.31	4.98	2.98	4.48	2.71	4.07	2.48	3.73	2.29	3.45
1/2	4.32	6.50	3.89	5.85	3.54	5.32	3.24	4.88	2.99	4.50
9/16	5.47	8.23	4.93	7.40	4.48	6.73	4.11	6.17	3.79	5.70
5/8	6.76	10.2	6.08	9.14	5.53	8.31	5.07	7.62	4.68	7.03
11/16	8.18	12.3	7.36	11.1	6.69	10.1	6.13	9.22	5.66	8.51
3/4	9.73	14.6	8.76	13.2	7.96	12.0	7.30	11.0	6.74	10.1
13/16	11.4	17.2	10.3	15.4	9.34	14.0	8.56	12.9	7.91	11.9
7/8	13.2	19.9	11.9	17.9	10.8	16.3	9.93	14.9	9.17	13.8
15/16	15.2	22.9	13.7	20.6	12.4	18.7	11.4	17.1	10.5	15.8
1	17.3	26.0	15.6	23.4	14.2	21.3	13.0	19.5	12.0	18.0
1 1/16	19.5	29.4	17.6	26.4	16.0	24.0	14.6	22.0	13.5	20.3
1 1/8	21.9	32.9	19.7	29.6	17.9	26.9	16.4	24.7	15.2	22.8
1 3/16	24.4	36.7	22.0	33.0	20.0	30.0	18.3	27.5	16.9	25.4
1 1/4	27.0	40.6	24.3	36.6	22.1	33.2	20.3	30.5	18.7	28.1

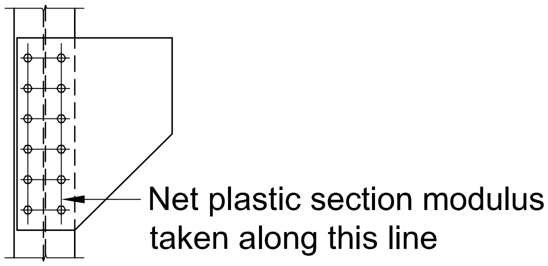
Table 15-3
Net Plastic Section Modulus, Z_{net}, in.^{3}
(Standard Holes)



# Bolts in One Vertical Row, *n*	Bracket Plate Depth, *a*, in.	Nominal Bolt Diameter, *d*, in.							
		3/4					7/8		
		Bracket Plate Thickness, *t*, in.							
		1/4	3/8	1/2	5/8	3/4	3/8	1/2	5/8
2	**6**	1.59	2.39	3.19	3.98	4.78	2.25	3.00	3.75
3	**9**	3.70	5.55	7.40	9.26	11.1	5.25	7.00	8.75
4	**12**	6.38	9.56	12.8	15.9	19.1	9.00	12.0	15.0
5	**15**	10.1	15.1	20.2	25.2	30.2	14.3	19.0	23.8
6	**18**	14.3	21.5	28.7	35.9	43.0	20.3	27.0	33.8
7	**21**	19.6	29.5	39.3	49.1	58.9	27.8	37.0	46.3
8	**24**	25.5	38.3	51.0	63.8	76.5	36.0	48.0	60.0
9	**27**	32.4	48.6	64.8	81.0	97.2	45.8	61.0	76.3
10	**30**	39.8	59.8	79.7	99.6	120	56.3	75.0	93.8
12	**36**	57.4	86.1	115	143	172	81.0	108	135
14	**42**	78.1	117	156	195	234	110	147	184
16	**48**	102	153	204	255	306	144	192	240
18	**54**	129	194	258	323	387	182	243	304
20	**60**	159	239	319	398	478	225	300	375
22	**66**	193	289	386	482	579	272	363	454
24	**72**	230	344	459	574	689	324	432	540
26	**78**	269	404	539	673	808	380	507	634
28	**84**	312	469	625	781	937	441	588	735
30	**90**	359	538	717	896	1080	506	675	844
32	**96**	408	612	816	1020	1220	576	768	960
34	**102**	461	691	921	1150	1380	650	867	1080
36	**108**	516	775	1030	1290	1550	729	972	1220

Notes:
The area reduction per hole is assumed to be $d_h + 1/16$ in.
Bolts spaced 3 in. vertically with $1^1/_2$-in. edge distance at top and bottom.
Values are based on Equations 15-4 and 15-5.

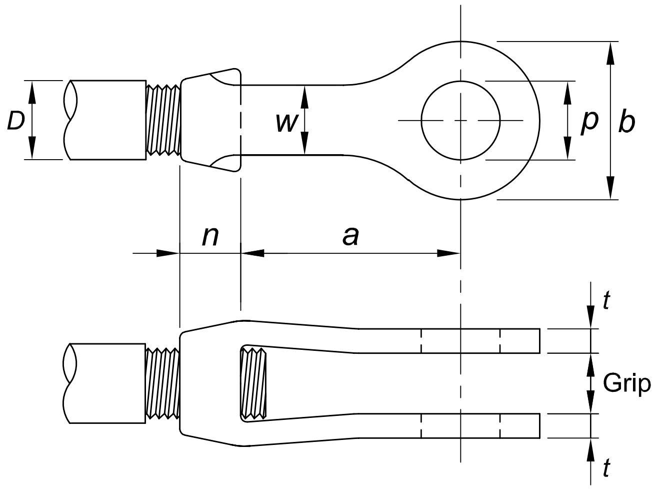
Table 15-3 (continued)
Net Plastic Section Modulus, Z_{net}, in.^{3}
(Standard Holes)



# Bolts in One Vertical Row, n	Bracket Plate Depth, a, in.	Nominal Bolt Diameter, d, in. — 7/8		1				
		Bracket Plate Thickness, t, in.						
		3/4	7/8	1/2	5/8	3/4	7/8	1
2	**6**	4.50	5.25	2.72	3.40	4.08	4.76	5.44
3	**9**	10.5	12.3	6.39	7.98	9.58	11.2	12.8
4	**12**	18.0	21.0	10.9	13.6	16.3	19.0	21.8
5	**15**	28.5	33.3	17.3	21.6	25.9	30.2	34.5
6	**18**	40.5	47.3	24.5	30.6	36.7	42.8	48.9
7	**21**	55.5	64.8	33.6	42.0	50.4	58.8	67.1
8	**24**	72.0	84.0	43.5	54.4	65.3	76.1	87.0
9	**27**	91.5	107	55.3	69.2	83.0	96.8	111
10	**30**	113	131	68.0	85.0	102	119	136
12	**36**	162	189	97.9	122	147	171	196
14	**42**	221	257	133	167	200	233	266
16	**48**	288	336	174	218	261	305	348
18	**54**	365	425	220	275	330	385	440
20	**60**	450	525	272	340	408	476	544
22	**66**	545	635	329	411	493	576	658
24	**72**	648	756	392	489	587	685	783
26	**78**	761	887	459	574	689	804	919
28	**84**	882	1030	533	666	799	933	1070
30	**90**	1010	1180	612	765	918	1070	1220
32	**96**	1150	1340	696	870	1040	1220	1390
34	**102**	1300	1520	786	982	1180	1380	1570
36	**108**	1460	1700	881	1100	1320	1540	1760

Notes:
The area reduction per hole is assumed to be d_h + 1/16 in. for 7/8-in.-diameter bolts and d_h + 1/8 in. for 1-in.-diameter bolts.
Bolts spaced 3 in. vertically with 1 1/2-in. edge distance at top and bottom.
Values are based on Equations 15-4 and 15-5.

Table 15-4
Dimensions and Weights of Clevises



Clevis Number	Dimensions, in.							Weight, lb	Available Strength, kips*	
	Max. *D*	Max. *p*	*b*	*n*	*a*	*w*	*t*		ASD	LRFD
2	5/8	3/4	1 7/16	5/8	3 9/16	1 1/16	5/16 (+1/32, -0)	1	5.83	8.75
2 1/2	7/8	1 1/2	2 1/2	1	4	1 1/4	5/16 (+1/32, -0)	2.5	12.5	18.8
3	1 3/8	1 3/4	3	1 1/4	5 1/16	1 1/2	1/2 (+1/16, -1/32)	4	25.0	37.5
3 1/2	1 1/2	2	3 1/2	1 1/2	6	1 3/4	1/2 (+1/16, -1/16)	6	30.0	45.0
4	1 3/4	2 1/4	4	1 3/4	5 15/16	2	1/2 (+1/16, -1/16)	9	35.0	52.5
5	2 1/8	2 1/2	5	2 1/4	7	2 1/2	5/8 (+3/32, -0)	16	62.5	93.8
6	2 1/2	3	6	2 3/4	8	3	3/4 (+3/32, -0)	26	90.0	135
7	3	3 3/4	7	3	9	3 1/2	7/8 (+1/8, -1/16)	36	114	171
8	4	4 1/4	8	4	10 1/8	4	1 1/2 (+1/8, -1/16)	90	225	338

ASD	LRFD
$\Omega = 3.00$	$\phi = 0.50$

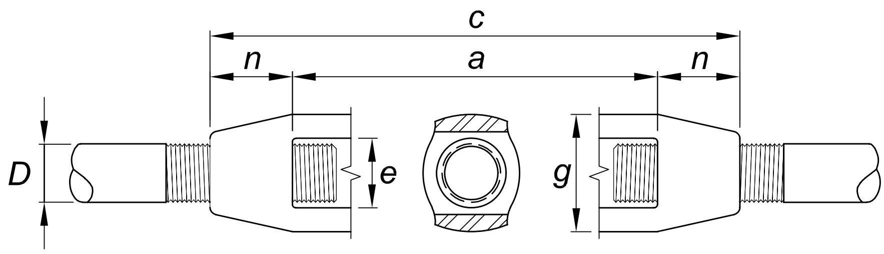
Notes:
Weights and dimensions of clevises are typical; products of all suppliers are essentially similar. User shall verify with the manufacturer that product meets available strength specifications above.
* Strength at service load corresponds to a 3:1 safety factor using maximum pin diameter.

Table 15-5
Clevis Numbers Compatible with Various Rods and Pins

Dia. of Tap, in.	Diameter of Pin, in.																	
	1/2	5/8	3/4	7/8	1	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	2 3/4	3	3 1/4	3 1/2	3 3/4	4	4 1/4
3/8	2	2	2	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
1/2	2	2	2	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
5/8	2	2	2	2 1/2	2 1/2	2 1/2	2 1/2	–	–	–	–	–	–	–	–	–	–	–
3/4	–	–	2 1/2	2 1/2	2 1/2	2 1/2	2 1/2	–	–	–	–	–	–	–	–	–	–	–
7/8	–	–	–	2 1/2	2 1/2	2 1/2	2 1/2	3	–	–	–	–	–	–	–	–	–	–
1	–	–	–	–	3	3	3	3	–	–	–	–	–	–	–	–	–	–
1 1/8	–	–	–	–	3	3	3	3	3 1/2	–	–	–	–	–	–	–	–	–
1 1/4	–	–	–	–	3	3	3	3	3 1/2	–	–	–	–	–	–	–	–	–
1 3/8	–	–	–	–	–	3	3	3 1/2	3 1/2	4	–	–	–	–	–	–	–	–
1 1/2	–	–	–	–	–	3 1/2	3 1/2	4	4	5	–	–	–	–	–	–	–	–
1 5/8	–	–	–	–	–	4	4	4	5	5	5	–	–	–	–	–	–	–
1 3/4	–	–	–	–	–	–	4	5	5	5	5	–	–	–	–	–	–	–
1 7/8	–	–	–	–	–	–	5	5	5	5	5	–	–	–	–	–	–	–
2	–	–	–	–	–	–	5	5	5	5	5	6	6	–	–	–	–	–
2 1/8	–	–	–	–	–	–	–	5	5	6	6	6	6	–	–	–	–	–
2 1/4	–	–	–	–	–	–	–	–	6	6	6	6	6	7	7	–	–	–
2 3/8	–	–	–	–	–	–	–	–	6	6	6	6	7	7	7	7	–	–
2 1/2	–	–	–	–	–	–	–	–	6	6	6	7	7	7	7	7	–	–
2 5/8	–	–	–	–	–	–	–	–	–	–	7	7	7	7	7	8	–	–
2 3/4	–	–	–	–	–	–	–	–	–	–	7	7	7	7	8	8	–	–
2 7/8	–	–	–	–	–	–	–	–	–	–	7	8	8	8	8	8	8	8
3	–	–	–	–	–	–	–	–	–	–	7	8	8	8	8	8	8	8
3 1/8	–	–	–	–	–	–	–	–	–	–	–	8	8	8	8	8	8	8
3 1/4	–	–	–	–	–	–	–	–	–	–	–	8	8	8	8	8	8	8
3 3/8	–	–	–	–	–	–	–	–	–	–	–	8	8	8	8	8	8	8
3 1/2	–	–	–	–	–	–	–	–	–	–	–	–	8	8	8	8	8	8
3 5/8	–	–	–	–	–	–	–	–	–	–	–	–	8	8	8	8	8	–
3 3/4	–	–	–	–	–	–	–	–	–	–	–	–	8	8	8	8	8	–
3 7/8	–	–	–	–	–	–	–	–	–	–	–	–	–	8	8	8	–	–
4	–	–	–	–	–	–	–	–	–	–	–	–	–	8	8	–	–	–

Notes:
Tabular values assume that the net area of the clevis through the pin hole is greater than or equal to 125% of the net area of the rod, and is applicable to round rods without upset ends. For other net area ratios, the required clevis size may be calculated by referring to the dimensions tabulated in Tables 15-4 and 7-17.

Table 15-6
Dimensions and Weights of Turnbuckles



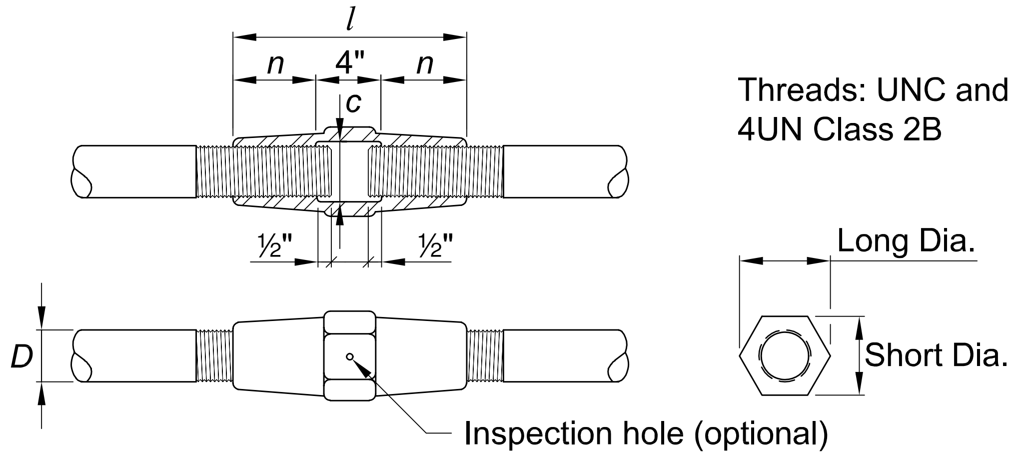
Threads: UNC and 4UN Class 2B

Diameter D, in.	Dimensions, in.					Weight (lb) for Length a, in.						Available Strength, kips	
	a	n	c	e	g	6	9	12	18	24	26	ASD R_n/Ω	LRFD ϕR_n
$\frac{3}{8}$	6	$\frac{9}{16}$	$7\frac{1}{8}$	$\frac{9}{16}$	$1\frac{1}{32}$	0.42	–	–	–	–	–	2.00	3.00
$\frac{1}{2}$	6	$\frac{25}{32}$	$7\frac{9}{16}$	$\frac{11}{16}$	$1\frac{5}{16}$	0.65	0.90	1.20	–	–	–	3.67	5.50
$\frac{5}{8}$	6	$\frac{15}{16}$	$7\frac{7}{8}$	$\frac{13}{16}$	$1\frac{1}{2}$	0.98	1.35	1.58	2.43	–	–	5.83	8.75
$\frac{3}{4}$	6	$1\frac{1}{16}$	$8\frac{1}{8}$	$\frac{15}{16}$	$1\frac{23}{32}$	1.45	1.84	2.35	3.06	4.25	–	8.67	13.0
$\frac{7}{8}$	6	$1\frac{5}{16}$	$8\frac{5}{8}$	$1\frac{3}{32}$	$1\frac{7}{8}$	1.85	–	3.02	4.20	5.43	–	12.0	18.0
1	6	$1\frac{7}{16}$	$8\frac{7}{8}$	$1\frac{9}{32}$	$2\frac{1}{32}$	2.60	–	4.02	4.40	6.85	10.0	15.5	23.3
$1\frac{1}{8}$	6	$1\frac{9}{16}$	$9\frac{1}{8}$	$1\frac{13}{32}$	$2\frac{9}{32}$	4.06	–	4.70	6.10	–	–	19.3	29.0
$1\frac{1}{4}$	6	$1\frac{9}{16}$	$9\frac{1}{8}$	$1\frac{9}{16}$	$2\frac{17}{32}$	4.00	–	6.49	7.13	11.3	13.1	25.3	38.0
$1\frac{3}{8}$	6	$1\frac{13}{16}$	$9\frac{5}{8}$	$1\frac{11}{16}$	$2\frac{3}{4}$	6.15	–	–	–	–	–	29.0	43.5
$1\frac{1}{2}$	6	$1\frac{7}{8}$	$9\frac{3}{4}$	$1\frac{27}{32}$	$3\frac{1}{32}$	6.15	–	9.70	9.13	16.8	19.4	35.0	52.5
$1\frac{5}{8}$	6	$2\frac{1}{2}$	11	$1\frac{31}{32}$	$3\frac{9}{32}$	9.80	–	–	–	–	–	40.9	61.3
$1\frac{3}{4}$	6	$2\frac{1}{2}$	11	$2\frac{1}{8}$	$3\frac{9}{16}$	9.80	–	15.3	16.0	19.5	–	47.2	70.8
$1\frac{7}{8}$	6	$2\frac{13}{16}$	$11\frac{5}{8}$	$2\frac{3}{8}$	4	14.0	–	15.3	–	–	–	62.0	93.0
2	6	$2\frac{13}{16}$	$11\frac{5}{8}$	$2\frac{3}{8}$	4	14.0	–	15.3	–	27.5	–	62.0	93.0
$2\frac{1}{4}$	6	$3\frac{5}{16}$	$12\frac{5}{8}$	$2\frac{11}{16}$	$4\frac{5}{8}$	19.6	–	30.9	–	43.5	–	80.0	120
$2\frac{1}{2}$	6	$3\frac{3}{4}$	$13\frac{1}{2}$	3	5	23.3	–	30.9	–	42.4	–	100	150
$2\frac{3}{4}$	6	$4\frac{3}{16}$	$14\frac{3}{8}$	$3\frac{1}{4}$	$5\frac{5}{8}$	31.5	–	–	–	54.0	–	125	188
3	6	$4\frac{5}{16}$	$14\frac{5}{8}$	$3\frac{5}{8}$	$6\frac{1}{8}$	39.5	–	–	–	–	–	161	242
$3\frac{1}{4}$	6	$5\frac{7}{16}$	$16\frac{7}{8}$	$3\frac{7}{8}$	$6\frac{3}{4}$	60.5	–	79.5	–	–	–	203	305
$3\frac{1}{2}$	6	$5\frac{7}{16}$	$16\frac{7}{8}$	$3\frac{7}{8}$	$6\frac{3}{4}$	60.5	70.0	79.5	–	–	–	203	305
$3\frac{3}{4}$	6	6	18	$4\frac{5}{8}$	$8\frac{1}{2}$	95.0	–	–	–	–	–	280	420
4	6	6	18	$4\frac{5}{8}$	$8\frac{1}{2}$	95.0	–	–	–	–	–	280	420
$4\frac{1}{4}$	9	$6\frac{3}{4}$	$22\frac{1}{2}$	$5\frac{1}{4}$	$9\frac{3}{4}$	–	152	–	–	–	–	390	585
$4\frac{1}{2}$	9	$6\frac{3}{4}$	$22\frac{1}{2}$	$5\frac{1}{4}$	$9\frac{3}{4}$	–	152	–	–	–	–	390	585
$4\frac{3}{4}$	9	$6\frac{3}{4}$	$22\frac{1}{2}$	$5\frac{1}{4}$	$9\frac{3}{4}$	–	152	–	–	–	–	390	585
5	9	$7\frac{1}{2}$	24	6	10	–	200	–	–	–	–	491	737

ASD	LRFD
$\Omega = 3.00$	$\phi = 0.50$

Notes:
Weights and dimensions of turnbuckles are typical; products of all suppliers are essentially similar. Users shall verify with the manufacturer that product meets strength specifications above.

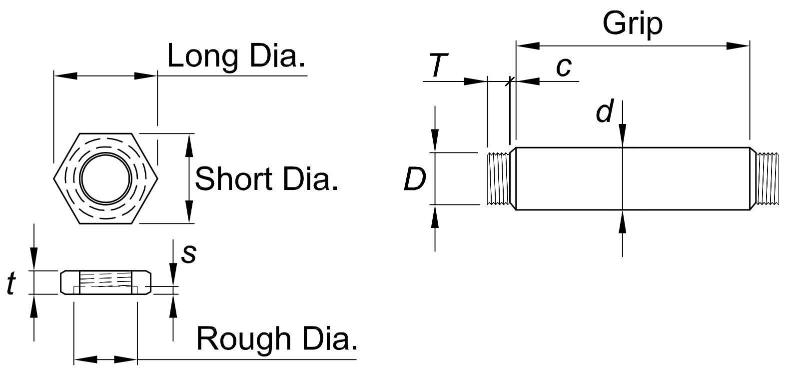
Table 15-7
Dimensions and Weights of Sleeve Nuts



Screw Dia., *D*, in.	Dimensions, in.					Weight, lb
	Short Dia.	Long Dia.	Length, *l*	Nut, *n*	Clear, *c*	
3/8	11/16	25/32	4	—	—	0.27
7/16	25/32	7/8	4	—	—	0.34
1/2	7/8	1	4	—	—	0.43
9/16	15/16	1 1/16	5	—	—	0.64
5/8	1 1/16	1 7/32	5	—	—	0.93
3/4	1 1/4	1 7/16	5	—	—	1.12
7/8	1 7/16	1 5/8	7	1 7/16	1	1.75
1	1 5/8	1 13/16	7	1 7/16	1 1/8	2.46
1 1/8	1 13/16	2 1/16	7 1/2	1 5/8	1 1/4	3.10
1 1/4	2	2 1/4	7 1/2	1 5/8	1 3/8	4.04
1 3/8	2 3/16	2 1/2	8	1 7/8	1 1/2	4.97
1 1/2	2 3/8	2 11/16	8	1 7/8	1 5/8	6.16
1 5/8	2 9/16	2 15/16	8 1/2	2 1/16	1 3/4	7.36
1 3/4	2 3/4	3 1/8	8 1/2	2 1/16	1 7/8	8.87
1 7/8	2 15/16	3 5/16	9	2 5/16	2	10.4
2	3 1/8	3 1/2	9	2 5/16	2 1/8	12.2
2 1/4	3 1/2	3 15/16	9 1/2	2 1/2	2 3/8	16.2
2 1/2	3 7/8	4 3/8	10	2 3/4	2 5/8	21.1
2 3/4	4 1/4	4 13/16	10 1/2	2 15/16	2 7/8	26.7
3	4 5/8	5 1/4	11	3 3/16	3 1/8	33.2
3 1/4	5	5 5/8	11 1/2	3 3/8	3 3/8	40.6
3 1/2	5 3/8	6	12	3 5/8	3 5/8	49.1
3 3/4	5 3/4	6 3/8	12 1/2	3 13/16	3 7/8	58.6
4	6 1/8	6 7/8	13	4 1/16	4 1/8	69.2
4 1/4	6 1/2	7 1/2	13 1/2	4 3/4	4 3/8	75.0
4 1/2	6 7/8	7 15/16	14	5	4 3/4	90.0
4 3/4	7 1/4	8 3/8	14 1/2	5 1/4	5	98.0
5	7 5/8	8 7/8	15	5 1/2	5 1/4	110
5 1/4	8	9 1/4	15 1/2	5 3/4	5 1/2	122
5 1/2	8 3/8	9 3/4	16	6	5 3/4	142
5 3/4	8 3/4	10 1/8	16 1/2	6 1/4	6	157
6	9 1/8	10 5/8	17	6 1/2	6 1/4	176

Notes:
Weights and dimensions of sleeve nuts are typical; products of all suppliers are essentially similar. User shall verify with the manufacturer that strengths of sleeve nut are greater than the corresponding connecting rod when the same material is used.

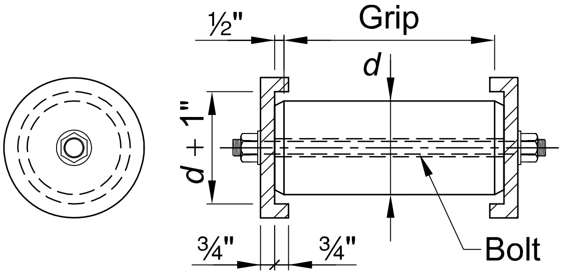
Table 15-8
Dimensions and Weights of Recessed-Pin Nuts



Material: Steel

Threads: 6UN Class 2A/2B

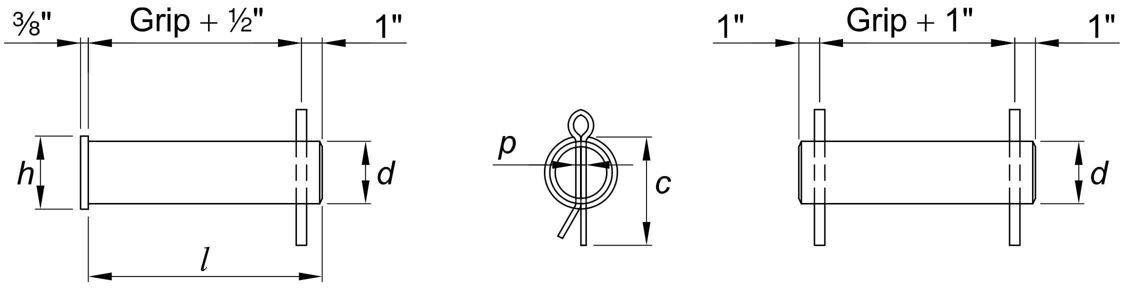
Pin Dia. *d*, in.	Pin Dimensions, in. — Thread *D*	Pin Dimensions, in. — Thread *T*	Pin Dimensions, in. — *c*	Nut Dimensions, in. — Thick-ness, *t*	Nut Dimensions, in. — Diameter Short Dia.	Nut Dimensions, in. — Diameter Long Dia.	Nut Dimensions, in. — Recess Rough Dia.	Nut Dimensions, in. — Recess *s*	Weight, lb
2, 2 1/4	1 1/2	1	1/8	7/8	3	3 3/8	2 5/8	1/4	1
2 1/2, 2 3/4	2	1 1/8	1/8	1	3 5/8	4 1/8	3 1/8	1/4	2
3, 3 1/4, 3 1/2	2 1/2	1 1/4	1/8	1 1/8	4 3/8	5	3 7/8	3/8	3
3 3/4, 4	3	1 3/8	1/4	1 1/4	4 7/8	5 5/8	4 3/8	3/8	4
4 1/4, 4 1/2, 4 3/4	3 1/2	1 1/2	1/4	1 3/8	5 3/4	6 5/8	5 1/4	1/2	5
5, 5 1/4	4	1 5/8	1/4	1 1/2	6 1/4	7 1/4	5 3/4	1/2	6
5 1/2, 5 3/4, 6	4 1/2	1 3/4	1/4	1 5/8	7	8 1/8	6 1/2	5/8	8
6 1/4, 6 1/2	5	1 7/8	3/8	1 3/4	7 5/8	8 7/8	7	5/8	10
6 3/4, 7	5 1/2	2	3/8	1 7/8	8 1/8	9 3/8	7 1/2	3/4	12
7 1/4, 7 1/2	5 1/2	2	3/8	1 7/8	8 5/8	10	8	3/4	14
7 3/4, 8, 8 1/4	6	2 1/4	3/8	2 1/8	9 3/8	10 7/8	8 3/4	3/4	19
8 1/2, 8 3/4, 9	6	2 1/4	3/8	2 1/8	10 1/4	11 7/8	9 5/8	3/4	24
9 1/4, 9 1/2	6	2 3/8	3/8	2 1/4	11 1/4	13	10 5/8	3/4	32
9 3/4, 10	6	2 3/8	3/8	2 1/4	11 1/4	13	10 5/8	3/4	32



Typical Pin Cap Detail
for Pins over 10 in. Diameter
(dimensions shown are approximate)

Notes:
Although nuts may be used on all sizes of pins as shown above, a detail similar to that shown at the left is preferable for pin diameters over 10 in. In this detail, the pin is held in place by a recessed cap at each end and secured by a bolt passing completely through the caps and pin. Suitable provisions must be made for attaching pilots and driving nuts.

Table 15-9
Dimensions and Weights of Clevis and Cotter Pins



l = Length of pin, in.

Horizontal or Vertical Pin

Horizontal Pin

Pin Diameter *d*, in.	Pins with Heads		Cotter		
	Head Diameter *h*, in.	Weight of One, lb	Length *c*, in.	Diameter *p*, in.	Weight per 100, lb
$1\frac{1}{4}$	$1\frac{1}{2}$	$0.19 + 0.35l$	2	$\frac{1}{4}$	2.64
$1\frac{1}{2}$	$1\frac{3}{4}$	$0.26 + 0.50l$	$2\frac{1}{2}$	$\frac{1}{4}$	3.10
$1\frac{3}{4}$	2	$0.33 + 0.68l$	$2\frac{3}{4}$	$\frac{1}{4}$	3.50
2	$2\frac{3}{8}$	$0.47 + 0.89l$	3	$\frac{3}{8}$	9.00
$2\frac{1}{4}$	$2\frac{5}{8}$	$0.58 + 1.13l$	$3\frac{1}{4}$	$\frac{3}{8}$	9.40
$2\frac{1}{2}$	$2\frac{7}{8}$	$0.70 + 1.39l$	$3\frac{3}{4}$	$\frac{3}{8}$	10.9
$2\frac{3}{4}$	$3\frac{1}{8}$	$0.82 + 1.68l$	4	$\frac{3}{8}$	11.4
3	$3\frac{1}{2}$	$1.02 + 2.00l$	5	$\frac{1}{2}$	28.5
$3\frac{1}{4}$	$3\frac{3}{4}$	$1.17 + 2.35l$	5	$\frac{1}{2}$	28.5
$3\frac{1}{2}$	4	$1.34 + 2.73l$	6	$\frac{1}{2}$	33.8
$3\frac{3}{4}$	$4\frac{1}{4}$	$1.51 + 3.13l$	6	$\frac{1}{2}$	33.8

PART 16

SPECIFICATIONS AND CODES

ANSI/AISC 360-16
An American National Standard

Specification for Structural Steel Buildings

July 7, 2016

Supersedes the *Specification for Structural Steel Buildings* dated June 22, 2010 and all previous versions

Approved by the Committee on Specifcations

AMERICAN INSTITUTE OF STEEL CONSTRUCTION
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PREFACE

(This Preface is not part of ANSI/AISC 360-16, *Specification for Structural Steel Buildings*, but is included for informational purposes only.)

This Specification is based upon past successful usage, advances in the state of knowledge, and changes in design practice. The 2016 American Institute of Steel Construction's *Specification for Structural Steel Buildings* provides an integrated treatment of allowable strength design (ASD) and load and resistance factor design (LRFD), and replaces earlier Specifications. As indicated in Chapter B of the Specification, designs can be made according to either ASD or LRFD provisions.

This ANSI-approved Specification has been developed as a consensus document using ANSI-accredited procedures to provide a uniform practice in the design of steel-framed buildings and other structures. The intention is to provide design criteria for routine use and not to provide specific criteria for infrequently encountered problems, which occur in the full range of structural design.

This Specification is the result of the consensus deliberations of a committee of structural engineers with wide experience and high professional standing, representing a wide geographical distribution throughout the United States. The committee includes approximately equal numbers of engineers in private practice and code agencies, engineers involved in research and teaching, and engineers employed by steel fabricating and producing companies. The contributions and assistance of more than 50 additional professional volunteers working in task committees are also hereby acknowledged.

The Symbols, Glossary, Abbreviations and Appendices to this Specification are an integral part of the Specification. A nonmandatory Commentary has been prepared to provide background for the Specification provisions and the user is encouraged to consult it. Additionally, nonmandatory User Notes are interspersed throughout the Specification to provide concise and practical guidance in the application of the provisions.

A number of significant technical modifications have also been made since the 2010 edition of the Specification, including the following:

- Adopted an ASTM umbrella bolt specification, ASTM F3125, that includes Grades A325, A325M, A490, A490M, F1852 and F2280
- Adopted new ASTM HSS material specifications, ASTM A1085/A1085M and A1065/A1065M, that permit use of a design thickness equal to the full nominal thickness of the member
- Expanded the structural integrity provisions applicable to connection design
- Added a shear lag factor for welded plates or connected elements with unequal length longitudinal welds
- The available compressive strength for double angles and tees is determined by the general flexural-torsional buckling equation for members without slender elements
- Added a constrained-axis torsional buckling limit state for members with lateral bracing offset from the shear center
- Revised the available compressive strength formulation for members with slender compression elements
- Reformulated the available flexural strength provisions for tees and double angles

- Revised the shear strength of webs of certain I-shapes and channels without tension field action and when considering tension field action
- Increased the limit on rebar strength to 80 ksi for composite columns
- Incorporated provisions for applying the direct analysis method to composite members
- Inserted general requirements to address minimum composite action in composite beams
- Revised the provisions for bolts in combination with welds
- Increased minimum pretension for $1\frac{1}{8}$-in.-diameter and larger bolts
- Increased standard hole sizes and short-slot and long-slot widths for 1-in.-diameter and larger bolts
- Reorganized the HSS connection design provisions in Chapter K, including reference to Chapter J for some limit states
- Expanded provisions in Appendix 1 for direct modeling of member imperfections and inelasticity that may be used with the direct analysis method
- Inserted a table of properties of high-strength bolts at elevated temperatures in Appendix 4

The reader is cautioned that professional judgment must be exercised when data or recommendations in the Specification are applied, as described more fully in the disclaimer notice preceding this Preface.

This Specification was approved by the Committee on Specifications,

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The Committee gratefully acknowledges advisory members, Carlos Aguirre, Edward E. Garvin and Alfred F. Wong, for their contributions, and the following task committee members for their involvement in the development of this document.

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TABLE OF CONTENTS

SYMBOLS

Some definitions in the list below have been simplified in the interest of brevity. In all cases, the definitions given in the body of this Specification govern. Symbols without text definitions, or used only in one location and defined at that location, are omitted in some cases. The section or table number in the righthand column refers to the Section where the symbol is first defined.

Symbol	Definition	Section
A	Cross-sectional area of angle, in.^{2} (mm^2)	F10.2
A_{BM}	Cross-sectional area of the base metal, in.^{2} (mm^2)	J2.4
A_b	Nominal unthreaded body area of bolt or threaded part, in.^{2} (mm^2)	J3.6
A_c	Area of concrete, in.^{2} (mm^2)	I2.1b
A_c	Area of concrete slab within effective width, in.^{2} (mm^2)	I3.2d
A_e	Effective area, in.^{2} (mm^2)	E7.2
A_e	Effective net area, in.^{2} (mm^2)	D2
A_e	Summation of the effective areas of the cross section based on the reduced effective widths, b_e, d_e or h_e, in.^{2} (mm^2)	E7
A_{fc}	Area of compression flange, in.^{2} (mm^2)	G2.2
A_{fg}	Gross area of tension flange, in.^{2} (mm^2)	F13.1
A_{fn}	Net area of tension flange, in.^{2} (mm^2)	F13.1
A_{ft}	Area of tension flange, in.^{2} (mm^2)	G2.2
A_g	Gross area of member, in.^{2} (mm^2)	B4.3a
A_g	Gross area of composite member, in.^{2} (mm^2)	I2.1
A_{gv}	Gross area subject to shear, in.^{2} (mm^2)	J4.2
A_n	Net area of member, in.^{2} (mm^2)	B4.3b
A_{nt}	Net area subject to tension, in.^{2} (mm^2)	J4.3
A_{nv}	Net area subject to shear, in.^{2} (mm^2)	J4.2
A_{pb}	Projected area in bearing, in.^{2} (mm^2)	J7
A_s	Cross-sectional area of steel section, in.^{2} (mm^2)	I2.1b
A_{sa}	Cross-sectional area of steel headed stud anchor, in.^{2} (mm^2)	I8.2a
A_{sf}	Area on the shear failure path, in.^{2} (mm^2)	D5.1
A_{sr}	Area of continuous reinforcing bars, in.^{2} (mm^2)	I2.1a
A_{sr}	Area of developed longitudinal reinforcing steel within the effective width of the concrete slab, in.^{2} (mm^2)	I3.2d.2
A_t	Net area in tension, in.^{2} (mm^2)	App. 3.4
A_T	Nominal forces and deformations due to the design-basis fire defined in Section 4.2.1	App. 4.1.4
A_w	Area of web, the overall depth times the web thickness, dt_w, in.^{2} (mm^2)	G2.1
A_{we}	Effective area of the weld, in.^{2} (mm^2)	J2.4
A_1	Loaded area of concrete, in.^{2} (mm^2)	I6.3a
A_1	Area of steel concentrically bearing on a concrete support, in.^{2} (mm^2)	J8
A_2	Maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, in.^{2} (mm^2)	J8

Symbol	Definition	Section
B	Overall width of rectangular HSS main member, measured 90° to the plane of the connection, in. (mm)	Table D3.1
B_b	Overall width of rectangular HSS branch member or plate, measured 90° to the plane of the connection, in. (mm)	K1.1
B_e	Effective width of rectangular HSS branch member or plate, in. (mm)	K1.1
B_1	Multiplier to account for P-δ effects	App. 8.2
B_2	Multiplier to account for P-Δ effects	App. 8.2
C	HSS torsional constant	H3.1
C_b	Lateral-torsional buckling modification factor for nonuniform moment diagrams when both ends of the segment are braced	F1
C_f	Constant from Table A-3.1 for the fatigue category	App. 3.3
C_m	Equivalent uniform moment factor assuming no relative translation of member ends	App. 8.2.1
C_{v1}	Web shear strength coefficient	G2.1
C_{v2}	Web shear buckling coefficient	G2.2
C_w	Warping constant, in.^{6} (mm^6)	E4
C_1	Coefficient for calculation of effective rigidity of encased composite compression member	I2.1b
C_2	Edge distance increment, in. (mm)	Table J3.5
C_3	Coefficient for calculation of effective rigidity of filled composite compression member	I2.2b
D	Outside diameter of round HSS, in. (mm)	E7.2
D	Outside diameter of round HSS main member, in. (mm)	K1.1
D	Nominal dead load, kips (N)	B3.9
D	Nominal dead load rating	App. 5.4.1
D_b	Outside diameter of round HSS branch member, in. (mm)	K1.1
D_u	In slip-critical connections, a multiplier that reflects the ratio of the mean installed bolt pretension to the specified minimum bolt pretension	J3.8
E	Modulus of elasticity of steel = 29,000 ksi (200 000 MPa)	Table B4.1
E_c	Modulus of elasticity of concrete = $w_c^{1.5}\sqrt{f_c'}$, ksi ($0.043w_c^{1.5}\sqrt{f_c'}$, MPa)	I2.1b
E_s	Modulus of elasticity of steel = 29,000 ksi (200 000 MPa)	I2.1b
EI_{eff}	Effective stiffness of composite section, kip-in.^{2} (N-mm^2)	I2.1b
F_c	Available stress in main member, ksi (MPa)	K1.1
F_{ca}	Available axial stress at the point of consideration, ksi (MPa)	H2
F_{cbw}, F_{cbz}	Available flexural stress at the point of consideration, ksi (MPa)	H2
F_{cr}	Buckling stress for the section as determined by analysis, ksi (MPa)	H3.3
F_{cr}	Critical stress, ksi (MPa)	E3
F_{cr}	Lateral-torsional buckling stress for the section as determined by analysis, ksi (MPa)	F12.2
F_{cr}	Local buckling stress for the section as determined by analysis, ksi (MPa)	F12.3
F_e	Elastic buckling stress, ksi (MPa)	E3
F_{el}	Elastic local buckling stress, ksi (MPa)	E7.1
F_{EXX}	Filler metal classification strength, ksi (MPa)	J2.4
F_{in}	Nominal bond stress, ksi (MPa)	I6.3c

Symbol	Definition	Section
F_L	Nominal compressive strength above which the inelastic buckling limit states apply, ksi (MPa)	F4.2
F_{nBM}	Nominal stress of the base metal, ksi (MPa)	J2.4
F_{nt}	Nominal tensile stress from Table J3.2, ksi (MPa)	J3.6
F'_{nt}	Nominal tensile stress modified to include the effects of shear stress, ksi (MPa)	J3.7
F_{nv}	Nominal shear stress from Table J3.2, ksi (MPa)	J3.6
F_{nw}	Nominal stress of the weld metal, ksi (MPa)	J2.4
F_{nw}	Nominal stress of the weld metal (Chapter J) with no increase in strength due to directionality of load for fillet welds, ksi (MPa)	K5
F_{SR}	Allowable stress range, ksi (MPa)	App. 3.3
F_{TH}	Threshold allowable stress range, maximum stress range for indefinite design life from Table A-3.1, ksi (MPa)	App. 3.3
F_u	Specified minimum tensile strength, ksi (MPa)	D2
F_y	Specified minimum yield stress, ksi (MPa). As used in this Specification, "yield stress" denotes either the specified minimum yield point (for those steels that have a yield point) or specified yield strength (for those steels that do not have a yield point)	B3.3
F_{yb}	Specified minimum yield stress of HSS branch member or plate material, ksi (MPa)	K1.1
F_{yf}	Specified minimum yield stress of the flange, ksi (MPa)	J10.1
F_{ysr}	Specified minimum yield stress of reinforcing steel, ksi (MPa)	I2.1b
F_{yst}	Specified minimum yield stress of the stiffener material, ksi (MPa)	G2.3
F_{yw}	Specified minimum yield stress of the web material, ksi (MPa)	G2.3
G	Shear modulus of elasticity of steel = 11,200 ksi (77 200 MPa)	E4
H	Maximum transverse dimension of rectangular steel member, in. (mm)	16.3c
H	Total story shear, in the direction of translation being considered, produced by the lateral forces used to compute Δ_H, kips (N)	App. 8.2.2
H	Overall height of rectangular HSS member, measured in the plane of the connection, in. (mm)	K1.1
H_b	Overall height of rectangular HSS branch member, measured in the plane of the connection, in. (mm)	K1.1
I	Moment of inertia in the plane of bending, in.^{4} (mm^4)	App. 8.2.1
I_c	Moment of inertia of the concrete section about the elastic neutral axis of the composite section, in.^{4} (mm^4)	I2.1b
I_d	Moment of inertia of the steel deck supported on secondary members, in.^{4} (mm^4)	App. 2.1
I_p	Moment of inertia of primary members, in.^{4} (mm^4)	App. 2.1
I_s	Moment of inertia of secondary members, in.^{4} (mm^4)	App. 2.1
I_s	Moment of inertia of steel shape about the elastic neutral axis of the composite section, in.^{4} (mm^4)	I2.1b
I_{sr}	Moment of inertia of reinforcing bars about the elastic neutral axis of the composite section, in.^{4} (mm^4)	I2.1b
I_{st}	Moment of inertia of transverse stiffeners about an axis in the web center for stiffener pairs, or about the face in contact with the web plate for single stiffeners, in.^{4} (mm^4)	G2.3

Symbol	Definition	Section
I_{st1}	Minimum moment of inertia of transverse stiffeners required for development of the full shear post buckling resistance of the stiffened web panels, in.^{4} (mm^4)	G2.3
I_{st2}	Minimum moment of inertia of transverse stiffeners required for development of web shear buckling resistance, in.^{4} (mm^4)	G2.3
I_x, I_y	Moment of inertia about the principal axes, in.^{4} (mm^4)	E4
I_{yeff}	Effective out-of-plane moment of inertia, in.^{4} (mm^4)	App. 6.3.2a
I_{yc}	Moment of inertia of the compression flange about the y-axis, in.^{4} (mm^4)	F4.2
I_{yt}	Moment of inertia of the tension flange about the y-axis, in.^{4} (mm^4)	App. 6.3.2a
J	Torsional constant, in.^{4} (mm^4)	E4
K	Effective length factor	E2
K_x	Effective length factor for flexural buckling about x-axis	E4
K_y	Effective length factor for flexural buckling about y-axis	E4
K_z	Effective length factor for torsional buckling about the longitudinal axis	E4
L	Length of member, in. (mm)	H3.1
L	Laterally unbraced length of member, in. (mm)	E2
L	Length of span, in. (mm)	App. 6.3.2a
L	Length of member between work points at truss chord centerlines, in. (mm)	E5
L	Nominal live load	B3.9
L	Nominal live load rating	App. 5.4.1
L	Nominal occupancy live load, kips (N)	App. 4.1.4
L	Height of story, in. (mm)	App. 7.3.2
L_b	Length between points that are either braced against lateral displacement of compression flange or braced against twist of the cross section, in. (mm)	F2.2
L_b	Largest laterally unbraced length along either flange at the point of load, in. (mm)	J10.4
L_{br}	Unbraced length within the panel under consideration, in. (mm)	App. 6.2.1
L_{br}	Unbraced length adjacent to the point brace, in. (mm)	App. 6.2.2
L_c	Effective length of member, in. (mm)	E2
L_{cx}	Effective length of member for buckling about x-axis, in. (mm)	E4
L_{cy}	Effective length of member for buckling about y-axis, in. (mm)	E4
L_{cz}	Effective length of member for buckling about longitudinal axis, in. (mm)	E4
L_{c1}	Effective length in the plane of bending, calculated based on the assumption of no lateral translation at the member ends, set equal to the laterally unbraced length of the member unless analysis justifies a smaller value, in. (mm)	App. 8.2.1
L_{in}	Load introduction length, in. (mm)	I6.3c
L_p	Limiting laterally unbraced length for the limit state of yielding, in. (mm)	F2.2
L_p	Length of primary members, ft (m)	App. 2.1

Symbol	Definition	Section
L_r	Limiting laterally unbraced length for the limit state of inelastic lateral-torsional buckling, in. (mm)	F2.2
L_r	Nominal roof live load	App. 5.4.1
L_s	Length of secondary members, ft (m)	App. 2.1
L_v	Distance from maximum to zero shear force, in. (mm)	G5
L_x, L_y, L_z	Laterally unbraced length of the member for each axis, in. (mm)	E4
M_A	Absolute value of moment at quarter point of the unbraced segment, kip-in. (N-mm)	F1
M_a	Required flexural strength using ASD load combinations, kip-in. (N-mm)	J10.4
M_B	Absolute value of moment at centerline of the unbraced segment, kip-in. (N-mm)	F1
M_C	Absolute value of moment at three-quarter point of the unbraced segment, kip-in. (N-mm)	F1
M_c	Available flexural strength, kip-in. (N-mm)	H1.1
M_{cr}	Elastic lateral-torsional buckling moment, kip-in. (N-mm)	F10.2
M_{cx}, M_{cy}	Available flexural strength determined in accordance with Chapter F, kip-in. (N-mm)	H1.1
M_{cx}	Available lateral-torsional strength for major axis flexure determined in accordance with Chapter F using $C_b = 1.0$, kip-in. (N-mm)	H1.3
M_{cx}	Available flexural strength about *x*-axis for the limit state of tensile rupture of the flange, determined according to Section F13.1, kip-in. (N-mm)	H4
M_{lt}	First-order moment using LRFD or ASD load combinations, due to lateral translation of the structure only, kip-in. (N-mm)	App. 8.2
M_{max}	Absolute value of maximum moment in the unbraced segment, kip-in. (N-mm)	F1
M_n	Nominal flexural strength, kip-in. (N-mm)	F1
M_{nt}	First-order moment using LRFD or ASD load combinations, with the structure restrained against lateral translation, kip-in. (N-mm)	App. 8.2
M_p	Plastic bending moment, kip-in. (N-mm)	Table B4.1
M_p	Moment corresponding to plastic stress distribution over the composite cross section, kip-in. (N-mm)	I3.4b
M_r	Required second-order flexural strength using LRFD or ASD load combinations, kip-in. (N-mm)	App. 8.2
M_r	Required flexural strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kip-in. (N-mm)	H1.1
M_r	Required flexural strength of the beam within the panel under consideration using LRFD or ASD load combinations, kip-in. (N-mm)	App. 6.3.1a
M_r	Largest of the required flexural strengths of the beam within the unbraced lengths adjacent to the point brace using LRFD or ASD load combinations, kip-in. (N-mm)	App. 6.3.1b

Symbol	Definition	Section
M_{br}	Required flexural strength of the brace, kip-in. (N-mm)	App. 6.3.2a
M_{ro}	Required flexural strength in chord at a joint, on the side of joint with lower compression stress, kips (N)	Table K2.1
$M_{r\text{-}ip}$	Required in-plane flexural strength in branch using LRFD or ASD load combinations, kip-in. (N-mm)	Table K4.1
$M_{r\text{-}op}$	Required out-of-plane flexural strength in branch using LRFD or ASD load combinations, kip-in. (N-mm)	Table K4.1
M_{rx}, M_{ry}	Required flexural strength, kip-in. (N-mm)	H1.1
M_{rx}	Required flexural strength at the location of the bolt holes, determined in accordance with Chapter C, positive for tension in the flange under consideration, negative for compression, kip-in. (N-mm)	H4
M_u	Required flexural strength using LRFD load combinations, kip-in. (N-mm)	J10.4
M_y	Moment at yielding of the extreme fiber, kip-in. (N-mm)	Table B4.1
M_y	Yield moment corresponding to yielding of the tension flange and first yield of the compression flange, kip-in. (N-mm)	I3.4b
M_y	Yield moment about the axis of bending, kip-in. (N-mm)	F9.1
M_{yc}	Yield moment in the compression flange, kip-in. (N-mm)	F4.1
M_{yt}	Yield moment in the tension flange, kip-in. (N-mm)	F4.4
M_1'	Effective moment at the end of the unbraced length opposite from M_2, kip-in. (N-mm)	App. 1.3.2c
M_1	Smaller moment at end of unbraced length, kip-in. (N-mm)	F13.5
M_2	Larger moment at end of unbraced length, kip-in. (N-mm)	F13.5
N_i	Notional load applied at level *i*, kips (N)	C2.2b
N_i	Additional lateral load, kips (N)	App. 7.3.2
O_v	Overlap connection coefficient	K3.1
P_a	Required axial strength in chord using ASD load combinations, kips (N)	Table K2.1
P_{br}	Required end and intermediate point brace strength using LRFD or ASD load combinations, kips (N)	App. 6.2.2
P_c	Available axial strength, kips (N)	H1.1
P_c	Available axial strength for the limit state of tensile rupture of the net section at the location of bolt holes, kips (N)	H4
P_{cy}	Available axial compressive strength out of the plane of bending, kips (N)	H1.3
P_e	Elastic critical buckling load determined in accordance with Chapter C or Appendix 7, kips (N)	I2.1b
$P_{e\ story}$	Elastic critical buckling strength for the story in the direction of translation being considered, kips (N)	App 8.2.2
P_{e1}	Elastic critical buckling strength of the member in the plane of bending, kips (N)	App. 8.2.1
P_{lt}	First-order axial force using LRFD or ASD load combinations, due to lateral translation of the structure only, kips (N)	App. 8.2
P_{mf}	Total vertical load in columns in the story that are part of moment frames, if any, in the direction of translation being considered, kips (N)	App. 8.2.2

Symbol	Definition	Section
P_n	Nominal axial strength, kips (N)	D2
P_n	Nominal compressive strength, kips (N)	E1
P_{no}	Nominal axial compressive strength of zero length, doubly symmetric, axially loaded composite member, kips (N)	I2.1b
P_{no}	Available compressive strength of axially loaded doubly symmetric filled composite members, kips (N)	I2.2b
P_{ns}	Cross-section compressive strength, kips (N)	C2.3
P_{nt}	First-order axial force using LRFD and ASD load combinations, with the structure restrained against lateral translation, kips (N)	App. 8.2
P_p	Nominal bearing strength, kips (N)	J8
P_r	Largest of the required axial strengths of the column within the unbraced lengths adjacent to the point brace, using LRFD or ASD load combinations, kips (N)	App. 6.2.2
P_r	Required axial compressive strength using LRFD or ASD load combinations, kips (N)	C2.3
P_r	Required axial strength of the column within the panel under consideration, using LRFD or ASD load combinations, kips (N)	App. 6.2.1
P_r	Required second-order axial strength using LRFD or ASD load combinations, kips (N)	App. 8.2
P_r	Required axial strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kips (N)	H1.1
P_r	Required axial strength of the member at the location of the bolt holes; positive in tension, negative in compression, kips (N)	H4
P_r	Required external force applied to the composite member, kips (N)	I6.2a
P_{ro}	Required axial strength in chord at a joint, on the side of joint with lower compression stress, kips (N)	Table K2.1
P_{story}	Total vertical load supported by the story using LRFD or ASD load combinations, as applicable, including loads in columns that are not part of the lateral force-resisting system, kips (N)	App. 8.2.2
P_u	Required axial strength in chord using LRFD load combinations, kips (N)	Table K2.1
P_u	Required axial strength in compression using LRFD load combinations, kips (N)	App. 1.3.2b
P_y	Axial yield strength of the column, kips (N)	J10.6
Q_{ct}	Available tensile strength, kips (N)	I8.3c
Q_{cv}	Available shear strength, kips (N)	I8.3c
Q_f	Chord-stress interaction parameter	J10.3
Q_g	Gapped truss joint parameter accounting for geometric effects	Table K3.1
Q_n	Nominal strength of one steel headed stud or steel channel anchors, kips (N)	I3.2d.1
Q_{nt}	Nominal tensile strength of steel headed stud anchor, kips (N)	I8.3b
Q_{nv}	Nominal shear strength of steel headed stud anchor, kips (N)	I8.3a
Q_{rt}	Required tensile strength, kips (N)	I8.3b
Q_{rv}	Required shear strength, kips (N)	I8.3c
R	Radius of joint surface, in. (mm)	Table J2.2
R_a	Required strength using ASD load combinations	B3.2

Symbol	Definition	Section
R_{FIL}	Reduction factor for joints using a pair of transverse fillet welds only	App. 3.3
R_g	Coefficient to account for group effect	I8.2a
R_M	Coefficient to account for influence of P-δ on P-Δ	App. 8.2.2
R_n	Nominal strength, specified in this Specification	B3.1
R_n	Nominal slip resistance, kips (N)	J1.8
R_n	Nominal strength of the applicable force transfer mechanism, kips (N)	I6.3
R_{nwl}	Total nominal strength of longitudinally loaded fillet welds, as determined in accordance with Table J2.5, kips (N)	J2.4
R_{nwt}	Total nominal strength of transversely loaded fillet welds, as determined in accordance with Table J2.5 without the alternate in Section J2.4(a), kips (N)	J2.4
R_p	Position effect factor for shear studs	I8.2a
R_{pc}	Web plastification factor	F4.1
R_{pg}	Bending strength reduction factor	F5.2
R_{PJP}	Reduction factor for reinforced or nonreinforced transverse partial-joint-penetration (PJP) groove welds	App. 3.3
R_{pt}	Web plastification factor corresponding to the tension flange yielding limit state	F4.4
R_u	Required strength using LRFD load combinations	B3.1
S	Elastic section modulus about the axis of bending, in.^{3} (mm^3)	F7.2
S	Nominal snow load, kips (N)	App. 4.1.4
S	Spacing of secondary members, ft (m)	App. 2.1
S_c	Elastic section modulus to the toe in compression relative to the axis of bending, in.^{3} (mm^3).	F10.3
S_e	Effective section modulus determined with the effective width of the compression flange, in.^{3} (mm^3)	F7.2
S_{ip}	Effective elastic section modulus of welds for in-plane bending, in.^{3} (mm^3)	K5
S_{min}	Minimum elastic section modulus relative to the axis of bending, in.^{3} (mm^3)	F12
S_{op}	Effective elastic section modulus of welds for out-of-plane bending, in.^{3} (mm^3)	K5
S_{xc}, S_{xt}	Elastic section modulus referred to compression and tension flanges, respectively, in.^{3} (mm^3)	Table B4.1
S_x	Elastic section modulus taken about the x-axis, in.^{3} (mm^3)	F2.2
S_x	Minimum elastic section modulus taken about the x-axis, in.^{3} (mm^3)	F13.1
S_y	Elastic section modulus taken about the y-axis, in.^{3} (mm^3)	F6.1
T	Elevated temperature of steel due to unintended fire exposure, °F (°C)	App. 4.2.4d
T_a	Required tension force using ASD load combinations, kips (kN)	J3.9
T_b	Minimum fastener tension given in Table J3.1 or J3.1M, kips (kN)	J3.8
T_c	Available torsional strength, kip-in. (N-mm)	H3.2

Symbol	Definition	Section
T_n	Nominal torsional strength, kip-in. (N-mm)	H3.1
T_r	Required torsional strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kip-in. (N-mm)	H3.2
T_u	Required tension force using LRFD load combinations, kips (kN)	J3.9
U	Shear lag factor	D3
U	Utilization ratio	Table K2.1
U_{bs}	Reduction coefficient, used in calculating block shear rupture strength	J4.3
U_p	Stress index for primary members	App. 2.2
U_s	Stress index for secondary members	App. 2.2
V'	Nominal shear force between the steel beam and the concrete slab transferred by steel anchors, kips (N)	I3.2d
V_{br}	Required shear strength of the bracing system in the direction perpendicular to the longitudinal axis of the column, kips (N)	App. 6.2.1
V_c	Available shear strength, kips (N)	H3.2
V_{c1}	Available shear strength calculated with V_n as defined in Section G2.1 or G2.2. as applicable, kips (N)	G2.3
V_{c2}	Available shear buckling strength, kips (N)	G2.3
V_n	Nominal shear strength, kips (N)	G1
V_r	Required shear strength in the panel being considered, kips (N)	G2.3
V_r	Required shear strength determined in accordance with Chapter C, using LRFD or ASD load combinations, kips (N)	H3.2
V_r'	Required longitudinal shear force to be transferred to the steel or concrete, kips (N)	I6.1
Y_i	Gravity load applied at level *i* from the LRFD load combination or ASD load combination, as applicable, kips (N)	C2.2b
Z	Plastic section modulus taken about the axis of bending, in.^{3} (mm^3)	F7.1
Z_b	Plastic section modulus of branch taken about the axis of bending, in.^{3} (mm^3)	K4.1
Z_x	Plastic section modulus about the *x*-axis, in.^{3} (mm^3)	Table B4.1
Z_y	Plastic section modulus about the *y*-axis, in.^{3} (mm^3)	F6.1
a	Clear distance between transverse stiffeners, in. (mm)	F13.2
a	Distance between connectors, in. (mm)	E6.1
a	Shortest distance from edge of pin hole to edge of member measured parallel to the direction of force, in. (mm)	D5.1
a	Half the length of the nonwelded root face in the direction of the thickness of the tension-loaded plate, in. (mm)	App. 3.3
a'	Weld length along both edges of the cover plate termination to the beam or girder, in. (mm)	F13.3
a_w	Ratio of two times the web area in compression due to application of major axis bending moment alone to the area of the compression flange components	F4.2
b	Full width of leg in compression, in. (mm)	F10.3
b	For flanges of I-shaped members, half the full-flange width, in. (mm)	B4.1a

Symbol	Definition	Section
b	For legs of angles and flanges of channels and zees, full leg or flange width, in. (mm)	B4.1a
b	For plates, the distance from the free edge to the first row of fasteners or line of welds, in. (mm)	B4.1a
b	Width of the element, in. (mm)	E7.1
b	Width of unstiffened compression element; width of stiffened compression element, in. (mm)	B4.1
b	Width of the leg resisting the shear force or depth of tee stem, in. (mm)	G3
b	Width of leg, in. (mm)	F10.2
b_{cf}	Width of column flange, in. (mm)	J10.6
b_e	Reduced effective width, in. (mm)	E7.1
b_e	Effective edge distance for calculation of tensile rupture strength of pin-connected member, in. (mm)	D5.1
b_f	Width of flange, in. (mm)	B4.1
b_{fc}	Width of compression flange, in. (mm)	F4.2
b_{ft}	Width of tension flange, in. (mm)	G2.2
b_l	Length of longer leg of angle, in. (mm)	E5
b_p	Smaller of the dimension a and h, in. (mm)	G2.3
b_s	Length of shorter leg of angle, in. (mm)	E5
b_s	Stiffener width for one-sided stiffeners; twice the individual stiffener width for pairs of stiffeners, in. (mm)	App. 6.3.2a
c	Distance from the neutral axis to the extreme compressive fibers, in. (mm)	App. 6.3.2a
c_1	Effective width imperfection adjustment factor determined from Table E7.1	E7.1
d	Depth of section from which the tee was cut, in. (mm)	Table D3.1
d	Depth of tee or width of web leg in compression, in. (mm)	F9.2
d	Nominal fastener diameter, in. (mm)	J3.3
d	Full nominal depth of the member, in. (mm)	B4.1
d	Depth of rectangular bar, in. (mm)	F11.1
d	Diameter, in. (mm)	J7
d	Diameter of pin, in. (mm)	D5.1
d_b	Depth of beam, in. (mm)	J10.6
d_b	Nominal diameter (body or shank diameter), in. (mm)	App. 3.4
d_c	Depth of column, in. (mm)	J10.6
d_e	Effective width for tees, in. (mm)	E7.1
d_{sa}	Diameter of steel headed stud anchor, in. (mm)	I8.1
e	Eccentricity in a truss connection, positive being away from the branches, in. (mm)	K3.1
$e_{mid\text{-}ht}$	Distance from the edge of steel headed stud anchor shank to the steel deck web, in. (mm)	I8.2a
f_c'	Specified compressive strength of concrete, ksi (MPa)	I1.2b
f_o	Stress due to impounded water due to either nominal rain or snow loads (exclusive of the ponding contribution), and other loads acting concurrently as specified in Section B2, ksi (MPa)	App. 2.2

Symbol	Definition	Section
f_{ra}	Required axial stress at the point of consideration, determined in accordance with Chapter C, using LRFD or ASD load combinations, ksi (MPa)	H2
f_{rbw}, f_{rbz}	Required flexural stress at the point of consideration, determined in accordance with Chapter C, using LRFD or ASD load combinations, ksi (MPa)	H2
f_{rv}	Required shear stress using LRFD or ASD load combinations, ksi (MPa)	J3.7
g	Transverse center-to-center spacing (gage) between fastener gage lines, in. (mm)	B4.3
g	Gap between toes of branch members in a gapped K-connection, neglecting the welds, in. (mm)	K3.1
h	For webs of rolled or formed sections, the clear distance between flanges less the fillet or corner radius at each flange; for webs of built-up sections, the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used; for webs of rectangular HSS, the clear distance between the flanges less the inside corner radius on each side, in. (mm)	B4.1b
h	Width resisting the shear force, taken as the clear distance between the flanges less the inside corner radius on each side for HSS or the clear distance between flanges for box sections, in. (mm)	G4
h_c	Twice the distance from the center of gravity to the following: the inside face of the compression flange less the fillet or corner radius, for rolled shapes; the nearest line of fasteners at the compression flange or the inside faces of the compression flange when welds are used, for built-up sections, in. (mm)	B4.1
h_e	Effective width for webs, in. (mm)	F7.1
h_f	Factor for fillers	E3.8
h_o	Distance between flange centroids, in. (mm)	F2.2
h_p	Twice the distance from the plastic neutral axis to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used, in. (mm)	B4.1b
k	Distance from outer face of flange to the web toe of fillet, in. (mm)	J10.2
k_c	Coefficient for slender unstiffened elements	Table B4.1
k_{sc}	Slip-critical combined tension and shear coefficient	J3.9
k_v	Web plate shear buckling coefficient	G2.1
l	Actual length of end-loaded weld, in. (mm)	J2.2
l	Length of connection, in. (mm)	Table D3.1
l_a	Length of channel anchor, in. (mm)	I8.2b
l_b	Bearing length of the load, measured parallel to the axis of the HSS member (or measured across the width of the HSS in the case of loaded cap plates), in. (mm)	K2.1
l_b	Length of bearing, in. (mm)	J7
l_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material, in. (mm)	J3.10

Symbol	Definition	Section
l_e	Total effective weld length of groove and fillet welds to rectangular HSS for weld strength calculations, in. (mm)	K5
l_{end}	Distance from the near side of the connecting branch or plate to end of chord, in. (mm)	K1.1
l_{ov}	Overlap length measured along the connecting face of the chord beneath the two branches, in. (mm)	K3.1
l_p	Projected length of the overlapping branch on the chord, in. (mm)	K3.1
l_1, l_2	Connection weld length, in. (mm)	Table D3.1
n	Number of braced points within the span	App. 6.3.2a
n	Threads per inch (per mm)	App. 3.4
n_b	Number of bolts carrying the applied tension	J3.9
n_s	Number of slip planes required to permit the connection to slip	J3.8
n_{SR}	Number of stress range fluctuations in design life	App. 3.3
p	Pitch, in. per thread (mm per thread)	App. 3.4
p_b	Perimeter of the steel-concrete bond interface within the composite cross section, in. (mm)	I6.3c
r	Radius of gyration, in. (mm)	E2
r	Retention factor depending on bottom flange temperature	App. 4.2.4d
r_a	Radius of gyration about the geometric axis parallel to the connected leg, in. (mm)	E5
r_i	Minimum radius of gyration of individual component, in. (mm)	E6.1
$\overline{r_o}$	Polar radius of gyration about the shear center, in. (mm)	E4
r_t	Effective radius of gyration for lateral-torsional buckling. For I-shapes with a channel cap or a cover plate attached to the compression flange, radius of gyration of the flange components in flexural compression plus one-third of the web area in compression due to application of major axis bending moment alone, in. (mm)	F4.2
r_x	Radius of gyration about the *x*-axis, in. (mm)	E4
r_y	Radius of gyration about *y*-axis, in. (mm)	E4
r_z	Radius of gyration about the minor principal axis, in. (mm)	E5
s	Longitudinal center-to-center spacing (pitch) of any two consecutive holes, in. (mm)	B4.3b
t	Distance from the neutral axis to the extreme tensile fibers, in. (mm)	App. 6.3.2a
t	Thickness of wall, in. (mm)	E7.2
t	Thickness of angle leg, in. (mm)	F10.2
t	Width of rectangular bar parallel to axis of bending, in. (mm)	F11.1
t	Thickness of connected material, in. (mm)	J3.10
t	Thickness of plate, in. (mm)	D5.1
t	Total thickness of fillers, in. (mm)	J5.2
t	Design wall thickness of HSS member, in. (mm)	B4.2
t	Design wall thickness of HSS main member, in. (mm)	K1.1
t	Thickness of angle leg or of tee stem, in. (mm)	G3
t_b	Design wall thickness of HSS branch member or thickness of plate, in. (mm)	K1.1
t_{bi}	Thickness of overlapping branch, in. (mm)	Table K3.2

Symbol	Definition	Section
t_{bj}	Thickness of overlapped branch, in. (mm)	Table K3.2
t_{cf}	Thickness of column flange, in. (mm)	J10.6
t_f	Thickness of flange, in. (mm)	F3.2
t_f	Thickness of the loaded flange, in. (mm)	J10.1
t_f	Thickness of flange of channel anchor, in. (mm)	I8.2b
t_{fc}	Thickness of compression flange, in. (mm)	F4.2
t_p	Thickness of tension loaded plate, in. (mm)	App. 3.3
t_{st}	Thickness of web stiffener, in. (mm)	App. 6.3.2a
t_w	Thickness of web, in. (mm)	F4.2
t_w	Smallest effective weld throat thickness around the perimeter of branch or plate, in. (mm)	K5
t_w	Thickness of channel anchor web, in. (mm)	I8.2b
w	Width of cover plate, in. (mm)	F13.3
w	Size of weld leg, in. (mm)	J2.2b
w	Subscript relating symbol to major principal axis bending	H2
w	Width of plate, in. (mm)	Table D3.1
w	Leg size of the reinforcing or contouring fillet, if any, in the direction of the thickness of the tension-loaded plate, in. (mm)	App. 3.3
w_c	Weight of concrete per unit volume ($90 \leq w_c \leq 155$ lb/ft^3 or $1\,500 \leq w_c \leq 2\,500$ kg/m^3)	I2.1b
w_r	Average width of concrete rib or haunch, in. (mm)	I3.2c
x	Subscript relating symbol to major axis bending	H1.1
x_o, y_o	Coordinates of the shear center with respect to the centroid, in. (mm)	E4
$\bar{x}$	Eccentricity of connection, in. (mm)	Table D3.1
y	Subscript relating symbol to minor axis bending	H1.1
z	Subscript relating symbol to minor principal axis bending	H2
α	ASD/LRFD force level adjustment factor	C2.3
β	Length reduction factor given by Equation J2-1	J2.2b
β	Width ratio; the ratio of branch diameter to chord diameter for round HSS; the ratio of overall branch width to chord width for rectangular HSS	K3.1
β_T	Overall brace system required stiffness, kip-in./rad (N-mm/rad)	App. 6.3.2a
β_{br}	Required shear stiffness of the bracing system, kip/in. (N/mm)	App. 6.2.1a
β_{br}	Required flexural stiffness of the brace, kip/in. (N/mm)	App. 6.3.2a
β_{eff}	Effective width ratio; the sum of the perimeters of the two branch members in a K-connection divided by eight times the chord width	K3.1
β_{eop}	Effective outside punching parameter	K3.2
β_{sec}	Web distortional stiffness, including the effect of web transverse stiffeners, if any, kip-in./rad (N-mm/rad)	App. 6.3.2a
β_w	Section property for single angles about major principal axis, in. (mm)	F10.2
Δ	First-order interstory drift due to the LRFD or ASD load combinations, in. (mm)	App. 7.3.2

Symbol	Definition	Section
Δ_H	First-order interstory drift, in the direction of translation being considered, due to lateral forces, in. (mm)	App. 8.2.2
γ	Chord slenderness ratio; the ratio of one-half the diameter to the wall thickness for round HSS; the ratio of one-half the width to wall thickness for rectangular HSS	K3.1
ζ	Gap ratio; the ratio of the gap between the branches of a gapped K-connection to the width of the chord for rectangular HSS	K3.1
η	Load length parameter, applicable only to rectangular HSS; the ratio of the length of contact of the branch with the chord in the plane of the connection to the chord width	K3.1
λ	Width-to-thickness ratio for the element as defined in Section B4.1	E7.1
λ_p	Limiting width-to-thickness parameter for compact element	B4.1
λ_{pd}	Limiting width-to-thickness parameter for plastic design	App. 1.2.2b
λ_{pf}	Limiting width-to-thickness parameter for compact flange	F3.2
λ_{pw}	Limiting width-to-thickness parameter for compact web	F4.2
λ_r	Limiting width-to-thickness parameter for noncompact element	B4.1
λ_{rf}	Limiting width-to-thickness parameter for noncompact flange	F3.2
λ_{rw}	Limiting width-to-thickness parameter for noncompact web	F4.2
μ	Mean slip coefficient for Class A or B surfaces, as applicable, or as established by tests	J3.8
ϕ	Resistance factor	B3.1
ϕ_B	Resistance factor for bearing on concrete	I6.3a
ϕ_b	Resistance factor for flexure	H1.1
ϕ_c	Resistance factor for compression	H1.1
ϕ_c	Resistance factor for axially loaded composite columns	I2.1b
ϕ_{sf}	Resistance factor for shear on the failure path	D5.1
ϕ_T	Resistance factor for torsion	H3.1
ϕ_t	Resistance factor for tension	H1.2
ϕ_t	Resistance factor for tensile rupture	H4
ϕ_t	Resistance factor for steel headed stud anchor in tension	I8.3b
ϕ_v	Resistance factor for shear	G1
ϕ_v	Resistance factor for steel headed stud anchor in shear	I8.3a
Ω	Safety factor	B3.2
Ω_B	Safety factor for bearing on concrete	I6.3a
Ω_b	Safety factor for flexure	H1.1
Ω_c	Safety factor for compression	H1.1
Ω_c	Safety factor for axially loaded composite columns	I2.1b
Ω_t	Safety factor for steel headed stud anchor in tension	I8.3b
Ω_{sf}	Safety factor for shear on the failure path	D5.1
Ω_T	Safety factor for torsion	H3.1
Ω_t	Safety factor for tension	H1.2
Ω_t	Safety factor for tensile rupture	H4
Ω_v	Safety factor for shear	G1
Ω_v	Safety factor for steel headed stud anchor in shear	I8.3a
ρ_w	Maximum shear ratio within the web panels on each side of the transverse stiffener	G2.3

Symbol	Definition	Section
ρ_{sr}	Minimum reinforcement ratio for longitudinal reinforcing	I2.1
θ	Angle between the line of action of the required force and the weld longitudinal axis, degrees	J2.4
θ	Acute angle between the branch and chord, degrees	K3.1
τ_b	Stiffness reduction parameter	C2.3

GLOSSARY

Notes:

(1) Terms designated with † are common AISI-AISC terms that are coordinated between the two standards development organizations.

(2) Terms designated with * are usually qualified by the type of load effect, for example, nominal tensile strength, available compressive strength, and design flexural strength.

(3) Terms designated with ** are usually qualified by the type of component, for example, web local buckling, and flange local bending.

Active fire protection. Building materials and systems that are activated by a fire to mitigate adverse effects or to notify people to take action to mitigate adverse effects.

*Allowable strength**†. Nominal strength divided by the safety factor, R_n/Ω.

*Allowable stress**. Allowable strength divided by the applicable section property, such as section modulus or cross-sectional area.

Applicable building code†. Building code under which the structure is designed.

ASD (allowable strength design)†. Method of proportioning structural components such that the allowable strength equals or exceeds the required strength of the component under the action of the ASD load combinations.

ASD load combination†. Load combination in the applicable building code intended for allowable strength design (allowable stress design).

Authority having jurisdiction (AHJ). Organization, political subdivision, office or individual charged with the responsibility of administering and enforcing the provisions of this *Specification*.

*Available strength**†. Design strength or allowable strength, as applicable.

*Available stress**. Design stress or allowable stress, as applicable.

Average rib width. In a formed steel deck, average width of the rib of a corrugation.

Beam. Nominally horizontal structural member that has the primary function of resisting bending moments.

Beam-column. Structural member that resists both axial force and bending moment.

Bearing†. In a connection, limit state of shear forces transmitted by the mechanical fastener to the connection elements.

Bearing (local compressive yielding)†. Limit state of local compressive yielding due to the action of a member bearing against another member or surface.

Bearing-type connection. Bolted connection where shear forces are transmitted by the bolt bearing against the connection elements.

Block shear rupture†. In a connection, limit state of tension rupture along one path and shear yielding or shear rupture along another path.

Box section. Square or rectangular doubly symmetric member made with four plates welded together at the corners such that it behaves as a single member.

Braced frame†. Essentially vertical truss system that provides resistance to lateral forces and provides stability for the structural system.

Bracing. Member or system that provides stiffness and strength to limit the out-of-plane movement of another member at a brace point.

Branch member. In an HSS connection, member that terminates at a chord member or main member.

Buckling†. Limit state of sudden change in the geometry of a structure or any of its elements under a critical loading condition.

Buckling strength. Strength for instability limit states.

Built-up member, cross section, section, shape. Member, cross section, section or shape fabricated from structural steel elements that are welded or bolted together.

Camber. Curvature fabricated into a beam or truss so as to compensate for deflection induced by loads.

Charpy V-notch impact test. Standard dynamic test measuring notch toughness of a specimen.

Chord member. In an HSS connection, primary member that extends through a truss connection.

Cladding. Exterior covering of structure.

Cold-formed steel structural member†. Shape manufactured by press-braking blanks sheared from sheets, cut lengths of coils or plates, or by roll forming cold- or hot-rolled coils or sheets; both forming operations being performed at ambient room temperature, that is, without manifest addition of heat such as would be required for hot forming.

Collector. Also known as drag strut; member that serves to transfer loads between floor diaphragms and the members of the lateral force-resisting system.

Column. Nominally vertical structural member that has the primary function of resisting axial compressive force.

Column base. Assemblage of structural shapes, plates, connectors, bolts and rods at the base of a column used to transmit forces between the steel superstructure and the foundation.

Compact section. Section capable of developing a fully plastic stress distribution and possessing a rotation capacity of approximately three before the onset of local buckling.

Compartmentation. Enclosure of a building space with elements that have a specific fire endurance.

Complete-joint-penetration (CJP) groove weld. Groove weld in which weld metal extends through the joint thickness, except as permitted for HSS connections.

Composite. Condition in which steel and concrete elements and members work as a unit in the distribution of internal forces.

Composite beam. Structural steel beam in contact with and acting compositely with a reinforced concrete slab.

Composite component. Member, connecting element or assemblage in which steel and concrete elements work as a unit in the distribution of internal forces, with the exception of the special case of composite beams where steel anchors are embedded in a solid concrete slab or in a slab cast on formed steel deck.

Concrete breakout surface. The surface delineating a volume of concrete surrounding a steel headed stud anchor that separates from the remaining concrete.

Concrete crushing. Limit state of compressive failure in concrete having reached the ultimate strain.

Concrete haunch. In a composite floor system constructed using a formed steel deck, the section of solid concrete that results from stopping the deck on each side of the girder.

Concrete-encased beam. Beam totally encased in concrete cast integrally with the slab.

Connection†. Combination of structural elements and joints used to transmit forces between two or more members.

Construction documents. Written, graphic and pictorial documents prepared or assembled for describing the design (including the structural system), location and physical characteristics of the elements of a building necessary to obtain a building permit and construct a building.

Cope. Cutout made in a structural member to remove a flange and conform to the shape of an intersecting member.

Cover plate. Plate welded or bolted to the flange of a member to increase cross-sectional area, section modulus or moment of inertia.

Cross connection. HSS connection in which forces in branch members or connecting elements transverse to the main member are primarily equilibrated by forces in other branch members or connecting elements on the opposite side of the main member.

Design. The process of establishing the physical and other properties of a structure for the purpose of achieving the desired strength, serviceability, durability, constructability, economy and other desired characteristics. Design for strength, as used in this *Specification*, includes analysis to determine required strength and proportioning to have adequate available strength.

Design-basis fire. Set of conditions that define the development of a fire and the spread of combustion products throughout a building or portion thereof.

Design drawings. Graphic and pictorial documents showing the design, location and dimensions of the work. These documents generally include plans, elevations, sections, details, schedules, diagrams and notes.

Design load†. Applied load determined in accordance with either LRFD load combinations or ASD load combinations, as applicable.

*Design strength**†. Resistance factor multiplied by the nominal strength, ϕR_n.

Design wall thickness. HSS wall thickness assumed in the determination of section properties.

Diagonal stiffener. Web stiffener at column panel zone oriented diagonally to the flanges, on one or both sides of the web.

Diaphragm†. Roof, floor or other membrane or bracing system that transfers in-plane forces to the lateral force-resisting system.

Diaphragm plate. Plate possessing in-plane shear stiffness and strength, used to transfer forces to the supporting elements.

Direct bond interaction. In a composite section, mechanism by which force is transferred between steel and concrete by bond stress.

Distortional failure. Limit state of an HSS truss connection based on distortion of a rectangular HSS chord member into a rhomboidal shape.

Distortional stiffness. Out-of-plane flexural stiffness of web.

Double curvature. Deformed shape of a beam with one or more inflection points within the span.

*Double-concentrated force*s. Two equal and opposite forces applied normal to the same flange, forming a couple.

Doubler. Plate added to, and parallel with, a beam or column web to increase strength at locations of concentrated forces.

Drift. Lateral deflection of structure.

Effective length factor, K. Ratio between the effective length and the unbraced length of the member.

Effective length. Length of an otherwise identical compression member with the same strength when analyzed with simple end conditions.

Effective net area. Net area modified to account for the effect of shear lag.

Effective section modulus. Section modulus reduced to account for buckling of slender compression elements.

Effective width. Reduced width of a plate or slab with an assumed uniform stress distribution which produces the same effect on the behavior of a structural member as the actual plate or slab width with its nonuniform stress distribution.

Elastic analysis. Structural analysis based on the assumption that the structure returns to its original geometry on removal of the load.

Elevated temperatures. Heating conditions experienced by building elements or structures as a result of fire which are in excess of the anticipated ambient conditions.

Encased composite member. Composite member consisting of a structural concrete member and one or more embedded steel shapes.

End panel. Web panel with an adjacent panel on one side only.

End return. Length of fillet weld that continues around a corner in the same plane.

Engineer of record. Licensed professional responsible for sealing the design drawings and specifications.

Expansion rocker. Support with curved surface on which a member bears that is able to tilt to accommodate expansion.

Expansion roller. Round steel bar on which a member bears that is able to roll to accommodate expansion.

Eyebar. Pin-connected tension member of uniform thickness, with forged or thermally cut head of greater width than the body, proportioned to provide approximately equal strength in the head and body.

Factored load†. Product of a load factor and the nominal load.

Fastener. Generic term for bolts, rivets or other connecting devices.

Fatigue†. Limit state of crack initiation and growth resulting from repeated application of live loads.

Faying surface. Contact surface of connection elements transmitting a shear force.

Filled composite member. Composite member consisting of an HSS or box section filled with structural concrete.

Filler metal. Metal or alloy added in making a welded joint.

Filler. Plate used to build up the thickness of one component.

Fillet weld reinforcement. Fillet welds added to groove welds.

Fillet weld. Weld of generally triangular cross section made between intersecting surfaces of elements.

Finished surface. Surfaces fabricated with a roughness height value measured in accordance with ANSI/ASME B46.1 that is equal to or less than 500.

Fire. Destructive burning, as manifested by any or all of the following: light, flame, heat or smoke.

Fire barrier. Element of construction formed of fire-resisting materials and tested in accordance with an approved standard fire resistance test, to demonstrate compliance with the applicable building code.

Fire resistance. Property of assemblies that prevents or retards the passage of excessive heat, hot gases or flames under conditions of use and enables the assemblies to continue to perform a stipulated function.

First-order analysis. Structural analysis in which equilibrium conditions are formulated on the undeformed structure; second-order effects are neglected.

Fitted bearing stiffener. Stiffener used at a support or concentrated load that fits tightly against one or both flanges of a beam so as to transmit load through bearing.

Flare bevel groove weld. Weld in a groove formed by a member with a curved surface in contact with a planar member.

Flare V-groove weld. Weld in a groove formed by two members with curved surfaces.

Flashover. Transition to a state of total surface involvement in a fire of combustible materials within an enclosure.

Flat width. Nominal width of rectangular HSS minus twice the outside corner radius. In the absence of knowledge of the corner radius, the flat width is permitted to be taken as the total section width minus three times the thickness.

Flexural buckling†. Buckling mode in which a compression member deflects laterally without twist or change in cross-sectional shape.

Flexural-torsional buckling†. Buckling mode in which a compression member bends and twists simultaneously without change in cross-sectional shape.

Force. Resultant of distribution of stress over a prescribed area.

Formed steel deck. In composite construction, steel cold formed into a decking profile used as a permanent concrete form.

Fully-restrained moment connection. Connection capable of transferring moment with negligible rotation between connected members.

Gage. Transverse center-to-center spacing of fasteners.

Gapped connection. HSS truss connection with a gap or space on the chord face between intersecting branch members.

Geometric axis. Axis parallel to web, flange or angle leg.

Girder filler. In a composite floor system constructed using a formed steel deck, narrow piece of sheet steel used as a fill between the edge of a deck sheet and the flange of a girder.

Girder. See *Beam*.

Gouge. Relatively smooth surface groove or cavity resulting from plastic deformation or removal of material.

Gravity load. Load acting in the downward direction, such as dead and live loads.

Grip (of bolt). Thickness of material through which a bolt passes.

Groove weld. Weld in a groove between connection elements. See also AWS D1.1/D1.1M.

Gusset plate. Plate element connecting truss members or a strut or brace to a beam or column.

Heat flux. Radiant energy per unit surface area.

Heat release rate. Rate at which thermal energy is generated by a burning material.

Horizontal shear. In a composite beam, force at the interface between steel and concrete surfaces.

HSS (hollow structural section). Square, rectangular or round hollow structural steel section produced in accordance with one of the product specifications in Section A3.1a(b).

Inelastic analysis. Structural analysis that takes into account inelastic material behavior, including plastic analysis.

In-plane instability†. Limit state involving buckling in the plane of the frame or the member.

Instability†. Limit state reached in the loading of a structural component, frame or structure in which a slight disturbance in the loads or geometry produces large displacements.

Introduction length. The length along which the required longitudinal shear force is assumed to be transferred into or out of the steel shape in an encased or filled composite column.

Joint†. Area where two or more ends, surfaces, or edges are attached. Categorized by type of fastener or weld used and method of force transfer.

Joint eccentricity. In an HSS truss connection, perpendicular distance from chord member center-of-gravity to intersection of branch member work points.

k-area. The region of the web that extends from the tangent point of the web and the flange-web fillet (AISC k dimension) a distance $1^1/_2$ in. (38 mm) into the web beyond the k dimension.

K-connection. HSS connection in which forces in branch members or connecting elements transverse to the main member are primarily equilibriated by forces in other branch members or connecting elements on the same side of the main member.

Lacing. Plate, angle or other steel shape, in a lattice configuration, that connects two steel shapes together.

Lap joint. Joint between two overlapping connection elements in parallel planes.

Lateral bracing. Member or system that is designed to inhibit lateral buckling or lateral-torsional buckling of structural members.

Lateral force-resisting system. Structural system designed to resist lateral loads and provide stability for the structure as a whole.

Lateral load. Load acting in a lateral direction, such as wind or earthquake effects.

Lateral-torsional buckling†. Buckling mode of a flexural member involving deflection out of the plane of bending occurring simultaneously with twist about the shear center of the cross section.

Leaning column. Column designed to carry gravity loads only, with connections that are not intended to provide resistance to lateral loads.

Length effects. Consideration of the reduction in strength of a member based on its unbraced length.

Lightweight concrete. Structural concrete with an equilibrium density of 115 lb/ft^3 (1 840 kg/m^3) or less, as determined by ASTM C567.

Limit state†. Condition in which a structure or component becomes unfit for service and is judged either to be no longer useful for its intended function (serviceability limit state) or to have reached its ultimate load-carrying capacity (strength limit state).

Load†. Force or other action that results from the weight of building materials, occupants and their possessions, environmental effects, differential movement, or restrained dimensional changes.

Load effect†. Forces, stresses and deformations produced in a structural component by the applied loads.

Load factor. Factor that accounts for deviations of the nominal load from the actual load, for uncertainties in the analysis that transforms the load into a load effect and for the probability that more than one extreme load will occur simultaneously.

Load transfer region. Region of a composite member over which force is directly applied to the member, such as the depth of a connection plate.

*Local bending***†. Limit state of large deformation of a flange under a concentrated transverse force.

*Local buckling***. Limit state of buckling of a compression element within a cross section.

*Local yielding***†. Yielding that occurs in a local area of an element.

LRFD (load and resistance factor design)†. Method of proportioning structural components such that the design strength equals or exceeds the required strength of the component under the action of the LRFD load combinations.

LRFD load combination†. Load combination in the applicable building code intended for strength design (load and resistance factor design).

Main member. In an HSS connection, chord member, column or other HSS member to which branch members or other connecting elements are attached.

Member imperfection. Initial displacement of points along the length of individual members (between points of intersection of members) from their nominal locations, such as the out-of-straightness of members due to manufacturing and fabrication.

Mill scale. Oxide surface coating on steel formed by the hot rolling process.

Moment connection. Connection that transmits bending moment between connected members.

Moment frame†. Framing system that provides resistance to lateral loads and provides stability to the structural system, primarily by shear and flexure of the framing members and their connections.

Negative flexural strength. Flexural strength of a composite beam in regions with tension due to flexure on the top surface.

Net area. Gross area reduced to account for removed material.

Nominal dimension. Designated or theoretical dimension, as in tables of section properties.

Nominal load†. Magnitude of the load specified by the applicable building code.

Nominal rib height. In a formed steel deck, height of deck measured from the underside of the lowest point to the top of the highest point.

*Nominal strength**†. Strength of a structure or component (without the resistance factor or safety factor applied) to resist load effects, as determined in accordance with this Specification.

Noncompact section. Section that is able to develop the yield stress in its compression elements before local buckling occurs, but is unable to develop a rotation capacity of three.

Nondestructive testing. Inspection procedure wherein no material is destroyed and the integrity of the material or component is not affected.

Notch toughness. Energy absorbed at a specified temperature as measured in the Charpy V-notch impact test.

Notional load. Virtual load applied in a structural analysis to account for destabilizing effects that are not otherwise accounted for in the design provisions.

Out-of-plane buckling†. Limit state of a beam, column or beam-column involving lateral or lateral-torsional buckling.

Overlapped connection. HSS truss connection in which intersecting branch members overlap.

Panel brace. Brace that controls the relative movement of two adjacent brace points along the length of a beam or column or the relative lateral displacement of two stories in a frame (see *point brace*).

Panel zone. Web area of beam-to-column connection delineated by the extension of beam and column flanges through the connection, transmitting moment through a shear panel.

Partial-joint-penetration (PJP) groove weld. Groove weld in which the penetration is intentionally less than the complete thickness of the connected element.

Partially restrained moment connection. Connection capable of transferring moment with rotation between connected members that is not negligible.

Percent elongation. Measure of ductility, determined in a tensile test as the maximum elongation of the gage length divided by the original gage length expressed as a percentage.

Pipe. See *HSS*.

Pitch. Longitudinal center-to-center spacing of fasteners. Center-to-center spacing of bolt threads along axis of bolt.

Plastic analysis. Structural analysis based on the assumption of rigid-plastic behavior, that is, that equilibrium is satisfied and the stress is at or below the yield stress throughout the structure.

Plastic hinge. Fully yielded zone that forms in a structural member when the plastic moment is attained.

Plastic moment. Theoretical resisting moment developed within a fully yielded cross section.

Plastic stress distribution method. In a composite member, method for determining stresses assuming that the steel section and the concrete in the cross section are fully plastic.

Plastification. In an HSS connection, limit state based on an out-of-plane flexural yield line mechanism in the chord at a branch member connection.

Plate girder. Built-up beam.

Plug weld. Weld made in a circular hole in one element of a joint fusing that element to another element.

Point brace. Brace that prevents lateral movement or twist independently of other braces at adjacent brace points (see *panel brace*).

Ponding. Retention of water due solely to the deflection of flat roof framing.

Positive flexural strength. Flexural strength of a composite beam in regions with compression due to flexure on the top surface.

Pretensioned bolt. Bolt tightened to the specified minimum pretension.

Pretensioned joint. Joint with high-strength bolts tightened to the specified minimum pretension.

Properly developed. Reinforcing bars detailed to yield in a ductile manner before crushing of the concrete occurs. Bars meeting the provisions of ACI 318, insofar as development length, spacing and cover are deemed to be properly developed.

Prying action. Amplification of the tension force in a bolt caused by leverage between the point of applied load, the bolt, and the reaction of the connected elements.

Punching load. In an HSS connection, component of branch member force perpendicular to a chord.

P-δ effect. Effect of loads acting on the deflected shape of a member between joints or nodes.

P-Δ effect. Effect of loads acting on the displaced location of joints or nodes in a structure. In tiered building structures, this is the effect of loads acting on the laterally displaced location of floors and roofs.

Quality assurance. Monitoring and inspection tasks to ensure that the material provided and work performed by the fabricator and erector meet the requirements of the approved construction documents and referenced standards. Quality assurance includes those tasks designated "special inspection" by the applicable building code.

Quality assurance inspector (QAI). Individual designated to provide quality assurance inspection for the work being performed.

Quality assurance plan (QAP). Program in which the agency or firm responsible for quality assurance maintains detailed monitoring and inspection procedures to ensure conformance with the approved construction documents and referenced standards.

Quality control. Controls and inspections implemented by the fabricator or erector, as applicable, to ensure that the material provided and work performed meet the requirements of the approved construction documents and referenced standards.

Quality control inspector (QCI). Individual designated to perform quality control inspection tasks for the work being performed.

Quality control program (QCP). Program in which the fabricator or erector, as applicable, maintains detailed fabrication or erection and inspection procedures to ensure conformance with the approved design drawings, specifications, and referenced standards.

Reentrant. In a cope or weld access hole, a cut at an abrupt change in direction in which the exposed surface is concave.

*Required strength**†. Forces, stresses and deformations acting on a structural component, determined by either structural analysis, for the LRFD or ASD load combinations, as applicable, or as specified by this Specification or Standard.

Resistance factor, ϕ†. Factor that accounts for unavoidable deviations of the nominal strength from the actual strength and for the manner and consequences of failure.

Restrained construction. Floor and roof assemblies and individual beams in buildings where the surrounding or supporting structure is capable of resisting significant thermal expansion throughout the range of anticipated elevated temperatures.

Reverse curvature. See *double curvature*.

Root of joint. Portion of a joint to be welded where the members are closest to each other.

Rotation capacity. Incremental angular rotation defined as the ratio of the inelastic rotation attained to the idealized elastic rotation at first yield prior to significant load shedding.

Rupture strength†. Strength limited by breaking or tearing of members or connecting elements.

Safety factor, Ω†. Factor that accounts for deviations of the actual strength from the nominal strength, deviations of the actual load from the nominal load, uncertainties in the analysis that transforms the load into a load effect, and for the manner and consequences of failure.

Second-order effect. Effect of loads acting on the deformed configuration of a structure; includes P-δ effect and P-Δ effect.

Seismic force-resisting system. That part of the structural system that has been considered in the design to provide the required resistance to the seismic forces prescribed in ASCE/SEI 7.

Seismic response modification factor. Factor that reduces seismic load effects to strength level.

Service load combination. Load combination under which serviceability limit states are evaluated.

Service load†. Load under which serviceability limit states are evaluated.

Serviceability limit state†. Limiting condition affecting the ability of a structure to preserve its appearance, maintainability, durability, comfort of its occupants, or function of machinery, under typical usage.

Shear buckling†. Buckling mode in which a plate element, such as the web of a beam, deforms under pure shear applied in the plane of the plate.

Shear lag. Nonuniform tensile stress distribution in a member or connecting element in the vicinity of a connection.

Shear wall†. Wall that provides resistance to lateral loads in the plane of the wall and provides stability for the structural system.

Shear yielding (punching). In an HSS connection, limit state based on out-of-plane shear strength of the chord wall to which branch members are attached.

Sheet steel. In a composite floor system, steel used for closure plates or miscellaneous trimming in a formed steel deck.

Shim. Thin layer of material used to fill a space between faying or bearing surfaces.

Sidesway buckling (frame). Stability limit state involving lateral sidesway instability of a frame.

Simple connection. Connection that transmits negligible bending moment between connected members.

Single-concentrated force. Tensile or compressive force applied normal to the flange of a member.

Single curvature. Deformed shape of a beam with no inflection point within the span.

Slender-element section. Cross section possessing plate components of sufficient slenderness such that local buckling in the elastic range will occur.

Slip. In a bolted connection, limit state of relative motion of connected parts prior to the attainment of the available strength of the connection.

Slip-critical connection. Bolted connection designed to resist movement by friction on the faying surface of the connection under the clamping force of the bolts.

Slot weld. Weld made in an elongated hole fusing an element to another element.

Snug-tightened joint. Joint with the connected plies in firm contact as specified in Chapter J.

Specifications. Written documents containing the requirements for materials, standards and workmanship.

Specified minimum tensile strength. Lower limit of tensile strength specified for a material as defined by ASTM.

Specified minimum yield stress†. Lower limit of yield stress specified for a material as defined by ASTM.

Splice. Connection between two structural elements joined at their ends to form a single, longer element.

Stability. Condition in the loading of a structural component, frame or structure in which a slight disturbance in the loads or geometry does not produce large displacements.

Steel anchor. Headed stud or hot rolled channel welded to a steel member and embodied in concrete of a composite member to transmit shear, tension, or a combination of shear and tension at the interface of the two materials.

Stiffened element. Flat compression element with adjoining out-of-plane elements along both edges parallel to the direction of loading.

Stiffener. Structural element, typically an angle or plate, attached to a member to distribute load, transfer shear or prevent buckling.

Stiffness. Resistance to deformation of a member or structure, measured by the ratio of the applied force (or moment) to the corresponding displacement (or rotation).

Story drift. Horizontal deflection at the top of the story relative to the bottom of the story.

Story drift ratio. Story drift divided by the story height.

Strain compatibility method. In a composite member, method for determining the stresses considering the stress-strain relationships of each material and its location with respect to the neutral axis of the cross section.

Strength limit state†. Limiting condition in which the maximum strength of a structure or its components is reached.

Stress. Force per unit area caused by axial force, moment, shear or torsion.

Stress concentration. Localized stress considerably higher than average due to abrupt changes in geometry or localized loading.

Strong axis. Major principal centroidal axis of a cross section.

Structural analysis†. Determination of load effects on members and connections based on principles of structural mechanics.

Structural component†. Member, connector, connecting element or assemblage.

Structural Integrity. Performance characteristic of a structure indicating resistance to catastrophic failure.

Structural steel. Steel elements as defined in the AISC *Code of Standard Practice for Steel Buildings and Bridges* Section 2.1.

Structural system. An assemblage of load-carrying components that are joined together to provide interaction or interdependence.

System imperfection. Initial displacement of points of intersection of members from their nominal locations, such as the out-of-plumbness of columns due to erection tolerances.

T-connection. HSS connection in which the branch member or connecting element is perpendicular to the main member and in which forces transverse to the main member are primarily equilibrated by shear in the main member.

Tensile strength (of material)†. Maximum tensile stress that a material is capable of sustaining as defined by ASTM.

Tensile strength (of member). Maximum tension force that a member is capable of sustaining.

Tension and shear rupture†. In a bolt or other type of mechanical fastener, limit state of rupture due to simultaneous tension and shear force.

Tension field action. Behavior of a panel under shear in which diagonal tensile forces develop in the web and compressive forces develop in the transverse stiffeners in a manner similar to a Pratt truss.

Thermally cut. Cut with gas, plasma or laser.

Tie plate. Plate element used to join two parallel components of a built-up column, girder or strut rigidly connected to the parallel components and designed to transmit shear between them.

Toe of fillet. Junction of a fillet weld face and base metal. Tangent point of a fillet in a rolled shape.

Torsional bracing. Bracing resisting twist of a beam or column.

Torsional buckling†. Buckling mode in which a compression member twists about its shear center axis.

Transverse reinforcement. In an encased composite column, steel reinforcement in the form of closed ties or welded wire fabric providing confinement for the concrete surrounding the steel shape.

Transverse stiffener. Web stiffener oriented perpendicular to the flanges, attached to the web.

Tubing. See *HSS*.

Turn-of-nut method. Procedure whereby the specified pretension in high-strength bolts is controlled by rotating the fastener component a predetermined amount after the bolt has been snug tightened.

Unbraced length. Distance between braced points of a member, measured between the centers of gravity of the bracing members.

Uneven load distribution. In an HSS connection, condition in which the stress is not distributed uniformly through the cross section of connected elements.

Unframed end. The end of a member not restrained against rotation by stiffeners or connection elements.

Unstiffened element. Flat compression element with an adjoining out-of-plane element along one edge parallel to the direction of loading.

Unrestrained construction. Floor and roof assemblies and individual beams in buildings that are assumed to be free to rotate and expand throughout the range of anticipated elevated temperatures.

Weak axis. Minor principal centroidal axis of a cross section.

Weathering steel. High-strength, low-alloy steel that, with sufficient precautions, is able to be used in typical atmospheric exposures (not marine) without protective paint coating.

Web local crippling†. Limit state of local failure of web plate in the immediate vicinity of a concentrated load or reaction.

Web sidesway buckling. Limit state of lateral buckling of the tension flange opposite the location of a concentrated compression force.

Weld metal. Portion of a fusion weld that has been completely melted during welding. Weld metal has elements of filler metal and base metal melted in the weld thermal cycle.

Weld root. See *root of joint*.

Y-connection. HSS connection in which the branch member or connecting element is not perpendicular to the main member and in which forces transverse to the main member are primarily equilibrated by shear in the main member.

Yield moment†. In a member subjected to bending, the moment at which the extreme outer fiber first attains the yield stress.

Yield point†. First stress in a material at which an increase in strain occurs without an increase in stress as defined by ASTM.

Yield strength†. Stress at which a material exhibits a specified limiting deviation from the proportionality of stress to strain as defined by ASTM.

Yield stress†. Generic term to denote either yield point or yield strength, as applicable for the material.

Yielding†. Limit state of inelastic deformation that occurs when the yield stress is reached.

Yielding (plastic moment)†. Yielding throughout the cross section of a member as the bending moment reaches the plastic moment.

Yielding (yield moment)†. Yielding at the extreme fiber on the cross section of a member when the bending moment reaches the yield moment.

ABBREVIATIONS

The following abbreviations appear in this Specification. The abbreviations are written out where they first appear within a Section.

ACI (American Concrete Institute)

AHJ (authority having jurisdiction)

AISC (American Institute of Steel Construction)

AISI (American Iron and Steel Institute)

ANSI (American National Standards Institute)

ASCE (American Society of Civil Engineers)

ASD (allowable strength design)

ASME (American Society of Mechanical Engineers)

ASNT (American Society for Nondestructive Testing)

AWI (associate welding inspector)

AWS (American Welding Society)

CJP (complete joint penetration)

CVN (Charpy V-notch)

EOR (engineer of record)

ERW (electric resistance welded)

FCAW (flux cored arc welding)

FR (fully restrained)

GMAW (gas metal arc welding)

HSLA (high-strength low-alloy)

HSS (hollow structural section)

LRFD (load and resistance factor design)

MT (magnetic particle testing)

NDT (nondestructive testing)

OSHA (Occupational Safety and Health Administration)

PJP (partial joint penetration)

PQR (procedure qualification record)

PR (partially restrained)

PT (penetrant testing)

QA (quality assurance)

QAI (quality assurance inspector)

QAP (quality assurance plan)

QC (quality control)

QCI (quality control inspector)

QCP (quality control program)

RCSC (Research Council on Structural Connections)

RT (radiographic testing)

SAW (submerged arc welding)

SEI (Structural Engineering Institute)

SFPE (Society of Fire Protection Engineers)

SMAW (shielded metal arc welding)

SWI (senior welding inspector)

UNC (Unified National Coarse)

UT (ultrasonic testing)

WI (welding inspector)

WPQR (welder performance qualification records)

WPS (welding procedure specification)

CHAPTER A

GENERAL PROVISIONS

This chapter states the scope of this Specification, lists referenced specifications, codes and standards, and provides requirements for materials and structural design documents.

The chapter is organized as follows:

A1. SCOPE

The *Specification for Structural Steel Buildings* (ANSI/AISC 360), hereafter referred to as this Specification, shall apply to the design, fabrication and erection of the structural steel system or systems with structural steel acting compositely with reinforced concrete, where the steel elements are defined in Section 2.1 of the AISC *Code of Standard Practice for Steel Buildings and Bridges* (ANSI/AISC 303), hereafter referred to as the *Code of Standard Practice*.

This Specification includes the Symbols, the Glossary, Abbreviations, Chapters A through N, and Appendices 1 through 8. The Commentary to this Specification and the User Notes interspersed throughout are not part of this Specification. The phrases “is permitted” and “are permitted” in this document identify provisions that comply with this Specification, but are not mandatory.

User Note: User notes are intended to provide concise and practical guidance in the application of the Specification provisions.

This Specification sets forth criteria for the design, fabrication and erection of structural steel buildings and other structures, where other structures are defined as structures designed, fabricated and erected in a manner similar to buildings, with building-like vertical and lateral load-resisting elements.

Wherever this Specification refers to the applicable building code and there is none, the loads, load combinations, system limitations, and general design requirements shall be those in ASCE *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE/SEI 7).

Where conditions are not covered by this Specification, designs are permitted to be based on tests or analysis, subject to the approval of the authority having jurisdiction. Alternative methods of analysis and design are permitted, provided such alternative methods or criteria are acceptable to the authority having jurisdiction.

User Note: For the design of cold-formed steel structural members, the provisions in the AISI *North American Specification for the Design of Cold-Formed Steel Structural Members* (AISI S100) are recommended, except for cold-formed hollow structural sections (HSS), which are designed in accordance with this Specification.

1. Seismic Applications

The AISC *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341) shall apply to the design of seismic force-resisting systems of structural steel or of structural steel acting compositely with reinforced concrete, unless specifically exempted by the applicable building code.

User Note: ASCE/SEI 7 (Table 12.2-1, Item H) specifically exempts structural steel systems in seismic design categories B and C from the requirements in the AISC *Seismic Provisions for Structural Steel Buildings* if they are designed according to this Specification and the seismic loads are computed using a seismic response modification factor, R, of 3; composite systems are not covered by this exemption. The *Seismic Provisions for Structural Steel Buildings* do not apply in seismic design category A.

2. Nuclear Applications

The design, fabrication and erection of nuclear structures shall comply with the provisions of this Specification as modified by the requirements of the AISC *Specification for Safety-Related Steel Structures for Nuclear Facilities* (ANSI/AISC N690).

A2. REFERENCED SPECIFICATIONS, CODES AND STANDARDS

The following specifications, codes and standards are referenced in this Specification:

(a) American Concrete Institute (ACI)

ACI 318-14 *Building Code Requirements for Structural Concrete and Commentary*

ACI 318M-14 *Metric Building Code Requirements for Structural Concrete and Commentary*

ACI 349-13 *Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary*

ACI 349M-13 *Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary (Metric)*

(b) American Institute of Steel Construction (AISC)

ANSI/AISC 303-16 *Code of Standard Practice for Steel Buildings and Bridges*

ANSI/AISC 341-16 *Seismic Provisions for Structural Steel Buildings*

ANSI/AISC N690-12 *Specification for Safety-Related Steel Structures for Nuclear Facilities*

ANSI/AISC N690s1-15 *Specification for Safety-Related Steel Structures for Nuclear Facilities, Supplement No. 1*

(c) American Society of Civil Engineers (ASCE)

ASCE/SEI 7-16 *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*

ASCE/SEI/SFPE 29-05 *Standard Calculation Methods for Structural Fire Protection*

(d) American Society of Mechanical Engineers (ASME)

ASME B18.2.6-10 *Fasteners for Use in Structural Applications*

ASME B46.1-09 *Surface Texture, Surface Roughness, Waviness, and Lay*

(e) American Society for Nondestructive Testing (ASNT)

ANSI/ASNT CP-189-2011 *Standard for Qualification and Certification of Nondestructive Testing Personnel*

Recommended Practice No. SNT-TC-1A-2011 *Personnel Qualification and Certification in Nondestructive Testing*

(f) ASTM International (ASTM)

A6/A6M-14 *Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling*

A36/A36M-14 *Standard Specification for Carbon Structural Steel*

A53/A53M-12 *Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless*

A193/A193M-15 *Standard Specification for Alloy-Steel and Stainless Steel Bolting Materials for High Temperature or High Pressure Service and Other Special Purpose Applications*

A194/A194M-15 *Standard Specification for Carbon Steel, Alloy Steel, and Stainless Steel Nuts for Bolts for High Pressure or High Temperature Service, or Both*

A216/A216M-14e1 *Standard Specification for Steel Castings, Carbon, Suitable for Fusion Welding, for High-Temperature Service*

A242/A242M-13 *Standard Specification for High-Strength Low-Alloy Structural Steel*

A283/A283M-13 *Standard Specification for Low and Intermediate Tensile Strength Carbon Steel Plates*

A307-14 *Standard Specification for Carbon Steel Bolts, Studs, and Threaded Rod, 60,000 PSI Tensile Strength*

User Note: ASTM A325/A325M are now included as a Grade within ASTM F3125.

A354-11 *Standard Specification for Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners*

A370-15 *Standard Test Methods and Definitions for Mechanical Testing of Steel Products*

A449-14 *Standard Specification for Hex Cap Screws, Bolts and Studs, Steel, Heat Treated, 120/105/90 ksi Minimum Tensile Strength, General Use*

User Note: ASTM A490/A490M are now included as a Grade within ASTM F3125.

A500/A500M-13 *Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes*

A501/A501M-14 *Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing*

A502-03 (2015) *Standard Specification for Rivets, Steel, Structural*

A514/A514M-14 *Standard Specification for High-Yield-Strength, Quenched and Tempered Alloy Steel Plate, Suitable for Welding*

A529/A529M-14 *Standard Specification for High-Strength Carbon-Manganese Steel of Structural Quality*

A563-15 *Standard Specification for Carbon and Alloy Steel Nuts*

A563M-07(2013) *Standard Specification for Carbon and Alloy Steel Nuts (Metric)*

A568/A568M-15 *Standard Specification for Steel, Sheet, Carbon, Structural, and High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, General Requirements for*

A572/A572M-15 *Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel*

A588/A588M-15 *Standard Specification for High-Strength Low-Alloy Structural Steel, up to 50 ksi [345 MPa] Minimum Yield Point, with Atmospheric Corrosion Resistance*

A606/A606M-15 *Standard Specification for Steel, Sheet and Strip, High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance*

A618/A618M-04(2015) *Standard Specification for Hot-Formed Welded and Seamless High-Strength Low-Alloy Structural Tubing*

A668/A668M-15 *Standard Specification for Steel Forgings, Carbon and Alloy, for General Industrial Use*

A673/A673M-07(2012) *Standard Specification for Sampling Procedure for Impact Testing of Structural Steel*

A709/A709M-13a *Standard Specification for Structural Steel for Bridges*

A751-14a *Standard Test Methods, Practices, and Terminology for Chemical Analysis of Steel Products*

A847/A847M-14 *Standard Specification for Cold-Formed Welded and Seamless High-Strength, Low-Alloy Structural Tubing with Improved Atmospheric Corrosion Resistance*

A913/A913M-15 *Standard Specification for High-Strength Low-Alloy Steel Shapes of Structural Quality, Produced by Quenching and Self-Tempering Process (QST)*

A992/A992M-11(2015) *Standard Specification for Structural Steel Shapes*

A1011/A1011M-14 *Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy, High-Strength Low-Alloy with Improved Formability, and Ultra-High Strength*

A1043/A1043M-14 *Standard Specification for Structural Steel with Low Yield to Tensile Ratio for Use in Buildings*

A1065/A1065M-15 *Standard Specification for Cold-Formed Electric-Fusion (Arc) Welded High-Strength Low-Alloy Structural Tubing in Shapes, with 50 ksi [345 MPa] Minimum Yield Point*

A1066/A1066M-11(2015)e1 *Standard Specification for High-Strength Low-Alloy Structural Steel Plate Produced by Thermo-Mechanical Controlled Process (TMCP)*

A1085/A1085M-13 *Standard Specification for Cold-Formed Welded Carbon Steel Hollow Structural Sections (HSS)*

C567/C567M-14 *Standard Test Method for Determining Density of Structural Lightweight Concrete*

E119-15 *Standard Test Methods for Fire Tests of Building Construction and Materials*

E165/E165M-12 *Standard Practice for Liquid Penetrant Examination for General Industry*

E709-15 *Standard Guide for Magnetic Particle Examination*

F436-11 *Standard Specification for Hardened Steel Washers*

F436M-11 *Standard Specification for Hardened Steel Washers (Metric)*

F606/F606M-14a *Standard Test Methods for Determining the Mechanical Properties of Externally and Internally Threaded Fasteners, Washers, Direct Tension Indicators, and Rivets*

F844-07a(2013) *Standard Specification for Washers, Steel, Plain (Flat), Unhardened for General Use*

F959-15 *Standard Specification for Compressible-Washer-Type Direct Tension Indicators for Use with Structural Fasteners*

F959M-13 *Standard Specification for Compressible-Washer-Type Direct Tension Indicators for Use with Structural Fasteners (Metric)*

F1554-15 *Standard Specification for Anchor Bolts, Steel, 36, 55, and 105-ksi Yield Strength*

User Note: ASTM F1554 is the most commonly referenced specification for anchor rods. Grade and weldability must be specified.

User Note: ASTM F1852 and F2280 are now included as Grades within ASTM F3125.

F3043-14e1 *Standard Specification for "Twist Off" Type Tension Control Structural Bolt/Nut/Washer Assemblies, Alloy Steel, Heat Treated, 200 ksi Minimum Tensile Strength*

F3111-14 *Standard Specification for Heavy Hex Structural Bolt/Nut/Washer Assemblies, Alloy Steel, Heat Treated, 200 ksi Minimum Tensile Strength*

F3125/F3125M-15 *Standard Specification for High Strength Structural Bolts, Steel and Alloy Steel, Heat Treated, 120 ksi (830 MPa) and 150 ksi (1040 MPa) Minimum Tensile Strength, Inch and Metric Dimensions*

(g) American Welding Society (AWS)

AWS A5.1/A5.1M:2012 *Specification for Carbon Steel Electrodes for Shielded Metal Arc Welding*

AWS A5.5/A5.5M:2014 *Specification for Low-Alloy Steel Electrodes for Shielded Metal Arc Welding*

AWS A5.17/A5.17M:1997 (R2007) *Specification for Carbon Steel Electrodes and Fluxes for Submerged Arc Welding*

AWS A5.18/A5.18M:2005 *Specification for Carbon Steel Electrodes and Rods for Gas Shielded Arc Welding*

AWS A5.20/A5.20M:2005 (R2015) *Specification for Carbon Steel Electrodes for Flux Cored Arc Welding*

AWS A5.23/A5.23M:2011 *Specification for Low-Alloy Steel Electrodes and Fluxes for Submerged Arc Welding*

AWS A5.25/A5.25M:1997 (R2009) *Specification for Carbon and Low-Alloy Steel Electrodes and Fluxes for Electroslag Welding*

AWS A5.26/A5.26M:1997 (R2009) *Specification for Carbon and Low-Alloy Steel Electrodes for Electrogas Welding*

AWS A5.28/A5.28M:2005 (R2015) *Specification for Low-Alloy Steel Electrodes and Rods for Gas Shielded Arc Welding*

AWS A5.29/A5.29M:2010 *Specification for Low-Alloy Steel Electrodes for Flux Cored Arc Welding*

AWS A5.32/A5.32M:2011 *Welding Consumables—Gases and Gas Mixtures for Fusion Welding and Allied Processes*

AWS A5.36/A5.36M:2012 *Specification for Carbon and Low-Alloy Steel Flux Cored Electrodes for Flux Cored Arc Welding and Metal Cored Electrodes for Gas Metal Arc Welding*

AWS B5.1:2013-AMD1 *Specification for the Qualification of Welding Inspectors*

AWS D1.1/D1.1M:2015 *Structural Welding Code—Steel*

AWS D1.3/D1.3M:2008 *Structural Welding Code—Sheet Steel*

(h) Research Council on Structural Connections (RCSC)
Specification for Structural Joints Using High-Strength Bolts, 2014

(i) Steel Deck Institute (SDI)
ANSI/SDI QA/QC-2011 *Standard for Quality Control and Quality Assurance for Installation of Steel Deck*

A3. MATERIAL

1. Structural Steel Materials

Material test reports or reports of tests made by the fabricator or a testing laboratory shall constitute sufficient evidence of conformity with one of the ASTM standards listed in Section A3.1a. For hot-rolled structural shapes, plates, and bars, such tests shall be made in accordance with ASTM A6/A6M; for sheets, such tests shall be made in accordance with ASTM A568/A568M; for tubing and pipe, such tests shall be made in accordance with the requirements of the applicable ASTM standards listed above for those product forms.

1a. ASTM Designations

Structural steel material conforming to one of the following ASTM specifications is approved for use under this Specification:

(a) Hot-rolled structural shapes

ASTM A36/A36M	ASTM A709/A709M
ASTM A529/A529M	ASTM A913/A913M
ASTM A572/A572M	ASTM A992/ A992M
ASTM A588/A588M	ASTM A1043/A1043M

(b) Hollow structural sections (HSS)

ASTM A53/A53M Grade B	ASTM A847/A847M
ASTM A500/A500M	ASTM A1065/A1065M
ASTM A501/A501M	ASTM A1085/A1085M
ASTM A618/A618M	

(c) Plates

ASTM A36/A36M	ASTM A572/A572M
ASTM A242/A242M	ASTM A588/A588M
ASTM A283/A283M	ASTM A709/A709M
ASTM A514/A514M	ASTM A1043/A1043M
ASTM A529/A529M	ASTM A1066/A1066M

(d) Bars

ASTM A36/A36M	ASTM A572/A572M
ASTM A529/A529M	ASTM A709/A709M

(e) Sheets

ASTM A606/A606M
ASTM A1011/A1011M SS, HSLAS, AND HSLAS-F

1b. Unidentified Steel

Unidentified steel, free of injurious defects, is permitted to be used only for members or details whose failure will not reduce the strength of the structure, either locally or overall. Such use shall be subject to the approval of the engineer of record.

User Note: Unidentified steel may be used for details where the precise mechanical properties and weldability are not of concern. These are commonly curb plates, shims and other similar pieces.

1c. Rolled Heavy Shapes

ASTM A6/A6M hot-rolled shapes with a flange thickness exceeding 2 in. (50 mm) are considered to be rolled heavy shapes. Rolled heavy shapes used as members subject to primary (computed) tensile forces due to tension or flexure and spliced or connected using complete-joint-penetration groove welds that fuse through the thickness of the flange or the flange and the web, shall be specified as follows. The structural design documents shall require that such shapes be supplied with Charpy V-notch (CVN) impact test results in accordance with ASTM A6/A6M, Supplementary Requirement S30, Charpy V-Notch Impact Test for Structural Shapes—Alternate Core Location. The impact test shall meet a minimum average value of 20 ft-lb (27 J) absorbed energy at a maximum temperature of +70°F (+21°C).

The requirements in this section do not apply if the splices and connections are made by bolting. Where a rolled heavy shape is welded to the surface of another shape using groove welds, the requirements apply only to the shape that has weld metal fused through the cross section.

> **User Note:** Additional requirements for rolled heavy-shape welded joints are given in Sections J1.5, J1.6, J2.6 and M2.2.

1d. Built-Up Heavy Shapes

Built-up cross sections consisting of plates with a thickness exceeding 2 in. (50 mm) are considered built-up heavy shapes. Built-up heavy shapes used as members subject to primary (computed) tensile forces due to tension or flexure and spliced or connected to other members using complete-joint-penetration groove welds that fuse through the thickness of the plates, shall be specified as follows. The structural design documents shall require that the steel be supplied with Charpy V-notch impact test results in accordance with ASTM A6/A6M, Supplementary Requirement S5, Charpy V-Notch Impact Test. The impact test shall be conducted in accordance with ASTM A673/A673M, Frequency P, and shall meet a minimum average value of 20 ft-lb (27 J) absorbed energy at a maximum temperature of +70°F (+21°C).

When a built-up heavy shape is welded to the face of another member using groove welds, these requirements apply only to the shape that has weld metal fused through the cross section.

> **User Note:** Additional requirements for built-up heavy-shape welded joints are given in Sections J1.5, J1.6, J2.6 and M2.2.

2. Steel Castings and Forgings

Steel castings and forgings shall conform to an ASTM standard intended for structural applications and shall provide strength, ductility, weldability and toughness adequate for the purpose. Test reports produced in accordance with the ASTM reference standards shall constitute sufficient evidence of conformity with such standards.

3. Bolts, Washers and Nuts

Bolt, washer and nut material conforming to one of the following ASTM specifications is approved for use under this Specification:

> **User Note:** ASTM F3125 is an umbrella standard that incorporates Grades A325, A325M, A490, A490M, F1852 and F2280, which were previously separate standards.

(a) Bolts

ASTM A307
ASTM A354
ASTM A449
ASTM F3043
ASTM F3111
ASTM F3125/F3125M

(b) Nuts

ASTM A194/A194M	ASTM A563M
ASTM A563	

(c) Washers

ASTM F436	ASTM F844
ASTM F436M	

(d) Compressible-Washer-Type Direct Tension Indicators

ASTM F959	ASTM F959M

Manufacturer's certification shall constitute sufficient evidence of conformity with the standards.

4. Anchor Rods and Threaded Rods

Anchor rod and threaded rod material conforming to one of the following ASTM specifications is approved for use under this Specification:

ASTM A36/A36M	ASTM A572/A572M
ASTM A193/A193M	ASTM A588/A588M
ASTM A354	ASTM F1554
ASTM A449	

User Note: ASTM F1554 is the preferred material specification for anchor rods.

ASTM A449 material is permitted for high-strength anchor rods and threaded rods of any diameter.

Threads on anchor rods and threaded rods shall conform to the Unified Standard Series of ASME B18.2.6 and shall have Class 2A tolerances.

Manufacturer's certification shall constitute sufficient evidence of conformity with the standards.

5. Consumables for Welding

Filler metals and fluxes shall conform to one of the following specifications of the American Welding Society:

AWS A5.1/A5.1M	AWS A5.25/A5.25M
AWS A5.5/A5.5M	AWS A5.26/A5.26M
AWS A5.17/A5.17M	AWS A5.28/A5.28M
AWS A5.18/A5.18M	AWS A5.29/A5.29M
AWS A5.20/A5.20M	AWS A5.32/A5.32M
AWS A5.23/A5.23M	AWS A5.36/A5.36M

Manufacturer's certification shall constitute sufficient evidence of conformity with the standards.

6. Headed Stud Anchors

Steel headed stud anchors shall conform to the requirements of the *Structural Welding Code—Steel* (AWS D1.1/D1.1M).

Manufacturer's certification shall constitute sufficient evidence of conformity with AWS D1.1/D1.1M.

A4. STRUCTURAL DESIGN DRAWINGS AND SPECIFICATIONS

The structural design drawings and specifications shall meet the requirements of the *Code of Standard Practice*.

User Note: The *Code of Standard Practice* uses the term "design documents" in place of "design drawings" to generalize the term and to reflect both paper drawings and electronic models. Similarly, "fabrication documents" is used in place of "shop drawings," and "erection documents" is used in place of "erection drawings." The use of "drawings" in this standard is not intended to create a conflict.

User Note: Provisions in this Specification contain information that is to be shown on design drawings. These include:

- Section A3.1c: Rolled heavy shapes where alternate core Charpy V-notch toughness (CVN) is required
- Section A3.1d: Built-up heavy shapes where CVN toughness is required
- Section J3.1: Locations of connections using pretensioned bolts

Other information needed by the fabricator or erector should be shown on design drawings, including:

- Fatigue details requiring nondestructive testing
- Risk category (Chapter N)
- Indication of complete-joint-penetration (CJP) groove welds subject to tension (Chapter N)

CHAPTER B

DESIGN REQUIREMENTS

This chapter addresses general requirements for the design of steel structures applicable to all chapters of this Specification.

The chapter is organized as follows:

B1. GENERAL PROVISIONS

The design of members and connections shall be consistent with the intended behavior of the framing system and the assumptions made in the structural analysis.

B2. LOADS AND LOAD COMBINATIONS

The loads, nominal loads and load combinations shall be those stipulated by the applicable building code. In the absence of a building code, the loads, nominal loads and load combinations shall be those stipulated in *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE/SEI 7).

> **User Note:** When using ASCE/SEI 7 for design according to Section B3.1 (LRFD), the load combinations in ASCE/SEI 7 Section 2.3 apply. For design according to Section B3.2 (ASD), the load combinations in ASCE/SEI 7 Section 2.4 apply.

B3. DESIGN BASIS

Design shall be such that no applicable strength or serviceability limit state shall be exceeded when the structure is subjected to all applicable load combinations.

Design for strength shall be performed according to the provisions for load and resistance factor design (LRFD) or to the provisions for allowable strength design (ASD).

> **User Note:** The term “design”, as used in this Specification, is defined in the Glossary.

1. Design for Strength Using Load and Resistance Factor Design (LRFD)

Design according to the provisions for load and resistance factor design (LRFD) satisfies the requirements of this Specification when the design strength of each structural component equals or exceeds the required strength determined on the basis of the LRFD load combinations. All provisions of this Specification, except for those in Section B3.2, shall apply.

Design shall be performed in accordance with Equation B3-1:

$$R_u \leq \phi R_n \tag{B3-1}$$

where

R_u = required strength using LRFD load combinations
R_n = nominal strength
ϕ = resistance factor
ϕR_n = design strength

The nominal strength, R_n, and the resistance factor, ϕ, for the applicable limit states are specified in Chapters D through K.

2. Design for Strength Using Allowable Strength Design (ASD)

Design according to the provisions for allowable strength design (ASD) satisfies the requirements of this Specification when the allowable strength of each structural component equals or exceeds the required strength determined on the basis of the ASD load combinations. All provisions of this Specification, except those of Section B3.1, shall apply.

Design shall be performed in accordance with Equation B3-2:

$$R_a \leq \frac{R_n}{\Omega} \tag{B3-2}$$

where

R_a = required strength using ASD load combinations
R_n = nominal strength
Ω = safety factor
R_n/Ω = allowable strength

The nominal strength, R_n, and the safety factor, Ω, for the applicable limit states are specified in Chapters D through K.

3. Required Strength

The required strength of structural members and connections shall be determined by structural analysis for the applicable load combinations as stipulated in Section B2.

Design by elastic or inelastic analysis is permitted. Requirements for analysis are stipulated in Chapter C and Appendix 1.

The required flexural strength of indeterminate beams composed of compact sections, as defined in Section B4.1, carrying gravity loads only, and satisfying the unbraced length requirements of Section F13.5, is permitted to be taken as nine-tenths of the negative moments at the points of support, produced by the gravity loading and determined by an elastic analysis satisfying the requirements of Chapter C, provided that the maximum positive moment is increased by one-tenth of the average negative moment determined by an elastic analysis. This moment redistribution is not permitted for moments in members with F_y exceeding 65 ksi (450 MPa), for moments produced by loading on cantilevers, for design using partially restrained (PR) moment connections, or for design by inelastic analysis using the provisions of Appendix 1. This moment redistribution is permitted for design according to Section B3.1 (LRFD) and for design according to Section B3.2 (ASD). The required axial strength shall not exceed $0.15\phi_c F_y A_g$ for LRFD or $0.15F_y A_g/\Omega_c$ for ASD, where ϕ_c and Ω_c are determined from Section E1, A_g = gross area of member, in.^{2} (mm^2), and F_y = specified minimum yield stress, ksi (MPa).

4. Design of Connections and Supports

Connection elements shall be designed in accordance with the provisions of Chapters J and K. The forces and deformations used in design of the connections shall be consistent with the intended performance of the connection and the assumptions used in the design of the structure. Self-limiting inelastic deformations of the connections are permitted. At points of support, beams, girders and trusses shall be restrained against rotation about their longitudinal axis unless it can be shown by analysis that the restraint is not required.

User Note: Section 3.1.2 of the *Code of Standard Practice* addresses communication of necessary information for the design of connections.

4a. Simple Connections

A simple connection transmits a negligible moment. In the analysis of the structure, simple connections may be assumed to allow unrestrained relative rotation between the framing elements being connected. A simple connection shall have sufficient rotation capacity to accommodate the required rotation determined by the analysis of the structure.

4b. Moment Connections

Two types of moment connections, fully restrained and partially restrained, are permitted, as specified below.

(a) Fully Restrained (FR) Moment Connections
A fully restrained (FR) moment connection transfers moment with a negligible rotation between the connected members. In the analysis of the structure, the connection may be assumed to allow no relative rotation. An FR connection shall have sufficient strength and stiffness to maintain the initial angle between the connected members at the strength limit states.

(b) Partially Restrained (PR) Moment Connections
Partially restrained (PR) moment connections transfer moments, but the rotation between connected members is not negligible. In the analysis of the structure, the force-deformation response characteristics of the connection shall be included. The response characteristics of a PR connection shall be documented in the technical literature or established by analytical or experimental means. The component elements of a PR connection shall have sufficient strength, stiffness and deformation capacity at the strength limit states.

5. Design of Diaphragms and Collectors

Diaphragms and collectors shall be designed for forces that result from loads as stipulated in Section B2. They shall be designed in conformance with the provisions of Chapters C through K, as applicable.

6. Design of Anchorages to Concrete

Anchorage between steel and concrete acting compositely shall be designed in accordance with Chapter I. The design of column bases and anchor rods shall be in accordance with Chapter J.

7. Design for Stability

The structure and its elements shall be designed for stability in accordance with Chapter C.

8. Design for Serviceability

The overall structure and the individual members and connections shall be evaluated for serviceability limit states in accordance with Chapter L.

9. Design for Structural Integrity

When design for structural integrity is required by the applicable building code, the requirements in this section shall be met.

(a) Column splices shall have a nominal tensile strength equal to or greater than $D + L$ for the area tributary to the column between the splice and the splice or base immediately below,

where
D = nominal dead load, kips (N)
L = nominal live load, kips (N)

(b) Beam and girder end connections shall have a minimum nominal axial tensile strength equal to (i) two-thirds of the required vertical shear strength for design according to Section B3.1 (LRFD) or (ii) the required vertical shear strength for design according to Section B3.2 (ASD), but not less than 10 kips in either case.

(c) End connections of members bracing columns shall have a nominal tensile strength equal to or greater than (i) 1% of two-thirds of the required column axial strength at that level for design according to Section B3.1 (LRFD) or (ii) 1% of the required column axial strength at that level for design according to Section B3.2 (ASD).

The strength requirements for structural integrity in this section shall be evaluated independently of other strength requirements. For the purpose of satisfying these requirements, bearing bolts in connections with short-slotted holes parallel to the direction of the tension force and inelastic deformation of the connection are permitted.

10. Design for Ponding

The roof system shall be investigated through structural analysis to ensure strength and stability under ponding conditions, unless the roof surface is configured to prevent the accumulation of water.

Methods of evaluating stability and strength under ponding conditions are provided in Appendix 2.

11. Design for Fatigue

Fatigue shall be considered in accordance with Appendix 3, for members and their connections subject to repeated loading. Fatigue need not be considered for seismic effects or for the effects of wind loading on typical building lateral force-resisting systems and building enclosure components.

12. Design for Fire Conditions

Two methods of design for fire conditions are provided in Appendix 4: (a) by analysis and (b) by qualification testing. Compliance with the fire-protection requirements in the applicable building code shall be deemed to satisfy the requirements of Appendix 4.

This section is not intended to create or imply a contractual requirement for the engineer of record responsible for the structural design or any other member of the design team.

User Note: Design by qualification testing is the prescriptive method specified in most building codes. Traditionally, on most projects where the architect is the prime professional, the architect has been the responsible party to specify and coordinate fire-protection requirements. Design by analysis is a newer engineering approach to fire protection. Designation of the person(s) responsible for designing for fire conditions is a contractual matter to be addressed on each project.

13. Design for Corrosion Effects

Where corrosion could impair the strength or serviceability of a structure, structural components shall be designed to tolerate corrosion or shall be protected against corrosion.

B4. MEMBER PROPERTIES

1. Classification of Sections for Local Buckling

For members subject to axial compression, sections are classified as nonslender-element or slender-element sections. For a nonslender-element section, the width-to-thickness ratios of its compression elements shall not exceed λ_r from Table B4.1a. If the width-to-thickness ratio of any compression element exceeds λ_r, the section is a slender-element section.

For members subject to flexure, sections are classified as compact, noncompact or slender-element sections. For a section to qualify as compact, its flanges must be continuously connected to the web or webs, and the width-to-thickness ratios of its compression elements shall not exceed the limiting width-to-thickness ratios, λ_p, from Table B4.1b. If the width-to-thickness ratio of one or more compression elements exceeds λ_p, but does not exceed λ_r from Table B4.1b, the section is noncompact. If the width-to-thickness ratio of any compression element exceeds λ_r, the section is a slender-element section.

1a. Unstiffened Elements

For unstiffened elements supported along only one edge parallel to the direction of the compression force, the width shall be taken as follows:

(a) For flanges of I-shaped members and tees, the width, b, is one-half the full-flange width, b_f.

(b) For legs of angles and flanges of channels and zees, the width, b, is the full leg or flange width.

(c) For plates, the width, b, is the distance from the free edge to the first row of fasteners or line of welds.

(d) For stems of tees, d is the full depth of the section.

User Note: Refer to Table B4.1 for the graphic representation of unstiffened element dimensions.

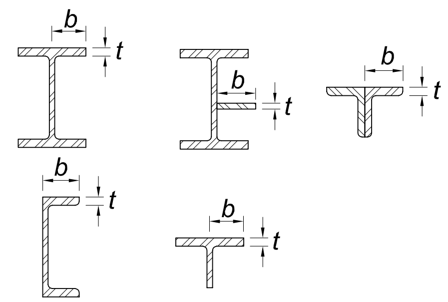
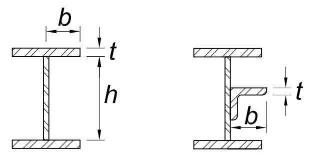
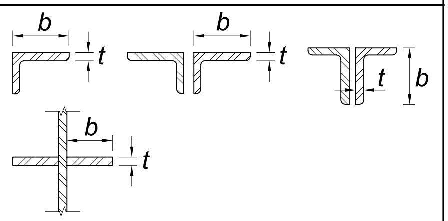
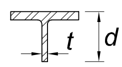
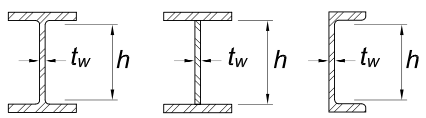
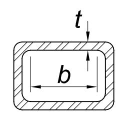
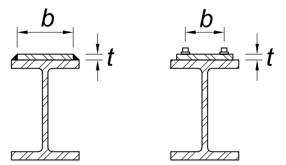
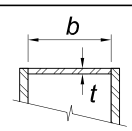
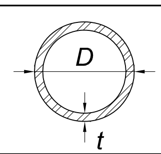
1b. Stiffened Elements

For stiffened elements supported along two edges parallel to the direction of the compression force, the width shall be taken as follows:

(a) For webs of rolled sections, h is the clear distance between flanges less the fillet at each flange; h_c is twice the distance from the centroid to the inside face of the compression flange less the fillet or corner radius.

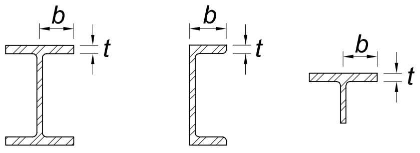
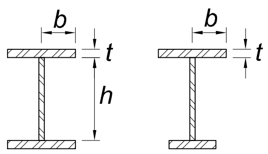
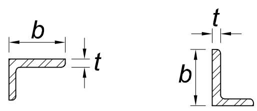
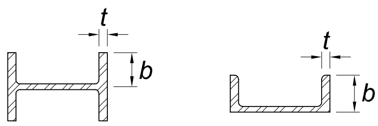
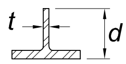
(b) For webs of built-up sections, h is the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used, and h_c is twice the distance from the centroid to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used; h_p is twice the distance from the plastic neutral axis to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used.

TABLE B4.1a
Width-to-Thickness Ratios: Compression Elements Members Subject to Axial Compression

	Case	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio λ_r (nonslender/slender)	Examples
Unstiffened Elements	1	Flanges of rolled I-shaped sections, plates projecting from rolled I-shaped sections, outstanding legs of pairs of angles connected with continuous contact, flanges of channels, and flanges of tees	b/t	$0.56\sqrt{\dfrac{E}{F_y}}$	
	2	Flanges of built-up I-shaped sections and plates or angle legs projecting from built-up I-shaped sections	b/t	$0.64\sqrt{\dfrac{k_c E}{F_y}}$ [a]	
	3	Legs of single angles, legs of double angles with separators, and all other unstiffened elements	b/t	$0.45\sqrt{\dfrac{E}{F_y}}$	
	4	Stems of tees	d/t	$0.75\sqrt{\dfrac{E}{F_y}}$	
Stiffened Elements	5	Webs of doubly symmetric rolled and built-up I-shaped sections and channels	h/t_w	$1.49\sqrt{\dfrac{E}{F_y}}$	
	6	Walls of rectangular HSS	b/t	$1.40\sqrt{\dfrac{E}{F_y}}$	
	7	Flange cover plates and diaphragm plates between lines of fasteners or welds	b/t	$1.40\sqrt{\dfrac{E}{F_y}}$	
	8	All other stiffened elements	b/t	$1.49\sqrt{\dfrac{E}{F_y}}$	
	9	Round HSS	D/t	$0.11\dfrac{E}{F_y}$	

[a] $k_c = 4/\sqrt{h/t_w}$, but shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes.

TABLE B4.1b
Width-to-Thickness Ratios: Compression Elements Members Subject to Flexure

	Case	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio: λ_p (compact/noncompact)	Limiting Width-to-Thickness Ratio: λ_r (noncompact/slender)	Examples
Unstiffened Elements	10	Flanges of rolled I-shaped sections, channels, and tees	b/t	$0.38\sqrt{\dfrac{E}{F_y}}$	$1.0\sqrt{\dfrac{E}{F_y}}$	
Unstiffened Elements	11	Flanges of doubly and singly symmetric I-shaped built-up sections	b/t	$0.38\sqrt{\dfrac{E}{F_y}}$	[a] [b] $0.95\sqrt{\dfrac{k_cE}{F_L}}$	
Unstiffened Elements	12	Legs of single angles	b/t	$0.54\sqrt{\dfrac{E}{F_y}}$	$0.91\sqrt{\dfrac{E}{F_y}}$	
Unstiffened Elements	13	Flanges of all I-shaped sections and channels in flexure about the minor axis	b/t	$0.38\sqrt{\dfrac{E}{F_y}}$	$1.0\sqrt{\dfrac{E}{F_y}}$	
Unstiffened Elements	14	Stems of tees	d/t	$0.84\sqrt{\dfrac{E}{F_y}}$	$1.52\sqrt{\dfrac{E}{F_y}}$	

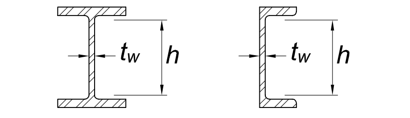
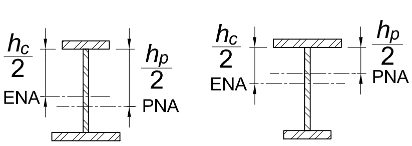
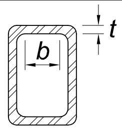
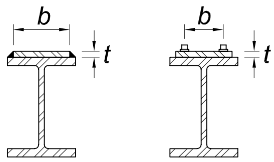
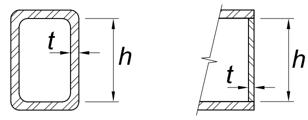
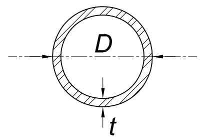
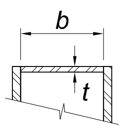
(c) For flange or diaphragm plates in built-up sections, the width, b, is the distance between adjacent lines of fasteners or lines of welds.

(d) For flanges of rectangular hollow structural sections (HSS), the width, b, is the clear distance between webs less the inside corner radius on each side. For webs of rectangular HSS, h is the clear distance between the flanges less the inside corner radius on each side. If the corner radius is not known, b and h shall be taken as the corresponding outside dimension minus three times the thickness. The thickness, t, shall be taken as the design wall thickness, per Section B4.2.

(e) For flanges or webs of box sections and other stiffened elements, the width, b, is the clear distance between the elements providing stiffening.

(f) For perforated cover plates, b is the transverse distance between the nearest line of fasteners, and the net area of the plate is taken at the widest hole.

TABLE B4.1b (continued)
Width-to-Thickness Ratios: Compression Elements
Members Subject to Flexure

	Case	Description of Element	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio λ_p (compact/ noncompact)	Limiting Width-to-Thickness Ratio λ_r (noncompact/ slender)	Examples
Stiffened Elements	15	Webs of doubly symmetric I-shaped sections and channels	h/t_w	$3.76\sqrt{\frac{E}{F_y}}$	$5.70\sqrt{\frac{E}{F_y}}$	
	16	Webs of singly symmetric I-shaped sections	h_c/t_w	$\dfrac{\frac{h_c}{h_p}\sqrt{\frac{E}{F_y}}}{\left(0.54\frac{M_p}{M_y}-0.09\right)^2}$ [c] $\le \lambda_r$	$5.70\sqrt{\frac{E}{F_y}}$	
	17	Flanges of rectangular HSS	b/t	$1.12\sqrt{\frac{E}{F_y}}$	$1.40\sqrt{\frac{E}{F_y}}$	
	18	Flange cover plates and diaphragm plates between lines of fasteners or welds	b/t	$1.12\sqrt{\frac{E}{F_y}}$	$1.40\sqrt{\frac{E}{F_y}}$	
	19	Webs of rectangular HSS and box sections	h/t	$2.42\sqrt{\frac{E}{F_y}}$	$5.70\sqrt{\frac{E}{F_y}}$	
	20	Round HSS	D/t	$0.07\frac{E}{F_y}$	$0.31\frac{E}{F_y}$	
	21	Flanges of box sections	b/t	$1.12\sqrt{\frac{E}{F_y}}$	$1.49\sqrt{\frac{E}{F_y}}$	

[a] $k_c = 4/\sqrt{h/t_w}$, shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes.
[b] $F_L = 0.7F_y$ for slender web I-shaped members and major-axis bending of compact and noncompact web built-up I-shaped members with $S_{xt}/S_{xc} \ge 0.7$; $F_L = F_yS_{xt}/S_{xc} \ge 0.5F_y$ for major-axis bending of compact and noncompact web built-up I-shaped members with $S_{xt}/S_{xc} < 0.7$, where S_{xc}, S_{xt} = elastic section modulus referred to compression and tension flanges, respectively, in.^{3} (mm^3).
[c] M_y is the moment at yielding of the extreme fiber. $M_p = F_yZ_x$, plastic bending moment, kip-in. (N-mm), where Z_x = plastic section modulus taken about x-axis, in.^{3} (mm^3).
E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)
F_y = specified minimum yield stress, ksi (MPa)
ENA = elastic neutral axis
PNA = plastic neutral axis

User Note: Refer to Table B4.1 for the graphic representation of stiffened element dimensions.

For tapered flanges of rolled sections, the thickness is the nominal value halfway between the free edge and the corresponding face of the web.

2. Design Wall Thickness for HSS

The design wall thickness, t, shall be used in calculations involving the wall thickness of hollow structural sections (HSS). The design wall thickness, t, shall be taken equal to the nominal thickness for box sections and HSS produced according to ASTM A1065/A1065M or ASTM A1085/A1085M. For HSS produced according to other standards approved for use under this Specification, the design wall thickness, t, shall be taken equal to 0.93 times the nominal wall thickness.

User Note: A pipe can be designed using the provisions of this Specification for round HSS sections as long as the pipe conforms to ASTM A53/A53M Grade B and the appropriate limitations of this Specification are used.

3. Gross and Net Area Determination

3a. Gross Area

The gross area, A_g, of a member is the total cross-sectional area.

3b. Net Area

The net area, A_n, of a member is the sum of the products of the thickness and the net width of each element computed as follows:

In computing net area for tension and shear, the width of a bolt hole shall be taken as $^1/_{16}$ in. (2 mm) greater than the nominal dimension of the hole.

For a chain of holes extending across a part in any diagonal or zigzag line, the net width of the part shall be obtained by deducting from the gross width the sum of the diameters or slot dimensions as provided in this section, of all holes in the chain, and adding, for each gage space in the chain, the quantity $s^2/4g$,

where

g = transverse center-to-center spacing (gage) between fastener gage lines, in. (mm)

s = longitudinal center-to-center spacing (pitch) of any two consecutive holes, in. (mm)

For angles, the gage for holes in opposite adjacent legs shall be the sum of the gages from the back of the angles less the thickness.

For slotted HSS welded to a gusset plate, the net area, A_n, is the gross area minus the product of the thickness and the total width of material that is removed to form the slot.

In determining the net area across plug or slot welds, the weld metal shall not be considered as adding to the net area.

For members without holes, the net area, A_n, is equal to the gross area, A_g.

B5. FABRICATION AND ERECTION

Shop drawings, fabrication, shop painting and erection shall satisfy the requirements stipulated in Chapter M.

B6. QUALITY CONTROL AND QUALITY ASSURANCE

Quality control and quality assurance activities shall satisfy the requirements stipulated in Chapter N.

B7. EVALUATION OF EXISTING STRUCTURES

The evaluation of existing structures shall satisfy the requirements stipulated in Appendix 5.

CHAPTER C

DESIGN FOR STABILITY

This chapter addresses requirements for the design of structures for stability. The direct analysis method is presented herein.

The chapter is organized as follows:

C1. General Stability Requirements
C2. Calculation of Required Strengths
C3. Calculation of Available Strengths

User Note: Alternative methods for the design of structures for stability are provided in Appendices 1 and 7. Appendix 1 provides alternatives that allow for considering member imperfections and/or inelasticity directly within the analysis and may be particularly useful for more complex structures. Appendix 7 provides the effective length method and a first-order elastic method.

C1. GENERAL STABILITY REQUIREMENTS

Stability shall be provided for the structure as a whole and for each of its elements. The effects of all of the following on the stability of the structure and its elements shall be considered: (a) flexural, shear and axial member deformations, and all other component and connection deformations that contribute to the displacements of the structure; (b) second-order effects (including P-Δ and P-δ effects); (c) geometric imperfections; (d) stiffness reductions due to inelasticity, including the effect of partial yielding of the cross section which may be accentuated by the presence of residual stresses; and (e) uncertainty in system, member, and connection strength and stiffness. All load-dependent effects shall be calculated at a level of loading corresponding to LRFD load combinations or 1.6 times ASD load combinations.

Any rational method of design for stability that considers all of the listed effects is permitted; this includes the methods identified in Sections C1.1 and C1.2.

User Note: See Commentary Section C1 and Table C-C1.1 for an explanation of how requirements (a) through (e) of Section C1 are satisfied in the methods of design listed in Sections C1.1 and C1.2.

1. Direct Analysis Method of Design

The direct analysis method of design is permitted for all structures, and can be based on either elastic or inelastic analysis. For design by elastic analysis, required strengths shall be calculated in accordance with Section C2 and the calculation of available strengths in accordance with Section C3. For design by advanced analysis, the provisions of Section 1.1 and Sections 1.2 or 1.3 of Appendix 1 shall be satisfied.

2. Alternative Methods of Design

The effective length method and the first-order analysis method, both defined in Appendix 7, are based on elastic analysis and are permitted as alternatives to the direct analysis method for structures that satisfy the limitations specified in that appendix.

C2. CALCULATION OF REQUIRED STRENGTHS

For the direct analysis method of design, the required strengths of components of the structure shall be determined from an elastic analysis conforming to Section C2.1. The analysis shall include consideration of initial imperfections in accordance with Section C2.2 and adjustments to stiffness in accordance with Section C2.3.

1. General Analysis Requirements

The analysis of the structure shall conform to the following requirements:

(a) The analysis shall consider flexural, shear and axial member deformations, and all other component and connection deformations that contribute to displacements of the structure. The analysis shall incorporate reductions in all stiffnesses that are considered to contribute to the stability of the structure, as specified in Section C2.3.

(b) The analysis shall be a second-order analysis that considers both P-Δ and P-δ effects, except that it is permissible to neglect the effect of P-δ on the response of the structure when the following conditions are satisfied: (1) the structure supports gravity loads primarily through nominally vertical columns, walls or frames; (2) the ratio of maximum second-order drift to maximum first-order drift (both determined for LRFD load combinations or 1.6 times ASD load combinations, with stiffnesses adjusted as specified in Section C2.3) in all stories is equal to or less than 1.7; and (3) no more than one-third of the total gravity load on the structure is supported by columns that are part of moment-resisting frames in the direction of translation being considered. It is necessary in all cases to consider P-δ effects in the evaluation of individual members subject to compression and flexure.

> **User Note:** A P-Δ-only second-order analysis (one that neglects the effects of P-δ on the response of the structure) is permitted under the conditions listed. In this case, the requirement for considering P-δ effects in the evaluation of individual members can be satisfied by applying the B_1 multiplier defined in Appendix 8 to the required flexural strength of the member.

Use of the approximate method of second-order analysis provided in Appendix 8 is permitted.

(c) The analysis shall consider all gravity and other applied loads that may influence the stability of the structure.

> **User Note**: It is important to include in the analysis all gravity loads, including loads on leaning columns and other elements that are not part of the lateral force-resisting system.

(d) For design by LRFD, the second-order analysis shall be carried out under LRFD load combinations. For design by ASD, the second-order analysis shall be carried out under 1.6 times the ASD load combinations, and the results shall be divided by 1.6 to obtain the required strengths of components.

2. Consideration of Initial System Imperfections

The effect of initial imperfections in the position of points of intersection of members on the stability of the structure shall be taken into account either by direct modeling of these imperfections in the analysis as specified in Section C2.2a or by the application of notional loads as specified in Section C2.2b.

> **User Note:** The imperfections required to be considered in this section are imperfections in the locations of points of intersection of members (system imperfections). In typical building structures, the important imperfection of this type is the out-of-plumbness of columns. Consideration of initial out-of-straightness of individual members (member imperfections) is not required in the structural analysis when using the provisions of this section; it is accounted for in the compression member design provisions of Chapter E and need not be considered explicitly in the analysis as long as it is within the limits specified in the *Code of Standard Practice*. Appendix 1, Section 1.2 provides an extension to the direct analysis method that includes modeling of member imperfections (initial out-of-straightness) within the structural analysis.

2a. Direct Modeling of Imperfections

In all cases, it is permissible to account for the effect of initial system imperfections by including the imperfections directly in the analysis. The structure shall be analyzed with points of intersection of members displaced from their nominal locations. The magnitude of the initial displacements shall be the maximum amount considered in the design; the pattern of initial displacements shall be such that it provides the greatest destabilizing effect.

> **User Note:** Initial displacements similar in configuration to both displacements due to loading and anticipated buckling modes should be considered in the modeling of imperfections. The magnitude of the initial displacements should be based on permissible construction tolerances, as specified in the *Code of Standard Practice* or other governing requirements, or on actual imperfections if known.

In the analysis of structures that support gravity loads primarily through nominally vertical columns, walls or frames, where the ratio of maximum second-order story drift to maximum first-order story drift (both determined for LRFD load combinations or 1.6 times ASD load combinations, with stiffnesses adjusted as specified in Section C2.3) in all stories is equal to or less than 1.7, it is permissible to include initial system imperfections in the analysis for gravity-only load combinations and not in the analysis for load combinations that include applied lateral loads.

2b. Use of Notional Loads to Represent Imperfections

For structures that support gravity loads primarily through nominally vertical columns, walls or frames, it is permissible to use notional loads to represent the effects of initial system imperfections in the position of points of intersection of members in accordance with the requirements of this section. The notional load shall be applied to a model of the structure based on its nominal geometry.

> **User Note:** In general, the notional load concept is applicable to all types of structures and to imperfections in the positions of both points of intersection of members and points along members, but the specific requirements in Sections C2.2b(a) through C2.2b(d) are applicable only for the particular class of structure and type of system imperfection identified here.

(a) Notional loads shall be applied as lateral loads at all levels. The notional loads shall be additive to other lateral loads and shall be applied in all load combinations, except as indicated in Section C2.2b(d). The magnitude of the notional loads shall be:

$$N_i = 0.002\alpha Y_i \tag{C2-1}$$

where

α = 1.0 (LRFD); α = 1.6 (ASD)

N_i = notional load applied at level i, kips (N)

Y_i = gravity load applied at level i from the LRFD load combination or ASD load combination, as applicable, kips (N)

> **User Note:** The use of notional loads can lead to additional (generally small) fictitious base shears in the structure. The correct horizontal reactions at the foundation may be obtained by applying an additional horizontal force at the base of the structure, equal and opposite in direction to the sum of all notional loads, distributed among vertical load-carrying elements in the same proportion as the gravity load supported by those elements. The notional loads can also lead to additional overturning effects, which are not fictitious.

(b) The notional load at any level, N_i, shall be distributed over that level in the same manner as the gravity load at the level. The notional loads shall be applied in the direction that provides the greatest destabilizing effect.

> **User Note:** For most building structures, the requirement regarding notional load direction may be satisfied as follows: for load combinations that do not include lateral loading, consider two alternative orthogonal directions of notional load application, in a positive and a negative sense in each of the two directions, in the same direction at all levels; for load combinations that include lateral loading, apply all notional loads in the direction of the resultant of all lateral loads in the combination.

(c) The notional load coefficient of 0.002 in Equation C2-1 is based on a nominal initial story out-of-plumbness ratio of 1/500; where the use of a different maximum out-of-plumbness is justified, it is permissible to adjust the notional load coefficient proportionally.

> **User Note:** An out-of-plumbness of 1/500 represents the maximum tolerance on column plumbness specified in the *Code of Standard Practice*. In some cases, other specified tolerances, such as those on plan location of columns, will govern and will require a tighter plumbness tolerance.

(d) For structures in which the ratio of maximum second-order drift to maximum first-order drift (both determined for LRFD load combinations or 1.6 times ASD load combinations, with stiffnesses adjusted as specified in Section C2.3) in all stories is equal to or less than 1.7, it is permissible to apply the notional load, N_i, only in gravity-only load combinations and not in combinations that include other lateral loads.

3. Adjustments to Stiffness

The analysis of the structure to determine the required strengths of components shall use reduced stiffnesses, as follows:

(a) A factor of 0.80 shall be applied to all stiffnesses that are considered to contribute to the stability of the structure. It is permissible to apply this reduction factor to all stiffnesses in the structure.

> **User Note:** Applying the stiffness reduction to some members and not others can, in some cases, result in artificial distortion of the structure under load and possible unintended redistribution of forces. This can be avoided by applying the reduction to all members, including those that do not contribute to the stability of the structure.

(b) An additional factor, τ_b, shall be applied to the flexural stiffnesses of all members whose flexural stiffnesses are considered to contribute to the stability of the structure. For noncomposite members, τ_b shall be defined as follows (see Section I1.5 for the definition of τ_b for composite members).

(1) When $\alpha P_r/P_{ns} \leq 0.5$

$$\tau_b = 1.0 \tag{C2-2a}$$

(2) When $\alpha P_r/P_{ns} > 0.5$

$$\tau_b = 4(\alpha P_r/P_{ns})[1-(\alpha P_r/P_{ns})] \tag{C2-2b}$$

where

α = 1.0 (LRFD); α = 1.6 (ASD)

P_r = required axial compressive strength using LRFD or ASD load combinations, kips (N)

P_{ns} = cross-section compressive strength; for nonslender-element sections, $P_{ns} = F_yA_g$, and for slender-element sections, $P_{ns} = F_yA_e$, where A_e is as defined in Section E7, kips (N)

User Note: Taken together, Sections (a) and (b) require the use of $0.8\tau_b$ times the nominal elastic flexural stiffness and 0.8 times other nominal elastic stiffnesses for structural steel members in the analysis.

(c) In structures to which Section C2.2b is applicable, in lieu of using $\tau_b < 1.0$ where $\alpha P_r/P_{ns} > 0.5$, it is permissible to use $\tau_b = 1.0$ for all noncomposite members if a notional load of $0.001\alpha Y_i$ [where Y_i is as defined in Section C2.2b(a)] is applied at all levels, in the direction specified in Section C2.2b(b), in all load combinations. These notional loads shall be added to those, if any, used to account for the effects of initial imperfections in the position of points of intersection of members and shall not be subject to the provisions of Section C2.2b(d).

(d) Where components comprised of materials other than structural steel are considered to contribute to the stability of the structure and the governing codes and specifications for the other materials require greater reductions in stiffness, such greater stiffness reductions shall be applied to those components.

C3. CALCULATION OF AVAILABLE STRENGTHS

For the direct analysis method of design, the available strengths of members and connections shall be calculated in accordance with the provisions of Chapters D through K, as applicable, with no further consideration of overall structure stability. The effective length for flexural buckling of all members shall be taken as the unbraced length unless a smaller value is justified by rational analysis.

Bracing intended to define the unbraced lengths of members shall have sufficient stiffness and strength to control member movement at the braced points.

User Note: Methods of satisfying this bracing requirement are provided in Appendix 6. The requirements of Appendix 6 are not applicable to bracing that is included in the design of the lateral force-resisting system of the overall structure.

CHAPTER D

DESIGN OF MEMBERS FOR TENSION

This chapter applies to members subject to axial tension.

The chapter is organized as follows:

D1. Slenderness Limitations
D2. Tensile Strength
D3. Effective Net Area
D4. Built-Up Members
D5. Pin-Connected Members
D6. Eyebars

User Note: For cases not included in this chapter, the following sections apply:
- B3.11 Members subject to fatigue
- Chapter H Members subject to combined axial tension and flexure
- J3 Threaded rods
- J4.1 Connecting elements in tension
- J4.3 Block shear rupture strength at end connections of tension members

D1. SLENDERNESS LIMITATIONS

There is no maximum slenderness limit for members in tension.

User Note: For members designed on the basis of tension, the slenderness ratio, L/r, preferably should not exceed 300. This suggestion does not apply to rods or hangers in tension.

D2. TENSILE STRENGTH

The design tensile strength, $\phi_t P_n$, and the allowable tensile strength, P_n/Ω_t, of tension members shall be the lower value obtained according to the limit states of tensile yielding in the gross section and tensile rupture in the net section.

(a) For tensile yielding in the gross section

$$P_n = F_y A_g \tag{D2-1}$$

$$\phi_t = 0.90 \text{ (LRFD)} \qquad \Omega_t = 1.67 \text{ (ASD)}$$

(b) For tensile rupture in the net section

$$P_n = F_u A_e \tag{D2-2}$$

$$\phi_t = 0.75 \text{ (LRFD)} \qquad \Omega_t = 2.00 \text{ (ASD)}$$

where
A_e = effective net area, in.^{2} (mm^2)
A_g = gross area of member, in.^{2} (mm^2)
F_y = specified minimum yield stress, ksi (MPa)
F_u = specified minimum tensile strength, ksi (MPa)

Where connections use plug, slot or fillet welds in holes or slots, the effective net area through the holes shall be used in Equation D2-2.

D3. EFFECTIVE NET AREA

The gross area, A_g, and net area, A_n, of tension members shall be determined in accordance with the provisions of Section B4.3.

The effective net area of tension members shall be determined as

$$A_e = A_n U \tag{D3-1}$$

where U, the shear lag factor, is determined as shown in Table D3.1.

For open cross sections such as W, M, S, C, or HP shapes, WTs, STs, and single and double angles, the shear lag factor, U, need not be less than the ratio of the gross area of the connected element(s) to the member gross area. This provision does not apply to closed sections, such as HSS sections, nor to plates.

D4. BUILT-UP MEMBERS

For limitations on the longitudinal spacing of connectors between elements in continuous contact consisting of a plate and a shape, or two plates, see Section J3.5.

Lacing, perforated cover plates, or tie plates without lacing are permitted to be used on the open sides of built-up tension members. Tie plates shall have a length not less than two-thirds the distance between the lines of welds or fasteners connecting them to the components of the member. The thickness of such tie plates shall not be less than one-fiftieth of the distance between these lines. The longitudinal spacing of intermittent welds or fasteners at tie plates shall not exceed 6 in. (150 mm).

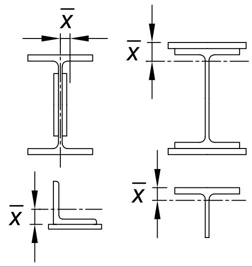
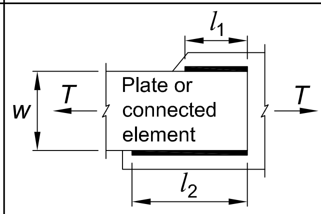
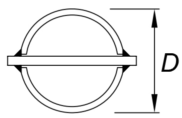
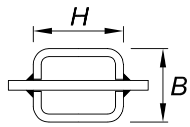
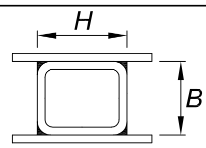
User Note: The longitudinal spacing of connectors between components should preferably limit the slenderness ratio in any component between the connectors to 300.

D5. PIN-CONNECTED MEMBERS

1. Tensile Strength

The design tensile strength, $\phi_t P_n$, and the allowable tensile strength, P_n/Ω_t, of pin-connected members, shall be the lower value determined according to the limit states of tensile rupture, shear rupture, bearing and yielding.

TABLE D3.1
Shear Lag Factors for Connections to Tension Members

Case	Description of Element		Shear Lag Factor, *U*	Example
1	All tension members where the tension load is transmitted directly to each of the cross-sectional elements by fasteners or welds (except as in Cases 4, 5 and 6).		$U = 1.0$	–
2	All tension members, except HSS, where the tension load is transmitted to some but not all of the cross-sectional elements by fasteners or by longitudinal welds in combination with transverse welds. Alternatively, Case 7 is permitted for W, M, S and HP shapes. (For angles, Case 8 is permitted to be used.)		$U = 1 - \dfrac{\bar{x}}{l}$	
3	All tension members where the tension load is transmitted only by transverse welds to some but not all of the cross-sectional elements.		$U = 1.0$ and A_n = area of the directly connected elements	–
4[a]	Plates, angles, channels with welds at heels, tees, and W-shapes with connected elements, where the tension load is transmitted by longitudinal welds only. See Case 2 for definition of $\bar{x}$.		$U = \dfrac{3l^2}{3l^2 + w^2}\left(1 - \dfrac{\bar{x}}{l}\right)$	
5	Round HSS with a single concentric gusset plate through slots in the HSS.		$l \geq 1.3D$, $U = 1.0$ $D \leq l < 1.3D$, $U = 1 - \dfrac{\bar{x}}{l}$ $\bar{x} = \dfrac{D}{\pi}$	
6	Rectangular HSS.	with a single concentric gusset plate	$l \geq H$, $U = 1 - \dfrac{\bar{x}}{l}$ $\bar{x} = \dfrac{B^2 + 2BH}{4(B+H)}$	
		with two side gusset plates	$l \geq H$, $U = 1 - \dfrac{\bar{x}}{l}$ $\bar{x} = \dfrac{B^2}{4(B+H)}$	
7	W-, M-, S- or HP-shapes, or tees cut from these shapes. (If *U* is calculated per Case 2, the larger value is permitted to be used.)	with flange connected with three or more fasteners per line in the direction of loading	$b_f \geq \dfrac{2}{3}d$, $U = 0.90$ $b_f < \dfrac{2}{3}d$, $U = 0.85$	–
		with web connected with four or more fasteners per line in the direction of loading	$U = 0.70$	–
8	Single and double angles. (If *U* is calculated per Case 2, the larger value is permitted to be used.)	with four or more fasteners per line in the direction of loading	$U = 0.80$	–
		with three fasteners per line in the direction of loading (with fewer than three fasteners per line in the direction of loading, use Case 2)	$U = 0.60$	–

B = overall width of rectangular HSS member, measured 90° to the plane of the connection, in. (mm); D = outside diameter of round HSS, in. (mm); H = overall height of rectangular HSS member, measured in the plane of the connection, in. (mm); d = depth of section, in. (mm); for tees, d = depth of the section from which the tee was cut, in. (mm); l = length of connection, in. (mm); w = width of plate, in. (mm); $\bar{x}$ = eccentricity of connection, in. (mm).

[a] $l = \dfrac{l_1 + l_2}{2}$, where l_1 and l_2 shall not be less than 4 times the weld size.

(a) For tensile rupture on the net effective area

$$P_n = F_u(2tb_e) \tag{D5-1}$$

$$\phi_t = 0.75 \text{ (LRFD)} \qquad \Omega_t = 2.00 \text{ (ASD)}$$

(b) For shear rupture on the effective area

$$P_n = 0.6F_uA_{sf} \tag{D5-2}$$

$$\phi_{sf} = 0.75 \text{ (LRFD)} \qquad \Omega_{sf} = 2.00 \text{ (ASD)}$$

where

$A_{sf} = 2t(a + d/2)$
= area on the shear failure path, in.^{2} (mm^2)

a = shortest distance from edge of the pin hole to the edge of the member measured parallel to the direction of the force, in. (mm)

b_e = $2t + 0.63$, in. (= $2t + 16$, mm), but not more than the actual distance from the edge of the hole to the edge of the part measured in the direction normal to the applied force, in. (mm)

d = diameter of pin, in. (mm)

t = thickness of plate, in. (mm)

(c) For bearing on the projected area of the pin, use Section J7.

(d) For yielding on the gross section, use Section D2(a).

2. Dimensional Requirements

Pin-connected members shall meet the following requirements:

(a) The pin hole shall be located midway between the edges of the member in the direction normal to the applied force.

(b) When the pin is expected to provide for relative movement between connected parts while under full load, the diameter of the pin hole shall not be more than $^1/_{32}$ in. (1 mm) greater than the diameter of the pin.

(c) The width of the plate at the pin hole shall not be less than $2b_e + d$ and the minimum extension, a, beyond the bearing end of the pin hole, parallel to the axis of the member, shall not be less than $1.33b_e$.

(d) The corners beyond the pin hole are permitted to be cut at 45° to the axis of the member, provided the net area beyond the pin hole, on a plane perpendicular to the cut, is not less than that required beyond the pin hole parallel to the axis of the member.

D6. EYEBARS

1. Tensile Strength

The available tensile strength of eyebars shall be determined in accordance with Section D2, with A_g taken as the cross-sectional area of the body.

For calculation purposes, the width of the body of the eyebars shall not exceed eight times its thickness.

2. Dimensional Requirements

Eyebars shall meet the following requirements:

(a) Eyebars shall be of uniform thickness, without reinforcement at the pin holes, and have circular heads with the periphery concentric with the pin hole.

(b) The radius of transition between the circular head and the eyebar body shall not be less than the head diameter.

(c) The pin diameter shall not be less than seven-eighths times the eyebar body width, and the pin-hole diameter shall not be more than $^1/_{32}$ in. (1 mm) greater than the pin diameter.

(d) For steels having F_y greater than 70 ksi (485 MPa), the hole diameter shall not exceed five times the plate thickness, and the width of the eyebar body shall be reduced accordingly.

(e) A thickness of less than $^1/_2$ in. (13 mm) is permissible only if external nuts are provided to tighten pin plates and filler plates into snug contact.

(f) The width from the hole edge to the plate edge perpendicular to the direction of applied load shall be greater than two-thirds and, for the purpose of calculation, not more than three-fourths times the eyebar body width.

CHAPTER E

DESIGN OF MEMBERS FOR COMPRESSION

This chapter addresses members subject to axial compression.

The chapter is organized as follows:

User Note: For cases not included in this chapter, the following sections apply:

- H1 – H2 Members subject to combined axial compression and flexure
- H3 Members subject to axial compression and torsion
- I2 Composite axially loaded members
- J4.4 Compressive strength of connecting elements

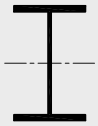
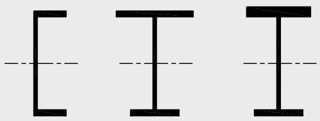
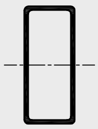
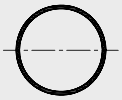
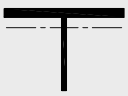
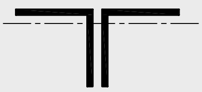
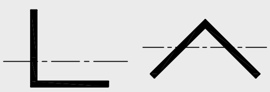
E1. GENERAL PROVISIONS

The design compressive strength, $\phi_c P_n$, and the allowable compressive strength, P_n/Ω_c, are determined as follows.

The nominal compressive strength, P_n, shall be the lowest value obtained based on the applicable limit states of flexural buckling, torsional buckling, and flexural-torsional buckling.

$$\phi_c = 0.90 \text{ (LRFD)} \qquad \Omega_c = 1.67 \text{ (ASD)}$$

TABLE USER NOTE E1.1
Selection Table for the Application of Chapter E Sections

Cross Section	Without Slender Elements		With Slender Elements	
	Sections in Chapter E	Limit States	Sections in Chapter E	Limit States
	E3 E4	FB TB	E7	LB FB TB
	E3 E4	FB FTB	E7	LB FB FTB
	E3	FB	E7	LB FB
	E3	FB	E7	LB FB
	E3 E4	FB FTB	E7	LB FB FTB
	E6 E3 E4	FB FTB	E6 E7	LB FB FTB
	E5		E5	
	E3	FB	N/A	N/A
Unsymmetrical shapes other than single angles	E4	FTB	E7	LB FTB

FB = flexural buckling, TB = torsional buckling, FTB = flexural-torsional buckling, LB = local buckling, N/A = not applicable

E2. EFFECTIVE LENGTH

The effective length, L_c, for calculation of member slenderness, L_c/r, shall be determined in accordance with Chapter C or Appendix 7,

where
K = effective length factor
$L_c = KL$ = effective length of member, in. (mm)
L = laterally unbraced length of the member, in. (mm)
r = radius of gyration, in. (mm)

User Note: For members designed on the basis of compression, the effective slenderness ratio, L_c/r, preferably should not exceed 200.

User Note: The effective length, L_c, can be determined through methods other than those using the effective length factor, K.

E3. FLEXURAL BUCKLING OF MEMBERS WITHOUT SLENDER ELEMENTS

This section applies to nonslender-element compression members, as defined in Section B4.1, for elements in axial compression.

User Note: When the torsional effective length is larger than the lateral effective length, Section E4 may control the design of wide-flange and similarly shaped columns.

The nominal compressive strength, P_n, shall be determined based on the limit state of flexural buckling:

$$P_n = F_{cr} A_g \tag{E3-1}$$

The critical stress, F_{cr}, is determined as follows:

(a) When $\dfrac{L_c}{r} \leq 4.71\sqrt{\dfrac{E}{F_y}}$ (or $\dfrac{F_y}{F_e} \leq 2.25$)

$$F_{cr} = \left(0.658^{\frac{F_y}{F_e}}\right) F_y \tag{E3-2}$$

(b) When $\dfrac{L_c}{r} > 4.71\sqrt{\dfrac{E}{F_y}}$ (or $\dfrac{F_y}{F_e} > 2.25$)

$$F_{cr} = 0.877 F_e \tag{E3-3}$$

where
A_g = gross cross-sectional area of member, in.^{2} (mm^2)
E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)

F_e = elastic buckling stress determined according to Equation E3-4, as specified in Appendix 7, Section 7.2.3(b), or through an elastic buckling analysis, as applicable, ksi (MPa)

$$= \frac{\pi^2 E}{\left(\frac{L_c}{r}\right)^2} \quad \text{(E3-4)}$$

F_y = specified minimum yield stress of the type of steel being used, ksi (MPa)
r = radius of gyration, in. (mm)

User Note: The two inequalities for calculating the limits of applicability of Sections E3(a) and E3(b), one based on L_c/r and one based on F_y/F_e, provide the same result for flexural buckling.

E4. TORSIONAL AND FLEXURAL-TORSIONAL BUCKLING OF SINGLE ANGLES AND MEMBERS WITHOUT SLENDER ELEMENTS

This section applies to singly symmetric and unsymmetric members, certain doubly symmetric members, such as cruciform or built-up members, and doubly symmetric members when the torsional unbraced length exceeds the lateral unbraced length, all without slender elements. These provisions also apply to single angles with $b/t > 0.71\sqrt{E/F_y}$, where b is the width of the longest leg and t is the thickness.

The nominal compressive strength, P_n, shall be determined based on the limit states of torsional and flexural-torsional buckling:

$$P_n = F_{cr} A_g \quad \text{(E4-1)}$$

The critical stress, F_{cr}, shall be determined according to Equation E3-2 or E3-3, using the torsional or flexural-torsional elastic buckling stress, F_e, determined as follows:

(a) For doubly symmetric members twisting about the shear center

$$F_e = \left(\frac{\pi^2 E C_w}{L_{cz}^2} + GJ\right)\frac{1}{I_x + I_y} \quad \text{(E4-2)}$$

(b) For singly symmetric members twisting about the shear center where y is the axis of symmetry

$$F_e = \left(\frac{F_{ey} + F_{ez}}{2H}\right)\left[1 - \sqrt{1 - \frac{4 F_{ey} F_{ez} H}{\left(F_{ey} + F_{ez}\right)^2}}\right] \quad \text{(E4-3)}$$

User Note: For singly symmetric members with the x-axis as the axis of symmetry, such as channels, Equation E4-3 is applicable with F_{ey} replaced by F_{ex}.

(c) For unsymmetric members twisting about the shear center, F_e is the lowest root of the cubic equation

$$(F_e - F_{ex})(F_e - F_{ey})(F_e - F_{ez}) - F_e^2(F_e - F_{ey})\left(\frac{x_o}{\bar{r}_o}\right)^2 - F_e^2(F_e - F_{ex})\left(\frac{y_o}{\bar{r}_o}\right)^2 = 0 \quad \text{(E4-4)}$$

where

C_w = warping constant, in.^{6} (mm^6)

$$F_{ex} = \frac{\pi^2 E}{\left(\frac{L_{cx}}{r_x}\right)^2} \tag{E4-5}$$

$$F_{ey} = \frac{\pi^2 E}{\left(\frac{L_{cy}}{r_y}\right)^2} \tag{E4-6}$$

$$F_{ez} = \left(\frac{\pi^2 E C_w}{L_{cz}^2} + GJ\right)\frac{1}{A_g \bar{r}_o^2} \tag{E4-7}$$

G = shear modulus of elasticity of steel = 11,200 ksi (77 200 MPa)

H = flexural constant

$$= 1 - \frac{x_o^2 + y_o^2}{\bar{r}_o^2} \tag{E4-8}$$

I_x, I_y = moment of inertia about the principal axes, in.^{4} (mm^4)

J = torsional constant, in.^{4} (mm^4)

K_x = effective length factor for flexural buckling about x-axis

K_y = effective length factor for flexural buckling about y-axis

K_z = effective length factor for torsional buckling about the longitudinal axis

L_{cx} = K_xL_x = effective length of member for buckling about x-axis, in. (mm)

L_{cy} = K_yL_y = effective length of member for buckling about y-axis, in. (mm)

L_{cz} = K_zL_z = effective length of member for buckling about longitudinal axis, in. (mm)

L_x, L_y, L_z = laterally unbraced length of the member for each axis, in. (mm)

$\bar{r}_o$ = polar radius of gyration about the shear center, in. (mm)

$$\bar{r}_o^2 = x_o^2 + y_o^2 + \frac{I_x + I_y}{A_g} \tag{E4-9}$$

r_x = radius of gyration about x-axis, in. (mm)

r_y = radius of gyration about y-axis, in. (mm)

x_o, y_o = coordinates of the shear center with respect to the centroid, in. (mm)

> **User Note:** For doubly symmetric I-shaped sections, C_w may be taken as $I_y h_o^2/4$, where h_o is the distance between flange centroids, in lieu of a more precise analysis. For tees and double angles, omit the term with C_w when computing F_{ez} and take x_o as 0.

(d) For members with lateral bracing offset from the shear center, the elastic buckling stress, F_e, shall be determined by analysis.

> **User Note:** Members with lateral bracing offset from the shear center are susceptible to constrained-axis torsional buckling, which is discussed in the Commentary.

E5. SINGLE-ANGLE COMPRESSION MEMBERS

The nominal compressive strength, P_n, of single-angle members shall be the lowest value based on the limit states of flexural buckling in accordance with Section E3 or Section E7, as applicable, or flexural-torsional buckling in accordance with Section E4. Flexural-torsional buckling need not be considered when $b/t \le 0.71\sqrt{E/F_y}$.

The effects of eccentricity on single-angle members are permitted to be neglected and the member evaluated as axially loaded using one of the effective slenderness ratios specified in Section E5(a) or E5(b), provided that the following requirements are met:

(1) Members are loaded at the ends in compression through the same one leg.

(2) Members are attached by welding or by connections with a minimum of two bolts.

(3) There are no intermediate transverse loads.

(4) L_c/r as determined in this section does not exceed 200.

(5) For unequal leg angles, the ratio of long leg width to short leg width is less than 1.7.

Single-angle members that do not meet these requirements or the requirements described in Section E5(a) or (b) shall be evaluated for combined axial load and flexure using the provisions of Chapter H.

(a) For angles that are individual members or are web members of planar trusses with adjacent web members attached to the same side of the gusset plate or chord

(1) For equal-leg angles or unequal-leg angles connected through the longer leg

(i) When $\dfrac{L}{r_a} \le 80$

$$\frac{L_c}{r} = 72 + 0.75\frac{L}{r_a} \qquad \text{(E5-1)}$$

(ii) When $\dfrac{L}{r_a} > 80$

$$\frac{L_c}{r} = 32 + 1.25\frac{L}{r_a} \qquad \text{(E5-2)}$$

(2) For unequal-leg angles connected through the shorter leg, L_c/r from Equations E5-1 and E5-2 shall be increased by adding $4[(b_l/b_s)^2 - 1]$, but L_c/r of the members shall not be taken as less than $0.95L/r_z$.

(b) For angles that are web members of box or space trusses with adjacent web members attached to the same side of the gusset plate or chord

(1) For equal-leg angles or unequal-leg angles connected through the longer leg

(i) When $\dfrac{L}{r_a} \le 75$

$$\frac{L_c}{r} = 60 + 0.8\frac{L}{r_a} \qquad \text{(E5-3)}$$

(ii) When $\dfrac{L}{r_a} > 75$

$$\frac{L_c}{r} = 45 + \frac{L}{r_a} \tag{E5-4}$$

(2) For unequal-leg angles with leg length ratios less than 1.7 and connected through the shorter leg, L_c/r from Equations E5-3 and E5-4 shall be increased by adding $6\left[(b_l/b_s)^2 - 1\right]$, but L_c/r of the member shall not be taken as less than $0.82L/r_z$

where

L = length of member between work points at truss chord centerlines, in. (mm)
L_c = effective length of the member for buckling about the minor axis, in. (mm)
b_l = length of longer leg of angle, in. (mm)
b_s = length of shorter leg of angle, in. (mm)
r_a = radius of gyration about the geometric axis parallel to the connected leg, in. (mm)
r_z = radius of gyration about the minor principal axis, in. (mm)

E6. BUILT-UP MEMBERS

1. Compressive Strength

This section applies to built-up members composed of two shapes either (a) interconnected by bolts or welds or (b) with at least one open side interconnected by perforated cover plates or lacing with tie plates. The end connection shall be welded or connected by means of pretensioned bolts with Class A or B faying surfaces.

User Note: It is acceptable to design a bolted end connection of a built-up compression member for the full compressive load with bolts in bearing and bolt design based on the shear strength; however, the bolts must be pretensioned. In built-up compression members, such as double-angle struts in trusses, a small relative slip between the elements can significantly reduce the compressive strength of the strut. Therefore, the connection between the elements at the ends of built-up members should be designed to resist slip.

The nominal compressive strength of built-up members composed of two shapes that are interconnected by bolts or welds shall be determined in accordance with Sections E3, E4 or E7, subject to the following modification. In lieu of more accurate analysis, if the buckling mode involves relative deformations that produce shear forces in the connectors between individual shapes, L_c/r is replaced by $(L_c/r)_m$, determined as follows:

(a) For intermediate connectors that are bolted snug-tight

$$\left(\frac{L_c}{r}\right)_m = \sqrt{\left(\frac{L_c}{r}\right)_o^2 + \left(\frac{a}{r_i}\right)^2} \tag{E6-1}$$

(b) For intermediate connectors that are welded or are connected by means of pretensioned bolts with Class A or B faying surfaces

(1) When $\dfrac{a}{r_i} \le 40$

$$\left(\frac{L_c}{r}\right)_m = \left(\frac{L_c}{r}\right)_o \tag{E6-2a}$$

(2) When $\dfrac{a}{r_i} > 40$

$$\left(\frac{L_c}{r}\right)_m = \sqrt{\left(\frac{L_c}{r}\right)_o^2 + \left(\frac{K_i a}{r_i}\right)^2} \tag{E6-2b}$$

where

$\left(\dfrac{L_c}{r}\right)_m$ = modified slenderness ratio of built-up member

$\left(\dfrac{L_c}{r}\right)_o$ = slenderness ratio of built-up member acting as a unit in the buckling direction being addressed

L_c = effective length of built-up member, in. (mm)

K_i = 0.50 for angles back-to-back
= 0.75 for channels back-to-back
= 0.86 for all other cases

a = distance between connectors, in. (mm)

r_i = minimum radius of gyration of individual component, in. (mm)

2. Dimensional Requirements

Built-up members shall meet the following requirements:

(a) Individual components of compression members composed of two or more shapes shall be connected to one another at intervals, a, such that the slenderness ratio, a/r_i, of each of the component shapes between the fasteners does not exceed three-fourths times the governing slenderness ratio of the built-up member. The least radius of gyration, r_i, shall be used in computing the slenderness ratio of each component part.

(b) At the ends of built-up compression members bearing on base plates or finished surfaces, all components in contact with one another shall be connected by a weld having a length not less than the maximum width of the member or by bolts spaced longitudinally not more than four diameters apart for a distance equal to $1^1/_2$ times the maximum width of the member.

Along the length of built-up compression members between the end connections required in the foregoing, longitudinal spacing of intermittent welds or bolts shall be adequate to provide the required strength. For limitations on the longitudinal spacing of fasteners between elements in continuous contact consisting of a plate and a shape, or two plates, see Section J3.5. Where a component of a built-up compression member consists of an outside plate, the maximum spacing shall not exceed the thickness of the thinner outside plate times $0.75\sqrt{E/F_y}$, nor 12 in.

(300 mm), when intermittent welds are provided along the edges of the components or when fasteners are provided on all gage lines at each section. When fasteners are staggered, the maximum spacing of fasteners on each gage line shall not exceed the thickness of the thinner outside plate times $1.12\sqrt{E/F_y}$ nor 18 in. (460 mm).

(c) Open sides of compression members built up from plates or shapes shall be provided with continuous cover plates perforated with a succession of access holes. The unsupported width of such plates at access holes, as defined in Section B4.1, is assumed to contribute to the available strength provided the following requirements are met:

(1) The width-to-thickness ratio shall conform to the limitations of Section B4.1.

> **User Note:** It is conservative to use the limiting width-to-thickness ratio for Case 7 in Table B4.1a with the width, *b*, taken as the transverse distance between the nearest lines of fasteners. The net area of the plate is taken at the widest hole. In lieu of this approach, the limiting width-to-thickness ratio may be determined through analysis.

(2) The ratio of length (in direction of stress) to width of hole shall not exceed 2.

(3) The clear distance between holes in the direction of stress shall be not less than the transverse distance between nearest lines of connecting fasteners or welds.

(4) The periphery of the holes at all points shall have a minimum radius of $1^1/_2$ in. (38 mm).

(d) As an alternative to perforated cover plates, lacing with tie plates is permitted at each end and at intermediate points if the lacing is interrupted. Tie plates shall be as near the ends as practicable. In members providing available strength, the end tie plates shall have a length of not less than the distance between the lines of fasteners or welds connecting them to the components of the member. Intermediate tie plates shall have a length not less than one-half of this distance. The thickness of tie plates shall be not less than one-fiftieth of the distance between lines of welds or fasteners connecting them to the segments of the members. In welded construction, the welding on each line connecting a tie plate shall total not less than one-third the length of the plate. In bolted construction, the spacing in the direction of stress in tie plates shall be not more than six diameters and the tie plates shall be connected to each segment by at least three fasteners.

(e) Lacing, including flat bars, angles, channels or other shapes employed as lacing, shall be so spaced that L/r of the flange element included between their connections shall not exceed three-fourths times the governing slenderness ratio for the member as a whole. Lacing shall be proportioned to provide a shearing strength normal to the axis of the member equal to 2% of the available compressive strength of the member. For lacing bars arranged in single systems, L/r shall not exceed 140. For double lacing, this ratio shall not exceed 200. Double lacing bars shall be joined at the intersections. For lacing bars in compression, L is permitted to be taken as the unsupported length of the lacing bar between welds or fasteners connecting it to the components of the built-up member for single lacing, and 70% of that distance for double lacing.

User Note: The inclination of lacing bars to the axis of the member shall preferably be not less than 60° for single lacing and 45° for double lacing. When the distance between the lines of welds or fasteners in the flanges is more than 15 in. (380 mm), the lacing should preferably be double or made of angles.

For additional spacing requirements, see Section J3.5.

E7. MEMBERS WITH SLENDER ELEMENTS

This section applies to slender-element compression members, as defined in Section B4.1 for elements in axial compression.

The nominal compressive strength, P_n, shall be the lowest value based on the applicable limit states of flexural buckling, torsional buckling, and flexural-torsional buckling in interaction with local buckling.

$$P_n = F_{cr} A_e \tag{E7-1}$$

where

A_e = summation of the effective areas of the cross section based on reduced effective widths, b_e, d_e or h_e, or the area as given by Equations E7-6 or E7-7, in.^{2} (mm^2).

F_{cr} = critical stress determined in accordance with Section E3 or E4, ksi (MPa). For single angles, determine F_{cr} in accordance with Section E3 only.

User Note: The effective area, A_e, may be determined by deducting from the gross area, A_g, the reduction in area of each slender element determined as $(b - b_e)t$.

1. Slender Element Members Excluding Round HSS

The effective width, b_e, (for tees, this is d_e; for webs, this is h_e) for slender elements is determined as follows:

(a) When $\lambda \le \lambda_r \sqrt{\dfrac{F_y}{F_{cr}}}$

$$b_e = b \tag{E7-2}$$

(b) When $\lambda > \lambda_r \sqrt{\dfrac{F_y}{F_{cr}}}$

$$b_e = b\left(1 - c_1\sqrt{\frac{F_{el}}{F_{cr}}}\right)\sqrt{\frac{F_{el}}{F_{cr}}} \tag{E7-3}$$

TABLE E7.1
Effective Width Imperfection Adjustment Factors, c_1 and c_2

Case	Slender Element	c_1	c_2
(a)	Stiffened elements except walls of square and rectangular HSS	0.18	1.31
(b)	Walls of square and rectangular HSS	0.20	1.38
(c)	All other elements	0.22	1.49

where

b = width of the element (for tees this is d; for webs this is h), in. (mm)

c_1 = effective width imperfection adjustment factor determined from Table E7.1

$$c_2 = \frac{1-\sqrt{1-4c_1}}{2c_1} \tag{E7-4}$$

λ = width-to-thickness ratio for the element as defined in Section B4.1

λ_r = limiting width-to-thickness ratio as defined in Table B4.1a

$$F_{el} = \left(c_2 \frac{\lambda_r}{\lambda}\right)^2 F_y \tag{E7-5}$$

= elastic local buckling stress determined according to Equation E7-5 or an elastic local buckling analysis, ksi (MPa)

2. Round HSS

The effective area, A_e, is determined as follows:

(a) When $\dfrac{D}{t} \le 0.11\dfrac{E}{F_y}$

$$A_e = A_g \tag{E7-6}$$

(b) When $0.11\dfrac{E}{F_y} < \dfrac{D}{t} < 0.45\dfrac{E}{F_y}$

$$A_e = \left[\frac{0.038E}{F_y(D/t)} + \frac{2}{3}\right]A_g \tag{E7-7}$$

where

D = outside diameter of round HSS, in. (mm)

t = thickness of wall, in. (mm)

CHAPTER F

DESIGN OF MEMBERS FOR FLEXURE

This chapter applies to members subject to simple bending about one principal axis. For simple bending, the member is loaded in a plane parallel to a principal axis that passes through the shear center or is restrained against twisting at load points and supports.

The chapter is organized as follows:

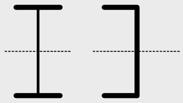
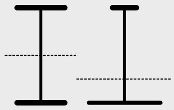
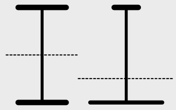
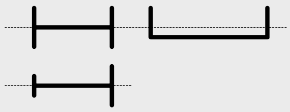
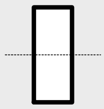
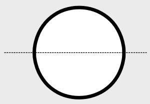
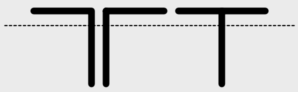
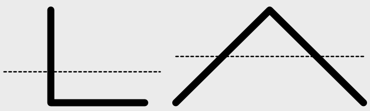
F1. General Provisions
F2. Doubly Symmetric Compact I-Shaped Members and Channels Bent about Their Major Axis
F3. Doubly Symmetric I-Shaped Members with Compact Webs and Noncompact or Slender Flanges Bent about Their Major Axis
F4. Other I-Shaped Members with Compact or Noncompact Webs Bent about Their Major Axis
F5. Doubly Symmetric and Singly Symmetric I-Shaped Members with Slender Webs Bent about Their Major Axis
F6. I-Shaped Members and Channels Bent about Their Minor Axis
F7. Square and Rectangular HSS and Box Sections
F8. Round HSS
F9. Tees and Double Angles Loaded in the Plane of Symmetry
F10. Single Angles
F11. Rectangular Bars and Rounds
F12. Unsymmetrical Shapes
F13. Proportions of Beams and Girders

User Note: For cases not included in this chapter, the following sections apply:

- Chapter G — Design provisions for shear
- H1–H3 — Members subject to biaxial flexure or to combined flexure and axial force
- H3 — Members subject to flexure and torsion
- Appendix 3 — Members subject to fatigue

For guidance in determining the appropriate sections of this chapter to apply, Table User Note F1.1 may be used.

TABLE USER NOTE F1.1
Selection Table for the Application of Chapter F Sections

Section in Chapter F	Cross Section	Flange Slenderness	Web Slenderness	Limit States
F2		C	C	Y, LTB
F3		NC, S	C	LTB, FLB
F4		C, NC, S	C, NC	CFY, LTB, FLB, TFY
F5		C, NC, S	S	CFY, LTB, FLB, TFY
F6		C, NC, S	N/A	Y, FLB
F7		C, NC, S	C, NC, S	Y, FLB, WLB, LTB
F8		N/A	N/A	Y, LB
F9		C, NC, S	N/A	Y, LTB, FLB, WLB
F10		N/A	N/A	Y, LTB, LLB
F11		N/A	N/A	Y, LTB
F12	Unsymmetrical shapes, other than single angles	N/A	N/A	All limit states

Y = yielding, CFY = compression flange yielding, LTB = lateral-torsional buckling, FLB = flange local buckling, WLB = web local buckling, TFY = tension flange yielding, LLB = leg local buckling, LB = local buckling, C = compact, NC = noncompact, S = slender, N/A = not applicable

F1. GENERAL PROVISIONS

The design flexural strength, $\phi_b M_n$, and the allowable flexural strength, M_n/Ω_b, shall be determined as follows:

(a) For all provisions in this chapter

$$\phi_b = 0.90 \text{ (LRFD)} \qquad \Omega_b = 1.67 \text{ (ASD)}$$

and the nominal flexural strength, M_n, shall be determined according to Sections F2 through F13.

(b) The provisions in this chapter are based on the assumption that points of support for beams and girders are restrained against rotation about their longitudinal axis.

(c) For singly symmetric members in single curvature and all doubly symmetric members

The lateral-torsional buckling modification factor, C_b, for nonuniform moment diagrams when both ends of the segment are braced is determined as follows:

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} \tag{F1-1}$$

where

M_{max} = absolute value of maximum moment in the unbraced segment, kip-in. (N-mm)

M_A = absolute value of moment at quarter point of the unbraced segment, kip-in. (N-mm)

M_B = absolute value of moment at centerline of the unbraced segment, kip-in. (N-mm)

M_C = absolute value of moment at three-quarter point of the unbraced segment, kip-in. (N-mm)

User Note: For doubly symmetric members with no transverse loading between brace points, Equation F1-1 reduces to 1.0 for the case of equal end moments of opposite sign (uniform moment), 2.27 for the case of equal end moments of the same sign (reverse curvature bending), and to 1.67 when one end moment equals zero. For singly symmetric members, a more detailed analysis for C_b is presented in the Commentary. The Commentary provides additional equations for C_b that provide improved characterization of the effects of a variety of member boundary conditions.

For cantilevers where warping is prevented at the support and where the free end is unbraced, $C_b = 1.0$.

(d) In singly symmetric members subject to reverse curvature bending, the lateral-torsional buckling strength shall be checked for both flanges. The available flexural strength shall be greater than or equal to the maximum required moment causing compression within the flange under consideration.

F2. DOUBLY SYMMETRIC COMPACT I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MAJOR AXIS

This section applies to doubly symmetric I-shaped members and channels bent about their major axis, having compact webs and compact flanges as defined in Section B4.1 for flexure.

> **User Note:** All current ASTM A6 W, S, M, C and MC shapes except W21×48, W14×99, W14×90, W12×65, W10×12, W8×31, W8×10, W6×15, W6×9, W6×8.5 and M4×6 have compact flanges for F_y = 50 ksi (345 MPa); all current ASTM A6 W, S, M, HP, C and MC shapes have compact webs at $F_y \le 70$ ksi (485 MPa).

The nominal flexural strength, M_n, shall be the lower value obtained according to the limit states of yielding (plastic moment) and lateral-torsional buckling.

1. Yielding

$$M_n = M_p = F_y Z_x \tag{F2-1}$$

where

F_y = specified minimum yield stress of the type of steel being used, ksi (MPa)
Z_x = plastic section modulus about the x-axis, in.^{3} (mm^3)

2. Lateral-Torsional Buckling

(a) When $L_b \le L_p$, the limit state of lateral-torsional buckling does not apply.

(b) When $L_p < L_b \le L_r$

$$M_n = C_b \left[M_p - \left(M_p - 0.7 F_y S_x \right) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \le M_p \tag{F2-2}$$

(c) When $L_b > L_r$

$$M_n = F_{cr} S_x \le M_p \tag{F2-3}$$

where

L_b = length between points that are either braced against lateral displacement of the compression flange or braced against twist of the cross section, in. (mm)

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\dfrac{L_b}{r_{ts}} \right)^2} \sqrt{1 + 0.078 \frac{Jc}{S_x h_o} \left(\frac{L_b}{r_{ts}} \right)^2} \tag{F2-4}$$

= critical stress, ksi (MPa)

E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)
J = torsional constant, in.^{4} (mm^4)
S_x = elastic section modulus taken about the x-axis, in.^{3} (mm^3)
h_o = distance between the flange centroids, in. (mm)

User Note: The square root term in Equation F2-4 may be conservatively taken equal to 1.0.

User Note: Equations F2-3 and F2-4 provide identical solutions to the following expression for lateral-torsional buckling of doubly symmetric sections that has been presented in past editions of this Specification:

$$M_{cr} = C_b \frac{\pi}{L_b} \sqrt{EI_y GJ + \left(\frac{\pi E}{L_b}\right)^2 I_y C_w}$$

The advantage of Equations F2-3 and F2-4 is that the form is very similar to the expression for lateral-torsional buckling of singly symmetric sections given in Equations F4-4 and F4-5.

L_p, the limiting laterally unbraced length for the limit state of yielding, in. (mm), is:

$$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} \tag{F2-5}$$

L_r, the limiting unbraced length for the limit state of inelastic lateral-torsional buckling, in. (mm), is:

$$L_r = 1.95 r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{S_x h_o} + \sqrt{\left(\frac{Jc}{S_x h_o}\right)^2 + 6.76\left(\frac{0.7F_y}{E}\right)^2}} \tag{F2-6}$$

where

r_y = radius of gyration about y-axis, in. (mm)

$$r_{ts}^2 = \frac{\sqrt{I_y C_w}}{S_x} \tag{F2-7}$$

and the coefficient c is determined as follows:

(1) For doubly symmetric I-shapes

$$c = 1 \tag{F2-8a}$$

(2) For channels

$$c = \frac{h_o}{2}\sqrt{\frac{I_y}{C_w}} \tag{F2-8b}$$

where

I_y = moment of inertia about the y-axis, in.^{4} (mm^4)

User Note:
For doubly symmetric I-shapes with rectangular flanges, $C_w = \dfrac{I_y h_o^2}{4}$, and thus, Equation F2-7 becomes

$$r_{ts}^2 = \frac{I_y h_o}{2S_x}$$

r_{ts} may be approximated accurately and conservatively as the radius of gyration of the compression flange plus one-sixth of the web:

$$r_{ts} = \frac{b_f}{\sqrt{12\left(1 + \dfrac{1}{6}\dfrac{ht_w}{b_f t_f}\right)}}$$

F3. DOUBLY SYMMETRIC I-SHAPED MEMBERS WITH COMPACT WEBS AND NONCOMPACT OR SLENDER FLANGES BENT ABOUT THEIR MAJOR AXIS

This section applies to doubly symmetric I-shaped members bent about their major axis having compact webs and noncompact or slender flanges as defined in Section B4.1 for flexure.

User Note: The following shapes have noncompact flanges for F_y = 50 ksi (345 MPa): W21×48, W14×99, W14×90, W12×65, W10×12, W8×31, W8×10, W6×15, W6×9, W6×8.5 and M4×6. All other ASTM A6 W, S and M shapes have compact flanges for $F_y \le$ 50 ksi (345 MPa).

The nominal flexural strength, M_n, shall be the lower value obtained according to the limit states of lateral-torsional buckling and compression flange local buckling.

1. Lateral-Torsional Buckling

For lateral-torsional buckling, the provisions of Section F2.2 shall apply.

2. Compression Flange Local Buckling

(a) For sections with noncompact flanges

$$M_n = M_p - \left(M_p - 0.7F_y S_x\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \quad \text{(F3-1)}$$

(b) For sections with slender flanges

$$M_n = \frac{0.9Ek_c S_x}{\lambda^2} \quad \text{(F3-2)}$$

where

k_c = $\dfrac{4}{\sqrt{h/t_w}}$ and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes

h = distance as defined in Section B4.1b, in. (mm)

λ = $\dfrac{b_f}{2t_f}$

b_f = width of the flange, in. (mm)

t_f = thickness of the flange, in. (mm)

λ_{pf} = λ_p is the limiting slenderness for a compact flange, defined in Table B4.1b

λ_{rf} = λ_r is the limiting slenderness for a noncompact flange, defined in Table B4.1b

F4. OTHER I-SHAPED MEMBERS WITH COMPACT OR NONCOMPACT WEBS BENT ABOUT THEIR MAJOR AXIS

This section applies to doubly symmetric I-shaped members bent about their major axis with noncompact webs and singly symmetric I-shaped members with webs attached to the mid-width of the flanges, bent about their major axis, with compact or noncompact webs, as defined in Section B4.1 for flexure.

User Note: I-shaped members for which this section is applicable may be designed conservatively using Section F5.

The nominal flexural strength, M_n, shall be the lowest value obtained according to the limit states of compression flange yielding, lateral-torsional buckling, compression flange local buckling, and tension flange yielding.

1. Compression Flange Yielding

$$M_n = R_{pc} M_{yc} \tag{F4-1}$$

where

M_{yc} = $F_y S_{xc}$ = yield moment in the compression flange, kip-in. (N-mm)

R_{pc} = web plastification factor, determined in accordance with Section F4.2(c)(6)

S_{xc} = elastic section modulus referred to compression flange, in.^{3} (mm^3)

2. Lateral-Torsional Buckling

(a) When $L_b \le L_p$, the limit state of lateral-torsional buckling does not apply.

(b) When $L_p < L_b \le L_r$

$$M_n = C_b \left[R_{pc} M_{yc} - \left(R_{pc} M_{yc} - F_L S_{xc} \right) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \le R_{pc} M_{yc} \tag{F4-2}$$

(c) When $L_b > L_r$

$$M_n = F_{cr} S_{xc} \le R_{pc} M_{yc} \tag{F4-3}$$

where

(1) M_{yc}, the yield moment in the compression flange, kip-in. (N-mm), is:

$$M_{yc} = F_y S_{xc} \tag{F4-4}$$

(2) F_{cr}, the critical stress, ksi (MPa), is:

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\dfrac{L_b}{r_t}\right)^2} \sqrt{1 + 0.078 \frac{J}{S_{xc} h_o} \left(\frac{L_b}{r_t}\right)^2} \tag{F4-5}$$

For $\dfrac{I_{yc}}{I_y} \le 0.23$, J shall be taken as zero,

where

I_{yc} = moment of inertia of the compression flange about the y-axis, in.^{4} (mm^4)

(3) F_L, nominal compression flange stress above which the inelastic buckling limit states apply, ksi (MPa), is determined as follows:

(i) When $\dfrac{S_{xt}}{S_{xc}} \ge 0.7$

$$F_L = 0.7 F_y \tag{F4-6a}$$

(ii) When $\dfrac{S_{xt}}{S_{xc}} < 0.7$

$$F_L = F_y \frac{S_{xt}}{S_{xc}} \ge 0.5 F_y \tag{F4-6b}$$

where

S_{xt} = elastic section modulus referred to tension flange, in.^{3} (mm^3)

(4) L_p, the limiting laterally unbraced length for the limit state of yielding, in. (mm) is:

$$L_p = 1.1 r_t \sqrt{\frac{E}{F_y}} \tag{F4-7}$$

(5) L_r, the limiting unbraced length for the limit state of inelastic lateral-torsional buckling, in. (mm), is:

$$L_r = 1.95 r_t \frac{E}{F_L} \sqrt{\frac{J}{S_{xc} h_o} + \sqrt{\left(\frac{J}{S_{xc} h_o}\right)^2 + 6.76 \left(\frac{F_L}{E}\right)^2}} \tag{F4-8}$$

(6) R_{pc}, the web plastification factor, is determined as follows:

(i) When $I_{yc}/I_y > 0.23$

(a) When $\dfrac{h_c}{t_w} \le \lambda_{pw}$

$$R_{pc} = \frac{M_p}{M_{yc}} \tag{F4-9a}$$

(b) When $\dfrac{h_c}{t_w} > \lambda_{pw}$

$$R_{pc} = \left[\frac{M_p}{M_{yc}} - \left(\frac{M_p}{M_{yc}} - 1\right)\left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right)\right] \le \frac{M_p}{M_{yc}} \tag{F4-9b}$$

(ii) When $I_{yc}/I_y \le 0.23$

$$R_{pc} = 1.0 \tag{F4-10}$$

where

M_p = $F_yZ_x \le 1.6F_yS_x$

h_c = twice the distance from the centroid to the following: the inside face of the compression flange less the fillet or corner radius, for rolled shapes; the nearest line of fasteners at the compression flange or the inside faces of the compression flange when welds are used, for built-up sections, in. (mm)

λ = $\dfrac{h_c}{t_w}$

λ_{pw} = λ_p, the limiting slenderness for a compact web, given in Table B4.1b

λ_{rw} = λ_r, the limiting slenderness for a noncompact web, given in Table B4.1b

(7) r_t, the effective radius of gyration for lateral-torsional buckling, in. (mm), is determined as follows:

(i) For I-shapes with a rectangular compression flange

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1 + \frac{1}{6}a_w\right)}} \tag{F4-11}$$

where

$$a_w = \frac{h_c t_w}{b_{fc} t_{fc}} \tag{F4-12}$$

b_{fc} = width of compression flange, in. (mm)

t_{fc} = thickness of compression flange, in. (mm)

t_w = thickness of web, in. (mm)

(ii) For I-shapes with a channel cap or a cover plate attached to the compression flange

r_t = radius of gyration of the flange components in flexural compression plus one-third of the web area in compression due to application of major axis bending moment alone, in. (mm)

3. Compression Flange Local Buckling

(a) For sections with compact flanges, the limit state of local buckling does not apply.

(b) For sections with noncompact flanges

$$M_n = R_{pc}M_{yc} - \left(R_{pc}M_{yc} - F_L S_{xc}\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \quad \text{(F4-13)}$$

(c) For sections with slender flanges

$$M_n = \frac{0.9 E k_c S_{xc}}{\lambda^2} \quad \text{(F4-14)}$$

where

F_L is defined in Equations F4-6a and F4-6b

R_{pc} is the web plastification factor, determined by Equation F4-9a, F4-9b or F4-10

$k_c = \dfrac{4}{\sqrt{h/t_w}}$ and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes

$\lambda = \dfrac{b_{fc}}{2t_{fc}}$

$\lambda_{pf} = \lambda_p$, the limiting slenderness for a compact flange, defined in Table B4.1b

$\lambda_{rf} = \lambda_r$, the limiting slenderness for a noncompact flange, defined in Table B4.1b

4. Tension Flange Yielding

(a) When $S_{xt} \ge S_{xc}$, the limit state of tension flange yielding does not apply.

(b) When $S_{xt} < S_{xc}$

$$M_n = R_{pt} M_{yt} \quad \text{(F4-15)}$$

where

$M_{yt} = F_y S_{xt}$ = yield moment in the tension flange, kip-in. (N-mm)

R_{pt}, the web plastification factor corresponding to the tension flange yielding limit state, is determined as follows:

(1) When $I_{yc}/I_y > 0.23$

(i) When $\dfrac{h_c}{t_w} \le \lambda_{pw}$

$$R_{pt} = \frac{M_p}{M_{yt}} \quad \text{(F4-16a)}$$

(ii) When $\dfrac{h_c}{t_w} > \lambda_{pw}$

$$R_{pt} = \left[\frac{M_p}{M_{yt}} - \left(\frac{M_p}{M_{yt}} - 1\right)\left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right)\right] \le \frac{M_p}{M_{yt}} \tag{F4-16b}$$

(2) When $I_{yc}/I_y \le 0.23$

$$R_{pt} = 1.0 \tag{F4-17}$$

where

$M_p = F_y Z_x \le 1.6 F_y S_x$

$\lambda = \dfrac{h_c}{t_w}$

$\lambda_{pw} = \lambda_p$, the limiting slenderness for a compact web, defined in Table B4.1b

$\lambda_{rw} = \lambda_r$, the limiting slenderness for a noncompact web, defined in Table B4.1b

F5. DOUBLY SYMMETRIC AND SINGLY SYMMETRIC I-SHAPED MEMBERS WITH SLENDER WEBS BENT ABOUT THEIR MAJOR AXIS

This section applies to doubly symmetric and singly symmetric I-shaped members with slender webs attached to the mid-width of the flanges and bent about their major axis as defined in Section B4.1 for flexure.

The nominal flexural strength, M_n, shall be the lowest value obtained according to the limit states of compression flange yielding, lateral-torsional buckling, compression flange local buckling, and tension flange yielding.

1. Compression Flange Yielding

$$M_n = R_{pg} F_y S_{xc} \tag{F5-1}$$

2. Lateral-Torsional Buckling

$$M_n = R_{pg} F_{cr} S_{xc} \tag{F5-2}$$

(a) When $L_b \le L_p$, the limit state of lateral-torsional buckling does not apply.

(b) When $L_p < L_b \le L_r$

$$F_{cr} = C_b\left[F_y - \left(0.3F_y\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le F_y \tag{F5-3}$$

(c) When $L_b > L_r$

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\dfrac{L_b}{r_t}\right)^2} \le F_y \tag{F5-4}$$

where

L_p is defined by Equation F4-7

$$L_r = \pi r_t \sqrt{\frac{E}{0.7F_y}} \tag{F5-5}$$

r_t = effective radius of gyration for lateral-torsional buckling as defined in Section F4, in. (mm)

R_{pg}, the bending strength reduction factor, is:

$$R_{pg} = 1 - \frac{a_w}{1{,}200 + 300a_w}\left(\frac{h_c}{t_w} - 5.7\sqrt{\frac{E}{F_y}}\right) \leq 1.0 \tag{F5-6}$$

and

a_w is defined by Equation F4-12, but shall not exceed 10

3. Compression Flange Local Buckling

$$M_n = R_{pg}\,F_{cr}S_{xc} \tag{F5-7}$$

(a) For sections with compact flanges, the limit state of compression flange local buckling does not apply.

(b) For sections with noncompact flanges

$$F_{cr} = \left[F_y - \left(0.3F_y\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] \tag{F5-8}$$

(c) For sections with slender flanges

$$F_{cr} = \frac{0.9Ek_c}{\left(\frac{b_f}{2t_f}\right)^2} \tag{F5-9}$$

where

$k_c = \frac{4}{\sqrt{h/t_w}}$ and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes

$\lambda = \frac{b_{fc}}{2t_{fc}}$

$\lambda_{pf} = \lambda_p$, the limiting slenderness for a compact flange, defined in Table B4.1b

$\lambda_{rf} = \lambda_r$, the limiting slenderness for a noncompact flange, defined in Table B4.1b

4. Tension Flange Yielding

(a) When $S_{xt} \geq S_{xc}$, the limit state of tension flange yielding does not apply.

(b) When $S_{xt} < S_{xc}$

$$M_n = F_yS_{xt} \tag{F5-10}$$

F6. I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MINOR AXIS

This section applies to I-shaped members and channels bent about their minor axis.

The nominal flexural strength, M_n, shall be the lower value obtained according to the limit states of yielding (plastic moment) and flange local buckling.

1. Yielding

$$M_n = M_p = F_y Z_y \leq 1.6 F_y S_y \tag{F6-1}$$

where

S_y = elastic section modulus taken about the y-axis, in.^{3} (mm^3)

Z_y = plastic section modulus taken about the y-axis, in.^{3} (mm^3)

2. Flange Local Buckling

(a) For sections with compact flanges, the limit state of flange local buckling does not apply.

> **User Note:** All current ASTM A6 W, S, M, C and MC shapes except W21×48, W14×99, W14×90, W12×65, W10×12, W8×31, W8×10, W6×15, W6×9, W6×8.5 and M4×6 have compact flanges at F_y = 50 ksi (345 MPa).

(b) For sections with noncompact flanges

$$M_n = M_p - \left(M_p - 0.7 F_y S_y\right)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right) \tag{F6-2}$$

(c) For sections with slender flanges

$$M_n = F_{cr} S_y \tag{F6-3}$$

where

$$F_{cr} = \frac{0.69E}{\left(\dfrac{b}{t_f}\right)^2} \tag{F6-4}$$

b = for flanges of I-shaped members, half the full flange width, b_f; for flanges of channels, the full nominal dimension of the flange, in. (mm)

t_f = thickness of the flange, in. (mm)

$\lambda = \dfrac{b}{t_f}$

λ_{pf} = λ_p, the limiting slenderness for a compact flange, defined in Table B4.1b

λ_{rf} = λ_r, the limiting slenderness for a noncompact flange, defined in Table B4.1b

F7. SQUARE AND RECTANGULAR HSS AND BOX SECTIONS

This section applies to square and rectangular HSS, and box sections bent about either axis, having compact, noncompact or slender webs or flanges, as defined in Section B4.1 for flexure.

The nominal flexural strength, M_n, shall be the lowest value obtained according to the limit states of yielding (plastic moment), flange local buckling, web local buckling, and lateral-torsional buckling under pure flexure.

1. Yielding

$$M_n = M_p = F_y Z \tag{F7-1}$$

where

Z = plastic section modulus about the axis of bending, in.^{3} (mm^3)

2. Flange Local Buckling

(a) For compact sections, the limit state of flange local buckling does not apply.

(b) For sections with noncompact flanges

$$M_n = M_p - \left(M_p - F_y S\right)\left(3.57\frac{b}{t_f}\sqrt{\frac{F_y}{E}} - 4.0\right) \le M_p \tag{F7-2}$$

where

S = elastic section modulus about the axis of bending, in.^{3} (mm^3)

b = width of compression flange as defined in Section B4.1b, in. (mm)

(c) For sections with slender flanges

$$M_n = F_y S_e \tag{F7-3}$$

where

S_e = effective section modulus determined with the effective width, b_e, of the compression flange taken as:

(1) For HSS

$$b_e = 1.92 t_f \sqrt{\frac{E}{F_y}}\left(1 - \frac{0.38}{b/t_f}\sqrt{\frac{E}{F_y}}\right) \le b \tag{F7-4}$$

(2) For box sections

$$b_e = 1.92 t_f \sqrt{\frac{E}{F_y}}\left(1 - \frac{0.34}{b/t_f}\sqrt{\frac{E}{F_y}}\right) \le b \tag{F7-5}$$

3. Web Local Buckling

(a) For compact sections, the limit state of web local buckling does not apply.

(b) For sections with noncompact webs

$$M_n = M_p - \left(M_p - F_y S\right)\left(0.305\frac{h}{t_w}\sqrt{\frac{F_y}{E}} - 0.738\right) \le M_p \tag{F7-6}$$

where
h = depth of web, as defined in Section B4.1b, in. (mm)

(c) For sections with slender webs

(1) Compression flange yielding

$$M_n = R_{pg}\, F_y S \tag{F7-7}$$

(2) Compression flange local buckling

$$M_n = R_{pg}\, F_{cr} S_{xc} \tag{F7-8}$$

and

$$F_{cr} = \frac{0.9 E k_c}{\left(\dfrac{b}{t_f}\right)^2} \tag{F7-9}$$

where
R_{pg} is defined by Equation F5-6 with $a_w = 2ht_w/(bt_f)$
$k_c = 4.0$

User Note: When Equation F7-9 results in the stress, F_{cr}, being greater than F_y, member strength will be limited by one of the other limit states in Section F7.

User Note: There are no HSS with slender webs.

4. Lateral-Torsional Buckling

(a) When $L_b \le L_p$, the limit state of lateral-torsional buckling does not apply.

(b) When $L_p < L_b \le L_r$

$$M_n = C_b\left[M_p - \left(M_p - 0.7F_y S_x\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p \tag{F7-10}$$

(c) When $L_b > L_r$

$$M_n = 2EC_b \frac{\sqrt{JA_g}}{L_b/r_y} \le M_p \tag{F7-11}$$

where
A_g = gross cross-sectional area of member, in.^{2} (mm^2)
L_p, the limiting laterally unbraced length for the limit state of yielding, in. (mm), is:

$$L_p = 0.13 E r_y \frac{\sqrt{JA_g}}{M_p} \tag{F7-12}$$

L_r, the limiting laterally unbraced length for the limit state of inelastic lateral-torsional buckling, in. (mm), is:

$$L_r = 2Er_y \frac{\sqrt{JA_g}}{0.7F_y S_x} \quad \text{(F7-13)}$$

User Note: Lateral-torsional buckling will not occur in square sections or sections bending about their minor axis. In HSS sizes, deflection will usually control before there is a significant reduction in flexural strength due to lateral-torsional buckling. The same is true for box sections, and lateral-torsional buckling will usually only be a consideration for sections with high depth-to-width ratios.

F8. ROUND HSS

This section applies to round HSS having D/t ratios of less than $\dfrac{0.45E}{F_y}$.

The nominal flexural strength, M_n, shall be the lower value obtained according to the limit states of yielding (plastic moment) and local buckling.

1. Yielding

$$M_n = M_p = F_y Z \quad \text{(F8-1)}$$

2. Local Buckling

(a) For compact sections, the limit state of flange local buckling does not apply.

(b) For noncompact sections

$$M_n = \left[\frac{0.021E}{\left(\dfrac{D}{t}\right)} + F_y\right] S \quad \text{(F8-2)}$$

(c) For sections with slender walls

$$M_n = F_{cr} S \quad \text{(F8-3)}$$

where

D = outside diameter of round HSS, in. (mm)

$$F_{cr} = \frac{0.33E}{\left(\dfrac{D}{t}\right)} \quad \text{(F8-4)}$$

t = design wall thickness of HSS member, in. (mm)

F9. TEES AND DOUBLE ANGLES LOADED IN THE PLANE OF SYMMETRY

This section applies to tees and double angles loaded in the plane of symmetry.

The nominal flexural strength, M_n, shall be the lowest value obtained according to the limit states of yielding (plastic moment), lateral-torsional buckling, flange local buckling, and local buckling of tee stems and double angle web legs.

1. Yielding

$$M_n = M_p \tag{F9-1}$$

where

(a) For tee stems and web legs in tension

$$M_p = F_y Z_x \leq 1.6M_y \tag{F9-2}$$

where

M_y = yield moment about the axis of bending, kip-in. (N-mm)

$$= F_y S_x \tag{F9-3}$$

(b) For tee stems in compression

$$M_p = M_y \tag{F9-4}$$

(c) For double angles with web legs in compression

$$M_p = 1.5M_y \tag{F9-5}$$

2. Lateral-Torsional Buckling

(a) For stems and web legs in tension

(1) When $L_b \leq L_p$, the limit state of lateral-torsional buckling does not apply.

(2) When $L_p < L_b \leq L_r$

$$M_n = M_p - (M_p - M_y)\left(\frac{L_b - L_p}{L_r - L_p}\right) \tag{F9-6}$$

(3) When $L_b > L_r$

$$M_n = M_{cr} \tag{F9-7}$$

where

$$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} \tag{F9-8}$$

$$L_r = 1.95 \left(\frac{E}{F_y}\right) \frac{\sqrt{I_y J}}{S_x} \sqrt{2.36 \left(\frac{F_y}{E}\right) \frac{dS_x}{J} + 1} \tag{F9-9}$$

$$M_{cr} = \frac{1.95E}{L_b}\sqrt{I_y J}\left(B + \sqrt{1+B^2}\right) \quad \text{(F9-10)}$$

$$B = 2.3\left(\frac{d}{L_b}\right)\sqrt{\frac{I_y}{J}} \quad \text{(F9-11)}$$

d = depth of tee or width of web leg in tension, in. (mm)

(b) For stems and web legs in compression anywhere along the unbraced length, M_{cr} is given by Equation F9-10 with

$$B = -2.3\left(\frac{d}{L_b}\right)\sqrt{\frac{I_y}{J}} \quad \text{(F9-12)}$$

where

d = depth of tee or width of web leg in compression, in. (mm)

(1) For tee stems

$$M_n = M_{cr} \leq M_y \quad \text{(F9-13)}$$

(2) For double-angle web legs, M_n shall be determined using Equations F10-2 and F10-3 with M_{cr} determined using Equation F9-10 and M_y determined using Equation F9-3.

3. Flange Local Buckling of Tees and Double-Angle Legs

(a) For tee flanges

(1) For sections with a compact flange in flexural compression, the limit state of flange local buckling does not apply.

(2) For sections with a noncompact flange in flexural compression

$$M_n = \left[M_p - (M_p - 0.7F_y S_{xc})\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] \leq 1.6M_y \quad \text{(F9-14)}$$

(3) For sections with a slender flange in flexural compression

$$M_n = \frac{0.7ES_{xc}}{\left(\frac{b_f}{2t_f}\right)^2} \quad \text{(F9-15)}$$

where

S_{xc} = elastic section modulus referred to the compression flange, in.^{3} (mm^3)

$\lambda = \frac{b_f}{2t_f}$

λ_{pf} = λ_p, the limiting slenderness for a compact flange, defined in Table B4.1b

λ_{rf} = λ_r, the limiting slenderness for a noncompact flange, defined in Table B4.1b

(b) For double-angle flange legs

The nominal moment strength, M_n, for double angles with the flange legs in compression shall be determined in accordance with Section F10.3, with S_c referred to the compression flange.

4. Local Buckling of Tee Stems and Double-Angle Web Legs in Flexural Compression

(a) For tee stems

$$M_n = F_{cr} S_x \quad \text{(F9-16)}$$

where

S_x = elastic section modulus, in.^{3} (mm^3)

F_{cr}, the critical stress, is determined as follows:

(1) When $\frac{d}{t_w} \le 0.84\sqrt{\frac{E}{F_y}}$

$$F_{cr} = F_y \quad \text{(F9-17)}$$

(2) When $0.84\sqrt{\frac{E}{F_y}} < \frac{d}{t_w} \le 1.52\sqrt{\frac{E}{F_y}}$

$$F_{cr} = \left(1.43 - 0.515\frac{d}{t_w}\sqrt{\frac{F_y}{E}}\right)F_y \quad \text{(F9-18)}$$

(3) When $\frac{d}{t_w} > 1.52\sqrt{\frac{E}{F_y}}$

$$F_{cr} = \frac{1.52E}{\left(\frac{d}{t_w}\right)^2} \quad \text{(F9-19)}$$

(b) For double-angle web legs

The nominal moment strength, M_n, for double angles with the web legs in compression shall be determined in accordance with Section F10.3, with S_c taken as the elastic section modulus.

F10. SINGLE ANGLES

This section applies to single angles with and without continuous lateral restraint along their length.

Single angles with continuous lateral-torsional restraint along the length are permitted to be designed on the basis of geometric axis (x, y) bending. Single angles without continuous lateral-torsional restraint along the length shall be designed using the provisions for principal axis bending except where the provision for bending about a geometric axis is permitted.

If the moment resultant has components about both principal axes, with or without axial load, or the moment is about one principal axis and there is axial load, the combined stress ratio shall be determined using the provisions of Section H2.

User Note: For geometric axis design, use section properties computed about the x- and y-axis of the angle, parallel and perpendicular to the legs. For principal axis design, use section properties computed about the major and minor principal axes of the angle.

The nominal flexural strength, M_n, shall be the lowest value obtained according to the limit states of yielding (plastic moment), lateral-torsional buckling, and leg local buckling.

User Note: For bending about the minor principal axis, only the limit states of yielding and leg local buckling apply.

1. Yielding

$$M_n = 1.5M_y \tag{F10-1}$$

2. Lateral-Torsional Buckling

For single angles without continuous lateral-torsional restraint along the length

(a) When $\dfrac{M_y}{M_{cr}} \le 1.0$

$$M_n = \left(1.92 - 1.17\sqrt{\frac{M_y}{M_{cr}}}\right)M_y \le 1.5M_y \tag{F10-2}$$

(b) When $\dfrac{M_y}{M_{cr}} > 1.0$

$$M_n = \left(0.92 - \frac{0.17M_{cr}}{M_y}\right)M_{cr} \tag{F10-3}$$

where

M_{cr} , the elastic lateral-torsional buckling moment, is determined as follows:

(1) For bending about the major principal axis of single angles

$$M_{cr} = \frac{9EAr_z tC_b}{8L_b}\left[\sqrt{1+\left(4.4\frac{\beta_w r_z}{L_b t}\right)^2} + 4.4\frac{\beta_w r_z}{L_b t}\right] \tag{F10-4}$$

where

C_b is computed using Equation F1-1 with a maximum value of 1.5

A = cross-sectional area of angle, in.^{2} (mm^2)

L_b = laterally unbraced length of member, in. (mm)

r_z = radius of gyration about the minor principal axis, in. (mm)
t = thickness of angle leg, in. (mm)
β_w = section property for single angles about major principal axis, in. (mm). β_w is positive with short legs in compression and negative with long legs in compression for unequal-leg angles, and zero for equal-leg angles. If the long leg is in compression anywhere along the unbraced length of the member, the negative value of β_w shall be used.

User Note: The equation for β_w and values for common angle sizes are listed in the Commentary.

(2) For bending about one of the geometric axes of an equal-leg angle with no axial compression

(i) With no lateral-torsional restraint:

(a) With maximum compression at the toe

$$M_{cr} = \frac{0.58Eb^4tC_b}{L_b^2}\left[\sqrt{1+0.88\left(\frac{L_bt}{b^2}\right)^2}-1\right] \tag{F10-5a}$$

(b) With maximum tension at the toe

$$M_{cr} = \frac{0.58Eb^4tC_b}{L_b^2}\left[\sqrt{1+0.88\left(\frac{L_bt}{b^2}\right)^2}+1\right] \tag{F10-5b}$$

where

M_y shall be taken as 0.80 times the yield moment calculated using the geometric section modulus.
b = width of leg, in. (mm)

(ii) With lateral-torsional restraint at the point of maximum moment only:

M_{cr} shall be taken as 1.25 times M_{cr} computed using Equation F10-5a or F10-5b.
M_y shall be taken as the yield moment calculated using the geometric section modulus.

User Note: M_n may be taken as M_y for single angles with their vertical leg toe in compression, and having a span-to-depth ratio less than or equal to

$$\frac{1.64E}{F_y}\sqrt{\left(\frac{t}{b}\right)^2-1.4\frac{F_y}{E}}$$

3. Leg Local Buckling

The limit state of leg local buckling applies when the toe of the leg is in compression.

(a) For compact sections, the limit state of leg local buckling does not apply.

(b) For sections with noncompact legs

$$M_n = F_y S_c \left[2.43 - 1.72\left(\frac{b}{t}\right)\sqrt{\frac{F_y}{E}}\right] \tag{F10-6}$$

(c) For sections with slender legs

$$M_n = F_{cr} S_c \tag{F10-7}$$

where

$$F_{cr} = \frac{0.71E}{\left(\frac{b}{t}\right)^2} \tag{F10-8}$$

S_c = elastic section modulus to the toe in compression relative to the axis of bending, in.^{3} (mm^3). For bending about one of the geometric axes of an equal-leg angle with no lateral-torsional restraint, S_c shall be 0.80 of the geometric axis section modulus.

b = full width of leg in compression, in. (mm)

F11. RECTANGULAR BARS AND ROUNDS

This section applies to rectangular bars bent about either geometric axis and rounds.

The nominal flexural strength, M_n, shall be the lower value obtained according to the limit states of yielding (plastic moment) and lateral-torsional buckling.

1. Yielding

For rectangular bars with $\frac{L_b d}{t^2} \le \frac{0.08E}{F_y}$ bent about their major axis, rectangular bars bent about their minor axis, and rounds

$$M_n = M_p = F_y Z \le 1.6 F_y S_x \tag{F11-1}$$

where

d = depth of rectangular bar, in. (mm)

t = width of rectangular bar parallel to axis of bending, in. (mm)

2. Lateral-Torsional Buckling

(a) For rectangular bars with $\frac{L_b d}{t^2} \le \frac{0.08E}{F_y}$ bent about their major axis, the limit state of lateral-torsional buckling does not apply.

(b) For rectangular bars with $\dfrac{0.08E}{F_y} < \dfrac{L_b d}{t^2} \le \dfrac{1.9E}{F_y}$ bent about their major axis

$$M_n = C_b\left[1.52 - 0.274\left(\frac{L_b d}{t^2}\right)\frac{F_y}{E}\right]M_y \le M_p \tag{F11-2}$$

where

L_b = length between points that are either braced against lateral displacement of the compression region or between points braced to prevent twist of the cross section, in. (mm)

(c) For rectangular bars with $\dfrac{L_b d}{t^2} > \dfrac{1.9E}{F_y}$ bent about their major axis

$$M_n = F_{cr} S_x \le M_p \tag{F11-3}$$

where

$$F_{cr} = \frac{1.9EC_b}{\dfrac{L_b d}{t^2}} \tag{F11-4}$$

(d) For rounds and rectangular bars bent about their minor axis, the limit state of lateral-torsional buckling need not be considered.

F12. UNSYMMETRICAL SHAPES

This section applies to all unsymmetrical shapes except single angles.

The nominal flexural strength, M_n, shall be the lowest value obtained according to the limit states of yielding (yield moment), lateral-torsional buckling, and local buckling where

$$M_n = F_n S_{min} \tag{F12-1}$$

where

S_{min} = minimum elastic section modulus relative to the axis of bending, in.^{3} (mm^3)

User Note: The design provisions within this section can be overly conservative for certain shapes, unbraced lengths and moment diagrams. To improve economy, the provisions of Appendix 1.3 are recommended as an alternative for determining the nominal flexural strength of members of unsymmetrical shape.

1. Yielding

$$F_n = F_y \tag{F12-2}$$

2. Lateral-Torsional Buckling

$$F_n = F_{cr} \le F_y \tag{F12-3}$$

where

F_{cr} = lateral-torsional buckling stress for the section as determined by analysis, ksi (MPa)

User Note: In the case of Z-shaped members, it is recommended that F_{cr} be taken as $0.5F_{cr}$ of a channel with the same flange and web properties.

3. Local Buckling

$$F_n = F_{cr} \leq F_y \tag{F12-4}$$

where

F_{cr} = local buckling stress for the section as determined by analysis, ksi (MPa)

F13. PROPORTIONS OF BEAMS AND GIRDERS

1. Strength Reductions for Members with Holes in the Tension Flange

This section applies to rolled or built-up shapes and cover-plated beams with holes, proportioned on the basis of flexural strength of the gross section.

In addition to the limit states specified in other sections of this Chapter, the nominal flexural strength, M_n, shall be limited according to the limit state of tensile rupture of the tension flange.

(a) When $F_u A_{fn} \geq Y_t F_y A_{fg}$, the limit state of tensile rupture does not apply.

(b) When $F_u A_{fn} < Y_t F_y A_{fg}$, the nominal flexural strength, M_n, at the location of the holes in the tension flange shall not be taken greater than

$$M_n = \frac{F_u A_{fn}}{A_{fg}} S_x \tag{F13-1}$$

where

A_{fg} = gross area of tension flange, calculated in accordance with the provisions of Section B4.3a, in.^{2} (mm^2)

A_{fn} = net area of tension flange, calculated in accordance with the provisions of Section B4.3b, in.^{2} (mm^2)

F_u = specified minimum tensile strength, ksi (MPa)

S_x = minimum elastic section modulus taken about the x-axis, in.^{3} (mm^3)

Y_t = 1.0 for $F_y/F_u \leq 0.8$
 = 1.1 otherwise

2. Proportioning Limits for I-Shaped Members

Singly symmetric I-shaped members shall satisfy the following limit:

$$0.1 \leq \frac{I_{yc}}{I_y} \leq 0.9 \tag{F13-2}$$

I-shaped members with slender webs shall also satisfy the following limits:

(a) When $\frac{a}{h} \le 1.5$

$$\left(\frac{h}{t_w}\right)_{max} = 12.0\sqrt{\frac{E}{F_y}} \quad \text{(F13-3)}$$

(b) When $\frac{a}{h} > 1.5$

$$\left(\frac{h}{t_w}\right)_{max} = \frac{0.40E}{F_y} \quad \text{(F13-4)}$$

where

a = clear distance between transverse stiffeners, in. (mm)

In unstiffened girders, h/t_w shall not exceed 260. The ratio of the web area to the compression flange area shall not exceed 10.

3. Cover Plates

For members with cover plates, the following provisions apply:

(a) Flanges of welded beams or girders are permitted to be varied in thickness or width by splicing a series of plates or by the use of cover plates.

(b) High-strength bolts or welds connecting flange to web, or cover plate to flange, shall be proportioned to resist the total horizontal shear resulting from the bending forces on the girder. The longitudinal distribution of these bolts or intermittent welds shall be in proportion to the intensity of the shear.

(c) However, the longitudinal spacing shall not exceed the maximum specified for compression or tension members in Sections E6 or D4, respectively. Bolts or welds connecting flange to web shall also be proportioned to transmit to the web any loads applied directly to the flange, unless provision is made to transmit such loads by direct bearing.

(d) Partial-length cover plates shall be extended beyond the theoretical cutoff point and the extended portion shall be attached to the beam or girder by high-strength bolts in a slip-critical connection or fillet welds. The attachment shall, at the applicable strength given in Sections J2.2, J3.8 or B3.11, develop the cover plate's portion of the flexural strength in the beam or girder at the theoretical cutoff point.

(e) For welded cover plates, the welds connecting the cover plate termination to the beam or girder shall be continuous welds along both edges of the cover plate in the length a', defined in the following, and shall develop the cover plate's portion of the available strength of the beam or girder at the distance a' from the end of the cover plate.

(1) When there is a continuous weld equal to or larger than three-fourths of the plate thickness across the end of the plate

$$a' = w \tag{F13-5}$$

where
w = width of cover plate, in. (mm)

(2) When there is a continuous weld smaller than three-fourths of the plate thickness across the end of the plate

$$a' = 1.5w \tag{F13-6}$$

(3) When there is no weld across the end of the plate

$$a' = 2w \tag{F13-7}$$

4. Built-Up Beams

Where two or more beams or channels are used side by side to form a flexural member, they shall be connected together in compliance with Section E6.2. When concentrated loads are carried from one beam to another or distributed between the beams, diaphragms having sufficient stiffness to distribute the load shall be welded or bolted between the beams.

5. Unbraced Length for Moment Redistribution

For moment redistribution in indeterminate beams according to Section B3.3, the laterally unbraced length, L_b, of the compression flange adjacent to the redistributed end moment locations shall not exceed L_m determined as follows.

(a) For doubly symmetric and singly symmetric I-shaped beams with the compression flange equal to or larger than the tension flange loaded in the plane of the web

$$L_m = \left[0.12 + 0.076\left(\frac{M_1}{M_2}\right)\right]\left(\frac{E}{F_y}\right)r_y \tag{F13-8}$$

(b) For solid rectangular bars and symmetric box beams bent about their major axis

$$L_m = \left[0.17 + 0.10\left(\frac{M_1}{M_2}\right)\right]\left(\frac{E}{F_y}\right)r_y \geq 0.10\left(\frac{E}{F_y}\right)r_y \tag{F13-9}$$

where
F_y = specified minimum yield stress of the compression flange, ksi (MPa)
M_1 = smaller moment at end of unbraced length, kip-in. (N-mm)
M_2 = larger moment at end of unbraced length, kip-in. (N-mm)
r_y = radius of gyration about y-axis, in. (mm)
(M_1/M_2) is positive when moments cause reverse curvature and negative for single curvature

There is no limit on L_b for members with round or square cross sections or for any beam bent about its minor axis.

CHAPTER G

DESIGN OF MEMBERS FOR SHEAR

This chapter addresses webs of singly or doubly symmetric members subject to shear in the plane of the web, single angles and HSS subject to shear, and shear in the weak direction of singly or doubly symmetric shapes.

The chapter is organized as follows:

User Note: For cases not included in this chapter, the following sections apply:
- H3.3 Unsymmetric sections
- J4.2 Shear strength of connecting elements
- J10.6 Web panel zone shear

G1. GENERAL PROVISIONS

The design shear strength, $\phi_v V_n$, and the allowable shear strength, V_n / Ω_v, shall be determined as follows:

(a) For all provisions in this chapter except Section G2.1(a)

$$\phi_v = 0.90 \text{ (LRFD)} \qquad \Omega_v = 1.67 \text{ (ASD)}$$

(b) The nominal shear strength, V_n, shall be determined according to Sections G2 through G7.

G2. I-SHAPED MEMBERS AND CHANNELS

1. Shear Strength of Webs without Tension Field Action

The nominal shear strength, V_n, is:

$$V_n = 0.6 F_y A_w C_{v1} \qquad \text{(G2-1)}$$

where

F_y = specified minimum yield stress of the type of steel being used, ksi (MPa)
A_w = area of web, the overall depth times the web thickness, dt_w, in.^{2} (mm^2)

(a) For webs of rolled I-shaped members with $h/t_w \le 2.24\sqrt{E/F_y}$

$$\phi_v = 1.00 \text{ (LRFD)} \qquad \Omega_v = 1.50 \text{ (ASD)}$$

and

$$C_{v1} = 1.0 \tag{G2-2}$$

where

E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)
h = clear distance between flanges less the fillet at each flange, in. (mm)
t_w = thickness of web, in. (mm)

> **User Note:** All current ASTM A6 W, S and HP shapes except W44×230, W40×149, W36×135, W33×118, W30×90, W24×55, W16×26 and W12×14 meet the criteria stated in Section G2.1(a) for F_y = 50 ksi (345 MPa).

(b) For all other I-shaped members and channels

(1) The web shear strength coefficient, C_{v1}, is determined as follows:

(i) When $h/t_w \le 1.10\sqrt{k_v E/F_y}$

$$C_{v1} = 1.0 \tag{G2-3}$$

where

h = for built-up welded sections, the clear distance between flanges, in. (mm)
= for built-up bolted sections, the distance between fastener lines, in. (mm)

(ii) When $h/t_w > 1.10\sqrt{k_v E/F_y}$

$$C_{v1} = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} \tag{G2-4}$$

(2) The web plate shear buckling coefficient, k_v, is determined as follows:

(i) For webs without transverse stiffeners

$$k_v = 5.34$$

(ii) For webs with transverse stiffeners

$$k_v = 5 + \frac{5}{(a/h)^2} \tag{G2-5}$$

$$= 5.34 \text{ when } a/h > 3.0$$

where

a = clear distance between transverse stiffeners, in. (mm)

> **User Note:** For all ASTM A6 W, S, M and HP shapes except M12.5×12.4, M12.5×11.6, M12×11.8, M12×10.8, M12×10, M10×8 and M10×7.5, when F_y = 50 ksi (345 MPa), C_{v1} = 1.0.

2. Shear Strength of Interior Web Panels with $a/h \le 3$ Considering Tension Field Action

The nominal shear strength, V_n, is determined as follows:

(a) When $h/t_w \le 1.10\sqrt{k_v E/F_y}$

$$V_n = 0.6F_y A_w \tag{G2-6}$$

(b) When $h/t_w > 1.10\sqrt{k_v E/F_y}$

(1) When $2A_w/(A_{fc}+A_{ft}) \le 2.5$, $h/b_{fc} \le 6.0$ and $h/b_{ft} \le 6.0$

$$V_n = 0.6F_y A_w \left[C_{v2} + \frac{1-C_{v2}}{1.15\sqrt{1+(a/h)^2}}\right] \tag{G2-7}$$

(2) Otherwise

$$V_n = 0.6F_y A_w \left[C_{v2} + \frac{1-C_{v2}}{1.15\left[a/h+\sqrt{1+(a/h)^2}\,\right]}\right] \tag{G2-8}$$

where

The web shear buckling coefficient, C_{v2}, is determined as follows:

(i) When $h/t_w \le 1.10\sqrt{k_v E/F_y}$

$$C_{v2} = 1.0 \tag{G2-9}$$

(ii) When $1.10\sqrt{k_v E/F_y} < h/t_w \le 1.37\sqrt{k_v E/F_y}$

$$C_{v2} = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} \tag{G2-10}$$

(iii) When $h/t_w > 1.37\sqrt{k_v E/F_y}$

$$C_{v2} = \frac{1.51k_v E}{(h/t_w)^2 F_y} \tag{G2-11}$$

A_{fc} = area of compression flange, in.^{2} (mm^2)
A_{ft} = area of tension flange, in.^{2} (mm^2)
b_{fc} = width of compression flange, in. (mm)
b_{ft} = width of tension flange, in. (mm)
k_v is as defined in Section G2.1

The nominal shear strength is permitted to be taken as the larger of the values from Sections G2.1 and G2.2.

User Note: Section G2.1 may predict a higher strength for members that do not meet the requirements of Section G2.2(b)(1).

3. Transverse Stiffeners

For transverse stiffeners, the following shall apply.

(a) Transverse stiffeners are not required where $h / t_w \leq 2.46\sqrt{E / F_y}$, or where the available shear strength provided in accordance with Section G2.1 for $k_v = 5.34$ is greater than the required shear strength.

(b) Transverse stiffeners are permitted to be stopped short of the tension flange, provided bearing is not needed to transmit a concentrated load or reaction. The weld by which transverse stiffeners are attached to the web shall be terminated not less than four times nor more than six times the web thickness from the near toe of the web-to-flange weld or web-to-flange fillet. When single stiffeners are used, they shall be attached to the compression flange if it consists of a rectangular plate, to resist any uplift tendency due to torsion in the flange.

(c) Bolts connecting stiffeners to the girder web shall be spaced not more than 12 in. (300 mm) on center. If intermittent fillet welds are used, the clear distance between welds shall not be more than 16 times the web thickness nor more than 10 in. (250 mm).

(d) $$(b/t)_{st} \leq 0.56\sqrt{\frac{E}{F_{yst}}} \tag{G2-12}$$

(e) $$I_{st} \geq I_{st2} + (I_{st1} - I_{st2})\,\rho_w \tag{G2-13}$$

where

F_{yst} = specified minimum yield stress of the stiffener material, ksi (MPa)

F_{yw} = specified minimum yield stress of the web material, ksi (MPa)

I_{st} = moment of inertia of the transverse stiffeners about an axis in the web center for stiffener pairs, or about the face in contact with the web plate for single stiffeners, in.^{4} (mm^4)

$$I_{st1} = \frac{h^4 \rho_{st}^{1.3}}{40}\left(\frac{F_{yw}}{E}\right)^{1.5} \tag{G2-14}$$

= minimum moment of inertia of the transverse stiffeners required for development of the full shear post buckling resistance of the stiffened web panels, $V_r = V_{c1}$, in.^{4} (mm^4)

$$I_{st2} = \left[\frac{2.5}{(a/h)^2} - 2\right] b_p t_w^3 \geq 0.5\, b_p t_w^3 \tag{G2-15}$$

= minimum moment of inertia of the transverse stiffeners required for development of the web shear buckling resistance, $V_r = V_{c2}$, in.^{4} (mm^4)

V_{c1} = available shear strength calculated with V_n as defined in Section G2.1 or G2.2, as applicable, kips (N)

V_{c2} = available shear strength, kips (N), calculated with $V_n = 0.6F_yA_wC_{v2}$

V_r = required shear strength in the panel being considered, kips (N)
b_p = smaller of the dimension a and h, in. (mm)
$(b/t)_{st}$ = width-to-thickness ratio of the stiffener
ρ_{st} = larger of F_{yw}/F_{yst} and 1.0
ρ_w = maximum shear ratio, $\left(\dfrac{V_r - V_{c2}}{V_{c1} - V_{c2}}\right) \geq 0$, within the web panels on each side of the transverse stiffener

User Note: I_{st} may conservatively be taken as I_{st1}. Equation G2-15 provides the minimum stiffener moment of inertia required to attain the web shear post buckling resistance according to Sections G2.1 and G2.2, as applicable. If less post buckling shear strength is required, Equation G2-13 provides a linear interpolation between the minimum moment of inertia required to develop web shear buckling and that required to develop the web shear post buckling strength.

G3. SINGLE ANGLES AND TEES

The nominal shear strength, V_n, of a single-angle leg or a tee stem is:

$$V_n = 0.6F_y btC_{v2} \tag{G3-1}$$

where

C_{v2} = web shear buckling strength coefficient, as defined in Section G2.2 with $h/t_w = b/t$ and $k_v = 1.2$
b = width of the leg resisting the shear force or depth of the tee stem, in. (mm)
t = thickness of angle leg or tee stem, in. (mm)

G4. RECTANGULAR HSS, BOX SECTIONS, AND OTHER SINGLY AND DOUBLY SYMMETRIC MEMBERS

The nominal shear strength, V_n, is:

$$V_n = 0.6F_y A_w C_{v2} \tag{G4-1}$$

For rectangular HSS and box sections

A_w = $2ht$, in.^{2} (mm^2)
C_{v2} = web shear buckling strength coefficient, as defined in Section G2.2, with $h/t_w = h/t$ and $k_v = 5$
h = width resisting the shear force, taken as the clear distance between the flanges less the inside corner radius on each side for HSS or the clear distance between flanges for box sections, in. (mm). If the corner radius is not known, h shall be taken as the corresponding outside dimension minus 3 times the thickness.
t = design wall thickness, as defined in Section B4.2, in. (mm)

For other singly or doubly symmetric shapes

A_w = area of web or webs, taken as the sum of the overall depth times the web thickness, dt_w, in.^{2} (mm^2)
C_{v2} = web shear buckling strength coefficient, as defined in Section G2.2, with $h/t_w = h/t$ and $k_v = 5$

h = width resisting the shear force, in. (mm)
= for built-up welded sections, the clear distance between flanges, in. (mm)
= for built-up bolted sections, the distance between fastener lines, in. (mm)
t = web thickness, as defined in Section B4.2, in. (mm)

G5. ROUND HSS

The nominal shear strength, V_n, of round HSS, according to the limit states of shear yielding and shear buckling, shall be determined as:

$$V_n = F_{cr} A_g / 2 \tag{G5-1}$$

where

F_{cr} shall be the larger of

$$F_{cr} = \frac{1.60E}{\sqrt{\frac{L_v}{D}}\left(\frac{D}{t}\right)^{\frac{5}{4}}} \tag{G5-2a}$$

and

$$F_{cr} = \frac{0.78E}{\left(\frac{D}{t}\right)^{\frac{3}{2}}} \tag{G5-2b}$$

but shall not exceed $0.6F_y$

A_g = gross cross-sectional area of member, in.^{2} (mm^2)
D = outside diameter, in. (mm)
L_v = distance from maximum to zero shear force, in. (mm)
t = design wall thickness, in. (mm)

User Note: The shear buckling equations, Equations G5-2a and G5-2b, will control for D/t over 100, high-strength steels, and long lengths. For standard sections, shear yielding will usually control and $F_{cr} = 0.6F_y$.

G6. WEAK-AXIS SHEAR IN DOUBLY SYMMETRIC AND SINGLY SYMMETRIC SHAPES

For doubly and singly symmetric shapes loaded in the weak axis without torsion, the nominal shear strength, V_n, for each shear resisting element is:

$$V_n = 0.6F_y b_f t_f C_{v2} \tag{G6-1}$$

where

C_{v2} = web shear buckling strength coefficient, as defined in Section G2.2 with $h/t_w = b_f/2t_f$ for I-shaped members and tees, or $h/t_w = b_f/t_f$ for channels, and $k_v = 1.2$
b_f = width of flange, in. (mm)
t_f = thickness of flange, in. (mm)

User Note: For all ASTM A6 W, S, M and HP shapes, when $F_y \leq 70$ ksi (485 MPa), $C_{v2} = 1.0$.

G7. BEAMS AND GIRDERS WITH WEB OPENINGS

The effect of all web openings on the shear strength of steel and composite beams shall be determined. Reinforcement shall be provided when the required strength exceeds the available strength of the member at the opening.

CHAPTER H

DESIGN OF MEMBERS FOR COMBINED FORCES AND TORSION

This chapter addresses members subject to axial force and flexure about one or both axes, with or without torsion, and members subject to torsion only.

The chapter is organized as follows:

H1. Doubly and Singly Symmetric Members Subject to Flexure and Axial Force
H2. Unsymmetric and Other Members Subject to Flexure and Axial Force
H3. Members Subject to Torsion and Combined Torsion, Flexure, Shear, and/or Axial Force
H4. Rupture of Flanges with Holes Subjected to Tension

User Note: For composite members, see Chapter I.

H1. DOUBLY AND SINGLY SYMMETRIC MEMBERS SUBJECT TO FLEXURE AND AXIAL FORCE

1. Doubly and Singly Symmetric Members Subject to Flexure and Compression

The interaction of flexure and compression in doubly symmetric members and singly symmetric members constrained to bend about a geometric axis (*x* and/or *y*) shall be limited by Equations H1-1a and H1-1b.

User Note: Section H2 is permitted to be used in lieu of the provisions of this section.

(a) When $\dfrac{P_r}{P_c} \geq 0.2$

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0 \qquad \text{(H1-1a)}$$

(b) When $\dfrac{P_r}{P_c} < 0.2$

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0 \qquad \text{(H1-1b)}$$

where

P_r = required axial strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kips (N)

P_c = available axial strength determined in accordance with Chapter E, kips (N)

M_r = required flexural strength, determined in accordance with Chapter C, using LRFD or ASD load combinations, kip-in. (N-mm)

M_c = available flexural strength, determined in accordance with Chapter F, kip-in. (N-mm)
x = subscript relating symbol to major axis bending
y = subscript relating symbol to minor axis bending

For design according to Section B3.1 (LRFD):

P_r = required axial strength, determined in accordance with Chapter C, using LRFD load combinations, kips (N)
P_c = $\phi_c P_n$ = design axial strength, determined in accordance with Chapter E, kips (N)
M_r = required flexural strength, determined in accordance with Chapter C, using LRFD load combinations, kip-in. (N-mm)
M_c = $\phi_b M_n$ = design flexural strength determined in accordance with Chapter F, kip-in. (N-mm)
ϕ_c = resistance factor for compression = 0.90
ϕ_b = resistance factor for flexure = 0.90

For design according to Section B3.2 (ASD):

P_r = required axial strength, determined in accordance with Chapter C, using ASD load combinations, kips (N)
P_c = P_n / Ω_c = allowable axial strength, determined in accordance with Chapter E, kips (N)
M_r = required flexural strength, determined in accordance with Chapter C, using ASD load combinations, kip-in. (N-mm)
M_c = M_n / Ω_b = allowable flexural strength, determined in accordance with Chapter F, kip-in. (N-mm)
Ω_c = safety factor for compression = 1.67
Ω_b = safety factor for flexure = 1.67

2. Doubly and Singly Symmetric Members Subject to Flexure and Tension

The interaction of flexure and tension in doubly symmetric members and singly symmetric members constrained to bend about a geometric axis (x and/or y) shall be limited by Equations H1-1a and H1-1b,

where

For design according to Section B3.1 (LRFD):

P_r = required axial strength, determined in accordance with Chapter C, using LRFD load combinations, kips (N)
P_c = $\phi_t P_n$ = design axial strength, determined in accordance with Section D2, kips (N)
M_r = required flexural strength, determined in accordance with Chapter C, using LRFD load combinations, kip-in. (N-mm)
M_c = $\phi_b M_n$ = design flexural strength, determined in accordance with Chapter F, kip-in. (N-mm)
ϕ_t = resistance factor for tension (see Section D2)
ϕ_b = resistance factor for flexure = 0.90

For design according to Section B3.2 (ASD):

P_r = required axial strength, determined in accordance with Chapter C, using ASD load combinations, kips (N)
$P_c = P_n / \Omega_t$ = allowable axial strength, determined in accordance with Section D2, kips (N)
M_r = required flexural strength, determined in accordance with Chapter C, using ASD load combinations, kip-in. (N-mm)
$M_c = M_n / \Omega_b$ = allowable flexural strength, determined in accordance with Chapter F, kip-in. (N-mm)
Ω_t = safety factor for tension (see Section D2)
Ω_b = safety factor for flexure = 1.67

For doubly symmetric members, C_b in Chapter F is permitted to be multiplied by $\sqrt{1+\frac{\alpha P_r}{P_{ey}}}$ for axial tension that acts concurrently with flexure,

where

$$P_{ey} = \frac{\pi^2 E I_y}{L_b^2} \tag{H1-2}$$

α = 1.0 (LRFD); α = 1.6 (ASD)

and

E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)
I_y = moment of inertia about the y-axis, in.^{4} (mm^4)
L_b = length between points that are either braced against lateral displacement of the compression flange or braced against twist of the cross section, in.^{4} (mm^4)

3. Doubly Symmetric Rolled Compact Members Subject to Single-Axis Flexure and Compression

For doubly symmetric rolled compact members, with the effective length for torsional buckling less than or equal to the effective length for y-axis flexural buckling, $L_{cz} \le L_{cy}$, subjected to flexure and compression with moments primarily about their major axis, it is permissible to address the two independent limit states, in-plane instability and out-of-plane buckling or lateral-torsional buckling, separately in lieu of the combined approach provided in Section H1.1,

where

L_{cy} = effective length for buckling about the y-axis, in. (mm)
L_{cz} = effective length for buckling about the longitudinal axis, in. (mm)

For members with $M_{ry}/M_{cy} \ge 0.05$, the provisions of Section H1.1 shall be followed.

(a) For the limit state of in-plane instability, Equations H1-1a and H1-1b shall be used with P_c taken as the available compressive strength in the plane of bending and M_{cx} taken as the available flexural strength based on the limit state of yielding.

(b) For the limit state of out-of-plane buckling and lateral-torsional buckling

$$\frac{P_r}{P_{cy}}\left(1.5-0.5\frac{P_r}{P_{cy}}\right)+\left(\frac{M_{rx}}{C_b M_{cx}}\right)^2 \le 1.0 \tag{H1-3}$$

where

P_{cy} = available compressive strength out of the plane of bending, kips (N)

C_b = lateral-torsional buckling modification factor determined from Section F1

M_{cx} = available lateral-torsional strength for major axis flexure determined in accordance with Chapter F using C_b = 1.0, kip-in. (N-mm)

User Note: In Equation H1-3, $C_b M_{cx}$ may be larger than $\phi_b M_{px}$ in LRFD or M_{px}/Ω_b in ASD. The yielding resistance of the beam-column is captured by Equations H1-1.

H2. UNSYMMETRIC AND OTHER MEMBERS SUBJECT TO FLEXURE AND AXIAL FORCE

This section addresses the interaction of flexure and axial stress for shapes not covered in Section H1. It is permitted to use the provisions of this Section for any shape in lieu of the provisions of Section H1.

$$\left|\frac{f_{ra}}{F_{ca}}+\frac{f_{rbw}}{F_{cbw}}+\frac{f_{rbz}}{F_{cbz}}\right| \le 1.0 \tag{H2-1}$$

where

f_{ra} = required axial stress at the point of consideration, determined in accordance with Chapter C, using LRFD or ASD load combinations, ksi (MPa)

F_{ca} = available axial stress at the point of consideration, ksi (MPa)

f_{rbw}, f_{rbz} = required flexural stress at the point of consideration, determined in accordance with Chapter C, using LRFD or ASD load combinations, ksi (MPa)

F_{cbw},F_{cbz} = available flexural stress at the point of consideration, ksi (MPa)

w = subscript relating symbol to major principal axis bending

z = subscript relating symbol to minor principal axis bending

User Note: The subscripts w and z refer to the principal axes of the unsymmetric cross section. For doubly symmetric cross sections, these can be replaced by the x and y subscripts.

For design according to Section B3.1 (LRFD)

f_{ra} = required axial stress at the point of consideration, determined in accordance with Chapter C, using LRFD load combinations, ksi (MPa)

F_{ca} = $\phi_c F_{cr}$ = design axial stress, determined in accordance with Chapter E for compression or Section D2 for tension, ksi (MPa)

f_{rbw}, f_{rbz} = required flexural stress at the point of consideration, determined in accordance with Chapter C, using LRFD load combinations, ksi (MPa)

$F_{cbw}, F_{cbz} = \dfrac{\phi_b M_n}{S}$ = design flexural stress, determined in accordance with Chapter F, ksi (MPa). Use the section modulus, S, for the specific location in the cross section and consider the sign of the stress.

ϕ_c = resistance factor for compression = 0.90

ϕ_t = resistance factor for tension (Section D2)

ϕ_b = resistance factor for flexure = 0.90

For design according to Section B3.2 (ASD)

f_{ra} = required axial stress at the point of consideration, determined in accordance with Chapter C, using ASD load combinations, ksi (MPa)

F_{ca} = allowable axial stress, determined in accordance with Chapter E for compression, or Section D2 for tension, ksi (MPa)

f_{rbw}, f_{rbz} = required flexural stress at the point of consideration, determined in accordance with Chapter C, using ASD load combinations, ksi (MPa)

$F_{cbw}, F_{cbz} = \dfrac{M_n}{\Omega_b S}$ = allowable flexural stress, determined in accordance with Chapter F, ksi (MPa). Use the section modulus, S, for the specific location in the cross section and consider the sign of the stress.

Ω_c = safety factor for compression = 1.67

Ω_t = safety factor for tension (see Section D2)

Ω_b = safety factor for flexure = 1.67

Equation H2-1 shall be evaluated using the principal bending axes by considering the sense of the flexural stresses at the critical points of the cross section. The flexural terms are either added to or subtracted from the axial term as applicable. When the axial force is compression, second-order effects shall be included according to the provisions of Chapter C.

A more detailed analysis of the interaction of flexure and tension is permitted in lieu of Equation H2-1.

H3. MEMBERS SUBJECT TO TORSION AND COMBINED TORSION, FLEXURE, SHEAR, AND/OR AXIAL FORCE

1. Round and Rectangular HSS Subject to Torsion

The design torsional strength, $\phi_T T_n$, and the allowable torsional strength, T_n / Ω_T, for round and rectangular HSS according to the limit states of torsional yielding and torsional buckling shall be determined as follows:

$$T_n = F_{cr}C \tag{H3-1}$$

$$\phi_T = 0.90 \text{ (LRFD)} \qquad \Omega_T = 1.67 \text{ (ASD)}$$

where

C = HSS torsional constant, in.^{3} (mm^3)

The critical stress, F_{cr}, shall be determined as follows:

(a) For round HSS, F_{cr} shall be the larger of

(1) $$F_{cr} = \frac{1.23E}{\sqrt{\dfrac{L}{D}}\left(\dfrac{D}{t}\right)^{\frac{5}{4}}} \tag{H3-2a}$$

and

(2) $$F_{cr} = \frac{0.60E}{\left(\dfrac{D}{t}\right)^{\frac{3}{2}}} \tag{H3-2b}$$

but shall not exceed $0.6F_y$,

where

D = outside diameter, in. (mm)

L = length of member, in. (mm)

t = design wall thickness defined in Section B4.2, in. (mm)

(b) For rectangular HSS

(1) When $h/t \le 2.45\sqrt{E/F_y}$

$$F_{cr} = 0.6F_y \tag{H3-3}$$

(2) When $2.45\sqrt{E/F_y} < h/t \le 3.07\sqrt{E/F_y}$

$$F_{cr} = \frac{0.6F_y\left(2.45\sqrt{E/F_y}\right)}{\left(\dfrac{h}{t}\right)} \tag{H3-4}$$

(3) When $3.07\sqrt{E/F_y} < h/t \le 260$

$$F_{cr} = \frac{0.458\pi^2 E}{\left(\dfrac{h}{t}\right)^2} \tag{H3-5}$$

where

h = flat width of longer side, as defined in Section B4.1b(d), in. (mm)

User Note: The torsional constant, C, may be conservatively taken as:

For round HSS: $C = \dfrac{\pi(D-t)^2 t}{2}$

For rectangular HSS: $C = 2(B-t)(H-t)t - 4.5(4-\pi)t^3$

2. HSS Subject to Combined Torsion, Shear, Flexure and Axial Force

When the required torsional strength, T_r, is less than or equal to 20% of the available torsional strength, T_c, the interaction of torsion, shear, flexure and/or axial force for HSS may be determined by Section H1 and the torsional effects may be neglected. When T_r exceeds 20% of T_c, the interaction of torsion, shear, flexure and/or axial force shall be limited, at the point of consideration, by

$$\left(\frac{P_r}{P_c}+\frac{M_r}{M_c}\right)+\left(\frac{V_r}{V_c}+\frac{T_r}{T_c}\right)^2 \le 1.0 \tag{H3-6}$$

where

For design according to Section B3.1 (LRFD)

P_r = required axial strength, determined in accordance with Chapter C, using LRFD load combinations, kips (N)

P_c = ϕP_n = design tensile or compressive strength, determined in accordance with Chapter D or E, kips (N)

M_r = required flexural strength, determined in accordance with Chapter C, using LRFD load combinations, kip-in. (N-mm)

M_c = $\phi_b M_n$ = design flexural strength, determined in accordance with Chapter F, kip-in. (N-mm)

V_r = required shear strength, determined in accordance with Chapter C, using LRFD load combinations, kips (N)

V_c = $\phi_v V_n$ = design shear strength, determined in accordance with Chapter G, kips (N)

T_r = required torsional strength, determined in accordance with Chapter C, using LRFD load combinations, kip-in. (N-mm)

T_c = $\phi_T T_n$ = design torsional strength, determined in accordance with Section H3.1, kip-in. (N-mm)

For design according to Section B3.2 (ASD)

P_r = required axial strength, determined in accordance with Chapter C, using ASD load combinations, kips (N)

P_c = P_n / Ω = allowable tensile or compressive strength, determined in accordance with Chapter D or E, kips (N)

M_r = required flexural strength, determined in accordance with Chapter C, using ASD load combinations, kip-in. (N-mm)

M_c = M_n / Ω_b = allowable flexural strength, determined in accordance with Chapter F, kip-in. (N-mm)

V_r = required shear strength, determined in accordance with Chapter C, using ASD load combinations, kips (N)

V_c = V_n / Ω_v = allowable shear strength, determined in accordance with Chapter G, kips (N)

T_r = required torsional strength, determined in accordance with Chapter C, using ASD load combinations, kip-in. (N-mm)

T_c = T_n / Ω_T = allowable torsional strength, determined in accordance with Section H3.1, kip-in. (N-mm)

3. Non-HSS Members Subject to Torsion and Combined Stress

The available torsional strength for non-HSS members shall be the lowest value obtained according to the limit states of yielding under normal stress, shear yielding under shear stress, or buckling, determined as follows:

$$\phi_T = 0.90 \text{ (LRFD)} \qquad \Omega_T = 1.67 \text{ (ASD)}$$

(a) For the limit state of yielding under normal stress

$$F_n = F_y \tag{H3-7}$$

(b) For the limit state of shear yielding under shear stress

$$F_n = 0.6F_y \tag{H3-8}$$

(c) For the limit state of buckling

$$F_n = F_{cr} \tag{H3-9}$$

where

F_{cr} = buckling stress for the section as determined by analysis, ksi (MPa)

Constrained local yielding is permitted adjacent to areas that remain elastic.

H4. RUPTURE OF FLANGES WITH HOLES SUBJECTED TO TENSION

At locations of bolt holes in flanges subjected to tension under combined axial force and major axis flexure, flange tensile rupture strength shall be limited by Equation H4-1. Each flange subjected to tension due to axial force and flexure shall be checked separately.

$$\frac{P_r}{P_c} + \frac{M_{rx}}{M_{cx}} \leq 1.0 \tag{H4-1}$$

where

P_r = required axial strength of the member at the location of the bolt holes, determined in accordance with Chapter C, positive in tension and negative in compression, kips (N)

P_c = available axial strength for the limit state of tensile rupture of the net section at the location of bolt holes, kips (N)

M_{rx} = required flexural strength at the location of the bolt holes, determined in accordance with Chapter C, positive for tension in the flange under consideration and negative for compression, kip-in. (N-mm)

M_{cx} = available flexural strength about x-axis for the limit state of tensile rupture of the flange, determined according to Section F13.1. When the limit state of tensile rupture in flexure does not apply, use the plastic bending moment, M_p, determined with bolt holes not taken into consideration, kip-in. (N-mm)

For design according to Section B3.1 (LRFD):

P_r = required axial strength, determined in accordance with Chapter C, using LRFD load combinations, kips (N)

P_c = $\phi_t P_n$ = design axial strength for the limit state of tensile rupture, determined in accordance with Section D2(b), kips (N)

M_{rx} = required flexural strength, determined in accordance with Chapter C, using LRFD load combinations, kip-in. (N-mm)

M_{cx} = $\phi_b M_n$ = design flexural strength determined in accordance with Section F13.1 or the plastic bending moment, M_p, determined with bolt holes not taken into consideration, as applicable, kip-in. (N-mm)

ϕ_t = resistance factor for tensile rupture = 0.75

ϕ_b = resistance factor for flexure = 0.90

For design according to Section B3.2 (ASD):

P_r = required axial strength, determined in accordance with Chapter C, using ASD load combinations, kips (N)

P_c = P_n / Ω_t = allowable axial strength for the limit state of tensile rupture, determined in accordance with Section D2(b), kips (N)

M_{rx} = required flexural strength, determined in accordance with Chapter C, using ASD load combinations, kip-in. (N-mm)

M_{cx} = M_n / Ω_b = allowable flexural strength determined in accordance with Section F13.1, or the plastic bending moment, M_p, determined with bolt holes not taken into consideration, as applicable, kip-in. (N-mm)

Ω_t = safety factor for tensile rupture = 2.00

Ω_b = safety factor for flexure = 1.67

CHAPTER I

DESIGN OF COMPOSITE MEMBERS

This chapter addresses composite members composed of rolled or built-up structural steel shapes or HSS and structural concrete acting together, and steel beams supporting a reinforced concrete slab so interconnected that the beams and the slab act together to resist bending. Simple and continuous composite beams with steel headed stud anchors, encased and filled beams, constructed with or without temporary shores, are included.

The chapter is organized as follows:

I1. GENERAL PROVISIONS

In determining load effects in members and connections of a structure that includes composite members, consideration shall be given to the effective sections at the time each increment of load is applied.

1. Concrete and Steel Reinforcement

The design, detailing and material properties related to the concrete and reinforcing steel portions of composite construction shall comply with the reinforced concrete and reinforcing bar design specifications stipulated by the applicable building code. Additionally, the provisions in the *Building Code Requirements for Structural Concrete and Commentary* (ACI 318) and the *Metric Building Code Requirements for Structural Concrete and Commentary* (ACI 318M), subsequently referred to in Chapter I collectively as ACI 318, shall apply with the following exceptions and limitations:

(a) ACI 318 provisions specifically intended for composite columns shall be excluded in their entirety.

(b) Concrete and steel reinforcement material limitations shall be as specified in Section I1.3.

(c) Transverse reinforcement limitations shall be as specified in Section I2.1a(b) and I2.2a(c), in addition to those specified in ACI 318.

Minimum longitudinal reinforcement limitations shall be as specified in Sections I2.1a(c) and I2.2a(c). Concrete and steel reinforcement components designed in accordance with ACI 318 shall be based on a level of loading corresponding to LRFD load combinations.

User Note: It is the intent of this Specification that the concrete and reinforcing steel portions of composite concrete members are detailed utilizing the noncomposite provisions of ACI 318, as modified by this Specification. All requirements specific to composite members are covered in this Specification.

Note that the design basis for ACI 318 is strength design. Designers using ASD for steel must be conscious of the different load factors.

2. Nominal Strength of Composite Sections

The nominal strength of composite sections shall be determined in accordance with either the plastic stress distribution method, the strain compatibility method, the elastic stress distribution method, or the effective stress-strain method, as defined in this section.

The tensile strength of the concrete shall be neglected in the determination of the nominal strength of composite members.

Local buckling effects shall be evaluated for filled composite members, as defined in Section I1.4. Local buckling effects need not be evaluated for encased composite members.

2a. Plastic Stress Distribution Method

For the plastic stress distribution method, the nominal strength shall be computed assuming that steel components have reached a stress of F_y in either tension or compression, and concrete components in compression due to axial force and/or flexure have reached a stress of $0.85f'_c$, where f'_c is the specified compressive strength of concrete, ksi (MPa). For round HSS filled with concrete, a stress of $0.95f'_c$ is permitted to be used for concrete components in compression due to axial force and/or flexure to account for the effects of concrete confinement.

2b. Strain Compatibility Method

For the strain compatibility method, a linear distribution of strains across the section shall be assumed, with the maximum concrete compressive strain equal to 0.003 in./in. (mm/mm). The stress-strain relationships for steel and concrete shall be obtained from tests or from published results.

User Note: The strain compatibility method can be used to determine nominal strength for irregular sections and for cases where the steel does not exhibit elasto-plastic behavior. General guidelines for the strain compatibility method for encased members subjected to axial load, flexure or both are given in AISC Design Guide 6, *Load and Resistance Factor Design of W-Shapes Encased in Concrete*, and ACI 318.

2c. Elastic Stress Distribution Method

For the elastic stress distribution method, the nominal strength shall be determined from the superposition of elastic stresses for the limit state of yielding or concrete crushing.

2d. Effective Stress-Strain Method

For the effective stress-strain method, the nominal strength shall be computed assuming strain compatibility, and effective stress-strain relationships for steel and concrete components accounting for the effects of local buckling, yielding, interaction and concrete confinement.

3. Material Limitations

Concrete, structural steel, and steel reinforcing bars in composite systems shall meet the following limitations:

(a) For the determination of the available strength, concrete shall have a compressive strength, f'_c, of not less than 3 ksi (21 MPa) nor more than 10 ksi (69 MPa) for normal weight concrete and not less than 3 ksi (21 MPa) nor more than 6 ksi (41 MPa) for lightweight concrete.

> **User Note:** Higher strength concrete material properties may be used for stiffness calculations but may not be relied upon for strength calculations unless justified by testing or analysis.

(b) The specified minimum yield stress of structural steel used in calculating the strength of composite members shall not exceed 75 ksi (525 MPa).

(c) The specified minimum yield stress of reinforcing bars used in calculating the strength of composite members shall not exceed 80 ksi (550 MPa).

4. Classification of Filled Composite Sections for Local Buckling

For compression, filled composite sections are classified as compact, noncompact or slender. For a section to qualify as compact, the maximum width-to-thickness ratio of its compression steel elements shall not exceed the limiting width-to-thickness ratio, λ_p, from Table I1.1a. If the maximum width-to-thickness ratio of one or more steel compression elements exceeds λ_p, but does not exceed λ_r from Table I1.1a, the filled composite section is noncompact. If the maximum width-to-thickness ratio of any compression steel element exceeds λ_r, the section is slender. The maximum permitted width-to-thickness ratio shall be as specified in the table.

For flexure, filled composite sections are classified as compact, noncompact or slender. For a section to qualify as compact, the maximum width-to-thickness ratio of its compression steel elements shall not exceed the limiting width-to-thickness ratio, λ_p, from Table I1.1b. If the maximum width-to-thickness ratio of one or more steel compression elements exceeds λ_p, but does not exceed λ_r from Table I1.1b, the section is noncompact. If the width-to-thickness ratio of any steel element exceeds λ_r, the section is slender. The maximum permitted width-to-thickness ratio shall be as specified in the table.

Refer to Section B4.1b for definitions of width, b and D, and thickness, t, for rectangular and round HSS sections and box sections of uniform thickness.

TABLE I1.1a
Limiting Width-to-Thickness Ratios for Compression Steel Elements in Composite Members Subject to Axial Compression for Use with Section I2.2

Description of Element	Width-to-Thickness Ratio	λ_p Compact/ Noncompact	λ_r Noncompact/ Slender	Maximum Permitted
Walls of Rectangular HSS and Box Sections of Uniform Thickness	b/t	$2.26\sqrt{\dfrac{E}{F_y}}$	$3.00\sqrt{\dfrac{E}{F_y}}$	$5.00\sqrt{\dfrac{E}{F_y}}$
Round HSS	D/t	$\dfrac{0.15E}{F_y}$	$\dfrac{0.19E}{F_y}$	$\dfrac{0.31E}{F_y}$

TABLE I1.1b
Limiting Width-to-Thickness Ratios for Compression Steel Elements in Composite Members Subject to Flexure for Use with Section I3.4

Description of Element	Width-to-Thickness Ratio	λ_p Compact/ Noncompact	λ_r Noncompact/ Slender	Maximum Permitted
Flanges of Rectangular HSS and Box Sections of Uniform Thickness	b/t	$2.26\sqrt{\dfrac{E}{F_y}}$	$3.00\sqrt{\dfrac{E}{F_y}}$	$5.00\sqrt{\dfrac{E}{F_y}}$
Webs of Rectangular HSS and Box Sections of Uniform Thickness	h/t	$3.00\sqrt{\dfrac{E}{F_y}}$	$5.70\sqrt{\dfrac{E}{F_y}}$	$5.70\sqrt{\dfrac{E}{F_y}}$
Round HSS	D/t	$\dfrac{0.09E}{F_y}$	$\dfrac{0.31E}{F_y}$	$\dfrac{0.31E}{F_y}$

User Note: All current ASTM A500 Grade C square HSS sections are compact according to the limits of Table I1.1a and Table I1.1b, except HSS7×7×1/8, HSS8×8×1/8, HSS10×10×3/16 and HSS12×12×3/16, which are noncompact for both axial compression and flexure, and HSS9×9×1/8, which is slender for both axial compression and flexure.

All current ASTM A500 Grade C round HSS sections are compact according to the limits of Table I1.1a and Table I1.1b for both axial compression and flexure, with the exception of HSS6.625×0.125, HSS7.000×0.125, HSS10.000× 0.188, HSS14.000×0.250, HSS16.000×0.250, and HSS20.000×0.375, which are noncompact for flexure.

5. Stiffness for Calculation of Required Strengths

For the direct analysis method of design, the required strengths of encased composite members and filled composite members shall be determined using the provisions of Section C2 and the following requirements:

(1) The nominal flexural stiffness of members subject to net compression shall be taken as the effective stiffness of the composite section, EI_{eff}, as defined in Section I2.

(2) The nominal axial stiffness of members subject to net compression shall be taken as the summation of the elastic axial stiffnesses of each component.

(3) Stiffness of members subject to net tension shall be taken as the stiffness of the bare steel members in accordance with Chapter C.

(4) The stiffness reduction parameter, τ_b, shall be taken as 0.8.

User Note: Taken together, the stiffness reduction factors require the use of $0.64EI_{eff}$ for the flexural stiffness and 0.8 times the nominal axial stiffness of encased composite members and filled composite members subject to net compression in the analysis.

Stiffness values appropriate for the calculation of deflections and for use with the effective length method are discussed in the Commentary.

I2. AXIAL FORCE

This section applies to encased composite members and filled composite members subject to axial force.

1. Encased Composite Members

1a. Limitations

For encased composite members, the following limitations shall be met:

(a) The cross-sectional area of the steel core shall comprise at least 1% of the total composite cross section.

(b) Concrete encasement of the steel core shall be reinforced with continuous longitudinal bars and lateral ties or spirals.

Where lateral ties are used, a minimum of either a No. 3 (10 mm) bar spaced at a maximum of 12 in. (300 mm) on center, or a No. 4 (13 mm) bar or larger spaced at a maximum of 16 in. (400 mm) on center shall be used. Deformed wire or welded wire reinforcement of equivalent area are permitted.

Maximum spacing of lateral ties shall not exceed 0.5 times the least column dimension.

(c) The minimum reinforcement ratio for continuous longitudinal reinforcing, ρ_{sr}, shall be 0.004, where ρ_{sr} is given by:

$$\rho_{sr} = \frac{A_{sr}}{A_g} \tag{I2-1}$$

where

A_g = gross area of composite member, in.^{2} (mm^2)

A_{sr} = area of continuous reinforcing bars, in.^{2} (mm^2)

User Note: Refer to ACI 318 for additional tie and spiral reinforcing provisions.

1b. Compressive Strength

The design compressive strength, $\phi_c P_n$, and allowable compressive strength, P_n/Ω_c, of doubly symmetric axially loaded encased composite members shall be determined for the limit state of flexural buckling based on member slenderness as follows:

$$\phi_c = 0.75 \text{ (LRFD)} \qquad \Omega_c = 2.00 \text{ (ASD)}$$

(a) When $\dfrac{P_{no}}{P_e} \le 2.25$

$$P_n = P_{no}\left(0.658^{\frac{P_{no}}{P_e}}\right) \tag{I2-2}$$

(b) When $\dfrac{P_{no}}{P_e} > 2.25$

$$P_n = 0.877 P_e \tag{I2-3}$$

where

P_{no} $= F_y A_s + F_{ysr} A_{sr} + 0.85 f_c' A_c$ (I2-4)

P_e = elastic critical buckling load determined in accordance with Chapter C or Appendix 7, kips (N)

$= \pi^2 (EI_{eff})/L_c^2$ (I2-5)

A_c = area of concrete, in.^{2} (mm^2)

A_s = cross-sectional area of steel section, in.^{2} (mm^2)

E_c = modulus of elasticity of concrete

$= w_c^{1.5}\sqrt{f_c'}$, ksi $\left(0.043 w_c^{1.5}\sqrt{f_c'}\text{, MPa}\right)$

EI_{eff} = effective stiffness of composite section, kip-in.^{2} (N-mm^2)

$$= E_s I_s + E_s I_{sr} + C_1 E_c I_c \tag{I2-6}$$

C_1 = coefficient for calculation of effective rigidity of an encased composite compression member

$$= 0.25 + 3\left(\frac{A_s + A_{sr}}{A_g}\right) \le 0.7 \tag{I2-7}$$

E_s = modulus of elasticity of steel
= 29,000 ksi (200 000 MPa)

F_y = specified minimum yield stress of steel section, ksi (MPa)

F_{ysr} = specified minimum yield stress of reinforcing bars, ksi (MPa)

I_c = moment of inertia of the concrete section about the elastic neutral axis of the composite section, in.^{4} (mm^4)

I_s = moment of inertia of steel shape about the elastic neutral axis of the composite section, in.^{4} (mm^4)

I_{sr} = moment of inertia of reinforcing bars about the elastic neutral axis of the composite section, in.^{4} (mm^4)

K = effective length factor

L = laterally unbraced length of the member, in. (mm)

L_c = KL = effective length of the member, in. (mm)

f_c' = specified compressive strength of concrete, ksi (MPa)

w_c = weight of concrete per unit volume ($90 \le w_c \le 155$ lb/ft^3 or $1500 \le w_c \le 2500$ kg/m^3)

The available compressive strength need not be less than that specified for the bare steel member, as required by Chapter E.

1c. Tensile Strength

The available tensile strength of axially loaded encased composite members shall be determined for the limit state of yielding as:

$$P_n = F_y A_s + F_{ysr} A_{sr} \tag{I2-8}$$

$$\phi_t = 0.90 \text{ (LRFD)} \qquad \Omega_t = 1.67 \text{ (ASD)}$$

1d. Load Transfer

Load transfer requirements for encased composite members shall be determined in accordance with Section I6.

1e. Detailing Requirements

For encased composite members, the following detailing requirements shall be met:

(a) Clear spacing between the steel core and longitudinal reinforcing shall be a minimum of 1.5 reinforcing bar diameters, but not less than 1.5 in. (38 mm).

(b) If the composite cross section is built up from two or more encased steel shapes, the shapes shall be interconnected with lacing, tie plates or comparable components to prevent buckling of individual shapes due to loads applied prior to hardening of the concrete.

2. Filled Composite Members

2a. Limitations

For filled composite members:

(a) The cross-sectional area of the steel section shall comprise at least 1% of the total composite cross section.

(b) Filled composite members shall be classified for local buckling according to Section I1.4.

(c) Minimum longitudinal reinforcement is not required. If longitudinal reinforcement is provided, internal transverse reinforcement is not required for strength.

2b. Compressive Strength

The available compressive strength of axially loaded doubly symmetric filled composite members shall be determined for the limit state of flexural buckling in accordance with Section I2.1b with the following modifications:

(a) For compact sections

$$P_{no} = P_p \tag{I2-9a}$$

where

$$P_p = F_y A_s + C_2 f_c' \left(A_c + A_{sr} \frac{E_s}{E_c} \right) \tag{I2-9b}$$

C_2 = 0.85 for rectangular sections and 0.95 for round sections

(b) For noncompact sections

$$P_{no} = P_p - \frac{P_p - P_y}{\left(\lambda_r - \lambda_p\right)^2}\left(\lambda - \lambda_p\right)^2 \tag{I2-9c}$$

where

λ, λ_p and λ_r are slenderness ratios determined from Table I1.1a

P_p is determined from Equation I2-9b

$$P_y = F_y A_s + 0.7 f_c' \left(A_c + A_{sr} \frac{E_s}{E_c} \right) \tag{I2-9d}$$

(c) For slender sections

$$P_{no} = F_{cr} A_s + 0.7 f_c' \left(A_c + A_{sr} \frac{E_s}{E_c} \right) \tag{I2-9e}$$

where

(1) For rectangular filled sections

$$F_{cr} = \frac{9E_s}{\left(\dfrac{b}{t}\right)^2} \tag{I2-10}$$

(2) For round filled sections

$$F_{cr} = \frac{0.72F_y}{\left[\left(\frac{D}{t}\right)\frac{F_y}{E_s}\right]^{0.2}} \tag{I2-11}$$

The effective stiffness of the composite section, EI_{eff}, for all sections shall be:

$$EI_{eff} = E_sI_s + E_sI_{sr} + C_3E_cI_c \tag{I2-12}$$

where

C_3 = coefficient for calculation of effective rigidity of filled composite compression member

$$= 0.45 + 3\left(\frac{A_s + A_{sr}}{A_g}\right) \le 0.9 \tag{I2-13}$$

The available compressive strength need not be less than specified for the bare steel member, as required by Chapter E.

2c. Tensile Strength

The available tensile strength of axially loaded filled composite members shall be determined for the limit state of yielding as:

$$P_n = A_sF_y + A_{sr}F_{ysr} \tag{I2-14}$$

$$\phi_t = 0.90 \text{ (LRFD)} \qquad \Omega_t = 1.67 \text{ (ASD)}$$

2d. Load Transfer

Load transfer requirements for filled composite members shall be determined in accordance with Section I6.

I3. FLEXURE

This section applies to three types of composite members subject to flexure: composite beams with steel anchors consisting of steel headed stud anchors or steel channel anchors, concrete encased members, and concrete filled members.

1. General

1a. Effective Width

The effective width of the concrete slab shall be the sum of the effective widths for each side of the beam centerline, each of which shall not exceed:

(a) one-eighth of the beam span, center-to-center of supports;

(b) one-half the distance to the centerline of the adjacent beam; or

(c) the distance to the edge of the slab.

1b. Strength During Construction

When temporary shores are not used during construction, the steel section alone shall have sufficient strength to support all loads applied prior to the concrete attaining 75% of its specified strength, f_c'. The available flexural strength of the steel section shall be determined in accordance with Chapter F.

2. Composite Beams with Steel Headed Stud or Steel Channel Anchors

2a. Positive Flexural Strength

The design positive flexural strength, $\phi_b M_n$, and allowable positive flexural strength, M_n/Ω_b, shall be determined for the limit state of yielding as follows:

$$\phi_b = 0.90 \text{ (LRFD)} \qquad \Omega_b = 1.67 \text{ (ASD)}$$

(a) When $h/t_w \le 3.76\sqrt{E/F_y}$

M_n shall be determined from the plastic stress distribution on the composite section for the limit state of yielding (plastic moment).

> **User Note:** All current ASTM A6 W, S and HP shapes satisfy the limit given in Section I3.2a(a) for $F_y \le 70$ ksi (485 MPa).

(b) When $h/t_w > 3.76\sqrt{E/F_y}$

M_n shall be determined from the superposition of elastic stresses, considering the effects of shoring, for the limit state of yielding (yield moment).

2b. Negative Flexural Strength

The available negative flexural strength shall be determined for the steel section alone, in accordance with the requirements of Chapter F.

Alternatively, the available negative flexural strength shall be determined from the plastic stress distribution on the composite section, for the limit state of yielding (plastic moment), with

$$\phi_b = 0.90 \text{ (LRFD)} \qquad \Omega_b = 1.67 \text{ (ASD)}$$

provided that the following limitations are met:

(a) The steel beam is compact and is adequately braced in accordance with Chapter F.

(b) Steel headed stud or steel channel anchors connect the slab to the steel beam in the negative moment region.

(c) The slab reinforcement parallel to the steel beam, within the effective width of the slab, is developed.

2c. Composite Beams with Formed Steel Deck

1. General

The available flexural strength of composite construction consisting of concrete slabs on formed steel deck connected to steel beams shall be determined by the applicable portions of Sections I3.2a and I3.2b, with the following requirements:

(a) The nominal rib height shall not be greater than 3 in. (75 mm). The average width of concrete rib or haunch, w_r, shall be not less than 2 in. (50 mm), but shall not be taken in calculations as more than the minimum clear width near the top of the steel deck.

(b) The concrete slab shall be connected to the steel beam with steel headed stud anchors welded either through the deck or directly to the steel cross section. Steel headed stud anchors, after installation, shall extend not less than $1\frac{1}{2}$ in. (38 mm) above the top of the steel deck and there shall be at least $\frac{1}{2}$ in. (13 mm) of specified concrete cover above the top of the steel headed stud anchors.

(c) The slab thickness above the steel deck shall be not less than 2 in. (50 mm).

(d) Steel deck shall be anchored to all supporting members at a spacing not to exceed 18 in. (460 mm). Such anchorage shall be provided by steel headed stud anchors, a combination of steel headed stud anchors and arc spot (puddle) welds, or other devices specified by the contract documents.

2. Deck Ribs Oriented Perpendicular to Steel Beam

Concrete below the top of the steel deck shall be neglected in determining composite section properties and in calculating A_c for deck ribs oriented perpendicular to the steel beams.

3. Deck Ribs Oriented Parallel to Steel Beam

Concrete below the top of the steel deck is permitted to be included in determining composite section properties and in calculating A_c.

Formed steel deck ribs over supporting beams are permitted to be split longitudinally and separated to form a concrete haunch.

When the nominal depth of steel deck is $1\frac{1}{2}$ in. (38 mm) or greater, the average width, w_r, of the supported haunch or rib shall be not less than 2 in. (50 mm) for the first steel headed stud anchor in the transverse row plus four stud diameters for each additional steel headed stud anchor.

2d. Load Transfer Between Steel Beam and Concrete Slab

1. Load Transfer for Positive Flexural Strength

The entire horizontal shear at the interface between the steel beam and the concrete slab shall be assumed to be transferred by steel headed stud or steel channel anchors, except for concrete-encased beams as defined in Section I3.3. For composite action with concrete subject to flexural compression, the nominal shear force between the steel beam and the concrete slab transferred by steel anchors,

V', between the point of maximum positive moment and the point of zero moment shall be determined as the lowest value in accordance with the limit states of concrete crushing, tensile yielding of the steel section, or the shear strength of the steel anchors:

(a) Concrete crushing

$$V' = 0.85 f_c' A_c \tag{I3-1a}$$

(b) Tensile yielding of the steel section

$$V' = F_y A_s \tag{I3-1b}$$

(c) Shear strength of steel headed stud or steel channel anchors

$$V' = \Sigma Q_n \tag{I3-1c}$$

where

A_c = area of concrete slab within effective width, in.^{2} (mm^2)

A_s = cross-sectional area of steel section, in.^{2} (mm^2)

ΣQ_n = sum of nominal shear strengths of steel headed stud or steel channel anchors between the point of maximum positive moment and the point of zero moment, kips (N)

The effect of ductility (slip capacity) of the shear connection at the interface of the concrete slab and the steel beam shall be considered.

2. Load Transfer for Negative Flexural Strength

In continuous composite beams where longitudinal reinforcing steel in the negative moment regions is considered to act compositely with the steel beam, the total horizontal shear between the point of maximum negative moment and the point of zero moment shall be determined as the lower value in accordance with the following limit states:

(a) For the limit state of tensile yielding of the slab reinforcement

$$V' = F_{ysr} A_{sr} \tag{I3-2a}$$

where

A_{sr} = area of developed longitudinal reinforcing steel within the effective width of the concrete slab, in.^{2} (mm^2)

F_{ysr} = specified minimum yield stress of the reinforcing steel, ksi (MPa)

(b) For the limit state of shear strength of steel headed stud or steel channel anchors

$$V' = \Sigma Q_n \tag{I3-2b}$$

3. Encased Composite Members

The available flexural strength of concrete-encased members shall be determined as follows:

$$\phi_b = 0.90 \text{ (LRFD)} \qquad \Omega_b = 1.67 \text{ (ASD)}$$

The nominal flexural strength, M_n, shall be determined using one of the following methods:

(a) The superposition of elastic stresses on the composite section, considering the effects of shoring for the limit state of yielding (yield moment).

(b) The plastic stress distribution on the steel section alone, for the limit state of yielding (plastic moment) on the steel section.

(c) The plastic stress distribution on the composite section or the strain-compatibility method, for the limit state of yielding (plastic moment) on the composite section. For concrete-encased members, steel anchors shall be provided.

4. Filled Composite Members

4. Limitations

Filled composite sections shall be classified for local buckling according to Section I1.4.

4b. Flexural Strength

The available flexural strength of filled composite members shall be determined as follows:

$$\phi_b = 0.90 \text{ (LRFD)} \qquad \Omega_b = 1.67 \text{ (ASD)}$$

The nominal flexural strength, M_n, shall be determined as follows:

(a) For compact sections

$$M_n = M_p \tag{I3-3a}$$

where

M_p = moment corresponding to plastic stress distribution over the composite cross section, kip-in. (N-mm)

(b) For noncompact sections

$$M_n = M_p - \left(M_p - M_y\right)\left(\frac{\lambda - \lambda_p}{\lambda_r - \lambda_p}\right) \tag{I3-3b}$$

where

λ, λ_p and λ_r are slenderness ratios determined from Table I1.1b.

M_y = yield moment corresponding to yielding of the tension flange and first yield of the compression flange, kip-in. (N-mm). The capacity at first yield shall be calculated assuming a linear elastic stress distribution with the maximum concrete compressive stress limited to $0.70f'_c$ and the maximum steel stress limited to F_y.

(c) For slender sections, M_n, shall be determined as the first yield moment. The compression flange stress shall be limited to the local buckling stress, F_{cr}, determined using Equation I2-10 or I2-11. The concrete stress distribution shall be linear elastic with the maximum compressive stress limited to $0.70f'_c$.

I4. SHEAR

1. Filled and Encased Composite Members

The design shear strength, $\phi_v V_n$, and allowable shear strength, V_n/Ω_v, shall be determined based on one of the following:

(a) The available shear strength of the steel section alone as specified in Chapter G

(b) The available shear strength of the reinforced concrete portion (concrete plus steel reinforcement) alone as defined by ACI 318 with

$$\phi_v = 0.75 \text{ (LRFD)} \qquad \Omega_v = 2.00 \text{ (ASD)}$$

(c) The nominal shear strength of the steel section, as defined in Chapter G, plus the nominal strength of the reinforcing steel, as defined by ACI 318, with a combined resistance or safety factor of

$$\phi_v = 0.75 \text{ (LRFD)} \qquad \Omega_v = 2.00 \text{ (ASD)}$$

2. Composite Beams with Formed Steel Deck

The available shear strength of composite beams with steel headed stud or steel channel anchors shall be determined based upon the properties of the steel section alone in accordance with Chapter G.

I5. COMBINED FLEXURE AND AXIAL FORCE

The interaction between flexure and axial forces in composite members shall account for stability as required by Chapter C. The available compressive strength and the available flexural strength shall be determined as defined in Sections I2 and I3, respectively. To account for the influence of length effects on the axial strength of the member, the nominal axial strength of the member shall be determined in accordance with Section I2.

(a) For encased composite members and for filled composite members with compact sections, the interaction between axial force and flexure shall be based on the interaction equations of Section H1.1 or one of the methods defined in Section I1.2.

(b) For filled composite members with noncompact or slender sections, the interaction between axial force and flexure shall be based either on the interaction equations of Section H1.1, the method defined in Section I1.2d, or Equations I5-1a and b.

(1) When $\dfrac{P_r}{P_c} \geq c_p$

$$\frac{P_r}{P_c} + \frac{1 - c_p}{c_m}\left(\frac{M_r}{M_c}\right) \leq 1.0 \qquad \text{(I5-1a)}$$

(2) When $\dfrac{P_r}{P_c} < c_p$

$$\left(\frac{1 - c_m}{c_p}\right)\left(\frac{P_r}{P_c}\right) + \frac{M_r}{M_c} \leq 1.0 \qquad \text{(I5-1b)}$$

TABLE I5.1
Coefficients c_p and c_m for Use with Equations I5-1a and I5-1b

Filled Composite Member Type	c_p	c_m when $c_{sr} \geq 0.5$	c_m when $c_{sr} < 0.5$
Rectangular	$c_p = \dfrac{0.17}{{c_{sr}}^{0.4}}$	$c_m = \dfrac{1.06}{{c_{sr}}^{0.11}} \geq 1.0$	$c_m = \dfrac{0.90}{{c_{sr}}^{0.36}} \leq 1.67$
Round HSS	$c_p = \dfrac{0.27}{{c_{sr}}^{0.4}}$	$c_m = \dfrac{1.10}{{c_{sr}}^{0.08}} \geq 1.0$	$c_m = \dfrac{0.95}{{c_{sr}}^{0.32}} \leq 1.67$

where

M_c = available flexural strength, determined in accordance with Section I3, kip-in. (N-mm)

M_r = required flexural strength, determined in accordance with Section I1.5, using LRFD or ASD load combinations, kip-in. (N-mm)

P_c = available axial strength, determined in accordance with Section I2, kips (N)

P_r = required axial strength, determined in accordance with Section I1.5, using LRFD or ASD load combinations, kips (N)

For design according to Section B3.1 (LRFD):

$M_c = \phi_b M_n$ = design flexural strength determined in accordance with Section I3, kip-in. (N-mm)

M_r = required flexural strength, determined in accordance with Section I1.5, using LRFD load combinations, kip-in. (N-mm)

$P_c = \phi_c P_n$ = design axial strength, determined in accordance with Section I2, kips (N)

P_r = required axial strength, determined in accordance with Section I1.5, using LRFD load combinations, kips (N)

ϕ_c = resistance factor for compression = 0.75

ϕ_b = resistance factor for flexure = 0.90

For design according to Section B3.2 (ASD):

$M_c = M_n / \Omega_b$ = allowable flexural strength, determined in accordance with Section I3, kip-in. (N-mm)

M_r = required flexural strength, determined in accordance with Section I1.5, using ASD load combinations, kip-in. (N-mm)

$P_c = P_n / \Omega_c$ = allowable axial strength, determined in accordance with Section I2, kips (N)

P_r = required axial strength, determined in accordance with Section I1.5, using ASD load combinations, kips (N)
Ω_c = safety factor for compression = 2.00
Ω_b = safety factor for flexure = 1.67

c_m and c_p are determined from Table I5.1

$$c_{sr} = \frac{A_s F_y + A_{sr} F_{yr}}{A_c f_c'} \tag{I5-2}$$

I6. LOAD TRANSFER

1. General Requirements

When external forces are applied to an axially loaded encased or filled composite member, the introduction of force to the member and the transfer of longitudinal shear within the member shall be assessed in accordance with the requirements for force allocation presented in this section.

The design strength, ϕR_n, or the allowable strength, R_n/Ω, of the applicable force transfer mechanisms as determined in accordance with Section I6.3 shall equal or exceed the required longitudinal shear force to be transferred, V_r', as determined in accordance with Section I6.2. Force transfer mechanisms shall be located within the load transfer region as determined in accordance with Section I6.4.

2. Force Allocation

Force allocation shall be determined based upon the distribution of external force in accordance with the following requirements.

User Note: Bearing strength provisions for externally applied forces are provided in Section J8. For filled composite members, the term $\sqrt{A_2/A_1}$ in Equation J8-2 may be taken equal to 2.0 due to confinement effects.

2a. External Force Applied to Steel Section

When the entire external force is applied directly to the steel section, the force required to be transferred to the concrete, V_r', shall be determined as:

$$V_r' = P_r(1 - F_y A_s / P_{no}) \tag{I6-1}$$

where

P_{no} = nominal axial compressive strength without consideration of length effects, determined by Equation I2-4 for encased composite members, and Equation I2-9a or Equation I2-9c, as applicable, for compact or noncompact filled composite members, kips (N)
P_r = required external force applied to the composite member, kips (N)

User Note: Equation I6-1 does not apply to slender filled composite members for which the external force is applied directly to the concrete fill in accordance with Section I6.2b, or concurrently to the steel and concrete, in accordance with Section I6.2c.

2b. External Force Applied to Concrete

When the entire external force is applied directly to the concrete encasement or concrete fill, the force required to be transferred to the steel, V_r', shall be determined as follows:

(a) For encased or filled composite members that are compact or noncompact

$$V_r' = P_r\,(F_y A_s / P_{no}) \tag{I6-2a}$$

(b) For slender filled composite members

$$V_r' = P_r\,(F_{cr} A_s / P_{no}) \tag{I6-2b}$$

where

F_{cr} = critical buckling stress for steel elements of filled composite members determined using Equation I2-10 or Equation I2-11, as applicable, ksi (MPa)

P_{no} = nominal axial compressive strength without consideration of length effects, determined by Equation I2-4 for encased composite members, and Equation I2-9a for filled composite members, kips (N)

2c. External Force Applied Concurrently to Steel and Concrete

When the external force is applied concurrently to the steel section and concrete encasement or concrete fill, V_r' shall be determined as the force required to establish equilibrium of the cross section.

User Note: The Commentary provides an acceptable method of determining the longitudinal shear force required for equilibrium of the cross section.

3. Force Transfer Mechanisms

The nominal strength, R_n, of the force transfer mechanisms of direct bond interaction, shear connection and direct bearing shall be determined in accordance with this section. Use of the force transfer mechanism providing the largest nominal strength is permitted. Force transfer mechanisms shall not be superimposed.

The force transfer mechanism of direct bond interaction shall not be used for encased composite members.

3a. Direct Bearing

Where force is transferred in an encased or filled composite member by direct bearing from internal bearing mechanisms, the available bearing strength of the concrete for the limit state of concrete crushing shall be determined as:

$$R_n = 1.7 f_c' A_1 \tag{I6-3}$$

$$\phi_B = 0.65 \text{ (LRFD)} \qquad \Omega_B = 2.31 \text{ (ASD)}$$

where

A_1 = loaded area of concrete, in.^{2} (mm^2)

User Note: An example of force transfer via an internal bearing mechanism is the use of internal steel plates within a filled composite member.

3b. Shear Connection

Where force is transferred in an encased or filled composite member by shear connection, the available shear strength of steel headed stud or steel channel anchors shall be determined as:

$$R_c = \Sigma Q_{cv} \tag{I6-4}$$

where

ΣQ_{cv} = sum of available shear strengths, ϕQ_{nv} (LRFD) or Q_{nv}/Ω (ASD), as applicable, of steel headed stud or steel channel anchors, determined in accordance with Section I8.3a or Section I8.3d, respectively, placed within the load introduction length as defined in Section I6.4, kips (N)

3c. Direct Bond Interaction

Where force is transferred in a filled composite member by direct bond interaction, the available bond strength between the steel and concrete shall be determined as follows:

$$R_n = p_b L_{in} F_{in} \tag{I6-5}$$

$$\phi = 0.50 \text{ (LRFD)} \qquad \Omega = 3.00 \text{ (ASD)}$$

where

F_{in} = nominal bond stress, ksi (MPa)
= $12t/H^2 \le 0.1$, ksi ($2\,100t/H^2 \le 0.7$, MPa) for rectangular cross sections
= $30t/D^2 \le 0.2$, ksi ($5\,300t/D^2 \le 1.4$, MPa) for circular cross sections

D = outside diameter of round HSS, in. (mm)

H = maximum transverse dimension of rectangular steel member, in. (mm)

L_{in} = load introduction length, determined in accordance with Section I6.4, in. (mm)

R_n = nominal bond strength, kips (N)

p_b = perimeter of the steel-concrete bond interface within the composite cross section, in. (mm)

t = design wall thickness of HSS member as defined in Section B4.2, in. (mm)

4. Detailing Requirements

4a. Encased Composite Members

Force transfer mechanisms shall be distributed within the load introduction length, which shall not exceed a distance of two times the minimum transverse dimension of the encased composite member above and below the load transfer region. Anchors utilized to transfer longitudinal shear shall be placed on at least two faces of the steel shape in a generally symmetric configuration about the steel shape axes.

Steel anchor spacing, both within and outside of the load introduction length, shall conform to Section I8.3e.

4b. Filled Composite Members

Force transfer mechanisms shall be distributed within the load introduction length, which shall not exceed a distance of two times the minimum transverse dimension of a rectangular steel member or two times the diameter of a round steel member both above and below the load transfer region. For the specific case of load applied to the concrete of a filled composite member containing no internal reinforcement, the load introduction length shall extend beyond the load transfer region in only the direction of the applied force. Steel anchor spacing within the load introduction length shall conform to Section I8.3e.

I7. COMPOSITE DIAPHRAGMS AND COLLECTOR BEAMS

Composite slab diaphragms and collector beams shall be designed and detailed to transfer loads between the diaphragm, the diaphragm's boundary members and collector elements, and elements of the lateral force-resisting system.

> **User Note:** Design guidelines for composite diaphragms and collector beams can be found in the Commentary.

I8. STEEL ANCHORS

1. General

The diameter of a steel headed stud anchor, d_{sa}, shall be $^3/_4$ in. (19 mm) or less, except where anchors are utilized solely for shear transfer in solid slabs in which case $^7/_8$-in.- (22 mm) and 1-in.- (25 mm) diameter anchors are permitted. Additionally, d_{sa} shall not be greater than 2.5 times the thickness of the base metal to which it is welded, unless it is welded to a flange directly over a web.

Section I8.2 applies to a composite flexural member where steel anchors are embedded in a solid concrete slab or in a slab cast on formed steel deck. Section I8.3 applies to all other cases.

2. Steel Anchors in Composite Beams

The length of steel headed stud anchors shall not be less than four stud diameters from the base of the steel headed stud anchor to the top of the stud head after installation.

2a. Strength of Steel Headed Stud Anchors

The nominal shear strength of one steel headed stud anchor embedded in a solid concrete slab or in a composite slab with decking shall be determined as follows:

$$Q_n = 0.5 A_{sa} \sqrt{f_c' E_c} \leq R_g R_p A_{sa} F_u \tag{I8-1}$$

where

A_{sa} = cross-sectional area of steel headed stud anchor, in.^{2} (mm^2)

E_c = modulus of elasticity of concrete
= $w_c^{1.5}\sqrt{f_c'}$, ksi $\left(0.043 w_c^{1.5}\sqrt{f_c'}\text{, MPa}\right)$

F_u = specified minimum tensile strength of a steel headed stud anchor, ksi (MPa)

R_g = 1.0 for:
- (a) One steel headed stud anchor welded in a steel deck rib with the deck oriented perpendicular to the steel shape
- (b) Any number of steel headed stud anchors welded in a row directly to the steel shape
- (c) Any number of steel headed stud anchors welded in a row through steel deck with the deck oriented parallel to the steel shape and the ratio of the average rib width to rib depth ≥ 1.5

= 0.85 for:
- (a) Two steel headed stud anchors welded in a steel deck rib with the deck oriented perpendicular to the steel shape
- (b) One steel headed stud anchor welded through steel deck with the deck oriented parallel to the steel shape and the ratio of the average rib width to rib depth < 1.5

= 0.7 for three or more steel headed stud anchors welded in a steel deck rib with the deck oriented perpendicular to the steel shape

R_p = 0.75 for:
- (a) Steel headed stud anchors welded directly to the steel shape
- (b) Steel headed stud anchors welded in a composite slab with the deck oriented perpendicular to the beam and $e_{mid\text{-}ht} \geq 2$ in. (50 mm)
- (c) Steel headed stud anchors welded through steel deck, or steel sheet used as girder filler material, and embedded in a composite slab with the deck oriented parallel to the beam

= 0.6 for steel headed stud anchors welded in a composite slab with deck oriented perpendicular to the beam and $e_{mid\text{-}ht} < 2$ in. (50 mm)

$e_{mid\text{-}ht}$ = distance from the edge of steel headed stud anchor shank to the steel deck web, measured at mid-height of the deck rib, and in the load bearing direction of the steel headed stud anchor (in other words, in the direction of maximum moment for a simply supported beam), in. (mm)

User Note: The table below presents values for R_g and R_p for several cases. Available strengths for steel headed stud anchors can be found in the AISC *Steel Construction Manual*.

Condition	R_g	R_p
No decking	1.0	0.75
Decking oriented parallel to the steel shape		
$\frac{w_r}{h_r} \geq 1.5$	1.0	0.75
$\frac{w_r}{h_r} < 1.5$	0.85[a]	0.75
Decking oriented perpendicular to the steel shape Number of steel headed stud anchors occupying the same decking rib:		
1	1.0	0.6[b]
2	0.85	0.6[b]
3 or more	0.7	0.6[b]

h_r = nominal rib height, in. (mm)
w_r = average width of concrete rib or haunch (as defined in Section I3.2c), in. (mm)
[a] For a single steel headed stud anchor
[b] This value may be increased to 0.75 when $e_{mid\text{-}ht} \geq 2$ in. (50 mm).

2b. Strength of Steel Channel Anchors

The nominal shear strength of one hot-rolled channel anchor embedded in a solid concrete slab shall be determined as:

$$Q_n = 0.3(t_f + 0.5t_w)l_a\sqrt{f_c'E_c} \tag{I8-2}$$

where

l_a = length of channel anchor, in. (mm)
t_f = thickness of flange of channel anchor, in. (mm)
t_w = thickness of channel anchor web, in. (mm)

The strength of the channel anchor shall be developed by welding the channel to the beam flange for a force equal to Q_n, considering eccentricity on the anchor.

2c. Required Number of Steel Anchors

The number of anchors required between the section of maximum bending moment, positive or negative, and the adjacent section of zero moment shall be equal to the horizontal shear as determined in Sections I3.2d.1 and I3.2d.2 divided by the nominal shear strength of one steel anchor as determined from Section I8.2a or Section I8.2b. The number of steel anchors required between any concentrated load and the nearest point of zero moment shall be sufficient to develop the maximum moment required at the concentrated load point.

2d. Detailing Requirements

Steel anchors in composite beams shall meet the following requirements:

(a) Steel anchors required on each side of the point of maximum bending moment, positive or negative, shall be distributed uniformly between that point and the adjacent points of zero moment, unless specified otherwise on the contract documents.

(b) Steel anchors shall have at least 1 in. (25 mm) of lateral concrete cover in the direction perpendicular to the shear force, except for anchors installed in the ribs of formed steel decks.

(c) The minimum distance from the center of a steel anchor to a free edge in the direction of the shear force shall be 8 in. (200 mm) if normal weight concrete is used and 10 in. (250 mm) if lightweight concrete is used. The provisions of ACI 318 Chapter 17 are permitted to be used in lieu of these values.

(d) Minimum center-to-center spacing of steel headed stud anchors shall be four diameters in any direction. For composite beams that do not contain anchors located within formed steel deck oriented perpendicular to the beam span, an additional minimum spacing limit of six diameters along the longitudinal axis of the beam shall apply.

(e) The maximum center-to-center spacing of steel anchors shall not exceed eight times the total slab thickness or 36 in. (900 mm).

3. Steel Anchors in Composite Components

This section shall apply to the design of cast-in-place steel headed stud anchors and steel channel anchors in composite components.

The provisions of the applicable building code or ACI 318 Chapter 17 are permitted to be used in lieu of the provisions in this section.

> **User Note:** The steel headed stud anchor strength provisions in this section are applicable to anchors located primarily in the load transfer (connection) region of composite columns and beam-columns, concrete-encased and filled composite beams, composite coupling beams, and composite walls, where the steel and concrete are working compositely within a member. They are not intended for hybrid construction where the steel and concrete are not working compositely, such as with embed plates.

Section I8.2 specifies the strength of steel anchors embedded in a solid concrete slab or in a concrete slab with formed steel deck in a composite beam.

Limit states for the steel shank of the anchor and for concrete breakout in shear are covered directly in this Section. Additionally, the spacing and dimensional limitations provided in these provisions preclude the limit states of concrete pry-out for anchors loaded in shear and concrete breakout for anchors loaded in tension as defined by ACI 318 Chapter 17.

For normal weight concrete: Steel headed stud anchors subjected to shear only shall not be less than five stud diameters in length from the base of the steel headed stud to the top of the stud head after installation. Steel headed stud anchors subjected to tension or interaction of shear and tension shall not be less than eight stud diameters in length from the base of the stud to the top of the stud head after installation.

For lightweight concrete: Steel headed stud anchors subjected to shear only shall not be less than seven stud diameters in length from the base of the steel headed stud to the top of the stud head after installation. Steel headed stud anchors subjected to tension shall not be less than ten stud diameters in length from the base of the stud to the top of the stud head after installation. The nominal strength of steel headed stud anchors subjected to interaction of shear and tension for lightweight concrete shall be determined as stipulated by the applicable building code or ACI 318 Chapter 17.

Steel headed stud anchors subjected to tension or interaction of shear and tension shall have a diameter of the head greater than or equal to 1.6 times the diameter of the shank.

User Note: The following table presents values of minimum steel headed stud anchor h/d ratios for each condition covered in this Specification.

Loading Condition	Normal Weight Concrete	Lightweight Concrete
Shear	$h/d_{sa} \geq 5$	$h/d_{sa} \geq 7$
Tension	$h/d_{sa} \geq 8$	$h/d_{sa} \geq 10$
Shear and Tension	$h/d_{sa} \geq 8$	N/A[a]

h/d_{sa} = ratio of steel headed stud anchor shank length to the top of the stud head, to shank diameter.

[a] Refer to ACI 318 Chapter 17 for the calculation of interaction effects of anchors embedded in lightweight concrete.

3a. Shear Strength of Steel Headed Stud Anchors in Composite Components

Where concrete breakout strength in shear is not an applicable limit state, the design shear strength, $\phi_v Q_{nv}$, and allowable shear strength, Q_{nv}/Ω_v, of one steel headed stud anchor shall be determined as:

$$Q_{nv} = F_u A_{sa} \tag{I8-3}$$

$$\phi_v = 0.65 \text{ (LRFD)} \qquad \Omega_v = 2.31 \text{ (ASD)}$$

where

A_{sa} = cross-sectional area of a steel headed stud anchor, in.^{2} (mm^2)

F_u = specified minimum tensile strength of a steel headed stud anchor, ksi (MPa)

Q_{nv} = nominal shear strength of a steel headed stud anchor, kips (N)

Where concrete breakout strength in shear is an applicable limit state, the available shear strength of one steel headed stud anchor shall be determined by one of the following:

(a) Where anchor reinforcement is developed in accordance with ACI 318 on both sides of the concrete breakout surface for the steel headed stud anchor, the minimum of the steel nominal shear strength from Equation I8-3 and the nominal strength of the anchor reinforcement shall be used for the nominal shear strength, Q_{nv}, of the steel headed stud anchor.

(b) As stipulated by the applicable building code or ACI 318 Chapter 17.

> **User Note:** If concrete breakout strength in shear is an applicable limit state (for example, where the breakout prism is not restrained by an adjacent steel plate, flange or web), appropriate anchor reinforcement is required for the provisions of this Section to be used. Alternatively, the provisions of the applicable building code or ACI 318 Chapter 17 may be used.

3b. Tensile Strength of Steel Headed Stud Anchors in Composite Components

Where the distance from the center of an anchor to a free edge of concrete in the direction perpendicular to the height of the steel headed stud anchor is greater than or equal to 1.5 times the height of the steel headed stud anchor measured to the top of the stud head, and where the center-to-center spacing of steel headed stud anchors is greater than or equal to three times the height of the steel headed stud anchor measured to the top of the stud head, the available tensile strength of one steel headed stud anchor shall be determined as:

$$Q_{nt} = F_u A_{sa} \tag{I8-4}$$

$$\phi_t = 0.75 \text{ (LRFD)} \qquad \Omega_t = 2.00 \text{ (ASD)}$$

where

Q_{nt} = nominal tensile strength of steel headed stud anchor, kips (N)

Where the distance from the center of an anchor to a free edge of concrete in the direction perpendicular to the height of the steel headed stud anchor is less than 1.5 times the height of the steel headed stud anchor measured to the top of the stud head, or where the center-to-center spacing of steel headed stud anchors is less than three times the height of the steel headed stud anchor measured to the top of the stud head, the nominal tensile strength of one steel headed stud anchor shall be determined by one of the following:

(a) Where anchor reinforcement is developed in accordance with ACI 318 on both sides of the concrete breakout surface for the steel headed stud anchor, the minimum of the steel nominal tensile strength from Equation I8-4 and the nominal strength of the anchor reinforcement shall be used for the nominal tensile strength, Q_{nt}, of the steel headed stud anchor.

(b) As stipulated by the applicable building code or ACI 318 Chapter 17.

User Note: Supplemental confining reinforcement is recommended around the anchors for steel headed stud anchors subjected to tension or interaction of shear and tension to avoid edge effects or effects from closely spaced anchors. See the Commentary and ACI 318 for guidelines.

3c. Strength of Steel Headed Stud Anchors for Interaction of Shear and Tension in Composite Components

Where concrete breakout strength in shear is not a governing limit state, and where the distance from the center of an anchor to a free edge of concrete in the direction perpendicular to the height of the steel headed stud anchor is greater than or equal to 1.5 times the height of the steel headed stud anchor measured to the top of the stud head, and where the center-to-center spacing of steel headed stud anchors is greater than or equal to three times the height of the steel headed stud anchor measured to the top of the stud head, the nominal strength for interaction of shear and tension of one steel headed stud anchor shall be determined as:

$$\left(\frac{Q_{rt}}{Q_{ct}}\right)^{5/3} + \left(\frac{Q_{rv}}{Q_{cv}}\right)^{5/3} \le 1.0 \tag{I8-5}$$

where

Q_{ct} = available tensile strength, kips (N)

Q_{rt} = required tensile strength, kips (N)

Q_{cv} = available shear strength, kips (N)

Q_{rv} = required shear strength, kips (N)

For design in accordance with Section B3.3 (LRFD):

Q_{rt} = required tensile strength using LRFD load combinations, kips (N)
Q_{ct} = $\phi_t Q_{nt}$ = design tensile strength, determined in accordance with Section I8.3b, kips (N)
Q_{rv} = required shear strength using LRFD load combinations, kips (N)
Q_{cv} = $\phi_v Q_{nv}$ = design shear strength, determined in accordance with Section I8.3a, kips (N)
ϕ_t = resistance factor for tension = 0.75
ϕ_v = resistance factor for shear = 0.65

For design in accordance with Section B3.4 (ASD):

Q_{rt} = required tensile strength using ASD load combinations, kips (N)
Q_{ct} = Q_{nt}/Ω_t = allowable tensile strength, determined in accordance with Section I8.3b, kips (N)
Q_{rv} = required shear strength using ASD load combinations, kips (N)
Q_{cv} = Q_{nv}/Ω_v = allowable shear strength, determined in accordance with Section I8.3a, kips (N)
Ω_t = safety factor for tension = 2.00
Ω_v = safety factor for shear = 2.31

Where concrete breakout strength in shear is a governing limit state, or where the distance from the center of an anchor to a free edge of concrete in the direction perpendicular to the height of the steel headed stud anchor is less than 1.5 times the height of the steel headed stud anchor measured to the top of the stud head, or where the center-to-center spacing of steel headed stud anchors is less than three times the height of the steel headed stud anchor measured to the top of the stud head, the nominal strength for interaction of shear and tension of one steel headed stud anchor shall be determined by one of the following:

(a) Where anchor reinforcement is developed in accordance with ACI 318 on both sides of the concrete breakout surface for the steel headed stud anchor, the minimum of the steel nominal shear strength from Equation I8-3 and the nominal strength of the anchor reinforcement shall be used for the nominal shear strength, Q_{nv}, of the steel headed stud anchor, and the minimum of the steel nominal tensile strength from Equation I8-4 and the nominal strength of the anchor reinforcement shall be used for the nominal tensile strength, Q_{nt}, of the steel headed stud anchor for use in Equation I8-5.

(b) As stipulated by the applicable building code or ACI 318 Chapter 17.

3d. Shear Strength of Steel Channel Anchors in Composite Components

The available shear strength of steel channel anchors shall be based on the provisions of Section I8.2b with the following resistance factor and safety factor:

$$\phi_t = 0.75 \text{ (LRFD)} \qquad \Omega_t = 2.00 \text{ (ASD)}$$

3e. Detailing Requirements in Composite Components

Steel anchors in composite components shall meet the following requirements:

(a) Minimum concrete cover to steel anchors shall be in accordance with ACI 318 provisions for concrete protection of headed shear stud reinforcement.

(b) Minimum center-to-center spacing of steel headed stud anchors shall be four diameters in any direction.

(c) The maximum center-to-center spacing of steel headed stud anchors shall not exceed 32 times the shank diameter.

(d) The maximum center-to-center spacing of steel channel anchors shall be 24 in. (600 mm).

User Note: Detailing requirements provided in this section are absolute limits. See Sections I8.3a, I8.3b and I8.3c for additional limitations required to preclude edge and group effect considerations.

CHAPTER J

DESIGN OF CONNECTIONS

This chapter addresses connecting elements, connectors and the affected elements of connected members not subject to fatigue loads.

The chapter is organized as follows:

J1. General Provisions
J2. Welds
J3. Bolts and Threaded Parts
J4. Affected Elements of Members and Connecting Elements
J5. Fillers
J6. Splices
J7. Bearing Strength
J8. Column Bases and Bearing on Concrete
J9. Anchor Rods and Embedments
J10. Flanges and Webs with Concentrated Forces

User Note: For cases not included in this chapter, the following sections apply:
- Chapter K — Additional Requirements for HSS and Box-Section Connections
- Appendix 3 — Fatigue

J1. GENERAL PROVISIONS

1. Design Basis

The design strength, ϕR_n, and the allowable strength, R_n/Ω, of connections shall be determined in accordance with the provisions of this chapter and the provisions of Chapter B.

The required strength of the connections shall be determined by structural analysis for the specified design loads, consistent with the type of construction specified, or shall be a proportion of the required strength of the connected members when so specified herein.

Where the gravity axes of intersecting axially loaded members do not intersect at one point, the effects of eccentricity shall be considered.

2. Simple Connections

Simple connections of beams, girders and trusses shall be designed as flexible and are permitted to be proportioned for the reaction shears only, except as otherwise indicated in the design documents. Flexible beam connections shall accommodate end rotations of simple beams. Some inelastic but self-limiting deformation in the connection is permitted to accommodate the end rotation of a simple beam.

3. Moment Connections

End connections of restrained beams, girders and trusses shall be designed for the combined effect of forces resulting from moment and shear induced by the rigidity of the connections. Response criteria for moment connections are provided in Section B3.4b.

User Note: See Chapter C and Appendix 7 for analysis requirements to establish the required strength for the design of connections.

4. Compression Members with Bearing Joints

Compression members relying on bearing for load transfer shall meet the following requirements:

(a) For columns bearing on bearing plates or finished to bear at splices, there shall be sufficient connectors to hold all parts in place.

(b) For compression members other than columns finished to bear, the splice material and its connectors shall be arranged to hold all parts in line and their required strength shall be the lesser of:

(1) An axial tensile force equal to 50% of the required compressive strength of the member; or

(2) The moment and shear resulting from a transverse load equal to 2% of the required compressive strength of the member. The transverse load shall be applied at the location of the splice exclusive of other loads that act on the member. The member shall be taken as pinned for the determination of the shears and moments at the splice.

User Note: All compression joints should also be proportioned to resist any tension developed by the load combinations stipulated in Section B2.

5. Splices in Heavy Sections

When tensile forces due to applied tension or flexure are to be transmitted through splices in heavy sections, as defined in Sections A3.1c and A3.1d, by complete-joint-penetration (CJP) groove welds, the following provisions apply: (a) material notch-toughness requirements as given in Sections A3.1c and A3.1d; (b) weld access hole details as given in Section J1.6; (c) filler metal requirements as given in Section J2.6; and (d) thermal cut surface preparation and inspection requirements as given in Section M2.2. The foregoing provision is not applicable to splices of elements of built-up shapes that are welded prior to assembling the shape.

User Note: CJP groove welded splices of heavy sections can exhibit detrimental effects of weld shrinkage. Members that are sized for compression that are also subject to tensile forces may be less susceptible to damage from shrinkage if they are spliced using partial-joint-penetration (PJP) groove welds on the flanges and fillet-welded web plates, or using bolts for some or all of the splice.

6. Weld Access Holes

Weld access holes shall meet the following requirements:

(a) All weld access holes required to facilitate welding operations shall be detailed to provide room for weld backing as needed.

(b) The access hole shall have a length from the toe of the weld preparation not less than $1^1/_2$ times the thickness of the material in which the hole is made, nor less than $1^1/_2$ in. (38 mm).

(c) The access hole shall have a height not less than the thickness of the material with the access hole, nor less than $^3/_4$ in. (19 mm), nor does it need to exceed 2 in. (50 mm).

(d) For sections that are rolled or welded prior to cutting, the edge of the web shall be sloped or curved from the surface of the flange to the reentrant surface of the access hole.

(e) In hot-rolled shapes, and built-up shapes with CJP groove welds that join the web-to-flange, weld access holes shall be free of notches and sharp reentrant corners.

(f) No arc of the weld access hole shall have a radius less than $^3/_8$ in. (10 mm).

(g) In built-up shapes with fillet or partial-joint-penetration (PJP) groove welds that join the web-to-flange, weld access holes shall be free of notches and sharp reentrant corners.

(h) The access hole is permitted to terminate perpendicular to the flange, providing the weld is terminated at least a distance equal to the weld size away from the access hole.

(i) For heavy shapes, as defined in Sections A3.1c and A3.1d, the thermally cut surfaces of weld access holes shall be ground to bright metal.

(j) If the curved transition portion of weld access holes is formed by predrilled or sawed holes, that portion of the access hole need not be ground.

7. Placement of Welds and Bolts

Groups of welds or bolts at the ends of any member that transmit axial force into that member shall be sized so that the center of gravity of the group coincides with the center of gravity of the member, unless provision is made for the eccentricity. The foregoing provision is not applicable to end connections of single-angle, double-angle and similar members.

8. Bolts in Combination with Welds

Bolts shall not be considered as sharing the load in combination with welds, except in the design of shear connections on a common faying surface where strain compatibility between the bolts and welds is considered.

It is permitted to determine the available strength, ϕR_n and R_n / Ω, as applicable, of a joint combining the strengths of high-strength bolts and longitudinal fillet welds as the sum of (1) the nominal slip resistance, R_n, for bolts as defined in Equation J3-4 according to the requirements of a slip-critical connection and (2) the nominal weld strength, R_n, as defined in Section J2.4, when the following apply:

(a) ϕ = 0.75 (LRFD); Ω = 2.00 (ASD) for the combined joint.

(b) When the high-strength bolts are pretensioned according to the requirements of Table J3.1 or Table J3.1M, using the turn-of-nut method, the longitudinal fillet welds shall have an available strength of not less than 50% of the required strength of the connection.

(c) When the high-strength bolts are pretensioned according to the requirements of Table J3.1 or Table J3.1M, using any method other than the turn-of-nut method, the longitudinal fillet welds shall have an available strength of not less than 70% of the required strength of the connection.

(d) The high-strength bolts shall have an available strength of not less than 33% of the required strength of the connection.

In joints with combined bolts and longitudinal welds, the strength of the connection need not be taken as less than either the strength of the bolts alone or the strength of the welds alone.

9. Welded Alterations to Structures with Existing Rivets or Bolts

In making welded alterations to structures, existing rivets and high-strength bolts in standard or short-slotted holes transverse to the direction of load and tightened to the requirements of slip-critical connections are permitted to be utilized for resisting loads present at the time of alteration, and the welding need only provide the additional required strength. The weld available strength shall provide the additional required strength, but not less than 25% of the required strength of the connection.

User Note: The provisions of this section are generally recommended for alteration in building designs or for field corrections. Use of the combined strength of bolts and welds on a common faying surface is not recommended for new design.

10. High-Strength Bolts in Combination with Rivets

In both new work and alterations, in connections designed as slip-critical connections in accordance with the provisions of Section J3, high-strength bolts are permitted to be considered as sharing the load with existing rivets.

J2. WELDS

All provisions of the *Structural Welding Code—Steel* (AWS D1.1/D1.1M), hereafter referred to as AWS D1.1/D1.1M, apply under this Specification, with the exception that the provisions of the listed Specification sections apply under this Specification in lieu of the cited AWS provisions as follows:

(a) Section J1.6 in lieu of AWS D1.1/D1.1M clause 5.16

(b) Section J2.2a in lieu of AWS D1.1/D1.1M clauses 2.4.2.10 and 2.4.4.4

(c) Table J2.2 in lieu of AWS D1.1/D1.1M Table 2.1

(d) Table J2.5 in lieu of AWS D1.1/D1.1M Table 2.3

(e) Appendix 3, Table A-3.1 in lieu of AWS D1.1/D1.1M Table 2.5

<table>
<tr><th colspan="4">TABLE J2.1
Effective Throat of
Partial-Joint-Penetration Groove Welds</th></tr>
<tr><th>Welding Process</th><th>Welding Position
F (flat),
H (horizontal),
V (vertical),
OH (overhead)</th><th>Groove Type
(AWS D1.1,
Figure 3.3)</th><th>Effective Throat</th></tr>
<tr><td>Shielded metal arc (SMAW)</td><td rowspan="2">All</td><td rowspan="2">J or U groove

60° V</td><td rowspan="3">depth of groove</td></tr>
<tr><td>Gas metal arc (GMAW)
Flux cored arc (FCAW)</td></tr>
<tr><td>Submerged arc (SAW)</td><td>F</td><td>J or U groove

60° bevel or V</td></tr>
<tr><td>Gas metal arc (GMAW)
Flux cored arc (FCAW)</td><td>F, H</td><td>45° bevel</td><td>depth of groove</td></tr>
<tr><td>Shielded metal arc (SMAW)</td><td>All</td><td rowspan="2">45° bevel</td><td rowspan="2">depth of groove
minus $^1/_8$ in.
(3 mm)</td></tr>
<tr><td>Gas metal arc (GMAW)
Flux cored arc (FCAW)</td><td>V, OH</td></tr>
</table>

(f) Section B3.11 and Appendix 3 in lieu of AWS D1.1/D1.1M clause 2, Part C

(g) Section M2.2 in lieu of AWS D1.1/D1.1M clauses 5.14 and 5.15

1. Groove Welds

1a. Effective Area

The effective area of groove welds shall be taken as the length of the weld times the effective throat.

The effective throat of a CJP groove weld shall be the thickness of the thinner part joined.

When filled flush to the surface, the effective weld throat for a PJP groove weld shall be as given in Table J2.1 and the effective weld throat for a flare groove weld shall be as given in Table J2.2. The effective throat of a PJP groove weld or flare groove weld filled less than flush shall be as shown in Table J2.1 or Table J2.2, less the greatest perpendicular dimension measured from a line flush to the base metal surface to the weld surface.

User Note: The effective throat of a PJP groove weld is dependent on the process used and the weld position. The design drawings should either indicate the effective throat required or the weld strength required, and the fabricator should detail the joint based on the weld process and position to be used to weld the joint.

TABLE J2.2
Effective Throat of Flare Groove Welds

Welding Process	Flare Bevel Groove[a]	Flare V-Groove
GMAW and FCAW-G	$^5/_8R$	$^3/_4R$
SMAW and FCAW-S	$^5/_{16}R$	$^5/_8R$
SAW	$^5/_{16}R$	$^1/_2R$

[a] For flare bevel groove with $R < {}^3/_8$ in. (10 mm), use only reinforcing fillet weld on filled flush joint.
General note: R = radius of joint surface (is permitted to be $2t$ for HSS), in. (mm)

TABLE J2.3
Minimum Effective Throat of Partial-Joint-Penetration Groove Welds

Material Thickness of Thinner Part Joined, in. (mm)	Minimum Effective Throat,[a] in. (mm)
To $^1/_4$ (6) inclusive	$^1/_8$ (3)
Over $^1/_4$ (6) to $^1/_2$ (13)	$^3/_{16}$ (5)
Over $^1/_2$ (13) to $^3/_4$ (19)	$^1/_4$ (6)
Over $^3/_4$ (19) to $1^1/_2$ (38)	$^5/_{16}$ (8)
Over $1^1/_2$ (38) to $2^1/_4$ (57)	$^3/_8$ (10)
Over $2^1/_4$ (57) to 6 (150)	$^1/_2$ (13)
Over 6 (150)	$^5/_8$ (16)

[a] See Table J2.1.

Larger effective throats than those in Table J2.2 are permitted for a given welding procedure specification (WPS), provided the fabricator establishes by qualification the consistent production of such larger effective throat. Qualification shall consist of sectioning the weld normal to its axis, at mid-length, and terminal ends. Such sectioning shall be made on a number of combinations of material sizes representative of the range to be used in the fabrication.

1b. Limitations

The minimum effective throat of a partial-joint-penetration groove weld shall not be less than the size required to transmit calculated forces nor the size shown in Table J2.3. Minimum weld size is determined by the thinner of the two parts joined.

TABLE J2.4
Minimum Size of Fillet Welds

Material Thickness of Thinner Part Joined, in. (mm)	Minimum Size of Fillet Weld,[a] in. (mm)
To 1/4 (6) inclusive	1/8 (3)
Over 1/4 (6) to 1/2 (13)	3/16 (5)
Over 1/2 (13) to 3/4 (19)	1/4 (6)
Over 3/4 (19)	5/16 (8)

[a] Leg dimension of fillet welds. Single pass welds must be used.
Note: See Section J2.2b for maximum size of fillet welds.

2. Fillet Welds

2a. Effective Area

The effective area of a fillet weld shall be the effective length multiplied by the effective throat. The effective throat of a fillet weld shall be the shortest distance from the root to the face of the diagrammatic weld. An increase in effective throat is permitted if consistent penetration beyond the root of the diagrammatic weld is demonstrated by tests using the production process and procedure variables.

For fillet welds in holes and slots, the effective length shall be the length of the centerline of the weld along the center of the plane through the throat. In the case of overlapping fillets, the effective area shall not exceed the nominal cross-sectional area of the hole or slot, in the plane of the faying surface.

2b. Limitations

Fillet welds shall meet the following limitations:

(a) The minimum size of fillet welds shall be not less than the size required to transmit calculated forces, nor the size as shown in Table J2.4. These provisions do not apply to fillet weld reinforcements of PJP or CJP groove welds.

(b) The maximum size of fillet welds of connected parts shall be:

(1) Along edges of material less than 1/4 in. (6 mm) thick; not greater than the thickness of the material.

(2) Along edges of material 1/4 in. (6 mm) or more in thickness; not greater than the thickness of the material minus 1/16 in. (2 mm), unless the weld is especially designated on the drawings to be built out to obtain full-throat thickness. In the as-welded condition, the distance between the edge of the base metal and the toe of the weld is permitted to be less than 1/16 in. (2 mm), provided the weld size is clearly verifiable.

(c) The minimum length of fillet welds designed on the basis of strength shall be not less than four times the nominal weld size, or else the effective size of the weld shall not be taken to exceed one-quarter of its length. For the effect of longitudinal fillet weld length in end connections upon the effective area of the connected member, see Section D3.

(d) The effective length of fillet welds shall be determined as follows:

(1) For end-loaded fillet welds with a length up to 100 times the weld size, it is permitted to take the effective length equal to the actual length.

(2) When the length of the end-loaded fillet weld exceeds 100 times the weld size, the effective length shall be determined by multiplying the actual length by the reduction factor, β, determined as:

$$\beta = 1.2 - 0.002(l/w) \leq 1.0 \tag{J2-1}$$

where

l = actual length of end-loaded weld, in. (mm)

w = size of weld leg, in. (mm)

(3) When the length of the weld exceeds 300 times the leg size, w, the effective length shall be taken as $180w$.

(e) Intermittent fillet welds are permitted to be used to transfer calculated stress across a joint or faying surfaces and to join components of built-up members. The length of any segment of intermittent fillet welding shall be not less than four times the weld size, with a minimum of $1^1/_2$ in. (38 mm).

(f) In lap joints, the minimum amount of lap shall be five times the thickness of the thinner part joined, but not less than 1 in. (25 mm). Lap joints joining plates or bars subjected to axial stress that utilize transverse fillet welds only shall be fillet welded along the end of both lapped parts, except where the deflection of the lapped parts is sufficiently restrained to prevent opening of the joint under maximum loading.

(g) Fillet weld terminations shall be detailed in a manner that does not result in a notch in the base metal subject to applied tension loads. Components shall not be connected by welds where the weld would prevent the deformation required to provide assumed design conditions.

User Note: Fillet weld terminations should be detailed in a manner that does not result in a notch in the base metal transverse to applied tension loads that can occur as a result of normal fabrication. An accepted practice to avoid notches in base metal is to stop fillet welds short of the edge of the base metal by a length approximately equal to the size of the weld. In most welds, the effect of stopping short can be neglected in strength calculations.

There are two common details where welds are terminated short of the end of the joint to permit relative deformation between the connected parts:

- Welds on the outstanding legs of beam clip-angle connections are returned on the top of the outstanding leg and stopped no more than 4 times the weld size and not greater than half the leg width from the outer toe of the angle.
- Fillet welds connecting transverse stiffeners to webs of girders that are $^3/_4$ in. thick or less are stopped 4 to 6 times the web thickness from the web toe of the flange-to web fillet weld, except where the end of the stiffener is welded to the flange.

Details of fillet weld terminations may be shown on shop standard details.

(h) Fillet welds in holes or slots are permitted to be used to transmit shear and resist loads perpendicular to the faying surface in lap joints or to prevent the buckling or separation of lapped parts and to join components of built-up members. Such fillet welds are permitted to overlap, subject to the provisions of Section J2. Fillet welds in holes or slots are not to be considered plug or slot welds.

(i) For fillet welds in slots, the ends of the slot shall be semicircular or shall have the corners rounded to a radius of not less than the thickness of the part containing it, except those ends which extend to the edge of the part.

3. Plug and Slot Welds

3a. Effective Area

The effective shearing area of plug and slot welds shall be taken as the nominal cross-sectional area of the hole or slot in the plane of the faying surface.

3b. Limitations

Plug or slot welds are permitted to be used to transmit shear in lap joints or to prevent buckling or separation of lapped parts and to join component parts of built-up members, subject to the following limitations:

(a) The diameter of the holes for a plug weld shall not be less than the thickness of the part containing it plus $^5/_{16}$ in. (8 mm), rounded to the next larger odd $^1/_{16}$ in. (even mm), nor greater than the minimum diameter plus $^1/_8$ in. (3 mm) or $2^1/_4$ times the thickness of the weld.

(b) The minimum center-to-center spacing of plug welds shall be four times the diameter of the hole.

(c) The length of slot for a slot weld shall not exceed 10 times the thickness of the weld.

(d) The width of the slot shall be not less than the thickness of the part containing it plus $^5/_{16}$ in. (8 mm) rounded to the next larger odd $^1/_{16}$ in. (even mm), nor shall it be larger than $2^1/_4$ times the thickness of the weld.

(e) The ends of the slot shall be semicircular or shall have the corners rounded to a radius of not less than the thickness of the part containing it.

(f) The minimum spacing of lines of slot welds in a direction transverse to their length shall be four times the width of the slot.

(g) The minimum center-to-center spacing in a longitudinal direction on any line shall be two times the length of the slot.

(h) The thickness of plug or slot welds in material 5/8 in. (16 mm) or less in thickness shall be equal to the thickness of the material. In material over 5/8 in. (16 mm) thick, the thickness of the weld shall be at least one-half the thickness of the material, but not less than 5/8 in. (16 mm).

4. Strength

(a) The design strength, ϕR_n and the allowable strength, R_n / Ω, of welded joints shall be the lower value of the base material strength determined according to the limit states of tensile rupture and shear rupture and the weld metal strength determined according to the limit state of rupture as follows:

For the base metal

$$R_n = F_{nBM} A_{BM} \tag{J2-2}$$

For the weld metal

$$R_n = F_{nw} A_{we} \tag{J2-3}$$

where

A_{BM} = cross-sectional area of the base metal, in.^{2} (mm^2)
A_{we} = effective area of the weld, in.^{2} (mm^2)
F_{nBM} = nominal stress of the base metal, ksi (MPa)
F_{nw} = nominal stress of the weld metal, ksi (MPa)

The values of ϕ, Ω, F_{nBM} and F_{nw}, and limitations thereon, are given in Table J2.5.

(b) For fillet welds, the available strength is permitted to be determined accounting for a directional strength increase of $(1.0 + 0.50\sin^{1.5}\theta)$ if strain compatibility of the various weld elements is considered,

where

$\phi = 0.75$ (LRFD); $\Omega = 2.00$ (ASD)
θ = angle between the line of action of the required force and the weld longitudinal axis, degrees

(1) For a linear weld group with a uniform leg size, loaded through the center of gravity

$$R_n = F_{nw} A_{we} \tag{J2-4}$$

where

$$F_{nw} = 0.60F_{EXX}(1.0 + 0.50\sin^{1.5}\theta), \text{ ksi (MPa)} \tag{J2-5}$$

F_{EXX} = filler metal classification strength, ksi (MPa)

User Note: A linear weld group is one in which all elements are in a line or are parallel.

<table>
<tr><th colspan="6">TABLE J2.5
Available Strength of Welded Joints,
ksi (MPa)</th></tr>
<tr><th>Load Type and Direction Relative to Weld Axis</th><th>Pertinent Metal</th><th>ϕ and Ω</th><th>Nominal Stress (F_{nBM} or F_{nw}), ksi (MPa)</th><th>Effective Area (A_{BM} or A_{we}), in.^{2} (mm^2)</th><th>Required Filler Metal Strength Level[a][b]</th></tr>
<tr><td colspan="6">COMPLETE-JOINT-PENETRATION GROOVE WELDS</td></tr>
<tr><td>Tension—
Normal to weld axis</td><td colspan="4">Strength of the joint is controlled by the base metal.</td><td>Matching filler metal shall be used. For T- and corner-joints with backing left in place, notch tough filler metal is required. See Section J2.6.</td></tr>
<tr><td>Compression—
Normal to weld axis</td><td colspan="4">Strength of the joint is controlled by the base metal.</td><td>Filler metal with a strength level equal to or one strength level less than matching filler metal is permitted.</td></tr>
<tr><td>Tension or compression—
Parallel to weld axis</td><td colspan="4">Tension or compression in parts joined parallel to a weld is permitted to be neglected in design of welds joining the parts.</td><td>Filler metal with a strength level equal to or less than matching filler metal is permitted.</td></tr>
<tr><td>Shear</td><td colspan="4">Strength of the joint is controlled by the base metal.</td><td>Matching filler metal shall be used.[c]</td></tr>
<tr><td colspan="6">PARTIAL-JOINT-PENETRATION GROOVE WELDS INCLUDING FLARE V-GROOVE AND FLARE BEVEL GROOVE WELDS</td></tr>
<tr><td rowspan="2">Tension—
Normal to weld axis</td><td>Base</td><td>ϕ = 0.75
Ω = 2.00</td><td>F_u</td><td>See J4</td><td rowspan="10">Filler metal with a strength level equal to or less than matching filler metal is permitted.</td></tr>
<tr><td>Weld</td><td>ϕ = 0.80
Ω = 1.88</td><td>$0.60F_{EXX}$</td><td>See J2.1a</td></tr>
<tr><td>Compression—
Column to base plate and column splices designed per Section J1.4(a)</td><td colspan="4">Compressive stress is permitted to be neglected in design of welds joining the parts.</td></tr>
<tr><td rowspan="2">Compression—
Connections of members designed to bear other than columns as described in Section J1.4(b)</td><td>Base</td><td>ϕ = 0.90
Ω = 1.67</td><td>F_y</td><td>See J4</td></tr>
<tr><td>Weld</td><td>ϕ = 0.80
Ω = 1.88</td><td>$0.60F_{EXX}$</td><td>See J2.1a</td></tr>
<tr><td rowspan="2">Compression—
Connections not finished-to-bear</td><td>Base</td><td>ϕ = 0.90
Ω = 1.67</td><td>F_y</td><td>See J4</td></tr>
<tr><td>Weld</td><td>ϕ = 0.80
Ω = 1.88</td><td>$0.90F_{EXX}$</td><td>See J2.1a</td></tr>
<tr><td>Tension or compression—
Parallel to weld axis</td><td colspan="4">Tension or compression in parts joined parallel to a weld is permitted to be neglected in design of welds joining the parts.</td></tr>
<tr><td rowspan="2">Shear</td><td>Base</td><td colspan="3">Governed by J4</td></tr>
<tr><td>Weld</td><td>ϕ = 0.75
Ω = 2.00</td><td>$0.60F_{EXX}$</td><td>See J2.1a</td></tr>
</table>

TABLE J2.5 (continued)
Available Strength of Welded Joints, ksi (MPa)

Load Type and Direction Relative to Weld Axis	Pertinent Metal	ϕ and Ω	Nominal Stress (F_{nBM} or F_{nw}), ksi (MPa)	Effective Area (A_{BM} or A_{we}), in.^{2} (mm^2)	Required Filler Metal Strength Level[a][b]
FILLET WELDS INCLUDING FILLETS IN HOLES AND SLOTS AND SKEWED T-JOINTS					
Shear	Base	Governed by J4			Filler metal with a strength level equal to or less than matching filler metal is permitted.
	Weld	$\phi = 0.75$ $\Omega = 2.00$	$0.60F_{EXX}$[d]	See J2.2a	
Tension or compression—Parallel to weld axis	Tension or compression in parts joined parallel to a weld is permitted to be neglected in design of welds joining the parts.				
PLUG AND SLOT WELDS					
Shear—Parallel to faying surface on the effective area	Base	Governed by J4			Filler metal with a strength level equal to or less than matching filler metal is permitted.
	Weld	$\phi = 0.75$ $\Omega = 2.00$	$0.60F_{EXX}$	See J2.3a	

[a] For matching weld metal, see AWS D1.1/D1.1M clause 3.3.
[b] Filler metal with a strength level one strength level greater than matching is permitted.
[c] Filler metals with a strength level less than matching are permitted to be used for groove welds between the webs and flanges of built-up sections transferring shear loads, or in applications where high restraint is a concern. In these applications, the weld joint shall be detailed and the weld shall be designed using the thickness of the material as the effective throat, where $\phi = 0.80$, $\Omega = 1.88$ and $0.60F_{EXX}$ is the nominal strength.
[d] The provisions of Section J2.4(b) are also applicable.

(2) For fillet weld groups concentrically loaded and consisting of elements with a uniform leg size that are oriented both longitudinally and transversely to the direction of applied load, the combined strength, R_n, of the fillet weld group shall be determined as the greater of the following:

(i) $R_n = R_{nwl} + R_{nwt}$ (J2-6a)

or

(ii) $R_n = 0.85\,R_{nwl} + 1.5R_{nwt}$ (J2-6b)

where

R_{nwl} = total nominal strength of longitudinally loaded fillet welds, as determined in accordance with Table J2.5, kips (N)

R_{nwt} = total nominal strength of transversely loaded fillet welds, as determined in accordance with Table J2.5 without the increase in Section J2.4(b), kips (N)

User Note: The instantaneous center method is a valid way to calculate the strength of weld groups consisting of weld elements in various directions based on strain compatibility.

5. Combination of Welds

If two or more of the general types of welds (groove, fillet, plug, slot) are combined in a single joint, the strength of each shall be separately computed with reference to the axis of the group in order to determine the strength of the combination.

6. Filler Metal Requirements

The choice of filler metal for use with CJP groove welds subject to tension normal to the effective area shall comply with the requirements for matching filler metals given in AWS D1.1/D1.1M.

User Note: The following User Note Table summarizes the AWS D1.1/D1.1M provisions for matching filler metals. Other restrictions exist. For a complete list of base metals and prequalified matching filler metals, see AWS D1.1/D1.1M Table 3.1 and Table 3.2.

Base Metal (ASTM)	Matching Filler Metal
A36 ≤ ¾ in. thick	60- and 70-ksi filler metal
A36 > ¾ in., A588[a], A1011, A572 Gr. 50 and 55, A913 Gr. 50, A992, A1018	SMAW: E7015, E7016, E7018, E7028 Other processes: 70-ksi filler metal
A913 Gr. 60 and 65	80-ksi filler metal
A913 Gr. 70	90-ksi filler metal

[a] For corrosion resistance and color similar to the base metal, see AWS D1.1/D1.1M clause 3.7.3.

Notes:
In joints with base metals of different strengths, either a filler metal that matches the higher strength base metal or a filler metal that matches the lower strength and produces a low hydrogen deposit may be used when matching strength is required.

Filler metal with a specified minimum Charpy V-notch toughness of 20 ft-lb (27 J) at 40°F (4°C) or lower shall be used in the following joints:

(a) CJP groove welded T- and corner joints with steel backing left in place, subject to tension normal to the effective area, unless the joints are designed using the nominal strength and resistance factor or safety factor, as applicable, for a PJP groove weld

(b) CJP groove welded splices subject to tension normal to the effective area in heavy sections, as defined in Sections A3.1c and A3.1d

The manufacturer's Certificate of Conformance shall be sufficient evidence of compliance.

7. Mixed Weld Metal

When Charpy V-notch toughness is specified, the process consumables for all weld metal, tack welds, root pass and subsequent passes deposited in a joint shall be compatible to ensure notch-tough composite weld metal.

J3. BOLTS AND THREADED PARTS

ASTM A307 bolts are permitted except where pretensioning is specified.

1. High-Strength Bolts

Use of high-strength bolts shall conform to the provisions of the *Specification for Structural Joints Using High-Strength Bolts*, hereafter referred to as the RCSC *Specification*, as approved by the Research Council on Structural Connections, except as otherwise provided in this Specification. High-strength bolts in this Specification are grouped according to material strength as follows:

Group A—ASTM F3125/F3125M Grades A325, A325M, F1852 and ASTM A354 Grade BC

Group B—ASTM F3125/F3125M Grades A490, A490M, F2280 and ASTM A354 Grade BD

Group C—ASTM F3043 and F3111

Use of Group C high-strength bolt/nut/washer assemblies shall conform to the applicable provisions of their ASTM standard. ASTM F3043 and F3111 Grade 1 assemblies may be installed only to the snug-tight condition. ASTM F3043 and F3111 Grade 2 assemblies may be used in snug-tight, pretensioned and slip-critical connections, using procedures provided in the applicable ASTM standard.

> **User Note:** The use of Group C assemblies is limited to specific building locations and noncorrosive environmental conditions by the applicable ASTM standard.

When assembled, all joint surfaces, including those adjacent to the washers, shall be free of scale, except tight mill scale.

(a) Bolts are permitted to be installed to the snug-tight condition when used in:

(1) Bearing-type connections, except as stipulated in Section E6

(2) Tension or combined shear and tension applications, for Group A bolts only, where loosening or fatigue due to vibration or load fluctuations are not design considerations

(b) Bolts in the following connections shall be pretensioned:

(1) As required by the RCSC *Specification*

(2) Connections subjected to vibratory loads where bolt loosening is a consideration

(3) End connections of built-up members composed of two shapes either interconnected by bolts, or with at least one open side interconnected by perforated cover plates or lacing with tie plates, as required in Section E6.1

(c) The following connections shall be designed as slip critical:

(1) As required by the RCSC *Specification*

(2) The extended portion of bolted, partial-length cover plates, as required in Section F13.3

TABLE J3.1
Minimum Bolt Pretension, kips[a]

Bolt Size, in.	Group A[a] (e.g., A325 Bolts)	Group B[a] (e.g., A490 Bolts)	Group C, Grade 2[b] (e.g., F3043 Gr. 2 bolts)
1/2	12	15	–
5/8	19	24	–
3/4	28	35	–
7/8	39	49	–
1	51	64	90
1 1/8	64	80	113
1 1/4	81	102	143
1 3/8	97	121	–
1 1/2	118	148	–

[a] Equal to 0.70 times the minimum tensile strength of bolts as specified in ASTM F3125/F3125M for Grade A325 and Grade A490 bolts with UNC threads, rounded off to nearest kip.

[b] Equal to 0.70 times the minimum tensile strength of bolts, rounded off to nearest kip, for ASTM F3043 Grade 2 and ASTM F3111 Grade 2.

TABLE J3.1M
Minimum Bolt Pretension, kN[a]

Bolt Size, mm	Group A (e.g., A325M Bolts)	Group B (e.g., A490M Bolts)
M16	91	114
M20	142	179
M22	176	221
M24	205	257
M27	267	334
M30	326	408
M36	475	595

[a] Equal to 0.70 times the minimum tensile strength of bolts, rounded off to nearest kN, as specified in ASTM F3125/F3125M for Grade A325M and Grade A490M bolts with UNC threads.

The snug-tight condition is defined in the RCSC *Specification*. Bolts to be tightened to a condition other than snug tight shall be clearly identified on the design drawings. (See Table J3.1 or J3.1M for minimum bolt pretension for connections designated as pretensioned or slip critical.)

User Note: There are no specific minimum or maximum tension requirements for snug-tight bolts. Bolts that have been pretensioned are permitted in snug-tight connections unless specifically prohibited on design documents.

When bolt requirements cannot be provided within the RCSC *Specification* limitations because of requirements for lengths exceeding 12 diameters or diameters exceeding $1^{1}/_{2}$ in. (38 mm), bolts or threaded rods conforming to Group A or Group B materials are permitted to be used in accordance with the provisions for threaded parts in Table J3.2.

When ASTM A354 Grade BC, A354 Grade BD, or A449 bolts and threaded rods are used in pretensioned connections, the bolt geometry, including the thread pitch, thread length, head and nut(s), shall be equal to or (if larger in diameter) proportional to that required by the RCSC *Specification*. Installation shall comply with all applicable requirements of the RCSC *Specification* with modifications as required for the increased diameter and/or length to provide the design pretension.

2. Size and Use of Holes

The following requirements apply for bolted connections:

(a) The maximum sizes of holes for bolts are given in Table J3.3 or Table J3.3M, except that larger holes, required for tolerance on location of anchor rods in concrete foundations, are permitted in column base details.

(b) Standard holes or short-slotted holes transverse to the direction of the load shall be provided in accordance with the provisions of this Specification, unless oversized holes, short-slotted holes parallel to the load, or long-slotted holes are approved by the engineer of record.

(c) Finger shims up to $^{1}/_{4}$ in. (6 mm) are permitted in slip-critical connections designed on the basis of standard holes without reducing the nominal shear strength of the fastener to that specified for slotted holes.

(d) Oversized holes are permitted in any or all plies of slip-critical connections, but they shall not be used in bearing-type connections.

(e) Short-slotted holes are permitted in any or all plies of slip-critical or bearing-type connections. The slots are permitted without regard to direction of loading in slip-critical connections, but the length shall be normal to the direction of the loading in bearing-type connections.

(f) Long-slotted holes are permitted in only one of the connected parts of either a slip-critical or bearing-type connection at an individual faying surface. Long-slotted holes are permitted without regard to direction of loading in slip-critical connections, but shall be normal to the direction of loading in bearing-type connections.

(g) Washers shall be provided in accordance with the RCSC *Specification* Section 6, except for Group C assemblies, where washers shall be provided in accordance with the applicable ASTM standard.

User Note: When Group C heavy-hex fastener assemblies are used, a single washer is used under the bolt head and a single washer is used under the nut. When Group C twist-off bolt assemblies are used, a single washer is used under the nut. Washers are of the type specified in the ASTM standard for the assembly.

TABLE J3.2
Nominal Strength of Fasteners and Threaded Parts, ksi (MPa)

Description of Fasteners	Nominal Tensile Strength, F_{nt}, ksi (MPa)[a]	Nominal Shear Strength in Bearing-Type Connections, F_{nv}, ksi (MPa)[b]
A307 bolts	45 (310)[c]	27 (186)[c] [d]
Group A (e.g., A325) bolts, when threads are not excluded from shear planes	90 (620)	54 (372)
Group A (e.g., A325) bolts, when threads are excluded from shear planes	90 (620)	68 (469)
Group B (e.g., A490) bolts, when threads are not excluded from shear planes	113 (780)	68 (469)
Group B (e.g., A490) bolts, when threads are excluded from shear planes	113 (780)	84 (579)
Group C (e.g., F3043) bolt assemblies, when threads and transition area of shank are not excluded from the shear plane	150 (1040)	90 (620)
Group C (e.g., F3043) bolt assemblies, when threads and transition area of shank are excluded from the shear plane	150 (1040)	113 (779)
Threaded parts meeting the requirements of Section A3.4, when threads are not excluded from shear planes	$0.75F_u$	$0.450F_u$
Threaded parts meeting the requirements of Section A3.4, when threads are excluded from shear planes	$0.75F_u$	$0.563F_u$

[a] For high-strength bolts subject to tensile fatigue loading, see Appendix 3.
[b] For end loaded connections with a fastener pattern length greater than 38 in. (950 mm), F_{nv} shall be reduced to 83.3% of the tabulated values. Fastener pattern length is the maximum distance parallel to the line of force between the centerline of the bolts connecting two parts with one faying surface.
[c] For A307 bolts, the tabulated values shall be reduced by 1% for each $^1/_{16}$ in. (2 mm) over five diameters of length in the grip.
[d] Threads permitted in shear planes.

TABLE J3.3
Nominal Hole Dimensions, in.

Bolt Diameter, in.	Hole Dimensions			
	Standard (Dia.)	Oversize (Dia.)	Short-Slot (Width × Length)	Long-Slot (Width × Length)
1/2	9/16	5/8	9/16 × 11/16	9/16 × 1 1/4
5/8	11/16	13/16	11/16 × 7/8	11/16 × 1 9/16
3/4	13/16	15/16	13/16 × 1	13/16 × 1 7/8
7/8	15/16	1 1/16	15/16 × 1 1/8	15/16 × 2 3/16
1	1 1/8	1 1/4	1 1/8 × 1 5/16	1 1/8 × 2½
≥ 1 1/8	d + 1/8	d + 5/16	(d + 1/8) × (d + 3/8)	(d + 1/8) × 2.5d

TABLE J3.3M
Nominal Hole Dimensions, mm

Bolt Diameter, mm	Hole Dimensions			
	Standard (Dia.)	Oversize (Dia.)	Short-Slot (Width × Length)	Long-Slot (Width × Length)
M16	18	20	18 × 22	18 × 40
M20	22	24	22 × 26	22 × 50
M22	24	28	24 × 30	24 × 55
M24	27[a]	30	27 × 32	27 × 60
M27	30	35	30 × 37	30 × 67
M30	33	38	33 × 40	33 × 75
≥ M36	d + 3	d + 8	(d + 3) × (d + 10)	(d + 3) × 2.5d

[a] Clearance provided allows the use of a 1-in.-diameter bolt.

3. Minimum Spacing

The distance between centers of standard, oversized or slotted holes shall not be less than $2^2/_3$ times the nominal diameter, d, of the fastener. However, the clear distance between bolt holes or slots shall not be less than d.

User Note: A distance between centers of standard, oversize or slotted holes of 3d is preferred.

4. Minimum Edge Distance

The distance from the center of a standard hole to an edge of a connected part in any direction shall not be less than either the applicable value from Table J3.4 or Table J3.4M, or as required in Section J3.10. The distance from the center of an oversized or slotted hole to an edge of a connected part shall be not less than that required for a standard hole to an edge of a connected part plus the applicable increment, C_2, from Table J3.5 or Table J3.5M.

User Note: The edge distances in Tables J3.4 and J3.4M are minimum edge distances based on standard fabrication practices and workmanship tolerances. The appropriate provisions of Sections J3.10 and J4 must be satisfied.

5. Maximum Spacing and Edge Distance

The maximum distance from the center of any bolt to the nearest edge of parts in contact shall be 12 times the thickness of the connected part under consideration, but shall not exceed 6 in. (150 mm). The longitudinal spacing of fasteners between elements consisting of a plate and a shape, or two plates, in continuous contact shall be as follows:

(a) For painted members or unpainted members not subject to corrosion, the spacing shall not exceed 24 times the thickness of the thinner part or 12 in. (300 mm).

(b) For unpainted members of weathering steel subject to atmospheric corrosion, the spacing shall not exceed 14 times the thickness of the thinner part or 7 in. (180 mm).

User Note: The dimensions in (a) and (b) do not apply to elements consisting of two shapes in continuous contact.

6. Tensile and Shear Strength of Bolts and Threaded Parts

The design tensile or shear strength, ϕR_n, and the allowable tensile or shear strength, R_n/Ω, of a snug-tightened or pretensioned high-strength bolt or threaded part shall be determined according to the limit states of tension rupture and shear rupture as:

$$R_n = F_n A_b \tag{J3-1}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

where

A_b = nominal unthreaded body area of bolt or threaded part, in.^{2} (mm^2)

F_n = nominal tensile stress, F_{nt}, or shear stress, F_{nv}, from Table J3.2, ksi (MPa)

The required tensile strength shall include any tension resulting from prying action produced by deformation of the connected parts.

TABLE J3.4
Minimum Edge Distance[a] from Center of Standard Hole[b] to Edge of Connected Part, in.

Bolt Diameter, in.	Minimum Edge Distance
1/2	3/4
5/8	7/8
3/4	1
7/8	1 1/8
1	1 1/4
1 1/8	1 1/2
1 1/4	1 5/8
Over 1 1/4	1 1/4d

[a] If necessary, lesser edge distances are permitted provided the applicable provisions from Sections J3.10 and J4 are satisfied, but edge distances less than one bolt diameter are not permitted without approval from the engineer of record.
[b] For oversized or slotted holes, see Table J3.5.

TABLE J3.4M
Minimum Edge Distance[a] from Center of Standard Hole[b] to Edge of Connected Part, mm

Bolt Diameter, mm	Minimum Edge Distance
16	22
20	26
22	28
24	30
27	34
30	38
36	46
Over 36	1.25d

[a] If necessary, lesser edge distances are permitted provided the applicable provisions from Sections J3.10 and J4 are satisfied, but edge distances less than one bolt diameter are not permitted without approval from the engineer of record.
[b] For oversized or slotted holes, see Table J3.5M.

TABLE J3.5
Values of Edge Distance Increment C_2, in.

Nominal Diameter of Fastener	Oversized Holes	Slotted Holes: Long Axis Perpendicular to Edge — Short Slots	Slotted Holes: Long Axis Perpendicular to Edge — Long Slots[a]	Slotted Holes: Long Axis Parallel to Edge
≤ 7/8	1/16	1/8	3/4*d*	0
1	1/8	1/8		
≥ 1 1/8	1/8	3/16		

[a] When the length of the slot is less than the maximum allowable (see Table J3.3), C_2 is permitted to be reduced by one-half the difference between the maximum and actual slot lengths.

TABLE J3.5M
Values of Edge Distance Increment C_2, mm

Nominal Diameter of Fastener	Oversized Holes	Slotted Holes: Long Axis Perpendicular to Edge — Short Slots	Slotted Holes: Long Axis Perpendicular to Edge — Long Slots[a]	Slotted Holes: Long Axis Parallel to Edge
≤ 22	2	3	0.75*d*	0
24	3	3		
≥ 27	3	5		

[a] When the length of the slot is less than the maximum allowable (see Table J3.3M), C_2 is permitted to be reduced by one-half the difference between the maximum and actual slot lengths.

User Note: The force that can be resisted by a snug-tightened or pretensioned high-strength bolt or threaded part may be limited by the bearing strength at the bolt hole per Section J3.10. The effective strength of an individual fastener may be taken as the lesser of the fastener shear strength per Section J3.6 or the bearing strength at the bolt hole per Section J3.10. The strength of the bolt group is taken as the sum of the effective strengths of the individual fasteners.

7. Combined Tension and Shear in Bearing-Type Connections

The available tensile strength of a bolt subjected to combined tension and shear shall be determined according to the limit states of tension and shear rupture as:

$$R_n = F'_{nt} A_b \tag{J3-2}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

where

F'_{nt} = nominal tensile stress modified to include the effects of shear stress, ksi (MPa)

$$= 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_{rv} \le F_{nt} \quad \text{(LRFD)} \tag{J3-3a}$$

$$= 1.3F_{nt} - \frac{\Omega F_{nt}}{F_{nv}} f_{rv} \le F_{nt} \quad \text{(ASD)} \tag{J3-3b}$$

F_{nt} = nominal tensile stress from Table J3.2, ksi (MPa)
F_{nv} = nominal shear stress from Table J3.2, ksi (MPa)
f_{rv} = required shear stress using LRFD or ASD load combinations, ksi (MPa)

The available shear stress of the fastener shall equal or exceed the required shear stress, f_{rv}.

User Note: Note that when the required stress, f, in either shear or tension, is less than or equal to 30% of the corresponding available stress, the effects of combined stress need not be investigated. Also note that Equations J3-3a and J3-3b can be rewritten so as to find a nominal shear stress, F'_{nv}, as a function of the required tensile stress, f_t.

8. High-Strength Bolts in Slip-Critical Connections

Slip-critical connections shall be designed to prevent slip and for the limit states of bearing-type connections. When slip-critical bolts pass through fillers, all surfaces subject to slip shall be prepared to achieve design slip resistance.

The single bolt available slip resistance for the limit state of slip shall be determined as follows:

$$R_n = \mu D_u h_f T_b n_s \tag{J3-4}$$

(a) For standard size and short-slotted holes perpendicular to the direction of the load

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

(b) For oversized and short-slotted holes parallel to the direction of the load

$$\phi = 0.85 \text{ (LRFD)} \qquad \Omega = 1.76 \text{ (ASD)}$$

(c) For long-slotted holes

$$\phi = 0.70 \text{ (LRFD)} \qquad \Omega = 2.14 \text{ (ASD)}$$

where

D_u = 1.13, a multiplier that reflects the ratio of the mean installed bolt pretension to the specified minimum bolt pretension. The use of other values are permitted if approved by the engineer of record.
T_b = minimum fastener tension given in Table J3.1, kips, or Table J3.1M, kN

h_f = factor for fillers, determined as follows:

(1) For one filler between connected parts

$$h_f = 1.0$$

(2) For two or more fillers between connected parts

$$h_f = 0.85$$

n_s = number of slip planes required to permit the connection to slip
μ = mean slip coefficient for Class A or B surfaces, as applicable, and determined as follows, or as established by tests:

(1) For Class A surfaces (unpainted clean mill scale steel surfaces or surfaces with Class A coatings on blast-cleaned steel or hot-dipped galvanized and roughened surfaces)

$$\mu = 0.30$$

(2) For Class B surfaces (unpainted blast-cleaned steel surfaces or surfaces with Class B coatings on blast-cleaned steel)

$$\mu = 0.50$$

9. Combined Tension and Shear in Slip-Critical Connections

When a slip-critical connection is subjected to an applied tension that reduces the net clamping force, the available slip resistance per bolt from Section J3.8 shall be multiplied by the factor, k_{sc}, determined as follows:

$$k_{sc} = 1 - \frac{T_u}{D_u T_b n_b} \geq 0 \quad \text{(LRFD)} \tag{J3-5a}$$

$$k_{sc} = 1 - \frac{1.5T_a}{D_u T_b n_b} \geq 0 \quad \text{(ASD)} \tag{J3-5b}$$

where
T_a = required tension force using ASD load combinations, kips (kN)
T_u = required tension force using LRFD load combinations, kips (kN)
n_b = number of bolts carrying the applied tension

10. Bearing and Tearout Strength at Bolt Holes

The available strength, ϕR_n and R_n/Ω, at bolt holes shall be determined for the limit states of bearing and tearout, as follows:

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

The nominal strength of the connected material, R_n, is determined as follows:

(a) For a bolt in a connection with standard, oversized and short-slotted holes, independent of the direction of loading, or a long-slotted hole with the slot parallel to the direction of the bearing force

(1) Bearing

(i) When deformation at the bolt hole at service load is a design consideration

$$R_n = 2.4dtF_u \tag{J3-6a}$$

(ii) When deformation at the bolt hole at service load is not a design consideration

$$R_n = 3.0dtF_u \tag{J3-6b}$$

(2) Tearout

(i) When deformation at the bolt hole at service load is a design consideration

$$R_n = 1.2l_c tF_u \tag{J3-6c}$$

(ii) When deformation at the bolt hole at service load is not a design consideration

$$R_n = 1.5l_c tF_u \tag{J3-6d}$$

(b) For a bolt in a connection with long-slotted holes with the slot perpendicular to the direction of force

(1) Bearing

$$R_n = 2.0dtF_u \tag{J3-6e}$$

(2) Tearout

$$R_n = 1.0l_c tF_u \tag{J3-6f}$$

(c) For connections made using bolts that pass completely through an unstiffened box member or HSS, see Section J7 and Equation J7-1;

where

F_u = specified minimum tensile strength of the connected material, ksi (MPa)
d = nominal fastener diameter, in. (mm)
l_c = clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material, in. (mm)
t = thickness of connected material, in. (mm)

Bearing strength and tearout strength shall be checked for both bearing-type and slip-critical connections. The use of oversized holes and short- and long-slotted holes parallel to the line of force is restricted to slip-critical connections per Section J3.2.

11. Special Fasteners

The nominal strength of special fasteners other than the bolts presented in Table J3.2 shall be verified by tests.

12. Wall Strength at Tension Fasteners

When bolts or other fasteners in tension are attached to an unstiffened box or HSS wall, the strength of the wall shall be determined by rational analysis.

J4. AFFECTED ELEMENTS OF MEMBERS AND CONNECTING ELEMENTS

This section applies to elements of members at connections and connecting elements, such as plates, gussets, angles and brackets.

1. Strength of Elements in Tension

The design strength, ϕR_n, and the allowable strength, R_n/Ω, of affected and connecting elements loaded in tension shall be the lower value obtained according to the limit states of tensile yielding and tensile rupture.

(a) For tensile yielding of connecting elements

$$R_n = F_y A_g \tag{J4-1}$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

(b) For tensile rupture of connecting elements

$$R_n = F_u A_e \tag{J4-2}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

where
A_e = effective net area as defined in Section D3, in.^{2} (mm^2)

User Note: The effective net area of the connection plate may be limited due to stress distribution as calculated by methods such as the Whitmore section.

2. Strength of Elements in Shear

The available shear strength of affected and connecting elements in shear shall be the lower value obtained according to the limit states of shear yielding and shear rupture:

(a) For shear yielding of the element

$$R_n = 0.60 F_y A_{gv} \tag{J4-3}$$

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

where
A_{gv} = gross area subject to shear, in.^{2} (mm^2)

(b) For shear rupture of the element

$$R_n = 0.60 F_u A_{nv} \tag{J4-4}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

where
A_{nv} = net area subject to shear, in.^{2} (mm^2)

3. Block Shear Strength

The available strength for the limit state of block shear rupture along a shear failure path or paths and a perpendicular tension failure path shall be determined as follows:

$$R_n = 0.60F_uA_{nv} + U_{bs}F_uA_{nt} \leq 0.60F_yA_{gv} + U_{bs}F_uA_{nt} \tag{J4-5}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

where

A_{nt} = net area subject to tension, in.^{2} (mm^2)

Where the tension stress is uniform, $U_{bs} = 1$; where the tension stress is nonuniform, $U_{bs} = 0.5$.

User Note: Typical cases where U_{bs} should be taken equal to 0.5 are illustrated in the Commentary.

4. Strength of Elements in Compression

The available strength of connecting elements in compression for the limit states of yielding and buckling shall be determined as follows:

(a) When $L_c / r \leq 25$

$$P_n = F_yA_g \tag{J4-6}$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

(b) When $L_c / r > 25$, the provisions of Chapter E apply;

where

$L_c = KL$ = effective length, in. (mm)

K = effective length factor

L = laterally unbraced length of the member, in. (mm)

User Note: The effective length factors used in computing compressive strengths of connecting elements are specific to the end restraint provided and may not necessarily be taken as unity when the direct analysis method is employed.

5. Strength of Elements in Flexure

The available flexural strength of affected elements shall be the lower value obtained according to the limit states of flexural yielding, local buckling, flexural lateral-torsional buckling, and flexural rupture.

J5. FILLERS

1. Fillers in Welded Connections

Whenever it is necessary to use fillers in joints required to transfer applied force, the fillers and the connecting welds shall conform to the requirements of Section J5.1a or Section J5.1b, as applicable.

1a. Thin Fillers

Fillers less than 1/4 in. (6 mm) thick shall not be used to transfer stress. When the thickness of the fillers is less than 1/4 in. (6 mm), or when the thickness of the filler is 1/4 in. (6 mm) or greater but not sufficient to transfer the applied force between the connected parts, the filler shall be kept flush with the edge of the outside connected part, and the size of the weld shall be increased over the required size by an amount equal to the thickness of the filler.

1b. Thick Fillers

When the thickness of the fillers is sufficient to transfer the applied force between the connected parts, the filler shall extend beyond the edges of the outside connected base metal. The welds joining the outside connected base metal to the filler shall be sufficient to transmit the force to the filler and the area subjected to the applied force in the filler shall be sufficient to prevent overstressing the filler. The welds joining the filler to the inside connected base metal shall be sufficient to transmit the applied force.

2. Fillers in Bolted Bearing-Type Connections

When a bolt that carries load passes through fillers that are equal to or less than 1/4 in. (6 mm) thick, the shear strength shall be used without reduction. When a bolt that carries load passes through fillers that are greater than 1/4 in. (6 mm) thick, one of the following requirements shall apply:

(a) The shear strength of the bolts shall be multiplied by the factor

$$1 - 0.4(t - 0.25)$$

$$1 - 0.0154(t - 6) \quad \text{(S.I.)}$$

but not less than 0.85, where t is the total thickness of the fillers.

(b) The fillers shall be welded or extended beyond the joint and bolted to uniformly distribute the total force in the connected element over the combined cross section of the connected element and the fillers.

(c) The size of the joint shall be increased to accommodate a number of bolts that is equivalent to the total number required in (b).

J6. SPLICES

Groove-welded splices in plate girders and beams shall develop the nominal strength of the smaller spliced section. Other types of splices in cross sections of plate girders and beams shall develop the strength required by the forces at the point of the splice.

J7. BEARING STRENGTH

The design bearing strength, ϕR_n, and the allowable bearing strength, R_n / Ω, of surfaces in contact shall be determined for the limit state of bearing (local compressive yielding) as follows:

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

The nominal bearing strength, R_n, shall be determined as follows:

(a) For finished surfaces, pins in reamed, drilled, or bored holes, and ends of fitted bearing stiffeners

$$R_n = 1.8 F_y A_{pb} \tag{J7-1}$$

where

A_{pb} = projected area in bearing, in.^{2} (mm^2)

F_y = specified minimum yield stress, ksi (MPa)

(b) For expansion rollers and rockers

(1) When $d \le 25$ in. (630 mm)

$$R_n = \frac{1.2(F_y - 13)\, l_b\, d}{20} \tag{J7-2}$$

$$R_n = \frac{1.2(F_y - 90)\, l_b\, d}{20} \tag{J7-2M}$$

(2) When $d > 25$ in. (630 mm)

$$R_n = \frac{6.0(F_y - 13)\, l_b \sqrt{d}}{20} \tag{J7-3}$$

$$R_n = \frac{30.2(F_y - 90)\, l_b \sqrt{d}}{20} \tag{J7-3M}$$

where

d = diameter, in. (mm)

l_b = length of bearing, in. (mm)

J8. COLUMN BASES AND BEARING ON CONCRETE

Provisions shall be made to transfer the column loads and moments to the footings and foundations.

In the absence of code regulations, the design bearing strength, $\phi_c P_p$, and the allowable bearing strength, P_p / Ω_c, for the limit state of concrete crushing are permitted to be taken as follows:

$$\phi_c = 0.65 \text{ (LRFD)} \qquad \Omega_c = 2.31 \text{ (ASD)}$$

The nominal bearing strength, P_p, is determined as follows:

(a) On the full area of a concrete support

$$P_p = 0.85 f'_c A_1 \tag{J8-1}$$

(b) On less than the full area of a concrete support

$$P_p = 0.85 f'_c A_1 \sqrt{A_2 / A_1} \le 1.7 f'_c A_1 \tag{J8-2}$$

where

A_1 = area of steel concentrically bearing on a concrete support, in.^{2} (mm^2)
A_2 = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, in.^{2} (mm^2)
f'_c = specified compressive strength of concrete, ksi (MPa)

J9. ANCHOR RODS AND EMBEDMENTS

Anchor rods shall be designed to provide the required resistance to loads on the completed structure at the base of columns including the net tensile components of any bending moment resulting from load combinations stipulated in Section B2. The anchor rods shall be designed in accordance with the requirements for threaded parts in Table J3.2.

Design of anchor rods for the transfer of forces to the concrete foundation shall satisfy the requirements of ACI 318 (ACI 318M) or ACI 349 (ACI 349M).

User Note: Column bases should be designed considering bearing against concrete elements, including when columns are required to resist a horizontal force at the base plate. See AISC Design Guide 1, *Base Plate and Anchor Rod Design*, Second Edition, for column base design information.

When anchor rods are used to resist horizontal forces, hole size, anchor rod setting tolerance, and the horizontal movement of the column shall be considered in the design.

Larger oversized holes and slotted holes are permitted in base plates when adequate bearing is provided for the nut by using ASTM F844 washers or plate washers to bridge the hole.

User Note: The permitted hole sizes, corresponding washer dimensions and nuts are given in the AISC *Steel Construction Manual* and ASTM F1554. ASTM F1554 anchor rods may be furnished in accordance with product specifications with a body diameter less than the nominal diameter. Load effects such as bending and elongation should be calculated based on minimum diameters permitted by the product specification. See ASTM F1554 and the table, “Applicable ASTM Specifications for Various Types of Structural Fasteners,” in Part 2 of the AISC *Steel Construction Manual*.

User Note: See ACI 318 (ACI 318M) for embedment design and for shear friction design. See OSHA for special erection requirements for anchor rods.

J10. FLANGES AND WEBS WITH CONCENTRATED FORCES

This section applies to single- and double-concentrated forces applied normal to the flange(s) of wide-flange sections and similar built-up shapes. A single-concentrated force is either tensile or compressive. Double-concentrated forces are one tensile and one compressive and form a couple on the same side of the loaded member.

When the required strength exceeds the available strength as determined for the limit states listed in this section, stiffeners and/or doublers shall be provided and shall be sized for the difference between the required strength and the available strength for the applicable limit state. Stiffeners shall also meet the design requirements in Section J10.8. Doublers shall also meet the design requirement in Section J10.9.

User Note: See Appendix 6, Section 6.3 for requirements for the ends of cantilever members.

Stiffeners are required at unframed ends of beams in accordance with the requirements of Section J10.7.

User Note: Design guidance for members other than wide-flange sections and similar built-up shapes can be found in the Commentary.

1. Flange Local Bending

This section applies to tensile single-concentrated forces and the tensile component of double-concentrated forces.

The design strength, ϕR_n, and the allowable strength, R_n/Ω, for the limit state of flange local bending shall be determined as:

$$R_n = 6.25 F_{yf} t_f^2 \tag{J10-1}$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

where

F_{yf} = specified minimum yield stress of the flange, ksi (MPa)

t_f = thickness of the loaded flange, in. (mm)

If the length of loading across the member flange is less than $0.15b_f$, where b_f is the member flange width, Equation J10-1 need not be checked.

When the concentrated force to be resisted is applied at a distance from the member end that is less than $10t_f$, R_n shall be reduced by 50%.

When required, a pair of transverse stiffeners shall be provided.

2. Web Local Yielding

This section applies to single-concentrated forces and both components of double-concentrated forces.

The available strength for the limit state of web local yielding shall be determined as follows:

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

The nominal strength, R_n, shall be determined as follows:

(a) When the concentrated force to be resisted is applied at a distance from the member end that is greater than the full nominal depth of the member, d,

$$R_n = F_{yw} t_w (5k + l_b) \tag{J10-2}$$

(b) When the concentrated force to be resisted is applied at a distance from the member end that is less than or equal to the full nominal depth of the member, d,

$$R_n = F_{yw} t_w (2.5k + l_b) \tag{J10-3}$$

where

F_{yw} = specified minimum yield stress of the web material, ksi (MPa)
k = distance from outer face of the flange to the web toe of the fillet, in. (mm)
l_b = length of bearing (not less than k for end beam reactions), in. (mm)
t_w = thickness of web, in. (mm)

When required, a pair of transverse stiffeners or a doubler plate shall be provided.

3. Web Local Crippling

This section applies to compressive single-concentrated forces or the compressive component of double-concentrated forces.

The available strength for the limit state of web local crippling shall be determined as follows:

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

The nominal strength, R_n, shall be determined as follows:

(a) When the concentrated compressive force to be resisted is applied at a distance from the member end that is greater than or equal to $d/2$

$$R_n = 0.80 t_w^2 \left[1 + 3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5} \right] \sqrt{\frac{E F_{yw} t_f}{t_w}}\, Q_f \tag{J10-4}$$

(b) When the concentrated compressive force to be resisted is applied at a distance from the member end that is less than $d/2$

(1) For $l_b/d \le 0.2$

$$R_n = 0.40t_w^2\left[1+3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}}\,Q_f \qquad \text{(J10-5a)}$$

(2) For $l_b/d > 0.2$

$$R_n = 0.40t_w^2\left[1+\left(\frac{4l_b}{d}-0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}}\,Q_f \qquad \text{(J10-5b)}$$

where

d = full nominal depth of the member, in. (mm)

Q_f = 1.0 for wide-flange sections and for HSS (connecting surface) in tension
= as given in Table K3.2 for all other HSS conditions

When required, a transverse stiffener, a pair of transverse stiffeners, or a doubler plate extending at least three quarters of the depth of the web shall be provided.

4. Web Sidesway Buckling

This section applies only to compressive single-concentrated forces applied to members where relative lateral movement between the loaded compression flange and the tension flange is not restrained at the point of application of the concentrated force.

The available strength of the web for the limit state of sidesway buckling shall be determined as follows:

$$\phi = 0.85 \text{ (LRFD)} \qquad \Omega = 1.76 \text{ (ASD)}$$

The nominal strength, R_n, shall be determined as follows:

(a) If the compression flange is restrained against rotation

(1) When $(h/t_w)/(L_b/b_f) \le 2.3$

$$R_n = \frac{C_r t_w^3 t_f}{h^2}\left[1+0.4\left(\frac{h/t_w}{L_b/b_f}\right)^3\right] \qquad \text{(J10-6)}$$

(2) When $(h/t_w)/(L_b/b_f) > 2.3$, the limit state of web sidesway buckling does not apply.

When the required strength of the web exceeds the available strength, local lateral bracing shall be provided at the tension flange or either a pair of transverse stiffeners or a doubler plate shall be provided.

(b) If the compression flange is not restrained against rotation

(1) When $(h/t_w)/(L_b/b_f) \le 1.7$

$$R_n = \frac{C_r t_w^3 t_f}{h^2}\left[0.4\left(\frac{h/t_w}{L_b/b_f}\right)^3\right] \qquad \text{(J10-7)}$$

(2) When $(h/t_w)/(L_b/b_f) > 1.7$, the limit state of web sidesway buckling does not apply.

When the required strength of the web exceeds the available strength, local lateral bracing shall be provided at both flanges at the point of application of the concentrated forces.

In Equations J10-6 and J10-7, the following definitions apply:

C_r = 960,000 ksi (6.6×10^6 MPa), when $M_u < M_y$ (LRFD) or $1.5M_a < M_y$ (ASD) at the location of the force
= 480,000 ksi (3.3×10^6 MPa), when $M_u \geq M_y$ (LRFD) or $1.5M_a \geq M_y$ (ASD) at the location of the force

L_b = largest laterally unbraced length along either flange at the point of load, in. (mm)

M_a = required flexural strength using ASD load combinations, kip-in. (N-mm)

M_u = required flexural strength using LRFD load combinations, kip-in. (N-mm)

b_f = width of flange, in. (mm)

h = clear distance between flanges less the fillet or corner radius for rolled shapes; distance between adjacent lines of fasteners or the clear distance between flanges when welds are used for built-up shapes, in. (mm)

User Note: For determination of adequate restraint, refer to Appendix 6.

5. Web Compression Buckling

This section applies to a pair of compressive single-concentrated forces or the compressive components in a pair of double-concentrated forces, applied at both flanges of a member at the same location.

The available strength for the limit state of web compression buckling shall be determined as follows:

$$R_n = \left(\frac{24 t_w^3 \sqrt{E F_{yw}}}{h} \right) Q_f \quad \text{(J10-8)}$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

where

Q_f = 1.0 for wide-flange sections and for HSS (connecting surface) in tension
= as given in Table K3.2 for all other HSS conditions

When the pair of concentrated compressive forces to be resisted is applied at a distance from the member end that is less than $d/2$, R_n shall be reduced by 50%.

When required, a single transverse stiffener, a pair of transverse stiffeners, or a doubler plate extending the full depth of the web shall be provided.

6. Web Panel-Zone Shear

This section applies to double-concentrated forces applied to one or both flanges of a member at the same location.

The available strength of the web panel zone for the limit state of shear yielding shall be determined as follows:

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

The nominal strength, R_n, shall be determined as follows:

(a) When the effect of inelastic panel-zone deformation on frame stability is not accounted for in the analysis:

(1) For $\alpha P_r \leq 0.4P_y$

$$R_n = 0.60F_y d_c t_w \tag{J10-9}$$

(2) For $\alpha P_r > 0.4P_y$

$$R_n = 0.60F_y d_c t_w \left(1.4 - \frac{\alpha P_r}{P_y}\right) \tag{J10-10}$$

(b) When the effect of inelastic panel-zone deformation on frame stability is accounted for in the analysis:

(1) For $\alpha P_r \leq 0.75P_y$

$$R_n = 0.60F_y d_c t_w \left(1 + \frac{3b_{cf} t_{cf}^2}{d_b d_c t_w}\right) \tag{J10-11}$$

(2) For $\alpha P_r > 0.75P_y$

$$R_n = 0.60F_y d_c t_w \left(1 + \frac{3b_{cf} t_{cf}^2}{d_b d_c t_w}\right)\left(1.9 - \frac{1.2\alpha P_r}{P_y}\right) \tag{J10-12}$$

In Equations J10-9 through J10-12, the following definitions apply:

A_g = gross cross-sectional area of member, in.^{2} (mm^2)
F_y = specified minimum yield stress of the column web, ksi (MPa)
P_r = required axial strength using LRFD or ASD load combinations, kips (N)
P_y = $F_y A_g$, axial yield strength of the column, kips (N)
b_{cf} = width of column flange, in. (mm)
d_b = depth of beam, in. (mm)
d_c = depth of column, in. (mm)
t_{cf} = thickness of column flange, in. (mm)
t_w = thickness of column web, in. (mm)
α = 1.0 (LRFD); = 1.6 (ASD)

When required, doubler plate(s) or a pair of diagonal stiffeners shall be provided within the boundaries of the rigid connection whose webs lie in a common plane.

See Section J10.9 for doubler plate design requirements.

7. Unframed Ends of Beams and Girders

At unframed ends of beams and girders not otherwise restrained against rotation about their longitudinal axes, a pair of transverse stiffeners, extending the full depth of the web, shall be provided.

8. Additional Stiffener Requirements for Concentrated Forces

Stiffeners required to resist tensile concentrated forces shall be designed in accordance with the requirements of Section J4.1 and welded to the loaded flange and the web. The welds to the flange shall be sized for the difference between the required strength and available strength. The stiffener to web welds shall be sized to transfer to the web the algebraic difference in tensile force at the ends of the stiffener.

Stiffeners required to resist compressive concentrated forces shall be designed in accordance with the requirements in Section J4.4 and shall either bear on or be welded to the loaded flange and welded to the web. The welds to the flange shall be sized for the difference between the required strength and the applicable limit state strength. The weld to the web shall be sized to transfer to the web the algebraic difference in compression force at the ends of the stiffener. For fitted bearing stiffeners, see Section J7.

Transverse full depth bearing stiffeners for compressive forces applied to a beam or plate girder flange(s) shall be designed as axially compressed members (columns) in accordance with the requirements of Section E6.2 and Section J4.4. The member properties shall be determined using an effective length of $0.75h$ and a cross section composed of two stiffeners, and a strip of the web having a width of $25t_w$ at interior stiffeners and $12t_w$ at the ends of members. The weld connecting full depth bearing stiffeners to the web shall be sized to transmit the difference in compressive force at each of the stiffeners to the web.

Transverse and diagonal stiffeners shall comply with the following additional requirements:

(a) The width of each stiffener plus one-half the thickness of the column web shall not be less than one-third of the flange or moment connection plate width delivering the concentrated force.

(b) The thickness of a stiffener shall not be less than one-half the thickness of the flange or moment connection plate delivering the concentrated load, nor less than the width divided by 16.

(c) Transverse stiffeners shall extend a minimum of one-half the depth of the member except as required in Sections J10.3, J10.5 and J10.7.

9. Additional Doubler Plate Requirements for Concentrated Forces

Doubler plates required for compression strength shall be designed in accordance with the requirements of Chapter E.

Doubler plates required for tensile strength shall be designed in accordance with the requirements of Chapter D.

Doubler plates required for shear strength (see Section J10.6) shall be designed in accordance with the provisions of Chapter G.

Doubler plates shall comply with the following additional requirements:

(a) The thickness and extent of the doubler plate shall provide the additional material necessary to equal or exceed the strength requirements.

(b) The doubler plate shall be welded to develop the proportion of the total force transmitted to the doubler plate.

10. Transverse Forces on Plate Elements

When a force is applied transverse to the plane of a plate element, the nominal strength shall consider the limit states of shear and flexure in accordance with Sections J4.2 and J4.5.

User Note: The flexural strength can be checked based on yield-line theory and the shear strength can be determined based on a punching shear model. See AISC *Steel Construction Manual* Part 9 for further discussion.

CHAPTER K

ADDITIONAL REQUIREMENTS FOR HSS AND BOX-SECTION CONNECTIONS

This chapter addresses additional requirements for connections to HSS members and box sections of uniform wall thickness, where seam welds between box-section elements are complete-joint-penetration (CJP) groove welds in the connection region. The requirements of Chapter J also apply.

The chapter is organized as follows:

K1. General Provisions and Parameters for HSS Connections
K2. Concentrated Forces on HSS
K3. HSS-to-HSS Truss Connections
K4. HSS-to-HSS Moment Connections
K5. Welds of Plates and Branches to Rectangular HSS

K1. GENERAL PROVISIONS AND PARAMETERS FOR HSS CONNECTIONS

For the purposes of this chapter, the centerlines of branch members and chord members shall lie in a common plane. Rectangular HSS connections are further limited to having all members oriented with walls parallel to the plane.

The tables in this chapter are often accompanied by limits of applicability. Connections complying with the limits of applicability listed can be designed considering only those limit states provided for each joint configuration. Connections not complying with the limits of applicability listed are not prohibited and must be designed by rational analysis.

User Note: The connection strengths calculated in Chapter K, including the applicable sections of Chapter J, are based on strength limit states only. See the Commentary if excessive connection deformations may cause serviceability or stability concerns.

User Note: Connection strength is often governed by the size of HSS members, especially the wall thickness of truss chords, and this must be considered in the initial design. To ensure economical and dependable connections can be designed, the connections should be considered in the design of the members. Angles between the chord and the branch(es) of less than 30° can make welding and inspection difficult and should be avoided. The limits of applicability provided reflect limitations on tests conducted to date, measures to eliminate undesirable limit states, and other considerations. See Section J3.10(c) for through-bolt provisions.

This section provides parameters to be used in the design of plate-to-HSS and HSS-to-HSS connections.

The design strength, ϕR_n, ϕM_n and ϕP_n, and the allowable strength, R_n / Ω, M_n / Ω and P_n / Ω, of connections shall be determined in accordance with the provisions of this chapter and the provisions of Chapter B.

1. Definitions of Parameters

A_g = gross cross-sectional area of member, in.^{2} (mm^2)
B = overall width of rectangular HSS main member, measured 90° to the plane of the connection, in. (mm)
B_b = overall width of rectangular HSS branch member or plate, measured 90° to the plane of the connection, in. (mm)
B_e = effective width of rectangular HSS branch member or plate, in. (mm)
D = outside diameter of round HSS main member, in. (mm)
D_b = outside diameter of round HSS branch member, in. (mm)
F_c = available stress in main member, ksi (MPa)
= F_y for LRFD; $0.60F_y$ for ASD
F_u = specified minimum tensile strength of HSS member material, ksi (MPa)
F_y = specified minimum yield stress of HSS main member material, ksi (MPa)
F_{yb} = specified minimum yield stress of HSS branch member or plate material, ksi (MPa)
H = overall height of rectangular HSS main member, measured in the plane of the connection, in. (mm)
H_b = overall height of rectangular HSS branch member, measured in the plane of the connection, in. (mm)
l_{end} = distance from the near side of the connecting branch or plate to end of chord, in. (mm)
t = design wall thickness of HSS main member, in. (mm)
t_b = design wall thickness of HSS branch member or thickness of plate, in. (mm)

2. Rectangular HSS

2a. Effective Width for Connections to Rectangular HSS

The effective width of elements (plates or rectangular HSS branches) perpendicular to the longitudinal axis of a rectangular HSS member that deliver a force component transverse to the face of the member shall be taken as:

$$B_e = \left(\frac{10t}{B}\right)\left(\frac{F_y t}{F_{yb} t_b}\right) B_b \leq B_b \qquad \text{(K1-1)}$$

K2. CONCENTRATED FORCES ON HSS

1. Definitions of Parameters

l_b = bearing length of the load, measured parallel to the axis of the HSS member (or measured across the width of the HSS in the case of loaded cap plates), in. (mm)

2. Round HSS

The available strength of plate-to-round HSS connections, within the limits in Table K2.1A, shall be taken as shown in Table K2.1.

TABLE K2.1
Available Strengths of Plate-to-Round HSS Connections

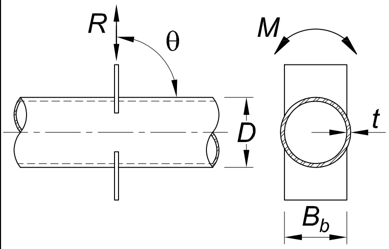
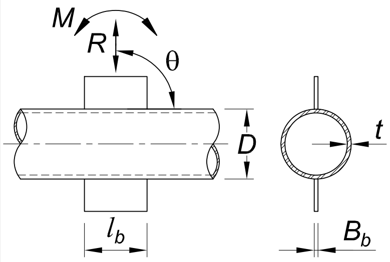
<table>
<tr><th>Connection Type</th><th>Connection Available Strength</th><th colspan="2">Plate Bending</th></tr>
<tr><td rowspan="4">Transverse Plate T- and Cross-Connections

</td><td colspan="3">Limit State: HSS Local Yielding</td></tr>
<tr><td>Plate Axial Load</td><td>In-Plane</td><td>Out-of-Plane</td></tr>
<tr><td>$R_n \sin\theta = F_y t^2 \left(\dfrac{5.5}{1-0.81\dfrac{B_b}{D}}\right) Q_f$
(K2-1a)</td><td>–</td><td>$M_n = 0.5 B_b R_n$
(K2-1b)</td></tr>
<tr><td colspan="3">$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)</td></tr>
<tr><td rowspan="4">Longitudinal Plate T-, Y- and Cross-Connections

</td><td colspan="3">Limit State: HSS Plastification</td></tr>
<tr><td>Plate Axial Load</td><td>In-Plane</td><td>Out-of-Plane</td></tr>
<tr><td>$R_n \sin\theta = 5.5 F_y t^2 \left(1+0.25\dfrac{l_b}{D}\right) Q_f$
(K2-2a)</td><td>$M_n = 0.8 l_b R_n$
(K2-2b)</td><td>–</td></tr>
<tr><td colspan="3">$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)</td></tr>
<tr><td colspan="4">Functions</td></tr>
<tr><td colspan="4">$Q_f = 1$ for HSS (connecting surface) in tension
$= 1.0 - 0.3U(1 + U)$ for HSS (connecting surface) in compression (K2-3)
$U = \left|\dfrac{P_{ro}}{F_c A_g} + \dfrac{M_{ro}}{F_c S}\right|$ (K2-4)
where P_{ro} and M_{ro} are determined on the side of the joint that has the lower compression stress. P_{ro} and M_{ro} refer to required strengths in the HSS: $P_{ro} = P_u$ for LRFD, and P_a for ASD; $M_{ro} = M_u$ for LRFD, and M_a for ASD.</td></tr>
</table>

TABLE K2.1A
Limits of Applicability of Table K2.1

HSS wall slenderness:	$D/t \le 50$ for T-connections under branch plate axial load or bending
	$D/t \le 40$ for cross-connections under branch plate axial load or bending
	$D/t \le 0.11E/F_y$ under branch plate shear loading
	$D/t \le 0.11E/F_y$ for cap plate connections in compression
Width ratio:	$0.2 < B_b/D \le 1.0$ for transverse branch plate connections
Material strength:	$F_y \le 52$ ksi (360 MPa)
Ductility:	$F_y/F_u \le 0.8$ Note: ASTM A500 Grade C is acceptable.
End distance:	$l_{end} \ge D\left(1.25 - \dfrac{B_b/D}{2}\right)$ for transverse and longitudinal branch plate connections under axial load

3. Rectangular HSS

The available strength of connections to rectangular HSS with concentrated loads shall be determined based on the applicable limit states from Chapter J.

K3. HSS-TO-HSS TRUSS CONNECTIONS

HSS-to-HSS truss connections are defined as connections that consist of one or more branch members that are directly welded to a continuous chord that passes through the connection and shall be classified as follows:

(a) When the punching load, $P_r \sin\theta$, in a branch member is equilibrated by beam shear in the chord member, the connection shall be classified as a T-connection when the branch is perpendicular to the chord, and classified as a Y-connection otherwise.

(b) When the punching load, $P_r \sin\theta$, in a branch member is essentially equilibrated (within 20%) by loads in other branch member(s) on the same side of the connection, the connection shall be classified as a K-connection. The relevant gap is between the primary branch members whose loads equilibrate. An N-connection can be considered as a type of K-connection.

User Note: A K-connection with one branch perpendicular to the chord is often called an N-connection.

(c) When the punching load, $P_r \sin\theta$, is transmitted through the chord member and is equilibrated by branch member(s) on the opposite side, the connection shall be classified as a cross-connection.

(d) When a connection has more than two primary branch members, or branch members in more than one plane, the connection shall be classified as a general or multiplanar connection.

When branch members transmit part of their load as K-connections and part of their load as T-, Y- or cross-connections, the adequacy of the connections shall be determined by interpolation on the proportion of the available strength of each in total.

For trusses that are made with HSS that are connected by welding branch members to chord members, eccentricities within the limits of applicability are permitted without consideration of the resulting moments for the design of the connection.

1. Definitions of Parameters

O_v = $l_{ov} / l_p \times 100$, %

e = eccentricity in a truss connection, positive being away from the branches, in. (mm)

g = gap between toes of branch members in a gapped K-connection, neglecting the welds, in. (mm)

l_b = $H_b / \sin\theta$, in. (mm)

l_{ov} = overlap length measured along the connecting face of the chord beneath the two branches, in. (mm)

l_p = projected length of the overlapping branch on the chord, in. (mm)

β = width ratio; the ratio of branch diameter to chord diameter = D_b / D for round HSS; the ratio of overall branch width to chord width = B_b / B for rectangular HSS

β_{eff} = effective width ratio; the sum of the perimeters of the two branch members in a K-connection divided by eight times the chord width

γ = chord slenderness ratio; the ratio of one-half the diameter to the wall thickness = $D / 2t$ for round HSS; the ratio of one-half the width to wall thickness = $B / 2t$ for rectangular HSS

η = load length parameter, applicable only to rectangular HSS; the ratio of the length of contact of the branch with the chord in the plane of the connection to the chord width = l_b / B

θ = acute angle between the branch and chord (degrees)

ζ = gap ratio; the ratio of the gap between the branches of a gapped K-connection to the width of the chord = g / B for rectangular HSS

2. Round HSS

The available strength of round HSS-to-HSS truss connections, within the limits in Table K3.1A, shall be taken as the lowest value obtained according to the limit states shown in Table K3.1.

3. Rectangular HSS

The available strength, ϕP_n and P_n / Ω, of rectangular HSS-to-HSS truss connections within the limits in Table K3.2A, shall be taken as the lowest value obtained according to limit states shown in Table K3.2 and Chapter J.

> **User Note:** Outside the limits in Table K3.2A, the limit states of Chapter J are still applicable and the applicable limit states of Chapter K are not defined.

> **User Note:** Maximum gap size in Table K3.2A will be controlled by the e / H limit. If the gap is large, treat as two Y-connections.

K4. HSS-TO-HSS MOMENT CONNECTIONS

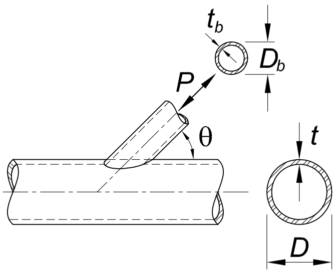
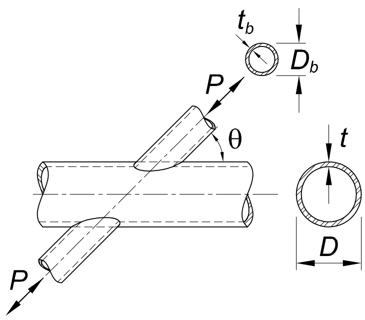
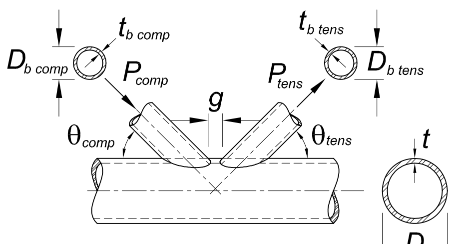
HSS-to-HSS moment connections are defined as connections that consist of one or two branch members that are directly welded to a continuous chord that passes through the connection, with the branch or branches loaded by bending moments.

A connection shall be classified as:

(a) A T-connection when there is one branch and it is perpendicular to the chord and as a Y-connection when there is one branch, but not perpendicular to the chord

(b) A cross-connection when there is a branch on each (opposite) side of the chord

TABLE K3.1
Available Strengths of Round HSS-to-HSS Truss Connections

Connection Type	Connection Available Axial Strength
General Check for T-, Y-, Cross- and K-Connections with gap, when $D_{b\,(tens/comp)} < (D - 2t)$	Limit State: Shear Yielding (punching) $P_n = 0.6F_y t\pi D_b\left(\dfrac{1+\sin\theta}{2\sin^2\theta}\right)$ (K3-1) $\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)
T- and Y-Connections 	Limit State: Chord Plastification $P_n\sin\theta = F_y t^2\left(3.1+15.6\beta^2\right)\gamma^{0.2}Q_f$ (K3-2) $\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)
Cross-Connections 	Limit State: Chord Plastification $P_n\sin\theta = F_y t^2\left(\dfrac{5.7}{1-0.81\beta}\right)Q_f$ (K3-3) $\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)
K-Connections with Gap or Overlap 	Limit State: Chord Plastification $(P_n\sin\theta)_{compression\ branch}$ (K3-4) $= F_y t^2\left(2.0+11.33\dfrac{D_{b\,comp}}{D}\right)Q_g Q_f$ $(P_n\sin\theta)_{tension\ branch}$ $= (P_n\sin\theta)_{compression\ branch}$ (K3-5) $\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)

Functions

$Q_f = 1$ for chord (connecting surface) in tension
$= 1.0 - 0.3U(1+U)$ for HSS (connecting surface) in compression (K2-3)

$$U = \left|\frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S}\right| \tag{K2-4}$$

where P_{ro} and M_{ro} are determined on the side of the joint that has the lower compression stress. P_{ro} and M_{ro} refer to required strengths in the HSS: $P_{ro} = P_u$ for LRFD, and P_a for ASD; $M_{ro} = M_u$ for LRFD, and M_a for ASD.

$$Q_g = \gamma^{0.2}\left[1+\frac{0.024\gamma^{1.2}}{\exp\left(\dfrac{0.5g}{t}-1.33\right)+1}\right] \tag{K3-6}$$

Note that $\exp(x)$ is equal to e^x, where e = 2.71828 is the base of the natural logarithm.

TABLE K3.1A
Limits of Applicability of Table K3.1

Joint eccentricity:	$-0.55 \le e/D \le 0.25$ for K-connections
Chord wall slenderness:	$D/t \le 50$ for T-, Y- and K-connections $D/t \le 40$ for cross-connections
Branch wall slenderness:	$D_b/t_b \le 50$ for tension and compression branch $D_b/t_b \le 0.05E/F_{yb}$ for compression branch
Width ratio:	$0.2 < D_b/D \le 1.0$ for T-, Y-, cross- and overlapped K-connections $0.4 < D_b/D \le 1.0$ for gapped K-connections
Gap:	$g \ge t_{b\,comp} + t_{b\,tens}$ for gapped K-connections
Overlap:	$25\% \le O_v \le 100\%$ for overlapped K-connections
Branch thickness:	$t_{b\,overlapping} \le t_{b\,overlapped}$ for branches in overlapped K-connections
Material strength:	F_y and $F_{yb} \le 52$ ksi (360 MPa)
Ductility:	F_y/F_u and $F_{yb}/F_{ub} \le 0.8$ Note: ASTM A500 Grade C is acceptable.
End distance:	$l_{end} \ge D\left(1.25 - \dfrac{\beta}{2}\right)$ for T-, Y-, cross- and K-connections

TABLE K3.2
Available Strengths of Rectangular HSS-to-HSS Truss Connections

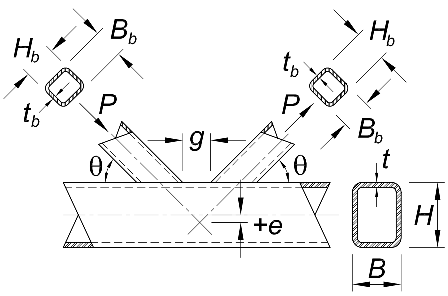
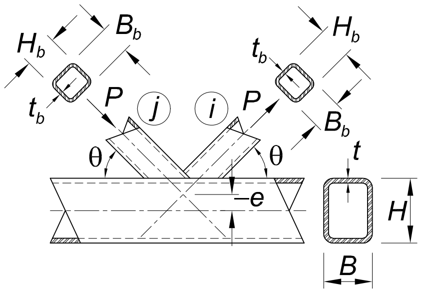
Connection Type	Connection Available Axial Strength
Gapped K-Connections 	Limit State: Chord Wall Plastification, for all β $P_n \sin\theta = F_y t^2 (9.8\beta_{eff}\gamma^{0.5})Q_f$ (K3-7) $\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)
	Limit State: Shear Yielding (punching), when $B_b < B - 2t$ This limit state need not be checked for square branches. $P_n \sin\theta = 0.6F_y tB(2\eta + \beta + \beta_{eop})$ (K3-8) $\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)
	Limit State: Shear of Chord Side Walls in the Gap Region Determine $P_n \sin\theta$ in accordance with Section G4. This limit state need not be checked for square chords.
	Limit State: Local Yielding of Branch/Branches due to Uneven Load Distribution This limit state need not be checked for square branches or where $B/t \ge 15$. $P_n = F_{yb} t_b (2H_b + B_b + B_e - 4t_b)$ (K3-9) $\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)

TABLE K3.2 (continued)
Available Strengths of Rectangular HSS-to-HSS Truss Connections

Connection Type	Connection Available Axial Strength
Overlapped K-Connections  Note that the force arrows shown for overlapped K-connections may be reversed; *i* and *j* control member identification.	Limit State: Local Yielding of Branch/Branches due to Uneven Load Distribution $\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD) When $25\% \le O_v < 50\%$ $P_{n,i} = F_{ybi} t_{bi} \left[\dfrac{O_v}{50} (2H_{bi} - 4t_{bi}) + B_{ei} + B_{ej} \right]$ (K3-10) When $50\% \le O_v < 80\%$ $P_{n,i} = F_{ybi} t_{bi} (2H_{bi} - 4t_{bi} + B_{ei} + B_{ej})$ (K3-11) When $80\% \le O_v \le 100\%$ $P_{n,i} = F_{ybi} t_{bi} (2H_{bi} - 4t_{bi} + B_{bi} + B_{ej})$ (K3-12) Subscript *i* refers to the overlapping branch Subscript *j* refers to the overlapped branch $P_{n,j} = P_{n,i} \left(\dfrac{F_{ybj} A_{bj}}{F_{ybi} A_{bi}} \right)$ (K3-13)

Functions

$Q_f = 1$ for chord (connecting surface) in tension

$$= 1.3 - 0.4 \frac{U}{\beta} \le 1.0 \qquad \text{(K3-14)}$$

for chord (connecting surface) in compression, for T-, Y- and cross-connections

$$= 1.3 - 0.4 \frac{U}{\beta_{eff}} \le 1.0 \qquad \text{(K3-15)}$$

for chord (connecting surface) in compression, for gapped K-connections

$$U = \left| \frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S} \right| \qquad \text{(K2-4)}$$

where P_{ro} and M_{ro} are determined on the side of the joint that has the lower compression stress. P_{ro} and M_{ro} refer to required strengths in the HSS: $P_{ro} = P_u$ for LRFD, and P_a for ASD; $M_{ro} = M_u$ for LRFD, and M_a for ASD.

$$\beta_{eff} = \left[(B_b + H_b)_{compression\ branch} + (B_b + H_b)_{tension\ branch} \right] \Big/ 4B \qquad \text{(K3-16)}$$

$$\beta_{eop} = \frac{5\beta}{\gamma} \le \beta \qquad \text{(K3-17)}$$

TABLE K3.2A
Limits of Applicability of Table K3.2

Joint eccentricity:	$-0.55 \le e/H \le 0.25$ for K-connections	
Chord wall slenderness:	B/t and $H/t \le 35$ for gapped K-connections and T-, Y-and cross-connections	
Branch wall slenderness:	$B/t \le 30$ for overlapped K-connections	
	$H/t \le 35$ for overlapped K-connections	
	B_b/t_b and $H_b/t_b \le 35$ for tension branch	
	$\le 1.25\sqrt{\dfrac{E}{F_{yb}}}$ for compression branch of gapped K-, T-, Y- and cross-connections	
	≤ 35 for compression branch of gapped K-, T-, Y- and cross-connections	
	$\le 1.1\sqrt{\dfrac{E}{F_{yb}}}$ for compression branch of overlapped K-connections	
Width ratio:	B_b/B and $H_b/B \ge 0.25$ for T-, Y- cross- and overlapped K-connections	
Aspect ratio:	$0.5 \le H_b/B_b \le 2.0$ and $0.5 \le H/B \le 2.0$	
Overlap:	$25\% \le O_v \le 100\%$ for overlapped K-connections	
Branch width ratio:	$B_{bi}/B_{bj} \ge 0.75$ for overlapped K-connections, where subscript i refers to the overlapping branch and subscript j refers to the overlapped branch	
Branch thickness ratio:	$t_{bi}/t_{bj} \le 1.0$ for overlapped K-connections, where subscript i refers to the overlapping branch and subscript j refers to the overlapped branch	
Material strength:	F_y and $F_{yb} \le 52$ ksi (360 MPa)	
Ductility:	F_y/F_u and $F_{yb}/F_{ub} \le 0.8$	Note: ASTM A500 Grade C is acceptable.
End distance:	$l_{end} \ge B\sqrt{1-\beta}$ for T- and Y-connections	
Additional Limits for Gapped K-Connections		
Width ratio:	$\dfrac{B_b}{B}$ and $\dfrac{H_b}{B} \ge 0.1+\dfrac{\gamma}{50}$	
	$\beta_{eff} \ge 0.35$	
Gap ratio:	$\zeta = g/B \ge 0.5(1-\beta_{eff})$	
Gap:	$g \ge t_{b\ compression\ branch} + t_{b\ tension\ branch}$	
Branch size:	smaller $B_b \ge 0.63$ (larger B_b), if both branches are square	

1. Definitions of Parameters

Z_b = Plastic section modulus of branch about the axis of bending, in.^{3} (mm^3)

β = width ratio
= D_b/D for round HSS; ratio of branch diameter to chord diameter
= B_b/B for rectangular HSS; ratio of overall branch width to chord width

γ = chord slenderness ratio
= $D/2t$ for round HSS; ratio of one-half the diameter to the wall thickness
= $B/2t$ for rectangular HSS; ratio of one-half the width to the wall thickness

η = load length parameter, applicable only to rectangular HSS
= l_b / B; the ratio of the length of contact of the branch with the chord in the plane of the connection to the chord width, where $l_b = H_b / \sin\theta$

θ = acute angle between the branch and chord (degrees)

2. Round HSS

The available strength of round HSS-to-HSS moment connections within the limits of Table K4.1A shall be taken as the lowest value of the applicable limit states shown in Table K4.1.

3. Rectangular HSS

The available strength, ϕP_n and P_n / Ω, of rectangular HSS-to-HSS moment connections within the limits in Table K4.2A shall be taken as the lowest value obtained according to limit states shown in Table K4.2 and Chapter J.

User Note: Outside the limits in Table K4.2A, the limit states of Chapter J are still applicable and the applicable limit states of Chapter K are not defined.

K5. WELDS OF PLATES AND BRANCHES TO RECTANGULAR HSS

The available strength of branch connections shall be determined considering the nonuniformity of load transfer along the line of weld, due to differences in relative stiffness of HSS walls in HSS-to-HSS connections and between elements in transverse plate-to-HSS connections, as follows:

$$R_n \text{ or } P_n = F_{nw} t_w l_e \qquad \text{(K5-1)}$$

$$M_{n\text{-}ip} = F_{nw} S_{ip} \qquad \text{(K5-2)}$$

$$M_{n\text{-}op} = F_{nw} S_{op} \qquad \text{(K5-3)}$$

Interaction shall be considered.

(a) For fillet welds

$\phi = 0.75$ (LRFD) $\quad \Omega = 2.00$ (ASD)

(b) For partial-joint-penetration groove welds

$\phi = 0.80$ (LRFD) $\quad \Omega = 1.88$ (ASD)

where

F_{nw} = nominal stress of weld metal (Chapter J) with no increase in strength due to directionality of load for fillet welds, ksi (MPa)

S_{ip} = effective elastic section modulus of welds for in-plane bending (Table K5.1), in.^{3} (mm^3)

S_{op} = effective elastic section modulus of welds for out-of-plane bending (Table K5.1), in.^{3} (mm^3)

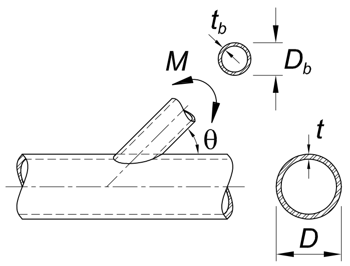
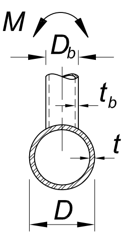
<table>
<tr><th colspan="2">TABLE K4.1
Available Strengths of Round
HSS-to-HSS Moment Connections</th></tr>
<tr><th>Connection Type</th><th>Connection Available Flexural Strength</th></tr>
<tr><td rowspan="2">Branch(es) Under In-Plane Bending
T-, Y- and Cross-Connections
</td><td>Limit State: Chord Plastification
$$M_{n\text{-}ip}\sin\theta = 5.39F_y t^2\gamma^{0.5}\beta D_b Q_f \quad \text{(K4-1)}$$
$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)</td></tr>
<tr><td>Limit State: Shear Yielding (punching),
when $D_b < (D - 2t)$
$$M_{n\text{-}ip} = 0.6F_y t D_b^2\left(\frac{1+3\sin\theta}{4\sin^2\theta}\right) \quad \text{(K4-2)}$$
$\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)</td></tr>
<tr><td rowspan="2">Branch(es) Under Out-of-Plane Bending
T-, Y- and Cross-Connections
</td><td>Limit State: Chord Plastification
$$M_{n\text{-}op} = \frac{F_y t^2 D_b}{\sin\theta}\left(\frac{3.0}{1-0.81\beta}\right)Q_f \quad \text{(K4-3)}$$
$\phi = 0.90$ (LRFD) $\Omega = 1.67$ (ASD)</td></tr>
<tr><td>Limit State: Shear Yielding (punching),
when $D_b < (D - 2t)$
$$M_{n\text{-}op} = 0.6F_y t D_b^2\left(\frac{3+\sin\theta}{4\sin^2\theta}\right) \quad \text{(K4-4)}$$
$\phi = 0.95$ (LRFD) $\Omega = 1.58$ (ASD)</td></tr>
<tr><td colspan="2">For T-, Y- and cross-connections, with branch(es) under combined axial load, in-plane bending, and out-of-plane bending, or any combination of these load effects:
LRFD: $\left[P_u/(\phi P_n)\right] + \left[M_{r\text{-}ip}/(\phi M_{n\text{-}ip})\right]^2 + \left[M_{r\text{-}op}/(\phi M_{n\text{-}op})\right] \le 1.0$ (K4-5)
ASD: $\left[P_a/(P_n/\Omega)\right] + \left[M_{r\text{-}ip}/(M_{n\text{-}ip}/\Omega)\right]^2 + \left[M_{r\text{-}op}/(M_{n\text{-}op}/\Omega)\right] \le 1.0$ (K4-6)
ϕP_n = design strength (or P_n/Ω = allowable strength) obtained from Table K3.1
$\phi M_{n\text{-}ip}$ = design strength (or $M_{n\text{-}ip}/\Omega$ = allowable strength) for in-plane bending
$\phi M_{n\text{-}op}$ = design strength (or $M_{n\text{-}op}/\Omega$ = allowable strength) for out-of-plane bending
$M_{r\text{-}ip}$ = $M_{u\text{-}ip}$ for LRFD; $M_{a\text{-}ip}$ for ASD
$M_{r\text{-}op}$ = $M_{u\text{-}op}$ for LRFD; $M_{a\text{-}op}$ for ASD</td></tr>
</table>

TABLE K4.1 (continued)
Available Strengths of Round HSS-to-HSS Moment Connections

Functions

Q_f = 1 for chord (connecting surface) in tension
$= 1.0 - 0.3U(1 + U)$ for chord (connecting surface) in compression (K2-3)

$$U = \left| \frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S} \right| \quad \text{(K2-4)}$$

where P_{ro} and M_{ro} are determined on the side of the joint that has the lower compression stress. P_{ro} and M_{ro} refer to required strengths in the HSS: $P_{ro} = P_u$ for LRFD, and P_a for ASD; $M_{ro} = M_u$ for LRFD, and M_a for ASD.

TABLE K4.1A
Limits of Applicability of Table K4.1

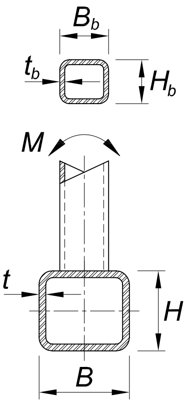
Chord wall slenderness:	D/t	≤ 50 for T- and Y-connections
	D/t	≤ 40 for cross-connections
Branch wall slenderness:	D_b/t_b	≤ 50
	D_b/t_b	$\le 0.05E/F_{yb}$
Width ratio:	0.2	$< D_b/D \le 1.0$
Material strength:	F_y and F_{yb}	≤ 52 ksi (360 MPa)
Ductility:	F_y/F_u and F_{yb}/F_{ub}	≤ 0.8 Note: ASTM A500 Grade C is acceptable.

l_e = total effective weld length of groove and fillet welds to rectangular HSS for weld strength calculations, in. (mm)

t_w = smallest effective weld throat around the perimeter of branch or plate, in. (mm)

When an overlapped K-connection has been designed in accordance with Table K3.2, and the branch member component forces normal to the chord are 80% balanced (i.e., the branch member forces normal to the chord face differ by no more than 20%), the hidden weld under an overlapping branch may be omitted if the remaining welds to the overlapped branch everywhere develop the full capacity of the overlapped branch member walls.

TABLE K4.2
Available Strengths of Rectangular HSS-to-HSS Moment Connections

Connection Type	Connection Available Flexural Strength
Branch(es) under Out-of-Plane Bending T- and Cross-Connections  	Limit state: Chord distortional failure, for T-connections and unbalanced cross-connections $M_n = 2F_y t\left[H_b t + \sqrt{BHt(B+H)}\right]$ (K4-7) $\phi = 1.00$ (LRFD) $\Omega = 1.50$ (ASD)

For T- and cross-connections, with branch(es) under combined axial load, in-plane bending, and out-of-plane bending, or any combination of these load effects:

LRFD: $\left[P_u/(\phi P_n)\right] + \left[M_{r\text{-}ip}/(\phi M_{n\text{-}ip})\right] + \left[M_{r\text{-}op}/(\phi M_{n\text{-}op})\right] \le 1.0$ (K4-8)

ASD: $\left[P_a/(P_n/\Omega)\right] + \left[M_{r\text{-}ip}/(M_{n\text{-}ip}/\Omega)\right] + \left[M_{r\text{-}op}/(M_{n\text{-}op}/\Omega)\right] \le 1.0$ (K4-9)

ϕP_n = design strength (or P_n/Ω = allowable strength)
$\phi M_{n\text{-}ip}$ = design strength (or $M_{n\text{-}ip}/\Omega$ = allowable strength) for in-plane bending
$\phi M_{n\text{-}op}$ = design strength (or $M_{n\text{-}op}/\Omega$ = allowable strength) for out-of-plane bending
$M_{r\text{-}ip}$ = $M_{u\text{-}ip}$ for LRFD; $M_{a\text{-}ip}$ for ASD
$M_{r\text{-}op}$ = $M_{u\text{-}op}$ for LRFD; $M_{a\text{-}op}$ for ASD

Functions

Q_f = 1 for chord (connecting surface) in tension

$= 1.3 - 0.4\dfrac{U}{\beta} \le 1.0$ for chord (connecting surface) in compression (K3-14)

$$U = \left|\frac{P_{ro}}{F_c A_g} + \frac{M_{ro}}{F_c S}\right| \quad \text{(K2-4)}$$

where P_{ro} and M_{ro} are determined on the side of the joint that has the lower compression stress. P_{ro} and M_{ro} refer to required strengths in the HSS: $P_{ro} = P_u$ for LRFD, and P_a for ASD; $M_{ro} = M_u$ for LRFD, and M_a for ASD.

TABLE K4.2A
Limits of Applicability of Table K4.2

Branch angle:	$\theta \cong 90°$
Chord wall slenderness:	B/t and $H/t \le 35$
Branch wall slenderness:	B_b/t_b and $H_b/t_b \le 35$
	$\le 1.25\sqrt{\dfrac{E}{F_{yb}}}$
Width ratio:	$B_b/B \ge 0.25$
Aspect ratio:	$0.5 \le H_b/B_b \le 2.0$ and $0.5 \le H/B \le 2.0$
Material strength:	F_y and $F_{yb} \le 52$ ksi (360 MPa)
Ductility:	F_y/F_u and $F_{yb}/F_{ub} \le 0.8$ Note: ASTM A500 Grade C is acceptable.

The weld checks in Table K5.1 are not required if the welds are capable of developing the full strength of the branch member wall along its entire perimeter (or a plate along its entire length).

User Note: The approach used here to allow downsizing of welds assumes a constant weld size around the full perimeter of the HSS branch. Special attention is required for equal width (or near-equal width) connections which combine partial-joint-penetration groove welds along the matched edges of the connection, with fillet welds generally across the main member face.

TABLE K5.1
Effective Weld Properties for Connections to Rectangular HSS

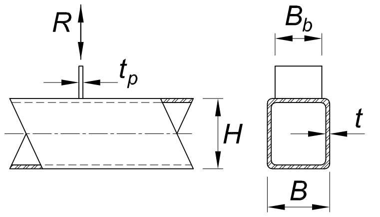
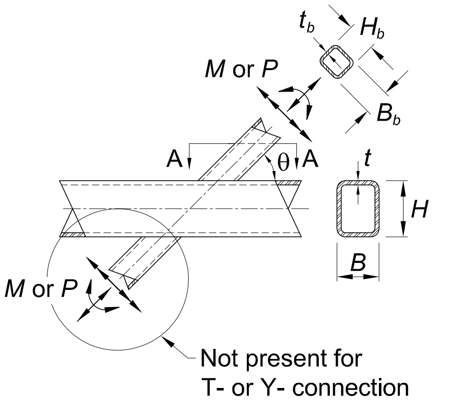
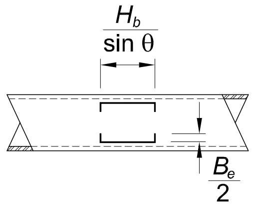
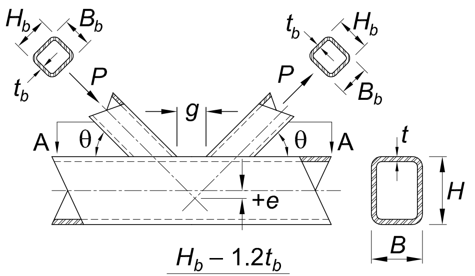
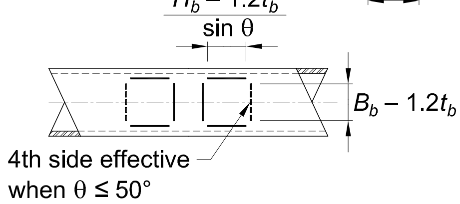
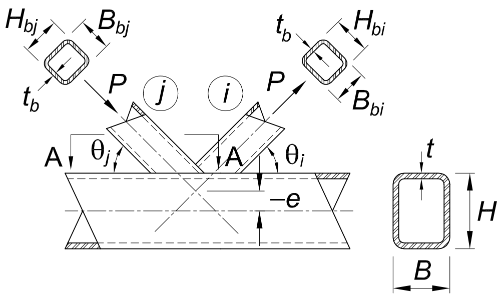
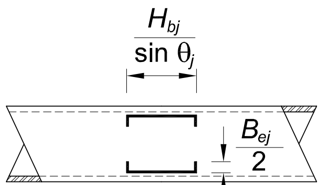
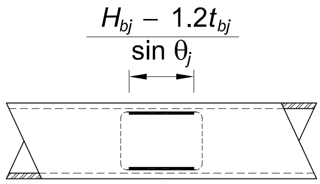
Connection Type	Weld Properties
Transverse Plate T- and Cross-Connections under Plate Axial Load 	Effective Weld Properties $l_e = 2B_e$ (K5-4) where l_e = total effective weld length for welds on both sides of the transverse plate
T-, Y- and Cross-Connections under Branch Axial Load or Bending Section A-A: Effective weld	Effective Weld Properties $l_e = \frac{2H_b}{\sin\theta} + 2B_e$ (K5-5) $S_{ip} = \frac{t_w}{3}\left(\frac{H_b}{\sin\theta}\right)^2 + t_w B_e\left(\frac{H_b}{\sin\theta}\right)$ (K5-6) $S_{op} = t_w\left(\frac{H_b}{\sin\theta}\right)B_b + \frac{t_w}{3}\left(B_b^2\right) - \frac{(t_w/3)(B_b - B_e)^3}{B_b}$ (K5-7) When $\beta > 0.85$ or $\theta > 50°$, $B_e/2$ shall not exceed $B_b/4$.
Gapped K-Connections under Branch Axial Load Section A-A: Effective weld for θ ≥ 60°	Effective Weld Properties When $\theta \le 50°$: $l_e = \frac{2(H_b - 1.2t_b)}{\sin\theta} + 2(B_b - 1.2t_b)$ (K5-8) When $\theta \ge 60°$: $l_e = \frac{2(H_b - 1.2t_b)}{\sin\theta} + B_b - 1.2t_b$ (K5-9) When $50° < \theta < 60°$, linear interpolation shall be used to determine l_e.

TABLE K5.1 (continued)
Effective Weld Properties for Connections to Rectangular HSS

Connection Type	Weld Properties
Overlapped K-Connections Under Branch Axial Load  Note that the force arrows shown for overlapped K-connections may be reversed; i and j control member identification.  Section A-A: Effective weld when $\frac{B_{bj}}{B} \le 0.85$ and $\theta_j \le 50°$  Section A-A: Effective weld when $\frac{B_{bj}}{B} > 0.85$ or $\theta_j > 50°$	Overlapping Member Effective Weld Properties (all dimensions are for the overlapping branch, i) When $25\% \le O_v < 50\%$: $l_{e,i} = \frac{2O_v}{50}\left[\left(1-\frac{O_v}{100}\right)\left(\frac{H_{bi}}{\sin\theta_i}\right)+\frac{O_v}{100}\left(\frac{H_{bi}}{\sin(\theta_i+\theta_j)}\right)\right]+B_{ei}+B_{ej}$ (K5-10) When $50\% \le O_v < 80\%$: $l_{e,i} = 2\left[\left(1-\frac{O_v}{100}\right)\left(\frac{H_{bi}}{\sin\theta_i}\right)+\frac{O_v}{100}\left(\frac{H_{bi}}{\sin(\theta_i+\theta_j)}\right)\right]+B_{ei}+B_{ej}$ (K5-11) When $80\% \le O_v \le 100\%$: $l_{e,i} = 2\left[\left(1-\frac{O_v}{100}\right)\left(\frac{H_{bi}}{\sin\theta_i}\right)+\frac{O_v}{100}\left(\frac{H_{bi}}{\sin(\theta_i+\theta_j)}\right)\right]+B_{bi}+B_{ej}$ (K5-12) When $B_{bi}/B > 0.85$ or $\theta_i > 50°$, $B_{ei}/2$ shall not exceed $B_{bi}/4$ and when $B_{bi}/B_{bj} > 0.85$ or $(180 - \theta_i - \theta_j) > 50°$, $B_{ej}/2$ shall not exceed $B_{bi}/4$. Subscript i refers to the overlapping branch Subscript j refers to the overlapped branch
	Overlapped Member Effective Weld Properties (all dimensions are for the overlapped branch, j) $l_{e,j} = \frac{2H_{bj}}{\sin\theta_j} + 2B_{ej}$ (K5-13) When $B_{bj}/B > 0.85$ or $\theta_j > 50°$, $l_{e,j} = 2(H_{bj} - 1.2t_{bj})/\sin\theta_j$ (K5-14)

CHAPTER L

DESIGN FOR SERVICEABILITY

This chapter addresses the evaluation of the structure and its components for the serviceability limit states of deflections, drift, vibration, wind-induced motion, thermal distortion, and connection slip.

The chapter is organized as follows:

L1. General Provisions
L2. Deflections
L3. Drift
L4. Vibration
L5. Wind-Induced Motion
L6. Thermal Expansion and Contraction
L7. Connection Slip

L1. GENERAL PROVISIONS

Serviceability is a state in which the function of a building, its appearance, maintainability, durability, and the comfort of its occupants are preserved under typical usage. Limiting values of structural behavior for serviceability (such as maximum deflections and accelerations) shall be chosen with due regard to the intended function of the structure. Serviceability shall be evaluated using applicable load combinations.

> **User Note**: Serviceability limit states, service loads, and appropriate load combinations for serviceability considerations can be found in *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE/SEI 7) Appendix C and its commentary. The performance requirements for serviceability in this chapter are consistent with ASCE/SEI 7 Appendix C. Service loads are those that act on the structure at an arbitrary point in time and are not usually taken as the nominal loads.
>
> Reduced stiffness values used in the direct analysis method, described in Chapter C, are not intended for use with the provisions of this chapter.

L2. DEFLECTIONS

Deflections in structural members and structural systems shall be limited so as not to impair the serviceability of the structure.

L3. DRIFT

Drift shall be limited so as not to impair the serviceability of the structure.

L4. VIBRATION

The effect of vibration on the comfort of the occupants and the function of the structure shall be considered. The sources of vibration to be considered include occupant loading, vibrating machinery and others identified for the structure.

L5. WIND-INDUCED MOTION

The effect of wind-induced motion of buildings on the comfort of occupants shall be considered.

L6. THERMAL EXPANSION AND CONTRACTION

The effects of thermal expansion and contraction of a building shall be considered.

L7. CONNECTION SLIP

The effects of connection slip shall be included in the design where slip at bolted connections may cause deformations that impair the serviceability of the structure. Where appropriate, the connection shall be designed to preclude slip.

User Note: For the design of slip-critical connections, see Sections J3.8 and J3.9. For more information on connection slip, refer to the RCSC *Specification for Structural Joints Using High-Strength Bolts*.

CHAPTER M

FABRICATION AND ERECTION

This chapter addresses requirements for shop drawings, fabrication, shop painting and erection.

The chapter is organized as follows:

M1. Shop and Erection Drawings
M2. Fabrication
M3. Shop Painting
M4. Erection

M1. SHOP AND ERECTION DRAWINGS

Shop and erection drawings are permitted to be prepared in stages. Shop drawings shall be prepared in advance of fabrication and give complete information necessary for the fabrication of the component parts of the structure, including the location, type and size of welds and bolts. Erection drawings shall be prepared in advance of erection and give information necessary for erection of the structure. Shop and erection drawings shall clearly distinguish between shop and field welds and bolts and shall clearly identify pretensioned and slip-critical high-strength bolted connections. Shop and erection drawings shall be made with due regard to speed and economy in fabrication and erection.

M2. FABRICATION

1. Cambering, Curving and Straightening

Local application of heat or mechanical means is permitted to be used to introduce or correct camber, curvature and straightness. The temperature of heated areas shall not exceed 1,100°F (590°C) for ASTM A514/A514M and ASTM A852/A852M steels nor 1,200°F (650°C) for other steels.

2. Thermal Cutting

Thermally cut edges shall meet the requirements of *Structural Welding Code—Steel* (AWS D1.1/D1.1M) clauses 5.14.5.2, 5.14.8.3 and 5.14.8.4, hereafter referred to as AWS D1.1M/D1.1M, with the exception that thermally cut free edges that will not be subject to fatigue shall be free of round-bottom gouges greater than $^3/_{16}$ in. (5 mm) deep and sharp V-shaped notches. Gouges deeper than $^3/_{16}$ in. (5 mm) and notches shall be removed by grinding or repaired by welding.

Reentrant corners shall be formed with a curved transition. The radius need not exceed that required to fit the connection. Discontinuous corners are permitted where the material on both sides of the discontinuous reentrant corner are connected to a mating piece to prevent deformation and associated stress concentration at the corner.

User Note: Reentrant corners with a radius of $^1/_2$ to $^3/_8$ in. (13 to 10 mm) are acceptable for statically loaded work. Where pieces need to fit tightly together, a discontinuous reentrant corner is acceptable if the pieces are connected close to the corner on both sides of the discontinuous corner. Slots in HSS for gussets may be made with semicircular ends or with curved corners. Square ends are acceptable provided the edge of the gusset is welded to the HSS.

Weld access holes shall meet the geometrical requirements of Section J1.6. Beam copes and welds access holes in shapes that are to be galvanized shall be ground to bright metal. For shapes with a flange thickness not exceeding 2 in. (50 mm), the roughness of thermally cut surfaces of copes shall be no greater than a surface roughness value of 2,000 μin. (50 μm) as defined in *Surface Texture, Surface Roughness, Waviness, and Lay* (ASME B46.1), hereafter referred to as ASTM B46.1. For beam copes and weld access holes in which the curved part of the access hole is thermally cut in ASTM A6/A6M hot-rolled shapes with a flange thickness exceeding 2 in. (50 mm) and welded built-up shapes with material thickness greater than 2 in. (50 mm), a preheat temperature of not less than 150°F (66°C) shall be applied prior to thermal cutting. The thermally cut surface of access holes in ASTM A6/A6M hot-rolled shapes with a flange thickness exceeding 2 in. (50 mm) and built-up shapes with a material thickness greater than 2 in. (50 mm) shall be ground.

User Note: The AWS *Surface Roughness Guide for Oxygen Cutting* (AWS C4.1-77) sample 2 may be used as a guide for evaluating the surface roughness of copes in shapes with flanges not exceeding 2 in. (50 mm) thick.

3. Planing of Edges

Planing or finishing of sheared or thermally cut edges of plates or shapes is not required unless specifically called for in the construction documents or included in a stipulated edge preparation for welding.

4. Welded Construction

Welding shall be performed in accordance with AWS D1.1/D1.1M, except as modified in Section J2.

User Note: Welder qualification tests on plate defined in AWS D1.1/D1.1M clause 4 are appropriate for welds connecting plates, shapes or HSS to other plates, shapes or rectangular HSS. The 6GR tubular welder qualification is required for unbacked complete-joint-penetration groove welds of HSS T-, Y- and K-connections.

5. Bolted Construction

Parts of bolted members shall be pinned or bolted and rigidly held together during assembly. Use of a drift pin in bolt holes during assembly shall not distort the metal or enlarge the holes. Poor matching of holes shall be cause for rejection.

Bolt holes shall comply with the provisions of the RCSC *Specification for Structural Joints Using High-Strength Bolts* Section 3.3, hereafter referred to as the RCSC *Specification*, except that thermally cut holes are permitted with a surface roughness profile not exceeding 1,000 μin. (25 μm), as defined in ASME B46.1. Gouges shall not exceed a depth of $^1/_{16}$ in. (2 mm). Water jet cut holes are also permitted.

User Note: The AWS *Surface Roughness Guide for Oxygen Cutting* (AWS C4.1-77) sample 3 may be used as a guide for evaluating the surface roughness of thermally cut holes.

Fully inserted finger shims, with a total thickness of not more than $^1/_4$ in. (6 mm) within a joint, are permitted without changing the strength (based upon hole type) for the design of connections. The orientation of such shims is independent of the direction of application of the load.

The use of high-strength bolts shall conform to the requirements of the RCSC *Specification*, except as modified in Section J3.

6. Compression Joints

Compression joints that depend on contact bearing as part of the splice strength shall have the bearing surfaces of individual fabricated pieces prepared by milling, sawing or other equivalent means.

7. Dimensional Tolerances

Dimensional tolerances shall be in accordance with Chapter 6 of the AISC *Code of Standard Practice for Steel Buildings and Bridges*, hereafter referred to as the *Code of Standard Practice*.

8. Finish of Column Bases

Column bases and base plates shall be finished in accordance with the following requirements:

(a) Steel bearing plates 2 in. (50 mm) or less in thickness are permitted without milling provided a smooth and notch-free contact bearing surface is obtained. Steel bearing plates over 2 in. (50 mm) but not over 4 in. (100 mm) in thickness are permitted to be straightened by pressing or, if presses are not available, by milling for bearing surfaces, except as stipulated in subparagraphs (b) and (c) of this section, to obtain a smooth and notch-free contact bearing surface. Steel bearing plates over 4 in. (100 mm) in thickness shall be milled for bearing surfaces, except as stipulated in subparagraphs (b) and (c) of this section.

(b) Bottom surfaces of bearing plates and column bases that are grouted to ensure full bearing contact on foundations need not be milled.

(c) Top surfaces of bearing plates need not be milled when complete-joint-penetration groove welds are provided between the column and the bearing plate.

9. Holes for Anchor Rods

Holes for anchor rods are permitted to be thermally cut in accordance with the provisions of Section M2.2.

10. Drain Holes

When water can collect inside HSS or box members, either during construction or during service, the member shall be sealed, provided with a drain hole at the base, or otherwise protected from water infiltration.

11. Requirements for Galvanized Members

Members and parts to be galvanized shall be designed, detailed and fabricated to provide for flow and drainage of pickling fluids and zinc and to prevent pressure buildup in enclosed parts.

> **User Note:** See *The Design of Products to be Hot-Dip Galvanized After Fabrication*, American Galvanizer's Association, and ASTM A123, F2329, A384 and A780 for useful information on design and detailing of galvanized members. See Section M2.2 for requirements for copes of members that are to be galvanized.

M3. SHOP PAINTING

1. General Requirements

Shop painting and surface preparation shall be in accordance with the provisions in *Code of Standard Practice* Chapter 6.

Shop paint is not required unless specified by the contract documents.

2. Inaccessible Surfaces

Except for contact surfaces, surfaces inaccessible after shop assembly shall be cleaned and painted prior to assembly, if required by the construction documents.

3. Contact Surfaces

Paint is permitted in bearing-type connections. For slip-critical connections, the faying surface requirements shall be in accordance with RCSC *Specification* Section 3.2.2.

4. Finished Surfaces

Machine-finished surfaces shall be protected against corrosion by a rust inhibitive coating that can be removed prior to erection or has characteristics that make removal prior to erection unnecessary.

5. Surfaces Adjacent to Field Welds

Unless otherwise specified in the design documents, surfaces within 2 in. (50 mm) of any field weld location shall be free of materials that would prevent weld quality from meeting the quality requirements of this Specification, or produce unsafe fumes during welding.

M4. ERECTION

1. Column Base Setting

Column bases shall be set level and to correct elevation with full bearing on concrete or masonry as defined in *Code of Standard Practice* Chapter 7.

2. Stability and Connections

The frame of structural steel buildings shall be carried up true and plumb within the limits defined in *Code of Standard Practice* Chapter 7. As erection progresses, the structure shall be secured to support dead, erection and other loads anticipated to occur during the period of erection. Temporary bracing shall be provided, in accordance with the requirements of the *Code of Standard Practice*, wherever necessary to support the loads to which the structure may be subjected, including equipment and the operation of same. Such bracing shall be left in place as long as required for safety.

3. Alignment

No permanent bolting or welding shall be performed until the affected portions of the structure have been aligned as required by the construction documents.

4. Fit of Column Compression Joints and Base Plates

Lack of contact bearing not exceeding a gap of $^1/_{16}$ in. (2 mm), regardless of the type of splice used (partial-joint-penetration groove welded or bolted), is permitted. If the gap exceeds $^1/_{16}$ in. (2 mm), but is equal to or less than $^1/_4$ in. (6 mm), and if an engineering investigation shows that sufficient contact area does not exist, the gap shall be packed out with nontapered steel shims. Shims need not be other than mild steel, regardless of the grade of the main material.

5. Field Welding

Surfaces in and adjacent to joints to be field welded shall be prepared as necessary to assure weld quality. This preparation shall include surface preparation necessary to correct for damage or contamination occurring subsequent to fabrication.

6. Field Painting

Responsibility for touch-up painting, cleaning, and field painting shall be allocated in accordance with accepted local practices, and this allocation shall be set forth explicitly in the contract documents.

CHAPTER N

QUALITY CONTROL AND QUALITY ASSURANCE

This chapter addresses minimum requirements for quality control, quality assurance and nondestructive testing for structural steel systems and steel elements of composite members for buildings and other structures.

User Note: This chapter does not address quality control or quality assurance for the following items:
(a) Steel (open web) joists and girders
(b) Tanks or pressure vessels
(c) Cables, cold-formed steel products, or gage material
(d) Concrete reinforcing bars, concrete materials, or placement of concrete for composite members
(e) Surface preparations or coatings

The Chapter is organized as follows:

N1. General Provisions
N2. Fabricator and Erector Quality Control Program
N3. Fabricator and Erector Documents
N4. Inspection and Nondestructive Testing Personnel
N5. Minimum Requirements for Inspection of Structural Steel Buildings
N6. Approved Fabricators and Erectors
N7. Nonconforming Material and Workmanship

N1. GENERAL PROVISIONS

Quality control (QC) as specified in this chapter shall be provided by the fabricator and erector. Quality assurance (QA) as specified in this chapter shall be provided by others when required by the authority having jurisdiction (AHJ), applicable building code, purchaser, owner, or engineer of record (EOR). Nondestructive testing (NDT) shall be performed by the agency or firm responsible for quality assurance, except as permitted in accordance with Section N6.

User Note: The QA/QC requirements in Chapter N are considered adequate and effective for most steel structures and are strongly encouraged without modification. When the applicable building code and AHJ requires the use of a QA plan, this chapter outlines the minimum requirements deemed effective to provide satisfactory results in steel building construction. There may be cases where supplemental inspections are advisable. Additionally, where the contractor's QC program has demonstrated the capability to perform some tasks this plan has assigned to QA, modification of the plan could be considered.

> **User Note:** The producers of materials manufactured in accordance with the standard specifications referenced in Section A3 and steel deck manufacturers are not considered to be fabricators or erectors.

N2. FABRICATOR AND ERECTOR QUALITY CONTROL PROGRAM

The fabricator and erector shall establish, maintain and implement QC procedures to ensure that their work is performed in accordance with this Specification and the construction documents.

1. Material Identification

Material identification procedures shall comply with the requirements of Section 6.1 of the AISC *Code of Standard Practice for Steel Buildings and Bridges*, hereafter referred to as the *Code of Standard Practice*, and shall be monitored by the fabricator's quality control inspector (QCI).

2. Fabricator Quality Control Procedures

The fabricator's QC procedures shall address inspection of the following as a minimum, as applicable:

(a) Shop welding, high-strength bolting, and details in accordance with Section N5

(b) Shop cut and finished surfaces in accordance with Section M2

(c) Shop heating for cambering, curving and straightening in accordance with Section M2.1

(d) Tolerances for shop fabrication in accordance with *Code of Standard Practice* Section 6.4

3. Erector Quality Control Procedures

The erector's quality control procedures shall address inspection of the following as a minimum, as applicable:

(a) Field welding, high-strength bolting, and details in accordance with Section N5

(b) Steel deck in accordance with SDI *Standard for Quality Control and Quality Assurance for Installation of Steel Deck*

(c) Headed steel stud anchor placement and attachment in accordance with Section N5.4

(d) Field cut surfaces in accordance with Section M2.2

(e) Field heating for straightening in accordance with Section M2.1

(f) Tolerances for field erection in accordance with *Code of Standard Practice* Section 7.13

N3. FABRICATOR AND ERECTOR DOCUMENTS

1. Submittals for Steel Construction

The fabricator or erector shall submit the following documents for review by the EOR or the EOR's designee, in accordance with *Code of Standard Practice* Section 4.4, prior to fabrication or erection, as applicable:

(a) Shop drawings, unless shop drawings have been furnished by others

(b) Erection drawings, unless erection drawings have been furnished by others

2. Available Documents for Steel Construction

The following documents shall be available in electronic or printed form for review by the EOR or the EOR's designee prior to fabrication or erection, as applicable, unless otherwise required in the construction documents to be submitted:

(a) For main structural steel elements, copies of material test reports in accordance with Section A3.1.

(b) For steel castings and forgings, copies of material test reports in accordance with Section A3.2.

(c) For fasteners, copies of manufacturer's certifications in accordance with Section A3.3.

(d) For anchor rods and threaded rods, copies of material test reports in accordance with Section A3.4.

(e) For welding consumables, copies of manufacturer's certifications in accordance with Section A3.5.

(f) For headed stud anchors, copies of manufacturer's certifications in accordance with Section A3.6.

(g) Manufacturer's product data sheets or catalog data for welding filler metals and fluxes to be used. The data sheets shall describe the product, limitations of use, recommended or typical welding parameters, and storage and exposure requirements, including baking, if applicable.

(h) Welding procedure specifications (WPS).

(i) Procedure qualification records (PQR) for WPS that are not prequalified in accordance with *Structural Welding Code—Steel* (AWS D1.1/D1.1M), hereafter referred to as AWS D1.1/D1.1M, or *Structural Welding Code—Sheet Steel* (AWS D1.3/D1.3M), as applicable.

(j) Welding personnel performance qualification records (WPQR) and continuity records.

(k) Fabricator's or erector's, as applicable, written QC manual that shall include, as a minimum:

    (1) Material control procedures

    (2) Inspection procedures

    (3) Nonconformance procedures

(l) Fabricator's or erector's, as applicable, QCI qualifications.

(m) Fabricator NDT personnel qualifications, if NDT is performed by the fabricator.

N4. INSPECTION AND NONDESTRUCTIVE TESTING PERSONNEL

1. Quality Control Inspector Qualifications

QC welding inspection personnel shall be qualified to the satisfaction of the fabricator's or erector's QC program, as applicable, and in accordance with either of the following:

(a) Associate welding inspectors (AWI) or higher as defined in *Standard for the Qualification of Welding Inspectors* (AWS B5.1), or

(b) Qualified under the provisions of AWS D1.1/D1.1M clause 6.1.4.

QC bolting inspection personnel shall be qualified on the basis of documented training and experience in structural bolting inspection.

2. Quality Assurance Inspector Qualifications

QA welding inspectors shall be qualified to the satisfaction of the QA agency's written practice, and in accordance with either of the following:

(a) Welding inspectors (WI) or senior welding inspectors (SWI), as defined in *Standard for the Qualification of Welding Inspectors* (AWS B5.1), except AWI are permitted to be used under the direct supervision of WI, who are on the premises and available when weld inspection is being conducted, or

(b) Qualified under the provisions of AWS D1.1/D1.1M clause 6.1.4.

QA bolting inspection personnel shall be qualified on the basis of documented training and experience in structural bolting inspection.

3. NDT Personnel Qualifications

NDT personnel, for NDT other than visual, shall be qualified in accordance with their employer's written practice, which shall meet or exceed the criteria of AWS D1.1/D1.1M clause 6.14.6, and,

(a) *Personnel Qualification and Certification Nondestructive Testing* (ASNT SNT-TC-1A), or

(b) *Standard for the Qualification and Certification of Nondestructive Testing Personnel* (ANSI/ASNT CP-189).

N5. MINIMUM REQUIREMENTS FOR INSPECTION OF STRUCTURAL STEEL BUILDINGS

1. Quality Control

QC inspection tasks shall be performed by the fabricator's or erector's QCI, as applicable, in accordance with Sections N5.4, N5.6 and N5.7.

Tasks in Tables N5.4-1 through N5.4-3 and Tables N5.6-1 through N5.6-3 listed for QC are those inspections performed by the QCI to ensure that the work is performed in accordance with the construction documents.

For QC inspection, the applicable construction documents are the shop drawings and the erection drawings, and the applicable referenced specifications, codes and standards.

User Note: The QCI need not refer to the design drawings and project specifications. The *Code of Standard Practice* Section 4.2.1(a) requires the transfer of information from the contract documents (design drawings and project specification) into accurate and complete shop and erection drawings, allowing QC inspection to be based upon shop and erection drawings alone.

2. Quality Assurance

The QAI shall review the material test reports and certifications as listed in Section N3.2 for compliance with the construction documents.

QA inspection tasks shall be performed by the QAI, in accordance with Sections N5.4, N5.6 and N5.7.

Tasks in Tables N5.4-1 through N5.4-3 and N5.6-1 through N5.6-3 listed for QA are those inspections performed by the QAI to ensure that the work is performed in accordance with the construction documents.

Concurrent with the submittal of such reports to the AHJ, EOR or owner, the QA agency shall submit to the fabricator and erector:

(a) Inspection reports

(b) NDT reports

3. Coordinated Inspection

When a task is noted to be performed by both QC and QA, it is permitted to coordinate the inspection function between the QCI and QAI so that the inspection functions are performed by only one party. When QA relies upon inspection functions performed by QC, the approval of the EOR and the AHJ is required.

4. Inspection of Welding

Observation of welding operations and visual inspection of in-process and completed welds shall be the primary method to confirm that the materials, procedures and workmanship are in conformance with the construction documents.

User Note: The technique, workmanship, appearance and quality of welded construction are addressed in Section M2.4.

As a minimum, welding inspection tasks shall be in accordance with Tables N5.4-1, N5.4-2 and N5.4-3. In these tables, the inspection tasks are as follows:

(a) Observe (O): The inspector shall observe these items on a random basis. Operations need not be delayed pending these inspections.

(b) Perform (P): These tasks shall be performed for each welded joint or member.

TABLE N5.4-1
Inspection Tasks Prior to Welding

Inspection Tasks Prior to Welding	QC	QA
Welder qualification records and continuity records	P	O
WPS available	P	P
Manufacturer certifications for welding consumables available	P	P
Material identification (type/grade)	O	O
Welder identification system[a]	O	O
Fit-up of groove welds (including joint geometry) • Joint preparations • Dimensions (alignment, root opening, root face, bevel) • Cleanliness (condition of steel surfaces) • Tacking (tack weld quality and location) • Backing type and fit (if applicable)	O	O
Fit-up of CJP groove welds of HSS T-, Y- and K-joints without backing (including joint geometry) • Joint preparations • Dimensions (alignment, root opening, root face, bevel) • Cleanliness (condition of steel surfaces) • Tacking (tack weld quality and location)	P	O
Configuration and finish of access holes	O	O
Fit-up of fillet welds • Dimensions (alignment, gaps at root) • Cleanliness (condition of steel surfaces) • Tacking (tack weld quality and location)	O	O
Check welding equipment	O	–

[a] The fabricator or erector, as applicable, shall maintain a system by which a welder who has welded a joint or member can be identified. Stamps, if used, shall be the low-stress type.

TABLE N5.4-2
Inspection Tasks During Welding

Inspection Tasks During Welding	QC	QA
Control and handling of welding consumables • Packaging • Exposure control	O	O
No welding over cracked tack welds	O	O
Environmental conditions • Wind speed within limits • Precipitation and temperature	O	O
WPS followed • Settings on welding equipment • Travel speed • Selected welding materials • Shielding gas type/flow rate • Preheat applied • Interpass temperature maintained (min./max.) • Proper position (F, V, H, OH)	O	O
Welding techniques • Interpass and final cleaning • Each pass within profile limitations • Each pass meets quality requirements	O	O
Placement and installation of steel headed stud anchors	P	P

TABLE N5.4-3
Inspection Tasks After Welding

Inspection Tasks After Welding	QC	QA
Welds cleaned	O	O
Size, length and location of welds	P	P
Welds meet visual acceptance criteria • Crack prohibition • Weld/base-metal fusion • Crater cross section • Weld profiles • Weld size • Undercut • Porosity	P	P
Arc strikes	P	P
k-area[a]	P	P
Weld access holes in rolled heavy shapes and built-up heavy shapes[b]	P	P
Backing removed and weld tabs removed (if required)	P	P
Repair activities	P	P
Document acceptance or rejection of welded joint or member	P	P
No prohibited welds have been added without the approval of the EOR	O	O

[a] When welding of doubler plates, continuity plates or stiffeners has been performed in the *k*-area, visually inspect the web *k*-area for cracks within 3 in. (75 mm) of the weld.

[b] After rolled heavy shapes (see Section A3.1c) and built-up heavy shapes (see Section A3.1d) are welded, visually inspect the weld access hole for cracks.

5. Nondestructive Testing of Welded Joints

5a. Procedures

Ultrasonic testing (UT), magnetic particle testing (MT), penetrant testing (PT), and radiographic testing (RT), where required, shall be performed by QA in accordance with AWS D1.1/D1.1M.

> **User Note:** The technique, workmanship, appearance and quality of welded construction is addressed in Section M2.4.

5b. CJP Groove Weld NDT

For structures in risk category III or IV, UT shall be performed by QA on all complete-joint-penetration (CJP) groove welds subject to transversely applied tension loading in butt, T- and corner joints, in material $^5/_{16}$ in. (8 mm) thick or greater. For structures in risk category II, UT shall be performed by QA on 10% of CJP groove welds in butt, T- and corner joints subject to transversely applied tension loading, in materials $^5/_{16}$ in. (8 mm) thick or greater.

> **User Note**: For structures in risk category I, NDT of CJP groove welds is not required. For all structures in all risk categories, NDT of CJP groove welds in materials less than $^5/_{16}$ in. (8 mm) thick is not required.

5c. Welded Joints Subjected to Fatigue

When required by Appendix 3, Table A-3.1, welded joints requiring weld soundness to be established by radiographic or ultrasonic inspection shall be tested by QA as prescribed. Reduction in the rate of UT is prohibited.

5d. Ultrasonic Testing Rejection Rate

The ultrasonic testing rejection rate shall be determined as the number of welds containing defects divided by the number of welds completed. Welds that contain acceptable discontinuities shall not be considered as having defects when the rejection rate is determined. For evaluating the rejection rate of continuous welds over 3 ft (1 m) in length where the effective throat is 1 in. (25 mm) or less, each 12 in. (300 mm) increment or fraction thereof shall be considered as one weld. For evaluating the rejection rate on continuous welds over 3 ft (1 m) in length where the effective throat is greater than 1 in. (25 mm), each 6 in. (150 mm) of length, or fraction thereof, shall be considered one weld.

5e. Reduction of Ultrasonic Testing Rate

For projects that contain 40 or fewer welds, there shall be no reduction in the ultrasonic testing rate. The rate of UT is permitted to be reduced if approved by the EOR and the AHJ. Where the initial rate of UT is 100%, the NDT rate for an individual welder or welding operator is permitted to be reduced to 25%, provided the rejection rate, the number of welds containing unacceptable defects divided by the number of welds completed, is demonstrated to be 5% or less of the welds tested for the welder or welding operator. A sampling of at least 40 completed welds shall be made for such reduced evaluation on each project.

5f. Increase in Ultrasonic Testing Rate

For structures in risk category II and higher (where the initial rate for UT is 10%) the NDT rate for an individual welder or welding operator shall be increased to 100% should the rejection rate (the number of welds containing unacceptable defects divided by the number of welds completed) exceed 5% of the welds tested for the welder or welding operator. A sampling of at least 20 completed welds on each project shall be made prior to implementing such an increase. If the rejection rate for the welder or welding operator falls to 5% or less on the basis of at least 40 completed welds, the rate of UT may be decreased to 10%.

5g. Documentation

All NDT performed shall be documented. For shop fabrication, the NDT report shall identify the tested weld by piece mark and location in the piece. For field work, the NDT report shall identify the tested weld by location in the structure, piece mark, and location in the piece.

When a weld is rejected on the basis of NDT, the NDT record shall indicate the location of the defect and the basis of rejection.

6. Inspection of High-Strength Bolting

Observation of bolting operations shall be the primary method used to confirm that the materials, procedures and workmanship incorporated in construction are in conformance with the construction documents and the provisions of the RCSC *Specification*.

(a) For snug-tight joints, pre-installation verification testing as specified in Table N5.6-1 and monitoring of the installation procedures as specified in Table N5.6-2 are not applicable. The QCI and QAI need not be present during the installation of fasteners in snug-tight joints.

(b) For pretensioned joints and slip-critical joints, when the installer is using the turn-of-nut method with matchmarking techniques, the direct-tension-indicator method, or the twist-off-type tension control bolt method, monitoring of bolt pretensioning procedures shall be as specified in Table N5.6-2. The QCI and QAI need not be present during the installation of fasteners when these methods are used by the installer.

(c) For pretensioned joints and slip-critical joints, when the installer is using the calibrated wrench method or the turn-of-nut method without matchmarking, monitoring of bolt pretensioning procedures shall be as specified in Table N5.6-2. The QCI and QAI shall be engaged in their assigned inspection duties during installation of fasteners when these methods are used by the installer.

As a minimum, bolting inspection tasks shall be in accordance with Tables N5.6-1, N5.6-2 and N5.6-3. In these tables, the inspection tasks are as follows:

(a) Observe (O): The inspector shall observe these items on a random basis. Operations need not be delayed pending these inspections.

(b) Perform (P): These tasks shall be performed for each bolted connection.

7. Inspection of Galvanized Structural Steel Main Members

Exposed cut surfaces of galvanized structural steel main members and exposed corners of rectangular HSS shall be visually inspected for cracks subsequent to galvanizing. Cracks shall be repaired or the member shall be rejected.

> **User Note:** It is normal practice for fabricated steel that requires hot dip galvanizing to be delivered to the galvanizer and then shipped to the jobsite. As a result, inspection on site is common.

8. Other Inspection Tasks

The fabricator's QCI shall inspect the fabricated steel to verify compliance with the details shown on the shop drawings.

> **User Note:** This includes such items as the correct application of shop joint details at each connection.

The erector's QCI shall inspect the erected steel frame to verify compliance with the field installed details shown on the erection drawings.

> **User Note:** This includes such items as braces, stiffeners, member locations, and correct application of field joint details at each connection.

The QAI shall be on the premises for inspection during the placement of anchor rods and other embedments supporting structural steel for compliance with the construction documents. As a minimum, the diameter, grade, type and length of the anchor rod or embedded item, and the extent or depth of embedment into the concrete, shall be verified and documented prior to placement of concrete.

The QAI shall inspect the fabricated steel or erected steel frame, as applicable, to verify compliance with the details shown on the construction documents.

> **User Note:** This includes such items as braces, stiffeners, member locations and the correct application of joint details at each connection.

The acceptance or rejection of joint details and the correct application of joint details shall be documented.

TABLE N5.6-1
Inspection Tasks Prior to Bolting

Inspection Tasks Prior to Bolting	QC	QA
Manufacturer's certifications available for fastener materials	O	P
Fasteners marked in accordance with ASTM requirements	O	O
Correct fasteners selected for the joint detail (grade, type, bolt length if threads are to be excluded from shear plane)	O	O
Correct bolting procedure selected for joint detail	O	O
Connecting elements, including the appropriate faying surface condition and hole preparation, if specified, meet applicable requirements	O	O
Pre-installation verification testing by installation personnel observed and documented for fastener assemblies and methods used	P	O
Protected storage provided for bolts, nuts, washers and other fastener components	O	O

TABLE N5.6-2
Inspection Tasks During Bolting

Inspection Tasks During Bolting	QC	QA
Fastener assemblies placed in all holes and washers and nuts are positioned as required	O	O
Joint brought to the snug-tight condition prior to the pretensioning operation	O	O
Fastener component not turned by the wrench prevented from rotating	O	O
Fasteners are pretensioned in accordance with the RCSC *Specification*, progressing systematically from the most rigid point toward the free edges	O	O

TABLE N5.6-3
Inspection Tasks After Bolting

Inspection Tasks After Bolting	QC	QA
Document acceptance or rejection of bolted connections	P	P

N6. APPROVED FABRICATORS AND ERECTORS

QA inspection is permitted to be waived when the work is performed in a fabricating shop or by an erector approved by the AHJ to perform the work without QA.

NDT of welds completed in an approved fabricator's shop is permitted to be performed by that fabricator when approved by the AHJ. When the fabricator performs the NDT, the QA agency shall review the fabricator's NDT reports.

At completion of fabrication, the approved fabricator shall submit a certificate of compliance to the AHJ stating that the materials supplied and work performed by the fabricator are in accordance with the construction documents. At completion of erection, the approved erector shall submit a certificate of compliance to the AHJ stating that the materials supplied and work performed by the erector are in accordance with the construction documents.

N7. NONCONFORMING MATERIAL AND WORKMANSHIP

Identification and rejection of material or workmanship that is not in conformance with the construction documents is permitted at any time during the progress of the work. However, this provision shall not relieve the owner or the inspector of the obligation for timely, in-sequence inspections. Nonconforming material and workmanship shall be brought to the immediate attention of the fabricator or erector, as applicable.

Nonconforming material or workmanship shall be brought into conformance or made suitable for its intended purpose as determined by the EOR.

Concurrent with the submittal of such reports to the AHJ, EOR or owner, the QA agency shall submit to the fabricator and erector:

(a) Nonconformance reports

(b) Reports of repair, replacement or acceptance of nonconforming items

APPENDIX 1

DESIGN BY ADVANCED ANALYSIS

This Appendix permits the use of more advanced methods of structural analysis to directly model system and member imperfections and/or allow for the redistribution of member and connection forces and moments as a result of localized yielding.

The appendix is organized as follows:

1.1. GENERAL REQUIREMENTS

The analysis methods permitted in this Appendix shall ensure that equilibrium and compatibility are satisfied for the structure in its deformed shape, including all flexural, shear, axial and torsional deformations, and all other component and connection deformations that contribute to the displacements of the structure.

Design by the methods of this Appendix shall be conducted in accordance with Section B3.1, using load and resistance factor design (LRFD).

1.2. DESIGN BY ELASTIC ANALYSIS

1. General Stability Requirements

Design by a second-order elastic analysis that includes the direct modeling of system and member imperfections is permitted for all structures subject to the limitations defined in this section. All requirements of Section C1 apply, with additional requirements and exceptions as noted below. All load-dependent effects shall be calculated at a level of loading corresponding to LRFD load combinations.

The influence of torsion shall be considered, including its impact on member deformations and second-order effects.

The provisions of this method apply only to doubly symmetric members, including I-shapes, HSS and box sections, unless evidence is provided that the method is applicable to other member types.

2. Calculation of Required Strengths

For design using a second-order elastic analysis that includes the direct modeling of system and member imperfections, the required strengths of components of the structure shall be determined from an analysis conforming to Section C2, with additional requirements and exceptions as noted in the following.

2a. General Analysis Requirements

The analysis of the structure shall also conform to the following requirements:

(a) Torsional member deformations shall be considered in the analysis.

(b) The analysis shall consider geometric nonlinearities, including P-Δ, P-δ and twisting effects as applicable to the structure. The use of the approximate procedures appearing in Appendix 8 is not permitted.

> **User Note:** A rigorous second-order analysis of the structure is an important requirement for this method of design. Many analysis routines common in design offices are based on a more traditional second-order analysis approach that includes only P-Δ and P-δ effects without consideration of additional second-order effects related to member twist, which can be significant for some members with unbraced lengths near or exceeding L_r. The type of second-order analysis defined herein also includes the beneficial effects of additional member torsional strength and stiffness due to warping restraint, which can be conservatively neglected. Refer to the Commentary for additional information and guidance.

(c) In all cases, the analysis shall directly model the effects of initial imperfections due to both points of intersection of members displaced from their nominal locations (system imperfections), and initial out-of-straightness or offsets of members along their length (member imperfections). The magnitude of the initial displacements shall be the maximum amount considered in the design; the pattern of initial displacements shall be such that it provides the greatest destabilizing effect for the load combination being considered. The use of notional loads to represent either type of imperfection is not permitted.

> **User Note:** Initial displacements similar in configuration to both displacements due to loading and anticipated buckling modes should be considered in the modeling of imperfections. The magnitude of the initial points of intersection of members displaced from their nominal locations (system imperfections) should be based on permissible construction tolerances, as specified in the AISC *Code of Standard Practice for Steel Buildings and Bridges* or other governing requirements, or on actual imperfections, if known. When these displacements are due to erection tolerances, 1/500 is often considered, based on the tolerance of the out-of-plumbness ratio specified in the *Code of Standard Practice*. For out-of-straightness of members (member imperfections), a 1/1000 out-of-straightness ratio is often considered. Refer to the Commentary for additional guidance.

2b. Adjustments to Stiffness

The analysis of the structure to determine the required strengths of components shall use reduced stiffnesses as defined in Section C2.3. Such stiffness reduction, including factors of 0.8 and τ_b, shall be applied to all stiffnesses that are considered to contribute to the stability of the structure. The use of notional loads to represent τ_b is not permitted.

> **User Note:** Stiffness reduction should be applied to all member properties including torsional properties (GJ and EC_w) affecting twist of the member cross section. One practical method of including stiffness reduction is to reduce E and G by $0.8\tau_b$, thereby leaving all cross-section geometric properties at their nominal value.
>
> Applying this stiffness reduction to some members and not others can, in some cases, result in artificial distortion of the structure under load and thereby lead to an unintended redistribution of forces. This can be avoided by applying the reduction to all members, including those that do not contribute to the stability of the structure.

3. Calculation of Available Strengths

For design using a second-order elastic analysis that includes the direct modeling of system and member imperfections, the available strengths of members and connections shall be calculated in accordance with the provisions of Chapters D through K, as applicable, except as defined below, with no further consideration of overall structure stability.

The nominal compressive strength of members, P_n, may be taken as the cross-section compressive strength, F_yA_g, or as F_yA_e for members with slender elements, where A_e is defined in Section E7.

1.3. DESIGN BY INELASTIC ANALYSIS

> **User Note:** Design by the provisions of this section is independent of the requirements of Section 1.2.

1. General Requirements

The design strength of the structural system and its members and connections shall equal or exceed the required strength as determined by the inelastic analysis. The provisions of Section 1.3 do not apply to seismic design.

The inelastic analysis shall take into account: (a) flexural, shear, axial and torsional member deformations, and all other component and connection deformations that contribute to the displacements of the structure; (b) second-order effects (including P-Δ, P-δ and twisting effects); (c) geometric imperfections; (d) stiffness reductions due to inelasticity, including partial yielding of the cross section that may be accentuated by the presence of residual stresses; and (e) uncertainty in system, member, and connection strength and stiffness.

Strength limit states detected by an inelastic analysis that incorporates all of the preceding requirements in this Section are not subject to the corresponding provisions of this Specification when a comparable or higher level of reliability is provided by the analysis. Strength limit states not detected by the inelastic analysis shall be evaluated using the corresponding provisions of Chapters D through K.

Connections shall meet the requirements of Section B3.4.

Members and connections subject to inelastic deformations shall be shown to have ductility consistent with the intended behavior of the structural system. Force redistribution due to rupture of a member or connection is not permitted.

Any method that uses inelastic analysis to proportion members and connections to satisfy these general requirements is permitted. A design method based on inelastic analysis that meets the preceding strength requirements, the ductility requirements of Section 1.3.2, and the analysis requirements of Section 1.3.3 satisfies these general requirements.

2. Ductility Requirements

Members and connections with elements subject to yielding shall be proportioned such that all inelastic deformation demands are less than or equal to their inelastic deformation capacities. In lieu of explicitly ensuring that the inelastic deformation demands are less than or equal to their inelastic deformation capacities, the following requirements shall be satisfied for steel members subject to plastic hinging.

2a. Material

The specified minimum yield stress, F_y, of members subject to plastic hinging shall not exceed 65 ksi (450 MPa).

2b. Cross Section

The cross section of members at plastic hinge locations shall be doubly symmetric with width-to-thickness ratios of their compression elements not exceeding λ_{pd}, where λ_{pd} is equal to λ_p from Table B4.1b, except as modified below:

(a) For the width-to-thickness ratio, h/t_w, of webs of I-shaped members, rectangular HSS, and box sections subject to combined flexure and compression

(1) When $P_u/\phi_c P_y \le 0.125$

$$\lambda_{pd} = 3.76\sqrt{\frac{E}{F_y}}\left(1 - \frac{2.75P_u}{\phi_c P_y}\right) \tag{A-1-1}$$

(2) When $P_u/\phi_c P_y > 0.125$

$$\lambda_{pd} = 1.12\sqrt{\frac{E}{F_y}}\left(2.33 - \frac{P_u}{\phi_c P_y}\right) \ge 1.49\sqrt{\frac{E}{F_y}} \tag{A-1-2}$$

where

P_u = required axial strength in compression, using LRFD load combinations, kips (N)

P_y = $F_y A_g$ = axial yield strength, kips (N)

h = as defined in Section B4.1, in. (mm)

t_w = web thickness, in. (mm)

ϕ_c = resistance factor for compression = 0.90

(b) For the width-to-thickness ratio, b/t, of flanges of rectangular HSS and box sections, and for flange cover plates, and diaphragm plates between lines of fasteners or welds

$$\lambda_{pd} = 0.94\sqrt{E/F_y} \tag{A-1-3}$$

where

b = as defined in Section B4.1, in. (mm)

t = as defined in Section B4.1, in. (mm)

(c) For the diameter-to-thickness ratio, D/t, of round HSS in flexure

$$\lambda_{pd} = 0.045E/F_y \tag{A-1-4}$$

where

D = outside diameter of round HSS, in. (mm)

2c. Unbraced Length

In prismatic member segments that contain plastic hinges, the laterally unbraced length, L_b, shall not exceed L_{pd}, determined as follows. For members subject to flexure only, or to flexure and axial tension, L_b shall be taken as the length between points braced against lateral displacement of the compression flange, or between points braced to prevent twist of the cross section. For members subject to flexure and axial compression, L_b shall be taken as the length between points braced against both lateral displacement in the minor axis direction and twist of the cross section.

(a) For I-shaped members bent about their major axis:

$$L_{pd} = \left(0.12 - 0.076\frac{M_1'}{M_2}\right)\frac{E}{F_y}r_y \tag{A-1-5}$$

where

r_y = radius of gyration about minor axis, in. (mm)

(1) When the magnitude of the bending moment at any location within the unbraced length exceeds M_2

$$M_1'/M_2 = +1 \tag{A-1-6a}$$

Otherwise:

(2) When $M_{mid} \leq (M_1 + M_2)/2$

$$M_1' = M_1 \tag{A-1-6b}$$

(3) When $M_{mid} > (M_1 + M_2)/2$

$$M_1' = (2M_{mid} - M_2) < M_2 \tag{A-1-6c}$$

where

M_1 = smaller moment at end of unbraced length, kip-in. (N-mm)

M_2 = larger moment at end of unbraced length, kip-in. (N-mm) (shall be taken as positive in all cases)

M_{mid} = moment at middle of unbraced length, kip-in. (N-mm)

M_1' = effective moment at end of unbraced length opposite from M_2, kip-in. (N-mm)

The moments M_1 and M_{mid} are individually taken as positive when they cause compression in the same flange as the moment, M_2, and taken as negative otherwise.

(b) For solid rectangular bars and for rectangular HSS and box sections bent about their major axis

$$L_{pd} = \left(0.17 - 0.10\frac{M_1'}{M_2}\right)\frac{E}{F_y}r_y \geq 0.10\frac{E}{F_y}r_y \tag{A-1-7}$$

For all types of members subject to axial compression and containing plastic hinges, the laterally unbraced lengths about the cross-section major and minor axes shall not exceed $4.71r_x\sqrt{E/F_y}$ and $4.71r_y\sqrt{E/F_y}$, respectively.

There is no L_{pd} limit for member segments containing plastic hinges in the following cases:

(a) Members with round or square cross sections subject only to flexure or to combined flexure and tension

(b) Members subject only to flexure about their minor axis or combined tension and flexure about their minor axis

(c) Members subject only to tension

2d. Axial Force

To ensure ductility in compression members with plastic hinges, the design strength in compression shall not exceed $0.75F_yA_g$.

3. Analysis Requirements

The structural analysis shall satisfy the general requirements of Section 1.3.1. These requirements are permitted to be satisfied by a second-order inelastic analysis meeting the requirements of this Section.

Exception: For continuous beams not subject to axial compression, a first-order inelastic or plastic analysis is permitted and the requirements of Sections 1.3.3b and 1.3.3c are waived.

User Note: Refer to the Commentary for guidance in conducting a traditional plastic analysis and design in conformance with these provisions.

3a. Material Properties and Yield Criteria

The specified minimum yield stress, F_y, and the stiffness of all steel members and connections shall be reduced by a factor of 0.9 for the analysis, except as stipulated in Section 1.3.3c.

The influence of axial force, major axis bending moment, and minor axis bending moment shall be included in the calculation of the inelastic response.

The plastic strength of the member cross section shall be represented in the analysis either by an elastic-perfectly-plastic yield criterion expressed in terms of the axial force, major axis bending moment, and minor axis bending moment, or by explicit modeling of the material stress-strain response as elastic-perfectly-plastic.

3b. Geometric Imperfections

In all cases, the analysis shall directly model the effects of initial imperfections due to both points of intersection of members displaced from their nominal locations (system imperfections), and initial out-of-straightness or offsets of members along their length (member imperfections). The magnitude of the initial displacements shall be the maximum amount considered in the design; the pattern of initial displacements shall be such that it provides the greatest destabilizing effect.

3c. Residual Stress and Partial Yielding Effects

The analysis shall include the influence of residual stresses and partial yielding. This shall be done by explicitly modeling these effects in the analysis or by reducing the stiffness of all structural components as specified in Section C2.3.

If the provisions of Section C2.3 are used, then:

(a) The 0.9 stiffness reduction factor specified in Section 1.3.3a shall be replaced by the reduction of the elastic modulus, E, by 0.8 as specified in Section C2.3, and

(b) The elastic-perfectly-plastic yield criterion, expressed in terms of the axial force, major axis bending moment, and minor axis bending moment, shall satisfy the cross-section strength limit defined by Equations H1-1a and H1-1b using $P_c = 0.9P_y$, $M_{cx} = 0.9M_{px}$, and $M_{cy} = 0.9M_{py}$.

APPENDIX 2

DESIGN FOR PONDING

This appendix provides methods for determining whether a roof system has adequate strength and stiffness to resist ponding. These methods are valid for flat roofs with rectangular bays where the beams are uniformly spaced and the girders are considered to be uniformly loaded.

The appendix is organized as follows:

2.1. Simplified Design for Ponding
2.2. Improved Design for Ponding

The members of a roof system shall be considered to have adequate strength and stiffness against ponding by satisfying the requirements of Sections 2.1 or 2.2.

2.1. SIMPLIFIED DESIGN FOR PONDING

The roof system shall be considered stable for ponding and no further investigation is needed if both of the following two conditions are met:

$$C_p + 0.9C_s \leq 0.25 \tag{A-2-1}$$

$$I_d \geq 25(S^4)10^{-6} \tag{A-2-2}$$

$$I_d \geq 3\,940S^4 \tag{A-2-2M}$$

where

$$C_p = \frac{32L_sL_p^4}{10^7 I_p} \tag{A-2-3}$$

$$C_p = \frac{504L_sL_p^4}{I_p} \tag{A-2-3M}$$

$$C_s = \frac{32SL_s^4}{10^7 I_s} \tag{A-2-4}$$

$$C_s = \frac{504SL_s^4}{I_s} \tag{A-2-4M}$$

I_d = moment of inertia of the steel deck supported on secondary members, in.^{4} per ft (mm^4 per m)
I_p = moment of inertia of primary members, in.^{4} (mm^4)
I_s = moment of inertia of secondary members, in.^{4} (mm^4)
L_p = length of primary members, ft (m)
L_s = length of secondary members, ft (m)
S = spacing of secondary members, ft (m)

For trusses and steel joists, the calculation of the moments of inertia, I_p and I_s, shall include the effects of web member strain when used in the above equation.

User Note: When the moment of inertia is calculated using only the truss or joist chord areas, the reduction in the moment of inertia due to web member strain can typically be taken as 15%.

A steel deck shall be considered a secondary member when it is directly supported by the primary members.

2.2. IMPROVED DESIGN FOR PONDING

It is permitted to use the provisions in this section when a more accurate evaluation of framing stiffness is needed than that given by Equations A-2-1 and A-2-2.

Define the stress indexes

$$U_p = \left(\frac{0.8F_y - f_o}{f_o}\right)_p \quad \text{for the primary member} \tag{A-2-5}$$

$$U_s = \left(\frac{0.8F_y - f_o}{f_o}\right)_s \quad \text{for the secondary member} \tag{A-2-6}$$

where

F_y = specified minimum yield stress, ksi (MPa)

f_o = stress due to impounded water due to either nominal rain or snow loads (exclusive of the ponding contribution), and other loads acting concurrently as specified in Section B2, ksi (MPa)

For roof framing consisting of primary and secondary members, evaluate the combined stiffness as follows. Enter Figure A-2.1 at the level of the computed stress index, U_p, determined for the primary beam; move horizontally to the computed C_s value of the secondary beams and then downward to the abscissa scale. The combined stiffness of the primary and secondary framing is sufficient to prevent ponding if the flexibility coefficient read from this latter scale is more than the value of C_p computed for the given primary member; if not, a stiffer primary or secondary beam, or combination of both, is required.

A similar procedure must be followed using Figure A-2.2.

For roof framing consisting of a series of equally spaced wall bearing beams, evaluate the stiffness as follows. The beams are considered as secondary members supported on an infinitely stiff primary member. For this case, enter Figure A-2.2 with the computed stress index, U_s. The limiting value of C_s is determined by the intercept of a horizontal line representing the U_s value and the curve for $C_p = 0$.

Evaluate the stability against ponding of a roof consisting of a metal roof deck of relatively slender depth-to-span ratio, spanning between beams supported directly on columns, as follows. Use Figure A-2.1 or A-2.2, using as C_s the flexibility coefficient for a one-foot (one-meter) width of the roof deck (S = 1.0).

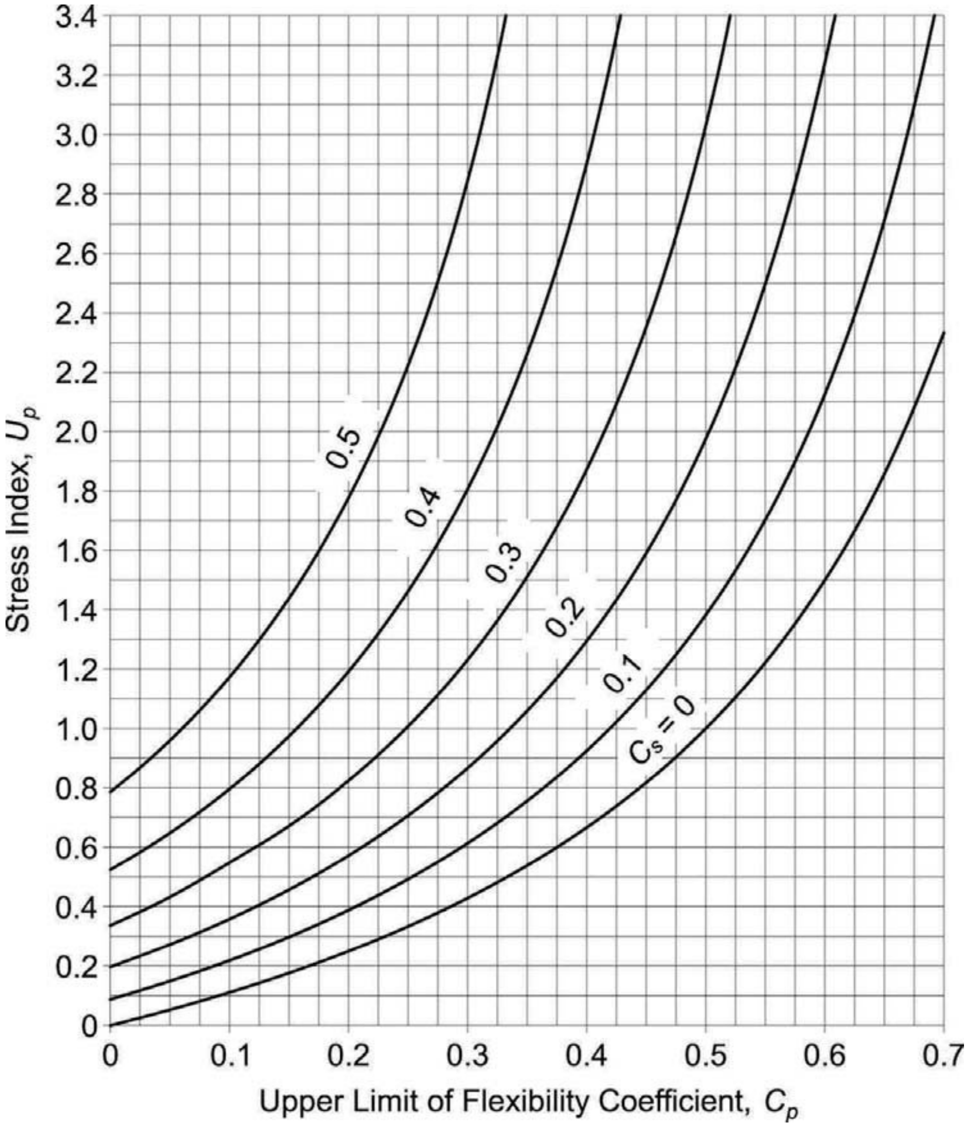


Fig. A-2.1. Limiting flexibility coefficient for the primary systems.

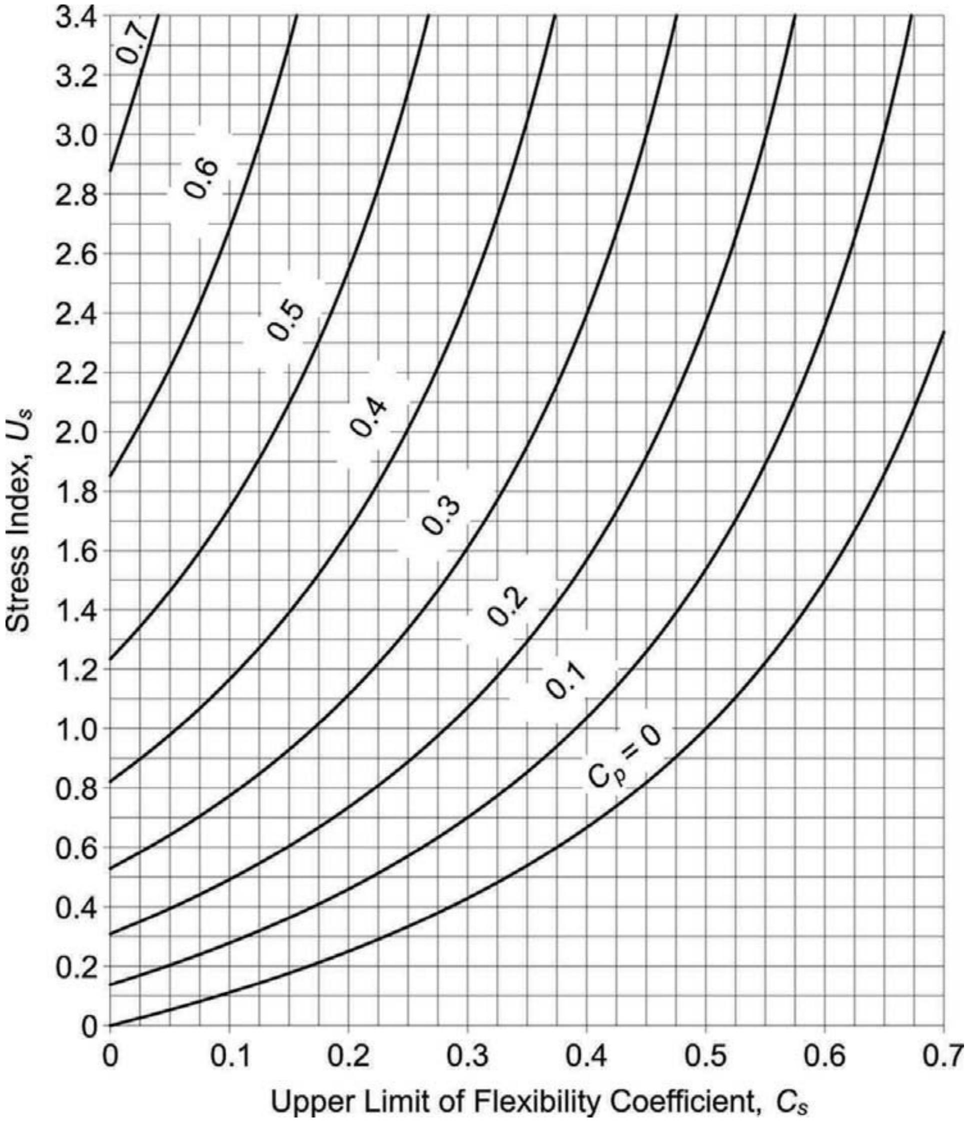


Fig. A-2.2. Limiting flexibility coefficient for the secondary systems.

APPENDIX 3

FATIGUE

This appendix applies to members and connections subject to high-cycle loading within the elastic range of stresses of frequency and magnitude sufficient to initiate cracking and progressive failure.

User Note: See AISC *Seismic Provisions for Structural Steel Buildings* for structures subject to seismic loads.

The appendix is organized as follows:

3.1. General Provisions
3.2. Calculation of Maximum Stresses and Stress Ranges
3.3. Plain Material and Welded Joints
3.4. Bolts and Threaded Parts
3.5. Fabrication and Erection Requirements for Fatigue
3.6. Nondestructive Examination Requirements for Fatigue

3.1. GENERAL PROVISIONS

The fatigue resistance of members consisting of shapes or plate shall be determined when the number of cycles of application of live load exceeds 20,000. No evaluation of fatigue resistance of members consisting of HSS in building-type structures subject to code mandated wind loads is required. When the applied cyclic stress range is less than the threshold allowable stress range, F_{TH}, no further evaluation of fatigue resistance is required. See Table A-3.1.

The engineer of record shall provide either complete details including weld sizes or shall specify the planned cycle life and the maximum range of moments, shears and reactions for the connections.

The provisions of this Appendix shall apply to stresses calculated on the basis of the applied cyclic load spectrum. The maximum permitted stress due to peak cyclic loads shall be $0.66F_y$. In the case of a stress reversal, the stress range shall be computed as the numerical sum of maximum repeated tensile and compressive stresses or the numerical sum of maximum shearing stresses of opposite direction at the point of probable crack initiation.

The cyclic load resistance determined by the provisions of this Appendix is applicable to structures with suitable corrosion protection or subject only to mildly corrosive atmospheres, such as normal atmospheric conditions.

The cyclic load resistance determined by the provisions of this Appendix is applicable only to structures subject to temperatures not exceeding 300°F (150°C).

3.2. CALCULATION OF MAXIMUM STRESSES AND STRESS RANGES

Calculated stresses shall be based upon elastic analysis. Stresses shall not be amplified by stress concentration factors for geometrical discontinuities.

For bolts and threaded rods subject to axial tension, the calculated stresses shall include the effects of prying action, if any. In the case of axial stress combined with bending, the maximum stresses of each kind shall be those determined for concurrent arrangements of the applied load.

For members having symmetric cross sections, the fasteners and welds shall be arranged symmetrically about the axis of the member, or the total stresses including those due to eccentricity shall be included in the calculation of the stress range.

For axially loaded angle members where the center of gravity of the connecting welds lies between the line of the center of gravity of the angle cross section and the center of the connected leg, the effects of eccentricity shall be ignored. If the center of gravity of the connecting welds lies outside this zone, the total stresses, including those due to joint eccentricity, shall be included in the calculation of stress range.

3.3. PLAIN MATERIAL AND WELDED JOINTS

In plain material and welded joints, the range of stress due to the applied cyclic loads shall not exceed the allowable stress range computed as follows.

(a) For stress categories A, B, B′, C, D, E and E′, the allowable stress range, F_{SR}, shall be determined by Equation A-3-1 or A-3-1M, as follows:

$$F_{SR} = 1{,}000\left(\frac{C_f}{n_{SR}}\right)^{0.333} \geq F_{TH} \tag{A-3-1}$$

$$F_{SR} = 6\,900\left(\frac{C_f}{n_{SR}}\right)^{0.333} \geq F_{TH} \tag{A-3-1M}$$

where

C_f = constant from Table A-3.1 for the fatigue category

F_{SR} = allowable stress range, ksi (MPa)

F_{TH} = threshold allowable stress range, maximum stress range for indefinite design life from Table A-3.1, ksi (MPa)

n_{SR} = number of stress range fluctuations in design life

(b) For stress category F, the allowable stress range, F_{SR}, shall be determined by Equation A-3-2 or A-3-2M as follows:

$$F_{SR} = 100\left(\frac{1.5}{n_{SR}}\right)^{0.167} \geq 8 \text{ ksi} \tag{A-3-2}$$

$$F_{SR} = 690\left(\frac{1.5}{n_{SR}}\right)^{0.167} \geq 55 \text{ MPa} \tag{A-3-2M}$$

(c) For tension-loaded plate elements connected at their end by cruciform, T or corner details with partial-joint-penetration (PJP) groove welds transverse to the direction of stress, with or without reinforcing or contouring fillet welds, or if joined with only fillet welds, the allowable stress range on the cross section of the tension-loaded plate element shall be determined as the lesser of the following:

(1) Based upon crack initiation from the toe of the weld on the tension-loaded plate element (i.e., when $R_{PJP} = 1.0$), the allowable stress range, F_{SR}, shall be determined by Equation A-3-1 or A-3-1M for stress category C.

(2) Based upon crack initiation from the root of the weld, the allowable stress range, F_{SR}, on the tension loaded plate element using transverse PJP groove welds, with or without reinforcing or contouring fillet welds, the allowable stress range on the cross section at the root of the weld shall be determined by Equation A-3-3 or A-3-3M, for stress category C′ as follows:

$$F_{SR} = 1{,}000 R_{PJP} \left(\frac{4.4}{n_{SR}} \right)^{0.333} \tag{A-3-3}$$

$$F_{SR} = 6\,900 R_{PJP} \left(\frac{4.4}{n_{SR}} \right)^{0.333} \tag{A-3-3M}$$

where

R_{PJP}, the reduction factor for reinforced or nonreinforced transverse PJP groove welds, is determined as follows:

$$R_{PJP} = \frac{0.65 - 0.59\left(\dfrac{2a}{t_p}\right) + 0.72\left(\dfrac{w}{t_p}\right)}{t_p^{0.167}} \le 1.0 \tag{A-3-4}$$

$$R_{PJP} = \frac{1.12 - 1.01\left(\dfrac{2a}{t_p}\right) + 1.24\left(\dfrac{w}{t_p}\right)}{t_p^{0.167}} \le 1.0 \tag{A-3-4M}$$

$2a$ = length of the nonwelded root face in the direction of the thickness of the tension-loaded plate, in. (mm)

t_p = thickness of tension loaded plate, in. (mm)

w = leg size of the reinforcing or contouring fillet, if any, in the direction of the thickness of the tension-loaded plate, in. (mm)

If $R_{PJP} = 1.0$, the stress range will be limited by the weld toe and category C will control.

(3) Based upon crack initiation from the roots of a pair of transverse fillet welds on opposite sides of the tension loaded plate element, the allowable stress range, F_{SR}, on the cross section at the root of the welds shall be determined by Equation A-3-5 or A-3-5M, for stress category C″ as follows:

$$F_{SR} = 1{,}000 R_{FIL} \left(\frac{4.4}{n_{SR}} \right)^{0.333} \tag{A-3-5}$$

$$F_{SR} = 6\,900 R_{FIL} \left(\frac{4.4}{n_{SR}} \right)^{0.333} \tag{A-3-5M}$$

where

R_{FIL} = reduction factor for joints using a pair of transverse fillet welds only

$$= \frac{0.06 + 0.72\left(w / t_p\right)}{t_p^{0.167}} \leq 1.0 \tag{A-3-6}$$

$$= \frac{0.103 + 1.24\left(w / t_p\right)}{t_p^{0.167}} \leq 1.0 \tag{A-3-6M}$$

If R_{FIL} = 1.0, the stress range will be limited by the weld toe and category C will control.

User Note: Stress categories C′ and C″ are cases where the fatigue crack initiates in the root of the weld. These cases do not have a fatigue threshold and cannot be designed for an infinite life. Infinite life can be approximated by use of a very high cycle life such as 2×10^8. Alternatively, if the size of the weld is increased such that R_{FIL} or R_{PJP} is equal to 1.0, then the base metal controls, resulting in stress category C, where there is a fatigue threshold and the crack initiates at the toe of the weld.

3.4. BOLTS AND THREADED PARTS

In bolts and threaded parts, the range of stress of the applied cyclic load shall not exceed the allowable stress range computed as follows.

(a) For mechanically fastened connections loaded in shear, the maximum range of stress in the connected material of the applied cyclic load shall not exceed the allowable stress range computed using Equation A-3-1 or A-3-1M, where C_f and F_{TH} are taken from Section 2 of Table A-3.1.

(b) For high-strength bolts, common bolts, threaded anchor rods, and hanger rods with cut, ground or rolled threads, the maximum range of tensile stress on the net tensile area from applied axial load and moment plus load due to prying action shall not exceed the allowable stress range computed using Equation A-3-1 or A-3-1M, where C_f and F_{TH} are taken from Case 8.5 (stress category G). The net area in tension, A_t, is given by Equation A-3-7 or A-3-7M.

$$A_t = \frac{\pi}{4}\left(d_b - \frac{0.9743}{n}\right)^2 \qquad \text{(A-3-7)}$$

$$A_t = \frac{\pi}{4}\left(d_b - 0.9382p\right)^2 \qquad \text{(A-3-7M)}$$

where
d_b = nominal diameter (body or shank diameter), in. (mm)
n = threads per in. (per mm)
p = pitch, in. per thread (mm per thread)

For joints in which the material within the grip is not limited to steel or joints that are not tensioned to the requirements of Table J3.1 or J3.1M, all axial load and moment applied to the joint plus effects of any prying action shall be assumed to be carried exclusively by the bolts or rods.

For joints in which the material within the grip is limited to steel and which are pretensioned to the requirements of Table J3.1 or J3.1M, an analysis of the relative stiffness of the connected parts and bolts is permitted to be used to determine the tensile stress range in the pretensioned bolts due to the total applied cyclic load and moment, plus effects of any prying action. Alternatively, the stress range in the bolts shall be assumed to be equal to the stress on the net tensile area due to 20% of the absolute value of the applied cyclic axial load and moment from dead, live and other loads.

3.5. FABRICATION AND ERECTION REQUIREMENTS FOR FATIGUE

Longitudinal steel backing, if used, shall be continuous. If splicing of steel backing is required for long joints, the splice shall be made with a complete-joint-penetration (CJP) groove weld, ground flush to permit a tight fit. If fillet welds are used to attach left-in-place longitudinal backing, they shall be continuous.

In transverse CJP groove welded T- and corner-joints, a reinforcing fillet weld, not less than 1/4 in. (6 mm) in size, shall be added at reentrant corners.

The surface roughness of thermally cut edges subject to cyclic stress ranges, that include tension, shall not exceed 1,000 μin. (25 μm), where *Surface Texture, Surface Roughness, Waviness, and Lay* (ASME B46.1) is the reference standard.

> **User Note:** AWS C4.1 Sample 3 may be used to evaluate compliance with this requirement.

Reentrant corners at cuts, copes and weld access holes shall form a radius not less than the prescribed radius in Table A-3.1 by predrilling or subpunching and reaming a hole, or by thermal cutting to form the radius of the cut.

For transverse butt joints in regions of tensile stress, weld tabs shall be used to provide for cascading the weld termination outside the finished joint. End dams shall not be used. Weld tabs shall be removed and the end of the weld finished flush with the edge of the member.

Fillet welds subject to cyclic loading normal to the outstanding legs of angles or on the outer edges of end plates shall have end returns around the corner for a distance not less than two times the weld size; the end return distance shall not exceed four times the weld size.

3.6. NONDESTRUCTIVE EXAMINATION REQUIREMENTS FOR FATIGUE

In the case of CJP groove welds, the maximum allowable stress range calculated by Equation A-3-1 or A-3-1M applies only to welds that have been ultrasonically or radiographically tested and meet the acceptance requirements of *Structural Welding Code—Steel* (AWS D1.1/D1.1M) clause 6.12.2 or clause 6.13.2.

TABLE A-3.1
Fatigue Design Parameters

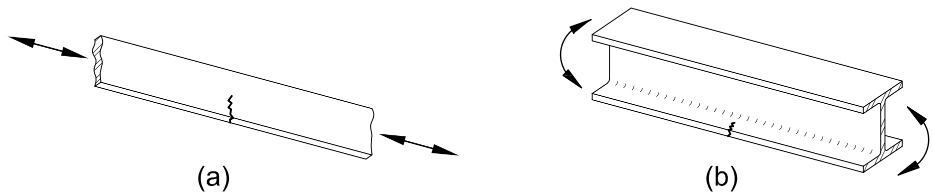
Description	Stress Category	Constant C_f	Threshold F_{TH}, ksi (MPa)	Potential Crack Initiation Point
SECTION 1—PLAIN MATERIAL AWAY FROM ANY WELDING				
1.1 Base metal, except noncoated weathering steel, with as-rolled or cleaned surfaces; flame-cut edges with surface roughness value of 1,000 μin. (25 μm) or less, but without reentrant corners	A	25	24 (165)	Away from all welds or structural connections
1.2 Noncoated weathering steel base metal with as-rolled or cleaned surfaces; flame-cut edges with surface roughness value of 1,000 μin. (25 μm) or less, but without reentrant corners	B	12	16 (110)	Away from all welds or structural connections
1.3 Member with reentrant corners at copes, cuts, block-outs or other geometrical discontinuities, except weld access holes				At any external edge or at hole perimeter
$R \geq 1$ in. (25 mm), with radius, R, formed by predrilling, subpunching and reaming or thermally cut and ground to a bright metal surface	C	4.4	10 (69)	
$R \geq 3/8$ in. (10 mm) and the radius, R, need not be ground to a bright metal surface	E′	0.39	2.6 (18)	
1.4 Rolled cross sections with weld access holes made to requirements of Section J1.6				At reentrant corner of weld access hole
Access hole $R \geq 1$ in. (25 mm) with radius, R, formed by predrilling, subpunching and reaming or thermally cut and ground to a bright metal surface	C	4.4	10 (69)	
Access hole $R \geq 3/8$ in. (10 mm) and the radius, R, need not be ground to a bright metal surface	E′	0.39	2.6 (18)	
1.5 Members with drilled or reamed holes				In net section originating at side of the hole
Holes containing pretensioned bolts	C	4.4	10 (69)	
Open holes without bolts	D	2.2	7 (48)	

TABLE A-3.1 (continued)
Fatigue Design Parameters

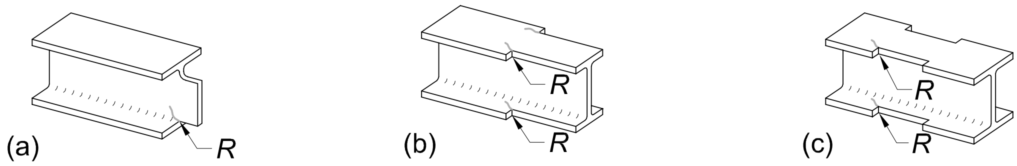
Illustrative Typical Examples

SECTION 1—PLAIN MATERIAL AWAY FROM ANY WELDING

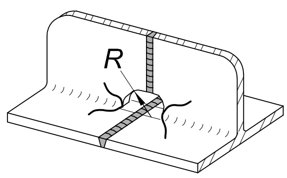
1.1 and 1.2



1.3



1.4



1.5

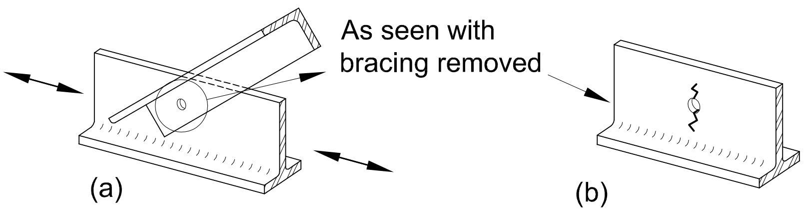


TABLE A-3.1 (continued)
Fatigue Design Parameters

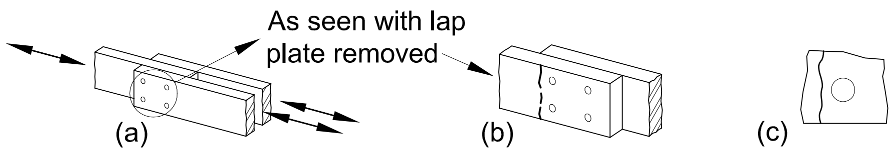
Description	Stress Category	Constant C_f	Threshold F_{TH}, ksi (MPa)	Potential Crack Initiation Point
SECTION 2—CONNECTED MATERIAL IN MECHANICALLY FASTENED JOINTS				
2.1 Gross area of base metal in lap joints connected by high-strength bolts in joints satisfying all requirements for slip-critical connections	B	12	16 (110)	Through gross section near hole
2.2 Base metal at net section of high-strength bolted joints, designed on the basis of bearing resistance, but fabricated and installed to all requirements for slip-critical connections	B	12	16 (110)	In net section originating at side of hole
2.3 Base metal at the net section of riveted joints	C	4.4	10 (69)	In net section originating at side of hole
2.4 Base metal at net section of eyebar head or pin plate	E	1.1	4.5 (31)	In net section originating at side of hole

TABLE A-3.1 (continued)
Fatigue Design Parameters

Illustrative Typical Examples

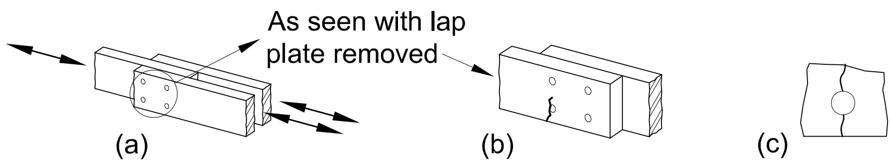
SECTION 2—CONNECTED MATERIAL IN MECHANICALLY FASTENED JOINTS

2.1



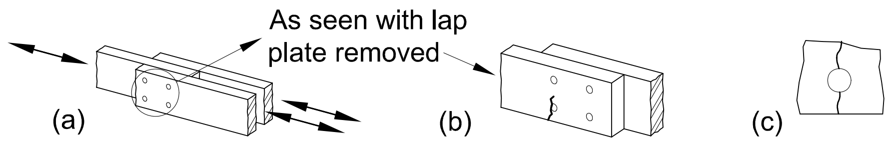
(Note: Figures are for slip-critical bolted connections.)

2.2



(Note: Figures are for bolted connections designed to bear, meeting the requirements of slip-critical connections.)

2.3



(Note: Figures are for snug-tightened bolts, rivets, or other mechanical fasteners.)

2.4

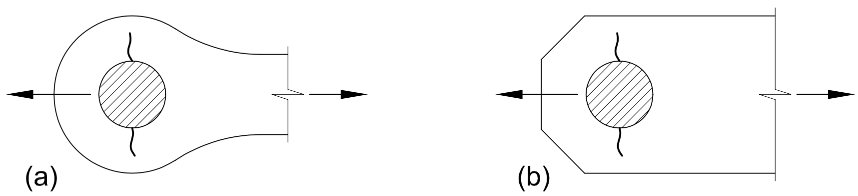


TABLE A-3.1 (continued)
Fatigue Design Parameters

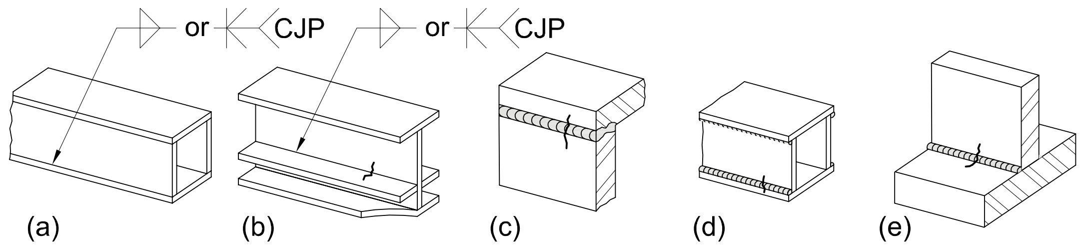
Description	Stress Category	Constant C_f	Threshold F_{TH}, ksi (MPa)	Potential Crack Initiation Point
SECTION 3—WELDED JOINTS JOINING COMPONENTS OF BUILT-UP MEMBERS				
3.1 Base metal and weld metal in members without attachments built up of plates or shapes connected by continuous longitudinal CJP groove welds, back gouged and welded from second side, or by continuous fillet welds	B	12	16 (110)	From surface or internal discontinuities in weld
3.2 Base metal and weld metal in members without attachments built up of plates or shapes, connected by continuous longitudinal CJP groove welds with left-in-place continuous steel backing, or by continuous PJP groove welds	B′	6.1	12 (83)	From surface or internal discontinuities in weld
3.3 Base metal at the ends of longitudinal welds that terminate at weld access holes in connected built-up members, as well as weld toes of fillet welds that wrap around ends of weld access holes				From the weld termination into the web or flange
Access hole $R \geq 1$ in. (25 mm) with radius, R, formed by predrilling, subpunching and reaming, or thermally cut and ground to bright metal surface	D	2.2	7 (48)	
Access hole $R \geq 3/8$ in. (10 mm) and the radius, R, need not be ground to a bright metal surface	E′	0.39	2.6 (18)	
3.4 Base metal at ends of longitudinal intermittent fillet weld segments	E	1.1	4.5 (31)	In connected material at start and stop locations of any weld
3.5 Base metal at ends of partial length welded coverplates narrower than the flange having square or tapered ends, with or without welds across the ends				In flange at toe of end weld (if present) or in flange at termination of longitudinal weld
$t_f \leq 0.8$ in. (20 mm)	E	1.1	4.5 (31)	
$t_f > 0.8$ in. (20 mm) where t_f = thickness of member flange, in. (mm)	E′	0.39	2.6 (18)	

TABLE A-3.1 (continued)
Fatigue Design Parameters

Illustrative Typical Examples

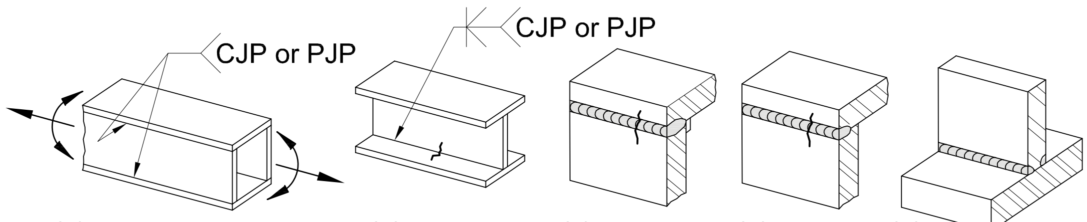
SECTION 3—WELDED JOINTS JOINING COMPONENTS OF BUILT-UP MEMBERS

3.1



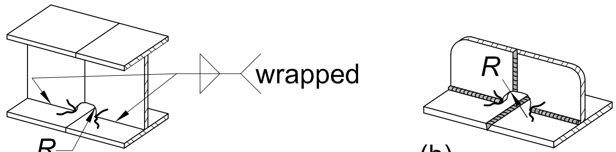
(a) (b) (c) (d) (e)

3.2



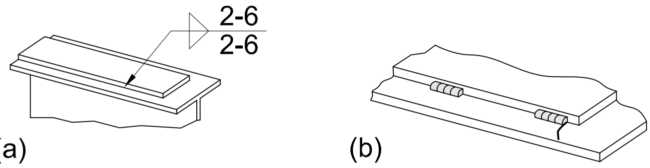
(a) (b) (c) (d) (e)

3.3



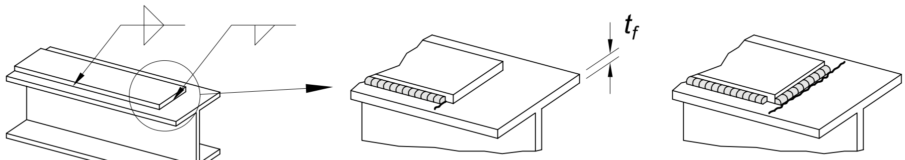
(a) (b)

3.4



(a) (b)

3.5



(a) (b) (c)

TABLE A-3.1 (continued)
Fatigue Design Parameters

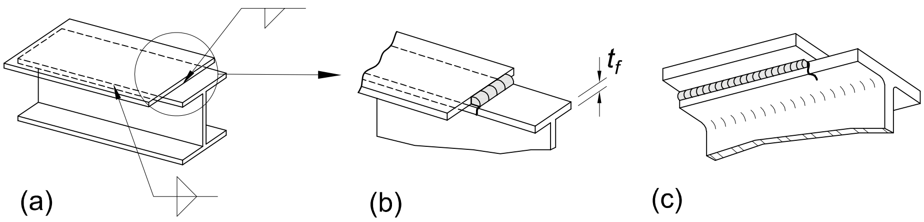
Description	Stress Category	Constant C_f	Threshold F_{TH}, ksi (MPa)	Potential Crack Initiation Point
SECTION 3—WELDED JOINTS JOINING COMPONENTS OF BUILT-UP MEMBERS (cont'd)				
3.6 Base metal at ends of partial length welded coverplates or other attachments wider than the flange with welds across the ends				In flange at toe of end weld or in flange at termination of longitudinal weld or in edge of flange
$t_f \leq 0.8$ in. (20 mm)	E	1.1	4.5 (31)	
$t_f > 0.8$ in. (20 mm)	E′	0.39	2.6 (18)	
3.7 Base metal at ends of partial length welded coverplates wider than the flange without welds across the ends				In edge of flange at end of coverplate weld
$t_f \leq 0.8$ in. (20 mm)	E′	0.39	2.6 (18)	
$t_f > 0.8$ in. (20 mm) is not permitted	None	–	–	
SECTION 4—LONGITUDINAL FILLET WELDED END CONNECTIONS				
4.1 Base metal at junction of axially loaded members with longitudinally welded end connections; welds are on each side of the axis of the member to balance weld stresses				Initiating from end of any weld termination extending into the base metal
$t_f \leq 0.5$ in. (13 mm)	E	1.1	4.5 (31)	
$t_f > 0.5$ in. (13 mm)	E′	0.39	2.6 (18)	
where t = connected member thickness, as shown in Case 4.1 figure, in. (mm)				

TABLE A-3.1 (continued)
Fatigue Design Parameters

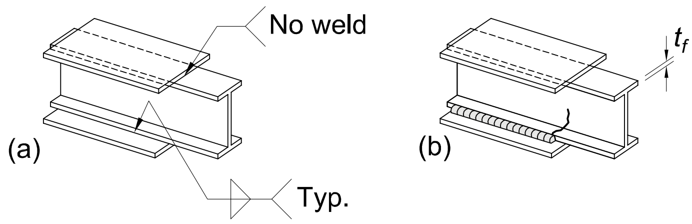
Illustrative Typical Examples

SECTION 3—WELDED JOINTS JOINING COMPONENTS OF BUILT-UP MEMBERS (cont'd)

3.6



3.7



SECTION 4—LONGITUDINAL FILLET WELDED END CONNECTIONS

4.1

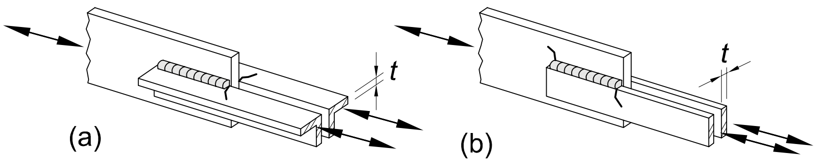


TABLE A-3.1 (continued)
Fatigue Design Parameters

Description	Stress Category	Constant C_f	Threshold F_{TH}, ksi (MPa)	Potential Crack Initiation Point
SECTION 5—WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS				
5.1 Weld metal and base metal in or adjacent to CJP groove welded splices in plate, rolled shapes, or built-up cross sections with no change in cross section with welds ground essentially parallel to the direction of stress and inspected in accordance with Section 3.6	B	12	16 (110)	From internal discontinuities in weld metal or along the fusion boundary
5.2 Weld metal and base metal in or adjacent to CJP groove welded splices with welds ground essentially parallel to the direction of stress at transitions in thickness or width made on a slope no greater than 1:2$^{1}/_{2}$ and inspected in accordance with Section 3.6				From internal discontinuities in metal or along the fusion boundary or at start of transition when $F_y \geq$ 90 ksi (620 MPa)
$F_y <$ 90 ksi (620 MPa)	B	12	16 (110)	
$F_y \geq$ 90 ksi (620 MPa)	B′	6.1	12 (83)	
5.3 Base metal and weld metal in or adjacent to CJP groove welded splices with welds ground essentially parallel to the direction of stress at transitions in width made on a radius, R, of not less than 24 in. (600 mm) with the point of tangency at the end of the groove weld and inspected in accordance with Section 3.6.	B	12	16 (110)	From internal discontinuities in weld metal or along the fusion boundary
5.4 Weld metal and base metal in or adjacent to CJP groove welds in T- or corner-joints or splices, without transitions in thickness or with transition in thickness having slopes no greater than 1:2$^{1}/_{2}$, when weld reinforcement is not removed, and is inspected in accordance with Section 3.6	C	4.4	10 (69)	From weld extending into base metal or into weld metal

TABLE A-3.1 (continued)
Fatigue Design Parameters

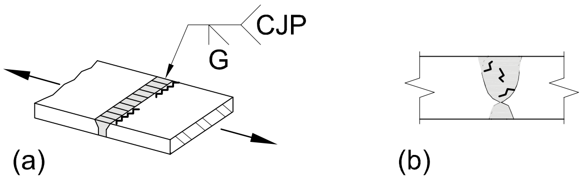
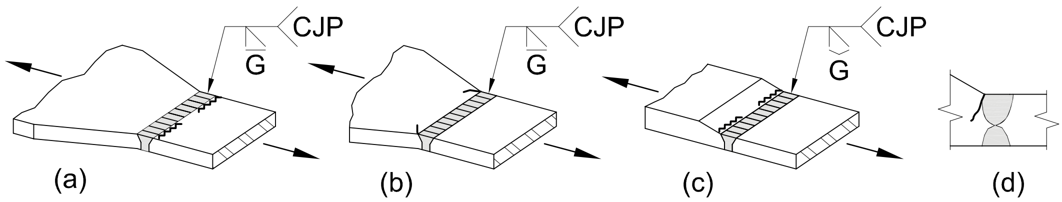
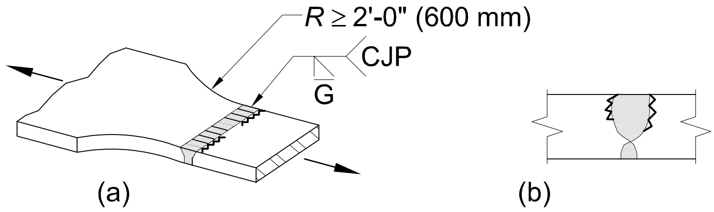
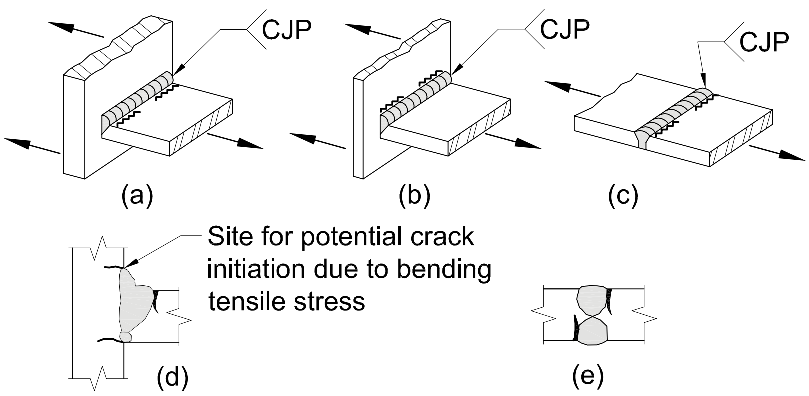
Illustrative Typical Examples
SECTION 5—WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS
5.1 
5.2 
5.3 
5.4 

TABLE A-3.1 (continued)
Fatigue Design Parameters

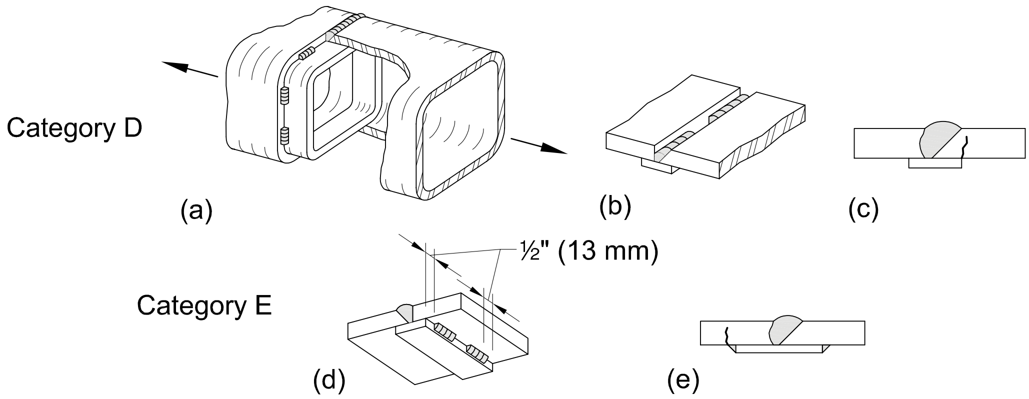
Description	Stress Category	Constant C_f	Threshold F_{TH}, ksi (MPa)	Potential Crack Initiation Point
SECTION 5—WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS				
5.5 Base metal and weld metal in or adjacent to transverse CJP groove welded butt splices with backing left in place				From the toe of the groove weld or the toe of the weld attaching backing when applicable
Tack welds inside groove	D	2.2	7 (48)	
Tack welds outside the groove and not closer than 1/2 in. (13 mm) to the edge of base metal	E	1.1	4.5 (31)	
5.6 Base metal and weld metal at transverse end connections of tension-loaded plate elements using PJP groove welds in butt, T- or corner-joints, with reinforcing or contouring fillets; F_{SR} shall be the smaller of the toe crack or root crack allowable stress range				
Crack initiating from weld toe	C	4.4	10 (69)	Initiating from weld toe extending into base metal
Crack initiating from weld root	C′	See Eq. A-3-3 or A-3-3M	None	Initiating at weld root extending into and through weld
5.7 Base metal and weld metal at transverse end connections of tension-loaded plate elements using a pair of fillet welds on opposite sides of the plate; F_{SR} shall be the smaller of the weld toe crack or weld root crack allowable stress range				
Crack initiating from weld toe	C	4.4	10 (69)	Initiating from weld toe extending into base metal
Crack initiating from weld root	C″	See Eq. A-3-5 or A-3-5M	None	Initiating at weld root extending into and through weld
5.8 Base metal of tension-loaded plate elements, and on built-up shapes and rolled beam webs or flanges at toe of transverse fillet welds adjacent to welded transverse stiffeners	C	4.4	10 (69)	From geometrical discontinuity at toe of fillet extending into base metal

TABLE A-3.1 (continued)
Fatigue Design Parameters

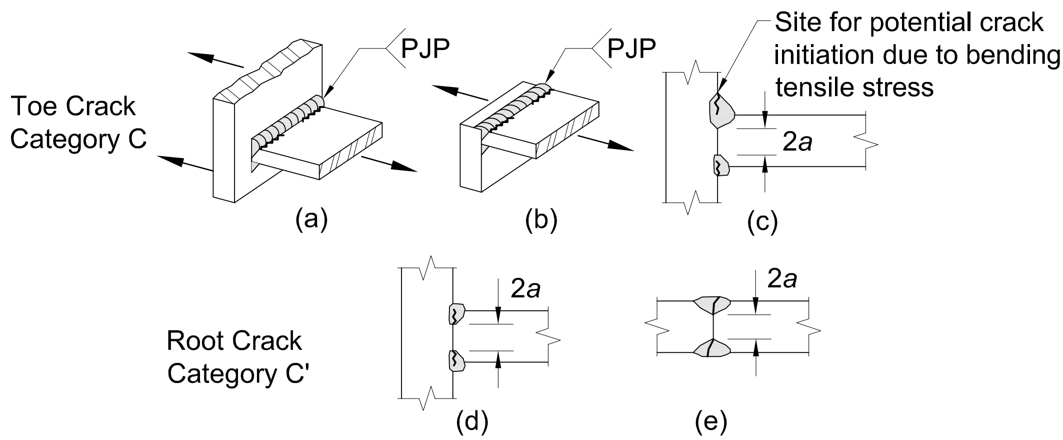
Illustrative Typical Examples

SECTION 5—WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS

5.5



5.6



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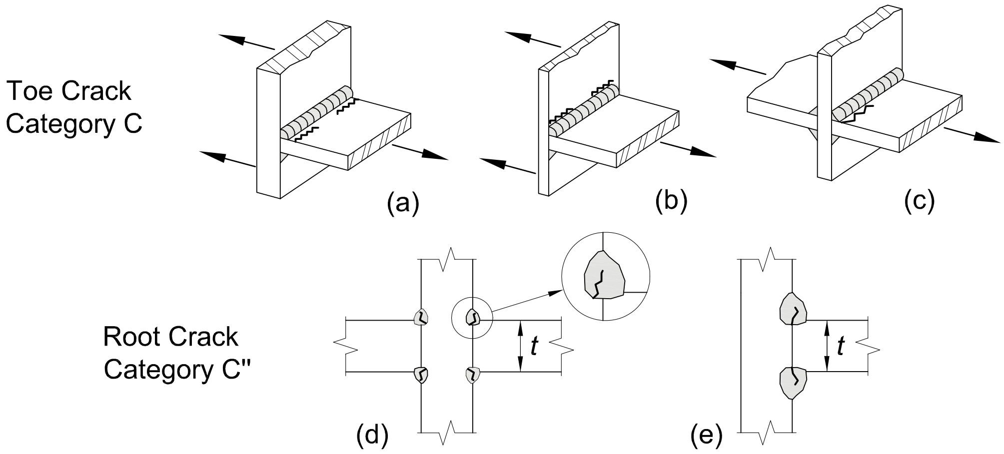

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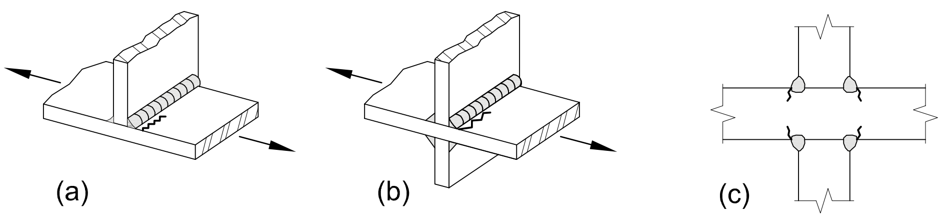


TABLE A-3.1 (continued)
Fatigue Design Parameters

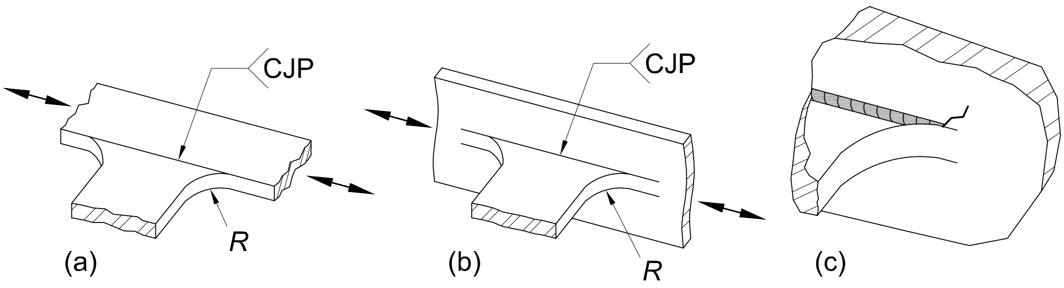
Description	Stress Category	Constant C_f	Threshold F_{TH}, ksi (MPa)	Potential Crack Initiation Point
SECTION 6—BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS				
6.1 Base metal of equal or unequal thickness at details attached by CJP groove welds subject to longitudinal loading only when the detail embodies a transition radius, *R*, with the weld termination ground smooth and inspected in accordance with Section 3.6				Near point of tangency of radius at edge of member
$R \geq$ 24 in. (600 mm)	B	12	16 (110)	
6 in. $\leq R <$ 24 in. (150 mm $\leq R <$ 600 mm)	C	4.4	10 (69)	
2 in. $\leq R <$ 6 in. (50 mm $\leq R <$ 150 mm)	D	2.2	7 (48)	
$R <$ 2 in. (50 mm)	E	1.1	4.5 (31)	
6.2 Base metal at details of equal thickness attached by CJP groove welds, subject to transverse loading, with or without longitudinal loading, when the detail embodies a transition radius, *R*, with the weld termination ground smooth and inspected in accordance with Section 3.6				
(a) When weld reinforcement is removed				Near point of tangency of radius or in the weld or at fusion boundary or member or attachment
$R \geq$ 24 in. (600 mm)	B	12	16 (110)	
6 in. $\leq R <$ 24 in. (150 mm $\leq R <$ 600 mm)	C	4.4	10 (69)	
2 in. $\leq R <$ 6 in. (50 mm $\leq R <$ 150 mm)	D	2.2	7 (48)	
$R <$ 2 in. (50 mm)	E	1.1	4.5 (31)	
(b) When weld reinforcement is not removed				At toe of the weld either along edge of member or the attachment
$R \geq$ 6 in. (150 mm)	C	4.4	10 (69)	
2 in. $\leq R <$ 6 in. (50 mm $\leq R <$ 150 mm)	D	2.2	7 (48)	
$R <$ 2 in. (50 mm)	E	1.1	4.5 (31)	

TABLE A-3.1 (continued)
Fatigue Design Parameters

Illustrative Typical Examples

SECTION 6—BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS

6.1



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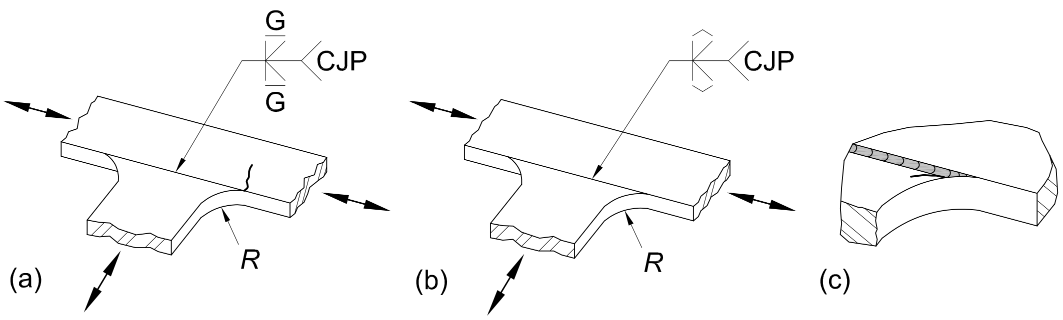


TABLE A-3.1 (continued) Fatigue Design Parameters

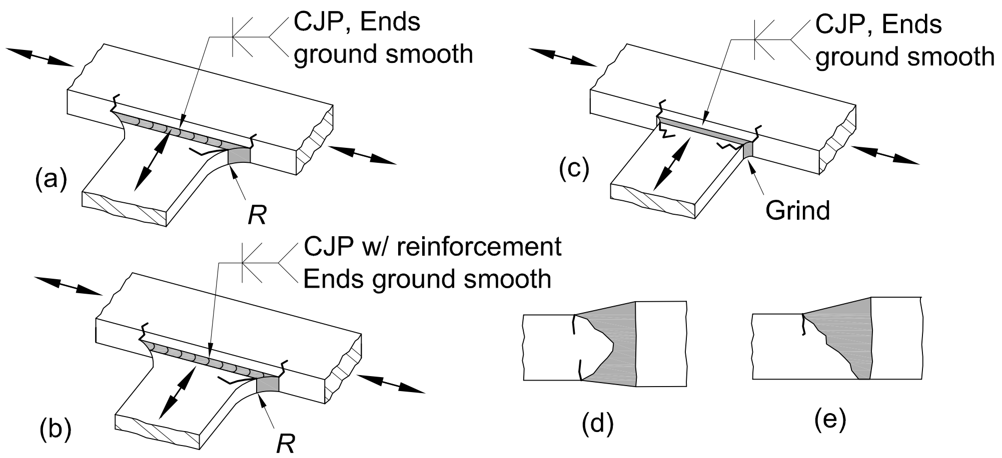
Description	Stress Category	Constant C_f	Threshold F_{TH}, ksi (MPa)	Potential Crack Initiation Point
SECTION 6—BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)				
6.3 Base metal at details of unequal thickness attached by CJP groove welds, subject to transverse loading, with or without longitudinal loading, when the detail embodies a transition radius, R, with the weld termination ground smooth and in accordance with Section 3.6				
(a) When weld reinforcement is removed				
R > 2 in. (50 mm)	D	2.2	7 (48)	At toe of weld along edge of thinner material
$R \leq$ 2 in. (50 mm)	E	1.1	4.5 (31)	In weld termination in small radius
(b) When reinforcement is not removed				
Any radius	E	1.1	4.5 (31)	At toe of weld along edge of thinner material
6.4 Base metal of equal or unequal thickness, subject to longitudinal stress at transverse members, with or without transverse stress, attached by fillet or PJP groove welds parallel to direction of stress when the detail embodies a transition radius, R, with weld termination ground smooth				Initiating in base metal at the weld termination or at the toe of the weld extending into the base metal
R > 2 in. (50 mm)	D	2.2	7 (48)	
$R \leq$ 2 in. (50 mm)	E	1.1	4.5 (31)	

TABLE A-3.1 (continued)
Fatigue Design Parameters

Illustrative Typical Examples

SECTION 6—BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)

6.3



6.4

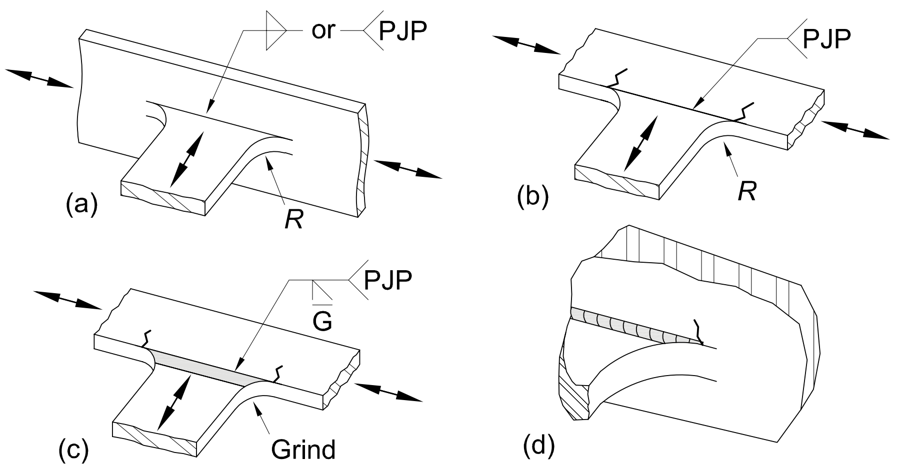


TABLE A-3.1 (continued)
Fatigue Design Parameters

Description	Stress Category	Constant C_f	Threshold F_{TH}, ksi (MPa)	Potential Crack Initiation Point
SECTION 7—BASE METAL AT SHORT ATTACHMENTS[a]				
7.1 Base metal subject to longitudinal loading at details with welds parallel or transverse to the direction of stress, with or without transverse load on the detail, where the detail embodies no transition radius, *R*, and with detail length, *a*, and thickness of the attachment, *b*:				Initiating in base metal at the weld termination or at the toe of the weld extending into the base metal
$a < 2$ in. (50 mm) for any thickness, *b*	C	4.4	10 (69)	
2 in. (50 mm) $\le a \le$ lesser of 12*b* or 4 in. (100 mm)	D	2.2	7 (48)	
$a >$ lesser of 12*b* or 4 in. (100 mm) when $b \le 0.8$ in. (20 mm)	E	1.1	4.5 (31)	
$a > 4$ in. (100 mm) when $b > 0.8$ in. (20 mm)	E′	0.39	2.6 (18)	
7.2 Base metal subject to longitudinal stress at details attached by fillet or PJP groove welds, with or without transverse load on detail, when the detail embodies a transition radius, *R*, with weld termination ground smooth:				Initiating in base metal at the weld termination, extending into the base metal
$R > 2$ in. (50 mm)	D	2.2	7 (48)	
$R \le 2$ in. (50 mm)	E	1.1	4.5 (31)	

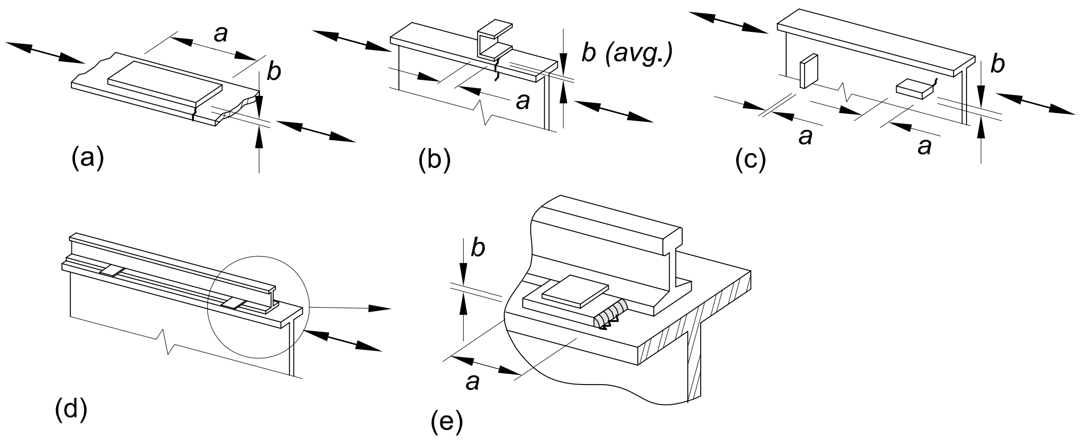
[a] "Attachment," as used herein, is defined as any steel detail welded to a member that causes a deviation in the stress flow in the member and, thus, reduces the fatigue resistance. The reduction is due to the presence of the attachment, not due to the loading on the attachment.

TABLE A-3.1 (continued)
Fatigue Design Parameters

Illustrative Typical Examples

SECTION 7—BASE METAL AT SHORT ATTACHMENTS[a]

7.1



7.2

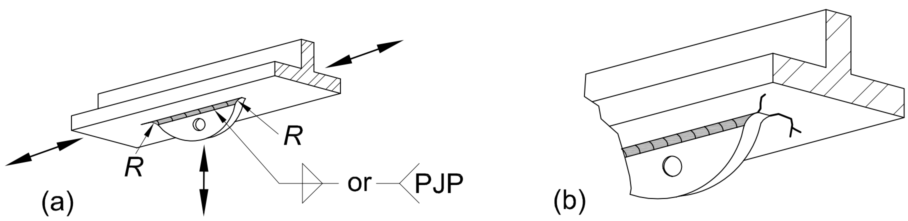
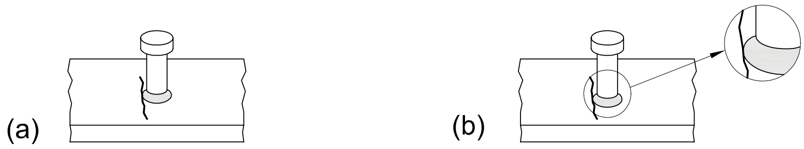
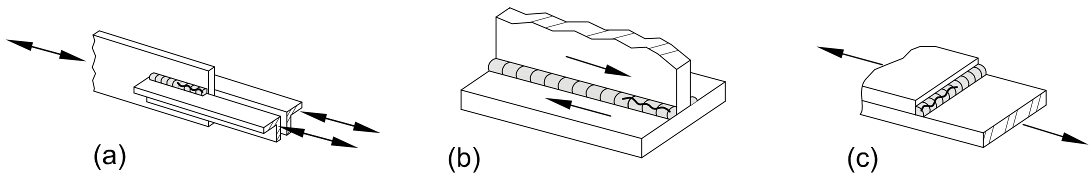
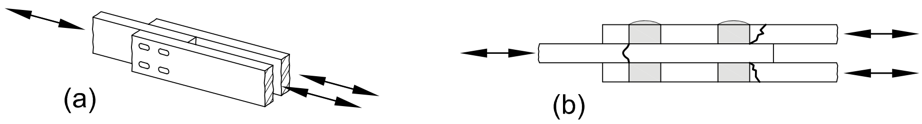
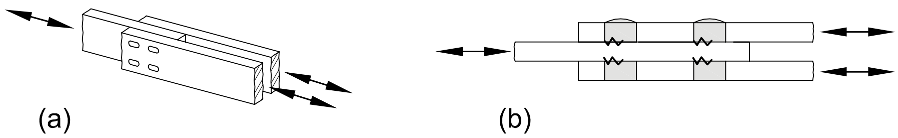
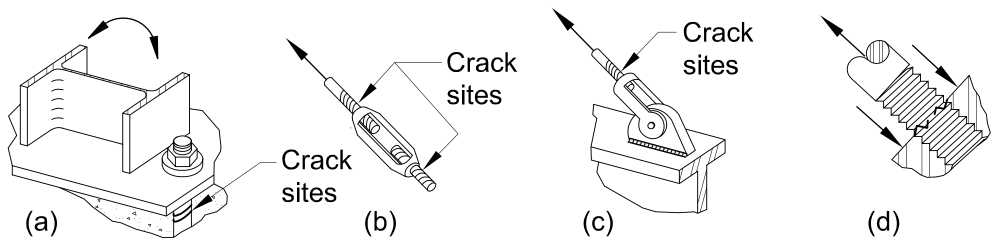


TABLE A-3.1 (continued)
Fatigue Design Parameters

Description	Stress Category	Constant C_f	Threshold F_{TH}, ksi (MPa)	Potential Crack Initiation Point
SECTION 8—MISCELLANEOUS				
8.1 Base metal at steel headed stud anchors attached by fillet weld or automatic stud welding	C	4.4	10 (69)	At toe of weld in base metal
8.2 Shear on throat of any fillet weld, continuous or intermittent, longitudinal or transverse	F	See Eq. A-3-2 or A-3-2M	See Eq. A-3-2 or A-3-2M	Initiating at the root of the fillet weld, extending into the weld
8.3 Base metal at plug or slot welds	E	1.1	4.5 (31)	Initiating in the base metal at the end of the plug or slot weld, extending into the base metal
8.4 Shear on plug or slot welds	F	See Eq. A-3-2 or A-3-2M	See Eq. A-3-2 or A-3-2M	Initiating in the weld at the faying surface, extending into the weld
8.5 High-strength bolts, common bolts, threaded anchor rods, and hanger rods, whether pretensioned in accordance with Table J3.1 or J3.1M, or snug-tightened with cut, ground or rolled threads; stress range on tensile stress area due to applied cyclic load plus prying action, when applicable	G	0.39	7 (48)	Initiating at the root of the threads, extending into the fastener

TABLE A-3.1 (continued)
Fatigue Design Parameters

Illustrative Typical Examples
SECTION 8—MISCELLANEOUS
8.1 
8.2 
8.3 
8.4 
8.5 

APPENDIX 4

STRUCTURAL DESIGN FOR FIRE CONDITIONS

This appendix provides criteria for the design and evaluation of structural steel components, systems and frames for fire conditions. These criteria provide for the determination of the heat input, thermal expansion and degradation in mechanical properties of materials at elevated temperatures that cause progressive decrease in strength and stiffness of structural components and systems at elevated temperatures.

User Note: Throughout this chapter, the term "elevated temperatures" refers to temperatures due to unintended fire exposure only.

The appendix is organized as follows:

4.1. General Provisions
4.2. Structural Design for Fire Conditions by Analysis
4.3. Design by Qualification Testing

4.1. GENERAL PROVISIONS

The methods contained in this appendix provide regulatory evidence of compliance in accordance with the design applications outlined in this section.

1. Performance Objective

Structural components, members, and building frame systems shall be designed so as to maintain their load-bearing function during the design-basis fire and to satisfy other performance requirements specified for the building occupancy.

Deformation criteria shall be applied where the means of providing structural fire resistance, or the design criteria for fire barriers, requires evaluation of the deformation of the load-carrying structure.

Within the compartment of fire origin, forces and deformations from the design-basis fire shall not cause a breach of horizontal or vertical compartmentation.

2. Design by Engineering Analysis

The analysis methods in Section 4.2 are permitted to be used to document the anticipated performance of steel framing when subjected to design-basis fire scenarios. Methods in Section 4.2 provide evidence of compliance with performance objectives established in Section 4.1.1.

The analysis methods in Section 4.2 are permitted to be used to demonstrate an equivalency for an alternative material or method, as permitted by the applicable building code (ABC).

Structural design for fire conditions using Appendix 4.2 shall be performed using the load and resistance factor design method in accordance with the provisions of Section B3.1 (LRFD).

3. Design by Qualification Testing

The qualification testing methods in Section 4.3 are permitted to be used to document the fire resistance of steel framing subject to the standardized fire testing protocols required by the ABC.

4. Load Combinations and Required Strength

In the absence of ABC provisions for design under fire exposures, the required strength of the structure and its elements shall be determined from the gravity load combination as follows:

$$(0.9 \text{ or } 1.2)\, D + A_T + 0.5L + 0.2S \qquad \text{(A-4-1)}$$

where

A_T = nominal forces and deformations due to the design-basis fire defined in Section 4.2.1

D = nominal dead load

L = nominal occupancy live load

S = nominal snow load

> **User Note:** ASCE/SEI 7 Section 2.5 contains this load combination for extraordinary events, which includes fire.

A notional load, $N_i = 0.002Y_i$, as defined in Section C2.2b, where N_i = notional load applied at framing level i and Y_i = gravity load from Equation A-4-1 acting on framing level i, shall be applied in combination with the loads stipulated in Equation A-4-1. Unless otherwise stipulated by the applicable building code, D, L and S shall be the nominal loads specified in ASCE/SEI 7.

> **User Note:** The effect of initial imperfections may be taken into account by direct modeling of imperfections in the analysis. In typical building structures, when evaluating frame stability, the important imperfection is the out-of-plumbness of columns.

4.2. STRUCTURAL DESIGN FOR FIRE CONDITIONS BY ANALYSIS

It is permitted to design structural members, components and building frames for elevated temperatures in accordance with the requirements of this section.

1. Design-Basis Fire

A design-basis fire shall be identified to describe the heating conditions for the structure. These heating conditions shall relate to the fuel commodities and compartment characteristics present in the assumed fire area. The fuel load density based on the occupancy of the space shall be considered when determining the total fuel load. Heating conditions shall be specified either in terms of a heat flux or temperature of the upper gas layer created by the fire. The variation of the heating conditions with time shall be determined for the duration of the fire.

The analysis methods in Section 4.2 shall be used in accordance with the provisions for alternative materials, designs and methods as permitted by the ABC. When the analysis methods in Section 4.2 are used to demonstrate equivalency to hourly ratings based on qualification testing in Section 4.3, the design-basis fire shall be permitted to be determined in accordance with ASTM E119.

1a. Localized Fire

Where the heat release rate from the fire is insufficient to cause flashover, a localized fire exposure shall be assumed. In such cases, the fuel composition, arrangement of the fuel array, and floor area occupied by the fuel shall be used to determine the radiant heat flux from the flame and smoke plume to the structure.

1b. Post-Flashover Compartment Fires

Where the heat release rate from the fire is sufficient to cause flashover, a post-flashover compartment fire shall be assumed. The determination of the temperature versus time profile resulting from the fire shall include fuel load, ventilation characteristics of the space (natural and mechanical), compartment dimensions, and thermal characteristics of the compartment boundary.

The fire duration in a particular area shall be determined from the total combustible mass, or fuel load in the space. In the case of either a localized fire or a post-flashover compartment fire, the fire duration shall be determined as the total combustible mass divided by the mass loss rate.

1c. Exterior Fires

The exposure effects of the exterior structure to flames projecting from windows or other wall openings as a result of a post-flashover compartment fire shall be addressed along with the radiation from the interior fire through the opening. The shape and length of the flame projection shall be used along with the distance between the flame and the exterior steelwork to determine the heat flux to the steel. The method identified in Section 4.2.1b shall be used for describing the characteristics of the interior compartment fire.

1d. Active Fire-Protection Systems

The effects of active fire-protection systems shall be addressed when describing the design-basis fire.

Where automatic smoke and heat vents are installed in nonsprinklered spaces, the resulting smoke temperature shall be determined from calculation.

2. Temperatures in Structural Systems under Fire Conditions

Temperatures within structural members, components and frames due to the heating conditions posed by the design-basis fire shall be determined by a heat transfer analysis.

3. Material Strengths at Elevated Temperatures

Material properties at elevated temperatures shall be determined from test data. In the absence of such data, it is permitted to use the material properties stipulated in this section. These relationships do not apply for steels with yield strengths in excess of 65 ksi (450 MPa) or concretes with specified compressive strength in excess of 8,000 psi (55 MPa).

3a. Thermal Elongation

The coefficients of expansion shall be taken as follows:

(a) For structural and reinforcing steels: For calculations at temperatures above 150°F (66°C), the coefficient of thermal expansion shall be 7.8×10^{-6}/°F (1.4×10^{-5}/°C).

(b) For normal weight concrete: For calculations at temperatures above 150°F (66°C), the coefficient of thermal expansion shall be 1.0×10^{-5}/°F (1.8×10^{-5}/°C).

(c) For lightweight concrete: For calculations at temperatures above 150°F (66°C), the coefficient of thermal expansion shall be 4.4×10^{-6}/°F (7.9×10^{-6}/°C).

3b. Mechanical Properties at Elevated Temperatures

The deterioration in strength and stiffness of structural members, components and systems shall be taken into account in the structural analysis of the frame.

(a) For steel, the values $F_y(T)$, $F_p(T)$, $F_u(T)$, $E(T)$ and $G(T)$ at elevated temperature to be used in structural analysis, expressed as the ratio with respect to the property at ambient, which is assumed to be 68°F (20°C), shall be defined as in Tables A-4.2.1. $F_p(T)$ is the proportional limit at elevated temperatures, which is calculated as a ratio to yield strength as specified in Table A-4.2.1. It is permitted to interpolate between these values.

(b) For concrete, the values $f'_c(T)$, $E_c(T)$ and $\varepsilon_{cu}(T)$ at elevated temperature to be used in structural analysis, expressed as the ratio with respect to the property at ambient, which is assumed to be 68°F (20°C), shall be defined as in Table A-4.2.2. It is permitted to interpolate between these values. For lightweight concrete, values of ε_{cu} shall be obtained from tests.

(c) For bolts, the values of $F_{nt}(T)$ and $F_{nv}(T)$ at elevated temperature to be used in structural analysis, expressed as the ratio with respect to the property at ambient, which is assumed to be 68°F (20°C), shall be defined as in Table A-4.2.3. It is permitted to interpolate between these values.

TABLE A-4.2.1
Properties of Steel at Elevated Temperatures

Steel Temperature, °F (°C)	$k_E = E(T)/E = G(T)/G$	$k_p = F_p(T)/F_y$	$k_y = F_y(T)/F_y$	$k_u = F_u(T)/F_y$
68 (20)	1.00	1.00	*	*
200 (93)	1.00	1.00	*	*
400 (200)	0.90	0.80	*	*
600 (320)	0.78	0.58	*	*
750 (400)	0.70	0.42	1.00	1.00
800 (430)	0.67	0.40	0.94	0.94
1000 (540)	0.49	0.29	0.66	0.66
1200 (650)	0.22	0.13	0.35	0.35
1400 (760)	0.11	0.06	0.16	0.16
1600 (870)	0.07	0.04	0.07	0.07
1800 (980)	0.05	0.03	0.04	0.04
2000 (1100)	0.02	0.01	0.02	0.02
2200 (1200)	0.00	0.00	0.00	0.00

*Use ambient properties

4. Structural Design Requirements

4a. General Structural Integrity

The structural frame and foundation shall be capable of providing the strength and deformation capacity to withstand, as a system, the structural actions developed during the fire within the prescribed limits of deformation. The structural system shall be designed to sustain local damage with the structural system as a whole remaining stable. Frame stability and required strength shall be determined in accordance with the requirements of Section C1.

Continuous load paths shall be provided to transfer all forces from the exposed region to the final point of resistance.

4b. Strength Requirements and Deformation Limits

Conformance of the structural system to these requirements shall be demonstrated by constructing a mathematical model of the structure based on principles of structural mechanics and evaluating this model for the internal forces and deformations in the members of the structure developed by the temperatures from the design-basis fire.

TABLE A-4.2.2
Properties of Concrete at Elevated Temperatures

Concrete Temperature, °F (°C)	$k_c = f'_c(T)/f'_c$		$E_c(T)/E_c$	$\varepsilon_{cu}(T)$, %
	Normal Weight Concrete	Lightweight Concrete		Normal Weight Concrete
68 (20)	1.00	1.00	1.00	0.25
200 (93)	0.95	1.00	0.93	0.34
400 (200)	0.90	1.00	0.75	0.46
550 (290)	0.86	1.00	0.61	0.58
600 (320)	0.83	0.98	0.57	0.62
800 (430)	0.71	0.85	0.38	0.80
1000 (540)	0.54	0.71	0.20	1.06
1200 (650)	0.38	0.58	0.092	1.32
1400 (760)	0.21	0.45	0.073	1.43
1600 (870)	0.10	0.31	0.055	1.49
1800 (980)	0.05	0.18	0.036	1.50
2000 (1100)	0.01	0.05	0.018	1.50
2200 (1200)	0.00	0.00	0.000	0.00

Individual members shall have the design strength necessary to resist the shears, axial forces and moments determined in accordance with these provisions.

Connections shall develop the strength of the connected members or the forces. Where the means of providing fire resistance requires the evaluation of deformation criteria, the deformation of the structural system, or members thereof, under the design-basis fire shall not exceed the prescribed limits.

It shall be permitted to include membrane action of composite floor slabs for fire resistance if the design provides for the effects of increased connection tensile forces and redistributed gravity load demands on the adjacent framing supports.

4c. Design by Advanced Methods of Analysis

Design by advanced methods of analysis is permitted for the design of all steel building structures for fire conditions. The design-basis fire exposure shall be that determined in Section 4.2.1. The analysis shall include both a thermal response and the mechanical response to the design-basis fire.

The thermal response shall produce a temperature field in each structural element as a result of the design-basis fire and shall incorporate temperature-dependent thermal properties of the structural elements and fire-resistive materials, as per Section 4.2.2.

TABLE A-4.2.3
Properties of Group A and Group B High-Strength Bolts at Elevated Temperatures

Bolt Temperature, °F (°C)	$F_{nt}(T)/F_{nt}$ or $F_{nv}(T)/F_{nv}$
68 (20)	1.00
200 (93)	0.97
300 (150)	0.95
400 (200)	0.93
600 (320)	0.88
800 (430)	0.71
900 (480)	0.59
1000 (540)	0.42
1200 (650)	0.16
1400 (760)	0.08
1600 (870)	0.04
1800 (980)	0.01
2000 (1100)	0.00

The mechanical response results in forces and deformations in the structural system subjected to the thermal response calculated from the design-basis fire. The mechanical response shall take into account explicitly the deterioration in strength and stiffness with increasing temperature, the effects of thermal expansions, inelastic behavior and load redistribution, large deformations, time-dependent effects such as creep, and uncertainties resulting from variability in material properties at elevated temperature. Boundary conditions and connection fixity must represent the proposed structural design. Material properties shall be defined as per Section 4.2.3.

The resulting analysis shall address all relevant limit states, such as excessive deflections, connection ruptures, and overall or local buckling.

4d. Design by Simple Methods of Analysis

The methods of analysis in this section are permitted to be used for the evaluation of the performance of individual members at elevated temperatures during exposure to fire.

The support and restraint conditions (forces, moments and boundary conditions) applicable at normal temperatures are permitted to be assumed to remain unchanged throughout the fire exposure.

It is permitted to model the thermal response of steel and composite members using a one-dimensional heat transfer equation with heat input as determined by the design-basis fire defined in Section 4.2.1, using the temperature equal to the maximum steel temperature. For flexural members, the maximum steel temperature shall be assigned to the bottom flange.

For steel temperatures less than or equal to 400°F (200°C), the member and connection design strengths shall be determined without consideration of temperature effects.

The design strength shall be determined as in Section B3.1. The nominal strength, R_n, shall be calculated using material properties, as provided in Section 4.2.3b, at the temperature developed by the design-basis fire and as stipulated in Sections 4.2.4d(a) through (f).

> **User Note:** At temperatures below 400°F (200°C), the reduction in steel properties need not be considered in calculating member strengths for the simple method of analysis; however, forces and deformations induced by elevated temperatures must be considered.

(a) Design for Tension
Nominal strength for tension shall be determined using the provisions of Chapter D, with steel properties as stipulated in Section 4.2.3b and assuming a uniform temperature over the cross section using the temperature equal to the maximum steel temperature.

(b) Design for Compression
The nominal strength for compression shall be determined using the provisions of Chapter E with steel properties as stipulated in Section 4.2.3b and Equation A-4-2 used in lieu of Equations E3-2 and E3-3 to calculate the nominal compressive strength for flexural buckling:

$$F_{cr}(T) = \left[0.42^{\sqrt{\frac{F_y(T)}{F_e(T)}}}\right] F_y(T) \qquad \text{(A-4-2)}$$

where $F_y(T)$ is the yield stress at elevated temperature and $F_e(T)$ is the critical elastic buckling stress calculated from Equation E3-4 with the elastic modulus, $E(T)$, at elevated temperature. $F_y(T)$ and $E(T)$ are obtained using coefficients from Table A-4.2.1.

> **User Note:** For most fire conditions, uniform heating and temperatures govern the design for compression. A method to account for the effects of nonuniform heating and resulting thermal gradients on the design strength of compression members is referenced in the Commentary. The strength of leaning (gravity) columns may be increased by rotational restraints from cooler columns in the stories above and below the story exposed to the fire. A method to account for the beneficial influence of rotational restraints is discussed in the Commentary.

(c) Design for Flexure

For steel beams, it is permitted to assume that the calculated bottom flange temperature is constant over the depth of the member.

Nominal strength for flexure shall be determined using the provisions of Chapter F with steel properties as stipulated in Section 4.2.3b and Equations A-4-3 through A-4-10 used in lieu of Equations F2-2 through F2-6 to calculate the nominal flexural strength for lateral-torsional buckling of laterally unbraced doubly symmetric members:

(1) When $L_b \le L_r(T)$

$$M_n(T) = C_b\left\{M_r(T) + \left[M_p(T) - M_r(T)\right]\left[1 - \frac{L_b}{L_r(T)}\right]^{c_x}\right\} \le M_p(T) \quad \text{(A-4-3)}$$

(2) When $L_b > L_r(T)$

$$M_n(T) = F_{cr}(T)S_x \le M_p(T) \quad \text{(A-4-4)}$$

where

$$F_{cr}(T) = \frac{C_b \pi^2 E(T)}{\left(\dfrac{L_b}{r_{ts}}\right)^2}\sqrt{1 + 0.078\frac{Jc}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2} \quad \text{(A-4-5)}$$

$$L_r(T) = 1.95 r_{ts}\frac{E(T)}{F_L(T)}\sqrt{\frac{Jc}{S_x h_o} + \sqrt{\left(\frac{Jc}{S_x h_o}\right)^2 + 6.76\left[\frac{F_L(T)}{E(T)}\right]^2}} \quad \text{(A-4-6)}$$

$$M_r(T) = F_L(T)S_x \quad \text{(A-4-7)}$$

$$F_L(T) = F_y(k_p - 0.3k_y) \quad \text{(A-4-8)}$$

$$M_p(T) = F_y(T)Z_x \quad \text{(A-4-9)}$$

$$c_x = 0.53 + \frac{T}{450} \le 3.0 \text{ where } T \text{ is in °F} \quad \text{(A-4-10)}$$

$$c_x = 0.6 + \frac{T}{250} \le 3.0 \text{ where } T \text{ is in °C} \quad \text{(A-4-10M)}$$

and

T = elevated temperature of steel due to unintended fire exposure, °F (°C)

The material properties at elevated temperatures, $E(T)$ and $F_y(T)$, and the k_p and k_y coefficients are calculated in accordance with Table A-4.2.1, and other terms are as defined in Chapter F.

(d) Design for Flexure in Composite Beams

For composite beams, the calculated bottom flange temperature shall be taken as constant between the bottom flange and mid-depth of the web and shall decrease linearly by no more than 25% from the mid-depth of the web to the top flange of the beam.

The nominal strength of a composite flexural member shall be determined using the provisions of Chapter I, with reduced yield stresses in the steel consistent with the temperature variation described under thermal response.

Alternatively, the nominal flexural strength of a composite beam, $M_n(T)$, is permitted to be calculated using the bottom flange temperature, T, as follows:

$$M_n(T) = r(T)M_n \tag{A-4-11}$$

where

M_n = nominal flexural strength at ambient temperature calculated in accordance with provisions of Chapter I, kip-in. (N-mm)

$r(T)$ = retention factor depending on bottom flange temperature, T, as given in Table A-4.2.4

(e) Design for Shear

Nominal strength for shear shall be determined in accordance with the provisions of Chapter G, with steel properties as stipulated in Section 4.2.3b and assuming a uniform temperature over the cross section.

(f) Design for Combined Forces and Torsion

Nominal strength for combinations of axial force and flexure about one or both axes, with or without torsion, shall be in accordance with the provisions of Chapter H with the design axial and flexural strengths as stipulated in Sections 4.2.4d(a) to (d). Nominal strength for torsion shall be determined in accordance with the provisions of Chapter H, with the steel properties as stipulated in Section 4.2.3b, assuming uniform temperature over the cross section.

4.3. DESIGN BY QUALIFICATION TESTING

1. Qualification Standards

Structural members and components in steel buildings shall be qualified for the rating period in conformance with ASTM E119. Demonstration of compliance with these requirements using the procedures specified for steel construction in Section 5 of *Standard Calculation Methods for Structural Fire Protection* (ASCE/SEI/SFPE 29) is permitted.

2. Restrained Construction

For floor and roof assemblies and individual beams in buildings, a restrained condition exists when the surrounding or supporting structure is capable of resisting forces and accommodating deformations caused by thermal expansion throughout the range of anticipated elevated temperatures.

Steel beams, girders and frames supporting concrete slabs that are welded or bolted to integral framing members shall be considered restrained construction.

TABLE A-4.2.4
Retention Factor for Composite Flexural Members

Bottom Flange Temperature, °F (°C)	$r(T)$
68 (20)	1.00
300 (150)	0.98
600 (320)	0.95
800 (430)	0.89
1000 (540)	0.71
1200 (650)	0.49
1400 (760)	0.26
1600 (870)	0.12
1800 (980)	0.05
2000 (1100)	0.00

3. Unrestrained Construction

Steel beams, girders and frames that do not support a concrete slab shall be considered unrestrained unless the members are bolted or welded to surrounding construction that has been specifically designed and detailed to resist effects of elevated temperatures.

A steel member bearing on a wall in a single span or at the end span of multiple spans shall be considered unrestrained unless the wall has been designed and detailed to resist effects of thermal expansion.

APPENDIX 5

EVALUATION OF EXISTING STRUCTURES

This appendix applies to the evaluation of the strength and stiffness under static loads of existing structures by structural analysis, by load tests, or by a combination of structural analysis and load tests when specified by the engineer of record or in the contract documents. For such evaluation, the steel grades are not limited to those listed in Section A3.1. This appendix does not address load testing for the effects of seismic loads or moving loads (vibrations). Section 5.4 is only applicable to static vertical gravity loads applied to existing roofs or floors.

The Appendix is organized as follows:

5.1. GENERAL PROVISIONS

These provisions shall be applicable when the evaluation of an existing steel structure is specified for (a) verification of a specific set of design loadings or (b) determination of the available strength of a load-resisting member or system. The evaluation shall be performed by structural analysis (Section 5.3), by load tests (Section 5.4), or by a combination of structural analysis and load tests, when specified in the contract documents by the engineer of record (EOR). Where load tests are used, the EOR shall first analyze the structure, prepare a testing plan, and develop a written procedure for the test. The plan shall consider catastrophic collapse and/or excessive levels of permanent deformation, as defined by the EOR, and shall include procedures to preclude either occurrence during testing.

5.2. MATERIAL PROPERTIES

1. Determination of Required Tests

The EOR shall determine the specific tests that are required from Sections 5.2.2 through 5.2.6 and specify the locations where they are required. Where available, the use of applicable project records is permitted to reduce or eliminate the need for testing.

2. Tensile Properties

Tensile properties of members shall be considered in evaluation by structural analysis (Section 5.3) or load tests (Section 5.4). Such properties shall include the yield stress, tensile strength and percent elongation. Where available, certified material test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM A6/A6M or A568/A568M, as applicable, is permitted for this purpose. Otherwise, tensile tests shall be conducted in accordance with ASTM A370 from samples taken from components of the structure.

3. Chemical Composition

Where welding is anticipated for repair or modification of existing structures, the chemical composition of the steel shall be determined for use in preparing a welding procedure specification. Where available, results from certified material test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM procedures is permitted for this purpose. Otherwise, analyses shall be conducted in accordance with ASTM A751 from the samples used to determine tensile properties or from samples taken from the same locations.

4. Base Metal Notch Toughness

Where welded tension splices in heavy shapes and plates as defined in Section A3.1d are critical to the performance of the structure, the Charpy V-notch toughness shall be determined in accordance with the provisions of Section A3.1d. If the notch toughness so determined does not meet the provisions of Section A3.1d, the EOR shall determine if remedial actions are required.

5. Weld Metal

Where structural performance is dependent on existing welded connections, representative samples of weld metal shall be obtained. Chemical analysis and mechanical tests shall be made to characterize the weld metal. A determination shall be made of the magnitude and consequences of imperfections. If the requirements of *Structural Welding Code—Steel*, AWS D1.1/D1.1M, are not met, the EOR shall determine if remedial actions are required.

6. Bolts and Rivets

Representative samples of bolts shall be inspected to determine markings and classifications. Where bolts cannot be properly identified, representative samples shall be taken and tested to determine tensile strength in accordance with ASTM F606/F606M and the bolt classified accordingly. Alternatively, the assumption that the bolts are ASTM A307 is permitted. Rivets shall be assumed to be ASTM A502 Grade 1 unless a higher grade is established through documentation or testing.

5.3. EVALUATION BY STRUCTURAL ANALYSIS

1. Dimensional Data

All dimensions used in the evaluation, such as spans, column heights, member spacings, bracing locations, cross-section dimensions, thicknesses, and connection details, shall be determined from a field survey. Alternatively, when available, it is permitted to determine such dimensions from applicable project design or shop drawings with field verification of critical values.

2. Strength Evaluation

Forces (load effects) in members and connections shall be determined by structural analysis applicable to the type of structure evaluated. The load effects shall be determined for the loads and factored load combinations stipulated in Section B2.

The available strength of members and connections shall be determined from applicable provisions of Chapters B through K of this Specification.

3. Serviceability Evaluation

Where required, the deformations at service loads shall be calculated and reported.

5.4. EVALUATION BY LOAD TESTS

1. Determination of Load Rating by Testing

To determine the load rating of an existing floor or roof structure by testing, a test load shall be applied incrementally in accordance with the EOR's plan. The structure shall be visually inspected for signs of distress or imminent failure at each load level. Appropriate measures shall be taken if these or any other unusual conditions are encountered.

The tested strength of the structure shall be taken as the maximum applied test load plus the in-situ dead load. The live load rating of a floor structure shall be determined by setting the tested strength equal to $1.2D + 1.6L$, where D is the nominal dead load and L is the nominal live load rating for the structure. For roof structures, L_r, S or R shall be substituted for L,

where

L_r = nominal roof live load

R = nominal load due to rainwater or snow, exclusive of the ponding contribution

S = nominal snow load

More severe load combinations shall be used where required by the applicable building codes.

Periodic unloading shall be considered once the service load level is attained and after the onset of inelastic structural behavior is identified to document the amount of permanent set and the magnitude of the inelastic deformations. Deformations of the structure, such as member deflections, shall be monitored at critical locations during the test, referenced to the initial position before loading. It shall be demonstrated, while maintaining maximum test load for one hour, that the deformation of the structure does not increase by more than 10% above that at the beginning of the holding period. It is permissible to repeat the sequence if necessary to demonstrate compliance.

Deformations of the structure shall also be recorded 24 hours after the test loading is removed to determine the amount of permanent set. Because the amount of acceptable permanent deformation depends on the specific structure, no limit is specified for permanent deformation at maximum loading. Where it is not feasible to load test the entire structure, a segment or zone of not less than one complete bay representative of the most critical conditions shall be selected.

2. Serviceability Evaluation

When load tests are prescribed, the structure shall be loaded incrementally to the service load level. The service test load shall be held for a period of one hour, and deformations shall be recorded at the beginning and at the end of the one-hour holding period.

5.5. EVALUATION REPORT

After the evaluation of an existing structure has been completed, the EOR shall prepare a report documenting the evaluation. The report shall indicate whether the evaluation was performed by structural analysis, by load testing, or by a combination of structural analysis and load testing. Furthermore, when testing is performed, the report shall include the loads and load combination used and the load-deformation and time-deformation relationships observed. All relevant information obtained from design drawings, material test reports, and auxiliary material testing shall also be reported. Finally, the report shall indicate whether the structure, including all members and connections, is adequate to withstand the load effects.

APPENDIX 6

MEMBER STABILITY BRACING

This appendix addresses the minimum strength and stiffness necessary to provide a braced point in a column, beam or beam-column.

The appendix is organized as follows:

6.1. General Provisions
6.2. Column Bracing
6.3. Beam Bracing
6.4. Beam-Column Bracing

User Note: Stability requirements for lateral force-resisting systems are provided in Chapter C. The provisions in this appendix apply to bracing that is not generally included in the analysis model of the overall structure, but is provided to stabilize individual columns, beams and beam-columns. Guidance for applying these provisions to stabilize trusses is provided in the Commentary.

6.1. GENERAL PROVISIONS

Bracing systems shall have the strength and stiffness specified in this Appendix, as applicable. Where such a system braces more than one member, the strength and stiffness of the bracing shall be based on the sum of the required strengths of all members being braced. The evaluation of the stiffness furnished by the bracing shall include the effects of connections and anchoring details.

User Note: More detailed analyses for bracing strength and stiffness are presented in the Commentary.

A panel brace (formerly referred to as a relative brace) controls the angular deviation of a segment of the braced member between braced points (that is, the lateral displacement of one end of the segment relative to the other). A point brace (formerly referred to as a nodal brace) controls the movement at the braced point without direct interaction with adjacent braced points. A continuous bracing system consists of bracing that is attached along the entire member length.

The available strength and stiffness of the bracing members and connections shall equal or exceed the required strength and stiffness, respectively, unless analysis indicates that smaller values are justified.

Columns, beams and beam-columns with end and intermediate braced points designed to meet the requirements in Sections 6.2, 6.3 and 6.4, as applicable, are permitted to be designed based on lengths L_c and L_b, as defined in Chapters E and F, taken equal to the distance between the braced points.

In lieu of the requirements of Sections 6.2, 6.3 and 6.4,

(a) The required brace strength and stiffness can be obtained using a second-order analysis that satisfies the provisions of Chapter C or Appendix 1, as appropriate, and includes brace points displaced from their nominal locations in a pattern that provides for the greatest demand on the bracing.

(b) The required bracing stiffness can be obtained as $2/\phi$ (LRFD) or 2Ω (ASD) times the ideal bracing stiffness determined from a buckling analysis. The required brace strength can be determined using the provisions of Sections 6.2, 6.3 and 6.4, as applicable.

(c) For either of the above analysis methods, members with end or intermediate braced points meeting these requirements may be designed based on effective lengths, L_c and L_b, taken less than the distance between braced points.

User Note: The stability bracing requirements in Sections 6.2, 6.3 and 6.4 are based on buckling analysis models involving idealizations of common bracing conditions. Computational analysis methods may be used for greater generality, accuracy and efficiency for more complex bracing conditions. The Commentary to Section 6.1 provides guidance on these considerations.

6.2. COLUMN BRACING

It is permitted to laterally brace an individual column at end and intermediate points along its length using either panel or point bracing.

User Note: This section provides requirements only for lateral bracing. Column lateral bracing is assumed to be located at the shear center of the column. When lateral bracing does not prevent twist, the column is susceptible to torsional buckling, as addressed in Section E4. When the lateral bracing is offset from the shear center, the column is susceptible to constrained-axis torsional buckling, which is addressed in the commentary to Section E4.

1. Panel Bracing

The panel bracing system shall have the strength and stiffness specified in this section. The connection of the bracing system to the column shall have the strength specified in Section 6.2.2 for a point brace at that location.

User Note: If the stiffness of the connection to the panel bracing system is comparable to the stiffness of the panel bracing system itself, the panel bracing system and its connection to the column function as a panel and point bracing system arranged in series. Such cases may be evaluated using the alternative analysis methods listed in Section 6.1.

In the direction perpendicular to the longitudinal axis of the column, the required shear strength of the bracing system is:

$$V_{br} = 0.005P_r \tag{A-6-1}$$

and, the required shear stiffness of the bracing system is:

$$\beta_{br} = \frac{1}{\phi}\left(\frac{2P_r}{L_{br}}\right) \text{ (LRFD)} \tag{A-6-2a}$$

$$\beta_{br} = \Omega\left(\frac{2P_r}{L_{br}}\right) \text{ (ASD)} \tag{A-6-2b}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

where

L_{br} = unbraced length within the panel under consideration, in. (mm)

P_r = required axial strength of the column within the panel under consideration, using LRFD or ASD load combinations, kips (N)

2. Point Bracing

In the direction perpendicular to the longitudinal axis of the column, the required strength of end and intermediate point braces is

$$P_{br} = 0.01P_r \tag{A-6-3}$$

and, the required stiffness of the brace is

$$\beta_{br} = \frac{1}{\phi}\left(\frac{8P_r}{L_{br}}\right) \text{ (LRFD)} \tag{A-6-4a}$$

$$\beta_{br} = \Omega\left(\frac{8P_r}{L_{br}}\right) \text{ (ASD)} \tag{A-6-4b}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

where

L_{br} = unbraced length adjacent to the point brace, in. (mm)

P_r = largest of the required axial strengths of the column within the unbraced lengths adjacent to the point brace using LRFD or ASD load combinations, kips (N)

When the unbraced lengths adjacent to a point brace have different P_r / L_{br} values, the larger value shall be used to determine the required brace stiffness.

For intermediate point bracing of an individual column, L_{br} in Equations A-6-4a or A-6-4b need not be taken less than the maximum effective length, L_c, permitted for the column based upon the required axial strength, P_r.

6.3. BEAM BRACING

Beams shall be restrained against rotation about their longitudinal axis at points of support. When a braced point is assumed in the design between points of support, lateral bracing, torsional bracing, or a combination of the two shall be provided to prevent the relative displacement of the top and bottom flanges (i.e., to prevent twist). In members subject to double curvature bending, the inflection point shall not be considered a braced point unless bracing is provided at that location.

The requirements of this section shall apply to bracing of doubly and singly symmetric I-shaped members subjected to flexure within a plane of symmetry and zero net axial force.

1. Lateral Bracing

Lateral bracing shall be attached at or near the beam compression flange, except as follows:

(a) At the free end of a cantilevered beam, lateral bracing shall be attached at or near the top (tension) flange.

(b) For braced beams subject to double curvature bending, bracing shall be attached at or near both flanges at the braced point nearest the inflection point.

It is permitted to use either panel or point bracing to provide lateral bracing for beams.

1a. Panel Bracing

The panel bracing system shall have the strength and stiffness specified in this section. The connection of the bracing system to the member shall have the strength specified in Section 6.3.1b for a point brace at that location.

User Note: The stiffness contribution of the connection to the panel bracing system should be assessed as provided in the User Note to Section 6.2.1.

The required shear strength of the bracing system is

$$V_{br} = 0.01\left(\frac{M_r C_d}{h_o}\right) \tag{A-6-5}$$

and, the required shear stiffness of the bracing system is

$$\beta_{br} = \frac{1}{\phi}\left(\frac{4M_r C_d}{L_{br} h_o}\right) \quad \text{(LRFD)} \tag{A-6-6a}$$

$$\beta_{br} = \Omega\left(\frac{4M_r C_d}{L_{br} h_o}\right) \quad \text{(ASD)} \tag{A-6-6b}$$

$\phi = 0.75$ (LRFD) $\quad \Omega = 2.00$ (ASD)

where

C_d = 1.0, except in the following case:
= 2.0 for the brace closest to the inflection point in a beam subject to double curvature bending

L_{br} = unbraced length within the panel under consideration, in. (mm)

M_r = required flexural strength of the beam within the panel under consideration, using LRFD or ASD load combinations, kip-in. (N-mm)

h_o = distance between flange centroids, in. (mm)

1b. Point Bracing

In the direction perpendicular to the longitudinal axis of the beam, the required strength of end and intermediate point braces is

$$P_{br} = 0.02\left(\frac{M_r C_d}{h_o}\right) \quad \text{(A-6-7)}$$

and, the required stiffness of the brace is

$$\beta_{br} = \frac{1}{\phi}\left(\frac{10 M_r C_d}{L_{br} h_o}\right) \text{ (LRFD)} \quad \text{(A-6-8a)}$$

$$\beta_{br} = \Omega\left(\frac{10 M_r C_d}{L_{br} h_o}\right) \text{ (ASD)} \quad \text{(A-6-8b)}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

where

L_{br} = unbraced length adjacent to the point brace, in. (mm)

M_r = largest of the required flexural strengths of the beam within the unbraced lengths adjacent to the point brace using LRFD or ASD load combinations, kip-in. (N-mm)

When the unbraced lengths adjacent to a point brace have different M_r / L_{br} values, the larger value shall be used to determine the required brace stiffness.

For intermediate point bracing of an individual beam, L_{br} in Equations A-6-8a or A-6-8b need not be taken less than the maximum effective length, L_b, permitted for the beam based upon the required flexural strength, M_r.

2. Torsional Bracing

It is permitted to attach torsional bracing at any cross-section location, and it need not be attached near the compression flange.

> **User Note:** Torsional bracing can be provided as point bracing, such as cross-frames, moment-connected beams or vertical diaphragm elements, or as continuous bracing, such as slabs or decks.

2a. Point Bracing

About the longitudinal axis of the beam, the required flexural strength of the brace is:

$$M_{br} = 0.02M_r \tag{A-6-9}$$

and, the required flexural stiffness of the brace is:

$$\beta_{br} = \frac{\beta_T}{\left(1 - \dfrac{\beta_T}{\beta_{sec}}\right)} \tag{A-6-10}$$

where

$$\beta_T = \frac{1}{\phi}\frac{2.4L}{nEI_{yeff}}\left(\frac{M_r}{C_b}\right)^2 \quad \text{(LRFD)} \tag{A-6-11a}$$

$$\beta_T = \Omega\frac{2.4L}{nEI_{yeff}}\left(\frac{M_r}{C_b}\right)^2 \quad \text{(ASD)} \tag{A-6-11b}$$

$$\beta_{sec} = \frac{3.3E}{h_o}\left(\frac{1.5h_o t_w^3}{12} + \frac{t_{st} b_s^3}{12}\right) \tag{A-6-12}$$

and

$\phi = 0.75$ (LRFD); $\Omega = 3.00$ (ASD)

> **User Note:** $\Omega = 1.5^2/\phi = 3.00$ in Equations A-6-11a or A-6-11b, because the moment term is squared.
>
> β_{sec} can be taken equal to infinity, and $\beta_{br} = \beta_T$, when a cross-frame is attached near both flanges or a vertical diaphragm element is used that is approximately the same depth as the beam being braced.

E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)
I_{yeff} = effective out-of-plane moment of inertia, in.^{4} (mm^4)
= $I_{yc} + (t/c)I_{yt}$
I_{yc} = moment of inertia of the compression flange about the y-axis, in.^{4} (mm^4)
I_{yt} = moment of inertia of the tension flange about the y-axis, in.^{4} (mm^4)
L = length of span, in. (mm)
M_r = largest of the required flexural strengths of the beam within the unbraced lengths adjacent to the point brace, using LRFD or ASD load combinations, kip-in. (N-mm)
$\dfrac{M_r}{C_b}$ = maximum value of the required flexural strength of the beam divided by the moment gradient factor, within the unbraced lengths adjacent to the point brace, using LRFD or ASD load combinations, kip-in. (N-mm)
b_s = stiffener width for one-sided stiffeners, in. (mm)
= twice the individual stiffener width for pairs of stiffeners, in. (mm)

c = distance from the neutral axis to the extreme compressive fibers, in. (mm)
n = number of braced points within the span
t = distance from the neutral axis to the extreme tensile fibers, in. (mm)
t_w = thickness of beam web, in. (mm)
t_{st} = thickness of web stiffener, in. (mm)
β_T = overall brace system required stiffness, kip-in./rad (N-mm/rad)
β_{sec} = web distortional stiffness, including the effect of web transverse stiffeners, if any, kip-in./rad (N-mm/rad)

User Note: If $\beta_{sec} < \beta_T$, Equation A-6-10 is negative, which indicates that torsional beam bracing will not be effective due to inadequate web distortional stiffness.

User Note: For doubly symmetric members, $c = t$ and I_{yeff} = out-of-plane moment of inertia, I_y, in.^{4} (mm^4).

When required, a web stiffener shall extend the full depth of the braced member and shall be attached to the flange if the torsional brace is also attached to the flange. Alternatively, it is permissible to stop the stiffener short by a distance equal to $4t_w$ from any beam flange that is not directly attached to the torsional brace.

2b. Continuous Bracing

For continuous torsional bracing:

(a) The brace strength requirement per unit length along the beam shall be taken as Equation A-6-9 divided by the maximum unbraced length permitted for the beam based upon the required flexural strength, M_r. The required flexural strength, M_r, shall be taken as the maximum value throughout the beam span.

(b) The brace stiffness requirement per unit length shall be given by Equations A-6-10 and A-6-11 with $L/n = 1.0$.

(c) The web distortional stiffness shall be taken as:

$$\beta_{sec} = \frac{3.3Et_w^3}{12h_o} \tag{A-6-13}$$

6.4. BEAM-COLUMN BRACING

For bracing of beam-columns, the required strength and stiffness for the axial force shall be determined as specified in Section 6.2, and the required strength and stiffness for flexure shall be determined as specified in Section 6.3. The values so determined shall be combined as follows:

(a) When panel bracing is used, the required strength shall be taken as the sum of the values determined using Equations A-6-1 and A-6-5, and the required stiffness shall be taken as the sum of the values determined using Equations A-6-2 and A-6-6.

(b) When point bracing is used, the required strength shall be taken as the sum of the values determined using Equations A-6-3 and A-6-7, and the required stiffness shall be taken as the sum of the values determined using Equations A-6-4 and A-6-8. In Equations A-6-4 and A-6-8, L_{br} for beam-columns shall be taken as the actual unbraced length; the provisions in Sections 6.2.2 and 6.3.1b, that L_{br} need not be taken less than the maximum permitted effective length based upon P_r and M_r, shall not be applied.

(c) When torsional bracing is provided for flexure in combination with panel or point bracing for the axial force, the required strength and stiffness shall be combined or distributed in a manner that is consistent with the resistance provided by the element(s) of the actual bracing details.

(d) When the combined stress effect from axial force and flexure results in compression to both flanges, either lateral bracing shall be added to both flanges or both flanges shall be laterally restrained by a combination of lateral and torsional bracing.

User Note: For case (d), additional guidelines are provided in the Commentary.

APPENDIX 7

ALTERNATIVE METHODS OF DESIGN FOR STABILITY

This appendix presents alternatives to the direct analysis method of design for stability defined in Chapter C. The two alternative methods covered are the effective length method and the first-order analysis method.

The appendix is organized as follows:

7.1. General Stability Requirements
7.2. Effective Length Method
7.3. First-Order Analysis Method

7.1. GENERAL STABILITY REQUIREMENTS

The general requirements of Section C1 shall apply. As an alternative to the direct analysis method (defined in Sections C1 and C2), it is permissible to design structures for stability in accordance with either the effective length method, specified in Section 7.2, or the first-order analysis method, specified in Section 7.3, subject to the limitations indicated in those sections.

7.2. EFFECTIVE LENGTH METHOD

1. Limitations

The use of the effective length method shall be limited to the following conditions:

(a) The structure supports gravity loads primarily through nominally vertical columns, walls or frames.

(b) The ratio of maximum second-order drift to maximum first-order drift (both determined for load and resistance factor design (LRFD) load combinations or 1.6 times allowable strength design (ASD) load combinations, with stiffness not adjusted as specified in Section C2.3) in all stories is equal to or less than 1.5.

User Note: The ratio of second-order drift to first-order drift in a story may be taken as the B_2 multiplier, calculated as specified in Appendix 8.

2. Required Strengths

The required strengths of components shall be determined from an elastic analysis conforming to the requirements of Section C2.1, except that the stiffness reduction indicated in Section C2.1(a) shall not be applied; the nominal stiffnesses of all structural steel components shall be used. Notional loads shall be applied in the analysis in accordance with Section C2.2b.

User Note: Since the condition specified in Section C2.2b(d) will be satisfied in all cases where the effective length method is applicable, the notional load need only be applied in gravity-only load cases.

3. Available Strengths

The available strengths of members and connections shall be calculated in accordance with the provisions of Chapters D through K, as applicable.

For flexural buckling, the effective length, L_c, of members subject to compression shall be taken as KL, where K is as specified in (a) or (b), in the following, as applicable, and L is the laterally unbraced length of the member.

(a) In braced-frame systems, shear-wall systems, and other structural systems where lateral stability and resistance to lateral loads does not rely on the flexural stiffness of columns, the effective length factor, K, of members subject to compression shall be taken as unity unless a smaller value is justified by rational analysis.

(b) In moment-frame systems and other structural systems in which the flexural stiffnesses of columns are considered to contribute to lateral stability and resistance to lateral loads, the effective length factor, K, or elastic critical buckling stress, F_e, of those columns whose flexural stiffnesses are considered to contribute to lateral stability and resistance to lateral loads shall be determined from a sidesway buckling analysis of the structure; K shall be taken as 1.0 for columns whose flexural stiffnesses are not considered to contribute to lateral stability and resistance to lateral loads.

Exception: It is permitted to use $K = 1.0$ in the design of all columns if the ratio of maximum second-order drift to maximum first-order drift (both determined for LRFD load combinations or 1.6 times ASD load combinations) in all stories is equal to or less than 1.1.

User Note: Methods of calculating the effective length factor, K, are discussed in the Commentary.

Bracing intended to define the unbraced lengths of members shall have sufficient stiffness and strength to control member movement at the braced points.

User Note: Methods of satisfying the bracing requirement are provided in Appendix 6. The requirements of Appendix 6 are not applicable to bracing that is included in the design of the lateral force-resisting system of the overall structure.

7.3. FIRST-ORDER ANALYSIS METHOD

1. Limitations

The use of the first-order analysis method shall be limited to the following conditions:

(a) The structure supports gravity loads primarily through nominally vertical columns, walls or frames.

(b) The ratio of maximum second-order drift to maximum first-order drift (both determined for LRFD load combinations or 1.6 times ASD load combinations, with stiffness not adjusted as specified in Section C2.3) in all stories is equal to or less than 1.5.

User Note: The ratio of second-order drift to first-order drift in a story may be taken as the B_2 multiplier, calculated as specified in Appendix 8.

(c) The required axial compressive strengths of all members whose flexural stiffnesses are considered to contribute to the lateral stability of the structure satisfy the limitation:

$$\alpha P_r \leq 0.5 P_{ns} \tag{A-7-1}$$

where

α = 1.0 (LRFD); α = 1.6 (ASD)

P_r = required axial compressive strength under LRFD or ASD load combinations, kips (N)

P_{ns} = cross-section compressive strength; for nonslender-element sections, $P_{ns} = F_y A_g$, and for slender-element sections, $P_{ns} = F_y A_e$, where A_e is as defined in Section E7, kips (N)

2. Required Strengths

The required strengths of components shall be determined from a first-order analysis, with additional requirements (a) and (b) given in the following. The analysis shall consider flexural, shear and axial member deformations, and all other deformations that contribute to displacements of the structure.

(a) All load combinations shall include an additional lateral load, N_i, applied in combination with other loads at each level of the structure:

$$N_i = 2.1\alpha(\Delta/L)Y_i \geq 0.0042Y_i \tag{A-7-2}$$

where

α = 1.0 (LRFD); α = 1.6 (ASD)

Y_i = gravity load applied at level i from the LRFD load combination or ASD load combination, as applicable, kips (N)

Δ/L = maximum ratio of Δ to L for all stories in the structure

Δ = first-order interstory drift due to the LRFD or ASD load combination, as applicable, in. (mm). Where Δ varies over the plan area of the structure, Δ shall be the average drift weighted in proportion to vertical load or, alternatively, the maximum drift.

L = height of story, in. (mm)

The additional lateral load at any level, N_i, shall be distributed over that level in the same manner as the gravity load at the level. The additional lateral loads shall be applied in the direction that provides the greatest destabilizing effect.

> **User Note:** For most building structures, the requirement regarding the direction of N_i may be satisfied as follows: (a) For load combinations that do not include lateral loading, consider two alternative orthogonal directions for the additional lateral load in a positive and a negative sense in each of the two directions, same direction at all levels; (b) for load combinations that include lateral loading, apply all the additional lateral loads in the direction of the resultant of all lateral loads in the combination.

(b) The nonsway amplification of beam-column moments shall be included by applying the B_1 amplifier of Appendix 8 to the total member moments.

> **User Note:** Since there is no second-order analysis involved in the first-order analysis method for design by ASD, it is not necessary to amplify ASD load combinations by 1.6 before performing the analysis, as required in the direct analysis method and the effective length method.

3. Available Strengths

The available strengths of members and connections shall be calculated in accordance with the provisions of Chapters D through K, as applicable.

The effective length for flexural buckling of all members shall be taken as the unbraced length unless a smaller value is justified by rational analysis.

Bracing intended to define the unbraced lengths of members shall have sufficient stiffness and strength to control member movement at the braced points.

> **User Note:** Methods of satisfying this requirement are provided in Appendix 6. The requirements of Appendix 6 are not applicable to bracing that is included in the analysis of the overall structure as part of the overall force-resisting system.

APPENDIX 8

APPROXIMATE SECOND-ORDER ANALYSIS

This appendix provides an approximate procedure to account for second-order effects in structures by amplifying the required strengths indicated by two first-order elastic analyses.

The appendix is organized as follows:

8.1. LIMITATIONS

The use of this procedure is limited to structures that support gravity loads primarily through nominally vertical columns, walls or frames, except that it is permissible to use the procedure specified for determining P-δ effects for any individual compression member.

8.2. CALCULATION PROCEDURE

The required second-order flexural strength, M_r, and axial strength, P_r, of all members shall be determined as:

$$M_r = B_1 M_{nt} + B_2 M_{lt} \tag{A-8-1}$$

$$P_r = P_{nt} + B_2 P_{lt} \tag{A-8-2}$$

where

B_1 = multiplier to account for P-δ effects, determined for each member subject to compression and flexure, and each direction of bending of the member in accordance with Section 8.2.1. B_1 shall be taken as 1.0 for members not subject to compression.

B_2 = multiplier to account for P-Δ effects, determined for each story of the structure and each direction of lateral translation of the story in accordance with Section 8.2.2

M_{lt} = first-order moment using LRFD or ASD load combinations, due to lateral translation of the structure only, kip-in. (N-mm)

M_{nt} = first-order moment using LRFD or ASD load combinations, with the structure restrained against lateral translation, kip-in. (N-mm)

M_r = required second-order flexural strength using LRFD or ASD load combinations, kip-in. (N-mm)

P_{lt} = first-order axial force using LRFD or ASD load combinations, due to lateral translation of the structure only, kips (N)

P_{nt} = first-order axial force using LRFD or ASD load combinations, with the structure restrained against lateral translation, kips (N)

P_r = required second-order axial strength using LRFD or ASD load combinations, kips (N)

User Note: Equations A-8-1 and A-8-2 are applicable to all members in all structures. Note, however, that B_1 values other than unity apply only to moments in beam-columns; B_2 applies to moments and axial forces in components of the lateral force-resisting system (including columns, beams, bracing members and shear walls). See the Commentary for more on the application of Equations A-8-1 and A-8-2.

1. Multiplier B_1 for P-δ Effects

The B_1 multiplier for each member subject to compression and each direction of bending of the member is calculated as:

$$B_1 = \frac{C_m}{1 - \alpha P_r / P_{e1}} \geq 1 \qquad \text{(A-8-3)}$$

where

α = 1.0 (LRFD); α = 1.6 (ASD)

C_m = equivalent uniform moment factor, assuming no relative translation of the member ends, determined as follows:

(a) For beam-columns not subject to transverse loading between supports in the plane of bending

$$C_m = 0.6 - 0.4(M_1 / M_2) \qquad \text{(A-8-4)}$$

where M_1 and M_2, calculated from a first-order analysis, are the smaller and larger moments, respectively, at the ends of that portion of the member unbraced in the plane of bending under consideration. M_1 / M_2 is positive when the member is bent in reverse curvature and negative when bent in single curvature.

(b) For beam-columns subject to transverse loading between supports, the value of C_m shall be determined either by analysis or conservatively taken as 1.0 for all cases.

P_{e1} = elastic critical buckling strength of the member in the plane of bending, calculated based on the assumption of no lateral translation at the member ends, kips (N)

$$= \frac{\pi^2 EI^*}{(L_{c1})^2} \qquad \text{(A-8-5)}$$

where

EI^* = flexural rigidity required to be used in the analysis (= $0.8\tau_b EI$ when used in the direct analysis method, where τ_b is as defined in Chapter C; = EI for the effective length and first-order analysis methods)

E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)

I = moment of inertia in the plane of bending, in.^{4} (mm^4)

L_{c1} = effective length in the plane of bending, calculated based on the assumption of no lateral translation at the member ends, set equal to the laterally unbraced length of the member unless analysis justifies a smaller value, in. (mm)

It is permitted to use the first-order estimate of P_r (i.e., $P_r = P_{nt} + P_{lt}$) in Equation A-8-3.

2. Multiplier B_2 for P-Δ Effects

The B_2 multiplier for each story and each direction of lateral translation is calculated as:

$$B_2 = \frac{1}{1 - \dfrac{\alpha P_{story}}{P_{e\,story}}} \geq 1 \tag{A-8-6}$$

where

α = 1.0 (LRFD); α = 1.6 (ASD)

P_{story} = total vertical load supported by the story using LRFD or ASD load combinations, as applicable, including loads in columns that are not part of the lateral force-resisting system, kips (N)

$P_{e\,story}$ = elastic critical buckling strength for the story in the direction of translation being considered, kips (N), determined by sidesway buckling analysis or as:

$$= R_M \frac{HL}{\Delta_H} \tag{A-8-7}$$

and

H = total story shear, in the direction of translation being considered, produced by the lateral forces used to compute Δ_H, kips (N)

L = height of story, in. (mm)

R_M = $1 - 0.15\,(P_{mf} / P_{story})$ (A-8-8)

P_{mf} = total vertical load in columns in the story that are part of moment frames, if any, in the direction of translation being considered (= 0 for braced-frame systems), kips (N)

Δ_H = first-order interstory drift, in the direction of translation being considered, due to lateral forces, in. (mm), computed using the stiffness required to be used in the analysis. (When the direct analysis method is used, stiffness is reduced according to Section C2.3.) Where Δ_H varies over the plan area of the structure, it shall be the average drift weighted in proportion to vertical load or, alternatively, the maximum drift.

User Note: R_M can be taken as 0.85 as a lower bound value for stories that include moment frames, and R_M = 1 if there are no moment frames in the story. H and Δ_H in Equation A-8-7 may be based on any lateral loading that provides a representative value of story lateral stiffness, H / Δ_H.

COMMENTARY

on the Specification for Structural Steel Buildings

July 7, 2016

(The Commentary is not a part of ANSI/AISC 360-16, *Specification for Structural Steel Buildings*, but is included for informational purposes only.)

INTRODUCTION

The Specification is intended to be complete for normal design usage.

The Commentary furnishes background information and references for the benefit of the design professional seeking further understanding of the basis, derivations and limits of the Specification.

The Specification and Commentary are intended for use by design professionals with demonstrated engineering competence.

COMMENTARY SYMBOLS

The Commentary uses the following symbols in addition to the symbols defined in the Specification. The section number in the right-hand column refers to the Commentary section where the symbol is first used.

Symbol	Definition	Section
B	Overall width of rectangular HSS, in. (mm)	I3
C_f	Compression force in concrete slab for fully composite beam; smaller of $F_y A_s$ and $0.85 f_c' A_c$, kips (N)	I3.2
F_y	Reported yield stress, ksi (MPa)	App. 5.2.2
F_{ys}	Static yield stress, ksi (MPa)	App. 5.2.2
H	Overall height of rectangular HSS, in. (mm)	I3
H	Height of anchor, in. (mm)	I8.2
I_{LB}	Lower bound moment of inertia, in.^{4} (mm^4)	I3.2
I_{neg}	Effective moment of inertia for negative moment, in.^{4} (mm^4)	I3.2
I_{pos}	Effective moment of inertia for positive moment, in.^{4} (mm^4)	I3.2
I_s	Moment of inertia for the structural steel section, in.^{4} (mm^4)	I3.2
I_{tr}	Moment of inertia for fully composite uncracked transformed section, in.^{4} (mm^4)	I3.2
$I_{y\,Top}$	Moment of inertia of the top flange about an axis through the web, in.^{4} (mm^4)	F1
K_S	Secant stiffness, kip-in. (N-mm)	B3.4
M_{CL}	Moment at the middle of the unbraced length, kip-in. (N-mm)	F1
M_o	Maximum first-order moment within the member due to the transverse loading, kip-in. (N-mm)	App. 8
Ms	Moment at service loads, kip-in. (N-mm)	B3.4
M_T	Torsional moment, kip-in. (N-mm)	G3
N	Number of cycles to failure	App. 3.3
P_{br}	Required brace strength, kips (N)	App. 6.1
Q_m	Mean value of the load effect Q	B3.1
R_{cap}	Minimum rotation capacity	App. 1.3.1
R_m	Mean value of the resistance R	B3.1
S_r	Stress range	App. 3.3
S_s	Section modulus for the structural steel section, referred to the tension flange, in.^{3} (mm^3)	I3.2
S_{tr}	Section modulus for the fully composite uncracked transformed section, referred to the tension flange of the steel section, in.^{3} (mm^3)	I3.2
V_b	Component of the shear force parallel to the angle leg with width b and thickness t, kips (N)	G3
V_Q	Coefficient of variation of the load effect Q	B3.1
V_R	Coefficient of variation of the resistance R	B3.1

a	Bracing offset measured from the shear center in x-direction, in. (mm)	E4
a_{cr}	Neutral axis location for force equilibrium, slender section, in. (mm)	I3.4
a_p	Neutral axis location for force equilibrium, compact section, in. (mm)	I3.4
a_y	Neutral axis location for force equilibrium, noncompact section, in. (mm)	I3.4
b	Bracing offset measured from the shear center in y-direction, in. (mm)	E4
f_v	Shear stress in angle, ksi (MPa)	G3
k	Plate buckling coefficient characteristic of the type of plate edge-restraint	E7.1
β	Reliability index	B3.1
β	Brace stiffness, kip/in. (N/mm)	App. 6.1
β_{act}	Actual bracing stiffness provided	App. 6.1
δ_o	Maximum deflection due to transverse loading, in. (mm)	App. 8
θ_S	Rotation at service loads, rad	B3.4
ν	Poisson's ratio = 11,200 ksi (77 200 MPa)	E7.1
ω	Empirical adjustment factor	E4

COMMENTARY GLOSSARY

The Commentary uses the following terms in addition to the terms defined in the Glossary of the Specification.

Alignment chart. Nomograph for determining the effective length factor, K, for some types of columns.

Biaxial bending. Simultaneous bending of a member about two perpendicular axes.

Brittle fracture. Abrupt cleavage with little or no prior ductile deformation.

Column curve. Curve expressing the relationship between axial column strength and slenderness ratio.

Critical load. Load at which a perfectly straight member under compression may either assume a deflected position or may remain undeflected, or a beam under flexure may either deflect and twist out of plane or remain in its in-plane deflected position, as determined by a theoretical stability analysis.

Cyclic load. Repeatedly applied external load that may subject the structure to fatigue.

Drift damage index. Parameter used to measure the potential damage caused by interstory drift.

Effective moment of inertia. Moment of inertia of the cross section of a member that remains elastic when partial plastification of the cross section takes place, usually under the combination of residual stress and applied stress; also, the moment of inertia based on effective widths of elements that buckle locally; also, the moment of inertia used in the design of partially composite members.

Effective stiffness. Stiffness of a member computed using the effective moment of inertia of its cross section.

Fatigue threshold. Stress range at which fatigue cracking will not initiate regardless of the number of cycles of loading.

First-order plastic analysis. Structural analysis based on the assumption of rigid-plastic behavior—in other words, that equilibrium is satisfied throughout the structure and the stress is at or below the yield stress—and in which equilibrium conditions are formulated on the undeformed structure.

Flexible connection. Connection permitting a portion, but not all, of the simple beam rotation of a member end.

Inelastic action. Material deformation that does not disappear on removal of the force that produced it.

Interstory drift. Lateral deflection of a floor relative to the lateral deflection of the floor immediately below, divided by the distance between floors, $(\delta_n - \delta_{n-1})/h$.

Permanent load. Load in which variations over time are rare or of small magnitude. All other loads are variable loads.

Plastic plateau. Portion of the stress-strain curve for uniaxial tension or compression in which the stress remains essentially constant during a period of substantially increased strain.

Primary member. For ponding analysis, beam or girder that supports the concentrated reactions from the secondary members framing into it.

Residual stress. Stress that remains in an unloaded member after it has been formed into a finished product. (Examples of such stresses include, but are not limited to, those induced by cold bending, cooling after rolling, or welding.)

Rigid frame. Structure in which connections maintain the angular relationship between beam and column members under load.

Secondary member. For ponding analysis, beam or joist that directly supports the distributed ponding loads on the roof of the structure.

Sidesway. Lateral movement of a structure under the action of lateral loads, unsymmetrical vertical loads, or unsymmetrical properties of the structure.

Sidesway buckling. Buckling mode of a multistory frame precipitated by the relative lateral displacements of joints, leading to failure by sidesway of the frame.

Shape Factor. Ratio of the plastic moment to the yield moment, M_p/M_y, also given by Z/S.

St. Venant torsion. Portion of the torsion in a member that induces only shear stresses in the member.

Strain hardening. Phenomenon wherein ductile steel, after undergoing considerable deformation at or just above yield point, exhibits the capacity to resist substantially higher loading than that which caused initial yielding.

Stub-column. A short compression test specimen utilizing the complete cross section, sufficiently long to provide a valid measure of the stress-strain relationship as averaged over the cross section, but short enough so that it will not buckle as a column in the elastic or plastic range.

Total building drift. Lateral frame deflection at the top of the most occupied floor divided by the height of the building to that level, Δ/H.

Undercut. Notch resulting from the melting and removal of base metal at the edge of a weld.

Variable load. Load with substantial variation over time.

Warping torsion. Portion of the total resistance to torsion that is provided by resistance to warping of the cross section.

CHAPTER A
GENERAL PROVISIONS

A1. SCOPE

The scope of this Specification is essentially the same as the 2010 *Specification for Structural Steel Buildings* (AISC, 2010) that it replaces.

The basic purpose of the provisions in this Specification is the determination of the nominal and available strengths of the members, connections and other components of steel building structures.

This Specification provides two methods of design:

(a) Load and Resistance Factor Design (LRFD): The nominal strength is multiplied by a resistance factor, ϕ, resulting in the design strength, which is then required to equal or exceed the required strength determined by structural analysis for the appropriate LRFD load combinations specified by the applicable building code.

(b) Allowable Strength Design (ASD): The nominal strength is divided by a safety factor, Ω, resulting in the allowable strength, which is then required to equal or exceed the required strength determined by structural analysis for the appropriate ASD load combinations specified by the applicable building code.

This Specification gives provisions for determining the values of the nominal strengths according to the applicable limit states and lists the corresponding values of the resistance factor, ϕ, and the safety factor, Ω. Nominal strength is usually defined in terms of resistance to a load effect, such as axial force, bending moment, shear or torque, but in some instances it is expressed in terms of a stress. The ASD safety factors are calibrated to give the same structural reliability and the same component size as the LRFD method at a live-to-dead load ratio of 3. The term available strength is used throughout the Specification to denote design strength and allowable strength, as applicable.

This Specification is applicable to both buildings and other structures. Many structures found in petrochemical plants, power plants, and other industrial applications are designed, fabricated and erected in a manner similar to buildings. It is not intended that this Specification address steel structures with vertical and lateral force-resisting systems that are not similar to buildings, nor those constructed of shells or catenary cables.

The Specification may be used for the design of structural steel elements, as defined in the AISC *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2016a), hereafter referred to as the *Code of Standard Practice*, when used as components of nonbuilding structures or other structures. Engineering judgment must be applied to the Specification requirements when the structural steel elements are exposed to environmental or service conditions and/or loads not usually applicable to building structures.

The *Code of Standard Practice* defines the practices that are the commonly accepted standards of custom and usage for structural steel fabrication and erection. As such, the *Code of Standard Practice* is primarily intended to serve as a contractual document to be incorporated into the contract between the buyer and seller of fabricated structural steel. Some parts of the *Code of Standard Practice*, however, form the basis for some of the provisions in this Specification. Therefore, the *Code of Standard Practice* is referenced in selected locations in this Specification to maintain the ties between these documents, where appropriate.

The Specification disallows seismic design of buildings and other structures using the provisions of Appendix 1, Section 1.3. The *R*-factor specified in ASCE/SEI 7-16 (ASCE, 2016) used to determine the seismic loads is based on a nominal value of system overstrength and ductility that is inherent in steel structures designed by elastic analysis using this Specification. Therefore, it would be inappropriate to take advantage of the additional strength afforded by the inelastic design approach presented in Appendix 1, Section 1.3, while simultaneously using the code specified *R*-factor. In addition, the provisions for ductility in Appendix 1, Section 1.3.2, are not fully consistent with the intended levels for seismic design.

A2. REFERENCED SPECIFICATIONS, CODES AND STANDARDS

Section A2 provides references to documents cited in this Specification. The date of the referenced document found in this Specification is the intended date referenced in this Commentary unless specifically indicated otherwise. Note that not all grades of a particular material specification are necessarily approved for use according to this Specification. For a list of approved materials and grades, see Section A3.

A3. MATERIAL

1. Structural Steel Materials

1a. ASTM Designations

There are hundreds of steel materials and products. This Specification lists those products/materials that are commonly useful to structural engineers and those that have a history of satisfactory performance. Other materials may be suitable for specific applications, but the evaluation of those materials is the responsibility of the engineer specifying them. In addition to typical strength properties, considerations for materials may include, but are not limited to strength properties in transverse directions, ductility, formability, soundness, weldability including sensitivity to thermal cycles, notch toughness, and other forms of crack sensitivity, coatings, and corrosivity. Consideration for product form may include material considerations in addition to effects of production, tolerances, testing, reporting and surface profiles.

Hot-Rolled Structural Shapes. The grades of steel approved for use under this Specification, covered by ASTM Specifications, extend to a yield stress of 100 ksi (690 MPa). Some of the ASTM Specifications specify a minimum yield point, while others specify a minimum yield strength. The term “yield stress” is used in this Specification as a generic term to denote either the yield point or the yield strength.

It is important to be aware of limitations of availability that may exist for some combinations of strength and size. Not all structural section sizes are included in the various material specifications. For example, the 60-ksi (415 MPa) yield stress steel in the ASTM A572/A572M Specification includes plate only up to 1¹/4 in. (32 mm) in thickness. Another limitation on availability is that even when a product is included in this Specification, it may be infrequently produced by the mills. Specifying these products may result in procurement delays or require ordering large quantities directly from the producing mills. Consequently, it is prudent to check availability before completing the details of a design. The AISC web site provides this information (**www.aisc.org**).

Properties in the direction of rolling are of principal interest in the design of steel structures. Hence, yield stress as determined by the standard tensile test is the principal mechanical property recognized in the selection of the steels approved for use under this Specification. It must be recognized that other mechanical and physical properties of rolled steel, such as anisotropy, ductility, notch toughness, formability, corrosion resistance, etc., may also be important to the satisfactory performance of a structure.

For rotary-straightened W-shapes, an area of reduced notch toughness has been documented in a limited region of the web immediately adjacent to the flange. Considerations in design and detailing that recognize this situation are presented in Chapter J.

It is not possible to incorporate in the Commentary adequate information to impart full understanding of all factors that might merit consideration in the selection and specification of materials for unique or especially demanding applications. In such a situation, the user of this Specification is advised to make use of reference material contained in the literature on the specific properties of concern and to specify supplementary material production or quality requirements as provided for in ASTM material specifications. One such case is the design of highly restrained welded connections (AISC, 1973). Rolled steel is anisotropic, especially insofar as ductility is concerned; therefore, weld contraction strains in the region of highly restrained welded connections may exceed the strength of the material if special attention is not given to material selection, details, workmanship and inspection.

Another special situation is that of fracture control design for certain types of service conditions (AASHTO, 2014). For especially demanding service conditions such as structures exposed to low temperatures, particularly those with impact loading, the specification of steels with superior notch toughness may be warranted. However, for most buildings, the steel is relatively warm, strain rates are essentially static, and the stress intensity and number of cycles of full design stress are low. Accordingly, the probability of fracture in most building structures is low. Good workmanship and good design details incorporating joint geometry that avoids severe stress concentrations are generally the most effective means of providing fracture-resistant construction.

TABLE C-A3.1
Minimum Tensile Properties of HSS Steels

Specification	Grade	F_y, ksi (MPa)	F_u, ksi (MPa)
ASTM A53/A53M	B	35 (240)	60 (415)
ASTM A500/A500M (round)	B C	42 (290) 46 (315)	58 (400) 62 (425)
ASTM A500/A500M (rectangular)	B C	46 (315) 50 (345)	58 (400) 62 (425)
ASTM A501/A501M	A B	36 (250) 50 (345)	58 (400) 70 (485)
ASTM A618/A618M (round)	I and II ($t \le w$, in. (MPa)) III	50 (345) 50 (345)	70 (485) 65 (450)
ASTM A847/A847M	–	50 (345)	70 (485)
CAN/CSA-G40.20/G40.21	350W	51 (350)	65 (450)
ASTM A1085/A1085M	–	50 (345)	65 (450)
ASTM A1065/A1065M	50 50W	50 (345) 50 (345)	60 (415) 70 (480)

Hollow Structural Sections (HSS). Specified minimum tensile properties are summarized in Table C-A3.1 for various HSS material specifications and grades. ASTM A53/A53M Grade B is a pipe specification included as an approved HSS material specification because it is the most readily available round product in the United States. Other North American HSS products that have properties and characteristics that are similar to the approved ASTM products are produced in Canada under the *General Requirements for Rolled or Welded Structural Quality Steel* (CSA, 2013). In addition, pipe is produced to other specifications that meet the strength, ductility and weldability requirements of the materials in Section A3, but may have additional requirements for notch toughness or pressure testing. As stated in the preamble to Section A3.1, for materials not specifically listed in Section A3, evidence of conformity to the specified ASTM specification must be shown.

Round HSS can be readily obtained in ASTM A53/A53M material and ASTM A500/A500M Grade C is also common. For rectangular HSS, ASTM A500/A500M Grade C is the most commonly available material and a special order would be required for any other material. Depending upon size, either welded or seamless round HSS can be obtained. In North America, however, all ASTM A500/A500M rectangular HSS for structural purposes are welded. Rectangular HSS differ from box sections in that they have uniform thickness, except for some thickening in the rounded corners.

Nominal strengths of direct welded T-, Y- and K-connections of HSS have been developed analytically and empirically. Connection deformation is anticipated and is an acceptance limit for connection tests. Ductility is necessary to achieve the expected deformations. The ratio of the specified minimum yield strength to the specified minimum tensile strength (yield/tensile ratio) is one measure of material ductility. Materials in HSS used in connection tests have had a yield/tensile ratio of up to 0.80 and therefore that ratio has been adopted as a limit of applicability for direct welded HSS connections. ASTM A500/A500M Grade A material does not meet this ductility "limit of applicability" for direct connections in Chapter K. ASTM A500/A500M Grade C has a yield/tensile ratio of 0.807 but it is reasonable to use the rounding method described in ASTM E29 and find this material acceptable for use.

Even though ASTM A501/A501M includes rectangular HSS, hot-formed rectangular HSS are not currently produced in the United States. The *General Requirements for Rolled or Welded Structural Quality Steel* (CSA, 2013) includes Class C (cold-formed) and Class H (cold-formed and stress relieved) HSS. Class H HSS have relatively low levels of residual stress, which enhances their performance in compression and may provide better ductility in the corners of rectangular HSS.

API 5L (API, 2012) is a line pipe specification that has some mechanical characteristics that make it advantageous in specific structural applications, such as in long span roofs with long unbraced lengths or large composite columns in heavy unbraced frames. Note, however, that Section A3.1 states, for materials not specifically listed in Section A3, evidence of conformity to the specified ASTM Specification must be shown. The specified minimum yield strength of API 5L ranges from 25 to 80 ksi (170 to 550 MPa) and specified minimum tensile strength ranges from 45 to 90 ksi (310 to 620 MPa), depending on product specification level and material grade. For Grades X42 and higher, additional elements may be used upon agreement between the purchaser and the manufacturer; however, care should be exercised in determining the alloying content for any given size and wall thickness of pipe, because the addition of such otherwise desirable elements may affect the weldability of the pipe. PSL2 pipe is a common structural choice and Grade X52 is probably the most common grade for structural purposes. Some pertinent mechanical and geometric properties for PSL2 X52N are: F_y = 52 ksi (360 MPa); F_u = 66 ksi (460 MPa); Toughness = 20 ft-lb @ 32°F (27 J @ 0°C) for $D \leq 30$ in. (760 mm); a wall thickness lower tolerance of −10% for $^3/_{16}$ in. $< t <$ $^{19}/_{32}$ in. (5 mm $< t <$ 15 mm), and −0.02 in. (−0.5 mm) for $t <$ $^3/_{16}$ in. ($t <$ 5 mm); a mass or area tolerance of −3.5% for regular plain-ended. With a diameter range from $^{13}/_{32}$ in. to 84 in. (10 mm to 2100 mm), this high-quality pipe material addresses a frequent need for either large diameter or thick-walled round hollow sections. Other special features of PSL2 pipe are an upper bound on the yield strength [e.g., for X52 the minimum and maximum yield strengths are 52 ksi (360 MPa) and 76 ksi (530 MPa), respectively], and a maximum yield-to-tensile stress ratio of 0.93 in the as-delivered pipe [for $D >$ 12.75 in. (320 mm)].

1c. Rolled Heavy Shapes

The web-to-flange intersection and the web center of heavy hot-rolled shapes, as well as the interior portions of heavy plates, may contain a more coarse grain structure and/or lower notch toughness material than other areas of these products. This is probably caused by ingot segregation, the somewhat lesser deformation during hot rolling, higher finishing temperature, and the slower cooling rate after rolling for these heavy sections. This characteristic is not detrimental to suitability for compression members or for nonwelded members. However, when heavy cross sections are joined by splices or connections using complete-joint-penetration groove welds that extend through the coarser and/or lower notch-tough interior portions, tensile strains induced by weld shrinkage may result in cracking. An example is a complete-joint-penetration groove welded connection of a heavy cross-section beam to any column section. When members of lesser thickness are joined by complete-joint-penetration groove welds, which induce smaller weld shrinkage strains, to the finer grained and/or more notch-tough surface material of ASTM A6/A6M shapes and heavy built-up cross sections, the potential for cracking is significantly lower. An example is a complete-joint-penetration groove welded connection of a nonheavy cross-section beam to a heavy cross-section column.

For critical applications, such as primary tension members, material should be specified to provide adequate notch toughness at service temperatures. Because of differences in the strain rate between the Charpy V-notch (CVN) impact test and the strain rate experienced in actual structures, the CVN test is conducted at a temperature higher than the anticipated service temperature for the structure. The location of the CVN test specimens ("alternate core location") is specified in ASTM A6/A6M, Supplemental Requirement S30.

The notch toughness requirements of Section A3.1c are intended only to provide material of reasonable notch toughness for ordinary service applications. For unusual applications and/or low temperature service, more restrictive requirements and/or notch toughness requirements for other section sizes and thicknesses may be appropriate. To minimize the potential for fracture, the notch toughness requirements of Section A3.1c must be used in conjunction with good design and fabrication procedures. Specific requirements are given in Sections J1.5, J1.6, J2.6 and J2.7.

2. Steel Castings and Forgings

Design and fabrication of cast and forged steel components are not covered in this Specification.

Steel Castings. There are a number of ASTM Specifications for steel castings. The Steel Founders' Society of America (SFSA) *Steel Castings Handbook* (SFSA, 1995) discusses a number of standards useful for steel structures. In addition to the requirements of this Specification, SFSA recommends that various other requirements be considered for cast steel products. Continued quality assurance is critical to ensure confidence in the cast product. This includes testing of first article components as well as production testing. It may be appropriate to inspect the first piece cast using magnetic particle inspection (MPI) in accordance with ASTM E125, degree 1a, b or c (ASTM, 2013a). Radiographic inspection level III may be desirable for the first piece cast. Ultrasonic testing (UT) in compliance with ASTM A609/A609M (ASTM, 2012b) may be appropriate for the first cast piece over 6 in. (150 mm) thick. UT and MPI of production castings are also advisable. Design approval, sample approval, periodic nondestructive testing, chemical testing, and selection of the correct welding specification should be among the issues defined in the selection and procurement of cast steel products. Refer to SFSA (1995) for design information about cast steel products. For visual examination, refer to ASTM A802/A802M (ASTM, 2015d); for magnetic particle and liquid penetrant surface and subsurface examination, refer to ASTM A903/A903M (ASTM, 2012a); for radiographic examination, refer to ASTM E1030/E1030M (ASTM, 2015e); and for ultrasonic examination, refer to ASTM A609/A609M (ASTM, 2012b). ASTM A958/A958M is a cast steel used in the Kaiser Bolted Bracket Moment Connection, a prequalified moment connection in ANSI/AISC 358 (AISC, 2016c), but it may also be specified in some nonseismic applications. Additional information about cast steels can be found in the *Steel Castings Handbook*, *Supplement 2* (SFSA, 2009).

Steel Forgings. There are a number of ASTM specifications for steel forgings. The Forging Industry Association's *Forging Industry Handbook* (FIA, 1985) discusses some typical forging issues, but more detailed information can be obtained at **www.forging.org**. Steel forgings should conform to ASTM A668/A668M and the related ASTM testing requirements. UT should be in compliance with ASTM A388/A388M (ASTM, 2016) and MPI in accordance with ASTM A275/A275M. Many of the frequently used structural forgings are catalog items for which the testing has been established. For custom forgings, the frequency and type of testing required should be established to conform to ASTM requirements.

3. Bolts, Washers and Nuts

ASTM F3125 is an umbrella specification that covers what were ASTM A325/A325M, A490/A490M, F1852, and F2280 fasteners. These previously separate standards have been unified, coordinated, and made consistent with each other, turning them into Grades of ASTM F3125. From the user perspective, not much has changed, as the head marks remain the same, and handling and installation remain the same. Nevertheless, the specifier should be aware that ASTM F3125 now contains Grade A325, A325M, A490, A490M, F1852 and F2280 fasteners. One change of note is that under F3125, Grade A325 and A325M fasteners are uniformly 120 ksi (830 MPa); Grade A325 and A325M had a drop in strength to 105 ksi (725 MPa) for diameters over one inch (25 mm) in previous standards.

The ASTM standard specification for A307 bolts covers two grades of fasteners. Either grade may be used under this Specification; however, it should be noted that Grade B is intended for pipe-flange bolting and Grade A is the grade long in use for structural applications.

4. Anchor Rods and Threaded Rods

ASTM F1554 is the primary specification for anchor rods. Since there is a limit on the maximum available length of structural bolts, the attempt to use these bolts for anchor rods with design lengths longer than the maximum available lengths has presented problems in the past. The inclusion of ASTM A449 and A354 materials in this Specification allows the use of higher strength material for bolts longer than structural bolts.

The engineer of record should specify the required strength for threaded rods used as load-carrying members.

5. Consumables for Welding

The AWS filler metal specifications listed in Section A3.5 are general specifications that include filler metal classifications suitable for building construction, as well as classifications that may not be suitable for building construction. *Structural Welding Code—Steel* (AWS D1.1/D1.1M) (AWS, 2015) Table 3.2 lists the various electrodes that may be used for prequalified welding procedure specifications, for the various steels that are to be joined. This list specifically does not include various classifications of filler metals that are not suitable for structural steel applications. Filler metals listed under the various AWS A5 filler metal specifications may or may not have specified notch toughness properties, depending on the specific electrode classification. Section J2.6 identifies certain welded joints where notch toughness of filler metal is needed in building construction. There may be other situations where the engineer of record may elect to specify the use of filler metals with specified notch toughness properties, such as for structures subject to high loading rate, cyclic loading, or seismic loading. Since AWS D1.1/D1.1M does not automatically require that the filler metal used have specified notch toughness properties, it is important that filler metals used for such applications be of an AWS classification, where such properties are required. This information can be found in the various AWS filler metal specifications and is often contained on the filler metal manufacturer's certificate of conformance or product specification sheets.

When specifying filler metal and/or flux by AWS designation, the applicable standard specifications should be carefully reviewed to assure a complete understanding of the designation reference. This is necessary because the AWS designation systems are not consistent. For example, in the case of electrodes for shielded metal arc welding, AWS A5.1/A5.1M, *Specification for Carbon Steel Electrodes for Shielded Metal Arc Welding* (AWS, 2012), the first two or three digits indicate the nominal tensile strength classification, in ksi, of the filler metal and the final two digits indicate the type of coating. For metric designations, the first two digits times 10 indicate the nominal tensile strength classification in MPa. In the case of mild steel electrodes for submerged arc welding, AWS A5.17/A5.17M, *Specification for Carbon Steel Electrodes and Fluxes for Submerged Arc Welding* (AWS, 2007), the first one or two digits times 10 indicate the nominal tensile strength classification for both U.S. customary and metric units, while the final digit or digits times 10 indicate the testing temperature in °F, for filler metal impact tests. In the case of low-alloy steel covered arc welding electrodes, AWS A5.5/A5.5M, *Specification for Low-Alloy Steel Electrodes for Shielded Metal Arc Welding* (AWS, 2014), certain portions of the designation indicate a requirement for stress relief, while others indicate no stress relief requirement.

Engineers do not, in general, specify the exact filler metal to be employed on a particular structure. Rather, the decision as to which welding process and which filler metal is to be utilized is usually left with the fabricator or erector. Codes restrict the usage of certain filler materials, or impose qualification testing to prove the suitability of the specific electrode, so as to make certain that the proper filler metals are used.

A4. STRUCTURAL DESIGN DRAWINGS AND SPECIFICATIONS

The abbreviated list of requirements in this Specification is intended to be compatible with and a summary of the more extensive requirements in the *Code of Standard Practice* Section 3. The user should refer there for further information.

CHAPTER B

DESIGN REQUIREMENTS

B1. GENERAL PROVISIONS

This Specification is meant to be primarily applicable to the common types of building frames with gravity loads carried by beams and girders and lateral loads carried by moment frames, braced frames or shear walls. However, there are many unusual buildings or building-like structures for which this Specification is also applicable. Rather than attempt to establish the purview of the Specification with an exhaustive classification of construction types, Section B1 requires that the design of members and their connections be consistent with the intended use of the structure and the assumptions made in the analysis of the structure.

Section B1 widens the purview of this Specification to a class of construction types broader than those addressed in previous editions of the Specification. Section B1 recognizes that a structural system is a combination of members connected in such a way that the structure can respond in different ways to meet different design objectives under different loads. Even within the purview of ordinary buildings, there can be enormous variety in the design details.

Previous to the 2005 edition, the Specification contained a section entitled "Types of Construction"; for example, Section A2 in the 1999 *Load and Resistance Factor Design Specification for Structural Steel Buildings* (AISC, 2000b). In this Specification there is no such section and the requirements related to "types of construction" have been divided between Section B1, Section B3.4 and Section J1.

B2. LOADS AND LOAD COMBINATIONS

The loads, load combinations and nominal loads for use with this Specification are given in the applicable building code. In the absence of an applicable specific local, regional or national building code, the loads (for example, D, L, L_r, S, R, W and E), load factors, load combinations and nominal loads (numeric values for D, L and other loads) are as specified in *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, ASCE/SEI 7 (ASCE, 2016). This edition of ASCE/SEI 7 has adopted many of the seismic design provisions of the *NEHRP Recommended Seismic Provisions for New Buildings and Other Structures* (BSSC, 2015), as have the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2016b). The reader is referred to the commentaries of these documents for an expanded discussion on loads, load factors and seismic design.

This Specification is based on strength limit states that apply to structural steel design in general. The Specification permits design for strength using either load and resistance factor design (LRFD) or allowable strength design (ASD). It should be noted that the terms strength and stress reflect whether the appropriate section property has been applied in the calculation of the available strength. In most instances, the Specification uses strength rather than stress. In all cases it is a simple matter to recast the provisions into a stress format. The terminology used to describe load combinations in ASCE/SEI 7 is somewhat different from that used by this Specification. ASCE/SEI 7 Section 2.3 defines Combining Factored Loads Using Strength Design; these combinations are applicable to design using the LRFD approach. ASCE/SEI 7 Section 2.4 defines Combining Nominal Loads Using Allowable Stress Design; these combinations are applicable to design using the ASD load approach.

LRFD Load Combinations. If the LRFD approach is selected, the load combination requirements are defined in ASCE/SEI 7 Section 2.3.

The load combinations in ASCE/SEI 7 Section 2.3 are based on modern probabilistic load modeling and a comprehensive survey of reliabilities inherent in traditional design practice (Galambos et al., 1982; Ellingwood et al., 1982). These load combinations utilize a "principal action-companion action format," which is based on the notion that the maximum combined load effect occurs when one of the time-varying loads takes on its maximum lifetime value (principal action) while the other variable loads are at "arbitrary point-in-time" values (companion actions), the latter being loads that would be measured in a load survey at any arbitrary time. The dead load, which is considered to be permanent, is the same for all combinations in which the load effects are additive. Research has shown that this approach to load combination analysis is consistent with the manner in which loads actually combine on structural elements and systems in situations in which strength limit states may be approached. The load factors reflect uncertainty in individual load magnitudes and in the analysis that transforms load to load effect. The nominal loads in ASCE/SEI 7 are substantially in excess of the arbitrary point-in-time values. The basis for the LRFD load combinations can be found in the Commentary to ASCE/SEI 7 Section 2.3.

The return period associated with earthquake loads was revised in both the 2003 and 2009 editions of the *NEHRP Recommended Seismic Provisions for New Buildings and Other Structures* (BSSC, 2003, 2009). In the 2009 edition, adopted as the basis for ASCE/SEI 7-10 (ASCE, 2010), the earthquake loads at most locations are intended to produce a collapse probability of 1% in a 50 year period, a performance objective that is achieved by requiring that the probability of incipient collapse, given the occurrence of the Maximum Considered Earthquake (MCE), is less than 10%. At some sites in regions of high seismic activity, where high intensity events occur frequently, deterministic limits on the ground motion result in somewhat higher collapse probabilities. The Commentary to Chapter 1 of ASCE/SEI 7 provides information on the intended maximum probability of structural failure under earthquake and other loads.

Load combinations of ASCE/SEI 7 Section 2.3, which apply specifically to cases in which the structural actions due to lateral forces and gravity loads counteract one another and the dead load stabilizes the structure, incorporate a load factor on dead load of 0.9.

ASD Load Combinations. If the ASD approach is selected, the load combination requirements are defined in ASCE/SEI 7 Section 2.4.

The load combinations in ASCE/SEI 7 Section 2.4 are similar to those traditionally used in allowable stress design. In ASD, safety is provided by the safety factor, Ω, and the nominal loads in the basic combinations involving gravity loads, earth pressure or fluid pressure are not factored. The reduction in the combined time-varying load effect in combinations incorporating wind or earthquake load is achieved by the load combination factor 0.75. This load combination factor dates back to the 1972 edition of ANSI Standard A58.1 (ANSI, 1972), the predecessor of ASCE/SEI 7. It should be noted that in ASCE/SEI 7, the 0.75 factor applies only to combinations of variable loads; it is irrational to reduce the dead load because it is always present and does not fluctuate with time. It should also be noted that certain ASD load combinations may actually result in a higher required strength than similar load combinations for LRFD. Load combinations that apply specifically to cases in which the structural actions due to lateral forces and gravity loads counteract one another, where the dead load stabilizes the structure, incorporate a load factor on dead load of 0.6. This eliminates a deficiency in the traditional treatment of counteracting loads in ASD and emphasizes the importance of checking stability. The earthquake load effect is multiplied by 0.7 in applicable combinations involving that load to align ASD for earthquake effects with the definition of E in the sections of ASCE/SEI 7 defining seismic load effects and combinations.

The load combinations in Sections 2.3 and 2.4 of ASCE/SEI 7 apply to design for strength limit states. They do not account for gross error or negligence. Loads and load combinations for nonbuilding structures and other structures may be defined in ASCE/SEI 7 or other applicable industry standards and practices.

B3. DESIGN BASIS

As stated in this Specification: "design shall be based on the principle that no applicable strength or serviceability limit state shall be exceeded when the structure is subjected to load from all appropriate load combinations."

A limit state is a condition in which a structural system or component becomes unfit for its intended purpose (serviceability limit state), or has reached its ultimate load-carrying capacity (strength limit state). Limit states may be dictated by functional requirements, such as maximum deflections or drift; they may be related to structural behavior, such as the formation of a plastic hinge or mechanism; or they may represent the collapse of the whole or part of the structure, such as by instability or fracture. The design provisions in this Specification ensure that the probability of exceeding a limit state is acceptably small by stipulating the combination of load factors, resistance or safety factors, nominal loads, and nominal strengths consistent with the design assumptions.

Two kinds of limit states apply to structures: (a) strength limit states, which define safety against local or overall failure conditions during the intended life of the structure; and (b) serviceability limit states, which define functional requirements. This Specification, like other structural design codes, focuses primarily on strength limit states because of overriding considerations of public safety. This does not mean that limit states of serviceability (see Chapter L) are not important to the designer, who must provide for functional performance and economy of design. However, serviceability considerations permit more exercise of judgment on the part of the designer.

Load and resistance factor design (LRFD) and allowable strength design (ASD) are distinct methods for satisfying strength limit states. They are equally acceptable by this Specification, but their provisions are not interchangeable. Indiscriminate use of combinations of the two methods could result in unpredictable performance or unsafe design. Thus, the LRFD and ASD methods are specified as alternatives. There are, however, circumstances in which the two methods could be used in the design, modification or renovation of a structural system without conflict, such as providing modifications to a structural floor system of an older building after assessing the as-built conditions.

Strength limit states vary from element to element, and several limit states may apply to a given element. The most common strength limit states are yielding, buckling and rupture. The most common serviceability limit states include deflections or drift, and vibrations.

1. Design for Strength Using Load and Resistance Factor Design (LRFD)

Design for strength by LRFD is performed in accordance with Equation B3-1. The left side of Equation B3-1, R_u, represents the required strength computed by structural analysis based on load combinations stipulated in ASCE/SEI 7 Section 2.3 (or their equivalent) (ASCE, 2016), while the right side, ϕR_n, represents the limiting structural resistance, or design strength, provided by the member or element.

The resistance factor, ϕ, in this Specification is equal to or less than 1.00. When compared to the nominal strength, R_n, computed according to the methods given in Chapters D through K, a ϕ of less than 1.00 accounts for approximations in the theory and variations in mechanical properties and dimensions of members and frames. For limit states where $\phi = 1.00$, the nominal strength is judged to be sufficiently conservative when compared to the actual strength that no reduction is needed.

The LRFD provisions are based on (1) probabilistic models of loads and resistance, (2) a calibration of the LRFD provisions to the 1978 edition of the ASD Specification for selected members (AISC, 1978), and (3) the evaluation of the resulting provisions by judgment and past experience aided by comparative design office studies of representative structures.

In the probabilistic basis for LRFD (Ravindra and Galambos, 1978; Ellingwood et al., 1982), the load effects, Q, and the resistances, R, are modeled as statistically independent random variables. In Figure C-B3.1, relative frequency distributions for Q and R are portrayed as separate curves on a common plot for a hypothetical case. As long as the resistance, R, is greater than (to the right of) the effects of the loads, Q, a margin of safety for the particular limit state exists. However, because Q and R are random variables, there is a small probability that R may be less than Q. The probability of this limit state is related to the degree of overlap of the frequency distributions in Figure C-B3.1, which depends on the positioning of their mean values (R_m versus Q_m) and their dispersions.

The probability that R is less than Q depends on the distributions of the many variables (material, loads, etc.) that determine resistance and total load effect. Often, only the means and the standard deviations or coefficients of variation of the variables involved in the determination of R and Q can be estimated. However, this information is sufficient to build an approximate design provision that is independent of the knowledge of these distributions, by stipulating the following design condition:

$$\beta\sqrt{V_R^2 + V_Q^2} \le \ln\left(R_m/Q_m\right) \tag{C-B3-1}$$

where

R_m = mean value of the resistance, R
Q_m = mean value of the load effect, Q
V_R = coefficient of variation of the resistance, R
V_Q = coefficient of variation of the load effect, Q

For structural elements and the usual loading, R_m, Q_m, and the coefficients of variation, V_R and V_Q, can be estimated, so a calculation of

$$\beta = \frac{\ln\left(R_m / Q_m\right)}{\sqrt{V_R^2 + V_Q^2}} \tag{C-B3-2}$$

will give a comparative measure of reliability of a structure or component. The parameter β is denoted the reliability index. Extensions to the determination of β in Equation C-B3-2 to accommodate additional probabilistic information and more complex design situations are described in Ellingwood et al. (1982) and have been used in the development of the recommended load combinations in ASCE/SEI 7.

The original studies that determined the statistical properties (mean values and coefficients of variation) for the basic material properties and for steel beams, columns, composite beams, plate girders, beam-columns and connection elements that were used to develop the LRFD provisions are presented in a series of eight articles in the September 1978 issue of the *Journal of the Structural Division* (ASCE, 1978). The

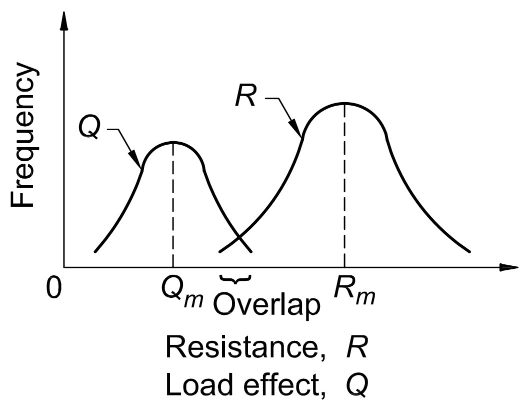


Fig. C-B3.1. Frequency distribution of load effect, Q, *and resistance,* R.

corresponding load statistics are given in Galambos et al. (1982). Based on these statistics, the values of β inherent in the 1978 *Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings* (AISC, 1978) were evaluated under different load combinations (live/dead, wind/dead, etc.) and for various tributary areas for typical members (beams, columns, beam-columns, structural components, etc.). As might be expected, there was a considerable variation in the range of β-values. For example, compact rolled beams (flexure) and tension members (yielding) had β-values that decreased from about 3.1 at L/D = 0.50 to 2.4 at L/D = 4. This decrease is a result of ASD applying the same factor to dead load, which is relatively predictable, and live load, which is more variable. For bolted or welded connections, β was in the range of 4 to 5.

The variation in β that was inherent to ASD is reduced substantially in LRFD by specifying several target β-values and selecting load and resistance factors to meet these targets. The Committee on Specifications set the point at which LRFD is calibrated to ASD at L/D = 3.0 for braced compact beams in flexure and tension members at yield. The resistance factor, ϕ, for these limit states is 0.90, and the implied β is approximately 2.6 for members and 4.0 for connections. The larger β-value for connections reflects the complexity in modeling their behavior, effects of workmanship, and the benefit provided by additional strength. Limit states for other members are handled similarly.

The databases on steel strength used in previous editions of the LRFD *Specification* (AISC, 1986, 1993, 2000b) were based mainly on research conducted prior to 1970. An important recent study of the material properties of structural shapes (Bartlett et al., 2003) reflected changes in steel production methods and steel materials that have occurred over the past 15 years. This study indicated that the new steel material characteristics did not warrant changes in the ϕ-values.

2. Design for Strength Using Allowable Strength Design (ASD)

The ASD method is provided in this Specification as an alternative to LRFD for use by engineers who prefer to deal with ASD load combinations and allowable stresses in the traditional ASD format. The term “allowable strength” has been introduced to emphasize that the basic equations of structural mechanics that underlie the provisions are the same for LRFD and ASD.

Traditional ASD is based on the concept that the maximum stress in a component shall not exceed a specified allowable stress under normal service conditions. The load effects are determined on the basis of an elastic analysis of the structure, while the allowable stress is the limiting stress (at yielding, instability, rupture, etc.) divided by a safety factor. The magnitude of the safety factor and the resulting allowable stress depend on the particular governing limit state against which the design must produce a certain margin of safety. For any single element, there may be a number of different allowable stresses that must be checked.

The safety factor in traditional ASD provisions was a function of both the material and the component being considered. It may have been influenced by factors such as member length, member behavior, load source, and anticipated quality of workmanship. The traditional safety factors were based solely on experience and have remained unchanged for over 50 years. Although ASD-designed structures have performed adequately over the years, the actual level of safety provided was never known. This was a principal drawback of the traditional ASD approach. An illustration of typical performance data is provided in Bjorhovde (1978), where theoretical and actual safety factors for columns are examined.

Design for strength by ASD is performed in accordance with Equation B3-2. The ASD method provided in the Specification recognizes that the controlling modes of failure are the same for structures designed by ASD and LRFD. Thus, the nominal strength that forms the foundation of LRFD is the same nominal strength that provides the foundation for ASD. When considering available strength, the only difference between the two methods is the resistance factor in LRFD, ϕ, and the safety factor in ASD, Ω.

In developing appropriate values of Ω for use in this Specification, the aim was to ensure similar levels of safety and reliability for the two methods. A straightforward approach for relating the resistance factor and the safety factor was developed. As already mentioned, the original LRFD *Specification* (AISC, 1986) was calibrated to the 1978 ASD *Specification* (AISC, 1978) at a live load-to-dead load ratio of 3. Thus, by equating the designs for the two methods at a ratio of live-to-dead load of 3, the relationship between ϕ and Ω can be determined. Using the live plus dead load combinations, with $L = 3D$, yields the following relationships.

For design according to Section B3.1 (LRFD)

$$\phi R_n = 1.2D + 1.6L = 1.2D + 1.6(3D) = 6D \tag{C-B3-3}$$

$$R_n = \frac{6D}{\phi}$$

For design according to Section B3.2 (ASD)

$$\frac{R_n}{\Omega} = D + L = D + 3D = 4D \tag{C-B3-4}$$

$$R_n = \Omega\ (4D)$$

Equating R_n from the LRFD and ASD formulations and solving for Ω yields

$$\Omega = \frac{6D}{\phi}\left(\frac{1}{4D}\right) = \frac{1.5}{\phi} \tag{C-B3-5}$$

Throughout this Specification, the values of Ω were obtained from the values of ϕ by Equation C-B3-5.

3. Required Strength

This Specification permits the use of elastic or inelastic, which includes plastic, structural analysis. Generally, design is performed by elastic analysis. Provisions for inelastic and plastic analysis are given in Appendix 1. The required strength is determined by the appropriate methods of structural analysis.

In some circumstances, as in the proportioning of stability bracing members that carry no calculated forces (see, for example, Appendix 6), the required strength is explicitly stated in this Specification.

A beam that is reliably restrained at one or both ends by connection to other members or by a support will have reserve capacity past yielding at the point with the greatest moment predicted by an elastic analysis. The additional capacity is the result of inelastic redistribution of moments. This Specification bases the design of the member on providing a resisting moment greater than the demand represented by the greatest moment predicted by the elastic analysis. This approach ignores the reserve capacity associated with inelastic redistribution. The 10% reduction of the greatest moment, predicted by elastic analysis with the accompanying 10% increase in the moment on the reverse side of the moment diagram, is an attempt to account approximately for this reserve capacity.

This adjustment is appropriate only for cases where the inelastic redistribution of moments is possible. For statically determinate spans (e.g., beams that are simply supported at both ends or for cantilevers), redistribution is not possible; therefore, the adjustment is not allowable in these cases. Members with fixed ends or beams continuous over a support can sustain redistribution. Members with cross sections that are unable to accommodate the inelastic rotation associated with the redistribution (e.g., because of local buckling) are also not permitted to use this redistribution. Thus, only compact sections qualify for redistribution in this Specification.

An inelastic analysis will automatically account for any redistribution. Therefore, the redistribution of moments only applies to moments computed from an elastic analysis.

The 10% reduction rule applies only to beams. Inelastic redistribution is possible in more complicated structures, but the 10% amount is only verified, at present, for beams. For other structures, the provisions of Appendix 1 should be used.

4. Design of Connections and Supports

This section provides the charging language for Chapter J and Chapter K on the design of connections and supports. Chapter J covers the proportioning of the individual elements of a connection (angles, welds, bolts, etc.) once the load effects on the connection are known. According to the provisions of this section, the modeling assumptions associated with the structural analysis must be consistent with the conditions used in Chapter J to proportion the connecting elements.

In many situations, it is not necessary to include the connection elements as part of the analysis of the structural system. For example, simple and fully restrained (FR) connections may often be idealized as pinned or fixed, respectively, for the purposes of structural analysis. Once the analysis has been completed, the deformations or forces computed at the joints may be used to proportion the connection elements. The classifications of FR and simple connections are meant to justify these idealizations for analysis with the provision that if, for example, one assumes a connection to be FR for the purposes of analysis, the actual connection must meet the FR conditions. In other words, it must have adequate strength and stiffness, as described in the provisions and discussed in the following.

In certain cases, the deformation of the connection elements affects the way the structure resists load and hence the connections must be included in the analysis of the structural system. These connections are referred to as partially restrained (PR) moment connections. For structures with PR connections, the connection flexibility must be estimated and included in the structural analysis, as described in the following sections. Once the analysis is complete, the load effects and deformations computed for the connection can be used to check the adequacy of the connecting elements.

For simple and FR connections, the connection proportions are established after the final analysis of the structural design is completed, thereby greatly simplifying the design cycle. In contrast, the design of PR connections (like member selection) is inherently iterative because one must assume values of the connection proportions in order to establish the force-deformation characteristics of the connection needed to perform the structural analysis. The life-cycle performance characteristics must also be considered. The adequacy of the assumed proportions of the connection elements can be verified once the outcome of the structural analysis is known. If the connection elements are inadequate, then the values must be revised and the structural analysis repeated. The potential benefits of using PR connections for various types of framing systems are discussed in the literature referenced in the following.

Connection Classification. The basic assumption made in classifying connections is that the most important behavioral characteristics of the connection can be modeled by a moment-rotation (M-θ) curve. Figure C-B3.2 shows a typical M-θ curve. Implicit in the moment-rotation curve is the definition of the connection as being a region of the column and beam along with the connecting elements. The connection response is defined this way because the rotation of the member in a physical test is generally measured over a length that incorporates the contributions of not only the connecting elements, but also the ends of the members being connected and the column panel zone.

Examples of connection classification schemes include those in Bjorhovde et al. (1990) and Eurocode 3 (CEN, 2005a). These classifications account directly for the stiffness, strength and ductility of the connections.

Connection Stiffness. Because the nonlinear behavior of the connection manifests itself even at low moment-rotation levels, the initial stiffness of the connection, K_i, (shown in Figure C-B3.2) does not adequately characterize connection response at service levels. Furthermore, many connection types do not exhibit a reliable initial stiffness, or it exists only for a very small moment-rotation range. The secant stiffness, K_s, at service loads is taken as an index property of connection stiffness. Specifically,

$$K_s = M_s / \theta_s \tag{C-B3-6}$$

where

M_s = moment at service loads, kip-in. (N-mm)

θ_s = rotation at service loads, rad

In the following discussion, L and EI are the length and bending rigidity, respectively, of the beam.

If $K_s L / EI \geq 20$, it is acceptable to consider the connection to be fully restrained (in other words, able to maintain the angles between members). If $K_s L / EI \leq 2$, it is acceptable to consider the connection to be simple (in other words, it rotates without developing moment). Connections with stiffnesses between these two limits are partially restrained and the stiffness, strength and ductility of the connection must be considered in the design (Leon, 1994). Examples of FR, PR and simple connection response curves are shown in Figure C-B3.3. The points marked θ_s indicate the service load states for the example connections and thereby define the secant stiffnesses for those connections.

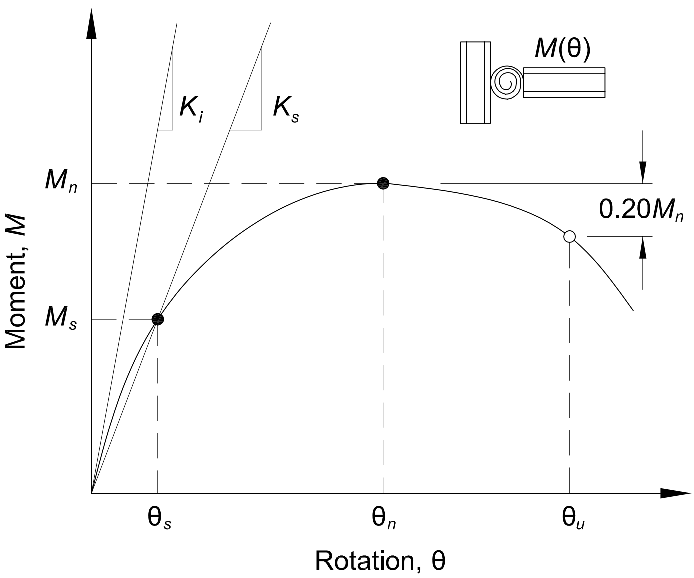


Fig. C-B3.2. Definition of stiffness, strength and ductility characteristics of the moment-rotation response of a partially restrained connection.

Connection Strength. The strength of a connection is the maximum moment that it is capable of carrying, M_n, as shown in Figure C-B3.2. The strength of a connection can be determined on the basis of an ultimate limit-state model of the connection, or from physical tests. If the moment-rotation response does not exhibit a peak load then the strength can be taken as the moment at a rotation of 0.02 rad (Hsieh and Deierlein, 1991; Leon et al., 1996).

It is also useful to define a lower limit on strength below which the connection may be treated as a simple connection. Connections that transmit less than 20% of the fully plastic moment of the beam at a rotation of 0.02 rad may be considered to have no flexural strength for design. However, it should be recognized that the aggregate strength of many weak connections can be important when compared to that of a few strong connections (FEMA, 1997).

In Figure C-B3.3, the points marked M_n indicate the maximum strength states of the example connections. The points marked θ_u indicate the maximum rotation states of the example connections. Note that it is possible for an FR connection to have a strength less than the strength of the beam. It is also possible for a PR connection to have a strength greater than the strength of the beam. The strength of the connection must be adequate to resist the moment demands implied by the design loads.

Connection Ductility. If the connection strength substantially exceeds the fully plastic moment strength of the beam, then the ductility of the structural system is controlled by the beam and the connection can be considered elastic. If the connection strength

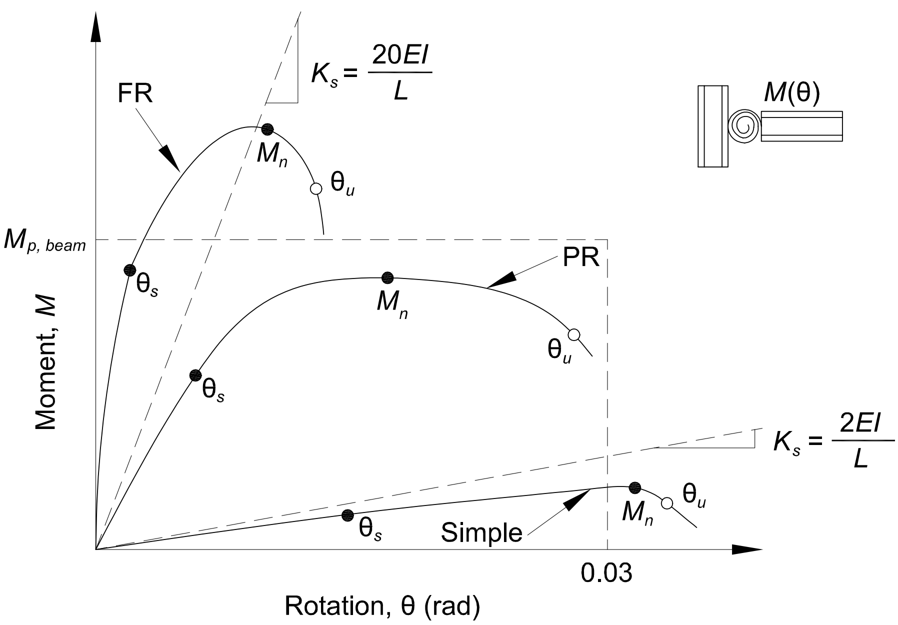


Fig. C-B3.3. Classification of moment-rotation response of fully restrained (FR), partially restrained (PR), and simple connections.

only marginally exceeds the fully plastic moment strength of the beam, then the connection may experience substantial inelastic deformation before the beam reaches its full strength. If the beam strength exceeds the connection strength, then deformations can concentrate in the connection. The ductility required of a connection will depend upon the particular application. For example, the ductility requirement for a braced frame in a nonseismic area will generally be less than the ductility required in a high seismic area. The rotation ductility requirements for seismic design depend upon the structural system (AISC, 2016b).

In Figure C-B3.2, the rotation capacity, θ_u, can be defined as the value of the connection rotation at the point where either (a) the resisting strength of the connection has dropped to $0.8M_n$ or (b) the connection has deformed beyond 0.03 rad. This second criterion is intended to apply to connections where there is no loss in strength until very large rotations occur. It is not prudent to rely on these large rotations in design.

The available rotation capacity, θ_u, should be compared with the rotation required at the strength limit state, as determined by an analysis that takes into account the nonlinear behavior of the connection. (Note that for design by ASD, the rotation required at the strength limit state should be assessed using analyses conducted at 1.6 times the ASD load combinations.) In the absence of an accurate analysis, a rotation capacity of 0.03 rad is considered adequate. This rotation is equal to the minimum beam-to-column connection capacity as specified in the seismic provisions for special moment frames (AISC, 2016b). Many types of PR connections, such as top and seat-angle connections, meet this criterion.

Structural Analysis and Design. When a connection is classified as PR, the relevant response characteristics of the connection must be included in the analysis of the structure to determine the member and connection forces, displacements, and the frame stability. Therefore, PR construction requires, first, that the moment-rotation characteristics of the connection be known and, second, that these characteristics be incorporated in the analysis and member design.

Typical moment-rotation curves for many PR connections are available from one of several databases (Goverdhan, 1983; Ang and Morris, 1984; Nethercot, 1985; Kishi and Chen, 1986). Care should be exercised when utilizing tabulated moment-rotation curves not to extrapolate to sizes or conditions beyond those used to develop the database because other failure modes may control (ASCE, 1997). When the connections to be modeled do not fall within the range of the databases, it may be possible to determine the response characteristics from tests, simple component modeling, or finite element studies (FEMA, 1995). Examples of procedures to model connection behavior are given in the literature (Bjorhovde et al., 1988; Chen and Lui, 1991; Bjorhovde et al., 1992; Lorenz et al., 1993; Chen and Toma, 1994; Chen et al., 1995; Bjorhovde et al., 1996; Leon et al., 1996; Leon and Easterling, 2002; Bijlaard et al., 2005; Bjorhovde et al., 2008).

The degree of sophistication of the analysis depends on the problem at hand. Design for PR construction usually requires separate analyses for the serviceability and strength limit states. For serviceability, an analysis using linear springs with a stiffness given by K_s (see Figure C-B3.2) is sufficient if the resistance demanded of the connection is well below the strength. When subjected to strength load combinations, a procedure is needed whereby the characteristics assumed in the analysis are consistent with those of the connection response. The response is especially nonlinear as the applied moment approaches the connection strength. In particular, the effect of the connection nonlinearity on second-order moments and other stability checks needs to be considered (ASCE, 1997). The use of the direct analysis method with PR connections has been demonstrated (Surovek et al., 2005; White and Goverdhan, 2008).

5. Design of Diaphragms and Collectors

This section provides charging language for the design of structural steel components (members and their connections) of diaphragms and collector systems.

Diaphragms transfer in-plane lateral loads to the lateral force-resisting system. Typical diaphragm elements in a building structure are the floor and roof systems, which accumulate lateral forces due to gravity, wind and/or seismic loads, and distribute these forces to individual elements (braced frames, moment frames, shear walls, etc.) of the vertically oriented lateral force-resisting system of the building structure. Collectors (also known as drag struts) are often used to collect and deliver diaphragm forces to the lateral force-resisting system.

Diaphragms are classified into one of three categories: rigid, semi-rigid or flexible. Rigid diaphragms distribute the in-plane forces to the lateral force-resisting system with negligible in-plane deformation of the diaphragm. A rigid diaphragm may be assumed to distribute the lateral loads in proportion to the relative stiffness of the individual elements of the lateral force-resisting system. A semi-rigid diaphragm distributes the lateral loads in proportion to the in-plane stiffness of the diaphragm and the relative stiffness of the individual elements of the lateral force-resisting system. The in-plane stiffness of a flexible diaphragm is negligible compared to the stiffness of the lateral force-resisting system and, therefore, the distribution of lateral forces is independent of the relative stiffness of the individual elements of the lateral force-resisting system. In this case, the distribution of lateral forces may be computed in a manner analogous to a series of simple beams spanning between the lateral force-resisting system elements.

Diaphragms should be designed for the shear, moment and axial forces resulting from the design loads. The diaphragm response may be considered analogous to a deep beam where the flanges (often referred to as chords of the diaphragm) develop tension and compression forces, and the web resists the shear. The component elements of the diaphragm need to have strength and deformation capacity consistent with assumptions and intended behavior.

6. Design of Anchorages to Concrete

This section provides the charging language for Chapter I and Chapter J on design of anchorages to concrete.

7. Design for Stability

This section provides the charging language for Chapter C on design for stability.

8. Design for Serviceability

This section provides the charging language for Chapter L on design for serviceability.

9. Design for Structural Integrity

This section provides the minimum connection design criteria for satisfying structural integrity requirements where required by the applicable building code. Section 1615 of the International Building Code (ICC, 2015) assigns structural integrity requirements to high-rise buildings in risk category III or IV, which means that the number of buildings to which this requirement currently applies is limited.

Evaluation of built structures that have been subjected to extraordinary events indicates that structures that have a higher level of connectivity perform better than those that do not. The intent of the integrity requirements is to achieve this improved connectivity by limiting the possibility of a connection failure when it is subjected to unanticipated tension forces. The forces can result from a wide range of events such as cool-down after a fire, failure of adjacent structural members, and blast or impact loads on columns. The Specification integrity checks are similar in principle to those defined in other model codes and international codes which have provided good historical performance (Geschwindner and Gustafson, 2010). The fundamental aspect of the integrity requirement is that it is a connection design requirement only and is not a design force applied to any part of the structure other than the connection itself. In addition, the forces determined for the integrity check are not to be combined with any other forces and the integrity connection design check is to be conducted separately. The structural integrity requirements are a detailing requirement for the connection and not a load or force applied to the structure.

Section B3.9(a) provides the nominal tensile strength for column splices. The intent of this requirement is to provide a minimum splice capacity for the resistance of unanticipated forces. This requirement is based on the assumption that two floors are supported by the splice. Any live load reduction should be the same as that used for the design of the connections of the floor members framing to the column. The tension design force should be distributed reasonably uniformly between the flanges and web so that some bending and shear capacity is provided in addition to the tension capacity. A load path for this tension force does not need to be provided.

Section B3.9(b) provides the minimum nominal axial tensile strength of the end connection of beams that frame to girders and also for beams or girders that frame to columns. Geschwindner and Gustafson (2010) have shown that single-plate connections designed to resist shear according to this Specification will satisfy this requirement. Since inelastic deformation is permitted for the integrity check, it is expected that most other framed connections, such as double-angle connections, can be shown to satisfy this requirement through nonlinear analysis or yield line analysis. The forces determined in this section are to be applied to only the connection design itself and are not to be included in the member design. In particular, checking the local bending of column and beam webs induced by the tension is not required by this section.

Section B3.9(c) provides the minimum nominal tensile force to brace columns. Maintaining column bracing is one of the fundamental principles for providing structural integrity. Since column bracing elements are usually much lighter than the column, extraordinary events have more potential to affect the bracing member or the slab surrounding the column than the column itself. This is the reason that the steel connection itself is required to provide the bracing force. The assumption is that the extraordinary event has compromised the ability of the column to be braced by the slab or by one of the beams framing to the column. This tensile bracing force requirement is to be applied separately from other bracing requirements, as specified in Appendix 6. Note that the requirements of this section will usually govern for the lower stories of high-rise buildings, whereas Section B3.9(a) will govern in most other situations.

Although the integrity requirements need be applied only when required by code, they should be considered for any building where improved structural performance under undefined extraordinary events is desired. For structures that have a defined extraordinary load, reference should be made to ASCE/SEI 7. For structures that are required to be designed to resist progressive (disproportionate) collapse, reference should be made to the ASCE/SEI 7 Commentary.

10. Design for Ponding

As used in this Specification, ponding refers to the retention of water due solely to the deflection of flat roof framing. The amount of accumulated water is dependent on the stiffness of the framing. Unbounded incremental deflections due to the incremental increase in retained water can result in the collapse of the roof. The problem becomes catastrophic when more water causes more deflection, resulting in more room for more water until the roof collapses.

Previous editions of this Specification suggested that ponding instability could be avoided by providing a minimum roof slope of 1/4 in. per ft (20 mm per meter). There are cases where this minimum roof slope is not enough to prevent ponding instability (Fisher and Pugh, 2007). This edition of the Specification requires that design for ponding be considered if water is impounded on the roof, irrespective of roof slope. Camber and deflections due to loads acting concurrently with rain loads must be considered in establishing the initial conditions.

Determination of ponding stability is typically done by structural analysis where the rain loads are increased by the incremental deflections of the framing system to the accumulated rain water, assuming the primary roof drains are blocked.

Detailed provisions and design aids for determining ponding stability and strength are given in Appendix 2.

11. Design for Fatigue

This section provides the charging language for Appendix 3 on design for fatigue.

12. Design for Fire Conditions

This section provides the charging language for Appendix 4 on structural design for fire resistance. Qualification testing is an acceptable alternative to design by analysis for providing fire resistance. Qualification testing is addressed in ASCE/SEI/SFPE Standard 29 (ASCE, 2008), ASTM E119 (ASTM, 2009b), and similar documents.

13. Design for Corrosion Effects

Steel members may deteriorate in some service environments. This deterioration may appear either as external corrosion, which would be visible upon inspection, or in undetected changes that would reduce member strength. The designer should recognize these problems by either factoring a specific amount of tolerance for damage into the design or providing adequate protection (for example, coatings or cathodic protection) and/or planned maintenance programs so that such problems do not occur.

Because the interior of an HSS is difficult to inspect, some concern has been expressed regarding internal corrosion. However, good design practice can eliminate the concern and the need for expensive protection. Corrosion occurs in the presence of oxygen and water. In an enclosed building, it is improbable that there would be sufficient reintroduction of moisture to cause severe corrosion. Therefore, internal corrosion protection is a consideration only in HSS exposed to weather.

In a sealed HSS, internal corrosion cannot progress beyond the point where the oxygen or moisture necessary for chemical oxidation is consumed (AISI, 1970). The oxidation depth is insignificant when the corrosion process must stop, even when a corrosive atmosphere exists at the time of sealing. If fine openings exist at connections, moisture and air can enter the HSS through capillary action or by aspiration due to the partial vacuum that is created if the HSS is cooled rapidly (Blodgett, 1967). This can be prevented by providing pressure-equalizing holes in locations that make it impossible for water to flow into the HSS by gravity.

Situations where conservative practice would recommend an internal protective coating include (a) open HSS where changes in the air volume by ventilation or direct flow of water is possible, and (b) open HSS subject to a temperature gradient that would cause condensation.

HSS that are filled or partially filled with concrete should not be sealed. In the event of fire, water in the concrete will vaporize and may create pressure sufficient to burst a sealed HSS. Care should be taken to keep water from remaining in the HSS during or after construction, since the expansion caused by freezing can create pressure that is sufficient to burst an HSS.

Galvanized HSS assemblies should not be completely sealed because rapid pressure changes during the galvanizing process tend to burst sealed assemblies.

B4. MEMBER PROPERTIES

1. Classification of Sections for Local Buckling

Cross sections with a limiting width-to-thickness ratio, λ, greater than those provided in Table B4.1 are subject to local buckling limit states. Since the 2010 AISC *Specification* (AISC, 2010), Table B4.1 has been separated into two parts: B4.1a for compression members and B4.1b for flexural members. Separation of Table B4.1 into two parts reflects the fact that compression members are only categorized as either slender or nonslender, while flexural members may be slender, noncompact or compact. In addition, separation of Table B4.1 into two parts clarifies ambiguities in λ_r. The width-to-thickness ratio, λ_r, may be different for columns and beams, even for the same element in a cross section, reflecting both the underlying stress state of the connected elements and the different design methodologies between columns (Chapter E and Appendix 1) and beams (Chapter F and Appendix 1).

Limiting Width-to-Thickness Ratios for Compression Elements in Members Subject to Axial Compression. Compression members containing any elements with width-to-thickness ratios greater than λ_r provided in Table B4.1a are designated as slender and are subject to the local buckling reductions detailed in Section E7. Nonslender compression members (all elements having width-to-thickness ratio $\leq \lambda_r$) are not subject to local buckling reductions.

Flanges of Built-Up I-Shaped Sections. In the 1993 *Load and Resistance Factor Design Specification for Structural Steel Buildings* (AISC, 1993), for built-up I-shaped sections under axial compression (Case 2 in Table B4.1a), modifications were made to the flange local buckling criterion to include web-flange interaction. The k_c in the λ_r limit is the same as that used for flexural members. Theory indicates that the web-flange interaction in axial compression is at least as severe as in flexure. Rolled shapes are excluded from this provision because there are no standard sections with proportions where the interaction would occur at commonly available yield stresses. In built-up sections where the interaction causes a reduction in the flange local buckling strength, it is likely that the web is also a thin stiffened element. The k_c factor accounts for the interaction of flange and web local buckling demonstrated in experiments reported in Johnson (1985). The maximum limit of 0.76 corresponds to $F_{cr} = 0.69E/\lambda^2$, which was used as the local buckling strength in earlier editions of both the ASD and LRFD Specifications. An $h/t_w = 27.5$ is required to reach $k_c = 0.76$. Fully fixed restraint for an unstiffened compression element corresponds to $k_c = 1.3$ while zero restraint gives $k_c = 0.42$. Because of web-flange interactions, it is possible to get $k_c < 0.42$ from the k_c formula. If $h/t_w > 5.70\sqrt{E/F_y}$, use $h/t_w = 5.70\sqrt{E/F_y}$ in the k_c equation, which corresponds to the 0.35 limit.

Rectangular HSS in Compression. The limits for rectangular HSS walls in uniform compression (Case 6 in Table B4.1a) have been used in AISC Specifications since 1969 (AISC, 1969). They are based on Winter (1968), where adjacent stiffened compression elements in box sections of uniform thickness were observed to provide negligible torsional restraint for one another along their corner edges.

Round HSS in Compression. The λ_r limit for round HSS in compression (Case 9 in Table B4.1a) was first used in the 1978 *Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings* (AISC, 1978). It was recommended in Schilling (1965) based upon research reported in Winter (1968). Excluding the use of round HSS with $D/t > 0.45E/F_y$ was also recommended in Schilling (1965). This is implied in Sections E7 and F8 where no criteria are given for round HSS with D/t greater than this limit.

Limiting Width-to-Thickness Ratios for Compression Elements in Members Subject to Flexure. Flexural members containing compression elements, all with width-to-thickness ratios less than or equal to λ_p as provided in Table B4.1b, are designated as compact. Compact sections are capable of developing a fully plastic stress distribution and they possess a rotation capacity, R_{cap}, of approximately 3 (see Figure C-A-1.2) before the onset of local buckling (Yura et al., 1978). Flexural members containing any compression element with width-to-thickness ratios greater than λ_p, but still with all compression elements having width-to-thickness ratios less than or equal to λ_r, are designated as noncompact. Noncompact sections can develop partial yielding in compression elements before local buckling occurs, but will not resist inelastic local buckling at the strain levels required for a fully plastic stress distribution. Flexural members containing any compression elements with width-to-thickness ratios greater than λ_r are designated as slender. Slender-element sections have one or more compression elements that will buckle elastically before the yield stress is achieved. Noncompact and slender-element sections are subject to flange local buckling and/or web local buckling reductions as provided in Chapter F and summarized in Table User Note F1.1, or in Appendix 1.

The values of the limiting ratios, λ_p and λ_r, specified in Table B4.1b are similar to those in the 1989 *Specification for Structural Steel Buildings—Allowable Stress Design and Plastic Design* (AISC, 1989) and Table 2.3.3.3 of Galambos (1978), except that $\lambda_p = 0.38\sqrt{E/F_y}$, limited in Galambos (1978) to determinate beams and to indeterminate beams when moments are determined by elastic analysis, was adopted for all conditions on the basis of Yura et al. (1978). For greater inelastic rotation capacities than provided by the limiting value of λ_p given in Table B4.1b, and/or for structures in areas of high seismicity, see Chapter D and Table D1.1 of the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2016b).

Webs in Flexure. In the 2010 *Specification for Structural Steel Buildings* (AISC, 2010), formulas for λ_p were added as Case 16 in Table B4.1b for I-shaped beams with unequal flanges based on White (2008). In extreme cases where the plastic neutral axis is located in the compression flange, $h_p = 0$ and the web is considered to be compact.

Rectangular HSS in Flexure. The λ_p limit for compact sections is adopted from *Limit States Design of Steel Structures* (CSA, 2009). Lower values of λ_p are specified for high-seismic design in the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2016b) based upon tests (Lui and Goel, 1987) that have shown that rectangular HSS braces subjected to reversed axial load fracture catastrophically under relatively few cycles if a local buckle forms. This was confirmed in tests (Sherman, 1995a) where rectangular HSS braces sustained over 500 cycles when a local buckle did not form, even though general column buckling had occurred, but failed in less than 40 cycles when a local buckle developed. Since 2005, the λ_p limit for webs in rectangular HSS flexural members (Case 19 in Table B4.1b) has been reduced from $\lambda_p = 3.76\sqrt{E/F_y}$ to $\lambda_p = 2.42\sqrt{E/F_y}$ based on the work of Wilkinson and Hancock (1998, 2002).

Box Sections in Flexure. In the 2016 *Specification*, box sections are defined separately from rectangular HSS. Thus, Case 21 has been added to Table B4.1b for flanges of box sections and box sections have been included in Case 19 for webs.

Round HSS in Flexure. The λ_p values for round HSS in flexure (Case 20, Table B4.1b) are based on Sherman (1976), Sherman and Tanavde (1984), and Ziemian (2010). Beyond this, the local buckling strength decreases rapidly, making it impractical to use these sections in building construction.

2. Design Wall Thickness for HSS

ASTM A500/A500M tolerances allow for a wall thickness that is not greater than ±10% of the nominal value. Because the plate and strip from which these HSS are made are produced to a much smaller thickness tolerance, manufacturers in the consistently produce these HSS with a wall thickness that is near the lower-bound wall thickness limit. Consequently, AISC and the Steel Tube Institute of North America (STI) recommend that 0.93 times the nominal wall thickness be used for calculations involving engineering design properties of these HSS. This results in a weight (mass) variation that is similar to that found in other structural shapes. The design wall thickness and section properties based upon this reduced thickness have been tabulated in AISC and STI publications since 1997.

Two new HSS material standards have been added to the 2016 *Specification*. ASTM A1085/A1085M is a standard in which the wall thickness is permitted to be no more than 5% under the nominal thickness and the mass is permitted to be no more than 3.5% under the nominal mass. This is in addition to a Charpy V-notch toughness limit and a limit on the range of yield strength that makes A1085/A1085M suitable for seismic applications. With these tolerances, the design wall thickness may be taken as the nominal thickness of the HSS. Other acceptable HSS products that do not have the same thickness and mass tolerances must still use the design thickness as 0.93 times the nominal thickness as discussed previously.

The other new material standard is ASTM A1065/A1065M. These HSS are produced by cold-forming two C-shaped sections and joining them with two electric-fusion seam welds to form a square or rectangular HSS. These sections are available in larger sizes than those produced in a tube mill. Since the thickness meets plate tolerance limits, the design wall thickness may be taken as the nominal thickness. In previous Specifications, they were classified as box sections because they were not produced according to an ASTM standard. With the new ASTM A1065/1065M standard, they are included as acceptable HSS and the term box section is used for sections made by corner welding four plates to form a hollow box.

3. Gross and Net Area Determination

3a. Gross Area

Gross area is the total area of the cross section without deductions for holes or ineffective portions of elements subject to local buckling.

3b. Net Area

The net area is based on net width and load transfer at a particular chain. Because of possible damage around a hole during drilling or punching operations, $^1/_{16}$ in. (2 mm) is added to the nominal hole diameter when computing the net area.

B5. FABRICATION AND ERECTION

Section B5 provides the charging language for Chapter M on fabrication and erection.

B6. QUALITY CONTROL AND QUALITY ASSURANCE

Section B6 provides the charging language for Chapter N on quality control and quality assurance.

B7. EVALUATION OF EXISTING STRUCTURES

Section B7 provides the charging language for Appendix 5 on the evaluation of existing structures.

CHAPTER C

DESIGN FOR STABILITY

Design for stability is the combination of analysis to determine the required strengths of components and proportioning of components to have adequate available strengths. Various methods are available to provide for stability (Ziemian, 2010).

Chapter C addresses the stability design requirements for steel buildings and other structures. It is based upon the direct analysis method, which can be used in all cases. The effective length method and first-order analysis method are addressed in Appendix 7 as alternative methods of design for stability, and may be used when the limits in Appendix 7, Sections 7.2.1 and 7.3.1, respectively, are satisfied. A complete discussion of each of these methods, along with example problems, may be found in AISC Design Guide 28, *Stability Design of Steel Buildings* (Griffis and White, 2013). Other approaches are permitted provided the general requirements in Section C1 are satisfied. For example, Appendix 1 provides logical extensions to the direct analysis method, in which design provisions are provided for explicitly modeling member imperfections and/or inelasticity. First-order elastic structural analysis without stiffness reductions for inelasticity is not sufficient to assess stability because the analysis and the equations for component strengths are inextricably interdependent.

C1. GENERAL STABILITY REQUIREMENTS

There are many parameters and behavioral effects that influence the stability of steel-framed structures (Birnstiel and Iffland, 1980; McGuire, 1992; White and Chen, 1993; ASCE, 1997; Ziemian, 2010). The stability of structures and individual elements must be considered from the standpoint of the structure as a whole, including not only compression members, but also beams, bracing systems and connections.

Stiffness requirements for control of seismic drift are included in many building codes that prohibit sidesway amplification, $\Delta_{2nd\text{-}order}/\Delta_{1st\text{-}order}$ or B_2, calculated with nominal stiffness, from exceeding approximately 1.5 to 1.6 (ICC, 2015). This limit usually is well within the more general recommendation that sidesway amplification, calculated with reduced stiffness, should be equal to or less than 2.5. The latter recommendation is made because at larger levels of amplification, small changes in gravity loads and/or structural stiffness can result in relatively larger changes in sidesway deflections and second-order effects, due to large geometric nonlinearities.

Table C-C1.1 shows how the five general requirements provided in Section C1 are addressed in the direct analysis method (Sections C2 and C3) and the effective length method (Appendix 7, Section 7.2). The first-order analysis method (Appendix 7, Section 7.3) is not included in Table C-C1.1 because it addresses these requirements in an indirect manner using a mathematical manipulation of the direct analysis method. The additional lateral load required in Appendix 7, Section 7.3.2(a) is calibrated to achieve roughly the same result as the collective effects of notional loads

<table>
<tr><th colspan="4">TABLE C-C1.1
Comparison of Basic Stability Requirements
with Specific Provisions</th></tr>
<tr><th colspan="2">Basic Requirement in Section C1</th><th>Provision in Direct Analysis Method (DM)</th><th>Provision in Effective Length Method (ELM)</th></tr>
<tr><td colspan="2">(1) Consider all deformations</td><td>C2.1(a). Consider all deformations</td><td>Same as DM
(by reference to C2.1)</td></tr>
<tr><td colspan="2">(2) Consider second-order effects (both P-Δ and P-δ)</td><td>C2.1(b). Consider second-order effects (P-Δ and P-δ)[b]</td><td>Same as DM
(by reference to C2.1)</td></tr>
<tr><td rowspan="3">(3) Consider geometric imperfections
This includes joint-position imperfections[a] (which affect structure response) and member imperfections (which affect structure response and member strength)</td><td>Effect of system imperfections on structure response</td><td>C2.2a. Direct modeling
or
C2.2b. Notional loads</td><td>Same as DM, second option only (by reference to C2.2b)</td></tr>
<tr><td>Effect of member imperfections on structure response</td><td>Included in the stiffness reduction specified in C2.3</td><td rowspan="6">All these effects are considered by using $L_c = KL$ from a sidesway buckling analysis in the member strength check. Note that the differences between DM and ELM are:
• DM uses reduced stiffness in the analysis and $L_c = L$ in the member strength check
• ELM uses full stiffness in the analysis and $L_c = KL$ from sidesway buckling analysis in the member strength check</td></tr>
<tr><td>Effect of member imperfections on member strength</td><td>Included in member strength formulas, with $L_c = L$</td></tr>
<tr><td rowspan="2">(4) Consider stiffness reduction due to inelasticity
This affects structure response and member strength</td><td>Effect of stiffness reduction on structure response</td><td>Included in the stiffness reduction specified in C2.3</td></tr>
<tr><td>Effect of stiffness reduction on member strength</td><td>Included in member strength formulas, with $L_c = L$</td></tr>
<tr><td rowspan="2">(5) Consider uncertainty in strength and stiffness
This affects structure response and member strength</td><td>Effect of stiffness/strength uncertainty on structure response</td><td>Included in the stiffness reduction specified in C2.3</td></tr>
<tr><td>Effect of stiffness/strength uncertainty on member strength</td><td>Included in member strength formulas, with $L_c = L$</td></tr>
<tr><td colspan="4">[a] In typical building structures, the “joint-position imperfections” refers to column out-of-plumbness.
[b] Second-order effects may be considered either by a computational P-Δ and P-δ analysis or by the approximate method (using B_1 and B_2 multipliers) specified in Appendix 8.</td></tr>
</table>

required in Section C2.2b, P-Δ effects required in Section C2.1(b), and the stiffness reduction required in Section C2.3. Additionally, a B_1 multiplier addresses P-δ effects as defined in Appendix 8, Section 8.2.1.

In the 2010 AISC *Specification* (AISC, 2010), uncertainties in stiffness and strength was added to the list of effects that should be considered when designing for stability. Although all methods detailed in this Specification, including the direct analysis method, the effective length method, and the first-order elastic method, satisfy this requirement, the effect is listed to ensure that it is included, along with the original four other effects, when any other rational method of designing for stability is employed.

C2. CALCULATION OF REQUIRED STRENGTHS

Analysis to determine required strengths in accordance with this Section and the assessment of member and connection available strengths in accordance with Section C3 form the basis of the direct analysis method of design for stability. This method is useful for the stability design of all structural steel systems, including moment frames, braced frames, shear walls, and combinations of these and similar systems (AISC-SSRC, 2003a). While the precise formulation of this method is unique to the AISC *Specification*, some of its features are similar to those found in other major design specifications around the world, including the Eurocodes, the Australian standard, the Canadian standard, and ACI 318 (ACI, 2014).

The direct analysis method allows a more accurate determination of the load effects in the structure through the inclusion of the effects of geometric imperfections and stiffness reductions directly within the structural analysis. This also allows the use of $K = 1.0$ in calculating the in-plane column strength, P_c, within the beam-column interaction equations of Chapter H. This is a significant simplification in the design of steel moment frames and combined systems. Verification studies for the direct analysis method are provided by Deierlein et al. (2002), Maleck and White (2003), and Martinez-Garcia and Ziemian (2006).

1. General Analysis Requirements

Deformations to be Considered in the Analysis. It is required that the analysis consider flexural, shear and axial deformations, and all other component and connection deformations that contribute to the displacement of the structure. However, it is important to note that "consider" is not synonymous with "include," and some deformations can be neglected after rational consideration of their likely effect. For example, the in-plane deformation of a concrete-on-steel deck floor diaphragm in an office building usually can be neglected, but that of a cold-formed steel roof deck in a large warehouse with widely spaced lateral force-resisting elements usually cannot. As another example, shear deformations in beams and columns in a low-rise moment frame usually can be neglected, but this may not be true in a high-rise framed-tube system with relatively deep members and short spans. For such frames, the use of rigid offsets to account for member depths may significantly overestimate frame stiffness and consequently underestimate second-order effects due to high shear stresses within the panel zone of the connections. For example, Charney and Johnson (1986) found that for the range of columns and beam sizes they studied the deflections of a subassembly modeled using centerline dimensions could vary from an overestimation of 23% to an underestimation of 20% when compared to a finite element model. Charney and Johnson conclude that analysis based on centerline dimensions may either underestimate or overestimate drift, with results depending on the span of the girder and on the web thickness of the column.

Second-Order Effects. The direct analysis method includes the basic requirement to calculate the internal load effects using a second-order analysis that accounts for both P-Δ and P-δ effects (see Figure C-C2.1). P-Δ effects are the effects of loads acting on the displaced location of joints or member-end nodes in a structure. P-δ effects are the effect of loads acting on the deflected shape of a member between joints or member-end nodes.

Many, but not all, modern commercial structural analysis programs are capable of accurately and directly modeling all significant P-Δ and P-δ second-order effects. Programs that accurately estimate second-order effects typically solve the governing differential equations either through the use of a geometric stiffness approach (McGuire et al., 2000; Ziemian, 2010) or the use of stability functions (Chen and Lui, 1987). What is, and just as importantly what is not, included in the analysis should be verified by the user for each particular program. Some programs neglect P-δ effects in the analysis of the structure, and because this is a common approximation that is permitted under certain conditions, it is discussed at the end of this section.

Methods that modify first-order analysis results through second-order multipliers are permitted. The use of the B_1 and B_2 multipliers provided in Appendix 8 is one such method. The accuracy of other methods should be verified.

Analysis Benchmark Problems. The following benchmark problems are recommended as a first-level check to determine whether an analysis procedure meets the requirements of a P-Δ and P-δ second-order analysis adequate for use in the direct analysis method (and the effective length method in Appendix 7). Some second-order analysis procedures may not include the effects of P-δ on the overall response of the structure. These benchmark problems are intended to reveal whether or not these effects are included in the analysis. It should be noted that in accordance with the requirements of Section C2.1(b), it is not always necessary to include P-δ effects in the second-order analysis (additional discussion of the consequences of neglecting these effects will follow).

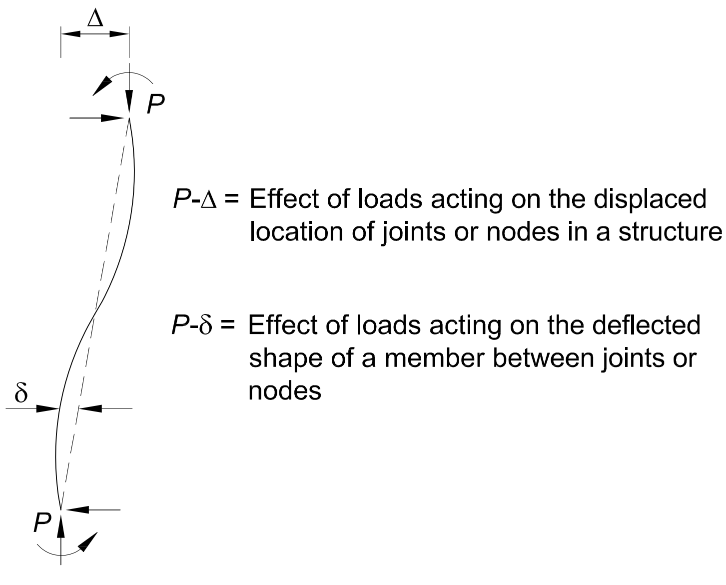
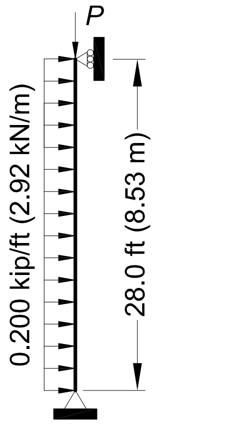


Fig. C-C2.1. P-*Δ and* P-*δ effects in beam-columns.*

The benchmark problem descriptions and solutions are shown in Figures C-C2.2 and C-C2.3. Proportional loading is assumed and axial, flexural and shear deformations are included. Case 1 is a simply supported beam-column subjected to an axial load concurrent with a uniformly distributed transverse load between supports. This problem contains only P-δ effects because there is no translation of one end of the member relative to the other. Case 2 is a fixed-base cantilevered beam-column subjected to an axial load concurrent with a lateral load at its top. This problem contains both P-Δ and P-δ effects. In confirming the accuracy of the analysis method, both moments and deflections should be checked at the locations shown for the various levels of axial load on the member and in all cases should agree within 3% and 5%, respectively.



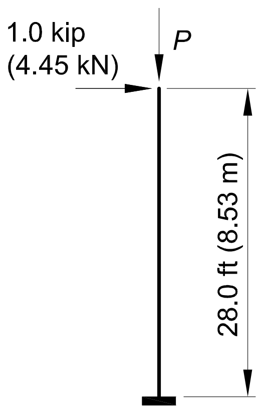
Major axis bending
W14x48 (W360x72)
E=29,000 ksi (200 GPa)

Axial Force, P (kips)	0	150	300	450
M_{mid} (kip-in.)	235 [235]	270 [269]	316 [313]	380 [375]
Δ_{mid} (in.)	0.202 [0.197]	0.230 [0.224]	0.269 [0.261]	0.322 [0.311]

Axial Force, P (kN)	0	667	1334	2001
M_{mid} (kN-m)	26.6 [26.6]	30.5 [30.4]	35.7 [35.4]	43.0 [42.4]
Δ_{mid} (mm)	5.13 [5.02]	5.86 [5.71]	6.84 [6.63]	8.21 [7.91]

Analyses include axial, flexural and shear deformations.
[Values in brackets] exclude shear deformations.

Fig. C-C2.2. Benchmark problem Case 1.



Major axis bending
W14x48 (W360x72)
E = 29,000 ksi (200 GPa)

Axial Force, P (kips)	0	100	150	200
M_{base} (kip-in.)	336 [336]	470 [469]	601 [598]	856 [848]
Δ_{tip} (in.)	0.907 [0.901]	1.34 [1.33]	1.77 [1.75]	2.60 [2.56]

Axial Force, P (kN)	0	445	667	890
M_{base} (kN-m)	38.0 [38.0]	53.2 [53.1]	68.1 [67.7]	97.2 [96.2]
Δ_{tip} (mm)	23.1 [22.9]	34.2 [33.9]	45.1 [44.6]	66.6 [65.4]

Analyses include axial, flexural and shear deformations.
[Values in brackets] exclude shear deformations.

Fig. C-C2.3. Benchmark problem Case 2.

Given that there are many attributes that must be studied to confirm the accuracy of a given analysis method for routine use in the design of general framing systems, a wide range of benchmark problems should be employed. Several other targeted analysis benchmark problems can be found in Kaehler et al. (2010), Chen and Lui (1987), and McGuire et al. (2000). When using benchmark problems to assess the correctness of a second-order procedure, the details of the analysis used in the benchmark study, such as the number of elements used to represent the member and the numerical solution scheme employed, should be replicated in the analysis used to design the actual structure. Because the ratio of design load to elastic buckling load is a strong indicator of the influence of second-order effects, benchmark problems with such ratios on the order of 0.6 to 0.7 should be included.

***Effect of Neglecting* P-δ.** A common type of approximate analysis is one that captures only P-Δ effects due to member end translations (for example, interstory drift) but fails to capture P-δ effects due to curvature of the member relative to its chord. This type of analysis is referred to as a P-Δ analysis. Where P-δ effects are significant, errors arise in approximate methods that do not accurately account for the effect of P-δ moments on amplification of both local (δ) and global (Δ) displacements and corresponding internal moments. These errors can occur both with second-order computer analysis programs and with the B_1 and B_2 amplifiers. For instance, the R_M modifier in Equation A-8-7 is an adjustment factor that approximates the effects of P-δ (due to column curvature) on the overall sidesway displacements, Δ, and the corresponding moments. For regular rectangular moment frames, a single-element-per-member P-Δ analysis is equivalent to using the B_2 amplifier of Equation A-8-6 with $R_M = 1$, and hence, such an analysis neglects the effect of P-δ on the response of the structure.

Section C2.1(b) indicates that a P-Δ-only analysis (one that neglects the effect of P-δ deformations on the response of the structure) is permissible for typical building structures when the ratio of second-order drift to first-order drift is less than 1.7 and no more than one-third of the total gravity load on the building is on columns that are part of moment-resisting frames. The latter condition is equivalent to an R_M value of 0.95 or greater. When these conditions are satisfied, the error in lateral displacement from a P-Δ-only analysis typically will be less than 3%. However, when the P-δ effect in one or more members is large (corresponding to a B_1 multiplier of more than about 1.2), use of a P-Δ-only analysis may lead to larger errors in the nonsway moments in components connected to the high-P-δ members.

The engineer should be aware of this possible error before using a P-Δ-only analysis in such cases. For example, consider the evaluation of the fixed-base cantilevered beam-column shown in Figure C-C2.4 using the direct analysis method. The sidesway displacement amplification factor is 3.83 and the base moment amplifier is 3.32, giving M_u = 1,394 kip-in. (158 kN-m).

For the loads shown, the beam-column strength interaction according to Equation H1-1a is equal to 1.0. The sidesway displacement and base moment amplification determined by a single-element P-Δ analysis, which ignores the effect of P-δ on the response of the structure, is 2.55, resulting in an estimated M_u = 1,070 kip-in. (120×10^6 N-mm)—an error of 23.2% relative to the more accurate value of M_u—and a beam-column interaction value of 0.91.

P-δ effects can be captured in some (but not all) P-Δ-only analysis methods by subdividing the members into multiple elements. For this example, three equal-length P-Δ analysis elements are required to reduce the errors in the second-order base moment and sidesway displacement to less than 3% and 5%, respectively.

It should be noted that, in this case, the unconservative error that results from ignoring the effect of P-δ on the response of the structure is removed through the use of Equation A-8-8. For the loads shown in Figure C-C2.4, Equations A-8-6 and A-8-7 with $R_M = 0.85$ gives a B_2 amplifier of 3.52. This corresponds to $M_u = 1{,}480$ kip-in. (170×10^6 N-mm) in the preceding example; approximately 6% over that determined from a computational second-order analysis that includes both P-Δ and P-δ effects.

For sway columns with nominally simply supported base conditions, the errors in the second-order internal moment and in the second-order displacements from a P-Δ-only analysis are generally smaller than 3% and 5%, respectively, when $\alpha P_r / P_{eL} \leq 0.05$,

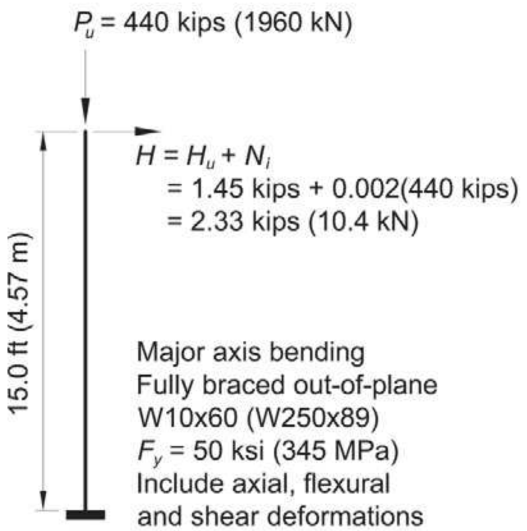
where

α = 1.0 (LRFD)
= 1.6 (ASD)

P_{eL} = $\pi^2 EI / L^2$ if the analysis uses nominal stiffness, kips (N)
= $0.8\tau_b \pi^2 EI / L^2$ if the analysis uses a flexural stiffness reduction of $0.8\tau_b$, kips (N)

P_r = required axial force, ASD or LRFD, kips (N)

For sway columns with rotational restraint at both ends of at least $1.5(EI/L)$ if the analysis uses nominal stiffness or $1.5(0.8\tau_b EI/L)$ if the analysis uses a flexural stiffness reduction of $0.8\tau_b$, the errors in the second-order internal moments and displacements from a P-Δ-only analysis are generally smaller than 3% and 5%, respectively, when $\alpha P_r/P_{eL} \leq 0.12$.



$P_u / P_y = 0.50,\ \tau = 1.0$

$E = 0.80\tau(29{,}000 \text{ ksi}) = 23{,}200$ ksi (160 GPa)

$G = E / 2(1+\nu) = 8{,}920$ ksi (61.5 GPa)

Computational P-Δ and P-δ analysis:

$\Delta_{2nd} = 2.22$ in. (56.5 mm)

$M_u = 1{,}394$ kip-in. (158 kN-m)

$P_u / \phi_c P_n + (8/9)(M_u / \phi_b M_n) = 1.00$

Single-element P-Δ analysis:

$\Delta_{1st} = 0.580$ in. (15 mm)

$M_{1st} = HL = 419$ kip-in. (48 kN-m)

$$\frac{1}{1-\left[P_u / (HL / \Delta_{1st})\right]} = 2.55$$

$M_u^{P\text{-}\Delta} = 2.55 M_{1st} = 1{,}070$ kip-in. (121 kN-m)

$P_u / \phi_c P_n + (8/9)\left(M_u^{P\text{-}\Delta} / \phi_b M_n\right) = 0.910$

Fig. C-C2.4. Illustration of potential errors associated with the use of a single-element-per-member P-Δ *analysis.*

For members subjected predominantly to nonsway end conditions, the errors in the second-order internal moments and displacements from a P-Δ-only analysis are generally smaller than 3% and 5%, respectively, when $\alpha P_r/P_{eL} \le 0.05$.

In meeting these limitations for use of a P-Δ-only analysis, it is important to note that in accordance with Section C2.1(b) the moments along the length of the member (i.e., the moments between the member-end nodal locations) should be amplified as necessary to include P-δ effects. One device for achieving this is the use of a B_1 factor.

Kaehler et al. (2010) provide further guidelines for the appropriate number of P-Δ analysis elements in cases where the P-Δ-only analysis limits are exceeded, as well as guidelines for calculating internal element second-order moments. They also provide relaxed guidelines for the number of elements required per member when using typical second-order analysis capabilities that include both P-Δ and P-δ effects.

As previously indicated, the engineer should verify the accuracy of second-order analysis software by comparisons to known solutions for a range of representative loadings. In addition to the examples presented in Chen and Lui (1987) and McGuire et al. (2000), Kaehler et al. (2010) provides five useful benchmark problems for testing second-order analysis of frames composed of prismatic members. In addition, they provide benchmarks for evaluation of second-order analysis capabilities for web-tapered members.

Analysis with Factored Loads. It is essential that the analysis of the system be made with loads factored to the strength limit state level because of the nonlinearity associated with second-order effects. For design by ASD, this load level is estimated as 1.6 times the ASD load combinations, and the analysis must be conducted at this elevated load to capture second-order effects at the strength level.

Because second-order effects are dependent on the ratios of applied loads and member forces to structural and member stiffnesses, equivalent results may be obtained by using 1.0 times ASD load combinations if all stiffnesses are reduced by a factor of 1.6—i.e., using $0.5E$ instead of $0.8E$ in the second-order analysis (note that the use of $0.5E$ is similar to the 12/23 factor used in the definition of F'_e in earlier ASD Specifications). With this approach, required member strengths are provided directly by the analysis and do not have to be divided by 1.6 when evaluating member capacities using ASD design. Notional loads, N_i, would also be defined using 1.0 times ASD load combinations, i.e., $\alpha = 1.0$. τ_b would be redefined as $\tau_b = 1.0$ when $P_r/P_{ns} \le 0.3$ and $\tau_b = 4(P_r/0.6P_{ns})(1 - P_r/0.6P_{ns})$ when $P_r/P_{ns} > 0.3$. The stiffness of components comprised of other materials should be evaluated at design loads and reduced by the same 1.6 factor, although this may be overly conservative if these stiffnesses already include ϕ factors. Serviceability criteria may be assessed using 50% of the deflections from this analysis, although this will overestimate second-order effects at service loads.

2. Consideration of Initial System Imperfections

Current stability design provisions are based on the premise that the member forces are calculated by second-order elastic analysis, where equilibrium is satisfied on the deformed geometry of the structure. Initial imperfections in the structure, such as out-of-plumbness and material and fabrication tolerances, create additional destabilizing effects.

In the development and calibration of the direct analysis method, initial geometric imperfections are conservatively assumed equal to the maximum material, fabrication and erection tolerances permitted in the AISC *Code of Standard Practice* (AISC, 2016a): a member out-of-straightness equal to L/1,000, where L is the member length between brace or framing points, and a frame out-of-plumbness equal to H/500, where H is the story height. The permitted out-of-plumbness may be smaller in some cases, as specified in the AISC *Code of Standard Practice*.

Initial imperfections may be accounted for in the direct analysis method through direct modeling (Section C2.2a) or the inclusion of notional loads (Section C2.2b). When second-order effects are such that the maximum sidesway amplification $\Delta_{2nd\text{-}order}/\Delta_{1st\text{-}order}$ or $B_2 \leq 1.7$ using the reduced elastic stiffness (or 1.5 using the unreduced elastic stiffness) for all lateral load combinations, it is permitted to apply notional loads only in gravity load-only combinations and not in combination with other lateral loads. At this low range of sidesway amplification or B_2, the errors in internal forces caused by not applying the notional loads in combination with other lateral loads are relatively small. When B_2 is above this threshold, notional loads must also be applied in combination with other lateral loads.

In the 2016 AISC *Specification*, Appendix 1, Section 1.2 includes an extension to the direct analysis method that permits direct modeling of initial imperfections along the lengths of members (member imperfections) as well as at member ends (system imperfections). This extension permits axially loaded members (columns and beam-columns according to Chapters E and H, respectively) to be designed by employing a nominal compressive strength that is taken as the cross-sectional strength; this is equivalent to the use of an effective member length, L_c = 0, when computing the nominal compressive strength, P_n, of compression members.

The Specification requirements for consideration of initial imperfections are intended to apply only to analyses for strength limit states. It is not necessary, in most cases, to consider initial imperfections in analyses for serviceability conditions such as drift, deflection and vibration.

3. Adjustments to Stiffness

Partial yielding accentuated by residual stresses in members can produce a general softening of the structure at the strength limit state that further creates additional destabilizing effects. The direct analysis method is also calibrated against inelastic distributed-plasticity analyses that account for the spread of plasticity through the member cross section and along the member length. In these calibration studies, residual stresses in wide-flange shapes were assumed to have a maximum value of $0.3F_y$ in compression at the flange tips, and a distribution matching the so-called Lehigh pattern—a linear variation across the flanges and uniform tension in the web (Ziemian, 2010).

Reduced stiffness ($EI^* = 0.8\tau_b EI$ and $EA^* = 0.8\tau_b EA$) is used in the direct analysis method for two reasons. First, for frames with slender members, where the limit state is governed by elastic stability, the 0.8 factor on stiffness results in a system available strength equal to 0.8 times the elastic stability limit. This is roughly equivalent to the margin of safety implied in the design provisions for slender columns by the effective length procedure where, from Equation E3-3, $\phi P_n = 0.90(0.877P_e) = 0.79P_e$. Second, for frames with intermediate or stocky columns, the $0.8\tau_b$ factor reduces the stiffness to account for inelastic softening prior to the members reaching their design strength. The τ_b factor is similar to the inelastic stiffness reduction factor implied in the column curve to account for loss of stiffness under high compression loads ($\alpha P_r > 0.5P_{ns}$), and the 0.8 factor accounts for additional softening under combined axial compression and bending. It is a fortuitous coincidence that the reduction coefficients for both slender and stocky columns are close enough, such that the single reduction factor of $0.8\tau_b$ works over the full range of slenderness. For the 2016 AISC *Specification*, the definition for τ_b has been modified to account for the effects of local buckling of slender elements in compression members.

The use of reduced stiffness only pertains to analyses for strength and stability limit states. It does not apply to analyses for other stiffness-based conditions and criteria, such as for drift, deflection, vibration and period determination.

For ease of application in design practice, where $\tau_b = 1$, the reduction on EI and EA can be applied by modifying E in the analysis. However, for computer programs that do semi-automated design, one should ensure that the reduced E is applied only for the second-order analysis. The elastic modulus should not be reduced in nominal strength equations that include E (for example, M_n for lateral-torsional buckling in an unbraced beam).

As shown in Figure C-C2.5, the net effect of modifying the analysis in the manner just described is to amplify the second-order forces such that they are closer to the actual internal forces in the structure. It is for this reason that the beam-column interaction for in-plane flexural buckling is checked using an axial strength, P_{nL}, calculated from the column curve using the actual unbraced member length, $L_c = L$, in other words, with $K = 1.0$.

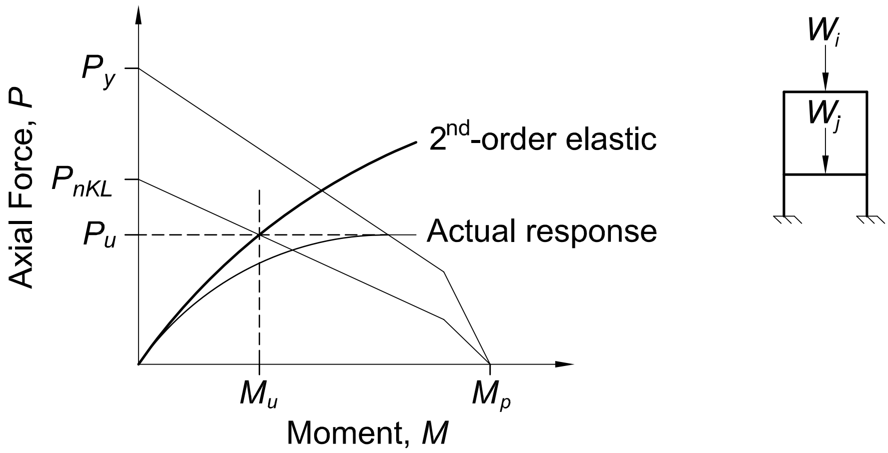
In cases where the flexibility of other structural components (connections, column base details, horizontal trusses acting as diaphragms) is modeled explicitly in the analysis, the stiffness of these components also should be reduced. The stiffness reduction may be taken conservatively as $EA^* = 0.8EA$ and/or $EI^* = 0.8EI$ for all cases. Surovek et al. (2005) discusses the appropriate reduction of connection stiffness in the analysis of partially restrained frames.

Where concrete or masonry shear walls or other nonsteel components contribute to the stability of the structure and the governing codes or standards for those elements specify a greater stiffness reduction, the greater reduction should be applied.

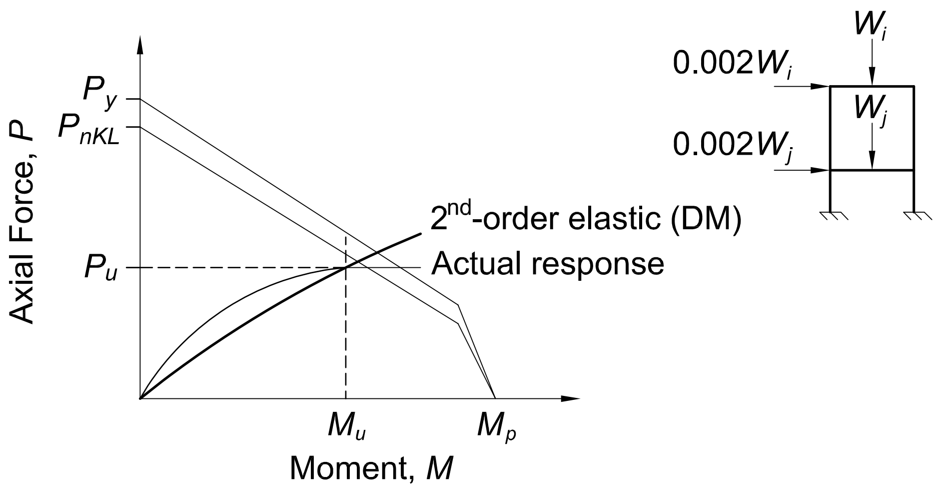
C3. CALCULATION OF AVAILABLE STRENGTHS

Section C3 provides that when the analysis meets the requirements in Section C2, the member provisions for available strength in Chapters D through I and connection provisions in Chapters J and K complete the process of design by the direct analysis method. The effective length for flexural buckling may be taken as the unbraced length for all members in the strength checks.

Where beams and columns rely upon braces that are not part of the lateral force-resisting system to define their unbraced length, the braces themselves must have sufficient strength and stiffness to control member movement at the brace points (see Appendix 6). Design requirements for braces that are part of the lateral force-resisting system (that is, braces that are included within the analysis of the structure) are included within Chapter C.



(a) Effective length method (P_{nKL} *is the nominal compressive strength used in the effective length method; see Appendix 7)*



(b) Direct analysis method (DM)

Fig. C-C2.5. Comparison of in-plane beam-column interaction checks for (a) the effective length method and (b) the direct analysis method (DM).

For beam-columns in single-axis flexure and compression, the analysis results from the direct analysis method may be used directly with the interaction equations in Section H1.3, which address in-plane flexural buckling and out-of-plane lateral-torsional instability separately. These separated interaction equations reduce the conservatism of the Section H1.1 provisions, which combine the two limit state checks into one equation that uses the most severe combination of in-plane and out-of-plane limits for P_r/P_c and M_r/M_c. A significant advantage of the direct analysis method is that the in-plane check with P_c in the interaction equation is determined using the unbraced length of the member as its effective length.

CHAPTER D

DESIGN OF MEMBERS FOR TENSION

The provisions of Chapter D do not account for eccentricities between the lines of action of connected assemblies.

D1. SLENDERNESS LIMITATIONS

The advisory upper limit on slenderness in the User Note is based on professional judgment and practical considerations of economics, ease of handling, and care required so as to minimize inadvertent damage during fabrication, transport and erection. This slenderness limit is not essential to the structural integrity of tension members; it merely assures a degree of stiffness such that undesirable lateral movement ("slapping" or vibration) will be unlikely. Out-of-straightness within reasonable tolerances does not affect the strength of tension members. Applied tension tends to reduce, whereas compression tends to amplify, out-of-straightness.

For single angles, the radius of gyration about the z-axis produces the maximum L/r and, except for very unusual support conditions, the maximum effective slenderness ratio.

D2. TENSILE STRENGTH

Because of strain hardening, a ductile steel bar loaded in axial tension can resist without rupture a force greater than the product of its gross area and its specified minimum yield stress. However, excessive elongation of a tension member due to uncontrolled yielding of its gross area not only marks the limit of its usefulness but can precipitate failure of the structural system of which it is a part. On the other hand, depending upon the reduction of area and other mechanical properties of the steel, the member can fail by rupture of the net area at a load smaller than required to yield the gross area. Hence, general yielding of the gross area and rupture of the net area both constitute limit states.

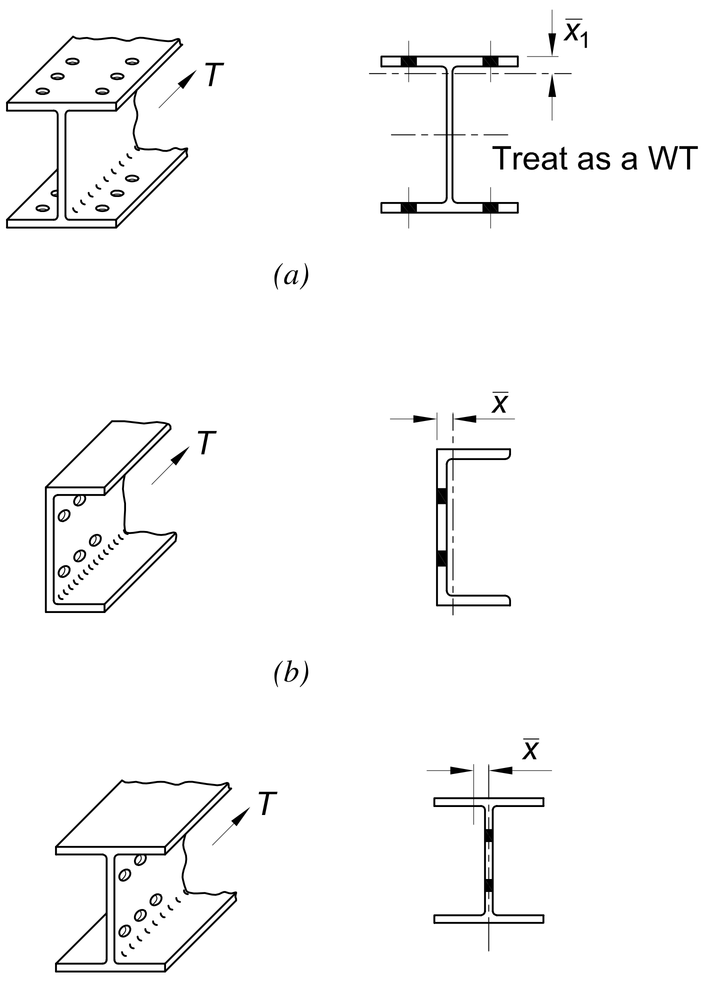
The length of the member in the net area is generally negligible relative to the total length of the member. Strain hardening is easily reached in the vicinity of holes and yielding of the net area at fastener holes does not constitute a limit state of practical significance.

Except for HSS that are subjected to cyclic load reversals, there is no information that the factors governing the strength of HSS in tension differ from those for other structural shapes, and the provisions in Section D2 apply.

D3. EFFECTIVE NET AREA

This section deals with the effect of shear lag, applicable to both welded and bolted tension members. Shear lag is a concept used to account for uneven stress distribution in connected members where some but not all of their elements (flange, web, leg, etc.) are connected. The reduction coefficient, U, is applied to the net area, A_n, of bolted members and to the gross area, A_g, of welded members. As the length of the connection, l, is increased, the shear lag effect diminishes. This concept is expressed empirically by the equation for U. Using this expression to compute the effective area, the estimated strength of some 1,000 bolted and riveted connection test specimens, with few exceptions, correlated with observed test results within a scatterband of ±10% (Munse and Chesson, 1963). Newer research provides further justification for the current provisions (Easterling and Gonzales, 1993).

For any given profile and configuration of connected elements, $\bar{x}$ is the perpendicular distance from the connection plane, or face of the member, to the centroid of the member section resisting the connection force, as shown in Figure C-D3.1. The length, l, is a function of the number of rows of fasteners or the length of weld. The length, l, is illustrated as the distance, parallel to the line of force, between the first and last row of fasteners in a line for bolted connections. The number of bolts in a line, for the purpose of the determination of l, is determined by the line with the maximum number of bolts in the connection. For staggered bolts, the out-to-out dimension is used for l, as shown in Figure C-D3.2.



(a)

(b)

(c)

Fig. C-D3.1. Determination of $\bar{x}$ *for* U.

For tension members with connections similar to that shown in Figure C-D3.1, the distance from the force in the member to the shear plane of the connection must be determined. For the I-shaped member with bolts in the flanges as shown in Figure C-D3.1(a), the member is treated as two WT-shapes. Because the section shown is symmetric about the horizontal axis and that axis is also the plastic neutral axis, the first moment of the area above the plastic neutral axis is $Z_x/2$, where Z_x is the plastic section modulus of the entire section, $Z = \sum|A_i\, d_i|$. The area above the plastic neutral axis is $A/2$; therefore, by definition $\bar{x}_1 = Z_x/A$. Thus, for use in calculating U, $\bar{x}_1 = d/2 - Z_x/A$. For the I-shaped member with bolts in the web as shown in Figure C-D3.1(c), the shape is treated as two channels and the shear plane is assumed to be at the web centerline. Using the definitions just discussed, but related now to the y-axis, yields $\bar{x} = Z_y/A$. Note that the plastic neutral axis must be an axis of symmetry for this relationship to apply. Thus, it cannot be used for the case shown in Figure C-D3.1(b) where $\bar{x}$ would simply be determined from the properties of a channel.

There is insufficient data for establishing a value of U if all lines have only one bolt, but it is probably conservative to use A_e equal to the net area of the connected element. The limit states of block shear (Section J4.3) and bearing and tearout (Section J3.10), which must be checked, will probably control the design.

The ratio of the area of the connected element to the gross area is a reasonable lower bound for U and allows for cases where the calculated U based on $(1-\bar{x}/l)$ is very small or nonexistent, such as when a single bolt per gage line is used and $l = 0$. This lower bound is similar to other design specifications; for example, the AASHTO *Standard Specifications for Highway Bridges* (AASHTO, 2002), which allow a U based on the area of the connected portion plus half the gross area of the unconnected portion.

The effect of connection eccentricity is a function of connection and member stiffness and may sometimes need to be considered in the design of the tension connection or member. Historically, engineers have neglected the effect of eccentricity in both the member and the connection when designing tension-only bracing. In Cases 1a and 1b shown in Figure C-D3.3, the length of the connection required to

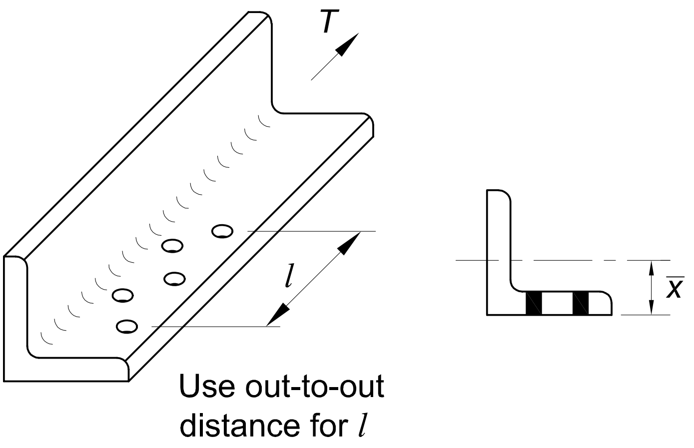
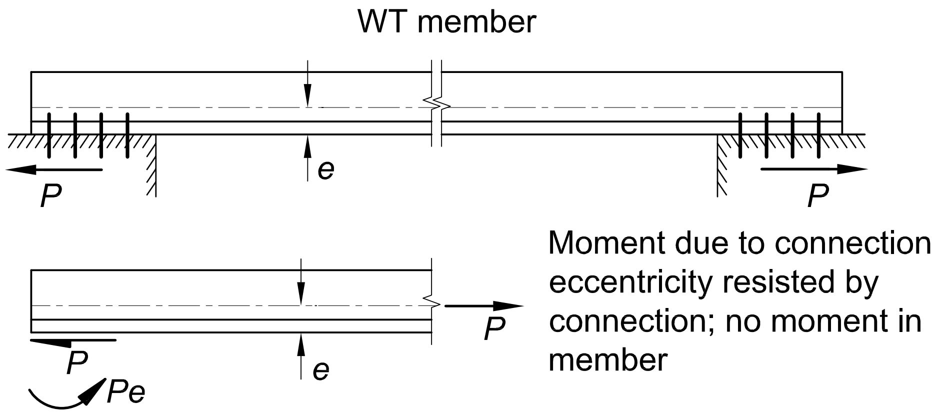
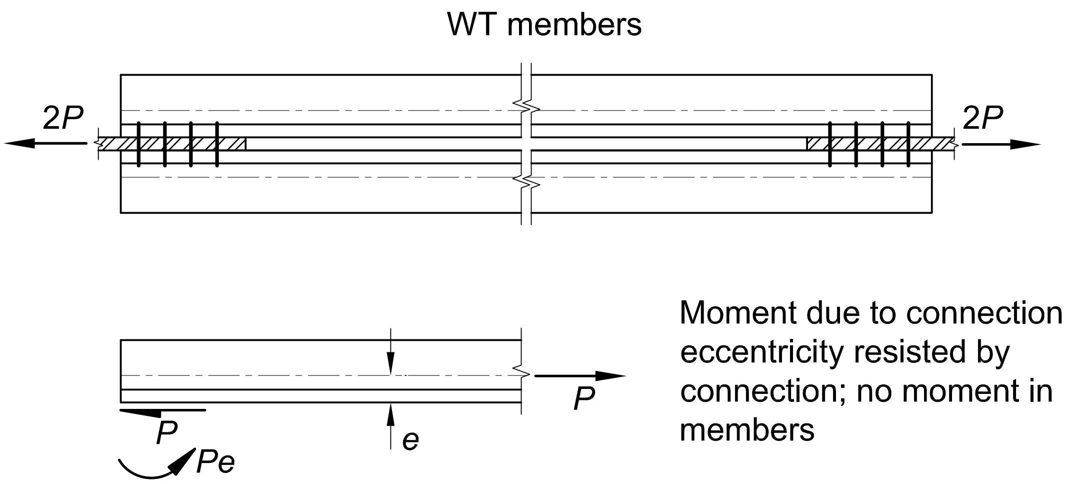


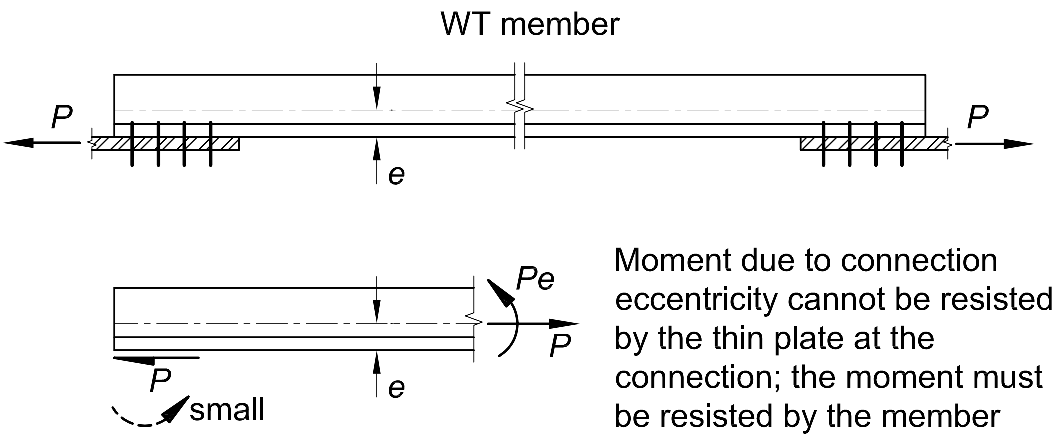
Fig. C-D3.2. Determination of l *for* U *of bolted connections with staggered holes.*



(a) Case 1a. End rotation restrained by connection to rigid abutments



(b) Case 1b. End rotation restrained by symmetry



(c) Case 2. End rotation not restrained—connection to thin plate

Fig. C-D3.3. The effect of connection restraint on eccentricity.

resist the axial loads will usually reduce the applied axial load on the bolts to a negligible value. For Case 2, the flexibility of the member and the connections will allow the member to deform such that the resulting eccentricity is relieved to a considerable extent.

For welded connections, l is the length of the weld parallel to the line of force as shown in Figure C-D3.4 for longitudinal and longitudinal plus transverse welds. For welds with unequal lengths, use the average length.

End connections for HSS in tension are commonly made by welding around the perimeter of the HSS; in this case, there is no shear lag or reduction in the gross area. Alternatively, an end connection with gusset plates can be used. Single gusset plates may be welded in longitudinal slots that are located at the centerline of the cross section. Welding around the end of the gusset plate may be omitted for statically loaded connections to prevent possible undercutting of the gusset and having to bridge the gap at the end of the slot. In such cases, the net area at the end of the slot is the critical area as illustrated in Figure C-D3.5. Alternatively, a pair of gusset plates can be welded to opposite sides of a rectangular HSS with flare bevel groove welds with no reduction in the gross area.

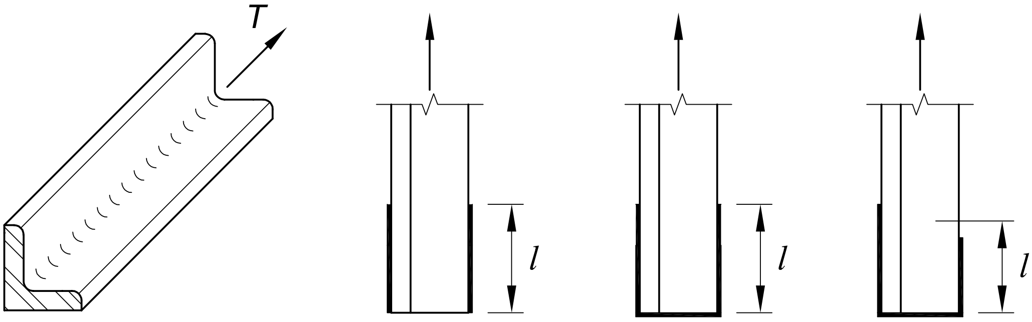


Fig. C-D3.4. Determination of l *for calculation of* U *for connections with longitudinal and transverse welds.*

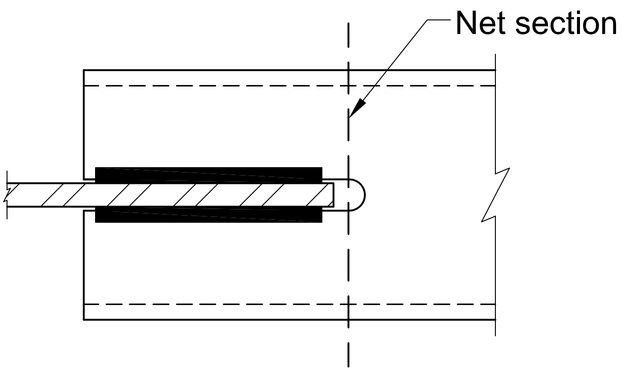


Fig. C-D3.5. Net area through slot for a single gusset plate.

For end connections with gusset plates, the general provisions for shear lag in Case 2 of Table D3.1 can be simplified and the connection eccentricity, $\bar{x}$, can be explicitly defined as in Cases 5 and 6. In Cases 5 and 6 it is implied that the weld length, l, should not be less than the depth of the HSS. In Case 5, the use of $U = 1$ when $l \geq 1.3D$ is based on research (Cheng and Kulak, 2000) that shows rupture occurs only in short connections and in long connections the round HSS tension member necks within its length and failure is by member yielding and eventual rupture. Case 6 of Table D3.1 can also be applied to box sections of uniform wall thickness. However, the welds joining the plates in the box section should be at least as large as the welds attaching the gusset plate to the box section wall for a length required to resist the force in the connected elements plus the length l.

Prior to 2016, two plates connected with welds shorter in length than the distance between the welds were not accommodated in Table D3.1. In light of the need for this condition, a shear lag factor was derived and is now shown in Case 4. The shear lag factor is based on a fixed-fixed beam model for the welded section of the connected part. The derivation of the factor is presented in Fortney and Thornton (2012).

The shear lag factors given in Cases 7 and 8 of Table D3.1 are given as alternate U values to the value determined from $1 - \bar{x}/l$ given for Case 2 in Table D3.1. It is permissible to use the larger of the two values.

D4. BUILT-UP MEMBERS

Although not commonly used, built-up member configurations using lacing, tie plates and perforated cover plates are permitted by this Specification. The length and thickness of tie plates are limited by the distance between the lines of fasteners, h, which may be either bolts or welds.

D5. PIN-CONNECTED MEMBERS

Pin-connected members are occasionally used as tension members with very large dead loads. Pin-connected members are not recommended when there is sufficient variation in live loading to cause wearing of the pins in the holes. The dimensional requirements presented in Section D5.2 must be met to provide for the proper functioning of the pin.

1. Tensile Strength

The tensile strength requirements for pin-connected members use the same ϕ and Ω values as elsewhere in this Specification for similar limit states. However, the definitions of effective net area for tension and shear are different.

2. Dimensional Requirements

Dimensional requirements for pin-connected members are illustrated in Figure C-D5.1.

D6. EYEBARS

Forged eyebars have generally been replaced by pin-connected plates or eyebars thermally cut from plates. Provisions for the proportioning of eyebars contained in this Specification are based upon standards evolved from long experience with forged eyebars. Through extensive destructive testing, eyebars have been found to provide balanced designs when they are thermally cut instead of forged. The more conservative rules for pin-connected members of nonuniform cross section and for members not having enlarged "circular" heads are likewise based on the results of experimental research (Johnston, 1939).

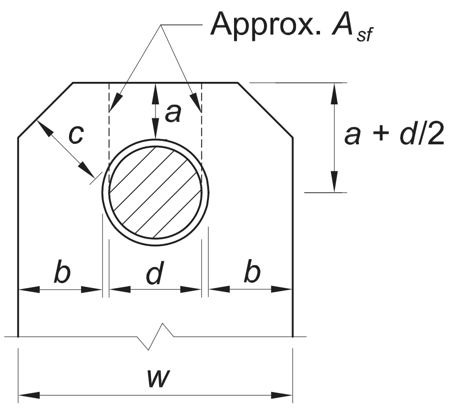
Stockier proportions are required for eyebars fabricated from steel having a yield stress greater than 70 ksi (485 MPa) to eliminate any possibility of their "dishing" under the higher design stress.

1. Tensile Strength

The tensile strength of eyebars is determined as for general tension members, except that, for calculation purposes, the width of the body of the eyebar is limited to eight times its thickness.

2. Dimensional Requirements

Dimensional limitations for eyebars are illustrated in Figure C-D6.1. Adherence to these limits assures that the controlling limit state will be tensile yielding of the body; thus, additional limit state checks are unnecessary.



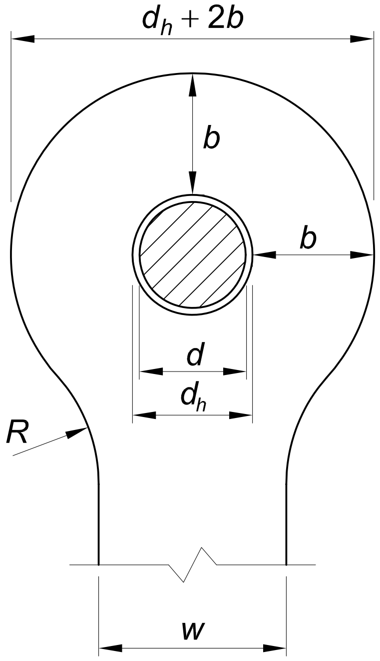
Dimensional Requirements

1. $a \ge 1.33b_e$
2. $w \ge 2b_e + d$
3. $c \ge a$

where

$b_e = 2t + 0.63$ in. $(2t + 16$ mm$) \le b$

Fig. C-D5.1. Dimensional requirements for pin-connected members.



Dimensional Requirements

1. $t \geq ½$ in. (13 mm) (Exception is provided in Section D6.2(e))
2. $w \leq 8t$ (For calculation purposes only)
3. $d \geq ⅞w$
4. $d_h \leq d + 1/32$ in. ($d + 1$ mm)
5. $R \geq d_h + 2b$
6. $⅔w \leq b \leq ¾w$ (Upper limit is for calculation purposes only)

Fig. C-D6.1. Dimensional limitations for eyebars.

CHAPTER E

DESIGN OF MEMBERS FOR COMPRESSION

E1. GENERAL PROVISIONS

The column equations in Section E3 are based on a conversion of research data into strength equations (Ziemian, 2010; Tide, 1985, 2001). These equations are the same as those that have been used since the 2005 AISC *Specification for Structural Steel Buildings* (AISC, 2005) and are essentially the same as those created for the initial LRFD *Specification* (AISC, 1986). The resistance factor, ϕ, was increased from 0.85 to 0.90 in the 2005 AISC *Specification*, recognizing substantial numbers of additional column strength analyses and test results, combined with the changes in industry practice that had taken place since the original calibrations were performed in the 1970s and 1980s.

In the original research on the probability-based strength of steel columns (Bjorhovde, 1972, 1978, 1988), three column curves were recommended. The three column curves were the approximate means of bands of strength curves for columns of similar manufacture, based on extensive analyses and confirmed by full-scale tests (Bjorhovde, 1972). For example, hot-formed and cold-formed heat treated HSS columns fell into the data band of highest strength [SSRC Column Category 1P (Bjorhovde, 1972, 1988; Bjorhovde and Birkemoe, 1979; Ziemian, 2010)], while welded built-up wide-flange columns made from universal mill plates were included in the data band of lowest strength (SSRC Column Category 3P). The largest group of data clustered around SSRC Column Category 2P. Had the original LRFD *Specification* opted for using all three column curves for the respective column categories, probabilistic analysis would have resulted in a resistance factor $\phi = 0.90$ or even slightly higher (Galambos, 1983; Bjorhovde, 1988; Ziemian, 2010). However, it was decided to use only one column curve, SSRC Column Category 2P, for all column types. This resulted in a larger data spread and thus a larger coefficient of variation, and so a resistance factor $\phi = 0.85$ was adopted for the column equations to achieve a level of reliability comparable to that of beams (AISC, 1986).

Since then, a number of changes in industry practice have taken place: (a) welded built-up shapes are no longer manufactured from universal mill plates; (b) the most commonly used structural steel is now ASTM A992/A992M, with a specified minimum yield stress of 50 ksi (345 MPa); and (c) changes in steelmaking practice have resulted in materials of higher quality and much better defined properties. The level and variability of the yield stress thus have led to a reduced coefficient of variation for the relevant material properties (Bartlett et al., 2003).

An examination of the SSRC Column Curve Selection Table (Bjorhovde, 1988; Ziemian, 2010) shows that the SSRC 3P Column Curve Category is no longer needed. It is now possible to use only the statistical data for SSRC Column Category

2P for the probabilistic determination of the reliability of columns. The curves in Figures C-E1.1 and C-E1.2 show the variation of the reliability index, β, with the live-to-dead load ratio, L/D, in the range of 1 to 5 for LRFD with $\phi = 0.90$ and ASD with $\Omega = 1.67$, respectively, for $F_y = 50$ ksi (345 MPa). The reliability index does not fall below $\beta = 2.6$. This is comparable to the reliability of beams.

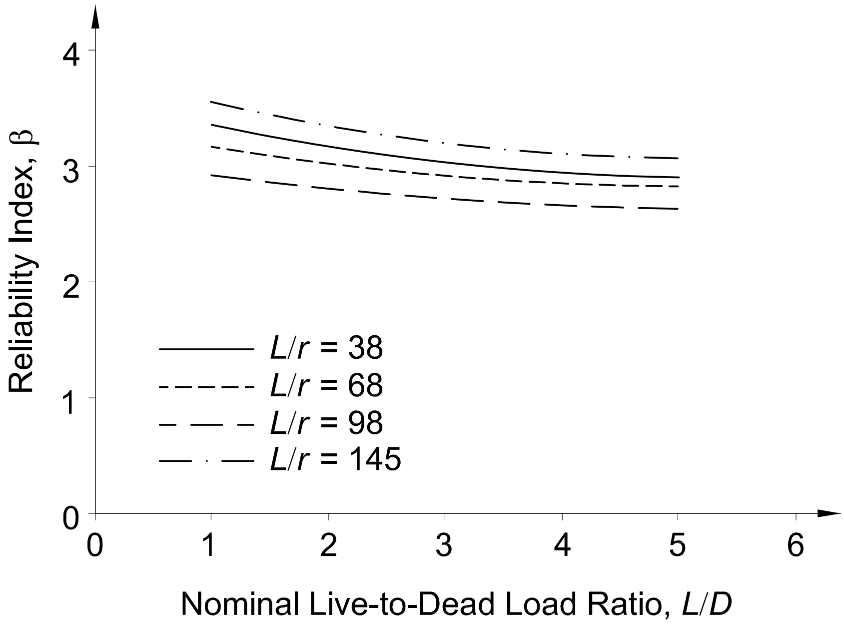


Fig. C-E1.1. Reliability of columns (LRFD).

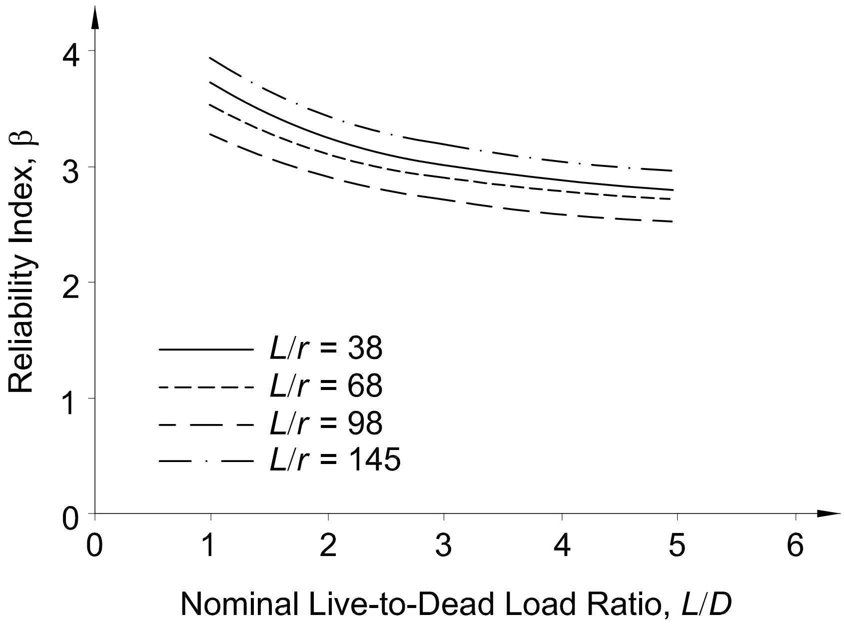


Fig. C-E1.2. Reliability of columns (ASD).

E2. EFFECTIVE LENGTH

In the 2016 AISC *Specification*, the effective length, which since the 1963 AISC *Specification* (AISC, 1963) had been given as KL, is changed to L_c. This was done to simplify the definition of effective length for the various modes of buckling without having to define a specific effective length factor, K. The effective length is then defined as KL in those situations where effective length factors, K, are appropriate. This change recognizes that there are several ways to determine the effective length that do not involve the direct determination of an effective length factor. It also recognizes that for some modes of buckling, such as torsional and flexural-torsional buckling, the traditional use of K is not the best approach. The direct use of effective length without the K-factor can be seen as a return to the approach used in the 1961 AISC *Specification* (AISC, 1961), when column strength equations based on effective length were first introduced by AISC.

The concept of a maximum limiting slenderness ratio has experienced an evolutionary change from a mandatory "…The slenderness ratio, KL/r, of compression members shall not exceed 200…" in the 1978 AISC *Specification* (AISC, 1978) to no restriction at all in the 2005 AISC *Specification* (AISC, 2005). The 1978 ASD and the 1999 LRFD *Specifications* (AISC, 2000b) provided a transition from the mandatory limit to a limit that was defined in the 2005 AISC *Specification* by a User Note, with the observation that "…the slenderness ratio, KL/r, preferably should not exceed 200…." However, the designer should keep in mind that columns with a slenderness ratio of more than 200 will have a critical stress (Equation E3-3) less than 6.3 ksi (43 MPa). The traditional upper limit of 200 was based on professional judgment and practical construction economics, ease of handling, and care required to minimize inadvertent damage during fabrication, transport and erection. These criteria are still valid and it is not recommended to exceed this limit for compression members except for cases where special care is exercised by the fabricator and erector.

E3. FLEXURAL BUCKLING OF MEMBERS WITHOUT SLENDER ELEMENTS

Section E3 applies to compression members with all nonslender elements, as defined in Section B4.

The column strength equations in Section E3 are the same as those in the previous editions of the LRFD Specification, with the exception of the cosmetic replacement of the slenderness term, $\lambda_c = \dfrac{KL}{\pi r}\sqrt{\dfrac{F_y}{E}}$, by the more familiar slenderness ratio, $\dfrac{KL}{r}$, for 2005 and 2010, and by the simpler form of the slenderness ratio, L_c/r, for 2016. For the convenience of those calculating the elastic buckling stress, F_e, directly, without first calculating an effective length, the limits on the use of Equations E3-2 and E3-3 are also provided in terms of the ratio F_y/F_e, as shown in the following discussion.

Comparisons between the previous column design curves and those introduced in the 2005 AISC *Specification* and continued in this Specification are shown in Figures C-E3.1 and C-E3.2 for the case of F_y = 50 ksi (345 MPa). The curves show the variation of the available column strength with the slenderness ratio for LRFD and ASD, respectively. The LRFD curves reflect the change of the resistance factor, ϕ, from 0.85 to 0.90, as was explained in Commentary Section E1. These column equations provide improved economy in comparison with the previous editions of the Specification.

The limit between elastic and inelastic buckling is defined to be $\dfrac{L_c}{r} = 4.71\sqrt{\dfrac{E}{F_y}}$ or $\dfrac{F_y}{F_e} = 2.25$. These are the same as $F_e = 0.44F_y$ that was used in the 2005 AISC *Specification*. For convenience, these limits are defined in Table C-E3.1 for the common values of F_y.

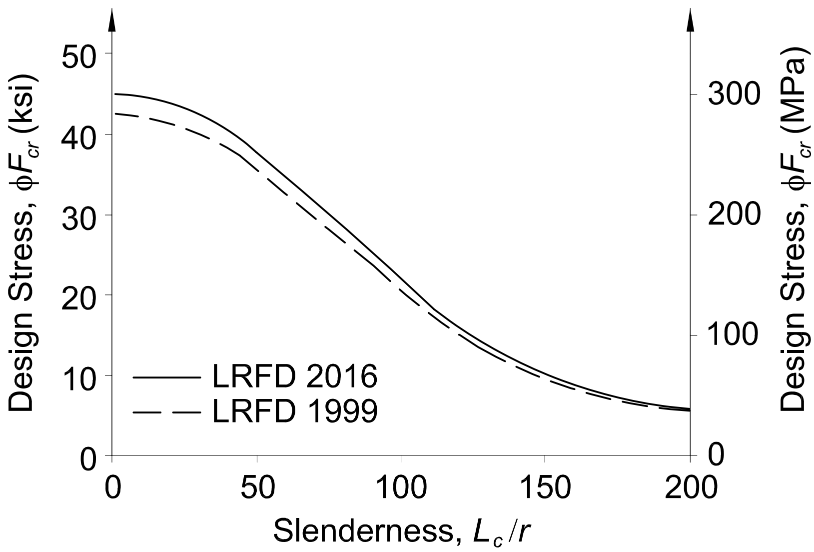


Fig. C-E3.1. LRFD column curves compared.

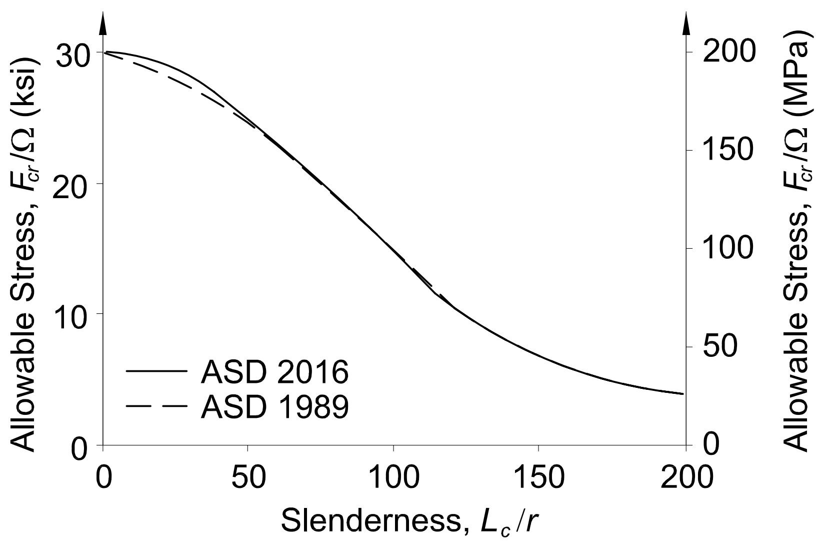


Fig. C-E3.2. ASD column curves compared.

TABLE C-E3.1
Limiting values of L_c/r and F_e

F_y, ksi (MPa)	Limiting $\frac{L_c}{r}$	F_e, ksi (MPa)
36 (250)	134	16.0 (110)
50 (345)	113	22.2 (150)
65 (450)	99.5	28.9 (200)
70 (485)	95.9	31.1 (210)

One of the key parameters in the column strength equations is the elastic critical stress, F_e. Equation E3-4 presents the familiar Euler form for F_e. However, F_e can also be determined by other means, including a direct frame buckling analysis or a torsional or flexural-torsional buckling analysis as addressed in Section E4.

The column strength equations of Section E3 can also be used for frame buckling and for torsional or flexural-torsional buckling (Section E4). They may also be entered with a modified slenderness ratio for single-angle members (Section E5).

E4. TORSIONAL AND FLEXURAL-TORSIONAL BUCKLING OF SINGLE ANGLES AND MEMBERS WITHOUT SLENDER ELEMENTS

Section E4 applies to singly symmetric and unsymmetric members and certain doubly symmetric members, such as cruciform or built-up columns with all nonslender elements, as defined in Section B4 for uniformly compressed elements. It also applies to doubly symmetric members when the torsional buckling length is greater than the flexural buckling length of the member. In addition, Section E4 applies to single angles with $b/t > 0.71\sqrt{E/F_y}$, although there are no ASTM A36/A36M hot-rolled angles for which this applies.

The equations in Section E4 for determining the torsional and flexural-torsional elastic buckling loads of columns are derived in textbooks and monographs on structural stability (Bleich, 1952; Timoshenko and Gere, 1961; Galambos, 1968a; Chen and Atsuta, 1977; Galambos and Surovek, 2008; and Ziemian, 2010). Since these equations apply only to elastic buckling, they must be modified for inelastic buckling by the appropriate equations of Section E3. Inelasticity has a more significant impact on warping torsion than St. Venant torsion. For consideration of inelastic effects, the full elastic torsional or flexural-torsional buckling stress is conservatively used to determine F_e for use in the column equations of Section E3.

Torsional buckling of symmetric shapes and flexural-torsional buckling of unsymmetrical shapes are failure modes usually not considered in the design of hot-rolled columns. They generally do not govern, or the critical load differs very little from the minor-axis flexural buckling load. Torsional and flexural-torsional buckling modes may, however, control the strength of symmetric columns manufactured from relatively thin plate elements and unsymmetric columns and symmetric columns having torsional unbraced lengths significantly larger than the minor-axis flexural unbraced lengths.

Equations for determining the elastic critical stress for columns are given in Section E4. Table C-E4.1 serves as a guide for selecting the appropriate equations. Equation E4-4 is the general buckling expression that is applicable to doubly symmetric, singly symmetric and unsymmetric shapes. Equation E4-3 was derived from Equation E4-4 for the specific case of a singly symmetric shape in which the y-axis is the axis of symmetry (such as in WT sections). For members, such as channels, in which the x-axis is the axis of symmetry, F_{ey} in Equation E4-3 should be replaced with F_{ex}.

For doubly symmetric shapes, the geometric centroid and shear center coincide resulting in $x_o = y_o = 0$. Therefore, for a doubly symmetric section, Equation E4-4 results in three roots: flexural buckling about the x-axis, flexural buckling about the y-axis, and torsional buckling about the shear center of the section, with the lowest root controlling the capacity of the cross section. Most designers are familiar with evaluating the strength of a wide-flange column by considering flexural buckling about the x-axis and y-axis; however, torsional buckling as given by Equation E4-2 is another potential buckling mode that should be considered and may control when the unbraced length for torsional buckling exceeds the unbraced length for minor-axis flexural buckling. Equation E4-2 is applicable for columns that twist about the shear center of the section, which will be the case when lateral bracing details like that shown in Figure C-E4.1 are used. The rod that is used for the brace in this case restrains the column from lateral movement about the minor axis, but does not generally prevent twist of the section and therefore the unbraced length for torsional buckling may be larger than for minor-axis flexure, which is a case where torsional

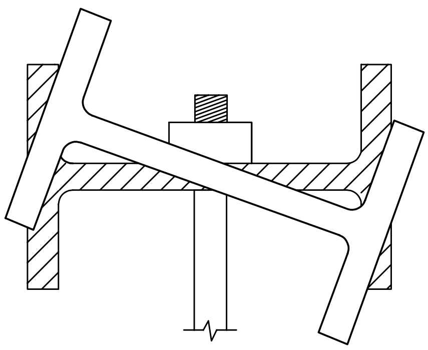
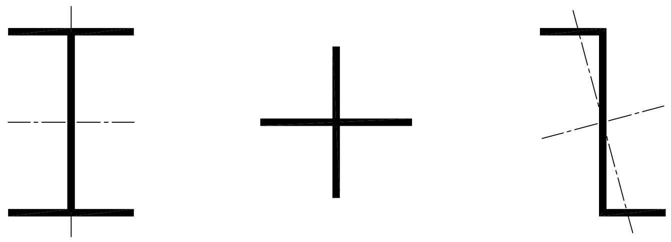
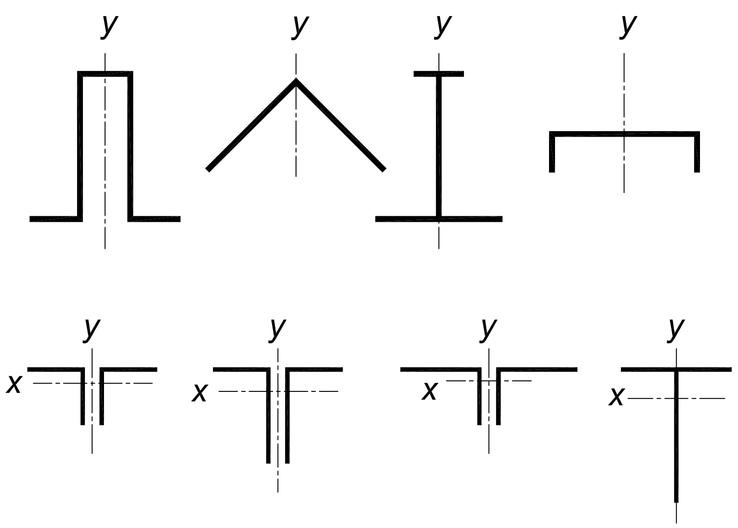
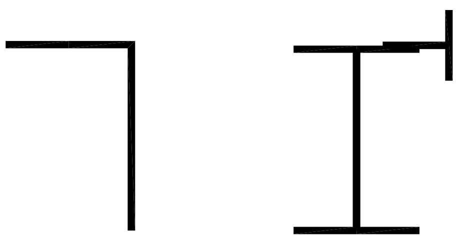


Fig. C-E4.1. Lateral bracing detail resulting in twist about the shear center.

TABLE C-E4.1
Selection of Equations for Torsional and Flexural-Torsional Buckling About the Shear Center

Type of Cross Section	**Applicable Equations in Section E4**
All doubly symmetric shapes and Z-shapes— Case (a) in Section E4 	E4-2
Singly symmetric members including double angles and tee-shaped members— Case (b) in Section E4 	E4-3
Unsymmetric shapes— Case (c) in Section E4 	E4-4

buckling may control. Most typical column base plate details will restrain twist at the base of the column. In addition, twist will often also be adequately restrained by relatively simple framing to beams. Many of the cases where inadequate torsional restraint is provided at a brace point will often occur at intermediate (between the ends of the column) brace locations.

Many common bracing details may result in situations where the lateral bracing is offset from the shear center of the section, such as columns or roof trusses restrained by a shear diaphragm that is connected to girts or purlins on the outside of the column or chord flange. Depending on the orientation of the primary member, the bracing may be offset along either the minor axis or the major axis as depicted in Figure C-E4.2. Since girts or purlins often have relatively simple connections that do not restrain twist, columns or truss chords can be susceptible to torsional buckling. However, in common cases due to the offset of the bracing relative to the shear center, the members are susceptible to constrained-axis torsional buckling.

Timoshenko and Gere (1961) developed the following expressions for constrained-axis torsional buckling:

Bracing offset along the minor axis by an amount "*a*" [see Figure C-E4.2(a)]:

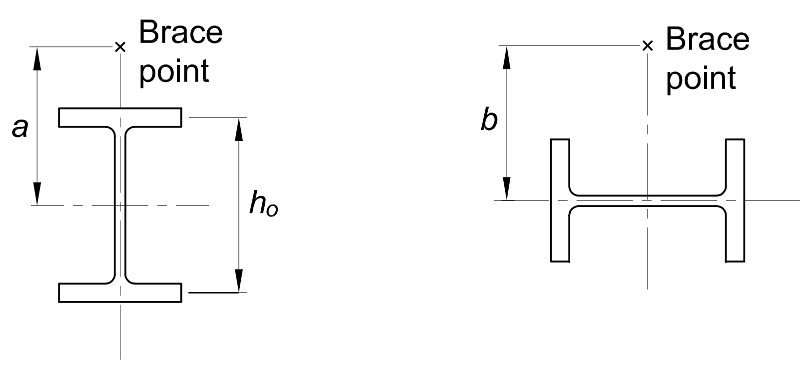
$$F_e = \omega \left[\frac{\pi^2 EI_y}{(L_{cz})^2} \left(\frac{h_o^2}{4} + a^2 \right) + GJ \right] \frac{1}{Ar_o^2} \tag{C-E4-1}$$

Bracing offset along the major axis by an amount "*b*" [(see Figure C-E4.2(b)]:

$$F_e = \omega \left[\frac{\pi^2 EI_y}{(L_{cz})^2} \left(\frac{h_o^2}{4} + \frac{I_x}{I_y} b^2 \right) + GJ \right] \frac{1}{Ar_o^2} \tag{C-E4-2}$$

where the polar radius of gyration is given by the expression:

$$r_o^2 = \left(r_x^2 + r_y^2 + a^2 + b^2 \right) \tag{C-E4-3}$$



(a) Bracing offset along minor axis *(b) Bracing offset along major axis*

Fig. C-E4.2. Bracing details resulting in an offset relative to the shear center.

The terms in these equations are as defined in Section E4 with the exception of *a*, *b* and ω. The bracing offsets, *a* and *b*, are measured relative to the shear center and h_o is the distance between flange centroids as indicated in Figure C-E4.2. The empirical factor ω was included to address some of the assumptions made in the original derivation. The expressions from Timoshenko and Gere (1961) were developed assuming that continuous lateral restraint was provided that is infinitely stiff. The impact of the continuous bracing assumption is not that significant since the column will generally be checked for buckling between discrete brace points. However, the assumption of the infinitely stiff lateral bracing will result in a reduction in the capacity for systems with finite brace stiffness. The ω-factor that is shown in Equations C-E4-1 and C-E4-2 is included to account for the reduction due to a finite brace stiffness. With a modest stiffness of the bracing (such as stiffness values recommended in the Appendix 6 lateral bracing provisions), the reduction is relatively small and a value of 0.9 is recommended based upon finite element studies (Errera, 1976; Helwig and Yura, 1999).

The specific method of calculating the buckling strength of double-angle and tee-shaped members that had been given in the 2010 AISC *Specification* (AISC, 2010) has been deleted in preference for the use of the general flexural-torsional buckling equations because the deleted equation was usually more conservative than necessary.

Equations E4-2 and E4-7 contain a torsional buckling effective length, L_{cz}. This effective length may be conservatively taken as the length of the column. For greater accuracy, if both ends of the column have a connection that restrains warping, say by boxing the end over a length at least equal to the depth of the member, the effective length may be taken as 0.5 times the column length. If one end of the member is restrained from warping and the other end is free to warp, then the effective length may be taken as 0.7 times the column length.

At points of bracing both lateral and/or torsional bracing shall be provided, as required in Appendix 6. AISC Design Guide 9, *Torsional Analysis of Structural Steel Members* (Seaburg and Carter, 1997), provides an overview of the fundamentals of torsional loading for structural steel members. Design examples are also included.

E5. SINGLE-ANGLE COMPRESSION MEMBERS

The compressive strength of single angles is to be determined in accordance with Sections E3 or E7 for the limit state of flexural buckling and Section E4 for the limit state of flexural-torsional buckling. However, single angles with $b/t \le 0.71\sqrt{E/F_y}$ do not require consideration of flexural-torsional buckling according to Section E4. This applies to all currently produced hot-rolled angles with $F_y = 36$ ksi. Use Section E4 to compute F_e for single angles only when $b/t > 0.71\sqrt{E/F_y}$.

Section E5 also provides a simplified procedure for the design of single angles subjected to an axial compressive load introduced through one connected leg. The angle is treated as an axially loaded member by adjusting the member slenderness. The attached leg is to be attached to a gusset plate or the projecting leg of another member by welding or by a bolted connection with at least two bolts. The equivalent slenderness expressions in this section presume significant restraint about the axis, which is perpendicular to the connected leg. This leads to the angle member tending to bend and buckle primarily about the axis parallel to the attached gusset. For this reason, L/r_a is the slenderness parameter used, where the subscript, *a*, represents the axis parallel to the attached leg. This may be the *x*- or *y*-axis of the angle, depending on which leg is the attached leg. The modified slenderness ratios indirectly account for bending in the angles due to the eccentricity of loading and for the effects of end restraint from the members to which they are attached.

The equivalent slenderness expressions also presume a degree of rotational restraint. Equations E5-3 and E5-4 [Section E5(b), referred to as case (b)] assume a higher degree of rotational restraint about the axis parallel to the attached leg than do Equations E5-1 and E5-2 [Section E5(a), referred to as case (a)]. Equations E5-3 and E5-4 are essentially equivalent to those employed for equal-leg angles as web members in latticed transmission towers in ASCE 10-97 (ASCE, 2000).

In space trusses, the web members framing in from one face typically restrain the twist of the chord at the panel points and thus provide significant restraint about the axis parallel to the attached leg for the angles under consideration. It is possible that the chords of a planar truss well restrained against twist justify use of case (b), in other words, Equations E5-3 and E5-4. Similarly, simple single-angle diagonal braces in braced frames could be considered to have enough end restraint such that case (a), in other words, Equations E5-1 and E5-2, could be employed for their design. This procedure, however, is not intended for the evaluation of the compressive strength of X-brace single angles.

The procedure in Section E5 permits use of unequal-leg angles attached by the smaller leg provided that the equivalent slenderness is increased by an amount that is a function of the ratio of the longer to the shorter leg lengths, and has an upper limit on L/r_z.

If the single-angle compression members cannot be evaluated using the procedures in this section, use the provisions of Section H2. In evaluating P_n, the effective length due to end restraint should be considered. With values of effective length about the geometric axes, one can use the procedure in Lutz (1992) to compute an effective radius of gyration for the column. To obtain results that are not too conservative, one must also consider that end restraint reduces the eccentricity of the axial load of single-angle struts and thus the value of f_{rbw} or f_{rbz} used in the flexural term(s) in Equation H2-1.

E6. BUILT-UP MEMBERS

Section E6 addresses the strength and dimensional requirements of built-up members composed of two or more shapes interconnected by stitch bolts or welds.

Two types of built-up members are commonly used for steel construction: closely spaced steel shapes interconnected at intervals using welds or fasteners, and laced or battened members with widely spaced flange components. The compressive strength of built-up members is affected by the interaction between the global buckling mode of the member and the localized component buckling mode between lacing points or intermediate connectors. Duan et al. (2002) refer to this type of buckling as compound buckling.

For both types of built-up members, limiting the slenderness ratio of each component shape between connection fasteners or welds, or between lacing points, as applicable, to 75% of the governing global slenderness ratio of the built-up member effectively mitigates the effect of compound buckling (Duan et al., 2002).

1. Compressive Strength

This section applies to built-up members such as double-angle or double-channel members with closely spaced individual components. The longitudinal spacing of connectors connecting components of built-up compression members must be such that the slenderness ratio, L_c / r, of individual shapes does not exceed three-fourths of the slenderness ratio of the entire member. However, this requirement does not necessarily ensure that the effective slenderness ratio of the built-up member is equal to that of a built-up member acting as a single unit.

For a built-up member to be effective as a structural member, the end connection must be welded or pretensioned bolted with Class A or B faying surfaces. Even so, the compressive strength will be affected by the shearing deformation of the intermediate connectors. This Specification uses the effective slenderness ratio to consider this effect. Based mainly on the test data of Zandonini (1985), Zahn and Haaijer (1987) developed an empirical formulation of the effective slenderness ratio for the 1986 LRFD *Specification* (AISC, 1986). When pretensioned bolted or welded intermediate connectors are used, Aslani and Goel (1991) developed a semi-analytical formula for use in the 1993, 1999 and 2005 AISC *Specifications* (AISC, 1993, 2000b, 2005). As more test data became available, a statistical evaluation (Sato and Uang, 2007) showed that the simplified expressions used in this Specification achieve the same level of accuracy.

Fastener spacing less than the maximum required for strength may be needed to ensure a close fit over the entire faying surface of components in continuous contact. Special requirements for weathering steel members exposed to atmospheric corrosion are given in Brockenbrough (1983).

2. Dimensional Requirements

This section provides additional requirements on connector spacing and end connection for built-up member design. Design requirements for laced built-up members where the individual components are widely spaced are also provided. Some dimensioning requirements are based upon judgment and experience. The provisions governing the proportioning of perforated cover plates are based upon extensive experimental research (Stang and Jaffe, 1948).

E7. MEMBERS WITH SLENDER ELEMENTS

The structural engineer designing with hot-rolled shapes will seldom find an occasion to turn to Section E7. Among rolled shapes, the most frequently encountered cases requiring the application of this section are beam shapes used as columns, columns containing angles with thin legs, and tee-shaped columns having slender stems. Special attention to the determination of effective area must be given when columns are made by welding or bolting thin plates together or ultra-high strength steels are employed.

The provisions of Section E7 address the modifications to be made when one or more plate elements in the column cross section are slender. A plate element is considered to be slender if its width-to-thickness ratio exceeds the limiting value, λ_r, defined in Table B4.1a. As long as the plate element is not slender, it can support the full yield stress without local buckling. When the cross section contains slender elements, the potential reduction in capacity due to local-global buckling interaction must be accounted for.

The Q-factor approach to dealing with columns with slender elements was adopted in the 1969 AISC *Specification* (AISC, 1969), emulating the 1969 AISI *Specification for the Design of Cold-Formed Steel Structural Members* (AISI, 1969). Prior to 1969, the AISC practice was to remove the width of the plate that exceeded the λ_r limit and check the remaining cross section for conformance with the allowable stress, which proved inefficient and uneconomical. Two separate philosophies were used: Unstiffened elements were considered to have attained their limit state when they reach the theoretical local buckling stress; stiffened elements, on the other hand, make use of the post-buckling strength inherent in a plate that is supported on both of its longitudinal edges, such as in HSS columns and webs of I-shaped columns. The effective width concept is used to obtain the added post-buckling strength. This dual philosophy reflects the 1969 practice in the design of cold-formed columns. Subsequent editions of the AISI Specifications, in particular, the *North American Specification for the Design of Cold-Formed Steel Structural Members* (AISI, 2001, 2007, 2012), hereafter referred to as the AISI *North American Specification,* adopted the effective width concept for both stiffened and unstiffened elements. This approach is adopted in this Specification.

1. Slender Element Members Excluding Round HSS

The effective width method is employed for determining the reduction in capacity due to local buckling. The effective width method was developed by von Kármán et al. (1932), empirically modified by Winter (1947), and generalized for local-global buckling interaction by Peköz (1987); see Ziemian (2010) for a complete summary. The point at which the slender element begins to influence column strength, $\lambda_r\sqrt{F_y/F_{cr}}$, is a function of element slenderness from Table B4.1a and column slenderness as reflected through F_{cr}. This reflects the unified effective width approach where the maximum stress in the effective width formulation is the column stress, F_{cr} (as opposed to F_y). This implies that columns designated as having slender elements by Table B4.1a may not necessarily see any reduction in strength due to local buckling, depending on the column stress, F_{cr}.

Prior to this Specification, the effective width, b_e, of a stiffened element was expressed as

$$b_e = 1.92t\sqrt{\frac{E}{f}}\left(1 - \frac{0.34}{(b/t)}\sqrt{\frac{E}{f}}\right) \leq b \tag{C-E7-1}$$

where

E = modulus of elasticity, ksi (MPa)
b = width of stiffened compression element, in. (mm)
f = critical stress when slender element is not considered, ksi (MPa)
t = thickness of element, in. (mm)

This may be compared with the new generalized effective width Equation E7-3:

$$b_e = b\left(1 - c_1\sqrt{\frac{F_{el}}{F_{cr}}}\right)\sqrt{\frac{F_{el}}{F_{cr}}} \tag{C-E7-2}$$

where F_{el} is the local elastic buckling stress, and c_1 is the empirical correction factor typically associated with imperfection sensitivity. The two expressions are essentially equivalent if one recognizes that

$$F_{el} = k\,\frac{\pi^2 E}{12(1 - v^2)}\left(\frac{t}{b}\right)^2 \tag{C-E7-3}$$

where

v = Poisson's ratio = 0.3

and utilizes $k = 4.0$ for the stiffened element, $c_1 = 0.18$ for the imperfection sensitivity factor, and sets $f = F_{cr}$.

Equation E7-3 provides an effective width expression applicable to both stiffened and unstiffened elements. Further, by making elastic local buckling explicit in the expression, the potential to use analysis to provide F_{el} is also allowed [see Seif and Schafer (2010)]. For ultra-high-strength steel sections or sections built-up from thin plates, this can be especially useful.

Equation E7-5 provides an explicit expression for elastic local buckling, F_{el}. This expression is based on the assumptions implicit in Table B4.1a and was determined as follows. At the limiting width-to-thickness ratio: $\lambda = \lambda_r$, $b = b_e$, $F_{el} = F_{el\text{-}r}$; therefore, at this limit, local elastic buckling implies:

$$F_{el\text{-}r} = k\,\frac{\pi^2 E}{12(1 - v^2)}\left(\frac{t}{b}\right)^2 = k\,\frac{\pi^2 E}{12(1 - v^2)}\left(\frac{t}{\lambda_r}\right)^2 \tag{C-E7-4}$$

and the effective width expression simplifies to:

$$1 = \left(1 - c_1\sqrt{\frac{F_{el\text{-}r}}{F_y}}\right)\sqrt{\frac{F_{el\text{-}r}}{F_y}} \tag{C-E7-5}$$

which may be used to back-calculate the plate buckling coefficient, k, assumed in Table B4.1a:

$$k = \left(\frac{1 - \sqrt{1 - 4c_1}}{2c_1}\right)^2 \frac{12(1 - v^2)}{\pi^2}\,\frac{F_y}{E}\left(\frac{1}{\lambda_r}\right)^2 \tag{C-E7-6}$$

This relationship provides a prediction of the elastic local buckling stress consistent with the k implicit in Table B4.1a, after substitution:

$$F_{el} = \left(\frac{1-\sqrt{1-4c_1}}{2c_1} \frac{\lambda_r}{\lambda} \right)^2 F_y = \left(c_2 \frac{\lambda_r}{\lambda} \right)^2 F_y \qquad \text{(C-E7-7)}$$

Thus, λ_r from Table B4.1a may be used to determine k, which may be used to determine the elastic local buckling stress. Further, c_2 is shown to be determined by c_1 alone, and is used only for convenience.

Equation E7-3 has long been used in the AISI *North American Specification* with $c_1 = 0.22$ for both stiffened and unstiffened elements. The same c_1 factor is adopted here for all elements, except those that prior to the 2016 AISC *Specification* had explicit (and calibrated) effective width expressions.

One disadvantage of Equation E7-3, and the explicit use of F_{el}, is the loss of convenience when working with a particular slender element. If Equation E7-5 is utilized directly, then Equation E7-3 may be simplified to

$$b_e = \left(1 - c_1 c_2 \frac{\lambda_r}{\lambda} \sqrt{\frac{F_y}{F_{cr}}} \right) c_2 \frac{\lambda_r}{\lambda} \sqrt{\frac{F_y}{F_{cr}}}\, b \qquad \text{(C-E7-8)}$$

or, more specifically, for case (a), stiffened elements, except walls of square and rectangular sections of uniform thickness:

$$b_e = \left(1 - 0.24 \frac{\lambda_r}{\lambda} \sqrt{\frac{F_y}{F_{cr}}} \right) 1.31 \frac{\lambda_r}{\lambda} \sqrt{\frac{F_y}{F_{cr}}}\, b \qquad \text{(C-E7-9)}$$

for case (b), walls of square and rectangular sections of uniform thickness:

$$b_e = \left(1 - 0.28 \frac{\lambda_r}{\lambda} \sqrt{\frac{F_y}{F_{cr}}} \right) 1.38 \frac{\lambda_r}{\lambda} \sqrt{\frac{F_y}{F_{cr}}}\, b \qquad \text{(C-E7-10)}$$

or, case (c), all other elements:

$$b_e = \left(1 - 0.33 \frac{\lambda_r}{\lambda} \sqrt{\frac{F_y}{F_{cr}}} \right) 1.49 \frac{\lambda_r}{\lambda} \sqrt{\frac{F_y}{F_{cr}}}\, b \qquad \text{(C-E7-11)}$$

These equations may be further simplified if the constants associated with the slenderness limit, λ_r, are combined with the constants in Table E7.1. This results in

$$b_e = c_2 c_3 t \sqrt{\frac{k_c E}{F_{cr}}} \left(1 - \frac{c_1 c_2 c_3}{(b/t)} \sqrt{\frac{k_c E}{F_{cr}}} \right) \qquad \text{(C-E7-12)}$$

TABLE C-E7.1
Constants for Use in Equations C-E7-12 and C-E7-13

Table B4.1a Case	Table E7.1 Case	k_c	c_1	c_2	c_3	c_4	c_5
1	(c)	1.0	0.22	1.49	0.56	0.834	0.184
2	(c)	k_c	0.22	1.49	0.64	0.954	0.210
3	(c)	1.0	0.22	1.49	0.45	0.671	0.148
4	(c)	1.0	0.22	1.49	0.75	1.12	0.246
5	(a)	1.0	0.18	1.31	1.49	1.95	0.351
6	(b)	1.0	0.20	1.38	1.40	1.93	0.386
7	(a)	1.0	0.18	1.31	1.40	1.83	0.330
8	(a)	1.0	0.18	1.31	1.49	1.95	0.351

where c_3 is the constant associated with slenderness limits given in Table B4.1a (Geschwindner and Troemner, 2016). Combining the constants in Equation C-E7-12 with $c_4 = c_2c_3$ and $c_5 = c_1c_2c_3$ yields

$$b_e = c_4 t\sqrt{\frac{k_cE}{F_{cr}}}\left(1 - \frac{c_5}{(b/t)}\sqrt{\frac{k_cE}{F_{cr}}}\right) \quad \text{(C-E7-13)}$$

The constants c_4 and c_5 are given in Table C-E7.1 for each of the cases in Table B4.1a.

The impact of the changes in this Specification for treatment of slender element compression members is greatest for unstiffened element compression members and may be negligible for stiffened element compression members as shown by Geschwindner and Troemner (2016).

2. Round HSS

The classical theory of longitudinally compressed cylinders overestimates the actual buckling strength, often by 200% or more. Inevitable imperfections of shape and the eccentricity of the load are responsible for the reduction in actual strength below the theoretical strength. The limits in this section are based upon test evidence (Sherman, 1976), rather than theoretical calculations, that local buckling will not occur if $\frac{D}{t} \le \frac{0.11E}{F_y}$. When D/t exceeds this value but is less than $\frac{0.45E}{F_y}$, Equation E7-7 provides a reduction in the local buckling effective area. This Specification does not recommend the use of round HSS or pipe columns with $\frac{D}{t} > \frac{0.45E}{F_y}$.

Following the SSRC recommendations (Ziemian, 2010) and the approach used for other shapes with slender compression elements, an effective area is used in Section E7 for round sections to account for interaction between local and column buckling. The effective area is determined based on the ratio between the local buckling stress and the yield stress. The local buckling stress for the round section is taken from AISI provisions based on inelastic action (Winter, 1970) and is based on tests conducted on fabricated and manufactured cylinders. Subsequent tests on fabricated cylinders (Ziemian, 2010) confirm that this equation is conservative.

CHAPTER F

DESIGN OF MEMBERS FOR FLEXURE

Chapter F applies to members subject to simple bending about one principal axis of the cross section. That is, the member is loaded in a plane parallel to a principal axis that passes through the shear center. Simple bending may also be attained if all load points and supports are restrained against twisting about the longitudinal axis. In all cases, the provisions of this chapter are based on the assumption that points of support for all members are restrained against rotation about their longitudinal axis.

Section F2 gives the provisions for the flexural strength of doubly symmetric compact I-shaped and channel members subject to bending about their major axis. For most designers, the provisions in this section will be sufficient to perform their everyday designs. The remaining sections of Chapter F address less frequently occurring cases encountered by structural engineers. Since there are many such cases, many equations and many pages in the Specification, the table in User Note F1.1 is provided as a map for navigating through the cases considered in Chapter F. The coverage of the chapter is extensive and there are many equations that appear formidable; however, it is stressed again that for most designs, the engineer need seldom go beyond Section F2. AISC Design Guide 25, *Frame Design Using Web-Tapered Members* (Kaehler et al., 2010), addresses flexural strength for web-tapered members.

For all sections covered in Chapter F, the highest possible nominal flexural strength is the plastic moment, $M_n = M_p$. Being able to use this value in design represents the optimum use of the steel. In order to attain M_p, the beam cross section must be compact and the member must have sufficient lateral bracing.

Compactness depends on the flange and web width-to-thickness ratios, as defined in Section B4. When these conditions are not met, the nominal flexural strength diminishes. All sections in Chapter F treat this reduction in the same way. For laterally braced beams, the plastic moment region extends over the range of width-to-thickness ratios, λ, terminating at λ_p. This is the compact condition. Beyond these limits, the nominal flexural strength reduces linearly until λ reaches λ_r. This is the range where the section is noncompact. Beyond λ_r the section is a slender-element section. These three ranges are illustrated in Figure C-F1.1 for the case of rolled wide-flange members for the limit state of flange local buckling. The curve in Figure C-F1.1 shows the relationship between the flange width-to-thickness ratio, $b_f/2t_f$, and the nominal flexural strength, M_n.

The basic relationship between the nominal flexural strength, M_n, and the unbraced length, L_b, for the limit state of lateral-torsional buckling is shown by the solid curve in Figure C-F1.2 for a compact section that is simply supported and subjected to uniform bending with $C_b = 1.0$.

There are four principal zones defined on the basic curve by the lengths L_m, L_p and L_r. Equation F2-5 defines the maximum unbraced length, L_p, to reach M_p with uniform moment. Elastic lateral-torsional buckling will occur when the unbraced length is greater than L_r, given by Equation F2-6. Equation F2-2 defines the range of inelastic lateral-torsional buckling as a straight line between the defined limits M_p at L_p and $0.7F_yS_x$ at L_r. Buckling strength in the elastic region is given by Equation F2-3 in combination with Equation F2-4.

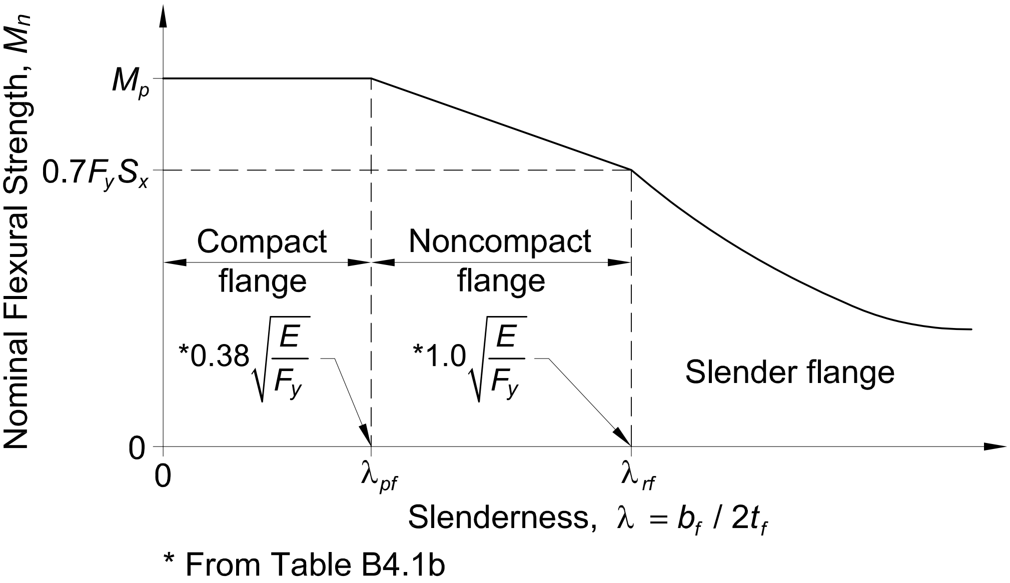


Fig. C-F1.1. Nominal flexural strength as a function of the flange width-to-thickness ratio of rolled I-shapes.

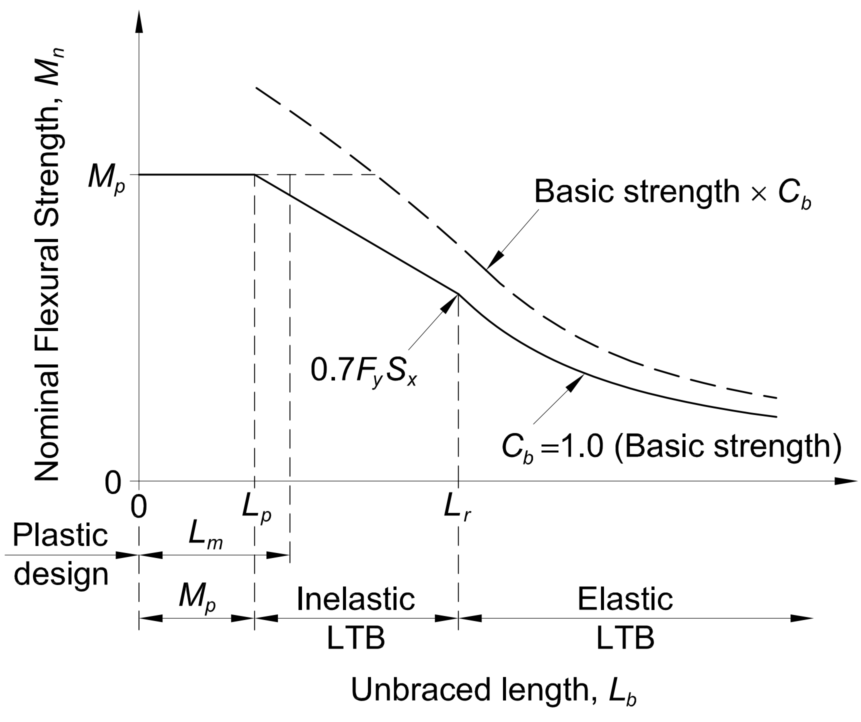


Fig. C-F1.2. Nominal flexural strength as a function of unbraced length and moment gradient.

The length L_m is defined in Section F13.5 as the limiting unbraced length needed for plastic design. Although plastic design methods generally require more stringent limits on the unbraced length compared to elastic design, the magnitude of L_m is often larger than L_p. The reason for this is because the L_m expression accounts for moment gradient directly, while designs based upon an elastic analysis rely on C_b factors to account for the benefits of moment gradient as outlined in the following.

For other than uniform moment along the member length, the lateral buckling strength is obtained by multiplying the basic strength in the elastic and inelastic region by C_b as shown in Figure C-F1.2. However, in no case can the maximum nominal flexural strength exceed the plastic moment, M_p. Note that L_p given by Equation F2-5 has physical meaning only for $C_b = 1.0$. For C_b greater than 1.0, members with larger unbraced lengths can reach M_p, as shown by the dashed curve for $C_b > 1.0$ in Figure C-F1.2. The largest length at which $M_n = M_p$ is calculated by setting Equation F2-2 equal to M_p and solving for L_b using the actual value of C_b.

F1. GENERAL PROVISIONS

Throughout Chapter F, the resistance factor and the safety factor remain unchanged, regardless of the controlling limit state. This includes the limit state defined in Section F13 for design of flexural members with holes in the tension flange where rupture is the controlling limit state (Geschwindner, 2010a).

In addition, the requirement that all supports for flexural members be restrained against rotation about the longitudinal axis is stipulated. Although there are provisions for members unbraced along their length, under no circumstances can the supports remain unrestrained torsionally.

Beginning with the 1961 AISC *Specification* (AISC, 1961) and continuing through the 1986 LRFD *Specification* (AISC, 1986), the following equation was used to adjust the lateral-torsional buckling equations for variations in the moment diagram within the unbraced length.

$$C_b = 1.75 + 1.05\left(\frac{M_1}{M_2}\right) + 0.3\left(\frac{M_1}{M_2}\right)^2 \leq 2.3 \qquad \text{(C-F1-1)}$$

where

M_1 = smaller moment at end of unbraced length, kip-in. (N-mm)

M_2 = larger moment at end of unbraced length, kip-in. (N-mm)

(M_1/M_2) is positive when moments cause reverse curvature and negative for single curvature

This equation is applicable strictly only to moment diagrams that consist of straight lines between braced points—a condition that is rare in beam design. The equation provides a lower bound to the solutions developed in Salvadori (1956). Equation C-F1-1 can be applied to nonlinear moment diagrams by using a straight line between M_2 and the moment at the middle of the unbraced length, and taking M_1 as the value on this straight line at the opposite end of the unbraced length (AASHTO, 2014). If the moment at the middle of the unbraced length is greater than M_2, C_b is conservatively taken equal to 1.0 when applying Equation C-F1.1 in this manner.

Kirby and Nethercot (1979) present an equation that is a direct fit to various nonlinear moment diagrams within the unbraced segment. Their original equation was slightly adjusted to give Equation C-F1-2a (Equation F1-1 in this Specification):

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} \tag{C-F1-2a}$$

This equation gives a more accurate solution for unbraced lengths in which the moment diagram deviates substantially from a straight line, such as the case of a fixed-end beam with no lateral bracing within the span, subjected to a uniformly distributed transverse load. It gives slightly conservative results compared to Equation C-F1-1, in most cases, for moment diagrams with straight lines between points of bracing. The absolute values of the three quarter-point moments and the maximum moment, regardless of its location, are used in Equation C-F1-2a. Wong and Driver (2010) review a number of approaches and recommend the following alternative quarter-point equation for use with doubly symmetric I-shaped members:

$$C_b = \frac{4\,M_{max}}{\sqrt{M_{max}^2 + 4M_A^2 + 7M_B^2 + 4M_C^2}} \tag{C-F1-2b}$$

This equation gives improved predictions for a number of important cases, including cases with moderately nonlinear moment diagrams. The maximum moment in the unbraced segment is used in all cases for comparison with the nominal moment, M_n. In addition, the length between braces, not the distance to inflection points, is used in all cases.

The lateral-torsional buckling modification factor given by Equation C-F1-2a is applicable for doubly symmetric sections and singly symmetric sections in single curvature. It should be modified for application with singly symmetric sections in reverse curvature. Previous work considered the behavior of singly symmetric I-shaped beams subjected to gravity loading (Helwig et al., 1997). The study resulted in the following expression:

$$C_b = \left(\frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C}\right)R_m \le 3.0 \tag{C-F1-3}$$

For single curvature bending

$$R_m = 1.0$$

For reverse curvature bending

$$R_m = 0.5 + 2\left(\frac{I_{y\,Top}}{I_y}\right)^2 \tag{C-F1-4}$$

where

$I_{y\,Top}$ = moment of inertia of the top flange about an axis in the plane of the web, in.^{4} (mm^4)

I_y = moment of inertia of the entire section about an axis in the plane of the web, in.^{4} (mm^4)

Equation C-F1-3 was developed for gravity loading on beams with a horizontal orientation of the longitudinal axis. For more general cases, the top flange is defined as the flange on the opposite side of the web mid-depth from the direction of the transverse loading. The term in parentheses in Equation C-F1-3 is identical to Equation C-F1-2a, while the factor R_m is a modifier for singly symmetric sections that is greater than unity when the top flange is the larger flange and less than unity when the top flange is the smaller flange. For singly symmetric sections subjected to reverse curvature bending, the lateral-torsional buckling strength should be evaluated by separately treating each flange as the compression flange and comparing the available flexural strength with the required moment that causes compression in the flange under consideration.

The C_b factors discussed in the foregoing are defined as a function of the spacing between braced points. However, many situations arise where a beam may be subjected to reverse curvature bending and have one of the flanges continuously braced laterally by closely spaced joists and/or light gauge decking normally used for roofing or flooring systems. Although the lateral bracing provides significant restraint to one of the flanges, the other flange can still buckle laterally due to the compression caused by the reverse curvature bending. A variety of C_b expressions have been developed that are a function of the type of loading, distribution of the moment, and the support conditions. For gravity loaded rolled I-section beams with the top flange laterally restrained, the following expression is applicable (Yura, 1995; Yura and Helwig, 2010):

$$C_b = 3.0 - \frac{2}{3}\left(\frac{M_1}{M_o}\right) - \frac{8}{3}\left[\frac{M_{CL}}{\left(M_o + M_1\right)^*}\right] \tag{C-F1-5}$$

where

M_o = moment at the end of the unbraced length that gives the largest compressive stress in the bottom flange, kip-in. (N-mm)

M_1 = moment at other end of the unbraced length, kip-in. (N-mm)

M_{CL} = moment at the middle of the unbraced length, kip-in. (N-mm)

$(M_o + M_1)^*$ = M_o, if M_1 is positive, causing tension on the bottom flange

The unbraced length is defined as the spacing between locations where twist is restrained. The sign convention for the moments is shown in Figure C-F1.3. M_o, M_1 and M_{CL} are all taken as positive when they cause compression on the top flange, and they are taken as negative when they cause compression on the bottom flange, as shown in the figure. The asterisk on the last term in Equation C-F1-5 indicates that M_1 is taken as zero in the last term if it is positive. For example, considering the distribution of moment shown in Figure C-F1.4, the C_b value would be:

$$C_b = 3.0 - \frac{2}{3}\left(\frac{+200}{-100}\right) - \frac{8}{3}\left(\frac{+50}{-100}\right) = 5.67$$

Note that $(M_o + M_1)^*$ is taken as M_o since M_1 is positive.

In this case, $C_b = 5.67$ would be used with the lateral-torsional buckling strength for the beam using an unbraced length of 20 ft, which is defined by the locations where twist or lateral movement of both flanges is restrained.

A similar buckling problem occurs with rolled I-shaped roofing beams subjected to uplift from wind loading. The light gauge metal decking that is used for the roofing system usually provides continuous restraint to the top flange of the beam; however, the uplift can be large enough to cause the bottom flange to be in compression. The sign convention for the moment is the same as indicated in Figure C-F1.3. The moment must cause compression in the bottom flange (M_{CL} negative) for the beam to buckle. Three different expressions are given in Figure C-F1.5 depending on whether the end moments are positive or negative (Yura and Helwig, 2010). As outlined in the foregoing, the unbraced length is defined as the spacing between points where both the top and bottom flange are restrained from lateral movement or between points restrained from twist.

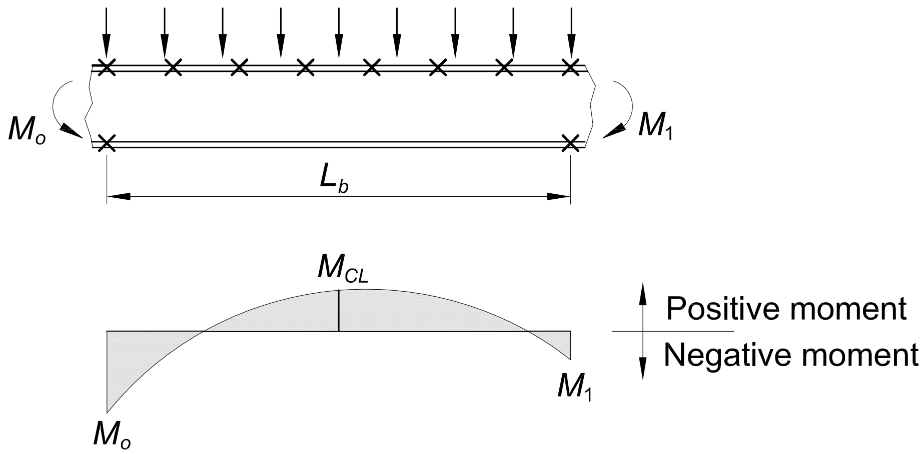


Fig. C-F1.3. Sign convention for moments in Equation C-F1-5.

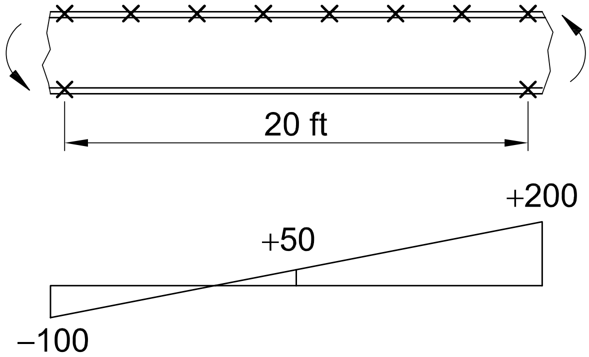


Fig. C-F1.4. Moment diagram for numerical example of application of Equation C-F1-5.

The equations for the limit state of lateral-torsional buckling in Chapter F assume that the loads are applied along the beam centroidal axis. C_b may be conservatively taken equal to 1.0, with the exception of some cases involving unbraced overhangs or members with no bracing within the span and with significant loading applied to the top flange. If the load is placed on the top flange and the flange is not braced, there is a tipping effect that reduces the critical moment; conversely, if the load is suspended from an unbraced bottom flange, there is a stabilizing effect that increases the critical moment (Ziemian, 2010). For unbraced top flange loading on compact I-shaped members, the reduced critical moment may be conservatively approximated by setting the square root expression in Equation F2-4 equal to unity.

An effective length factor of unity is implied in the critical moment equations to represent the worst-case simply supported unbraced segment. Consideration of any end restraint due to adjacent unbuckled segments on the critical segment can increase its strength. The effects of beam continuity on lateral-torsional buckling have been studied, and a simple conservative design method based on the analogy to end-restrained nonsway columns with an effective length less than unity is proposed in Ziemian (2010).

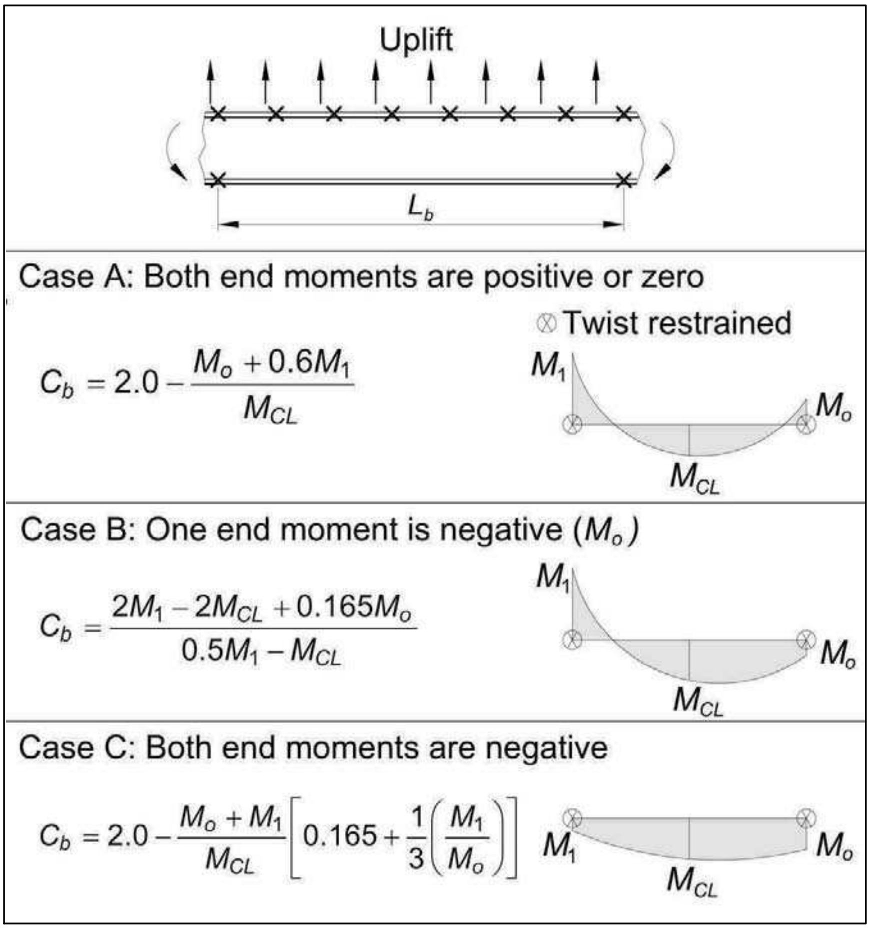


Fig. C-F1.5. C_b *factors for uplift loading on rolled I-shaped beams with the top flange continuously restrained laterally.*

TABLE C-F2.1
Comparison of Equations for Nominal Flexural Strength

1999 AISC LRFD *Specification* Equations	2005 and later *Specification* Equations
F1-1	F2-1
F1-2	F2-2
F1-13	F2-3

F2. DOUBLY SYMMETRIC COMPACT I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MAJOR AXIS

Section F2 applies to members with compact I-shaped or channel cross sections subject to bending about their major axis; hence, the only limit state to consider is lateral-torsional buckling. Almost all rolled wide-flange shapes listed in the AISC *Steel Construction Manual* (AISC, 2011) are eligible to be designed by the provisions of this section, as indicated in the User Note in this section.

The flexural strength equations in Section F2 are nearly identical to the corresponding equations in Section F1 of the 1999 LRFD *Specification* (AISC, 2000b), and are the same as those in the 2005 and 2010 *Specifications* (AISC, 2005, 2010). Table C-F2.1 gives the list of equivalent equations.

The only difference between the 1999 LRFD *Specification* (AISC, 2000b) and this Specification is that the stress at the interface between inelastic and elastic buckling has been changed from $F_y - F_r$ in the 1999 edition to $0.7F_y$.

In the Specifications prior to the 2005 AISC *Specification* the residual stress, F_r, for rolled and welded shapes was different, namely 10 ksi (69 MPa) and 16.5 ksi (110 MPa), respectively, while since the 2005 AISC *Specification* the residual stress has been taken as $0.3F_y$ so that the value of $F_y - F_r = 0.7F_y$ is adopted. This change was made in the interest of simplicity; in addition, this modification provides a slightly improved correlation with experimental data (White, 2008).

The elastic lateral-torsional buckling stress, F_{cr}, of Equation F2-4:

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\dfrac{L_b}{r_{ts}}\right)^2} \sqrt{1 + 0.078 \frac{Jc}{S_x h_o} \left(\frac{L_b}{r_{ts}}\right)^2} \tag{C-F2-1}$$

is identical to Equation F1-13 in the 1999 LRFD *Specification*:

$$F_{cr} = \frac{M_{cr}}{S_x} = \frac{C_b \pi}{L_b S_x} \sqrt{E I_y G J + \left(\frac{\pi E}{L_b}\right)^2 I_y C_w} \tag{C-F2-2}$$

This equation may be rearranged to the form:

$$F_{cr} = \frac{C_b \pi^2 E}{L_b^2} \frac{\sqrt{I_y C_w}}{S_x} \sqrt{1 + \frac{GJ}{EC_w}\left(\frac{L_b}{\pi}\right)^2} \tag{C-F2-3}$$

By using the definitions:

$$r_{ts}{}^2 = \frac{\sqrt{I_y C_w}}{S_x},\; C_w = \frac{I_y h_o^2}{4} \text{ and } c = 1$$

for doubly symmetric I-shaped members, Equation C-F2-1 is obtained after some algebraic arrangement. Section F2 provides an alternate definition for c, based on the expression for C_w of channels, which allows the use of Equation C-F2-1 for channel shapes.

Equation F2-5 is the same as F1-4 in the 1999 LRFD *Specification* and Equation F2-6 corresponds to F1-6. It is obtained by setting $F_{cr} = 0.7F_y$ in Equation F2-4 and solving for L_b. The format of Equation F2-6 was changed for the 2010 AISC *Specification* so that it is not undefined at the limit when $J = 0$; otherwise it gives identical results. The term r_{ts} can be approximated accurately as the radius of gyration of the compression flange plus one-sixth of the web.

These provisions are much simpler than the previous ASD provisions and are based on a more informed understanding of beam limit states behavior (White and Chang, 2007). The maximum allowable stress obtained in these provisions may be slightly higher than the previous limit of $0.66F_y$, because the true plastic strength of the member is reflected by use of the plastic section modulus in Equation F2-1. The Section F2 provisions for unbraced length are satisfied through the use of two equations: one for inelastic lateral-torsional buckling (Equation F2-2), and one for elastic lateral-torsional buckling (Equation F2-3). Previous ASD provisions placed an arbitrary stress limit of $0.6F_y$ when a beam was not fully braced and required that three equations be checked with the selection of the largest stress to determine the strength of a laterally unbraced beam. With the current provisions, once the unbraced length is determined, the member strength can be obtained directly from these equations.

F3. DOUBLY SYMMETRIC I-SHAPED MEMBERS WITH COMPACT WEBS AND NONCOMPACT OR SLENDER FLANGES BENT ABOUT THEIR MAJOR AXIS

Section F3 is a supplement to Section F2 for the case where the flange of the section is either noncompact or slender (see Figure C-F1.1 where the linear variation of M_n between λ_{pf} and λ_{rf} addresses the noncompact behavior and the curve beyond λ_{rf} addresses the slender behavior). As pointed out in the User Note of Section F2, very few rolled wide-flange shapes are subject to this criterion. However, any built-up doubly symmetric I-shaped member with a compact web and a noncompact or slender flange would require use of the provisions in this section.

F4. OTHER I-SHAPED MEMBERS WITH COMPACT OR NONCOMPACT WEBS BENT ABOUT THEIR MAJOR AXIS

The provisions of Section F4 are applicable to doubly symmetric I-shaped beams with noncompact webs and to singly symmetric I-shaped members with compact or noncompact webs (see the Table in User Note F1.1). This section addresses welded I-shaped beams where the webs are not slender. The flanges may be compact, noncompact or slender. The following section, F5, considers I-shapes with slender webs. The contents of Section F4 are based on White (2008).

Four limit states are considered in Section F4: (a) compression flange yielding; (b) lateral-torsional buckling; (c) compression flange local buckling; and (d) tension flange yielding . The effect of inelastic local buckling of the web is addressed indirectly by multiplying the moment causing yielding in the compression flange by a factor, R_{pc}, and the moment causing yielding in the tension flange by a factor, R_{pt}. These two factors can vary from unity to as high as M_p/M_{yc} and $M_p/M_{yt} \le 1.6$. The maximum limit of 1.6 is intended to ensure against substantial early yielding potentially leading to inelastic response under service conditions. They can be assumed to conservatively equal 1.0 although in many circumstances this will be much too conservative to be a reasonable assumption. The following steps are provided as a guide to the determination of R_{pc} and R_{pt}.

Step 1. Calculate h_p and h_c, as defined in Figure C-F4.1.

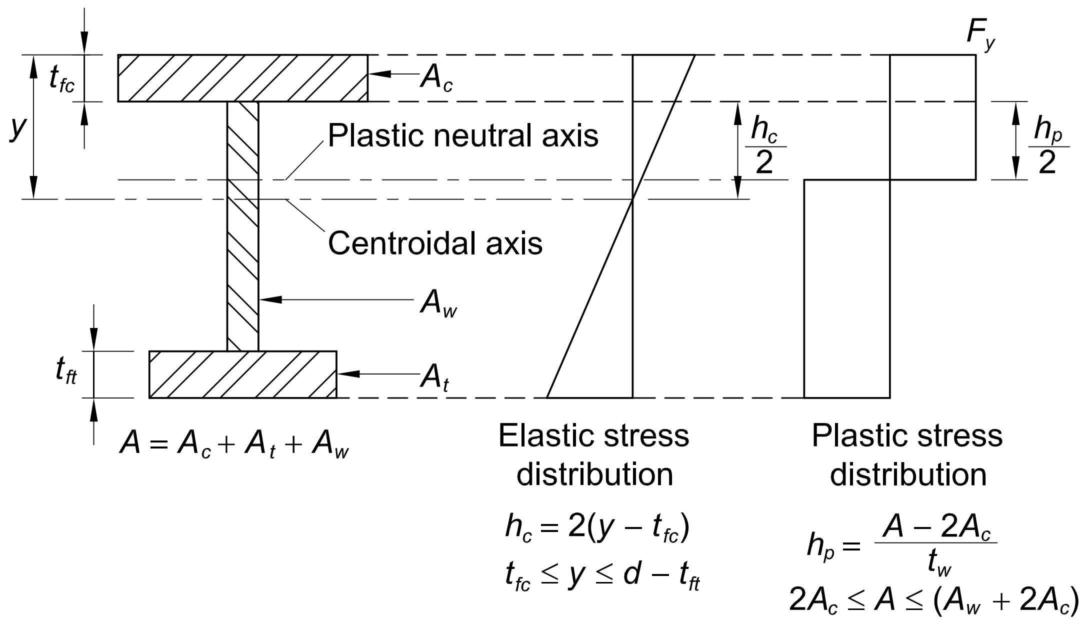


Fig. C-F4.1. Elastic and plastic stress distributions.

Step 2. Determine the web slenderness and the yield moments in compression and tension:

$$\left\{\begin{array}{l} \lambda = \dfrac{h_c}{t_w} \\ S_{xc} = \dfrac{I_x}{y}; \quad S_{xt} = \dfrac{I_x}{d-y} \\ M_{yc} = F_y S_{xc}; \quad M_{yt} = F_y S_{xt} \end{array}\right\} \tag{C-F4-1}$$

Step 3. Determine λ_{pw} and λ_{rw}:

$$\left\{\begin{array}{l} \lambda_{pw} = \dfrac{\dfrac{h_c}{h_p}\sqrt{\dfrac{E}{F_y}}}{\left(\dfrac{0.54M_p}{M_y} - 0.09\right)^2} \le 5.70\sqrt{\dfrac{E}{F_y}} \\ \lambda_{rw} = 5.70\sqrt{\dfrac{E}{F_y}} \end{array}\right\} \tag{C-F4-2}$$

If $\lambda > \lambda_{rw}$, then the web is slender and the design is governed by Section F5. Otherwise, in extreme cases where the plastic neutral axis is located in the compression flange, $h_p = 0$ and the web is considered to be compact.

Step 4. Calculate R_{pc} and R_{pt} using Section F4.

The basic maximum nominal moment is $R_{pc}M_{yc} = R_{pc}F_yS_{xc}$ corresponding to the compression flange, and $R_{pt}M_{yt} = R_{pt}F_yS_{xt}$ corresponding to tension flange yielding, which is applicable only when $M_{yt} < M_{yc}$, or $S_{xt} < S_{xc}$ (beams with the larger flange in compression). The Section F4 provisions parallel the rules for doubly symmetric members in Sections F2 and F3. Equations F2-4 and F2-6 are nearly the same as Equations F4-5 and F4-8, with the former using S_x and the latter using S_{xc}, both representing the elastic section modulus to the compression side. This is a simplification that tends to be somewhat conservative if the compression flange is smaller than the tension flange, and it is somewhat unconservative when the reverse is true (White and Jung, 2003). It is required to check for tension flange yielding if the tension flange is smaller than the compression flange (Section F4.4).

For a more accurate solution, especially when the loads are not applied at the centroid of the member, the designer is directed to Galambos (2001), White and Jung (2003), and Ziemian (2010). The following alternative equations in lieu of Equations F4-5 and F4-8 are provided by White and Jung:

$$M_n = C_b \frac{\pi^2 EI_y}{L_b^2}\left[\frac{\beta_x}{2} + \sqrt{\left(\frac{\beta_x}{2}\right)^2 + \frac{C_w}{I_y}\left(1 + 0.0390\frac{J}{C_w}L_b^2\right)}\right] \tag{C-F4-3}$$

$$L_r = \frac{1.38E\sqrt{I_y J}}{S_{xc}F_L}\sqrt{\frac{2.6\beta_x F_L S_{xc}}{EJ} + 1 + \sqrt{\left(\frac{2.6\beta_x F_L S_{xc}}{EJ} + 1\right)^2 + \frac{27.0C_w}{I_y}\left(\frac{F_L S_{xc}}{EJ}\right)^2}} \tag{C-F4-4}$$

where the coefficient of monosymmetry, $\beta_x = 0.9h\alpha\left(\frac{I_{yc}}{I_{yt}} - 1\right)$, the warping constant, $C_w = h^2 I_{yc}\alpha$, where $\alpha = \frac{1}{\frac{I_{yc}}{I_{yt}} + 1}$, and F_L is the magnitude of the flexural stress in compression at which the lateral-torsional buckling is influenced by yielding. In Equations F4-6a and F4-6b, this stress level is taken generally as the smaller of $0.7F_y$ in the compression flange, or the compression flange stress when the tension flange reaches the yield strength, but not less than $0.5F_y$.

F5. DOUBLY SYMMETRIC AND SINGLY SYMMETRIC I-SHAPED MEMBERS WITH SLENDER WEBS BENT ABOUT THEIR MAJOR AXIS

This section applies to doubly and singly symmetric I-shaped members with a slender web, that is, $\frac{h_c}{t_w} > \lambda_r = 5.70\sqrt{\frac{E}{F_y}}$. As is the case in Section F4, four limit states are considered: (a) compression flange yielding; (b) lateral-torsional buckling; (c) compression flange local buckling; and (d) tension flange yielding. The provisions in this section have changed little since 1963. The provisions are based on research reported in Basler and Thürlimann (1963).

There is no seamless transition between the equations in Section F4 and F5. The bending strength of a girder with F_y = 50 ksi (345 MPa) and a web slenderness, $h/t_w = 137$, is not close to that of a girder with $h/t_w = 138$. These two slenderness ratios are on either side of the limiting ratio. This gap is caused by the discontinuity between the lateral-torsional buckling resistances predicted by Section F4 and those predicted by Section F5 due to the implicit use of $J = 0$ in Section F5. However, for typical I-shaped members with webs close to the noncompact web limit, the influence of J on the lateral-torsional buckling resistance is relatively small (for example, the calculated L_r values including J versus using $J = 0$ typically differ by less than 10%). The implicit use of $J = 0$ in Section F5 is intended to account for the influence of web distortional flexibility on the lateral-torsional buckling resistance for slender-web I-section members.

F6. I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MINOR AXIS

I-shaped members and channels bent about their minor axis do not experience lateral-torsional buckling or web local buckling. The only limit states to consider are yielding and flange local buckling. The user note informs the designer of the few rolled shapes that need to be checked for flange local buckling. The limiting width-to-thickness ratios for rolled I-shaped members given in Table B4.1b are the same for major- and minor-axis bending. This is a conservative simplification. The limit of $1.6F_yS_y$ in Equation F6-1 is intended to ensure against substantial early yielding in channels subjected to minor-axis bending, potentially leading to inelastic response under service conditions. The minor-axis plastic moment capacity of I-sections rarely exceeds this limit.

F7. SQUARE AND RECTANGULAR HSS AND BOX SECTIONS

The provisions for the nominal flexural strength of HSS and box sections include the limit states of yielding, flange local buckling, web local buckling, and lateral-torsional buckling.

The provisions for local buckling of noncompact rectangular HSS are also the same as those in the previous sections of this chapter: $M_n = M_p$ for $b/t \le \lambda_p$, and a linear transition from M_p to F_yS_x when $\lambda_p < b/t \le \lambda_r$. The equation for the effective width of the compression flange when b/t exceeds λ_r is the same as that used for rectangular HSS in axial compression in the 2010 AISC *Specification*, except that the stress is taken as the yield stress. This implies that the stress in the corners of the compression flange is at yield when the ultimate post-buckling strength of the flange is reached. When using the effective width, the nominal flexural strength is determined from the effective section modulus referred to the compression flange using the distance from the shifted neutral axis. A slightly conservative estimate of the nominal flexural strength can be obtained by using the effective width for both the compression and tension flange, thereby maintaining the symmetry of the cross section and simplifying the calculations. For box sections, λ_r is the same as that used for uniformly compressed slender elements under compression in the 2010 AISC *Specification*.

Although there are no HSS with slender webs in flexural compression, Section F7.3(c) has been added to account for box sections which may have slender webs. The provisions of Section F5 for I-shaped members have been adopted with a doubling of a_w to account for two webs.

Because of the high torsional resistance of the closed cross section, the critical unbraced lengths, L_p and L_r, which correspond to the development of the plastic moment and the yield moment, respectively, are typically relatively large. For example, as shown in Figure C-F7.1, an HSS20×4×5/16 (HSS508×101.6×7.9), which has one of the largest depth-to-width ratios among standard HSS, has L_p of 6.7 ft (2.0 m) and L_r of 137 ft (42 m). An extreme deflection limit might correspond to a length-to-depth ratio of 24 or a length of 40 ft (12 m) for this member. Using the specified linear reduction between the plastic moment and the yield moment for lateral-torsional buckling, the plastic moment is reduced by only 7% for the 40 ft (12 m) length. In most practical designs with HSS where there is a moment gradient and the lateral-torsional buckling modification factor, C_b, is larger than unity, the reduction will be nonexistent or insignificant.

Section F7.4 has been added to account for the lateral-torsional buckling of very narrow box sections and box sections with plates thinner than HSS with the largest depth-to-width ratio. The provisions are those from the 1989 AISC *Specification* (AISC, 1989), which were removed in subsequent editions where only HSS were considered.

F8. ROUND HSS

Round HSS are not subject to lateral-torsional buckling. The failure modes and post-buckling behavior of round HSS can be grouped into three categories (Sherman, 1992; Ziemian, 2010):

(a) For low values of D/t, a long plastic plateau occurs in the moment-rotation curve. The cross section gradually ovalizes, local wave buckles eventually form, and the moment resistance subsequently decays slowly. Flexural strength may exceed the theoretical plastic moment due to strain hardening.

(b) For intermediate values of D/t, the plastic moment is nearly achieved but a single local buckle develops and the flexural strength decays slowly with little or no plastic plateau region.

(c) For high values of D/t, multiple buckles form suddenly with very little ovalization and the flexural strength drops quickly.

The flexural strength provisions for round HSS reflect these three regions of behavior and are based upon five experimental programs involving hot-formed seamless pipe, electric-resistance-welded pipe, and fabricated tubing (Ziemian, 2010).

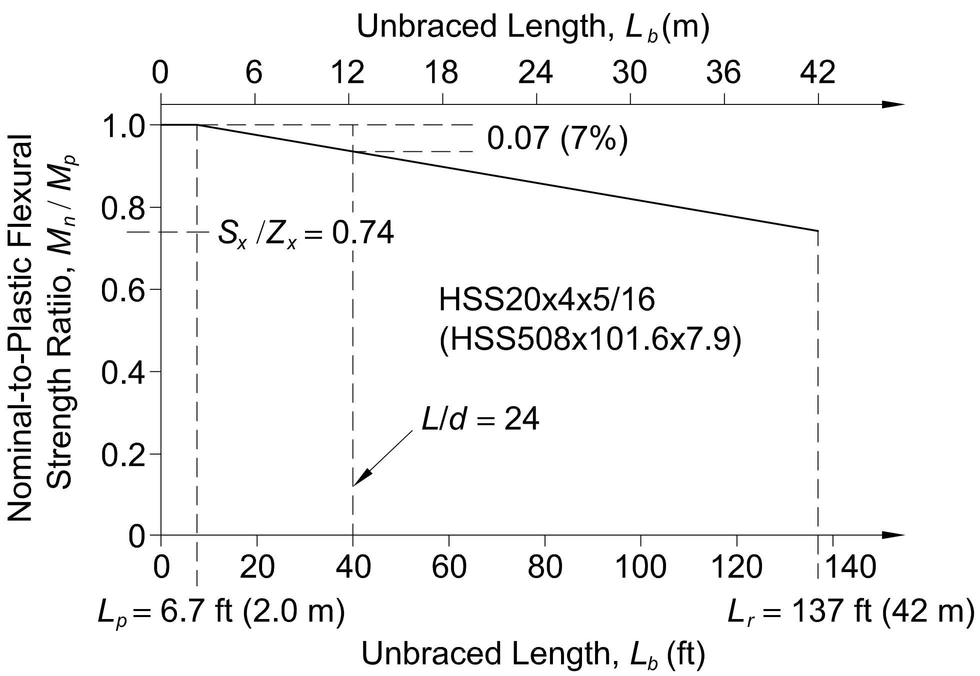


Fig. C-F7.1. Lateral-torsional buckling of rectangular HSS [F_y *= 46 ksi (310 MPa)].*

F9. TEES AND DOUBLE ANGLES LOADED IN THE PLANE OF SYMMETRY

This section addresses both tees and double angles loaded in the plane of symmetry. Prior editions of the Specification did not distinguish between tees and double angles and as a result, there were instances when double angles would appear to have less strength than two single angles. This Specification has addressed this concern by providing separate provisions for tees and double angles. In those cases where double angles should have the same strength as two single angles, the provisions reference Section F10.

The lateral-torsional buckling strength of singly symmetric tee beams is given by a fairly complex formula (Ziemian, 2010). Equation F9-4 in the 2010 AISC *Specification* (AISC, 2010) is a simplified formulation based on Kitipornchai and Trahair (1980). See also Ellifritt et al. (1992).

This Specification has introduced a substantial change in Section F9.2 for the limit state of lateral-torsional buckling when the stem of the member is in tension; that is, when the flange is in compression. The 2010 AISC *Specification* transitioned abruptly from the full plastic moment to the elastic buckling range. The plastic range then often extended for a considerable length of the beam. A new linear transition from full plastic moment, M_p, to the yield moment, M_y, as shown by the dashed line in Figure C-F9.1, has been introduced to bring the members into conformance with the lateral-torsional buckling rules for I-shaped beams. It should be noted that the ratio of the plastic moment to the yield moment, M_p/M_y, is in excess of 1.6, and is

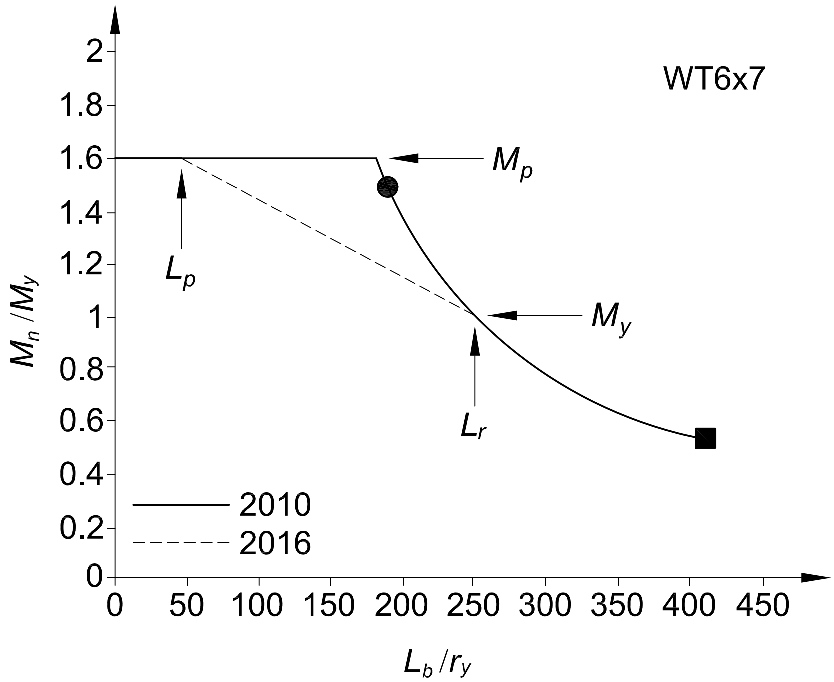


Fig.C-F9.1 Comparison of the 2016 and 2010 Specification lateral-torsional buckling formulas when the stem is in tension.

usually around 1.8 for tee and double-angle beams in flexure. The plastic moment value is limited to $1.6M_y$ to preclude potential early yielding under service loading conditions. For double-angle legs in compression, the plastic moment is limited to $1.5M_y$, while for tee stems in compression the plastic moment value is limited to M_y. The committee is unaware of any studies that show what strength tee stems in compression can achieve. Thus, this conservative limit from previous editions of this Specification has been continued.

The solid curve in Figure C-F9.1 defines the nominal moment criteria in the 2010 AISC *Specification* and the dashed line shows the modified form defined in the 2016 edition. The WT6×7 illustrated is an extreme case. For most shapes, the length, L_r, is impractically long. Also shown in Figure C-F9.1 are two additional points: the square symbol is the length when the center deflection of the member equals $L_b/1000$ under its self-weight. The round symbol defines the length when the length-to-depth ratio equals 24.

The C_b factor used for I-shaped beams is unconservative for tee beams with the stem in compression. For such cases, $C_b = 1.0$ is appropriate. When beams are bent in reverse curvature, the portion with the stem in compression may control the lateral-torsional buckling resistance even though the moments may be small relative to other portions of the unbraced length with $C_b \approx 1.0$. This is because the lateral-torsional buckling strength of a tee with the stem in compression may be only about one-fourth of the strength for the stem in tension. Since the buckling strength is sensitive to the moment diagram, C_b has been conservatively taken as 1.0 in Section F9.2. In cases where the stem is in tension, connection details should be designed to minimize any end restraining moments that might cause the stem to be in compression.

The 2005 AISC *Specification* did not have provisions for the local buckling strength of the stems of tee sections and the legs of double-angle sections under a flexural compressive stress gradient. The Commentary to this Section in the 2005 AISC *Specification* explained that the local buckling strength was accounted for in the equation for the lateral-torsional buckling limit state, Equation F9-4, when the unbraced length, L_b, approached zero. While this was thought to be an acceptable approximation at the time, it led to confusion and to many questions by users of the Specification. For this reason, Section F9.4, "Local Buckling of Tee Stems in Flexural Compression," was added to provide an explicit set of formulas for the 2010 AISC *Specification*.

The derivation of these formulas is provided here to explain the changes. The classical formula for the elastic buckling of a rectangular plate is (Ziemian, 2010):

$$F_{cr} = \frac{\pi^2 Ek}{12\left(1-\nu^2\right)\left(\frac{b}{t}\right)^2} \tag{C-F9-1}$$

where

ν = 0.3 (Poisson's ratio)
b/t = plate width-to-thickness ratio
k = plate buckling coefficient

For the stem of tee sections, the width-to-thickness ratio is equal to d/t_w. The two rectangular plates in Figure C-F9.2 are fixed at the top, free at the bottom, and loaded, respectively, with a uniform and a linearly varying compressive stress. The corresponding plate buckling coefficients, k, are 1.33 and 1.61 (Figure 4.4, Ziemian, 2010). The graph in Figure C-F9.3 shows the general scheme used historically in developing the local buckling criteria in AISC Specifications. The ordinate is the critical stress divided by the yield stress, and the abscissa is a nondimensional width-to-thickness ratio,

$$\bar{\lambda} = \frac{b}{t}\sqrt{\frac{F_y}{E}}\sqrt{\frac{12\left(1-\nu^2\right)}{\pi^2 k}} \tag{C-F9-2}$$

In the traditional scheme, it is assumed the critical stress is the yield stress, F_y, as long as $\bar{\lambda} \le 0.7$. Elastic buckling, governed by Equation C-F9-1, commences when $\bar{\lambda} = 1.24$ and $F_{cr} = 0.65F_y$. Between these two points, the transition is assumed linear to account for initial deflections and residual stresses. While these assumptions are arbitrary empirical values, they have proven satisfactory. The curve in Figure C-F9.3 shows the graph of the formulas adopted for the stem of tee sections when these elements are subject to flexural compression. The limiting width-to-thickness ratio up to which $F_{cr} = F_y$ is (using $\nu = 0.3$ and $k = 1.61$):

$$\bar{\lambda} = 0.7 = \frac{b}{t}\sqrt{\frac{F_y}{E}}\sqrt{\frac{12\left(1-\nu^2\right)}{\pi^2 k}} \rightarrow \frac{b}{t} = \frac{d}{t_w} = 0.84\sqrt{\frac{E}{F_y}}$$

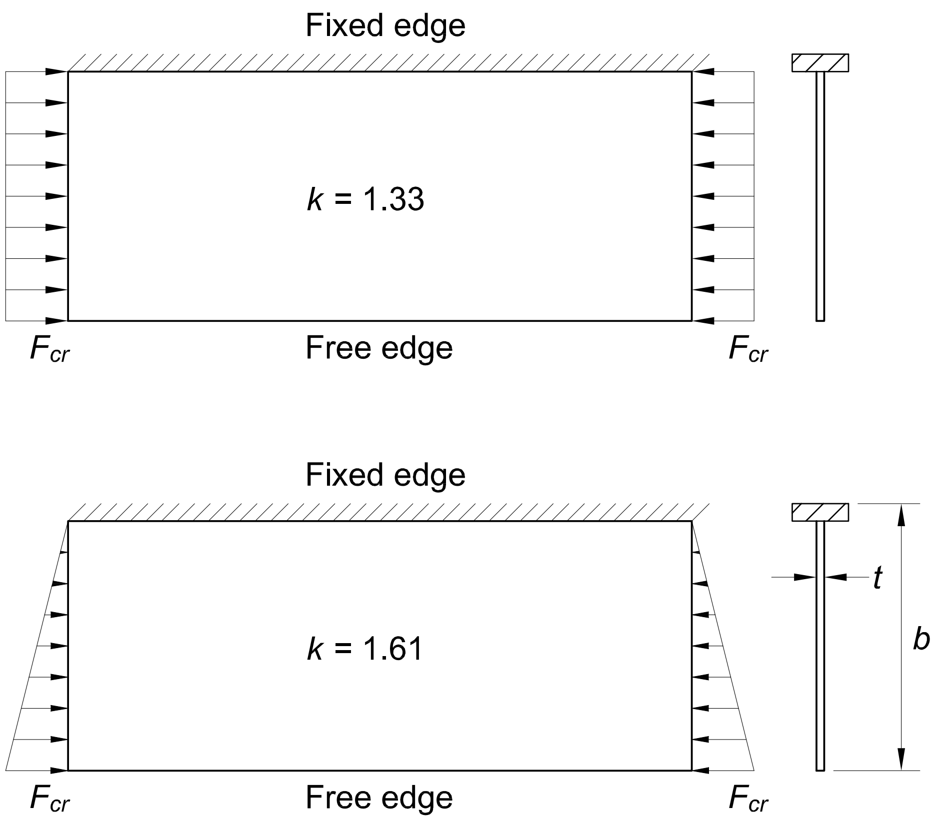


Fig. C-F9.2. Plate buckling coefficients for uniform compression and for linearly varying compressive stresses

The elastic buckling range was assumed to be governed by the same equation as the local buckling of the flanges of a wide-flange beam bent about its minor axis (Equation F6-4):

$$F_{cr} = \frac{0.69E}{\left(\dfrac{d}{t_w}\right)^2}$$

The underlying plate buckling coefficient for this equation is $k = 0.76$, which is a very conservative assumption for tee stems in flexural compression. An extensive direct analysis was performed by Richard Kaehler and Benjamin Schafer of the AISC Committee on Specifications Task Committee 4, on the elastic plate stability of a rolled WT-beam under bending causing compression at the tip of the stem, and it was found that the appropriate value for the plate-buckling coefficient is $k = 1.68$, resulting in Equation F9-19:

$$F_{cr} = \frac{\pi^2 Ek}{12\,(1-\nu^2)\left(\dfrac{b}{t}\right)^2} = \frac{1.52E}{\left(\dfrac{d}{t_w}\right)^2}$$

The transition point between the noncompact and slender range is:

$$\left(\frac{d}{t_w}\right)_r = \lambda_r = 1.52\sqrt{\frac{E}{F_y}}$$

as listed in Table B4.1b, Case 14.

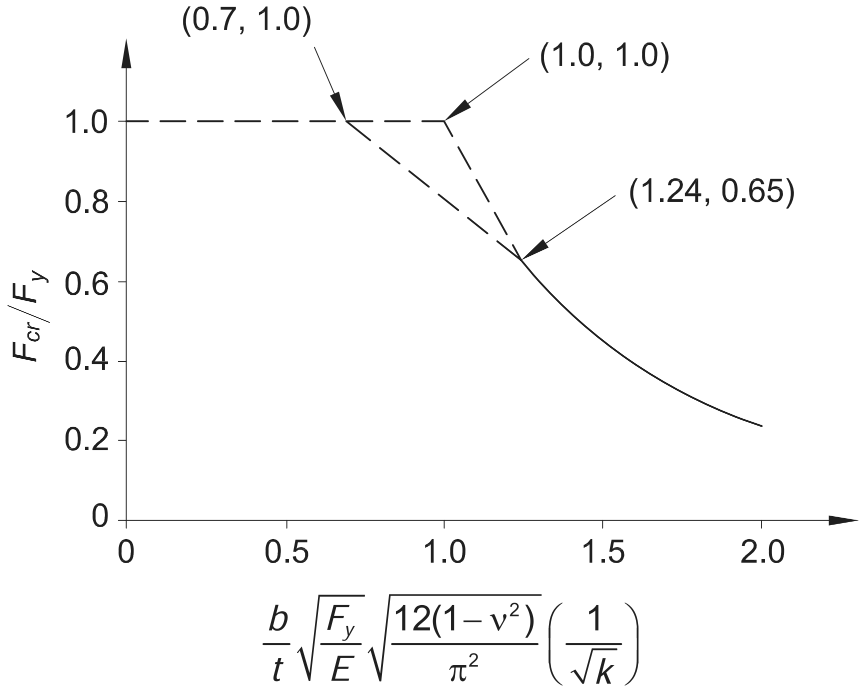


Fig. C-F9.3. General scheme for plate local buckling limit states.

The comparison between the web local buckling curves in the 2016 and the 2010 editions of the AISC *Specification* are illustrated in Figure C-F9.4.

Flexure about the *y*-axis of tees and double angles does not occur frequently and is not covered in this Specification. However, guidance is given here to address this condition. The yield limit state and the local buckling limit state of the flange can be checked by using Equations F6-1 through F6-3. Lateral-torsional buckling can conservatively be calculated by assuming the flange acts alone as a rectangular beam, using Equations F11-2 through F11-4. Alternately, an elastic critical moment given as:

$$M_e = \frac{\pi}{L_b}\sqrt{EI_xGJ} \tag{C-F9-3}$$

may be used in Equations F10-2 or F10-3 to obtain the nominal flexural strength.

F10. SINGLE ANGLES

Flexural strength limits are established for the limit states of yielding, lateral-torsional buckling, and leg local buckling of single-angle beams. In addition to addressing the general case of unequal-leg single angles, the equal-leg angle is treated as a special case. Furthermore, bending of equal-leg angles about a geometric axis, an axis parallel to one of the legs, is addressed separately as it is a common case of angle bending.

The tips of an angle refer to the free edges of the two legs. In most cases of unrestrained bending, the flexural stresses at the two tips will have the same sign (tension or compression). For constrained bending about a geometric axis, the tip stresses will differ in sign. Provisions for both tension and compression at the tip should be checked, as appropriate, but in most cases it will be evident which controls.

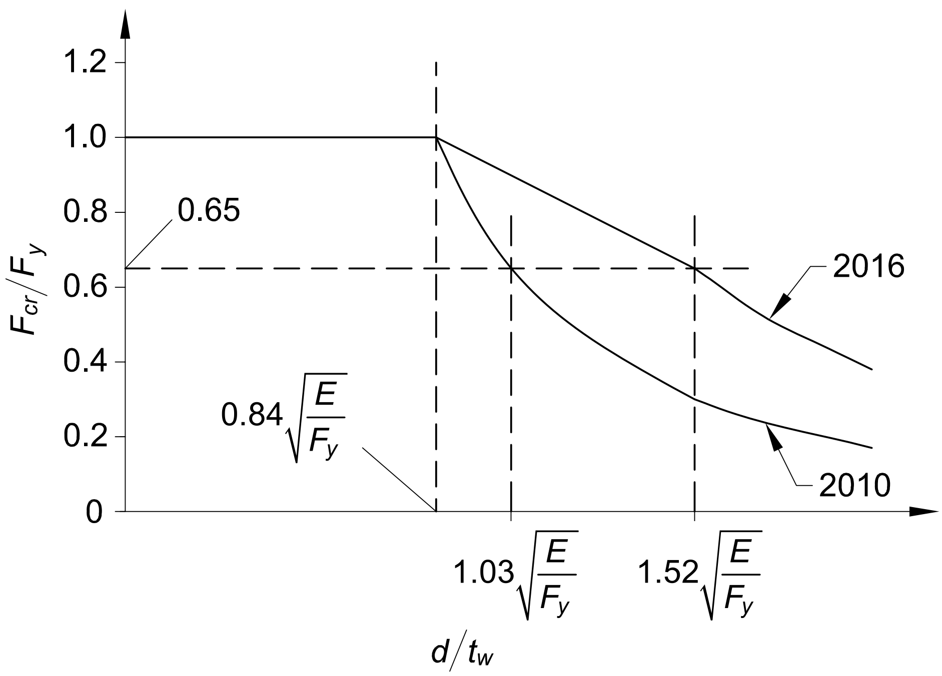


Fig. C-F9.4. Local buckling of tee stem in flexural compression.

Appropriate serviceability limits for single-angle beams need to also be considered. In particular, for longer members subjected to unrestrained bending, deflections are likely to control rather than lateral-torsional buckling or leg local buckling strength.

The provisions in this section follow the general format for nominal flexural resistance (see Figure C-F1.2). There is a region of full plastification, a linear transition to the yield moment, and a region of local buckling.

1. Yielding

The strength at full yielding is limited to 1.5 times the yield moment. This limit acts as a limit on the ratio of plastic moment to yield moment, M_p/M_y, which can also be represented as Z/S. This ratio is also known as the shape factor. The limit in Equation F10-1 assures an upper bound plastic moment for an angle that could be bent about any axis, inasmuch as these provisions are applicable to all flexural conditions. A 1.25 factor had been used in the past and was known to be a conservative value. Research work (Earls and Galambos, 1997) has indicated that the 1.5 factor represents a better upper bound value. Since the shape factor for angles is in excess of 1.5, the nominal design strength, $M_n = 1.5M_y$, for compact members is justified provided that instability does not control.

2. Lateral-Torsional Buckling

Lateral-torsional buckling may limit the flexural strength of an unbraced single-angle beam. As illustrated in Figure C-F10.1, Equation F10-3 represents the elastic buckling portion with the maximum nominal flexural strength, M_n, equal to 75% of the theoretical buckling moment, M_{cr}. Equation F10-2 represents the inelastic buckling transition expression between $0.75M_y$ and $1.5M_y$. The maximum beam flexural

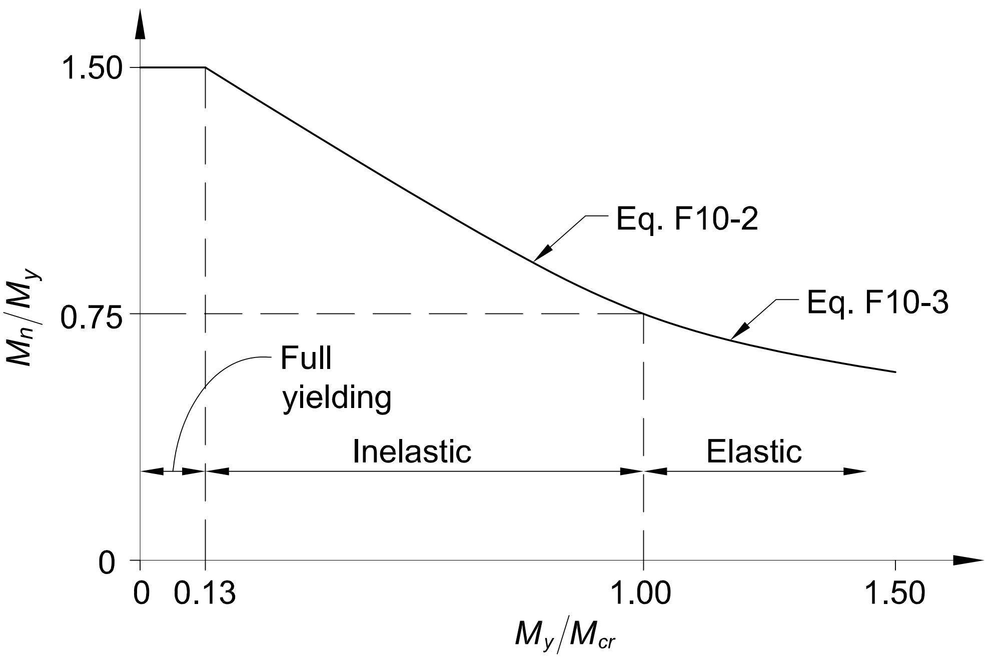


Fig. C-F10.1. Lateral-torsional buckling limits of a single-angle beam.

strength, $M_n = 1.5M_y$, will occur when the theoretical buckling moment, M_{cr}, reaches or exceeds $7.7M_y$. M_y is the moment at first yield in Equations F10-2 and F10-3, the same as the M_y in Equation F10-1. These equations are modifications of those developed from the results of Australian research on single angles in flexure and on an analytical model consisting of two rectangular elements of length equal to the actual angle leg width minus one-half the thickness (AISC, 1975; Leigh and Lay, 1978, 1984; Madugula and Kennedy, 1985).

When bending is applied about one leg of a laterally unrestrained single angle, the angle will deflect laterally as well as in the bending direction. Its behavior can be evaluated by resolving the load and/or moments into principal axis components and determining the sum of these principal axis flexural effects. Subsection (i) of Section F10.2(2) is provided to simplify and expedite the calculations for this common situation with equal-leg angles. For such unrestrained bending of an equal-leg angle, the resulting maximum normal stress at the angle tip (in the direction of bending) will be approximately 25% greater than the calculated stress using the geometric axis section modulus. The value of M_{cr} given by Equations F10-5a and F10-5b and the evaluation of M_y using 0.80 of the geometric axis section modulus reflect bending about the inclined axis shown in Figure C-F10.2. Dumonteil (2009) compares the results using the geometric axis approach with that of the principal axis approach for lateral-torsional buckling.

The deflection calculated using the geometric axis moment of inertia has to be increased 82% to approximate the total deflection. Deflection has two components: a vertical component (in the direction of applied load) of 1.56 times the calculated value and a horizontal component of 0.94 times the calculated value. The resultant total deflection is in the general direction of the minor principal axis bending of the

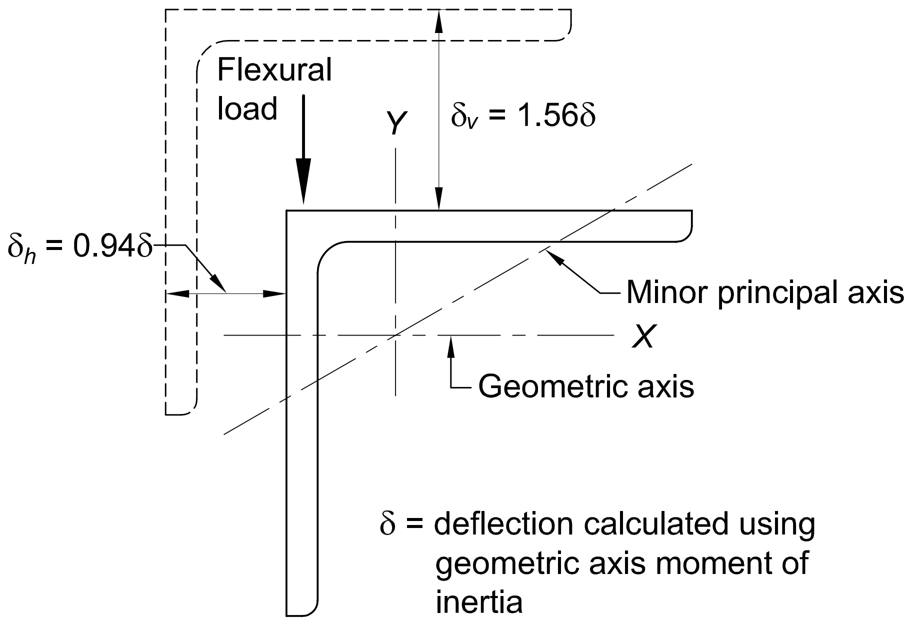


Fig. C-F10.2. Deflection for geometric axis bending of laterally unrestrained equal-leg angles.

angle (see Figure C-F10.2). These unrestrained bending deflections should be considered in evaluating serviceability and will often control the design over lateral-torsional buckling.

The horizontal component of deflection being approximately 60% of the vertical deflection means that the lateral restraining force required to achieve purely vertical deflection must be 60% of the applied load value (or produce a moment 60% of the applied value), which is very significant.

Lateral-torsional buckling is limited by M_{cr} (Leigh and Lay, 1978, 1984) as defined in Equation F10-5a, which is based on

$$M_{cr} = \frac{2.33Eb^4 t}{(1+3\cos^2\theta)(KL)^2}\left[\sqrt{\sin^2\theta + \frac{0.156(1+3\cos^2\theta)(KL)^2 t^2}{b^4}} + \sin\theta\right] \quad \text{(C-F10-1)}$$

(the general expression for the critical moment of an equal-leg angle) with $\theta = -45°$ for the condition where the angle tip stress is compressive (see Figure C-F10.3). Lateral-torsional buckling can also limit the flexural strength of the cross section when the maximum angle tip stress is tensile from geometric axis flexure, especially with use of the flexural strength limits in Section F10.2. Using $\theta = 45°$ in Equation C-F10-1, the resulting expression is Equation F10-5b with a +1 instead of −1 as the last term.

Stress at the tip of the angle leg parallel to the applied bending axis is of the same sign as the maximum stress at the tip of the other leg when the single angle is unrestrained. For an equal-leg angle this stress is about one-third of the maximum stress. It is only necessary to check the nominal bending strength based on the tip of the angle leg with the maximum stress when evaluating such an angle. If an angle is sub-

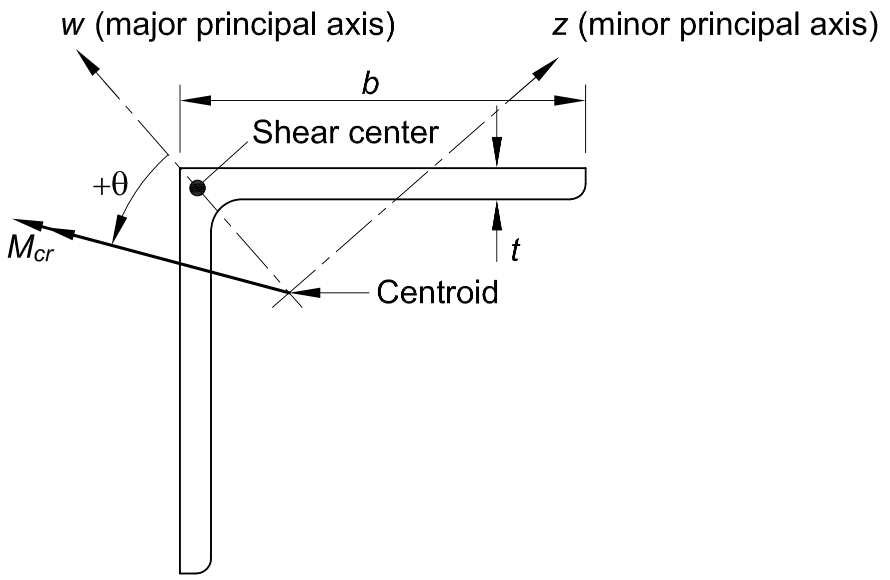


Fig. C-F10.3. Equal-leg angle with general moment loading.

jected to an axial compressive load, the flexural limits obtained from Section F10.2(2) cannot be used due to the inability to calculate a proper moment magnification factor for use in the interaction equations.

For unequal-leg angles and for equal-leg angles in compression without lateral-torsional restraint, the applied load or moment must be resolved into components along the two principal axes in all cases and design must be for biaxial bending using the interaction equations in Chapter H.

Under major-axis bending of single angles, Equation F10-4 in combination with Equations F10-2 and F10-3 control the available moment against overall lateral-torsional buckling of the angle. This is based on M_{cr} given in Equation C-F10-1 with $\theta = 0°$.

Lateral-torsional buckling will reduce the stress below $1.5M_y$ only for $M_{cr} < 7.7M_y$. For an equal-leg angle bent about its major principal axis, this occurs for $L_b/t \geq 3{,}700C_b/F_y$. If the $L_b t/b^2$ parameter is small (less than approximately $0.44C_b$ for this case), local buckling will control the available moment and M_n based on lateral-torsional buckling need not be evaluated. Local buckling must be checked using Section F10.3.

Lateral-torsional buckling about the major principal axis, *w*-axis, of an angle is controlled by M_{cr} in Equation F10-4. The section property, β_w, which is nonzero for unequal-leg angles reflects the location of the shear center relative to the principal axis of the section and the bending direction under uniform bending. Positive β_w and maximum M_{cr} occurs when the shear center is in flexural compression while negative β_w and minimum M_{cr} occur when the shear center is in flexural tension (see Figure C-F10.4). This β_w effect is consistent with the behavior of singly symmetric I-shaped beams, which are more stable when the compression flange is larger than the tension flange.

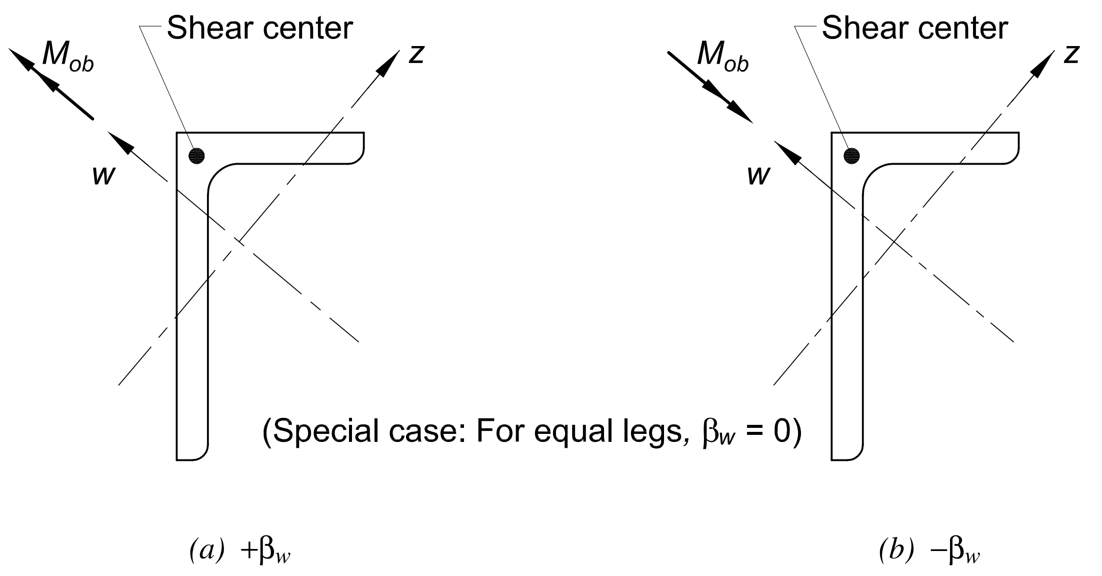


Fig. C-F10.4. Unequal-leg angle in bending.

TABLE C-F10.1
β_w Values for Angles

Angle size, in. (mm)	β_w, in. (mm)[a]
8 × 6 (203 × 152) 8 × 4 (203 × 102)	3.31 (84.1) 5.48 (139)
7 × 4 (178 × 102)	4.37 (111)
6 × 4 (152 × 102) 6 × 3½ (152 × 89)	3.14 (79.8) 3.69 (93.7)
5 × 3½ (127 × 89) 5 × 3 (127 × 76)	2.40 (61.0) 2.99 (75.9)
4 × 3½ (102 × 89) 4 × 3 (102 × 76)	0.87 (22.1) 1.65 (41.9)
3½ × 3 (89 × 76) 3½ × 2½ (89 × 64)	0.87 (22.1) 1.62 (41.1)
3 × 2½ (76 × 64) 3 × 2 (76 × 51)	0.86 (21.8) 1.56 (39.6)
2½ × 2 (64 × 51)	0.85 (21.6)
2½ × 1½ (64 × 38)	1.49 (37.8)
Equal legs	0.00

[a] $\beta_w = \dfrac{1}{I_w}\displaystyle\int_A z\left(w^2 + z^2\right)dA - 2z_o$

where

z_o = coordinate along the z-axis of the shear center with respect to the centroid, in. (mm)

I_w = moment of inertia for the major principal axis, in.^{4} (mm^4)

β_w has a positive or negative value depending on the direction of bending (see Figure C-F10.4).

For reverse curvature bending, part of the unbraced length has positive β_w, while the remainder has negative β_w; conservatively, the negative value is assigned for that entire unbraced segment.

The factor β_w is essentially independent of angle thickness (less than 1% variation from mean value) and is primarily a function of the leg widths. The average values shown in Table C-F10.1 may be used for design.

3. Leg Local Buckling

The b/t limits were modified for the 2010 AISC *Specification* to be more representative of flexural limits rather than using those for single angles under uniform compression. Typically, the flexural stresses will vary along the leg length permitting the use of the stress limits given. Even for the geometric axis flexure case, which produces uniform compression along one leg, use of these limits will provide a conservative value when compared to the results reported in Earls and Galambos (1997).

F11. RECTANGULAR BARS AND ROUNDS

The provisions in Section F11 apply to solid bars with round and rectangular cross section. The prevalent limit state for such members is the attainment of the full plastic moment, M_p. The exception is the lateral-torsional buckling of rectangular bars where the depth is larger than the width. The requirements for design are identical to those given previously in Table A-F1.1 in the 1999 LRFD *Specification* (AISC, 2000b) and the same as those in use since the 2005 AISC *Specification* (AISC, 2005). Since the shape factor, Z/S, for a rectangular cross section is 1.5 and for a round section is 1.7, consideration must be given to serviceability issues such as excessive deflection or permanent deformation under service-load conditions.

F12. UNSYMMETRICAL SHAPES

When the design engineer encounters beams that do not contain an axis of symmetry, or any other shape for which there are no provisions in the other sections of Chapter F, the stresses are to be limited by the yield stress or the elastic buckling stress. The stress distribution and/or the elastic buckling stress must be determined from principles of structural mechanics, textbooks or handbooks, such as the SSRC Guide (Ziemian, 2010), papers in journals, or finite element analyses. Alternatively, the designer can avoid the problem by selecting cross sections from among the many choices addressed in the previous sections of Chapter F.

F13. PROPORTIONS OF BEAMS AND GIRDERS

1. Strength Reductions for Members with Holes in the Tension Flange

Historically, provisions for proportions of rolled beams and girders with holes in the tension flange were based upon either a percentage reduction independent of material strength or a calculated relationship between the tension rupture and tension yield strengths of the flange, with resistance factors or safety factors included in the calculation. In both cases, the provisions were developed based upon tests of steel with a specified minimum yield stress of 36 ksi (250 MPa) or less.

More recent tests (Dexter and Altstadt, 2004; Yuan et al., 2004) indicate that the flexural strength on the net section is better predicted by comparison of the quantities F_yA_{fg} and F_uA_{fn}, with a slight adjustment when the ratio of F_y to F_u exceeds 0.8. If the holes remove enough material to affect the member strength, the critical stress is adjusted from F_y to F_uA_{fn}/A_{fg} and this value is conservatively applied to the elastic section modulus, S_x.

The resistance factor and safety factor used throughout this chapter, $\phi = 0.90$ and $\Omega = 1.67$, are those normally applied for the limit state of yielding. In the case of rupture of the tension flange due to the presence of holes, the provisions of this chapter continue to apply the same resistance and safety factors. Since the effect of Equation F13-1 is to multiply the elastic section modulus by a stress that is always less than the yield stress, it can be shown that this resistance and safety factor always give conservative results when $Z/S \le 1.2$. It can also be shown to be conservative when $Z/S > 1.2$, and a more accurate model for the rupture strength is used (Geschwindner, 2010a).

2. Proportioning Limits for I-Shaped Members

The provisions of this section were taken directly from Appendix G, Section G1 of the 1999 LRFD *Specification* (AISC, 2000b) and have been the same since the 2005 AISC *Specification* (AISC, 2005). They have been part of the plate-girder design requirements since 1963 and are derived from Basler and Thürlimann (1963). The web depth-to-thickness limitations are provided so as to prevent the flange from buckling into the web. Equation F13-4 was slightly modified from the corresponding Equation A-G1-2 in the 1999 LRFD *Specification* to recognize the change in the definition of residual stress from a constant 16.5 ksi (110 MPa) to 30% of the yield stress in the 2005 AISC *Specification*, as shown by the following derivation:

$$\frac{0.48E}{\sqrt{F_y\left(F_y+16.5\right)}} \approx \frac{0.48E}{\sqrt{F_y\left(F_y+0.3F_y\right)}} = \frac{0.42E}{F_y} \qquad \text{(C-F13-1)}$$

3. Cover Plates

Cover plates need not extend the entire length of the beam or girder. The end connection between the cover plate and beam must be designed to resist the full force in the cover plate at the theoretical cutoff point. The end force in a cover plate on a beam whose required strength exceeds the available yield strength, $\phi M_y = \phi F_y S_x$ (LFRD) or $M_y / \Omega = F_y S_x / \Omega$ (ASD), of the combined shape can be determined by an elastic-plastic analysis of the cross section but can conservatively be taken as the full yield strength of the cover plate for LRFD or the full yield strength of the cover plate divided by 1.5 for ASD. The forces in a cover plate on a beam whose required strength does not exceed the available yield strength of the combined section can be determined using the elastic distribution, MQ / I.

The requirements for minimum weld lengths on the sides of cover plates at each end reflect uneven stress distribution in the welds due to shear lag in short connections.

The requirement that the area of cover plates on bolted girders be limited was removed for this Specification since there was no justification to treat bolted girders any differently than welded girders when considering the size of the cover plate.

5. Unbraced Length for Moment Redistribution

The moment redistribution provisions of Section B3.3 refer to this section for setting the maximum unbraced length, L_m, when moments are to be redistributed. These provisions have been a part of the AISC *Specification* since the 1949 edition (AISC, 1949). Portions of members that would be required to rotate inelastically while the moments are redistributed need more closely spaced bracing than similar parts of a continuous beam. However, the magnitude of L_m is often larger than L_p. This is because the L_m expression accounts for moment gradient directly, while designs based upon an elastic analysis rely on C_b factors from Section F1.1 to account for the benefits of moment gradient. Equations F13-8 and F13-9 define the maximum permitted unbraced length in the vicinity of redistributed moment for doubly symmetric and singly symmetric I-shaped members with a compression flange equal to or larger than the tension flange bent about their major axis, and for solid rectangular bars and symmetric box beams bent about their major axis, respectively. These equations are identical to those in Appendix 1 of the 2005 AISC *Specification* (AISC, 2005) and the 1999 LRFD *Specification* (AISC, 2000b), and are based on research reported in Yura et al. (1978). They are different from the corresponding equations in Chapter N of the 1989 AISC *Specification* (AISC, 1989).

CHAPTER G
DESIGN OF MEMBERS FOR SHEAR

G1. GENERAL PROVISIONS

Chapter G applies to webs of I-shaped members subject to shear in the plane of the web, single angles, tees, and HSS. It also applies to flanges of I-shaped members and tees subject to shear in the *y*-direction.

G2. I-SHAPED MEMBERS AND CHANNELS

Two shear strength prediction methods are presented. The method in Section G2.1 accounts for the web shear post-bucking strength in members with unstiffened webs, members with transverse stiffeners spaced wider than $3h$, and end panels of members with transverse stiffeners spaced closer than $3h$. The method of Section G2.2 accounts for the web shear post-buckling strength of interior panels of members with stiffeners spaced at $3h$ or smaller. Consideration of shear and bending interaction is not required because the shear and flexural resistances can be calculated with a sufficient margin of safety without considering this effect (White et al., 2008; Daley et al., 2016).

1. Shear Strength of Webs without Tension Field Action

Section G2.1 addresses the shear strength of I-shaped members subject to shear and bending in the plane of the web. The provisions in this section apply when post-buckling strength develops due to web stress redistribution but classical tension field action is not developed. They may be conservatively applied where it is desired to not use the tension field action enhancement for convenience in design.

The nominal shear strength of a web is defined by Equation G2-1, a product of the shear yield force, $0.6F_yA_w$, and the shear post-buckling strength reduction factor, C_{v1}. The formulation is based on the Rotated Stress Field Theory (Höglund, 1997), which includes post-buckling strength due to web stress redistribution in members with or without transverse stiffeners. Höglund presented equations for members with rigid end posts (in essence, vertical beams spanning between flanges) and nonrigid end posts such as regular bearing stiffeners. The latter equation was written in the form of the familiar C_v formulation from prior AISC *Specifications* and modified slightly for use in Section G2.1 (Daley et al., 2016; Studer et al., 2015).

The provisions in Section G2.1(a) for rolled I-shaped members with $h/t_w \le 2.24\sqrt{E/F_y}$ are similar to the 1999 and earlier LRFD provisions, with the exception that ϕ has been increased from 0.90 to 1.00 (with a corresponding decrease of the safety factor from 1.67 to 1.50), thus making these provisions consistent with the 1989 provisions for allowable stress design (AISC, 1989). The value of ϕ of 1.00 is justified by comparison with experimental test data and recognizes the minor consequences of shear yielding, as compared to tension and compression yielding, on the overall performance of rolled I-shaped members. This increase is applicable only to the shear yielding limit state of rolled I-shaped members.

Section G2.1(b) uses the shear post-buckling strength reduction factor, C_{v1}, shown in Figure C-G2.1. The curve for C_{v1} has two segments whereas the previous AISC Section G2.1 provisions for C_v had three (AISC, 2010).

For webs with $h/t_w \le 1.10\sqrt{k_v E / F_{yw}}$, the nominal shear strength, V_n, is based on shear yielding of the web, with $C_{v1} = 1.0$ as given by Equation G2-3. This h/t_w yielding limit was determined by slightly increasing the limit from Höglund (1997) to match the previous yielding limit which was based on Cooper et al. (1978).

When $h/t_w > 1.10\sqrt{k_v E / F_{yw}}$, the web shear strength is based on the shear buckling and subsequent post-buckling strength of a web with nonrigid end posts. The resulting strength reduction factor, C_{v1}, given by Equation G2-4, was determined by dividing the Höglund (1997) buckling plus post-buckling strength by the shear yield strength and increasing that ratio slightly to better match experimental measurements (Daley et al., 2016; Studer et al., 2015).

The plate buckling coefficient, k_v, for panels subject to pure shear having simple supports on all four sides is given by the following (Ziemian, 2010).

$$k_v = \begin{cases} 4.00 + \dfrac{5.34}{(a/h)^2} & \text{for } a/h \le 1 \\[2ex] 5.34 + \dfrac{4.00}{(a/h)^2} & \text{for } a/h > 1 \end{cases} \tag{C-G2-1}$$

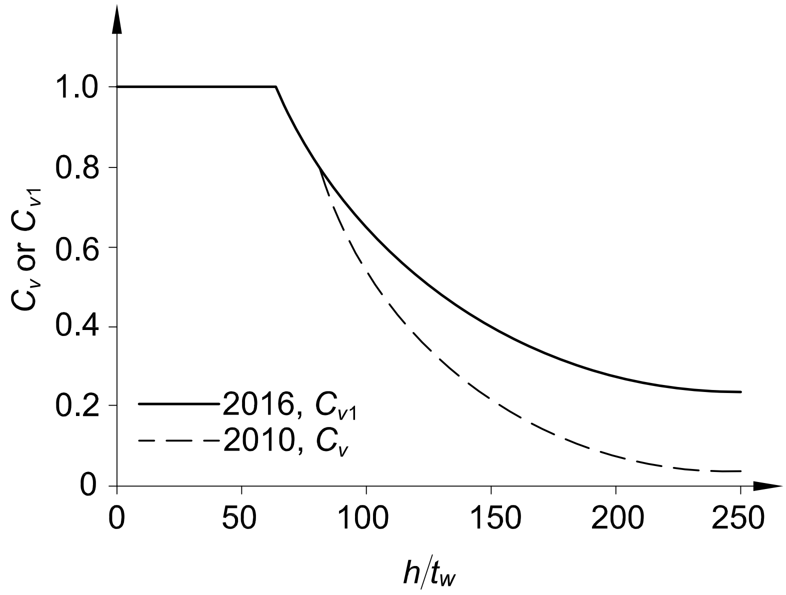


Fig. C-G2.1. Shear buckling coefficient for F_y *= 50 ksi (345 MPa).*

For simplicity, these equations have been simplified without loss of accuracy herein and in AASHTO (2014) to the following equation which is based on Vincent (1969).

$$k_v = 5 + \frac{5}{(a/h)^2} \quad \text{(C-G2-2)}$$

The plate buckling coefficient, k_v, is 5.34, for web panels with an aspect ratio, a/h, exceeding 3.0. This value is slightly larger than the value of 5.0 used in prior AISC *Specifications*, and is consistent with Höglund's developments (Höglund, 1997).

Prior AISC *Specifications* limited a/h to $[260/(h/t_w)]^2$, which was based on the following statement by Basler (1961): "In the range of high web slenderness ratios, the stiffener spacing should not be arbitrarily large. Although the web might still be sufficient to carry the shear, the distortions could be almost beyond control in fabrication and under load." The experimental evidence shows that I-shaped members develop the calculated resistances without the need for this restriction (White and Barker, 2008; White et al., 2008). Furthermore, for $a/h > 1.5$, Equation F13-4 limits the maximum h/t_w to 232 for $F_y = 50$ ksi, and for $a/h \le 1.5$, Equation F13-3 limits the web slenderness to 289 for $F_y = 50$ ksi. These limits are considered sufficient to limit distortions during fabrication and handling. The engineer should be aware of the fact that sections with highly slender webs are more apt to be controlled by the web local yielding, web local crippling, and/or web compression buckling limit states of Sections J10.2, J10.3 and J10.5. Therefore, these limit states may limit the maximum practical web slenderness in some situations.

The provisions of Section G2.1 assume monotonically increasing loads. If a flexural member is subjected to load reversals causing cyclic yielding over large portions of a web, such as may occur during a major earthquake, special design considerations may apply (Popov, 1980).

Lee et al. (2008) presented a strength prediction method that applies when $a/h \le 6$, and does not directly apply to members with long web panels. This method is accurate on average, but is not conservative enough to be used with $\phi = 0.90$ (Daley et al., 2016); it also involves more calculations than the proposed method based on Höglund (1997).

2. Shear Strength of Interior Web Panels with $a/h \le 3$ Considering Tension Field Action

The panels of the web of a built-up member, bounded on the top and bottom by the flanges and on each side by transverse stiffeners, are capable of carrying loads far in excess of their web buckling load. Upon reaching the theoretical web buckling limit, slight lateral web displacements will have developed. These deformations are of no structural significance, because other means are still present to provide further strength.

When transverse stiffeners are properly spaced and are stiff enough to resist out-of-plane movement of the post-buckled web, significant diagonal tension fields form in the web panels prior to the shear resistance limit. The web in effect acts like a Pratt truss composed of tension diagonals and compression verticals that are stabilized by the transverse stiffeners. This effective Pratt truss furnishes the strength to resist applied shear forces unaccounted for by the linear buckling theory.

The key requirement in the development of tension field action in the web of plate girders is the ability of the stiffeners to provide sufficient flexural rigidity to stabilize the web along their length. In the case of end panels there is a panel only on one side. The anchorage of the tension field is limited in many situations at these locations and is thus neglected. In addition, the enhanced resistance due to tension field forces is reduced when the panel aspect ratio becomes large. For this reason, the inclusion of tension field action is not permitted when a/h exceeds 3.0.

Analytical methods based on tension field action have been developed (Basler and Thürlimann, 1963; Basler, 1961) and corroborated in an extensive program of tests (Basler et al., 1960). Equation G2-7 is based on this research. The second term in the bracket represents the relative increase of the panel shear strength due to tension field action. The merits of Equation G2-7 relative to various alternative representations of web shear resistance are evaluated and Equation G2-7 is recommended for characterization of the shear strength of stiffened interior web panels in White and Barker (2008).

AISC *Specifications* prior to 2005 required explicit consideration of the interaction between the flexural and shear strengths when the web is designed using tension field action. White et al. (2008) show that the interaction between the shear and flexural resistances may be neglected by using a smaller tension field action shear strength for girders with $2A_w/(A_{ft}+A_{fc}) > 2.5$ or $h/b_{ft} > 6$ or $h/b_{fc} > 6$. Section G2.2 disallows the use of the traditional complete tension field action, Equation G2-7, for I-shaped members with relatively small flange-to-web proportions identified by these limits. For cases where these limits are violated, Equation G2-8 gives an applicable reduced tension field action resistance referred to as the "true Basler" tension field resistance. The true Basler resistance is based on the development of only a partial tension field, whereas Equation G2-7 is based on the development of a theoretical complete tension field. Similar limits are specified in AASHTO (2014).

3. Transverse Stiffeners

Numerous studies (Horne and Grayson, 1983; Rahal and Harding, 1990a, 1990b, 1991; Stanway et al., 1993, 1996; Lee et al., 2002b; Xie and Chapman, 2003; Kim et al., 2007; Kim and White, 2014) have shown that transverse stiffeners in I-girders designed for shear post-buckling strength, including tension field action, are loaded predominantly in bending due to the restraint they provide to lateral deflection of the web. Generally, there is evidence of some axial compression in the transverse stiffeners due to the tension field, but even in the most slender web plates permitted by this Specification, the effect of the axial compression transmitted from the post-buckled web plate is typically minor compared to the lateral loading effect. Therefore, the transverse stiffener area requirement from prior AISC *Specifications* is no longer specified. Rather, the demands on the stiffener flexural rigidity are increased in situations where the post-buckling resistance of the web is relied upon. Equation G2-13 is the same requirement as specified in AASHTO (2014).

G3. SINGLE ANGLES AND TEES

Shear stresses in single-angle members and tee stems are the result of the gradient of the bending moment along the length (flexural shear) and the torsional moment.

For angles, the maximum elastic stress due to flexural shear is:

$$f_v = \frac{1.5V_b}{bt} \tag{C-G3-1}$$

where V_b is the component of the shear force parallel to the angle leg with width b and thickness t. The stress is constant throughout the thickness and it should be calculated for both legs to determine the maximum. The coefficient 1.5 is the calculated value for equal-leg angles loaded along one of the principal axes. For equal-leg angles loaded along one of the geometric axes, this factor is 1.35. Factors between these limits may be calculated conservatively from V_bQ/It to determine the maximum stress at the neutral axis. Alternatively, if only flexural shear is considered, a uniform flexural shear stress in the leg of V_b/bt may be used due to inelastic material behavior and stress redistribution.

If the angle is not laterally braced against twist, a torsional moment is produced equal to the applied transverse load times the perpendicular distance, e, to the shear center, which is at the point of intersection of the centerlines of the two legs. Torsional moments are resisted by two types of shear behavior: pure torsion (St. Venant torsion) and warping torsion (Seaburg and Carter, 1997). The shear stresses due to restrained warping are small compared to the St. Venant torsion (typically less than 20%) and they can be neglected for practical purposes. The applied torsional moment is then resisted by pure shear stresses that are constant along the width of the leg (except for localized regions at the toe of the leg), and the maximum value can be approximated by

$$f_v = \frac{M_T t}{J} = \frac{3M_T}{At} \tag{C-G3-2}$$

where

A = cross-sectional area of angle, in.^{2} (mm^2)

J = torsional constant (approximated by $\Sigma(bt^3/3)$ when precomputed value is unavailable), in.^{4} (mm^4)

M_T = torsional moment, kip-in. (N-mm)

For a study of the effects of warping, see Gjelsvik (1981). Torsional moments from laterally unrestrained transverse loads also produce warping normal stresses that are superimposed on the bending stresses. However, since the warping strength of single angles is relatively small, this additional bending effect, just like the warping shear effect, can be neglected for practical purposes.

G4. RECTANGULAR HSS, BOX SECTIONS, AND OTHER SINGLY AND DOUBLY SYMMETRIC MEMBERS

The shear strength of rectangular HSS and box section webs is taken as the shear yield strength if web slenderness, h/t_w, does not exceed the yielding limit, or the shear buckling strength. Post-buckling strength from Section G2.1 is not included due to lack of experimental verification.

G5. ROUND HSS

Little information is available on round HSS subjected to transverse shear; therefore, the recommendations are based on local buckling of cylinders due to torsion. However, since torsion is generally constant along the member length and transverse shear usually has a gradient, it is recommended to take the critical stress for transverse shear as 1.3 times the critical stress for torsion (Brockenbrough and Johnston, 1981; Ziemian, 2010). The torsion equations apply over the full length of the member, but for transverse shear it is reasonable to use the length between the points of maximum and zero shear force. Only thin HSS may require a reduction in the shear strength based upon first shear yield.

In the equation for the nominal shear strength, V_n, it is assumed that the shear stress at the neutral axis, VQ/Ib, is at F_{cr}. For a thin round section with radius R and thickness t, $I = \pi R^3 t$, $Q = 2R^2 t$ and $b = 2t$. This gives the stress at the centroid as $V/\pi Rt$, in which the denominator is recognized as half the area of the round HSS.

G6. WEAK-AXIS SHEAR IN DOUBLY SYMMETRIC AND SINGLY SYMMETRIC SHAPES

The weak-axis shear strength of I-shaped members and channel flanges is the shear yield strength if flange slenderness, $b_f/2t_f$ for I-shapes or b_f/t_f for channels, does not exceed the limit $1.10\sqrt{k_v E/F_y}$, or the shear buckling strength, otherwise. Because shear post-buckling strength is not included for these cases due to lack of experimental verification, the shear buckling coefficient, C_{v2}, from Section G2.2 is used. The plate buckling coefficient, k_v, is 1.2 due to the presence of a free edge.

The maximum plate slenderness of all rolled shapes is $b_f/t_f = b_f/2t_f = 13.8$. The lower bound of $1.10\sqrt{k_v E/F_y}$, computed using F_y = 100 ksi, is

$$1.10\sqrt{(1.2)(29{,}000\,\text{ksi})/100} = 20.5$$

The maximum plate slenderness does not exceed the lower bound of the yielding limit; therefore, $C_{v2} = 1.0$, except for built-up shapes with very slender flanges.

G7. BEAMS AND GIRDERS WITH WEB OPENINGS

Web openings may be used to accommodate various mechanical, electrical and other systems. Strength limit states, including local buckling of the compression flange or of the web, local buckling or yielding of the tee-shaped compression zone above or below the opening, lateral buckling and moment-shear interaction, or serviceability may control the design of a flexural member with web openings. The location, size, and number of openings are important and empirical limits for them have been identified. One general procedure for assessing these effects and the design of any needed reinforcement for both steel and composite beams is given in the ASCE *Specification for Structural Steel Beams with Web Openings* (ASCE, 1999), with background information provided in AISC Design Guide 2, *Steel and Composite Beams with Web Openings* (Darwin, 1990), and in ASCE (1992a, 1992b).

CHAPTER H

DESIGN OF MEMBERS FOR COMBINED FORCES AND TORSION

Chapters D, E, F and G of this Specification address members subject to only one type of force: axial tension, axial compression, flexure and shear, respectively, or to multiple forces that can be treated as only one type of force. This chapter addresses members subject to a combination of two or more of these individual forces, as well as possibly by additional forces due to torsion. The provisions fall into two categories: (a) the majority of the cases that can be handled by an interaction equation involving sums of ratios of required strengths to the available strengths; and (b) cases where the stresses due to the applied forces are added and compared to limiting buckling or yield stresses. Designers will have to consult the provisions of Sections H2 and H3 only in rarely occurring cases.

H1. DOUBLY AND SINGLY SYMMETRIC MEMBERS SUBJECT TO FLEXURE AND AXIAL FORCE

1. Doubly and Singly Symmetric Members Subject to Flexure and Compression

This section contains design provisions for doubly symmetric and singly symmetric members under combined flexure and compression, and under combined flexure and tension. The provisions of this section apply typically to rolled wide-flange shapes, channels, tee-shapes, round, square and rectangular HSS, solid rounds, squares, rectangles or diamonds, and any of the many possible combinations of doubly or singly symmetric shapes fabricated from plates and/or shapes by welding or bolting. The interaction equations accommodate flexure about one or both principal axes as well as axial compression or tension. The restriction on the ratio I_{yc}/I_y previously included in Section H1.1 was found to be unnecessary and has been removed.

In 1923, the first AISC *Specification* (AISC, 1923) required that the stresses due to flexure and compression be added and that the sum not exceed the allowable value. An interaction equation appeared first in the 1936 AISC *Specification* (AISC, 1936), stating "Members subject to both axial and bending stresses shall be so proportioned that the quantity $\dfrac{f_a}{F_a}+\dfrac{f_b}{F_b}$ shall not exceed unity," in which F_a and F_b are, respectively, the axial and flexural allowable stresses permitted by this Specification, and f_a and f_b are the corresponding stresses due to the axial force and the bending moment, respectively. This linear interaction equation was in force until the 1961 AISC *Specification* (AISC, 1961), when it was modified to account for frame stability and for the P-δ effect, that is, the secondary bending between the ends of the members (Equation C-H1-1). The P-Δ effect, that is, the second-order bending moment due to story sway, was not accommodated.

$$\frac{f_a}{F_a}+\frac{C_m f_b}{\left(1-\dfrac{f_a}{F'_e}\right)F_b}\le 1.0 \tag{C-H1-1}$$

The allowable axial stress, F_a, was usually determined for an effective length that is larger than the actual member length for moment frames. The term $\dfrac{1}{1-\dfrac{f_a}{F'_e}}$ is the amplification of the interspan moment due to member deflection multiplied by the axial force (the P-δ effect). C_m accounts for the effect of the moment gradient. This interaction equation was part of all the subsequent editions of the AISC ASD *Specifications* from 1961 through 1989.

A new approach to the interaction of flexural and axial forces was introduced in the 1986 AISC *Load and Resistance Factor Design Specification for Structural Steel Buildings* (AISC, 1986). The following is an explanation of the thinking behind the interaction curves used. The equations

$$\frac{P}{P_y}+\frac{8}{9}\frac{M_{pc}}{M_p}=1 \;\text{ for } \frac{P}{P_y}\ge 0.2 \tag{C-H1-2a}$$

$$\frac{P}{2P_y}+\frac{M_{pc}}{M_p}=1 \;\text{ for } \frac{P}{P_y}< 0.2 \tag{C-H1-2b}$$

define the lower-bound curve for the interaction of the nondimensional axial strength, P/P_y, and flexural strength, M_{pc}/M_p, for compact wide-flange stub-columns bent about their x-axis. The cross section is assumed to be fully yielded in tension and compression. The symbol M_{pc} is the plastic moment strength of the cross section in the presence of an axial force, P. The curve representing Equations C-H1-2 almost overlaps the analytically exact curve for the major-axis bending of a W8×31 cross section (see Figure C-H1.1). The major-axis bending equations for the exact yield capacity of a wide-flange shape are (ASCE, 1971):

For $0\le \dfrac{P}{P_y}\le \dfrac{t_w\left(d-2t_f\right)}{A}$ (for the plastic neutral axis in the web)

$$\frac{M_{pc}}{M_p}=1-\frac{A^2\left(\dfrac{P}{P_y}\right)^2}{4t_wZ_x} \tag{C-H1-3a}$$

For $\dfrac{t_w\left(d-2t_f\right)}{A}<\dfrac{P}{P_y}\le 1$ (for the plastic neutral axis in the flange)

$$\frac{M_{pc}}{M_p}=\frac{A\left(1-\dfrac{P}{P_y}\right)}{2Z_x}\left[d-\frac{A\left(1-\dfrac{P}{P_y}\right)}{2b_f}\right] \tag{C-H1-3b}$$

For major-axis bending, an equation approximating the average yield strength of wide-flange shapes when $P \geq 1.5P_y$ is given as

$$\frac{M_{pc}}{M_p} = 1.18\left(1 - \frac{P}{P_y}\right) \leq 1 \tag{C-H1-4}$$

When $P < 0.15P_y$, M_{pc} may be taken as M_p.

The curves in Figure C-H1.2 show the exact and approximate yield interaction curves for wide-flange shapes bent about the y-axis, and the exact curves for the solid rectangular and round shapes. It is evident that the lower-bound AISC interaction curves are very conservative for these shapes.

The idea of portraying the strength of stub beam-columns was extended to actual beam-columns with actual lengths by normalizing the required flexural strength, M_u, of the beam by the nominal strength of a beam without axial force, M_n, and the required axial strength, P_u, by the nominal strength of a column without bending moment, P_n. This rearrangement results in a translation and rotation of the original stub-column interaction curve, as seen in Figure C-H1.3.

The normalized equations corresponding to the beam-column with length effects included are shown as Equation C-H1-5:

$$\frac{P_u}{P_n} + \frac{8}{9}\frac{M_u}{M_n} = 1 \quad \text{for } \frac{P_u}{P_n} \geq 0.2 \tag{C-H1-5a}$$

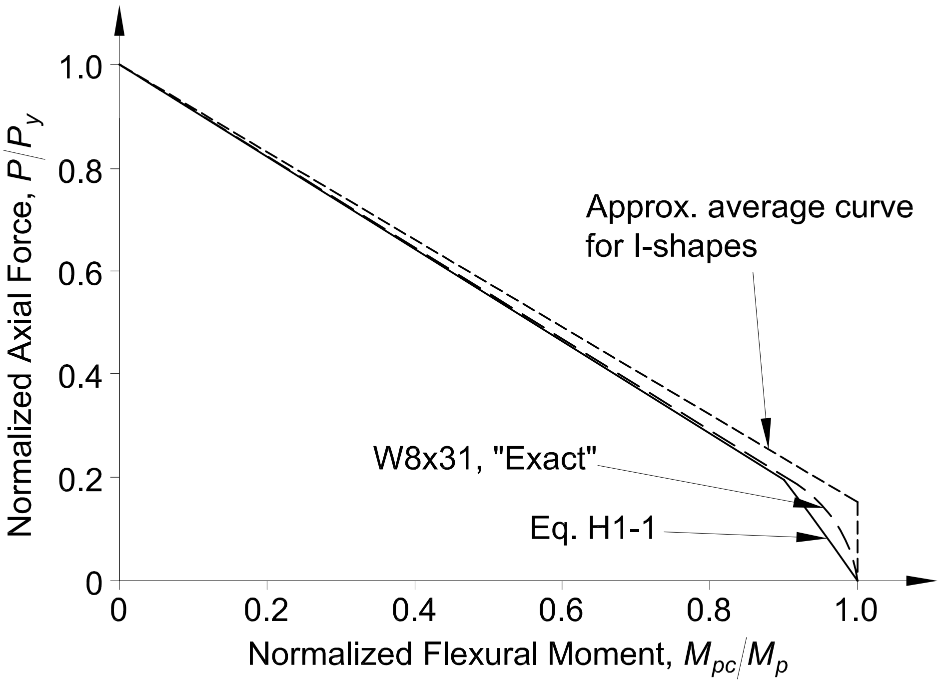


Fig. C-H1.1. Stub-column interaction curves: plastic moment versus axial force for wide-flange shapes, major-axis flexure [W8×31, F_y = 50 ksi (345 MPa)].

$$\frac{P_u}{2P_n} + \frac{M_u}{M_n} = 1 \quad \text{for } \frac{P_u}{P_n} < 0.2 \tag{C-H1-5b}$$

The interaction equations are designed to be very versatile. The terms in the denominator fix the endpoints of the interaction curve. The nominal flexural strength, M_n, is determined by the appropriate provisions from Chapter F. It encompasses the limit states of yielding, lateral-torsional buckling, flange local buckling, and web local buckling.

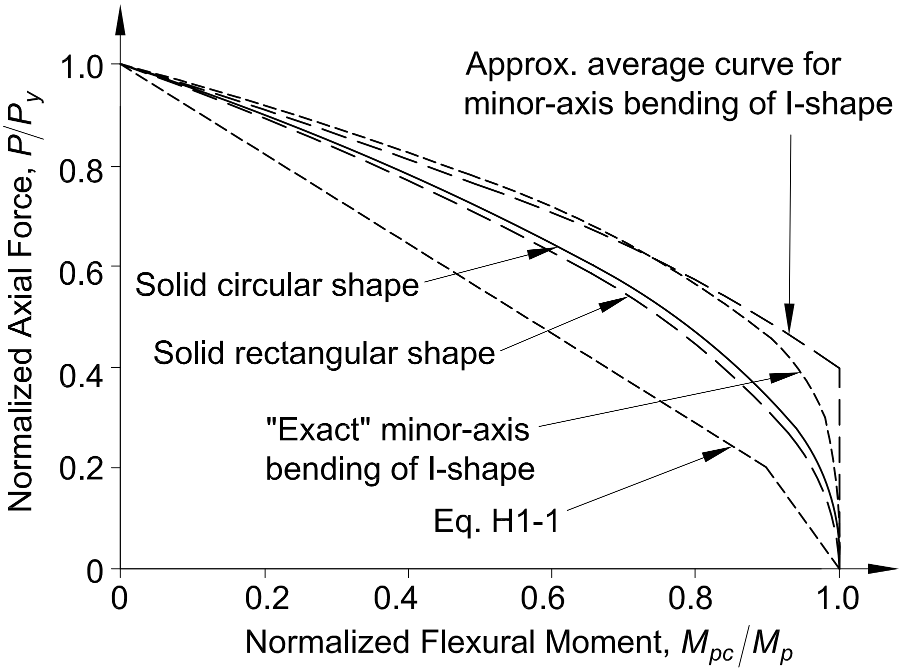


Fig. C-H1.2. Stub-column interaction curves: plastic moment versus axial force for solid round and rectangular sections and for wide-flange shapes, minor-axis flexure.

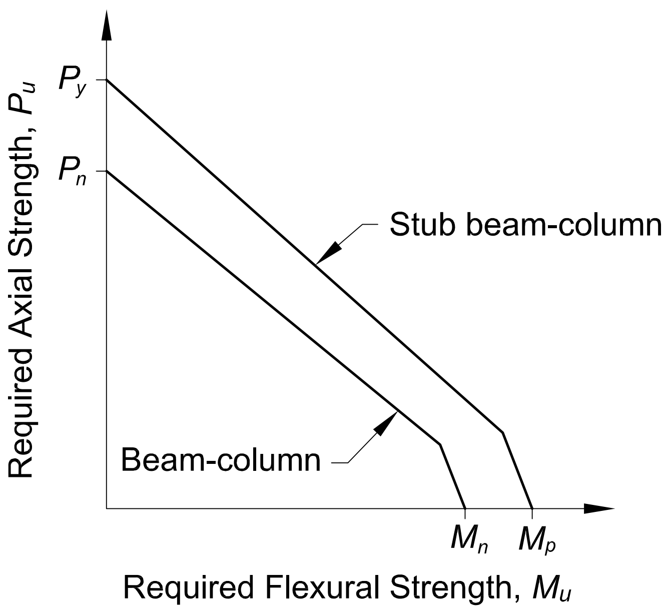


Fig. C-H1.3. Interaction curve for stub beam-column and beam-column.

The axial term, P_n, is governed by the provisions of Chapter E, and it can accommodate nonslender or slender element columns, as well as the limit states of major- and minor-axis buckling, and torsional and flexural-torsional buckling. Furthermore, P_n is calculated for the applicable effective length of the column to take care of frame stability effects, if the procedures of Appendix 7, Section 7.2 are used to determine the required moments and axial forces. These required moments and axial forces must include the amplification due to second-order effects.

The utility of the interaction equations is further enhanced by the fact that they also permit the consideration of biaxial bending without the presence of axial load.

2. Doubly and Singly Symmetric Members Subject to Flexure and Tension

Section H1.1 considers the most frequently occurring cases in design: members under flexure and axial compression. Section H1.2 addresses the less frequent cases of flexure and axial tension. Since axial tension increases the bending stiffness of the member to some extent, Section H1.2 permits the increase of C_b in Chapter F. Thus, when the bending term is controlled by lateral-torsional buckling, the moment gradient factor, C_b, is increased by

$$\sqrt{1+\frac{\alpha P_r}{P_{ey}}}$$

For the 2010 AISC *Specification* (AISC, 2010), this multiplier was altered slightly as shown here to use the same constant, α, as is used throughout the Specification when results at the ultimate strength level are required.

3. Doubly Symmetric Rolled Compact Members Subject to Single-Axis Flexure and Compression

For doubly symmetric wide-flange sections with moment applied about the *x*-axis, the bilinear interaction Equation C-H1-5 is conservative for cases where the axial limit state is out-of-plane buckling and the flexural limit state is lateral-torsional buckling (Ziemian, 2010). Since this condition is common in building structures, the provisions of this section may be quite useful to the designer and lead to a more economical structure than solutions using Section H1.1. Section H1.3 gives an optional equation for checking the out-of-plane resistance of such beam-columns.

The two curves labeled Equation H1-1 (out-of-plane) and Equation H1-3 (out-of-plane) in Figure C-H1.4 illustrate the difference between the bilinear and the parabolic interaction equations for out-of-plane resistance for the case of a W27×84 beam-column, L_b = 10 ft (3.1 m) and F_y = 50 ksi (345 MPa), subjected to a linearly varying major-axis moment with zero moment at one end and maximum moment at the other end (C_b = 1.67). In addition, the figure shows the in-plane bilinear strength interaction for this member obtained from Equation H1-1. Note that the resistance term C_bM_{cx} may be larger than ϕ_bM_p in LRFD and M_p/Ω_b in ASD. The smaller ordinate from the out-of-plane and in-plane resistance curves is the controlling strength.

Equation H1-3 is developed from the following fundamental form for the out-of-plane lateral-torsional buckling strength of doubly symmetric I-section members, in LRFD:

$$\left(\frac{M_u}{C_b \phi_b M_{nx(C_b=1)}}\right)^2 \le \left(1 - \frac{P_u}{\phi_c P_{ny}}\right)\left(1 - \frac{P_u}{\phi_c P_{ez}}\right) \qquad \text{(C-H1-6)}$$

Equation H1-3 is obtained by substituting a lower-bound of 2.0 for the ratio of the elastic torsional buckling resistance to the out-of-plane nominal flexural buckling resistance, P_{ez}/P_{ny}, for W-shape members with $L_{cy} = L_{cz}$. The 2005 AISC *Specification* (AISC, 2005) assumed an upper bound, $P_{ez}/P_{ny} = \infty$, in Equation C-H1-6 in the development of Equation H1-3 which lead to some cases where the out-of-plane strength was overestimated. In addition, the fact that the nominal out-of-plane flexural resistance term, $C_b M_{nx(C_b = 1)}$, may be larger than M_p was not apparent in the 2005 AISC *Specification*. These changes that were implemented for the 2010 AISC *Specification* have been maintained for this Specification.

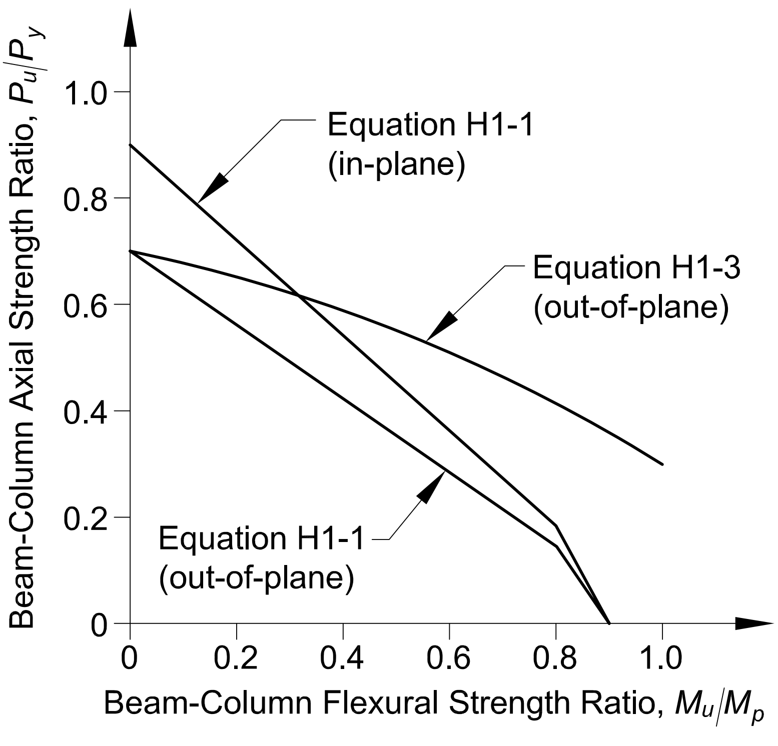


Fig. C-H1.4. Comparison between bilinear (Equation H1-1), parabolic (Equation H1-3) out-of-plane strength interaction equations, and bilinear (Equation H1-1) in-plane strength interaction equation (W27×84, F_y = 50 ksi, L_b = 10 ft, C_b =1.75).

The relationship between Equations H1-1 and H1-3 is further illustrated in Figures C-H1.5 (for LRFD) and C-H1.6 (for ASD). The curves relate the required axial force, P (ordinate), and the required bending moment, M (abscissa), when the interaction Equations H1-1 and H1-3 are equal to unity. The positive values of P are compression and the negative values are tension. The curves are for a 10 ft (3 m) long W16×26 [F_y = 50 ksi (345 MPa)] member subjected to uniform major-axis bending, C_b = 1. The solid curve is for in-plane behavior, that is, lateral bracing prevents lateral-torsional buckling. The dotted curve represents Equation H1-1 for the case when there are no lateral braces between the ends of the beam-column. In the region of the tensile axial force, the curve is modified by the term

$$\sqrt{1+\frac{\alpha P_r}{P_{ey}}}$$

as permitted in Section H1.2. The dashed curve is Equation H1-3 for the case of axial compression, and it is taken as the lower-bound determined using Equation C-H1-6 with P_{ez}/P_{ny} taken equal to infinity for the case of axial tension. For a given compressive or tensile axial force, Equations H1-3 and C-H1-6 allow a larger bending moment over most of their applicable range.

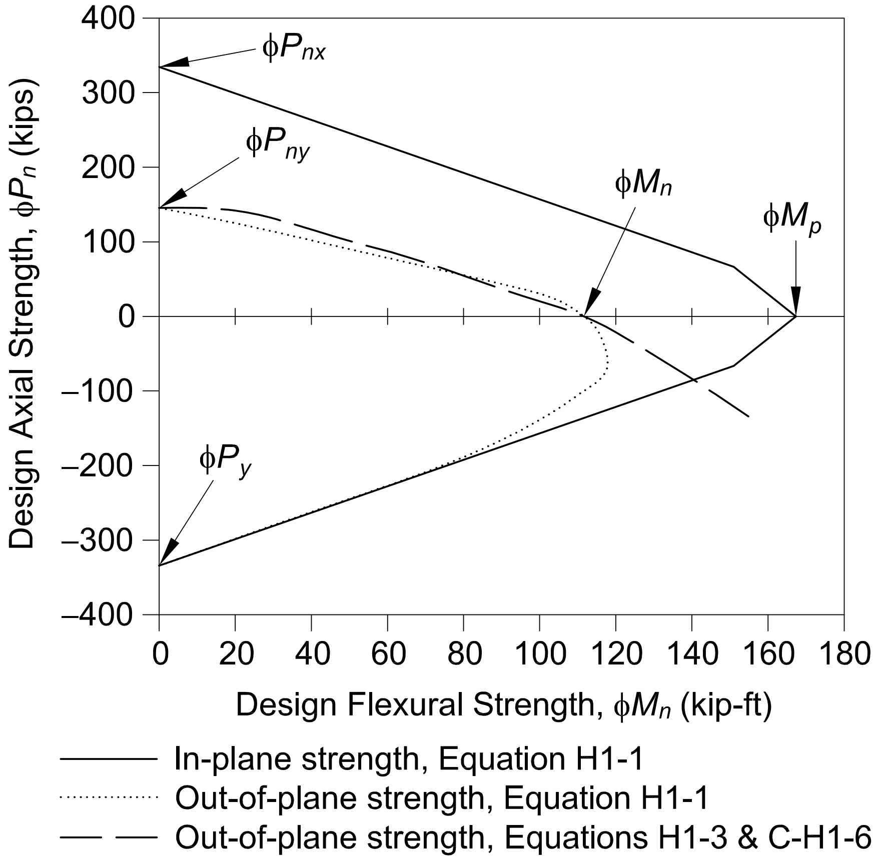


Fig. C-H1.5. Beam-columns under compressive and tensile axial force (tension is shown as negative) (LRFD) (W16×26, F_y = 50 ksi, L_b = 10 ft, C_b = 1).

H2. UNSYMMETRIC AND OTHER MEMBERS SUBJECT TO FLEXURE AND AXIAL FORCE

The provisions of Section H1 apply to beam-columns with cross sections that are either doubly or singly symmetric. However, there are many cross sections that are unsymmetrical, such as unequal-leg angles and any number of possible fabricated sections. For these situations, the interaction equations of Section H1 may not be appropriate. The linear interaction

$$\left| \frac{f_{ra}}{F_{ca}} + \frac{f_{rbw}}{F_{cbw}} + \frac{f_{rbz}}{F_{cbz}} \right| \leq 1.0$$

provides a conservative and simple way to deal with such problems. The lower case stresses, f, are the required axial and flexural stresses computed by elastic analysis for the applicable loads, including second-order effects where appropriate, and the upper case stresses, F, are the available stresses corresponding to the limit state of yielding or buckling. The subscripts r and c refer to the required and available stresses, respectively, while the subscripts w and z refer to the principal axes of the unsymmetric cross section. This Specification leaves the option to the designer to use the Section H2 interaction equation for cross sections that would qualify for the more liberal interaction equation of Section H1.

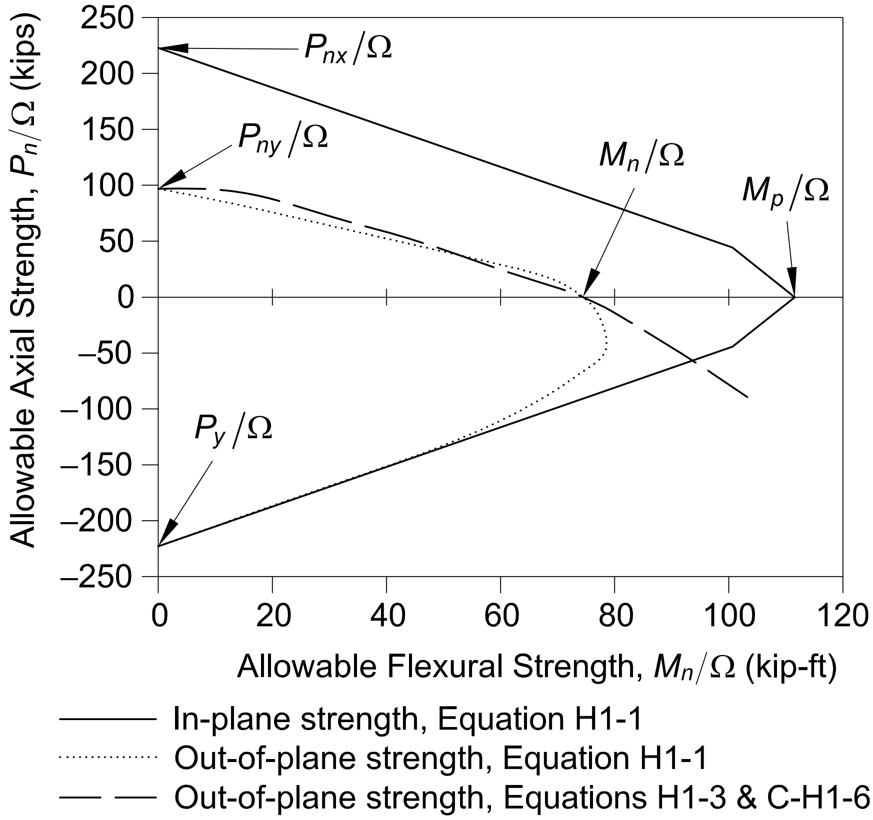


Fig. C-H1.6. Beam-columns under compressive and tensile axial force (tension is shown as negative) (ASD) (W16×26, F_y = 50 ksi, L_b = 10 ft, C_b = 1).

The interaction equation, Equation H2-1, applies equally to the case where the axial force is in tension. Equation H2-1 was written in stress format as an aid in examining the condition at the various critical locations of the unsymmetric member. For unsymmetrical sections with uniaxial or biaxial flexure, the critical condition is dependent on the resultant direction of the moment. This is also true for singly symmetric members, such as for *x*-axis flexure of tees. The same elastic section properties are used to compute the corresponding required and available flexural stress terms which means that the moment ratio will be the same as the stress ratio.

There are two approaches for using Equation H2-1:

(a) Strictly using Equation H2-1 for the interaction of the critical moment about each principal axis, there is only one flexural stress ratio term for every critical location because moment and stress ratios are the same as noted previously. In this case, one would algebraically add the value of each of the ratio terms to obtain the critical condition at one of the extreme fibers.

Using Equation H2-1 is the conservative approach and is recommended for examining members such as single angles. The available flexural stresses at a particular location (tip of short or long leg or at the heel) are based on the yielding limit moment, the local buckling limit moment, or the lateral-torsional buckling moment consistent with the sign of the required flexural stress. In each case, the yield moment should be based on the smallest section modulus about the axis being considered. One would check the stress condition at the tip of the long and short legs and at the heel and find that at one of the locations the stress ratios would be critical.

(b) For certain load components, where the critical stress can transition from tension at one point on the cross section to compression at another, it may be advantageous to consider two interaction relationships depending on the magnitude of each component. This is permitted by the sentence at the end of Section H2 that permits a more detailed analysis in lieu of Equation H2-1 for the interaction of flexure and tension.

As an example, for a tee with flexure about both the *x*- and *y*-axes creating tension at the tip of the stem, compression at the flange could control or tension at the stem could control the design. If *y*-axis flexure is large relative to *x*-axis flexure, the stress ratio need only be checked for compression at the flange using corresponding design compressive stress limits. However, if the *y*-axis flexure is small relative to the *x*-axis flexure, then one would check the tensile stress condition at the tip of the stem, this limit being independent of the amount of the *y*-axis flexure. The two differing interaction expressions are

$$\left|\frac{f_{ra}}{F_{ca}}+\frac{f_{rby}}{F_{cby}}+\frac{f_{rbx}}{F_{cbx}}\right| \le 1.0 \text{ at tee flange}$$

and

$$\left|\frac{f_{ra}}{F_{ca}}+\frac{f_{rbx}}{F_{cbx}}\right| \le 1.0 \text{ at tee stem}$$

The interaction diagrams for biaxial flexure of a WT using both approaches are illustrated in Figure C-H2.1.

Another situation in which one could benefit from consideration of more than one interaction relationship occurs when axial tension is combined with a flexural compression limit based on local buckling or lateral-torsional buckling. An example of this is when the stem of a tee in flexural compression is combined with axial tension. The introduction of the axial tension will reduce the compression which imposed the buckling stress limit. With a required large axial tension and a relatively small flexural compression, the design flexural stress could be set at the yield limit at the stem. The interaction equation is then,

$$\left|\frac{f_{ra}}{F_{ca}} + \frac{f_{rbx}}{F_{cbx}}\right| \le 1.0 \tag{C-H2-1}$$

where F_{cbx} is the flange tension stress based on reaching ϕF_y in the stem. There could be justification for using F_{cbx} equal to ϕF_y in this expression.

This interaction relationship would hold until the interaction between the flexural compressive stress at the stem with F_{cbx} based on the local or lateral-torsional buckling limit, as increased by the axial tension, would control, resulting in the following interaction.

$$\left|\frac{f_{ra}}{F_{ca}} - \frac{f_{rbx}}{F_{cbx}}\right| \le 1.0 \tag{C-H2-2}$$

The interaction diagrams for this case, using both approaches, are illustrated in Figure C-H2.2.

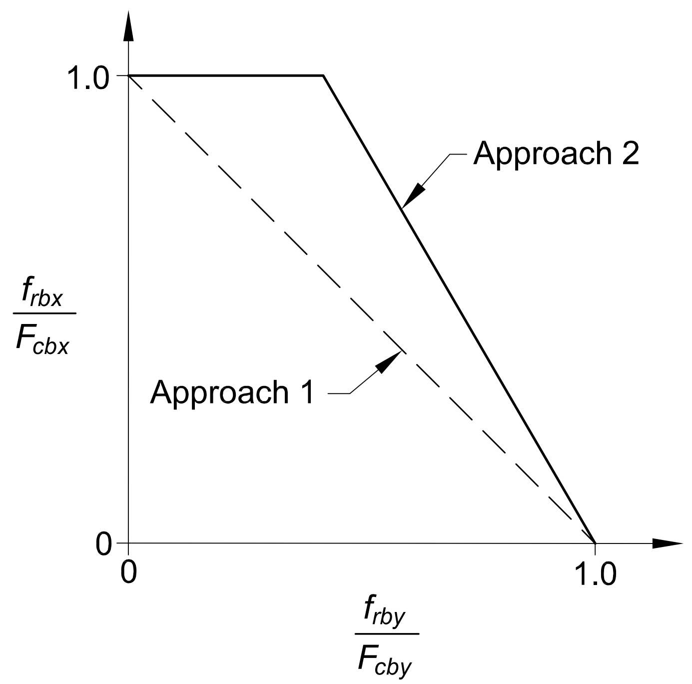


Fig. C-H2.1. WT with biaxial flexure.

H3. MEMBERS SUBJECT TO TORSION AND COMBINED TORSION, FLEXURE, SHEAR, AND/OR AXIAL FORCE

Section H3 provides provisions for cases not covered in the previous two sections. The first two parts of this section address the design of HSS members, and the third part is a general provision directed to cases where the designer encounters torsion in addition to normal stresses and shear stresses.

1. Round and Rectangular HSS Subject to Torsion

Hollow structural sections (HSS) are frequently used in space-frame construction and in other situations wherein significant torsional moments must be resisted by the members. Because of its closed cross section, an HSS is far more efficient in resisting torsion than an open cross section, such as an I-shape or a channel. While normal and shear stresses due to restrained warping are usually significant in shapes of open cross section, they are insignificant in closed cross sections. The total torsional moment can be assumed to be resisted by pure torsional shear stresses. These are often referred to in the literature as St. Venant torsional stresses.

The pure torsional shear stress in HSS sections is assumed to be uniformly distributed along the wall of the cross section, and it is equal to the torsional moment divided by a torsional shear constant for the cross section, C. In a limit state format, the nominal torsional resisting moment is the shear constant times the critical shear stress, F_{cr}.

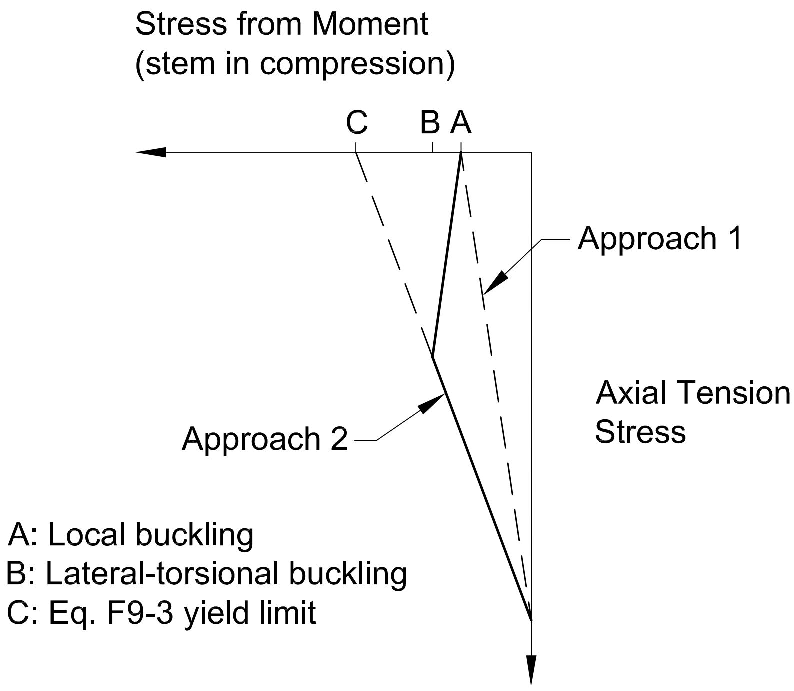


Fig. C-H2.2. WT with flexural compression on the stem plus axial tension.

For round HSS, the torsional shear constant is equal to the polar moment of inertia divided by the radius:

$$C = \frac{\pi\left(D^4 - D_i^4\right)}{32D/2} \approx \frac{\pi t\left(D-t\right)^2}{2} \tag{C-H3-1}$$

where D_i is the inside diameter.

For rectangular HSS, the torsional shear constant is obtained as $2tA_o$ using the membrane analogy (Timoshenko, 1956), where A_o is the area bounded by the midline of the section. Conservatively assuming an outside corner radius of $2t$, the midline radius is $1.5t$ and

$$A_o = \left(B-t\right)\left(H-t\right) - 9t^2\frac{\left(4-\pi\right)}{4} \tag{C-H3-2}$$

resulting in

$$C = 2t\left(B-t\right)\left(H-t\right) - 4.5t^3\left(4-\pi\right) \tag{C-H3-3}$$

The resistance factor, ϕ, and the safety factor, Ω, are the same as for flexural shear in Chapter G.

When considering local buckling in round HSS subjected to torsion, most structural members will either be long or of moderate length and the provisions for short cylinders will not apply. The elastic local buckling strength of long cylinders is unaffected by end conditions and the critical stress is given in Ziemian (2010) as

$$F_{cr} = \frac{K_t E}{\left(\dfrac{D}{t}\right)^{\frac{3}{2}}} \tag{C-H3-4}$$

The theoretical value of K_t is 0.73 but a value of 0.6 is recommended to account for initial imperfections. An equation for the elastic local buckling stress for round HSS of moderate length where the edges are not fixed at the ends against rotation is given in Schilling (1965) and Ziemian (2010) as

$$F_{cr} = \frac{1.23E}{\left(\dfrac{D}{t}\right)^{\frac{5}{4}}\sqrt{\dfrac{L}{D}}} \tag{C-H3-5}$$

This equation includes a 15% reduction to account for initial imperfections. The length effect is included in this equation for simple end conditions, and the approximately 10% increase in buckling strength is neglected for edges fixed at the end. A limitation is provided so that the shear yield strength, $0.6F_y$, is not exceeded.

The critical stress provisions for rectangular HSS are identical to the flexural shear provisions of Section G4 with the shear buckling coefficient equal to $k_v = 5.0$. The shear distribution due to torsion is uniform in the longest sides of a rectangular HSS, and this is the same distribution that is assumed to exist in the web of an I-shape beam. Therefore, it is reasonable that the provisions for buckling are the same in both cases.

2. HSS Subject to Combined Torsion, Shear, Flexure and Axial Force

Several interaction equation forms have been proposed in the literature for load combinations that produce both normal and shear stresses. In one common form, the normal and shear stresses are combined elliptically with the sum of the squares (Felton and Dobbs, 1967):

$$\left(\frac{f}{F_{cr}}\right)^2 + \left(\frac{f_v}{F_{vcr}}\right)^2 \le 1 \qquad \text{(C-H3-6)}$$

In a second form, the first power of the ratio of the normal stresses is used:

$$\left(\frac{f}{F_{cr}}\right) + \left(\frac{f_v}{F_{vcr}}\right)^2 \le 1 \qquad \text{(C-H3-7)}$$

The latter form is somewhat more conservative, but not overly so (Schilling, 1965), and this is the form used in this Specification:

$$\left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right) + \left(\frac{V_r}{V_c} + \frac{T_r}{T_c}\right)^2 \le 1.0 \qquad \text{(C-H3-8)}$$

where the terms with the subscript r represent the required strengths, and the ones with the subscript c are the corresponding available strengths. Normal effects due to flexural and axial load effects are combined linearly and then combined with the square of the linear combination of flexural and torsional shear effects. When an axial compressive load effect is present, the required flexural strength, M_c, is to be determined by second-order analysis. When normal effects due to flexural and axial load effects are not present, the square of the linear combination of flexural and torsional shear effects underestimates the actual interaction. A more accurate measure is obtained without squaring this combination.

3. Non-HSS Members Subject to Torsion and Combined Stress

This section covers all the cases not previously covered. Examples are built-up unsymmetric crane girders and many other types of odd-shaped built-up cross sections. The required stresses are determined by elastic stress analysis based on established theories of structural mechanics. The three limit states to consider and the corresponding available stresses are:

(a) Yielding under normal stress—F_y

(b) Yielding under shear stress—$0.6F_y$

(b) Buckling—F_{cr}

In most cases it is sufficient to consider normal stresses and shear stresses separately because maximum values rarely occur in the same place in the cross section or at the same place in the span. AISC Design Guide 9, *Torsional Analysis of Structural Steel Members* (Seaburg and Carter, 1997), provides a complete discussion on torsional analysis of open shapes.

H4. RUPTURE OF FLANGES WITH HOLES SUBJECTED TO TENSION

Equation H4-1 is provided to evaluate the limit state of tensile rupture of the flanges of beam-columns. This provision is only applicable in cases where there are one or more holes in the flange in net tension under the combined effect of flexure and axial forces. When both the axial and flexural stresses are tensile, their effects are additive. When the stresses are of opposite sign, the tensile effect is reduced by the compression effect.

CHAPTER I

DESIGN OF COMPOSITE MEMBERS

Chapter I includes the following major changes and additions in this edition of the Specification:

(1) References to ACI 318 have been updated to reflect the complete reorganization of that document.

(2) A new method for calculating cross-sectional strength for noncompact and slender composite sections—the effective stress-strain method—has been added in Section I1.2d.

(3) The minimum yield stress specified for reinforcing bars has been set at 80 ksi (550 MPa) in Section I1.3(c).

(4) Provisions for the stiffness of encased composite members and filled composite members to be used with the direct analysis method of design have been added in the new Section I1.5.

(5) The coefficients C_1 in Equation I2-6 and C_3 in Equation I2-12 have been modified to reflect new research. The change to Equation I2-7 reduced the considerable conservatism from the corresponding equation in the previous editions. The changes in Equation I2-13, on the other hand, will sometimes reduce the capacity from that in previous editions.

(6) A requirement to consider steel-anchor ductility has been added to Section I3.2d.

(7) Section I5 now has explicit equations for calculating axial and bending strength of noncompact and slender sections.

(8) Section I6 was revised to address load transfer in noncompact and slender composite cross sections and to update the direct bond force transfer mechanism to allow for explicit consideration of the section type and slenderness ratio.

(9) Steel headed stud anchor diameters larger than $^3/_4$ in. (19 mm) are now permitted for shear transfer in solid slabs in Section I8.1.

I1. GENERAL PROVISIONS

Design of composite sections requires consideration of both steel and concrete behavior. These provisions were developed with the intent both to minimize conflicts between current steel and concrete design and detailing provisions, and to give proper recognition to the advantages of composite design.

As a result of the attempt to minimize design conflicts, this Specification uses a cross-sectional strength approach for compression member design consistent with that used in reinforced concrete design (ACI, 2014). This approach, in addition, results in a consistent treatment of cross-sectional strengths for both composite columns and beams.

The provisions in Chapter I address strength design of the composite sections only. The designer needs to consider the loads resisted by the steel section alone when determining load effects during the construction phase. The designer also needs to consider deformations throughout the life of the structure and the appropriate cross section for those deformations. When considering these latter limit states, due allowance should be made for the additional long-term changes in stresses and deformations due to creep and shrinkage of the concrete.

1. Concrete and Steel Reinforcement

Reference is made to ACI 318 and ACI 318M (ACI, 2014), subsequently referred to as ACI 318, for provisions related to the concrete and reinforcing steel portion of composite design and detailing, such as anchorage and splice lengths, intermediate column ties, reinforcing spirals, and shear and torsion provisions.

Exceptions and limitations are provided as follows:

(1) The intent of this Specification is to exclude provisions of ACI 318 that are specifically related to composite columns and that conflict with the Specification to take advantage of recent research into composite behavior (Ziemian, 2010; Hajjar, 2000; Shanmugam and Lakshmi, 2001; Leon et al., 2007; Jacobs and Goverdhan, 2010; Lai et al., 2014; Lai et al., 2016; Denavit et al., 2016a; Lai and Varma, 2015).

The previous edition of the Specification excluded specific sections of ACI 318-08 (ACI, 2008); however, due to the reorganization of ACI 318-14 (ACI, 2014), a compact listing of affected sections is no longer practical, thus the exclusion now takes the form of a general statement of intent. Specific sections of ACI 318-14 covered by this statement include, but are not limited to, the following:

Section 6.2.5.2 (radius of gyration for composite columns)
Section 6.6.4.4.5 (calculation of $(EI)_{eff}$ for composite columns)
Section 10.3.1.6 (thickness of steel encasement)
Section 10.5.2.2 (force transfer between steel section and concrete)
Section 10.6.1.2 (limits for area of longitudinal bars)
Section 10.7.3.2 (placement of longitudinal bars)
Section 10.7.5.3.2 (bearing at ends of composite columns)
Section 10.7.6.1.4 (limits for ties)
Section 16.3.1.3 (bases of composite columns)
Section 19.2.1.1 (limits on concrete material strength)
Section 20.4.2.2 (limit on f_y for encased structural steel)
Section 22.4.2.1 (nominal axial compressive strength)
Section 25.7.2.1 (spacing of ties)

(2) Concrete limitations in addition to those given in ACI 318 are provided to reflect the applicable range of test data on composite members.

(3) ACI 318 provisions for tie reinforcing of noncomposite reinforced concrete compression members should be followed in addition to the provisions specified in Section I2.1a(b).

The limitation of $0.01A_g$ in ACI 318 for the minimum longitudinal reinforcing ratio of reinforced concrete compression members is based upon the phenomenon of stress transfer under service load levels from the concrete to the reinforcement due to creep and shrinkage. It is also intended for resisting incidental bending not captured in the analysis. The inclusion of an encased structural steel section meeting the requirements of Section I2.1a aids in mitigating these effects and consequently allows a reduction in minimum longitudinal reinforcing requirements. See also Commentary Section I2.1a(c).

The design basis for ACI 318 is strength design. Designers using allowable strength design for steel design must be aware of the different load factors between the two specifications.

2. Nominal Strength of Composite Sections

The cross-section strength of composite members is computed based on one of the four methods presented in this section of the Specification. This forms the basis for calculating the nominal axial and flexural strength for composite members which are then used to determine member strength under interaction. The first method is the plastic stress distribution method, which provides a general method for calculating the cross-section strength for composite members with compact cross sections. The second method is the strain compatibility method, which provides an alternative method for calculating the cross-section strength for composite members with compact cross sections. The third approach is the elastic stress distribution method, which has been retained from previous editions of the Specification to allow for the calculation of the strength of composite beams with noncompact webs. The fourth method, added in this edition of the Specification, is the effective stress-strain method, which provides guidance for calculating the cross-section strength (axial force-moment strength interaction) for composite members with noncompact or slender cross sections. The plastic stress distribution method provides a simple and convenient calculation method for the most common design situations, and is thus treated first. Further discussion related to the effects of member slenderness and second-order forces on interaction equations are given in Commentary Section I5.

2a. Plastic Stress Distribution Method

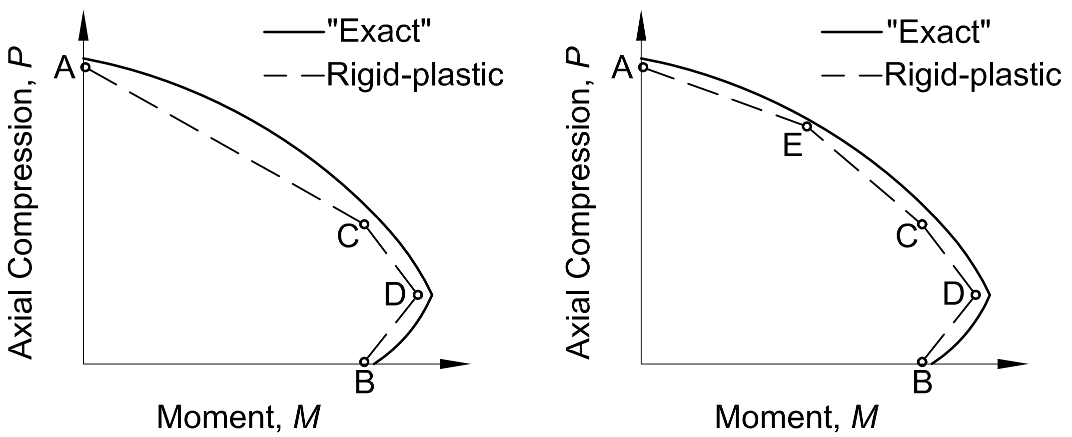
The plastic stress distribution method is based on the plastic limit analysis of the cross section, which is assumed to undergo complete plastification and form a mechanism (plastic hinge). Steel and concrete materials are assumed to have rigid-plastic uniaxial behavior with the steel yield stress equal to F_y in either tension or compression and the concrete compressive stress equal to $0.85f'_c$ (for most cases). The concrete is assumed to have zero tension stress capacity. Force equilibrium is established over the cross section to calculate points for the axial force-plastic moment section strength for the composite cross section. The actual cross-section strength for a composite section based on the plastic stress distribution method is similar to that of a reinforced concrete cross section, as shown in Figure C-I1.1. As a simplification, a conservative linear relation between four or five anchor points can be used (Roik and Bergmann, 1992; Ziemian, 2010). These points are identified as A, B, C, D and

E in Figure C-I1.1. Note that the formulas originally utilized for Point E have since been revised (Denavit et al., 2016b).

The plastic stress distribution method assumes (a) that sufficient strains have developed in the steel and concrete for both to reach their yield strength and (b) that local buckling is delayed until yielding and concrete crushing have taken place, based on the use of a compact section. Tests and analyses have shown that these are reasonable assumptions for both concrete-encased steel sections with steel anchors and for HSS sections that comply with these provisions (Hajjar, 2000; Shanmugam and Lakshmi, 2001; Varma et al., 2002; Leon et al., 2007; Ziemian, 2010). For round HSS, these provisions allow for the increase of the usable concrete stress to $0.95f'_c$ for calculating both axial compressive and flexural strengths to account for the beneficial effects of the restraining hoop action arising from transverse confinement (Leon et al., 2007).

Based on similar assumptions, but allowing for slip between the steel beam and the composite slab, simplified expressions can also be derived for typical composite beam sections. Strictly speaking, these distributions are not based on slip, but on the strength of the shear connection. Full interaction is assumed if the shear connection strength exceeds that of either (a) the tensile yield strength of the steel section or the compressive strength of the concrete slab when the composite beam is loaded in positive moment, or (b) the tensile yield strength of the longitudinal reinforcing bars in the slab or the compressive strength of the steel section when loaded in negative moment.

When steel anchors are provided in sufficient numbers to fully develop this flexural strength, any slip that occurs prior to yielding has a negligible effect on behavior. When full interaction is not present, the beam is said to be partially composite. The effects of slip on the elastic properties of a partially composite beam can be significant and should be accounted for in calculations of deflections and stresses at service loads. Approximate elastic properties of partially composite beams are given in Commentary Section I3.



(a) Strong axis *(b) Weak axis*

Fig. C-I1.1. Comparison between exact and simplified moment-axial compressive force cross-section strength envelopes.

2b. Strain Compatibility Method

The principles used to calculate cross-sectional strength in Section I1.2a may not be applicable to all design situations or possible cross sections. As an alternative, Section I1.2b permits the use of a generalized strain-compatibility approach that allows the use of any reasonable uniaxial stress-strain model for the steel and concrete. This method is focused on ultimate strength and does not contemplate the use of pseudo-material properties to explicitly account for three-dimensional phenomena like local buckling and confinement that may arise in noncompact and slender sections.

2c. Elastic Stress Distribution Method

The use of an elastic stress distribution is recognized for composite beams, encased composite members, and filled composite members for which the plastic stress distribution method is not applicable. Additional discussion for this method can be found in Commentary Sections I3.2a and I3.3.

2d. Effective Stress-Strain Method

This methodology has been added to provide one alternative for calculating the cross-section strength (axial force-moment strength interaction) for composite members, including noncompact and slender cross sections. The effective stress-strain method is applicable when using a fiber-based approach for calculating the cross section axial force-moment strength interaction, while assuming strain compatibility and utilizing modified material stress-strain curves to account implicitly for the effects of steel HSS local buckling, yielding, residual stresses, concrete cracking, concrete crushing, confinement, and any other effects that significantly impact the strength of the cross section (Sakino et al., 2004; Han et al., 2005; Liang, 2009; Lai and Varma, 2016).

3. Material Limitations

The material limitations given in Section I1.3 reflect the range of material properties available from experimental testing (Ziemian, 2010; Hajjar, 2000; Shanmugam and Lakshmi, 2001; Varma et al., 2002; Leon et al., 2007). As for reinforced concrete design, a limit of 10 ksi (70 MPa) is imposed for strength calculations, both to reflect the scant data available above this strength and the changes in behavior observed (Varma et al., 2002). A lower limit of 3 ksi (21 MPa) is specified for both normal and lightweight concrete and an upper limit of 6 ksi (42 MPa) is specified for lightweight concrete to encourage the use of good quality, yet readily available, grades of structural concrete. The use of higher strengths in computing the modulus of elasticity is permitted, and the limits given can be extended for strength calculations if appropriate testing and analyses are carried out. The specified minimum yield stress of reinforcing bars has been increased to 80 ksi (550 MPa) in coordination with ACI 318 (2014).

4. Classification of Filled Composite Sections for Local Buckling

The behavior of filled composite members is fundamentally different from the behavior of hollow steel members. The concrete infill has a significant influence on the stiffness, strength and ductility of composite members. As the steel section area decreases, the concrete contribution becomes even more significant.

The elastic local buckling of the steel HSS is influenced significantly by the presence of the concrete infill. The concrete infill changes the buckling mode of the steel HSS (both within the cross section and along the length of the member) by preventing it from deforming inwards as shown in Figures C-I1.2 and C-I1.3. Bradford et al. (1998) analyzed the elastic local buckling behavior of filled composite compression members, showing that for rectangular steel HSS, the plate buckling coefficient, *k*-factor, in the elastic plate buckling equation (Ziemian, 2010) changes from 4.00 for hollow tubes to 10.6 for filled sections. As a result, the elastic plate buckling stress increases by a factor of 2.65 for filled sections as compared to hollow structural sections. Similarly, Bradford et al. (2002) showed that the elastic local buckling stress for filled round sections is 1.73 times that for hollow round sections.

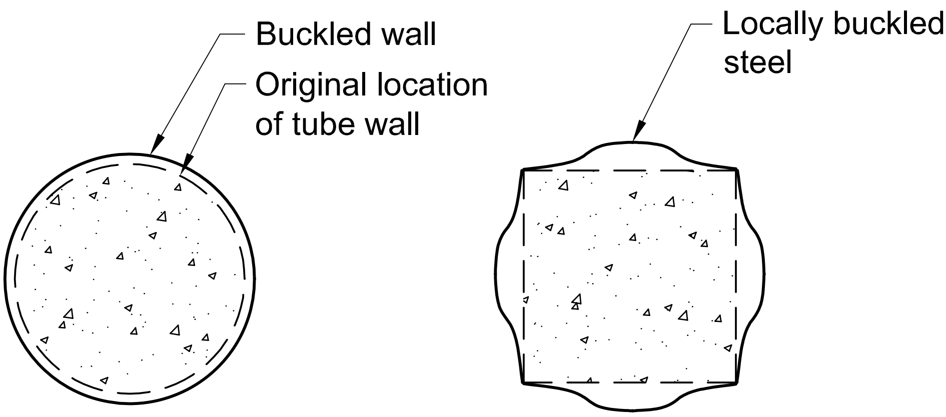


Fig. C-I1.2. Cross-sectional buckling mode with concrete infill.

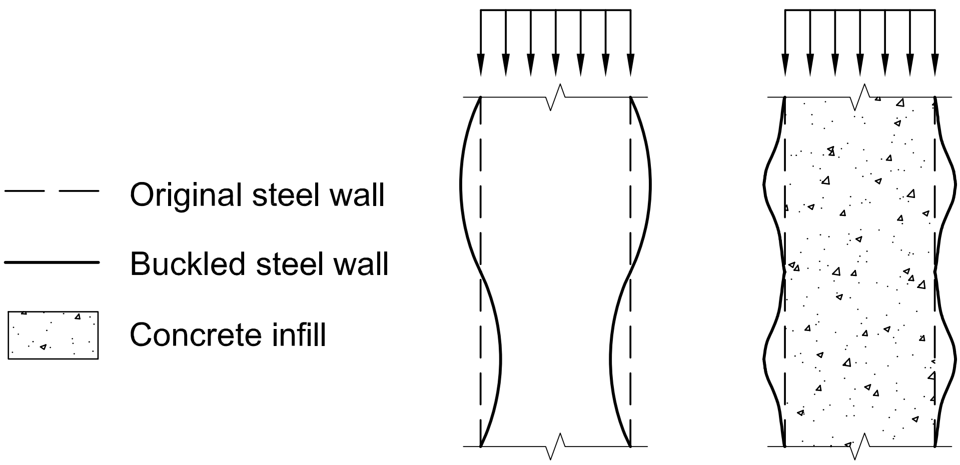


Fig. C-I1.3. Changes in buckling mode with length due to the presence of infill.

For rectangular filled sections, the elastic local buckling stress, F_{cr}, from the plate buckling equation simplifies to Equation I2-10. This equation indicates that yielding will occur for plates with b/t less than or equal to $3.00\sqrt{E/F_y}$, which designates the limit between noncompact and slender sections, λ_r. This limit does not account for the effects of residual stresses or geometric imperfections because the concrete contribution governs for these larger b/t ratios and the effects of reducing steel stresses is small. The maximum permitted b/t value is based on the lack of experimental data above the limit of $5.00\sqrt{E/F_y}$, and the potential effects (plate deflections and locked-in stresses) of concrete placement in extremely slender filled HSS cross sections. For flexure, the b/t limits for the flanges are the same as those for walls in axial compression due to the similarities in loading and behavior. The compact/noncompact limit, λ_p, for webs in flexure was established conservatively as $3.00\sqrt{E/F_y}$. The noncompact/slender limit, λ_r, for the web was established conservatively as $5.70\sqrt{E/F_y}$, which is also the maximum permitted for hollow structural sections. This limit was also established as the maximum permitted value due to the lack of experimental data and concrete placement concerns for thinner filled HSS cross sections (Lai et al., 2014).

For round filled sections in axial compression, the noncompact/slender limit, λ_r, was established as $0.19E/F_y$, which is 1.73 times the limit ($0.11E/F_y$) for hollow round sections. This was based on the findings of Bradford et al. (2002) and it compares well with experimental data. The maximum permitted D/t equal to $0.31E/F_y$ is based on the lack of experimental data and the potential effects of concrete placement in extremely slender filled HSS cross sections. For round filled sections in flexure, the compact/noncompact limit, λ_p, in Table I1.1b was developed conservatively as 1.25 times the limit $0.07E/F_y$ for round hollow structural sections. The noncompact/slender limit, λ_r, was assumed conservatively to be the same as for round hollow structural sections, $0.31E/F_y$. This limit was also established as the maximum permitted value due to lack of experimental data and concrete placement concerns for thinner filled HSS cross sections (Lai and Varma, 2015).

5. Stiffness for Calculation of Required Strengths

This section along with Chapter C forms the basis of the direct analysis method of design for structural systems including encased composite members or filled composite members. The method is identical to the method for bare steel with the exception of the adjustments to stiffness prescribed for the analysis to determine required strengths. The reasoning for the reduced stiffness, $EI^* = 0.8\tau_b EI_{eff}$, where EI_{eff} is as calculated in Section I2, and $EA^* = 0.8(E_sA_s + E_sA_{sr} + E_cA_c)$, mirrors that of bare steel. First, for frames with slender members, where the limit state is governed by elastic stability, the factor of $0.8\tau_b$ (= 0.64) on the effective flexural stiffness results in a system available strength equal to 0.64 times the elastic stability limit. This is roughly equivalent to the margin of safety implied in the design provisions for slender columns by the effective length procedure, where from Equation I2-3, $\phi P_n = 0.75(0.877P_e) = 0.66P_e$. Second, for frames with intermediate or stocky columns, the $0.8\tau_b$ factor reduces the stiffness to account for inelastic softening (e.g., concrete cracking and steel partial yielding) prior to the members reaching their design strength.

Unlike for bare steel, the τ_b factor is a constant value and does not vary with required axial compressive strength. As a consequence, the use of $\tau_b = 1.0$ by applying additional notional load such as described in Section C2.3(3) is inaccurate and not permitted. For the case of a structure containing both composite members and highly loaded ($\alpha P_r > 0.5P_y$) bare steel members, a conservative approach to avoid a variable stiffness in the analysis would be to apply the additional notional load so that $\tau_b = 1.0$ can be used for the bare steel members and maintain $\tau_b = 0.8$ for the composite members.

Research indicates that the stiffness prescribed in this section may result in unconservative errors for very stability sensitive structures (Denavit et al., 2016a).

The Specification has traditionally not accounted for long-term effects due to creep and shrinkage; as such, the stiffness prescribed in this section was developed based on studies examining only short-term behavior. Refer to Commentary Sections I1 and I3.2 for additional discussion.

The use of reduced stiffness only pertains to analyses for strength and stability limit states. It does not apply to analyses for other stiffness-based conditions and criteria, such as for drift, deflection, vibration and period determination. The effective stiffness, EI_{eff}, has been found to provide a reasonable value for use in determining drifts (Denavit and Hajjar, 2014).

This section does not apply to the effective length method. It is recommended that when using the effective length method with composite compression members that either (a) the nominal stiffness be taken as the effective stiffness ($EI = EI_{eff}$) and the interaction strength of Section H1.1 be used, or (b) the nominal stiffness be taken as 0.8 times the effective stiffness ($EI = 0.8EI_{eff}$) and the interaction strength be determined using one of the methods described in Commentary Section I5.

I2. AXIAL FORCE

The design of encased and filled composite members is treated separately, although they have much in common. The intent is to facilitate design by keeping the general principles and detailing requirements for each type of compression member separate.

An ultimate strength cross-section model is used to determine the section strength (Leon et al., 2007; Leon and Hajjar, 2008). This model is similar to that used in previous LRFD *Specifications*. The design equations in Section I2 for computing compressive axial strength including length effects apply only to doubly symmetric sections. For singly symmetric and unsymmetric sections, only the strain compatibility approach utilizing reasonable limitations on strains (e.g., 0.003 for concrete and 0.02 for steel) is applicable for determining cross-sectional strength. As for steel-only columns, more advanced methods are necessary to design singly symmetric and unsymmetric columns to include length effects. Generalized approaches, such as those in Chapters E and F for steel-only columns, are not yet available for composite columns as the variety of sections possible does not lend itself to simplifications.

The design for length effects is consistent with that for steel compression members. The equations used are the same as those in Chapter E modified for use in composite design. As the percentage of concrete in the section decreases, the design defaults to that of a steel section, although with different resistance and safety factors. The equations for EI_{eff} were updated in this Specification following a reevaluation of the experimental data and an analytical investigation. The changes represent a significant increase in strength for some encased composite members and a moderate decrease in strength for some filled composite members (Denavit et al., 2016a). Comparisons between the provisions in the Specification and experimental data show that the method is generally accurate; however, the coefficient of variation resulting from the application of the strength prediction model is significant given the relatively large statistical scatter associated with the experimental data (Leon et al., 2007).

1. Encased Composite Members

1a. Limitations

(a) Encased composite compression members must have a minimum area of steel core such that the steel core area divided by the gross area of the member is equal to or greater than 1%.

(b) The requirements for transverse reinforcement are intended to provide good confinement to the concrete. According to Section I1.1(c), the transverse tie provisions of ACI 318 are to be followed, in addition to the limits provided.

(c) A minimum amount of longitudinal reinforcing steel is prescribed to ensure that unreinforced concrete encasements are not designed with these provisions. Continuous longitudinal bars should be placed at each corner of the cross section. Other longitudinal bars may be needed to provide the required restraint to the cross-ties, but that longitudinal steel cannot be counted towards the minimum area of longitudinal reinforcing nor the cross-sectional strength unless it is continuous and properly anchored.

1b. Compressive Strength

The compressive strength of the cross section, P_{no}, is given as the sum of the ultimate strengths of the components. The nominal strength, P_n, is not capped as in reinforced concrete compression member design for a combination of the following reasons: (a) the resistance factor is 0.75; (b) the required transverse steel provides better performance than a typical reinforced concrete compression member; (c) the presence of a steel section near the center of the section reduces the possibility of a sudden failure due to buckling of the longitudinal reinforcing steel; and (d) there will typically be moment present due to the manner in which stability is addressed in the Specification.

1c. Tensile Strength

This section clarifies the tensile strength to be used in situations where uplift is a concern and for computations related to beam-column interaction. The provision focuses on the limit state of yielding of the gross area. Where appropriate for the structural configuration, consideration should also be given to other tensile strength and connection strength limit states as specified in Chapters D and J.

2. Filled Composite Members

2a. Limitations

(a) As discussed for encased compression members, it is permissible to design filled composite compression members with a steel ratio as low as 1%.

(b) Filled composite sections are classified as compact, noncompact or slender depending on the hollow structural section (HSS) slenderness, b/t or D/t, and the limits in Table I1.1a.

(c) Walls of rectangular filled sections may be susceptible to deformations during casting if a large hydrostatic pressure is exerted. These deformations will affect the location and initiation of local buckling. To control these deformations, the following serviceability limits are suggested by Leon et al. (2011):

$$\sigma_{max} = \max \begin{bmatrix} \left(\dfrac{2h_c}{b_c + 4h_c} \right) \dfrac{ph_c^2}{t^2} \\ \dfrac{1}{3} \left(\dfrac{3b_c + 4h_c}{b_c + 4h_c} \right) \dfrac{ph_c^2}{t^2} \end{bmatrix} \le 0.5F_y \qquad \text{(C-I2-1)}$$

$$\delta_{max} = \frac{1}{32} \left(\frac{5b_c + 4h_c}{b_c + 4h_c} \right) \frac{ph_c^4}{E_s t^3} \le \frac{L}{2{,}000} \qquad \text{(C-I2-2)}$$

where, h_c and b_c are, respectively, the longer and the shorter inner widths of the rectangular cross section ($h_c = h - 2t$; $b_c = b - 2t$), t is the wall thickness, b and h are the overall outside dimensions, L is the pressure length, and p is the hydrostatic pressure. If either the corresponding stresses or deformations in rectangular filled composite cross sections exceed the limits given in Equations C-I2-1 or C-I2-2, it is recommended that external supports be added during casting.

2b. Compressive Strength

A compact hollow structural section (HSS) has sufficient thickness to develop yielding of the steel HSS in longitudinal compression, and to provide confinement to the concrete infill to develop its compressive strength (0.85 or $0.95f'_c$). A noncompact section has sufficient HSS thickness to develop yielding of the HSS in the longitudinal direction, but it cannot adequately confine the concrete infill after it reaches $0.70f'_c$ compressive stress in the concrete and starts undergoing significant inelasticity and volumetric dilation, thus pushing against the steel HSS. A slender section can neither develop yielding of the steel HSS in the longitudinal direction, nor confine the concrete after it reaches $0.70f'_c$ compressive stress in the concrete and starts undergoing inelastic strains and significant volumetric dilation pushing against the HSS (Lai et al., 2014; Lai and Varma, 2015).

Figure C-I2.1 shows the variation of the nominal axial compressive strength, P_{no}, of the composite section with respect to the HSS wall slenderness. As shown, compact sections can develop the full plastic strength, P_p, in compression. The nominal axial strength, P_{no}, of noncompact sections can be determined using a quadratic interpolation between the plastic strength, P_p, and the yield strength, P_y, with respect to the

HSS slenderness. This interpolation is quadratic because the ability of the HSS to confine the concrete infill undergoing inelasticity and volumetric dilation decreases rapidly with HSS wall slenderness. Slender sections are limited to developing the critical buckling stress, F_{cr}, of the steel HSS and $0.70f'_c$ of the concrete infill (Lai et al., 2014; Lai and Varma, 2015).

The nominal axial strength, P_n, of composite compression members, including length effects, may be determined using Equations I2-2 and I2-3, while using EI_{eff} (from Equation I2-12) to account for composite section rigidity and P_{no} to account for the effects of local buckling as described in the preceding. This approach is slightly different than the one used for HSS found in Section E7. This approach was not implemented for filled compression members because (a) their axial strength is governed significantly by the contribution of the concrete infill, (b) concrete inelasticity occurs within the compression member failure segment irrespective of the buckling load, and (c) the calculated nominal strengths compare conservatively with experimental results (Lai et al., 2014; Lai and Varma, 2015).

2c. Tensile Strength

As for encased compression members, this section specifies the tensile strength for filled composite members. Similarly, while the provision focuses on the limit state of yielding of the gross area, where appropriate, consideration should also be given to other tensile strength and connection strength limit states as specified in Chapters D and J.

I3. FLEXURE

1. General

Three types of composite flexural members are addressed in this section: fully encased steel beams, filled HSS, and steel beams with mechanical anchorage to a concrete slab which are generally referred to as composite beams.

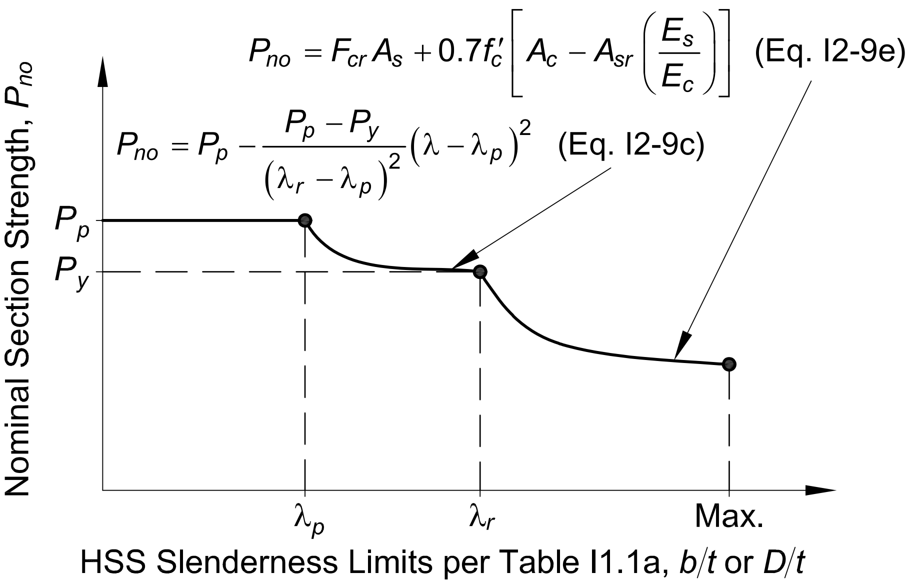


Fig. C-I2.1. Nominal axial strength, P_{no}, *versus HSS wall slenderness.*

1a. Effective Width

The same effective width rules apply to composite beams with a slab on either one side or both sides of the beam. In cases where the effective stiffness of a beam with a one-sided slab is important, special care should be exercised because this model can substantially overestimate stiffness (Brosnan and Uang, 1995). To simplify design, the effective width is based on the full span, center-to-center of supports, for both simple and continuous beams.

1b. Strength During Construction

Composite beam design requires care in considering the loading history. Loads applied to an unshored beam before the concrete has cured are resisted by the steel section alone; total loads applied before and after the concrete has cured are considered to be resisted by the composite section. It is usually assumed for design purposes that concrete has hardened when it attains 75% of its design strength. Unshored beam deflection caused by fresh concrete tends to increase slab thickness and dead load. For longer spans this may lead to instability analogous to roof ponding. Excessive increase of slab thickness may be avoided by beam camber. Pouring the slab to a constant thickness will also help eliminate the possibility of ponding instability (Ruddy, 1986). When forms are not attached to the top flange, lateral bracing of the steel beam during construction may not be continuous and the unbraced length may control flexural strength, as defined in Chapter F.

This Specification does not include special requirements for strength during construction. For these noncomposite beams, the provisions of Chapter F apply.

Load combinations for construction loads should be determined for individual projects considering the project-specific circumstances, using ASCE/SEI 37-14 (ASCE, 2014) as a guide.

2. Composite Beams with Steel Headed Stud or Steel Channel Anchors

This section applies to simple and continuous composite beams with steel anchors, constructed with or without temporary shores.

When a composite beam is controlled by deflection, the design should limit the behavior of the beam to the elastic range under serviceability load combinations. Alternatively, the amplification effects of inelastic behavior should be considered when deflection is checked.

Accurate prediction of flexural stiffness for composite beam members is difficult to achieve, and an examination of previous studies (Leon, 1990; Leon and Alsamsam, 1993) indicates a wide variation between predicted and experimental deflections. More recent studies indicate that the use of the equivalent moment of inertia, I_{equiv}, for deflection calculations results in a prediction of short-term deflections roughly equivalent to the statistical average of the experimental tests reviewed (Zhao and Leon, 2013). Previous editions of the Specification recommended an additional reduction factor of 0.75 be applied to I_{equiv} to form an effective moment of inertia; however, this approach has been removed as its basis could not be substantiated. An alternate approach is the lower bound moment of inertia, I_{LB}, which is, as the name implies, a lower bound approach that provides a conservative estimate of short-term deflections; values obtained by the I_{LB} approach correspond roughly to the mean plus one standard deviation (84%) based on the 120 tests examined (Zhao and Leon, 2013).

The lower bound moment of inertia, I_{LB}, is defined as

$$I_{LB} = I_s + A_s(Y_{ENA} - d_3)^2 + (\Sigma Q_n / F_y)(2d_3 + d_1 - Y_{ENA})^2 \quad \text{(C-I3-1)}$$

where

A_s = area of steel cross section, in.^{2} (mm^2)

d_1 = distance from the compression force in the concrete to the top of the steel section, in. (mm)

d_3 = distance from the resultant steel tension force for full section tension yield to the top of the steel, in. (mm)

I_{LB} = lower bound moment of inertia, in.^{4} (mm^4)

I_s = moment of inertia for the structural steel section, in.^{4} (mm^4)

ΣQ_n = sum of the nominal strengths of steel anchors between the point of maximum positive moment and the point of zero moment to either side, kips (kN)

$Y_{ENA} = \left[A_s d_3 + (\Sigma Q_n / F_y)(2d_3 + d_1)\right] / \left[A_s + (\Sigma Q_n / F_y)\right]$, in. (mm) (C-I3-2)

The equivalent moment of inertia, I_{equiv}, is defined as

$$I_{equiv} = I_s + \sqrt{\left(\Sigma Q_n / C_f\right)}\left(I_{tr} - I_s\right) \quad \text{(C-I3-3)}$$

where

C_f = compression force in concrete slab for fully composite beam; smaller of $A_s F_y$ and $0.85 f'_c A_c$, kips (N)

A_c = area of concrete slab within the effective width, in.^{2} (mm^2)

I_{tr} = moment of inertia for the fully composite uncracked transformed section, in.^{4} (mm^4)

The effective section modulus, S_{eff}, referred to the tension flange of the steel section for a partially composite beam, may be approximated by

$$S_{eff} = S_s + \sqrt{\left(\Sigma Q_n / C_f\right)}\left(S_{tr} - S_s\right) \quad \text{(C-I3-4)}$$

where

S_s = section modulus for the structural steel section, referred to the tension flange, in.^{3} (mm^3)

S_{tr} = section modulus for the fully composite uncracked transformed section, referred to the tension flange of the steel section, in.^{3} (mm^3)

Equations C-I3-3 and C-I3-4 should not be used for ratios, $\Sigma Q_n / C_f$, less than 0.25. This restriction is to prevent excessive slip and the resulting substantial loss in beam stiffness. Studies indicate that Equations C-I3-3 and C-I3-4 adequately reflect the reduction in beam stiffness and strength, respectively, when fewer anchors are used than required for full composite action (Grant et al., 1977).

The use of a constant stiffness in elastic analyses of continuous beams is analogous to the practice in reinforced concrete design. The stiffness calculated using a weighted average of moments of inertia in the positive moment region and negative moment regions may take the following form:

$$I_t = aI_{pos} + bI_{neg} \tag{C-I3-5}$$

where

I_{pos} = effective moment of inertia for positive moment, in.^{4} (mm^4)
I_{neg} = effective moment of inertia for negative moment, in.^{4} (mm^4)

For continuous beams subjected to gravity loads only, the value of a may be taken as 0.6 and the value of b may be taken as 0.4. For composite beams used as part of a lateral force-resisting system in moment frames, the value of a and b may be taken as 0.5 for calculations related to drift.

U.S. practice does not generally require the following items to be considered. These items are highlighted here for designers evaluating atypical conditions for which they might apply.

(a) Horizontal shear strength of the slab: For the case of girders with decks with narrow troughs or thin slabs, shear strength of the slab may govern the design (for example, see Figure C-I3.1). Although the configuration of decks built in the U.S. tends to preclude this mode of failure, it is important that it be checked if the force in the slab is large or an unconventional assembly is chosen. The shear strength of the slab may be calculated as the superposition of the shear strength of the concrete plus the contribution of any slab steel crossing the shear plane. The required shear strength, as shown in the figure, is given by the difference in the force between the regions inside and outside the potential failure surface. Where experience has shown that longitudinal cracking detrimental to serviceability is likely to occur, the slab should be reinforced in the direction transverse to the supporting steel section. It is recommended that the area of such reinforcement be at least 0.002 times the concrete area in the longitudinal direction of the beam and that it be uniformly distributed.

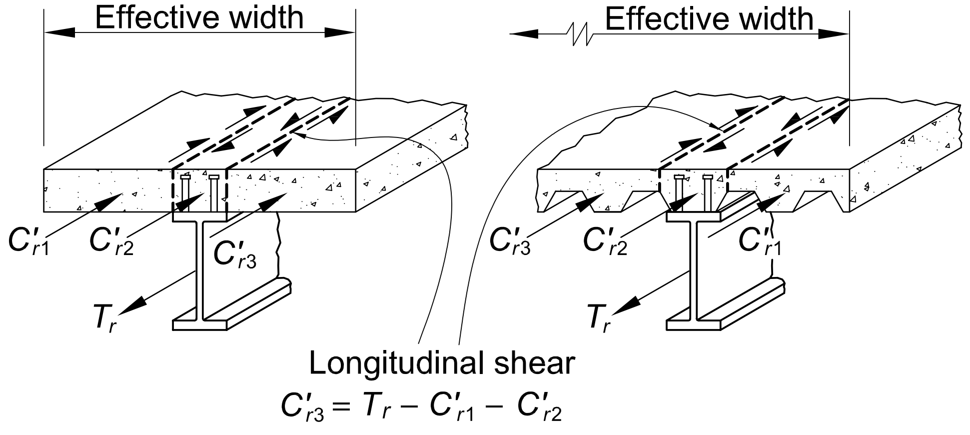


Fig. C-I3.1. Longitudinal shear in the slab [after Chien and Ritchie (1984)].

(b) Rotational capacity of hinging zones: There is no required rotational capacity for hinging zones. Where plastic redistribution to collapse is allowed, the moments at a cross section may be as much as 30% lower than those given by a corresponding elastic analysis. This reduction in load effects is predicated, however, on the ability of the system to deform through very large rotations. To achieve these rotations, very strict local buckling and lateral-torsional buckling requirements must be fulfilled (Dekker et al., 1995). For cases in which a 10% redistribution is utilized, as permitted in Section B3.3, the required rotation capacity is within the limits provided by the local and lateral-torsional buckling provisions of Chapter F. Therefore, a rotational capacity check is not normally required for designs using this provision.

(c) Long-term deformations due to shrinkage and creep: There is no direct guidance in the computation of the long-term deformations of composite beams due to creep and shrinkage. The long-term deformation due to shrinkage can be calculated with the simplified model shown in Figure C-I3.2, in which the effect of shrinkage is taken as an equivalent set of end moments given by the shrinkage force (long-term restrained shrinkage strain times modulus of concrete times effective area of concrete) times the eccentricity between the center of the slab and the elastic neutral axis. If the restrained shrinkage coefficient for the aggregates is not known, the shrinkage strain, e_{sh}, for these calculations may be taken as 0.02%. The long-term deformations due to creep, which can be quantified using a model similar to that shown in the figure, are small unless the spans are long and the permanent live loads large. For shrinkage and creep effects, special attention should be given to lightweight aggregates, which tend to have higher

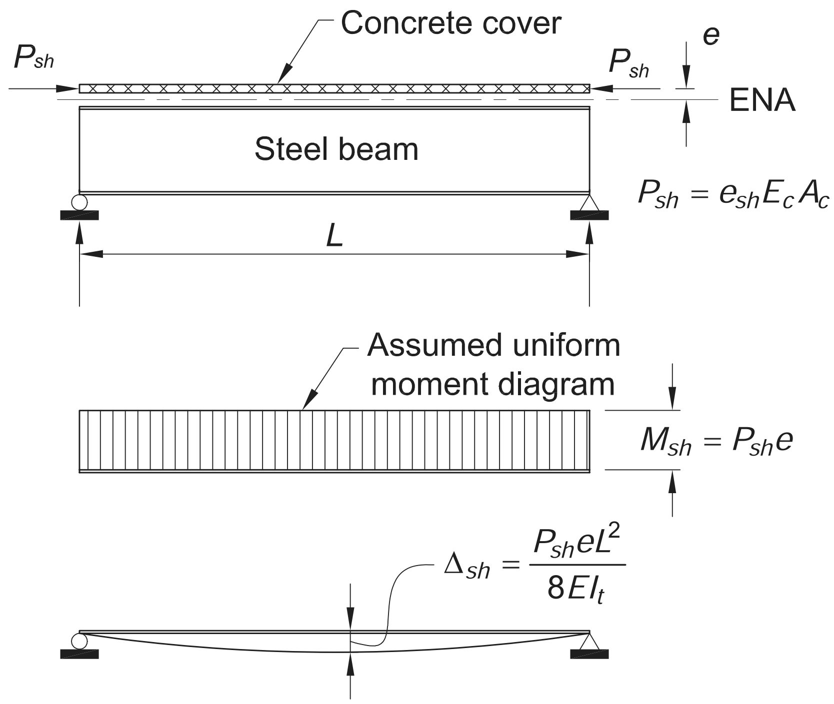


Fig. C-I3.2. Calculation of shrinkage effects [from Chien and Ritchie (1984)].

creep coefficients and moisture absorption and lower modulus of elasticity than conventional aggregates, exacerbating any potential deflection problems. Engineering judgment is required, as calculations for long-term deformations require consideration of the many variables involved and because linear superposition of these effects is not strictly correct (ACI, 1997; Viest et al., 1997).

2a. Positive Flexural Strength

The flexural strength of a composite beam in the positive moment region may be controlled by the strength of the steel section, the concrete slab, or the steel headed stud anchors. In addition, web buckling may limit flexural strength if the web is slender and a sufficient portion of the web is in compression.

Plastic Stress Distribution for Positive Moment. When flexural strength is determined from the plastic stress distribution shown in Figure C-I3.3, the compression force, C, in the concrete slab is the smallest of:

$$C = A_s F_y \tag{C-I3-6}$$

$$C = 0.85 f_c' A_c \tag{C-I3-7}$$

$$C = \Sigma Q_n \tag{C-I3-8}$$

where

A_c = area of concrete slab within effective width, in.^{2} (mm^2)
A_s = area of steel cross section, in.^{2} (mm^2)
F_y = specified minimum yield stress of steel, ksi (MPa)
ΣQ_n = sum of nominal strengths of steel headed stud anchors between the point of maximum positive moment and the point of zero moment to either side, kips (N)
f_c' = specified compressive strength of concrete, ksi (MPa)

Longitudinal slab reinforcement makes a negligible contribution to the compression force, except when Equation C-I3-7 governs. In this case, the area of longitudinal

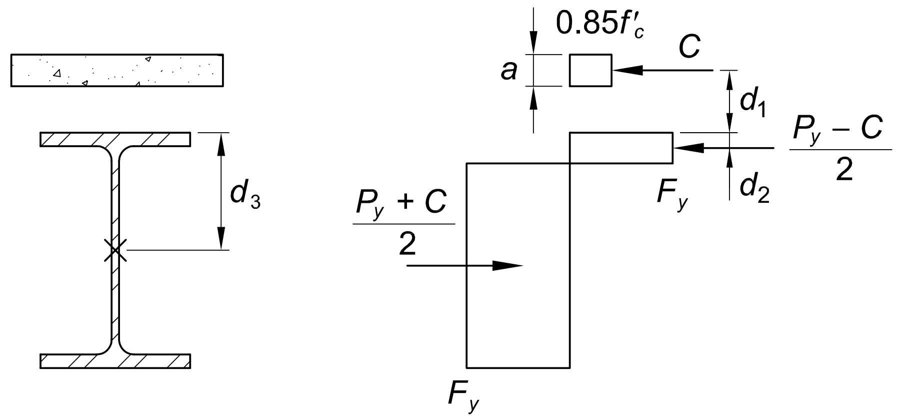


Fig. C-I3.3. Plastic stress distribution for positive moment in composite beams.

reinforcement within the effective width of the concrete slab times the yield stress of the reinforcement may be added in determining C.

The depth of the compression block is:

$$a = \frac{C}{0.85 f_c' b} \tag{C-I3-9}$$

where

b = effective width of concrete slab, in. (mm)

A fully composite beam corresponds to the case where C is governed by either Equation C-I3-6 or C-I3-7. If C is governed by Equation C-I3-8, the beam is partially composite.

The plastic stress distribution may have the plastic neutral axis, PNA, in the web, in the top flange of the steel section, or in the slab, depending on the governing C.

Using Figure C-I3.3, the nominal plastic moment strength of a composite beam in positive bending is given by:

$$M_n = C(d_1 + d_2) + P_y(d_3 - d_2) \tag{C-I3-10}$$

where

P_y = tensile strength of the steel section; $P_y = F_y A_s$, kips (N)

d_1 = distance from the centroid of the compression force, C, in the concrete to the top of the steel section, in. (mm)

d_2 = distance from the centroid of the compression force in the steel section to the top of the steel section, in. (mm). For the case of no compression in the steel section, $d_2 = 0$.

d_3 = distance from P_y to the top of the steel section, in. (mm)

Equation C-I3-10 is applicable for steel sections symmetrical about one or two axes.

According to Table B4.1b, Case 15, local web buckling does not reduce the plastic strength of a bare steel beam if the width-to-thickness ratio of the web is not larger than $3.76\sqrt{E/F_y}$. In the absence of web buckling research on composite beams, the same ratio is conservatively applied to composite beams. All current ASTM A6 W-shapes have compact webs for $F_y \le 70$ ksi (485 MPa).

Elastic Stress Distribution. For beams with more slender webs, this Specification conservatively adopts first yield as the flexural strength limit using the elastic stress distribution method. In this case, stresses on the steel section from permanent loads applied to unshored beams before the concrete has cured must be superimposed on stresses on the composite section from loads applied to the beams after hardening of concrete. For shored beams, all loads may be assumed to be resisted by the composite section.

When first yield is the flexural strength limit, the elastic transformed section is used to calculate stresses on the composite section. The modular ratio, $n = E_s / E_c$, used to determine the transformed section, depends on the specified unit weight and strength of concrete.

2b. Negative Flexural Strength

Plastic Stress Distribution for Negative Moment. When an adequately braced compact steel section and adequately developed longitudinal reinforcing bars act compositely in the negative moment region, the nominal flexural strength is determined from the plastic stress distribution, as shown in Figure C-I3.4. Loads applied to a continuous composite beam with steel anchors throughout its length, after the slab is cracked in the negative moment region, are resisted in that region by the steel section and by properly anchored longitudinal slab reinforcement.

The tensile force, T, in the reinforcing bars is the smaller of:

$$T = F_{yr} A_r \tag{C-I3-11}$$

$$T = \Sigma Q_n \tag{C-I3-12}$$

where

A_r = area of properly developed slab reinforcement parallel to the steel beam and within the effective width of the slab, in.^{2} (mm^2)

F_{yr} = specified minimum yield stress of the slab reinforcement, ksi (MPa)

ΣQ_n = sum of the nominal strengths of steel headed stud anchors between the point of maximum negative moment and the point of zero moment to either side, kips (N)

A third theoretical limit on T is the product of the area and yield stress of the steel section; however, this limit is redundant in view of practical limitations for slab reinforcement.

Using Figure C-I3.4, the nominal plastic moment strength of a composite beam in negative bending is given by:

$$M_n = T(d_1 + d_2) + P_{yc}(d_3 - d_2) \tag{C-I3-13}$$

where

P_{yc} = compressive strength of the steel section; $P_{yc} = A_s F_y$, kips (N)

d_1 = distance from the centroid of the longitudinal slab reinforcement to the top of the steel section, in. (mm)

d_2 = distance from the centroid of the tension force in the steel section to the top of the steel section, in. (mm)

d_3 = distance from P_{yc} to the top of the steel section, in. (mm)

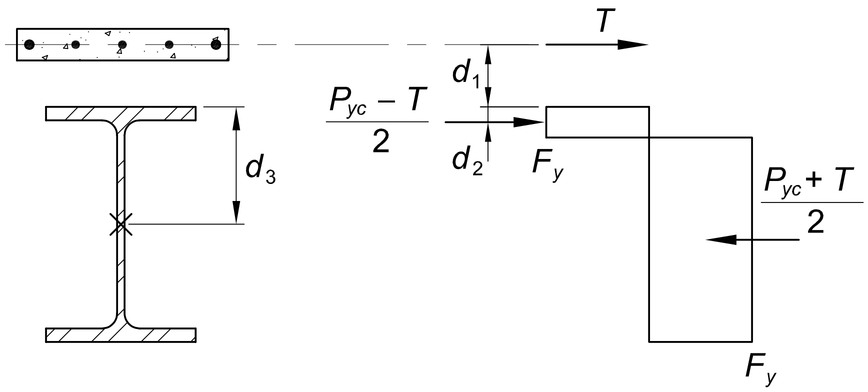


Fig. C-I3.4. Plastic stress distribution for negative moment.

2c. Composite Beams with Formed Steel Deck

Figure C-I3.5 is a graphic presentation of the terminology used in Section I3.2c.

The design rules for composite construction with formed steel deck are based upon a study (Grant et al., 1977) of the then-available test results. The limiting parameters

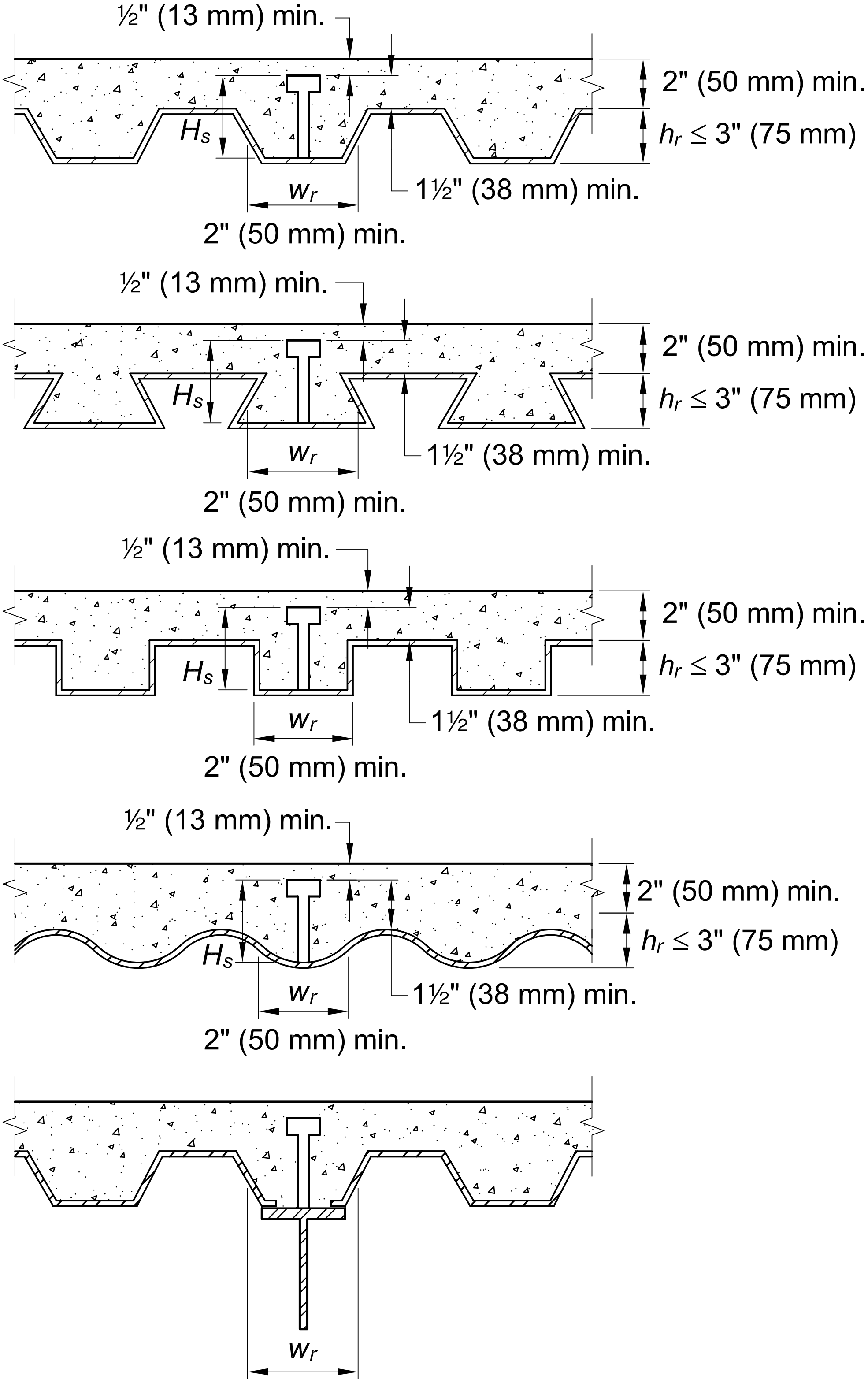


Fig. C-I3.5. Steel deck limits.

listed in Section I3.2c were established to keep composite construction with formed steel deck within the available research data.

The Specification requires steel headed stud anchors to project a minimum of $1^1/_2$ in. (38 mm) above the deck flutes. This is intended to be the minimum in-place projection, and stud lengths prior to installation should account for any shortening of the stud that could occur during the welding process. The minimum specified cover over a steel headed stud anchor of $^1/_2$ in. (13 mm) after installation is intended to prevent the anchor from being exposed after construction is complete. In achieving this requirement, the designer should carefully consider tolerances on steel beam camber, concrete placement and finishing tolerances, and the accuracy with which steel beam deflections can be calculated. In order to minimize the possibility of exposed anchors in the final construction, the designer should consider increasing the bare steel beam size to reduce or eliminate camber requirements (this also improves floor vibration performance), checking beam camber tolerances in the fabrication shop, and monitoring concrete placement operations in the field. Wherever possible, the designer should also consider providing for anchor cover requirements above the $^1/_2$ in. (13 mm) minimum by increasing the slab thickness while maintaining the $1^1/_2$ in. (38 mm) requirement for anchor projection above the top of the steel deck as required by the Specification.

The maximum spacing of 18 in. (450 mm) for connecting composite decking to the support is intended to address a minimum uplift requirement during the construction phase prior to placing concrete (SDI, 2001).

2d. Load Transfer between Steel Beam and Concrete Slab

1. Load Transfer for Positive Flexural Strength

Shear connection at the interface of a concrete slab and supporting steel members is an assembly consisting of the connector, typically a steel headed stud anchor, its weld to the steel member, and the surrounding concrete with a specific deck flute geometry. Shear connection deforms when subjected to shear at the interface. Its ability to deform without fracturing is known as slip capacity or ductility of the shear connection. It is important to note that the term ductility does not merely relate to the ductility of the connector itself, but to the ductility of the overall shear connection assembly. While the slip capacity of the shear connection consisting of a $^3/_4$ in. (19 mm) diameter steel headed stud anchor embedded in a solid slab is about $^1/_4$ in. (6 mm) (Oehlers and Coughlan, 1986), the shear connection with the same connector embedded in a slender concrete slab rib will possess only a fraction of this slip capacity (Lyons et al., 1994; Roddenberry et al., 2002a). Similarly, these same sources show that shear connection ductility can be significantly larger for some configurations.

Flexural strength of a composite section based on a plastic stress distribution is the most typical manner for establishing the member strength. It assumes sufficiently ductile steel and concrete components capable of developing a fully plastic stress block across the depth of the composite section. This analysis also assumes a sufficiently ductile shear connection, allowing for the shear at the interface to be evenly shared among the connectors located between the points of zero and maximum moment. Reliability studies evaluating the computational models for the flexural strength of composite beams (Galambos and Ravindra, 1978; Roddenberry et al., 2002a; Mujagic and Easterling, 2009) are based on the plastic stress distribution methodology. An implicit assumption of the theory is that the shear demands at the interface can be uniformly distributed over the shear span because the connectors are ductile and can redistribute the demands (Viest et al., 1997). Even when shear connections possess adequate ductility to accommodate interfacial slip, excessive slip demand at the interface will cause excessive discontinuities in the strain diagram at the interface of the concrete slab and cause early departure from the elastic behavior, and as a consequence, invalidate the design approach. It is therefore important to limit the shear connection ductility demand at the interface.

The determination of flexural strength based on the plastic stress distribution method without any specific slip capacity checks was reasonable until the mid-1980s, given that low amounts of interaction were uncommon in design and that most spans were relatively short. Today, the design for composite beams often involves much longer spans that are governed by serviceability criteria and therefore require less composite action to achieve their required strength. The simultaneous use of lower levels of composite action and longer spans results in additional deformation demands on the shear studs. Mujagic et al. (2015) and Selden et al. (2015) indicate that long beams designed at low levels of partial interaction may not reach their nominal strength due to lack of connector deformation capacity.

The consideration of ductility demand at the interface of composite beams can come in the form of a number of different approaches of varying degrees of complexity. These approaches generally fall into two groups. First, the effect of shear at the interface can be taken into account directly in the determination of member strength through modeling of the interface slip. The complexity of such an analysis varies greatly based upon whether all components of the composite beams are idealized as linearly elastic (Newmark, et al., 1951; Robinson and Naraine, 1988; Viest et al., 1997), or considered using a nonlinear analysis by capturing inelastic behavior of a partially yielded section and nonlinear behavior of the shear connection along the span (Salari et al., 1998; Salari and Spacone, 2001; Zona and Ranzi, 2014). Second, various indirect analytical models have been proposed. Such models aim to provide convenient computational models suitable for routine design use by either idealizing various components of the composite beam as fully elastic or fully plastic and capturing most dominant elements driving the shear connection ductility demand (Oehlers and Sved, 1995) or by parametrically relating the results of rigorous nonlinear finite element analyses to the most critical design properties affecting shear connection ductility through simple algebraic relationships (Johnson and Molenstra, 1991).

Configurations of composite beams as designed in routine practice depend on strength and serviceability requirements, detailing rules, construction sequence, framing details, fabrication logistics, fire rating requirements, as well as various other considerations related to standard practice. Many of these elements will directly or indirectly affect the ductility performance of the shear connection, and the effect of some is difficult to quantify. Based on the available studies (Mujagic et al., 2015; Selden et al., 2015), beams are not susceptible to connector failure due to insufficient deformation capacity, and thus, need not be checked for this limit state if they meet one or more of the following conditions:

(1) Beams with span not exceeding 30 ft (9.1 m);

(2) Beams with a degree of composite action of at least 50%; or

(3) Beams with an average nominal shear connector capacity of at least 16 kips per ft (233 kN per m) along their shear span, corresponding to a 3/4-in. (19 mm) steel headed stud anchor placed at 12-in. (300 mm) spacing on average.

Beams that do not meet the foregoing criteria may still be acceptable, and can be evaluated through direct nonlinear modeling of the member capturing all sources of deformation. Such modeling should be performed under factored loads using strength and stiffness properties of the member. The analysis should meet the pertinent requirements of Appendix 1. Furthermore, the analytical model should be validated using experimental data with respect to the load-deformation properties of both the member global behavior and the behavior of the shear connection at the interface of the slab and beam. Such validation should utilize strength and stiffness properties that match the constitutive models reflected in the actual experimental data. Experimental data providing insight into the load-deformation response of shear connection and the composite member as a whole is provided by Lyons et al. (1994) and Roddenberry et al. (2002a). As another alternative, the mixed analysis approach provided by Oehlers and Sved (1995) can be used.

When steel headed stud anchors are used on beams with formed steel deck, they may be welded directly through the deck or through prepunched or cut-in-place holes in the deck. The usual procedure is to install steel headed stud anchors by welding directly through the deck. However, special precautions and procedures recommended by the stud manufacturer should be followed when:

(1) The deck thickness is greater than 16 gage (1.5 mm) for single thickness or 18 gage (1.2 mm) for each sheet of double thickness; or

(2) The total thickness of galvanized coating is greater than 1.25 ounces/ft^2 (0.38 kg/m^2).

Composite beam tests in which the longitudinal spacing of steel headed stud anchors was varied according to the intensity of the static shear, and duplicate beams in which the anchors were uniformly spaced, exhibited approximately the same ultimate strength and approximately the same amount of deflection at nominal loads. Under distributed load conditions, only a slight deformation in the concrete near the more heavily stressed anchors is needed to redistribute the horizontal shear to other less heavily stressed anchors. The important consideration is that the total number of anchors be sufficient to develop the shear on either side of the point of maximum moment. The provisions of this Specification are based upon this concept of composite action.

2. Load Transfer for Negative Flexural Strength

In computing the available flexural strength at points of maximum negative bending, reinforcement parallel to the steel beam within the effective width of the slab may be included, provided such reinforcement is properly anchored beyond the region of negative moment. However, steel anchors are required to transfer the ultimate tensile force in the reinforcement from the slab to the steel beam.

When the steel deck includes units for carrying electrical wiring, crossover headers are commonly installed over the cellular deck perpendicular to the ribs. These create trenches that completely or partially replace sections of the concrete slab above the deck. These trenches, running parallel to or transverse to a composite beam, may reduce the effectiveness of the concrete flange. Without special provisions to replace the concrete displaced by the trench, the trench should be considered as a complete structural discontinuity in the concrete flange.

When trenches are parallel to the composite beam, the effective flange width should be determined from the known position of the trench.

Trenches oriented transverse to composite beams should, if possible, be located in areas of low bending moment and the full required number of studs should be placed between the trench and the point of maximum positive moment. Where the trench cannot be located in an area of low moment, the beam should be designed as noncomposite.

3. Encased Composite Members

Tests of concrete-encased beams demonstrate that (a) the encasement drastically reduces the possibility of lateral-torsional instability and prevents local buckling of the encased steel, (b) the restrictions imposed on the encasement practically prevent bond failure prior to first yielding of the steel section, and (c) bond failure does not necessarily limit the moment strength of an encased steel beam (ASCE, 1979). Accordingly, this Specification permits three alternative design methods for the determination of the nominal flexural strength: (a) based on an elastic stress distribution using first yield in the tension flange of the composite section; (b) based on the plastic flexural strength of the steel section alone; and (c) based on the strength of the composite section obtained from the plastic stress distribution method or the strain-compatibility method. An assessment of the data indicates that the same resistance and safety factors may be used for all three approaches (Leon et al., 2007). For concrete-encased composite beams, method (c) is applicable only when shear anchors are provided along the steel section and reinforcement of the concrete encasement meets the specified detailing requirements. For concrete-encased composite beams, no limitations are placed on the slenderness of either the composite beam or the elements of the steel section, because the encasement effectively inhibits both local and lateral buckling.

In method (a), stresses on the steel section from permanent loads applied to unshored beams before the concrete has hardened must be superimposed on stresses on the composite section from loads applied to the beams after hardening of the concrete. In this superposition, all permanent loads should be multiplied by the dead load factor and all live loads should be multiplied by the live load factor. For shored beams, all loads may be assumed as resisted by the composite section. Complete interaction (no slip) between the concrete and steel is assumed.

Insufficient research is available to allow coverage of partially composite encased or filled sections subjected to flexure.

4. Filled Composite Members

Tests of filled composite beams indicate that (a) the steel HSS drastically reduces the possibility of lateral-torsional instability, (b) the concrete infill changes the local buckling mode of the steel HSS, and (c) bond failure does not necessarily limit the moment strength of a filled composite beam (Leon et al., 2007).

Figure C-I3.6 shows the variation of the nominal flexural strength, M_n, of the filled section with respect to the HSS wall slenderness. As shown, compact sections can develop the full plastic strength, M_p, in flexure. The nominal flexural strength, M_n, of noncompact sections can be determined using a linear interpolation between the plastic strength, M_p, and the elastic strength based on the yield strength, M_y, with respect to the HSS wall slenderness. Slender sections are limited to developing the first yield moment, M_{cr}, of the composite section where the tension flange reaches first yielding, while the compression flange is limited to the critical buckling stress, F_{cr}, and the concrete is limited to linear elastic behavior with maximum compressive stress equal to $0.7f'_c$ (Lai et al., 2014). The nominal flexural strengths calculated using the Specification compare conservatively with experimental results (Lai et al., 2014; Lai and Varma, 2015). Figure C-I3.7 shows typical stress blocks for determining the nominal flexural strengths of compact, noncompact and slender filled rectangular box sections.

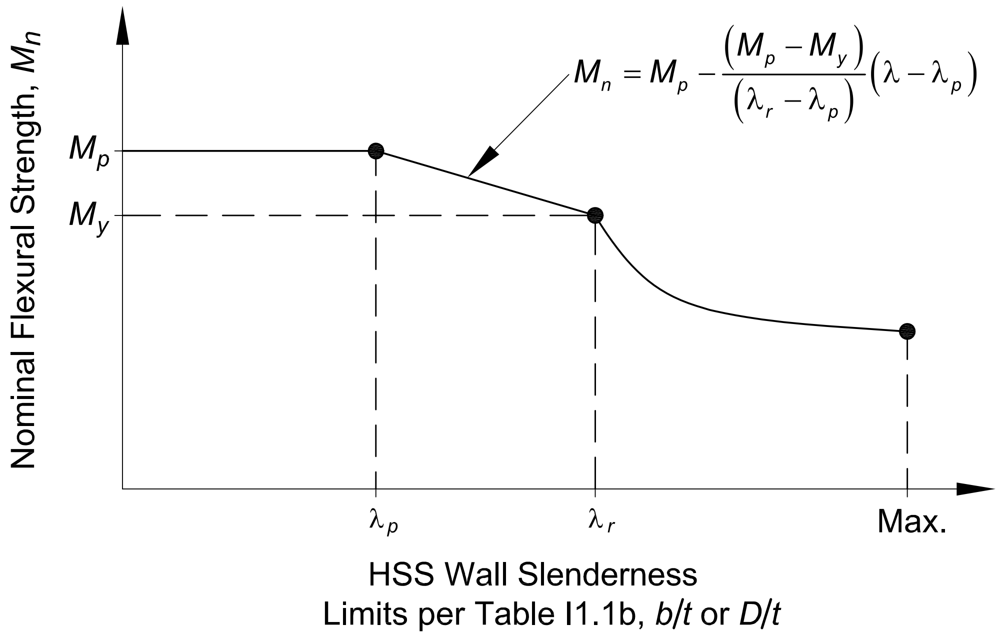


Fig. C-I3.6. Nominal flexural strength of a filled beam versus HSS wall slenderness.

I4. SHEAR

1. Filled and Encased Composite Members

Three methods for determining the shear strength of filled and encased composite members are provided:

(a) The intent is to allow the designer to ignore the concrete contribution entirely and simply use the provisions of Chapter G with their associated resistance or safety factors.

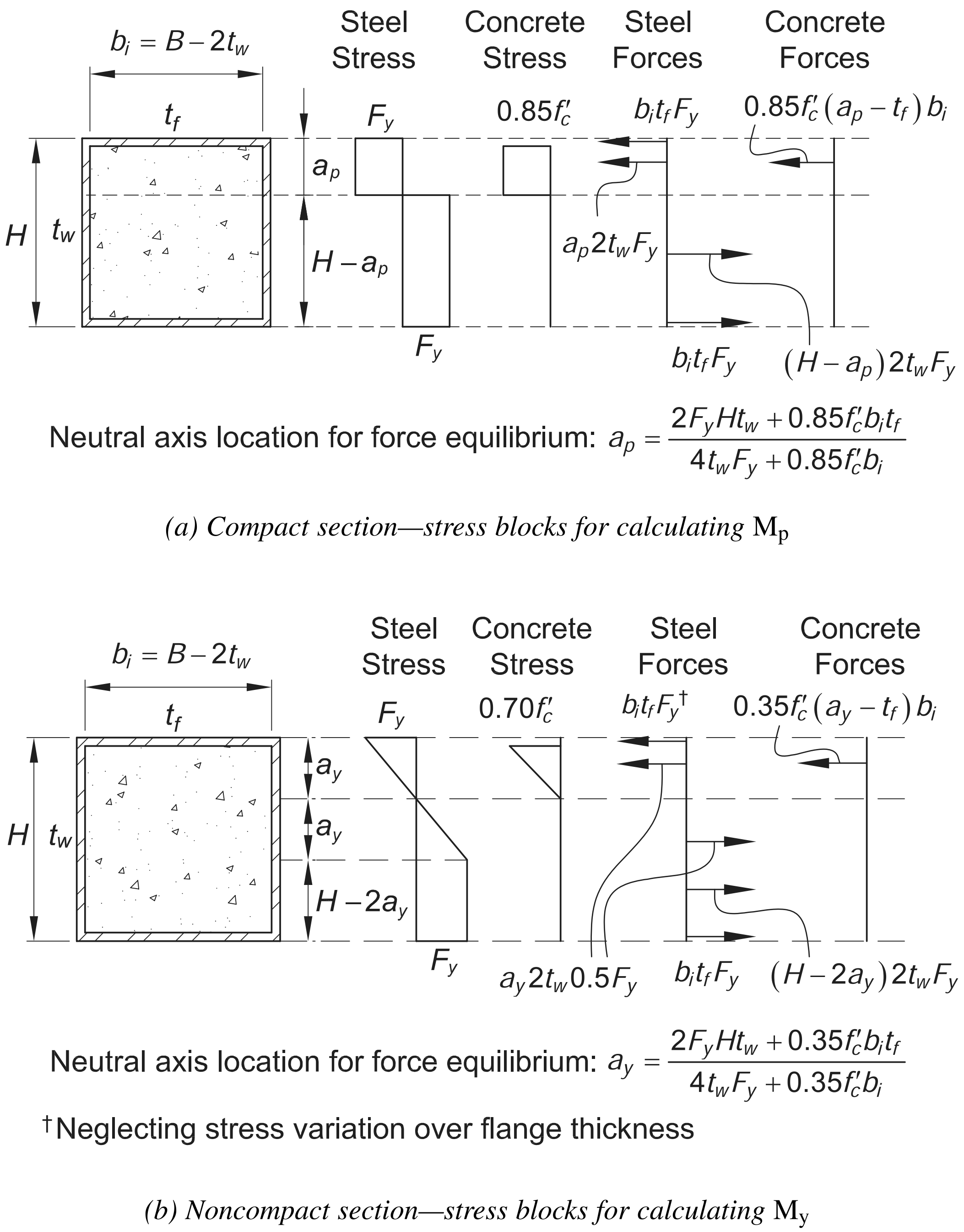


Fig. C-I3.7. Stress blocks for calculating nominal flexural strengths of filled rectangular box sections (Lai et al., 2014).

(b) When using only the strength of the reinforcing and concrete, a resistance factor of 0.75 or the corresponding safety factor of 2.00 is to be applied, which is consistent with ACI 318.

(c) When using the strength of the steel section in combination with the contribution of the transverse reinforcing bars, the nominal shear strength of the steel section alone should be determined according to the provisions of Chapter G and then combined with the nominal shear strength of the transverse reinforcing as determined by ACI 318. This combined nominal strength should then be multiplied by an overall resistance factor of 0.75 or divided by the safety factor of 2.00 to determine the available shear strength of the member.

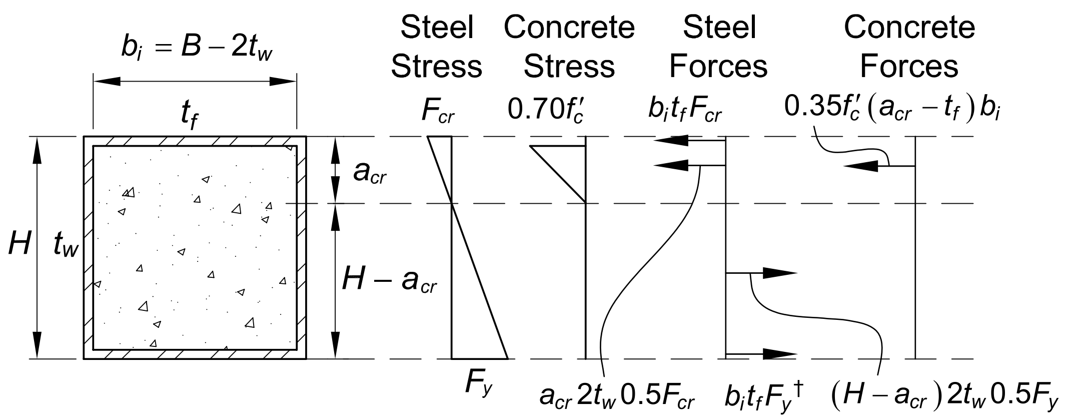
Though it would be logical to suggest provisions where both the contributions of the steel section and reinforced concrete are superimposed, there is insufficient research available to justify such a combination.

2. Composite Beams with Formed Steel Deck

A conservative approach to shear provisions for composite beams with steel headed stud or steel channel anchors is adopted by assigning all shear to the steel section in accordance with Chapter G. This method neglects any concrete contribution and serves to simplify design.

I5. COMBINED FLEXURE AND AXIAL FORCE

As with all frame analyses in this Specification, required strengths for composite beam-columns should be obtained from second-order analysis or amplified first-order analysis, as specified in Chapter C and Appendix 7, respectively. Section I1.5



Neutral axis location for force equilibrium: $a_{cr} = \dfrac{F_y H t_w + (0.35f'_c + F_y - F_{cr})b_i t_f}{t_w(F_{cr} + F_y) + 0.35f'_c b_i}$

†Neglecting stress variation over flange thickness

(c) Slender section—stress blocks for calculating first yield moment, M_{cr}

Fig. C-I3.7 (continued). Stress blocks for calculating nominal flexural strengths of filled rectangular box sections (Lai et al., 2014).

provides the appropriate stiffness for composite members to be used with the direct analysis method of Chapter C. For the assessment of available strength, the Specification provisions for interaction between axial force and flexure in composite members are the same as for bare steel members as covered in Section H1.1. The provisions also permit an analysis based on the strength provisions of Section I1.2 that leads to an interaction diagram similar to those used in reinforced concrete design. The latter approach is discussed here.

For encased composite members, the available axial strength, including the effects of buckling, and the available flexural strength can be calculated using either the plastic stress distribution method or the strain-compatibility method (Leon et al., 2007; Leon and Hajjar, 2008). For filled composite members, the available axial and flexural strengths can be calculated using Sections I2.2 and I3.4, respectively, which also include the effects of local buckling for noncompact and slender sections (classified according to Section I1.4).

The following commentary describes three different approaches to designing composite beam-columns that are applicable to both concrete-encased steel shapes and compact filled HSS, and a fourth approach that is applicable to noncompact or slender filled sections. The first two approaches are based on variations in the plastic stress distribution method while the third method references AISC Design Guide 6, *Load and Resistance Factor Design of W-Shapes Encased in Concrete* (Griffis, 1992), which is based on an earlier version of the Specification. The strain compatibility method is similar to that used in the design of concrete compression members as specified in ACI 318 Chapter 22 (ACI, 2014).

Method 1—Interaction Equations of Section H1. The first approach applies to doubly symmetric composite beam-columns, the most common geometry found in building construction. For this case, the interaction equations of Section H1 provide a conservative assessment of the available strength of the member for combined axial compression and flexure (see Figure C-I5.1). These provisions may also be used for combined axial tension and flexure. The degree of conservatism generally depends on the extent of concrete contribution to the overall strength relative to the steel contribution. The larger the load carrying contribution coming from the steel section, the less conservative the strength prediction of the interaction equations from Section H1. Thus, for example, the equations are generally more conservative for members with high concrete compressive strength as compared to members with low concrete compressive strength. The advantages of this method include the following: (a) the same interaction equations used for steel beam-columns are applicable; and (b) only two anchor points are needed to define the interaction curves—one for pure flexure (point B) and one for pure axial load (point A). Point A is determined using Equations I2-2 or I2-3, as applicable. Point B is determined as the flexural strength of the section according to the provisions of Section I3. Note that slenderness must also be considered using the provisions of Section I2.

The nominal strengths predicted using the equations of Section H1 compare conservatively with a wide range of experimental data for noncompact/slender rectangular and round filled sections (Lai et al., 2014; Lai and Varma, 2015).

Method 2—Interaction Curves from the Plastic Stress Distribution Method. The second approach applies to doubly symmetric encased and compact filled composite beam-columns and is based on developing interaction surfaces for combined axial compression and flexure at the nominal strength level using the plastic stress distribution method. This approach results in interaction surfaces similar to those shown in Figure C-I5.2. The four points, A through D, identified in Figure C-I5.2, are defined by the plastic stress distribution used in their determination. The strength equations for concrete encased W-shapes and filled HSS shapes used to define each point are provided in Geschwindner (2010b) and will be available in the 15th Edition AISC *Steel Construction Manual* Part 6. Point A is the pure axial strength determined according to Section I2. Point B is determined as the flexural strength of the section according to the provisions of Section I3. Point C corresponds to a plastic neutral axis location that results in the same flexural strength as point B, but including axial compression. Point D corresponds to an axial compressive strength of one-half of that determined for point C. An additional point E (see Figure C-I1.1b) is included (between points A and C) for encased W-shapes bent about their weak axis. Point E is an arbitrary point, generally corresponding to a plastic neutral axis location at the flange tips of the encased W-shape, necessary to better reflect bending strength for weak-axis bending of encased shapes. Linear interpolation between these anchor points may be used. However, with this approach, care should be taken in reducing point D by a resistance factor or to account for member slenderness, as this may lead to an unsafe situation whereby additional flexural strength is permitted at a lower axial compressive strength than predicted by the cross-section strength of the member. This potential problem may be avoided through a simplification to this method whereby point D is removed from the interaction surface. Figure C-I5.3 demonstrates this simplification with the vertical dashed line that connects point C″ to point B″. Once the nominal strength interaction surface is determined, length effects according

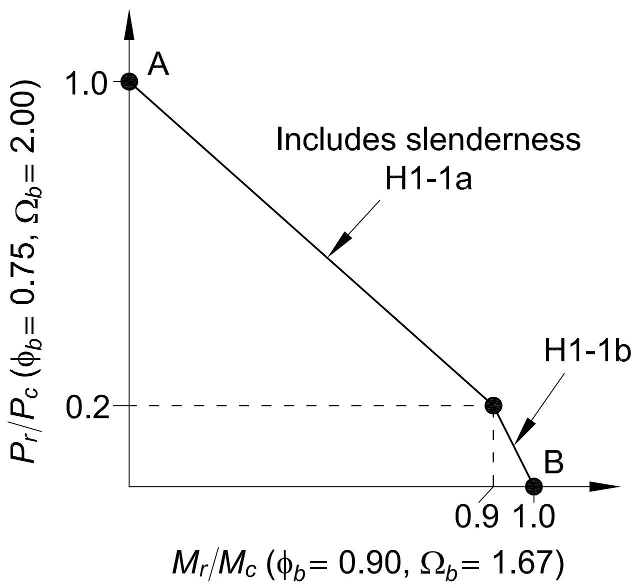


Fig. C-I5.1. Interaction diagram for composite beam-column design—Method 1.

to Equations I2-2 and I2-3 must be applied to obtain points A′ through E′. Note that the same slenderness reduction factor (λ = A′/A in Figure C-I5.2, equal to P_n/P_{no}, where P_n and P_{no} are calculated from Section I2) applies to points A, C, D and E. The available strength is then determined by applying the compression and bending resistance factors or safety factors to points A″ through E″.

Using linear interpolation between points A″, C″ and B″ in Figure C-I5.3, the following interaction equations may be derived for composite beam-columns subjected to combined axial compression plus biaxial flexure:

(a) If $P_r < P_C$

$$\frac{M_{rx}}{M_{Cx}} + \frac{M_{ry}}{M_{Cy}} \leq 1 \qquad \text{(C-I5-1a)}$$

(b) If $P_r \geq P_C$

$$\frac{P_r - P_C}{P_A - P_C} + \frac{M_{rx}}{M_{Cx}} + \frac{M_{ry}}{M_{Cy}} \leq 1 \qquad \text{(C-I5-1b)}$$

where

P_r = required compressive strength, kips (N)
P_A = available axial compressive strength at point A″, kips (N)
P_C = available axial compressive strength at point C″, kips (N)
M_r = required flexural strength, kip-in. (N-mm)
M_C = available flexural strength at point C″, kip-in. (N-mm)
x = subscript relating symbol to strong-axis bending
y = subscript relating symbol to weak-axis bending

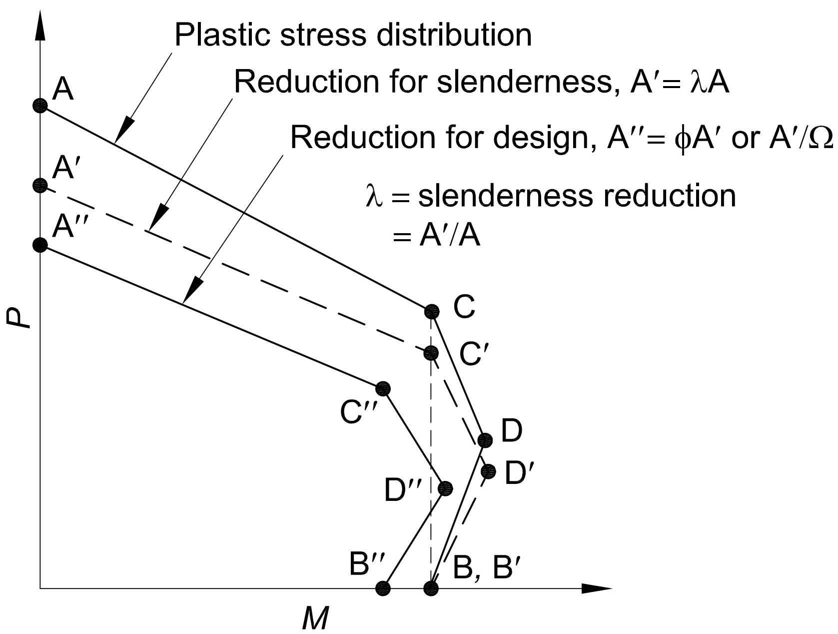


Fig. C-I5.2. Interaction diagram for composite beam-columns—Method 2.

For design according to Section B3.3 (LRFD):

P_r = P_u = required compressive strength using LRFD load combinations, kips (N)

P_A = design axial compressive strength at point A″ in Figure C-I5.3, determined in accordance with Section I2, kips (N)

P_C = design axial compressive strength at point C″, kips (N)

M_r = required flexural strength using LRFD load combinations, kip-in. (N-mm)

M_C = design flexural strength at point C″, determined in accordance with Section I3, kip-in. (N-mm)

For design according to Section B3.4 (ASD):

P_r = P_a = required compressive strength using ASD load combinations, kips (N)

P_A = allowable compressive strength at point A″ in Figure C-I5.3, determined in accordance with Section I2, kips (N)

P_C = allowable axial compressive strength at point C″, kips (N)

M_r = required flexural strength using ASD load combinations, kip-in. (N-mm)

M_C = allowable flexural strength at point C″, determined in accordance with Section I3, kip-in. (N-mm)

For biaxial bending, the value of the axial compressive strength at point C may be different when computed for the strong and weak axis. The smaller of the two values should be used in Equation C-I5-1b and for the limits in Equations C-I5-1a and b.

Method 3—Design Guide 6. The approach presented in AISC Design Guide 6 (Griffis, 1992) may also be used to determine the beam-column strength of concrete encased W-shapes. Although this method is based on an earlier version of the Specification, axial load and moment strengths can conservatively be determined directly from the tables in this design guide. The difference in resistance factors from the earlier Specification may safely be ignored.

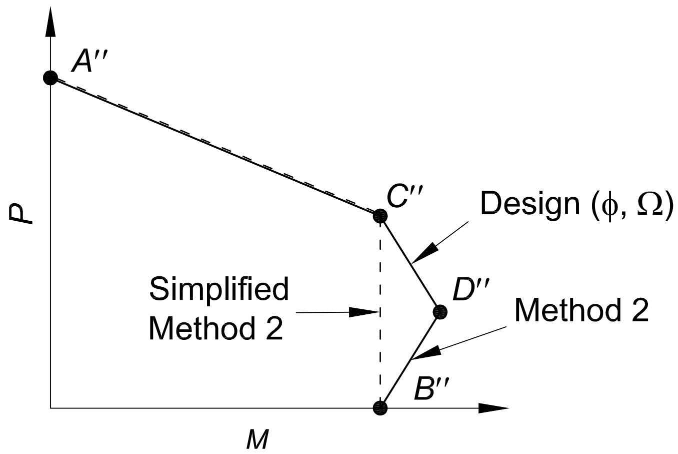


Fig. C-I5.3. Interaction diagram for composite beam-columns—Method 2 simplified.

Method 4—Direct Interaction Method for Noncompact and Slender Filled Sections. For filled noncompact and slender composite members, the interaction equations in Section H1.1 can be conservative (Lai et al., 2016). The interaction between axial compression, P, and flexure, M, in filled composite members is typically seen to vary with the strength ratio, c_{sr}, which is calculated using Equation I5-2 as the yield strength of the steel components divided by the compressive strength of the concrete component. As the c_{sr} ratio increases, the steel component dominates. As the ratio decreases, the concrete component dominates.

This behavior is illustrated in Figure C-I5.4, which shows interaction curves developed using Equations I5-1a and b. Lai et al. (2016) developed these equations as a bilinear simplification of the parabolic P-M interaction curves for filled composite members with noncompact or slender cross sections. The three anchor points of the normalized interaction curve include: (a) the member available axial compressive strength as a column, P_c, determined using Section I2.2b; (b) the member available flexural strength, M_c, determined using Section I3.4; and (c) the balance point with coordinates (c_m, c_p). The balance point coordinates are functions of the strength ratio, c_{sr}, and calculated using Table I5.1 for rectangular and round filled composite members.

The interaction curve developed using Equations I5-1a and b is recommended for noncompact or slender filled composite members: (a) with governing unsupported length-to-diameter, L/D, or length-to-width, L/B ratios, less than or equal to 20; and (b) not providing stability support to leaning or gravity-only columns with significant

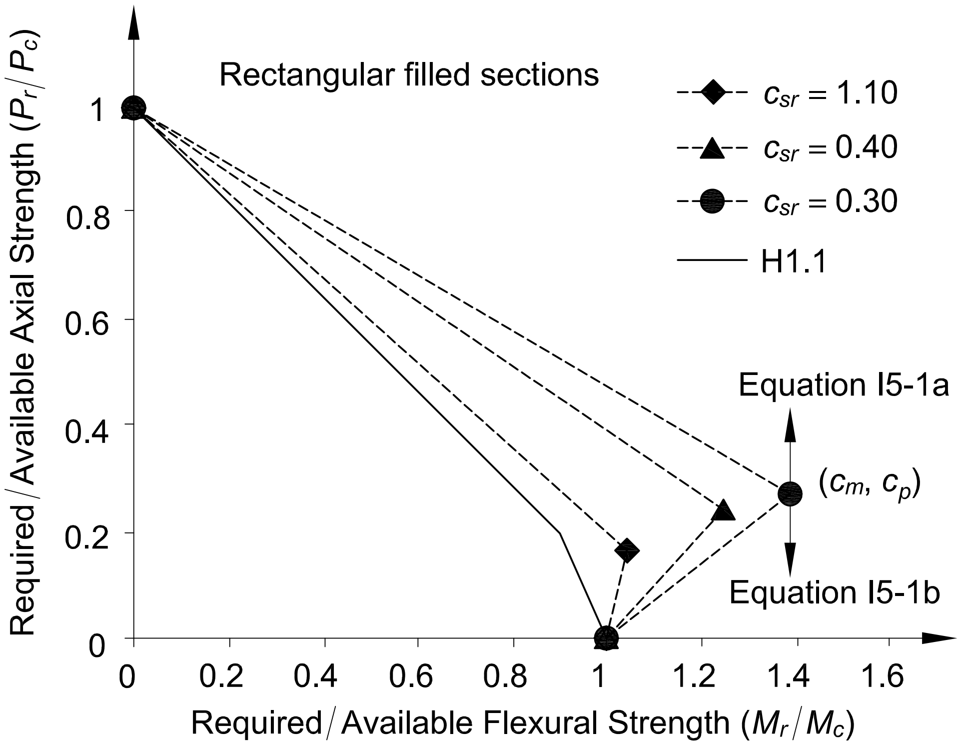


Fig. C-I5.4. Interaction diagram for filled composite members with noncompact or slender cross section developed using Equations I5-1a and b.

axial loading. In situations where (a) and (b) are not met, the balance point with coordinates (c_m, c_p) may be reduced further due to slenderness effects. This potential problem can be addressed through the simplified method described earlier and shown in Figure C-I5.3, where the increased available flexural strength due to axial compression is removed from the interaction curve.

I6. LOAD TRANSFER

1. General Requirements

External forces are typically applied to composite members through direct connection to the steel member, bearing on the concrete, or a combination thereof. Design of the connection for force application shall follow the applicable limit states within Chapters J and K of the Specification as well as the provisions of Section I6. Note that for concrete bearing checks on filled composite members, confinement can affect the bearing strength for external force application as discussed in Commentary Section I6.2.

Once a load path has been provided for the introduction of external force to the member, the interface between the concrete and steel must be designed to transfer the longitudinal shear required to obtain force equilibrium within the composite section. Section I6.2 contains provisions for determining the magnitude of longitudinal shear to be transferred between the steel and concrete depending upon the external force application condition. Section I6.3 contains provisions addressing mechanisms for the transfer of longitudinal shear.

The load transfer provisions of this Specification are primarily intended for the transfer of longitudinal shear due to applied axial forces. Load transfer of longitudinal shear due to applied bending moments is beyond the scope of the Specification; however, tests (Lu and Kennedy, 1994; Prion and Boehme, 1994; Wheeler and Bridge, 2006) indicate that filled composite members can develop their full plastic moment capacity based on bond alone without the use of additional anchorage.

2. Force Allocation

This Specification addresses conditions in which the entire external force is applied to the steel or concrete as well as conditions in which the external force is applied to both materials concurrently. The provisions are based upon the assumption that in order to achieve equilibrium across the cross section, transfer of longitudinal shears along the interface between the concrete and steel shall occur such that the resulting force levels within the two materials may be proportioned according to the relative cross-sectional strength contributions of each material. Load allocation based on the cross-sectional strength contribution model is represented by Equations I6-1 and I6-2. Equation I6-1 represents the magnitude of force that is present within the concrete encasement or concrete fill at equilibrium. The longitudinal shear generated by loads applied directly to the steel section is determined based on the amount of force to be distributed to the concrete according to Equation I6-1. Conversely, when load is applied to the concrete section only, the longitudinal shear required for cross-sectional equilibrium is based upon the amount of force to be distributed to the steel according to Equation I6-2. Where loads are applied concurrently to the two materials, the longitudinal shear force to be transferred to achieve cross-sectional equilibrium can be taken as either the difference in magnitudes between the portion of external force applied directly to the concrete and that required by Equation I6-1 or the portion of external force applied directly to the steel section and that required by Equations I6-2a and b. The steel contribution to the overall nominal axial compressive strength of the cross section decreases with increasing slenderness ratio (B/t or D/t). As a result, it is not permitted to apply axial force directly to the steel wall of filled composite sections classified as slender because the stress concentrations associated with force application could cause premature local buckling. Additionally, the magnitude of longitudinal shears required to be transferred to the concrete infill would require impractical load transfer lengths.

When external forces are applied to the concrete of a filled composite member via bearing, it is acceptable to assume that adequate confinement is provided by the steel encasement to allow the maximum available bearing strength permitted by Equation J8-2 to be used. This strength is obtained by setting the term $\sqrt{A_2/A_1}$ equal to 2. This discussion is in reference to the introduction of external load to the compression member. The transfer of longitudinal shear within the compression member via bearing mechanisms such as internal steel plates is addressed directly in Section I6.3a.

The Specification provisions assume that the required external force to be allocated imparts compression to the composite section. For applied tensile force, it is generally acceptable to design the component of the composite member to which the force is applied (i.e., either the steel section or the longitudinal reinforcement) to resist the entire tensile force, and no further force transfer calculations are necessary. For atypical conditions where the magnitude of required external tensile force necessitates the use of longitudinal reinforcement in conjunction with the steel section, force allocation to each component may be determined as follows.

When the entire external tensile force is applied directly to the steel section:

$$V_r' = P_r(1 - F_y A_s / P_n) \qquad \text{(C-I6-1)}$$

When the entire external tensile force is applied directly to the longitudinal reinforcement:

$$V_r' = P_r(F_y A_s / P_n) \qquad \text{(C-I6-2)}$$

where

P_n = nominal axial tensile strength, determined by Equation I2-8 for encased composite members, and Equation I2-14 for filled composite members, kips (N)

P_r = required external tensile force applied to the composite member, kips (N)

V_r' = required longitudinal shear force to be transferred to the steel section or longitudinal reinforcement, kips (N)

Where longitudinal reinforcing bars are used to resist tension forces, they must use appropriate lap splices in accordance with ACI 318 as directed by Section I1. For sustained tension, mechanical splices are required by ACI 318 Section 25.5.7.4 (ACI, 2014).

3. Force Transfer Mechanisms

Transfer of longitudinal shear by direct bearing via internal bearing mechanisms, such as internal bearing plates or shear connection via steel anchors, is permitted for both filled and encased composite members. Transfer of longitudinal shear via direct bond interaction is permitted solely for compact and noncompact filled composite members. Although it is recognized that force transfer also occurs by direct bond interaction between the steel and concrete for encased composite columns, this mechanism is typically ignored and shear transfer is generally carried out solely with steel anchors (Griffis, 1992).

The use of the force transfer mechanism providing the largest resistance is permissible. Superposition of force transfer mechanisms is not permitted as the experimental data indicate that direct bearing or shear connection often does not initiate until after direct bond interaction has been breached, and little experimental data is available regarding the interaction of direct bearing and shear connection via steel anchors.

3a. Direct Bearing

For the general condition of load applied directly to concrete in bearing, and considering a supporting concrete area that is wider on all sides than the loaded area, the nominal bearing strength for concrete may be taken as

$$R_n = 0.85 f_c' A_1 \sqrt{A_2 / A_1} \tag{C-I6-3}$$

where

A_1 = loaded area of concrete, in.^{2} (mm^2)

A_2 = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, in.^{2} (mm^2)

f_c' = specified compressive concrete strength, ksi (MPa)

The value of $\sqrt{A_2 / A_1}$ must be less than or equal to 2 (ACI, 2014).

For the specific condition of transferring longitudinal shear by direct bearing via internal bearing mechanisms, the Specification uses the maximum nominal bearing strength allowed by Equation C-I6-1 of $1.7 f_c' A_1$ as indicated in Equation I6-3. The resistance factor for bearing, ϕ_B, is 0.65 (and the associated safety factor, Ω_B, is 2.31) in accordance with ACI 318.

3b. Shear Connection

Steel anchors shall be designed according to the provisions for composite components in Section I8.3.

3c. Direct Bond Interaction

Force transfer by direct bond is commonly used in filled composite members as long as the connections are detailed to limit local deformations (API, 1993; Roeder et al., 1999). While chemical adhesion provides some contribution, direct bond is primarily a frictional resistance mechanism. There is large scatter in the experimental data on the bond of filled composite compression members; however, some trends have been identified (Roeder et al., 1999; Zhang et al., 2012). Larger cross sections, thinner walls, rectangular shapes, smoothed or oiled interfaces, and high-shrinkage concrete contribute to lower apparent bond strengths. Smaller cross sections, thicker walls, circular shapes, rougher interfaces, expansive concrete, and the presence of bending moment (including eccentric loading such as from shear tabs) contribute to higher apparent bond strengths.

The equations for direct bond interaction for filled composite compression members assume the entire interface perimeter is engaged in the transfer of stress. Accordingly, and in contrast to the previous edition of the Specification, the strength is compared to the sum of the force required to be transferred from connecting elements framing in from all sides. The scatter in the experimental data leads to the recommended low value of the resistance factor, ϕ, and the corresponding high value of the safety factor, Ω.

4. Detailing Requirements

To avoid overstressing the structural steel section or the concrete at connections in encased or filled composite members, transfer of longitudinal shear is required to occur within the load introduction length. The load introduction length is taken as two times the minimum transverse dimension of the composite member both above and below the load transfer region. The load transfer region is generally taken as the depth of the connecting element as indicated in Figure C-I6.1. In cases where the applied forces are of such a magnitude that the required longitudinal shear transfer cannot take place within the prescribed load introduction length, the designer should treat the compression member as noncomposite along the additional length required for shear transfer.

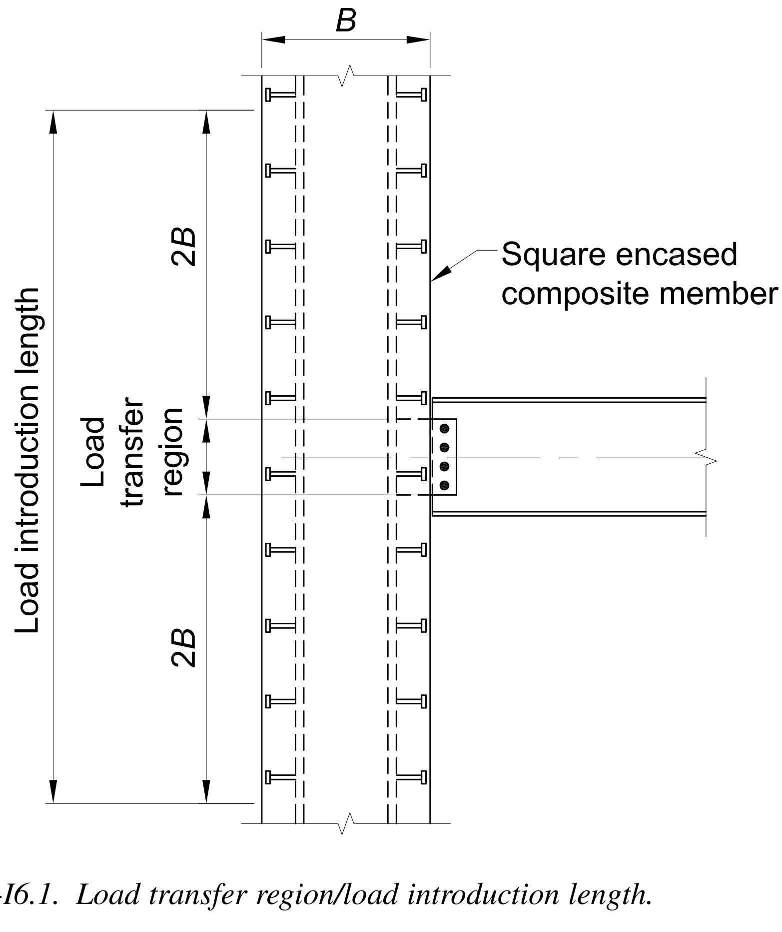


Fig. C-I6.1. Load transfer region/load introduction length.

For encased composite members, steel anchors are required throughout the compression member length in order to maintain composite action of the member under incidental moments (including flexure induced by incipient buckling). These anchors are typically placed at the maximum permitted spacing according to Section I8.3e. Additional anchors required for longitudinal shear transfer shall be located within the load introduction length as described previously.

Unlike concrete encased members, steel anchors in filled members are required only when used for longitudinal shear transfer and are not required along the length of the member outside of the introduction region. This difference is due to the adequate confinement provided by the steel encasement which prevents the loss of composite action under incidental moments.

I7. COMPOSITE DIAPHRAGMS AND COLLECTOR BEAMS

In composite construction, floor or roof slabs consisting of composite metal deck and concrete fill are typically connected to the structural framing to form composite diaphragms. Diaphragms are horizontally spanning members, analogous to deep beams, which distribute lateral loads from their origin to the lateral force-resisting system either directly or in combination with load transfer elements known as collectors or collector beams (also known as diaphragm struts and drag struts).

Diaphragms serve the important structural function of interconnecting the components of a structure to help it behave as a unit. Diaphragms are commonly analyzed as simple-span or continuously spanning deep beams, and hence, are subject to shear, moment and axial forces, as well as the associated deformations. Further information on diaphragm classifications and behavior can be found in AISC (2012) and SDI (2015).

Composite Diaphragm Strength. Diaphragms should be designed to resist all forces associated with the collection and distribution of lateral forces to the lateral force-resisting system. In some cases, loads from other floors should also be included, such as at a level where a horizontal offset in the lateral force-resisting system exists. Several methods exist for determining the in-place shear strength of composite diaphragms. Three such methods are as follows:

(a) As determined for the combined strength of composite deck and concrete fill, including the considerations of composite deck configuration, as well as type and layout of deck attachments. One publication which is considered to provide such guidance is the SDI *Diaphragm Design Manual* (SDI, 2015). This publication covers many aspects of diaphragm design, including strength and stiffness calculations. Calculation procedures are also provided for alternative deck-to-framing connection methods, such as puddle welding and mechanical fasteners in cases where anchors are not used. Where stud anchors are used, stud shear strength values shall be as determined according to Section I8.

(b) As the thickness of concrete over the steel deck is increased, the shear strength can approach that for a concrete slab of the same thickness. For example, in composite floor deck diaphragms having cover depths between 2 in. (50 mm) and

6 in. (150 mm), measured shear stresses on the order of $0.11\sqrt{f_c'}$ (where f_c' is in units of ksi) have been reported. In such cases, the diaphragm strength of concrete metal deck slabs can conservatively be based on the principles of reinforced concrete design (ACI, 2014) using the concrete and reinforcement above the metal deck ribs and ignoring the beneficial effect of the concrete in the flutes.

(c) Results from in-plane tests of filled diaphragms.

Collector Beams and Other Composite Elements. Horizontal diaphragm forces are transferred to the steel lateral force-resisting frame as axial forces in collector beams (also known as diaphragm struts or drag struts). The design of collector beams has not been addressed directly in this Chapter. The rigorous design of composite beam-columns (collector beams) is complex and few detailed guidelines exist on such members. Until additional research becomes available, a reasonable simplified design approach is provided as follows:

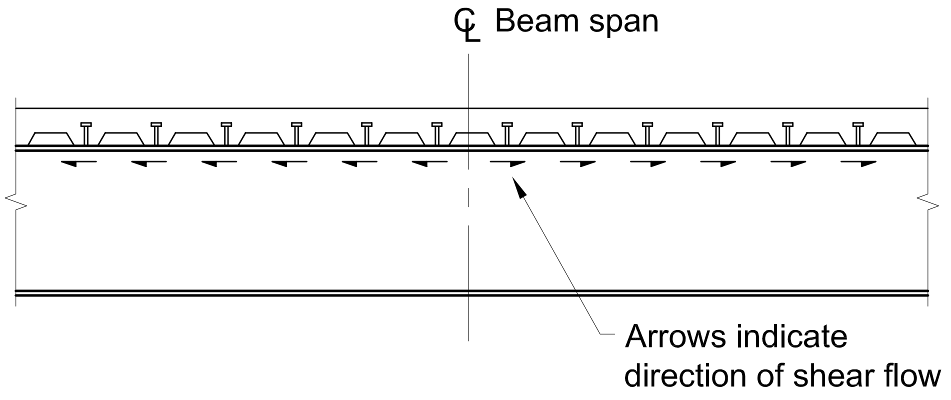
Force Application. Collector beams can be designed for the combined effects of axial load due to diaphragm forces, as well as flexure due to gravity and/or lateral loads. The effect of the vertical offset (eccentricity) between the plane of the diaphragm and the centerline of the collector element results in additional shear reactions that should be investigated for design.

Axial Strength. The available axial strength of collector beams can be determined according to the noncomposite provisions of Chapter D and Chapter E. For compressive loading, collector beams are generally considered unbraced for buckling between braced points about their strong axis, and fully braced by the composite diaphragm for buckling about the weak axis. The limit state of constrained-axis torsional buckling about the top flange as discussed in the Commentary Section E4 may also apply.

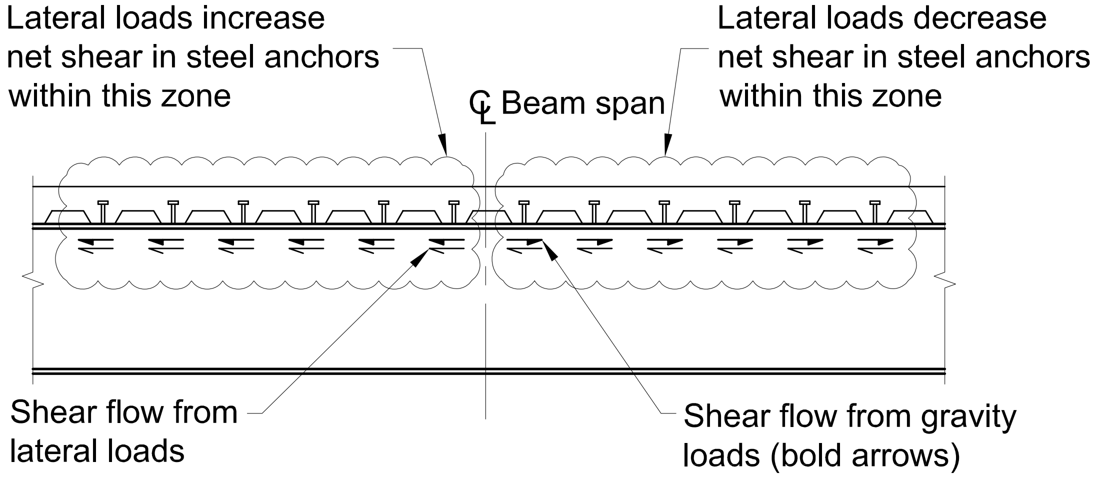
Flexural Strength. The available flexural strength of collector beams can be determined using either the composite provisions of Chapter I or the noncomposite provisions of Chapter F. It is recommended that all collector beams, even those designed as noncomposite members, should consider shear connector slip capacity as discussed in Commentary Section I3. This recommendation is intended to prevent designers from utilizing a small number of anchors solely to transfer diaphragm forces on a beam designed as a noncomposite member. Anchors designed only to transfer horizontal shear due to lateral forces will still be subjected to horizontal shear due to flexure from gravity loads superimposed on the composite section and could become overloaded under gravity loading conditions. Overloading the anchors could result in loss of stud strength, which could inhibit the ability of the collector beam to function as required for the transfer of diaphragm forces due to lateral loads.

Interaction. Combined axial force and flexure can be assessed using the interaction equations provided in Chapter H. As a reasonable simplification for design purposes, it is acceptable to use the noncomposite axial strength and the composite flexural strength in combination for determining interaction.

Shear Connection. It is not required to superimpose the horizontal shear due to lateral forces with the horizontal shear due to flexure for the determination of steel anchor requirements. The reasoning behind this methodology is twofold. First, the load combinations as presented in ASCE/SEI 7 (ASCE, 2016) provide reduced live load levels for load combinations containing lateral loads. This reduction decreases the demand on the steel anchors and provides additional capacity for diaphragm force transfer. Secondly, horizontal shear due to flexure in a simply supported member flows in two directions. For a uniformly loaded beam, the shear flow emanates outwards from the center of the beam as illustrated in Figure C-I7.1(a). Lateral loads on collector beams induce shear in one direction. As these shears are superimposed, the horizontal shears on one portion of the beam are increased and the horizontal shears on the opposite portion of the beam are decreased as illustrated in Figure C-I7.1(b). In lieu of additional research, it is considered acceptable for the localized additional loading of the steel anchors in the additive beam segment to be considered offset by the concurrent unloading of the steel anchors in the subtractive beam segment up to a force level corresponding to the summation of the nominal strengths of all studs placed on the beam. It is considered that the shear connectors in typically practical configurations possess an adequate degree of slip capacity to accommodate this mechanism.



(a) Shear flow due to gravity loads only



(b) Shear flow due to gravity and lateral loads in combination

Fig. C-I7.1. Shear flow at collector beams.

I8. STEEL ANCHORS

1. General

This section covers the strength, placement and limitations on the use of steel anchors in composite construction. The term “steel anchor,” first introduced in the 2010 AISC *Specification*, includes the traditional “shear connector,” now defined as a “steel headed stud anchor” and a “steel channel anchor” both of which have been part of previous Specifications. Both steel headed stud anchors and hot-rolled steel channel anchors are addressed in the Specification. The design provisions for steel anchors are given for composite beams with solid slabs or with formed steel deck and for composite components. A composite component is defined as a member, connecting element or assemblage in which steel and concrete elements work as a unit in the distribution of internal forces. This term excludes composite beams with solid slabs or formed steel deck. The provisions for composite components include the use of a resistance factor or safety factor applied to the nominal strength of the steel anchor, while for composite beams the resistance factor and safety factor are part of the composite beam resistance and safety factor.

Steel headed stud anchors up to 1 in. (25 mm) in diameter are now permitted for use in beams with solid slabs based on a review of available data and their history of successful performance in bridge applications. The limitation of $^{3}/_{4}$-in. (19 mm) anchors for all other conditions represents the limits of push-out data for decked members as well as the limits of applicability of the current composite component provisions. Though larger anchors for use in composite components are not addressed by this Specification, their strength may be determined by ACI 318 Chapter 17 (ACI, 2014).

Studs not located directly over the web of a beam tend to tear out of a thin flange before attaining full shear strength. To guard against this contingency, the size of a stud not located over the beam web is limited to $2^{1}/_{2}$ times the flange thickness (Goble, 1968). The practical application of this limitation is to select only beams with flanges thicker than the stud diameter divided by 2.5.

Section I8.2 requires a minimum overall headed stud anchor height to the shank diameter ratio of four when calculating the nominal shear strength of a steel headed stud anchor in a composite beam. This requirement has been used in previous Specifications and has had a record of successful performance. For calculating the nominal shear strength of a steel headed stud anchor in other composite components, Section I8.3 increases this minimum ratio to five for normal weight concrete and seven for lightweight concrete. Additional increases in the minimum ratio are required for computing the nominal tensile strength or the nominal strength for interaction of shear and tension in Section I8.3. The provisions of Section I8.3 also establish minimum edge distances and center-to-center spacings for steel headed stud anchors if the nominal strength equations in that section are to be used. These limits are established in recognition of the fact that only steel failure modes are checked in the calculation of the nominal anchor strengths in Equations I8-3, I8-4 and I8-5. Concrete failure modes are not checked explicitly in these equations (Pallarés and Hajjar, 2010a, 2010b), whereas concrete failure is checked in Equation I8-1. This is discussed further in Commentary Section I8.3.

2. Steel Anchors in Composite Beams

2a. Strength of Steel Headed Stud Anchors

The present strength equations for composite beams and steel headed stud anchors are based on the considerable research that has been published in recent years (Jayas and Hosain, 1988a, 1988b; Mottram and Johnson, 1990; Easterling et al., 1993; Roddenberry et al., 2002a). Equation I8-1 contains R_g and R_p factors to bring these composite beam strength requirements to a comparable level with other codes around the world. Other codes use a stud strength expression similar to the AISC *Specification* but the stud strength is reduced by a ϕ factor of 0.8 in the Canadian code (CSA, 2009) and by an even lower partial safety factor ($\phi = 0.60$) for the corresponding stud strength equations in *Eurocode 4* (CEN, 2009). The AISC *Specification* includes the stud anchor resistance factor as part of the overall composite beam resistance factor.

The majority of composite steel floor decks used today have a stiffening rib in the middle of each deck flute. Because of the stiffener, studs must be welded off-center in the deck rib. Studies have shown that steel studs behave differently depending upon their location within the deck rib (Lawson, 1992; Easterling et al., 1993; Van der Sanden, 1996; Yuan, 1996; Johnson and Yuan, 1998; Roddenberry et al., 2002a, 2002b). The so-called "weak" (unfavorable) and "strong" (favorable) positions are illustrated in Figure C-I8.1. Furthermore, the maximum value shown in these studies for studs welded through steel deck is on the order of 0.7 to $0.75F_uA_{sc}$. Studs placed in the weak position have strengths as low as $0.5F_uA_{sc}$.

The strength of stud anchors installed in the ribs of concrete slabs on formed steel deck with the ribs oriented perpendicular to the steel beam is reasonably estimated by the strength of stud anchors computed from Equation I8-1, which sets the default value for steel stud strength equal to that for the weak stud position. Both AISC (1997a) and the Steel Deck Institute (SDI, 2001) recommend that studs be detailed in the strong position, but ensuring that studs are placed in the strong position is not necessarily an easy task because it is not always easy for the installer to determine where along the beam the particular rib is located relative to the end, midspan, or point of zero shear. Therefore, the installer may not be clear on which location is the strong, and which is the weak position.

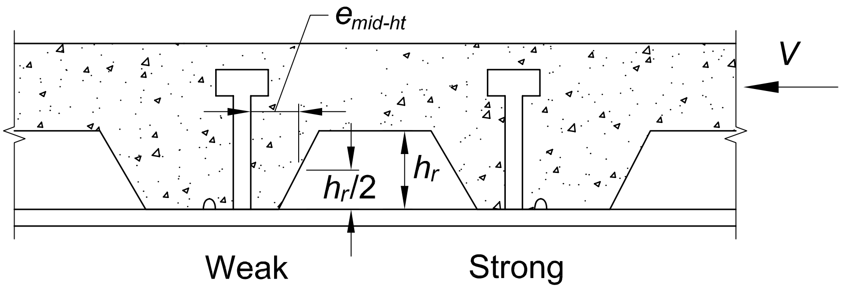


Fig. C-I8.1. Weak and strong stud positions [Roddenberry et al. (2002b)].

In most composite floors designed today, the ultimate strength of the composite section is governed by the strength of the shear connection, as full composite action is typically not the most economical solution to resist the required strength. The degree of composite action, as represented by the ratio of the total shear connection strength divided by the lesser of the yield strength of the steel cross section and the compressive strength of the concrete slab, $\Sigma Q_n/[\min(F_y A_s,\ 0.85 f_c' A_c)]$, influences the flexural strength as shown in Figure C-I8.2.

It can be seen from Figure C-I8.2 that a relatively large change in shear connection strength results in a much smaller change in flexural strength. Thus, formulating the influence of steel deck on shear anchor strength by conducting beam tests and back-calculating through the flexural model, as was done in the past, leads to an inaccurate assessment of stud strength when installed in metal deck.

The changes in stud anchor requirements that occurred in the 2005 AISC *Specification* (AISC, 2005) were not a result of either structural failures or performance problems. Designers concerned about the strength of existing structures based on earlier Specification requirements should note that the slope of the curve shown in Figure C-I8.2 is rather flat as the degree of composite action approaches one. Thus, even a large change in steel stud strength does not result in a proportional decrease of the flexural strength. In addition, the current expression does not account for all the possible shear force transfer mechanisms, primarily because many of them are difficult or impossible to quantify. However, as noted in Commentary Section I3.1, as the degree of composite action decreases, the deformation demands on steel studs increase. This effect is reflected by the increasing slope of the relationship shown in Figure C-I8.2 as the degree of composite action decreases. Thus, designers should

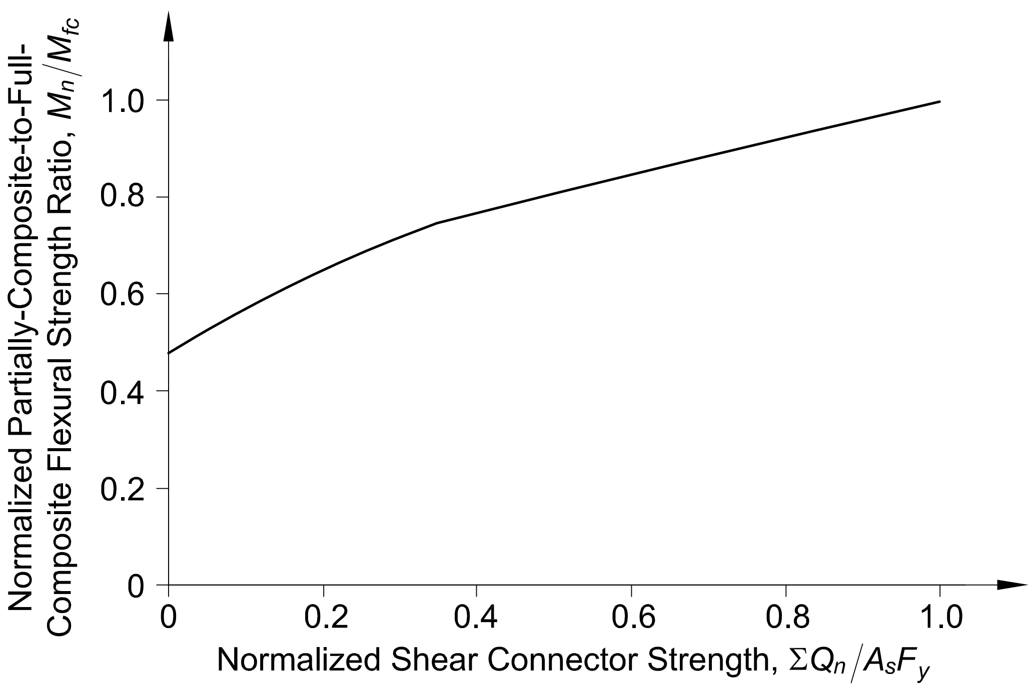


Fig. C-I8.2. Normalized flexural strength versus shear connection strength ratio (W16×31, F_y = 50 ksi, Y2 = 4.5 in.) (Easterling et al., 1993).

consider the influence of increased ductility demand, when evaluating existing composite beams with less than 50% composite action.

The reduction factor, R_p, for headed stud anchors used in composite beams with no decking was reduced from 1.0 to 0.75 in the 2010 AISC *Specification*. The methodology used for headed stud anchors that incorporates R_g and R_p was implemented in the 2005 AISC *Specification*. The research (Roddenberry et al., 2002a) in which the factors R_g and R_p were developed focused almost exclusively on cases involving the use of headed stud anchors welded through the steel deck. The research pointed to the likelihood that the solid slab case should use $R_p = 0.75$; however, the body of test data had not been established to support the change. More recent research has shown that the 0.75 factor is appropriate (Pallarés and Hajjar, 2010a).

2b. Strength of Steel Channel Anchors

Equation I8-2 is a modified form of the formula for the strength of channel anchors presented in Slutter and Driscoll (1965), which was based on the results of pushout tests and a few simply supported beam tests with solid slabs by Viest et al. (1952). The modification has extended its use to lightweight concrete.

Eccentricities need not be considered in the weld design for cases where the welds at the toe and heel of the channel are greater than $^3/_{16}$ in. (5 mm) and the anchor meets the following requirements:

$$1.0 \le \frac{t_f}{t_w} \le 5.5$$

$$\frac{H}{t_w} \ge 8.0$$

$$\frac{L_c}{t_f} \ge 6.0$$

$$0.5 \le \frac{R}{t_w} \le 1.6$$

where

H = height of anchor, in. (mm)

L_c = length of anchor, in. (mm)

R = radius of the fillet between the flange and the web of the channel anchor, in. (mm)

t_f = thickness of channel anchor flange, in. (mm)

t_w = thickness of channel anchor web, in. (mm)

2d. Detailing Requirements

Uniform spacing of shear anchors is permitted, except in the presence of heavy concentrated loads.

The minimum distances from the center of an anchor to a free edge in the direction of the shear force that are shown in this Specification are based on data reported by Nelson Stud Welding Division (Nelson, 1977). Data for various steel headed anchor diameters, concrete compressive strengths, and unit weights are reported. The provisions selected for inclusion in the Specification result in no reduced strength for 3/4-in.-(19 mm) diameter anchors in 4-ksi (28 MPa) concrete, which were deemed to be representative of most composite beam construction. Other values are available in the report for use by the designer if deemed to be more applicable.

The minimum spacing of anchors along the length of the beam, in both flat soffit concrete slabs and in formed steel deck with ribs parallel to the beam, is six diameters; this spacing reflects the development of shear planes in the concrete slab (Ollgaard et al., 1971). Because most test data are based on the minimum transverse spacing of four diameters, this transverse spacing was set as the minimum permitted. If the steel beam flange is narrow, this spacing requirement may be achieved by staggering the studs with a minimum transverse spacing of three diameters between the staggered row of studs. When deck ribs are parallel to the beam and the design requires more studs than can be placed in the rib, the deck may be split so that adequate spacing is available for stud installation. Figure C-I8.3 shows possible anchor arrangements.

3. Steel Anchors in Composite Components

This section applies to steel headed stud anchors used primarily in the load transfer (connection) region of composite compression members and beam-columns, encased and filled composite beams, composite coupling beams, and composite walls, where the steel and concrete are working compositely within a member. An example of the use of steel headed stud anchors in a composite wall is shown in Figure C-I8.4. In such cases, it is possible that the steel anchor will be subjected to shear, tension, or interaction of shear and tension. As the strength of the connectors in the load transfer region must be assessed directly, rather than implicitly within the strength assessment of a composite member, a resistance or safety factor should be applied, comparable to the design of bolted connections in Chapter J.

These provisions are not intended for hybrid construction where the steel and concrete are not working compositely, such as with embed plates. Section I8.2 specifies the strength of steel anchors embedded in a solid concrete slab or in a concrete slab with formed steel deck in a composite beam.

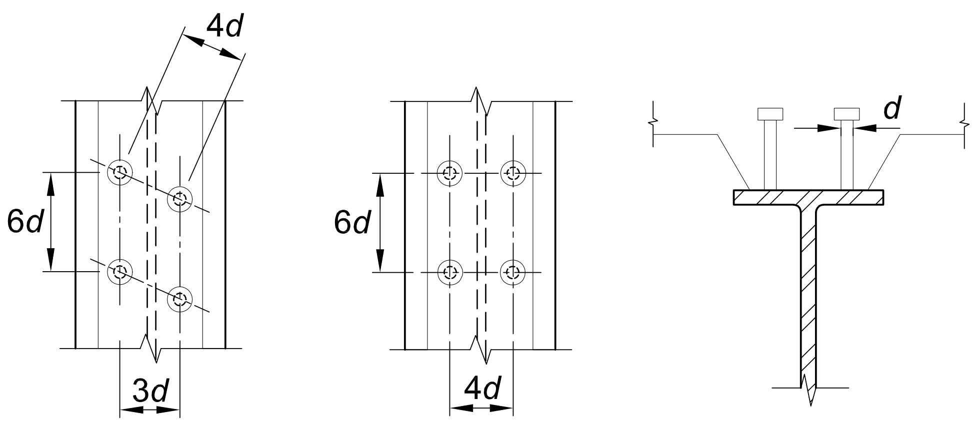


Fig. C-I8.3. Steel anchor arrangements.

Data from a wide range of experiments indicate that the failure of steel headed stud anchors subjected to shear occurs in the steel shank or weld in a large percentage of cases if the ratio of the overall height to the shank diameter of the steel headed stud anchor is greater than five for normal weight concrete. In the case of lightweight concrete, the necessary minimum ratio between the overall height of the stud and the diameter increases up to seven (Pallarés and Hajjar, 2010a). Use of anchors meeting the dimensional limitations for shear loading preclude the limit state of concrete pryout as defined by ACI 318 Chapter 17 (ACI, 2014). A similarly large percentage of failures occur in the steel shank or weld of steel headed stud anchors subjected to tension or interaction of shear and tension if the ratio of the overall height to shank diameter of the steel headed stud anchor is greater than eight for normal weight concrete. In the case of lightweight concrete, the necessary minimum ratio between the overall height of the stud and the diameter increases up to ten for steel headed stud anchors subjected to tension (Pallarés and Hajjar, 2010b). For steel headed stud anchors subjected to interaction of shear and tension in lightweight concrete, there are so few experiments available that it is not possible to discern sufficiently when the steel material will control the failure mode. For the strength of steel headed stud anchors in lightweight concrete subjected to interaction of shear and tension, it is recommended that the provisions of ACI 318 Chapter 17 be used. Use of anchors

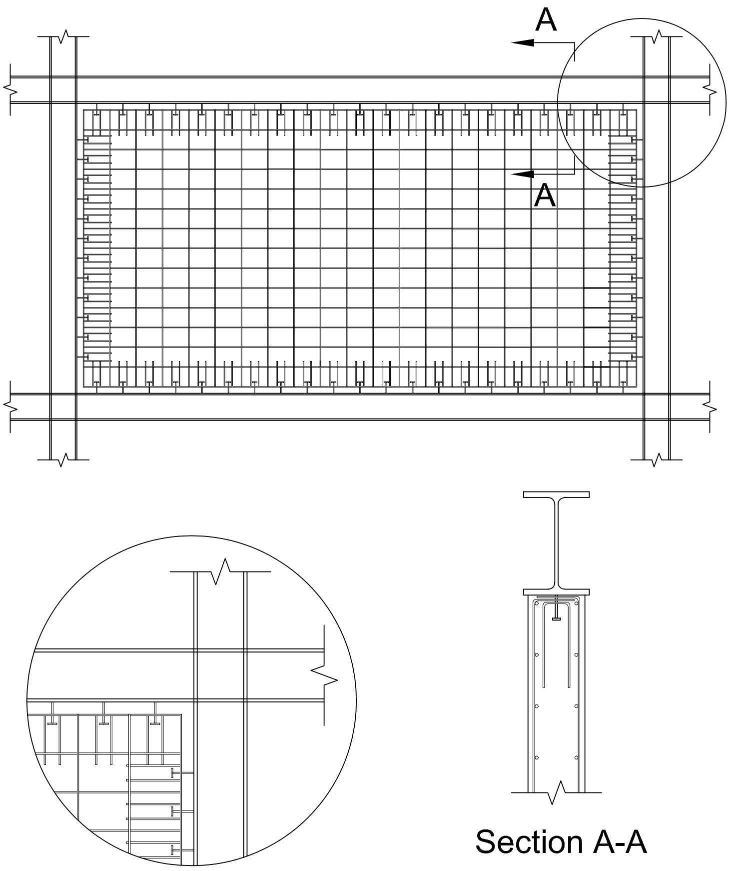


Fig. C-I8.4. Typical reinforcement detailing in a composite wall for steel headed stud anchors subjected to tension.

meeting the dimensional limitations for tension loading preclude the limit states of concrete breakout and pryout as defined by ACI 318 Chapter 17 where analysis indicates no cracking at service load levels, as would generally be the case in compression zones and regions of high confinement typical of composite construction. Where the engineer determines that concrete cracking under service load levels can occur, it is recommended that the provisions of ACI 318 Chapter 17 be used.

The use of edge distances in ACI 318 Chapter 17 to compute the strength of a steel anchor subjected to concrete crushing failure is complex. It is rare in composite construction that there is a nearby edge that is not uniformly supported in a way that prevents the possibility of concrete breakout failure due to a close edge. Thus, for brevity, the provisions in this Specification simplify the assessment of whether it is warranted to check for a concrete failure mode. Additionally, if an edge is supported uniformly, as would be common in composite construction, it is assumed that a concrete failure mode will not occur due to the edge condition. Thus, if these provisions are to be used, it is important that it be deemed by the engineer that a concrete breakout failure mode in shear is directly avoided through having the edges perpendicular to the line of force supported, and the edges parallel to the line of force sufficiently distant that concrete breakout through a side edge is not deemed viable. For loading in shear, the determination of whether breakout failure in the concrete is a viable failure mode for the stud anchor is left to the engineer. Alternatively, the provisions call for required anchor reinforcement with provisions comparable to those of ACI 318 Section 17.5.2.9 (ACI, 2014). In addition, the provisions of the applicable building code or ACI 318 Chapter 17 may be used directly to compute the strength of the steel headed stud anchor.

The steel limit states, resistance factors and corresponding safety factors covered in this section match with the corresponding limit states of ACI 318 Chapter 17 (ACI, 2014), although they were assessed independently for these provisions. As only steel limit states are required to be checked if there are no edge conditions, experiments that satisfy the minimum height/diameter ratio but that included failure of the steel headed stud anchor either in the steel or in the concrete were included in the assessment of the resistance and safety factors (Pallarés and Hajjar, 2010a, 2010b).

For steel headed stud anchors subjected to tension or combined shear and tension interaction, it is recommended that anchor reinforcement always be included around the stud to mitigate premature failure in the concrete. If the ratio of the diameter of the head of the stud to the shank diameter is too small, the provisions call for use of ACI 318 Chapter 17 to compute the strength of the steel headed stud anchor. If the distance to the edge of the concrete or the distance to the neighboring anchor is too small, the provisions call for required anchor reinforcement with provisions comparable to those of ACI 318 Section 17.4.2.9 (ACI, 2014). Alternatively, the provisions of the applicable building code or ACI 318 Chapter 17 may also be used directly to compute the strength of the steel headed stud anchor.

CHAPTER J
DESIGN OF CONNECTIONS

The provisions of Chapter J cover the design of connections not subject to cyclic loads. Wind and other environmental loads are generally not considered to be cyclic loads. The provisions generally apply to connections other than HSS and box sections. See Chapter K for provisions specific to HSS and box-section connections, and Appendix 3 for fatigue provisions.

J1. GENERAL PROVISIONS

1. Design Basis

In the absence of defined design loads, a minimum design load should be considered. Historically, a value of 10 kips (44 kN) for LRFD and 6 kips (27 kN) for ASD have been used as reasonable values. For smaller elements such as lacing, sag rods, girts or similar small members, a load more appropriate to the size and use of the part should be used. Both design requirements and construction loads should be considered when specifying minimum loads for connections.

2. Simple Connections

Simple connections are considered in this section and Section B3.4a. In Section B3.4a, simple connections are defined in an idealized manner for the purpose of analysis. The assumptions made in the analysis determine the outcome of the analysis that serves as the basis for design; for connections, that means the force and deformation demands that the connection must resist. This section focuses on the actual proportioning of the connection elements to achieve the required resistance. Thus, Section B3.4a establishes the modeling assumptions that determine the design forces and deformations for use in Section J1.2.

This section and Section B3.4a are not mutually exclusive. If a “simple” connection is assumed for analysis, the actual connection, as finally designed, must perform consistent with that assumption. A simple connection must be able to meet the required rotation and must not introduce strength and stiffness that significantly alters the rotational response.

3. Moment Connections

Two types of moment connections are defined in Section B3.4b: fully restrained (FR) and partially restrained (PR). FR moment connections must have sufficient strength and stiffness to transfer moment and maintain the angle between connected members. PR moment connections are designed to transfer moments but also allow rotation between connected members as the loads are resisted. The response characteristics of a PR connection must be documented in the technical literature or established by analytical or experimental means. The component elements of a PR connection must have sufficient strength, stiffness and deformation capacity to satisfy the design assumptions.

4. Compression Members with Bearing Joints

The provisions in Section J1.4(b), for compression members other than columns finished to bear, are intended to account for member out-of-straightness and also to provide a degree of robustness in the structure to resist unintended or accidental lateral loadings that may not have been considered explicitly in the design.

A provision analogous to that in Section J1.4(b)(1), requiring that splice materials and connectors have an available strength of at least 50% of the required compressive strength, has been in the AISC *Specification* since 1946 (AISC, 1946). The current Specification clarifies this requirement by stating that the force for proportioning the splice materials and connectors is a tensile force. This avoids uncertainty as to how to handle situations where compression on the connection imposes no force on the connectors.

Proportioning the splice materials and connectors for 50% of the required member strength is simple, but can be very conservative. In Section J1.4(b)(2), the Specification offers an alternative that addresses directly the design intent of these provisions. The lateral load of 2% of the required compressive strength of the member simulates the effect of a kink at the splice caused by an end finished slightly out-of-square or other construction condition. Proportioning the connection for the resulting moment and shear also provides a degree of robustness in the structure.

5. Splices in Heavy Sections

Solidified but still hot weld metal contracts significantly as it cools to ambient temperature. Shrinkage of large groove welds between elements that are not free to move so as to accommodate the shrinkage causes strains in the material adjacent to the weld that can exceed the yield point strain. In thick material, the weld shrinkage is restrained in the thickness direction and in the width and length directions causing triaxial stresses to develop that may inhibit the ability to deform in a ductile manner. Under these conditions, the possibility of brittle fracture increases.

The web-to-flange intersection and the web center of heavy hot-rolled shapes, as well as the interior portions of heavy plates, may contain a coarser grain structure and/or lower notch toughness than other areas of these products.

When splicing hot-rolled shapes with flange thickness exceeding 2 in. (50 mm) or heavy welded built-up members, these potentially harmful weld shrinkage strains can be avoided by using bolted splices, fillet-welded lap splices, or splices that combine a welded and bolted detail as seen in Figure C-J1.1. Details and techniques that perform well for materials of modest thickness usually must be changed or supplemented by more demanding requirements when welding thick material.

The provisions of AWS D1.1/D1.1M (AWS, 2015) are minimum requirements that apply to most structural welding situations. However, when designing and fabricating welded splices of hot-rolled shapes with flange thicknesses exceeding 2 in. (50 mm) and similar built-up cross sections, special consideration must be given to all aspects of the welded splice detail:

(1) Notch-toughness requirements are required to be specified for tension members as discussed in Commentary Section A3.1c.

(2) Generously sized weld access holes (see Section J1.6) are required to provide increased relief from concentrated weld shrinkage strains, to avoid close juncture of welds in orthogonal directions, and to provide adequate clearance for the exercise of high quality workmanship in hole preparation, welding, and for ease of inspection.

(3) Preheating for thermal cutting is required to minimize the formation of a hard surface layer. (See Section M2.2.)

(4) Grinding of copes and weld access holes to bright metal to remove the hard surface layer is required.

In addition to tension splices of truss chord members and tension flanges of flexural members, other joints fabricated from heavy sections subject to tension should be given special consideration during design and fabrication.

Alternative details that do not generate shrinkage strains can be used. In connections where the forces transferred approach the member strength, direct welded groove joints may still be the most effective choice.

Until 1999, the Specification mandated that backing bars and weld tabs be removed from all splices of heavy sections. These requirements were deliberately removed, being judged unnecessary and, in some situations, potentially resulting in more harm than good. This Specification still permits the engineer of record to specify their removal when this is judged appropriate.

The previous requirement for the removal of backing bars necessitated, in some situations, that such operations be performed out-of-position; that is, the welding required to restore the backgouged area had to be applied in the overhead position. This may necessitate difficult equipment for gaining access, different welding equipment, processes and/or procedures, and other practical constraints. When box sections

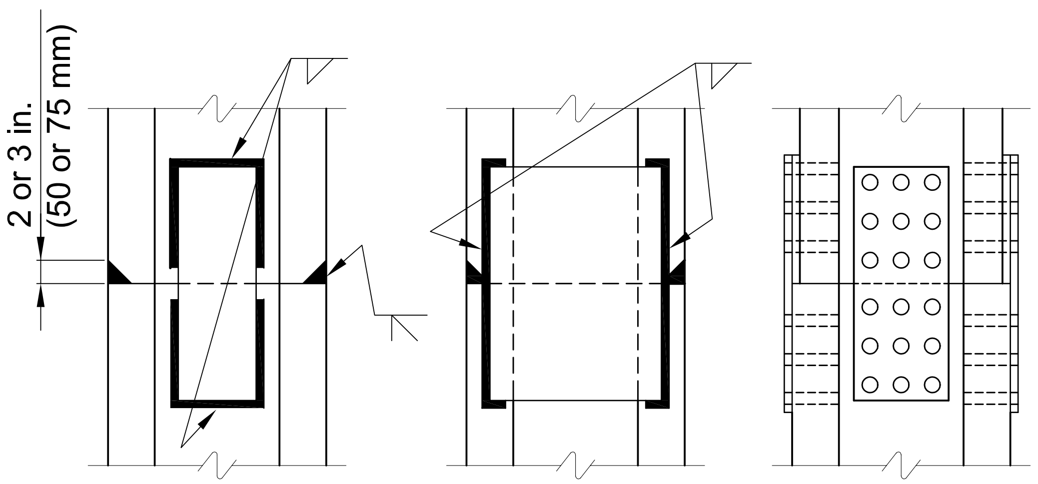


Fig. C-J1.1. Alternative splices that minimize weld restraint tensile stresses.

made of plate are spliced, access to the interior side, necessary for backing removal, is typically impossible.

Weld tabs that are left in place on splices act as "short attachments" and attract little stress. Even though it is acknowledged that weld tabs might contain regions of inferior quality weld metal, the stress concentration effect is minimized since little stress is conducted through the attachment.

Previous editions of this Specification required magnetic particle or dye-penetrant inspection of thermally cut weld access holes for splices in heavy sections. This requirement was deliberately removed as anecdotal evidence suggested this inspection was not necessary because cracks from thermal cutting rarely occurred when the other Specification requirements were met. The previously prescribed magnetic particle testing or penetrant testing was replaced with a requirement for visual inspection of weld access holes after welding (see Table N5.4-3).

6. Weld Access Holes

Weld access holes are frequently required in the fabrication of structural components. The geometry of these structural details can affect the components' performance. The size and shape of beam copes and weld access holes can have a significant effect on the ease of depositing sound weld metal, the ability to conduct nondestructive examinations, and the magnitude of the stresses at the geometric discontinuities produced by these details.

Weld access holes used to facilitate welding operations are required to have a minimum length from the toe of the weld preparation (see Figure C-J1.2) equal to 1.5 times the thickness of the material in which the hole is made. This minimum length is expected to accommodate a significant amount of the weld shrinkage strains at the web-to-flange intersection.

The height of the weld access hole must provide sufficient clearance for ease of welding and inspection and must be large enough to allow the welder to deposit sound weld metal through and beyond the web. A weld access hole height equal to 1.0 times the thickness of the material with the access hole, but not less than $^{3}/_{4}$ in. (19 mm), has been judged to satisfy these welding and inspection requirements. The height of the weld access hole need not exceed 2 in. (50 mm).

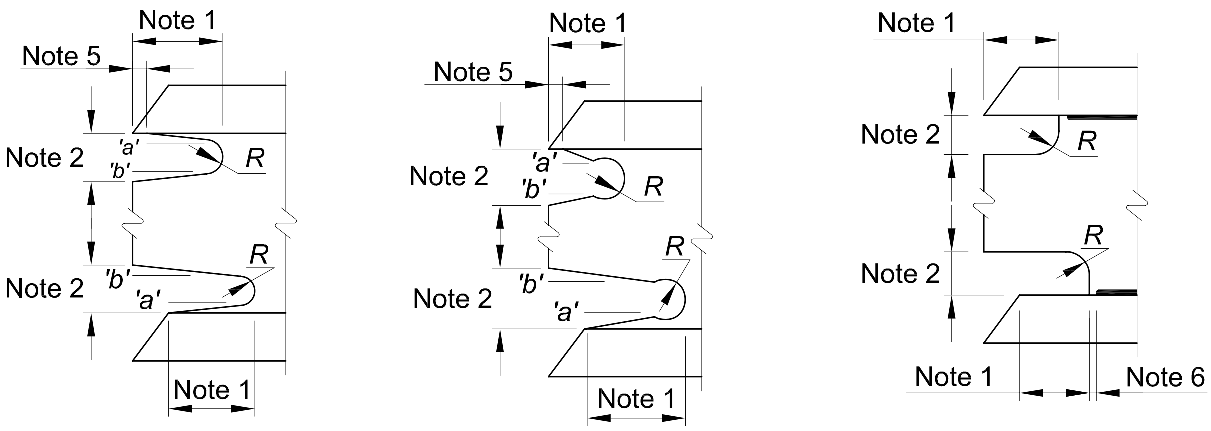
The geometry of the reentrant corner between the web and the flange determines the level of stress concentration at that location. A 90° reentrant corner having a very small radius produces a very high stress concentration that may lead to rupture of the flange. Consequently, to minimize the stress concentration at this location, the edge of the web is sloped or curved from the surface of the flange to the reentrant surface of the weld access hole.

Stress concentrations along the perimeter of weld access holes also can affect the performance of the joint. Consequently, weld access holes are required to be free of stress raisers such as notches and gouges. The NDT requirement of access holes in earlier editions of the Specification has been removed in response to reports that these examinations had revealed no defects.

Stress concentrations at web-to-flange intersections of built-up shapes can be decreased by terminating the weld away from the access hole. Thus, for built-up shapes with fillet welds or partial-joint-penetration groove welds that join the web to the flange, the weld access hole may terminate perpendicular to the flange, provided that the weld is terminated a distance equal to or greater than one weld size away from the access hole.

7. Placement of Welds and Bolts

Slight eccentricities between the gravity axis of single- and double-angle members and the center of gravity of connecting bolts or rivets have long been ignored as having negligible effect on the static strength of such members. Tests have shown that similar practice is warranted in the case of welded members in statically loaded structures (Gibson and Wake, 1942).



Alternate 1 Alternate 2 Alternate 3

Rolled shapes and built-up shapes assembled prior to cutting the weld access hole.

Built-up shapes assembled after cutting the weld access hole.

Notes: These are typical details for joints welded from one side against steel backing. Alternative details are discussed in the commentary text.

1. Length: Greater of $1.5t_w$ or $1^1/_2$ in. (38 mm)
2. Height: Greater of $1.0t_w$ or $^3/_4$ in. (19 mm) but need not exceed 2 in. (50 mm)
3. *R*: $^3/_8$ in. min. (10 mm). Grind the thermally cut surfaces of weld access holes in heavy shapes as defined in Sections A3.1(c) and (d).
4. Slope *'a'* forms a transition from the web to the flange. Slope *'b'* may be horizontal.
5. The bottom of the top flange is to be contoured to permit the tight fit of backing bars where they are to be used.
6. The web-to-flange weld of built-up members is to be held back a distance of at least the weld size from the edge of the access hole.

Fig. C-J1.2. Weld access hole geometry.

However, the fatigue life of eccentrically loaded welded angles has been shown to be very short (Klöppel and Seeger, 1964). Notches at the roots of fillet welds are harmful when alternating tensile stresses are normal to the axis of the weld, as could occur due to bending when axial cyclic loading is applied to angles with end welds not balanced about the neutral axis. Accordingly, balanced welds are required when such members are subjected to cyclic loading (see Figure C-J1.3).

8. Bolts in Combination with Welds

As in previous editions, this Specification does not permit bolts or rivets to share the load with welds except for conditions where shear is resisted at the faying surface. In joints where the strength is based on the strength of bolts and welds acting together, the compatibility of deformations of the various components of the connection at the ultimate load level are important factors in determining the connection strength. Physical tests (Kulak and Grondin, 2003) and finite element models (Shi et al., 2011) have shown that bolts designed as part of a slip-critical connection and properly tightened according to the requirements for a slip-critical connection can share the load with longitudinal fillet welds, provided a reasonable proportion of the load is carried by each. The limits established are 50% minimum for the welds and 33% minimum for the high-strength bolts. The strength of transverse welds is not permitted to be included with the strength of bolts because these welds have less ductility. The provisions of this section are generally intended to be applied in cases where retrofit work is required to accommodate higher design loads, or cases where the mean slip coefficient in the field may not have complied with the value assumed in the design [special testing is required according to Appendix A of the RCSC *Specification* (RCSC, 2014) in such cases to validate the slip coefficient, μ, value used in the final retrofitted design].

The intent of this 2016 provision as prescribed in the second paragraph is to ensure the combined joint will provide the required strength just prior to when the welds fracture, which defines the ultimate load level. The ultimate load is defined by the

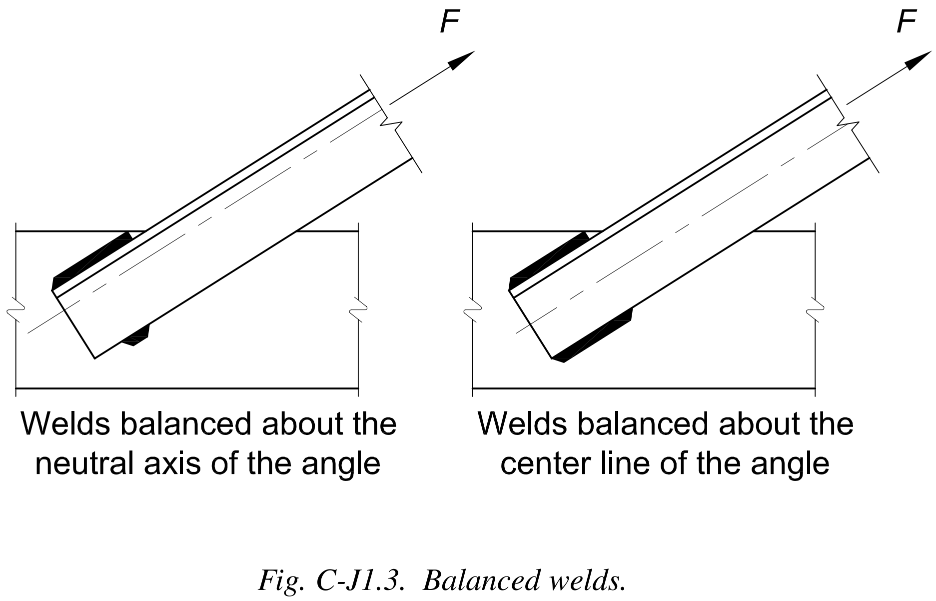


Fig. C-J1.3. Balanced welds.

capacity of the welds and the slip resistance from the bolt pretension clamping force. No additional bearing or tearout capacity check is required. The use of a single resistance factor ($\phi = 0.75$) or safety factor ($\Omega = 2.00$) on the nominal strength of the bolts and welds combined is intended to improve the reliability of the connection compared to the use of the higher resistance factor ($\phi = 1.00$) and lower safety factor ($\Omega = 1.50$) permitted for standard holes in slip-critical bolted connections alone. For existing connections with high-strength bolts originally tightened by other methods than turn-of-nut, an additional $^1/_3$ turn for ASTM F3125 Grades A325 or A325M and $^1/_2$ turn for Grades A490 or A490M bolts would allow the bolts to be considered pretensioned by turn-of-nut relative to this section. Over-rotation of a bolt is not cause for rejection per the RCSC *Specification.* The additional rotation may occasionally result in bolt rupture which will occur at the time the bolts are rotated. Broken bolts can be replaced with equivalent bolts installed using the turn-of-nut method. Note that the connection strength need not be taken as less than the strength of the bolts alone or the strength of the welds alone. The heat of welding near bolts will not alter the mechanical properties of the bolts.

The restrictions on bolts in combination with welds do not apply to typical bolted/welded beam-to-girder and beam-to-column connections, and other comparable connections where the bolts and welds are used on separate faying surfaces (Kulak et al., 1987).

10. High-Strength Bolts in Combination with Rivets

When high-strength bolts are used in combination with rivets, the ductility of the rivets permits the direct addition of the strengths of the two fastener types.

J2. WELDS

Selection of weld type [complete-joint-penetration (CJP) groove weld versus fillet versus partial-joint-penetration (PJP) groove weld] depends on base connection geometry (butt versus T or corner), in addition to required strength, and other issues discussed in the following. Notch effects and the ability to evaluate with nondestructive testing may affect joint selection for cyclically loaded joints or joints expected to deform plastically.

1. Groove Welds

1a. Effective Area

Tables J2.1 and J2.2 show that the effective throat of PJP and flare groove welds is dependent upon the weld process and the position of the weld. It is recommended that the design drawings show either the required strength or the required effective throat size and allow the fabricator to select the process and determine the position required to meet the specified requirements. Effective throats larger than those in Table J2.2 can be qualified by tests. Weld reinforcement is not used in determining the effective throat of a groove weld, but reinforcing fillets on T- and corner-joints are accounted for in the effective throat. See AWS D1.1/D1.1M Annex A (AWS, 2015).

1b. Limitations

Table J2.3 gives the minimum effective throat thickness of a PJP groove weld. Notice that for PJP groove welds Table J2.3 goes up to a plate thickness of over 6 in. (150 mm) and a minimum weld throat of $^5/_8$ in. (16 mm), whereas for fillet welds Table J2.4 goes up to a plate thickness of over $^3/_4$ in. (19 mm) and a minimum leg size of fillet weld of only $^5/_{16}$ in. (8 mm). The additional thickness for PJP groove welds is intended to provide for reasonable proportionality between weld and material thickness. The use of single-sided PJP groove welds in joints subject to rotation about the toe of the weld is discouraged.

2. Fillet Welds

2a. Effective Area

The effective throat of a fillet weld does not include the weld reinforcement, nor any penetration beyond the weld root. Some welding procedures produce a consistent penetration beyond the root of the weld. This penetration contributes to the strength of the weld. However, it is necessary to demonstrate that the weld procedure to be used produces this increased penetration. In practice, this can be done initially by cross-sectioning the runoff tabs of the joint. Once this is done, no further testing is required, as long as the welding procedure is not changed.

2b. Limitations

Table J2.4 provides the minimum size of a fillet weld for a given thickness of the thinner part joined. The requirements are not based on strength considerations, but on the quench effect of thick material on small welds. Very rapid cooling of weld metal may result in a loss of ductility. Furthermore, the restraint to weld metal shrinkage provided by thick material may result in weld cracking.

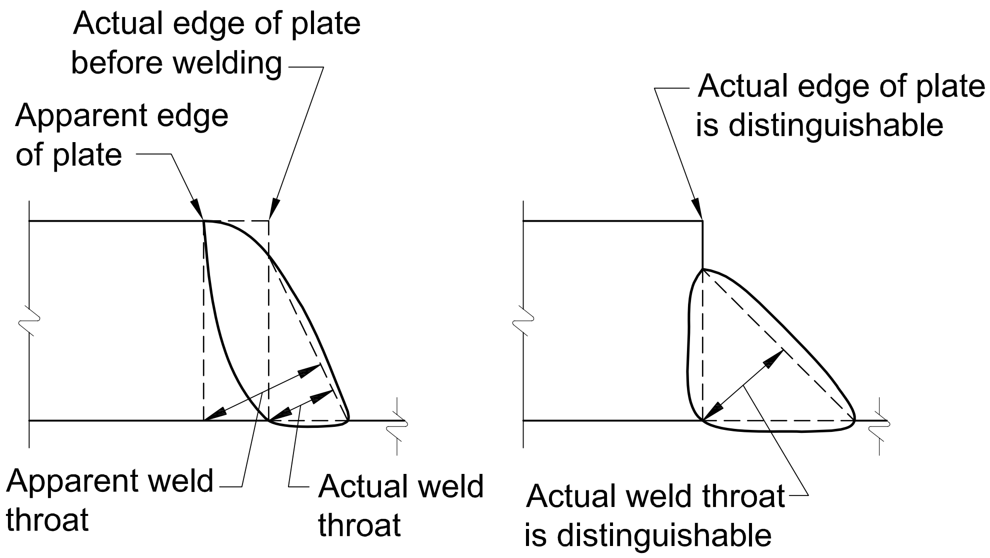
The use of the thinner part to determine the minimum size weld is based on the prevalence of the use of filler metal considered to be "low hydrogen." Because a $^5/_{16}$-in. (8 mm) fillet weld is the largest that can be deposited in a single pass by the SMAW process and still be considered prequalified under AWS D1.1/D1.1M (AWS, 2015), $^5/_{16}$ in. (8 mm) applies to all material greater than $^3/_4$ in. (19 mm) in thickness, but minimum preheat and interpass temperatures are required by AWS D1.1/D1.1M. The design drawings should reflect these minimum sizes and the production welds should be of these minimum sizes.

For thicker members in lap joints, it is possible for the welder to melt away the upper corner, resulting in a weld that appears to be full size but actually lacks the required weld throat dimension. See Figure C-J2.1(a). On thinner members, the full weld throat is likely to be achieved, even if the edge is melted away. Accordingly, when the plate is $^1/_4$ in. (6 mm) or thicker, the maximum fillet weld size is $^1/_{16}$ in. (2 mm) less than the plate thickness, t, which is sufficient to ensure that the edge remains. See Figure C-J2.1(b).

By providing a minimum lap of five times the thickness of the thinner part of a lap joint, the resulting rotation of the joint when pulled will not be excessive, as shown in Figure C-J2.2, where the condition shown in the righthand figure subjects the fillet weld to torsion. Fillet welded lap joints under tension tend to open and apply a tearing action at the root of the weld as shown in Figure C-J2.3(b), unless restrained by a force, *F*, as shown in Figure C-J2.3(a). The minimum length reduces stresses due to Poisson effects.

The use of single-sided fillet welds in joints subject to rotation around the toe of the weld is discouraged. End returns are not essential for developing the full length of fillet welded connections and have a negligible effect on their strength. Their use has been encouraged to ensure that the weld size is maintained over the length of the weld, to enhance the fatigue resistance of cyclically loaded flexible end connections, and to increase the plastic deformation capability of such connections.

The weld strength database on which the Specification was developed had no end returns. This includes the study reported in Higgins and Preece (1968), the seat angle tests in Lyse and Schreiner (1935), the seat and top angle tests in Lyse and Gibson (1937), the tests on beam webs welded directly to a column or girder by fillet welds in Johnston and Deits (1942), and the tests on eccentrically loaded welded connections reported by Butler et al. (1972). Hence, the current strength values and joint design



(a) Incorrect for t ≥ $^1/_4$ *in.* *(b) Correct for* t ≥ $^1/_4$ *in.*

Fig. C-J2.1. Identification of plate edge.

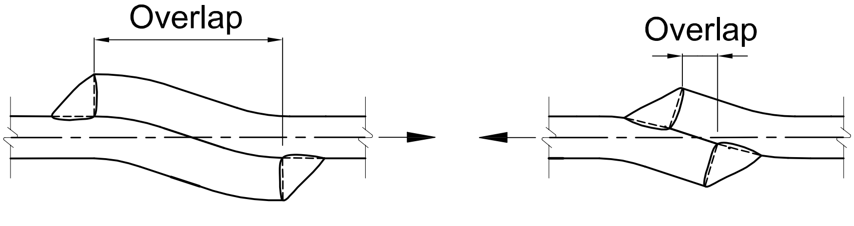
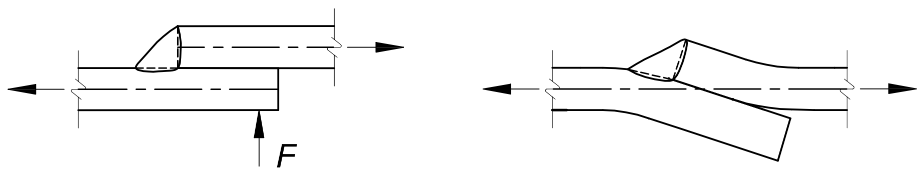


Fig. C-J2.2. Minimum lap.

models do not require end returns when the required weld size is provided. Johnston and Green (1940) noted that movement consistent with the design assumption of no end restraint (in other words, joint flexibility) was enhanced without end returns. They also verified that greater plastic deformation of the connection was achieved when end returns existed, although the strength was not significantly different.

When longitudinal fillet welds parallel to the stress are used to transmit the load to the end of an axially loaded member, the welds are termed "end loaded." Typical examples of such welds include, but are not limited to (a) longitudinally welded lap joints at the end of axially loaded members, (b) welds attaching bearing stiffeners, and (c) similar cases. Typical examples of longitudinally loaded fillet welds that are not considered end loaded include, but are not limited to (a) welds that connect plates or shapes to form built-up cross sections in which the shear force is applied to each increment of length of weld depending upon the distribution of the shear along the length of the member, and (b) welds attaching beam web connection angles and shear plates because the flow of shear force from the beam or girder web to the weld is essentially uniform throughout the weld length; that is, the weld is not end-loaded despite the fact that it is loaded parallel to the weld axis. Neither does the reduction coefficient, β, apply to welds attaching stiffeners to webs because the stiffeners and welds are not subject to calculated axial stress but merely serve to keep the web flat.

The distribution of stress along the length of end-loaded fillet welds is not uniform and is dependent upon complex relationships between the stiffness of the longitudinal fillet weld relative to the stiffness of the connected materials. Experience has shown that when the length of the weld is equal to approximately 100 times the weld size or less, it is reasonable to assume that the full length is effective. For weld lengths greater than 100 times the weld size, the effective length should be taken less than the actual length. The reduction factor, β, provided in Section J2.2b is the equivalent to that given in CEN (2005a), which is a simplified approximation of exponential formulas developed by finite element studies and tests performed in Europe over many years. The provision is based on the combined consideration of the nominal strength for fillet welds with leg size less than $^1/_4$ in. (6 mm) and of a judgment-based serviceability limit of slightly less than $^1/_{32}$ in. (1 mm) displacement at the end of the weld for welds with leg size $^1/_4$ in. (6 mm) and larger. Given the empirically derived mathematical form of the β factor, as the ratio of weld length to weld size, w, increases beyond 300, the effective length of the weld begins to decrease, illogically causing a weld of greater length to have progressively less strength. Therefore, the effective length is taken as $0.6(300)w = 180w$ when the weld length is greater than 300 times the leg size.



(a) Restrained *(b) Unrestrained*

Fig. C-J2.3. Restraint of lap joints.

In most cases, fillet weld terminations do not affect the strength or serviceability of connections. However, in certain cases the disposition of welds affect the planned function of the connection, and notches may affect the static strength and/or the resistance to crack initiation if cyclic loads of sufficient magnitude and frequency occur. For these cases, termination details at the end of the joint are specified to provide the desired profile and performance. In cases where profile and notches are less critical, terminations are permitted to run to the end. In most cases, stopping the weld short of the end of the joint will not reduce the strength of the weld. The small loss of weld area due to stopping the weld short of the end of the joint by one to two weld sizes is not typically considered in the calculation of weld strength. Only short weld lengths will be significantly affected by this.

The following situations require special attention:

(1) For lapped joints where one part extends beyond the end or edge of the part to which it is welded and if the parts are subject to calculated tensile stress at the start of the overlap, it is important that the weld terminate a short distance from the stressed edge. For one typical example, the lap joint between the tee chord and the web members of a truss, the weld should not extend to the edge of the tee stem (see Figure C-J2.4). The best technique to avoid inadvertent notches at this critical location is to strike the welding arc at a point slightly back from the edge and proceed with welding in the direction away from the edge (see Figure C-J2.5). Where framing angles extend beyond the end of the beam web to which they are welded, the free end of the beam web is subject to zero stress; thus, it is permissible for the fillet weld to extend continuously across the top end, along the side, and along the bottom end of the angle to the extreme end of the beam (see Figure C-J2.6).

(2) For connections such as framing angles and framing tees, which are assumed in the design of the structure to be flexible connections, the tension edges of the outstanding legs or flanges must be left unwelded over a substantial portion of their

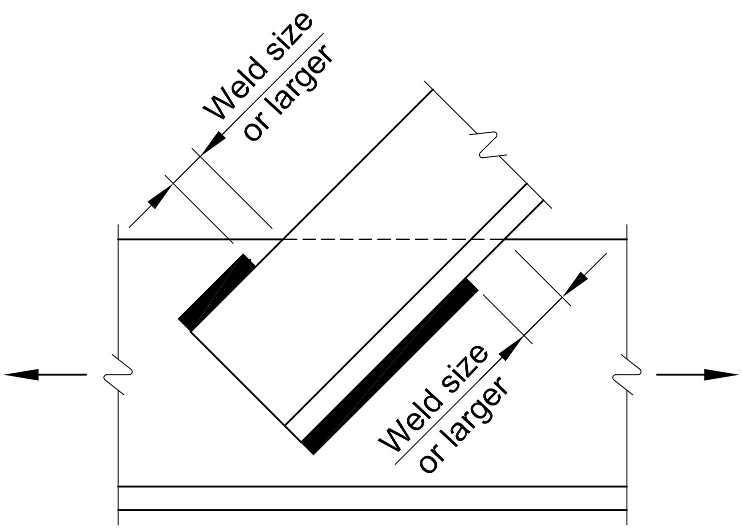


Fig. C-J2.4. Fillet welds near tension edges.

length to provide flexibility in the connection. Tests have shown that the static strength of the connection is the same with or without end returns; therefore, the use of returns is optional, but if used, their length must be restricted to not more than four times the weld size (Johnston and Green, 1940) (see Figure C-J2.7).

(3) Experience has shown that when ends of intermediate transverse stiffeners on the webs of plate girders are not welded to the flanges (the usual practice), small torsional distortions of the flange occur near shipping bearing points in the normal course of shipping by rail or truck and may cause high out-of-plane bending stresses (up to the yield point) and fatigue cracking at the toe of the web-to-flange welds. This has been observed even with closely fitted stiffeners. The

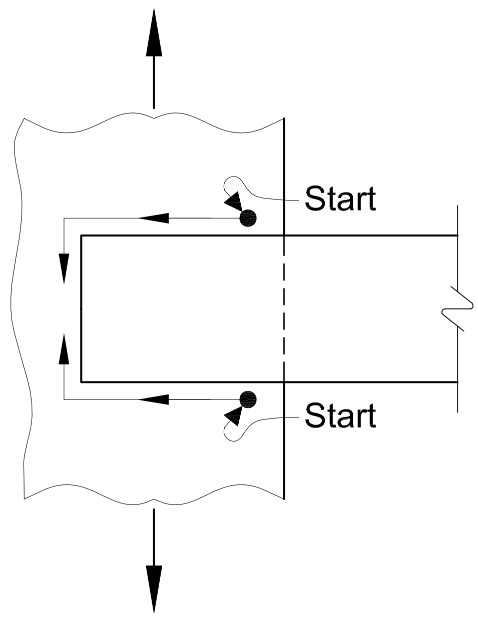


Fig. C-J2.5. Suggested direction of welding travel to avoid notches.

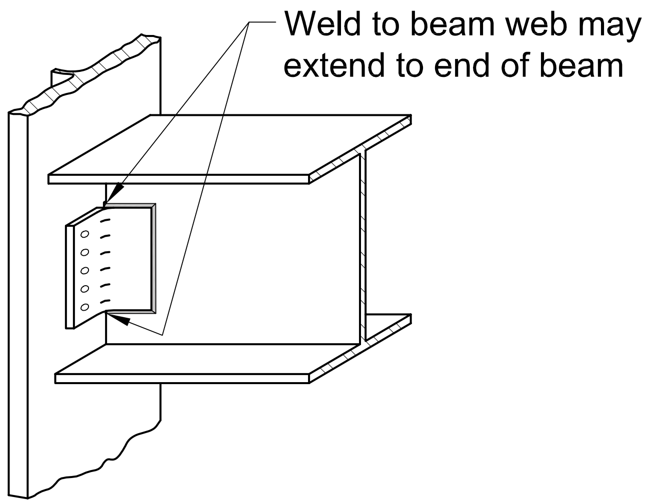


Fig. C-J2.6. Fillet weld details on framing angles.

intensity of these out-of-plane stresses may be effectively limited and cracking prevented if "breathing room" is provided by terminating the stiffener weld away from the web-to-flange welds. The unwelded distance should not exceed six times the web thickness so that column buckling of the web within the unwelded length does not occur.

(4) For fillet welds that occur on opposite sides of a common plane, it is difficult to deposit a weld continuously around the corner from one side to the other without causing a gouge in the corner of the parts joined; therefore, the welds must be interrupted at the corner (see Figure C-J2.8). AWS D1.1/D1.1M (AWS, 2015) added a specific exception that permits continuous welds around opposite sides of a common plane where the engineer requires sealed joints.

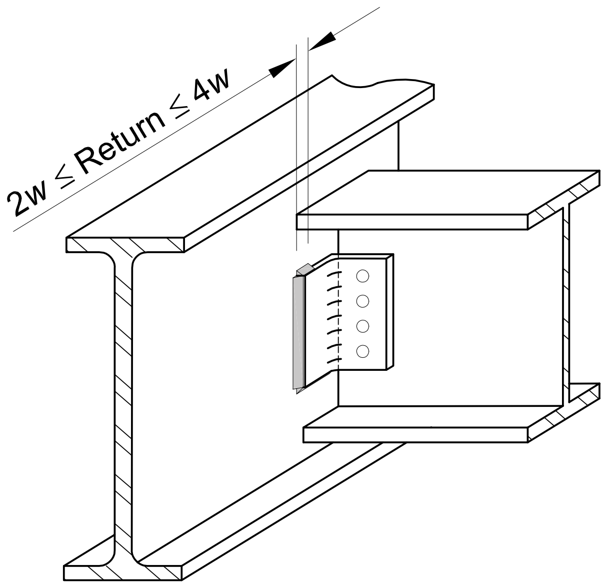


Fig. C-J2.7. Flexible connection returns optional unless subject to fatigue.

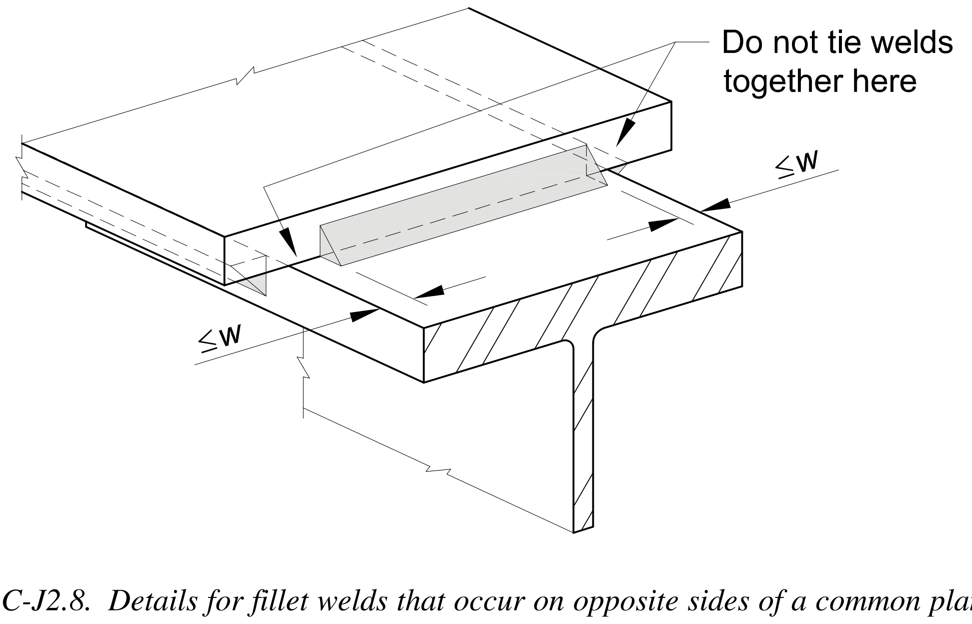


Fig. C-J2.8. Details for fillet welds that occur on opposite sides of a common plane.

3. Plug and Slot Welds

A plug weld is a weld made in a circular hole in one member of a joint fusing that member to another member. A slot weld is a weld made in an elongated hole in one member of a joint fusing that member to another member. Both plug and slot welds are only applied to lap joints. Care should be taken when plug or slot welds are applied to structures subject to cyclic loading as the fatigue performance of these welds is limited.

A fillet weld inside a hole or slot is not a plug weld. A "puddle weld," typically used for joining decking to the supporting steel, is not the same as a plug weld.

3a. Effective Area

When plug and slot welds are detailed in accordance with Section J2.3b, the strength of the weld is controlled by the size of the fused area between the weld and the base metal. The total area of the hole or slot is used to determine the effective area.

3b. Limitations

Plug and slot welds are limited to situations where they are loaded in shear, or where they are used to prevent elements of a cross section from buckling, such as for web doubler plates on deeper rolled sections. Plug and slot welds are only allowed where the applied loads result in shear between the joined materials—they are not to be used to resist direct tensile loads. This restriction does not apply to fillets in holes or slots.

The geometric limitations on hole and slot sizes are prescribed in order to provide a geometry that is conducive to good fusion. Deep, narrow slots and holes make it difficult for the welder to gain access and see the bottom of the cavity into which weld metal must be placed. Where access is difficult, fusion may be limited, and the strength of the connection reduced.

4. Strength

The strength of welds is governed by the strength of either the base material or the deposited weld metal. Table J2.5 presents the nominal weld strengths and the ϕ and Ω factors, as well as the limitations on filler metal strength levels.

The strength of a joint that contains a complete-joint-penetration (CJP) groove weld, whether loaded in tension or compression, is dependent upon the strength of the base metal, and no computation of the strength of the CJP groove weld is required. For tension applications, matching strength filler metal is required, as defined in AWS D1.1/D1.1M Table 3.1 (AWS, 2015). For compression applications, up to a 10 ksi (69 MPa) decrease in filler metal strength is permitted, which is equivalent to one strength level.

CJP groove welds loaded in tension or compression parallel to the weld axis, such as for the groove welded corners of box columns, do not transfer primary loads across the joint. In cases such as this, no computation of the strength of the CJP groove weld strength is required.

CJP groove welded tension joints are intended to provide strength equivalent to the base metal; therefore, matching filler metal is required. CJP groove welds have been shown not to exhibit compression failure even when they are undermatched. The amount of undermatching before unacceptable deformation occurs has not been established, but one standard strength level is conservative and therefore permitted. Joints in which the weld strength is calculated based on filler metal classification strength can be designed using any filler metal strength equal to or less than matching. Filler metal selection is still subject to compliance with AWS D1.1/D1.1M.

The nominal strength of partial-joint-penetration (PJP) groove welded joints in compression is higher than for other joints because compression limit states are not observed on weld metal until significantly above the yield strength.

Connections that contain PJP groove welds designed to bear in accordance with Section J1.4(b), and where the connection is loaded in compression, are not limited in strength by the weld since the surrounding base metal can transfer compression loads. When not designed in accordance with Section J1.4(b), an otherwise similar connection must be designed considering the possibility that either the weld or the base metal may be the critical component in the connection.

The factor of 0.6 on F_{EXX} for the tensile strength of PJP groove welds has been used since the early 1960s to compensate for factors such as the notch effect of the unfused area of the joint and uncertain quality in the root of the weld due to the difficulty in performing nondestructive evaluation. It does not imply that the tensile failure mode is by shear stress on the effective throat, as in fillet welds.

Column splices have historically been connected with relatively small PJP groove welds. Frequently, erection aids are available to resist construction loads. Columns are intended to be in bearing in splices and on base plates. Section M4.4 recognizes that, in the as-fitted product, the contact may not be consistent across the joint and therefore provides rules assuring some contact that limits the potential deformation of weld metal and the material surrounding it. These welds are intended to hold the columns in place, not to transfer the compressive loads. Additionally, the effects of very small deformation in column splices are accommodated by normal construction practices. Similarly, the requirements for base plates and normal construction practice assure some bearing at bases. Therefore, the compressive stress in the weld metal does not need to be considered, as the weld metal will deform and subsequently stop when the columns bear.

Other PJP groove welded joints connect members that may be subject to unanticipated loads and may fit with a gap. Where these connections are finished to bear, fit-up may not be as good as that specified in Section M4.4, but some bearing is anticipated and the weld is designed to resist loads defined in Section J1.4(b) using the factors, strengths and effective areas in Table J2.5. Where the joints connect members that are not finished to bear, the welds are designed for the total load using the available strengths and areas in Table J2.5.

In Table J2.5, the nominal strength of fillet welds is determined from the effective throat area, whereas the strengths of the connected parts are governed by their respective thicknesses. Figure C-J2.9 illustrates the shear planes for fillet welds and base material:

(1) Plane 1-1, in which the strength is governed by the shear strength of material A

(2) Plane 2-2, in which the strength is governed by the shear strength of the weld metal

(3) Plane 3-3, in which the strength is governed by the shear strength of material B

The strength of the welded joint is the lowest of the strengths calculated in each plane of shear transfer. Note that planes 1-1 and 3-3 are positioned away from the fusion areas between the weld and the base material. Tests have demonstrated that the stress on this fusion area is not critical in determining the shear strength of fillet welds (Preece, 1968).

The shear planes for plug and PJP groove welds are shown in Figure C-J2.10 for the weld and base metal. Generally, the base metal will govern the shear strength.

The instantaneous center of rotation method is a valid approach to calculate the strength of weld groups consisting of elements oriented in various directions relative to the load. The instantaneous center of rotation method considers strain compatibility among the elements in the weld group. Aspects of the method were previously included in the Specification. These aspects along with a more comprehensive explanation of the method are discussed in the AISC *Steel Construction Manual* (AISC, 2011).

5. Combination of Welds

When determining the strength of a combination PJP groove weld and fillet weld contained within the same joint, the total throat dimension is not the simple addition of the fillet weld throat and the groove weld throat. In such cases, the resultant throat of the combined weld (shortest dimension from the root to face of the final weld) must be determined and the design based upon this dimension.

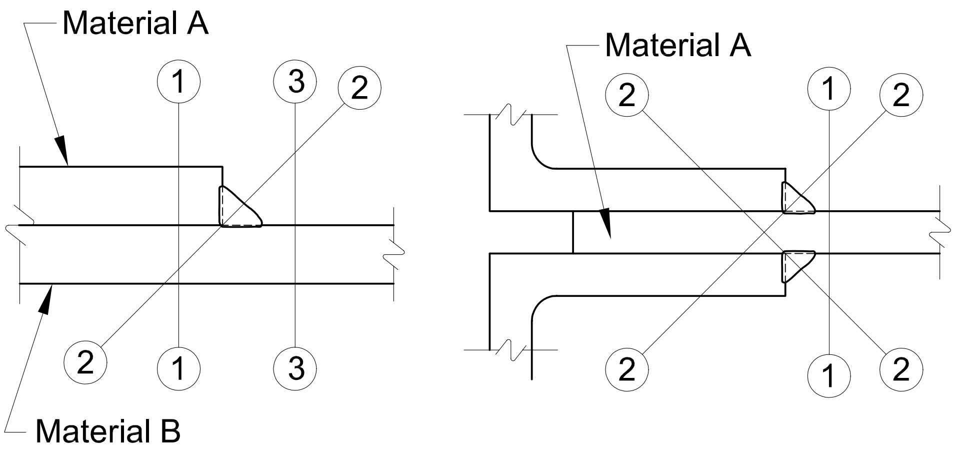
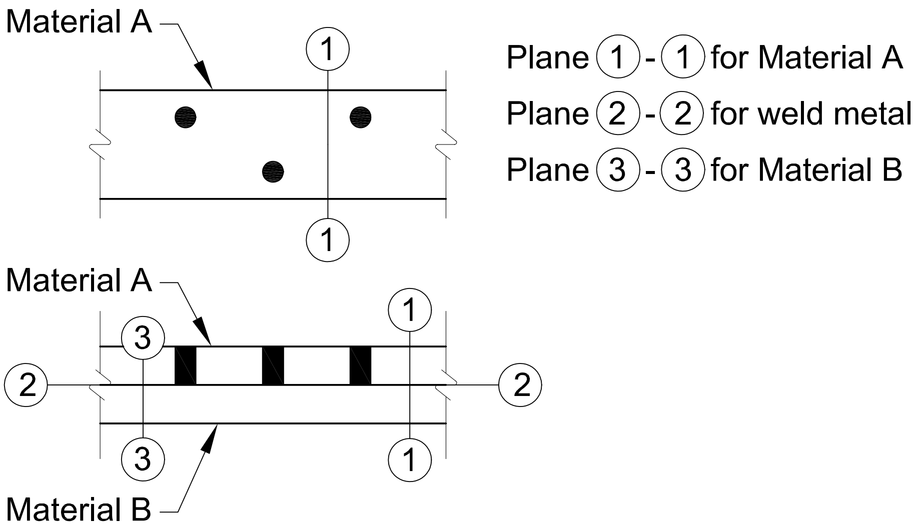


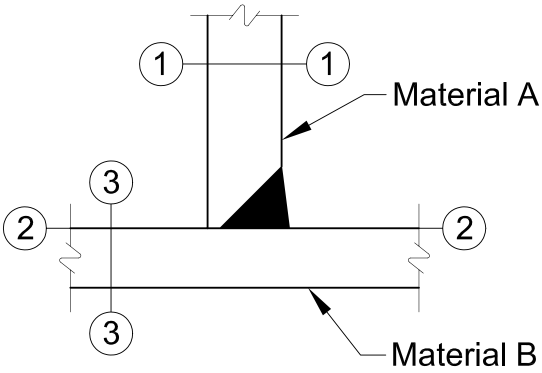
Fig. C-J2.9. Shear planes for fillet welds loaded in longitudinal shear.

6. Filler Metal Requirements

Applied and residual stresses and geometrical discontinuities from backing bars with associated notch effects contribute to sensitivity to fracture. Additionally, some weld metals in combination with certain procedures result in welds with low notch toughness. Accordingly, this Specification requires a minimum specified toughness for weld metals in those joints that are subject to more significant applied stresses and toughness demands. The level of toughness required is selected as one level more conservative than the base metal requirement for hot-rolled shapes with a flange thickness exceeding 2 in. (50 mm).



(a) Plug welds



(b) Partial-joint-penetration groove welds

Fig. C-J2.10. Shear planes for plug and partial-joint-penetration groove welds.

7. Mixed Weld Metal

Problems can occur when incompatible weld metals are used in combination and notch-tough composite weld metal is required. For instance, tack welds deposited using a self-shielded process with aluminum deoxidizers in the electrodes and subsequently covered by SAW weld passes can result in a composite weld metal with low notch-toughness, despite the fact that each process by itself could provide notch-tough weld metal.

Potential concern about intermixing weld metal types is limited to situations where one of the two weld metals is deposited by the self-shielded flux-cored arc welding (FCAW-s) process. Changes in tensile and elongation properties have been demonstrated to be of insignificant consequence. Notch toughness is the property that can be affected the most. Many compatible combinations of FCAW-s and other processes are commercially available.

J3. BOLTS AND THREADED PARTS

1. High-Strength Bolts

In general, except as provided in this Specification, the use of high-strength bolts is required to conform to the provisions of the *Specification for Structural Joints Using High-Strength Bolts* (RCSC, 2014) as approved by the Research Council on Structural Connections. Kulak (2002) provides an overview of the properties and use of high-strength bolts.

Provisions in this Specification vary from the RCSC *Specification* as follows:

(a) RCSC *Specification* limits bolt grades to ASTM A325, A325M, A490 and A490M. This Specification allows bolt grades of ASTM F3125 Grades A325, A325M, A490, A490M, F1852 and F2280.

(b) This Specification also allows the use of ASTM F3043, F3111, A354 Grade BC, A354 Grade BD, and A449 bolts.

(c) This Specification designates the following bolt groups:

  (1) Group A: ASTM F3125 Grades A325, A325M, F1852 and ASTM A354 Grade BC

  (2) Group B: ASTM F3125 Grades A490, A490M, F2280 and ASTM A354 Grade BD

  (3) Group C: ASTM F3043 and F3111

(d) Bolt hole sizes listed in RCSC *Specification* Table 3.1 are as listed in this Specification Table J3.3.

(e) Bolt strengths listed in RCSC *Specification* Table 5.1 are as listed in this Specification Table J3.2.

(f) Minimum bolt pretensions listed in RCSC *Specification* Table 8.1 are as listed in this Specification Table J3.1.

(g) RCSC *Specification* Section 5.2 shall be replaced with Section J3.7 of this Specification.

Occasionally the need arises for the use of high-strength bolts of diameters in excess of those permitted for ASTM F3125 Grades A325 or A325M and Grades A490 or A490M bolts (or lengths exceeding those available in these grades). For joints requiring diameters in excess of $1^1/_2$ in. (38 mm) or lengths in excess of about 8 in. (200 mm), Section J3.1 permits the use of ASTM A449 bolts and ASTM A354 Grade BC and BD threaded rods. When ASTM A354 or A449 bolts are to be pretensioned they must have geometry matching that of A325 (A325M) or A490 (A490M) bolts. The fastener dimensions should be specified as heavy hex structural bolts and the threads specified as Unified Coarse Thread Series with Class 2A tolerances per ASME B18.2.6 (ASME, 2010). The minimum tensile strength of ASTM A449 bolts reduces for bolts greater than one inch (25 mm) in diameter and again for bolts greater than $1^1/_2$ in. (38 mm) in diameter. Therefore, these bolts should be designed as threaded parts in Table J3.2. Note that anchor rods are more preferably specified as ASTM F1554 material. Fasteners made of materials with 150-ksi (1030 MPa) tensile strength or higher, such as ASTM A354 Grade BD, and pretensioned to near the yield strength may be susceptible to hydrogen embrittlement. Designers are cautioned to evaluate the effects of galvanizing and of threads rolled after heat treatment. Some exposures can increase susceptibility. ASTM A143 (ASTM, 2014) includes some information helpful in reducing internal hydrogen embrittlement. External hydrogen embrittlement should also be considered.

High-strength bolts have been grouped by strength levels into three categories:

(1) Group A bolts, which have a strength similar to ASTM F3125 Grade A325 bolts

(2) Group B bolts, which have a strength similar to ASTM F3125 Grade A490 bolts

(3) Group C bolts, which are 200-ksi (830 MPa) strength as in ASTM F3111 bolts

Group C fastener assemblies have been added in this Specification. They are based upon fastener assemblies of strength designation Grade 14.9 [200-ksi (1400 MPa) tensile strength] used in building structures in Japan. The bolt steel is produced to minimize risk of internal hydrogen embrittlement, with bolt design features to minimize stress and strain concentrations including increased radius under the bolt head, a shank transition near the threads, and a wider, smoother radius at the thread root. Basic head, shank and nut dimensions are compatible with installation tools used for Group A and Group B fasteners, as specified in ASME B18.2.6 (ASME, 2010). Group C Grade 1 assemblies use an ASME B1.15 UNJ thread root profile, and Grade 2 assemblies use a proprietary thread root profile (ASME, 1995).

The use of Group C fasteners is limited to applications and locations that would not subject the fastener assembly to environmental hydrogen embrittlement. Use is intended for building interiors that are normally dry, including where the structural steel is embedded in concrete, encased in masonry, or protected by membrane or noncorrosive contact type fireproofing, as well as for building interiors and exteriors that are normally dry and under roof with the installed assemblies soundly protected by a shop-applied or field-applied coating to the structural steel system. Use is not intended for the following: (1) structural steel framing not under roof; (2) chemical or heavy industrial environments where strong concentrations of highly corrosive gases, fumes or chemicals, either in solution or as concentrated liquids or solids, contact the fasteners or the structural steel coating system; (3) locations with high humidity environments maintaining almost continuous condensation; (4) locations submerged in water or soil; or, (5) cathodically protected environments where current is applied to the structural steel system by the sacrificial anode method or the DC power method.

Group C Grade 2 fasteners have been subjected to testing to validate the prescribed pretensioning methods, and have their thread root profile performance validated by successful performance in numerous projects. Group C Grade 1 fasteners, as of the date of this standard, have not been subjected to testing to validate the prescribed pretensioning methods, and have not been tested to validate that the sharper UNJ thread root profile is adequate for performance in a pretensioned application. Therefore, Grade 2 fasteners are permitted to be used in snug-tight, pretensioned and slip-critical joints, and Grade 1 fasteners are restricted to use in the snug-tight condition.

The Group C transition shank cross-sectional area approximates the tensile stress area of the bolt. The tensile stress area of the Grade 2 assembly is approximately 4% greater than that for Grade 1. For simplicity, the nominal shear strength for transition shank or threads included in the shear plane is based upon 80% of the full shank cross-sectional area. Nominal tensile strength is based upon 75% of the bolt's specified minimum tensile strength. As only the Grade 2 is permitted to be pretensioned, the bolt pretension is based upon the Grade 2 tensile stress area.

Snug-tightened installation is the most economical installation procedure and is permitted for bolts in bearing-type connections, except where pretensioning is required in the Specification. Only Group A bolts in tension or combined shear and tension, and Group B bolts in shear, where loosening or fatigue are not design considerations, are permitted to be installed snug tight. Two studies have been conducted to investigate possible reductions in strength because of varying levels of pretension in bolts within the same connection. The studies found that no significant loss of strength resulted from having different pretensions in bolts within the same connection, even with ASTM F3125 Grade A490 or A490M fasteners. See Commentary Section J3.6 for more details.

There are no specified minimum or maximum pretensions for snug-tight installation of bolts. The only requirement is that the bolts bring the plies into firm contact. Depending on the thickness of material and the possible distortion due to welding, portions of the connection may not be in contact.

There are practical cases in the design of structures where slip of the connection is desirable to allow for expansion and contraction of a joint in a controlled manner. Regardless of whether force transfer is required in the direction normal to the slip direction, the nuts should be hand-tightened with a spud wrench and then backed off one-quarter turn. Furthermore, it is advisable to deform the bolt threads or use a locking nut or jamb nut to ensure that the nut does not back off further under service conditions. Thread deformation is commonly accomplished with a cold chisel and hammer applied at one location. Note that tack-welding of the nut to the bolt threads is not recommended.

2. Size and Use of Holes

Standard holes or short-slotted holes transverse to the direction of load are permitted for all applications complying with the requirements of this Specification. To accommodate manufacturing process tolerances and provide fit and rotation capacity proportional to the size of connections typically using large diameter bolts, the size of standard holes for bolts 1 in. diameter and larger was increased to 1/8 in. over the bolt diameter. The size of standard holes in S.I. units already provided sufficient tolerance and were not increased. In addition, to provide some latitude for adjustment in plumbing a frame during erection, three types of enlarged holes are permitted, subject to the approval of the designer. The nominal maximum sizes of these holes are given in Table J3.3 or J3.3M. The use of these enlarged holes is restricted to connections assembled with high-strength bolts and is subject to the provisions of Sections J3.3 and J3.4.

3. Minimum Spacing

The minimum spacing dimension of 2 2/3 times the nominal diameter is to facilitate construction and does not necessarily satisfy the bearing and tearout strength requirements in Section J3.10.

4. Minimum Edge Distance

Prior to the 2010 AISC *Specification* (AISC, 2010), separate minimum edge distances were given in Tables J3.4 and J3.4M for sheared edges and for rolled or thermally cut edges. Sections J3.10 and J4 are used to prevent exceeding bearing and tearout limits, are suitable for use with both thermally cut, sawed and sheared edges, and must be met for all bolt holes. Accordingly, the edge distances in Tables J3.4 and J3.4M are workmanship standards and are no longer dependent on edge condition or fabrication method.

5. Maximum Spacing and Edge Distance

Limiting the edge distance to not more than 12 times the thickness of the connected part under consideration, but not more than 6 in. (150 mm), is intended to provide for the exclusion of moisture in the event of paint failure, thus preventing corrosion between the parts that might accumulate and force these parts to separate. More restrictive limitations are required for connected parts of unpainted weathering steel exposed to atmospheric corrosion.

The longitudinal spacing applies only to elements consisting of a shape and a plate, or two plates. For elements, such as back-to-back angles not subject to corrosion, the longitudinal spacing may be as required for structural requirements.

6. Tension and Shear Strength of Bolts and Threaded Parts

Tension loading of fasteners is usually accompanied by some bending due to the deformation of the connected parts. Hence, the resistance factor, ϕ, and the safety factor, Ω, are relatively conservative. The nominal tensile strength values in Table J3.2 were obtained from the equation

$$F_{nt} = 0.75F_u \tag{C-J3-2}$$

The factor of 0.75 included in this equation accounts for the approximate ratio of the effective tension area of the threaded portion of the bolt to the area of the shank of the bolt for common sizes. Thus A_b is defined as the area of the unthreaded body of the bolt, and the value given for F_{nt} in Table J3.2 is calculated as $0.75F_u$.

The tensile strength given by Equation C-J3-2 is independent of whether the bolt was initially installed pretensioned or snug-tightened. Tests confirm that the performance of ASTM F3125 Grade A325 and A325M bolts in tension not subjected to fatigue are unaffected by the original installation condition (Amrine and Swanson, 2004; Johnson, 1996; Murray et al., 1992). While the equation was developed for bolted connections, it was also conservatively applied to threaded parts (Kulak et al., 1987).

Previously for ASTM A325 and A325M, the specified minimum tensile strength, F_u, was lower for bolts with diameters in excess of 1 in. (25 mm). This difference no longer exists under the ASTM F3125 standard. This is also reflected in the minimum bolt pretensions provided in Table J3.1.

The values of nominal shear strength in Table J3.2 were obtained from the following equations rounded to the nearest whole ksi (MPa):

(a) When threads are excluded from the shear planes

$$F_{nv} = 0.563F_u \quad \text{(C-J3-3)}$$

(b) When threads are not excluded from the shear plane

$$F_{nv} = 0.45F_u \quad \text{(C-J3-4)}$$

The factor 0.563 accounts for the effect of a shear/tension ratio of 0.625 and a 0.90 length reduction factor. The factor of 0.45 is 80% of 0.563, which accounts for the reduced area of the threaded portion of the fastener when the threads are not excluded from the shear plane. The initial reduction factor of 0.90 is imposed on connections with lengths up to and including 38 in. (950 mm). The resistance factor, ϕ, and the safety factor, Ω, for shear in bearing-type connections in combination with the initial 0.90 factor accommodate the effects of differential strain and second-order effects in connections less than or equal to 38 in. (950 mm) in length.

In connections consisting of only a few fasteners and length not exceeding approximately 16 in. (400 mm), the effect of differential strain on the shear in bearing fasteners is negligible (Kulak et al., 1987; Fisher et al., 1978; Tide, 2010). In longer tension and compression joints, the differential strain produces an uneven distribution of load between fasteners, those near the end taking a disproportionate part of the total load, so that the maximum strength per fastener is reduced. This Specification does not limit the length but requires that the initial 0.90 factor be replaced by 0.75 when determining bolt shear strength for connections longer than 38 in. (950 mm). In lieu of another column of design values, the appropriate values are obtained by multiplying the tabulated values by 0.75/0.90 = 0.833, as given in the Table J3.2 footnote.

The foregoing discussion is primarily applicable to end-loaded tension and compression connections, but for connection lengths less than or equal to 38 in. (950 mm) it is applied to all connections to maintain simplicity. For shear-type connections used in beams and girders with lengths greater than 38 in. (950 mm), there is no need to make the second reduction. Examples of end-loaded and non-end-loaded connections are shown in Figure C-J3.1.

When determining the shear strength of a fastener, the area, A_b, is multiplied by the number of shear planes. While developed for bolted connections, the equations were also conservatively applied to threaded parts. The value given for ASTM A307 bolts was obtained from Equation C-J3-4 but is specified for all cases regardless of the position of threads.

Additional information regarding the development of the provisions in this section can be found in the Commentary to the RCSC *Specification* (RCSC, 2014).

In Table J3.2, footnote c, the specified reduction of 1% for each $^1/_{16}$ in. (2 mm) over 5 diameters for ASTM A307 bolts is a carryover from the reduction that was specified for long rivets. Because the material strengths are similar, it was decided a similar reduction was appropriate.

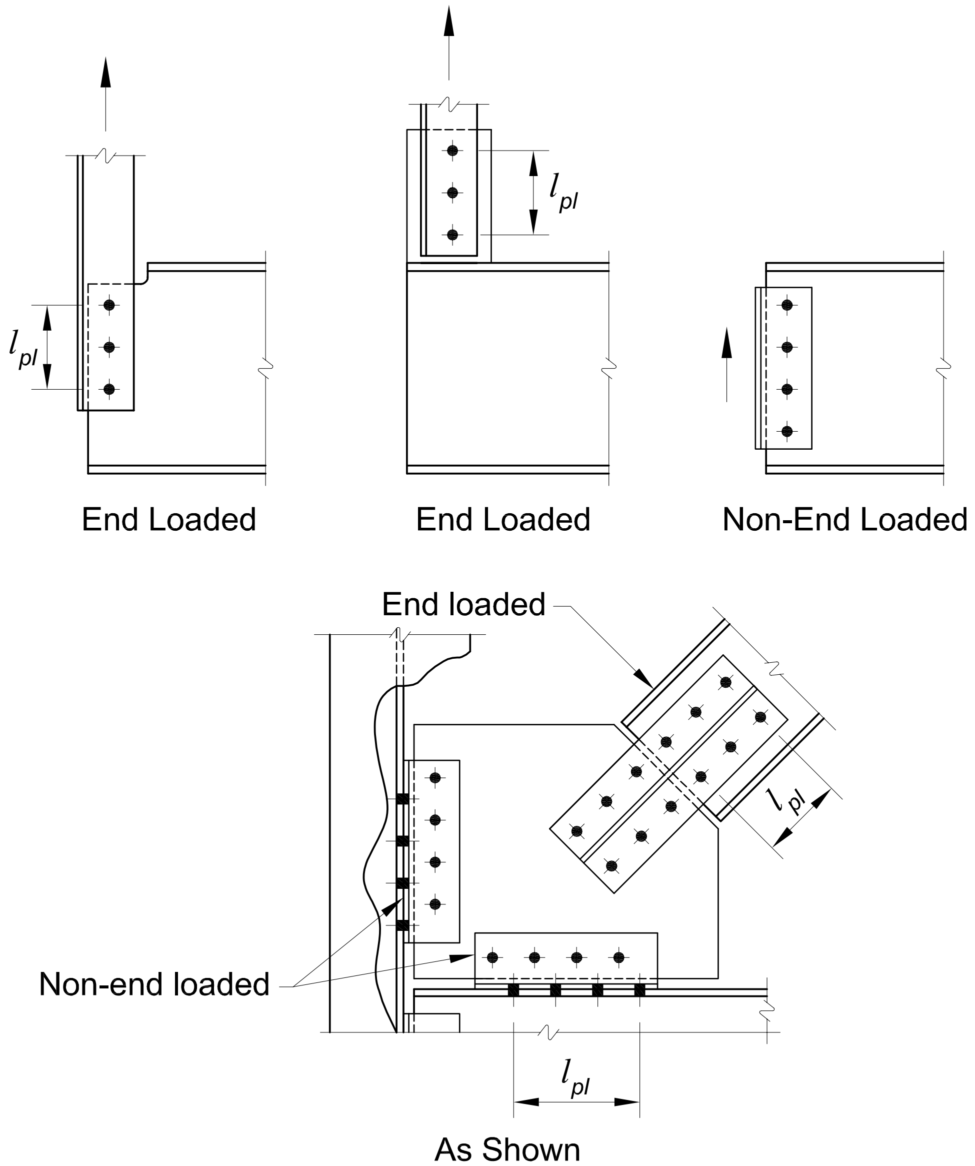


Fig. C-J3.1. End-loaded and non-end-loaded connection examples; l_{pl} = *fastener pattern length.*

7. Combined Tension and Shear in Bearing-Type Connections

Tests have shown that the strength of bearing fasteners subject to combined shear and tension resulting from externally applied forces can be closely defined by an ellipse (Kulak et al., 1987). The relationship is expressed as:

For design according to Section B3.1 (LRFD)

$$\left(\frac{f_t}{\phi F_{nt}}\right)^2 + \left(\frac{f_v}{\phi F_{nv}}\right)^2 = 1 \qquad \text{(C-J3-5a)}$$

For design according to Section B3.2 (ASD)

$$\left(\frac{\Omega f_t}{F_{nt}}\right)^2 + \left(\frac{\Omega f_v}{F_{nv}}\right)^2 = 1 \qquad \text{(C-J3-5b)}$$

where

F_{nt} = nominal tensile stress, ksi (MPa)
F_{nv} = nominal shear stress, ksi (MPa)
f_t = required tensile stress, ksi (MPa)
f_v = required shear stress, ksi (MPa)

The elliptical relationship can be replaced, with only minor deviations, by three straight lines as shown in Figure C-J3.2. The sloped portion of the straight-line representation follows.

For design according to Section B3.1 (LRFD)

$$\left(\frac{f_t}{\phi F_{nt}}\right) + \left(\frac{f_v}{\phi F_{nv}}\right) = 1.3 \qquad \text{(C-J3-6a)}$$

For design according to Section B3.2 (ASD)

$$\left(\frac{\Omega f_t}{F_{nt}}\right) + \left(\frac{\Omega f_v}{F_{nv}}\right) = 1.3 \qquad \text{(C-J3-6b)}$$

which results in Equations J3-3a and J3-3b (Carter et al., 1997).

This latter representation offers the advantage that no modification of either type of stress is required in the presence of fairly large magnitudes of the other type. Note that Equations J3-3a and J3-3b can be rewritten so as to find the nominal shear strength per unit area, F'_{nv}, as a function of the required tensile stress, f_t. These formulations are:

For design according to Section B3.1 (LRFD)

$$F'_{nv} = 1.3F_{nv} - \frac{F_{nv}}{\phi F_{nt}} f_t \le F_{nv} \qquad \text{(C-J3-7a)}$$

For design according to Section B3.2 (ASD)

$$F'_{nv} = 1.3F_{nv} - \frac{\Omega F_{nv}}{F_{nt}} f_t \le F_{nv} \qquad \text{(C-J3-7b)}$$

The linear relationship was adopted for use in Section J3.7; generally, use of the elliptical relationship is acceptable (see Figure C-J3.2). A similar formulation using the elliptical solution follows.

For design according to Section B3.1 (LRFD)

$$F'_{nv} = F_{nv}\sqrt{1-\left(\frac{f_t}{\phi F_{nt}}\right)^2} \tag{C-J3-8a}$$

For design according to Section B3.2 (ASD)

$$F'_{nv} = F_{nv}\sqrt{1-\left(\frac{\Omega f_t}{F_{nt}}\right)^2} \tag{C-J3-8b}$$

8. High-Strength Bolts in Slip-Critical Connections

The design provisions for slip-critical connections have remained substantially the same for many years. The original provisions, using standard holes with $^1/_{16}$-in. (2 mm) clearance, were based on a 10% probability of slip at code loads when tightened by the calibrated wrench method. This was comparable to a design for slip at approximately 1.4 to 1.5 times code loads. Because slip resistance was considered to be a serviceability design issue, this was determined to be an adequate safety factor. Per the RCSC *Guide to the Design Criteria for Bolted and Riveted Joints* (Kulak et al., 1987), the provisions were revised to include oversized and slotted holes (Allan and Fisher, 1968). The revised provisions included a reduction in the allowable strength of 15% for oversize holes, 30% for long slots perpendicular, and 40% for long slots parallel to the direction of the load.

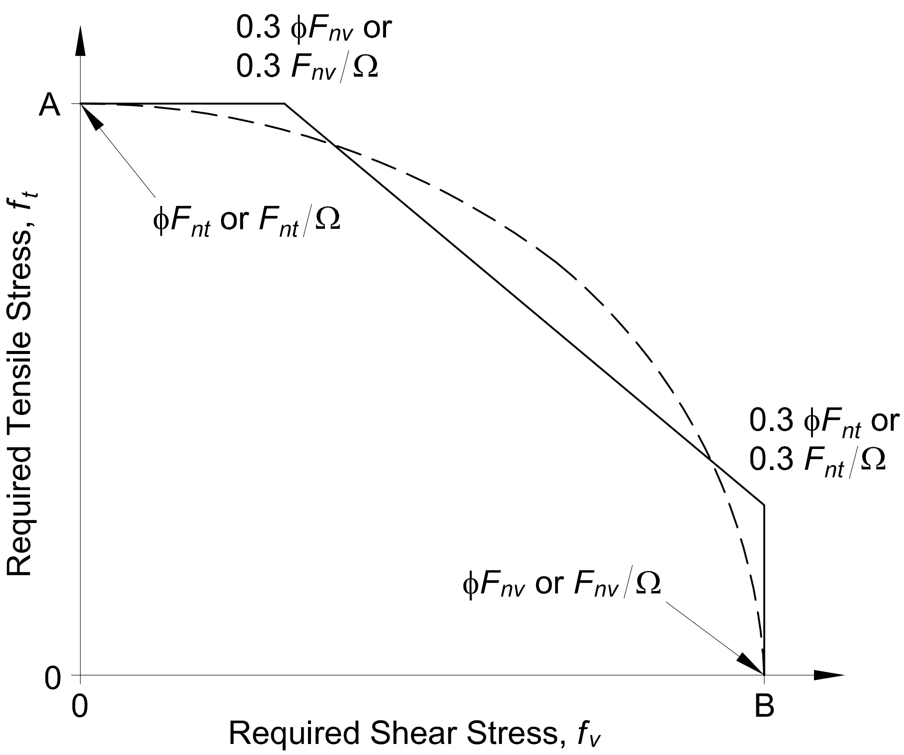


Fig. C-J3.2. Straight-line representation of elliptical solution.

Except for minor changes and adding provisions for LRFD, the design of slip-critical connections was unchanged until the 2005 AISC *Specification* (AISC, 2005) added a higher reliability level for slip-critical connections designed for use where selected by the engineer of record. The reason for this added provision was twofold. First, the use of slip-critical connections with oversize holes had become very popular because of the economy they afforded, especially with large bolted trusses and heavy vertical bracing systems. While the Commentary to the RCSC *Specification* (RCSC, 2014) indicated that only the engineer of record can determine if potential slippage at service loads could reduce the ability of the frame to resist factored loads, it did not give any guidance on how to do this. The 2005 AISC *Specification* provided a procedure to design to resist slip at factored loads if slip at service loads could reduce the ability of the structure to support factored loads.

Second, many of these connection details require large filler plates. There was a question about the need to develop these fills and how to do it. The 1999 LRFD *Specification* (AISC, 2000b) stated that as an alternative to developing the filler "the joint shall be designed as slip critical." The RCSC *Specification* at this time stated, "The joint shall be designed as a slip-critical joint. The slip resistance of the joint shall not be reduced for the presence of fillers or shims." Both Specifications required the joint to be checked as a bearing connection, which normally would require development of large fillers.

The answer to both of these issues seemed to provide a method for designing a connection with oversize holes to resist slip at the strength level and not require the bearing strength check for the connection. In order to do this, it was necessary to first determine as closely as possible what the slip resistance currently was for oversize holes. Then it was necessary to establish what would be an adequate level of slip resistance to be able to say the connection could resist slip at factored loads.

Three major research projects formed the primary sources for the development of the 2010 AISC *Specification* (AISC, 2010) provisions for slip-critical connections:

(1) Dusicka and Iwai (2007) evaluated slip-critical connections with fills for the Research Council on Structural Connections. The work provides results relevant to all slip-critical connections with fills.

(2) Grondin et al. (2007) is a two-part study that assembles slip resistance data from all known sources and analyzes reliability of SC connections indicated by that data. A structural system configuration—a long span roof truss—is evaluated to see if slip required more reliability in slip-critical connections.

(3) Borello et al. (2009) conducted 16 large-scale tests of slip-critical connections in both standard and oversize holes, with and without thick fillers.

Deliberations considered in development of the 2010 AISC *Specification* slip-critical provisions include the following:

Slip Coefficient for Class A Surfaces. Grondin et al. (2007) rigorously evaluated the test procedures and eliminated a substantial number of tests that did not meet the required protocol. The result was a recommended slip coefficient for Class A surfaces between 0.31 and 0.32. Part of the problem is the variability of what is considered to be clean mill scale. Current data on galvanized surfaces indicated more research was required and the American Galvanizers Association is sponsoring a series of tests to determine if further changes in the slip coefficient for these types of surfaces is needed.

Oversized Holes and Loss of Pretension. Borello et al. (2009) confirms that there is no additional loss of pretension and that connections with oversized holes had similar slip resistance to the control group with standard holes.

Higher Pretension with Turn-of-Nut Method. The difficulty in knowing in advance what method of pretensioning would be used resulted in leaving the value of D_u at 1.13 as established for the calibrated wrench method. The Specification does, however, allow the use of a higher D_u value when approved by the engineer of record.

Shear/Bearing Strength. Borello et al. (2009) verified that connections with oversized holes, regardless of fill size, can develop the available bearing strength when the fill is developed. There was some variation in shear strength with filler size but the maximum reduction for thick fillers was approximately 15% when undeveloped.

Fillers in Slip-Critical Connections. Borello et al. (2009) indicated that filler thickness did not reduce the slip resistance of the connection. Borello et al. (2009) and Dusicka and Iwai (2007) indicated that multiple fillers, as shown in Figure C-J3.3, reduced the slip resistance. It was determined that a factor for the number of fillers should be included in the design equation. A plate welded to the connected member or connection plate is not a filler plate and does not require this reduction factor.

The 2010 AISC *Specification* provisions for slip-critical connections were based on the following conclusions:

(1) The mean and coefficient of variation in Class A slip-critical connections supports the use of a $\mu = 0.31$, not 0.33 or 0.35. It was expected that the use of $\mu = 0.30$ would achieve more consistent reliability while using the same resistance factors for both slip classes. The value of $\mu = 0.30$ was selected and the resistance and safety factors reflect this value.

(2) A factor, h_f, to reflect the use of multiple filler plates was added to the equation for nominal slip resistance resulting in

$$R_n = \mu D_u h_f T_b n_s \qquad \text{(C-J3-9)}$$

where

h_f = factor for fillers; coefficient to reflect the reduction in slip due to multiple fills

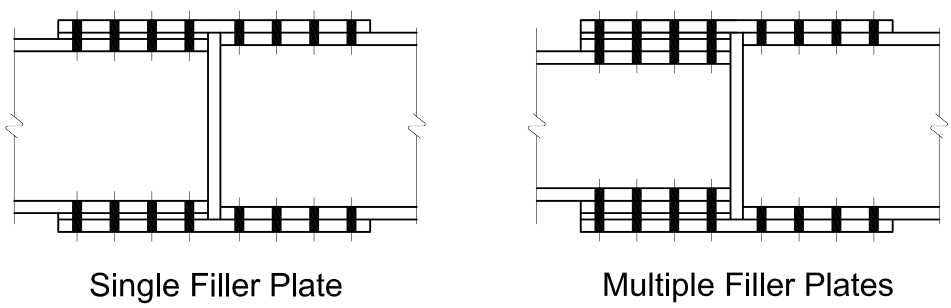


Fig C-J3.3. Single and multiple filler plate configurations.

TABLE C-J3.1
Reliability Factors, β, for Slip Resistance

Group	Class	Turn-of-Nut Method		Other Methods	
		Standard Holes, Parallel Slots	Oversized Holes	Standard Holes, Parallel Slots	Oversized Holes
Group A (A325, A325M)	Class A ($\mu = 0.30$)	2.39	2.92	1.82	2.41
	Class B ($\mu = 0.50$)	2.78	3.52	2.17	2.83
Group B (A490, A490M)	Class A ($\mu = 0.30$)	2.01	2.63	1.53	2.13
	Class B ($\mu = 0.50$)	2.47	3.20	1.86	2.54

(3) D_u is defined as a parameter derived from statistical analysis to calculate nominal slip resistance from statistical means developed as a function of installation method and minimum specified pretension and the level of slip probability selected.

(4) The surfaces of fills must be prepared to the same or higher slip coefficient as the other faying surfaces in the connection.

(5) The reduction in design slip resistance for oversized and slotted holes is not due to a reduction in tested slip resistance but is a factor used to reflect the consequence of slip. It was continued at the 0.85 level but clearly documented as a factor increasing the slip resistance of the connection.

Slip-critical connections with a single filler of any thickness with proper surface preparation may be designed without any reduction in slip resistance. Slip-critical connections with multiple fillers may be designed without any reduction in slip resistance provided the joint has either all faying surfaces with Class B surfaces or Class A surfaces with turn-of-nut tensioning. This provision for multiple fillers is based on the additional reliability of Class B surfaces or on the higher pretension achieved with turn-of-nut tensioning.

The Specification also recognizes a special type of slip-resistant connection for use in built-up compression members in Section E6 where pretensioned bolts and a minimum of Class A surfaces are required but the connection is designed using the bearing strength of the bolts. This is based on the need to prevent relative movement between elements of the compression member at the ends.

Reliability levels for slip resistance in oversized holes and slots parallel to the load given in Table C-J3.1 exceed reliability levels associated with the nominal strength of main members in the Specification when turn-of-nut pretensioning is used. Reliability of slip resistance when other tightening methods are used exceeds previous levels and is sufficient to prevent slip at load levels where inelastic deformation of the connected parts is expected. Since the effect of slip in standard holes is less than that of slip in oversized holes, the reliability factors permitted for standard holes are lower than those for oversized holes. This increased data on the reliability of these connections allowed the return to a single design level of slip resistance similar to the RCSC *Specification* (RCSC, 2014) and previous AISC *Specifications*.

10. Bearing and Tearout Strength at Bolt Holes

Provisions for bearing strength of pins differ from those for bearing strength of bolts; refer to Section J7.

Bearing strength values are provided as a measure of the strength of the material upon which a bolt bears, not as a protection to the fastener, which needs no such protection. Accordingly, the same bearing value applies to all joints assembled by bolts, regardless of fastener shear strength or the presence or absence of threads in the bearing area.

Material bearing strength may be limited either by bearing deformation of the hole or by tearout (a bolt-by-bolt block shear rupture) of the material upon which the bolt bears. In previous editions of the Specification, both limit states were defined by one equation and termed bearing limit states. For this edition, the limit states were separated to permit clear reference to each of the limits and their corresponding equations. Kim and Yura (1996) and Lewis and Zwerneman (1996) confirmed the bearing strength provisions for the bearing case wherein the nominal bearing strength, R_n, is equal to $CdtF_u$ and C is equal to 2.4, 3.0 or 2.0 depending upon hole type and/or acceptability of hole ovalization at ultimate load, as indicated in Section J3.10. However, this same research indicated the need for different bearing strength provisions when tearout failure would control. Appropriate equations for bearing strength as a function of clear distance, l_c, are therefore provided and this formulation is consistent with that in the RCSC *Specification* (RCSC, 2014).

Frank and Yura (1981) demonstrated that hole elongation greater than 1/4 in. (6 mm) will generally begin to develop as the bearing force is increased beyond $2.4dtF_u$, especially if it is combined with high tensile stress on the net section, even though rupture does not occur. For a long-slotted hole with the slot perpendicular to the direction of force, the same is true for a bearing force greater than $2.0dtF_u$. An upper bound of $3.0dtF_u$ anticipates hole ovalization [deformation greater than 1/4 in. (6 mm)] at maximum strength.

Additionally, to simplify and generalize such bearing strength calculations, the current provisions have been based upon a clear-distance formulation. Provisions prior to 1999 utilized edge distances and bolt spacings measured to hole centerlines with adjustment factors to account for varying hole type and orientation, as well as minimum edge distance requirements. The effective strength of an individual fastener is the lesser of the fastener shear strength per Section J3.6 and the bearing and tearout strength at the bolt hole per Section J3.10. The strength of a bolt group is a function of strain compatibility and is dependent on the relative stiffnesses of the bolts and connected parts. For typical connections, such as those shown in the AISC *Steel Construction Manual* (AISC, 2011) it is acceptable to calculate the shear, bearing and tearout limit states for each bolt in the same connected part and sum the lowest value of the bolt shear or the controlling bearing or tearout limit for each bolt to determine the group strength. The intent is that the separate bearing and tearout equations in this Specification be treated in the same way as the combined equations in the 2010 AISC *Specification*. This ignores the potential for interaction of these limit states in multiple connected parts, but that impact is small enough in common connection details within the range of the connections shown in Part 10 of the AISC *Manual,* to allow the benefit of this practical simplification in design. Nonstandard connections may be more sensitive to this interaction; if so, a more exact approach may be necessary.

12. Wall Strength at Tension Fasteners

With any connection configuration where the fasteners transmit a tensile force to the HSS wall, a rational analysis must be used to determine the appropriate limit states. These may include a yield-line mechanism in the HSS wall and/or pull-out through the HSS wall, in addition to applicable limit states for the fasteners subject to tension.

J4. AFFECTED ELEMENTS OF MEMBERS AND CONNECTING ELEMENTS

1. Strength of Elements in Tension

Tests have shown that A_e may be limited by the ability of the stress to distribute in the member. Analysis procedures such as the Whitmore section should be used to determine A_e in these cases.

2. Strength of Elements in Shear

Prior to the 2005 AISC *Specification*, the resistance factor for shear yielding had been 0.90, which was equivalent to a safety factor of 1.67. In the 1989 ASD *Specification* (AISC, 1989), the allowable shear yielding stress was $0.4F_y$, which was equivalent to a safety factor of 1.5. To make the LRFD approach in the 2005 AISC *Specification* consistent with prior editions of the ASD *Specification*, the resistance and safety factors for shear yielding became 1.00 and 1.50, respectively. The resulting increase in LRFD design strength of approximately 10% is justified by the long history of satisfactory performance of ASD use.

3. Block Shear Strength

Tests on coped beams indicated that a tearing failure mode (rupture) can occur along the perimeter of the bolt holes as shown in Figure C-J4.1 (Birkemoe and Gilmor, 1978). This block shear mode combines tensile failure on one plane and shear failure on a perpendicular plane. The failure path is defined by the centerlines of the bolt holes. This same condition exists on welded connections at beam copes. The tensile plane is the length of the horizontal portion of the weld and the shear plane runs from the horizontal weld to the bottom of the cope.

The block shear failure mode is not limited to coped ends of beams; other examples are shown in Figures C-J4.1 and C-J4.2. The block shear failure mode must also be checked around the periphery of welded connections.

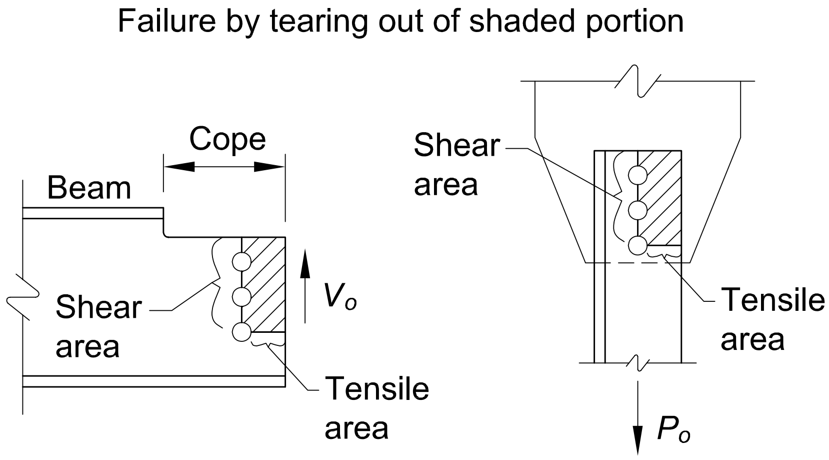
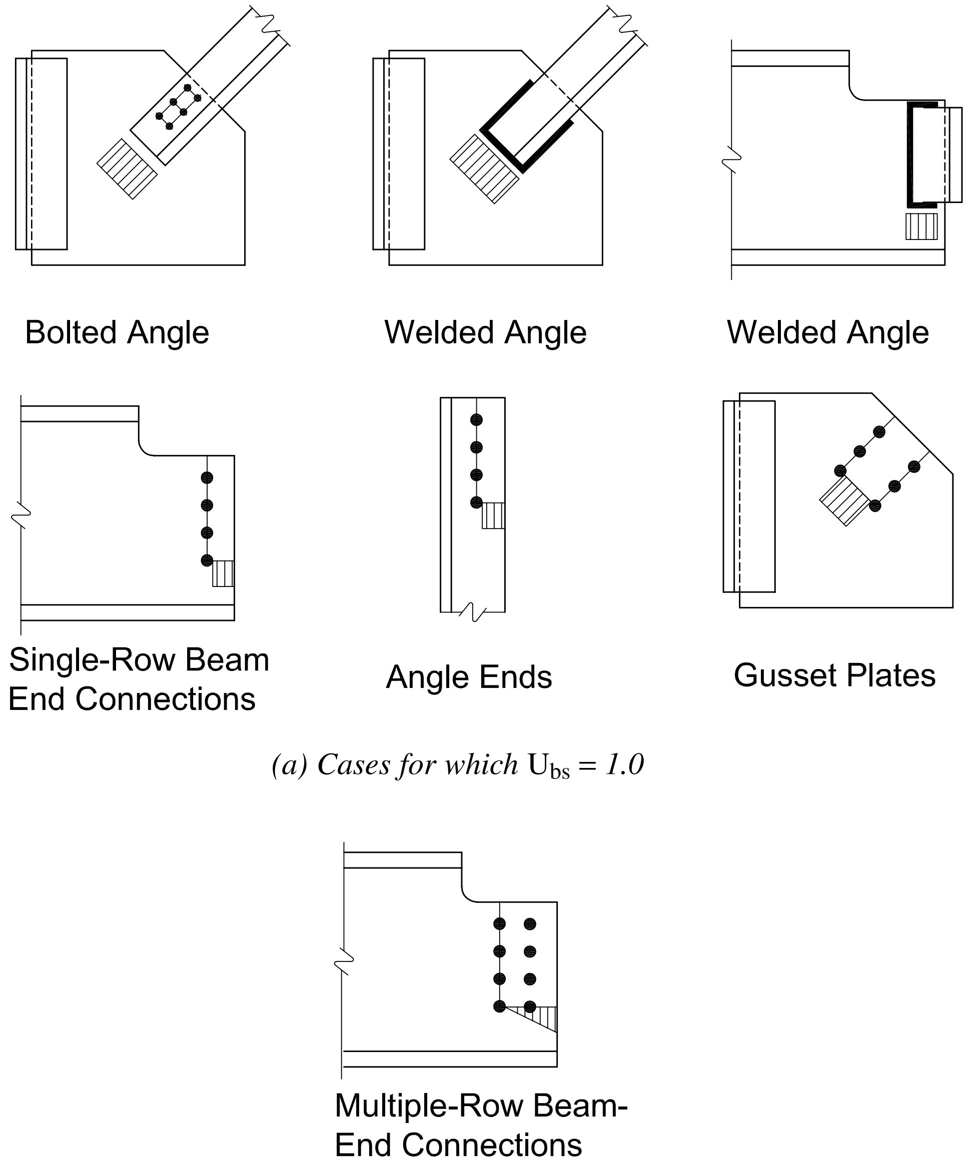


Fig. C-J4.1. Failure surface for block shear rupture limit state.



(a) Cases for which U_{bs} *= 1.0*

(b) Cases for which U_{bs} *= 0.5*

Fig. C-J4.2. Block shear tensile stress distributions.

This Specification has adopted a conservative model to predict block shear strength. The mode of failure in coped beam webs and angles is different than that of gusset plates because the shear resistance is present on only one plane, in which case there must be some rotation of the block of material that is providing the total resistance.

Although tensile failure is observed through the net section on the end plane, the distribution of tensile stresses is not always uniform (Ricles and Yura, 1983; Kulak and Grondin, 2001; Hardash and Bjorhovde, 1985). A reduction factor, U_{bs}, has been included in Equation J4-5 to approximate the nonuniform stress distribution on the tensile plane. The tensile stress distribution is nonuniform in the two row connection in Figure C-J4.2(b) because the rows of bolts nearest the beam end pick up most of the shear load. For conditions not shown in Figure C-J4.2, U_{bs} may be taken as $(1 - e/l)$, where e/l is the ratio of the eccentricity of the load to the centroid of the resistance divided by the block length. This fits data reported by Kulak and Grondin (2001), Kulak and Grondin (2002), and Yura et al. (1982).

Block shear is a rupture or tearing phenomenon, not a yielding limit state. However, gross yielding on the shear plane can occur when tearing on the tensile plane commences if $0.6F_uA_{nv}$ exceeds $0.6F_yA_{gv}$. Hence, Equation J4-5 limits the term $0.6F_uA_{nv}$ to not greater than $0.6F_yA_{gv}$ (Hardash and Bjorhovde, 1985). Equation J4-5 is consistent with the philosophy in Chapter D for tension members where the gross area is used for the limit state of yielding and the net area is used for the limit state of rupture.

4. Strength of Elements in Compression

To simplify connection calculations, the nominal strength of elements in compression when the element slenderness ratio is not greater than 25 is F_yA_g. This is a very slight increase over that obtained if the provisions of Chapter E are used. For more slender elements, the provisions of Chapter E apply.

Since a corner gusset plate is restrained along two edges, it is difficult to establish either L, the laterally unbraced length of the element, or K, the effective length factor. Dowswell (2006) provides guidance for determining K and L based on empirical data. When the gusset is found to be compact ($t_g > t_b$), the slenderness ratio can be assumed to be less than or equal to 25, though buckling need not be checked.

5. Strength of Elements in Flexure

Affected and connecting elements are often short enough and thick enough that flexural effects, if present at all, do not impact the design. When such elements are long enough and thin enough that flexural effects must be considered, the AISC *Manual* provides guidance relative to several specific conditions. Part 9 of the AISC *Manual* contains procedures to check the flexural strength of a coped beam. Part 9 also contains a discussion of prying action, which incorporates a weak-axis flexural strength check for framing-angle connections, end-plate connections, flanges, and other similar elements. Part 10 contains procedures to determine the flexural strength of plates used in the extended configuration of the single-plate shear connection. For all other conditions, the checks provided in Section F11 can be used.

The available flexural strength of connecting elements in LRFD can be calculated as the minimum of $0.9F_yZ_{gross}$ and $0.75F_uZ_{net}$, or in ASD as the minimum of $F_yZ_{gross}/1.67$ and $F_uZ_{net}/2.00$. Consequences of large deflections and supported member or plate instability must be considered when these values are used. If deflection is a concern, the factored loads should also be checked against $0.9F_yS_{gross}$ (Mohr and Murray, 2008).

The net plastic section modulus, Z_{net}, for an odd number of rows of bolts is:

$$Z_{net} = \frac{1}{4} t\,(s - d_h')(n^2 s - d_h') \tag{C-J4-1}$$

and for an even number of rows of bolts is:

$$Z_{net} = \frac{1}{4} t\,(s - d_h')n^2 s \tag{C-J4-2}$$

Section F13.1 contains checks related to the strength reduction for members with holes in the tension flange, which in some instances may be governed by net flexural rupture.

J5. FILLERS

As noted in Commentary Section J3.8, research reported in Borello et al. (2009) resulted in significant changes in the design of bolted connections with fillers. Starting with the 2010 AISC *Specification* (AISC, 2010), bearing connections with fillers over $^3/_4$ in. (19 mm) thick were no longer required to be developed provided the bolts were designed by multiplying the shear strength by a 0.85 factor. A test has shown that fillers welded to resist their proportion of the load will prevent a loss of shear strength in the bolts (Borello et. al., 2009).

Filler plates may be used in lap joints of welded connections that splice parts of different thickness, or where there may be an offset in the joint.

J7. BEARING STRENGTH

In general, the bearing strength design of finished surfaces is governed by the limit state of bearing (local compressive yielding) at nominal loads. The nominal bearing strength of milled contact surfaces exceeds the yield strength because adequate safety is provided by post-yield strength as deformation increases. Tests on pin connections (Johnston, 1939) and rockers (Wilson, 1934) have confirmed this behavior.

J8. COLUMN BASES AND BEARING ON CONCRETE

The provisions of this section are identical to equivalent provisions in ACI 318 and ACI 318M (ACI, 2014).

J9. ANCHOR RODS AND EMBEDMENTS

The term “anchor rod” is used for threaded rods embedded in concrete to anchor structural steel. The term “rod” is intended to clearly indicate that these are threaded rods, not structural bolts, and should be designed as threaded parts per Table J3.2 using the material specified in Section A3.4.

Generally, the largest tensile force for which anchor rods must be designed is that produced by bending moment at the column base and augmented by any uplift caused by the overturning tendency of a building under lateral load.

Shear at the base of a column is seldom resisted by bearing of the column base plate against the anchor rods. Even considering the lowest conceivable slip coefficient, the friction due to the vertical load on a column is generally more than sufficient to transfer the shear from the column base to the foundation. The possible exception is at the base of braced frames and moment frames where larger shear forces may require that shear transfer be accomplished by embedding the column base or providing a shear key at the top of the foundation.

The anchor rod hole sizes listed in Tables C-J9.1 and C-J9.1M are recommended to accommodate the variations that are common for setting anchor rods cast in concrete. These larger hole sizes are not detrimental to the integrity of the supported structure when used with proper washers. The slightly conical hole that results from punching operations or thermal cutting is acceptable.

If plate washers are utilized to resolve horizontal shear, bending in the anchor rod must be considered in the design and the layout of anchor rods must accommodate plate washer clearances. In this case, special attention must be given to weld clearances, accessibility, edge distances on plate washers, and the effect of the tolerances between the anchor rod and the edge of the hole.

It is important that the placement of anchor rods be coordinated with the placement and design of reinforcing steel in the foundations as well as the design and overall size of base plates. It is recommended that the anchorage device at the anchor rod bottom be as small as possible to avoid interference with the reinforcing steel in the foundation. A heavy-hex nut or forged head is adequate to develop the concrete shear cone. See AISC Design Guide 1, *Base Plate and Anchor Rod Design* (Fisher and Kloiber, 2006) for design of base plates and anchor rods. See also ACI 318 and ACI 318M (ACI, 2014) and ACI 349 (ACI, 2013) for embedment design; and OSHA *Safety and Health Regulations for Construction*, Standards—29 CFR 1926 Subpart R—Steel Erection (OSHA, 2015) for anchor rod design and construction requirements for erection safety.

J10. FLANGES AND WEBS WITH CONCENTRATED FORCES

This Specification separates flange and web strength requirements into distinct categories representing different limit states: flange local bending (Section J10.1), web local yielding (Section J10.2), web local crippling (Section J10.3), web sidesway buckling (Section J10.4), web compression buckling (Section J10.5), and web panel-zone shear (Section J10.6). These limit state provisions are applied to two distinct types of concentrated forces normal to member flanges:

(1) Single concentrated forces may be tensile (such as those delivered by tension hangers) or compressive (such as those delivered by bearing plates at beam interior positions, reactions at beam ends, and other bearing connections).

TABLE C-J9.1
Anchor Rod Hole Diameters, in.

Anchor Rod Diameter	Anchor Rod Hole Diameter
$^{1}/_{2}$	$1^{1}/_{16}$
$^{5}/_{8}$	$1^{3}/_{16}$
$^{3}/_{4}$	$1^{5}/_{16}$
$^{7}/_{8}$	$1^{9}/_{16}$
1	$1^{13}/_{16}$
$1^{1}/_{4}$	$2^{1}/_{16}$
$1^{1}/_{2}$	$2^{5}/_{16}$
$1^{3}/_{4}$	$2^{3}/_{4}$
≥ 2	$d_b + 1^{1}/_{4}$

TABLE C-J9.1M
Anchor Rod Hole Diameters, mm

Anchor Rod Diameter	Anchor Rod Hole Diameter
18	32
22	36
24	42
27	48
30	51
33	54
36	60
39	63
42	74

(2) Double concentrated forces, one tensile and one compressive, form a couple on the same side of the loaded member, such as that delivered to column flanges through welded and bolted moment connections.

Flange local bending applies only for tensile forces, web local yielding applies to both tensile and compressive forces, and the remainder of these limit states apply only to compressive forces.

Transverse stiffeners, also called continuity plates, and web doubler plates are only required when the concentrated force exceeds the available strength given for the applicable limit state. It is often more economical to choose a heavier member than to provide such reinforcement (Carter, 1999; Troup, 1999). The demand may be determined as the largest flange force from the various load cases, although the demand may also be taken as the gross area of the attachment delivering the force multiplied by the specified minimum yield strength, F_y. Stiffeners and/or doublers and their attaching welds are sized for the difference between the demand and the applicable limit state strength. Detailing and other requirements for stiffeners are provided in Section J10.7 and Section J10.8; requirements for doublers are provided in Section J10.9.

The provisions in J10 have been developed for use with wide-flange sections and similar built-up shapes. With some judgment they can also be applied to other shapes. The Commentary related to the individual subsections provides further detail relative to testing and assumptions. A brief guidance related the application of these checks to other sections is provided here. When applied to members with multiple webs, such as rectangular HSS and box sections, the strength calculated in this section should be multiplied by the number of webs.

Flange local bending assumes a single concentrated line load applied transverse to the beam web. It is not generally applicable to other shapes or other loading conditions. For instance, point loads, such as those delivered through bolts in tension, are typically addressed using yield-line methods (Dowswell, 2013). The web local yielding provisions assume that concentrated loads are distributed into the member spread out with a slope of 2.5:1. This model is likely appropriate for conditions beyond rolled wide flanges. For example, it could be used to determine the local yielding strength for C-shapes where the concentrated load is delivered opposite the web. It has also been applied to HSS where k is typically taken as the outside corner radius. If the radius is not known, it can be assumed to be $1.5t$, as implied in Section B4.1b(d). If a fillet weld is present at the juncture of the web and the flange, additional distribution of stress through this weld is often assumed. Web local crippling has been applied to HSS members assuming t_f and t_w are both equal to the design wall thickness and the depth, d, is equal to the flat dimension of the HSS sidewall. When the radius is not known, it is typically assumed to be $1.5t$, leading to a depth of $H - 3t$. For box sections, d and h can be taken as the clear distance between the flanges. Equations J10-4, J10-5a and J10-5b assume restraint between the flange and the web, which may not be present when small and/or intermittent welds join the elements of built-up sections. Web sidesway buckling is not generally a consideration for typical closed sections like HSS members. Web compression buckling has been applied to HSS members assuming t_f and t_w are both equal to the design wall thickness and the depth, d, is equal to the flat dimension of the HSS sidewall. For box sections, h can be taken as the clear distance between the flanges. Equation J10-8 assumes pinned restraints at the ends of the web. The web panel-zone shear equations are applicable to rolled wide-flange sections and similar built-up shapes. The equations in Section J10.6 neglect web stability. For deep members with thin webs, stability should not be neglected. See Chapter G and AISC Design Guide 16, *Flush and Extended Multiple-Row Moment End-Plate Connections* (Murray and Shoemaker, 2002). Additional inelastic shear strength due to flange deformation is recognized in Equations J10-11 and J10-12, which should not be applied to sections other than rolled wide-flange sections and similar built-up shapes. Though the Specification only provides explicit equations for rolled wide-flange sections, panel-zone shear is a consideration for other member types, such as HSS and box sections where moment is transferred at a panel zone.

1. Flange Local Bending

Where a tensile force is applied through a plate welded across a flange, that flange must be sufficiently rigid to prevent deformation of the flange and the corresponding high stress concentration in the weld in line with the web.

The effective column flange length for local flange bending is $12t_f$ (Graham et al., 1960). Thus, it is assumed that yield lines form in the flange at $6t_f$ in each direction from the point of the applied concentrated force. To develop the fixed edge consistent with the assumptions of this model, an additional $4t_f$, and therefore a total of $10t_f$, is required for the full flange-bending strength given by Equation J10-1. In the absence of applicable research, a 50% reduction has been introduced for cases wherein the applied concentrated force is less than $10t_f$ from the member end.

The strength given by Equation J10-1 was originally developed for moment connections but also applies to single concentrated forces, such as tension hangers consisting of a plate welded to the bottom flange of a beam and transverse to the beam web. In the original tests, the strength given by Equation J10-1 was intended to provide a lower bound to the force required for weld fracture, which was aggravated by the uneven stress and strain demand on the weld caused by the flange deformation (Graham et al., 1959).

Recent tests on welds with minimum Charpy V-notch (CVN) toughness requirements show that weld fracture is no longer the failure mode when the strength given by Equation J10-1 is exceeded. Rather, it was found that the strength given by Equation J10-1 is consistently less than the force required to separate the flanges in typical column sections by 1/4 in. (6 mm) (Hajjar et al., 2003; Prochnow et al., 2000). This amount of flange deformation is on the order of the tolerances in ASTM A6/A6M, and it is believed that if the flange deformation exceeded this level it could be detrimental to other aspects of the performance of the member, such as flange local buckling. Although this deformation could also occur under compressive normal forces, it is customary that flange local bending is checked only for tensile forces (because the original concern was weld fracture). Therefore, it is not required to check flange local bending for compressive forces.

The provision in Section J10.1 is not applicable to moment end-plate and tee-stub type connections. For these connections, see AISC Design Guide 13, *Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications* (Carter, 1999) or the AISC *Steel Construction Manual* (AISC, 2011).

2. Web Local Yielding

The web local yielding provisions (Equations J10-2 and J10-3) apply to both compressive and tensile forces of bearing and moment connections. These provisions are intended to limit the extent of yielding in the web of a member into which a force is being transmitted. The provisions are based on tests on two-sided directly welded girder-to-column connections (cruciform tests) (Sherbourne and Jensen, 1957) and were derived by considering a stress zone that spreads out with a slope of 2:1. Graham et al. (1960) report pull-plate tests and suggest that a 2.5:1 stress gradient is more appropriate. Recent tests confirm that the provisions given by Equations J10-2 and J10-3 are slightly conservative and that the yielding is confined to a length consistent with the slope of 2.5:1 (Hajjar et al., 2003; Prochnow et al., 2000).

3. Web Local Crippling

The web local crippling provisions (Equations J10-4 and J10-5) apply only to compressive forces. Originally, the term “web crippling” was used to characterize a phenomenon now called web local yielding, which was then thought to also adequately predict web crippling. The first edition of the AISC LRFD *Specification* (AISC, 1986) was the first AISC *Specification* to distinguish between web local yielding and web local crippling. Web local crippling was defined as crumpling of the web into buckled waves directly beneath the load, occurring in more slender webs, whereas web local yielding is yielding of that same area, occurring in stockier webs.

Equations J10-4 and J10-5 are based on research reported in Roberts (1981). The increase in Equation J10-5b for $l_b/d > 0.2$ was developed after additional testing to better represent the effect of longer bearing lengths at ends of members (Elgaaly and Salkar, 1991). All tests were conducted on bare steel beams without the expected beneficial contributions of any connection or floor attachments. Thus, the resulting provisions are considered conservative for such applications. Kaczinski et al. (1994) reported tests on cellular box beams with slender webs and confirmed that these provisions are appropriate in this type of member as well.

The equations were developed for bearing connections but are also generally applicable to moment connections. Equation J10-5a and J10-5b are intended to be applied to beam ends where the web of the beam end is not supported, for example, at the end of a seated connection. Where beam end connections are accomplished with the use of web connections, Equation J10-4 should be used to calculate the available strength for the limit state of web local crippling. Figure C-J10.1 illustrates examples of appropriate applications of Equations J10-4 and J10-5 when checking web local crippling for various framing conditions.

The web local crippling phenomenon has been observed to occur in the web adjacent to the loaded flange. For this reason, a three-quarter stiffener (or stiffeners) or a doubler plate is needed to eliminate this limit state. The stiffener depth was changed in this Specification in response to research by Salker et al. (2015).

4. Web Sidesway Buckling

The web sidesway buckling provisions (Equations J10-6 and J10-7) apply only to compressive forces in bearing connections and do not apply to moment connections. The web sidesway buckling provisions were developed after observing several unexpected failures in tested beams (Summers and Yura, 1982; Elgaaly, 1983). In those tests, the compression flanges were braced at the concentrated load, the web was subjected to compression from a concentrated load applied to the flange, and the tension flange buckled (see Figure C-J10.2).

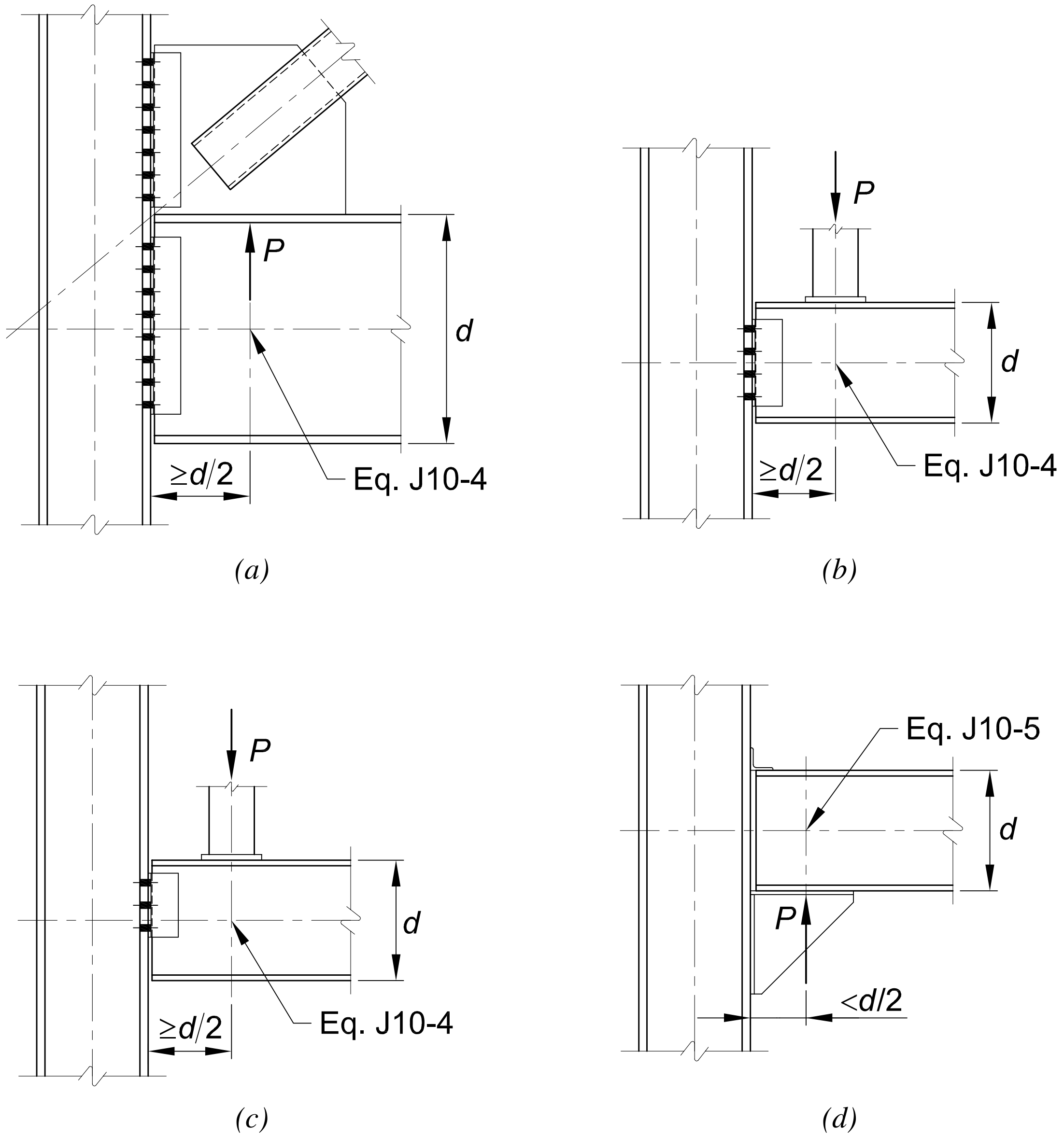


Fig. C-J10.1. Examples of application of the web local crippling equations.

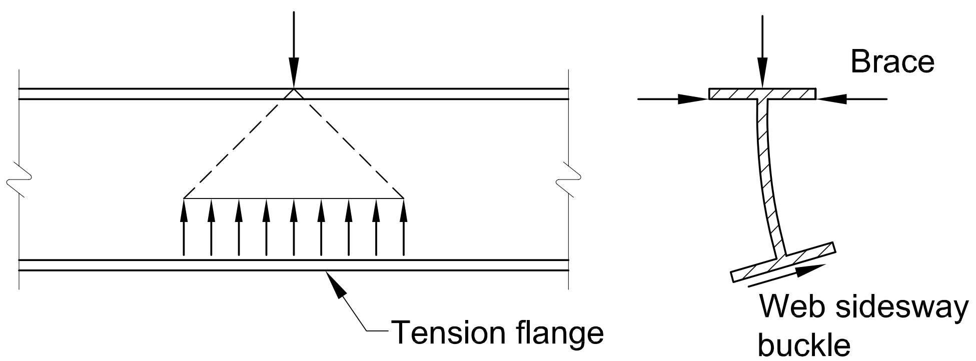


Fig. C-J10.2. Web sidesway buckling.

Web sidesway buckling will not occur in the following cases:

(a) For flanges restrained against rotation (such as when connected to a slab), when

$$\frac{h/t_w}{L_b/b_f} > 2.3 \qquad \text{(C-J10-1)}$$

(b) For flanges not restrained against rotation, when

$$\frac{h/t_w}{L_b/b_f} > 1.7 \qquad \text{(C-J10-2)}$$

where L_b is as shown in Figure C-J10.3.

Web sidesway buckling can be prevented by the proper design of lateral bracing or stiffeners at the load point. It is suggested that local bracing at both flanges be designed for 1% of the concentrated force applied at that point. If stiffeners are used, they must extend from the load point through at least one-half the beam or girder depth. In addition, the pair of stiffeners must be designed to carry the full load. If flange rotation is permitted at the loaded flange, neither stiffeners nor doubler plates are effective.

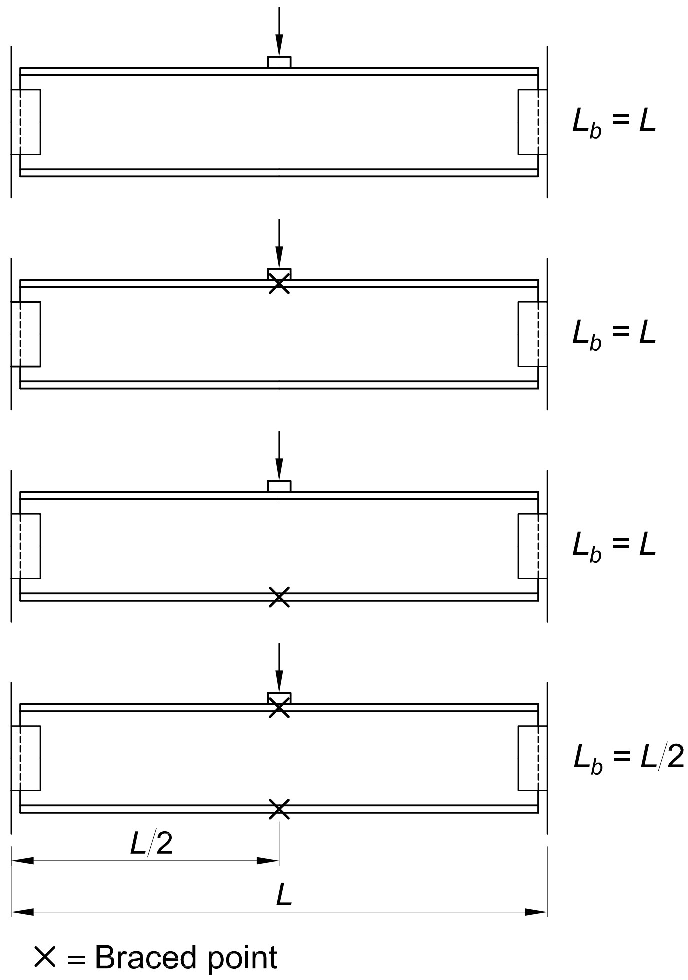


Fig. C-J10.3. Unbraced flange length for web sidesway buckling.

5. Web Compression Buckling

The web compression buckling provision (Equation J10-8) applies only when there are compressive forces on both flanges of a member at the same cross section, such as might occur at the bottom flange of two back-to-back moment connections under gravity loads. Under these conditions, the slenderness of the member web must be limited to avoid the possibility of buckling. Equation J10-8 is applicable to a pair of moment connections and to other pairs of compressive forces applied at both flanges of a member, for which l_b / d is approximately less than 1, where l_b is the length of bearing and d is the depth of the member. When l_b / d is not small, the member web should be designed as a compression member in accordance with Chapter E.

Equation J10-8 is predicated on an interior member loading condition. In the absence of applicable research, a 50% reduction has been introduced for cases wherein the compressive forces are close to the member end.

6. Web Panel-Zone Shear

This section addresses panel-zone behavior of wide-flange sections and similar built-up shapes. Panel-zone shear can also occur in other members, such as HSS and deep and tapered built-up shapes. For these general conditions, the shear strength should be determined in accordance with Chapter G.

Column web shear stresses may be significant within the boundaries of the rigid connection of two or more members with their webs in a common plane. Such webs must be reinforced when the required force, ΣR_u for LRFD or ΣR_a for ASD, along plane A-A in Figure C-J10.4 exceeds the column web available strength, ϕR_n or R_n / Ω, respectively.

For design according to Section B3.1 (LRFD)

$$\Sigma F_u = \frac{M_{u1}}{d_{m1}} + \frac{M_{u2}}{d_{m2}} - V_u \qquad \text{(C-J10-3a)}$$

where

M_{u1} $= M_{u1L} + M_{u1G}$
= sum of the moments due to the factored lateral loads, M_{u1L}, and the moments due to factored gravity loads, M_{u1G}, on the windward side of the connection, kip-in. (N-mm)

M_{u2} $= M_{u2L} - M_{u2G}$
= difference between the moments due to the factored lateral loads, M_{u2L}, and the moments due to factored gravity loads, M_{u2G}, on the leeward side of the connection, kip-in. (N-mm)

d_{m1}, d_{m2} = distance between flange forces in the moment connection, in. (mm)

For design according to Section B3.2 (ASD)

$$\Sigma F_a = \frac{M_{a1}}{d_{m1}} + \frac{M_{a2}}{d_{m2}} - V_a \qquad \text{(C-J10-3b)}$$

where

$M_{a1} = M_{a1L} + M_{a1G}$

= sum of the moments due to the nominal lateral loads, M_{a1L}, and the moments due to nominal gravity loads, M_{a1G}, on the leeward side of the connection, kip-in. (N-mm)

$M_{a2} = M_{a2L} - M_{a2G}$

= difference between the moments due to the nominal lateral loads, M_{a2L}, and the moments due to nominal gravity loads, M_{a2G}, on the windward side of the connection, kip-in. (N-mm)

Historically (and conservatively), 0.95 times the beam depth has been used for d_m.

If, for LRFD, $\Sigma F_u \leq \phi R_n$, or for ASD, $\Sigma F_a \leq R_n / \Omega$, no reinforcement is necessary; in other words, $t_{req} \leq t_w$, where t_w is the column web thickness.

Equations J10-9 and J10-10 limit panel-zone behavior to the elastic range. While such connection panels possess large reserve capacity beyond initial general shear yielding, the corresponding inelastic joint deformations may adversely affect the strength and stability of the frame or story (Fielding and Huang, 1971; Fielding and Chen, 1973). Panel-zone shear yielding affects the overall frame stiffness and, therefore, the resulting second-order effects may be significant. The shear/axial interaction expression of Equation J10-10, as shown in Figure C-J10.5, provides elastic panel behavior.

If adequate connection ductility is provided and the frame analysis considers the inelastic panel-zone deformations, the additional inelastic shear strength is recognized in Equations J10-11 and J10-12 by the factor

$$\left(1+\frac{3b_{cf}t_{cf}^2}{d_b d_c t_w}\right)$$

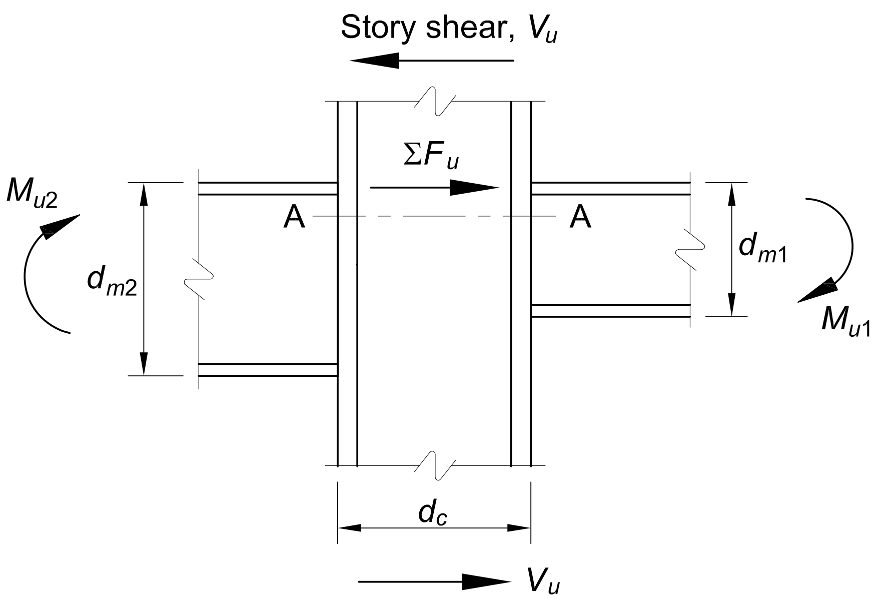


Fig. C-J10.4. LRFD forces in panel zone (ASD forces are similar).

This increase in shear strength due to inelasticity has been most often utilized for the design of frames in high-seismic applications and should be used when the panel zone is designed to develop the strength of the members from which it is formed.

The shear/axial interaction expression incorporated in Equation J10-12 (see Figure C-J10.6) recognizes that when the panel-zone web has completely yielded in shear, the axial column load is resisted by the flanges.

7. Unframed Ends of Beams and Girders

Full-depth stiffeners are required at unframed ends of beams and girders not otherwise restrained to avoid twisting about their longitudinal axes. These stiffeners are full depth but not fitted. They connect to the restrained flange but do not need to continue beyond the toe of the fillet at the far flange unless connection to the far flange is necessary for other purposes, such as resisting compression from a concentrated load on the far flange.

8. Additional Stiffener Requirements for Concentrated Forces

For guidelines on column stiffener design, see Carter (1999), Troup (1999), and Murray and Sumner (2004).

For rotary-straightened W-shapes, an area of reduced notch toughness is sometimes found in a limited region of the web immediately adjacent to the flange, referred to as the "*k*-area," as illustrated in Figure C-J10.7 (Kaufmann et al., 2001). The *k*-area

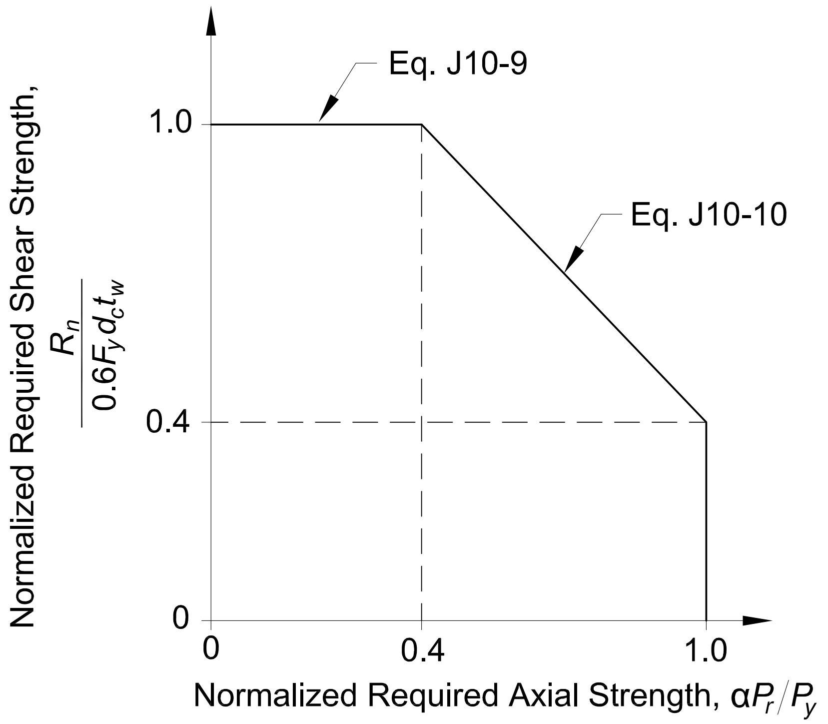


Fig. C-J10.5. Interaction of shear and axial force—elastic.

is defined as the region of the web that extends from the tangent point of the web and the flange-web fillet (AISC *k*-dimension) a distance 1½ in. (38 mm) into the web beyond the *k*-dimension. Following the 1994 Northridge earthquake, there was a tendency to specify thicker transverse stiffeners that were groove welded to the web and flange, and thicker doubler plates that were often groove welded in the gap between the doubler plate and the flanges. These welds were highly restrained and may have caused cracking during fabrication in some cases (Tide, 1999). AISC (1997b) recommended that the welds for continuity plates terminate away from the *k*-area.

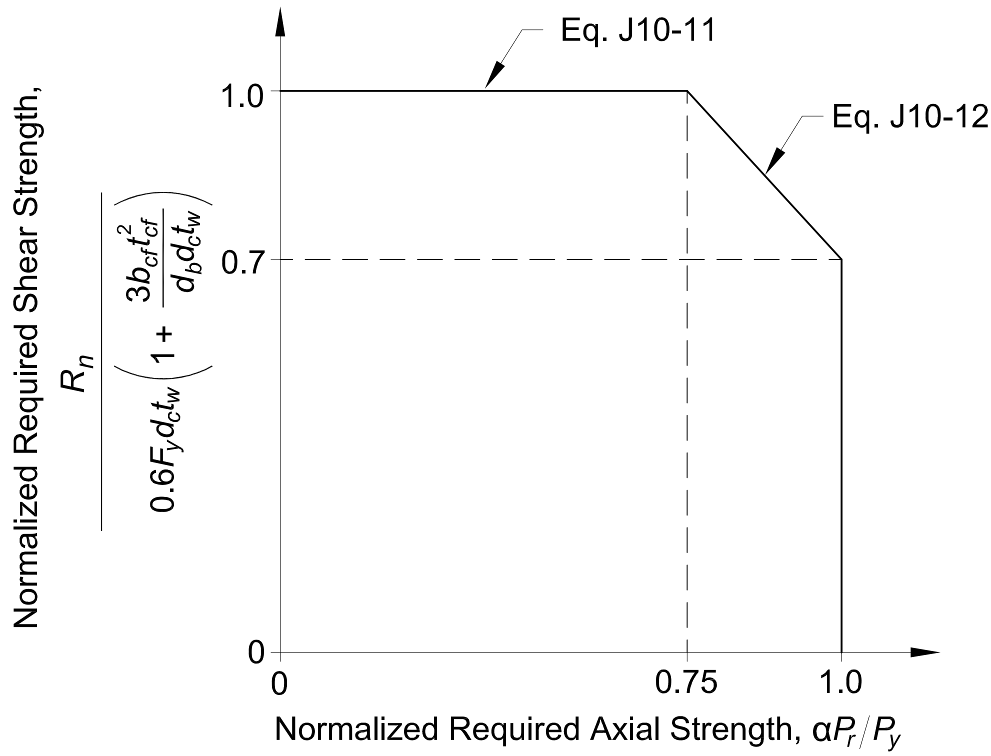


Fig. C-J10.6. Interaction of shear and axial force—inelastic.

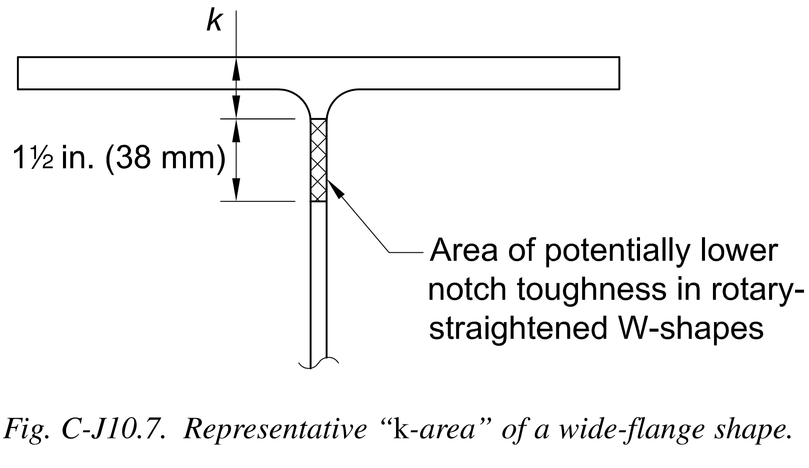


*Fig. C-J10.7. Representative "*k*-area" of a wide-flange shape.*

Pull-plate tests (Dexter and Melendrez, 2000; Prochnow et al., 2000; Hajjar et al., 2003) and full-scale beam-column joint testing (Bjorhovde et al., 1999; Dexter et al., 2001; Lee et al., 2002a) have shown that this problem can be avoided if the column stiffeners are fillet welded to both the web and the flange, the corner is clipped at least $1^1/_2$ in. (38 mm), and the fillet welds are stopped short by a weld leg length from the edges of the cutout, as shown in Figure C-J10.8. These tests also show that groove welding the stiffeners to the flanges or the web is unnecessary, and that the fillet welds performed well with no problems. If there is concern regarding the development of the stiffeners using fillet welds, the corner clip can be made so that the dimension along the flange is $^3/_4$ in. (20 mm) and the dimension along the web is $1^1/_2$ in. (38 mm).

Tests have also shown the viability of fillet welding doubler plates to the flanges, as shown in Figure C-J10.9 (Prochnow et al., 2000; Dexter et al., 2001; Lee et al., 2002a; Hajjar et al., 2003). It was found that it is not necessary to groove weld the doubler plates and that they do not need to be in contact with the column web to be fully effective.

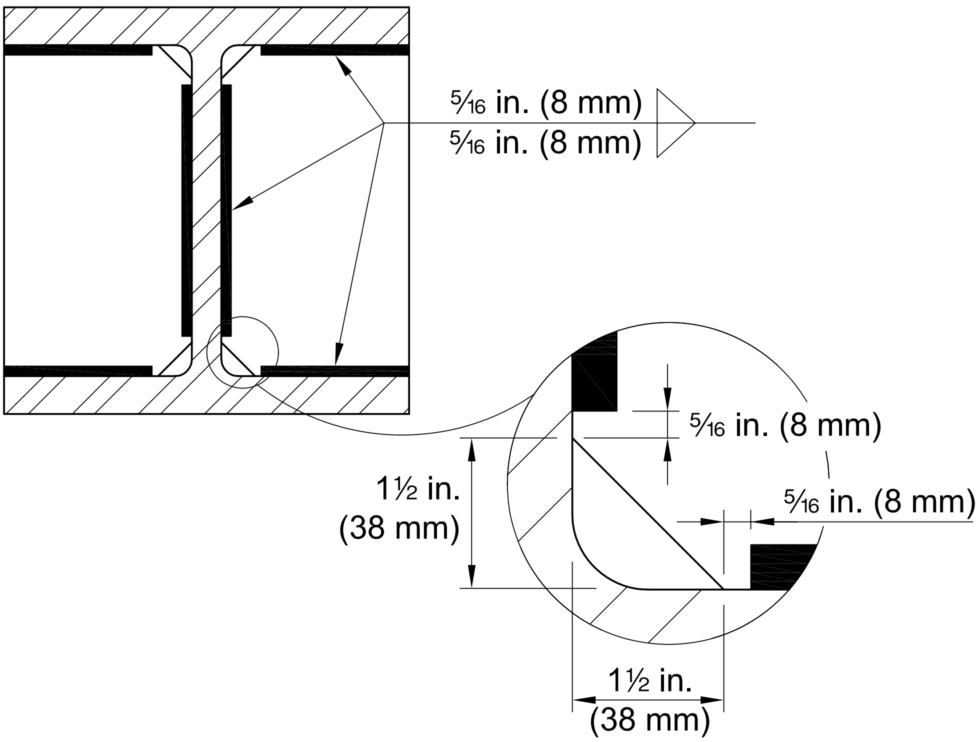


*Fig. C-J10.8. Recommended placement of stiffener fillet welds to avoid contact with "*k*-area."*

9. Additional Doubler Plate Requirements for Concentrated Forces

When required, doubler plates are to be designed using the appropriate limit state requirements for the type of loading. The sum of the strengths of the member element and the doubler plate(s) must exceed the required strength, and the doubler plate must be welded to the member element.

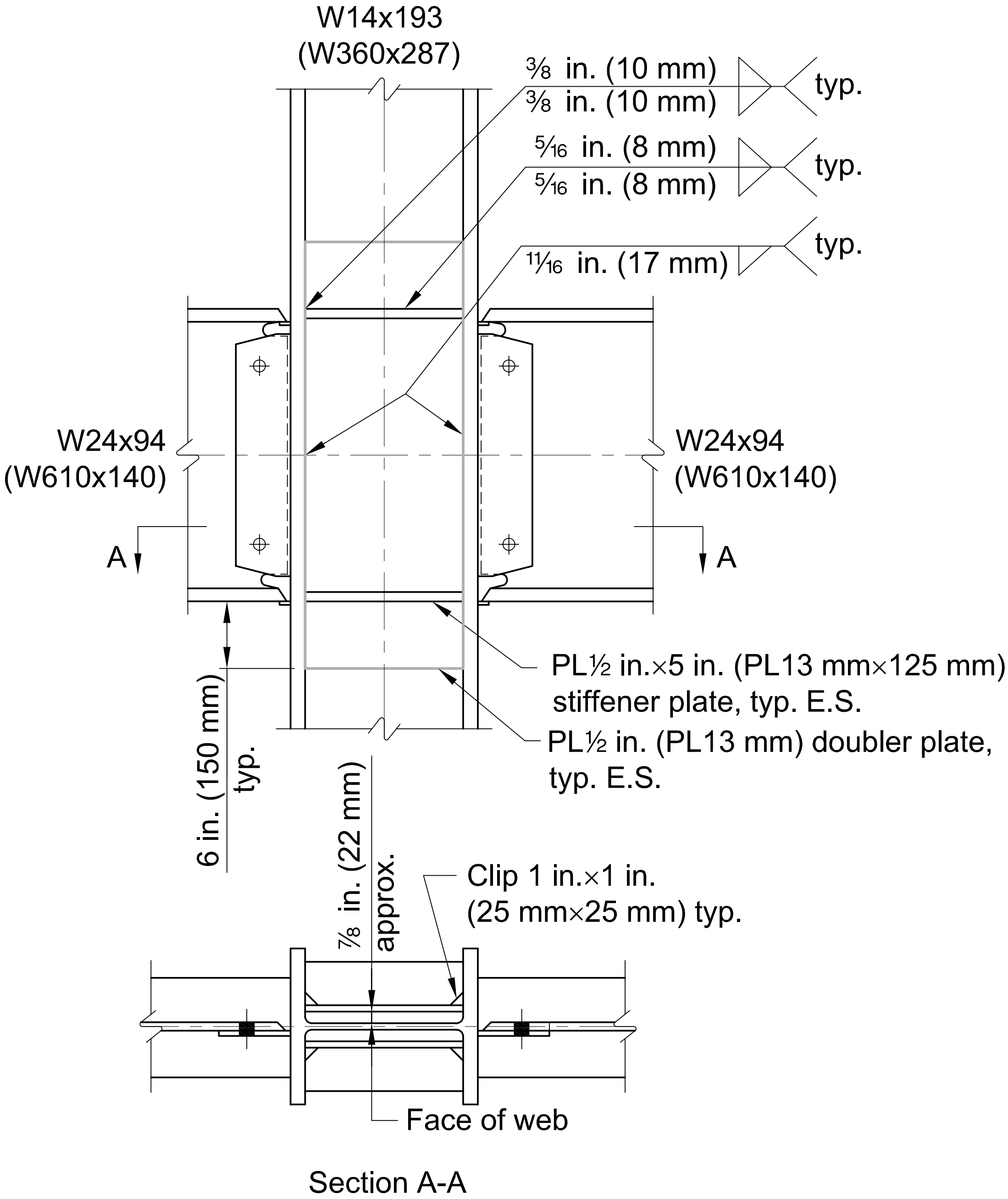


Fig. C-J10.9. Example of fillet welded doubler plate and stiffener details.

10. Transverse Forces on Plate Elements

Designing connections to resist forces transverse to the plane of plate elements as shown in Figure C-J10.10 is often not the best solution but, where it is required, there must be sufficient flexure and shear strength. This section addresses only strength. Stiffness may also be a consideration; in particular, for moment connections, Section B3.4b must be satisfied. Simple beam connections are required to provide for rotational ductility and usually do not need to have transverse plate elements designed for flexure.

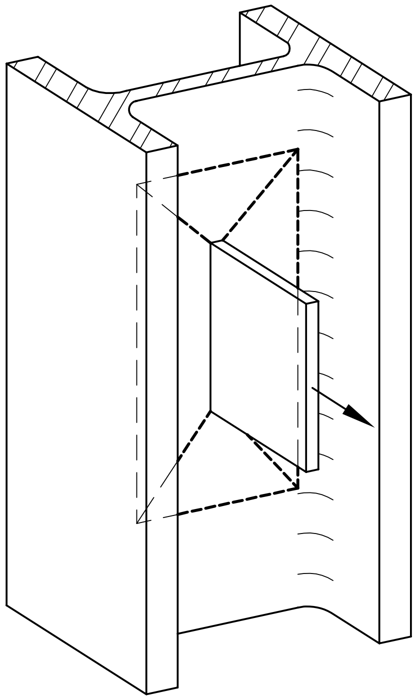


Fig. C-J10.10. Yield lines due to transverse forces on plate elements.

CHAPTER K

ADDITIONAL REQUIREMENTS FOR HSS AND BOX-SECTION CONNECTIONS

Chapter K addresses the strength of connections to hollow structural sections (HSS) and box sections of uniform wall thickness, where seam welds between box-section elements are complete-joint-penetration groove welds in the connection region. The provisions are based on failure modes that have been reported in international research on HSS, much of which has been sponsored and synthesized by CIDECT (International Committee for the Development and Study of Tubular Construction) since the 1960s. This work has also received critical review by the International Institute of Welding (IIW) Subcommission XV-E on "Tubular Structures." The HSS connection design recommendations are generally in accord with the design recommendations by this Subcommission (IIW, 1989). Some minor modifications to the IIW recommended provisions for some limit states have been made by the adoption of the formulations for the same limit states elsewhere in this Specification. These IIW connection design recommendations have also been implemented and supplemented in later design guides by CIDECT (Wardenier et al., 1991; Packer et al., 1992), in the design guide by the Canadian Institute of Steel Construction (Packer and Henderson, 1997), and in CEN (2005a). Parts of these IIW design recommendations are also incorporated in AWS (2015). A large amount of research data generated by CIDECT research programs up to the mid-1980s is summarized in CIDECT Monograph No. 6 (Giddings and Wardenier, 1986). Further information on CIDECT publications and reports can be obtained from their website: **www.cidect.com**.

Chapter K does not prohibit using joints which fall outside the listed limits of applicability; however, this Specification and commentary do not provide connection capacities or guidance when doing so. A rational approach to their design is left to the designer. This commentary gives some insight into the failure modes that should be considered. However, some of the discussions presented later concerning which limit states need to be checked, which can be eliminated, and when they can be eliminated, may or may not apply when outside the limits of applicability. There is also one notable failure mode (local buckling of the chord face) that has been eliminated from consideration in both the Specification and commentary due to the fact that, in tests, it did not control the connection strength when staying within the limits. All potential failure modes should be investigated by the designer when working outside the limits of applicability listed in Chapter K.

When inelastic finite element analysis is used, peak strains in the thick shell ($T \times T \times T$) elements should not exceed $0.02/T$ at the nominal capacity, where T is the thickness in inches.

The connection capacities calculated in Chapter K are based on strength limit states only. There is no connection deformation limit state considered in these provisions. Sub-commission XV-E of IIW, in their most recent design recommendations (IIW, 2012), have now adopted a limit of $0.03D$ for round and $0.03B$ for rectangular HSS as the maximum acceptable connection displacement, perpendicular to the main member face at the ultimate load capacity. This limit state equates to approximately 1% of connection deformation at service loads.

While the majority of Chapter K is in agreement with the previous IIW design recommendations (IIW, 1989), it was determined that adopting a connection deformation limit state for HSS would not be consistent with this Specification, which does not include deformation limit states for connections; however, designers should be aware of the potential for relatively large connection deformations in certain HSS joint configurations. In order to meet the new deformation limit state, IIW and ISO have made some modifications to the range of validity of T, Y, X and K gap connections and to the calculations of connection strengths, including changes to the strength reduction based on the chord or main member stress function, Q_f. The change in the chord stress function is particularly noticeable in high tension areas of main members where no chord stress reduction is necessary when using strength checks only. Q_f currently is 1.0 for main members in tension.

Where connection deformations would be a concern due to serviceability or stability, the IIW (2012) or CIDECT (Wardenier et al., 2008; Packer et al., 2009) recommendations could be used.

The scopes of Sections K2 and K3 note that the centerlines of the branch member(s) and the chord members must lie in a single plane. For other configurations, such as multi-planar connections, connections with partially or fully flattened branch member ends, double-chord connections, connections with a branch member that is offset so that its centerline does not intersect with the centerline of the chord, or connections with round branch members joined to a square or rectangular chord member, the provisions of IIW (1989), CIDECT (Wardenier et al., 1991; Packer et al., 1992), CISC (Packer and Henderson, 1997; Marshall, 1992; AWS, 2015), or other verified design guidance or tests can be used.

To be consistent with the requirements of Chapter K, box-section members require complete-joint-penetration groove seam welds in the connection region to ensure that each of the member's faces acts as a single element and is able to develop the full capacity of that element for all viable failure modes depending on the type of connection, geometric parameters, and loading. This constraint guarantees that box-section connections behave in a manner similar to HSS member connections with the same applicable failure modes. The length of the connection region along each member is determined based on the maximum extent of influence of all possible failure modes for the connection. These failure modes are described by Wardenier (1982) for both rectangular HSS truss connections and rectangular HSS moment connections. A conservative distance equal to the width of the member away from the face of the intersecting member in the connection can be used to define the connection region.

Connection available strengths in Chapter K assume a main member with sufficient end distances, l_{end}, on both sides of the connection. A new limit of applicability has been added to Tables K2.1A, K3.1A and K3.2A, which limits how close a branch or plate can be connected to the end of the chord. When a branch or plate is connected near to the end of a chord, there is not enough length to develop the typically assumed yield line patterns. A modified yield line pattern can be shown to develop an equal strength if the branch or plate is at least a distance equal to the limiting l_{end} from the chord end. Where the end distance is less than the limit, a cap plate or a reduction in the resistance are commonly accepted alternatives. The reduction in resistance may not be a linear proportion of the end distance. When the branch or plate is closer to the unreinforced end of a chord than indicated, the strengths predicted in Tables K3.1 and K3.2 can conservatively be reduced by 50%. The branch member or plate supplying the load must have sufficient lateral restraint.

Cap plates attached to the ends of round and rectangular HSS members contribute to stiffening the end of the member. If a cap plate is welded on all sides, a transverse load applied near the end of the member can be conservatively treated as if it were applied to a continuous member with load applied far from the end of the member. Therefore, there is no minimum end distance requirement in the case of a cap plate. The cap plate will allow the HSS member to develop either the strength of the connected face (plastification or shear yielding) or the strength of the sidewalls (yielding or crippling).

K1. GENERAL PROVISIONS AND PARAMETERS FOR HSS CONNECTIONS

The classification of HSS truss-type connections as K- (which includes N-), Y- (which includes T-), or cross- (also known as X-) connections is based on the method of force transfer in the connection, not on the physical appearance of the connection. Examples of such classification are shown in Figure C-K1.1.

When branch members transmit part of their load as K-connections and part of their load as T-, Y- or cross-connections, the adequacy of each branch is determined by linear interaction of the proportion of the branch load involved in each type of load transfer. One K-connection, shown in Figure C-K1.1(b), illustrates that the branch force components normal to the chord member may differ by as much as 20% and still be deemed to exhibit K-connection behavior. This is to accommodate slight variations in branch member forces along a typical truss, caused by a series of panel-point loads. The N-connection in Figure C-K1.1(c), however, has a ratio of branch force components normal to the chord member of 2:1. In this case, the connection is analyzed as both a “pure” K-connection (with balanced branch forces) and a cross-connection (because the remainder of the diagonal branch load is being transferred through the connection), as shown in Figure C-K3.3. For the diagonal tension branch in that connection, the following check is also made:

$$(0.5P_r \sin\theta\,/\,\text{K-connection available strength}) + (0.5P_r \sin\theta\,/\,\text{cross-connection available strength}) \leq 1.0$$

2. Rectangular HSS

Due primarily to the flexibility of the connecting face of the chord, the full width of a branch may not be effective. The resulting uneven load distribution is manifested by local buckling of a compression branch or premature yield failure of a tension branch. For plates framing transverse to the longitudinal axis of an HSS chord member, the full area of the plate may not be effective. For T-, Y- and cross-connections, the two walls of the HSS branch transverse to the chord may only be partially effective, whereas for gapped K-connections only one wall of the branch transverse to the chord is likely to be partially effective, because the HSS chord will be “reinforced” by the equal and opposite force from the other member. This is reflected in the equations for gapped K-connections. The effective width term, B_e, is introduced in Section

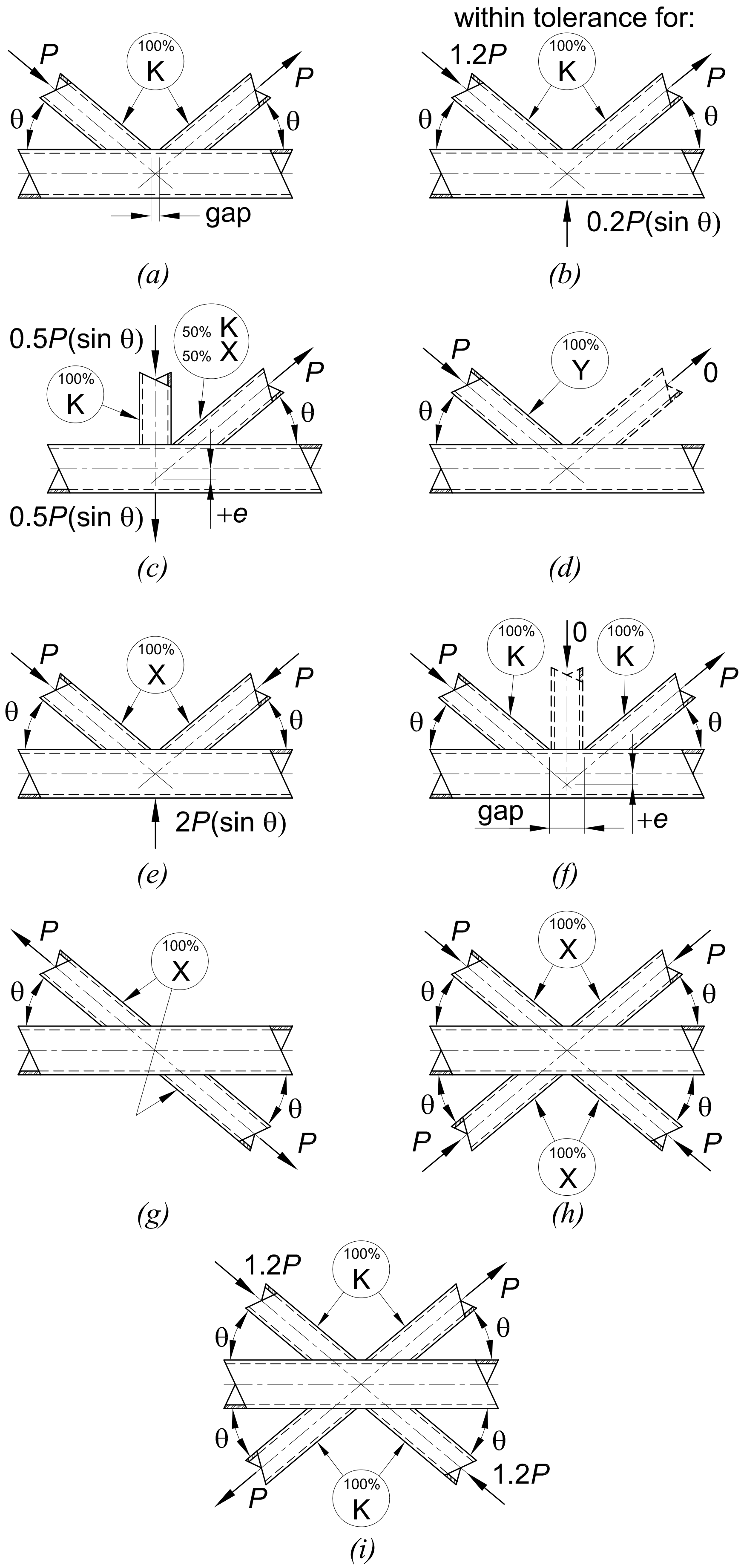


Fig. C-K1.1. Examples of HSS connection classification.

K1.2 to consolidate separate effective width terms, such as b_{eoi} and b_{eov}, that appeared in the 2010 AISC *Specification* but which provided equivalent information. The effective width parameter has been derived from research on transverse plate-to-HSS connections (Davies and Packer, 1982), and the constant of 10 in the calculation of the effective width, B_e, incorporates a ϕ factor of 0.80 or Ω factor of 1.88. Applying the same logic as for the limit state of punching shear, a global ϕ factor of 0.95 or Ω factor of 1.58 has been adopted in AWS D1.1/D1.1M (AWS, 2015), and this has been carried over to this Specification. A ϕ factor of 1.00 is used in IIW (1989).

When the branch (plate or HSS) width exceeds 85% of the connecting chord width, the transverse force from the branch can be assumed to be transferred predominately from the branch to the sidewalls of the chord. In such cases, the limit states associated with concentrated forces on the webs of I-sections, web local yielding (Section J10.2), web local crippling (Section J10.3), and web compression buckling (Section J10.5), can be used to determine the strength of the sidewalls of the chord.

When the branch (plate or HSS) width is less than 85% of the connecting chord width, the transverse force from the branch must pass though the face of the chord to be delivered to the sidewalls. Bending and shear on the chord face must be checked.

An analytical yield-line solution for flexure of the connecting chord face serves to limit connection deformations and is known to be well below the ultimate connection strength. A ϕ factor of 1.00 or Ω factor of 1.50 is thus appropriate. When the branch width exceeds 85% of the chord width, a yield-line failure mechanism will result in a noncritical connection capacity.

Punching shear can be based on the effective punching shear perimeter around the branch considering the effective width from Section K1.2 with the total branch perimeter being an upper limit on this length.

K2. CONCENTRATED FORCES ON HSS

Sections K2.2 and K2.3, although pertaining to all concentrated forces on HSS, are particularly oriented towards plate-to-HSS welded connections.

Wide-flange beam-to-HSS PR moment connections can be modelled as a pair of transverse plates at the beam flanges, neglecting the effect of the web. The beam moment is thus produced by a force couple in the beam flanges. The connection flexural strength is then given by the plate-to-HSS connection strength multiplied by the distance between the beam flange centers.

1. Definitions of Parameters

Some of the notation used in Chapter K is illustrated in Figure C-K2.1.

2. Round HSS

The limits of applicability in Table K2.1A stem primarily from limitations on tests conducted to date.

3. Rectangular HSS

When connecting single-plate shear connections to HSS columns the AISC *Manual* (AISC, 2011) includes recommendations based on Sherman and Ales (1991) and Sherman (1995b, 1996), where a large number of simple framing connections between wide-flange beams and rectangular HSS columns are investigated, in which the load transferred was predominantly shear. A review of costs also showed that single-plate and single-angle connections were the most economical, with double-angle and fillet-welded tee connections being more expensive. Through-plate and flare-bevel welded tee connections were among the most expensive (Sherman, 1995b). Over a wide range of connections tested, only one limit state was identified for the rectangular HSS column: punching shear failure related to end rotation of the beam, when a thick shear plate was joined to a relatively thin-walled HSS. In previous editions of the Specification, the available shear strength of the HSS wall was compared to the available tensile strength of the plate. Because the check only applies to single-plate shear connections, it has been removed from the Specification. A check has been added to the AISC *Manual* that investigates punching shear of the HSS wall due to a beam end reaction applied eccentrically, with the eccentricity being the distance from the HSS wall to the center-of-gravity of the bolt group.

The strength of a square or rectangular HSS wall with load transferred through a cap plate (or the flange of a T-stub), as shown in Figure C-K2.2, can be calculated by considering the limits states of local yielding and local crippling. In general, the

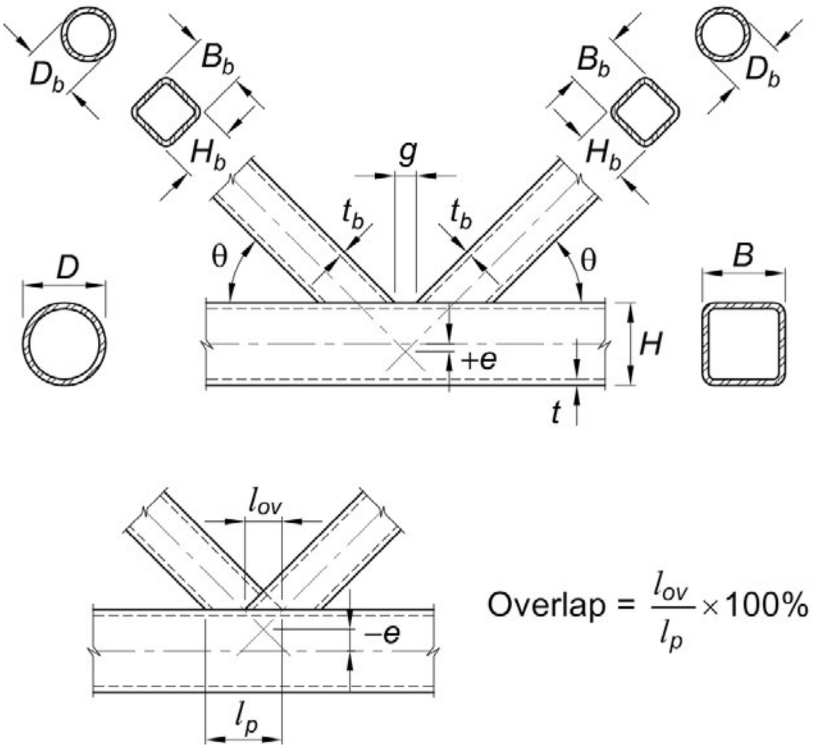


Fig. C-K2.1. Common notation for HSS connections.

rectangular HSS could have dimensions of $B \times H$, but the illustration shows the bearing length (or width), b, oriented for lateral load dispersion into the wall of dimension B. A conservative distribution slope can be assumed as 2.5:1 from each face of the tee web (Wardenier et al., 1991; Kitipornchai and Traves, 1989), which produces a dispersed load width of $(5t_p + b)$ relative to local yielding. If this is less than B, only the two side walls of dimension B are effective in resisting the load, and even they will both be only partially effective. If $(5t_p + b) \geq B$, all four walls of the rectangular HSS will be engaged, and all will be fully effective; however, the cap plate (or T-stub flange) must be sufficiently thick for this to happen. If the weld leg size is known, it is acceptable to assume load dispersion from the toes of the welds. Neglecting the effect of the weld is conservative. The same load dispersion model as shown in Figure C-K2.2 can also be applied to round HSS-to-cap plate connections.

If a longitudinal plate-to-rectangular HSS connection is made by passing the plate through a slot in the HSS and then welding the plate to both the front and back HSS faces to form a "through-plate connection," the nominal strength can be taken as equal to the sum of the yield-line strength of each wall (Kosteski and Packer, 2003).

K3. HSS-TO-HSS TRUSS CONNECTIONS

A 30° minimum branch angle is a practical limit for good fabrication. Smaller branch angles are possible, but prior agreement with the fabricator should be made.

The limits of applicability in Table K3.1A and Table K3.2A generally represent the parameter range over which the equations have been verified in experiments. The following limitations bear explanation.

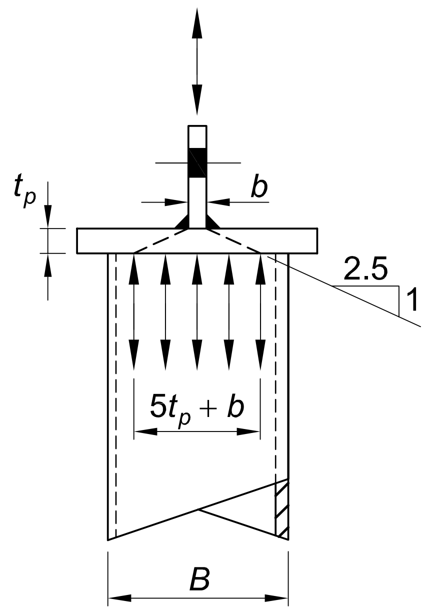


Fig. C-K2.2. Load dispersion from a concentrated force through a cap plate.

The restriction on the minimum overlap is applied so that there is an adequate interconnection of the branches to enable effective shear transfer from one branch to the other.

If the gap size in a gapped K- (or N-) connection [for example, Figure C-K1.1(a)] becomes large and exceeds the value permitted by the eccentricity limit, the K-connection should be treated as two independent Y-connections. In cross-connections, such as Figure C-K1.1(e), where the branches are close together or overlapping, the combined "footprint" of the two branches can be taken as the loaded area on the chord member. In K-connections, such as Figure C-K1.1(d), where a branch has very little or no loading, the connection can be treated as a Y-connection, as shown.

The design of welded HSS connections is based on potential limit states that may arise for particular connection geometry and loading, which in turn, represent possible failure modes that may occur within prescribed limits of applicability. Some typical failure modes for truss-type connections, shown for rectangular HSS, are given in Figure C-K3.1.

Connections in Tables K3.1 and K3.2 are for branches subject to axial loading only. Two analysis methods that will result in branches with axial loads are:

(a) Pin-jointed analysis, or

(b) Analysis using web members pin-connected to continuous chord members, as shown in Figure C-K3.2.

1. Definitions of Parameters

Some parameters are defined in Figure C-K2.1.

2. Round HSS

The wall slenderness limit for the compression branch is a restriction so that connection strength is not reduced by branch local buckling.

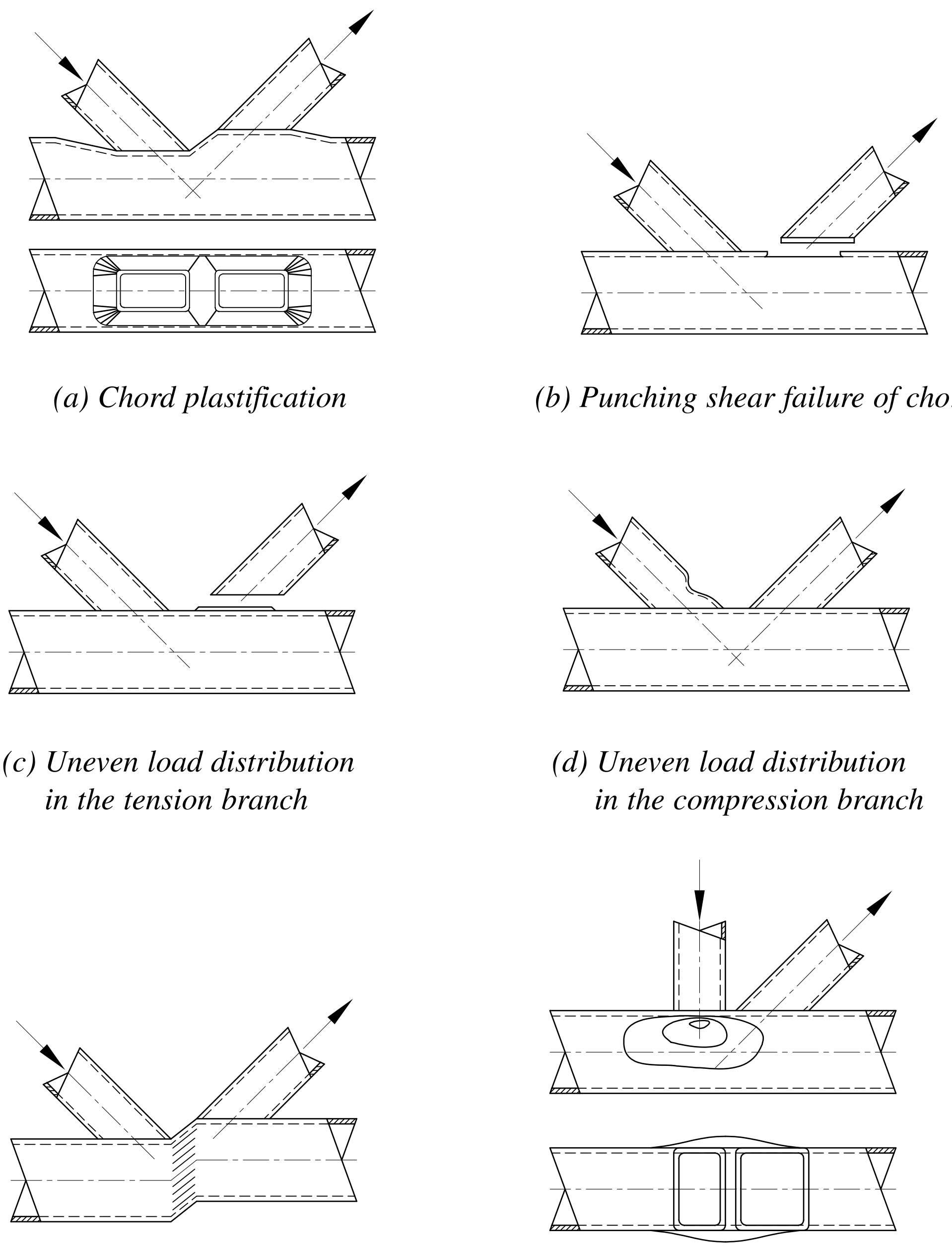
The minimum width ratio limit for gapped K-connections is based on Packer (2004), who showed that for width ratios less than 0.4, Equation K3-4 may be potentially unconservative when evaluated against proposed equations for the design of such connections by the American Petroleum Institute (API, 1993).

The restriction on the minimum gap size is stated so that adequate space is available to enable welding at the toes of the branches to be satisfactorily performed. The minimum gap limit also ensures no development of excessive load concentration and reflects the limits of testing.

The restriction on the minimum overlap is applied so that there is an adequate interconnection of the branches to enable effective shear transfer from one branch to the other.

The provisions given in Table K3.1 for T-, Y-, cross and K-connections are generally based, with the exception of the punching shear provision, on semi-empirical "characteristic strength" expressions that have a confidence of 95%, taking into account the variation in experimental test results as well as typical variations in mechanical and geometric properties. These "characteristic strength" expressions are then multiplied by resistance factors for LRFD or divided by safety factors for ASD to further allow for the relevant failure mode.

In the case of the chord plastification failure mode, $\phi = 0.90$ and $\Omega = 1.67$, whereas in the case of punching shear, $\phi = 0.95$ and $\Omega = 1.58$. For the case of punching shear, $\phi = 1.00$ (equivalent to $\Omega = 1.50$) in many recommendations or specifications [for example, IIW (1989), Wardenier et al. (1991) and Packer and Henderson (1997)] to reflect the large degree of reserve strength beyond the analytical nominal strength expression, which is itself based on the shear yield (rather than ultimate) strength of the material. In this Specification, however, $\phi = 0.95$ and $\Omega = 1.58$ are used to maintain consistency with the factors for similar failure modes in Table K3.2.



(a) Chord plastification

(b) Punching shear failure of chord

(c) Uneven load distribution in the tension branch

(d) Uneven load distribution in the compression branch

(e) Shear yielding of the chord

(f) Chord sidewall failure

Fig. C-K3.1. Typical limit states for HSS-to-HSS truss connections.

If the tensile stress, F_u, were adopted as a basis for a punching shear rupture criterion, the accompanying ϕ would be 0.75 and Ω would be 2.00, as elsewhere in this Specification. Then $0.75(0.6F_u) = 0.45F_u$ would yield a very similar value to $0.95(0.6F_y) = 0.57F_y$, and in fact the latter is even more conservative for HSS with specified nominal F_y / F_u ratios less than 0.79. Equation K3-1 need not be checked when $D_b > (D - 2t)$ because this is the physical limit at which the branch can punch into (or out of) the main tubular member.

With round HSS in axially loaded K-connections, the size of the compression branch dominates the determination of the connection strength. Hence, the term $D_{b\ comp}$ in Equation K3-4 pertains only to the compression branch and is not an average of the two branches. Thus, if one requires the connection strength expressed as a force in the tension branch, one can resolve the answer from Equation K3-4 into the direction of the tension branch, using Equation K3-5. That is, it is not necessary to repeat a calculation similar to Equation K3-4 with D_b as the tension branch.

3. Rectangular HSS

The restriction on the minimum gap ratio in Table K3.2A is modified from IIW (1989), according to Packer and Henderson (1997), to be more practical. The minimum gap size, g, is only specified so that adequate space is available to enable welding at the toes of the branches to be satisfactorily performed. The minimum gap limit also ensures no development of excessive load concentration and reflects the limits of testing.

The limit state of punching shear, evident in Equation K3-8, is based on the effective punching shear perimeter around the branch, with the total branch perimeter being an upper limit on this length. The term β_{eop} in Equation K3-8 represents the chord face

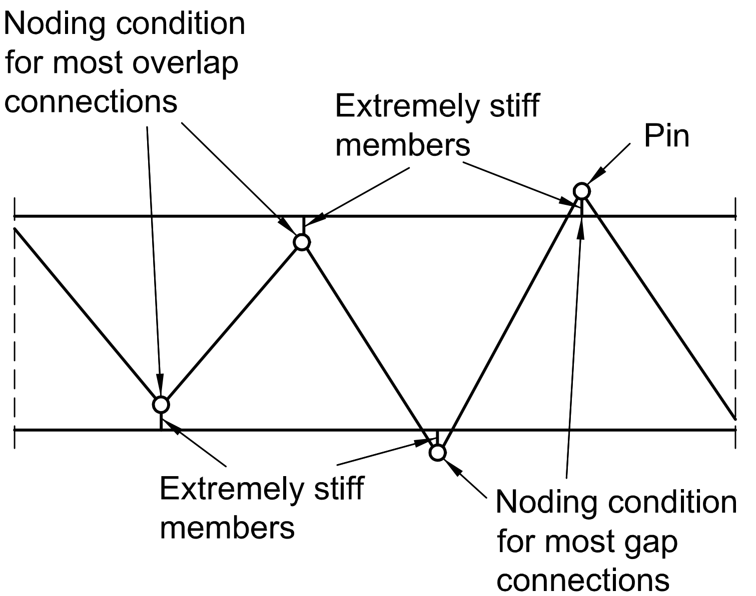


Fig. C-K3.2. Modeling assumption using web members pin-connected to continuous chord members.

effective punching shear width ratio adjacent to one of the branch walls transverse to the chord axis. This β_{eop} term incorporates $\phi = 0.80$ or $\Omega = 1.88$. Applying to generally one dimension of the rectangular branch footprint, this was deemed by AWS to be similar to a global $\phi = 0.95$ or $\Omega = 1.58$ for the whole expression, so this expression for punching shear appears in AWS (2015) with an overall $\phi = 0.95$. This $\phi = 0.95$ or $\Omega = 1.58$ has been carried over to this Specification, and this topic is discussed further in Commentary Section K3.2. The limitation specified for Equation K3-8 in Table K3.2 indicates when this failure mode is either physically impossible or noncritical. In particular, note that shear yielding is noncritical for square HSS branches.

For axially loaded, gapped K-connections, plastification of the chord connecting face under the "push-pull" action of the branches is by far the most prevalent and critical failure mode. Indeed, if all the HSS members are square, this failure mode is critical and Equation K3-7 is the only one to be checked. This formula for chord face plastification is a semi-empirical "characteristic strength" expression, which has a confidence of 95%, taking into account the variation in experimental test results as well as typical variations in mechanical and geometric properties. Equation K3-7 is then multiplied by a ϕ factor for LRFD or divided by an Ω factor for ASD, to further allow for the failure mode and provide an appropriate safety margin. A reliability calibration (Packer et al., 1984) for this equation, using a database of 263 gapped K-connections and the exponential expression for the resistance factor (with a safety index of 3.0 and a coefficient of separation of 0.55) derived a $\phi = 0.89$ and a corresponding $\Omega = 1.69$, while also imposing the parameter limits of validity. Since this failure mode dominates the test database, there is insufficient supporting test data to calibrate Equations K3-8 and K3-9.

For the limit state of shear yielding of the chord in the gap of gapped K-connections, Table K3.2 differs from international practice [for example, IIW (1989)] by recommending application of another section of this Specification—Section G4. This limit state need only be checked if the chord member is rectangular, not square, and is also oriented such that the shorter wall of the chord section lies in the plane of the truss, hence providing a more critical chord shear condition due to the short "webs." The axial force present in the gap region of the chord member may also have an influence on the shear strength of the chord sidewalls in the gap region.

For K-connections, the scope covers both gapped and overlapped connections. Note that the latter are generally more difficult and more expensive to fabricate than K-connections with a gap. However, an overlapped connection will, in general, produce a connection with a higher static strength and fatigue resistance, as well as a stiffer truss than its gapped connection counterpart.

For rectangular HSS meeting the limits of applicability in Table K3.2A, the sole failure mode to be considered for design of overlapped connections is the limit state of uneven load distribution in the branches, manifested by either local buckling of the compression branch or premature yield failure of the tension branch. The design procedure presumes that one branch is welded solely to the chord and hence only has a single cut at its end. This can be considered good practice and the "thru member" is termed the overlapped member. For partial overlaps of less than 100%, the other branch is then double-cut at its end and welded to both the thru branch as well as the chord.

The branch to be selected as the "thru" or overlapped member should be the one with the larger overall width. If both branches have the same width, the thicker branch should be the overlapped branch.

For a single failure mode to be controlling (and not have failure by one branch punching into or pulling out of the other branch, for example), limits are placed on various connection parameters, including the relative width and relative thickness of the two branches. The foregoing fabrication advice for rectangular HSS also pertains to round HSS overlapped K-connections, but the latter involves more complicated profiling of the branch ends to provide good saddle fits.

Overlapped rectangular HSS K-connection strength calculations (Equations K3-10, K3-11 and K3-12) are performed initially just for the overlapping branch, regardless of whether it is in tension or compression, and then the resistance of the overlapped branch is determined from that. The equations for connection strength, expressed as a force in a branch, are based on the load-carrying contributions of the four side walls of the overlapping branch and follow the design recommendations of the International Institute of Welding (IIW, 1989; Packer and Henderson, 1997; AWS, 2015). The effective widths of overlapping branch member walls transverse to the chord, B_e, depend on the flexibility of the surface on which they land, and are derived from plate-to-HSS effective width measurements (Rolloos, 1969; Wardenier et al., 1981; Davies and Packer, 1982).

The applicability of Equations K3-10, K3-11 and K3-12 depends on the amount of overlap, O_v, where $O_v = (l_{ov} / l_p) \times 100$. It is important to note that l_p is the projected length (or imaginary footprint) of the overlapping branch on the connecting face of the chord, even though it does not physically contact the chord. Also, l_{ov} is the overlap length measured along the connecting face of the chord beneath the region of overlap of the branches. This is illustrated in Figure C-K2.1.

A maximum overlap of 100% occurs when one branch sits completely on the other branch. In such cases, the overlapping branch is sometimes moved slightly up the overlapped branch so that the heel of the overlapping branch can be fillet welded to the face of the overlapped branch. If the connection is fabricated in this manner, an overlap slightly greater than 100% is created. In such cases, the connection strength for a rectangular HSS connection can be calculated by Equation K3-12 but with the B_{bi} term replaced by another B_e term. Also, with regard to welding details, it has been found experimentally that it is permissible to just tack weld the "hidden toe" of the overlapped branch, providing that the components of the two branch member forces normal to the chord substantially balance each other and providing that the welds are designed for the yield capacity of the connected branch walls. The "hidden toe" should be fully welded to the chord if the normal components of the two branch forces differ by more than 20% or the welds to the branches are designed using an effective length approach. More discussion is provided in Commentary Section K5. If the components of the two branch forces normal to the chord do in fact differ significantly, the connection should also be checked for behavior as a T-, Y- or cross-connection, using the combined footprint and the net force normal to the chord (see Figure C-K3.3).

For the design of round branches connecting to rectangular chords in T-, Y-, X- and K-gapped connections under static loading, a conversion method can be used to check chord wall plastification if the branch to chord width ratio, D_b/B, is less than 0.85. Supported by Packer et al. (2007), the conversion involves the replacement of the round branch (or branches) of diameter D_b by equivalent square branches of width $B_b = \pi D_b/4$ and the same thickness; then the design rule for chord wall plastification in rectangular HSS-to-rectangular HSS connections in Table K3.2 can be applied to round HSS-to-rectangular HSS connections. For round HSS-to-rectangular HSS K-overlapped connections, the conversion method can be used if the chord width ratio, D_b/B, is less than 0.8 to check local yielding of branch/branches due to uneven load distribution for $\beta \geq 0.25$. Many failure modes for HSS connections depend on the perimeter or cross-sectional area of the branch member, and both the perimeter and area of a round HSS, when compared to that of a square HSS, have a ratio of $\pi:4$.

K4. HSS-TO-HSS MOMENT CONNECTIONS

Section K4 on HSS-to-HSS connections under moment loading is applicable to frames with partially restrained or fully restrained moment connections, such as Vierendeel girders. The provisions of Section K4 are not generally applicable to typical planar triangulated trusses, which are covered by Section K3, because the latter should be analyzed in a manner that results in no bending moments in the web members (see Commentary Section K3). Thus, K-connections with moment loading on the branches are not covered by this Specification.

Available testing for HSS-to-HSS moment connections is much less extensive than that for axially-loaded T-, Y-, cross- and K-connections. Hence, the governing limit states to be checked for axially loaded connections can be used as a basis for the possible limit states in moment-loaded connections. Thus, the design criteria for round

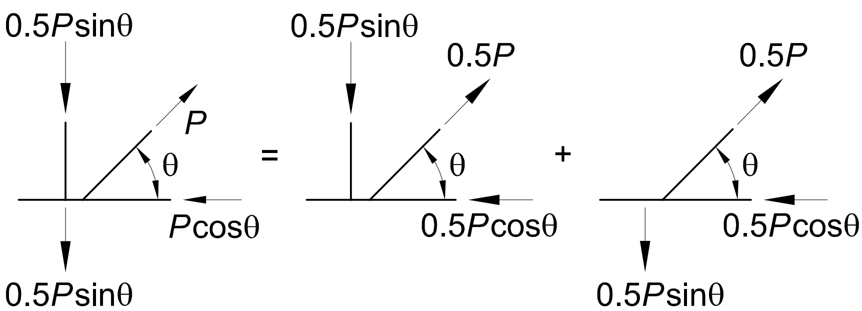


Fig. C-K3.3. Checking of K-connection with imbalanced branch member loads.

HSS moment connections are based on the limit states of chord plastification and punching shear failure, with ϕ and Ω factors consistent with Section K3, while the design criteria for rectangular HSS moment connections can be based on the limit states of plastification of the chord connecting face, uneven load distribution, and chord sidewall local yielding, crippling and buckling. The "chord distortional failure" mode is applicable only to rectangular HSS T-connections with an out-of-plane bending moment on the branch. Rhomboidal distortion of the branch can be prevented by the use of stiffeners or diaphragms to maintain the rectangular cross-sectional shape of the chord. The limits of applicability of the equations in Section K4 are predominantly reproduced from Section K3. The equations in Section K4 have also been adopted in CIDECT Design Guide No. 9 (Kurobane et al., 2004).

K5. WELDS OF PLATES AND BRANCHES TO RECTANGULAR HSS

Section K5 consolidates all the welding rules for plates and branch members to the face of an HSS into one section.

Due to differences in relative flexibilities of main members loaded normal to its surface and the branch member carrying membrane forces parallel to its surface, transfer of load across the weld is highly nonuniform and local yielding can be expected before the connection reaches its design load.

To prevent progressive failure of the weld and ensure ductile behavior of the joint, simple T-,Y- and K-connection welds shall be capable of developing at their ultimate strength the branch member yield strength. This requirement is presumed to be satisfied when using matching filler metal and either the prequalified joint details in AWS D1.1/D1.1M (AWS, 2015) for T-, Y- and K-connections, or when the effective throat of the fillet weld is equal to 1.1 times the branch member thickness for branch members with $F_y \leq 50$ ksi (345 MPa) per Eurocode 3 (CEN, 2005a).

Alternately, welds of rectangular hollow structural sections (RHSS) may be designed as "fit for purpose" to resist branch forces that are typically known in RHSS truss-type connections by using what is known as the "effective length concept." Many HSS truss web members are subjected to low axial loads and in such situations, this weld design philosophy is ideal. However, the nonuniform loading of the weld perimeter due to the flexibility of the connecting HSS face must be taken into account by using weld effective lengths. Suitable effective lengths for plates and various rectangular HSS connections subject to branch axial loading (and/or moment loading in some cases) are given in Table K5.1. Several of these provisions are similar to those given in AWS (2015) and are based on full-scale HSS connection and truss tests that studied weld failures (Frater and Packer, 1992a, 1992b; Packer and Cassidy, 1995).

Effective lengths used to determine weld sizing for T-, Y- and cross connections with moments and for overlapped connections are based on a rational extrapolation of the effective length concept used for design of the member itself. Diagrams that show the locations of the effective weld lengths (most of which are less than 100% of the total weld length) are shown in Table K5.1.

The effective length approach to weld design recognizes that a branch-to-main member connection becomes stiffer along its edges, relative to the center of the HSS face, as the angle of the branch to the connecting face and/or the width ratio (the width of a branch member relative to the connecting face) increase. Thus, the effective length used for sizing the weld may decrease as either the angle of the branch member (when over 50° relative to the connecting face) or the branch member width (creating width ratios over 0.85) increase. Note that for ease of calculation and because the error is insignificant, the weld corners were assumed as square for determination of the weld line section properties in certain cases.

As noted in Commentary Section K3, when the welds in overlapped joints are adequate to develop the strength of the remaining member walls, it has been found experimentally that it is permissible to eliminate the weld on the "hidden toe" of the overlapped branch, provided that the components of the two branch member forces normal to the chord substantially balance each other. The "hidden toe" should be fully welded to the chord if the normal components of the two branch forces differ by more than 20%. If the "fit for purpose" weld design philosophy is used in an overlapped joint, the hidden weld should be completed even though the effective weld length may be much less than the perimeter of the HSS. This helps account for the moments that can occur in typical HSS connections due to joint rotations and face deformations, but are not directly accounted for in design.

Until further investigation proves otherwise, directional strength increases typically used in the design of fillet welds are not allowed in Section K5 when welding to the face of HSS members in truss-type connections. Additionally, the design weld size in all cases shown in Table K5.1, including the hidden weld underneath an overlapped member as discussed in the foregoing, is the smallest weld throat around the connection perimeter; adding up the strengths of individual sections of a weld group with varying throat sizes around the perimeter of the cross section is not a viable approach to HSS connection design.

CHAPTER L

DESIGN FOR SERVICEABILITY

L1. GENERAL PROVISIONS

Serviceability limit states are conditions in which the functions of a building are impaired because of local damage, deterioration or deformation of building components, or occupant discomfort. While serviceability limit states generally do not involve collapse of a building, loss of life, or injury, they can seriously impair the usefulness of a building and lead to costly repairs and other economic consequences. Serviceability provisions are essential to provide satisfactory performance of building structural systems. Neglect of serviceability may result in structures that are excessively flexible or otherwise perform unacceptably in service.

The general types of structural behavior that are indicative of impaired serviceability in steel structures are:

(1) Excessive deflections or rotations that may affect the appearance, function, or drainage of the building, or may cause damaging transfer of load to nonstructural components and attachments

(2) Excessive drift due to wind that may damage cladding and nonstructural walls and partitions

(3) Excessive vibrations produced by the activities of the building occupants or mechanical equipment, that may cause occupant discomfort or malfunction of building service equipment

(4) Excessive wind-induced motions that may cause occupant discomfort

(5) Excessive effects of expansion and contraction caused by temperature differences as well as creep and shrinkage of concrete and yielding of steel

(6) Effects of connection slip, resulting in excessive deflections and rotations that may have deleterious effects similar to those produced by load effects

In addition, excessive local damage (local yielding, buckling, slip or cracking) or deterioration (weathering, corrosion and discoloration) may also affect the function and serviceability of the structure during its service life.

Serviceability limit states depend on the occupancy or function of the building, the perceptions of its occupants, and the type of structural system. Limiting values of structural behavior intended to provide adequate levels of serviceability should be determined by a team consisting of the building owner/developer, the architect, and the structural engineer after a careful analysis of all functional and economic requirements and constraints. In arriving at serviceability limits, the team should recognize that building occupants are able to perceive structural deformations, motions, cracking, or other signs of distress at levels that are much lower than those that would indicate impending structural damage or failure. Such signs of distress may be viewed as an indication that the building is unsafe and diminish its economic value and, therefore, must be considered at the time of design.

Service loads that may require consideration in checking serviceability include: (1) static loads from the occupants, snow or rain on the roof, or temperature fluctuations; and (2) dynamic loads from human activities, wind effects, the operation of mechanical or building service equipment, or traffic near the building. Service loads are loads that act on the structure at an arbitrary point in time and may be only a fraction of the corresponding nominal load. The response of the structure to service loads generally can be analyzed assuming elastic behavior. Members that accumulate residual deformations under service loads also may require examination with respect to this long-term behavior.

Serviceability limit states and appropriate load combinations for checking conformance to serviceability requirements can be found in ASCE/SEI 7, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, Section 1.3.2, Commentary 1.3.2, Appendix C, and Commentary Appendix C (ASCE, 2016).

L2. DEFLECTIONS

Excessive vertical deflections and misalignment arise primarily from three sources: (a) gravity loads, such as dead, live and snow loads; (b) effects of temperature, creep and differential settlement; and (c) construction tolerances and errors. Such deformations may be visually objectionable; cause separation, cracking or leakage of exterior cladding, doors, windows and seals; and cause damage to interior components and finishes. Appropriate limiting values of deformations depend on the type of structure, detailing and intended use (Galambos and Ellingwood, 1986). Historically, common deflection limits for horizontal members have been 1/360 of the span for floors subjected to reduced live load and 1/240 of the span for roof members. Deflections of about 1/300 of the span (for cantilevers, 1/150 of the length) are visible and may lead to general architectural damage or cladding leakage. Deflections greater than 1/200 of the span may impair operation of moveable components such as doors, windows and sliding partitions.

Deflection limits depend very much on the function of the structure and the nature of the supported construction. Traditional limits expressed as a fraction of the span length should not be extrapolated beyond experience. For example, the traditional limit of 1/360 of the span worked well for controlling cracks in plaster ceilings with spans common in the first half of the twentieth century. Many structures with more flexibility have performed satisfactorily with the now common, and more forgiving, ceiling systems. On the other hand, with the advent of longer structural spans, serviceability problems have been observed with flexible grid ceilings where actual deflections were far less than 1/360 of the span, because the distance between partitions or other elements that may interfere with ceiling deflection are far less than the span of the structural member. Proper control of deflections is a complex subject requiring careful application of professional judgment. AISC Design Guide 3, *Serviceability Design Considerations for Steel Buildings*, 2nd Edition (West and Fisher, 2003) provide an extensive discussion of the issues.

Deflection computations for composite beams should include an allowance for slip, creep and shrinkage as discussed in Commentary Section I3.

In certain long-span floor systems, it may be necessary to place a limit, independent of span, on the maximum deflection to minimize the possibility of damage of adjacent nonstructural elements (ISO, 1977). For example, damage to non-load-bearing partitions may occur if vertical deflections exceed more than about $^3/_8$ in. (10 mm) unless special provision is made for differential movement (Cooney and King, 1988); however, many components can and do accept larger deformations.

Load combinations for checking static deflections can be developed using first-order reliability analysis (Galambos and Ellingwood, 1986). Current static deflection guidelines for floor and roof systems are adequate for limiting superficial damage in most buildings. A combined load with an annual probability of being exceeded of 5% is appropriate in most instances. For serviceability limit states involving visually objectionable deformations, repairable cracking, or other damage to interior finishes, and other short-term effects, the suggested load combinations are:

$$D + L$$

$$D + 0.5S$$

For serviceability limit states involving creep, settlement or similar long-term or permanent effects, the suggested load combination is:

$$D + 0.5L$$

The dead load effect, D, may be that portion of dead load that occurs following attachment of nonstructural elements. For example, in composite construction, the dead load effects frequently are taken as those imposed after the concrete has cured. For ceiling related calculations, the dead load effects may include only those loads placed after the ceiling structure is in place.

L3. DRIFT

Drift (lateral deflection) in a steel building is a serviceability issue primarily from the effects of wind. Drift limits are imposed on buildings to minimize damage to cladding and to nonstructural walls and partitions. Lateral frame deflection is evaluated for the building as a whole, where the applicable parameter is the total building drift, defined as the lateral frame deflection at the top of the most occupied floor divided by the height of the building to that level, Δ / H. For each floor, the applicable parameter is interstory drift, defined as the lateral deflection of a floor relative to the lateral deflection of the floor immediately below, divided by the distance between floors, $(\delta_n - \delta_{n-1}) / h$.

Typical drift limits in common usage vary from $H/100$ to $H/600$ for total building drift and $h/200$ to $h/600$ for interstory drift, depending on building type and the type of cladding or partition materials used. The most widely used values are H (or h)/400 to H (or h)/500 (ASCE, 1988). These limits generally are sufficient to minimize damage to cladding and nonstructural walls and partitions. Smaller drift limits may be appropriate if the cladding is brittle. AISC Design Guide 3 (West and

Fisher, 2003) contains recommendations for higher drift limits that have successfully been used in low-rise buildings with various cladding types. It also contains recommendations for buildings containing cranes. An absolute limit on interstory drift is sometimes imposed by designers in light of evidence that damage to nonstructural partitions, cladding and glazing may occur if the interstory drift exceeds about $^3/_8$ in. (10 mm), unless special detailing practices are employed to accommodate larger movements (Cooney and King, 1988; Freeman, 1977). Many components can accept deformations that are significantly larger. More specific information on the damage threshold for building materials is available in the literature (Griffis, 1993).

It is important to recognize that frame racking or shear distortion is the real cause of damage to building elements such as cladding and partitions. Lateral drift only captures the horizontal component of the racking and does not include potential vertical racking, as from differential column shortening in tall buildings, which also contributes to damage. Moreover, some lateral drift may be caused by rigid body rotation of the cladding or partition which by itself does not cause strain and, therefore, damage. A more precise parameter, the drift damage index used to measure the potential damage, has been proposed (Griffis, 1993).

It must be emphasized that a reasonably accurate estimate of building drift is essential to controlling damage. The structural analysis must capture all significant components of potential frame deflection, including flexural deformation of beams and columns, axial deformation of columns and braces, shear deformation of beams and columns, beam-column joint rotation (panel-zone deformation), the effect of member joint size, and the P-Δ effect (Charney, 1990). For many low-rise steel frames with normal bay widths of 30 to 40 ft (9 to 12 m), use of center-to-center dimensions between columns without consideration of actual beam-to-column joint size and panel zone effects will usually suffice for checking drift limits. The stiffening effect of nonstructural cladding, walls and partitions may be taken into account if substantiating information (stress versus strain behavior) regarding their effect is available.

The level of wind load used in drift limit checks varies among designers depending upon the frequency with which the potential damage can be tolerated. Many designers use a 50-year, 20-year or 10-year mean recurrence interval wind load when checking serviceability limit states (Griffis, 1993; ASCE, 2016).

It is important to recognize that drift control limits by themselves, in wind-sensitive buildings, do not provide comfort of the occupants under wind load. See Section L5 for additional information regarding perception of motion in wind sensitive buildings.

L4. VIBRATION

The increasing use of high-strength materials with efficient structural systems and open plan architectural layouts leads to longer spans and more flexible floor systems having less damping. Therefore, floor vibrations have become an important design consideration. Acceleration is the recommended standard for evaluation.

An extensive treatment of vibration in steel-framed floor systems and pedestrian bridges is found in AISC Design Guide 11, *Vibrations of Steel-Framed Structural*

Systems Due to Human Activity (Murray et al., 2016). This guide provides basic principles and simple analytical tools to evaluate steel-framed floor systems and footbridges for vibration serviceability due to human activities, including walking and rhythmic activities. Both human comfort and the need to control movement for sensitive equipment are considered.

L5. WIND-INDUCED MOTION

Designers of wind-sensitive buildings have long recognized the need for controlling annoying vibrations under the action of wind to protect the psychological well-being of the occupants (Chen and Robertson, 1972). The perception of building motion under the action of wind may be described by various physical quantities including maximum displacement, velocity, acceleration, and rate of change of acceleration (sometimes called "jerk"). Acceleration has become the standard for evaluation because it is readily measured in the field and can be easily calculated analytically. Human response to building motion is a complex phenomenon involving many psychological and physiological factors. Perception and tolerance thresholds of acceleration as a measure of building motion are known to depend on factors such as frequency of the building, occupant gender, age, body posture (sitting, standing or reclining), body orientation, expectation of motion, body movement, visual cues, acoustic cues, and the type of motion (translational or torsional) (ASCE, 1981). Different thresholds and tolerance levels exist for different people and responses can be very subjective. It is known that some people can become accustomed to building motion and tolerate higher levels than others. Limited research exists on this subject, but certain standards have been applied for design as discussed in the following.

Acceleration in wind-sensitive buildings may be expressed as either root mean square (RMS) or peak acceleration. Both measures are used in practice and there is no clear agreement as to which is the more appropriate measure of motion perception. Some researchers believe that peak acceleration during wind storms is a better measure of actual perception but that RMS acceleration during the entire course of a wind storm is a better measure of actual discomfort. Target peak accelerations of 21 milli-g (0.021 times the acceleration of gravity) for commercial buildings (occupied mostly during daylight hours) and 15 milli-g for residential buildings (occupied during the entire day) under a 10-year mean recurrence interval wind storm have been successfully used in practice for many tall building designs (Griffis, 1993). The target is generally more strict for residential buildings because of the continuous occupancy, the perception that people are less sensitive and more tolerant at work than at home, the fact that there is more turnover in commercial buildings, and the fact that commercial buildings are more easily evacuated for peak wind events. Peak acceleration and RMS acceleration in wind-sensitive buildings are related by the "peak factor" best determined in a wind tunnel study and generally in the range of 3.5 for tall buildings (in other words, peak acceleration = peak factor × RMS acceleration). Guidance for design acceleration levels used in building design may be found in the literature (Chen and Robertson, 1972; Hansen et al., 1973; Irwin, 1986; NRCC, 1990; Griffis, 1993).

It is important to recognize that perception to building motion is strongly influenced by building mass and available damping as well as stiffness (Vickery et al., 1983). For this reason, building drift limits by themselves should not be used as the sole measure of controlling building motion (Islam et al., 1990). Damping levels for use in evaluating building motion under wind events are generally taken as approximately 1% of critical damping for steel buildings.

L6. THERMAL EXPANSION AND CONTRACTION

The satisfactory accommodation of thermal expansion and contraction cannot be reduced to a few simple rules, but must depend largely upon the judgment of a qualified engineer. The problem is likely to be more serious in buildings with masonry walls than with prefabricated units. Complete separation of the framing at widely spaced expansion joints is generally more satisfactory than more frequently located devices that depend upon the sliding of parts in bearing, and usually less expensive than rocker or roller expansion bearings.

Creep and shrinkage of concrete and yielding of steel are among the causes, other than temperature, for dimensional changes. Conditions during construction, such as temperature effects before enclosure of the structure, should also be considered. Engineers may also consider that damage to building cladding can cause water penetration and may lead to corrosion. Guidelines for the recommended size and spacing of expansion joints in buildings may be found in NRC (1974).

L7. CONNECTION SLIP

In bolted connections with bolts in holes having only small clearances, such as standard holes and slotted holes loaded transversely to the axis of the slot, the amount of possible slip is small. Slip at these connections is not likely to have serviceability implications. Possible exceptions include certain unusual situations where the effect of slip is magnified by the configuration of the structure, such as a connection at the base of a shallow cantilever beam or post where a small amount of bolt slip may produce unacceptable rotation and deflection.

This Specification requires that connections with oversized holes or slotted holes loaded parallel to the axis of the slot be designed as slip-critical connections. For a discussion of slip at these connections, see Commentary Section J3.8. Where slip at service loads is a realistic possibility in these connections, the effect of connection slip on the serviceability of the structure must be considered.

CHAPTER M

FABRICATION AND ERECTION

M1. SHOP AND ERECTION DRAWINGS

Supplementary information relevant to shop drawing documentation and associated fabrication, erection and inspection practices may be found in the AISC *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2016a) and in Schuster (1997).

M2. FABRICATION

1. Cambering, Curving and Straightening

In addition to mechanical means, local application of heat is permitted for curving, cambering and straightening. Maximum temperatures are specified to avoid metallurgical damage and inadvertent alteration of mechanical properties: for ASTM A514/A514M and A852/A852M steels, the maximum is 1,100°F (590°C); for other steels, the maximum is 1,200°F (650°C). In general, these should not be viewed as absolute maximums; they include an allowance for a variation of about 100°F (38°C), which is a common range achieved by experienced fabricators (FHWA, 1999).

Temperatures should be measured by appropriate means, such as temperature-indicating crayons and steel color. Precise temperature measurements are seldom called for. Also, surface temperature measurements should not be made immediately after removing the heating torch because it takes a few seconds for the heat to soak into the steel.

Local application of heat has long been used as a means of straightening or cambering beams and girders. With this method, selected zones are rapidly heated and tend to expand. But the expansion is resisted by the restraint provided by the surrounding unheated areas. Thus, the heated areas are "upset" (increase in thickness) and, upon cooling, they shorten to effect a change in curvature. In the case of trusses and girders, cambering can be built in during assembly of the component parts.

Although the desired curvature or camber can be obtained by these various methods, including at room temperature (cold cambering) (Bjorhovde, 2006), it must be realized that some deviation due to workmanship considerations, as well as some permanent change due to handling, is inevitable. Camber is usually defined by one mid-ordinate, because control of more than one point is difficult and not normally needed. Reverse cambers are difficult to achieve and are discouraged. Long cantilevers are sensitive to camber and may deserve closer control.

2. Thermal Cutting

Thermal cutting is preferably done by machine. The requirement in Section M2.2 for preheat before thermal cutting is to minimize the creation of a hard surface layer and the formation of cracks. This requirement for preheat for thermal cutting does not apply when the radius portion of the access hole or cope is drilled and the thermally cut portion is essentially linear. Such thermally cut surfaces are required to be ground in accordance with Section J1.6. After welding, the weld access hole surface is to be visually inspected in accordance with Table N5.4-3. The surface resulting from two straight torch cuts meeting at a point is not considered to be a curve.

4. Welded Construction

To avoid weld contamination, the light oil coating that is generally present after manufacturing an HSS should be removed with a suitable solvent in locations where welding will be performed. In cases where an external coating has been applied at the mill, the coating should be removed at the location of welding or the manufacturer should be consulted regarding the suitability of welding in the presence of the coating.

5. Bolted Construction

In most connections made with high-strength bolts, it is only required to install the bolts to the snug-tight condition. This includes bearing-type connections where slip is permitted and, for ASTM F3125 Grade A325 or A325M bolts only, tension (or combined shear and tension) applications where loosening or fatigue due to vibration or load fluctuations are not design considerations.

It is suggested that snug-tight bearing-type connections with ASTM F3125 Grade A325 or A325M or ASTM F3125 Grade A490 or A490M bolts be used in applications where ASTM A307 bolts are permitted.

This section provides rules for the use of oversized and slotted holes paralleling the provisions that have been in the RCSC *Specification for High-Strength Bolts* since 1972 (RCSC, 2014), extended to include ASTM A307 bolts, which are outside the scope of the RCSC *Specification*.

The Specification previously limited the methods used to form holes, based on common practice and equipment capabilities. Fabrication methods have changed and will continue to do so. To reflect these changes, this Specification has been revised to define acceptable quality instead of specifying the method used to form the holes, and specifically to permit thermally cut holes. AWS C4.1, Sample 3, is useful as an indication of the thermally cut profile that is acceptable (AWS, 2015). The use of numerically controlled or mechanically guided equipment is anticipated for the forming of thermally cut holes. To the extent that the previous limits may have related to safe operation in the fabrication shop, fabricators are referred to equipment manufacturers for equipment and tool operating limits.

10. Drain Holes

Because the interior of an HSS is difficult to inspect, concern is sometimes expressed regarding internal corrosion. However, good design practice can eliminate the concern and the need for expensive protection. Corrosion occurs in the presence of oxygen and water. In an enclosed building, it is improbable that there would be sufficient reintroduction of moisture to cause severe corrosion. Therefore, internal corrosion protection is a consideration only in HSS that are exposed to weather. In a sealed HSS, internal corrosion cannot progress beyond the point where the oxygen or moisture necessary for chemical oxidation is consumed (AISI, 1970). The oxidation depth is insignificant when the corrosion process must stop, even when a corrosive atmosphere exists at the time of sealing. If fine openings exist at connections, moisture and air can enter the HSS through capillary action or by aspiration due to the partial vacuum that is created if the HSS is cooled rapidly (Blodgett, 1967). This can be prevented by providing pressure-equalizing holes in locations that make it impossible for water to flow into the HSS by gravity.

Situations where an internal protective coating may be required include (1) open HSS where changes in the air volume by ventilation or direct flow of water is possible, and (2) open HSS subject to a temperature gradient that causes condensation. In such instances, it may also be prudent to use a minimum $^5/_{16}$-in. (8 mm) wall thickness.

HSS that are filled or partially filled with concrete should not be sealed. In the event of fire, water in the concrete will vaporize and may create pressure sufficient to burst a sealed HSS. Care should be taken to ensure that water does not remain in the HSS during or after construction, since the expansion for HSS exposed to freezing can create pressure that is sufficient to burst an HSS.

Galvanized HSS assemblies should not be completely sealed because rapid pressure changes during the galvanizing process tend to burst sealed assemblies.

11. Requirements for Galvanized Members

Cracking has been observed in steel members during hot-dip galvanizing. The occurrence of these cracks has been correlated to several characteristics including, but not limited to, highly restrained details, base material chemistry, galvanizing practices, and fabrication workmanship. The requirement to grind beam copes before galvanizing will not prevent all cope cracks from occurring during galvanizing. However, it has been shown to be an effective means to reduce the occurrence of this phenomenon.

Galvanizing of structural steel and hardware, such as fasteners, is a process that depends on special design, detailing and fabrication to achieve the desired level of corrosion protection. ASTM publishes the following standards related to galvanized structural steel.

ASTM A123 (ASTM, 2015c) provides a standard for the galvanized coating and its measurement, and includes provisions for the materials and fabrication of the products to be galvanized.

ASTM A153/153M (ASTM, 2009a) is a standard for galvanized hardware, such as fasteners, that are to be centrifuged.

ASTM A384/384M (ASTM, 2013b) is the *Standard Practice for Safeguarding Against Warpage and Distortion During Hot-Dip Galvanizing of Steel Assemblies*. It includes information on factors that contribute to warpage and distortion as well as suggestions for correction for fabricated assemblies.

ASTM A385/385M (ASTM, 2015a) is the *Standard Practice for Providing High Quality Zinc Coatings* (*Hot-Dip*). It includes information on base materials, venting, treatment of contacting surfaces, and cleaning. Many of these provisions should be indicated on the design and detail drawings.

ASTM A780/A780M (ASTM, 2015b) provides for repair of damaged and uncoated areas of hot-dip galvanized coatings.

M3. SHOP PAINTING

1. General Requirements

The surface condition of unpainted steel framing of long-standing buildings that have been demolished has been found to be unchanged from the time of its erection, except at isolated spots where leakage may have occurred. Even in the presence of leakage, the shop coat is of minor influence (Bigos et al., 1954).

This Specification does not define the type of paint to be used when a shop coat is required. Final exposure and individual preference with regard to finish paint are factors that determine the selection of a proper primer. A comprehensive treatment of the subject is found in various SSPC publications.

3. Contact Surfaces

Special concerns regarding contact surfaces of HSS should be considered. As a result of manufacturing, a light oil coating is generally present on the outer surface of the HSS. If paint is specified, HSS must be cleaned of this oil coating with a suitable solvent.

5. Surfaces Adjacent to Field Welds

This Specification allows for welding through surface materials, including appropriate shop coatings that do not adversely affect weld quality nor create objectionable fumes.

M4. ERECTION

2. Stability and Connections

For information on the design of temporary lateral support systems and components for low-rise buildings, see AISC Design Guide 10, *Erection Bracing of Low-Rise Structural Steel Buildings* (Fisher and West, 1997).

4. Fit of Column Compression Joints and Base Plates

Tests on spliced full-size columns with joints that had been intentionally milled out-of-square, relative to either the strong- or weak-axis, demonstrated that the load-carrying capacity was the same as that for similar columns without splices (Popov and Stephen, 1977). In the tests, gaps of $^1/_{16}$ in. (2 mm) were not shimmed; gaps of $^1/_4$ in. (6 mm) were shimmed with nontapered mild steel shims. Minimum size partial-joint-penetration groove welds were used in all tests. No tests were performed on specimens with gaps greater than $^1/_4$ in. (6 mm).

5. Field Welding

The Specification incorporates AWS D1.1/D1.1M (AWS, 2015) by reference. Surface preparation requirements are defined in that code. The erector is responsible for repair of routine damage and corrosion occurring after fabrication. Welding on coated surfaces demands consideration of quality and safety. Wire brushing has been shown to result in adequate quality welds in many cases. Erector weld procedures accommodate project site conditions within the range of variables normally used on structural steel welding. Welds to material in contact with concrete and welded assemblies in which shrinkage may add up to a substantial dimensional variance may be improved by judicious selection of weld procedure variables and fit up. These conditions are dependent on other variables such as the condition and content of the concrete and the design details of the welded joint. The range of variables permitted in the class of weld procedures, considered to be prequalified in the process used by the erector, is the range normally used.

CHAPTER N

QUALITY CONTROL AND QUALITY ASSURANCE

This chapter on quality control and quality assurance does not address a number of applications associated with structural steel. The following is a list of references that may help with quality control and quality assurance for some of these items:

(1) Steel (open web) joists and joist girders—Each model specification of the Steel Joist Institute contains a section on quality.

(2) Concrete reinforcing bars, concrete materials, or placement of concrete for composite members—ACI 318 and ACI 318M (ACI, 2014).

(3) Surface preparations for painting or coatings—SSPC *Painting Manual, Volumes 1 and 2* (SSPC, 2002, 2012).

N1. GENERAL PROVISIONS

This chapter provides minimum requirements for quality control (QC), quality assurance (QA) and nondestructive testing (NDT) for structural steel systems for buildings and other structures. Chapter N also addresses the inspection of field installed shear stud connectors of composite slab construction that are frequently within the scope of the fabricator and/or erector. The inspection requirements for the other elements of composite construction, such as concrete, formwork, reinforcement, and the related dimensional tolerances, are addressed elsewhere. Three publications of the American Concrete Institute may be applicable. These are ACI 117-10, *Specifications for Tolerances for Concrete Construction and Commentary* (ACI, 2010a), ACI 301-10, *Specifications for Structural Concrete* (ACI, 2010b), and ACI 318 and ACI 318M, *Building Code Requirements for Structural Concrete and Commentary* (ACI, 2014). Minimum observation and inspection tasks deemed necessary to ensure quality structural steel construction are defined.

This chapter also defines a comprehensive system of "Quality Control" requirements on the part of the steel fabricator and erector and similar requirements for "Quality Assurance" on the part of the project owner's representatives when such is deemed necessary to complement the contractor's quality control function. These requirements exemplify recognized principles of developing involvement of all levels of management and the workforce in the quality control process as the most effective method of achieving quality in the constructed product. The chapter supplements these quality control requirements with quality assurance responsibilities as are deemed suitable for a specific task. The requirements follow the same requirements for inspections utilized in AWS D1.1/D1.1M (AWS, 2015) and the RCSC *Specification* (RCSC, 2014).

Under AISC *Code of Standard Practice* Section 8 (AISC, 2016a), the fabricator or erector is to implement a QC system as part of their normal operations. Those that participate in AISC Quality Certification or similar programs are required to develop QC systems as part of those programs. The engineer of record should evaluate what is already a part of the fabricator's or erector's QC system in determining the QA needs for each project. Where the fabricator's or erector's QC system is considered adequate for the project, including compliance with any specific project needs, the special inspection or quality assurance plan may be modified to reflect this. Similarly, where additional needs are identified, supplementary requirements should be specified.

The terminology adopted is intended to provide a clear distinction between fabricator and erector requirements and the requirements of others. The definitions of QC and QA used here are consistent with usage in related industries, such as the steel bridge industry, and they are used for the purposes of this Specification. It is recognized that these definitions are not the only definitions in use. For example, QC and QA are defined differently in the AISC Quality Certification program in a fashion that is useful to that program and are consistent with the International Standards Organization and the American Society for Quality.

For the purposes of this Specification, QC includes those tasks performed by the steel fabricator and erector that have an effect on quality or are performed to measure or confirm quality. QA tasks performed by organizations other than the steel fabricator and erector are intended to provide a level of assurance that the product meets the project requirements.

The terms quality control and quality assurance are used throughout this Chapter to describe inspection tasks required to be performed by the steel fabricator and erector and project owner's representatives, respectively. The QA tasks are inspections often performed when required by the applicable building code or authority having jurisdiction (AHJ), and designated as "Special Inspections," or as otherwise required by the project owner or engineer of record.

Chapter N defines two inspection levels for required inspection tasks and labels them as either "observe" or "perform." The choice in terminology reflects the multi-task nature of welding and high-strength bolting operations, and the required inspections during each specific phase.

N2. FABRICATOR AND ERECTOR QUALITY CONTROL PROGRAM

Many quality requirements are common from project to project. Many of the processes used to produce structural steel have an effect on quality and are fundamental and integral to the fabricator's or erector's success. Consistency in imposing quality requirements between projects facilitates more efficient procedures for both.

The construction documents referred to in this chapter are, of necessity, the versions of the design drawings, specifications, and approved shop and erection drawings that have been released for construction, as defined in the AISC *Code of Standard Practice* (AISC, 2016a). When responses to requests for information and change orders exist that modify the construction documents, these also are part of the construction documents.

When a building information model is used on the project, it also is a part of the construction documents.

Elements of a quality control program can include a variety of documentation, such as policies, internal qualification requirements, and methods of tracking production progress. Any procedure that is not apparent subsequent to the performance of the work should be considered important enough to be part of the written procedures. Any documents and procedures made available to the quality assurance inspector (QAI) should be considered proprietary and not distributed inappropriately.

The inspection documentation should include the following information:

(1) The product inspected

(2) The inspection that was conducted

(3) The name of the inspector and the time period within which the inspection was conducted

(4) Nonconformances and corrections implemented

Records can include marks on pieces, notes on drawings, process paperwork, or electronic files. A record showing adherence to a sampling plan for pre-welding compliance during a given time period may be sufficient for pre-welding observation inspection.

The level of detail recorded should result in confidence that the product is in compliance with the requirements.

N3. FABRICATOR AND ERECTOR DOCUMENTS

1. Submittals for Steel Construction

The documents listed must be submitted so that the engineer of record (EOR) or the EOR's designee can evaluate that the items prepared by the fabricator or erector meet the EOR's design intent. This is usually done through the submittal of shop and erection drawings. In many cases, digital building models are produced in order to develop drawings for fabrication and erection. In lieu of submitting shop and erection drawings, the digital building model can be submitted and reviewed by the EOR for compliance with the design intent. For additional information concerning this process, refer to the AISC *Code of Standard Practice* (AISC, 2016a).

2. Available Documents for Steel Construction

The documents listed must be available for review by the EOR. Certain items are of a nature that submittal of substantial volumes of documentation is not practical, and therefore it is acceptable to have these documents reviewed at the fabricator's or erector's facility by the engineer or designee, such as the QA agency. Additional commentary on some of the documentation listed in this section follows:

(1) This section requires documentation to be available for the fastening of deck. For deck fasteners, such as screws and power fasteners, catalog cuts and/or manufacturers installation instructions are to be available for review. There is no requirement for certification of any deck fastening products.

(2) Because the selection and proper use of welding filler metals is critical to achieving the necessary levels of strength, notch toughness, and quality, the availability for review of welding filler metal documentation and welding procedure specifications (WPS) is required. This allows a thorough review on the part of the engineer and allows the engineer to have outside consultants review these documents, if needed.

(3) The fabricator and erector maintain written records of welding personnel qualification testing. Such records should contain information regarding date of testing, process, WPS, test plate, position, and the results of the testing. In order to verify the six-month limitation on welder qualification, the fabricator and erector should also maintain a record documenting the dates that each welder has used a particular welding process.

(4) The fabricator should consider AISC *Code of Standard Practice* Section 6.1, in establishing material control procedures for structural steel.

N4. INSPECTION AND NONDESTRUCTIVE TESTING PERSONNEL

1. Quality Control Inspector Qualifications

The fabricator or erector determines the qualifications, training and experience required for personnel conducting the specified inspections. Qualifications should be based on the actual work to be performed and should be incorporated into the fabricator's or erector's QC program. Inspection of welding should be performed by an individual who, by training and/or experience in metals fabrication, inspection and testing, is competent to perform inspection of the work. This is in compliance with AWS D1.1/D1.1M clause 6.1.4 (AWS, 2015). Recognized certification programs are a method of demonstrating some qualifications but they are not the only method nor are they required by Chapter N for QC inspectors.

2. Quality Assurance Inspector Qualifications

The QA agency determines the qualifications, training and experience required for personnel conducting the specified QA inspections. This may be based on the actual work to be performed on any particular project. AWS D1.1/D1.1M clause 6.1.4.1(3) states "An individual who, by training or experience, or both, in metals fabrication, inspection and testing, is competent to perform inspection of the work." Qualification for the QA inspector may include experience, knowledge and physical requirements. These qualification requirements are documented in the QA agency's written practice. AWS B5.1 (AWS, 2013) is a resource for qualification of a welding inspector.

The use of assistant welding inspectors under direct supervision is as permitted in AWS D1.1/D1.1M clause 6.1.4.3.

3. NDT Personnel Qualifications

NDT personnel should have sufficient education, training and experience in those NDT methods they will perform. ASNT SNT-TC-1a (ASNT, 2011a) and ASNT CP-189 (ASNT, 2011b) prescribe visual acuity testing, topical outlines for training, written knowledge, hands-on skills examinations, and experience levels for the NDT methods and levels of qualification.

As an example, under the provisions of ASNT SNT-TC-1a, an NDT Level II individual should be qualified to set up and calibrate equipment and to interpret and evaluate results with respect to applicable codes, standards and specifications. The NDT Level II individual should be thoroughly familiar with the scope and limitations of the methods for which they are qualified and should exercise assigned responsibility for on-the-job training and guidance of trainees and NDT Level I personnel. The NDT Level II individual should be able to organize and report the results of NDT tests.

N5. MINIMUM REQUIREMENTS FOR INSPECTION OF STRUCTURAL STEEL BUILDINGS

1. Quality Control

The welding inspection tasks listed in Tables N5.4-1 through N5.4-3 are inspection items contained in AWS D1.1/D1.1M (AWS, 2015), but have been organized in the tables in a more rational manner for scheduling and implementation using categories of before welding, during welding and after welding. Similarly, the bolting inspection tasks listed in Tables N5.6-1 through N5.6-3 are inspection items contained in the RCSC *Specification* (RCSC, 2014), but have been organized in a similar manner for scheduling and implementation using traditional categories of before bolting, during bolting and after bolting. The details of each table are discussed in Commentary Sections N5.4 and N5.6.

Typical model building codes, such as the 2015 *International Building Code* (IBC) (ICC, 2015) or NFPA 5000 (NFPA, 2015), make specific statements about inspecting to "approved construction documents"—the original and revised design drawings and specifications as approved by the building official or authority having jurisdiction (AHJ). AISC *Code of Standard Practice* Section 4.2(a) (AISC, 2016a) requires the transfer of information from the contract documents (design drawings and project specifications) into accurate and complete shop and erection drawings. Therefore, relevant items in the design drawings and project specifications that must be followed in fabrication and erection should be placed on the shop and erection drawings or in typical notes issued for the project. Because of this provision, QC inspection may be performed using shop drawings and erection drawings, not the original design drawings.

The applicable referenced standards in construction documents are commonly this standard, the AISC *Code of Standard Practice*, AWS D1.1/D1.1M, and the RCSC *Specification*.

2. Quality Assurance

AISC *Code of Standard Practice* Section 8.5.2 contains the following provisions regarding the scheduling of shop fabrication inspection: "Inspection of shop work by the Inspector shall be performed in the Fabricator's shop to the fullest extent possible. Such inspections shall be timely, in-sequence, and performed in such a manner as will not disrupt fabrication operations and will permit the repair of nonconforming work prior to any required painting while the material is still in-process in the fabrication shop."

Similarly, AISC *Code of Standard Practice* Section 8.5.3 states "Inspection of field work shall be promptly completed without delaying the progress or correction of the work."

AISC *Code of Standard Practice* Section 8.5.1 states "The Fabricator and the Erector shall provide the Inspector with access to all places where the work is being performed. A minimum of 24 hours notification shall be given prior to the commencement of work." However, the inspector's timely inspections are necessary for this to be achieved, while the scaffolding, lifts or other means provided by the fabricator or erector for their personnel are still in place or are readily available.

IBC Section 2203.1 (ICC, 2015) states "Identification of structural steel members shall comply with the requirements contained in AISC 360 .... Steel that is not readily identifiable as to grade from marking and test records shall be tested to determine conformity to such standards."

AISC *Code of Standard Practice* Section 6.1 states "Identification of Material. The fabricator shall be able to demonstrate by a written procedure and actual practice a method of material identification, visible up to the point of assembling members..."

AISC *Code of Standard Practice* Section 8.2 states "Material test reports shall constitute sufficient evidence that the mill product satisfies material order requirements. The Fabricator shall make a visual inspection of material that is received from the mill, ..." AISC *Code of Standard Practice* Sections 5.2 and 6.1 address the traceability of material test reports to individual pieces of steel, and the identification requirements for structural steel in the fabrication stage.

Model building codes, such as the IBC or NFPA 5000 (NFPA, 2015), make specific statements about inspecting to "approved construction documents" and the original and revised design drawings and specifications as approved by the building official or the authority having jurisdiction (AHJ). Because of these IBC provisions, the QAI should inspect using the original and revised design drawings and project specifications. The QAI may also use the shop drawings and erection drawings to assist in the inspection process.

3. Coordinated Inspection

Coordination of inspection tasks may be needed for fabricators in remote locations or distant from the project itself, or for erectors with projects in locations where inspection by a local firm or individual may not be feasible or where tasks are redundant.

The approval of both the AHJ and EOR is required for quality assurance to rely upon quality control, so there must be a level of assurance provided by the quality activities that are accepted. It may also serve as an intermediate step short of waiving QA as described in Section N6.

4. Inspection of Welding

AWS D1.1/D1.1M requires inspection, and any inspection task should be done by the fabricator or erector (termed contractor within AWS D1.1/D1.1M) under the terms of clause 6.1.2.1, as follows:

> Contractor's Inspection. This type of inspection and test shall be performed as necessary prior to assembly, during assembly, during welding, and after welding to ensure that materials and workmanship meet the requirements of the contract documents. Fabrication/erection inspection and testing shall be the responsibility of the Contractor unless otherwise provided in the contract documents.

This is further clarified in clause 6.1.3.3, which states:

> Inspector(s). When the term inspector is used without further qualification as to the specific inspector category described above, it applies equally to inspection and verification within the limits of responsibility described in 6.1.2.

The basis of Tables N5.4-1, N5.4-2 and N5.4-3 are inspection tasks, as well as quality requirements, and related detailed items contained within AWS D1.1/D1.1M. Commentary Tables C-N5.4-1, C-N5.4-2 and C-N5.4-3 provide specific references to clauses in AWS D1.1/D1.1M. In the determination of the task lists, and whether the task is designated "observe" or "perform," the pertinent terms of the following AWS D1.1/ D1.1M clauses were used:

> 6.5 Inspection of Work and Records
>
> 6.5.1 Size, Length, and Location of Welds. The Inspector shall ensure that the size, length, and location of all welds conform to the requirements of this code and to the detail drawings and that no unspecified welds have been added without the approval of the Engineer.
>
> 6.5.2 Scope of Examinations. The Inspector shall, at suitable intervals, observe joint preparation, assembly practice, the welding techniques, and performance of each welder, welding operator, and tack welder to ensure that the applicable requirements of this code are met.
>
> 6.5.3 Extent of Examination. The Inspector shall examine the work to ensure that it meets the requirements of this code. ... Size and contour of welds shall be measured with suitable gages. ...

"Observe" tasks are as described in clauses 6.5.2 and 6.5.3. Clause 6.5.2 uses the term "observe" and also defines the frequency to be "at suitable intervals." "Perform" tasks are required for each weld by AWS D1.1/D1.1M, as stated in clause 6.5.1 or 6.5.3, or are necessary for final acceptance of the weld or item. The use of the term "perform" is based upon the use in AWS D1.1/D1.1M of the phrases "shall examine the work" and "size and contour of welds shall be measured"; hence, "perform" items are limited to those functions typically performed at the completion of each weld.

TABLE C-N5.4-1
Reference to AWS D1.1/D1.1M (AWS, 2015) Clauses for Inspection Tasks Prior to Welding

Inspection Tasks Prior to Welding	Clauses
Welding procedure specifications (WPS) available	6.3
Manufacturer certifications for welding consumables available	6.2
Material identification (type/grade)	6.2
Welder identification system	6.4 (welder qualification) (identification system not required by AWS D1.1/D1.1M)
Fit-up of groove welds (including joint geometry) • Joint preparation • Dimensions (alignment, root opening, root face, bevel) • Cleanliness (condition of steel surfaces) • Tacking (tack weld quality and location) • Backing type and fit (if applicable)	 6.5.2 5.22 5.14 5.17 5.9, 5.21.1.1
Fit-up of CJP groove welds of HSS T-, Y- & K-joints without backing (including joint geometry) • Joint preparation • Dimensions (alignment, root opening, root face, bevel) • Cleanliness (condition of steel surfaces) • Tacking (tack weld quality and location)	9.11.2
Configuration and finish of access holes	6.5.2, 5.16 (also see Section J1.6)
Fit-up of fillet welds • Dimensions (alignment, gaps at root) • Cleanliness (condition of steel surfaces) • Tacking (tack weld quality and location)	 5.21.1 5.14 5.17
Check welding equipment	6.2, 5.10

The words “all welds” in clause 6.5.1 clearly indicate that all welds are required to be inspected for size, length and location in order to ensure conformity. Chapter N follows the same principle in labeling these tasks “perform,” which is defined as “Perform these tasks for each welded joint or member.”

The words “suitable intervals” used in clause 6.5.2 characterize that it is not necessary to inspect these tasks for each weld, but as necessary to ensure that the applicable requirements of AWS D1.1/D1.1M are met. Following the same principles and terminology, Chapter N labels these tasks as “observe,” which is defined as “Observe these items on a random basis.”

TABLE C-N5.4-2
Reference to AWS D1.1/D1.1M (AWS, 2015) Clauses for Inspection Tasks During Welding

Inspection Tasks During Welding	Clauses
Use of qualified welders	6.4
Control and handling of welding consumables • Packaging • Exposure control	6.2 5.3.1 5.3.2 (for SMAW), 5.3.3 (for SAW)
No welding over cracked tack welds	5.17
Environmental conditions • Wind speed within limits • Precipitation and temperature	 5.11.1 5.11.2
WPS followed • Settings on welding equipment • Travel speed • Selected welding materials • Shielding gas type/flow rate • Preheat applied • Interpass temperature maintained (min/max.) • Proper position (F, V, H, OH)	6.3.3, 6.5.2, 5.5, 5.20 5.6, 5.7
Welding techniques • Interpass and final cleaning • Each pass within profile limitations • Each pass meets quality requirements	6.5.2, 6.5.3, 5.23 5.29.1

The selection of suitable intervals as used in AWS D1.1/D1.1M is not defined within AWS D1.1/D1.1M, other than the AWS statement "to ensure that the applicable requirements of this code are met." The establishment of "at suitable intervals" is dependent upon the quality control program of the fabricator or erector, the skills and knowledge of the welders themselves, the type of weld, and the importance of the weld. During the initial stages of a project, it may be advisable to have increased levels of observation to establish the effectiveness of the fabricator's or erector's quality control program, but such increased levels need not be maintained for the duration of the project, nor to the extent of inspectors being on site. Rather, an appropriate level of observation intervals can be used which is commensurate with the observed performance of the contractor and their personnel. More inspection may be warranted for weld fit-up and monitoring of welding operations for complete-joint-penetration (CJP) and partial-joint-penetration (PJP) groove welds loaded in transverse tension, compared to the time spent on groove welds loaded in compression or shear, or time spent on fillet welds. More time may be warranted observing welding operations for multi-pass fillet welds, where poor quality root passes and poor fit-up may be obscured by subsequent weld beads, when compared to single pass fillet welds.

TABLE C-N5.4-3
Reference to AWS D1.1/D1.1M (AWS, 2015) Clauses for Inspection Tasks After Welding

Inspection Tasks After Welding	Clauses*
Welds cleaned	5.29.1
Size, length and location of welds	6.5.1
Welds meet visual acceptance criteria • Crack prohibition • Weld/base-metal fusion • Crater cross section • Weld profiles • Weld size • Undercut • Porosity	6.5.3 Table 6.1(1) Table 6.1(2) Table 6.1(3) Table 6.1(4), 5.24 Table 6.1(6) Table 6.1(7) Table 6.1(8)
Arc strikes	5.28
k-area*	not addressed in AWS
Weld access holes in rolled heavy shapes and built-up heavy shapes	5.16, 6.5.2 (see also Section J1.6)
Backing removed and weld tabs removed (if required)	5.9, 5.30
Repair activities	6.5.3, 5.25
Document acceptance or rejection of welded joint or member	6.5.4, 6.5.5

**k*-area issues were identified in AISC (1997b). See Commentary Section A3.1c and Section J10.8.

The terms “perform” and “observe” are not to be confused with the terms “periodic special inspection” and “continuous special inspection” used in the IBC for other construction materials. Both sets of terms establish two levels of inspection. The IBC terms specify whether the inspector is present at all times or not during the course of the work. Chapter N establishes inspection levels for specific tasks within each major inspection area. “Perform” indicates each item is to be inspected and “observe” indicates samples of the work are to be inspected. It is likely that the number of inspection tasks will determine whether an inspector has to be present full time but it is not in accordance with Chapter N to let the time an inspector is on site determine how many inspection tasks are done.

AWS D1.1/D1.1M clause 6.3 states that the contractor’s (fabricator/erector) inspector is specifically responsible for the WPS, verification of prequalification or proper qualification, and performance in compliance with the WPS. Quality assurance inspectors monitor welding to make sure QC is effective. For this reason, Tables N5.4-1 and N5.4-2 maintain an inspection task for the QA for these functions. For welding to be performed, and for this inspection work to be done, the WPS must be available to both welder and inspector. A separate inspection for tubular T-, Y-, K-connections was added to recognize the separate fit-up tolerances for these joints in AWS D1.1/D1.1M Table 9.8 and their importance to achieving an acceptable root.

Material verification of weld filler materials is accomplished by observing that the consumable markings correspond to those in the WPS and that certificates of compliance are available for consumables used.

The footnote to Table N5.4-1 states that "The fabricator or erector, as applicable, shall maintain a system by which a welder who has welded a joint or member can be identified. Stamps, if used, shall be the low-stress type." AWS D1.1/D1.1M does not require a welding personnel identification system. However, the inspector must verify the qualifications of welders, including identifying those welders whose work "appears to be below the requirements of this code." Also, if welds are to receive nondestructive testing (NDT), it is essential to have a welding personnel identification system to reduce the rate of NDT for good welders and increase the rate of NDT for welders whose welds frequently fail NDT. This welder identification system can also benefit the contractor by clearly identifying welders who may need additional training.

Table N5.4-3 includes requirements for observation that "No prohibited welds have been added without the approval of the engineer." AWS D1.1/D1.1M clause 5.17 includes specific provisions for tack welds incorporated into final welds, tack welds not incorporated into final welds, and construction aid welds.

AWS D1.1/D1.1M clause 7 on Stud Welding includes requirements regarding the stud welding materials and their condition, base metal condition, stud application qualification testing, pre-production welding inspection and bend testing, qualification of the welding operator, visual inspection of completed studs and bend testing of certain studs when required, and the repair of nonconforming studs. For manually welded studs, special requirements apply to the stud base and the welding procedures.

The proper fit-up for groove welds and fillet welds prior to welding should first be checked by the fitter and/or welder. Such detailed dimensions should be provided on the shop or erection drawings, as well as included in the WPS. Fitters and welders must be equipped with the necessary measurement tools to ensure proper fit-up prior to welding.

AWS D1.1/D1.1M clause 6.2 on Inspection of Materials and Equipment states that, "The Contractor's Inspector shall ensure that only materials and equipment conforming to the requirements of this code shall be used." For this reason, the check of welding equipment is assigned to QC only, and is not required for QA.

5. Nondestructive Testing of Welded Joints

5a. Procedures

Buildings are subjected to static loading unless fatigue is specifically addressed as prescribed in Appendix 3. Section J2 provisions contain exceptions to AWS D1.1/D1.1M.

5b. CJP Groove Weld NDT

For statically loaded structures, AWS D1.1/D1.1M and the Specification have no specific nondestructive testing (NDT) requirements, leaving it to the engineer to determine the appropriate NDT method(s), locations or categories of welds to be tested, and the frequency and type of testing (full, partial or spot), in accordance with AWS D1.1/D1.1M clause 6.15.

The Specification implements a selection of NDT methods and a rate of ultrasonic testing (UT) based upon a rational system of risk of failure. If based upon a model building code such as the International Building Code (ICC, 2015) or NFPA 5000 (NFPA, 2015), the applicable building code will assign every building or structure to one of four different risk categories. Where there is no applicable building code, then Section A1 requires that the risk category be assigned in accordance with ASCE/SEI 7 (ASCE, 2016).

Complete-joint-penetration (CJP) groove welds loaded in tension applied transversely to their axis are assumed to develop the capacity of the smaller steel element being joined, and therefore have the highest demand for quality. CJP groove welds in compression or shear are not subjected to the same crack propagation risks as welds subjected to tension. Partial-joint-penetration (PJP) groove welds are designed using a limited design strength when in tension, based upon the root condition, and therefore are not subjected to the same high stresses and subsequent crack propagation risk as a CJP groove weld. PJP groove welds in compression or shear are similarly at substantially less risk of crack propagation than CJP groove welds.

Fillet welds are designed using limited strengths, similar to PJP groove welds, and are designed for shear stresses regardless of load application, and therefore do not warrant NDT.

The selection of joint type and thickness ranges for ultrasonic testing (UT) are based upon AWS D1.1/D1.1M clause 6.19.1, which limits the procedures and standards as stated in Part F of AWS D1.1/D1.1M to groove welds and heat affected zones between the thicknesses of $^5/_{16}$ in. and 8 in. (8 mm and 200 mm), inclusive. The requirement to inspect 10% of CJP groove welds is a requirement that the full length of 10% of the CJP groove welds shall be inspected.

5c. Welded Joints Subjected to Fatigue

CJP groove welds in butt joints so designated in Appendix 3 Table A-3.1, Sections 5 and 6.1, require that internal soundness be verified using ultrasonic testing (UT) or radiographic testing (RT), meeting the acceptance requirements of AWS D1.1/D1.1M clause 6.12 or 6.13, as appropriate.

5e. Reduction of Ultrasonic Testing Rate

For statically loaded structures in risk categories III and IV, reduction of the rate of UT from 100% is permitted for individual welders who have demonstrated a high level of skill, proven after a significant number of their welds have been tested.

5f. Increase in Ultrasonic Testing Rate

For risk category II, where 10% of CJP groove welds loaded in transverse tension are tested, an increase in the rate of UT is required for individual welders who have failed to demonstrate a high level of skill, established as a failure rate of more than 5%, after a sufficient number of their welds have been tested. To implement this effectively, and not necessitate the retesting of welds previously deposited by a welder who has a high reject rate established after the 20 welds have been tested, it is suggested that at the start of the work, a higher rate of UT be performed on each welder's completed welds.

6. Inspection of High-Strength Bolting

The RCSC *Specification* (RCSC, 2014), like the referenced welding standard, defines bolting inspection requirements in terms of inspection tasks and scope of examinations. The RCSC *Specification* uses the term "routine observation" for the inspection of all pretensioned bolts, further validating the choice of the term "observe" in this chapter of the Specification.

Table N5.6-1 includes requirements for observation of "Fasteners marked in accordance with ASTM requirements." This includes the required package marking of the fasteners and the product marking of the fastener components in accordance with the applicable ASTM standard. As an example, ASTM F3125 Grade A325 requires the following items for package marking: ASTM designation and type; size; name and brand or trademark of the manufacturer; number of pieces; lot number; purchase order number; and country of origin. ASTM F3125 Grade A325 also requires manufacturer identification and grade identification on the head of each bolt.

Snug-tightened joints are required to be inspected to ensure that the proper fastener components are used and that the faying surfaces are brought into firm contact during installation of the bolts. The magnitude of the clamping force that exists in a snug-tightened joint is not a consideration and need not be verified.

Pretensioned joints and slip-critical joints are required to be inspected to ensure that the proper fastener components are used and that the faying surfaces are brought into firm contact during the initial installation of the bolts. Pre-installation verification testing is required for all pretensioned bolt installations, and the nature and scope of installation verification will vary based on the installation method used. The following provisions from the RCSC *Specification* serve as the basis for Tables N5.6-1, N5.6-2 and N5.6-3. In the following, underlining has been added for emphasis of terms:

> **9.2.1. Turn-of-Nut Pretensioning:** The inspector shall observe the pre-installation verification testing required in Section 8.2.1. Subsequently, it shall be ensured by <u>routine observation</u> that the bolting crew properly rotates the turned element relative to the unturned element by the amount specified in Table 8.2. Alternatively, when fastener assemblies are match-marked after the initial fit-up of the joint, but prior to pretensioning; visual inspection after pretensioning is permitted in lieu of routine observation.

9.2.2. Calibrated Wrench Pretensioning: The inspector shall observe the pre-installation verification testing required in Section 8.2.2. Subsequently, it shall be ensured by routine observation that the bolting crew properly applies the calibrated wrench to the turned element. No further evidence of conformity is required.

9.2.3. Twist-Off-Type Tension Control Bolt Pretensioning: The inspector shall observe the pre-installation verification testing required in Section 8.2.3. Subsequently, it shall be ensured by routine observation that the splined ends are properly severed during installation by the bolting crew.

9.2.4. Direct-Tension Indicator Pretensioning: The inspector shall observe the pre-installation verification testing required in Section 8.2.4. Subsequently, but prior to pretensioning, it shall be ensured by routine observation that the appropriate feeler gage is accepted in at least half of the spaces between the protrusions of the direct tension indicator and that the protrusions are properly oriented away from the work.

The presence of the inspector is dependent upon whether the installation method provides visual evidence of completed installation. Turn-of-nut installation with matchmarking, installation using twist-off bolts, and installation using direct tension indicators provides visual evidence of a completed installation, and therefore "observe" is stated for these methods. Turn-of-nut installation without matchmarking and calibrated wrench installation provides no such visual evidence, and the inspector is to be "engaged" onsite, although not necessarily watching every bolt or joint as it is being pretensioned.

The inspection provisions of the RCSC *Specification* rely upon observation of the work, hence all tables use "observe" for the designated tasks. Commentary Tables C-N5.6-1, C-N5.6-2 and C-N5.6-3 provide the applicable RCSC *Specification* references for inspection tasks prior to, during and after bolting.

7. Inspection of Galvanized Structural Steel Main Members

Cracks have been observed on the cut surfaces of rolled shapes, plates and on the corners of hollow structural sections (HSS) that have been galvanized. The propensity for cracking is related to residual and thermal stresses, geometric stress concentrations, and a potential for hydrogen or liquid metal assisted cracking. These characteristics can be modified, but no provisions have been found that eliminate all potential for cracking. Inspection should be focused near changes in direction of the cut surface, at edges of welded details, or at changes in section dimensions. In HSS, indications may appear on the inside corner near the exposed end. The word "exposed" in this section is intended to mean cut surface that is not covered by weld or a connected part.

8. Other Inspection Tasks

IBC requires that anchor rods for steel be set accurately to the pattern and dimensions called for on the plans. In addition, it is required that the protrusion of the threaded ends through the connected material be sufficient to fully engage the threads of the nuts, but not be greater than the length of the threads on the bolts.

TABLE C-N5.6-1
Reference to RCSC *Specification* (RCSC, 2014) Sections for Inspection Tasks Prior to Bolting

Inspection Tasks Prior to Bolting	Sections
Manufacturer's certifications available for fastener materials	2.1, 9.1
Fasteners marked in accordance with ASTM requirements	Figure C-2.1, 9.1 (also see ASTM standards)
Correct fasteners selected for the joint detail (grade, type, bolt length if threads are to be excluded from shear plane)	2.3.2, 2.7.2, 9.1
Correct bolting procedure selected for joint detail	4, 8
Connecting elements, including the appropriate faying surface condition and hole preparation, if specified, meet applicable requirements	3, 9.1, 9.3
Pre-installation verification testing by installation personnel observed and documented for fastener assemblies and methods used	7, 9.2
Protected storage provided for bolts, nuts, washers, and other fastener components	2.2, 8, 9.1

TABLE C-N5.6-2
Reference to RCSC *Specification* (RCSC, 2014) Sections for Inspection Tasks During Bolting

Inspection Tasks During Bolting	Sections
Fastener assemblies placed in all holes and washers (if required) are positioned as required	7.1(1), 8.1, 9.1
Joint brought to the snug-tight condition prior to the pretensioning operation	8.1, 9.1
Fastener component not turned by the wrench prevented from rotating	8.2, 9.2
Fasteners are pretensioned in accordance with a method approved by RCSC and progressing systematically from most rigid point toward free edges	8.2, 9.2

TABLE C-N5.6-3
Reference to RCSC *Specification* (RCSC, 2014) Sections for Inspection Tasks After Bolting

Inspection Tasks After Bolting	Sections
Document acceptance or rejection of bolted connections	not addressed by RCSC

AISC *Code of Standard Practice*, Section 7.5.1, states that anchor rods, foundation bolts, and other embedded items are to be set by the owner's designated representative for construction. The erector is likely not on site to verify placement, therefore it is assigned solely to the quality assurance inspector (QAI). Because it is not possible to verify proper anchor rod materials and embedment following installation, it is required that the QAI be onsite when the anchor rods are being set.

N6. APPROVED FABRICATORS AND ERECTORS

IBC Section 1704.2.5.1 (ICC, 2015) states that:

> Special inspections during fabrication are not required where the work is done on the premises of a fabricator registered and approved to perform such work without special inspection.
>
> Approval shall be based upon review of the fabricator's written procedural and quality control manuals and periodic auditing of fabrication practices by an approved agency.

An example of how these approvals may be made by the building official or authority having jurisdiction (AHJ) is the use of the AISC Certification program. A fabricator certified to the AISC Certification Program for Structural Steel Fabricators, *Standard for Steel Building Structures* (AISC, 2006), meets the criteria of having a quality control manual, written procedures, and annual onsite audits conducted by AISC's independent auditing company, Quality Management Company, LLC. Similarly, steel erectors may be an AISC Certified Erector or AISC Advanced Certified Steel Erector. The audits confirm that the company has the personnel, knowledge, organization, equipment, experience, capability, procedures and commitment to produce the required quality of work for a given certification category.

Granting a waiver of QA inspections in a fabrication shop does not eliminate the required NDT of welds; instead of being performed by QA, such inspections are instead performed by the fabricator's QC. Even when QA inspection is waived, the NDT reports prepared by the fabricator's QC are available for review by a third party QA.

APPENDIX 1
DESIGN BY ADVANCED ANALYSIS

General provisions for designing for stability are presented in Chapter C, in which specific details are provided for the direct analysis method. This Appendix provides details for explicitly modeling system and member imperfections and/or inelasticity within the analysis.

1.1. GENERAL REQUIREMENTS

The provisions of this Appendix permit the use of analysis methods that are more sophisticated than those required by Chapter C. The provisions also permit the use of computational analysis (e.g., the finite element method) to replace certain Specification equations used to evaluate limit states covered by Chapters D through H, J and K. The application of these provisions requires a complete understanding of the provisions of this Appendix as well as the equations they supersede. It is the responsibility of the engineer using these provisions to fully verify the completeness and accuracy of the analysis software used for this purpose.

1.2. DESIGN BY ELASTIC ANALYSIS

In more traditional approaches, design for stability involves the combination of employing an analysis to determine the required strengths of components and the use of prescriptive code equations to proportion components to ensure they have adequate available strengths. Many traditional second-order analysis methods commonly available to designers account for P-Δ and P-δ effects in flexure, but typically do not ensure equilibrium is satisfied on the deformed geometry of the system. They may also not account for twisting effects that can cause additional second-order effects that sometimes should be considered in design. The resulting effects of this approximation, such as neglecting twisting effects, have traditionally been accounted for when proportioning components and historically have been incorporated as part of the design requirements of Chapters D through K.

With more sophisticated analysis software being made available to designers, it is now possible to extend design methods, such as the direct analysis method, to provide engineers more opportunities to better approach complex design problems. Examples include, but are not limited to, defining the unbraced length of an arch or defining the effective length of an unbraced Vierendeel truss in which the axial force in the compression chord varies along its length.

A rigorous second-order elastic analysis, in which equilibrium and compatibility are satisfied on the deformed geometry of the system and its components, combined with adequate stiffness reductions for representing potential inelasticity, will indicate that deflections and internal force and moments will become unbounded as a structural system or any of its components approach instability. With instabilities such as flexural buckling of compressive members now being monitored by the analysis, the check for adequate design strength can be simplified to only needing to confirm adequate cross-section strength.

In this method of design, it is very important for a designer to ensure that the analysis adequately captures all applicable second-order effects (including twist of the member which can be important in some situations). Guidance is provided in this Commentary with a benchmark problem to ensure all significant second-order effects are being considered in order to use this method of design.

This new design approach is very useful in problems where it is not clearly evident what the unbraced lengths actually are for members in compression. Examples of such a situation occur when designing an arch structure for in-plane buckling effects under axial load, or when designing a through-truss (pony truss) where the top chord is continuous and seemingly unbraced out of plane. For these types of problems, the designer can perform a rigorous second-order elastic analysis as defined in this Appendix and avoid a direct consideration of length effects for axially loaded compression members, while using the member cross-sectional strength in the appropriate limit state design equations. In such a case, buckling and instability are accounted for in producing additional second-order moments and shears caused by member twist.

1. General Stability Requirements

This section references the five requirements from Chapter C that make up a rigorous second-order elastic analysis. The requirements are similar to those contained in earlier Specification requirements defining a traditional second-order analysis that considered P-Δ and P-δ effects in flexure, but now include the requirement to capture twisting and torsional effects that must be included as part of the design in some problems with long unbraced lengths that are subject to additional internal forces caused by member twist. A rigorous second-order analysis can also capture the beneficial effects of member torsional strength due to warping restraint, for software that is written to include this component of member torsional strength, which adds to the member strength when a member is subjected to twisting effects. Software programs that do consider member twisting by providing for equilibrium on the deformed shape under each increment of loading, but do not consider the beneficial effects of torsional strength due to warping restraint (consideration of the C_w member property), will provide a conservative solution to the member internal forces. The designer is cautioned to carefully examine the effects of twist and resulting second-order effects for each problem when using this method of analysis.

It is noted that this method is currently restricted to doubly symmetric sections, including I-shapes, HSS and box sections, because current analytical testing has generally taken place with these section types. The designer can consider using singly symmetric shapes or other shapes as long as an investigation is undertaken to ensure the results are properly capturing twisting effects and generally produce designs comparable to the traditional design approach as specified in Chapter C and the other design requirements contained in Chapters D through K.

2. Calculation of Required Strengths

The details for the level of second-order analysis required for use of this design method are contained in Section 1.2.2. Traditional second-order analysis methods readily available to designers in most modern software, and commonly used in recent years, have traditionally only included flexural second-order effects defined by the P-Δ and P-δ effects. These effects are explained in detail in Commentary Section C2. The difference between these traditional second-order analysis methods and the more rigorous second-order analysis referred to herein is the additional requirement for consideration of member twist, which results when an unbraced member is subjected to transverse load with or without axial load and containing an initial imperfection, such as camber or sweep, perpendicular to the plane of loading. Twist can also occur due to biaxial bending in a beam alone or in a beam-column with or without transverse loading that contains an out-of-plane imperfection.

Any analysis method that properly considers twist in a member due to an out-of-plane imperfection, or simply due to the tendency to twist under the effects of elastic or inelastic lateral-torsional buckling, will have additional moments caused by the twisting that must be resisted in the design. It has been found that twist becomes an important consideration in unbraced wide-flange sections as the unbraced length of the member approaches L_r and as the ratio of strong-axis to weak-axis moment stiffness and strength increases. For such cases, the member capacity is reached with twist of the cross section in the range of 0.03 to 0.05 rad (1.7 to 2.9°). Software that considers twisting effects using a large displacement analysis, where equilibrium is accounted for during each increment of loading in a line element model, or software that contains additional degrees of freedom (14 degrees of freedom) in a line element member model that includes twisting and warping restraint effects, is able to pick up these additional second-order effects not customarily accounted for in traditional software using a 12 degree of freedom line element member model considering only P-Δ and P-δ effects. While finite element models are able to pick up twisting and warping restraint effects and are readily available in some commercial software, they are not routinely used in a design office practice for most analyses.

Deformations to be Considered in the Analysis. The requirement for exactly what imperfection deformations are to be modeled as part of the rigorous second-order analysis is left to the designer for each particular design case. Normally, the imperfection limits (camber, sweep and twist) specified in the various ASTM Specifications for the member type would be consulted. Typically, for W-shaped column members, an imperfection out-of-plane of 1/1000 times the member's unbraced length would be used, unless a larger or smaller tolerance is justified by the fabrication. Generally, it is only necessary to consider the imperfection about the axis where buckling is likely to occur. It has been observed that the second-order internal forces in a member are incurred because of the natural tendency of the member to twist under loading, regardless of the imperfection used in the analysis. The important point is for some member perturbation to exist for the second-order effects to be picked up in the rigorous second-order analysis. For system imperfections caused by erection tolerances, the 1/500 out-of-plumbness, or a similar deviation in the nominal member end locations, should be included in the analysis unless justification exists for the particular design case that warrants use of a different level of erection tolerance. Regardless of the imperfection chosen for the analysis, it is important for the designer to understand the sensitivity of the analysis results to the level of imperfection chosen. Past studies have shown that the second-order internal forces can be very sensitive to the magnitude of the imperfection chosen, especially in the case of member twist. Some analysis software may only consider the effects of member twist by using a large displacement algorithm that considers member equilibrium in the deformed shape produced by each applied load increment, but without any consideration of the beneficial effects of torsional strength from warping resistance of the cross section. Using the internal member force results from such an analysis is conservative.

If these additional second-order effects caused by member twist are accounted for in the structural analysis, it is possible to design the members for axial load using their cross-sectional strength, without consideration of flexural or flexural-torsional buckling of members caused by unbraced length effects.

Adjustments to Stiffness. Partial yielding accentuated by residual stresses in members can produce a general softening of the structure at the strength limit state that further creates destabilizing effects. The design method provided in this section is similar to the direct analysis method presented in Chapter C, and is also calibrated against distributed-plasticity analyses that account for the spread of yielding through the member cross section and along the member length. In these calibration studies, the residual stresses in W-shapes were assumed to have a maximum value of $0.3F_y$ in compression at the flange tips, and a distribution matching the so-called Lehigh pattern—a linear variation across the flanges and uniform tension in the web (Ziemian, 2010).

Reduced stiffness ($EI^* = 0.8\tau_b EI$ and $EA^* = 0.8EA$) is used in the method provided in this section, just as it is for the direct analysis method of Chapter C. However, the stiffness reduction of 0.8 is also required for all other member properties, including J and C_w to properly account for twisting effects in the analysis. The τ_b factor is similar to the inelastic stiffness reduction factor implied in the column curve to account for loss of stiffness under high compression loads ($\alpha P_r > 0.5P_y$), and the 0.8 factor accounts for additional softening under combined axial compression and bending. It is a fortuitous coincidence that the reduction coefficients for both slender and stocky columns are close enough, such that the single reduction factor of $0.8\tau_b$ works fairly well over the full range of slenderness.

The use of reduced stiffness only pertains to analyses for strength and stability limit states. It does not apply to analyses for other stiffness-based conditions and criteria, such as for drift, deflection, vibration and period determination.

For ease of application in design practice, where $\tau_b = 1.0$, the 0.8 reduction on I, A, J and C_w can be applied by modifying E and G by 0.8 in the analysis. However, for computer software that does semi-automated design, one should ensure that the reduced E and G is applied only for the second-order analysis. The elastic modulus should not be reduced in nominal strength equations that include E for consideration of local buckling or slender-element effects.

Analysis Benchmark Problem. It is important for an engineer to understand the capabilities and limitations of the analysis software used in design. In order to provide a confidence level that a program is able to account for the second-order effects caused by the combination of axial force, flexure, and twist, it is strongly suggested that several benchmark problems be run to confirm the adequacy of the software being used. The following benchmark problem has been developed as one to consider in evaluating the accuracy of the analysis software required for application of the design method provided in Appendix 1, Section 1.2.

The results of an analysis procedure that does not consider member twist versus a procedure that does is demonstrated in Figure C-A-1.1. In this case, a member is subjected to loading that results in major-axis and minor-axis bending. As the figure indicates, a simply supported member with only ends restrained against twisting and simultaneously subjected to major-axis and minor-axis flexure will, in reality, twist to some extent. This twisting changes the magnitude of the components of major-axis and minor-axis moments when they are resolved to a coordinate system that references the cross-section axes of the deformed (twisted) state of the member.

Table C-A-1.1 provides numerical results at mid-span for a W18×65 beam-column spanning 20 ft and subjected to four different combinations of loading. The axial and uniformly distributed loads are assumed factored and are applied proportionally. The uniformly distributed load is applied in the vertical gravity direction throughout the loading history. The member is simply supported with rotation at the member ends restrained from twisting, with warping unrestrained. Therefore, the member ends are torsionally pinned (Seaburg and Carter, 1997). With a τ-factor equaling 1.0, because the axial force in all combinations is less than 0.5 times the axial yield force, the stiffness reduction applied to all section properties, including A, I_x, I_y, J and C_w, is 0.8 (or equivalently, the factor 0.8 could be applied to both E and G). To provide some degree of transparency in the results, and because the length-to-height and length-to-width ratios of the member are approximately 13 and 32, respectively, shear deformations have been assumed negligible and are therefore neglected[1]. For each combination of loading, three sets of results are provided.

In Table C-A-1.1, rows labeled (a) provide analysis results from a traditional second-order analysis meeting the expectations of Section C2.1. In this case, a nominally straight member is assumed and only in-plane P-δ effects on flexure need be considered. With no out-of-plane behavior occurring, there is no resulting twist, out-of-plane deflection, or minor-axis bending moments. According to Section C3, the interaction of axial force and flexure is assessed by the requirements of Chapter H, in which the nominal compressive strength defined in Section E3 is based on an effective length equaling the unbraced length of the member.

Analysis results provided in rows (b) and (c) of Table C-A-1.1 correspond to the requirements of Appendix 1, Section 1.2. In these cases, an out-of-plane member imperfection in the shape of a sine curve with an amplitude of L/1000 at midspan is included in the computational model. A more rigorous elastic analysis procedure is

[1] In many commercial analysis software, the default setting of including shear deformations should be turned off when making comparisons with the tabulated results provided.

employed that ensures equilibrium and compatibility are satisfied on the deformed shape of the member, and thereby includes second-order effects attributed to both P-δ and twist effects. Hence, the combination of the applied loads (P and w_y), initial out-of-plane imperfection ($\delta_{ox} = L/1000$), and the resulting deflections and twist (δ_x, δ_y and θ) produce both major-axis and minor-axis bending moments, which at midspan are

$$M'_{ux} = \left(\frac{w_y L^2}{8} + P\delta_y\right)\cos\theta - P(\delta_{ox} + \delta_x)\sin\theta \qquad \text{(C-A-1-1)}$$

$$M'_{uy} = \left(\frac{w_y L^2}{8} + P\delta_y\right)\sin\theta + P(\delta_{ox} + \delta_x)\cos\theta \qquad \text{(C-A-1-2)}$$

In calculating the results given in row (b), the warping resistance of the section produced by cross-flange bending along the length of the member is neglected ($EC_w = 0$) and torsional resistance is provided only by St. Venant stiffness (GJ). Such warping resistance, as well as the St. Venant stiffness, is included in the analysis results provided in row (c). The beneficial effects of including warping resistance are evident by significant reductions in deflections (δ_x, δ_y and θ) and minor-axis bending

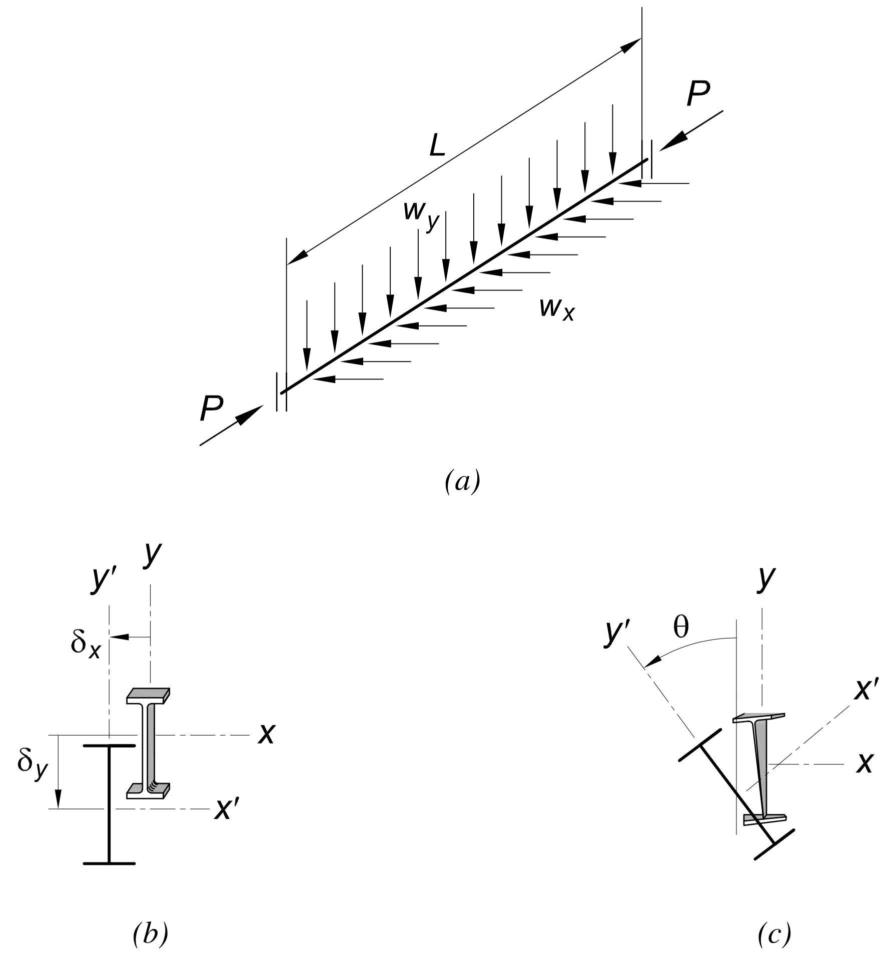


Fig. C-A-1.1. Deflection of cross section at midspan.

TABLE C-A-1.1
Results for Benchmark Problem Shown in Figure C-A-1.1
W18x65, *L* = 20 ft

***P*, kips**		**0**	**75**	**125**	**175**
($w_x = 0$) w_y, kip/ft		**4**	**3**	**2**	**1**
M'_{ux}, kip-in.	(a)	2400	1833	1237	626
	(b)	2386	1826	1235	624
	(c)	2399	1832	1237	626
M'_{uy}, kip-in.	(a)	0	0	0	0
	(b)	258	234	192	309
	(c)	55.8	104	140	284
δ_y, in.	(a)	0.580	0.443	0.299	0.151
	(b)	0.694	0.524	0.342	0.201
	(c)	0.589	0.460	0.318	0.186
δ_x, in.	(a)	0	0	0	0
	(b)	0.967	0.951	0.833	1.397
	(c)	0.214	0.435	0.616	1.292
θ, rad	(a)	0	0	0	0
	(b)	0.1078	0.0790	0.0471	0.0358
	(c)	0.0233	0.0290	0.0266	0.0260
Eq. H1-1	(a)	0.62	0.77	0.87	0.96
	(b)	0.87	0.75	0.58	0.62
	(c)	0.68	0.62	0.53	0.60

(a) Analysis per Section C2.1; without member imperfection
(b) Analysis per Appendix 1, Section 1.2; $\delta_{ox} = L/1000$; $EC_w = 0$
(c) Analysis per Appendix 1, Section 1.2; $\delta_{ox} = L/1000$

moments. Based on Appendix 1, Section 1.2.3, the interaction of axial force and flexure is assessed according to the requirements of Chapter H, in which the nominal compressive strength, P_n, is taken as the cross-section compressive strength, P_{ns}, which for this nonslender section is F_yA_g. In all cases, the nominal major-axis and minor-axis flexural strengths are determined according to the provisions of Chapter F.

Analysis with Factored Loads. As with the direct analysis method presented in Chapter C, and because of the high nonlinearity associated with second-order effects, it is essential that the analysis of the system be made with loads factored to the strength limit-state level.

The Specification requirements for consideration of initial imperfections are intended to apply only to analyses for strength limit states. It is not necessary, in most cases, to consider initial imperfections in analyses for serviceability conditions such as drift, deflection and vibration.

Where concrete shear walls or other nonsteel components contribute to the stability of the structure and the governing codes or standards for those elements specify a greater stiffness reduction, the greater reduction should be applied.

3. Calculation of Available Strengths

When the analysis meets the requirements of Appendix 1, Section 1.2.2, the member cross-sectional strength provisions for available strength in axially loaded members from Chapters D, E and H can be used. Otherwise, the provisions in Chapters F through K complete the process of design by this method. The effects of local buckling and reductions in member capacity because of slender elements of the member must still be considered for P_n. The effective length factor, K, and member buckling from length effects in axially loaded members in general need not be considered because they are directly accounted for in the structural analysis. The interaction of flexure and compression should be checked at all points along the member length, with the nominal flexural strengths, M_n, determined from Chapter F.

It should be noted that the AASHTO *Specification* (AASHTO, 2014) addresses the consideration of flange lateral bending due to minor-axis bending moment, plus warping due to torsion, in the design of general curved and straight I-section members for flexure. White and Grubb (2005) provides an overview of the background to these equations. For beam-columns with significant flange bending due to twist, the minor-axis flexural capacity ratio for the flange subjected to the largest combined lateral bending due to overall minor-axis bending plus torsion may be used with Equations H1-1 as a conservative assessment. Aghayere and Vigil (2014) provide a straightforward discussion of this type of calculation with references to additional background research studies.

Where beams and columns rely upon braces for stability, they should generally be included as part of the lateral force-resisting system in the analysis. As long as imperfections are considered as specified, sufficient strength and stiffness to control member movement at the brace points can automatically be assessed.

1.3. DESIGN BY INELASTIC ANALYSIS

This section contains provisions for the inelastic analysis and design of structural steel systems, including continuous beams, moment frames, braced frames and combined systems. The Appendix has been modified from the previous Specification to allow for the use of a wider range of inelastic analysis methods, varying from the traditional plastic design approaches to the more advanced nonlinear finite element analysis methods. In several ways, this Appendix represents a logical extension of the direct analysis method of Chapter C, in which second-order elastic analysis is used. The provision for moment redistribution in continuous beams, which is permitted for elastic analysis only, is provided in Section B3.3.

1. General Requirements

These requirements directly parallel the general requirements of Chapter C and are further discussed in Commentary Section C1.

Various levels of inelastic analysis are available to the designer (Ziemian, 2010; Chen and Toma, 1994). All are intended to account for the potential redistribution of member and connection forces and moments that are a result of localized yielding as a structural system reaches a strength limit state. At the higher levels, they have the ability to model complex forms of nonlinear behavior and detect member and/or frame instabilities well before the formation of a plastic mechanism. Many of the strength design equations in this Specification, for members subject to compression, flexure and combinations thereof, were developed using refined methods of inelastic analysis along with experimental results and engineering judgment (Yura et al., 1978; Kanchanalai and Lu, 1979; Bjorhovde, 1988; Ziemian, 2010). Also, research over the past twenty years has yielded significant advances in procedures for the direct application of second-order inelastic analysis in design (Ziemian et al., 1992; White and Chen, 1993; Liew et al., 1993; Ziemian and Miller, 1997; Chen and Kim, 1997; Surovek, 2010). Correspondingly, there has been a steady increase in the inclusion of provisions for inelastic analysis in commercial steel design software, but the level varies widely. Use of any analysis software requires an understanding of the aspects of structural behavior it simulates, the quality of its methods, and whether or not the software's ductility and analysis provisions are equivalent to those of Appendix 1, Sections 1.2 and 1.3. There are numerous studies available for verifying the accuracy of the inelastic analysis (Kanchanalai, 1977; El-Zanaty et al., 1980; White and Chen, 1993; Maleck and White, 2003; Martinez-Garcia and Ziemian, 2006; Ziemian, 2010).

With this background, it is the intent of this Appendix to allow certain levels of inelastic analysis to be used in place of the Specification design equations as a basis for confirming the adequacy of a member or system. In all cases, the strength limit state behavior being addressed by the corresponding provisions of the Specification needs to be considered. For example, Section E3 provides equations that define the nominal compressive strength corresponding to the flexural buckling of members without slender elements. The strengths determined by these equations account for many factors, which primarily include the initial out-of-straightness of the compression member, residual stresses that result from the fabrication process, and the reduction of flexural stiffness due to second-order effects and partial yielding of the cross section. If these factors are directly incorporated within the inelastic analysis and a comparable or higher level of reliability can be ensured, then the specific strength equations of Section E3 need not be evaluated. In other words, the inelastic analysis will indicate the limit state of flexural buckling and the design can be evaluated accordingly. On the other hand, suppose that the same inelastic analysis is not capable of modeling flexural-torsional buckling. In this case, the provisions of Section E4 would need to be evaluated. Other examples of strength limit states not detected by the analysis may include, but are not limited to, lateral-torsional buckling strength of flexural members, connection strength, and shear yielding or buckling strengths.

Item (e) of the General Requirements given in Appendix 1, Section 1.3.1 states that "…uncertainty in system, member, and connection strength and stiffness…" shall be taken into account. Member and connection reliability requirements are fulfilled by the probabilistically derived resistance factors and load factors of load and resistance factor design of this Specification. System reliability considerations are still a project-by-project exercise, and no overall methods have, as yet, been developed for steel building structures. Introduction to the topic of system reliability can be found in textbooks, for example, Ang and Tang (1984), Thoft-Christensen and Murotsu (1986), and Nowak and Collins (2000), as well as in many publications, for example, Buonopane and Schafer (2006).

Because this type of analysis is inherently conducted at ultimate load levels, the provisions of this Appendix are limited to the design basis of Section B3.1 (LRFD).

In accordance with Section B3.8, the serviceability of the design should be assessed with specific requirements given in Chapter L. In satisfying these requirements in conjunction with a design method based on inelastic analysis, consideration should be given to the degree of steel yielding permitted at service loads. Of particular concern are (a) permanent deflections that may occur due to steel yielding, and (b) stiffness degradation due to yielding and whether this is modeled in the inelastic analysis.

Although the use of inelastic analysis has great potential in earthquake engineering, the specific provisions beyond the general requirements of this Appendix do not apply to seismic design. The two primary reasons for this are:

(a) In defining "equivalent" static loads for use in elastic seismic design procedures, member yielding and inelastic force redistribution is already implied through the specification of seismic response modification factors (R-factors) that are greater than unity. Therefore, it would not be appropriate to use the equivalent seismic loads with a design approach based on inelastic analysis.

(b) The ductility requirements for seismic design based on inelastic analysis are more stringent than those provided in this Specification for nonseismic loads.

Criteria and guidelines for the use of inelastic analysis and design for seismic applications are provided in Chapter 16 of the ASCE/SEI 7 (ASCE, 2016), ASCE/SEI 41 (ASCE, 2013), and Resource Paper 9, "Seismic Design using Target Drift, Ductility, and Plastic Mechanism as Performance Criteria" in the *NEHRP Recommended Seismic Provisions for New Buildings and Other Structures* (BSSC, 2009). As described in these documents, when nonlinear (inelastic) static analysis is used for seismic design, the earthquake loading effects are typically quantified in terms of target displacements that are determined from ground motion spectral acceleration. Alternatively, for nonlinear (inelastic) dynamic analysis, the earthquake loading effects are defined in terms of input ground motions that are selected and scaled to match ground motion spectra. For the seismic design of new buildings, capacity design strategies are highly recommended to control the locations of inelastic action to well defined mechanisms (BSSC, 2009; Deierlein et al., 2010).

Connections adjacent to plastic hinges must be designed with sufficient strength and ductility to sustain the forces and deformations imposed under the required loads. The practical implementation of this rule is that the applicable requirements of Section B3.4 and Chapter J must be strictly adhered to. These provisions for connection design have been developed from plasticity theory and verified by extensive testing, as discussed in ASCE (1971) and in many books and papers. Thus, the connections that meet these provisions are inherently qualified for use in designing structures based on inelastic analysis.

Any method of design that is based on inelastic analysis and satisfies the given general requirements is permitted. These methods may include the use of nonlinear finite element analyses (Crisfield, 1991; Bathe, 1995) that are based on continuum elements to design a single structural component, such as a connection, or the use of second-order inelastic frame analyses (McGuire et al., 2000; Clarke et al., 1992) to design a structural system consisting of beams, columns and connections.

Appendix 1, Sections 1.3.2 and 1.3.3, collectively define provisions that can be used to satisfy the ductility and analysis requirements of Appendix 1, Section 1.3.1. They provide the basis for an approved second-order inelastic frame analysis method. These provisions are not intended to preclude other approaches meeting the requirements of Appendix 1, Section 1.3.1.

2. Ductility Requirements

Because an inelastic analysis will provide for the redistribution of internal forces due to yielding of structural components such as members and connections, it is imperative that these components have adequate ductility and be capable of maintaining their design strength while accommodating inelastic deformation demands. Factors that affect the inelastic deformation capacity of components include the material properties, the slenderness of cross-sectional elements, and the unbraced length. There are two general methods for assuring adequate ductility: (1) limiting the aforementioned factors, and (2) making direct comparisons of the actual inelastic deformation demands with predefined values of inelastic deformation capacities. The former is provided in this Appendix. It essentially decouples inelastic local buckling from inelastic lateral-torsional buckling. It has been part of the plastic design provisions for several previous editions of the Specification. Examples of the latter approach in which ductility demands are compared with defined capacities appear in Galambos (1968b), Kato (1990), Kemp (1996), Gioncu and Petcu (1997), FEMA 350 (FEMA, 2000), ASCE 41 (ASCE, 2013), and Ziemian (2010).

2a. Material

Extensive past research on the plastic and inelastic behavior of continuous beams, rigid frames and connections has amply demonstrated the suitability of steel with yield stress levels up to 65 ksi (450 MPa) (ASCE, 1971).

2b. Cross Section

Design by inelastic analysis requires that, up to the peak of the structure's load-deflection curve, the moments at the plastic hinge locations remain at the level of the plastic moment, which itself should be reduced for the presence of axial force. This implies that the member must have sufficient inelastic rotation capacity to permit the redistribution of additional moments. Sections that are designated as compact in Section B4 have a minimum rotation capacity of approximately $R_{cap} = 3$ (see Figure C-A-1.2) and are suitable for developing plastic hinges. The limiting width-to-thickness ratio designated as λ_p in Table B4.1b, and designated as λ_{pd} in this Appendix, is the maximum slenderness ratio that will permit this rotation capacity to be achieved. Further discussion of the antecedents of these provisions is given in Commentary Section B4.

The additional slenderness limits in Equations A-1-1 through A-1-4 apply to cases not covered in Table B4.1b. Equations A-1-1 and A-1-2, which define height-to-thickness ratio limits of webs of wide-flange and rectangular HSS sections under combined flexure and compression, have been part of the plastic design requirements since the 1969 AISC *Specification* (AISC, 1969) and are based on research documented in *Plastic Design in Steel, A Guide and a Commentary* (ASCE, 1971). The equations for the flanges of HSS and other box sections (Equation A-1-3), and for round HSS sections (Equation A-1-4), are from the *Specification for the Design of Steel Hollow Structural Sections* (AISC, 2000a).

Limiting the slenderness of elements in a cross section to ensure ductility at plastic hinge locations is permissible only for doubly symmetric shapes. In general, single-angle, tee and double-angle sections are not permitted for use in plastic design because the inelastic rotation capacity in the regions where the moment produces compression in an outstanding leg will typically not be sufficient.

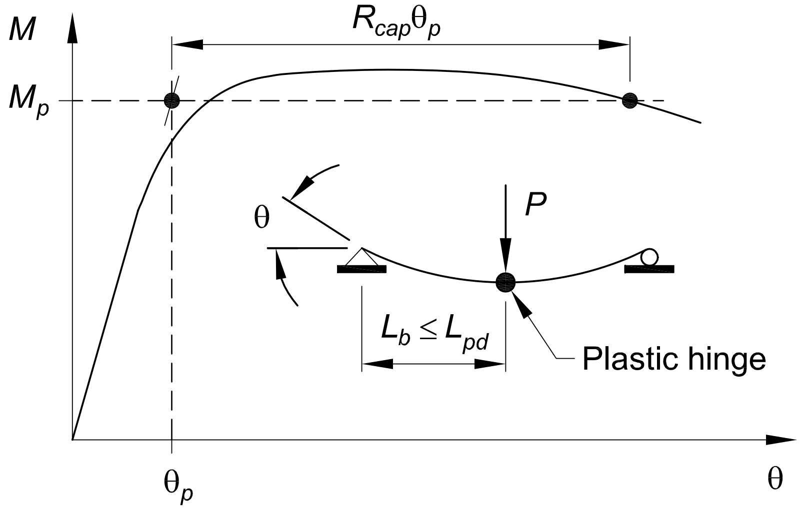


Fig. C-A-1.2. Definition of rotation capacity.

2c. Unbraced Length

The ductility of structural members with plastic hinges can be significantly reduced by the possibility of inelastic lateral-torsional buckling. In order to provide adequate rotation capacity, such members may need more closely spaced bracing than would be otherwise needed for design in accordance with elastic theory. Equations A-1-5 and A-1-7 define the maximum permitted unbraced length in the vicinity of plastic hinges for wide-flange shapes bent about their major axis, and for rectangular shapes and symmetric box-section beams, respectively. These equations are a modified version of those appearing in the 2005 AISC *Specification* (AISC, 2005), which were based on research reported by Yura et al. (1978) and others. The intent of these equations is to ensure a minimum rotation capacity, $R_{cap} \geq 3$, where R_{cap} is defined as shown in Figure C-A-1.2.

Equations A-1-5 and A-1-7 have been modified to account for nonlinear moment diagrams and for situations in which a plastic hinge does not develop at the brace location corresponding to the larger end moment. The moment M_2 in these equations is the larger moment at the end of the unbraced length, taken as positive in all cases. The moment M_1' is the moment at the opposite end of the unbraced length corresponding to an equivalent linear moment diagram that gives the same target rotation capacity. This equivalent linear moment diagram is defined as follows:

(a) For cases in which the magnitude of the bending moment at any location within the unbraced length, M_{max}, exceeds M_2, the equivalent linear moment diagram is taken as a uniform moment diagram with a value equal to M_{max} as illustrated in Figure C-A-1.3(a). Since the equivalent moment diagram is uniform, the appropriate value for L_{pd} can be obtained by using $M_1' / M_2 = +1$.

(b) For cases in which the internal moment distribution along the unbraced length of the beam is indeed linear, or when a linear moment diagram between M_2 and the actual moment M_1 at the opposite end of the unbraced length gives a larger magnitude moment in the vicinity of M_2 as illustrated in Figure C-A-1.3(b), M_1' is taken equal to the actual moment, M_1.

(c) For all other cases in which the internal moment distribution along the unbraced length of the beam is nonlinear and a linear moment diagram between M_2 and the actual moment, M_1, underestimates the moment in the vicinity of M_2, M_1' is defined as the opposite end moment for a line drawn between M_2 and the moment at the middle of the unbraced length, M_{mid}, as illustrated in Figure C-A-1.3(c).

The moments M_1 and M_{mid} are individually taken as positive when they cause compression in the same flange as the moment M_2, and negative, otherwise.

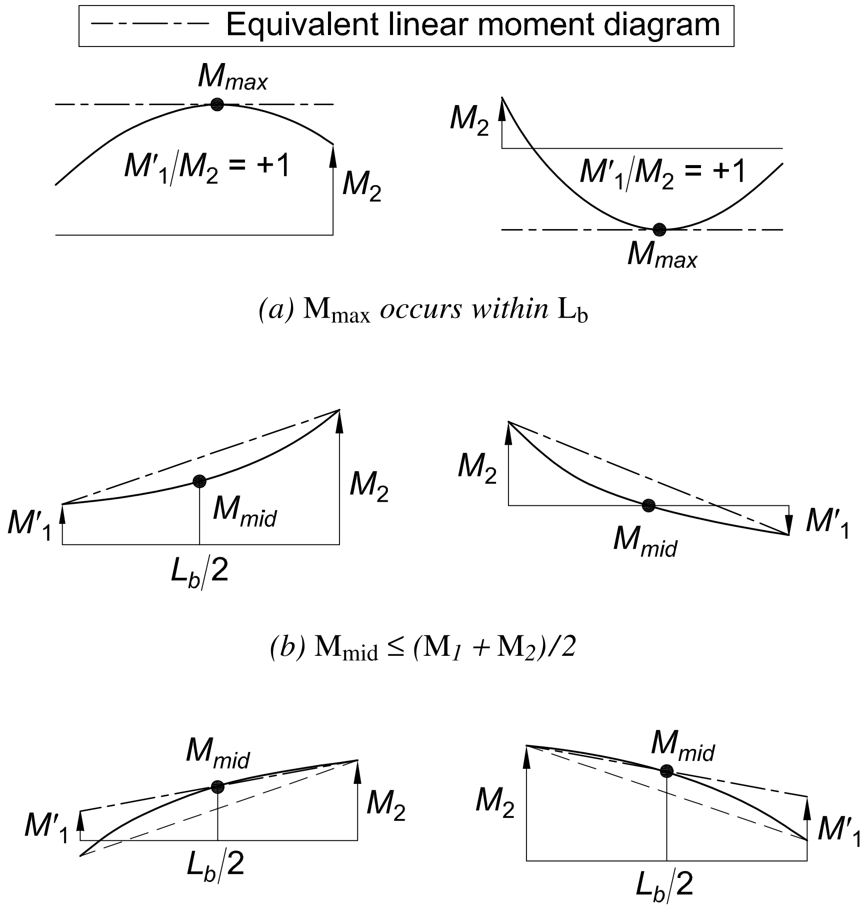
For conditions in which lateral-torsional buckling cannot occur, such as members with square and round compact cross sections and doubly symmetric compact sections subjected to minor-axis bending or sufficient tension, the ductility of the member is not a factor of the unbraced length.

2d. Axial Force

The provision in this section restricts the axial force in a compression member to $0.75F_yA_g$ or approximately 80% of the design yield load, $\phi_c F_y A$. This provision is a cautionary limitation, because insufficient research has been conducted to ensure that sufficient inelastic rotation capacity remains in members subject to high levels of axial force.

3. Analysis Requirements

For all structural systems with members subject to axial force, the equations of equilibrium must be formulated on the geometry of the deformed structure. The use of second-order inelastic analysis to determine load effects on members and connections is discussed in the *Guide to Stability Design Criteria for Metal Structures* (Ziemian, 2010). Textbooks [for example, Chen and Lui (1991), Chen and Sohal (1995), and McGuire et al. (2000)] present basic approaches to inelastic analysis, as well as worked examples and computer software for detailed study of the subject.



(a) M_{max} *occurs within* L_b

(b) $M_{mid} \le (M_1 + M_2)/2$

(c) $M_{mid} > (M_1 + M_2)/2$

Fig. C-A-1.3. Equivalent linear moment diagram used to calculate M'_1.

Continuous and properly braced beams not subject to axial loads can be designed by first-order inelastic analysis (traditional plastic analysis and design). First-order plastic analysis is treated in ASCE (1971), in steel design textbooks [for example, Salmon et al. (2008)], and in textbooks dedicated entirely to plastic design [for example, Beedle (1958), Horne and Morris (1982), Bruneau et al. (2011), and Wong (2009)]. Tools for plastic analysis of continuous beams are readily available to the designer from these and other books that provide simple ways of calculating plastic mechanism loads. It is important to note that such methods use LRFD load combinations, either directly or implicitly, and, therefore, should be modified to include a reduction in the plastic moment capacity of all members by a factor of 0.9. First-order inelastic analysis may also be used in the design of continuous steel-concrete composite beams. Design limits and ductility criteria for both the positive and negative plastic moments are given by Oehlers and Bradford (1995).

3a. Material Properties and Yield Criteria

This section provides an accepted method for including uncertainty in system, member, and connection strength and stiffness. The reduction in yield strength and member stiffness is equivalent to the reduction of member strength associated with the AISC resistance factors used in elastic design. In particular, the factor of 0.9 is based on the member and component resistance factors of Chapters E and F, which are appropriate when the structural system is composed of a single member and in cases where the system resistance depends critically on the resistance of a single member. For systems where this is not the case, the use of such a factor is conservative. The reduction in stiffness will contribute to larger deformations, and, in turn, increased second-order effects.

The inelastic behavior of most structural members is primarily the result of normal stresses in the direction of the longitudinal axis of the member equaling the yield strength of the material. Therefore, the normal stresses produced by the axial force and major- and minor-axis bending moments should be included in defining the plastic strength of member cross sections (Chen and Atsuta, 1976).

Modeling of strain hardening that results in strengths greater than the plastic strength of the cross section is not permitted.

3b. Geometric Imperfections

Because initial geometric imperfections may affect the nonlinear behavior of a structural system, it is imperative that they be included in the second-order analysis. Discussion on how frame out-of-plumbness may be modeled is provided in Commentary Section C2.2. Additional information is provided in ECCS (1984), Bridge and Bizzanelli (1997), Bridge (1998), and Ziemian (2010).

Member out-of-straightness should be included in situations in which it can have a significant impact on the inelastic behavior of the structural system. The significance of such effects is a function of (1) the relative magnitude of the member's applied axial force and bending moments, (2) whether the member is subject to single or reverse curvature bending, and (3) the slenderness of the member.

In all cases, initial geometric imperfections should be modeled to represent the potential maximum destabilizing effects.

3c. Residual Stresses and Partial Yielding Effects

Depending on the ratio of a member's plastic section modulus, Z, to its elastic section modulus, S, the partial yielding that occurs before the formation of a plastic hinge may significantly reduce the flexural stiffness of the member. This is particularly the case for minor-axis bending of I-shapes. Any change to bending stiffness may result in force redistribution and increased second-order effects, and thus needs to be considered in the inelastic analysis.

The impact of partial yielding is further accentuated by the presence of thermal residual stresses, which are due to nonuniform cooling during the manufacturing and fabrication processes. Because the relative magnitude and distribution of these stresses is dependent on the process and the cross-section geometry of the member, it is not possible to specify a single idealized pattern for use in all levels of inelastic analysis. Residual stress distributions used for common hot-rolled doubly symmetric shapes are provided in the literature, including ECCS (1984) and Ziemian (2010). In most cases, the maximum compressive residual stress is 30 to 50% of the yield stress.

The effects of partial yielding and residual stresses may either be included directly in inelastic distributed-plasticity analyses or by modifying plastic hinge based methods of analysis. An example of the latter is provided by Ziemian and McGuire (2002) and Ziemian et al. (2008), in which the flexural stiffness of members are reduced according to the amount of axial force and major- and minor-axis bending moments being resisted. This Specification permits the use of a similar strategy, which is provided in Section C2.3 and described in the Commentary to that section. If the residual stress effect is not included in the analysis and the provisions of Section C2.3 are employed, the stiffness reduction factor of 0.9 specified in Appendix 1, Section 1.3.3a (which accounts for uncertainty in strength and stiffness) must be changed to 0.8. The reason for this is that the provisions given in Section C2.3 assume that the analysis does not account for partial yielding. Also, to avoid cases in which the use of Section C2.3 may be unconservative, it is further required that the yield or plastic hinge criterion used in the inelastic analysis be defined by the interaction Equations H1-1a and H1-1b. This condition on cross-section strength does not have to be met when the residual stress and partial yielding effects are accounted for in the analysis.

APPENDIX 2
DESIGN FOR PONDING

Ponding stability is determined by ascertaining that the conditions of Appendix 2 Equations A-2-1 and A-2-2 are fulfilled. These equations provide a conservative evaluation of the stiffness required to avoid runaway deflection, giving a safety factor of four against ponding instability.

Since Equations A-2-1 and A-2-2 yield conservative results, it may be advantageous to perform a more detailed stress analysis to check whether a roof system that does not meet these equations is still safe against ponding failure.

For the purposes of this Appendix, secondary members are the beams or joists that directly support the distributed ponding loads on the roof of the structure, and primary members are the beams or girders that support the concentrated reactions from the secondary members framing into them. Representing the deflected shape of the primary and critical secondary member as a half-sine wave, the weight and distribution of the ponded water can be estimated and, from this, the contribution that the deflection of each of these members makes to the total ponding deflection can be expressed as follows (Marino, 1966):

For the primary member

$$\Delta_w = \frac{\alpha_p \Delta_o \left[1 + 0.25\pi\alpha_s + 0.25\pi\rho(1+\alpha_s)\right]}{1 - 0.25\pi\alpha_p\alpha_s} \tag{C-A-2-1}$$

For the secondary member

$$\delta_w = \frac{\alpha_s \delta_o \left[1 + \dfrac{\pi^3}{32}\alpha_p + \dfrac{\pi^2}{8\rho}(1+\alpha_p) + 0.185\alpha_s\alpha_p\right]}{1 - 0.25\pi\alpha_p\alpha_s} \tag{C-A-2-2}$$

In these expressions, Δ_o and δ_o are, respectively, the primary and secondary beam deflections due to loading present at the initiation of ponding, and

$$\alpha_p = C_p / \left(1 - C_p\right) \tag{C-A-2-3a}$$

$$\alpha_s = C_s / \left(1 - C_s\right) \tag{C-A-2-3b}$$

$$\rho = \delta_o / \Delta_o = C_s / C_p \tag{C-A-2-3c}$$

Using these expressions for Δ_w and δ_w, the ratios Δ_w/Δ_o and δ_w/δ_o can be computed for any given combination of primary and secondary beam framing using the computed values of coefficients C_p and C_s, respectively, defined in the Appendix.

Even on the basis of unlimited elastic behavior, it is seen that the ponding deflections would become infinitely large unless

$$\left(\frac{C_p}{1-C_p}\right)\left(\frac{C_s}{1-C_s}\right) < \frac{4}{\pi} \tag{C-A-2-4}$$

Since elastic behavior is not unlimited, the effective bending strength available in each member to resist the stress caused by ponding action is restricted to the difference between the yield stress of the member and the stress, f_o, produced by the total load supported by it before consideration of ponding is included.

Note that elastic deflection is directly proportional to stress. The admissible amount of ponding in either the primary or critical (midspan) secondary member, in terms of the applicable ratio, Δ_w/Δ_o and δ_w/δ_o, can be represented as $(0.8F_y - f_o)/f_o$, assuming a safety factor of 1.25 against yielding under the ponding load. Substituting this expression for Δ_w/Δ_o and δ_w/δ_o, and combining with the foregoing expressions for Δ_w and δ_w, the relationship between the critical values for C_p and C_s and the available elastic bending strength to resist ponding is obtained. The curves presented in Figures A-2.1 and A-2.2 are based upon this relationship. They constitute a design aid for use when a more exact determination of required flat roof framing stiffness is needed than given by the Specification provision: $C_p + 0.9C_s \le 0.25$.

Given any combination of primary and secondary framing, the stress index is computed as follows:

For the primary member

$$U_p = \left(\frac{0.8F_y - f_o}{f_o}\right)_p \tag{C-A-2-5}$$

For the secondary member

$$U_s = \left(\frac{0.8F_y - f_o}{f_o}\right)_s \tag{C-A-2-6}$$

where

f_o = stress due to impounded water due to either nominal rain or snow loads (exclusive of the ponding contribution), and other loads acting concurrently, as specified in Section B2, ksi (MPa)

Depending upon geographic location, this loading should include such amount of snow as might also be present, although ponding failures have occurred more frequently during torrential summer rains when the rate of precipitation exceeded the rate of drainage runoff and the resulting hydraulic gradient over large roof areas caused substantial accumulation of water some distance from the eaves.

Given the size, spacing and span of a tentatively selected combination of primary and secondary beams, for example, one may enter Figure A-2.1 at the level of the computed stress index, U_p, determined for the primary beam, move horizontally to the computed C_s value of the secondary beams, then move downward to the abscissa scale. The combined stiffness of the primary and secondary framing is sufficient to prevent ponding if the flexibility coefficient read from this latter scale is larger than the value of C_p computed for the given primary member; if not, a stiffer primary or secondary beam, or combination of both, is required.

If the roof framing consists of a series of equally spaced wall-bearing beams, the beams would be considered as secondary members, supported on an infinitely stiff primary member. For this case, one would use Figure A-2.2. The limiting value of C_s would be determined by the intercept of a horizontal line representing the U_s value and the curve for $C_p = 0$.

The ponding deflection contributed by a metal deck is usually such a small part of the total ponding deflection of a roof panel that it is sufficient merely to limit its moment of inertia to 0.000025 (3 940) times the fourth power of its span length [in.^{4} per foot (mm^4 per meter) of width normal to its span], as provided in Equation A-2-2. However, the stability against ponding of a roof consisting of a metal roof deck of relatively slender depth-to-span ratio, spanning between beams supported directly on columns, may need to be checked. This can be done using Figures A-2.1 or A-2.2 with the following computed values:

U_p = stress index for the supporting beam
U_s = stress index for the roof deck
C_p = flexibility coefficient for the supporting beams
C_s = flexibility coefficient for 1 ft (0.305 m) width of the roof deck (S = 1.0)

Since the shear rigidity of the web system is less than that of a solid plate, the moment of inertia of steel joists and trusses should be taken as somewhat less than that of their chords (Fisher and Pugh, 2007).

APPENDIX 3

FATIGUE

When the limit state of fatigue is a design consideration, its severity is most significantly affected by the number of load applications, the magnitude of the stress range, and the severity of the stress concentrations associated with particular details. Issues of fatigue are not normally encountered in building design; however, when encountered and if the severity is great enough, fatigue is of concern and all provisions of this Appendix must be satisfied.

3.1. GENERAL PROVISIONS

This Appendix deals with high cycle fatigue (i.e., > 20,000 cycles); this behavior occurs when elastic stresses are involved. In situations where inelastic (plastic) stresses are involved, fatigue cracks may initiate at far fewer than 20,000 cycles—perhaps as few as a dozen. However, unlike the conditions prescribed in this Appendix, low cycle fatigue involves cyclic, inelastic stresses. This is because the applicable cyclic allowable stress range will be limited by the static allowable stress. At low levels of cyclic tensile stress, a point is reached where the stress range is so low that fatigue cracking will not initiate regardless of the number of cycles of loading. This level of stress is defined as the fatigue threshold, F_{TH}.

Extensive test programs using full-size specimens, substantiated by theoretical stress analysis, have confirmed the following general conclusions (Fisher et al., 1970; Fisher et al., 1974):

(1) Stress range and notch severity are the dominant stress variables for welded details and beams.

(2) Other variables such as minimum stress, mean stress and maximum stress are not significant for design purposes.

(3) Structural steels with a specified minimum yield stress of 36 to 100 ksi (250 to 690 MPa) do not exhibit significantly different fatigue strengths for given welded details fabricated in the same manner.

Fatigue crack growth rates are generally inversely proportional to the modulus of elasticity and therefore, at higher temperatures, crack growth rates increase. At 500°F (260°C), crack growth rates on ASTM A212B steel (ASTM, 1967) are essentially the same as for room temperature (Hertzberg et al., 2012). The Appendix is conservatively limited to applications involving temperatures not to exceed 300°F (150°C). Elevated temperature applications may also have corrosion effects that are not considered by the Appendix.

The Appendix does not have a lower temperature limit because fatigue crack growth rates are lower. Fatigue tests as low as −100°F (−75°C) have been conducted with no observed change in crack growth rates (Roberts et al., 1980). It should be recognized that at low temperatures, brittle fracture concerns increase. The critical size to which a crack can grow before the onset of brittle fracture will be smaller for low temperature applications than will be the case for a room temperature application.

3.2. CALCULATION OF MAXIMUM STRESSES AND STRESS RANGES

Fluctuation in stress that does not involve tensile stress does not cause crack propagation and is not considered to be a fatigue situation. On the other hand, in elements of members subject solely to calculated compressive stress, fatigue cracks may initiate in regions of high tensile residual stress. In such situations, the cracks generally do not propagate beyond the region of the residual tensile stress, because the residual stress is relieved by the crack. For this reason, stress ranges that are completely in compression need not be investigated for fatigue. For cases involving cyclic reversal of stress, the calculated stress range must be taken as the sum of the compressive stress and the tensile stress caused by different directions or patterns of the applied live load. When part of the stress cycle is compressive, the stress range may exceed $0.66F_y$.

3.3. PLAIN MATERIAL AND WELDED JOINTS

Fatigue resistance has been derived from an exponential relationship between the number of cycles to failure, N, and the stress range, S_r, called an S-N relationship, of the form

$$N = \frac{C_f}{S_r^n} \tag{C-A-3-1}$$

The general relationship is often plotted as a linear log-log function (Log $N = A - n$ Log S_r). Figure C-A-3.1 shows the family of fatigue resistance curves identified as stress categories A, B, B′, C, D, E, E′ and G. These relationships were established based on an extensive database developed in the United States and abroad (Keating and Fisher, 1986). The allowable stress range has been developed by adjusting the coefficient, C_f, so that a design curve is provided that lies two standard deviations of the standard error of estimate of the fatigue cycle life below the mean S-N relationship of the actual test data. These values of C_f correspond to a probability of failure of 2.5% of the design life.

The number of stress range fluctuations in a design life, n_{SR}, in Equation A-3-1, can often be calculated as

$$\begin{aligned} n_{SR} = &\ (\text{number of stress fluctuations per day}) \times \\ &\ (365 \text{ days}) \times (\text{years in design life}) \end{aligned} \tag{C-A-3-2}$$

Stress category F is shown in Figure C-A-3.2 and has a slope different than the other stress categories. The fatigue resistance of stress category C′ or C″ details is determined by applying a reduction factor, R_{PJP} or R_{FIL}, respectively, to the stress category C stress range, which shifts the fatigue resistance curve for stress category C downward by a factor proportional to the reduction. Unlike stress category C, stress category C′ and C″ details do not have a fatigue threshold.

Prior to the 1999 AISC *Load and Resistance Factor Design Specification for Structural Steel Buildings* (AISC, 2000b), stepwise tables meeting the criteria discussed in the foregoing, including cycles of loading, stress categories, and allowable stress ranges were provided in the Specification. A single table format (Table A-3.1) was introduced in the 1999 AISC LRFD *Specification* that provides the stress categories, ingredients for the applicable equation, and information and examples, including the sites of concern for potential crack initiation (AISC, 2000b).

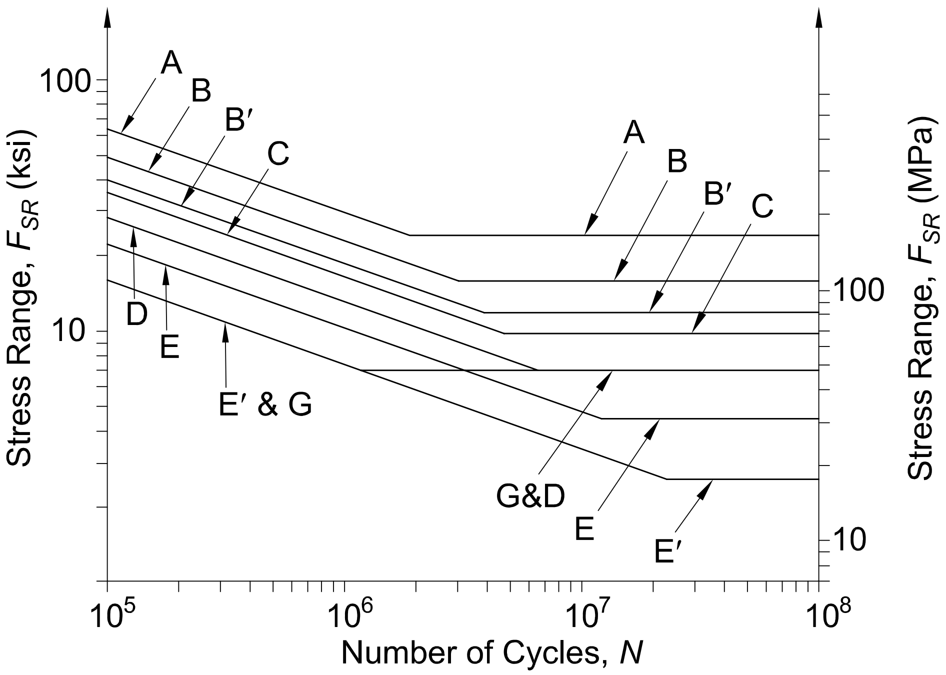


Fig. C-A-3.1. Fatigue resistance curves.

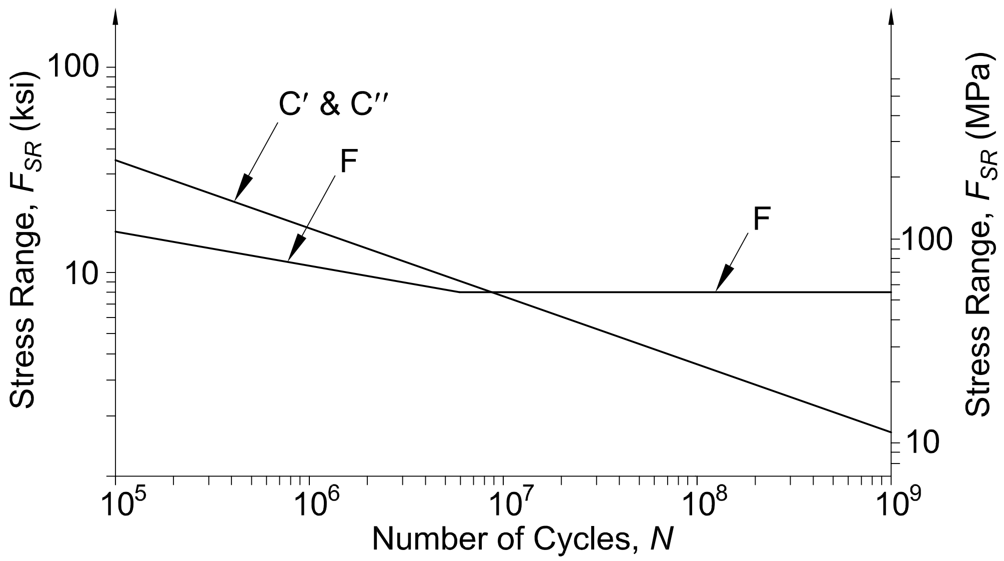


Fig. C-A-3.2. Fatigue resistance curves for stress categories C and F.

Table A-3.1 is organized into eight sections of general conditions for fatigue design, as follows:

(1) Section 1 provides information and examples for the steel material at copes, holes, cutouts or as produced.

(2) Section 2 provides information and examples for various types of mechanically fastened joints, including eyebars and pin plates.

(3) Section 3 provides information related to welded connections used to join built-up members, such as longitudinal welds, access holes and reinforcements.

(4) Section 4 deals only with longitudinal load carrying fillet welds at shear splices.

(5) Section 5 provides information for various types of groove and fillet welded joints that are transverse to the applied cyclic stress.

(6) Section 6 provides information on a variety of groove-welded attachments to flange tips and web plates, as well as similar attachments, connected with either fillet or partial-joint-penetration groove welds.

(7) Section 7 provides information on several short attachments to structural members.

(8) Section 8 collects several miscellaneous details, such as shear connectors, shear on the throat of fillet, plug and slot welds, and their impact on base metal. It also provides for tension on the stress area of various bolts, threaded anchor rods, and hangers.

A similar format and consistent criteria are used by other specifications.

When fabrication details involving more than one stress category occur at the same location in a member, the stress range at that location must be limited to that of the most restrictive category. The need for a member larger than required by static loading will often be eliminated by locating notch-producing fabrication details in regions subject to smaller ranges of stress.

A detail not explicitly covered before 1989 (AISC, 1989) was added in the 1999 AISC LRFD *Specification* (AISC, 2000b) to cover tension-loaded plate elements connected at their end by transverse partial-joint-penetration groove or fillet welds in which there is more than a single site for the initiation of fatigue cracking, one of which will be more critical than the others depending upon welded joint type and size, and material thickness (Frank and Fisher, 1979). Regardless of the site within the joint at which potential crack initiation is considered, the allowable stress range provided is applicable to connected material at the toe of the weld.

3.4. BOLTS AND THREADED PARTS

The fatigue resistance of bolts subject to tension is predictable in the absence of pretension and prying action; provisions are given for such nonpretensioned details as hanger rods and anchor rods. In the case of pretensioned bolts, deformation of the connected parts through which pretension is applied introduces prying action, the magnitude of which is not completely predictable (Kulak et al., 1987). The effect of prying is not limited to a change in the average axial tension on the bolt but includes bending in the threaded area under the nut. Because of the uncertainties in calculating prying effects, definitive provisions for the allowable stress range for bolts subject to applied axial tension are not included in this Specification. To limit the uncertainties regarding prying action on the fatigue of pretensioned bolts in details which introduce prying, the allowable stress range provided in Table A-3.1 is appropriate for extended cyclic loading only if the prying induced by the applied load is small.

The tensile stress range of bolts that are pretensioned to the requirements of Table J3.1 or J3.1M can be conservatively approximated as 20% of the absolute value of the applied cyclic axial load and moment from dead, live and other loads. AISC Design Guide 17, *High Strength Bolts: A Primer for Structural Engineers* (Kulak, 2002) states that the final bolt force is the initial pretension force plus a component of the externally applied load that depends on the relative areas of the bolt and the area of the connected material in compression. Test results show that this approach is a good predictor and that the increase in bolt pretension can be expected to be on the order of not more than about 5% to 10%, which affirms that the 20% rule is a conservative upper bound. The approximated stress range is compared with the allowable and threshold stress range.

Fatigue provisions in Appendix 3 and in the RCSC *Specification* (RCSC, 2014) are applied differently, but produce similar results. Some key differences are:

(1) Appendix 3 allows bolts that are pretensioned or not pretensioned to be subjected to cyclic axial loads, where the RCSC *Specification* only allows pretensioned bolts.

(2) Appendix 3 is applied using a bolt net area in tension, where RCSC *Specification* Table 5.2 is applied based upon the cross-sectional area determined from the nominal diameter.

(3) Appendix 3 is applied by determining a maximum allowable stress range and a stress range threshold regardless of the bolt material, where RCSC *Specification* Table 5.2 is applied by determining a maximum bolt stress, which does depend on the bolt material. Therefore, the stresses obtained from Appendix 3 should be compared to the tensile stress range including prying, while the stresses obtained from RCSC *Specification* Table 5.2 should be compared to the total applied tensile stress including prying.

Nonpretensioned fasteners are not permitted under this Specification for joints subject to cyclic shear forces. Bolts installed in joints meeting all the requirements for slip-critical connections survive unharmed when subject to cyclic shear stresses sufficient to fracture the connected parts; provisions for such bolts are given in Section 2 of Table A-3.1.

3.5. FABRICATION AND ERECTION REQUIREMENTS FOR FATIGUE

It is essential that when longitudinal backing bars are to be left in place, they be continuous or spliced using flush-ground complete-joint-penetration groove welds before attachment to the parts being joined. Otherwise, the transverse nonfused section constitutes a crack-like defect that can lead to premature fatigue failure or even brittle fracture of the built-up member.

Welds that attach left-in-place longitudinal backing to the structural member will affect the fatigue performance of the structural member. Continuous longitudinal fillet welds are stress category B; intermittent fillet welds are stress category E. Longitudinal backing may be attached to the joint by tack welding in the groove, attaching the backing to one member with a fillet weld, or attaching the backing to both members with fillet welds.

In transversely loaded joints subjected to tension, a lack-of-fusion plane in T-joints acts as an initial crack-like condition. In groove welds, the root at the backing bar often has discontinuities that can reduce the fatigue resistance of the connection. Removing the backing, back gouging the joint, and rewelding eliminates the undesirable discontinuities.

The addition of contoured fillet welds at transverse complete-joint-penetration groove welds in T- and corner joints and at reentrant corners reduces the stress concentration and improves fatigue resistance.

Experimental studies on welded built-up beams demonstrated that if the surface roughness of flame-cut edges was less than 1,000 μin. (25 μm), fatigue cracks would not develop from the flame-cut edge but from the longitudinal fillet welds connecting the beam flanges to the web (Fisher et al., 1970, 1974). This provides stress category B fatigue resistance without the necessity for grinding flame-cut edges.

Reentrant corners at cuts, copes and weld access holes provide a stress concentration point that can reduce fatigue resistance if discontinuities are introduced by punching or thermal cutting. Reaming sub-punched holes and grinding the thermally cut surface to bright metal prevents any significant reduction in fatigue resistance.

For Cases 1.4, 1.5 and 3.3 in Table A-3.1, Yam and Cheng (1990) reported that fatigue performance of reentrant corners less than 1 in. and not ground smooth is similar to stress category C when calculated with a stress concentration factor. To be consistent with other cases in this Appendix, reentrant corners with radii as small as $^3/_8$ in. and not ground are assigned stress category E′ and do not have to be calculated with a stress concentration factor. Reentrant corners with a radius of at least 1 in. and meeting surface requirements and NDE requirements are associated with stress category C, except for built-up members, where it is stress category D.

For Cases 3.5 and 3.6 in Table A-3.1, coverplates and other attachments wider than the flange with welds across the ends are subject to fatigue stress categories E and E′, depending on the thickness of the flange. There has been little research on connections with coverplates that are wider than the flange, where the flange is thicker than 0.8 in. and without welds across the ends; therefore, this detail is not recommended, as indicated for Case 3.7. Cover-plated flanges thicker than 0.8 in. are permitted when the ends are welded.

As shown in Case 7.1 in Table A-3.1, base metal subject to longitudinal loading at details with parallel or transverse welds with no transition radius is subject to stress category E or E′ fatigue stresses depending on the length and thickness of the attachment.

Attachments with no transition result in abrupt changes in stiffness of the stressed member corresponding to the stress the attachment attracts from the main member. Larger attachments attract more stress and make the attachment connection stiffer. These stiffness changes act as stress concentrations and aggravate fatigue crack growth.

The use of run-off tabs at transverse butt-joint groove welds enhances weld soundness at the ends of the joint. Subsequent removal of the tabs and grinding of the ends flush with the edge of the member removes discontinuities that are detrimental to fatigue resistance.

APPENDIX 4

STRUCTURAL DESIGN FOR FIRE CONDITIONS

4.1. GENERAL PROVISIONS

Appendix 4 provides structural engineers with criteria for designing steel-framed building systems and components, including columns, and floor and truss assemblies, for fire conditions. Compliance with the performance objective in Section 4.1.1 can be demonstrated by either structural analysis or component qualification testing.

Thermal expansion and progressive decrease in strength and stiffness are the primary structural responses to elevated temperatures that may occur during fires. An assessment of a design of building components and systems based on structural mechanics that allows designers to address the fire-induced restrained thermal expansions, deformations and material degradation at elevated temperatures can lead to a more robust structural design for fire conditions.

1. Performance Objective

The performance objective underlying the provisions in this Specification is that of life safety. Fire safety levels should depend on the building occupancy, height of the building, the presence of active fire mitigation measures, and the effectiveness of fire-fighting. Three limit states exist for elements serving as fire barriers (compartment walls and floors): (1) heat transmission leading to unacceptable rise of temperature on the unexposed surface; (2) breach of barrier due to cracking or loss of integrity; and (3) loss of load-bearing capacity. In general, all three must be considered by the engineer to achieve the desired performance. These three limit states are interrelated in fire-resistant design. For structural elements that are not part of a separating element, the governing limit state is loss of load-bearing capacity.

Specific performance objectives for a facility are determined by the stakeholders in the building process, within the context of the general performance objective and limit states discussed in the preceding paragraph. In some instances, applicable building codes may stipulate that steel in buildings of certain occupancies and heights be protected by fire-resistant materials or assemblies to achieve specified performance goals.

2. Design by Engineering Analysis

The strength design criteria for steel beams and columns at elevated temperatures are based on Tagaki and Deierlein (2007). These strength equations do not transition smoothly to the strength equations used to design steel members under ambient conditions. The practical implications of the discontinuity are minor, as the temperatures in the structural members during a fully developed fire are far in excess of the temperatures at which this discontinuity might otherwise be of concern in design. Nevertheless, to avoid the possibility of misinterpretation, the scope of applicability of the analysis methods in Appendix 4, Section 4.2 is limited to temperatures above 400°F (200°C).

Structural behavior under severe fire conditions is highly nonlinear in nature as a result of the constitutive behavior of materials at elevated temperatures and the relatively large deformations that may develop in structural systems at sustained elevated temperatures. As a result of this behavior, it is difficult to develop design equations to ensure the necessary level of structural performance during severe fires using elastically based ASD methods. Accordingly, structural design for fire conditions by analysis should be performed using LRFD methods, in which the nonlinear structural actions arising during severe fire exposures and the temperature-dependent design strengths can be properly taken into account.

4. Load Combinations and Required Strength

Fire safety measures are aimed at three levels of performance: (1) to prevent the outbreak of fires through elimination of ignition sources or hazardous practices; (2) to prevent uncontrolled fire development and flashover through early detection and suppression; and (3) to prevent loss of life or structural collapse through fire protection systems, compartmentation, exit ways, and provision of general structural integrity and other passive measures. Specific structural design provisions to check structural integrity and risk of progressive failure due to severe fires can be developed from principles of structural reliability theory (Ellingwood and Leyendecker, 1978; Ellingwood and Corotis, 1991).

The limit state probability of failure due to fire can be written as

$$P(F) = P(F \mid D,I)\ P(D \mid I)\ P(I) \tag{C-A-4-1}$$

where

$P(I)$ = probability of ignition
$P(D \mid I)$ = probability of development of a structurally significant fire
$P(F \mid D,I)$ = probability of failure, given the occurrence of the two preceding events

Measures taken to reduce $P(I)$ and $P(D \mid I)$ are mainly nonstructural in nature. Measures taken by the structural engineer to design fire resistance into the structure impact the term $P(F \mid D,I)$.

The development of structural design requirements requires a target reliability level, reliability being measured by $P(F)$ from Equation C-A-4-1. Analysis of reliability of structural systems for gravity dead and live load (Galambos et al., 1982) suggests that the limit state probability of individual steel members and connections is on the order of 10^{-5} to 10^{-4} per year. For redundant steel frame systems, $P(F)$ is on the order of 10^{-6} to 10^{-5}. The *de minimis* risk, that is, the level below which the risk is of regulatory or legal concern and the economic or social benefits of risk reduction are small, is on the order of 10^{-7} to 10^{-6} per year (Pate-Cornell, 1994). If $P(I)$ is on the order of 10^{-4} per year for typical buildings and $P(D \mid I)$ is on the order of 10^{-2} for office or commercial buildings in urban areas with suppression systems or other protective measures, then $P(F \mid D,I)$ should be approximately 0.1 to ascertain that the risk due to structural failure caused by fire is socially acceptable.

The use of first-order structural reliability analysis based on this target (conditional) limit state probability leads to the gravity load combination presented as Equation A-4-1. Load combination Equation A-4-1 is similar to Equation 2.5-1 that appears in ASCE/SEI 7-16 (ASCE, 2016), where the probabilistic basis for load combinations for extraordinary events is explained in detail. The factor 0.9 is applied to the dead load when the effect of the dead load is to stabilize the structure; otherwise, the factor 1.2 is applied. The companion action load factors on L and S in that equation reflect the fact that the probability of a coincidence of the peak time-varying load with the occurrence of a fire is negligible (Ellingwood and Corotis, 1991).

The overall stability of the structural system is checked by considering the effect of a small notional lateral load equal to 0.2% of the story gravity force, as defined in Section C2.2, acting in combination with the gravity loads. The required strength of the structural component or system, designed using the load combination given by Equation A-4-1, is on the order of 60 to 70% of the required strength under full gravity or wind load at normal temperature.

4.2. STRUCTURAL DESIGN FOR FIRE CONDITIONS BY ANALYSIS

1. Design-Basis Fire

Once a fuel load has been agreed upon for the occupancy, the designer should demonstrate the effect of various fires on the structure by assessing the temperature-time relationships for various ventilation factors. NFPA 557 (NFPA, 2012) and SFPE S.01 (SFPE, 2011), as well as other published standards, can be consulted in this regard. These heating effects may result in different structural responses, and it is useful to demonstrate the capability of the structure to withstand such exposures. The effects of a localized fire should also be assessed to ascertain that local damage is not excessive. Based on these results, members, connections and edge details can be specified to provide a structure that is sufficiently robust.

1a. Localized Fire

Localized fires may occur in large open spaces, such as the pedestrian area of covered malls, concourses of airport terminals, warehouses, and factories, where fuel packages are separated by large aisles or open spaces. In such cases, the radiant heat flux can be estimated by a point source approximation, requiring the heat release rate of the fire and separation distance between the center of the fuel package and the closest surface of the steelwork. The heat release rate can be determined from experimental results or may be estimated if the mass loss rate per unit floor area occupied by the fuel is known. Otherwise, a steady-state fire may be assumed.

1b. Post-Flashover Compartment Fires

Caution should be exercised when determining temperature-time profiles for spaces with high aspect ratios, for example 5:1 or greater, or for large spaces, for example those with an open (or exposed) floor area in excess of 5,000 ft^2 (460 m^2). In such cases, it is unlikely that all combustibles will burn in the space simultaneously. Instead, burning will be most intense in, or perhaps limited to, the combustibles nearest to a ventilation source. For modest-sized compartments with low aspect ratios, the temperature history of the design fire can be determined by algebraic equations or computer models, such as those described in the SFPE *Handbook of Fire Protection Engineering* (SFPE, 2002).

Caution should be exercised when determining the fire duration for spaces with high aspect ratios, for example 5:1 or greater, or for large spaces, for example those with a floor area in excess of 5,000 ft^2 (460 m^2). The principal difficulty lies in obtaining a realistic estimate for the mass loss rate, given that all combustibles within the space may not be burning simultaneously. Failure to recognize uneven burning will result in an overestimation of the mass burning rate and an underestimation of the fire duration by a significant margin. Note: Some computation methods may implicitly determine the duration of the fire, in which case the calculation of mass loss rate is unnecessary.

Where a parametric curve is used to define a post-flashover fire, the duration is determined by means of the fuel versus ventilation provisions, not explicitly by loss of mass. This clause should not limit the use of temperature-time relationships to those where duration is calculated, as stated in the foregoing, as these tend to be localized fires and external fire.

1c. Exterior Fires

A design guide is available for determining the exposure resulting from an exterior fire (AISI, 1979).

1d. Active Fire Protection Systems

Due consideration should be given to the reliability and effectiveness of active fire protection systems when describing the design-basis fire. When an automatic sprinkler system is installed, the total fuel load may be reduced by up to 60% based on Eurocode 1 (CEN, 1991). The maximum reduction in the fuel load should be considered only when the automatic sprinkler system is considered to be of the highest reliability, for example reliable and adequate water supply, supervision of control valves, regular schedule for maintenance of the automatic sprinkler system developed in accordance with NFPA (2002a), or alterations of the automatic sprinkler system are considered any time alterations for the space are considered.

For spaces with automatic smoke and heat vents, computer models are available to determine the smoke temperature (SFPE, 2002). Reduction in the temperature profile as a result of smoke and heat vents should only be considered for reliable installations of smoke and heat vents. As such, a regular maintenance schedule for the vents needs to be established in accordance with NFPA (2002b).

2. Temperatures in Structural Systems under Fire Conditions

The heat transfer analysis may range from one-dimensional analyses where the steel is assumed to be at uniform temperature to three-dimensional analyses. The uniform temperature assumption is appropriate in a “lumped heat capacity analysis” where a steel column, beam or truss element is uniformly heated along the entire length and around the entire perimeter of the exposed section and the protection system is uniform along the entire length and around the entire perimeter of the section. In cases with nonuniform heating or where different protection methods are used on different sides of the column, a one-dimensional analysis should be conducted for steel column assemblies. Two-dimensional analyses are appropriate for beams, bar joists or truss elements supporting floor or roof slabs.

Heat transfer analyses should consider changes in material properties with increasing temperature for all materials included in the assembly. This may be done in the lumped heat capacity analysis using an effective property value, determined at a temperature near the estimated midpoint of the temperature range expected to be experienced by that component over the duration of the exposure. In the one- and two-dimensional analyses, the variation in properties with temperature should be explicitly included.

The boundary conditions for the heat transfer analysis should consider radiation heat transfer in all cases and convection heat transfer if the exposed element is submerged in the smoke or is being subjected to flame impingement. The presence of fire-resistive materials in the form of insulation, heat screens, or other protective measures should be taken into account, if appropriate.

Lumped Heat Capacity Analysis. This first-order analysis to predict the temperature rise of steel structural members can be conducted using algebraic equations iteratively. This approach assumes that the steel member has a uniform temperature, applicable to cases where the steel member is unprotected or uniformly protected (on all sides), and is exposed to fire around the entire perimeter of the assembly containing the steel member. Caution should be used when applying this method to steel beams supporting floor and roof slabs, as the approach will overestimate the temperature rise in the beam. In addition, where this analysis is used as input for the structural analysis of a fire-exposed steel beam supporting a floor and roof slab, the thermally induced moments will not be simulated as a result of the uniform temperature assumption.

Unprotected Steel Members. The temperature rise in an unprotected steel section in a short time period is determined by:

$$\Delta T_s = \frac{a}{c_s\left(\dfrac{W}{D}\right)}\left(T_F - T_s\right)\Delta t \qquad \text{(C-A-4-2)}$$

where

a = heat transfer coefficient, Btu/(ft^2-s-°F) (W/m^2-°C)

= $a_c + a_r$ (C-A-4-3)

a_c = convective heat transfer coefficient

a_r = radiative heat transfer coefficient, given as:

$$= \frac{S_B \varepsilon_F}{T_F - T_S}\left(T_{FK}^4 - T_{SK}^4\right) \qquad \text{(C-A-4-4)}$$

c_s = specific heat of the steel, Btu/lb-°F (J/kg-°C)

D = heat perimeter, in. (m)

TABLE C-A-4.1
Guidelines for Estimating ε_F

Type of Assembly	ε_F
Column, exposed on all sides	0.7
Floor beam: Embedded in concrete floor slab, with only bottom flange of beam exposed to fire	0.5
Floor beam, with concrete slab resting on top flange of beam	
Flange width-to-beam depth ratio ≥ 0.5	0.5
Flange width-to-beam depth ratio < 0.5	0.7
Box girder and lattice girder	0.7

S_B = Stefan-Boltzmann constant = 3.97×10^{-14} Btu / ft-in-s-°F^4 (5.67×10^{-8} W/m^2-°C^4)
T_F = temperature of the fire, °F (°C)
T_{FK} = temperature of the fire, °K
= $(T_S + 459)/1.8$ for T_F in °F
= $(T_S + 273)$ for T_F in °C
T_S = temperature of the steel, °F (°C)
T_{SK} = temperature of the steel, °K
= $(T_S + 459)/1.8$ for T_S in °F
= $(T_S + 273)$ for T_S in °C
W = weight (mass) per unit length, lb/ft (kg/m)
ε_F = emissivity of the fire and view coefficient as suggested in Table C-A-4.1
Δt = time interval, s

For the standard exposure, the convective heat transfer coefficient, a_c, can be approximated as 1.02×10^{-4} Btu / (ft-in.-s-°F) (25 W/m^2-°C).

For accuracy reasons, a maximum limit for the time step, Δt, is suggested as 5 s.

The fire temperature needs to be determined based on the results of the design fire analysis. As alternatives, the standard time-temperature curves indicated in ASTM E119 (ASTM, 2009b) for building fires or ASTM E1529 (ASTM, 2006) for petrochemical fires may be selected.

Protected Steel Members. This method is most applicable for steel members with contour protection schemes, in other words where the insulating or (protection) material follows the shape of the section. Application of this method for box protection methods will generally result in the temperature rise being overestimated. The approach assumes that the outside insulation temperature is approximately equal to the fire temperature. Alternatively, a more complex analysis may be conducted that determines the exterior insulation temperature from a heat transfer analysis between the assembly and the exposing fire environment.

If the thermal capacity of the insulation is much less than that for the steel, such that the following inequality is satisfied:

$$c_s W/D > 2d_p \rho_p c_p \tag{C-A-4-5}$$

then, Equation C-A-4-6 can be applied to determine the temperature rise in the steel:

$$\Delta T_s = \frac{k_p}{c_s d_p \left(\dfrac{W}{D}\right)} \left(T_F - T_s\right) \Delta t \tag{C-A-4-6}$$

If the thermal capacity of the insulation needs to be considered (such that the inequality in Equation C-A-4-5 is not satisfied), then Equation C-A-4-7 should be applied:

$$\Delta T_s = \frac{k_p}{d_p} \left[\frac{T_F - T_s}{c_s \left(\dfrac{W}{D}\right) + \dfrac{c_p \rho_p d_p}{2}} \right] \Delta t \tag{C-A-4-7}$$

where

c_p = specific heat of the fire protection material, Btu/lb-°F (J/kg-°C)
d_p = thickness of the fire protection material, in. (m)
k_p = thermal conductivity of the fire protection material, Btu/ft-sec-°F (W/m-°C)
ρ_p = density of the fire protection material, lb/ft^3 (kg/m^3)

Note that the maximum limit for the time step, Δt, should be 5 s.

Ideally, material properties should be considered as a function of temperature. Alternatively, characteristic material properties may be evaluated at a mid-range temperature expected for that component or from calibrations to test data. For protected steel members, the material properties may be evaluated at 572°F (300°C), and for protection materials, a temperature of 932°F (500°C) may be considered.

External Steelwork. Temperature rise can be determined by applying the following equation:

$$\Delta T_s = \frac{q''}{c_s \left(\dfrac{W}{D}\right)} \Delta t \tag{C-A-4-8}$$

where q'' is the net heat flux incident on the steel member.

All given equations assume applications in consistent dimensional units within either the customary U.S. or SI systems. For Equations C-A-4-2, C-A-4-5, C-A-4-6 and C-A-4-7, the W/D in the U.S. customary system needs to be replaced by M/D for SI systems, where M is the mass per unit length. To convert W/D, typically given in lb/ft/in. to the appropriate M/D units of kg/m^2 in SI, multiply the lb/ft/in. value for W/D by 58.6.

Advanced Calculation Methods. The thermal response of steel members may be assessed by application of a computer model. A computer model for analyzing the thermal response of the steel members should consider the following:

(1) Exposure conditions are established based on the definition of a design fire. The exposure conditions need to be stipulated either in terms of a time-temperature history, along with radiation and convection heat transfer parameters associated with the exposure, or as an incident heat flux. The incident heat flux is depend-

ent on the design fire scenario and the location of the structural assembly. The heat flux emitted by the fire or smoke can be determined from a fire hazard analysis. Exposure conditions are established based on the definition of a design fire. The exposure conditions are stipulated either in terms of a time-temperature history, along with radiation and convection heat transfer parameters associated with the exposure, or as an incident heat flux.

(2) Temperature-dependent material properties.

(3) Temperature variation within the steel member and any protection components, especially where the exposure varies from side-to-side.

3. Material Strengths at Elevated Temperatures

The material properties used to assess the performance of steel and concrete structures at elevated temperatures should account for nonlinearities in stress versus strain response, thermal expansion, and time dependent creep effects. As these effects are highly variable, the uncertainties in the properties should be considered in measuring and using the derived properties to determine whether structural components and systems achieve the required reliability index target for deformation and strength limit states. While the Specification permits the determination of steel material properties from test data, in practice this is challenging, given that there are no universally accepted test methods to consistently establish all of the required properties.

In lieu of test data on material properties, this Specification allows the use of properties for steel and concrete at elevated temperatures adopted from the ECCS *Model Code on Fire Engineering* (ECCS, 2001), Section III.2, "Material Properties." These generic properties are consistent with those in Eurocode 3 (CEN, 2005b) and Eurocode 4 (CEN, 2009), and reflect the consensus of the international fire engineering and research community. As such, they are considered to implicitly incorporate the nonlinear stress versus strain response, including the effects of creep, as appropriate for evaluating structural response of buildings under fires. The background information for the mechanical properties of structural steel at elevated temperatures can be found in Cooke (1988) and Kirby and Preston (1988).

The stress-strain response of steel at elevated temperatures is more nonlinear than at room temperature and experiences less strain hardening. At elevated temperatures, the deviation from linear behavior is represented by the proportional limit, $F_p(T)$, and the yield strength, $F_y(T)$, is defined at a 2% strain as shown in Figure C-A4.1. At 1,000ºF (540ºC), the yield strength, $F_y(T)$, reduces to about 66% of its value at room temperature, and the proportional limit $F_p(T)$ occurs at 29% of the ambient temperature yield strength, F_y. Finally, at temperatures above 750ºF (400ºC), the elevated temperature ultimate strength is essentially the same as the elevated temperature yield strength; in other words, $F_y(T)$ is equal to $F_u(T)$.

Table A-4.2.3 provides properties for Group A and B high-strength bolts at elevated temperatures expressed as strength retention factors, which are the ratios of bolt shear or tension strength at high temperatures with respect to the corresponding property at ambient temperature. The strength retention factors are based on a review of available experimental data (Gonzalez and Lange, 2009; Hanus et al., 2010, 2011; Kirby, 1995;

Kodur et al., 2012; Li et al., 2001; Lou et al., 2010; Yu and Frank, 2009), and are consistent with values given in Eurocode 3 (CEN, 2005b). The available data indicates that retention factors are similar for both the shear and tensile strength of bolts, and are also similar for both Group A and B bolts. Consequently, Table A-4.2.3 specifies a single set of retention factors which are not, however, applicable to Group C bolts.

The strength of bolts depends both on temperature and temperature history. The strength retention factors given in Table A-4.2.3 assume the given temperature is the highest temperature to which the bolt has been exposed. For example, if a bolt is heated to 1,000°F (540°C), and this is the highest temperature the bolt has seen, the strength of the bolt at 1,000°F (540°C) can be computed as 42% of its normal room temperature value, as indicated in Table A-4.2.3. However, if the bolt has been heated to, say 1,600°F (870°C), and then cools to 1,000°F (540°C), then the strength of the bolt at 1,000°F (540°C) may be less than 42% of the room temperature value. Limited data on the temperature history dependence of bolt strength is provided by Hanus et al. (2011). The temperature history dependence of bolt strength can be important when evaluating connection strength during the cooling stage of a fire. An additional important consequence of this behavior is that bolts can suffer a significant permanent loss of strength after being heated in a fire and then cooled to room temperature. This permanent loss of strength can be important when evaluating the condition of a steel structure after a fire. Information on the post-fire properties of high-strength bolts are reported by Yu and Frank (2009).

Appendix 4 does not currently include provisions for computing the elevated temperature strength of welds because of the lack of experimental data on elevated temperature properties of welds made using typical U.S. welding processes, procedures and consumables. However, some guidance on the elevated temperature strength of welds is provided in Eurocode 3 (CEN, 2005b).

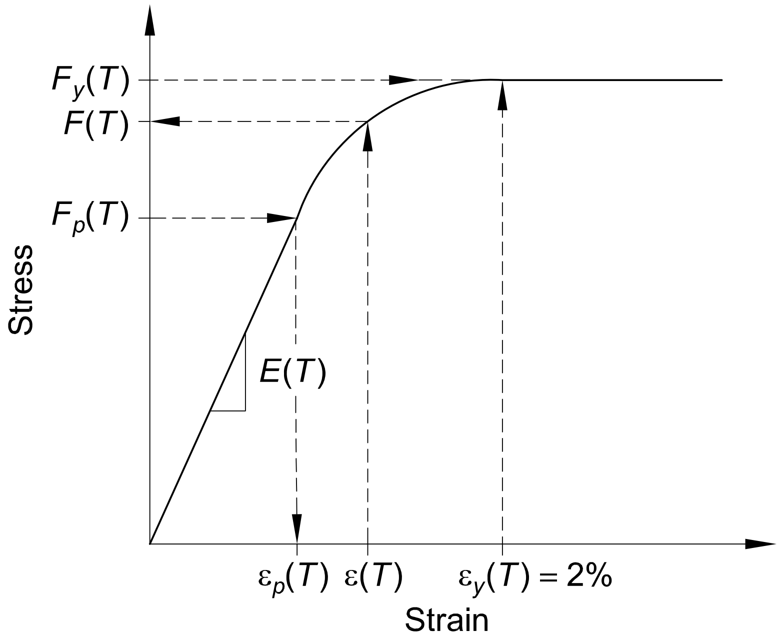


Fig. C-A-4.1. Parameters of idealized stress-strain curve at elevated temperatures (Takagi and Deierlein, 2007).

4. Structural Design Requirements

The resistance of the structural system in the design basis fire may be determined by:

(1) Structural analysis of individual elements where the effects of restraint to thermal expansion and bowing may be ignored but the reduction in strength and stiffness with increasing temperature is incorporated

(2) Structural analysis of assemblies/subframes where the effects of restrained thermal expansion and thermal bowing are considered by incorporating geometric and material nonlinearities

(3) Global structural analysis where restrained thermal expansion, thermal bowing, material degradation, and geometric nonlinearity are considered

4a. General Structural Integrity

The requirement for general structural integrity is consistent with that appearing in Section 1.4 of ASCE (2016). Structural integrity is the ability of the structural system to absorb and contain local damage or failure without developing into a progressive collapse that involves the entire structure or a disproportionately large part of it.

The Commentary C1.4 of ASCE (2016) contains guidelines for the provision of general structural integrity. Compartmentation (subdivision of buildings/stories in a building) is an effective means of achieving resistance to progressive collapse as well as preventing fire spread, as a cellular arrangement of structural components that are well tied together provides stability and integrity to the structural system as well as insulation.

Most typical structural steel connections will comply with the new Chapter B tie-force requirements for structural integrity at ambient conditions without reinforcement or other modifications. The exceptions to this generalization are seated, single-angle, and bolted-welded double-angle (“knife”) connections (Gustafson, 2009). However, these, and other types of simple shear connections, will likely need additional design enhancements for ductility and resistance to the higher tensile forces that may develop during the design basis fire exposure (Agarwal et al., 2014b; Fischer and Varma, 2015; Selden et al., 2016). A fire exposure will not only affect the magnitude of member end reactions, but may also change the nature of the reaction to a limit state different from the controlling mode at ambient.

4b. Strength Requirements and Deformation Limits

As structural elements are heated, their expansion is restrained by adjacent elements and connections. Material properties degrade with increasing temperature. Load transfer can occur from hotter elements to adjacent cooler elements. Excessive deformation may be of benefit in a fire as it allows release of thermally induced stresses. Deformation is acceptable once horizontal and vertical separation, as well as the overall load bearing capacity of the structural system, is maintained.

Membrane action of concrete floor slabs supported by steel beams has received growing international research attention over the last 15 years. Beginning with the landmark Cardington fire tests conducted during the mid-1990s in the United Kingdom (Newman, 1999), this high-temperature strength mechanism has been identified, better understood and developed as a fire resistant design alternative for steel beam and concrete floor slab systems. The novel advantage of this membrane action design is that it permits the secondary (infill) steel floor beams to be left unprotected, since they are designed for strength and stiffness primarily at ambient conditions. The tradeoffs are that the concrete slab, all the fire protected perimeter girders of the floor bays, and their end connections must have adequate strength to bridge over an entire floor bay and the severely thermally weakened infill beams such that an adequate load path is maintained to transmit the gravity design loads of the floor bay. Agarwal and Varma (2014) and Agarwal et al. (2014b) have demonstrated that the presence of steel reinforcement (greater than the minimum shrinkage reinforcement) in the concrete slabs, and fire protection of the single-plate connections facilitates the redistribution of gravity loading through membrane action and reduces the risk of progressive collapse of the structure. Bailey (2004) provides further background and the design criteria for how to effectively mobilize membrane action at large vertical deflections. There have been numerous other published papers on this research advancement, such as Zhao et al. (2008); Huang et al. (2004); and Bednar et al. (2013).

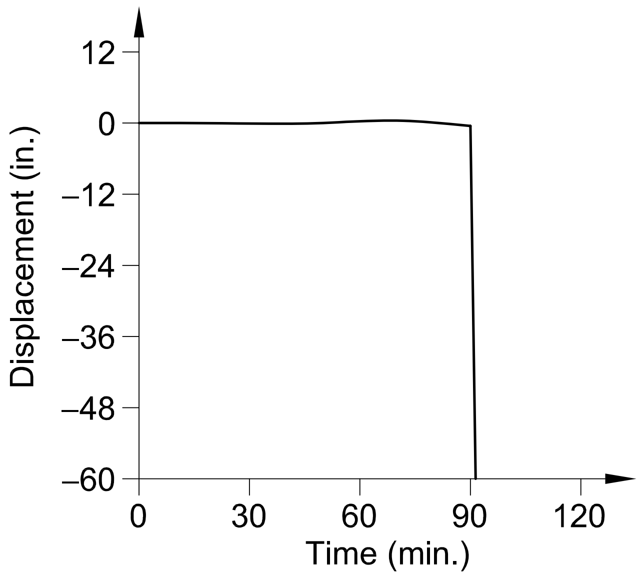
4c. Design by Advanced Methods of Analysis

Advanced methods are required when the overall structural system response to fire, the interaction between structural members and separating elements in fire, or the residual strength of the structural system following a fire must be considered. Advanced analysis should explicitly account for the deterioration in strength and stiffness with increasing temperature, the effects of thermal expansions, large deformations, time-dependent effects such as creep, and uncertainties resulting from variability in material properties at elevated temperature. Boundary conditions and connection fixity must represent the proposed structural design. The models for advanced analysis models should account for all potential limit states, such as excessive deflections, connection ruptures, and overall or local buckling. For example, Agarwal and Varma (2014) and Agarwal et al. (2014b) conducted advanced analysis of 3D building structures while accounting for all potential limit states, namely, inelastic column buckling, composite slab cracking, yielding of the steel floor beams and reinforcement in the slabs, and deformation and fracture of the various shear connections. They used the Eurocode stress-strain-temperature relationships to account for the deterioration in strength and stiffness with increasing temperature. Sample results from one of their advanced analyses are shown in Figure C-A-4.2.

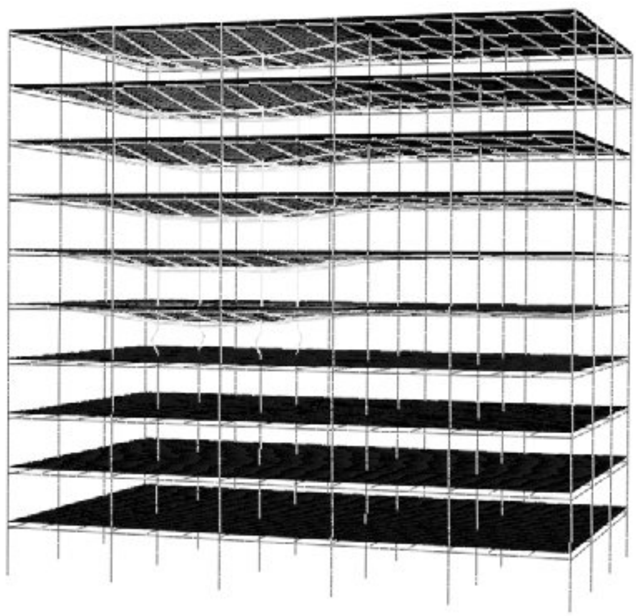
4d. Design by Simple Methods of Analysis

Simple methods may suffice when a structural member or component can be assumed to be subjected to uniform heat flux on all sides and the assumption of a uniform temperature is reasonable as, for example, in a free-standing column surrounded by fire.

Takagi and Deierlein (2007) have shown that the standard strength equations of this Specification (at ambient temperature), with steel properties (E, F_y and F_u) reduced for elevated temperatures, can overestimate considerably the strengths of members that are sensitive to stability effects. Special high temperature equations developed by Takagi and Deierlein (2007) more accurately represent the strength of compression members subjected to flexural buckling and flexural members subjected to lateral-torsional buckling. As shown in Figure C-A-4.3, these equations,



(a) Interior gravity column failure displacement history



(b) Failure mode

Fig. C-A-4.2. Advanced analysis of 3D building for design fire.

first introduced in the 2010 AISC *Specification* (AISC, 2010) and unchanged in this Specification, are much more accurate in comparison to equations from the ECCS (2001) and to detailed finite element method analyses (represented by the square symbol in the figure), which have been validated against test data.

The stability of steel structures under fire loading is governed by the fire resistance of gravity columns because they are most likely to reach critical temperatures and structural failure due to high utilization ratios (Agarwal and Varma, 2011, 2014). The fire resistance of gravity columns may be improved due to the rotational restraints offered by cooler columns in the stories above and below. The increase in design strength can be accounted for by reducing the column slenderness (L_c/r) used to calculate $F_e(T)$ in Equation A-4-2 to $(L_c/r)_T$ as follows:

$$\left(\frac{L_c}{r}\right)_T = \left(1 - \frac{T-32}{n(3{,}600)}\right)\left(\frac{L_c}{r}\right) - \frac{35}{n(3{,}600)}(T-32) \geq 0 \qquad \text{(C-A-4-9)}$$

where

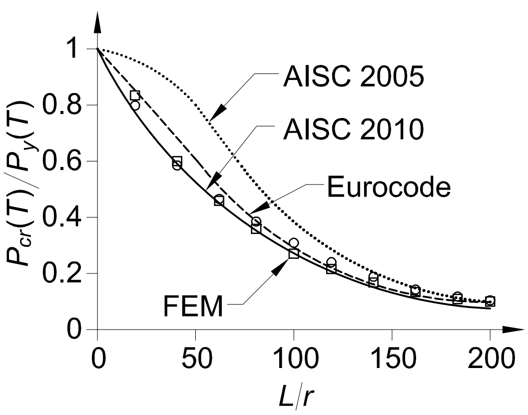
T = steel temperature,°F (°C)

n = 1 for columns with cooler columns both above and below

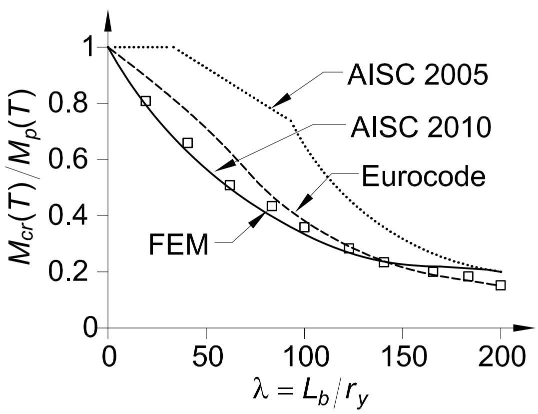
n = 2 for columns with cooler columns either above or below only

Figure C-A-4.4 shows this reduction in $(L_c/r)_T$ with increasing temperature for columns with rotational restraints at both ends and one end only.

Compression members subject to uniform heating have greater heat flux from all sides than members subjected to nonuniform heating. As a result, compression members subjected to uniform heating reach their failure temperatures much earlier than members subjected to nonuniform heating. Uniform heating will be the governing case for most fire scenarios (Agarwal et al., 2014a) in terms of time to failure.



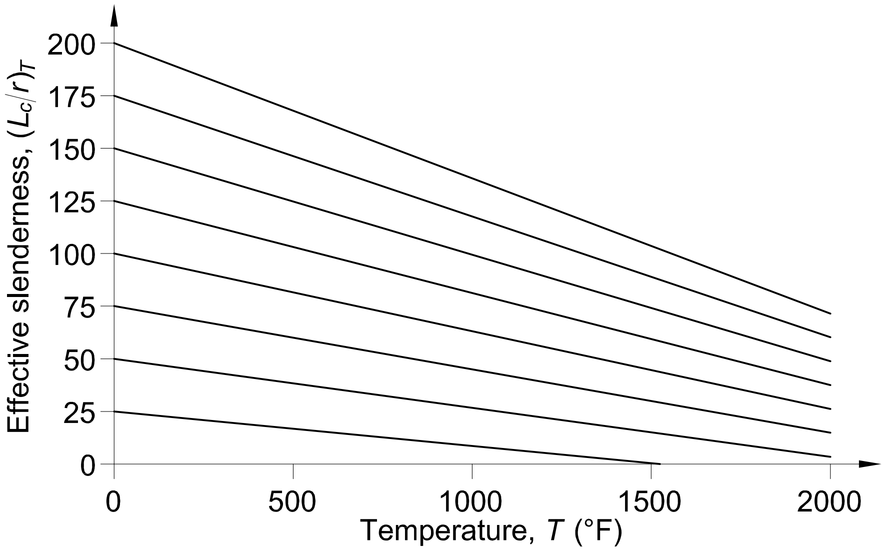
(a) Compression strength



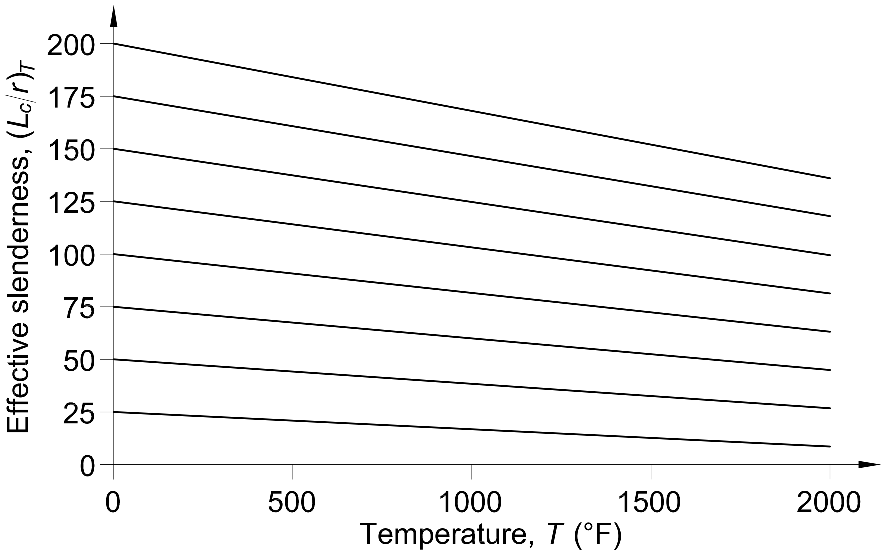
(b) Flexural strength

Fig. C-A-4.3. Comparison of compressive and flexural strengths at 500°C (930°F) (Takagi and Deierlein, 2007).

Thermal gradients due to nonuniform heating reduce the axial load capacity of compression members due to elevated temperatures, bowing deformations resulting from uneven thermal expansion, and asymmetry in the column cross section resulting from uneven degradation of material properties (yield stress and elastic modulus). Several researchers have discussed these effects and proposed alternate design methods for columns with thermal gradients. Agarwal et al. (2014a) and Choe et al. (2016)



(a) Rotational restraint at both ends



(b) Rotational restraint at one end

Fig. C-A-4.4. Effects of rotational restraints on column slenderness as a function of elevated temperature (from Equation C-A-4-9).

conducted experimental and numerical studies to develop and verify design equations for compression members with thermal gradients. The parameters included in the study were member length, cross section, and axial loading magnitude. Three different heating scenarios were considered: uniform heating, thermal gradient along the flanges, and thermal gradient along the web. The studies indicated that columns subjected to uniform heating have much greater heat influx, and therefore reach higher average temperatures faster than columns exposed to nonuniform heating. In most cases, uniformly heated columns reached their failure temperature earlier than nonuniform heated columns with thermal gradients. Exceptions were slender columns with very high axial compression (more than 50% of ambient capacity). The design strength of such columns can be calculated using equations presented by Agarwal et al. (2014a). These equations quantify the effects of elevated temperature, bowing, and cross-section asymmetry mentioned earlier. They were verified using the results of large-scale tests and numerical parametric studies.

The design strength for structural steel members and connections is calculated as ϕR_n, in which R_n = nominal strength when the deterioration in strength at elevated temperature is taken into account, and ϕ is the resistance factor. The nominal strength is determined from Chapters C through K and Appendix 4, using material strength and stiffnesses at elevated temperatures defined in Tables A-4.2.1, A-4.2.2 and A-4.2.3. For limit states governed by steel yielding or fracture, the ambient equations for nominal strength are used with elevated temperature material properties from Appendix 4, Section 4.2.3, and the corresponding Tables. For limit states governed by buckling or instability, equations for nominal strength are provided in this section. For example, nominal strength equations are provided for design for compression and for flexure governed by lateral-torsional buckling.

While ECCS (2001) and Eurocode 1 (CEN, 1991) specify partial material factors as equal to 1.0 for "accidental" limit states, the uncertainties in strength at elevated temperatures are substantial and in some cases are unknown. Research is continuing on this topic. In the interim, ambient resistance factors should be used when determining design strength.

For composite beams, Selden and Varma (2016) developed and benchmarked numerical models to determine their flexural strength at elevated temperatures, $M_n(T)$, while considering the distribution of temperatures over the depth of the composite section, the degree or percent composite action in the section, member length, and the effects of elevated temperature on the material stiffness and strength of the steel beam, concrete slab, steel reinforcement (if any), and the shear force-slip behavior of the stud anchors. The results of comprehensive parametric analyses conducted by Selden (2014) were used to develop Equation A-4-11 and the retention factor Table A-4-2.4.

4.3. DESIGN BY QUALIFICATION TESTING

1. Qualification Standards

Qualification testing is an acceptable alternative to design by analysis for providing fire resistance. Fire resistance ratings of building elements are generally determined in accordance with procedures set forth in ASTM E119, *Standard Test Methods for Fire Tests of Building Construction and Materials* (ASTM, 2009b). Tested building element designs, with their respective fire resistance ratings, may be found in special directories and reports published by testing agencies. Additionally, calculation procedures based on standard test results may be used as specified in *Standard Calculation Methods for Structural Fire Protection* (ASCE, 2005).

For building elements that are required to prevent the spread of fire, such as walls, floors and roofs, the test standard provides for measurement of the transmission of heat. For loadbearing building elements, such as columns, beams, floors, roofs and loadbearing walls, the test standard also provides for measurement of the load-carrying ability under the standard fire exposure.

For beam, floor and roof specimens tested under ASTM E119, two fire resistance classifications—restrained and unrestrained—may be determined, depending on the conditions of restraint and the acceptance criteria applied to the specimen.

2. Restrained Construction

The ASTM E119 standard provides for tests of loaded beam specimens only in the restrained condition, where the two ends of the beam specimen (including slab ends for composite steel-concrete beam specimens) are placed tightly against the test frame that supports the beam specimen. Therefore, during fire exposure, the thermal expansion and rotation of the beam specimen ends are resisted by the test frame. A similar restrained condition is provided in the ASTM E119 tests on restrained loaded floor or roof assemblies, where the entire perimeter of the assembly is placed tightly against the test frame.

The practice of restrained specimens dates back to the early fire tests (over 100 years ago), and it is predominant today in the qualification of structural steel framed and reinforced concrete floors, roofs and beams in North America. While the current ASTM E119 standard does provide for an option to test loaded floor and roof assemblies in the unrestrained condition, this testing option is rarely used for structural steel and concrete. However, unrestrained loaded floor and roof specimens, with sufficient space around the perimeter to allow for free thermal expansion and rotation, are common in the tests of wood and cold-formed-steel framed assemblies.

Gewain and Troup (2001) provide a detailed review of the background research and practices in the qualification fire resistance testing and rating of structural steel (and composite steel/concrete) girders, beams, and steel framed floors and roofs. The restrained assembly fire resistance ratings (developed from tests on loaded restrained floor or roof specimens) and the restrained beam fire resistance ratings (developed from tests on loaded restrained beam specimens) are commonly applicable to all types (with minor exceptions) of steel-framed floors, roofs, girders and beams, as recommended in Table X3.1 of ASTM E119, especially where they incorporate or support cast-in-place or prefabricated concrete slabs. AISC Design Guide 19, *Fire Resistance of Structural Steel Framing* (Ruddy et al., 2003), provides several detailed examples of steel-framed floor and roof designs by qualification testing.

3. Unrestrained Construction

An unrestrained condition is one in which thermal expansion at the support of load-carrying elements is not resisted by forces external to the element, and the supported ends are free to expand and rotate.

However, in the common practice for structural steel (and composite steel-concrete) beams and girders, the unrestrained beam ratings are developed from ASTM E119 tests on loaded restrained beam specimens or from ASTM E119 tests on loaded restrained floor or roof specimens, based only on temperature measurements on the surface of structural steel members. For steel-framed floors and roofs, the unrestrained assembly ratings are developed from ASTM E119 tests on loaded restrained floor and roof specimens, based only on temperature measurements on the surface of the steel deck (if any) and on the surface of structural steel members. As such, the unrestrained fire resistance ratings are temperature-based ratings indicative of the time when the steel reaches specified temperature limits. These unrestrained ratings do not bear much direct relevance to the unrestrained condition or the load-bearing functions of the specimens in fire tests.

Nevertheless, unrestrained ratings provide useful supplementary information and they are used as a conservative estimate of fire resistance (in lieu of the restrained ratings) in cases where the surrounding or supporting construction cannot be expected to accommodate the thermal expansion of steel beams or girders. For instance, as recommended in ASTM E119 Table X3.1, a steel member bearing on a wall in a single span, or at the end span of multiple spans, should be considered unrestrained when the wall has not been designed and detailed to resist thermal thrust.

ADDITIONAL BIBLIOGRAPHY

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APPENDIX 5

EVALUATION OF EXISTING STRUCTURES

5.1. GENERAL PROVISIONS

The load combinations referred to in this chapter pertain to static loading because it is the most prevalent condition encountered. If other loading conditions are a consideration, the appropriate load combination from ASCE/SEI 7 (ASCE, 2016) or from the applicable building code should be used.

For seismic evaluation of existing buildings, ASCE/SEI 31 (ASCE, 2003) provides a three-tiered process for determination of the design and construction adequacy of existing buildings to resist earthquakes. The standard defines evaluation requirements as well as detailed evaluation procedures. Buildings may be evaluated in accordance with this standard for life safety or immediate occupancy performance levels. Where seismic rehabilitation of existing structural steel buildings is required, engineering of seismic rehabilitation work may be performed in accordance with the ASCE/SEI 41 (ASCE, 2013) standard or other standards. Use of these two standards for seismic evaluation and seismic rehabilitation of existing structural steel buildings is subject to the approval of the authority having jurisdiction.

5.2. MATERIAL PROPERTIES

1. Determination of Required Tests

The extent of tests required depends on the nature of the project, the criticality of the structural system or member evaluated, and the availability of records pertinent to the project. Thus, the engineer of record has the responsibility to determine the specific tests required and the locations from which specimens are to be obtained.

2. Tensile Properties

Samples required for tensile tests should be removed from regions of reduced stress, such as at flange tips at beam ends and external plate edges, to minimize the effects of the reduced area. The number of tests required will depend on whether they are conducted to merely confirm the strength of a known material or to establish the strength of some other material.

It should be recognized that the yield stress determined by standard ASTM methods and reported by mills and testing laboratories is somewhat greater than the static yield stress because of dynamic effects of testing. Also, the test specimen location may have an effect. These effects have already been accounted for in the nominal strength equations in the Specification. However, when strength evaluation is done by load testing, this effect should be accounted for in test planning because yielding will tend to occur earlier than otherwise anticipated. The static yield stress, F_{ys}, can be estimated from that determined by routine application of ASTM methods, F_y, by the following equation (Ziemian, 2010):

$$F_{ys} = R\,(F_y - 4) \qquad \text{(C-A-5-1)}$$

$$F_{ys} = R\,(F_y - 27) \qquad \text{(C-A-5-1M)}$$

where

F_y = reported yield stress, ksi (MPa)

F_{ys} = static yield stress, ksi (MPa)

R = 0.95 for tests taken from web specimens
= 1.00 for tests taken from flange specimens

The R factor in Equation C-A-5-1 accounts for the effect of the coupon location on the reported yield stress. Prior to 1997, certified material test reports for structural shapes were based on specimens removed from the web, in accordance with ASTM A6/A6M. Subsequently, the specified coupon location was changed to the flange.

4. Base Metal Notch Toughness

The engineer of record should specify the location of samples. Samples should be cored, flame cut or saw cut. The distance from the edge of flat tension specimens (generally, specimens ½ in. (13 mm) thick or less) need to be made only large enough to obtain the grip width. The distance from the center of a cylindrical tension specimen to either of the thermal cut edges should be one inch (25 mm) or larger. The engineer of record will determine if remedial actions are required, such as the possible use of bolted splice plates.

5. Weld Metal

Because connections typically have a greater reliability index than structural members (see Commentary Section B3.1), strength testing of weld metal is not usually necessary. However, field investigations have sometimes indicated that complete-joint-penetration groove welds, such as at beam-to-column connections, were not made in accordance with AWS D1.1/D1.1M (AWS, 2015). The specified provisions in AWS D1.1/D1.1M provide a means for judging the quality of such a weld. Where feasible, any samples removed should be obtained from compression splices rather than tension splices, because the effects of repairs to restore the sampled area are less critical.

6. Bolts and Rivets

Because connections typically have a greater reliability index than structural members (see Commentary Section B3.1), removal and strength testing of fasteners is not usually necessary. However, strength testing of bolts is required where they cannot be properly identified otherwise. Because removal and testing of rivets is difficult, assuming the lowest rivet strength grade simplifies the investigation. Rivet strength can often be determined by referring to Section 1.3 of AISC Design Guide 15, *AISC Rehabilitation and Retrofit Guide, A Reference for Historic Shapes and Specifications* (Brockenbrough, 2002.)

5.3. EVALUATION BY STRUCTURAL ANALYSIS

2. Strength Evaluation

Resistance and safety factors reflect variations in determining strength of members and connections, such as uncertainty in theory and variations in material properties and dimensions. If an investigation of an existing structure indicates that there are variations in material properties or dimensions significantly greater than those anticipated in new construction, the engineer of record should consider the use of more conservative values.

5.4. EVALUATION BY LOAD TESTS

1. Determination of Load Rating by Testing

Generally, structures that can be designed according to the provisions of this Specification need no confirmation of calculated results by testing. However, special situations may arise when it is desirable to confirm by tests the results of calculations. Minimal test procedures are provided to determine the live load rating of a structure. The live load rating established by testing presumes $\phi = 1.0$ for all failure modes.

It is essential that the engineer of record take all necessary precautions to ascertain that the structure does not fail catastrophically during testing. A careful assessment of structural conditions before testing is a fundamental requirement. This includes accurate measurement and characterization of the size and strength of members, connections and details. All safety regulations of OSHA and other pertinent bodies must be strictly followed. Shoring and scaffolding should be used as required in the proximity of the test area to mitigate against unexpected circumstances. Deformations must be carefully monitored and structural conditions must be continually evaluated. In some cases, it may be desirable to monitor strains as well.

The engineer of record must use judgment to determine when deflections are becoming excessive and terminate the tests at a safe level even if the desired loading has not been achieved. Incremental loading is specified so that deformations can be accurately monitored and the performance of the structure carefully observed. Load increments should be small enough initially so that the onset of significant yielding can be determined. The increment can be reduced as the level of inelastic behavior increases, and the behavior at this level carefully evaluated to determine when to safely terminate the test. Periodic unloading, after the onset of inelastic behavior, will help the engineer of record determine when to terminate the test to avoid excessive permanent deformation or catastrophic failure.

It must be recognized that the margin of safety at the maximum load level used in the test may be very small, depending on such factors as the original design, the purpose of the tests, and the condition of the structure. Thus, it is imperative that all appropriate safety measures be adopted. It is recommended that the maximum live load used for load tests be selected conservatively. It should be noted that experience in testing more than one bay of a structure is limited.

The provision limiting increases in deformations for a period of one hour is given so as to have positive means to confirm that the structure is stable at the loads evaluated.

2. Serviceability Evaluation

In certain cases, serviceability performance must be determined by load testing. It should be recognized that complete recovery (in other words, return to the pre-tested deflected shape) after removal of maximum load is unlikely because of phenomena such as local yielding, slip at the slab interface in composite construction, creep in concrete slabs, localized crushing or deformation at shear connections in slabs, slip in bolted connections, and effects of continuity. Because most structures exhibit some slack when load is first applied, it is appropriate to project the load-deformation curve back to zero load to determine the slack and exclude it from the recorded deformations. Where desirable, the applied load sequence can be repeated to demonstrate that the structure is essentially elastic under service loads and that the permanent set is not detrimental.

5.5. EVALUATION REPORT

Extensive evaluation and load testing of existing structures is often performed when appropriate documentation no longer exists or when there is considerable disagreement about the condition of a structure. The resulting evaluation is only effective if well documented, particularly when load testing is involved. Furthermore, as time passes, various interpretations of the results can arise unless all parameters of the structural performance, including material properties, strength and stiffness, are well documented.

APPENDIX 6

MEMBER STABILITY BRACING

This Commentary provides background to the development of the Appendix 6 equations and explains their application in the design for bracing of beams, columns and beam-columns.

In the design of bracing for trusses, the compression chord may be treated as the compression flange of a beam. Further discussion of specific bracing applications for trusses and other systems can be found in this Commentary.

6.1. GENERAL PROVISIONS

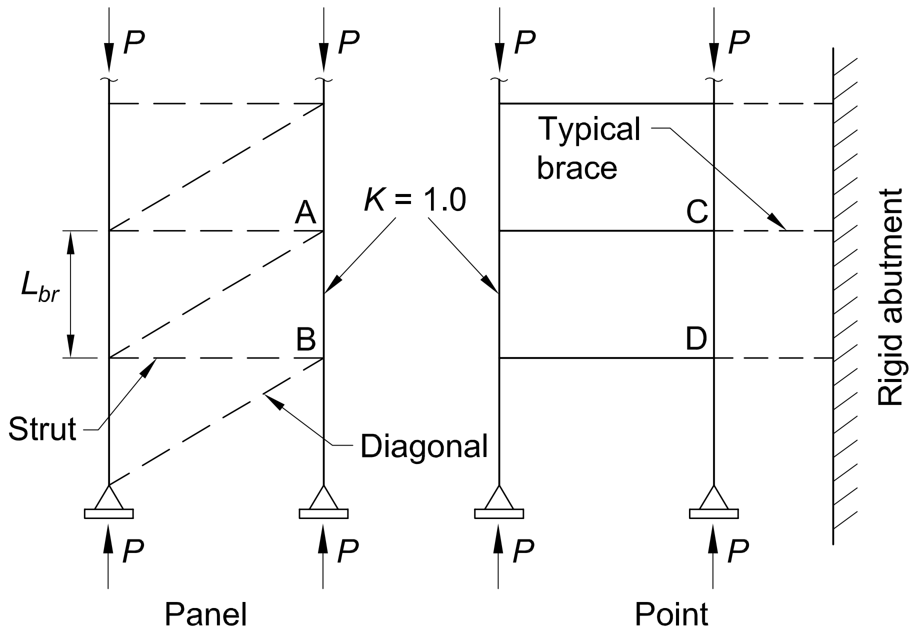
Winter (1958, 1960) developed the concept of a dual requirement for bracing design, which involves criteria for both strength and stiffness. Additional discussions are provided by Ziemian (2010). The design requirements of Appendix 6 are based upon this approach and consider two general types of bracing systems, panel and point, as shown in Figure C-A-6.1. In past editions of the Specification, the term relative bracing was used for panel bracing and nodal bracing was used for point bracing. The name change was made for clarity.

A panel-bracing system for a column is attached to two locations along the column length. The distance between these locations is the unbraced length, L_{br}, of the column. The panel bracing system shown in Figure C-A-6.1(a) consists of the diagonals and struts that control the movement at one end of the unbraced length, point A, with respect to the other end of the unbraced length, point B. The forces in these bracing elements are resolved by forces in the beams and columns in the frame that is braced. The diagonal and strut both contribute to the strength and stiffness of the panel-bracing system. However, when the strut is a floor beam and the diagonal a brace, the floor beam stiffness is usually large compared to the stiffness of the brace. In such a case, the brace strength and stiffness often controls the strength and stiffness of the panel-bracing system.

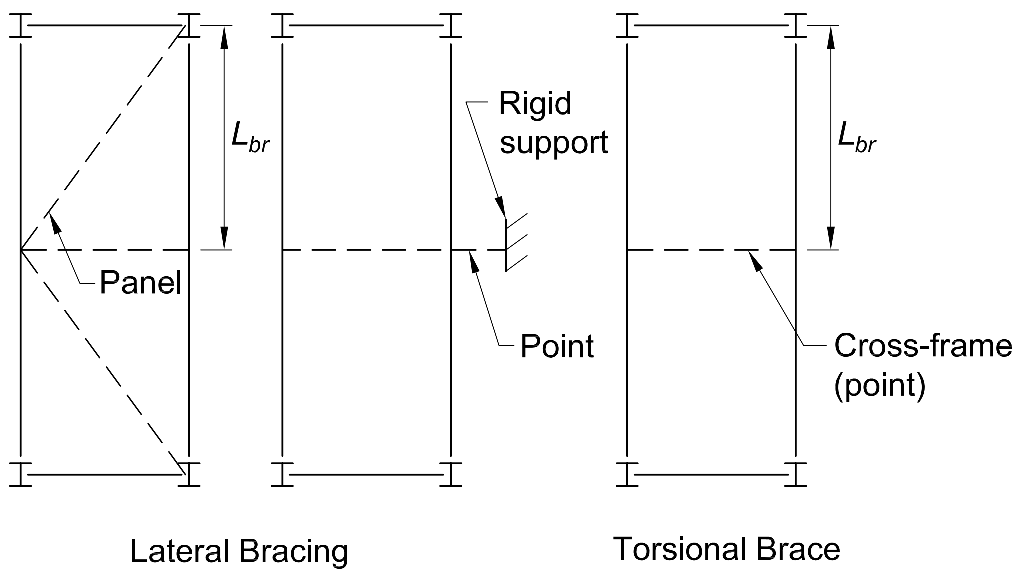
A point brace for a column controls movement only at the point it braces, and without direct interaction with adjacent braced points. The distance between adjacent braced points is the unbraced length, L_{br}, of the column. The point-bracing system shown in Figure C-A-6.1(a) consists of a series of independent braces, which connect to a rigid abutment from the braced points including point C and point D. The forces in these bracing elements are resolved by other structural elements not part of the frame that is braced.

As illustrated in Figure C-A-6.1(b), a panel-bracing system for a beam often consists of a system with diagonals; a point bracing system commonly exists when there is a link to an external support (such as another lateral brace) or a cross-frame torsional brace between two adjacent beams. The cross-frame prevents twist (not lateral displacement) of the beams at the particular cross-frame location. With the required lateral and rotational restraint provided at the beam ends, the unbraced length, L_{br}, in all of these cases is the distance from the support to the braced point.

The bracing requirements stipulated in Sections 6.2 and 6.3 allow for a member to develop a maximum load based on effective lengths, L_c and L_b, taken equal to the unbraced lengths between the brace points. The bracing requirements in Sections 6.2 and 6.3 generally are not sufficient to permit the development of member strengths based on L_c and/or L_b smaller than L_{br}; that is, the development of column or beam



(a) Column bracing



(b) Beam bracing

Fig. C-A-6.1. Types of bracing.

strengths based on a corresponding effective length factor of $K < 1$. Figure C-A-6.2 shows the critical buckling load versus the brace stiffness for an elastic cantilevered column with a brace of variable stiffness at its top. The ideal bracing stiffness for this column associated with $L_c = L_{br} = L$; that is, the bracing stiffness necessary to develop a column critical buckling load of $P_{cr} = P_e = \pi^2 EI/L_{br}^2$, is P_e/L_{br}. A brace having five times this stiffness is required for the column to reach a critical load of 95% of $P_{cr} = \pi^2 EI/(0.7L_{br})^2$ based on $L_c = 0.7L_{br}$. An infinitely stiff brace is required theoretically to reach $P_{cr} = \pi^2 EI/(0.7L_{br})^2$.

In addition, the determination of bracing required to reach specified rotation capacities or ductility limits is beyond the scope of the Appendix 6 provisions.

The provisions in Sections 6.2 and 6.3 for columns and beams, respectively, stipulate a required brace stiffness, β_{br}, equal to $2/\phi$ (LRFD) and 2Ω (ASD) times the ideal bracing stiffness, where $\phi = 0.75$ and $\Omega = 2.00$. The required brace strength, P_{br}, is a function of the initial out-of-alignment of the brace points (out-of-plumbness in the case of vertical columns), Δ_o, and the brace stiffness, β. The brace strength requirements are based on the nominal brace stiffness without the inclusion of ϕ and Ω. Separate resistance factors and safety factors are applied in the design of the bracing system components to resist these forces.

For a panel lateral bracing system on a column, the relationship between column load, brace stiffness and sway displacement is shown in Figure C-A-6.3. If the bracing stiffness, β, is equal to the ideal brace stiffness for a perfectly plumb member, β_i, the displacement of the bracing system becomes large as P approaches P_e. Such large displacements would produce large bracing forces, and Δ must be kept small for practical design.

For the panel-bracing system shown in Figure C-A-6.3, the use of $\beta_{br} = 2\beta_i$ and the assumption of an initial displacement of $\Delta_o = L_{br}/500$ results in V_{br} equal to 0.4% of P_e. In the foregoing, L_{br} is the distance between adjacent braced points as shown in

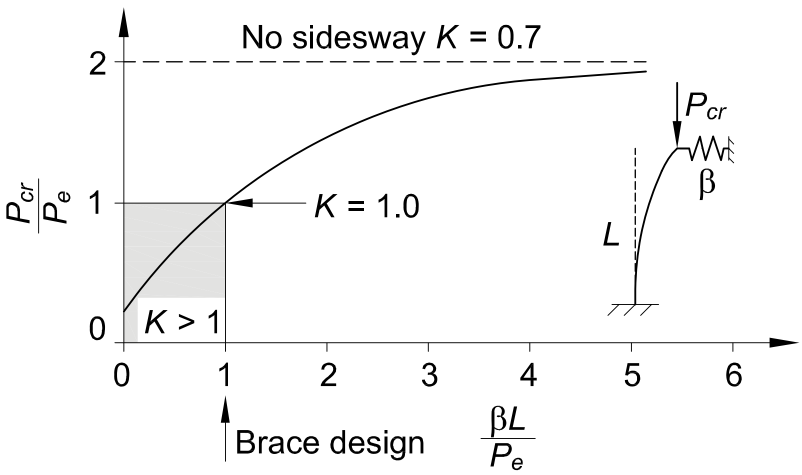


Fig. C-A-6.2. Cantilevered column with a variable stiffness brace at its top.

Figure C-A-6.4, and Δ_o is the relative lateral displacement of the braced points from the plumb (or aligned) position caused by erection tolerances, first-order effects from gravity and/or lateral loading on the structure, and first-order effects (i.e., the effects prior to amplification from member axial compression) from any other sources such as temperature movement, connection slip, etc.

As discussed in the user note in Chapter C, $\Delta_o = L_{br}/500$ corresponds to an erection tolerance equal to maximum frame out-of-plumbness specified in the AISC *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2016a). Similarly, for torsional bracing of beams an initial rotation, $\theta_o = L_{br}/(500h_o)$, is assumed, where h_o is the distance between flange centroids. For other values of Δ_o and θ_o, it is permissible to modify the bracing required strengths, V_{br}, P_{br} and M_{br}, by direct proportion.

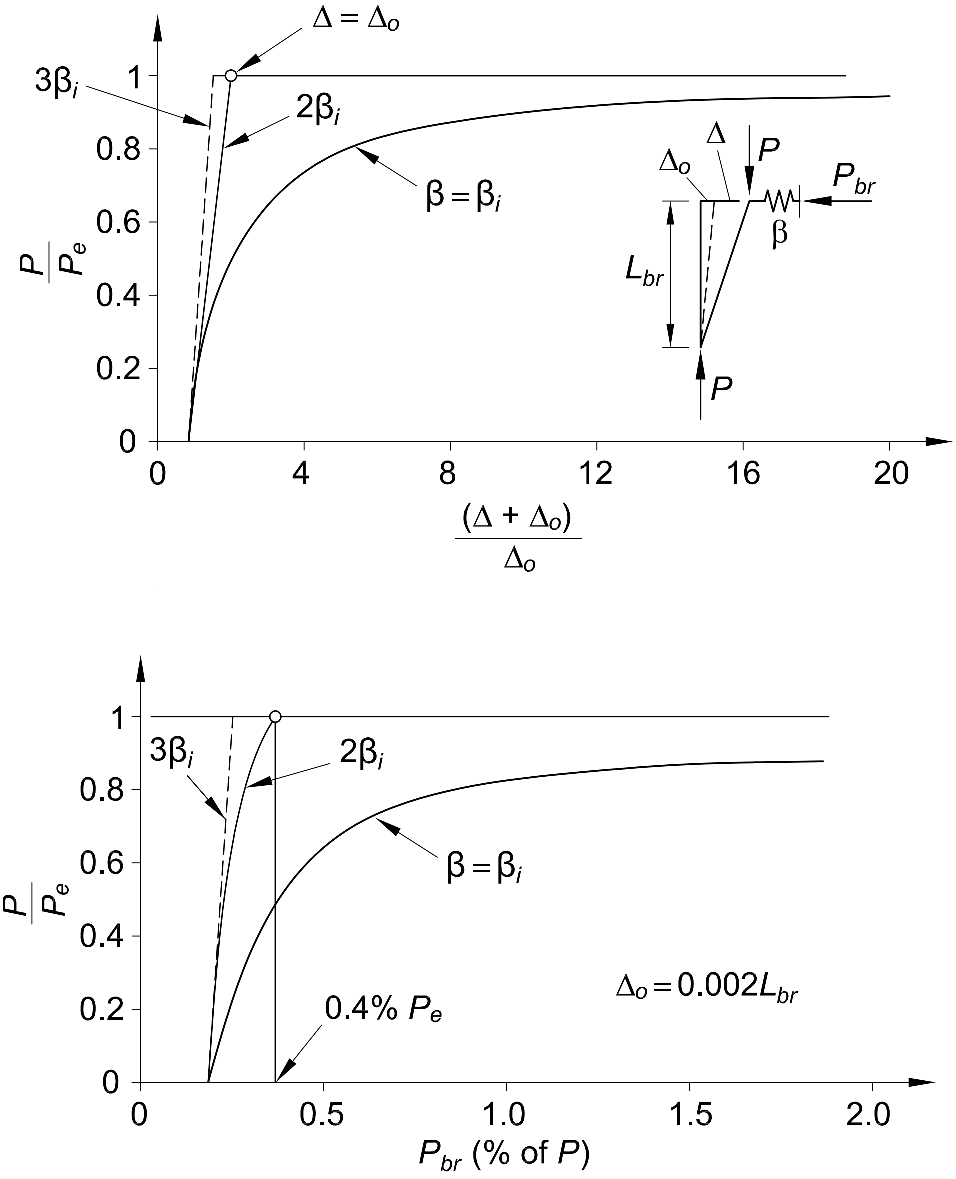


Fig. C-A-6.3. Effect of initial out-of-plumbness.

For cases where multiple columns are being braced and it is unlikely that all of the columns will be out-of-plumb in the same direction, Chen and Tong (1994) recommend the use of an average initial displacement due to erection tolerances of $\Delta_o = L_{br}/\left(500\sqrt{n_o}\right)$, where n_o is the number of columns, each with a random Δ_o, stabilized by the bracing system. This reduced Δ_o is added with the first-order effects causing any additional out-of-plumbness or out-of-alignment between the brace points to determine the total force in the bracing system. In this situation, assuming a panel-bracing system, the total shear force in the bracing system can be calculated as

$$V_{br} = V_{1st} + 2\,\frac{P_r}{L_{br}}\,\Delta_{o.total} \qquad \text{(C-A-6-1)}$$

where

L_{br} = unbraced length within the panel under consideration, in. (mm)

P_r = sum of the required axial forces in the columns being stabilized, kips (N)

V_{1st} = first-order shear force in the bracing system due to gravity and/or lateral loading on the structure, temperature effects, etc., kips (N)

$\Delta_{o.total}$ = total relative displacement between the ends of the unbraced length under consideration due to erection tolerances, first-order effects of gravity and/or lateral loads on the structure, and first-order effects (i.e., the effects prior to amplification from member axial compression) from any other sources such as temperature movement, connection slip, etc., in. (mm)

In the absence of any first-order forces in the bracing system, if the actual bracing stiffness provided (nominal stiffness with no stiffness reduction), β_{act}, is larger than β_{br}, the required brace strength, V_{br}, in the case of a panel lateral brace, or P_{br} in the case of a point lateral brace, can be multiplied by the following factor:

$$\frac{1}{2-\dfrac{\beta_{br}}{\beta_{act}}} \qquad \text{(C-A-6-2)}$$

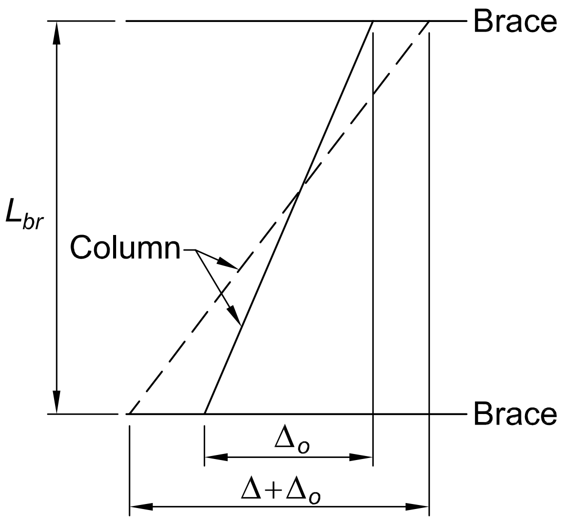


Fig. C-A-6.4. Definitions of initial displacements for panel and point braces.

In the case of a panel-bracing system that contains a first-order shear force, this factor can be applied to the second term of Equation C-A-6-1, giving

$$V_{br} = V_{1st} + \frac{1}{1 - \dfrac{\beta_{br}}{2\beta_{act}}} \frac{P_r}{L_{br}} \Delta_{o.total} \qquad \text{(C-A-6-3)}$$

By substituting the expression for β_{br} from Equation A-6-2, one can show that Equation C-A-6-3 states that the total shear force in the panel-bracing system is simply equal to the first-order shear force plus a P-Δ effect from the total vertical load being stabilized, P_r, acting through the second-order relative end displacement of the panel (Griffis and White, 2013). The second term in the previously given equations for V_{br} is based on the assumption of pins inserted in the column at each of the braced points, as in Winter's point bracing model (Winter, 1960). To account for the additional brace forces due to member curvature and member continuity across the braced points, the brace force, as defined by Equation A-6-1, is increased for point bracing as explained later in Commentary Appendix 6, Section 6.2.

Prado and White (2015) and Lokhande and White (2015) observed that Equation C-A-6-2 tends to overestimate the reduction in the torsional bracing brace strength requirement with increasing β_{act}/β_{br}. Therefore, this equation is not recommended for application with torsional bracing.

Connections in the bracing system, if they are flexible or can slip, should be considered in the assessment of the bracing requirements. The connection and the bracing system should be handled as components in series for the calculation of the bracing stiffness. As such, for point bracing, the actual bracing stiffness is related to the connection and the brace stiffnesses by the relationship

$$\frac{1}{\beta_{act}} = \frac{1}{\beta_{conn}} + \frac{1}{\beta_{brace}} \qquad \text{(C-A-6-4)}$$

The resulting bracing system stiffness, β_{act}, is less than the smaller of the connection stiffness, β_{conn}, and the brace stiffness, β_{brace}. Connection slip may be considered by increasing the value of Δ_o used in the calculation of the bracing force requirements, as long as Δ_o is small enough such that the brace is engaged well before the member reaches its maximum strength. Slip in connections with standard holes need not be considered, except when only a few bolts are used. This is in addition to the consideration of the fact that the initial Δ_o or θ_o is unlikely to be the same in each of the members, via recommendations such as those by Chen and Tong (1994) discussed previously.

When evaluating the bracing of rows of columns or beams, consideration must be given to the accumulation of the bracing forces, which may result in a different displacement at each column or beam location. In general, bracing forces can be minimized by increasing the number of braced bays and using stiff braces.

In certain cases, it may be more effective to obtain the bracing stiffness requirements as $2/\phi$ (LRFD) or 2Ω (ASD) times the ideal bracing stiffness determined from a computational buckling analysis. Although this approach can be applied to any column, beam and beam-column, specific cases of interest include members with brace spacings that vary significantly along the member length, members with stepped and/or tapered geometry, situations where it is desired to increase the bracing stiffness and/or strength to satisfy high demands in one portion of a member but use lighter bracing in other regions, and partially braced members. The buckling analysis should account for the reduction in stiffness associated with the member elastic and inelastic strength limit states. Togay et al. (2015) summarize the stiffness reduction factors corresponding to limit states of Chapter E column buckling and Chapter F I-shaped member lateral-torsional buckling. For design by ASD, the buckling analysis must be carried out under 1.6 times the ASD load combinations and the resulting ideal bracing stiffness values are then multiplied by 2Ω/1.6 to obtain the required bracing stiffnesses.

6.2. COLUMN BRACING

This section addresses lateral bracing of columns. Recommendations for torsional bracing of columns can be found in Helwig and Yura (1999). Commentary Section E4 discusses the calculation of the strength of columns that are restrained laterally at a location other than their shear center, and thus fail by constrained-axis torsional buckling. Lateral bracing requirements for the general case of beam-column members restrained laterally at a location other than their shear center are addressed later in Commentary Appendix 6, Section 6.4.

For point column bracing, the critical stiffness is a function of the number of intermediate braces (Winter, 1958, 1960). For one intermediate brace, $\beta_i = 2P_r / L_{br}$, and for many braces, $\beta_i = 4P_r / L_{br}$. The relationship between the critical stiffness and the number of braces, n, can be approximated (Yura, 1995) as:

$$\beta_i = \left(4 - \frac{2}{n}\right)\frac{P_r}{L_{br}} \tag{C-A-6-5}$$

The most severe case (many braces) is adopted for the brace stiffness requirement in Equation A-6-4, i.e., $\beta_{br} = 2 \times 4P_r / L_{br}$. The brace stiffness in Equation A-6-4 can be multiplied by the following ratio to account for the actual number of braces:

$$\left(\frac{2n-1}{2n}\right)$$

In Equation A-6-4, when the actual brace spacing is less than the value of the effective length, L_c, that enables the column to reach P_r, the calculated required stiffness may become quite conservative since the stiffness equations are inversely proportional to L_{br}. In such cases, L_{br} can be taken equal to L_c. This practice constitutes a simple method of designing for partial bracing; that is, bracing that is sufficient to develop the member's required strength but is not sufficient to develop the member's strength based on $L_c = L_{br}$. This substitution is also permitted for beam point lateral bracing in Equation A-6-8.

For example, a W12×53 (W310×79) with P_u = 400 kips (1800 kN) for LRFD or P_a = 267 kips (1200 kN) for ASD can have a maximum unbraced length of 18 ft

(5.5 m) for ASTM A992/A992M steel. If the actual brace spacing is 8 ft (2.4 m), 18 ft (5.5 m) may be used in Equation A-6-4 to determine the required stiffness. The use of L_{br} equal to the value of L_c in Equation A-6-4 provides reasonable estimates of the brace stiffness requirements; however, in some cases, this solution is significantly conservative. Improved accuracy can be obtained by treating the system as a continuous bracing system or directly determining the buckling strength of the partially braced member and the corresponding ideal bracing stiffness (Lutz and Fisher, 1985; Ziemian, 2010; Togay et al., 2015). The required bracing stiffness is taken as $2/\phi$ (LRFD) or 2Ω (ASD) times the ideal bracing stiffness. (Note that, as discussed in Commentary Appendix 6, Section 6.1, for ASD, the ideal bracing stiffness must be determined using 1.6 times the applicable load combinations and the resulting ideal bracing stiffness values are then multiplied by $2\Omega/1.6$ to obtain the required brace stiffnesses).

With regard to brace strength requirements, Winter's point bracing model only accounts for force effects from lateral displacement of the brace points and would derive a brace force equal to 0.8% of P_r. To account for the additional brace forces due to member curvature and member continuity across the brace points, this theoretical force is increased to 1% of P_r in Equation A-6-3. Member curvature and continuity across the brace points has a comparable effect on panel-bracing requirements. As such, the panel-bracing strength requirement of Equation A-6-1 is increased from 0.4 to 0.5% of P_r. Similar increases are applied to the panel and point lateral bracing strength requirements for beams in Equations A-6-5 and A-6-7.

6.3. BEAM BRACING

Beam bracing must control twist of the section, but need not prevent lateral displacement. Both lateral bracing, such as a steel joists attached to the compression flange of a simply supported beam, and torsional bracing, such as a cross-frame or vertical diaphragm element between adjacent girders, can be used to control twist. Note, however, that lateral bracing systems that are attached only near the beam shear center are generally ineffective in controlling twist.

For beams subject to reverse-curvature bending, an unbraced inflection point cannot be considered a braced point because significant twist can occur at that point (Ziemian, 2010). Bracing provided near an inflection point must be attached at or near both flanges to prevent twist; alternatively, torsional bracing can be provided. A lateral brace on one flange is ineffective near an inflection point.

The beam bracing requirements of this section are based predominantly on the recommendations from Yura (2001).

1. Lateral Bracing

For beam lateral bracing, the following stiffness requirement is derived following Winter's approach:

$$\beta_{br} = \frac{2N_i C_t P_f C_d}{\phi L_{br}} \tag{C-A-6-7}$$

where

C_d = double curvature factor, which accounts for the potential larger demands on the lateral bracing in unbraced lengths containing inflection points, applied only to the point brace closest to the inflection point or to the panel brace corresponding to the unbraced length containing the inflection point, as well as the panel brace in the adjacent unbraced length closest to the inflection point
= $1 + (M_S / M_L)^2$, where the C_d factor is applicable as defined
= 1.0, otherwise

C_t = 1.0 for centroidal loading
= $1 + (1.2/n)$ for top-flange loading

I_{yc} = moment of inertia of the compression flange about its principal axis within the plane of the web, in.^{4} (mm^4)

M_L = absolute value of the maximum moment causing compression in the braced flange within the overall length, composed of an unbraced length containing an inflection point and the adjacent unbraced length closest to the inflection point, kip-ft (N-mm)

M_S = absolute value of the maximum moment causing tension in the braced flange within the overall length, composed of the unbraced length containing an inflection point and the adjacent unbraced length closest to the inflection point, kip-ft (N-mm)

N_i = 1.0 for panel bracing
= $(4 - 2/n)$ for point bracing

P_f = beam compressive flange force, kips (N)

n = number of intermediate braces

The C_d factor varies between 1 and 2, and is applied only to the point brace closest to the inflection point or to the panel brace corresponding to the unbraced length containing the inflection point and the adjacent panel brace closest to the inflection point. The term $(2N_iC_t)$ can be conservatively approximated as 10 for any number of point braces and 4 for panel bracing, and P_f can be approximated by M_r / h_o, which simplifies Equation C-A-6-7 to the stiffness requirements given by Equations A-6-6 and A-6-8. Equation C-A-6-7 can be used in lieu of Equations A-6-6 and A-6-8.

The brace strength requirement for panel bracing is

$$P_{br} = \frac{0.005 M_r C_t C_d}{h_o} \qquad \text{(C-A-6-8a)}$$

and for point bracing is

$$P_{br} = \frac{0.01 M_r C_t C_d}{h_o} \qquad \text{(C-A-6-8b)}$$

These requirements are based on an assumed initial lateral displacement of the compression flange of $\Delta_o = 0.002L_{br}$. The brace strength requirements of Equations A-6-5 and A-6-7 are derived from Equations C-A-6-8a and C-A-6-8b by assuming top flange loading ($C_t = 2$). Equations C-A-6-8a and C-A-6-8b can be used in lieu of Equations A-6-5 and A-6-7, respectively.

2. Torsional Bracing

Torsional bracing can either be attached continuously along the length of the beam (for example, a metal deck or slab) or located at discrete points along the length of the member (for example, cross-frames or secondary beam members). With respect to the girder response, torsional bracing attached to the tension flange is just as effective as a brace attached at mid-depth or to the compression flange, as long as distortion of the beam cross section is controlled. Although the girder response is generally not sensitive to the brace location, the position of the brace on the cross section influences the stiffness of the brace itself. For example, a torsional brace attached on the bottom flange will tend to bend in single curvature (with a flexural stiffness of $2EI/L$ based on the brace properties), while a brace attached on the top flange will tend to bend in reverse curvature (with a flexural stiffness of $6EI/L$ based on the brace properties). Partially restrained connections of the bracing to the girder being braced can be used if their flexibility is considered in evaluating the torsional brace stiffness (Ziemian, 2010).

The torsional brace requirements are based on the buckling strength of a beam with a continuous torsional brace along its length, as presented in Taylor and Ojalvo (1966) and modified for cross-section distortion in Yura (2001):

$$M_r \le M_{cr} = \sqrt{\left(C_{bu}M_o\right)^2 + \frac{C_b^2 EI_{yeff}\,\overline{\beta}_T}{2C_{tt}}} \qquad \text{(C-A-6-9)}$$

The term $C_{bu}M_o$ is the buckling strength of the beam without torsional bracing. $C_{tt} = 1.2$ when there is top flange loading and C_{tt} = 1.0 for centroidal loading. $\overline{\beta}_T = n\beta_T/L$ is the continuous torsional brace stiffness per unit length or its equivalent when n point braces, each with a stiffness β_T, are used along the span, L, and the factor 2 accounts for initial out-of-straightness (the continuous torsional ideal bracing stiffness is thus taken as $\overline{\beta}_T/2$). Neglecting the unbraced beam buckling term gives a conservative estimate of the torsional brace stiffness requirement as expressed in Equation A-6-11. For a doubly symmetric I-shaped cross section, I_{yeff} is equal to the moment of inertia about the principal axis within the plane of the web of the section, I_y. For a singly symmetric I-shaped cross section,

$$I_{yeff} = I_{yc} + (t/c)I_{yt} \qquad \text{(C-A-6-10)}$$

where

I_{yc} and I_{yt} = respective moments of inertia of compression and tension flanges about their principal axes within the plane of the web, in.^{4} (mm^4)

c = distance from the neutral axis to the extreme compressive fibers, in. (mm)

t = distance from the neutral axis to the extreme tensile fibers, in. (mm)

The strength requirement for beam torsional bracing is developed based upon an assumed initial twist imperfection of $\theta_o = 0.002L_b/h_o$, where h_o is equal to the depth of the beam. Based on the use of an effective bracing stiffness equal to two times the ideal torsional bracing stiffness, the torsional bracing required moment resistance may be estimated as $M_{rb} = \beta_T\theta_o$. Using the formulation of Equation A-6-11 (without ϕ or Ω), the strength requirement for the torsional bracing is

$$M_{br} = \beta_T \theta_o = \left(\frac{2.4 L M_r^2}{n E I_{yeff} C_b^2} \right) \left(\frac{L_{br}}{500 h_o} \right) \tag{C-A-6-11}$$

The 2010 *Specification* commentary (AISC, 2010) showed the simplification of this equation as Equation A-6-9, provided here as Equation C-A-6-12:

$$M_{br} = \frac{0.024 M_r L}{n C_b L_{br}} \tag{C-A-6-12}$$

The underlying development of this equation involves the assumption that the rigidity of the fully elastic beam is available to assist the torsional bracing in resisting the brace point displacements. In addition, the derivation does not account for the fact that, in beams where the lateral-torsional buckling resistance is limited by the strength corresponding to the yielding limit state, the increase in lateral-torsional buckling resistance due to moment gradient effects is smaller than the factor C_b (e.g., for compact-section beams, the flexural resistance is never greater than $\phi_b M_p$, irrespective of the C_b value). Furthermore, in cases involving top flange loading, the C_{tt} factor of 1.2 tends to be offset by $C_b > 1.0$.

Prado and White (2015) and Lokhande and White (2015) investigated a range of member bracing cases having different degrees of inelasticity at the member strength limit. A value of 2% of the corresponding member moment was found to accurately capture the torsional bracing strength requirement in all cases. Equation A-6-9 has been simplified to $0.02M_r$ based on the results of these studies.

The β_{sec} term in Equation A-6-10, and defined in Equations A-6-12 and A-6-13 accounts for cross-section distortion. A web stiffener at the brace point reduces cross-sectional distortion and improves the effectiveness of a torsional brace. When a cross frame is attached near both flanges or a vertical diaphragm element is approximately the same depth as the girder, then web distortion will be insignificant and β_{sec} may be taken as infinity. The required bracing flexural stiffness, β_{br}, given by Equation A-6-10 is obtained by solving the following expression, which represents the brace system stiffness including distortion effects:

$$\frac{1}{\beta_T} = \frac{1}{\beta_{br}} + \frac{1}{\beta_{sec}} \tag{C-A-6-13}$$

Yura (2001) provides additional guidance regarding the handling of cross-section distortional flexibility for cases where the bracing system is attached through only a portion of the depth of the member being braced.

Parallel chord trusses with both chords subjected only to flexural loading and with both chords extended to the end of the span and attached to supports can be treated the same as beams. In Equations A-6-5 through A-6-9, M_r may be taken as the maximum compressive chord force times the depth of the truss to determine the torsional brace strength and stiffness requirements. Cross-section distortion effects, β_{sec}, need not be considered when full-depth cross-frames are used for bracing. When either chord does not extend to the end of the span, consideration should be given to the control of twisting near the ends of the span by the use of cross-frames or ties.

Beams—Point Torsional Bracing Combined with Lateral Bracing at the Compression Flange. Recent studies (Prado and White, 2015; Lokhande and White, 2015) have suggested that for beams having point torsional bracing combined with panel or point lateral bracing on the flange subjected to flexural compression, the required torsional and lateral brace stiffnesses can be reduced relative to the base values specified in Sections 6.3.1 and 6.3.2, but should satisfy this interaction equation:

$$\frac{\beta_{Tbr}}{\beta_{Tbro}} + \frac{\beta_{Lbr}}{\beta_{Lbro}} \geq 1.0 \qquad \text{(C-A-6-14)}$$

where

β_{Lbr} = actual or provided lateral brace stiffness, kip/in. (N/mm)

β_{Lbro} = required lateral brace stiffness given by Equation A-6-6 for panel bracing or Equation A-6-8 for point bracing acting alone, kip/in. (N/mm)

β_{Tbr} = actual or provided torsional brace stiffness, kip-in./rad (N-mm/rad)

β_{Tbro} = required torsional brace stiffness given by Equation A-6-10 acting alone, kip-in./rad (N-mm/rad)

Beams—Point Torsional Bracing Combined with Lateral Bracing at the Tension Flange. For beams having point torsional bracing combined with panel or point lateral bracing on the flange subjected to flexural tension, Equation C-A-6-14 applies and, in addition, the required torsional brace stiffness should be greater than or equal to the smaller of $\beta_{Pbro} h_o^2$ or β_{Tbro},

where

β_{Pbro} = required point lateral brace stiffness given by Equation A-6-8, calculated using the unbraced length between the torsional brace points, kip/in. (N/mm)

h_o = distance between the flange centroids, in. (mm)

The provisions of Sections 6.3.1 and 6.3.2 apply for the lateral and the torsional brace strength requirements.

Reduction in Beam Bracing Requirements with Combined Torsional and Lateral Bracing. Equation C-A-6-14 recognizes the typical reduction in the beam torsional and lateral bracing stiffness requirements when lateral and torsional bracing are used in combination, thus restraining both twisting and lateral movement at the braced points. This linear interaction equation is known to provide a conservative estimate of the bracing stiffness requirements in cases where the lateral bracing is provided at or near the flange subjected to flexural compression (Yura, et al., 1992; Prado and White, 2015; Lokhande and White, 2015).

For situations where the lateral bracing is located at or near the flange subjected to flexural tension, the lateral bracing system is ineffective on its own. However, a point torsional brace works effectively as a lateral brace to the compression flange, in the limit that the lateral bracing system stiffness becomes large. Prado and White (2015) and Lokhande and White (2015) show that the point lateral bracing stiffness requirement of Equation A-6-8, denoted by β_{Pbro}, when multiplied by h_o^2, serves as an accurate to conservative minimum limit on the required torsional bracing stiffness obtained from Equation C-A-6-14 for the case of point torsional bracing combined with lateral bracing at the tension flange.

Furthermore, where $\beta_{Pbro} h_o^2$ is greater than the base torsional bracing stiffness requirement from Equation A-6-10, the torsional bracing stiffness need not be greater than the requirement from Equation A-6-10.

The minimum required strength of the separate lateral and torsional bracing components is still governed by the provisions of Sections 6.3.1 and 6.3.2. The strength demands on the separate brace components are not necessarily reduced by the combination.

6.4. BEAM-COLUMN BRACING

The provisions for beam-column bracing are modified slightly in this edition of the Specification to reflect new research by Lokhande and White (2015) and White et al. (2011). In addition, this research proposed the following additional new guidelines for beam-column bracing.

Beam-Columns Braced by a Combination of Lateral and Torsional Bracing. For beam-columns braced by a combination of lateral and torsional bracing, the following rules apply:

(1) The required lateral bracing stiffness can be determined using Equations A-6-2 for panel lateral bracing, or Equations A-6-4 for point lateral bracing, based on the required member axial force, P_r. In Equations A-6-4, L_{br} can be taken as the actual unbraced length; the provision in Section 6.2.2 that L_{br} need not be taken less than the maximum permitted effective length based on P_r should not be applied.

(2) The required torsional bracing stiffness can be determined using Equation A-6-10, with an equivalent moment equal to $M_r + P_r h_o/2$, where P_r is the axial force in the member being braced.

(3) The required lateral brace strength can be determined using Equation A-6-1 for panel lateral bracing, or Equation A-6-3 for point lateral bracing, based on 1.3 of the required axial force, $1.3P_r$.

(4) The required torsional brace strength can be determined using Equation A-6-9 with an equivalent moment equal to $M_r + P_r h_o/2$, where P_r is the axial force in the member being braced.

Beam-Columns Braced by a Single Lateral Bracing System. For beam-columns braced by a single lateral bracing system attached at or near a flange subjected to flexural compression throughout the member length, the following rules apply:

(1) For panel bracing, when the opposite flange is subjected to a net tension force due to the axial and moment loading throughout the member length, the required bracing stiffness can be taken as the sum of the values determined using Equations A-6-2 with an equivalent axial force equal to $P_r/2$ and Equations A-6-6 with the required moment, M_r. The required bracing strength can be taken as the sum of the values determined using Equation A-6-1 with an equivalent axial force equal to $P_r/2$ and Equation A-6-5 with the required moment, M_r.

(2) For panel bracing, when the opposite flange is subjected to a net compression force due to the axial and moment loading at any position within the member length,

the required bracing stiffness can be taken as the sum of the values determined using Equation A-6-2 with an equivalent axial force equal to $2.5P_r$ and Equation A-6-6 with the required moment, M_r. The required bracing strength can be taken as the sum of the values determined using Equation A-6-1 with an equivalent axial force equal to $2.5P_r$ and Equation A-6-5 with the required moment, M_r.

(3) For point bracing, when the opposite flange is subjected to a net tension force due to the axial and moment loading throughout the member length, the required bracing stiffness can be taken as the sum of the values determined using Equations A-6-4 with an equivalent axial force equal to $P_r/2$ and Equations A-6-8 with the required moment, M_r. In Equations A-6-4 and A-6-8, L_{br} can be taken as the actual unbraced length; the provisions in Appendix 6, Sections 6.2.2 and 6.3.1b, that L_{br} need not be taken less than the maximum permitted effective length based on P_r and M_r, should not be applied. The required bracing strength can be taken as the sum of the values determined using Equation A-6-3 with an equivalent axial force equal to $P_r/2$ and Equation A-6-7 with the required moment, M_r.

(4) For point bracing, when the opposite flange is subjected to a net compression force due to the axial and moment loading at any position within the member length, the required bracing stiffness can be taken as the sum of the values determined using Equation A-6-4 with an equivalent axial force equal to $2.5P_r$ and Equation A-6-8 with the required moment, M_r. In Equations A-6-4 and A-6-8, L_{br} can be taken as the actual unbraced length; the provisions in Sections 6.2.2 and 6.3.1b, that L_{br} need not be taken less than the maximum permitted effective length based on P_r and M_r, should not be applied. The required bracing strength can be taken as the sum of the values determined using Equation A-6-3 with an equivalent axial force equal to $2.5P_r$ and Equation A-6-7 with the required moment, M_r.

In the application of these rules, where the member is subjected to axial compression larger than $P_c/10$, the slenderness ratio, L_{br}/r_{yf}, of the flange that does not have the additional lateral bracing should not be greater than 200,

where

L_{br} = unbraced length between the points where the flange having the larger unbraced length is restrained laterally, in. (mm)

P_c = available axial compressive strength of the member determined according to Chapter E, kip (N)

r_{yf} = radius of gyration of the flange having the larger unbraced length, taken about its principal axis parallel to the plane of the web, in. (mm)

This avoids potential excessive amplification of the bracing demands in cases where one flange is braced at closer intervals while the other flange has a large brace spacing.

Summary—Additional Guidelines for Beam-Column Bracing. The guidelines for beam-column bracing recommended in the preceding discussion utilize a simplified combination of the requirements for columns and for beams from Sections 6.2 and 6.3, respectively.

For beam-columns braced by a combination of lateral and torsional bracing, the lateral bracing is designed based on the column bracing provisions of Section 6.2 given the required axial compression of $1.0P_r$ for the lateral bracing stiffness requirement and $1.3P_r$ for the lateral bracing strength requirement. Correspondingly, the torsional bracing is designed based on the beam torsional bracing provisions of Section 6.3 using an equivalent moment equal to $M_r/C_b + P_r h_o/2$. The second term in this expression accounts for the increased demands on the torsional bracing caused by the presence of the axial compression force.

For beam-columns braced by a single lateral bracing system attached at or near a flange subjected to flexural compression throughout the member length, and when the opposite flange is subjected to a net tension due to the axial and moment loading at any position within the member length, the lateral bracing may be designed based on the sum of the requirements from the column bracing rules of Section 6.2 with an axial force of $P_r/2$ and the beam torsional bracing rules of Section 6.3 with the moment M_r.

The bracing requirements for other more general bracing configurations may be determined using a buckling analysis or a second-order load deflection analysis as discussed in Section 6.1.

APPENDIX 7

ALTERNATIVE METHODS OF DESIGN FOR STABILITY

The effective length method and first-order analysis method are addressed in this Appendix as alternatives to the direct analysis method, which is presented in Chapter C. These alternative methods of design for stability can be used when the limits on their use as defined in Appendix 7, Sections 7.2.1 and 7.3.1, respectively, are satisfied.

Both methods in this Appendix utilize the nominal geometry and the nominal elastic stiffnesses (*EI, EA*) in the analysis. Accordingly, it is important to note that the sidesway amplification ($\Delta_{2nd\text{-}order}/\Delta_{1st\text{-}order}$ or B_2) limits specified in Chapter C and this Appendix are different. For the direct analysis method in Chapter C, the limit of 1.7 for certain requirements is based upon the use of reduced stiffnesses (EI^* and EA^*). For the effective length method and first-order analysis method, the equivalent limit of 1.5 is based upon the use of unreduced stiffnesses (*EI, EA*).

7.2. EFFECTIVE LENGTH METHOD

The effective length method (though it was not originally identified by this name) has been used in various forms in the AISC *Specification* since 1961. The current provisions are essentially the same as those in Appendix 7 of the 2010 AISC *Specification* (AISC, 2010), with the following exceptions.

These provisions, together with the use of a column effective length greater than the actual length for calculating available strength in some cases, account for the effects of initial out-of-plumbness and member stiffness reductions due to the spread of plasticity. No stiffness reduction is required in the analysis.

The effective length, $L_c = KL$, for column buckling based upon elastic (or inelastic) stability theory, or alternatively the equivalent elastic column buckling stress, $F_e = \pi^2 E/(L_c/r)^2$, is used to calculate an axial compressive strength, P_c, through an empirical column curve that accounts for geometric imperfections and distributed yielding (including the effects of residual stresses). This column strength is then combined with the available flexural strength, M_c, and second-order member forces, P_r and M_r, in the beam-column interaction equations.

Braced Frames. Braced frames are commonly idealized as vertically cantilevered pin-connected truss systems, ignoring any secondary moments within the system. The effective length factor, K, of components of the braced frame is normally taken as 1.0, unless a smaller value is justified by structural analysis and the member and connection design is consistent with this assumption. If connection fixity is modeled in the analysis, the resulting member and connection moments must be accommodated in the design.

If $K < 1.0$ is used for the calculation of P_n in braced frames, the additional demands on the stability bracing systems and the influence on the second-order moments in beams providing restraint to the columns must be considered. The provisions in Appendix 6 do not address the additional demands on bracing members from the use of $K < 1.0$. Generally, a P-Δ and P-δ second-order elastic analysis is necessary for calculation of the second-order moments in beams providing restraint to column members designed based on $K < 1.0$. Therefore, design using $K = 1.0$ is recommended, except in those special situations where the additional calculations are deemed justified.

The effective length, L_c, may be taken as $0.5L$ for both in-plane and out-of-plane buckling of concentrically loaded compression braces in X-braced frames, where L is the overall length of the brace between work points, with identically sized brace members when the compression and tension braces are attached at the midpoint and the magnitude of compression and tension forces in the braces are approximately equal (McGuire et al., 2000). Greater unbraced lengths for out-of-plane buckling may be required for X-braced frames with unbalanced brace forces, particularly those with discontinuous midpoint connections (Davaran, 2001). Shorter unbraced lengths may also be justified (El-Tayem and Goel, 1986; Picard and Beaulieu, 1987; Nair, 1997; Moon et al., 2008).

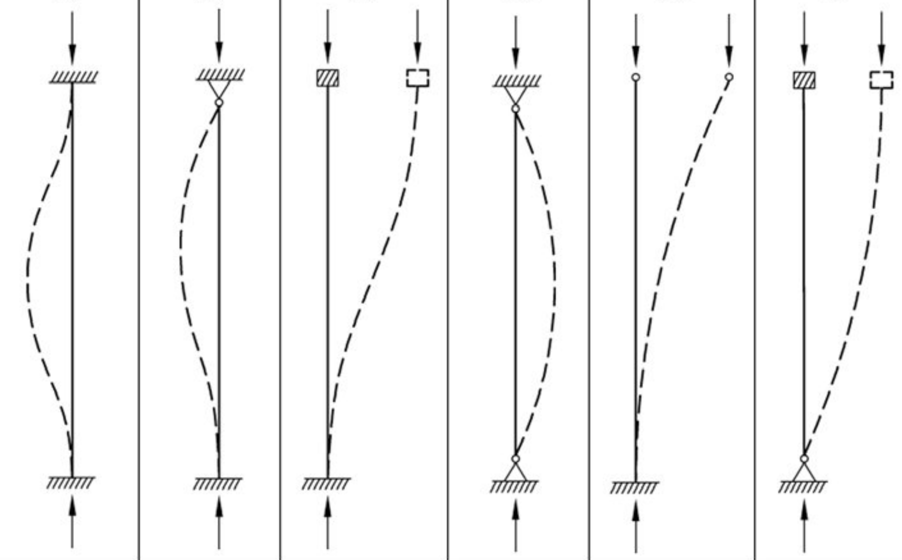
Moment Frames. Moment frames rely primarily upon the flexural stiffness of the connected beams and columns for stability. Stiffness reductions due to shear deformations may require consideration when bay sizes are small and/or members are deep.

When the effective length method is used, the design of all beam-columns in moment frames must be based on an effective length, $L_c = KL$, greater than the actual laterally unbraced length, L, except when specific exceptions based upon high structural stiffness are met. When the sidesway amplification ($\Delta_{2nd\text{-}order}/\Delta_{1st\text{-}order}$ or B_2) is equal to or less than 1.1, the frame design may be based on the use of $K = 1.0$ for the columns. This simplification for stiffer structures results in a 6% maximum error in the in-plane beam-column strength checks of Chapter H (White and Hajjar, 1997a). When the sidesway amplification is larger, K must be calculated.

A wide range of methods has been suggested in the literature for the calculation of K-factors (Kavanagh, 1962; Johnston, 1976; LeMessurier, 1977; ASCE, 1997; White and Hajjar, 1997b). These range from simple idealizations of single columns, as shown in Table C-A-7.1, to complex buckling solutions for specific frames and loading conditions. In some types of frames, K-factors are easily estimated or calculated and are a convenient tool for stability design. In other types of structures, the determination of accurate K-factors is determined by tedious hand procedures, and system stability may be assessed more effectively with the direct analysis method.

Alignment Charts. The most common method for determining K is through use of the alignment charts, which are shown in Figure C-A-7.1 for frames with sidesway inhibited and Figure C-A-7.2 for frames with sidesway uninhibited (Kavanagh,

TABLE C-A-7.1
Approximate Values of Effective Length Factor, *K*

	(a)	(b)	(c)	(d)	(e)	(f)
Buckled shape of column is shown by dashed line						
Theoretical K value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.1	2.0
End condition code	 Rotation fixed and translation fixed Rotation free and translation fixed Rotation fixed and translation free Rotation free and translation free					

1962). These charts are based on assumptions of idealized conditions, which seldom exist in real structures, as follows:

(1) Behavior is purely elastic.

(2) All members have constant cross section.

(3) All joints are rigid.

(4) For columns in frames with sidesway inhibited, rotations at opposite ends of the restraining beams are equal in magnitude and opposite in direction, producing single curvature bending.

(5) For columns in frames with sidesway uninhibited, rotations at opposite ends of the restraining beams are equal in magnitude and direction, producing reverse curvature bending.

(6) The stiffness parameter $L\sqrt{P/EI}$ of all columns is equal.

(7) Joint restraint is distributed to the column above and below the joint in proportion to EI/L for the two columns.

(8) All columns buckle simultaneously.

(9) No significant axial compression force exists in the girders.

(10) Shear deformations are neglected.

The alignment chart for sidesway inhibited frames shown in Figure C-A-7.1 is based on the following equation:

$$\frac{G_A G_B}{4}(\pi/K)^2 + \left(\frac{G_A + G_B}{2}\right)\left[1 - \frac{\pi/K}{\tan(\pi/K)}\right] + \frac{2\tan(\pi/2K)}{(\pi/K)} - 1 = 0 \quad \text{(C-A-7-1)}$$

The alignment chart for sidesway uninhibited frames shown in Figure C-A-7.2 is based on the following equation:

$$\frac{G_A G_B(\pi/K)^2 - 36}{6(G_A + G_B)} - \frac{(\pi/K)}{\tan(\pi/K)} = 0 \quad \text{(C-A-7-2)}$$

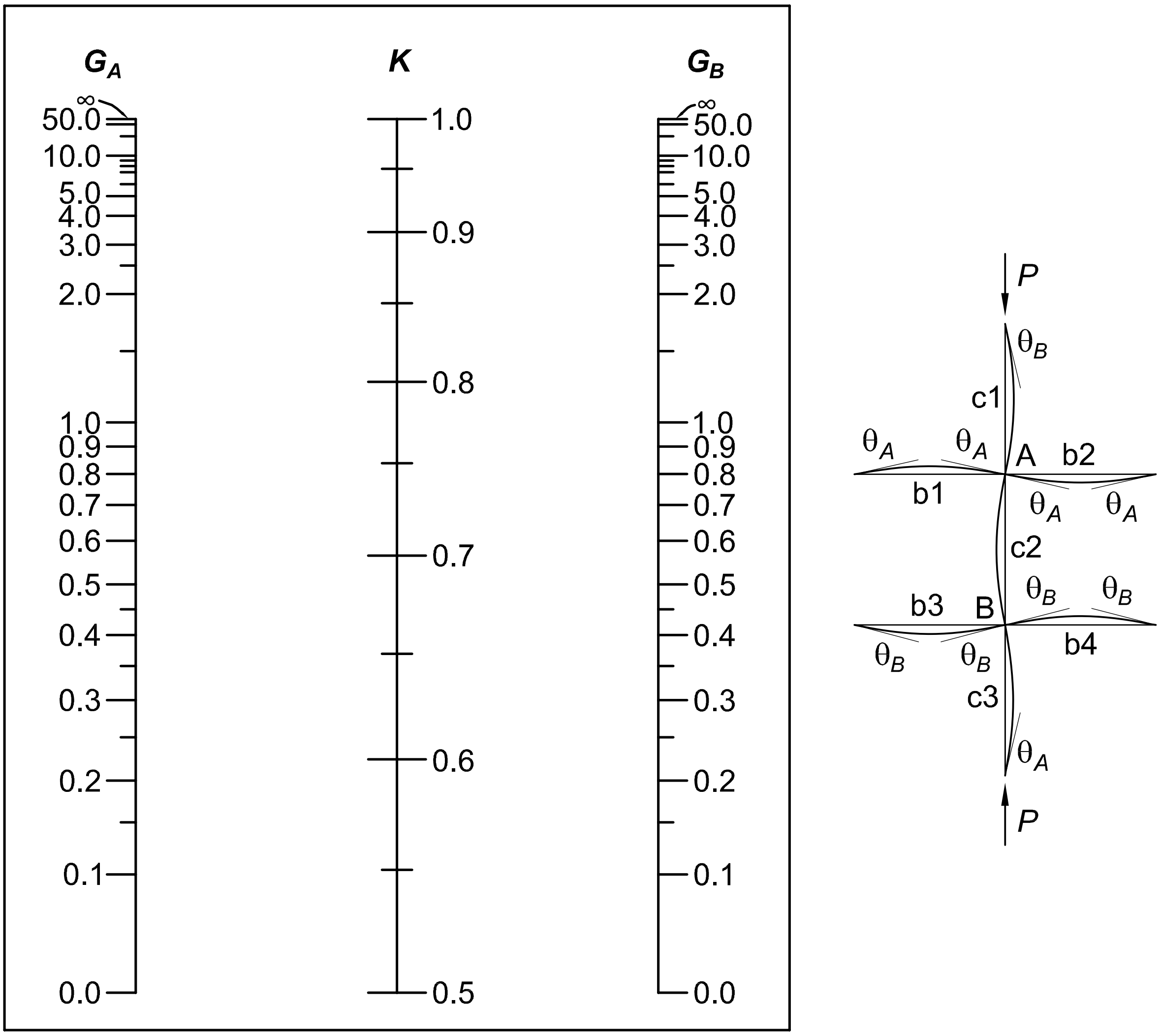


Fig. C-A-7.1. Alignment chart—sidesway inhibited (braced frame).

where

$$G = \frac{\Sigma\left(E_{col}I_{col} / L_{col}\right)}{\Sigma\left(E_g I_g / L_g\right)} = \frac{\Sigma\left(EI / L\right)_{col}}{\Sigma\left(EI / L\right)_g} \tag{C-A-7-3}$$

The subscripts *A* and *B* refer to the joints at the ends of the column being considered. The symbol Σ indicates a summation of all members rigidly connected to that joint and located in the plane in which buckling of the column is being considered. E_{col} is the elastic modulus of the column, I_{col} is the moment of inertia of the column, and L_{col} is the unsupported length of the column. E_g is the elastic modulus of the girder, I_g is the moment of inertia of the girder, and L_g is the unsupported length of the girder or other restraining member. I_{col} and L_g are taken about axes perpendicular to the plane of buckling being considered. The alignment charts are valid for different materials if an appropriate effective rigidity, *EI*, is used in the calculation of *G*.

It is important to remember that the alignment charts are based on the assumptions of idealized conditions previously discussed—and that these conditions seldom exist in real structures. Therefore, adjustments are often required.

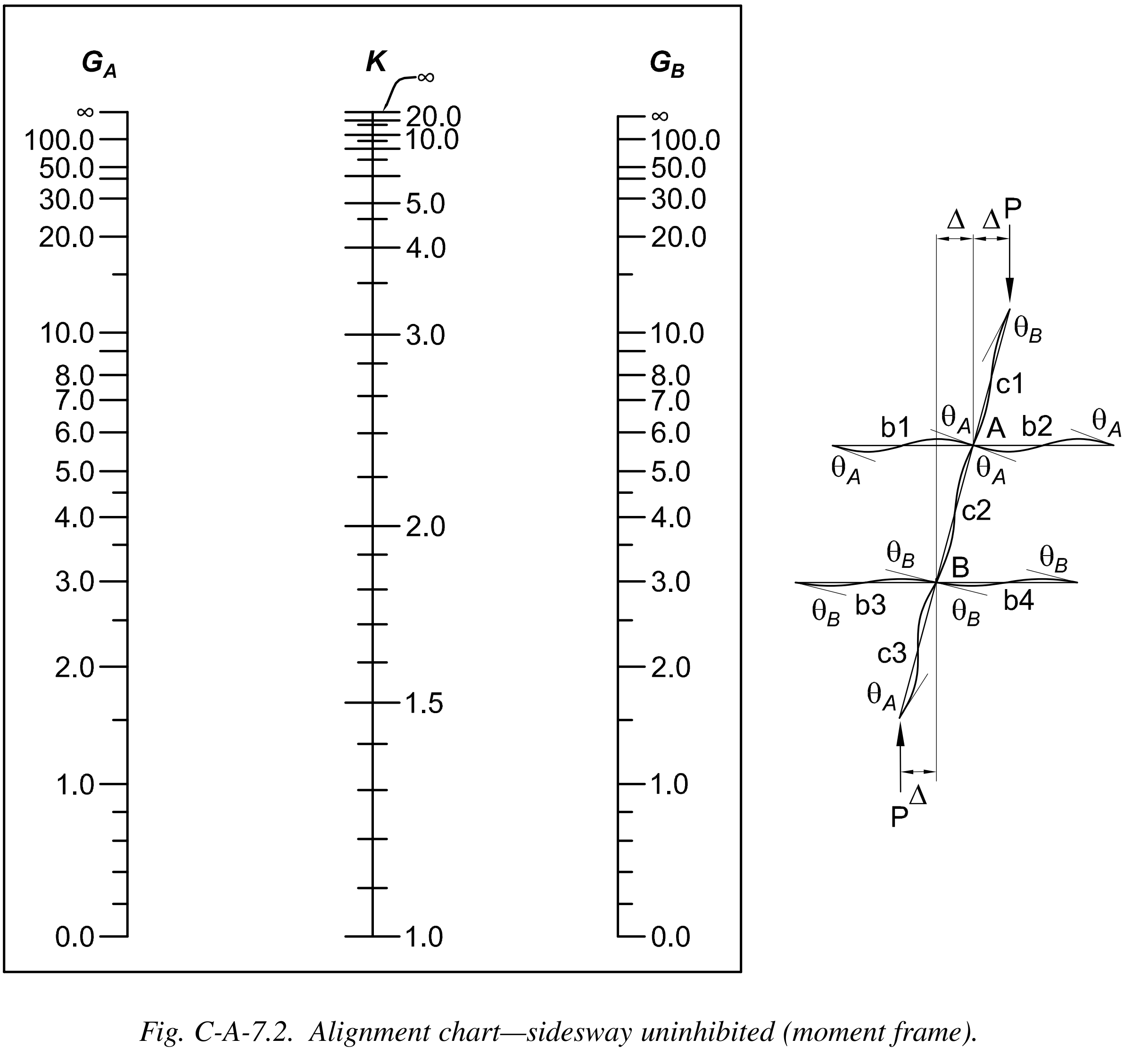


Fig. C-A-7.2. Alignment chart—sidesway uninhibited (moment frame).

Adjustments for Columns With Differing End Conditions. For column ends supported by, but not rigidly connected to, a footing or foundation, G is theoretically infinity but unless designed as a true friction-free pin, may be taken as 10 for practical designs. If the column end is rigidly attached to a properly designed footing, G may be taken as 1.0. Smaller values may be used if justified by analysis.

Adjustments for Girders With Differing End Conditions. For sidesway inhibited frames, these adjustments for different girder end conditions may be made:

(a) If rotation at the far end of a girder is prevented, multiply $(EI/L)_g$ of the member by 2.

(b) If the far end of the girder is pinned, multiply $(EI/L)_g$ of the member by 1.5.

For sidesway uninhibited frames and girders with different boundary conditions, the modified girder length, L_g', should be used in place of the actual girder length, where

$$L_g' = L_g\,(2 - M_F/M_N) \qquad \text{(C-A-7-4)}$$

M_F is the far end girder moment and M_N is the near end girder moment from a first-order lateral analysis of the frame. The ratio of the two moments is positive if the girder is in reverse curvature. If M_F/M_N is more than 2.0, then L_g' becomes negative, in which case G is negative and the alignment chart equation must be used. For sidesway uninhibited frames, the following adjustments for different girder end conditions may be made:

(a) If rotation at the far end of a girder is prevented, multiply $(EI/L)_g$ of the member by $^2/_3$.

(b) If the far end of the girder is pinned, multiply $(EI/L)_g$ of the member by $^1/_2$.

Adjustments for Girders with Significant Axial Load. For both sidesway conditions, multiply $(EI/L)_g$ by the factor $(1 - Q/Q_{cr})$, where Q is the axial load in the girder and Q_{cr} is the in-plane buckling load of the girder based on $K = 1.0$.

Adjustments for Column Inelasticity. For both sidesway conditions, replace $(E_{col}I_{col})$ with $\tau_b(E_{col}I_{col})$ for all columns in the expression for G_A and G_B. It is noted that τ_b is being used as an approximation for the τ_a expression that appeared in previous editions of the Commentary (AISC, 2005).

Adjustments for Connection Flexibility. One important assumption in the development of the alignment charts is that all beam-column connections are fully restrained (FR) connections. When the far end of a beam does not have an FR connection that behaves as assumed, an adjustment must be made. When a beam connection at the column under consideration is a shear-only connection, that is, there is no moment, then that beam cannot participate in the restraint of the column and it cannot be considered in the $\Sigma(EI/L)_g$ term of the equation for G. Only FR connections can be used directly in the determination of G. Partially restrained (PR) connections with a documented moment-rotation response can be utilized, but $(EI/L)_g$ of each beam must be adjusted to account for the connection flexibility. ASCE (1997) provides a detailed discussion of frame stability with PR connections.

Combined Systems. When combined systems are used, all the systems must be included in the structural analysis. Consideration must be given to the variation in stiffness inherent in concrete or masonry shear walls due to various degrees to which these elements may experience cracking. This applies to load combinations for serviceability as well as strength. It is prudent for the designer to consider a range of possible stiffnesses, as well as the effects of shrinkage, creep and load history, in order to envelope the likely behavior and provide sufficient strength in all interconnecting elements between systems. Following the analysis, the available strength of compression members in moment frames must be assessed with effective lengths calculated as required for moment-frame systems; other compression members may be assessed using $K = 1.0$.

Leaning Columns and Distribution of Sidesway Instability Effects. Columns in gravity framing systems can be designed as pin-ended columns with $K = 1.0$. However, the destabilizing effects (P-Δ effects) of the gravity loads on all such columns, and the load transfer from these columns to the lateral force-resisting system, must be accounted for in the design of the lateral force-resisting system.

It is important to recognize that sidesway instability of a building is a story phenomenon involving the sum of the sway resistances of all the lateral force-resisting elements in the story and the sum of the factored gravity loads in the columns in that story. No individual column in a story can buckle in a sidesway mode without the entire story buckling.

If every column in a story is part of a moment frame and each column is designed to support its own axial load, P, and P-Δ moment such that the contribution of each column to the lateral stiffness or to the story buckling load is proportional to the axial load supported by the column, all the columns will buckle simultaneously. Under this idealized condition, there is no interaction among the columns in the story; column sway instability and frame instability occur at the same time. Typical framing, however, does not meet this idealized condition, and real systems redistribute the story P-Δ effects to the lateral force-resisting elements in that story in proportion to their stiffnesses. This redistribution can be accomplished using such elements as floor diaphragms or horizontal trusses.

In a building that contains columns that contribute little or nothing to the sway stiffness of the story, such columns are referred to as leaning or gravity-only columns. These columns can be designed using $K = 1.0$, but the lateral force-resisting elements in the story must be designed to support the destabilizing P-Δ effects developed from the loads on these leaning columns. The redistribution of P-Δ effects among columns must be considered in the determination of K and F_e for all the columns in the story for the design of moment frames. The proper K-factor for calculation of P_c in moment frames, accounting for these effects, is denoted in the following by the symbol K_2.

Effective Length for Story Stability. Two approaches for evaluating story stability are recognized: the story stiffness approach (LeMessurier, 1976, 1977) and the story buckling approach (Yura, 1971). Additionally, a simplified approach proposed by LeMessurier (1995) is also discussed.

The column effective length factor associated with lateral story buckling is expressed as K_2 in the following discussions. The value of K_2 determined from Equation C-A-7-5 or Equation C-A-7-8 may be used directly in the equations of Chapter E. However, it is important to note that this substitution is not appropriate when calculating the story buckling mode as the summation of $\pi^2 EI/(K_2 L)^2$. Also, note that the value of P_c calculated using K_2 by either method cannot be taken greater than the value of P_c determined based on sidesway-inhibited buckling.

Story Stiffness Approach. For the story stiffness approach, K_2 is defined as

$$K_2 = \sqrt{\frac{P_{story}}{R_M P_r}\left(\frac{\pi^2 EI}{L^2}\right)\left(\frac{\Delta_H}{HL}\right)} \geq \sqrt{\frac{\pi^2 EI}{L^2}\left(\frac{\Delta_H}{1.7 H_{col} L}\right)} \tag{C-A-7-5}$$

in which R_M is used to approximate the influence of P-δ effects on the sidesway stiffness of the columns in a story and is defined in Equation A-8-8 as

$$R_M = 1 - 0.15(P_{mf}/P_{story}) \tag{C-A-7-6}$$

where P_{mf}, P_{story} and H are as defined in Appendix 8, Section 8.2.2.

It is possible that certain columns, having only a small contribution to the lateral force resistance in the overall frame, will have a K_2 value less than 1.0 based on the term to the left of the inequality. The limit on the righthand side is a minimum value for K_2 that accounts for the interaction between sidesway and non-sidesway buckling (ASCE, 1997; White and Hajjar, 1997b). The term H_{col} is the shear in the column under consideration, produced by the lateral forces used to compute Δ_H.

Equation C-A-7-5 can be reformulated to obtain the column buckling load, P_{e2}, as

$$P_{e2} = \left(\frac{HL}{\Delta_H}\right)\frac{P_r}{P_{story}} R_M \leq 1.7 H_{col} L/\Delta_H \tag{C-A-7-7}$$

P_{story} in Equations C-A-7-5 to C-A-7-7 includes all columns in the story, including any leaning columns, and P_r is for the column under consideration. The column buckling load, P_{e2}, calculated from Equation C-A-7-7 may be larger than $\pi^2 EI/L^2$, but may not be larger than the limit on the right-hand side of this equation.

In Appendix 8, the story stiffness approach is the basis for the B_2 calculation (for P-Δ effects). In Equation A-8-7, the buckling load for the story is expressed in terms of the story drift ratio, Δ_H/L, from a first-order lateral load analysis at a given applied lateral load level. In preliminary design, Δ_H/L may be taken in terms of a target maximum value for this drift ratio. This approach focuses the engineer's attention on the most fundamental stability requirement in building frames: providing adequate overall story stiffness in relation to the total vertical load, P_{story}, supported by the story. The elastic story stiffness expressed in terms of the drift ratio and the total horizontal load acting on the story is $H/(\Delta_H/L)$.

Story Buckling Approach. For the story buckling approach, K_2 is defined as

$$K_2 = \sqrt{\frac{\frac{\pi^2 EI}{L^2}}{P_r}\left[\frac{P_{story}}{\Sigma\frac{\pi^2 EI}{\left(K_{n2}L\right)^2}}\right]} \geq \sqrt{\frac{5}{8}}K_{n2} \qquad \text{(C-A-7-8)}$$

where K_{n2} is defined as the value of K determined directly from the alignment chart in Figure C-A-7.2.

The value of K_2 calculated from Equation C-A-7-8 may be less than 1.0. The limit on the righthand side is a minimum value for K_2 that accounts for the interaction between sidesway and non-sidesway buckling (ASCE, 1997; White and Hajjar, 1997b; Geschwindner, 2002; AISC-SSRC, 2003b). Other approaches to calculating K_2 are given in previous editions of this Commentary and the foregoing references.

Equation C-A-7-8 can be reformulated to obtain the column buckling load, P_{e2}, as

$$P_{e2} = \left(\frac{P_r}{P_{story}}\right)\Sigma\frac{\pi^2 EI}{\left(K_{n2}L\right)^2} \leq 1.6\frac{\pi^2 EI}{\left(K_{n2}L\right)^2} \qquad \text{(C-A-7-9)}$$

P_{story} in Equations C-A-7-8 and C-A-7-9 includes all columns in the story, including any leaning columns, and P_r is for the column under consideration. The column buckling load, P_{e2}, calculated from Equation C-A-7-9 may be larger than $\pi^2 EI/L^2$ but may not be larger than the limit on the righthand side of this equation.

LeMessurier Approach. Another simple approach for the determination of K_2 (LeMessurier, 1995), based only on the column end moments, is:

$$K_2 = \left[1+\left(1-\frac{M_1}{M_2}\right)^4\right]\sqrt{1+\frac{5}{6}\left(\frac{P_{story}-P_{mf}}{P_{mf}}\right)} \qquad \text{(C-A-7-10)}$$

In this equation, M_1 and M_2 are the smaller and larger end moments, respectively, in the column. These moments are determined from a first-order analysis of the frame under lateral load. Column inelasticity is considered in the derivation of this equation. The unconservative error in P_c, when it is based on K_2 determined from Equation C-A-7-10, is less than 3%, as long as the following inequality is satisfied:

$$\left(\frac{\Sigma P_{ymf}}{HL/\Delta_H}\right)\left(\frac{P_{story}}{P_{mf}}\right) \leq 0.45 \qquad \text{(C-A-7-11)}$$

where ΣP_{ymf} is the sum of the axial yield strengths of all columns in the story that are part of moment frames, if any, in the direction of translation being considered.

Some Conclusions Regarding K. Column design using K-factors can be tedious and confusing for complex building structures containing leaning columns and/or combined framing systems, particularly where column inelasticity is considered. This confusion can be avoided if the direct analysis method of Chapter C is used, where

P_c is always based on $K = 1.0$. Subject to certain limitations, the direct analysis method may be simplified to the first-order analysis method of Appendix 7, Section 7.3. Furthermore, when $\Delta_{2nd\text{-}order}/\Delta_{1st\text{-}order}$ or B_2 is sufficiently low, $K = 1.0$ may be assumed in the effective length method as specified in Appendix 7, Section 7.2.3(b).

Comparison of the Effective Length Method and the Direct Analysis Method. Figure C-C2.5(a) shows a plot of the in-plane interaction equation for the effective length method, where the anchor point on the vertical axis, P_{nKL}, is determined using an effective length, $L_c = KL$. Also shown in this plot is the same interaction equation with the first term based on the yield load, P_y. For W-shapes, this in-plane beam-column interaction equation is a reasonable estimate of the internal force state associated with full cross-section plastification.

The P versus M response of a typical member, obtained from second-order spread-of-plasticity analysis and labeled “actual response,” indicates the maximum axial force, P_r, that the member can sustain prior to the onset of instability. The load-deflection response from a second-order elastic analysis using the nominal geometry and elastic stiffness, as conducted with the effective length method, is also shown. The “actual response” curve has larger moments than the second-order elastic curve due to the combined effects of distributed yielding and geometric imperfections, which are not included in the second-order elastic analysis.

In the effective length method, the intersection of the second-order elastic analysis curve with the P_{nKL} interaction curve determines the member strength. The plot in Figure C-C2.5(a) shows that the effective length method is calibrated to give a resultant axial strength, P_c, consistent with the actual response. For slender columns, the calculation of the effective length, $L_c = KL$, (and P_{nKL}) is critical to achieving an accurate solution when using the effective length method.

One consequence of the procedure is that it underestimates the actual internal moments under the factored loads, as shown in Figure C-C2.5(a). This is inconsequential for the beam-column in-plane strength check because P_{nKL} reduces the effective strength in the correct proportion. However, the reduced moment can affect the design of the beams and connections, which provide rotational restraint to the column. This is of greatest concern when the calculated moments are small and axial loads are large, such that P-Δ moments induced by column out-of-plumbness can be significant.

The important difference between the direct analysis method and the effective length method is that where the former uses reduced stiffness in the analysis and $K = 1.0$ in the beam-column strength check, the latter uses nominal stiffness in the analysis and K from a sidesway buckling analysis in the beam-column strength check. The direct analysis method can be more sensitive to the accuracy of the second-order elastic analysis because analysis at reduced stiffness increases the magnitude of second-order effects. However, this difference is important only at high sidesway amplification levels; at those levels the accuracy of the calculation of K for the effective length method also becomes important.

7.3. FIRST-ORDER ANALYSIS METHOD

This section provides a method for designing frames using a first-order elastic analysis with the effective length, L_c, taken as the laterally unbraced length (K = 1.0), provided the limitations in Appendix 7, Section 7.3.1 are satisfied. This method is derived from the direct analysis method by mathematical manipulation [see AISC Design Guide 28, *Stability Design of Steel Buildings,* (Griffis and White, 2013)] so that the second-order internal forces and moments are determined directly as part of the first-order analysis. It is based upon a target maximum drift ratio, Δ/L, and assumptions, including:

(1) The sidesway amplification $\Delta_{2nd\ order}/\Delta_{1st\ order}$ or B_2 is assumed equal to 1.5.

(2) The initial out-of-plumbness in the structure is assumed as $\Delta_o/L = 1/500$, but the initial out-of-plumbness does not need to be considered in the calculation of Δ.

The first-order analysis is performed using the nominal (unreduced) stiffness; stiffness reduction is accounted for solely within the calculation of the amplification factors. The nonsway amplification of beam-column moments is addressed within the procedure specified in this section by applying the B_1 amplifier of Appendix 8, Section 8.2.1 conservatively to the total member moments. In many cases involving beam-columns not subject to transverse loading between supports in the plane of bending, B_1 = 1.0.

The target maximum drift ratio, corresponding to drifts under either the LRFD strength load combinations or 1.6 times the ASD strength load combinations, can be assumed at the start of design to determine the additional lateral load, N_i. As long as that drift ratio is not exceeded at any strength load level, the design will be conservative.

AISC Design Guide 28 presents the details of this method. If this approach is employed, it can be shown that, for $B_2 \le 1.5$ and $\tau_b = 1.0$, the required additional lateral load to be applied with other lateral loads in a first-order analysis of the structure, using the nominal (unreduced) stiffness, is

$$N_i = \left(\frac{B_2}{1-0.2B_2}\right)\frac{\Delta}{L}Y_i \ge \left(\frac{B_2}{1-0.2B_2}\right)0.002Y_i \qquad \text{(C-A-7-12)}$$

where these variables are as defined in Chapter C, Appendix 7 and Appendix 8. Note that if B_2 based on the unreduced stiffness is set equal to the 1.5 limit prescribed in Chapter C, then

$$N_i = 2.1\left(\frac{\Delta}{L}\right)Y_i \ge 0.0042Y_i \qquad \text{(C-A-7-13)}$$

This is the additional lateral load required in Appendix 7, Section 7.3.2. The minimum value of N_i of $0.0042Y_i$ is based on the assumption of a minimum first-order drift ratio, due to any effects, of $\Delta/L = 1/500$.

APPENDIX 8

APPROXIMATE SECOND-ORDER ANALYSIS

Section C2.1(b) states that a second-order analysis that captures both P-Δ and P-δ effects is required. As an alternative to a more rigorous second-order elastic analysis, the amplification and summation of first-order elastic analysis forces and moments by the approximate procedure in this Appendix is permitted. The main approximation in this technique is that it evaluates P-Δ and P-δ effects separately, through separate multipliers, B_2 and B_1, respectively, considering the influence of P-δ effects on the overall response of the structure (which, in turn, influences P-Δ) only indirectly, through the factor R_M. A more rigorous second-order elastic analysis is recommended for accurate determination of the frame internal forces when B_1 is larger than 1.2 in members that have a significant effect on the response of the overall structure.

This procedure uses a first-order elastic analysis with amplification factors that are applied to the first-order forces and moments so as to obtain an estimate of the second-order forces and moments. In the general case, a member may have first-order load effects not associated with sidesway that are multiplied by a factor B_1, and first-order load effects produced by sidesway that are multiplied by a factor B_2. The factor B_1 estimates the P-δ effects on the nonsway moments in compression members. The factor B_2 estimates the P-Δ effects on the forces and moments in all members. These effects are shown graphically in Figures C-C2.1 and C-A-8.1.

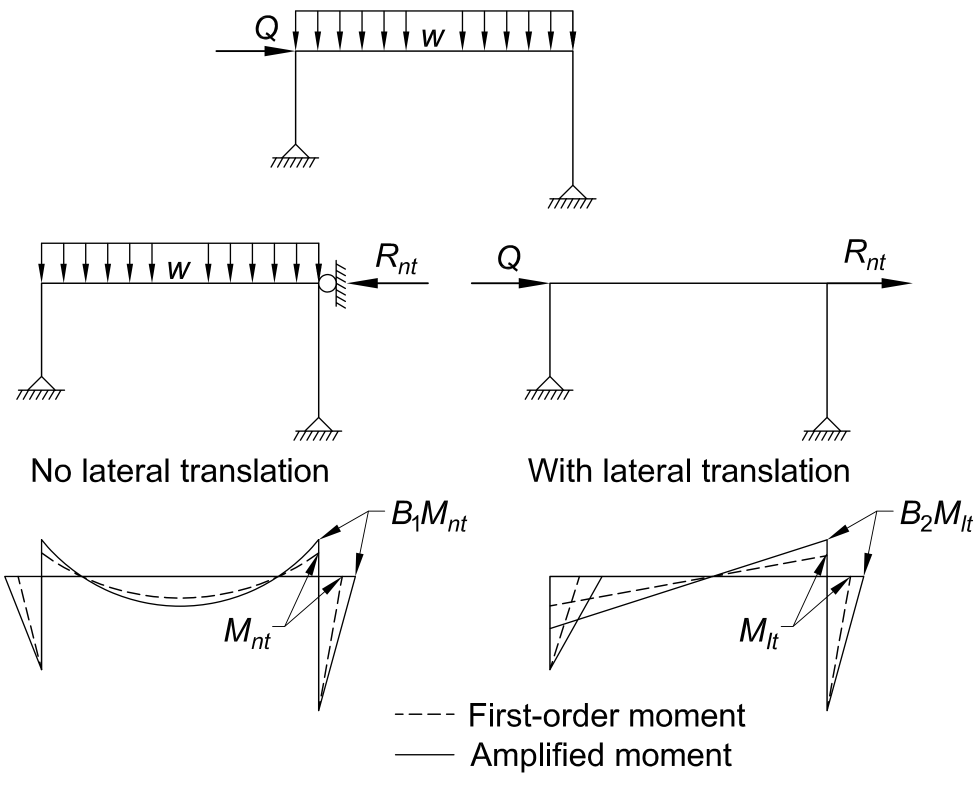


Fig. C-A-8.1. Moment amplification.

The factor B_2 applies only to internal forces associated with sidesway and is calculated for an entire story. In building frames designed to limit Δ_H/L to a predetermined value, the factor B_2 may be found in advance of designing individual members by using the target maximum limit on Δ_H/L within Equation A-8-7. Drift limits may also be set for design of various categories of buildings so that the effect of secondary bending is reduced (ATC, 1978; Kanchanalai and Lu, 1979). However, drift limits alone are not sufficient to allow stability effects to be neglected (LeMessurier, 1977).

In determining B_2 and the second-order effects on the lateral force-resisting system, it is important that Δ_H include not only the interstory displacement in the plane of the lateral force-resisting system, but also any additional displacement in the floor or roof diaphragm or horizontal framing system that may increase the overturning effect of columns attached to and "leaning" against the horizontal system. Either the maximum displacement or a weighted average displacement, weighted in proportion to column load, should be considered.

The current Specification provides only one equation, Equation A-8-7, for determining the elastic bucking strength of a story. This formula is based on the lateral stiffness of the story as determined from a first-order analysis and is applicable to all buildings. The 2005 AISC *Specification* (AISC, 2005) offered a second formula, Equation C2-6a, based on the lateral buckling strength of individual columns, applicable only to buildings in which lateral stiffness is provided entirely by moment frames. That equation is

$$P_{e\,story} = \Sigma \frac{\pi^2 EI}{(K_2 L)^2} \qquad \text{(C-A-8-1)}$$

where

K_2 = effective length factor in the plane of bending, calculated from a sidesway buckling analysis

L = story height, in. (mm)

$P_{e\,story}$ = elastic buckling strength of the story, kips (N)

This equation for the story elastic buckling strength was eliminated from the 2010 AISC *Specification* (AISC, 2010) because of its limited applicability, the difficulty involved in calculating K_2 correctly, and the greater ease of application of the story stiffness-based formula. Additionally, with the deletion of this equation, the symbol ΣP_{e2} was changed to $P_{e\,story}$ because the story buckling strength is not the summation of the strengths of individual columns, as implied by the earlier symbol.

First-order member forces and moments with the structure restrained against sidesway are labeled P_{nt} and M_{nt}; the first-order effects of lateral translation are labeled P_{lt} and M_{lt}. For structures where gravity load causes negligible lateral translation, P_{nt} and M_{nt} are the effects of gravity load and P_{lt} and M_{lt} are the effects of lateral load. In the general case, P_{nt} and M_{nt} are the results of an analysis with the structure restrained against sidesway; P_{lt} and M_{lt} are from an analysis with the lateral reactions from the first analysis (as used to find and P_{nt} and M_{nt}) applied as lateral loads. Algebraic addition of the two sets of forces and moments after application of multipliers B_1 and B_2, as specified in Equations A-8-1 and A-8-2, gives reasonably accurate values of the overall second-order forces and moments.

The B_2 multiplier is applicable to forces and moments, P_{lt} and M_{lt}, in all members, including beams, columns, bracing diagonals and shear walls, that participate in resisting lateral load. P_{lt} and M_{lt} will be zero in members that do not participate in resisting lateral load; hence, B_2 will have no effect on them. The B_1 multiplier is applicable only to compression members.

If B_2 for a particular direction of translation does not vary significantly among the stories of a building, it will be convenient to use the maximum value for all stories, leading to just two B_2 values, one for each direction, for the entire building. Where B_2 does vary significantly between stories, the multiplier for beams between stories should be the larger value.

When first-order end moments in columns are magnified by B_1 and B_2 factors, equilibrium requires that they be balanced by moments in the beams that connect to them (for example, see Figure C-A-8.1). The B_2 multiplier does not cause any difficulty in this regard, since it is applied to all members. The B_1 multiplier, however, is applied only to compression members; the associated second-order internal moments in the connected members can be accounted for by amplifying the moments in those members by the B_1 value of the compression member (using the largest B_1 value if there are two or more compression members at the joint). Alternatively, the difference between the magnified moment (considering B_1 only) and the first-order moment in the compression member(s) at a given joint may be distributed to any other moment-resisting members attached to the compression member (or members) in proportion to the relative stiffness of those members. Minor imbalances may be neglected, based upon engineering judgment. Complex conditions may be treated more expediently with a more rigorous second-order analysis.

In braced frames and moment frames, P_c is governed by the maximum slenderness ratio regardless of the plane of bending, if the member is subject to significant biaxial bending, or the provisions in Section H1.3 are not utilized. Section H1.3 is an alternative approach for checking beam-column strength that provides for the separate checking of beam-column in-plane and out-of-plane stability in members predominantly subject to bending within the plane of the frame. However, P_{e1} expressed by Equation A-8-5 is always calculated using the slenderness ratio in the plane of bending. Thus, when flexure in a beam-column is about the major axis only, two different values of slenderness ratio may be involved in the amplified first-order elastic analysis and strength check calculations.

The factor R_M in Equation A-8-7 accounts for the influence of P-δ effects on sidesway amplification. R_M can be taken as 0.85 as a lower bound value for stories that include moment frames (LeMessurier, 1977); $R_M = 1$ if there are no moment frames in the story. Equation A-8-8 can be used for greater precision between these extreme values.

Second-order internal forces from separate structural analyses cannot normally be combined by superposition since second-order amplification is a nonlinear effect based on the total axial forces within the structure; therefore, a separate analysis must be conducted for each load combination considered in the design. However, in the amplified first-order elastic analysis procedure of Appendix 8, the first-order internal forces, calculated prior to amplification may be superimposed to determine the total first-order internal forces.

***Equivalent Uniform Moment Factor,* $\mathbf{C_m}$*, and Effective Length Factor,* K.** Equations A-8-3 and A-8-4 are used to approximate the maximum second-order moments in compression members with no relative joint translation and no transverse loads between the ends of the member. Figure C-A-8.2 compares the approximation for C_m in Equation A-8-4 to the exact theoretical solution for beam-columns subjected to applied end moments (Chen and Lui, 1987). The approximate and analytical values of C_m are plotted versus the end-moment ratio, M_1/M_2, for several values of P/P_e ($P_e = P_{e1}$ with $K = 1$). The corresponding approximate and analytical solutions are shown in Figure C-A-8.3 for the maximum second-order elastic moment within the member, M_r, versus the axial load level, P/P_e, for several values of the end moment ratio, M_1/M_2.

For beam-columns with transverse loadings, the second-order moment can be approximated for simply supported members with

$$C_m = 1 + \psi\alpha P_r/P_{e1} \qquad \text{(C-A-8-2)}$$

where

M_o = maximum first-order moment within the member due to the transverse loading, kip-in. (N-mm)

α = 1.0 (LRFD) or 1.6 (ASD)

$\psi = \dfrac{\pi^2\delta_o EI}{M_o L^2} - 1$

δ_o = maximum deflection due to transverse loading, in. (mm)

For restrained ends, some limiting cases are given in Table C-A-8.1 together with two cases of simply supported beam-columns (Iwankiw, 1984). These values of C_m are always used with the maximum moment in the member. For the restrained-end cases, the values of B_1

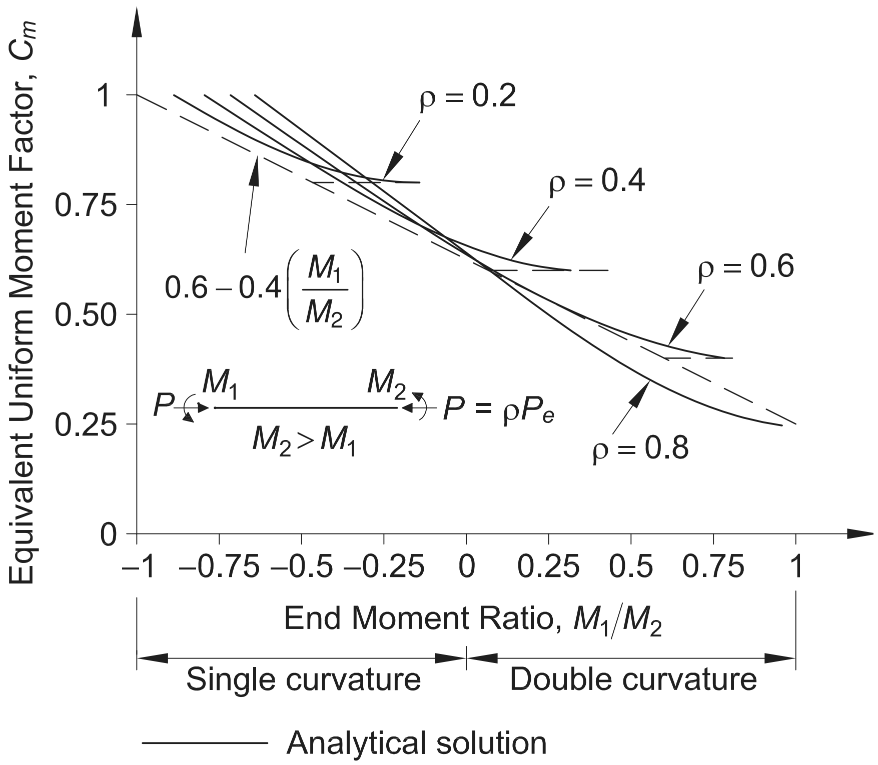


Fig. C-A-8.2. Equivalent uniform moment factor, C_m, *for beam-columns subjected to applied end moments.*

are most accurate if the effective length in the plane of bending, corresponding to the member end conditions, used in calculating P_{e1}, is less than the laterally unbraced length of the member ($K < 1.0$).

In lieu of using the equations given in Table C-A-8.1, the use of $C_m = 1.0$ is conservative for all transversely loaded members. It can be shown that the use of $C_m = 0.85$ for members with restrained ends, as specified in AISC *Specifications* prior to 2005, can sometimes result in a significant underestimation of the internal moments. Therefore, the use of $C_m = 1.0$ is recommended as a simple conservative approximation for all cases involving transversely loaded members.

In approximating a second-order analysis by amplification of the results of a first-order analysis, the effective length, L_{c1}, is used in the determination of the elastic critical buckling load, P_{e1}, for a member. This elastic critical buckling load is then used for calculation of the corresponding amplification factor, B_1.

B_1 is used to estimate the P-δ effects on the nonsway moments, M_{nt}, in compression members. The unbraced length, L_{c1}, is calculated in the plane of bending on the basis of no translation of the ends of the member and is normally set to the laterally unbraced length of the member, unless a smaller value is justified on the basis of analysis.

Since the amplified first-order elastic analysis involves the calculation of elastic buckling loads as a measure of frame and column stiffness, only elastic effective length K-factors are appropriate for this use.

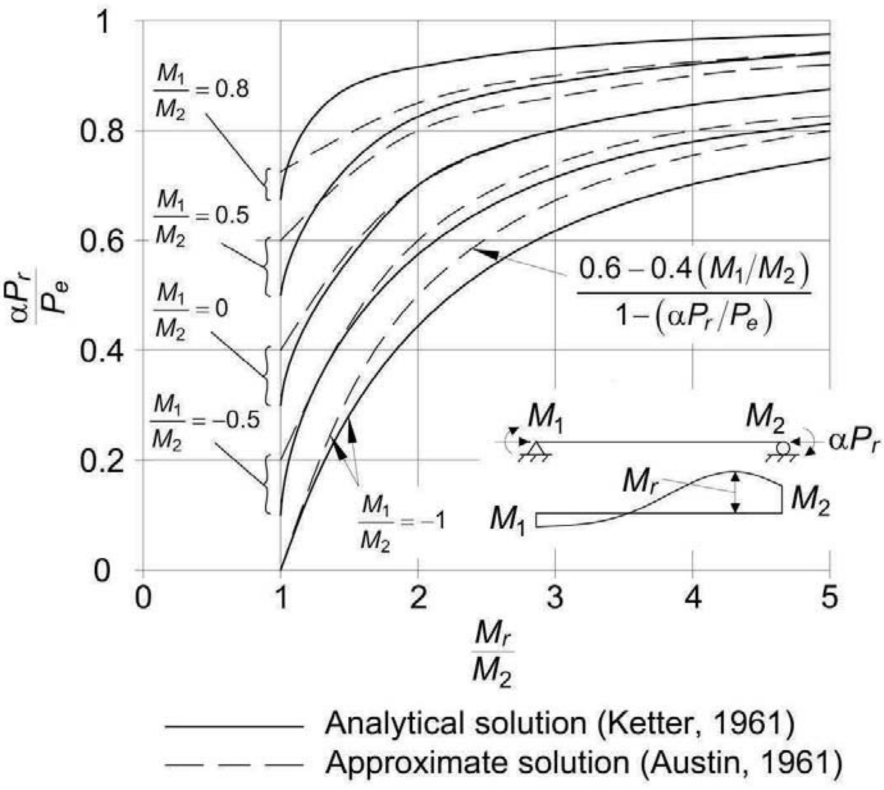
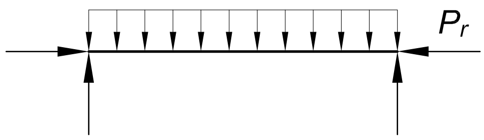
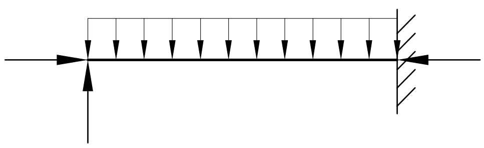
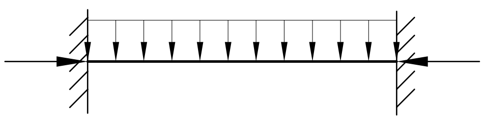
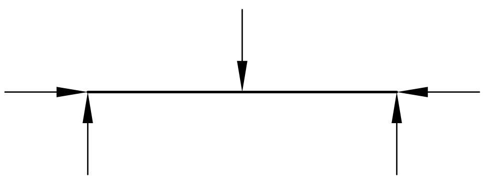
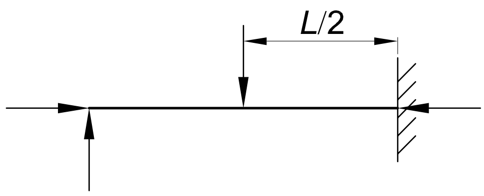
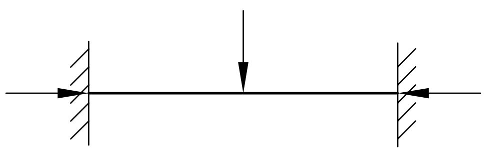


Fig. C-A-8.3. Maximum second-order moments, M_r, *for beam-columns subjected to applied end moments.*

TABLE C-A-8.1
Factor ψ and Equivalent Uniform Moment Factor, C_m

Case	Ψ	C_m
	0	1.0
	−0.4	$1-0.4\dfrac{\alpha P_r}{P_{e1}}$
	−0.4	$1-0.4\dfrac{\alpha P_r}{P_{e1}}$
	−0.2	$1-0.2\dfrac{\alpha P_r}{P_{e1}}$
	−0.3	$1-0.3\dfrac{\alpha P_r}{P_{e1}}$
	−0.2	$1-0.2\dfrac{\alpha P_r}{P_{e1}}$

Summary—Application of Multipliers B_1 and B_2. There is a single B_2 value for each story and each direction of lateral translation of the story, say B_{2X} and B_{2Y} for the two global directions. Multiplier B_{2X} is applicable to all axial and shear forces and moments produced by story translation in the global X-direction. Thus, in the common case where gravity load produces no lateral translation and all X translation is the result of lateral load in the X-direction, B_{2X} is applicable to all axial and shear forces, and moments produced by lateral load in the global X-direction. Similarly, B_{2Y} is applicable in the Y-direction.

Note that B_{2X} and B_{2Y} are associated with global axes X and Y and the direction of story translation or loading, but are completely unrelated to the direction of bending of individual members. Thus, for example, if lateral load or translation in the global X-direction causes moments M_x and M_y about member x- and y-axes in a particular member, B_{2X} must be applied to both M_x and M_y.

There is a separate B_1 value for every member subject to compression and flexure and each direction of bending of the member, say B_{1x} and B_{1y} for the two member axes. Multiplier B_{1x} is applicable to the member x-axis moment, regardless of the load that causes that moment. Similarly, B_{1y} is applicable to the member y-axis moment, regardless of the load that causes that moment.

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Metric Conversion Factors for Common Steel Design Units Used in the AISC Specification

Unit	Multiply	By	to Obtain
length	inch (in.)	25.4	millimeters (mm)
length	foot (ft)	0.304 8	meters (m)
mass	pound-mass (lbm)	0.453 6	kilogram (kg)
stress	ksi	6.895	megapascals (MPa), N/mm^2
moment	kip-in	113 000	N-mm
energy	ft-lbf	1.356	joule (J)
force	kip (1000 lbf)	4 448	newton (N)
force	psf	47.88	pascal (Pa), N/m^2
force	plf	14.59	N/m
temperature	To convert °F to °C: $t_c° = (t_f° - 32)/1.8$		

force in lbf or N = mass × g
where *g*, acceleration due to gravity = 32.2 ft/sec^2 = 9.81 m/sec^2

Specification for Structural Joints Using High-Strength Bolts

August 1, 2014

Supersedes the December 31, 2009 *Specification for Structural Joints Using High-Strength Bolts*.

Prepared by RCSC Committee A.1—Specifications and approved by the Research Council on Structural Connections.

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PREFACE

The purpose of the Research Council on Structural Connections (RCSC) is:

(1) To stimulate and support such investigation as may be deemed necessary and valuable to determine the suitability, strength and behavior of various types of structural connections;
(2) To promote the knowledge of economical and efficient practices relating to such structural connections; and,
(3) To prepare and publish related specifications and such other documents as necessary to achieving its purpose.

The Council membership consists of qualified structural engineers from academic and research institutions, practicing design engineers, suppliers and manufacturers of fastener components, fabricators, erectors and code-writing authorities.

The first Specification approved by the Council, called the *Specification for Assembly of Structural Joints Using High Tensile Steel Bolts*, was published in January 1951. Since that time the Council has published seventeen successive editions. Each was developed through the deliberations and approval of the full Council membership and based upon past successful usage, advances in the state of knowledge and changes in engineering design practice. This edition of the Council's *Specification for Structural Joints Using High-Strength Bolts* continues the tradition of earlier editions. The major changes are:

- Appendix B provisions were incorporated directly into Section 5 of the *Specification.*
- Tolerances for the Turn-of- Nut method were adjusted.
- Glossary definitions for "pretension" were added.
- F1136 coating on F1852 and F2280 bolts was deleted from Table 2.1 in recognition that this coating has not been approved by ASTM for use on TC bolts.
- Slip critical equations in Section 5.4 were updated for consistent with the AISC Specification.
- Clarification language was provided for approval requirements for hole sizes other than standard holes.
- The snug-tightened joint definition was redefined back to the 2004 definition due to issues regarding turn-of-nut tension requirements.

In addition, typographical changes have been made throughout this Specification.

By the Research Council on Structural Connections,

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SYMBOLS

The following symbols are used in this Specification.

A_b Cross-sectional area based upon the nominal diameter of bolt, in.^{2}

D Slip probability factor as described in Section 5.4.2

D_u Multiplier that reflects the ratio of the mean installed bolt pretension to the specified minimum bolt pretension, T_m, as described in Section 5.4.1

F_n Nominal strength (per unit area), ksi

F_u Specified minimum tensile strength (per unit area), ksi

I Moment of inertia of the built-up member about the axis of buckling (see the Commentary to Section 5.4), in.^{4}

L Total length of the built-up member (see the Commentary to Section 5.4), in.

L_s Length of a connection measured between extreme bolt hole centers parallel to the line of force (see Table 5.1), in.

L_c Clear distance, in the direction of load, between the edge of the hole and the edge of the adjacent hole or the edge of the material, in.

N_b Number of bolts in the joint

P_u Required strength in compression, kips; Axial compressive force in the built-up member (see the Commentary to Section 5.4), kips

Q First moment of area of one component about the axis of buckling of the built-up member (see the Commentary to Section 5.4), in.^{3}

R_n Nominal strength, kips

T Applied service load in tension, kips

T_m Specified minimum bolt pretension (for pretensioned joints as specified in Table 8.1), kips

T_u Required strength in tension (factored tensile load), kips

V_u	Required strength in shear (factored shear load), kips
d_b	Nominal diameter of bolt, in.
t	Thickness of the connected material, in.
t'	Total thickness of fillers or shims (see Section 5.1), in.
k_s	Slip coefficient for an individual specimen determined in accordance with Appendix A
ϕ	Resistance factor
ϕR_n	Design strength, kips
μ	Mean slip coefficient

GLOSSARY

The following terms are used in this Specification. Where used, they are italicized to alert the user that the term is defined in this Glossary.

Allowable Strength. Nominal strength, R_n, divided by the safety factor, Ω.

Available Strength. Design Strength or Allowable Strength, as appropriate.

ASD Load. Load due to a load combination in the applicable building code intended for allowable strength design (allowable stress design).

Coated Faying Surface. A *faying surface* that has been primed, primed and painted or protected against corrosion, except by hot-dip galvanizing.

Connection. An assembly of one or more *joints* that is used to transmit forces between two or more members.

Contractor. The party or parties responsible to provide, prepare and assemble the fastener components and connected parts described in this Specification.

Design Strength. ϕR_n, the resistance provided by an element or *connection*; the product of the *nominal strength*, R_n, and the resistance factor ϕ.

Engineer of Record. The party responsible for the design of the structure and for the approvals that are required in this Specification (see Section 1.6 and the corresponding Commentary).

Fastener Assembly. An assembly of fastener components that is supplied, tested and installed as a unit.

Faying Surface. The plane of contact between two plies of a *joint*.

Firm Contact. The condition that exists on a *faying surface* when the plies are solidly seated against each other, but not necessarily in continuous contact.

Galvanized Faying Surface. A *faying surface* that has been hot-dip galvanized.

Grip. The total thickness of the plies of a *joint* through which the bolt passes, exclusive of washers or direct-tension indicators.

Guide. The *Guide to Design Criteria for Bolted and Riveted Joints*, 2nd Edition (Kulak et al., 1987).

High-Strength Bolt. An ASTM A325 or A490 bolt, an ASTM F1852 or F2280 twist-off-type tension-control bolt or an alternative-design fastener that meets the requirements in Section 2.8.

Inspector. The party responsible to ensure that the *contractor* has satisfied the provisions of this Specification in the work.

Joint. A bolted assembly with or without collateral materials that is used to join two structural elements.

Lot. In this Specification, the term *lot* shall be taken as that given in the ASTM Standard as follows:

Product	ASTM Standard	See Lot Definition in ASTM Section
Conventional bolts	A325	9.4
	A490	11.4
Twist-off-type tension-control bolt assemblies	F1852	13.4
	F2280	3.1.1
Nuts	A563	9.2
Washers	F436	9.2
Compressible-washer-type direct tension indicators	F959	10.2.2

LRFD Load. Load due to a load combination in the applicable building code intended for strength design (load and resistance factor design).

Manufacturer. The party or parties that produce the components of the *fastener assembly*.

Mean Slip Coefficient. μ, the ratio of the frictional shear load at the *faying surface* to the total normal force when slip occurs.

Nominal Strength. The capacity of a structure or component to resist the effects of loads, as determined by computations using the specified material strengths and dimensions and equations derived from accepted principles of structural mechanics or by field tests or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions.

Pretension (noun). A level of tension achieved in a fastener assembly through its installation, as required for *pretensioned* and *slip-critical joints*.

Pretension (verb). The act of tightening a fastener assembly to a level required for *pretensioned* and *slip-critical joints*.

Pretensioned Joint. A *joint* that transmits shear and/or tensile loads in which the bolts have been installed in accordance with Section 8.2 to provide a pretension in the installed bolt.

Protected Storage. The continuous protection of fastener components in closed containers in a protected shelter as described in the Commentary to Section 2.2.

Prying Action. Lever action that exists in *connections* in which the line of application of the applied load is eccentric to the axis of the bolt, causing deformation of the fitting and an amplification of the axial tension in the bolt.

Required Strength. The load effect acting on an element or *connection* determined by structural analysis from the factored loads using the most appropriate critical load combination.

Routine Observation. Periodic monitoring of the work in progress.

Shear/Bearing Joint. A *snug-tightened joint* or *pretensioned joint* with bolts that transmit shear loads and for which the design criteria are based upon the shear strength of the bolts and the bearing strength of the connected materials.

Slip-Critical Joint. A *joint* that transmits shear loads or shear loads in combination with tensile loads in which the bolts have been installed in accordance with Section 8.2 to provide a pretension in the installed bolt (clamping force on the *faying surfaces*), and with *faying surfaces* that have been prepared to provide a calculable resistance against slip.

Snug-Tightened Joint. A *joint* in which the bolts have been installed in accordance with Section 8.1. The snug tightened condition is the tightness that is attained with a few impacts of an impact wrench or the full effort of an ironworker using an ordinary spud wrench to bring the plies into *firm contact.*

Start of Work. Any time prior to the installation of *high-strength bolts* in structural *connections* in accordance with Section 8.

Sufficient Thread Engagement. Having the end of the bolt extending beyond or at least flush with the outer face of the nut; a condition that develops the strength of the bolt.

Supplier. The party that sells the fastener components to the party that will install them in the work.

Tension Calibrator. A calibrated tension-indicating device that is used to verify the acceptability of the pretensioning method when a *pretensioned joint* or *slip-critical joint* is specified.

Uncoated Faying Surface. A *faying surface* that has neither been primed, painted, nor galvanized and is free of loose scale, dirt and other foreign material.

SPECIFICATION FOR STRUCTURAL JOINTS USING HIGH-STRENGTH BOLTS

SECTION 1. GENERAL REQUIREMENTS

1.1. Scope

This Specification covers the design of bolted *joints* and the installation and inspection of the assemblies of fastener components listed in Section 1.5, the use of alternative-design fasteners as permitted in Section 2.8 and alternative washer-type indicating devices as permitted in Section 2.6.2, in structural steel *joints*. This Specification relates only to those aspects of the connected materials that bear upon the performance of the fastener components. The Symbols, Glossary and Appendices are a part of this Specification.

Commentary:

This Specification deals principally with two strength grades of *high-strength bolts*, ASTM A325 and A490, and with their design, installation and inspection in structural steel *joints*. Equivalent fasteners, however, such as ASTM F1852 (equivalent to ASTM A325) and F2280 (equivalent to ASTM A490) twist-off-type tension-control bolt assemblies, are also covered. These provisions may not be relied upon for high-strength fasteners of other chemical composition, mechanical properties, or size. These provisions do not apply when material other than steel is included in the *grip*; nor are they applicable to anchor rods.

This Specification relates only to the performance of fasteners in structural steel *joints* and those few aspects of the connected material that affect this performance. Many other aspects of *connection* design and fabrication are of equal importance and must not be overlooked. For more general information on design and issues relating to *high-strength bolting* and the connected material, refer to current steel design textbooks and the *Guide to Design Criteria for Bolted and Riveted Joints*, 2nd Edition (Kulak et al., 1987).

1.2. Loads, Load Factors and Load Combinations

The design and construction of the structure shall conform to either an applicable load and resistance factor design specification for steel structures or to an applicable allowable strength design specification for steel structures. Because factored load combinations account for the reduced probabilities of maximum loads acting concurrently, the *design strengths* given in this Specification shall not be increased.

1.3 Design for Strength Using Load and Resistance Factor Design (LRFD)

Design according to the provisions for load and resistance factor design (LRFD) satisfies the requirements of this Specification when the design strength of each structural component or assemblage equals or exceeds the required strength determined on the basis of the LRFD load combinations.

Design shall be performed in accordance with Equation 1.1

$$R_u \leq \phi R_n \qquad \text{(Equation 1.1)}$$

Where

R_u = required strength using LRFD load combinations
R_n = *nominal strength*
ϕ = *resistance factor*
ϕR_n = design strength

1.4 Design for Strength Using Allowable Strength Design (ASD)

Design according to the provisions for allowable strength design (ASD) satisfies the requirements of this Specification when the design strength of each structural component or assemblage equals or exceeds the required strength determined on the basis of the ASD load combinations.

Design shall be performed in accordance with Equation 1.2

$$R_a \leq R_n / \Omega \qquad \text{(Equation 1.2)}$$

Where

R_a = required strength using ASD load combinations
R_n = *nominal strength*
Ω = *safety factor*
R_n / Ω = allowable strength

Commentary:
This Specification is written in a dual format covering both load and resistance factor design (LRFD) and allowable strength design (ASD). Both approaches provide a method of proportioning structural components such that no applicable limit state is exceeded when the structure is subject to all appropriate load combinations. When a structure or structural component ceases to fulfill the intended purpose in some way, it is said to have exceeded a limit state. Strength limit states concern maximum load-carrying capability, and are related to safety. Serviceability limit states are usually related to performance under normal service conditions, and usually are not related to strength or safety. The term "resistance" includes both strength limit states and serviceability limit states.

Although loads, load factors and load combinations are not explicitly specified in this Specification, the safety and resistance factors herein are based upon the loads, load factors and load combinations specified in ASCE 7. When the design is governed by other load criteria, the safety and resistance factors specified herein should be adjusted as appropriate.

1.5 Referenced Standards and Specifications

The following standards and specifications are referenced herein:

American Institute of Steel Construction

Specification for Structural Steel Buildings, June 22, 2010

American National Standards Institute

ANSI/ASME B18.2.6-10 *Fasteners for Use in Structural Applications*

ASTM International

ASTM A123-13 *Standard Specification for Zinc* (*Hot-Dip Galvanized*) *Coatings on Iron and Steel Products*

ASTM A194-14 *Specification for Carbon and Alloy Steel Nuts for Bolts for High Pressure or High-Temperature Service, or Both*

ASTM A325-14 *Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength*

ASTM A490-12 *Standard Specification for Heat-Treated Steel Structural Bolts, 150 ksi Minimum Tensile Strength*

ASTM A563-07a(2014) *Standard Specification for Carbon and Alloy Steel Nuts*

ASTM B695-04(2009) *Standard Specification for Coatings of Zinc Mechanically Deposited on Iron and Steel*

ASTM F436-11 *Standard Specification for Hardened Steel Washers*

ASTM F959-13 *Standard Specification for Compressible-Washer-Type Direct Tension Indicators for Use with Structural Fasteners*

ASTM F1136-11 *Standard Specification for Zinc/Aluminum Corrosion Protective Coatings for Fasteners*

ASTM F1852-14 *Standard Specification for "Twist Off" Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength*

ASTM F2280-14 *Standard Specification for "Twist Off" Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 150 ksi Minimum Tensile Strength*

ASTM F2329-13 *Standard Specification for Zinc Coating, Hot-Dip, Requirements for Application to Carbon and Alloy Steel Bolts, Screws, Washers, Nuts, and Special Threaded Fasteners*

American Society of Civil Engineers

ASCE 7-10 *Minimum Design Loads for Buildings and Other Structures*

IFI: Industrial Fastener Institute

IFI 144 (2000) *Test Evaluation Procedures for Coating Qualification Intended for Use on High-Strength Structural Bolts*

SSPC: The Society for Protective Coatings
SSPC-PA2 (5/2012) *Measurement of Dry Coating Thickness With Magnetic Gages*

Commentary:
Familiarity with the referenced AISC, ASCE, ASME, ASTM and SSPC specification requirements is necessary for the proper application of this Specification. The discussion of referenced specifications in this Commentary is limited to only a few frequently overlooked or misunderstood items.

1.6 Drawing Information

The *Engineer of Record* shall specify the following information in the contract documents:

(1) The ASTM designation and type (Section 2) of bolt to be used;
(2) The *joint* type (Section 4) and;
(3) The required class of slip resistance if *slip-critical joints* are specified (Section 4).

Commentary:
A summary of the information that the *Engineer of Record* is required to provide in the contract documents is provided in this Section. The parenthetical reference after each listed item indicates the location of the actual requirement in this Specification. In addition, the approval of the *Engineer of Record* is required in this Specification in the following cases:

(1) For the reuse of non-galvanized ASTM A325 bolts (Section 2.3.3);
(2) For the use of alternative washer-type indicating devices that differ from those that meet the requirements of ASTM F959, including the corresponding installation and inspection requirements that are provided by the *manufacturer* (Section 2.6.2);
(3) For the use of alternative-design fasteners, including the corresponding installation and inspection requirements that are provided by the *manufacturer* (Section 2.8);
(4) For the use of faying-surface coatings in *slip-critical joints* that provide a *mean slip coefficient* determined per Appendix A, but differing from Class A or Class B (Section 3.2.2(b));
(5) For the use of thermal cutting of bolt holes produced free hand or for use in cyclically loaded joints(Section 3.3);
(6) For the use of oversized (Section 3.3.2), short-slotted (Section 3.3.3) or long slotted holes (Section 3.3.4) in lieu of standard holes;
(7) For the use of a value of D_u other than 1.13 (Section 5.4).

SECTION 2. FASTENER COMPONENTS

2.1. Manufacturer Certification of Fastener Components

Manufacturer certifications documenting conformance to the applicable specifications required in Sections 2.3 through 2.8 for all fastener components used in the *fastener assemblies* shall be available to the *Engineer of Record* and *inspector* prior to assembly or erection of structural steel.

Commentary:
Certification by the *manufacturer* or *supplier* of *high-strength bolts*, nuts, washers and other components of the *fastener assembly* is required to ensure that the components to be used are identifiable and meet the requirements of the applicable ASTM Specifications.

2.2. Storage of Fastener Components

Fastener components shall be protected from dirt and moisture in closed containers at the site of installation. Only as many fastener components as are anticipated to be installed during the work shift shall be taken from *protected storage*. Fastener components that are not incorporated into the work shall be returned to *protected storage* at the end of the work shift. Fastener components shall not be cleaned or modified from the as-delivered condition.

Fastener components that accumulate rust or dirt shall not be incorporated into the work unless they are requalified as specified in Section 7. ASTM F1852 and F2280 twist-off-type tension-control bolt assemblies and alternative-design fasteners that meet the requirements in Section 2.8 shall not be relubricated, except by the *manufacturer*.

Commentary:
Protected storage requirements are specified for *high-strength bolts*, nuts, washers and other fastener components with the intent that the condition of the components be maintained as nearly as possible to the as-manufactured condition until they are installed in the work. This involves:

(1) The storage of the fastener components in closed containers to protect from dirt and corrosion;
(2) The storage of the closed containers in a protected shelter;
(3) The removal of fastener components from *protected storage* only as necessary; and,
(4) The prompt return of unused fastener components to *protected storage*.

To facilitate manufacture, prevent corrosion and facilitate installation, the *manufacturer* may apply various coatings and oils that are present in the as-manufactured condition. As such, the condition of supplied fastener components or the *fastener assembly* should not be altered to make them unsuitable for pretensioned installation.

If fastener components become dirty, rusty, or otherwise have their as-received condition altered, they may be unsuitable for pretensioned installation. It is also possible that a *fastener assembly* may not pass the pre-installation verification requirements of Section 7. Except for ASTM F1852 and F2280 twist-off-type tension-control bolt assemblies (Section 2.7) and some alternative-design fasteners (Section 2.8), fastener components can be cleaned and lubricated by the fabricator or the erector. Because the acceptability of their installation is dependent upon specific lubrication, ASTM F1852 and F2280 twist-off-type tension-control bolt assemblies and some alternative-design fasteners are suitable only if the *manufacturer* lubricates them.

2.3. Heavy-Hex Structural Bolts

2.3.1. Specifications: Heavy-hex structural bolts shall meet the requirements of ASTM A325 or ASTM A490. The *Engineer of Record* shall specify the ASTM designation and type of bolt (see Table 2.1) to be used.

2.3.2. Geometry: Heavy-hex structural bolt dimensions shall meet the requirements of ANSI/ASME B18.2.6. The bolt length used shall be such that the end of the bolt extends beyond or is at least flush with the outer face of the nut when properly installed.

2.3.3 Reuse: ASTM A490 bolts, ASTM F1852 and F2280 twist-off-type tension-control bolt assemblies, and galvanized or Zn/Al Inorganic coated ASTM A325 bolts shall not be reused. When approved by the *Engineer of Record*, black ASTM A325 bolts are permitted to be reused. Touching up or re-tightening bolts that may have been loosened by the installation of adjacent bolts shall not be considered to be a reuse.

Commentary:
ASTM A325 and ASTM A490 currently provide for two types (according to metallurgical classification) of *high-strength bolts*, supplied in diameters from ½ in. to 1½ in. inclusive. Type 1 covers medium carbon steel for ASTM A325 bolts and alloy steel for ASTM A490 bolts. Type 3 covers *high-strength bolts* that have improved atmospheric corrosion resistance and weathering characteristics. (Reference to Type 2 ASTM A325 and Type 2 A490 bolts, which appeared in previous editions of this Specification, has been removed following the removal of similar reference within the ASTM A325 and A490 Specifications). When the bolt type is not specified, either Type 1 or Type 3 may be supplied at the option of the *manufacturer*. Note that ASTM F1852 and ASTM F2280 twist-off-type tension-control bolt assemblies may be manufactured with a button head or hexagonal head; other requirements for these *fastener assemblies* are found in Section 2.7.

Table 2.1. Acceptable ASTM A563 Nut Grade and Finish and ASTM F436 Washer Type and Finish

ASTM Desig.	Bolt Type	Bolt Finish [d]	ASTM A563 Nut Grade and Finish [d]	ASTM F436 Washer Type and Finish [a,d]
A325	1	**Plain (uncoated)**	C, C3, D, DH [c] and DH3; plain	1; plain
		Galvanized	DH [c]; galvanized and lubricated	1; galvanized
		Zn/Al Inorganic, per ASTM F1136 Grade 3	DH [c]; Zn/Al Inorganic, per ASTM F1136 Grade 5	1; Zn/Al Inorganic, per ASTM F1136 Grade 3
	3	**Plain**	C3 and DH3; plain	3; plain
F1852	1	**Plain (uncoated)**	C, C3, DH [c] and DH3; plain	1; plain [b]
		Mechanically Galvanized	DH [c]; mechanically galvanized and lubricated	1; mechanically galvanized [b]
	3	**Plain**	C3 and DH3; plain	3; plain [b]
A490	1	**Plain**	DH[c] and DH3; plain	1; plain
		Zn/Al Inorganic, per ASTM F1136 Grade 3	DH [c]; Zn/Al Inorganic, per ASTM F1136 Grade 5	1; Zn/Al Inorganic, per ASTM F1136 Grade 3
	3	**Plain**	DH3; plain	3; plain
F2280	1	**Plain**	DH [c] and DH3; plain	1; plain [b]
	3	**Plain**	DH3; plain	3; plain [b]

[a] Applicable only if washer is required in Section 6.

[b] Required in all cases under nut per Section 6.

[c] The substitution of ASTM A194 grade 2H nuts in place of ASTM A563 grade DH nuts is permitted.

[d] “Galvanized” as used in this table refers to hot-dip galvanizing in accordance with ASTM F2329 or mechanical galvanizing in accordance with ASTM B695.

[e] "Zn/Al Inorganic" as used in this table refers to application of a Zn/Al Corrosion Protective Coating in accordance with ASTM F1136 which has met all the requirements of IFI-144.

Regular heavy-hex structural bolts and twist-off-type tension-control bolt assemblies are required by ASTM Specifications to be distinctively marked. Certain markings are mandatory. In addition to the mandatory markings, the *manufacturer* may apply additional distinguishing markings. The mandatory and sample optional markings are illustrated in Figure C-2.1.

ASTM Specifications permit the galvanizing of ASTM A325 bolts but not ASTM A490 bolts. Similarly, the application of zinc to ASTM A490 bolts by metallizing or mechanical coating is not permitted because the effect of mechanical galvanizing on embrittlement and delayed cracking of ASTM A490 bolts has not been fully investigated to date.

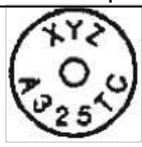
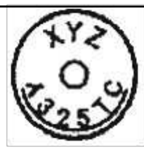
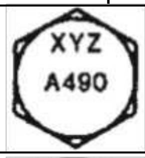
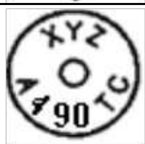
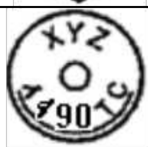
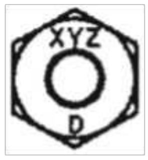
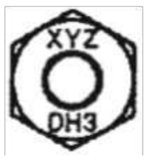
<table>
<tr><th>Bolt/Nut</th><th colspan="3">Type 1</th><th colspan="2">Type 3</th></tr>
<tr><td>ASTM A325 bolt</td><td colspan="3">
Three radial lines 120° apart are optional</td><td colspan="2"></td></tr>
<tr><td>ASTM F1852 bolt</td><td colspan="3">
Three radial lines 120° apart are optional</td><td colspan="2"></td></tr>
<tr><td>ASTM A490 bolt</td><td colspan="3"></td><td colspan="2"></td></tr>
<tr><td>ASTM F2280 bolt</td><td colspan="3"></td><td colspan="2"></td></tr>
<tr><td rowspan="2">ASTM A563 nut</td><td>
Arcs indicate Grade C</td><td colspan="2">
Arcs with “3” indicate Grade C3</td><td colspan="2">
Grade D</td></tr>
<tr><td colspan="2">
Grade DH</td><td colspan="3">
Grade DH3</td></tr>
<tr><td colspan="6">1. XYZ represents the manufacturer’s identification mark.
2. ASTM F1852 and ASTM F2280 twist-off-type tension-control bolt assemblies are also produced with a heavy-hex head that has similar markings.</td></tr>
</table>

Figure C-2.1. Required marks for acceptable bolt and nut assemblies.

An extensive investigation conducted in accordance with IFI-144 was completed in 2006 and presented to the ASTM F16 Committee on Fasteners (F16 Research Report RR: F16-1001). The investigation demonstrated that Zn/Al Inorganic Coating, when applied per ASTM F1136 Grade 3 to ASTM A490 bolts, does not cause delayed cracking by internal hydrogen embrittlement, nor does it accelerate environmental hydrogen embrittlement by cathodic hydrogen absorption. It was determined that this is an acceptable finish to be used on Type 1 ASTM A325 and A490 bolts.

Although these bolts are typically not used in this manner, prior to embedding bolts coated with Zn/Al Inorganic Coating in concrete, it should be confirmed that there is no negative impact (to the bolt or the concrete) caused by the reaction of the intended concrete mix and the aluminum in the coating.

Galvanized *high-strength bolts* and nuts must be considered as a manufactured *fastener assembly*. Insofar as the hot-dip galvanized bolt and nut assembly is concerned, four principal factors must be considered so that the provisions of this Specification are understood and properly applied. These are:

(1) The effect of the hot-dip galvanizing process on the mechanical properties of high-strength steels;
(2) The effect of over-tapping for hot-dip galvanized coatings on the nut stripping strength;
(3) The effect of galvanizing and lubrication on the torque required for pretensioning; and,
(4) Shipping requirements.

Birkemoe and Herrschaft (1970) showed that, in the as-galvanized condition, galvanizing increases the friction between the bolt and nut threads as well as the variability of the torque-induced pretension. A lower required torque and more consistent results are obtained if the nuts are lubricated. Thus, it is required in ASTM A325 that a galvanized bolt and lubricated galvanized or Zn/Al Inorganic coated nut be assembled in a steel *joint* with an equivalently coated washer and tested by the *supplier* prior to shipment. This testing must show that the galvanized or Zn/Al Inorganic coated nut with the lubricant provided may be rotated from the snug-tight condition well in excess of the rotation required for pretensioned installation without stripping. This requirement applies to hot-dip galvanized, mechanically galvanized, and Zn/Al Inorganic coated fasteners. The above requirements clearly indicate that:

(1) Galvanized and Zn/Al Inorganic coated *high-strength bolts* and nuts must be treated as a *fastener assembly*;
(2) The *supplier* must supply nuts that have been lubricated and tested with the supplied *high-strength bolts*;
(3) Nuts and *high-strength bolts* must be shipped together in the same shipping container; and,

(4) The purchase of galvanized *high-strength bolts* and galvanized nuts from separate *suppliers* is not in accordance with the intent of the ASTM Specifications because the control of over-tapping, the testing and application of lubricant and the *supplier* responsibility for the performance of the assembly would clearly not have been provided as required.

Because some of the lubricants used to meet the requirements of ASTM Specifications are water soluble, it is advisable that galvanized *high-strength bolts* and nuts be shipped and stored in plastic bags or in sealed wood or metal containers. Containers of fasteners with hot-wax-type lubricants should not be subjected to heat that would cause depletion or change in the properties of the lubricant.

Both the hot-dip galvanizing process (ASTM F2329) and the mechanical galvanizing process (ASTM B695) are recognized in ASTM A325. The effects of the two processes upon the performance characteristics and requirements for proper installation are distinctly different. Therefore, distinction between the two must be noted in the comments that follow. In accordance with ASTM A325, all threaded components of the *fastener assembly* must be galvanized by the same process and the *supplier's* option is limited to one process per item with no mixed processes in a *lot*. Mixing *high-strength bolts* that are galvanized by one process with nuts that are galvanized by the other may result in an unworkable assembly.

Steels in the 200 ksi and higher tensile-strength range are subject to embrittlement if hydrogen is permitted to remain in the steel and the steel is subjected to high tensile stress. The minimum tensile strength of ASTM A325 bolts is 105 ksi or 120 ksi, depending upon the diameter, and maximum hardness limits result in production tensile strengths well below the critical range. The maximum tensile strength for ASTM A490 bolts was set at 170 ksi to provide a little more than a ten-percent margin below 200 ksi. However, because *manufacturers* must target their production slightly higher than the required minimum, ASTM A490 bolts close to the critical range of tensile strength must be anticipated. For black *high-strength bolts*, this is not a cause for concern. However, if the bolt is hot-dip galvanized, delayed brittle fracture in service is a concern because of the possibility of the introduction of hydrogen during the pickling operation of the hot-dip galvanizing process and the subsequent "sealing-in" of the hydrogen by the zinc coating. There also exists the possibility of cathodic hydrogen absorption arising from the corrosion process in certain aggressive environments.

ASTM A325 and A490 bolts are manufactured to dimensions as specified in ANSI/ASME B18.2.6. The basic dimensions, as defined in Figure C-2.2, are shown in Table C-2.1.

Table C-2.1. Bolt and Nut Dimensions

Nominal Bolt Diameter, d_b, in.	Heavy-Hex Bolt Dimensions, in.			Heavy-Hex Nut Dims., in.	
	Width across flats, F	Height, H_1	Thread Length, T	Width across flats, W	Height, H_2
½	⅞	5⁄16	1	⅞	31⁄64
⅝	1 1⁄16	25⁄64	1¼	1 1⁄16	39⁄64
¾	1¼	15⁄32	1⅜	1¼	47⁄64
⅞	1 7⁄16	35⁄64	1½	1 7⁄16	55⁄64
1	1⅝	39⁄64	1¾	1⅝	63⁄64
1⅛	1 13⁄16	11⁄16	2	1 13⁄16	1 7⁄64
1¼	2	25⁄32	2	2	1 7⁄32
1⅜	2 13⁄16	27⁄32	2¼	2 3⁄16	1 11⁄32
1½	2⅜	15⁄16	2¼	2⅜	1 15⁄32

The principal geometric features of heavy-hex structural bolts that distinguish them from bolts for general application are the size of the head and the unthreaded body length. The head of the heavy-hex structural bolt is specified to be the same size as a heavy-hex nut of the same nominal diameter so that the ironworker may use the same wrench or socket either on the bolt head and/or on the nut. With the specific exception of fully threaded ASTM A325T bolts as discussed below, heavy-hex structural bolts have shorter threaded lengths than bolts for general applications. By making the body length of the bolt the control dimension, it has been possible to exclude the thread from all shear planes when desirable, except for the case of thin outside parts adjacent to the nut.

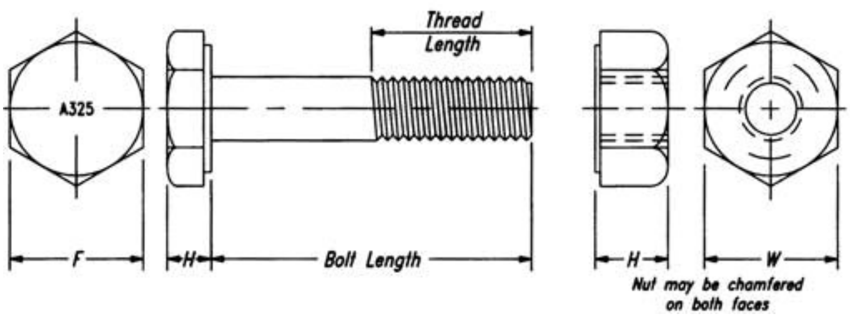


Figure C-2.2. Heavy-hex structural bolt and heavy-hex nut.

The shorter threaded lengths provided with heavy-hex structural bolts tend to minimize the threaded portion of the bolt within the *grip*. Accordingly, care must also be exercised to provide adequate threaded length between the nut and the bolt head to enable appropriate installation without jamming the nut on the thread run-out.

Depending upon the increments of supplied bolt lengths, the full thread may extend into the *grip* for an assembly without washers by as much as ⅜ in. for ½, ⅝, ¾, ⅞, 1¼, and 1½ in. diameter *high-strength bolts* and as much as ½ in. for 1, 1⅛, and 1⅜ in. diameter *high-strength bolts*. When the thickness of the ply closest to the nut is less than the ⅜ in. or ½ in. dimensions given above, it may still be possible to exclude the threads from the shear plane, when required, depending upon the specific combination of bolt length, *grip* and number of washers used under the nut (Carter, 1996). If necessary, the next increment of bolt length can be specified with ASTM F436 washers in sufficient number to both exclude the threads from the shear plane and ensure that the assembly can be installed with adequate threads included in the *grip* for proper installation.

At maximum accumulation of tolerances from all components in the *fastener assembly*, the thread run-out will cross the shear plane for the critical combination of bolt length and *grip* used to select the foregoing rules of thumb for ply thickness required to exclude the threads. This condition is not of concern, however, for two reasons. First, it is too unlikely that all component tolerances will accumulate at their maximum values to warrant consideration. Second, even if the maximum accumulation were to occur, the small reduction in shear strength due to the presence of the thread run-out (not a full thread) would be negligible.

There is an exception to the foregoing thread length requirements for ASTM A325 bolts, but not for ASTM A490 bolts, ASTM F1852 or ASTM F2280 twist-off-type tension-control bolt assemblies. Supplementary requirements in ASTM A325 permit the purchaser to specify a bolt that is threaded for the full length of the shank, when the bolt length is equal to or less than four times the nominal diameter. This exception is provided to increase economy through simplified ordering and inventory control in the fabrication and erection of some structures. It is particularly useful in those structures in which the strength of the *connection* is dependent upon the bearing strength of relatively thin connected material rather than the shear strength of the bolt, whether with threads in the shear plane or not. As required in ASTM A325, *high-strength bolts* ordered to such supplementary requirements must be marked with the symbol A325T.

To determine the required bolt length, the value shown in Table C-2.2 should be added to the *grip* (i.e., the total thickness of all connected material, exclusive of washers). For each ASTM F436 washer that is used, add 5⁄32 in.; for each beveled washer, add 15⁄16 in. The tabulated values provide appropriate allowances for manufacturing tolerances and also provide *sufficient thread*

Table C-2.2. Bolt Length Selection Increment

Nominal Bolt Diameter, d_b, in.	To Determine the Required Bolt Length, Add to Grip, in.
½	11⁄16
⅝	⅞
¾	1
⅞	1⅛
1	1¼
1⅛	1½
1¼	1⅝
1⅜	1¾
1½	1⅞

engagement with an installed heavy-hex nut. The length determined by the use of Table C-2.2 should be adjusted to the nearest ¼-in. length increment (½-in. length increment for lengths exceeding 6 in.). A more extensive table for bolt length selection based upon these rules is available (Carter, 1996).

Pretensioned installation involves the inelastic elongation of the portion of the threaded length between the nut and the thread run-out. ASTM A490 bolts and galvanized ASTM A325 bolts possess sufficient ductility to undergo one pretensioned installation, but are not consistently ductile enough to undergo a second pretensioned installation. Black ASTM A325 bolts, however, possess sufficient ductility to undergo more than one pretensioned installation as suggested in the *Guide* (Kulak et al., 1987). As a simple rule of thumb, a black ASTM A325 bolt is suitable for reuse if the nut can be run up the threads by hand.

2.4. Heavy-Hex Nuts

2.4.1. Specifications: Heavy-hex nuts shall meet the requirements of ASTM A563. The grade and finish of such nuts shall be as given in Table 2.1.

2.4.2. Geometry: Heavy-hex nut dimensions shall meet the requirements of ANSI/ASME B18.2.6.

Commentary:

Heavy-hex nuts are required by ASTM Specifications to be distinctively marked. Certain markings are mandatory. In addition to the mandatory markings, the *manufacturer* may apply additional distinguishing markings. The mandatory markings and sample optional markings are illustrated in Figure C-2.1.

Hot-dip galvanizing affects the stripping strength of the bolt-nut assembly because, to accommodate the relatively thick zinc coatings of non-uniform thickness on bolt threads, it is usual practice to hot-dip galvanize the blank nut and then to tap the nut over-size. This results in a reduction of thread engagement with a consequent reduction of the stripping strength. Only the stronger hardened nuts have adequate strength to meet ASTM thread strength requirements after over-tapping. Therefore, as specified in ASTM A325, only ASTM A563 grade DH are suitable for use as galvanized nuts. This requirement should not be overlooked if non-galvanized nuts are purchased and then sent to a local galvanizer for hot-dip galvanizing. Because the mechanical galvanizing process results in a more uniformly distributed and smooth zinc coating, nuts may be tapped over-size before galvanizing by an amount that is less than that required for the hot-dip process before galvanizing.

Despite the thin-film of the Zn/Al Inorganic Coating, tapping the nuts over-size may be necessary. Similar to mechanical galvanizing, the process results in a comparatively uniform and evenly distributed coating.

In earlier editions, this Specification permitted the use of ASTM A194 grade 2H nuts in the same finish as that permitted for ASTM A563 nuts in the following cases: with ASTM A325 Type 1 plain, Type 1 galvanized and Type 3 plain bolts and with ASTM A490 Type 1 plain bolts. Reference to ASTM A194 grade 2H nuts has been removed following the removal of similar reference within the ASTM A325 and A490 Specifications. However, it should be noted that ASTM A194 grade 2H nuts remain acceptable in these applications as indicated by footnote in Table 2.1, should they be available.

ASTM A563 nuts are manufactured to dimensions as specified in ANSI/ASME B18.2.6. The basic dimensions, as defined in Figure C-2.2, are shown in Table C-2.1

2.5. Washers

Flat circular washers and square or rectangular beveled washers shall meet the requirements of ASTM F436, except as provided in Table 6.1. The type and finish of such washers shall be as given in Table 2.1.

2.6. Washer-Type Indicating Devices

The use of washer-type indicating devices is permitted as described in Sections 2.6.1 and 2.6.2.

2.6.1. Compressible-Washer-Type Direct Tension Indicators: Compressible-washer-type direct tension indicators shall meet the requirements of ASTM F959.

2.6.2. Alternative Washer-Type Indicating Devices: When approved by the *Engineer of Record*, the use of alternative washer-type indicating devices that differ from those that meet the requirements of ASTM F959 is permitted.

Detailed installation instructions shall be prepared by the *manufacturer* in a supplemental specification that is approved by the *Engineer of Record* and shall provide for:

(1) The required character and frequency of pre-installation verification;
(2) The alignment of bolt holes to permit insertion of the bolt without undue damage to the threads;
(3) The placement of *fastener assemblies* in all types and sizes of holes, including placement and orientation of the alternative and regular washers;
(4) The systematic assembly of the *joint*, progressing from the most rigid part of the *joint* until the connected plies are in *firm contact*; and,
(5) The subsequent systematic pretensioning of all bolts in the *joint*, progressing from the most rigid part of the *joint* in a manner that will minimize relaxation of previously pretensioned bolts.

Detailed inspection instructions shall be prepared by the *manufacturer* in a supplemental specification that is approved by the *Engineer of Record* and shall provide for:

(1) Observation of the required pre-installation verification testing; and,
(2) Subsequent *routine observation* to ensure the proper use of the alternative washer-type indicating device.

2.7. Twist-Off-Type Tension-Control Bolt Assemblies

2.7.1. Specifications: Twist-off-type tension-control bolt assemblies shall meet the requirements of ASTM F1852 or F2280. The *Engineer of Record* shall specify the type of bolt (Table 2.1) to be used.

2.7.2. Geometry: Twist-off-type tension-control bolt assembly dimensions shall meet the requirements of ASTM F1852 or ASTM F2280. The bolt length used shall be such that the end of the bolt extends beyond or is at least flush with the outer face of the nut when properly installed.

Commentary:
It is the policy of the Research Council on Structural Connections to directly recognize only those fastener components that are manufactured to meet the requirements in an approved ASTM specification. Prior to this edition, the RCSC Specification provided for the use of ASTM A325 and A490 bolts, and F1852 twist-off-type tension-control bolt assemblies directly and alternative-design fasteners meeting detailed requirements similar to those in Section 2.8 when approved by the *Engineer of Record*. With this edition, ASTM F2280 twist-off-type tension-control bolt assemblies are now recognized directly. Essentially, ASTM F2280 relates an ASTM A490-equivalent product to a specific method of installation that is suitable for use in all *joint* types as described in Section 8. Provision has also been retained for approval by the

Engineer of Record of other alternative-design fasteners that meet the detailed requirements in Section 2.8.

If galvanized, ASTM F1852 twist-off-type tension-control bolt assemblies are required in ASTM F1852 to be mechanically galvanized.

2.8. Alternative-Design Fasteners

When approved by the *Engineer of Record*, the use of alternative-design fasteners is permitted if they:

(1) Meet the materials, manufacturing and chemical composition requirements of ASTM A325 or ASTM A490, as applicable;
(2) Meet the mechanical property requirements of ASTM A325 or ASTM A490 in full-size tests;
(3) Have a body diameter and bearing area under the bolt head and nut that is equal to or greater than those provided by a bolt and nut of the same nominal dimensions specified in Sections 2.3 and 2.4; and,
(4) Are supplied and used in the work as a *fastener assembly*.

Such alternative-design fasteners are permitted to differ in other dimensions from those of the specified *high-strength bolts* and nuts.

Detailed installation instructions shall be prepared by the *manufacturer* in a supplemental specification that is approved by the *Engineer of Record* and shall provide for:

(1) The required character and frequency of pre-installation verification;
(2) The alignment of bolt holes to permit insertion of the alternative-design fastener without undue damage;
(3) The placement of *fastener assemblies* in all holes, including any washer requirements as appropriate;
(4) The systematic assembly of the *joint*, progressing from the most rigid part of the *joint* until the connected plies are in *firm contact*; and,
(5) The subsequent systematic pretensioning of all *fastener assemblies* in the *joint*, progressing from the most rigid part of the *joint* in a manner that will minimize relaxation of previously pretensioned bolts.

Detailed inspection instructions shall be prepared by the *manufacturer* in a supplemental specification that is approved by the *Engineer of Record* and shall provide for:

(1) Observation of the required pre-installation verification testing; and,
(2) Subsequent *routine observation* to ensure the proper use of the alternative-design fastener.

SECTION 3. BOLTED PARTS

3.1. Connected Plies

All connected plies that are within the *grip* of the bolt and any materials that are used under the head or nut shall be steel with faying surfaces that are uncoated, coated or galvanized as defined in Section 3.2. Compressible materials shall not be placed within the *grip* of the bolt. The slope of the surfaces of parts in contact with the bolt head and nut shall be equal to or less than 1:20 with respect to a plane that is normal to the bolt axis.

Commentary:
The presence of gaskets, insulation or any compressible materials other than the specified coatings within the *grip* would preclude the development and/or retention of the installed pretensions in the bolts, when required.

ASTM A325, A490, F1852, and F2280 bolt assemblies are ductile enough to deform to a surface with a slope that is less than or equal to 1:20 with respect to a plane normal to the bolt axis. Greater slopes are undesirable because the resultant localized bending decreases both the strength and the ductility of the bolt.

3.2. Faying Surfaces

Faying surfaces and surfaces adjacent to the bolt head and nut shall be free of dirt and other foreign material. Additionally, *faying surfaces* shall meet the requirements in Sections 3.2.1 or 3.2.2.

3.2.1. *Snug-Tightened Joints* and *Pretensioned Joints*: The *faying surfaces* of *snug-tightened joints* and *pretensioned joints* as defined in Sections 4.1 and 4.2 are permitted to be uncoated, coated with coatings of any formulation or galvanized.

Commentary:
In both *snug-tightened joints* and *pretensioned joints*, the ultimate strength is dependent upon shear transmitted by the bolts and bearing of the bolts against the connected material. It is independent of any frictional resistance that may exist on the *faying surfaces*. Consequently, since slip resistance is not an issue, the *faying surfaces* are permitted to be uncoated, coated, or galvanized without regard to the resulting slip coefficient obtained.

For pretensioned joints, caution should be used in the specification and application of thick coatings within the *faying surface*. Although slip resistance is not required, fastener assemblies in joints with thick or multi-layer coatings may exhibit significant loss of pretension because of compressive creep in softer coatings such as epoxies, alkyds, vinyls, acrylics, and urethanes. Previous bolt relaxation studies have been conducted using uncoated steel with black bolts or galvanized steel with galvanized bolts. Galvanized surfaces ranged up to approximately 4 mils of thickness, of which approximately half the thickness was the compressible soft pure zinc surface layer. The underlying zinc-iron layers are very hard and would exhibit little creep. See *Guide*, Section 4.4. Tests have indicated that significant bolt pretension may be lost when the total coating thickness within the joint approaches 15 mils per surface, and that surface coatings beneath the bolt head and nut can contribute to additional reduction in pretension.

3.2.2 *Slip-Critical Joints*: The *faying surfaces* of *slip-critical joints* as defined in Section 4.3, including those of filler plates and finger shims, shall meet the following requirements:

(1) *Uncoated Faying Surfaces*: *Uncoated faying surfaces* shall be free of scale, except tight mill scale, and free of coatings, including inadvertent overspray, in areas closer than one bolt diameter but not less than 1 in. from the edge of any hole and in all areas within the bolt pattern or shall be blast cleaned (Class B).

(2) *Coated Faying Surfaces*: *Coated faying surfaces* shall first be blast cleaned and subsequently coated with a coating that is qualified in accordance with the requirements in Appendix A as a Class A or Class B coating as defined in Section 5.4. Alternatively, when approved by the *Engineer of Record*, coatings that provide a *mean slip coefficient* that differs from Class A or Class B are permitted when:

(i) The *mean slip coefficient* μ is established by testing in accordance with the requirements in Appendix A; and,

(ii) The design slip resistance is determined in accordance with Section 5.4 using this coefficient, except that, for design purposes, a value of μ greater than 0.50 shall not be used.

The plies of *slip-critical joints* with *coated faying surfaces* shall not be assembled before the coating has cured for the minimum time that was used in the qualifying tests.

(3) *Galvanized Faying Surfaces*: *Galvanized faying surfaces* shall first be hot dip galvanized in accordance with the requirements of ASTM A123 and subsequently roughened by means of hand wire brushing. Power wire brushing is not permitted. When prepared by roughening, the *galvanized faying surface* is designated as Class C for design.

Commentary:
Slip-critical joints are those *joints* that have specified *faying surface* conditions that, in the presence of the clamping force provided by pretensioned fasteners, resist a design load solely by friction and without displacement at the *faying*

surfaces. Consequently, it is necessary to prepare the *faying surfaces* in a manner so that the desired slip performance is achieved.

Clean mill scale steel surfaces (Class A, see Section 5.4.1) and blast-cleaned steel surfaces (Class B, see Section 5.4.1) can be used within *slip-critical joints*. When used, it is necessary to keep the *faying surfaces* free of coatings, including inadvertent overspray.

Corrosion often occurs on uncoated blast-cleaned steel surfaces (Class B, see Section 5.4.1) due to exposure between the time of fabrication and subsequent erection. In normal atmospheric exposures, this corrosion is not detrimental and may actually increase the slip resistance of the *joint*. Yura et al. (1981) found that the Class B slip coefficient could be maintained for up to one year prior to *joint* assembly.

Polyzois and Frank (1986) demonstrated that, for plate material with thickness in the range of ⅜ in. to ¾ in., the contact pressure caused by bolt pretension is concentrated on the *faying surfaces* in annular rings around and close to the bolts. In this study, unqualified paint on the *faying surfaces* away from the edge of the bolt hole by not less than 1 in. nor the bolt diameter did not reduce the slip resistance. However, this would not likely be the case for *joints* involving thicker material, particularly those with a large number of bolts on multiple gage lines; the Table 8.1 minimum bolt pretension might not be adequate to completely flatten and pull thicker material into tight contact around every bolt. Instead, the bolt pretension would be balanced by contact pressure on the regions of the *faying surfaces* that are in contact. To account for both possibilities, it is required in this Specification that all areas between the bolts be free of coatings, including overspray, as illustrated in Figure C-3.1.

As a practical matter, the smaller coating-free area can be laid out and protected more easily using masking located relative to the bolt-hole pattern than relative to the limits of the complete area of *faying surface* contact with varying and uncertain edge distance. Furthermore, the narrow coating strip around the perimeter of the *faying surface* minimizes the required field touch-up of uncoated material outside of the *joint*.

Polyzois and Frank (1986) also investigated the effect of various degrees of inadvertent overspray on slip resistance. It was found that even a small amount of overspray of unqualified paint (that is, not qualified as a Class A or Class B coating) within the specified coating-free area on clean mill scale can reduce the slip resistance significantly. On blast-cleaned surfaces, however, the presence of a small amount of overspray was not as detrimental. For simplicity, this Specification requires that all overspray be prohibited from areas that are required to be free of coatings in *slip-critical joints* regardless of whether the surface is clean mill scale steel or blast-cleaned steel.

In the 1980 edition of this Specification, generic names for coatings applied to *faying surfaces* were the basis for categories of allowable working stresses in *slip-critical* (friction) *joints*. Frank and Yura (1981) demonstrated that the slip coefficients for coatings described by a generic type are not unique values for a given generic coating description or product, but rather depend also upon the type of vehicle used. Small differences in formulation from *manufacturer* to *manufacturer* or from *lot* to *lot* with a single *manufacturer* can significantly affect slip coefficients if certain essential variables within a generic type are changed. Consequently, it is unrealistic to assign coatings to categories with relatively small incremental differences between categories based solely upon a generic description.

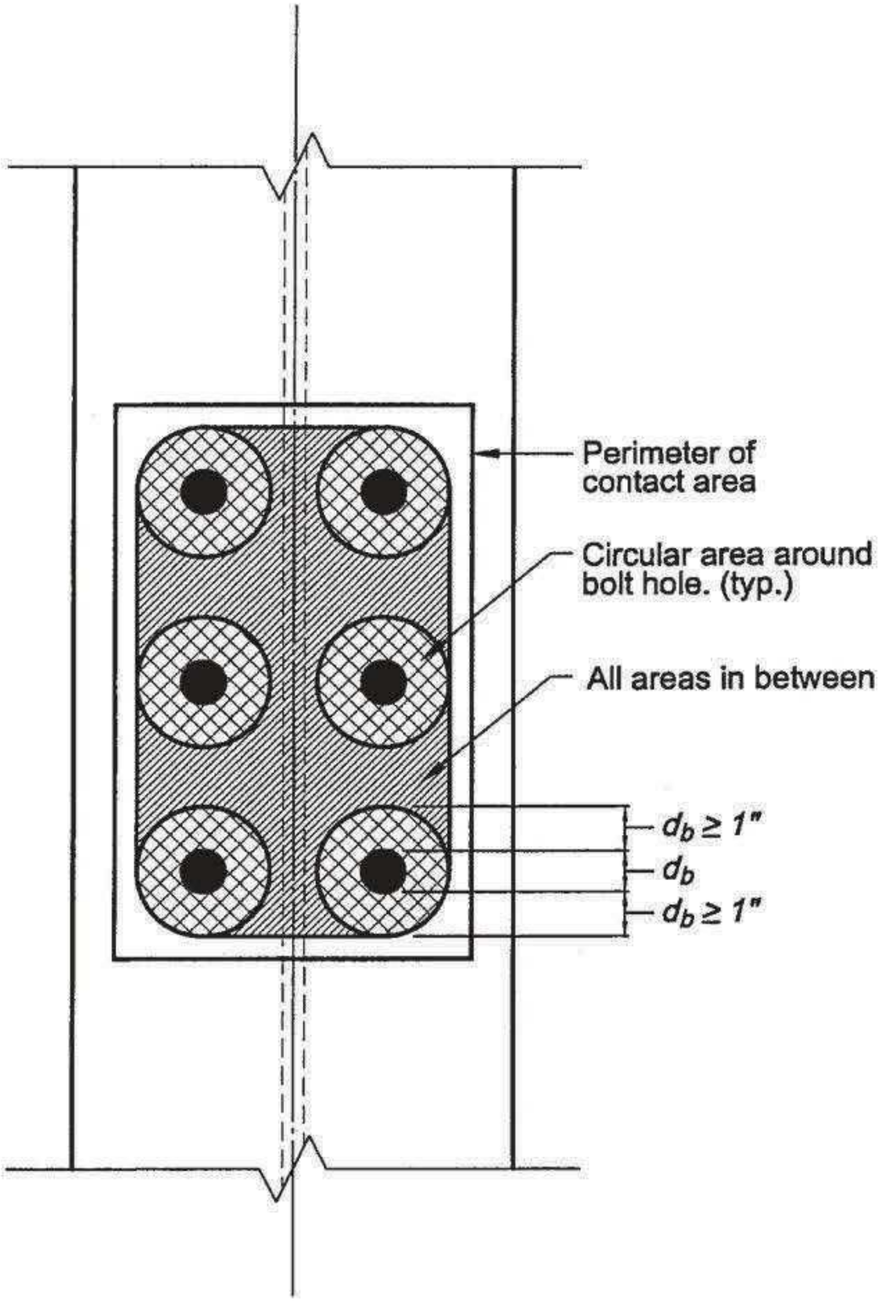


Figure C-3.1. Faying surfaces of slip-critical connections painted with unqualified paints.

When the *faying surfaces* of a *slip-critical joint* are to be protected against corrosion, a qualified coating must be used. A qualified coating is one that has been tested in accordance with Appendix A, the sole basis for qualification of any coating to be used in conjunction with this Specification. Coatings can be qualified as follows:

(1) As a Class A coating as defined in Section 5.4;
(2) As a Class B coating as defined in Section 5.4; or,
(3) As a coating with a *mean slip coefficient* μ of 0.30 (Class A) but not greater than 0.50 (Class B).

Requalification is required if any essential variable associated with surface preparation, paint manufacture, application method or curing requirements is changed. See Appendix A.

For slip-critical joints, coating testing as prescribed in Appendix A includes creep tests, which incorporate relaxation in the fastener and the effect of the coating itself. Users should verify the coating thicknesses used in the Appendix A testing and ensure that the actual coating thickness does not exceed that tested. See Appendix A, Commentary to Section A3.

Frank and Yura (1981) also investigated the effect of varying the time between coating the *faying surfaces* and assembly of the *joint* and pretensioning the bolts in order to ascertain if partially cured paint continued to cure within the assembled *joint* over a period of time. The results indicated that all curing effectively ceased at the time the *joint* was assembled and paint that was not fully cured at that time acted as a lubricant. The slip resistance of a *joint* that was assembled after a time less than the curing time used in the qualifying tests was severely reduced. Thus, the curing time prior to mating the *faying surfaces* is an essential parameter to be specified and controlled during construction.

The *mean slip coefficient* for clean hot-dip galvanized surfaces is on the order of 0.19 as compared with a factor of about 0.33 for clean mill scale. Birkemoe and Herrschaft (1970) showed that this *mean slip coefficient* can be significantly improved by treatments such as hand wire brushing or light "brush-off" grit blasting. In either case, the treatment must be controlled to achieve visible roughening or scoring. Power wire brushing is unsatisfactory because it may polish rather than roughen the surface, or remove the coating.

Field experience and test results have indicated that galvanized assemblies may continue to slip under sustained loading (Kulak et al., 1987; pp. 198-208). Tests of hot-dip galvanized *joints* subjected to sustained loading show a creep-type behavior that was not observed in short-duration or fatigue-type load application. See also the Commentary to Appendix A.

3.3. Bolt Holes

The nominal dimensions of standard, oversized, short-slotted and long-slotted holes for *high-strength bolts* shall be equal to or less than those shown in Table 3.1. Holes larger than those shown in Table 3.1 are permitted when specified or approved by the *Engineer of Record*. When complete connection design is not shown in the structural design drawings, the *Engineer of Record* shall be notified of the type and dimensions of holes to be used. Oversized holes, short slots not perpendicular to the applied load and long slots in any direction shall be subject to approval by the *Engineer of Record*. Any restrictions on the use of hole types permitted in Sections 3.3.1, 3.3.2, 3.3.3 and 3.3.4 shall be specified in the design documents.

Thermally cut holes produced by mechanically guided means are permitted in statically loaded *joints*. The surface roughness profile of the hole shall not exceed 1,000 microinches as defined in ASME B46.1. Occasional gouges not more than 1⁄16 in. in depth are permitted. Thermally cut holes produced free hand shall be permitted in statically loaded *joints* if approved by the *Engineer of Record*. For cyclically loaded *joints*, thermally cut holes shall be permitted if approved by the *Engineer of Record*.

Commentary:

The footnotes in Table 3.1 provide for slight variations in the dimensions of bolt holes from the nominal dimensions. When the dimensions of bolt holes are such that they exceed these permitted variations, the bolt hole must be treated as the next larger type.

Slots longer than standard long slots may be required to accommodate construction tolerances or expansion *joints*. Larger oversized holes may be necessary to accommodate construction tolerances or misalignments. In the latter two cases, the Specification provides no guidance for further reduction of *design strengths* or allowable loads. Engineering design considerations should include, as a minimum, the effects of edge distance, net section, reduction in clamping force in *slip-critical joints*, washer requirements, bearing capacity, and hole deformation.

For thermally cut holes produced free hand, it is usually necessary to grind the hole surface after thermal cutting in order to achieve a maximum surface roughness profile of 1,000 microinches.

Slotted holes in statically loaded *joints* are often produced by punching or drilling the hole ends and thermally cutting the sides of the slots by mechanically guided means. The sides of such slots should be ground smooth, particularly at the junctures of the thermal cuts to the hole ends.

For cyclically loaded *joints*, test results have indicated that when no major slip occurs in the *joint*, fretting fatigue failure usually occurs in the gross section prior to fatigue failure in the net section (Kulak et al., 1987, pp. 116, 117). Conversely, when slip occurs in the *joints* of cyclically loaded *connections*, failure usually occurs in the net section and the edge of a bolt hole becomes the point of crack initiation (Kulak et al., 1987, pp. 118). Therefore, for cyclically loaded *joints* designed as slip critical, the method used to produce bolt holes (either thermal cutting or drilling) should not influence the ultimate failure load, as failure usually occurs in the gross section when no major slip occurs.

3.3.1. Standard Holes: Standard holes are permitted to be used in all plies of bolted *joints*.

Table 3.1. Nominal Bolt Hole Dimensions

Nominal Bolt Diameter, d_b, in.	Nominal Bolt Hole Dimensions[a,b], in.			
	Standard (diameter)	**Oversized (diameter)**	**Short-slotted (width × length)**	**Long-slotted (width × length)**
1/2	9/16	5/8	9/16 × 11/16	9/16 × 1 1/4
5/8	11/16	13/16	11/16 × 7/8	11/16 × 1 9/16
3/4	13/16	15/16	3/16 × 1	13/16 × 1 7/8
7/8	15/16	1 1/16	5/16 × 1 1/8	15/16 × 2 3/16
1	1 1/16	1 1/4	1 1/16 × 1 5/16	1 1/16 × 2 1/2
≥1 1/8	d_b + 1/16	d_b + 5/16	(d_b + 1/16) × (d_b + 3/8)	(d_b + 1/16) × (2.5d_b)

[a] The upper tolerance on the tabulated nominal dimensions shall not exceed 1/32 in. Exception: In the width of slotted holes, gouges not more than 1/16 in. deep are permitted.

[b] The slightly conical hole that naturally results from punching operations with properly matched punches and dies is acceptable.

Commentary:
The use of bolt holes 1/16 in. larger than the bolt installed in them has been permitted since the first publication of this Specification. Allen and Fisher (1968) showed that larger holes could be permitted for *high-strength bolts* without adversely affecting the bolt shear or member bearing strength. However, the slip resistance can be reduced by the failure to achieve adequate pretension initially or by the relaxation of the bolt pretension as the highly compressed material yields at the edge of the hole or slot.

3.3.2. Oversized Holes: When approved by the *Engineer of Record*, oversized holes are permitted in any or all plies of *slip-critical joints* as defined in Section 4.3.

Commentary:
The provisions for oversized holes in this Specification are based upon the findings of Allen and Fisher (1968) and the additional concern for the consequences of a slip of significant magnitude that can occur as permitted by the oversized hole.

3.3.3. Short-Slotted Holes: Short-slotted holes are permitted in any one ply at each faying surface of *snug-tightened joints* as defined in Section 4.1, *pretensioned joints* as defined in Section 4.2 and *slip critical joints* as defined in Section 4.3, provided the applied load is approximately perpendicular (between 80 and 100 degrees) to the axis of the slot. When complete connection design is not shown in the structural design drawings, the *Engineer of Record* shall be notified when short-slotted holes are used in this manner. When approved by the *Engineer of Record*, short-slotted holes are permitted in more than one or all plies *snug tightened* joints as defined in Section 4.1 and *pretensioned* joints as defined in Section 4.2 provided the applied load is approximately perpendicular (between 80 and 100 degrees) to the axis of the slot and in any or all plies of *slip-critical joints* as defined in Section 4.3 without regard for the direction of the applied load.

Commentary:
For beam end connections, the use of short-slotted holes approximately perpendicular to the applied load in conjunction with snug tight bolts can provide the shear capacity and may allow the beam to rotate consistent with the design assumptions. Deformation of connections can be a concern where the beam is not laterally or torsionally restrained by floor, roof or other framing.

Short slots are used to account for minor adjustments in main members such as web thickness differences and member length. This practice is prevalent enough that this specification recognizes it and permits it unless it is specifically prohibited by the *Engineer of Record* in the design documents. This specification requires the *Engineer of Record* to be notified of the hole types and dimensions by showing this information on shop detail drawings or by obtaining prior approval of the *Engineer of Record*.

The provision of limiting the use of short slotted holes to one ply with snug tight bolts is to avoid the use of short slotted holes in opposing plies of a faying surface. The use of short slotted holes with snug tight bolts in connections with multiple plies that do not share a faying surface is still permitted. An example that would be permitted with multiple plies includes beam end connections on opposing sides of a column web.

3.3.4. Long-Slotted Holes: When approved by the *Engineer of Record*, long-slotted holes are permitted in only one ply at any individual *faying surface* of *snug-tightened joints* as defined in Section 4.1, and *pretensioned joints* as defined in Section 4.2, provided the applied load is approximately perpendicular (between 80 and 100 degrees) to the axis of the slot. When approved by the *Engineer of Record*, long-slotted holes are permitted in one ply only at any individual *faying surface* of *slip-critical joints* as defined in Section 4.3 without regard for the direction of the applied load. Fully inserted finger shims between the *faying surfaces* of load-transmitting elements of bolted *joints* are not considered a long-slotted element of a *joint*; nor are they considered to be a ply at any individual *faying surface*. However, finger shims must have the same faying surface as the rest of the plies.

Commentary:
See the Commentary to Section 3.3.1.

Finger shims are devices that are often used to permit the alignment and plumbing of structures. When these devices are fully and properly inserted, they do not have the same effect on bolt pretension relaxation or the *connection* performance, as do long-slotted holes in an outer ply. When fully inserted, the shim provides support around approximately 75 percent of the perimeter of the bolt in contrast to the greatly reduced area that exists with a bolt that is centered in a long slot. Furthermore, finger shims are always enclosed on both sides by the connected material, which should be effective in bridging the space between the fingers.

3.4. Burrs

Burrs less than or equal to $1/16$ in. in height are permitted to remain on *faying surfaces* of all *joints*. Burrs larger than $1/16$ in. in height shall be removed or reduced to $1/16$ in. or less from the *faying surfaces* of all *joints*.

Commentary:
Polyzois and Yura (1985) and McKinney and Zwerneman (1993) demonstrated that the slip resistance of *joints* was either unchanged or slightly improved by the presence of burrs. Therefore, small ($1/16$ in. or less) burrs need not be removed. On the other hand, parallel tests in the same program demonstrated that large burrs (over $1/16$ in.) could cause a small increase in the required nut rotation from the snug-tight condition to achieve the specified pretension with the turn-of-nut pretensioning method. Therefore, the Specification requires that all large burrs be removed or reduced in height.

Note that prior to pretensioning, the snug-tightening procedure is required to bring the plies into *firm contact*. If *firm contact* has not been achieved after snugging due to the presence of burrs, additional snugging is required to flatten the burrs, bringing the plies into *firm contact*.

SECTION 4. JOINT TYPE

For *joints* with fasteners that are loaded in shear or combined shear and tension, the *Engineer of Record* shall specify the *joint* type in the contract documents as snug-tightened, pretensioned or slip-critical. For *slip-critical joints*, the required class of slip resistance in accordance with Section 5.4 shall also be specified. For *joints* with fasteners that are loaded in tension only, the *Engineer of Record* shall specify the *joint* type in the contract documents as snug-tightened or pretensioned. Table 4.1 summarizes the applications and requirements of the three *joint* types.

Table 4.1. Summary of Applications and Requirements for Bolted Joints

Load Transfer	Application	*Joint* Type [a,b]	*Faying Surface* Prep.?	Install per Section	Inspect per Section	Arbitrate per Section 10?
Shear only	Resistance to shear load by shear/bearing	ST	No	8.1	9.1	No
	Resistance to shear by shear/bearing. Bolt pretension is required, but for reasons other than slip resistance.	PT	No	8.2	9.2	No
	Shear-load resistance by friction on faying surfaces is required.	SC	Yes [d]	8.2	9.3	If req'd to resolve dispute
Combined shear and tension	Resistance to shear load by shear/bearing. Tension load is static only. [c]	ST	No	8.1	9.1	No
	Resistance to shear by shear/bearing. Bolt pretension is required, but for reasons other than slip resistance.	PT	No	8.2	9.2	If req'd to resolve dispute
	Shear-load resistance by friction on faying surfaces is required.	SC	Yes [d]	8.2	9.3	If req'd to resolve dispute
Tension only	Static loading only. [c]	ST	No	8.1	9.1	No
	All other conditions of tension-only loading.	PT	No	8.2	9.2	If req'd to resolve dispute

[a] Under *Joint* Type: ST = snug-tightened, PT = pretensioned and SC = slip-critical; See Section 4.

[b] See Sections 4 and 5 for the design requirements for each *joint* type.

[c] Per Section 4.2, the use of ASTM A490 or F2280 bolts in *snug-tightened joints* with tensile loads is not permitted.

[d] See Section 3.2.2.

Commentary:
When first approved by the Research Council on Structural Connections, in January, 195l, the "Specification for Assembly of Structural Joints Using High-Strength Bolts" merely permitted the substitution of a like number of ASTM A325 bolts for hot driven ASTM A141[1] steel rivets of the same nominal diameter. Additionally, it was required that all bolts be pretensioned and that all *faying surfaces* be free of paint; hence, satisfying the requirements for a *slip-critical joint* by the present-day definition. As revised in 1954, the omission of paint was required to apply only to "*joints* subject to stress reversal, impact or vibration, or to cases where stress redistribution due to *joint* slippage would be undesirable." This relaxation of the earlier provision recognized the fact that, in many applications, movement of the connected parts that brings the bolts into bearing against the sides of their holes is in no way detrimental. Bolted *joints* were then designated as "bearing type," "friction type," or "direct tension." With the 1985 edition of this Specification, these designations were changed to "shear/bearing," "slip-critical," and "direct tension," respectively, and snug-tightened installation was permitted for many *shear/bearing joints*. *Snug-tightened joints* are also permitted for qualified applications involving ASTM A325 bolts in direct tension.

If non-pretensioned bolts are used in the type of *joint* that places the bolts in shear, load is transferred by shear in the bolts and bearing stress in the connected material. At the ultimate limit state, failure will occur by shear failure of the bolts, by bearing failure of the connected material or by failure of the member itself. On the other hand, if pretensioned bolts are used in such a *joint*, the frictional force that develops between the connected plies will initially transfer the load. Until the frictional force is exceeded, there is no shear in the bolts and no bearing stress in the connected components. Further increase of load places the bolts into shear and against the connected material in bearing, just as was the case when non-pretensioned bolts were used. Since it is known that the pretension in bolts will have been dissipated by the time bolt shear failure takes place (Kulak et al., 1987; p. 49), the ultimate limit state of a pretensioned bolted *joint* is the same as an otherwise identical *joint* that uses non-pretensioned bolts.

Because the consequences of slip into bearing vary from application to application, the determination of whether a *joint* can be designated as snug-tightened or as pretensioned or rather must be designated as slip-critical is best left to judgment and a decision on the part of the *Engineer of Record*. In the case of *joints* with three or more bolts in holes with only a small clearance, the freedom to slip generally does not exist. It is probable that normal fabrication tolerances and erection procedures are such that one or more bolts are in bearing even before additional load is applied. Such is the case for standard holes and for slotted holes loaded transverse to the axis of the slot.

Joints that are required to be *slip-critical joints* include:

(1) Those cases where slip movement could theoretically exceed an amount deemed by the *Engineer of Record* to affect the serviceability of the structure or through excessive distortion to cause a reduction in strength or stability, even though the

[1] ASTM A141 (discontinued in 1967) became identified as A502 Grade 1 (discontinued 1999).

resistance to fracture of the *connection* and yielding of the member may be adequate; and,

(2) Those cases where slip of any magnitude must be prevented, such as in *joints* subject to significant load reversal and *joints* between elements of built-up compression members in which any slip could cause a reduction of the flexural stiffness required for the stability of the built-up member.

In this Specification, the provisions for the design, installation and inspection of bolted *joints* are dependent upon the type of *joint* that is specified by the *Engineer of Record*. Consequently, it is required that the *Engineer of Record* identify the *joint* type in the contract documents.

4.1. Snug-Tightened Joints

Except as required in Sections 4.2 and 4.3, *snug-tightened joints* are permitted.

Bolts in *snug-tightened joints* shall be designed in accordance with the applicable provisions of Sections 5.1, 5.2 and 5.3, installed in accordance with Section 8.1 and inspected in accordance with Section 9.1. As indicated in Section 4 and Table 4.1, requirements for *faying surface* condition shall not apply to *snug-tightened joints*.

Commentary:

Recognizing that the ultimate strength of a *connection* is independent of the bolt pretension and slip movement, there are numerous practical cases in the design of structures where, if slip occurs, it will not be detrimental to the serviceability of the structure. Additionally, there are cases where slip of the *joint* is desirable to permit rotation in a *joint* or to minimize the transfer of moment. To provide for these cases while at the same time making use of the shear strength of *high-strength bolts*, *snug-tightened joints* are permitted.

The maximum amount of slip that can occur in a *joint* is, theoretically, equal to twice the hole clearance. In practical terms, it is observed in laboratory and field experience to be much less; usually, about one-half the hole clearance. Acceptable inaccuracies in the location of holes within a pattern of bolts usually cause one or more bolts to be in bearing in the initial, unloaded condition. Furthermore, even with perfectly positioned holes, the usual method of erection causes the weight of the connected elements to put some of the bolts into direct bearing at the time the member is supported on loose bolts and the lifting crane is unhooked. Additional loading in the same direction would not cause additional *joint* slip of any significance.

Snug-tightened joints are also permitted for statically loaded applications involving ASTM A325 bolts and ASTM F1852 twist-off-type tension-control bolt assemblies in direct tension. However, snug-tightened installation is not permitted for these fasteners in applications involving non-static loading, nor for applications involving ASTM A490 bolts and ASTM F2280 twist-off-type tension-control bolt assemblies.

4.2. Pretensioned Joints

Pretensioned joints are required in the following applications:

(1) *Joints* in which fastener pretension is required in the specification or code that invokes this Specification;
(2) *Joints* that are subject to significant load reversal;
(3) *Joints* that are subject to fatigue load with no reversal of the loading direction;
(4) *Joints* with ASTM A325 or F1852 bolts that are subject to tensile fatigue; and,
(5) *Joints* with ASTM A490 or F2280 bolts that are subject to tension or combined shear and tension, with or without fatigue.

Bolts in *pretensioned joints* subject to shear shall be designed in accordance with the applicable provisions of Sections 5.1 and 5.3, installed in accordance with Section 8.2 and inspected in accordance with Section 9.2. Bolts in *pretensioned joints* subject to tension or combined shear and tension shall be designed in accordance with the applicable provisions of Sections 5.1, 5.2, 5.3 and 5.5, installed in accordance with Section 8.2 and inspected in accordance with Section 9.2. As indicated in Section 4 and Table 4.1, requirements for *faying surface* condition shall not apply to *pretensioned joints*.

Commentary:
Under the provisions of some other specifications, certain shear *connections* are required to be pretensioned, but are not required to be slip-critical. Several cases are given, for example, in AISC Specification Section J1.10 (AISC, 2010) wherein certain bolted *joints* in bearing *connections* are to be pretensioned regardless of whether or not the potential for slip is a concern. The AISC Specification requires that *joints* be pretensioned in the following circumstances:

(1) Column splices in buildings with high ratios of height to width;
(2) *Connections* of members that provide bracing to columns in tall buildings;
(3) Various *connections* in buildings with cranes over 5-ton capacity; and,
(4) *Connections* for supports of running machinery and other sources of impact or stress reversal.

When pretension is desired for reasons other than the necessity to prevent slip, a *pretensioned joint* should be specified in the contract documents.

4.3. Slip-Critical Joints

Slip-critical joints are required in the following applications involving shear or combined shear and tension:

(1) *Joints* that are subject to fatigue load with reversal of the loading direction;
(2) *Joints* that utilize oversized holes;
(3) *Joints* that utilize slotted holes, except those with applied load approximately normal (within 80 to 100 degrees) to the direction of the long dimension of the slot; and,
(4) *Joints* in which slip at the *faying surfaces* would be detrimental to the performance of the structure.

Bolts in *slip-critical joints* shall be designed in accordance with the applicable provisions of Sections 5.1, 5.2, 5.3, 5.4 and 5.5, installed in accordance with Section 8.2 and inspected in accordance with Section 9.3.

Commentary:
In certain cases, slip of a bolted *joint* in shear under service loads would be undesirable or must be precluded. Clearly, *joints* that are subject to reversed fatigue load must be slip-critical since slip may result in back-and-forth movement of the *joint* and the potential for accelerated fatigue failure. Unless slip is intended, as desired in a sliding expansion *joint*, slip in *joints* with long-slotted holes that are parallel to the direction of the applied load might be large enough to invalidate structural analyses that are based upon the assumption of small displacements.

For *joints* subject to fatigue load with respect to shear of the bolts that does not involve a reversal of load direction, there are two alternatives for fatigue design. The designer can provide either a *slip-critical joint* that is proportioned on the basis of the applied stress range on the gross section, or a *pretensioned joint* that is proportioned on the basis of applied stress range on the net section.

SECTION 5. LIMIT STATES IN BOLTED JOINTS

The available shear strength and available tensile strength of bolts shall be determined in accordance with Section 5.1. The interaction of combined shear and tension on bolts shall be limited in accordance with Section 5.2. The available bearing strength of the connected parts at bolt holes shall be determined in accordance with Section 5.3. Each of these *available strengths* shall be equal to or greater than the *required strength*. The axial load in bolts that are subject to tension or combined shear and tension shall be calculated with consideration of the effects of the externally applied tensile load and any additional tension resulting from *prying action* produced by deformation of the connected parts.

When slip resistance is required at the *faying surfaces* subject to shear or combined shear and tension, slip resistance shall be checked at either the LRFD-load level or ASD-load level, at the option of the *Engineer of Record*. When slip of the *joint* under applied loads would affect the ability of the structure to support the loads, the *available strength* determined in accordance with Section 5.4 shall be equal to or greater than the *required strength*. In addition, slip-critical connections must meet the strength requirements of shear/bearing joints. Therefore, the strength requirements of Sections 5.1, 5.2 and 5.3 shall also be met.

When bolts are subject to cyclic application of axial tension, the stress determined in accordance with Section 5.5 shall be equal to or greater than the stress due to the effect of the service loads, including any additional tension resulting from prying action produced by deformation of the connected parts.

Commentary:
This section of the Specification provides the design requirements for *high-strength bolts* in bolted *joints*. However, this information is not intended to provide comprehensive coverage of the design of *high-strength bolted connections*. Other design considerations of importance to the satisfactory performance of the connected material, such as block shear rupture, shear lag, *prying action* and *connection* stiffness and its effect on the performance of the structure, are beyond the scope of this Specification and Commentary.

The design of bolted *joints* that transmit shear requires consideration of the shear strength of the bolts and the bearing strength of the connected material. If such *joints* are designated as *slip-critical joints*, the slip resistance must also be checked. This serviceability check can be made at the LRFD-load level or at the ASD-load level. Regardless of which load level is selected for the check of slip resistance, the prevention of slip in the service-load range is the design criterion.

Parameters that influence the shear strength of bolted *joints* include:

(1) Geometric parameters – the ratio of the net area to the gross area of the connected parts, the ratio of the net area of the connected parts to the total shear-resisting area of the bolts and the length of the *joint*; and,
(2) Material parameter – the ratio of the yield strength to the tensile strength of the connected parts.

Using both mathematical models and physical testing, it was possible to study the influences of these parameters (Kulak et al., 1987; pp. 89-116 and 126-132). These showed that, under the rules that existed at that time the longest (and often the most important) *joints* had the lowest factor of safety, about 2.0 based on ultimate strength.

In general, bolted *joints* that are designed in accordance with the provisions of this Specification will have a higher reliability than will the members they connect. This occurs primarily because the resistance factors used in limit states for the design of bolted *joints* were chosen to provide a reliability higher than that used for member design. Additionally, the controlling strength limit state in the structural member, such as yielding or deflection, is usually reached well before the strength limit state in the *connection*, such as bolt shear strength or bearing strength of the connected material. The installation requirements vary with *joint* type and influence the behavior of the *joints* within the service-load range, however, this influence is ignored in all strength calculations. Secondary tensile stresses that may be produced in bolts in *shear/bearing joints*, such as through the flexing of double-angle *connections* to accommodate the simple-beam end rotation, need not be considered.

It is sometimes necessary to use *high-strength bolts* and fillet welds in the same *connection*, particularly as the result of remedial work. When these fastening elements act in the same shear plane, the combined strength is a function of whether the bolts are snug-tightened or pretensioned, the location of the bolts relative to the holes in which they are located and the orientation of the fillet welds. The fillet welds can be parallel or transverse to the direction of load. Manuel and Kulak (1999) provide an approach that can be used to calculate the *design strength* of such *joints*.

5.1. Nominal Shear and Tensile Strengths

Shear and tensile strengths shall not be reduced by the installed bolt pretension. For *joints*, the nominal shear and tensile strengths shall be taken as the sum of the strengths of the individual bolts.

The *design strength* in shear or tension for an ASTM A325, A490, F1852 or F2280 bolt is ϕR_n, where $\phi = 0.75$ and the *allowable strength* in shear or tension is R_n/Ω, where $\Omega = 2.00$ and:

$$R_n = F_n A_b \qquad \text{(Equation 5.1)}$$

Where

R_n = nominal strength (shear strength per shear plane or tensile strength) of a bolt, kips;

Table 5.1. Nominal Strengths per Unit Area of Bolts

Applied Load Condition			*Nominal Strength* per Unit Area, F_n, ksi	
			ASTM A325 or F1852	ASTM A490 or F2280
Tension [a]	Static		90	113
	Fatigue		See Section 5.5	
Shear [a,b]	Threads included in shear plane	$L_s \leq 38$ in.	54	68
		$L_s > 38$ in.	45	56
	Threads excluded from shear plane	$L_s \leq 38$ in.	68	84
		$L_s > 38$ in.	56	70

[a] Except as required in Section 5.2.

[b] Reduction for values for $L_s > 38$ in. applies only when the joint is end loaded, such as splice plates on a beam or column flange.

F_n = nominal strength per unit area from Table 5.1 for the appropriate applied load conditions, ksi, adjusted for the presence of fillers as required below, and,

A_b = cross-sectional area based upon the nominal diameter of bolt, in.^{2}

When a bolt that carries load passes through fillers or shims in a shear plane that are equal to or less than ¼ in. thick, F_n from Table 5.1 shall be used without reduction. When a bolt that carries load passes through fillers or shims that are greater than ¼ in. thick, the connection shall be designed in accordance with one of the following procedures:

(1) F_n from Table 5.1 shall be multiplied by the factor $[1 - 0.4(t' - 0.25)]$ but not less than 0.85, where t' is the total thickness of fillers or shims, in.;
(2) The fillers or shims shall be extended beyond the *joint* and the filler or shim extension shall be secured with enough bolts to uniformly distribute the total force in the connected element over the combined cross-section of the connected element and the fillers or shims;
(3) The size of the *joint* shall be increased to accommodate a number of bolts that is equivalent to the total number required in (2) above; or,
(4) The *joint* shall be designed as a *slip-critical joint* using Class A surfaces with Turn-of-Nut pretensioning; or,
(5) The joint shall be designed as a *slip-critical joint* using Class B faying surfaces.

Commentary:
The nominal shear and tensile strengths of ASTM A325, F1852, A490 and F2280 bolts are given in Table 5.1. These values are based upon the work of a large number of researchers throughout the world, as reported in the *Guide* (Kulak et al., 1987; Tide, 2010).

The nominal shear strength is based upon the observation that the shear strength of a single *high-strength bolt* is about 0.62 times the tensile strength of that bolt (Kulak et al., 1987; pp. 44-50). In addition, a reduction factor of 0.90 is applied to joints up to 38 in. in length to account for an increase in bolt force due to minor secondary effects resulting from simplifying assumptions made in the modeling of structures that are commonly accepted in practice (e.g. truss bolted connections assumed pinned in the analysis model). Second order effects such as those resulting from the action of the applied loads on the deformed structure, should be accounted for through a second order analysis of the structure. As noted in Table 5.1, the average shear strength of bolts in *joints* longer than 38 in. in length is reduced by a factor of 0.75 instead of 0.90. This factor accounts for both the non-uniform force distribution between the bolts in a long joint and the minor secondary effects discussed above. Note that the 0.75 reduction factor does not apply in cases where the distribution of force is essentially uniform along the *joint*, such as the bolted *joints* in a shear *connection* at the end of a deep plate girder.

The average ratio of nominal shear strength for bolts with threads included in the shear plane to the nominal shear strength for bolts with threads excluded from the shear plane is 0.83 with a standard deviation of 0.03 (Frank and Yura, 1981). Conservatively, a reduction factor of 0.80 is used to account for the reduction in shear strength for a bolt with threads included in the shear plane but calculated with the area corresponding to the nominal bolt diameter. The case of a bolt in double shear with a non-threaded section in one shear plane and a threaded section in the other shear plane is not covered in this Specification for two reasons. First, the manner in which load is shared between these two dissimilar shear areas is uncertain. Second, the detailer's lack of certainty as to the orientation of the bolt placement might leave both shear planes in the threaded section. Thus, if threads are included in one shear plane, the conservative assumption is made that threads are included in all shear planes.

The tensile strength of a *high-strength bolt* is the product of its ultimate tensile strength per unit area and some area through the threaded portion. This area, called the tensile stress area, is a derived quantity that is a function of the relative thread size and pitch. For the usual sizes of structural bolts, it is about 75 percent of the nominal cross-sectional area of the bolt. Hence, the nominal tensile strengths per unit area given in Table 5.1 are 0.75 times the tensile strength of the bolt material. According to Equation 5.1, the nominal area of the bolt is then used to calculate the *design strength* or *allowable strength* in tension. The strengths so-calculated are intended to form the basis for comparison with the externally applied bolt tension plus any additional tension that results from *prying action* that is produced by deformation of the connected elements.

If pretensioned bolts are used in a *joint* that loads the bolts in tension, the question arises as to whether the pretension and the applied tension are additive. Because the compressed parts are being unloaded during the application of the external tensile force, the increase in bolt tension is minimal until the parts separate (Kulak et al., 1987; pp. 263-266). Thus, there will be little increase in bolt force above the pretension load under service loads. After the parts separate, the bolt acts as a tension member, as expected.

Pretensioned bolts have torsion present during the installation process. Once the installation is completed, any residual torsion is quite small and will disappear entirely when the fastener is loaded to the point of plate separation. Hence, there is no question of torsion-tension interaction when considering the ultimate tensile strength of a *high-strength bolt* (Kulak et al., 1987; pp. 41-47).

When required, pretension is induced in a bolt by imposing a small axial elongation during installation, as described in the Commentary to Section 8. When the *joint* is subsequently loaded in shear, tension or combined shear and tension, the bolts will undergo significant deformations prior to failure that have the effect of overriding the small axial elongation that was introduced during installation, thereby removing the pretension. Measurements taken in laboratory tests confirm that the pretension that would be sustained if the applied load were removed is essentially zero before the bolt fails in shear (Kulak et al., 1987; pp. 93-94). Thus, the shear and tensile strengths of a bolt are not affected by the presence of an initial pretension in the bolt.

See also the Commentary to Section 5.5.

Tests of 24 bolt A490 1⅛ diameter connections indicated the reduction in bolt shear strength in connections with filler as required in Section 5.1 (1) is limited to 85%. (Borello et al., 2009). Review of available data on slip critical connections revealed that connections with Class A surfaces pretensioned by Turn-of-Nut and connections with Class B surfaces provide a sufficient reliability against slip to eliminate the need to fasten the fills outside the connection or reduce the bolt shear capacity. (Grondin et al., 2008).

5.2. Combined Shear and Tension

When combined shear and tension loads are transmitted by an ASTM A325, A490, F1852 or F2280 bolt, the factored limit-state interaction shall be:

$$\left[\frac{T_u}{(\phi R_n)_t}\right]^2 + \left[\frac{V_u}{(\phi R_n)_v}\right]^2 \leq 1 \qquad \text{(Equation 5.2a)}$$

Where

T_u = *required strength* in tension (factored tensile load) per bolt, kips;

V_u	=	*required strength* in shear (factored shear load) per bolt, kips;
$(\phi R_n)_t$	=	*design strength* in tension determined in accordance with Section 5.1, kips; and,
$(\phi R_n)_v$	=	*design strength* in shear determined in accordance with Section 5.1, kips.

When combined shear and tension loads are transmitted by an ASTM A325, A490, F1852 or F2280 bolt, the allowable limit-state interaction shall be:

$$\left[\frac{T_a}{(R_n/\Omega)_t}\right]^2 + \left[\frac{V_a}{(R_n/\Omega)_v}\right]^2 \leq 1 \qquad \text{(Equation 5.2b)}$$

Where

T_a	=	*required strength* in tension (service tensile load) per bolt, kips;
V_a	=	*required strength* in shear (service shear load) per bolt, kips;
$(R_n/\Omega)_t$	=	*allowable strength* in tension determined in accordance with Section 5.1, kips; and,
$(R_n/\Omega)_v$	=	*allowable strength* in shear determined in accordance with Section 5.1, kips.

Commentary:
When both shear forces and tensile forces act on a bolt, the interaction can be conveniently expressed as an elliptical solution (Chesson et al., 1965) that includes the elements of the bolt acting in shear alone and the bolt acting in tension alone. Although the elliptical solution provides the best estimate of the strength of bolts subject to combined shear and tension and is thus used in this Specification, the nature of the elliptical solution is such that it can be approximated conveniently using three straight lines (Carter et al., 1997). Earlier editions of this specification have used such linear representations for the convenience of design calculations. The elliptical interaction equation in effect shows that, for design purposes, significant interaction does not occur until either force component exceeds 20 percent of the limiting strength for that component.

5.3. Nominal Bearing Strength at Bolt Holes

For *joints*, the nominal bearing strength shall be taken as the sum of the strengths of the connected material at the individual bolt holes.

The design bearing strength is ϕR_n, where ϕ = 0.75 and the allowable bearing strength is R_n/Ω, where Ω = 2.00 of the connected material at a standard bolt hole, oversized bolt hole, short-slotted bolt hole independent of the direction of loading or long-slotted bolt hole with the slot parallel to the direction of the bearing load and:

(1) when deformation of the bolt hole at service load is a design consideration;

$$R_n = 1.2L_c tF_u \leq 2.4d_b tF_u \qquad \text{(Equation 5.3)}$$

(2) when deformation of the bolt hole at service load is not a design consideration;

$$R_n = 1.5L_c tF_u \leq 3d_b tF_u \qquad \text{(Equation 5.4)}$$

The design bearing strength is ϕR_n, where $\phi = 0.75$ and the allowable bearing strength is R_n/Ω, where $\Omega = 2.00$ of the connected material at a long-slotted bolt hole with the slot perpendicular to the direction of the bearing load and:

$$R_n = L_c tF_u \leq 2d_b tF_u \qquad \text{(Equation 5.5)}$$

In Equations 5.3, 5.4 and 5.5,

R_n = *nominal strength* (bearing strength of the connected material), kips;
F_u = specified minimum tensile strength per unit area of the connected material, ksi;
L_c = clear distance, in the direction of load, between the edge of the hole and the edge of the adjacent hole or the edge of the material, in.;
d_b = nominal diameter of bolt, in.; and,
t = thickness of the connected material, in.

Commentary:
The contact pressure at the interface between a bolt and the connected material can be expressed as a bearing stress on the bolt or on the connected material. The connected material is always critical. For simplicity, the bearing area is expressed as the bolt diameter times the thickness of the connected material in bearing. The governing value of the bearing stress has been determined from extensive experimental research and a further limitation on strength was derived from the case of a bolt at the end of a tension member or near another fastener.

The design equations are based upon the models presented in the *Guide* (Kulak et al., 1987; pp. 141-143), except that the clear distance to another hole or edge is used in the Specification formulation rather than the bolt spacing or end distance as used in the *Guide* (see Figure C-5.1). Equation 5.3 is derived from tests (Kulak et al., 1987; pp. 112-116) that showed that the total elongation, including local bearing deformation, of a standard hole that is loaded to obtain the ultimate strength equal to $3d_b tF_u$ in Equation 5.4 was on the order of the diameter of the bolt.

This apparent hole elongation results largely from bearing deformation of the material that is immediately adjacent to the bolt. The lower value of $2.4d_btF_u$ in Equation 5.3 provides a bearing strength limit-state that is attainable at reasonable deformation (¼ in.). Strength and deformation limits were thus used to jointly evaluate bearing strength test results for design.

When long-slotted holes are oriented with the long dimension perpendicular to the direction of load, the bending component of the deformation in the material between adjacent holes or between the hole and the edge of the plate is increased. The nominal bearing strength is limited to $2d_btF_u$, which again provides a bearing strength limit-state that is attainable at reasonable deformation.

The design bearing strength has been expressed as that of a single bolt, although it is really that of the connected material that is immediately adjacent to the bolt. In calculating the design bearing strength of a connected part, the total bearing strength of the connected part can be taken as the sum of the bearing strengths of the individual bolts.

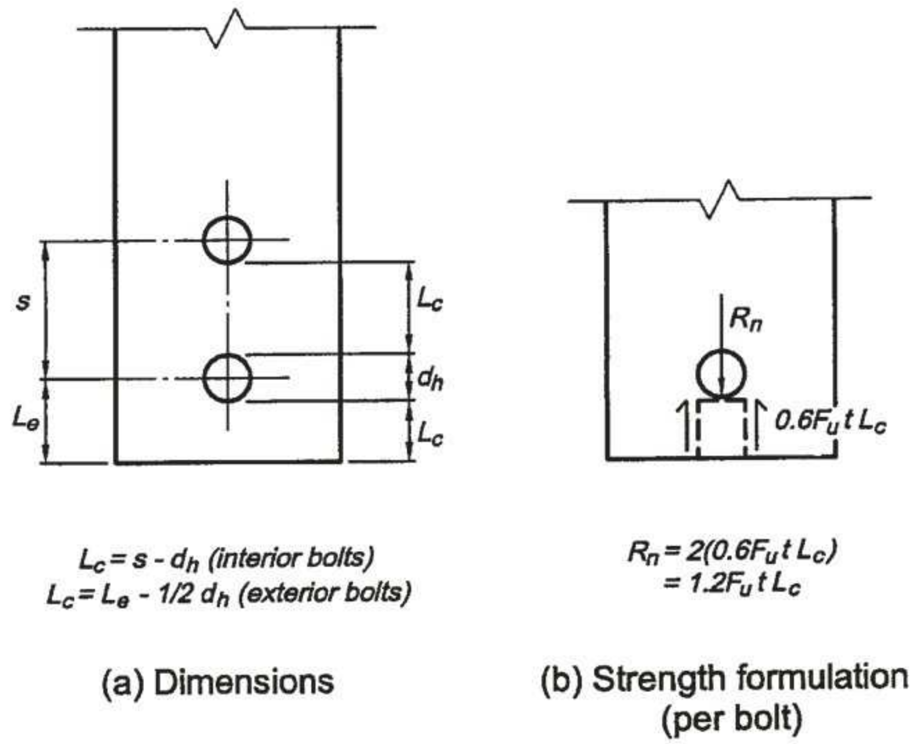


Figure. C-5.1. Bearing strength formulation.

5.4. Design Slip Resistance

Slip-critical connections shall be designed to prevent slip and for the limit states of bearing-type connections in accordance with Sections 5.1, 5.2 and 5.3. When slip-critical bolts pass through fillers, all *faying surfaces* subject to slip shall be prepared to achieve design slip resistance.

At LRFD load levels the design slip resistance is ϕR_n and at ASD load levels the allowable slip resistance is R_n/Ω where R_n, ϕ and Ω are defined below.

The nominal slip resistance per bolt for the limit state of slip shall be determined as follows:

$$R_n = \mu D_u h_f T_b n_s k_{sc} \qquad \text{(Equation 5.6)}$$

For standard size and short-slotted holes perpendicular to the direction of the load

$\phi = 1.00$ (LRFD) $\qquad \Omega = 1.50$ (ASD)

For oversized and short-slotted holes parallel to the direction of the load

$\phi = 0.85$ (LRFD) $\qquad \Omega = 1.76$ (ASD)

For long-slotted holes

$\phi = 0.70$ (LRFD) $\qquad \Omega = 2.14$ (ASD)

Where

μ = mean slip coefficient for Class A or B surfaces, as applicable, and determined as follows, or as established by tests:

(1) For Class A surfaces (unpainted clean mill scale steel surfaces or surfaces with Class A coatings on blast-cleaned steel or hot-dipped galvanized and roughened surfaces)

$\mu = 0.30$

(2) For Class B surfaces (unpainted blast-cleaned steel surfaces or surfaces with Class B coatings on blast-cleaned steel)

$\mu = 0.50$

D_u = 1.13; a multiplier that reflects the ratio of the mean installed bolt pretension to the specified minimum bolt pretension; the use of other values may be approved by the engineer of record.

T_b = minimum fastener tension given in Table 8.1, kips

h_f = factor for fillers, determined as follows:

(1) Where there are no fillers or bolts have been added to distribute loads in the filler

$h_f = 1.0$

(2) Where bolts have not been added to distribute the load in the filler:

(i) For one filler between connected parts

$h_f = 1.0$

(ii) For two or more fillers between connected parts

$h_f = 0.85$

n_s = number of slip planes required to permit the connection to slip

$$k_{sc} = 1 - \frac{T_u}{D_u T_b n_b} \geq 0 \qquad \text{(LRFD)}$$

$$= 1 - \frac{1.5T_a}{D_u T_b n_b} \geq 0 \qquad \text{(ASD)}$$

Where

T_a = required tension force using *ASD load combinations*, kips
T_u = required tension force using *LRFD load combinations*, kips
n_b = number of bolts carrying the applied tension

Commentary:

The *nominal strength* R_n represents the mean resistance, which is a function of the *mean slip coefficient* μ and the specified minimum bolt pretension (clamping force) T_m. The 1.13 multiplier in Equation 5.6 accounts for the statistical relationship between calculated slip resistance and historical measured test results. In the absence of other field test data, this value is used for all methods.

For most applications, the assumption that the slip resistance at each fastener is equal and additive with that at the other fasteners is based on the fact that all locations must develop the slip force before a total *joint* slip can occur at that plane. Similarly, the forces developed at various slip planes do not necessarily develop simultaneously, but one can assume that the full slip resistances must be mobilized at each plane before full *joint* slip can occur.

The nominal resistance in Section 5.4 results in a reliability consistent with the reliability of structural member design. The engineer should not need to design to a higher reliability in normal structural applications. The following comments reflect the collective thinking of the Council and are provided as guidance and an indication of the intent of the Specification (see also the Commentary to Sections 4.2 and 4.3):

(1) In built-up compression members, such as double-angle struts in trusses, a small relative slip between the elements especially at the end *connections* can increase the

effective length of the combined cross-section to that of the individual components and significantly reduce the compressive strength of the strut. Therefore, the *connection* between the elements at the ends of built-up members are checked to prevent slip, whether or not a *slip-critical joint* is required for serviceability. As given by Sherman and Yura (1998), the required slip resistance is $0.008P_uLQ/I$, where P_u is the axial compressive force in the built-up member, kips, L is the total length of the built-up member, in., Q is the first moment of area of one component about the axis of buckling of the built-up member, in.^{3}, and I is the moment of inertia of the built-up member about the axis of buckling, in.^{4};

(2) In *joints* with long-slotted holes that are parallel to the direction of the applied load, the *joint* is designed to prevent slip, however, the effect of the factored loads acting on the deformed structure (deformed by the maximum amount of slip in the long slots at all locations) must be included in the structural analysis; and,

(3) In *joints* subject to fatigue, design should be based upon service-load criteria and the design slip resistance of the governing cyclic design specification because fatigue is a function of the service load performance rather than that of the factored load.

Extensive data developed through research sponsored by the Council and others has been statistically analyzed to provide improved information on slip probability of *joints* in which the bolts have been pretensioned to the requirements of Table 8.1. Two variables, the *mean slip coefficient* of the *faying surfaces* and the bolt pretension, were found to affect the slip resistance of *joints*. Field studies (Kulak and Birkemoe, 1993) of installed bolts in various structural applications indicate that the Table 8.1 pretensions have been achieved as anticipated in the laboratory research.

An examination of the slip-coefficient data for a wide range of surface conditions indicates that the data are distributed normally and the standard deviation is essentially the same for each surface condition class. This means that different reduction factors should be applied to classes of surfaces with different *mean slip coefficients*—the smaller the mean value of the coefficient of friction, the smaller (more severe) the appropriate reduction factor—to provide equivalent reliability of slip resistance.

The bolt clamping force data indicate that bolt pretensions are distributed normally for each pretensioning method. However, the data also indicate that the mean value of the bolt pretension is different for each method. If the calibrated wrench method is used to pretension ASTM A325 bolts, the mean value of bolt pretension is about 1.13 times the specified minimum pretension in Table 8.1. If the turn-of-nut pretensioning method is used, the mean pretension is about 1.35 times the specified minimum pretension for ASTM A325 bolts and about 1.26 for ASTM A490 bolts.

The combined effects of the variability of the *mean slip coefficient* and bolt pretension have been accounted for approximately in the single value of the slip probability factor D_u in the equation for nominal slip resistance. This implies that slip will not occur with a reliability index, beta, of at least 2.6 regardless of the method of pretensioning.

The calibrated wrench installation method targets a specific bolt pretension, which is 5 percent greater than the specified minimum value given in Table 8.1. Thus, regardless of the actual strength of production bolts, this target value is unique for a given fastener grade. On the other hand, the turn-of-nut installation method imposes an elongation on the fastener. Consequently, the inherent strength of the bolts being installed will be reflected in the resulting pretension because this elongation will bring the fastener to its proportional limit under combined torsion and tension. As a result of these differences, the mean value and nature of the frequency distribution of pretensions for the two installation methods differ. Turn-of-nut installations result in higher mean levels of pretension than do calibrated wrench installations. Twist-off tension control bolt and direct tension indicator pretensions are similar to those of calibrated wrench. These differences were taken into account when the design criteria for *slip-critical joints* were developed.

In any of the foregoing installation methods, it can be expected that a portion of the bolt assembly (the threaded portion of the bolt within the *grip* length and/or the engaged threads of the nut and bolt) will reach the inelastic region of behavior. This permanent distortion has no undesirable effect on the subsequent performance of the bolt.

Although the design philosophy for *slip-critical joints* presumes that they do not slip into bearing when subject to loads in the service range, it is mandatory that *slip-critical joints* also meet the requirements of Sections 5.1, 5.2 and 5.3. Thus, they must meet the strength requirements to resist the factored loads as *shear/bearing joints*.

Section 3.2.2(b) permits the *Engineer of Record* to authorize the use of *faying surfaces* with a *mean slip coefficient,* μ, that is less than 0.50 (Class B) and other than 0.30 (Class A). This authorization requires that the *mean slip coefficient,* μ, must be determined in accordance with Appendix A.

Prior to the 1994 edition of this Specification, μ for galvanized surfaces was taken as 0.40. This value was reduced to 0.35 in the 1994 edition for better agreement with the available research (Kulak et al., 1987; pp. 78-82) and to 0.30 in the 2014 edition to be consistent with slip coefficients cited previously.

5.5. Tensile Fatigue

The tensile stress in the bolt that results from the cyclic application of externally applied service loads and the prying force, if any, but not the pretension, shall not exceed the stress in Table 5.2. The nominal diameter of the bolt shall be used in calculating the bolt stress. The connected parts shall be proportioned so that the calculated prying force does not exceed 30 percent of the externally applied load. *Joints* that are subject to tensile fatigue loading shall be specified as pretensioned in accordance with Section 4.2 or slip-critical in accordance with Section 4.3.

Table 5.2. Maximum Tensile Stress for Fatigue Loading

Number of Cycles	Maximum Bolt Stress for Design at Service Loads [a], ksi	
	ASTM A325 or F1852	ASTM A490 or F2280
Not more than 20,000	45	57
From 20,000 to 500,000	40	49
More than 500,000	31	38

[a] Including the effects of *prying action*, if any, but excluding the pretension.

Commentary:
As described in the Commentary to Section 5.1, *high-strength bolts* in *pretensioned joints* that are nominally loaded in tension will experience little, if any, increase in axial stress under service loads. For this reason, pretensioned bolts are not adversely affected by repeated application of service-load tensile stress. However, care must be taken to ensure that the calculated prying force is a relatively small part of the total applied bolt tension (Kulak et al., 1987; p. 272). The provisions that cover bolt fatigue in tension are based upon research results where various single-bolt assemblies and *joints* with bolts in tension were subjected to repeated external loads that produced fatigue failure of the pretensioned fasteners. A limited range of prying effects was investigated in this research.

SECTION 6. USE OF WASHERS

6.1. Snug-Tightened Joints

Washers are not required in snug-tightened joints, except as required in Sections 6.1.1 and 6.1.2.

6.1.1. Sloping Surfaces: When the outer face of the *joint* has a slope that is greater than 1:20 with respect to a plane that is normal to the bolt axis, an ASTM F436 beveled washer shall be used to compensate for the lack of parallelism.

6.1.2. Slotted Hole: When a slotted hole occurs in an outer ply, an ASTM F436 washer or 5⁄16 in. thick common plate washer shall be used as required to completely cover the hole.

6.2. Pretensioned Joints and Slip-Critical Joints

Washers are not required in *pretensioned joints* and *slip-critical joints*, except as required in Sections 6.1.1, 6.1.2, 6.2.1, 6.2.2, 6.2.3, 6.2.4 and 6.2.5.

6.2.1. Specified Minimum Yield Strength of Connected Material Less Than 40 ksi: When ASTM A490 or F2280 bolts are pretensioned in connected material of specified minimum yield strength less than 40 ksi, ASTM F436 washers shall be used under both the bolt head and nut, except that a washer is not needed under the head of an ASTM F2280 round head twist-off bolt.

6.2.2. Calibrated Wrench Pretensioning: When the calibrated wrench pretensioning method is used, an ASTM F436 washer shall be used under the turned element.

6.2.3. Twist-Off-Type Tension-Control Bolt Pretensioning: When the twist-off-type tension-control bolt pretensioning method is used, an ASTM F436 washer shall be used under the nut as part of the *fastener assembly*.

6.2.4. Direct-Tension-Indicator Pretensioning: When the direct-tension-indicator pretensioning method is used, an ASTM F436 washer shall be used as follows:

(1) When the nut is turned and the direct tension indicator is located under the bolt head, an ASTM F436 washer shall be used under the nut;
(2) When the nut is turned and the direct tension indicator is located under the nut, an ASTM F436 washer shall be used between the nut and the direct tension indicator;
(3) When the bolt head is turned and the direct tension indicator is located under the nut, an ASTM F436 washer shall be used under the bolt head; and,

Table 6.1. Washer Requirements for Pretensioned and Slip-Critical Bolted *Joints* with Oversized and Slotted Holes in the Outer Ply

<table>
<tr><th rowspan="2">ASTM Designation</th><th rowspan="2">Nominal Bolt Diameter, d_b, in.</th><th colspan="3">Hole Type in Outer Ply</th></tr>
<tr><th>Oversized</th><th>Short-Slotted</th><th>Long-Slotted</th></tr>
<tr><td>A325 or F1852</td><td>½ – 1 ½</td><td colspan="2" rowspan="2">ASTM F436 [a]</td><td rowspan="2">5⁄16 in. thick plate washer or continuous bar [b,c]</td></tr>
<tr><td rowspan="2">A490 or F2280</td><td>≤ 1</td></tr>
<tr><td>> 1</td><td colspan="2">ASTM F436 extra thick [a,b,d]</td><td>ASTM F436 washer with either a ⅜ in. thick plate washer or continuous bar [b,c]</td></tr>
<tr><td colspan="5">[a] This requirement shall not apply to heads of round head tension-control bolt assemblies that meet the requirements in Section 2.7 and provide a bearing circle diameter that meets the requirements of ASTM F1852 or F2280.
[b] See ASTM F436 Section 1.2.2.4. Multiple washers with a combined thickness of 5⁄16 in. or larger do not satisfy this requirement.
[c] The plate washer or bar shall be of structural-grade steel material, but need not be hardened.
[d] Alternatively, a ⅜ in. thick plate washer and an ordinary thickness F436 washer may be used. The plate washer need not be hardened.</td></tr>
</table>

(4) When the bolt head is turned and the direct tension indicator is located under the bolt head, an ASTM F436 washer shall be used between the bolt head and the direct tension indicator.

6.2.5. Oversized or Slotted Hole: When an oversized or slotted hole occurs in an outer ply, the washer requirements shall be as given in Table 6.1. The washer used shall be of sufficient size to completely cover the hole.

Commentary:

It is important that shop drawings and *connection* details clearly reflect the number and disposition of washers when they are required, especially the thick hardened washers or plate washers that are required for some slotted hole applications. The total thickness of washers in the *grip* affects the length of bolt that must be supplied and used.

The primary function of washers is to provide a hardened non-galling surface under the turned element, particularly for torque-based pretensioning methods such as the calibrated wrench pretensioning method and twist-off-type tension-control bolt pretensioning method. Circular flat washers that meet the requirements of ASTM F436 provide both a hardened non-galling surface and an increase in bearing area that is approximately 50 percent larger than that provided by a heavy-hex bolt head or nut. However, tests have shown that washers of the standard 5⁄32 in. thickness have a minor influence on the pressure distribution of the induced bolt pretension. Furthermore, they showed that a larger thickness is required when ASTM A490 bolts are used with material that has a minimum specified yield strength that is less than 40 ksi. This is necessary to mitigate the effects of local yielding of the material in the vicinity of the contact area of the head and nut. The requirement for standard thickness hardened washers, when such washers are specified, is waived for alternative design fasteners that incorporate a bearing surface under the head of the same diameter as the hardened washer.

With the 2011 revision of ASTM F436, special 5⁄16 in.-thick ASTM F436 washers are now called "extra thick". Extra thick ASTM F436 washers are required to cover oversized and short-slotted holes in external plies, when ASTM A490 or F2280 bolts of diameter larger than 1 in. are used, except as permitted by Table 6.1 footnotes a and d. This was found necessary to distribute the high clamping pressure so as to prevent collapse of the hole perimeter and enable the development of the desired clamping force. Preliminary investigation has shown that a similar but less severe deformation occurs when oversized or slotted holes are in the interior plies. The reduction in clamping force may be offset by "keying," which tends to increase the resistance to slip. These effects are accentuated in *joints* of thin plies. When long-slotted holes occur in an outer ply, 3⁄8 in. thick plate washers or continuous bars and one ASTM F436 washer are required in Table 6.1. This requirement can be satisfied with material of any structural grade. Alternatively, either of the following options can be used:

(1) The use of material with F_y greater than 40 ksi will eliminate the need to also provide ASTM F436 washers in accordance with the requirements in Section 6.2.1 for ASTM A490 or F2280 bolts of any diameter; or,
(2) Material with F_y equal to or less than 40 ksi can be used with ASTM F436 washers in accordance with the requirements in Section 6.2.1.

This specification previously required a washer under bolt heads with a bearing area smaller than that provided by an ASTM F436 washer. Tests indicate that the pretension achieved with a bolt having the minimum ASTM F1852 or F2280 bearing circle diameter is the same as that of a bolt with the larger bearing circle diameter equal to the size of an ASTM F436 washer, provided that the hole size meets the RCSC Specification limitations (Schnupp, 2003).

SECTION 7. PRE-INSTALLATION VERIFICATION

The requirements in this Section shall apply only as indicated in Section 8.2 to verify that the *fastener assemblies* and pretensioned installation procedures perform as required prior to installation.

7.1. Tension Calibrator

A *tension calibrator* shall be used where bolts are to be installed in *pretensioned joints* and *slip-critical joints* to:

(1) Confirm the suitability of the complete *fastener assembly*, including lubrication, for pretensioned installation; and,
(2) Confirm the procedure and proper use by the bolting crew of the pretensioning method to be used.

The accuracy of a hydraulic *tension calibrator* shall be confirmed through calibration at least annually.

Commentary:
A *tension calibrator* is a device that indicates the pretension that is developed in a bolt. It must be readily available whenever *high-strength bolts* are to be pretensioned. A bolt *tension calibrator* is essential for:

(1) The pre-installation verification of the suitability of the *fastener assembly*, including the lubrication that is applied by the *manufacturer* or specially applied, to develop the specified minimum pretension;
(2) Verifying the adequacy and proper use of the specified pretensioning method to be used;
(3) Determining the installation torque for the calibrated wrench pretensioning method; and,
(4) Determining an arbitration torque as specified in Section 10, if required to resolve dispute.

Hydraulic *tension calibrators* undergo a slight deformation during bolt pretensioning. Hence, when bolts are pretensioned according to Section 8.2.1, the nut rotation corresponding to a given pretension reading may be somewhat larger than it would be if the same bolt were pretensioned in a solid steel assembly. Stated differently, the reading of a hydraulic *tension calibrator* tends to underestimate the pretension that a given rotation of the turned element would induce in a bolt in a *pretensioned joint*.

Direct tension indicators (DTIs) may be used as tension calibrators, except in the case of turn-of-nut installation. This method is especially useful for, but not restricted to, bolts that are too short to fit into a hydraulic *tension calibrator*. The DTIs to be used for verification testing must first have the

Table 7.1 Minimum Bolt Pretension for Pre-Installation Verification

Nominal Bolt Diameter, d_b, in.	Minimum Bolt Pretension for Pre-Installation Verification, kips [a]	
	ASTM A325 and F1852	ASTM A490 and F2280
½	13	16
⅝	20	25
¾	29	37
⅞	41	51
1	54	67
1⅛	59	84
1¼	75	107
1⅜	89	127
1½	108	155

[a] Equal to 1.05 times the specified minimum bolt pretension required in Table 8.1, rounded to the nearest kip.

average gap determined for the specific level of pretension required by Table 7.1, measured to the nearest 0.001 in. This is termed the “calibrated gap.” Such measurements should be made for each lot of DTIs being used for verification testing, termed the “verification lot.” The fastener assembly may then be installed in a standard size hole with the additional verification DTI. The prescribed pretensioning procedure is followed, and it is verified that the average gap in the verification DTI is equal to or less than the calibrated gap for the verification lot. For calibrated wrench installation, the verification DTI should be placed at the fastener end opposite the installation wrench. For twist-off bolt installation, the verification DTI must be placed beneath the bolt head, with an additional ASTM F436 washer between bolt head and verification DTI, and the bolt head is not permitted to turn. For DTI installation, the verification DTI must be placed at the end opposite the placement of the production DTI.

This technique cannot be used for the turn-of-nut method because the deformation of the DTI consumes a portion of the turns provided. For turn-of-nut pre-installation verification of bolts too short to fit into a hydraulic calibration device, installing the fastener assembly in a solid plate with the proper size hole and applying the required turns is adequate. No verification is required for achieved pretension to meet Table 7.1.

7.2. Required Testing

A representative sample of not fewer than three complete *fastener assemblies* of each combination of diameter, length, grade and *lot* to be used in the work shall be checked at the site of installation in a *tension calibrator* to verify that the pretensioning method develops a pretension that is equal to or greater than that specified in Table 7.1. Washers shall be used in the pre-installation verification assemblies as required in the work in accordance with the requirements in Section 6.2.

If the actual pretension developed in any of the *fastener assemblies* is less than that specified in Table 7.1, the cause(s) shall be determined and resolved before the *fastener assemblies* are used in the work. Cleaning, lubrication and retesting of these *fastener assemblies*, except ASTM F1852 or F2280 twist-off-type tension-control bolt assemblies, (see Section 2.2) are permitted, provided that all assemblies are treated in the same manner.

Impact wrenches, if used, shall be of adequate capacity and supplied with sufficient air to perform the required pretensioning of each bolt within approximately 10 seconds for bolts to 1¼-in. diameter, and within approximately 15 seconds for larger bolts.

Commentary:

The fastener components listed in Section 1.5 are manufactured under separate ASTM specifications, each of which includes tolerances that are appropriate for the individual component covered. While these tolerances are intended to provide for a reasonable and workable fit between the components when used in an assembly, the cumulative effect of the individual tolerances permits a significant variation in the installation characteristics of the complete *fastener assembly*. It is the intent in this Specification that the responsibility rests with the *supplier* for proper performance of the *fastener assembly*, the components of which may have been produced by more than one *manufacturer*.

When pretensioned installation is required, it is essential that the effects of the accumulation of tolerances, surface condition and lubrication be taken into account. Hence, pre-installation verification testing of the complete *fastener assembly* is required as indicated in Section 8 to ensure that the *fastener assemblies* and installation method to be used in the work will provide a pretension that exceeds those specified in Table 8.1. It is not, however, intended simply to verify conformance with the individual ASTM specifications.

It is recognized in this Specification that a natural scatter is found in the results of the pre-installation verification testing that is required in Section 8. Furthermore, it is recognized that the pretensions developed in tests of a representative sample of the fastener components that will be installed in the work must be slightly higher to provide confidence that the majority of *fastener assemblies* will achieve the minimum required pretension as given in Table 8.1. Accordingly, the minimum pretension to be used in pre-installation verification is 1.05 times that required for installation and inspection, rounded to the nearest kip.

Pre-installation verification testing of as-received bolts and nuts is also a requirement in this Specification because of instances of under-strength and counterfeit bolts and nuts. Pre-installation verification testing provides a practical means for ensuring that non-conforming *fastener assemblies* are not incorporated into the work. Experience on many projects has shown that bolts and/or nuts not meeting the requirements of the applicable ASTM Specification would have been identified prior to installation if they had been tested as an assembly in a *tension calibrator*. The expense of replacing bolts installed in the structure when the non-conforming bolts were discovered at a later date would have been avoided.

Additionally, pre-installation verification testing clarifies for the bolting crew and the *inspector* the proper implementation of the selected pretensioning method and the adequacy of the installation equipment. It will also identify potential sources of problems, such as the need for lubrication to prevent failure of bolts by combined high torque with tension, under-strength assemblies resulting from excessive over-tapping of hot-dip galvanized nuts or other failures to meet strength or geometry requirements of applicable ASTM specifications.

The pre-installation verification requirements in this Section presume that *fastener assemblies* so verified will be pretensioned before the condition of the *fastener assemblies*, the equipment and the steelwork have changed significantly. Research by Kulak and Undershute (1998) on twist-off-type tension-control bolt assemblies from various *manufacturers* showed that installed pretensions could be a function of the time and environmental conditions of storage and exposure. The reduced performance of these bolts was caused by a deterioration of the lubricity of the assemblies. Furthermore, all bolt pretensioning that is achieved through rotation of the nut (or the head) is affected by the presence of torque, the excess of which has been demonstrated to adversely affect the development of the desired pretension. Thus, it is required that the condition of the *fastener assemblies* must be replicated in pre-installation verification. When time of exposure between the placement of *fastener assemblies* in the field work and the subsequent pretensioning of those *fastener assemblies* is of concern, pre-installation verification can be performed on *fastener assemblies* removed from the work or on extra *fastener assemblies* that, at the time of placement, were set aside to experience the same degree of exposure.

SECTION 8. INSTALLATION

Prior to installation, the fastener components shall be stored in accordance with Section 2.2. For *joints* that are designated in the contract documents as *snug-tightened joints*, the bolts shall be installed in accordance with Section 8.1. For *joints* that are designated in the contract documents as pretensioned or slip-critical, the bolts shall be installed in accordance with Section 8.2.

8.1. Snug-Tightened Joints

All bolt holes shall be aligned to permit insertion of the bolts without undue damage to the threads. Bolts shall be placed in all holes with washers positioned as required in Section 6.1 and nuts threaded to complete the assembly. Compacting the *joint* to the snug-tight condition shall progress systematically from the most rigid part of the *joint*. Snug tight is the condition that exists when all of the plies in a *connection* have been pulled into *firm contact* by the bolts in the *joint* and all of the bolts in the *joint* have been tightened sufficiently to prevent the removal of the nuts without the use of a wrench.

Commentary:

As discussed in the Commentary to Section 4, the bolted *joints* in most shear *connections* and in many tension *connections* can be specified as *snug-tightened joints*. The snug tightened condition is typically achieved with a few impacts of an impact wrench, application of an electric torque wrench until the wrench begins to slow or the full effort of a worker on an ordinary spud wrench. More than one cycle through the bolt pattern may be required to achieve the *snug-tightened joint*. The splines on twist-off type tension-control bolts may be twisted off or left in place in snug tightened joints.

The actual pretensions that result in individual fasteners in *snug-tightened joints* will vary from *joint* to *joint* depending upon the thickness, flatness, and degree of parallelism of the connected plies, as well as the effort applied. In most *joints*, plies of *joints* involving material of ordinary thickness and flatness can be drawn into complete contact at relatively low levels of pretension. However, in some *joints* in thick material or in material with large burrs, it may not be possible to reach continuous contact throughout the *faying surface* area as is commonly achieved in *joints* of thinner plates. This is generally not detrimental to the performance of the *joint*.

As used in Section 8.1, the term “undue damage” is intended to mean damage that would be sufficient to render the product unfit for its intended use.

The definition of a *snug-tightened joint* was temporarily changed in the 2009 specification and has now reverted back to the same definition specified in 2004. While the 2009 definition was suitable for inspection of bearing type connections, that definition was found to be inadequate to define a suitable starting point for the turn-of-nut method.

8.2. Pretensioned Joints and Slip-Critical Joints

One of the pretensioning methods in Sections 8.2.1 through 8.2.4 shall be used, except when alternative-design fasteners that meet the requirements of Section 2.8 or alternative washer-type indicating devices that meet the requirements of Section 2.6.2 are used, in which case, installation instructions provided by the *manufacturer* and approved by the *Engineer of Record* shall be followed.

Table 8.1. Minimum Bolt Pretension, *Pretensioned* and *Slip-Critical Joints*

Nominal Bolt Diameter, d_b, in.	Specified Minimum Bolt Pretension, T_m, kips [a]	
	ASTM A325 and F1852	ASTM A490 and F2280
½	12	15
⅝	19	24
¾	28	35
⅞	39	49
1	51	64
1⅛	56	80
1¼	71	102
1⅜	85	121
1½	103	148

[a] Equal to 70 percent of the specified minimum tensile strength of bolts as specified in ASTM Specifications for tests of full-size ASTM A325 and A490 bolts with UNC threads loaded in axial tension, rounded to the nearest kip.

When it is impractical to turn the nut, pretensioning by turning the bolt head is permitted while rotation of the nut is prevented, provided that the washer requirements in Section 6.2 are met. A pretension that is equal to or greater than the value in Table 8.1 shall be provided. The pre-installation verification procedures specified in Section 7 shall be performed using *fastener assemblies* that are representative of the condition of those that will be pretensioned in the work.

Pre-installation testing shall be performed for each fastener assembly lot prior to the use of that assembly lot in the work. The testing shall be done at the start of the work. For calibrated wrench pretensioning, this testing shall be performed daily for the calibration of the installation wrench.

Commentary:
The minimum pretension for ASTM A325 and A490 bolts is equal to 70 percent of the specified minimum tensile strength. As tabulated in Table 8.1, the values have been rounded to the nearest kip.

Four pretensioning methods are provided without preference in this Specification. Each method may be relied upon to provide satisfactory results when conscientiously implemented with the specified *fastener assembly* components in good condition. However, it must be recognized that misuse or abuse is possible with any method. With all methods, it is important to first install bolts in all holes of the *joint* and to compact the *joint* until the connected plies are in *firm contact*. Only after completion of this operation can the *joint* be reliably pretensioned. Both the initial phase of compacting the *joint* and the subsequent phase of pretensioning should begin at the most rigidly fixed or stiffest point.

In some *joints* in thick material, it may not be possible to reach continuous contact throughout the *faying surface* area, as is commonly achieved in *joints* of thinner plates. This is not detrimental to the performance of the *joint*. If the specified pretension is present in all bolts of the completed *joint*, the clamping force, which is equal to the total of the pretensions in all bolts, will be transferred at the locations that are in contact and the *joint* will be fully effective in resisting slip through friction.

If individual bolts are pretensioned in a single continuous operation in a *joint* that has not first been properly compacted or fitted up, the pretension in the bolts that are pretensioned first may be relaxed or removed by the pretensioning of adjacent bolts. The resulting reduction in total clamping force will reduce the slip resistance.

In the case of hot-dip galvanized coatings, especially if the *joint* consists of many plies of thickly coated material, relaxation of bolt pretension may be significant and re-pretensioning of the bolts may be required subsequent to the initial pretensioning. Munse (1967) showed that a loss of pretension of approximately 6.5 percent occurred for galvanized plates and bolts due to relaxation as compared with 2.5 percent for uncoated *joints*. This loss of bolt pretension occurred in five days; loss recorded thereafter was negligible. Either this loss can be allowed for in design, or pretension may be brought back to the prescribed level by re-pretensioning the bolts after an initial period of "settling-in."

As stated in the *Guide* (Kulak et al 1987; p. 61), "…it seems reasonable to expect an increase in bolt force relaxation as the *grip* length is decreased. Similarly, increasing the number of plies for a constant *grip* length might also lead to an increase in bolt relaxation."

8.2.1. Turn-of-Nut Pretensioning: All bolts shall be installed in accordance with the requirements in Section 8.1, with washers positioned as required in Section 6.2. Subsequently, the nut or head rotation specified in Table 8.2 shall be applied to all *fastener assemblies* in the *joint*, progressing systematically from the most rigid part of the *joint* in a manner that will minimize relaxation of previously pretensioned bolts. The part not turned by the wrench shall be prevented from rotating during this operation. Upon completion of the application of the required nut rotation for pretensioning, it is not permitted to turn the nut in the loosening direction except for the purpose of complete removal of the individual

Table 8.2. Nut Rotation from Snug-Tight Condition for Turn-of-Nut Pretensioning [a,b]

Bolt Length [c]	Disposition of Outer Faces of Bolted Parts		
	Both faces normal to bolt axis	One face normal to bolt axis, other sloped not more than 1:20 [d]	Both faces sloped not more than 1:20 from normal to bolt axis [d]
Not more than $4d_b$	⅓ turn	½ turn	⅔ turn
More than $4d_b$ but not more than $8d_b$	½ turn	⅔ turn	⅚ turn
More than $8d_b$ but not more than $12d_b$	⅔ turn	⅚ turn	1 turn

[a] Nut rotation is relative to bolt regardless of the element (nut or bolt) being turned. For all required nut rotations, the tolerance is plus 60 degrees (⅙ turn) and minus 30 degrees.

[b] Applicable only to *joints* in which all material within the *grip* is steel.

[c] When the bolt length exceeds $12d_b$, the required nut rotation shall be determined by actual testing in a suitable *tension calibrator* that simulates the conditions of solidly fitting steel.

[d] Beveled washer not used.

fastener assembly. Such fastener assemblies shall not be reused except as permitted in Section 2.3.3.

Commentary:
The turn-of-nut pretensioning method results in more uniform bolt pretensions than is generally provided with torque-controlled pretensioning methods. Strain-control that reaches the inelastic region of bolt behavior is inherently more reliable than a method that is dependent upon torque control. However, proper implementation is dependent upon ensuring that the *joint* is properly compacted prior to application of the required partial turn and that the bolt head (or nut) is securely held when the nut (or bolt head) is being turned.

Match-marking of the nut and protruding end of the bolt after snug-tightening can be helpful in the subsequent installation process and is certainly an aid to inspection.

As indicated in Table 8.2, there is no available research that establishes the required nut rotation for bolt lengths exceeding $12d_b$. The required turn for such bolts can be established on a case-by-case basis using a *tension calibrator*.

Significant research indicates that, at rotations exceeding those specified in Table 8.2, the level of pretension in the bolt will still be above the specified minimum pretension. In addition, the pretension is likely to remain high until just prior to twist-off of the fastener. The rotational margin against twist-off is large. A325 and A490 bolts ⅞ in. diameter and 5½ in. long with ⅛ in. of thread in the grip were tested. The installation condition for bolts of this length and diameter is ½ turn past snug. The A325 bolts did not fail until about 1¾ turns past snug, and the A490 bolts did not fail until about 1¼ turns past snug. Bolts with additional threads in the grip would exhibit additional ductility and tolerance for over-rotation.

Non-heat-treated nuts (A563 Grades C, C3 and D) manufactured near the lower range of permitted strength and hardness may strip if the bolt is tightened far beyond the specified level of pretension. For A325 bolts, nuts with a hardness of 89 HRB or higher should have adequate resistance to thread stripping. For A490 bolts, only heat-treated nuts are used. Deliberate over-rotation should be avoided to minimize risk of inducing nut stripping with low-hardness nuts, and inducing nut cracking with high-hardness and heat-treated nuts. Nut stripping or cracking would be considered cause for rejection of the installed fastener.

8.2.2. Calibrated Wrench Pretensioning: The pre-installation verification procedures specified in Section 7 shall be performed daily for the calibration of the installation wrench. Torque values determined from tables or from equations that claim to relate torque to pretension without verification shall not be used.

All bolts shall be installed in accordance with the requirements in Section 8.1, with washers positioned as required in Section 6.2. Subsequently, the installation torque determined in the pre-installation verification of the *fastener assembly* (Section 7) shall be applied to all bolts in the *joint*, progressing systematically from the most rigid part of the *joint* in a manner that will minimize relaxation of previously pretensioned bolts. The part not turned by the wrench shall be prevented from rotating during this operation. Application of the installation torque need not produce a relative rotation between the bolt and nut that is greater than the rotation specified in Table 8.2.

Commentary:
The scatter in installed pretension can be significant when torque-controlled methods of installation are used. The variables that affect the relationship between torque and pretension include:

(1) The finish and tolerance on the bolt and nut threads;
(2) The uniformity, degree and condition of lubrication;
(3) The shop or job-site conditions that contribute to dust and dirt or corrosion on the threads;
(4) The friction that exists to a varying degree between the turned element (the nut face or bearing area of the bolt head) and the supporting surface;
(5) The variability of the air supply parameters on impact wrenches that results from the length of air lines or number of wrenches operating from the same source;
(6) The condition, lubrication and power supply for the torque wrench, which may change within a work shift; and,
(7) The repeatability of the performance of any wrench that senses or responds to the level of the applied torque.

In the first edition of this Specification, which was published in 1951, a table of torque-to-pretension relationships for bolts of various diameters was included. It was soon demonstrated in research that a variation in the torque-to-pretension of as high as ±40 percent must be anticipated unless the relationship is established individually for each bolt *lot*, diameter, and fastener condition. Hence, in the 1954 edition of this Specification, recognition of relationships between torque and pretension in the form of tabulated values or equations was withdrawn. Recognition of the calibrated wrench pretensioning method was retained however until 1980, but with the requirement that the torque required for installation be determined specifically for the bolts being installed on a daily basis. Recognition of the method was withdrawn in 1980 because of the continuing controversy that resulted from the failure of users to adhere to the requirements for the valid use of the method during both installation and inspection.

In the 1985 edition of this Specification, the calibrated wrench pretensioning method was reinstated, but with more emphasis on detailed requirements that must be carefully followed. For calibrated wrench pretensioning, wrenches must be calibrated:

(1) Daily;
(2) When the *lot* of any component of the *fastener assembly* is changed;
(3) When the *lot* of any component of the *fastener assembly* is relubricated;
(4) When significant differences are noted in the surface condition of the bolt threads, nuts or washers; or,
(5) When any major component of the wrench including lubrication, hose and air supply are altered.

It is also important that:

(1) Fastener components be protected from dirt and moisture at the shop or job site as required in Section 2;

(2) Washers be used as specified in Section 6; and,
(3) The time between removal from *protected storage* and wrench calibration and final pretensioning be minimal.

8.2.3. Twist-Off-Type Tension-Control Bolt Pretensioning: Twist-off-type tension-control bolt assemblies that meet the requirements of ASTM F1852 or F2280 shall be used.

All *fastener assemblies* shall be installed in accordance with the requirements in Section 8.1 without severing the splined end and with washers positioned as required in Section 6.2. If a splined end is severed during this operation, the *fastener assembly* shall be removed and replaced. Subsequently, all bolts in the *joint* shall be pretensioned with the twist-off-type tension-control bolt installation wrench, progressing systematically from the most rigid part of the *joint* in a manner that will minimize relaxation of previously pretensioned bolts.

Commentary:
ASTM F1852 and F2280 twist-off-type tension-control bolt assemblies have a splined end that extends beyond the threaded portion of the bolt. During installation, this splined end is gripped by a specially designed wrench chuck and provides a means for turning the nut relative to the bolt. This product is, in fact, based upon a torque-controlled installation method to which the *fastener assembly* variables affecting torque that were discussed in the Commentary to Section 8.2.2 apply, except for wrench calibration, because torque is controlled within the *fastener assembly*.

Twist-off-type tension-control bolt assemblies must be used in the as-delivered, clean, lubricated condition as specified in Section 2. Adherence to the requirements in this Specification, especially those for storage, cleanliness and verification, is necessary for their proper use.

8.2.4. Direct-Tension-Indicator Pretensioning: Direct tension indicators that meet the requirements of ASTM F959 shall be used. The pre-installation verification procedures specified in Section 7 shall demonstrate that, when the pretension in the bolt reaches that required in Table 7.1, the gap is not less than the job inspection gap in accordance with ASTM F959.

All bolts shall be installed in accordance with the requirements in Section 8.1, with washers positioned as required in Section 6.2. The installer shall verify that the direct-tension-indicator protrusions have not been compressed to a gap that is less than the job inspection gap during this operation, and if this has occurred, the direct tension indicator shall be removed and replaced. Subsequently, all bolts in the *joint* shall be pretensioned, progressing systematically from the most rigid part of the *joint* in a manner that will minimize relaxation of previously pretensioned bolts. The installer shall verify that the direct tension indicator protrusions have been compressed to a gap that is less than the job inspection gap.

Commentary:

ASTM F959 direct tension indicators are recognized in this Specification as a bolt-tension-indicating device. Direct tension indicators are hardened, washer-shaped devices incorporating small arch-like protrusions on the bearing surface that are designed to deform in a controlled manner when subjected to compressive load.

During installation, care must be taken to ensure that the direct-tension-indicator arches are oriented to bear against the hardened bearing surface of the bolt head or nut, or against a hardened flat washer if used under turned element, whether that turned element is the nut or the bolt. Proper use and orientation is illustrated in Figure C-8.1.

In some cases, more than a single cycle of systematic partial pretensioning may be required to deform the direct-tension-indicator protrusions to the gap that is specified by the *manufacturer*. If the gaps fail to close or when the washer *lot* is changed, another verification procedure using the *tension calibrator* must be performed.

Provided the connected plies are in *firm contact*, partial compression of the direct tension indicator protrusions is commonly taken as an indication that the snug-tight condition has been achieved.

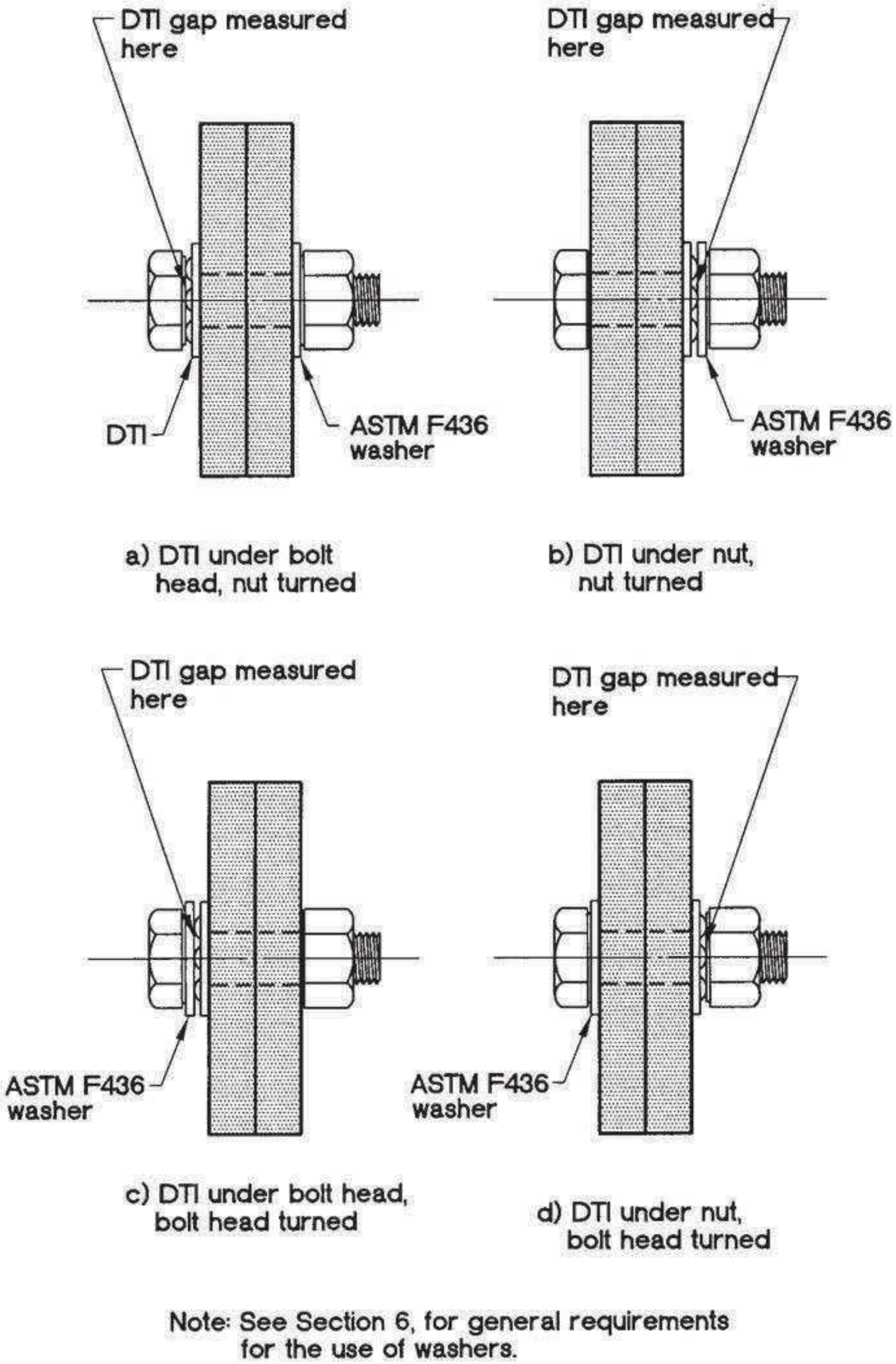


Figure C-8.1. Proper use and orientation of ASTM F959 direct-tension indicator

SECTION 9. INSPECTION

When inspection is required in the contract documents, the *inspector* shall ensure while the work is in progress that the requirements in this Specification are met. When inspection is not required in the contract documents, the *contractor* shall ensure while the work is in progress that the requirements in this Specification are met.

For *joints* that are designated in the contract documents as *snug-tightened joints*, the inspection shall be in accordance with Section 9.1. For *joints* that are designated in the contract documents as pretensioned, the inspection shall be in accordance with Section 9.2. For *joints* that are designated in the contract documents as slip-critical, the inspection shall be in accordance with Section 9.3.

9.1. Snug-Tightened Joints

Prior to the *start of work*, it shall be ensured that all fastener components to be used in the work meet the requirements in Section 2. Subsequently, it shall be ensured that all connected plies meet the requirements in Section 3.1 and all bolt holes meet the requirements in Sections 3.3 and 3.4. After the *connections* have been assembled, it shall be visually ensured that the plies of the connected elements have been brought into *firm contact* and that washers have been used as required in Section 6. It shall be determined that all of the bolts in the *joint* have been tightened sufficiently to prevent the turning of the nuts without the use of a wrench. No further evidence of conformity is required for *snug-tightened joints*. Where visual inspection indicates that the fastener may not have been sufficiently tightened to prevent the removal of the nut by hand, the inspector shall physically check for this condition for the fastener.

Commentary:
Inspection requirements for *snug-tightened joints* consist of verification that the proper fastener components were used, the connected elements were fabricated properly, the bolted *joint* was drawn into firm contact, and that the nuts could not be removed without the use of a wrench. Because pretension, beyond what is required to ensure that the nut cannot be removed from the bolt without the use of a wrench, is not required for the proper performance of a *snug-tightened joint*, the installed bolts should not be inspected to determine the actual installed pretension. Likewise, the arbitration procedures described in Section 10 are not applicable.

9.2. Pretensioned Joints

For *pretensioned joints*, the following inspection shall be performed in addition to that required in Section 9.1:

(1) When the turn-of-nut pretensioning method is used for installation, the inspection shall be in accordance with Section 9.2.1;
(2) When the calibrated wrench pretensioning method is used for installation, the inspection shall be in accordance with Section 9.2.2;

(3) When the twist-off-type tension-control bolt pretensioning method is used for installation, the inspection shall be in accordance with Section 9.2.3;
(4) When the direct-tension-indicator pretensioning method is used for installation, the inspection shall be in accordance with Section 9.2.4; and,
(5) When alternative-design fasteners that meet the requirements of Section 2.8 or alternative washer-type indicating devices that meet the requirements of Section 2.6.2 are used, the inspection shall be in accordance with inspection instructions provided by the *manufacturer* and approved by the *Engineer of Record*.

Commentary:
When *joints* are designated as pretensioned, they are not subject to the same faying-surface-treatment inspection requirements as is specified for *slip-critical joints* in Section 9.3.

9.2.1. Turn-of-Nut Pretensioning: The *inspector* shall observe the pre-installation verification testing required in Section 8.2. Subsequently, it shall be ensured by *routine observation* that the bolting crew properly rotates the turned element relative to the unturned element by the amount specified in Table 8.2. Alternatively, when *fastener assemblies* are match-marked after the initial fit-up of the *joint* but prior to pretensioning, visual inspection after pretensioning is permitted in lieu of routine observation. No further evidence of conformity is required. A pretension that is greater than the value specified in Table 8.1 shall not be cause for rejection. A rotation that exceeds the required values, including tolerance, specified in Table 8.2 shall not be cause for rejection.

Commentary:
Match-marking of the assembly during installation as discussed in the Commentary to Section 8.2.1 improves the ability to inspect bolts that have been pretensioned with the turn-of-nut pretensioning method. The sides of nuts and bolt heads that have been impacted sufficiently to induce the Table 8.1 minimum pretension will appear slightly peened.

The turn-of-nut pretensioning method, when properly applied and verified during the construction, provides more reliable installed pretensions than after-the-fact *inspection* testing. Therefore, proper inspection of the method is for the inspector to observe the required pre-installation verification testing of the *fastener assemblies* and the method to be used, followed by monitoring of the work in progress to ensure that the method is routinely and properly applied, or visual inspection of match-marked assemblies.

Some problems with the turn-of-nut pretensioning method have been encountered with hot-dip galvanized bolts. In some cases, the problems have been attributed to an especially effective lubricant applied by the *manufacturer* to ensure that bolts and nuts from stock will meet the ASTM Specification requirements for minimum turns testing of galvanized fasteners. Job-site testing in the *tension calibrator* demonstrated that the lubricant reduced the coefficient of friction between the bolt and nut to the degree that "the full effort of an ironworker using an ordinary spud wrench" to snug-tighten the *joint* actually induced the full required pretension. Also, because the nuts could be removed with an ordinary spud wrench, they were erroneously judged by the *inspector* to be improperly pretensioned. Excessively lubricated *high-strength bolts* may require significantly less torque to induce the specified pretension. The required pre-installation verification will reveal this potential problem.

Conversely, the absence of lubrication or lack of proper over-tapping can cause seizing of the nut and bolt threads, which will result in a twist failure of the bolt at less than the specified pretension. For such situations, the use of a *tension calibrator* to check the bolt assemblies to be installed will be helpful in establishing the need for lubrication.

9.2.2. Calibrated Wrench Pretensioning: The *inspector* shall observe the pre-installation verification testing required in Sections 8.2 and 8.2.2. Subsequently, it shall be ensured by *routine observation* that the bolting crew properly applies the calibrated wrench to the turned element. No further evidence of conformity is required. A pretension that is greater than the value specified in Table 8.1 shall not be cause for rejection.

Commentary:
For proper inspection of the method, it is necessary for the *inspector* to observe the required pre-installation verification testing of the *fastener assemblies* and the method to be used, followed by monitoring of the work in progress to ensure that the method is routinely and properly applied within the limits on time between removal from *protected storage* and final pretensioning.

9.2.3. Twist-Off-Type Tension-Control Bolt Pretensioning: The *inspector* shall observe the pre-installation verification testing required in Section 8.2. Subsequently, it shall be ensured by *routine observation* that the splined ends are properly severed during installation by the bolting crew. No further evidence of conformity is required. A pretension that is greater than the value specified in Table 8.1 shall not be cause for rejection.

Commentary:
The sheared-off splined end of an installed twist-off-type tension-control bolt assembly merely signifies that at some time the bolt was subjected to a torque that was adequate to cause the shearing. If in fact all fasteners are individually pretensioned in a single continuous operation without first properly snug-tightening all fasteners, they may give a misleading indication that the bolts have been properly pretensioned. Therefore, it is necessary that the *inspector* observe the required pre-installation verification testing of the *fastener assemblies*, and the ability to apply partial tension prior to twist-off is demonstrated. This is followed by monitoring of the work in progress to ensure

that the method is routinely and properly applied within the limits on time between removal from *protected storage* and final twist-off of the splined end.

9.2.4. Direct-Tension-Indicator Pretensioning: The *inspector* shall observe the pre-installation verification testing required in Sections 8.2 and 8.2.4. Subsequently, but prior to pretensioning, it shall be ensured by *routine observation* that the appropriate feeler gage is accepted in at least half of the spaces between the protrusions of the direct tension indicator and that the protrusions are properly oriented away from the work. If the appropriate feeler gage is accepted in fewer than half of the spaces, the direct tension indicator shall be removed and replaced. After pretensioning, it shall be ensured by *routine observation* that the appropriate feeler gage is refused entry into at least half of the spaces between the protrusions. No further evidence of conformity is required. A pretension that is greater than that specified in Table 8.1 shall not be cause for rejection.

Commentary:
When the *joint* is initially snug tightened, the direct tension indicator arch-like protrusions will generally compress partially. Whenever the snug-tightening operation causes one-half or more of the gaps between these arch-like protrusions to close to 0.015 in. or less (0.005 in. or less for coated direct tension indicators), the direct tension indicator should be replaced. Only after this initial operation should the bolts be pretensioned in a systematic manner. If the bolts are installed and pretensioned in a single continuous operation, direct tension indicators may give the *inspector* a misleading indication that the bolts have been properly pretensioned. Therefore, it is necessary that the *inspector* observe the required pre-installation verification testing of the *fastener assemblies* with the direct-tension indicators properly located and the method to be used. Following this operation, the *inspector* should monitor the work in progress to ensure that the method is routinely and properly applied.

9.3. Slip-Critical Joints

Prior to assembly, it shall be visually verified that the *faying surfaces* of *slip-critical joints* meet the requirements in Section 3.2.2. Subsequently, the inspection required in Section 9.2 shall be performed.

Commentary:
When *joints* are specified as slip-critical, it is necessary to verify that the *faying surface* condition meets the requirements as specified in the contract documents prior to assembly of the *joint* and that the bolts are properly pretensioned after they have been installed. Accordingly, the inspection requirements for *slip-critical joints* are identical to those specified in Section 9.2, with additional *faying surface* condition inspection requirements.

SECTION 10. ARBITRATION

When it is suspected after inspection in accordance with Section 9.2 or Section 9.3 that bolts in pretensioned or *slip-critical joints* do not have the proper pretension, the following arbitration procedure is permitted.

(1) A representative sample of five bolt and nut assemblies of each combination of diameter, length, grade and *lot* in question shall be installed in a *tension calibrator*. The material under the turned element shall be the same as in the actual installation, that is, structural steel or hardened washer. The bolt shall be partially pretensioned to approximately 15 percent of the pretension specified in Table 8.1. Subsequently, the bolt shall be pretensioned to the minimum value specified in Table 8.1;

(2) A manual torque wrench that indicates torque by means of a dial, or one that may be adjusted to give an indication that a defined torque has been reached, shall be applied to the pretensioned bolt. The torque that is necessary to rotate the nut or bolt head five degrees (approximately 1 in. at 12 in. radius) relative to its mating component in the tightening direction shall be determined. The arbitration torque shall be determined by rejecting the high and low values and averaging the remaining three; and,

(3) Bolts represented by the above sample shall be tested by applying, in the tightening direction, the arbitration torque to 10 percent of the bolts, but no fewer than two bolts, selected at random in each *joint* in question. If no nut or bolt head is turned relative to its mating component by application of the arbitration torque, the *joint* shall be accepted as properly pretensioned.

If verification of bolt pretension is required after the passage of a period of time and exposure of the completed *joints*, an alternative arbitration procedure that is appropriate to the specific situation shall be used.

If any nut or bolt is turned relative to its mating component by an attempted application of the arbitration torque, all bolts in the *joint* shall be tested. Those bolts whose nut or head is turned relative to its mating component by application of the arbitration torque shall be re-pretensioned by the Fabricator or Erector and reinspected. Alternatively, the Fabricator or Erector, at their option, is permitted to re-pretension all of the bolts in the *joint* and subsequently resubmit the *joint* for inspection.

Commentary:
When bolt pretension is arbitrated using torque wrenches after pretensioning, such arbitration is subject to all of the uncertainties of torque-controlled calibrated wrench installation that are discussed in the Commentary to Section 8.2.2. Additionally, the reliability of after-the-fact torque wrench arbitration is reduced by the absence of many of the controls that are necessary to minimize the variability of the torque-to-pretension relationship, such as:

(1) The use of hardened washers[2];
(2) Careful attention to lubrication; and,
(3) The uncertainty of the effect of passage of time and exposure in the installed condition.

Furthermore, in many cases such arbitration may have to be based upon an arbitration torque that is determined either using bolts that can only be assumed to be representative of the bolts used in the actual job or using bolts that are removed from completed *joints*. Ultimately, such arbitration may wrongly reject bolts that were subjected to a properly implemented installation procedure. The arbitration procedure contained in this Specification is provided, in spite of its limitations, as the most feasible available at this time.

Arbitration using an ultrasonic extensometer or a mechanical one capable of measuring changes in bolt length can be performed on a sample of bolts that is representative of those that have been installed in the work. Several *manufacturers* produce equipment specifically for this application. The use of appropriate techniques, which includes calibration, can produce a very accurate measurement of the actual pretension. The method involves measurement of the change in bolt length during the release of the nut, combined with either a load calibration of the removed *fastener assembly* or a theoretical calculation of the force corresponding to the measured elastic release or "stretch." Reinstallation of the released bolt or installation of a replacement bolt is required.

The required release suggests that the direct use of extensometers as an inspection tool be used in only the most critical cases. The problem of reinstallation may require bolt replacement unless torque can be applied slowly using a manual or hydraulic wrench, which will permit the restoration of the original elongation.

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[2] For example, because the reliability of the turn-of-nut pretensioning method is not dependent upon the presence or absence of washers under the turned element, washers are not generally required, except for other reasons as indicated in Section 6. Thus, in the absence of washers, after-the-fact, torque-based arbitration is particularly unreliable when the turn-of-nut pretensioning method has been used for installation.

APPENDIX A. TESTING METHOD TO DETERMINE THE SLIP COEFFICIENT FOR COATINGS USED IN BOLTED JOINTS

SECTION A1. GENERAL PROVISIONS

A1.1. Purpose and Scope

The purpose of this testing procedure is to determine the *mean slip coefficient* of a coating for use in the design of *slip-critical joints*. Adherence to this testing method provides that the creep deformation of the coating due to both the clamping force of the bolt and the service-load *joint* shear are such that the coating will provide satisfactory performance under sustained loading.

Commentary:

The Research Council on Structural Connections on June 14, 1984, first approved the testing method developed by Yura and Frank (1985). It has since been revised to incorporate changes resulting from the intervening years of experience with the testing method, and is now included as an appendix to this Specification.

The slip coefficient under short-term static loading has been found to be independent of the magnitude of the clamping force, variations in coating thickness and bolt hole diameter.

The proposed test methods are designed to provide the necessary information to evaluate the suitability of a coating for *slip-critical joints* and to determine the *mean slip coefficient* to be used in the design of the *joints*. The initial testing of the compression specimens provides a measure of the scatter of the slip coefficient.

The creep tests are designed to measure the creep behavior of the coating under the service loads, determined by the slip coefficient of the coating based upon the compression test results. The slip test conducted at the conclusion of the creep test is to ensure that the loss of clamping force in the bolt does not reduce the slip load below that associated with the design slip coefficient. ASTM A490 bolts are specified, since the loss of clamping force is larger for these bolts than that for ASTM A325 bolts. Qualification of the coating for use in a structure at an average thickness of 2 mils less than that to be used for the test specimen is to ensure that a casual buildup of the coating due to overspray and other causes does not jeopardize the coating's performance.

A1.2. Definition of Essential Variables

Essential variables are those that, if changed, will require retesting of the coating to determine its *mean slip coefficient*. The essential variables and the relationship of these variables to the limitations of application of the coating for structural *joints* are given below. The slip coefficient testing shall be repeated if there is any change in these essential variables.

A1.2.1. Time Interval: The time interval between application of the coating and the time of testing is an essential variable. The time interval must be recorded in hours and any special curing procedures detailed. Curing according to published *manufacturer's* recommendations would not be considered a special curing procedure. The coatings are qualified for use in structural *connections* that are assembled after coating for a time equal to or greater than the interval used in the test specimens. Special curing conditions used in the test specimens will also apply to the use of the coating in the structural *connections*.

A1.2.2. Coating Thickness: The coating thickness is an essential variable. The maximum average coating thickness, as per SSPC PA2 (SSPC 1993; SSPC 1991), allowed on the faying surfaces is 2 mils less than the average thickness, rounded to the nearest whole mil, of the coating that is used on the test specimens.

A1.2.3. Coating Composition and Method of Manufacture: The composition of the coating, including the thinners used, and its method of manufacture are essential variables.

A1.3. Retesting

A coating that fails to meet the creep or the post-creep slip test requirements in Section A4 may be retested in accordance with methods in Section A4 at a lower slip coefficient without repeating the static short-term tests specified in Section A3. Essential variables shall remain unchanged in the retest.

SECTION A2. TEST PLATES AND COATING OF THE SPECIMENS

A2.1. Test Plates

The test specimen plates for the short-term static tests are shown in Figure A1. The plates are 4 in. × 4 in. × ⅝ in. thick, with a 1 in. diameter hole drilled 1½ in. ± 1⁄16 in. from one edge. The test specimen plates for the creep tests are shown in Figure A2. The plates are 4 in. × 7 in. × ⅝ in. thick with two 1 in. diameter holes drilled 1½ in. ± 1⁄16 in. from each end. The edges of the plates may be milled, as-rolled or saw-cut; thermally cut edges are not permitted. The plates shall be flat enough to ensure that they will be in reasonably full contact over the *faying surface*. All burrs, lips or rough edges shall be removed. The arrangement of the specimen plates for the testing is shown in Figure A2. The plates shall be fabricated from a steel with a specified minimum yield strength that is between 36 and 50 ksi.

If specimens with more than one bolt are desired, the contact surface per bolt shall be 4 in. × 3 in. as shown for the single-bolt specimen in Figure A1.

Commentary:
The use of 1 in.-diameter bolt holes in the specimens is to ensure that adequate clearance is available for slip. Fabrication tolerances, coating buildup on the holes, and assembly tolerances tend to reduce the apparent clearances.

A2.2. Specimen Coating

Coatings are to be applied to the specimens in a manner that is consistent with that to be used in the actual intended structural application. The method of applying the coating and the surface preparation shall be given in the test report. The specimens are to be coated to an average thickness that is 2 mils greater than the maximum thickness to be used in the structure on both of the plate surfaces (the faying and outer surfaces). The thickness of the total coating and the primer, if used, shall be measured on the contact surface of the specimens. The thickness shall be measured in accordance with SSPC-PA2 (SSPC, 1993; SSPC, 1991). Two spot readings (six gage readings) shall be made for each contact surface. The overall average thickness from the three plates comprising a specimen is the average thickness for the specimen. This value shall be reported for each specimen. The average coating thickness of the creep specimens shall be calculated and reported.

The time between application of the coating and specimen assembly shall be the same for all specimens within ±4 hours. The average time shall be calculated and reported.

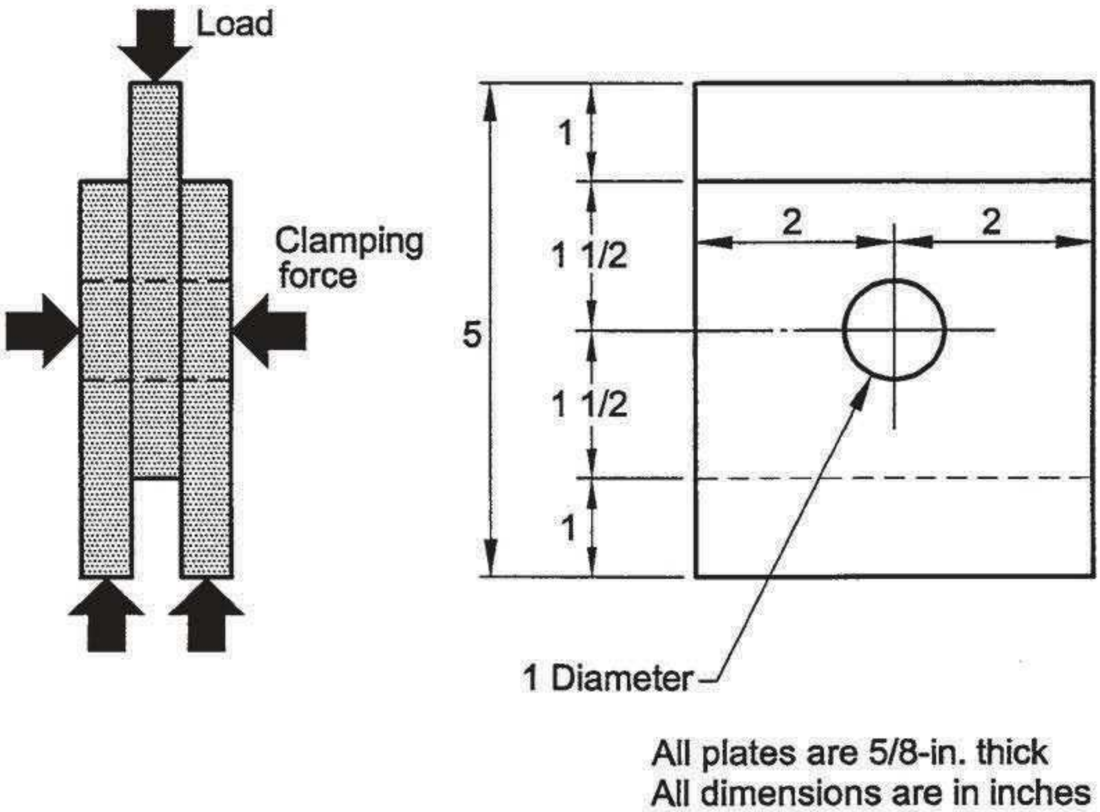

Figure A-1. Compression slip test specimen.

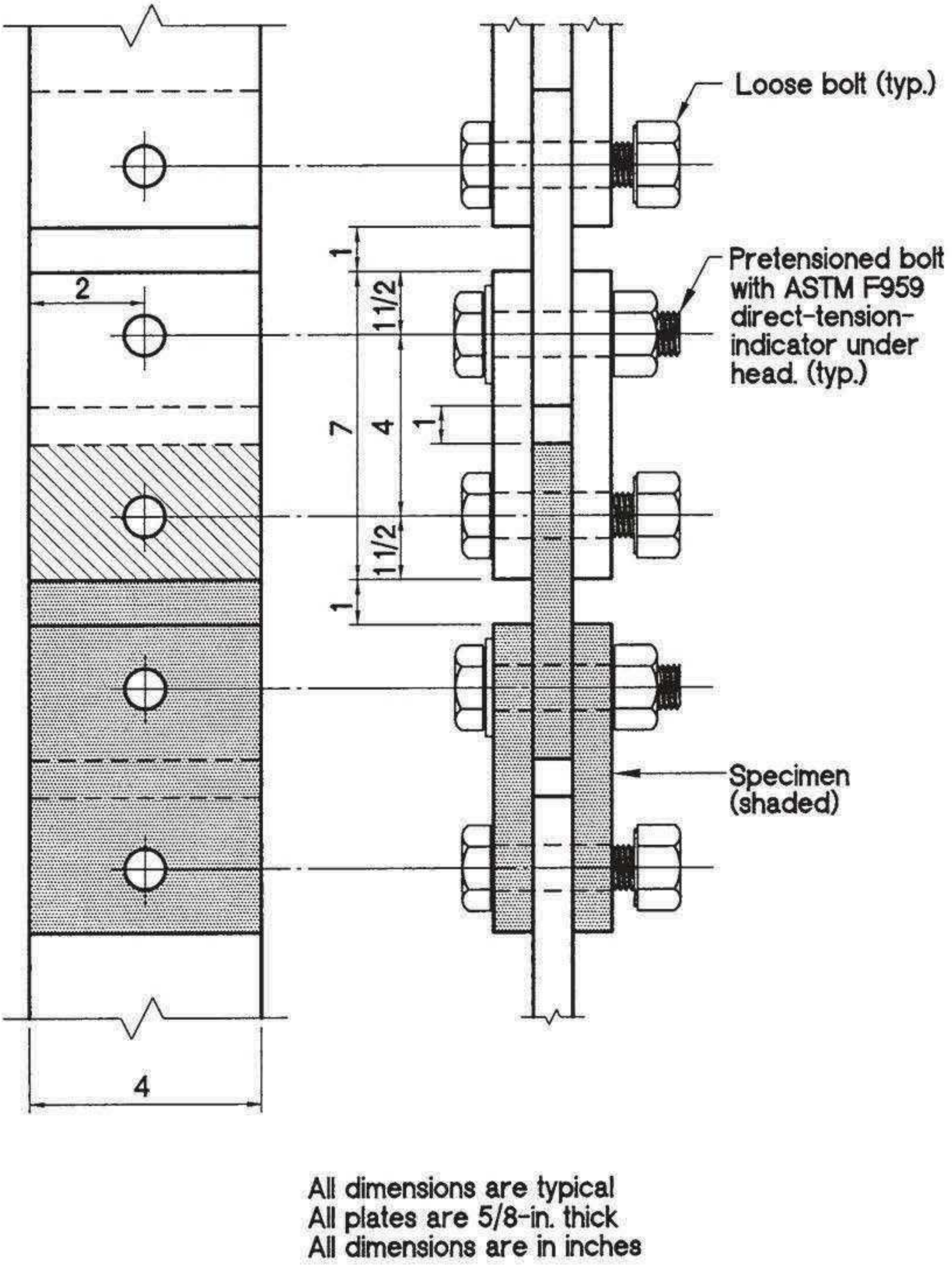

Figure A-2. Creep test specimen assembly.

SECTION A3. SLIP TESTS

The methods and procedures described herein are used to experimentally determine the *mean slip coefficient* under short-term static loading for *high-strength bolted joints*. The *mean slip coefficient* shall be determined by testing one set of five specimens.

Commentary:
The slip load measured in this setup yields the slip coefficient directly since the clamping force is controlled and measured directly. The resulting slip coefficient has been found to correlate with both tension and compression tests of bolted specimens. However, tests of bolted specimens revealed that the clamping force may not be constant but decreases with time due to the compressive creep of the coating on the *faying surfaces* and under the nut and bolt head. The reduction in clamping force can be considerable for *joints* with high clamping force and thick coatings (as much as a 20 percent loss). This reduction in clamping force causes a corresponding reduction in the slip load. The resulting reduction in slip load must be considered in the procedure used to determine the design allowable slip loads for the coating.

The loss in clamping force is a characteristic of the coating. Consequently, it cannot be accounted for by an increase in the factor of safety or a reduction in the clamping force used for design without unduly penalizing coatings that do not exhibit this behavior.

A3.1. Compression Test Setup

The test setup shown in Figure A3 has two major loading components, one to apply a clamping force to the specimen plates and another to apply a compressive load to the specimen so that the load is transferred across the *faying surfaces* by friction.

A3.1.1. Clamping Force System: The clamping force system consists of a ⅞ in. diameter threaded rod that passes through the specimen and a centerhole compression ram. An ASTM A563 grade DH nut is used at both ends of the rod and a hardened washer is used at each side of the test specimen. Between the ram and the specimen is a specially modified ⅞ in. diameter ASTM A563 grade DH nut in which the threads have been drilled out so that it will slide with little resistance along the rod. When oil is pumped into the centerhole ram, the piston rod extends, thus forcing the special nut against one of the outside plates of the specimen. This action puts tension in the threaded rod and applies a clamping force to the specimen, thereby simulating the effect of a pretensioned bolt. If the diameter of the centerhole ram is greater than 1 in., additional plate washers will be necessary at the ends of the ram. The clamping force system shall have a capability to apply a load of at least 49 kips and shall maintain this load during the test with an accuracy of 0.5 kips.

Commentary:
The slip coefficient can be easily determined using the hydraulic bolt test setup included in this Specification. The clamping force system simulates the clamping action of a pretensioned *high-strength bolt*. The centerhole ram

applies a clamping force to the specimen, simulating that due to a pretensioned bolt.

A3.1.2. Compressive Load System: A compressive load shall be applied to the specimen until slip occurs. This compressive load shall be applied with a compression test machine or a reaction frame using a hydraulic loading device. The loading device and the necessary supporting elements shall be able to support a force of 120 kips. The compression loading system shall have a minimum accuracy of 1 percent of the slip load.

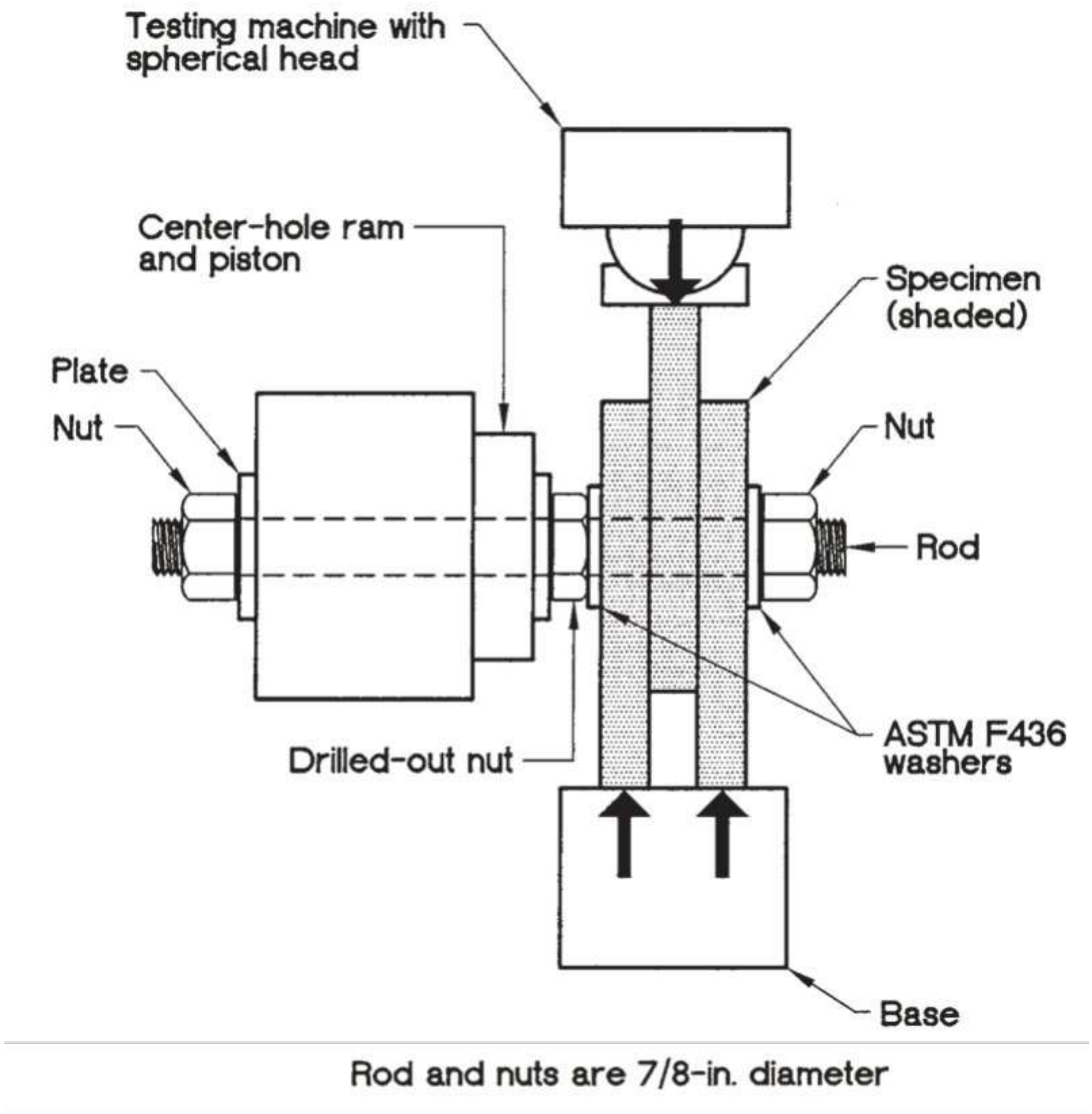

Figure A-3. Compression slip test setup.

A3.2. Instrumentation

A3.2.1. Clamping Force: The clamping force shall be measured within 0.5 kips. This is accomplished by measuring the pressure in the calibrated ram or placing a load cell in series with the ram.

A3.2.2. Compression Load: The compression load shall be measured during the test by direct reading from a compression testing machine, a load cell in series with the specimen and the compression loading device or pressure readings on a calibrated compression ram.

A3.2.3. Slip Deformation: The displacement of the center plate relative to the two outside plates shall be measured. This displacement, called "slip" for simplicity, shall be the average or that which occurs at the centerline of the specimen. This can be accomplished by using the average of two gages placed on the two exposed edges of the specimen or by monitoring the movement of the loading head relative to the base. If the latter method is used, due regard shall be taken for any slack that may be present in the loading system prior to application of the load. Deflections shall be measured by dial gages or any other calibrated device that has an accuracy of at least 0.001 in.

A3.3. Test Procedure

The specimen shall be installed in the test setup as shown in Figure A3. Before the hydraulic clamping force is applied, the individual plates shall be positioned so that they are in, or close to, full bearing contact with the ⅞ in. threaded rod in a direction that is opposite to the planned compressive loading to ensure obvious slip deformation. Care shall be taken in positioning the two outside plates so that the specimen is perpendicular to the base with both plates in contact with the base. After the plates are positioned, the centerhole ram shall be engaged to produce a clamping force of 49 kips. The applied clamping force shall be maintained within ±0.5 kips during the test until slip occurs.

The spherical head of the compression loading machine shall be brought into contact with the center plate of the specimen after the clamping force is applied. The spherical head or other appropriate device ensures concentric loading. When 1 kip or less of compressive load is applied, the slip gages shall be engaged or attached. The purpose of engaging the deflection gage(s), after a slight load is applied, is to eliminate initial specimen settling deformation from the slip reading.

When the slip gages are in place, the compression load shall be applied at a rate that does not exceed 25 kips per minute nor 0.003 in. of slip displacement per minute until the slip load is reached. The test should be terminated when a slip of 0.05 in. or greater is recorded. The load-slip relationship should preferably be monitored continuously on an X-Y plotter throughout the test, but in lieu of continuous data, sufficient load-slip data shall be recorded to evaluate the slip load defined below.

A3.4. Slip Load

Typical load-slip response is shown in Figure A4. Three types of curves are usually observed and the slip load associated with each type is defined as follows:

Curve (a) Slip load is the maximum load, provided this maximum occurs before a slip of 0.02 in. is recorded.
Curve (b) Slip load is the load at which the slip rate increases suddenly.
Curve (c) Slip load is the load corresponding to a deformation of 0.02 in. This definition applies when the load vs. slip curves show a gradual change in response.

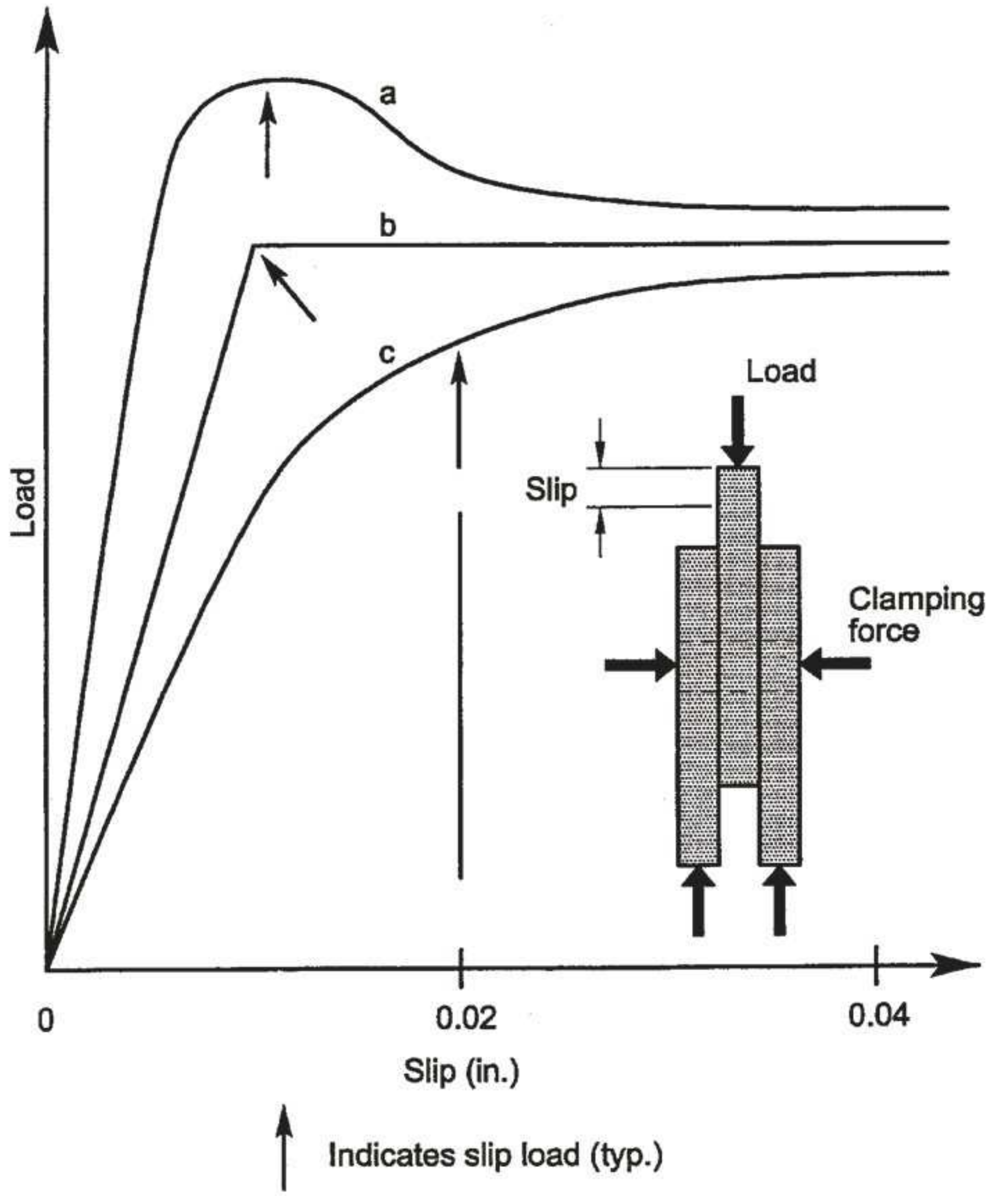

Figure A-4. Definition of slip load.

A3.5. Slip Coefficient

The slip coefficient for an individual specimen k_s shall be calculated as follows:

$$k_s = \frac{slip\ load}{2 \times clamping\ force} \qquad \text{(Equation A3.1)}$$

The *mean slip coefficient* μ for one set of five specimens shall be reported.

A3.6. Alternative Test Methods

Alternative test methods to determine slip are permitted, provided the accuracy of load measurement and clamping satisfies the conditions presented in the previous sections. For example, the slip load may be determined from a tension-type test setup rather than the compression-type test setup as long as the contact surface area per bolt of the test specimen is the same as that shown in Figure A1. The clamping force of at least 49 kips may be applied by any means, provided the force can be established within ± 1 percent.

Commentary:
Alternative test procedures and specimens may be used as long as the accuracy of load measurement and specimen geometry are maintained as prescribed. For example, strain-gaged bolts can usually provide the desired accuracy. However, bolts that are pretensioned by the turn-of-nut, calibrated wrench, alternative-design fastener, or direct-tension-indicator pretensioning method usually show too much variation to meet the ± 1 percent requirement of the slip test.

SECTION A4. TENSION CREEP TEST

The test method outlined is intended to ensure that the coating will not undergo significant creep deformation under sustained service loading. The test also indicates the loss in clamping force in the bolt due to the compression or creep of the coating. Three replicate specimens are to be tested.

Commentary:
The creep deformation of the bolted *joint* under the applied shear loading is also an important characteristic and a function of the coating applied. Thicker coatings tend to creep more than thinner coatings. Rate of creep deformation increases as the applied load approaches the slip load. Extensive testing has shown that the rate of creep is not constant with time, rather it decreases with time. After about 1,000 hours of loading, the additional creep deformation is negligible.

A4.1. Test Setup

Tension-type specimens, as shown in Figure A2, are to be used. The replicate specimens are to be linked together in a single chain-like arrangement, using loose pin bolts, so the same load is applied to all specimens. The specimens shall be assembled so the specimen plates are bearing against the bolt in a direction opposite to the applied tension loading. Care shall be taken in the assembly of the specimens to ensure the centerline of the holes used to accept the pin bolts is in line with the bolts used to assemble the *joint*. The load level, specified in Section A4.2, shall be maintained constant within ±1 percent by springs, load maintainers, servo controllers, dead weight or other suitable equipment. The bolts used to clamp the specimens together shall be ⅞ in. diameter ASTM A490 bolts. All bolts shall come from the same *lot*.

The clamping force in the bolts shall be a minimum of 49 kips. The clamping force shall be determined by calibrating the bolt force with bolt elongation, if standard bolts are used. Alternatively, special *fastener assemblies* that control the clamping force by other means, such as calibrated bolt torque or strain gages, are permitted. A minimum of three bolt calibrations shall be performed using the technique selected for bolt force determination. The average of the three-bolt calibration shall be calculated and reported. The method of measuring bolt force shall ensure the clamping force is within ±2 kips of the average value.

The relative slip between the outside plates and the center plates shall be measured to an accuracy of 0.001 in. These slips are to be measured on both sides of each specimen.

A4.2. Test Procedure

The load to be placed on the creep specimens is the service load permitted by Equation 5.7 for ⅞ in. diameter ASTM A490 bolts in *slip-critical joints* for the particular slip coefficient category under consideration. The load shall be placed on the specimen and held for 1,000 hours. The creep deformation of a specimen is calculated using the average reading of the two displacements on either side of the specimen. The difference between the average after 1,000 hours and the initial average reading taken within one-half hour after loading the specimens is defined as the creep deformation of the specimen. This value shall be reported for each specimen. If the creep deformation of any specimen exceeds 0.005 in., the coating has failed the test for the slip coefficient used. The coating may be retested using new specimens in accordance with this Section at a load corresponding to a lower value of slip coefficient.

If the value of creep deformation is less than 0.005 in. for all specimens, the specimens shall be loaded in tension to a load that is equal to the average clamping force times the design slip coefficient times 2, since there are two slip planes. The average slip deformation that occurs at this load shall be less than 0.015 in. for the three specimens. If the deformation is greater than this value, the coating is considered to have failed to meet the requirements for the particular *mean slip coefficient* used. The value of deformation for each specimen shall be reported.

Commentary:
See Commentary in Section A1.1.

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AISC 348-14

ANSI/AISC 303-16
An American National Standard

Code of Standard Practice for Steel Buildings and Bridges

June 15, 2016

Supersedes the *Code of Standard Practice for Steel Buildings and Bridges* dated April 14, 2010 and all previous versions

Approved by the Committee on the Code of Standard Practice

AMERICAN INSTITUTE OF STEEL CONSTRUCTION
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PREFACE

(This Preface is not part of ANSI/AISC 303-16, but is included for informational purposes only.)

As in any industry, trade practices have developed among those that are involved in the design, purchase, fabrication and erection of structural steel. This Code provides a useful framework for a common understanding of the acceptable standards when contracting for structural steel. As such, it is useful for owners, architects, engineers, general contractors, construction managers, fabricators, steel detailers, erectors and others associated with construction in structural steel. Unless specific provisions to the contrary are contained in the contract documents, the existing trade practices contained herein are considered to be the standard custom and usage of the industry and are thereby incorporated into the relationships between the parties to a contract.

It is important to note the differences in design requirements between buildings and bridges. ANSI/AISC 360 and 341 establish the design requirements for buildings and building-like structures, and this Code sets complementary commercial and technical requirements. For highway bridges, the governing design requirements are established by AASHTO and implemented by the contracting agency; the commercial provisions of the Code are applicable, but technical provisions, such as tolerances, are not addressed.

The Symbols and Glossary are an integral part of this Code. In many sections of this Code, a nonmandatory Commentary has been prepared to provide background and further explanation for the corresponding Code provisions. The user is encouraged to consult it.

This Code is written—and intended to be utilized in practice—as a unified document. Contract documents may supercede individual provisions of the Code as provided in Section 1.1, except when doing so would violate a requirement of the applicable building code.

Since the first edition of this Code was published in 1924, AISC has continuously surveyed the structural steel design community and construction industry to determine standard trade practices. Since then, this Code has been periodically updated to reflect new and changing technology and industry practices.

The 2000 edition was the fifth complete revision of this Code since it was first published. Like the 2005 and 2010 editions, the 2016 edition is not a complete revision but does add important changes and updates. It is the result of the deliberations of a fair and balanced Committee, the membership of which included structural engineers, architects, a code official, a general contractor, fabricators, a steel detailer, erectors, inspectors and an attorney. The following changes have been made in this revision:

- This Code is formally accredited by ANSI as an American National Standard.
- The language throughout the entire Code has been generalized to address contracts that utilize drawings, models, or drawings and models in combination, and Appendix A, which previously addressed models separately, has been eliminated.
- The Commentary in Section 1.1 has been updated to acknowledge that some portions of ANSI/AISC 303 are incorporated into the *International Building Code* through reference to those provisions in ANSI/AISC 360 and 341.
- The list of dates of referenced documents in Section 1.2 has been editorially updated.
- A new Section 1.4 has been added to address responsibility for identifying contract documents; subsequent sections have been renumbered.

- Section 1.10 has increased emphasis that the absence of a tolerance in this Code does not mean that tolerance is zero.
- Section 1.11 has been added to address marking requirements for protected zones in frames designed to meet the requirements of ANSI/AISC 341.
- A reference has been added in the Commentary to Section 2.2 to AISC Design Guide 27 for stainless steel.
- In Section 3.1, two items are added to the list of required information: preset requirements for free ends of cantilevered members and the drawing information required in ANSI/AISC 341.
- Sections 3.1.1 and 3.1.2 have been editorially switched in order. The resulting Section 3.1.2 (formerly Section 3.1.1) also has been improved to better address what is required for bidding when the owner's designated representative for design delegates the determination and design of member reinforcement at connections to the licensed engineer in responsible charge of the connection design.
- Section 3.2 has been updated to address revisions, if they are necessary, when referenced contract documents are not available at the time of design, bidding, detailing or fabrication.
- Section 3.3 has added emphasis that the fabricator need not discover design discrepancies.
- Sections 3.7 and 4.2.2 have been added to address intellectual property rights of the owner's designated representative for design and the fabricator, respectively.
- Section 4.4 has been clarified to better reflect the role of the connection design criteria required in Section 3.1.1 when connection design work is delegated.
- Commentary has been added to Section 4.5 to address potential pitfalls when fabrication and erection documents are not furnished by the fabricator.
- In Section 6.1.1, the listed shop-standard material grades have changed for HP-shapes and HSS.
- In Section 6.4.2, the tolerance for curved members has been improved.
- In Section 7.5.1, tolerances for anchor-rod placement have been revised for consistency with the hole sizes provided the AISC *Steel Construction Manual* and the tolerances given in ACI 117.
- In Section 7.8.3, the number of extra bolts required to be supplied has been increased to account for bolt loss and pre-installation verification testing requirements; also, backing has been clarified as steel backing.
- In Section 7.8.4, non-steel backing is now addressed.
- In Section 7.13, the term "building line" has been changed to "building exterior."
- Commentary has been added in Section 7.13.1.2(e) to coordinate with the cantilevered member preset information added in Section 3.1.
- Section 9.1.5 has been added to address allowances, when used.
- Section 10 has been significantly revised with multiple categories for AESS and different treatments required for each.
- The document has been editorially revised for consistency with current terms and other related documents.

The Committee thanks Jeffrey Dave, Douglas Fitzpatrick, Angela Stephens and Lawrence Kruth for their contributions to integrating treatment of model-based contracts throughout this Code; Walter Koppelaar, Terri Boake and Jack Petersen for their contributions to the update of Section 10; and, George Wendt, Charles Wood, John Rogers and Brian Smith for their contributions to the improvement of tolerances for curved members.

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By the AISC Committee on the Code of Standard Practice,

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GLOSSARY

The following abbreviations and terms are used in this Code. Where used, terms are italicized to alert the user that the term is defined in this Glossary.

AASHTO. American Association of State Highway and Transportation Officials.

Adjustable items. See Section 7.13.1.3.

AESS. See *architecturally exposed structural steel*.

AISC. American Institute of Steel Construction.

Allowance. A monetary amount included in a contract as a placeholder for work that is anticipated but not defined at the time the contract is executed.

Anchor bolt. See *anchor rod*.

Anchor rod. A mechanical device that is either cast or drilled and chemically adhered, grouted or wedged into concrete and/or masonry for the purpose of the subsequent attachment of *structural steel*.

ANSI. American National Standards Institute.

Approval documents. The *structural steel shop drawings*, *erection drawings*, and *embedment drawings*, or where the parties have agreed in the *contract documents* to provide digital model(s), the *fabrication* and *erection models*. A combination of drawings and digital models also may be provided.

Architect. The entity that is professionally qualified and duly licensed to perform architectural services.

Architecturally exposed structural steel. See Section 10.

AREMA. American Railway Engineering and Maintenance of Way Association.

ASME. American Society of Mechanical Engineers.

ASTM. American Society for Testing and Materials.

AWS. American Welding Society.

Bearing devices. Shop-attached base and bearing plates, loose base and bearing plates, and leveling devices, such as leveling plates, leveling nuts and washers, and leveling screws.

CASE. Council of American Structural Engineers.

Clarification. An interpretation, of the *design drawings* or *specifications* that have been *released for construction*, made in response to an *RFI* or a note on an approval drawing and providing an explanation that neither revises the information that has been *released for construction* nor alters the cost or schedule of performance of the work.

The Code, This Code. This document, the AISC *Code of Standard Practice for Steel Buildings and Bridges* as adopted by the American Institute of Steel Construction.

Column line. The grid line of column centers set in the field based on the dimensions shown on the structural *design documents* and using the building layout provided by the *owner's designated representative for construction*. Column offsets are taken from the *column line*. The *column line* may be straight or curved as shown in the structural *design documents.*

Connection. An assembly of one or more joints that is used to transmit forces between two or more members and/or *connection* elements.

Contract documents. The documents that define the responsibilities of the parties that are involved in bidding, fabricating and erecting *structural steel*. These documents normally include the *design documents*, the *specifications* and the contract.

Design documents. The *design drawings*, or where the parties have agreed in the *contract documents* to provide digital model(s), the *design model*. A combination of drawings and digital models also may be provided.

Design drawings. The graphic and pictorial portions of the *contract documents* showing the design, location and dimensions of the work. These documents generally include, but are not necessarily limited to, plans, elevations, sections, details, schedules, diagrams and notes.

Design model. A dimensionally accurate 3D digital model of the structure that conveys the *structural steel* requirements given in Section 3.1 for the building.

Detailer. See *steel detailer*.

Embedment drawings. Drawings that show the location and placement of items that are installed to receive *structural steel*.

EOR, engineer, engineer of record. See *structural engineer of record*.

Erection bracing drawings. Drawings that are prepared by the erector to illustrate the sequence of erection, any requirements for temporary supports and the requirements for raising, bolting and/or welding. These drawings are in addition to the *erection drawings*.

Erection documents. The *erection drawings*, or where the parties have agreed in the *contract documents* to provide digital model(s), the *erection model*. A combination of drawings and digital models also may be provided.

Erection drawings. Field-installation or member-placement drawings that are prepared by the *fabricator* to show the location and attachment of the individual *structural steel* shipping pieces.

Erection model. A dimensionally accurate 3D digital model produced to convey the information necessary to erect the *structural steel*. This may be the same digital model as the *fabrication model*, but it is not required to be.

Erector. The entity that is responsible for the erection of the *structural steel*.

Established column line. The actual field line that is most representative of the erected column centers along a line of columns placed using the dimensions shown in the structural *design drawings* or *design model* and the lines and benchmarks established by the *owner's designated representative for construction*, to be used in applying the erection tolerances given in this Code for column shipping pieces.

Fabrication documents. The *shop drawings*, or where the parties have agreed in the *contract documents* to provide digital model(s), the *fabrication model*. A combination of drawings and digital models also may be provided.

Fabrication model. A dimensionally accurate 3D digital model produced to convey the information necessary to fabricate the *structural steel*. This may be the same digital model as the *erection model*, but it is not required to be.

Fabricator. The entity that is responsible for detailing (except in Section 4.5) and fabricating the *structural steel*.

Hazardous materials. Components, compounds or devices that are either encountered during the performance of the contract work or incorporated into it containing substances that, not withstanding the application of reasonable care, present a threat of harm to persons and/or the environment.

Inspector. The *owner's* testing and inspection agency.

Levels of development, LOD. The levels of completeness of the digital model(s) or digital model elements.

MBMA. Metal Building Manufacturers Association.

Mill material. Steel mill products that are ordered expressly for the requirements of a specific project.

Owner. The entity that is identified as such in the *contract documents*.

Owner's designated representative for construction. The *owner* or the entity that is responsible to the *owner* for the overall construction of the project, including its planning, quality, and completion. This is usually the general contractor, the construction manager or similar authority at the job site.

Owner's designated representative for design. The *owner* or the entity that is responsible to the *owner* for the overall structural design of the project, including the *structural steel* frame. This is usually the *structural engineer of record*.

Plans. See *design drawings*.

RCSC. Research Council on Structural Connections.

Released for construction. The term that describes the status of *contract documents* that are in such a condition that the *fabricator* and the *erector* can rely upon them for the performance of their work, including the ordering of material and the preparation of *shop* and *erection drawings* or *fabrication* and *erection models*.

Revision. An instruction or directive providing information that differs from information that has been *released for construction*. A *revision* may, but does not always, impact the cost or schedule of performance of the work.

RFI. A written request for information or *clarification* generated during the construction phase of the project.

SER. See *structural engineer of record*.

Shop drawings. Drawings of the individual *structural steel* shipping pieces that are to be produced in the fabrication shop.

SJI. Steel Joist Institute.

Specifications. The portion of the *contract documents* that consists of the written requirements for materials, standards and workmanship.

SSPC. SSPC: The Society for Protective Coatings, which was formerly known as the Steel Structures Painting Council.

Standard structural shapes. Hot-rolled W-, S-, M- and HP-shapes, channels and angles listed in ASTM A6/A6M; structural tees split from the hot-rolled W-, S- and M- shapes listed in ASTM A6/A6M; hollow structural sections produced to ASTM A500/A500M, A501/A501M, A618/A618M, A847/A847M, A1065/A1065M, or A1085/A1085M; and, steel pipe produced to ASTM A53/A53M.

Steel detailer. The entity that produces the *approval documents*.

Structural engineer of record. The licensed professional who is responsible for sealing the *contract documents*, which indicates that he or she has performed or supervised the analysis, design and document preparation for the structure and has knowledge of the load-carrying structural system.

Structural steel. The elements of the structural frame as given in Section 2.1.

Substantiating connection information. Information submitted by the *fabricator*, if requested by the *owner's designated representative for design* in the *contract documents*, when Option 2 or Option 3 is designated for *connections* per Section 3.1.1.

Tier. The *structural steel* framing defined by a column shipping piece.

Weld show-through. In *architecturally exposed structural steel*, visual indication of the presence of a weld or welds on the side of the member opposite the weld.

CODE OF STANDARD PRACTICE FOR STEEL BUILDINGS AND BRIDGES

SECTION 1. GENERAL PROVISIONS

1.1. Scope

This Code sets forth criteria for the trade practices involved in steel buildings, bridges and other structures, where other structures are defined as those structures designed, fabricated and erected in a manner similar to buildings, with building-like vertical and lateral force-resisting elements. In the absence of specific instructions to the contrary in the *contract documents*, the trade practices that are defined in this Code shall govern the fabrication and erection of *structural steel*.

Commentary:
The practices defined in this Code are the commonly accepted standards of custom and usage for *structural steel* fabrication and erection, which generally represent the most efficient approach. Some provisions in this Code have been incorporated by reference into the International Building Code; see www.aisc.org/303IBC.

This Code is not intended to define a professional standard of care for the *owner's designated representative for design*; change the duties and responsibilities of the *owner*, contractor, *architect* or *structural engineer of record* from those set forth in the *contract documents*; nor assign to the *owner*, *architect* or *structural engineer of record* any duty or authority to undertake responsibility inconsistent with the provisions of the *contract documents*.

This Code is not applicable to steel joists or metal building systems, which are addressed by SJI and MBMA, respectively.

1.2. Dates of Referenced Specifications, Codes and Standards

The following dated versions of documents are referenced in this Code:

AASHTO Specification—2014 AASHTO *LRFD Bridge Design Specifications, 7th Edition*, with 2015 and 2016 Interim Revisions.

ANSI/AISC 341—ANSI/AISC 341-16, AISC *Seismic Provisions for Structural Steel Buildings*.

ANSI/AISC 360—ANSI/AISC 360-16, AISC *Specification for Structural Steel Buildings*.

ASME B46.1—ASME B46.1-09, Surface Texture (Surface Roughness, Waviness, and Lay).

AREMA Specification—2015 AREMA *Manual for Railway Engineering, Volume II—Structures, Chapter 15*.

ASTM A6/A6M-14, *Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling*.

ASTM A36/A36M-14, *Standard Specification for Carbon Structural Steel*.

ASTM A53/A53M-12, *Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless*.

ASTM A500/A500M-13, *Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes*.

ASTM A501/A501M-14, *Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing*.

ASTM A572/A572M-15, *Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel*.

ASTM A618/A618M-04(2015), *Standard Specification for Hot-Formed Welded and Seamless High-Strength Low-Alloy Structural Tubing*.

ASTM A847/A847M-14, *Standard Specification for Cold-Formed Welded and Seamless High-Strength, Low-Alloy Structural Tubing with Improved Atmospheric Corrosion Resistance*.

ASTM A992/A992M-11(2015), *Standard Specification for Structural Steel Shapes*.

ASTM A1065/A1065/M-15, *Standard Specification for Cold-Formed Electric-Fusion (Arc) Welded High-Strength Low-Alloy Structural Tubing in Shapes, with 50 ksi [345 MPa] Minimum Yield Point*.

ASTM A1085/A1085M-15, *Standard Specification for Cold-Formed Welded Carbon Steel Hollow Structural Sections (HSS)*.

AWS D1.1/D1.1M—AWS D1.1/D1.1M:2015 *Structural Welding Code—Steel*.

RCSC Specification—*Specification for Structural Joints Using High-Strength Bolts*, 2014.

SSPC SP1—SSPC *Surface Preparation Specification No. 1, Solvent Cleaning*, 2015.

SSPC SP2—SSPC *Surface Preparation Specification No. 2, Hand Tool Cleaning*, 2004.

SSPC SP6—SSPC *Surface Preparation Specification No. 6, Commercial Blast Cleaning*, 2007.

Commentary:

Additionally, the following dated versions of documents are referenced in the Commentary on this Code:

AIA Document E202—2008 Building Information Modeling Protocol Exhibit

AIA Document E203—2013 Building Information Modeling and Digital Data Exhibit

AIA Document G201—2013 Project Digital Data Protocol Form

AIA Document G202—2013 Project Building Information Modeling Protocol Form

ASTM A563-15, *Standard Specification for Carbon and Alloy Steel Nuts*.

ASTM A563M-07(2013), *Standard Specification for Carbon and Alloy Steel Nuts (Metric)*.

ASTM F3125/F3125M-15, *Standard Specification for High Strength Structural Bolts, Steel and Alloy Steel, Heat Treated, 120 ksi (830 MPa) and 150 ksi (1040 MPa) Minimum Tensile Strength, Inch and Metric Dimensions*.

BIMFORUM 2013 Level of Development Specification.
CASE Document 962—*National Practice Guidelines for the Structural Engineer of Record, 2012*.
Consensus Docs 301—2013 BIM Addendum.

1.3. Units

In this Code, the values stated in either U.S. customary units or metric units shall be used. Each system shall be used independently of the other.

Commentary:
In this Code, dimensions, weights and other measures are given in U.S. customary units with rounded or rationalized metric-unit equivalents in brackets. Because the values stated in each system are not exact equivalents, the selective combination of values from each of the two systems is not permitted.

1.4 Responsibility for Identifying Contract Documents

The *owner's designated representative for construction* shall identify all *contract documents*. When the *design drawings* and a *design model* are both provided, the *owner's designated representative for design* shall specify which document is the controlling *contract document*. The *contract documents* shall establish the procedures for communicating changes to the *contract documents*, permitted use of design and other digital models, and restrictions on the release of these digital models to other parties.

Commentary:
There can be many combinations of drawings and digital models used as part of the *contract documents*, and to transfer information between the many entities in the design and construction processes. The communication of design information to the *fabricator* through the *design model* is permitted in this Code. This Code does not designate which of these possible documents takes precedence because of the variation in current practice. The document hierarchy is left to the *owner's designated representative for design* and communicated through the *owner's designated representative for construction*. The *owner's designated representative for construction* must provide guidance as to which information is to be considered to have precedence if conflicts exist.

1.5. Design Criteria

For buildings and other structures, in the absence of other design criteria, the provisions in ANSI/AISC 360 shall govern the design of the *structural steel*. For bridges, in the absence of other design criteria, the provisions in the AASHTO Specification and AREMA Specification shall govern the design of the *structural steel*, as applicable.

1.6. Responsibility for Design

1.6.1. When the *owner's designated representative for design* provides the design, *design documents* and *specifications*, the *fabricator* and the *erector* are not responsible for the suitability, adequacy or building-code conformance of the design.

1.6.2. When the *owner* enters into a direct contract with the *fabricator* to both design and fabricate an entire, completed steel structure, the *fabricator* shall be responsible for the suitability, adequacy, conformance with *owner*-established performance criteria, and building-code conformance of the *structural steel* design. The owner shall be responsible for the suitability, adequacy and building-code conformance of the non-*structural steel* elements and shall establish the performance criteria for the *structural steel* frame.

1.7. Patents and Copyrights

The entity or entities that are responsible for the specification and/or selection of proprietary structural designs shall secure all intellectual property rights necessary for the use of those designs.

1.8. Existing Structures

1.8.1. Demolition and shoring of any part of an existing structure are not within the scope of work that is provided by either the *fabricator* or the *erector*. Such demolition and shoring shall be performed in a timely manner so as not to interfere with or delay the work of the *fabricator* or the *erector*.

1.8.2. Protection of an existing structure and its contents and equipment, so as to prevent damage from normal erection processes, is not within the scope of work that is provided by either the *fabricator* or the *erector*. Such protection shall be performed in a timely manner so as not to interfere with or delay the work of the *fabricator* or the *erector*.

1.8.3. Surveying or field dimensioning of an existing structure is not within the scope of work that is provided by either the *fabricator* or the *erector*. Such surveying or field dimensioning, which is necessary for the completion of the *approval documents* and fabrication, shall be performed and furnished to the *fabricator* in a timely manner so as not to interfere with or delay the work of the *fabricator* or the *erector*.

1.8.4. Abatement or removal of *hazardous materials* is not within the scope of work that is provided by either the *fabricator* or the *erector*. Such abatement or removal shall be performed in a timely manner so as not to interfere with or delay the work of the *fabricator* or the *erector*.

1.9. Means, Methods and Safety of Erection

1.9.1. The *erector* shall be responsible for the means, methods and safety of erection of the *structural steel* frame.

1.9.2. The *structural engineer of record* shall be responsible for the structural adequacy of the design of the structure in the completed project. The *structural engineer of record* shall not be responsible for the means, methods and safety of erection of the *structural steel* frame. See also Sections 3.1.4 and 7.10.

1.10. Tolerances

Tolerances for materials, fabrication and erection shall be as stipulated in Sections 5, 6, 7 and 10. Tolerances absent from this Code or the *contract documents* shall not be considered zero by default.

Commentary:
Tolerances are not necessarily specified in this Code for every possible variation that could be encountered. For most projects, where a tolerance is not specified or covered in this Code, it is not needed to ensure that the fabricated and erected *structural steel* complies with the requirements in Section 6 and 7. If a special design concept or system component requires a tolerance that is not specified in this Code, the necessary tolerance should be specified in the *contract documents*. If a tolerance is not shown and is deemed by the *fabricator* and/or *erector* to be important to the successful fabrication and erection of the structural steel, it should be requested from the *owner's designated representative for design*. The absence of a tolerance in this Code for a particular condition does not mean that the tolerance is zero; rather, it means that no tolerance has been established. In any case, the default tolerance is not zero.

1.11. Marking of Protected Zones in High-Seismic Applications

The *fabricator* shall permanently mark protected zones that are designated on the structural *design documents* in accordance with ANSI/AISC 341 Section A4.1. If these markings are obscured in the field, such as after the application of fire protection, the *owner's designated representative for construction* shall re-mark the protected zones as they are designated on the structural *design documents*.

SECTION 2. CLASSIFICATION OF MATERIALS

2.1. Definition of Structural Steel

Structural steel shall consist of the elements of the structural frame that are shown and sized in the structural *design documents*, essential to support the design loads and described as:

Anchor rods that will receive *structural steel*.
Base plates, if part of the *structural steel* frame.
Beams, including built-up beams, if made from *standard structural shapes* and/or plates.
Bearing plates, if part of the *structural steel* frame.
Bearings of steel for girders, trusses or bridges.
Bracing, if permanent.
Canopy framing, if made from standard *structural shapes* and/or plates.
Columns, including built-up columns, if made from *standard structural shapes* and/or plates.
Connection materials for framing *structural steel* to *structural steel*.
Crane stops, if made from *standard structural shapes* and/or plates.
Door frames, if made from *standard structural shapes* and/or plates and if part of the *structural steel* frame.
Edge angles and plates, if attached to the *structural steel* frame or steel (open-web) joists.
Embedded *structural steel* parts, other than bearing plates, that will receive *structural steel*.
Expansion joints, if attached to the *structural steel* frame.
Fasteners for connecting *structural steel* items: permanent shop bolts, nuts and washers; shop bolts, nuts and washers for shipment; field bolts, nuts and washers for permanent *connections*; and, permanent pins.
Floor-opening frames, if made from *standard structural shapes* and/or plates and attached to the *structural steel* frame or steel (open-web) joists.
Floor plates (checkered or plain), if attached to the *structural steel* frame.
Girders, including built-up girders, if made from *standard structural shapes* and/or plates.
Girts, if made from *standard structural shapes*.
Grillage beams and girders.
Hangers, if made from *standard structural shapes*, plates and/or rods and framing *structural steel* to *structural steel*.
Leveling nuts and washers.
Leveling plates.
Leveling screws.
Lintels, if attached to the *structural steel* frame.
Marquee framing, if made from *standard structural shapes* and/or plates.
Machinery supports, if made from *standard structural shapes* and/or plates and attached to the *structural steel* frame.
Monorail elements, if made from *standard structural shapes* and/or plates and attached to the *structural steel* frame.

Posts, if part of the *structural steel* frame.
Purlins, if made from *standard structural shapes*.
Relieving angles, if attached to the *structural steel* frame.
Roof-opening frames, if made from *standard structural shapes* and/or plates and attached to the *structural steel* frame or steel (open-web) joists.
Roof-screen support frames, if made from *standard structural shapes*.
Sag rods, if part of the *structural steel* frame and connecting *structural steel* to *structural steel*.
Shear stud connectors, if specified to be shop attached.
Shims, if permanent.
Struts, if permanent and part of the *structural steel* frame.
Tie rods, if part of the *structural steel* frame.
Trusses, if made from *standard structural shapes* and/or built-up members.
Wall-opening frames, if made from *standard structural shapes* and/or plates and attached to the *structural steel* frame.
Wedges, if permanent.

Commentary:
The *fabricator* normally fabricates the items listed in Section 2.1. Such items must be shown, sized and described in the structural *design documents*. Bracing includes vertical bracing for resistance to wind and seismic load and structural stability, horizontal bracing for floor and roof systems, and permanent stability bracing for components of the *structural steel* frame.

2.2. Other Steel, Iron or Metal Items

Structural steel shall not include other steel, iron or metal items that are not generally described in Section 2.1, even where such items are shown in the structural *design documents* or are attached to the *structural steel* frame. Other steel, iron or metal items include but are not limited to:

Base plates, if not part of the *structural steel* frame.
Bearing plates, if not part of the *structural steel* frame.
Bearings, if non-steel.
Cables for permanent bracing or suspension systems.
Castings.
Catwalks.
Chutes.
Cold-formed steel products.
Cold-rolled steel products, except those that are specifically covered in ANSI/AISC 360.
Corner guards.
Crane rails, splices, bolts and clamps.
Crane stops, if not made from *standard structural shapes* or plates.
Door guards.
Embedded steel parts, other than bearing plates, that do not receive *structural steel* or that are embedded in precast concrete.
Expansion joints, if not attached to the *structural steel* frame.

Flagpole support steel.
Floor plates (checkered or plain), if not attached to the *structural steel* frame.
Forgings.
Gage-metal products.
Grating.
Handrail.
Hangers, if not made from *standard structural shapes*, plates and/or rods or not framing *structural steel* to *structural steel*.
Hoppers.
Items that are required for the assembly or erection of materials that are furnished by trades other than the *fabricator* or *erector*.
Ladders.
Lintels, if not attached to the *structural steel* frame.
Masonry anchors.
Ornamental metal framing.
Other miscellaneous metal not already listed.
Pressure vessels.
Reinforcing steel for concrete or masonry.
Relieving angles, if not attached to the *structural steel* frame.
Roof screen support frames, if not made from *standard structural shapes*.
Safety cages.
Shear stud connectors, if specified to be field installed.
Stacks.
Stairs.
Steel deck.
Steel (open-web) joists.
Steel joist girders.
Tanks.
Toe plates.
Trench or pit covers.

Commentary:
Section 2.2 includes many items that may be furnished by the *fabricator* if contracted to do so by specific notation and detail in the *contract documents*. When such items are contracted to be provided by the *fabricator*, coordination will normally be required between the *fabricator* and other material suppliers and trades. The provisions in this Code are not intended to apply to items in Section 2.2.

In previous editions of this Code, provisions regarding who should normally furnish field-installed shear stud connectors and cold-formed steel deck support angles were included in Section 7.8. These provisions have been eliminated since field-installed shear stud connectors and steel deck support angles are not defined as *structural steel* in this Code.

Stainless steel is not covered in this Code. AISC Design Guide 27, *Structural Stainless Steel*, is a source of useful information regarding the practical fabrication and installation issues associated with structural stainless steel components.

SECTION 3. DESIGN DOCUMENTS AND SPECIFICATIONS

3.1. Structural Design Documents and Specifications

Unless otherwise indicated in the *contract documents*, the structural *design documents* shall be based upon consideration of the design loads and forces to be resisted by the *structural steel* frame in the completed project.

The structural *design documents* shall clearly show or note the work that is to be performed and shall give the following information with sufficient dimensions to accurately convey the quantity and complexity of the *structural steel* to be fabricated:

(a) The size, section, material grade and location of all members.
(b) All geometry and working points necessary for layout.
(c) Floor elevations.
(d) Column centers and offsets.
(e) The camber requirements for members.
(f) Preset elevation requirements, if any, at free ends of cantilevered members relative to their fixed-end elevations.
(g) Joining requirements between elements of built-up members.
(h) When the requirements of ANSI/AISC 341 are applicable, the information required in ANSI/AISC 341 Section A4.
(i) The information required in Sections 3.1.1 through 3.1.6.

The *structural steel specifications* shall include any special requirements for the fabrication and erection of the *structural steel*.

The structural *design documents*, *specifications* and addenda shall be numbered and dated for the purposes of identification. 3D digital models shall contain a unique identifier.

Commentary:
Contract documents vary greatly in complexity and completeness. Nonetheless, the *fabricator* and the *erector* must be able to rely upon the accuracy and completeness of the *contract documents*. This allows the *fabricator* and the *erector* to provide the *owner* with bids that are adequate and complete. It also enables the preparation of the *approval documents*, the ordering of materials, and the timely fabrication and erection of shipping pieces.

In some cases, the *owner* can benefit when reasonable latitude is allowed in the *contract documents* for alternatives that can reduce cost without compromising quality. However, critical requirements that are necessary to protect the *owner's* interest, that affect the integrity of the structure or that are necessary for the *fabricator* and the *erector* to proceed with their work must be included in the *contract documents*. Some examples of critical information may include, when applicable:

Standard specifications and codes that govern *structural steel* design and construction, including bolting and welding.
Material specifications.
Special material requirements to be reported on the material test reports.
Welded-joint configuration.

Weld-procedure qualification.
Special requirements for work of other trades.
Final disposition of backing and runoff tabs.
Lateral bracing.
Stability bracing.
Connections or data for *connection* selection and/or completion.
Restrictions on *connection* types.
Column stiffeners (also known as continuity plates).
Column web doubler plates.
Bearing stiffeners on beams and girders.
Web reinforcement.
Openings for other trades.
Surface preparation and shop painting requirements.
Shop and field inspection requirements.
Nondestructive testing requirements, including acceptance criteria.
Special requirements on delivery.
Special erection limitations.
Identification of non-*structural steel* elements that interact with the *structural steel* frame to provide for the lateral stability of the *structural steel* frame (see Section 3.1.4).
Column differential shortening information (see Commentary to Section 7.13).
Anticipated deflections and the associated loading conditions for major structural elements, such as transfer girders and trusses, supporting columns and hangers (see Commentary to Section 7.13).
Special fabrication and erection tolerances for *AESS*.
Special pay-weight provisions.

It may be necessary to specify a relative elevation to which the free end of a cantilever must be erected (preset) prior to load application, with the fixed end stabilized before the member is released from the crane or temporary support and any other load is applied to it. This is needed so that the cantilevered member can be detailed and fabricated to allow for any required preset. This does not apply to a beam that is continuous over a support, which is controlled by camber, not preset.

3.1.1. The *owner's designated representative for design* shall indicate one of the following options for each *connection*:

(1) Option 1: the complete *connection* design shall be shown in the structural *design documents*.
(2) Option 2: in the structural *design documents* or *specifications*, the *connection* shall be designated to be selected or completed by an experienced *steel detailer*.
(3) Option 3: in the structural *design documents* or *specifications*, the *connection* shall be designated to be designed by a licensed engineer working for the *fabricator*.

In all of the above options,

(a) The requirements of Section 3.1.2 shall apply.
(b) The approvals process in Section 4.4 shall be followed.

When Option 2 is specified, the experienced *steel detailer* shall utilize information provided in the structural *design documents* in the selection or completion of the *connections*. When such information is not provided, tables in the AISC *Steel Construction Manual*, or other reference information as approved by the *owner's designated representative for design*, shall be used.

When Option 2 or 3 is specified, the *owner's designated representative for design* shall provide the following *connection* design criteria in the structural *design documents* and *specifications*:

(a) Any restrictions on the types of *connections* that are permitted.
(b) Data concerning the loads, including shears, moments, axial forces and transfer forces, that are to be resisted by the individual members and their *connections*, sufficient to allow the selection, completion, or design of the *connection* details while preparing the *approval documents*.
(c) Whether the data required in (b) is given at the service-load level or the factored-load level.
(d) Whether LRFD or ASD is to be used in the selection, completion, or design of *connection* details.
(e) What *substantiating connection information*, if any, is to be provided with the *approval documents* to the *owner's designated representative for design*.

When Option 3 is specified:

(a) The *fabricator* shall submit in a timely manner representative samples of the required *substantiating connection information* to the *owner's designated representatives for design* and *construction*. The *owner's designated representative for design* shall confirm in writing in a timely manner that these representative samples are consistent with the requirements in the *contract documents*, or shall advise what modifications are required to bring the representative samples into compliance with the requirements in the *contract documents*. This initial submittal and review is in addition to the requirements in Section 4.4.
(b) The licensed engineer in responsible charge of the *connection* design shall review and confirm in writing as part of the *substantiating connection information*, that the *approval documents* properly incorporate the *connection* designs. However, this review by the licensed engineer in responsible charge of the *connection* design does not replace the approval process of the *approval documents* by the *owner's designated representative for design* in Section 4.4.
(c) The *fabricator* shall provide a means by which the *substantiating connection information* is referenced to the related *connections* on the *approval documents* for the purpose of review.

Commentary:
There are three options covered in this Section:

(1) In Option 1, the *owner's designated representative for design* shows the complete design of the *connections* in the structural *design documents*. The following information is included:

(a) All weld types, sizes, lengths and strengths.
(b) All bolt sizes, locations, quantities and grades.
(c) All plate and angle sizes, thicknesses, dimensions and grades.
(d) All work point locations and related information.

The intent of this approach is that complete design information necessary for detailing the *connection* is shown in the structural *design documents*. Typical details are shown for each *connection* type, set of geometric parameters, and adjacent framing conditions. The *steel detailer* will then be able to transfer this information to the *approval documents*, applying it to the individual pieces being detailed.

(2) In Option 2, the *owner's designated representative for design* allows an experienced *steel detailer* to select or complete the *connections*. This is commonly done by referring to loads embedded in the digital model, tables or schematic information in the structural *design documents*, tables in the AISC *Steel Construction Manual*, or other reference information approved by the *owner's designated representative for design*, such as journal papers and recognized software output. Tables and schematic information in the structural *design documents* should provide such information as weld types and sizes, plate thicknesses, and quantities of bolts. However, there may be some geometry and dimensional information that the *steel detailer* must develop. The *steel detailer* will then configure the connections based upon the design loads and other information given in the structural *design documents* and *specifications*.

The intent of this method is that the *steel detailer* will select the *connection* materials and configuration from the referenced tables or complete the specific *connection* configuration (e.g., dimensions, edge distances and bolt spacing) based upon the *connection* details that are shown in the structural *design documents*.

The *steel detailer* must be experienced and familiar with AISC requirements for *connection* configurations, the use of the *connection* tables in the AISC *Steel Construction Manual*, the calculation of dimensions, and adaptation of typical *connection* details to similar situations. Notations of loadings in the structural *design documents* are only to facilitate selection of the *connections* from the referenced tables. It is not the intent that this method be used when the practice of engineering is required.

(3) Option 3 reflects a practice in some areas of the U.S. to have a licensed engineer working for or retained by the *fabricator* design the *connections*, and recognizes the information required by the *fabricator* to do this work. The *owner's designated representative for design*, who has the knowledge of the structure as a whole, must review and approve the *approval documents*, and take such action on *substantiating connection information* as the *owner's designated representative for design* deems appropriate. See Section 4.4 for the approval process.

When, under Section 3.1.1, the *owner's designated representative for design* designates that *connections* are to be designed by a licensed engineer employed or retained by the *fabricator*, this work is incidental to, and part of, the overall means and methods of fabricating and constructing the steel frame. The licensed engineer performing the *connection* design is not providing a peer review of the *contract documents*.

The *owner's designated representative for design* reviews the *approval documents* during the approvals process as specified in Section 4.4 for conformance with the specified criteria and compatibility with the design of the primary structure.

One of these options should be indicated for each *connection* in a project. It is acceptable to group *connection* types and utilize a combination of these options for the various *connection* types involved in a project. Option 3 is not normally specified for *connections* that can be selected or completed as noted in Option 2 without practicing engineering.

If there are any restrictions as to the types of *connections* to be used, it is required that these limitations be set forth in the structural *design documents* and *specifications*. There are a variety of *connections* available in the AISC *Steel Construction Manual* for a given situation. Preference for a particular type will vary between *fabricators* and *erectors*. Stating these limitations, if any, in the structural *design documents* and *specifications* will help to avoid repeated changes to the *approval documents* due to the selection of a *connection* that is not acceptable to the *owner's designated representative for design*, thereby avoiding additional cost and/or delay for revising the *approval documents*.

The structural *design documents* must indicate the method of design used as LRFD or ASD. In order to conform to the spirit of ANSI/AISC 360, the *connections* must be selected using the same method and the corresponding references.

Substantiating connection information, when required, can take many forms. When Option 2 is designated, the *approval documents* may suffice with no additional *substantiating connection information* required. When Option 3 is designated, the *substantiating connection information* may take the form of hand calculations and/or software output.

When *substantiating connection information* is required, it is recommended that representative samples of that information be agreed upon prior to preparation of the *approval documents*, in order to avoid additional cost and/or delay for the *connection* redesign and/or revising that might otherwise result.

The *owner's designated representative for design* may require that the *substantiating connection information* be signed and sealed for Option 3. The signing and sealing of the cover letter transmitting the *approval documents* and *substantiating connection information* may suffice. This signing and sealing indicates that a licensed engineer performed the work but does not replace the approval process provided in Section 4.4.

A requirement to sign and seal each sheet of the *shop* and *erection drawings* is discouraged as it may serve to confuse the design responsibility between the *owner's designated representative for design* and the licensed engineer's work in performing the *connection* design. Such a requirement may not be possible when submitting *fabrication and erection models*.

3.1.2. Permanent bracing, openings in structural steel for other trades, and other special details, where required, shall be designed by the *owner's designated representative for design* and shown in sufficient detail in the structural *design documents* issued for bidding so that the quantity, detailing and fabrication requirements for these items can be readily understood.

At locations away from *connections*, stiffeners, web doubler plates, bearing stiffeners, and other member reinforcement, where required, shall be designed by the *owner's designated representative for design* and shown in sufficient detail in the structural *design documents* issued for bidding so that the quantity, detailing and fabrication requirements for these items can be readily understood.

At locations of *connections*, the following requirements shall apply to column stiffeners, web doubler plates, beam bearing stiffeners, and all other member reinforcement required to satisfy strength and equilibrium of forces through the *connection*:

(1) When Option 1 or 2 in Section 3.1.1 is specified for a *connection*, these items shall be designed by the *owner's designated representative for design* and shown in the structural *design documents* issued for bidding so that the quantity, detailing and fabrication requirements for member reinforcement at *connections* can be readily understood.

(2) When Option 3 in Section 3.1.1 is specified for a *connection*, two subsidiary options are available to the *owner's designated representative for design*; either:

(a) Option 3A: member reinforcement at *connections* shall be designed by the *owner's designated representative for design* and shown in the structural *design documents* issued for bidding so that the quantity, detailing and fabrication requirements for member reinforcement at *connections* can be readily understood, or;

(b) Option 3B: the *owner's designated representative for design* shall provide a bidding quantity of items required for member reinforcement at *connections* with corresponding project-specific details that show the conceptual configuration of reinforcement appropriate for the order of magnitude of forces to be transferred. These quantities and project-specific conceptual configurations will be relied upon for bidding purposes. If no quantities or conceptual configurations are shown, member reinforcement at *connections* will not be included in the bid.

Subsequently, member reinforcement at *connections*, where required, shall be designed in its final configuration by the licensed engineer in responsible charge of the *connection* design.

When the actual quantity and/or details of any of the foregoing items differ from the bidding quantity and/or details, the contract price and schedule shall be adjusted equitably in accordance with Sections 9.4 and 9.5.

Any limitations regarding type and connection of reinforcing shall be clearly provided.

Commentary:
Option 3A is most useful when the *owner's designated representative for design* delegates *connection* design work but has selected member sizes to eliminate or

minimize the need for member reinforcement at *connections*. Option 3A should not be used if the intent is to delegate the determination and design of member reinforcement at *connections* to the licensed engineer in responsible charge of the *connection* design.

Option 3B is necessary if the intent is to delegate the determination and design of member reinforcement at *connections* to the licensed engineer in responsible charge of the *connection* design. Because these requirements will not be known until *connections* are designed after award of the contract, bids prepared by multiple *fabricators* will not be comparable unless all bidders use the same assumptions in preparing their bids. The approach provided here allows for all bids to be comparable. The *owner's* final cost for the actual member reinforcement requirements at *connections* will be determined through equitable contract price adjustment.

When no quantities and details are shown for column stiffeners, web doubler plates, beam bearing stiffeners, and/or other member reinforcement required to satisfy strength and equilibrium of forces through *connections*, the *fabricator's* bid reflects no *allowance* for these items. Should it subsequently be determined that member reinforcement at *connections* is required, the provisions of Sections 9.4 and 9.5 then apply.

3.1.3. When leveling plates are to be furnished as part of the contract requirements, their locations and required thickness and sizes shall be specified in the *contract documents*.

3.1.4. When the *structural steel* frame, in the completely erected and fully connected state, requires interaction with non-*structural steel* elements (see Section 2) for strength and/or stability, those non-*structural steel* elements shall be identified in the *contract documents* as required in Section 7.10.

Commentary:
Examples of non-*structural steel* elements include diaphragms made of steel deck, diaphragms made of concrete on steel deck, and masonry and/or concrete shear walls.

3.1.5. When camber is required, the magnitude, direction and location of camber shall be specified in the structural *design documents*.

Commentary:
For cantilevers, the specified camber may be up or down, depending upon the framing and loading.

3.1.6. Specific members or portions thereof that are to be left unpainted shall be identified in the *contract documents*. When shop painting is required, the painting requirements shall be specified in the *contract documents*, including the following information:

(a) The identification of specific members or portions thereof to be painted.
(b) The surface preparation that is required for these members.
(c) The paint specifications and manufacturer's product identification, including color requirements, if any, that are required for these members.
(d) The minimum dry-film shop-coat thickness that is required for these members.

Commentary:
Some members or portions thereof may be required to be left unpainted, such as those that will be in contact and acting compositely with concrete, or those that will receive spray-applied fire protection materials.

3.2. Architectural, Electrical and Mechanical Design Documents and Specifications

All requirements for the quantities, sizes and locations of *structural steel* shall be shown or noted in the structural *design documents*. The structural *design documents* are permitted to reference the architectural, electrical and/or mechanical *design documents* as a supplement to the structural *design documents* for the purposes of defining detail configurations and construction information.

When the referenced information is not available at the time of structural design, bidding, detailing or fabrication, subsequent *revisions* shall be the responsibility of the *owner* and shall be made in accordance with Sections 3.5 and 9.3.

3.3. Discrepancies

When discrepancies exist between the *design documents* and *specifications*, the *design documents* shall govern. When discrepancies exist between scale dimensions in the *design documents* and the figures written in them, the figures shall govern. When discrepancies exist between the structural *design documents* and the architectural, electrical or mechanical *design documents*, or the *design documents* for other trades, the structural *design documents* shall govern. When discrepancies exist between the *design drawings* and the *design model*, the governing document shall be as identified per Section 1.4.

When a discrepancy is discovered in the *contract documents* in the course of the *fabricator's* work, the *fabricator* shall promptly notify the *owner's designated representative for construction* so that the discrepancy can be resolved. Such resolution shall be timely so as not to delay the *fabricator's* work. See Sections 3.5 and 9.3.

It is not the *fabricator's* responsibility to discover discrepancies, including those that are associated with the coordination of the various design disciplines.

3.4. Legibility of Design Drawings

Design *drawings* shall be clearly legible and drawn to an identified scale that is appropriate to clearly convey the information.

Commentary:
Historically, the most commonly accepted scale for *structural steel* drawings has been $^1/_8$ in. per ft (10 mm per 1 000 mm). There are, however, situations where a

smaller or larger scale is appropriate. Ultimately, consideration must be given to the clarity of the drawing.

The scaling of the *design drawings* to determine dimensions is not an accepted practice for detailing the *approval documents*. However, it should be remembered when preparing *design drawings* that scaling may be the only method available when early-submission drawings are used to determine dimensions for estimating and bidding purposes.

3.5. Revisions to the Design Documents and Specifications

Revisions to the *design documents* and *specifications* shall be made either by issuing new *design documents* and *specifications* or by reissuing the existing *design documents* and *specifications*. In either case, all *revisions*, including revisions that are communicated through responses to *RFIs* or the annotation of the *approval documents* (see Section 4.4.2), shall be clearly and individually indicated in the *contract documents*. The *contract documents* shall be dated and identified by *revision* number. When the *design documents* are communicated using *design drawings*, each *design drawing* shall be identified by the same drawing number throughout the duration of the project, regardless of the *revision*. See also Section 9.3.

When *revisions* are communicated using *design models*, *revisions* shall be made evident in the revised *design model* submitted by identifying within the *design model* which items are changed. Alternatively, the changes shall be submitted with a written document describing in explicit detail the items that are changed. A historic tracking of changes must either be present in the revised *design model* or maintained in the written record of changes.

The party or entity that is contractually assigned responsibility for managing the *design model* shall maintain accurate accounting and tracking records of the most current *design model*, as well as previously superseded *design models*, and shall facilitate a tracking mechanism so that all contracted parties are aware of, and have access to, the most current *design model*.

Commentary:

Revisions to the *design documents* and *specifications* can be made by issuing sketches and supplemental information separate from the design documents and *specifications*. These sketches and supplemental information become amendments to the *design documents* and *specifications* and are considered new *contract documents*. All sketches and supplemental information must be uniquely identified with a number and date as the latest instructions until such time as they may be superseded by new information.

When *revisions* are made by revising and reissuing the existing structural *design documents* and/or *specifications*, a unique *revision* number and date must be added to those documents to identify that information as the latest instructions until such time as they may be superseded by new information. When the *design documents* are communicated using *design drawings*, the same unique drawing number must identify each *design drawing* throughout the duration of the project so that *revisions* can be properly tracked, thus avoiding confusion and miscommunication among the various entities involved in the project.

When *revisions* are communicated through the annotation of the *approval documents* or contractor submissions, such changes must be confirmed in writing by one of the aforementioned methods. This written confirmation is imperative to maintain control of the cost and schedule of a project and to avoid potential errors in fabrication.

When *design models* are used, a similar unique method of identifying each *revision* must be used. This method can vary in various digital modeling software, but the same level of notation of changes must be present in the revised *design model* as would be used on *design drawings*.

3.6. Fast-Track Project Delivery

When the fast-track project delivery system is selected, release of the structural *design documents* and *specifications* shall constitute a *release for construction*, regardless of the status of the architectural, electrical, mechanical and other interfacing designs and *contract documents*. Subsequent *revisions*, if any, shall be the responsibility of the owner and shall be made in accordance with Sections 3.5 and 9.3.

Commentary:
The fast-track project delivery system generally provides for a condensed schedule for the design and construction of a project. Under this delivery system, the *owner* elects to *release for construction* the structural *design documents* and *specifications*, which may be partially complete, at a time that may precede the completion of and coordination with architectural, mechanical, electrical and other design work and *contract documents*. The release of the structural *design documents* and *specifications* may also precede the release of the General Conditions and Division 1 Specifications.

Release of the structural *design documents* and *specifications* to the *fabricator* for ordering of material constitutes a *release for construction*. Accordingly, the *fabricator* and the *erector* may begin their work based upon those partially complete documents. As the architectural, mechanical, electrical and other design elements of the project are completed, *revisions* may be required in design and/or construction. Thus, when considering the fast-track project delivery system, the owner should balance the potential benefits to the project schedule with the project cost contingency that may be required to allow for these subsequent *revisions*.

3.7 Intellectual Property

Any copyright or other property or proprietary rights owned by the *owner's designated representative for design* in any content included within the *contract documents*, whether created specifically for an individual project or otherwise made available for use on an individual project, shall remain the exclusive property of the *owner's designated representative for design*.

SECTION 4. APPROVAL DOCUMENTS

4.1. Owner Responsibility

The *owner* shall furnish, in a timely manner and in accordance with the *contract documents*, the complete structural *design documents* and *specifications* that have been *released for construction*. Unless otherwise noted, *design documents* and *specifications* that are provided as part of the contract bid documents shall constitute authorization by the *owner* that the *design documents* and *specifications* are *released for construction*.

Commentary:
When the *owner* issues *design documents* and *specifications* that are *released for construction*, the *fabricator* and the *erector* rely on the fact that these are the *owner's* requirements for the project. This release is required by the *fabricator* prior to the ordering of material and the preparation and completion of the *approval documents*.

To ensure the orderly flow of material procurement, detailing, fabrication and erection activities, on phased construction projects, it is essential that designs are not continuously revised after they have been *released for construction*. In essence, once a portion of a design is *released for construction*, the essential elements of that design should be "frozen" to ensure adherence to the contract price and construction schedule. Alternatively, all parties should reach a common understanding of the effects of future changes, if any, as they affect scheduled deliveries and added costs.

A pre-detailing conference, held after the *structural steel* fabrication contract is awarded, can benefit the project. Typical attendees may include the *owner's designated representative for construction*, the *owner's designated representative for design*, the *fabricator*, the *steel detailer*, and the *erector*. Topics of the meeting should relate to the specifics of the project and might include:

- Contract document review and general project overview, including *clarifications* of scope of work, tolerances, layouts and sequences, and special considerations.
- Detailing and coordination needs, such as bolting, welding, and *connection* considerations, constructability considerations, OSHA requirements, coordination with other trades, and the advanced bill of materials.
- The project communication system, including distribution of contact information for relevant parties to the contract, identification of the primary and alternate contacts in the general contractor's office, and the *RFI* system to be used on the project.
- The submittal schedule, including the method of submitting (electronic or hard copy); for hard copy, how many copies of documents are required; *connection* submittals; and identification of schedule-critical areas of the project, if any.
- If digital models will be used as part of the delivery method for the *design documents*, the parties should determine and convey the *levels of development (LOD)*, the digital model types that will be furnished, the authorized uses of

such digital models, the transmission of digital models to prevent the loss or alteration of data, interoperability, and methods of review and approval. The term *levels of development* refers to the level of completeness of elements within the digital model (see the BIMFORUM Level of Development Specification). The term "authorized uses" refers to the permitted uses of the digital model(s) and the digital data associated with the digital model(s). Such authorized uses may include the right to (1) store and view the digital model(s) for informational purposes only, (2) rely upon, store and view the digital model(s) to carry out the work on the project, (3) reproduce and distribute the digital model(s) for informational purposes only, (4) rely upon, reproduce and distribute the digital model(s) to carry out the work, (5) incorporate additional digital data into the digital model(s) without modifying the data received to carry out the work on the project, (6) modify the digital model(s) as required to carry out the work on the project, (7) produce the digital model(s) in an archival format for the *owner* to use as a reference for as-built construction data and/or for the operation of the project after completion, and/or (8) other authorized uses specified in the *contract documents*.

- Review of quality and inspection requirements, including the approvals process for corrective work.

Record of the meeting should be written and distributed to all parties. Subsequent meetings to discuss progress and issues that arise during construction also can be helpful, particularly when they are held on a regular schedule.

4.2. Fabricator Responsibility

4.2.1. Except as provided in Section 4.5, the *fabricator* shall produce the *approval documents* for the fabrication and erection of the structural steel and is responsible for the following:

(a) The transfer of information from the *contract documents* into accurate and complete *approval documents*.

(b) The development of accurate, detailed dimensional information to provide for the fit-up of parts in the field.

Commentary:
The *fabricator* is permitted to use the services of independent *steel detailers* to produce *approval documents* and to perform other support services, such as producing advanced bills of material and bolt summaries.

As the *fabricator* develops the detailed dimensional information for production of the *approval documents*, there may be discrepancies, missing information or conflicts discovered in the *contract documents*. See Section 3.3.

4.2.2. Any copyright or other property or proprietary rights owned by the *fabricator* in any content included within the *approval documents*, whether created specifically for an individual project or otherwise made available for use on an individual project, shall remain the exclusive property of the *fabricator*.

4.2.3. When the *approval documents* are *shop* and *erection drawings*, each *shop* and *erection drawing* shall be identified by the same drawing number throughout the duration of the project and shall be identified by *revision* number and date, with each specific *revision* clearly identified. When the *approval documents are fabrication* and *erection models*, each submittal shall be uniquely identified.

When the *fabricator* submits a request to change *connection* details that are described in the *contract documents*, the *fabricator* shall notify the *owner's designated representatives for design* and *construction* in writing in advance of the submission of the *approval documents*. The *owner's designated representative for design* shall review and approve or reject the request in a timely manner.

When requested to do so by the *owner's designated representative for design*, the *fabricator* shall provide to the *owner's designated representatives for design* and *construction* its schedule for the submittal of *approval documents* so as to facilitate the timely flow of information between all parties.

Commentary:
When the *fabricator* intends to make a submission of alternative *connection* details to those shown in the *contract documents*, the *fabricator* must notify the *owner's designated representatives for design* and *construction* in advance. This will allow the parties involved to plan for the increased effort that may be required to review the alternative *connection* details. In addition, the *owner* will be able to evaluate the potential for cost savings and/or schedule improvements against the additional design cost for review of the alternative *connection* details by the *owner's designated representative for design*. This evaluation by the *owner* may result in the rejection of the alternative *connection* details or acceptance of the submission for review based upon cost savings, schedule improvements and/or job efficiencies.

The *owner's designated representative for design* may request the *fabricator's* schedule for the submittal of the *approval documents*. This process is intended to allow the parties to plan for the staffing demands of the submission schedule. The *contract documents* may address this issue in more detail. In the absence of the requirement to provide this schedule, none need be provided.

When the *fabricator* provides a schedule for the submission of the *approval documents*, it must be recognized that this schedule may be affected by *revisions* and the response time to requests for missing information or the resolution of discrepancies.

4.3. Use of Digital Files or Copies of the Design Documents

The *fabricator* shall neither use nor reproduce any part of the *design documents* as part of the *approval documents* without the written permission of the *owner's designated representative for design*. When digital files or copies of the *design documents* are made available for the *fabricator's* use as part of the *approval documents*, the *fabricator* shall accept this information under the following conditions:

(a) All information contained in the digital files or copies of the *design documents* shall be considered instruments of service of the *owner's designated representative for design* and shall not be used for other projects, additions to the project or the completion of the project by others. Digital files or copies of the *design documents* shall remain the property of the *owner's designated representative for design* and in no case shall the transfer of these copies of the *design documents* be considered a sale or unrestricted license.

(b) CAD files or copies of the *design drawings* shall not be considered to be *contract documents*. In the event of a conflict between the *design drawings* and the CAD files or copies thereof, the *design drawings* shall govern.

(c) When a *design model* is made available for use by the *fabricator*, the *owner's designated representative for construction* shall designate whether the *design model* and/or other documents are to be considered the *contract documents*. See Section 1.4.

(d) Any party or entity that creates a copy of the *design model* does so at their own risk.

(e) The use of copies of the *design documents* shall not in any way obviate the *fabricator's* responsibility for proper checking and coordination of dimensions, details, member sizes and fit-up, and quantities of materials as required to facilitate the preparation of *approval documents* that are complete and accurate as required in Section 4.2.

(f) If copies of *design drawings* are used by the *fabricator*, the *fabricator* shall remove information that is not required for the fabrication or erection of the *structural steel* from the copies of the *design drawings*.

Commentary:

Copies of the *design documents* often are readily available to the *fabricator*. As a result, the *owner's designated representative for design* may have reduced control over the unauthorized use of the *design documents*. There are many copyright and other legal issues to be considered.

The *owner's designated representative for design* may choose to make copies of the *design documents* available to the *fabricator*, and may charge a service or licensing fee for this convenience. In doing so, a carefully negotiated agreement should be established to set out the specific responsibilities of both parties in view of the liabilities involved for both parties. For sample contracts, see Consensus Docs 301 BIM Addendum, AIA Document E202 Building Information Modeling Protocol Exhibit, AIA Document E203 Building Information Modeling and Digital Data Exhibit, AIA Document G201 Project Digital Data Protocol Form, and AIA Document G202 Project Building Information Modeling Protocol Form.

Once the *design model* has been accessed and/or modified by any entity other than the *owner's designated representative for design*, the resulting model is considered a copy of the *design model* and is no longer part of the *contract documents*.

The copies of the *design documents* are provided to the *fabricator* for convenience only. The information therein should be adapted for use only in reference to the placement of *structural steel* members during erection. The *fabricator* should treat this information as if it were fully produced by the *fabricator* and undertake the same level of checking and quality assurance. When amendments or *revisions* are made to the *contract documents*, the *fabricator* must update this reference material.

When copies of the *design drawings* are provided to the *fabricator*, they often contain other information, such as architectural backgrounds or references to other *contract documents*. This additional material should be removed when producing the *approval documents* to avoid the potential for confusion.

Just like the transmission of the *design documents* created by the *owner's designated representative for design* does not convey ownership rights in the *design documents*, the transmission of the approval documents created by the *fabricator* does not convey ownership rights in the *approval documents*.

4.4. Approval

Except as provided in Section 4.5, the *approval documents* shall be submitted to the *owner's designated representatives for design* and construction for review and approval. The *approval documents* shall be returned to the *fabricator* within 14 calendar days.

Final *substantiating connection information*, if any, shall also be submitted with the *approval documents*. The *owner's designated representative for design* is the final authority in the event of a disagreement between parties regarding the design of *connections* to be incorporated into the overall *structural steel* frame. The *fabricator* and licensed engineer in responsible charge of *connection* design are entitled to rely upon the *connection* design criteria provided in accordance with Section 3.1.1. *Revisions* to these criteria shall be addressed in accordance with Sections 9.3 and 9.4.

Approved *approval documents* shall be individually annotated by the *owner's designated representatives for design* and *construction* as either approved or approved subject to corrections noted. When so required, the *fabricator* shall subsequently make the corrections noted and furnish corrected *fabrication* and *erection documents* to the *owner's designated representatives for design* and *construction*.

Commentary:
As used in this Code, the 14-day allotment for the return of *approval documents* is intended to represent the *fabricator's* portal-to-portal time. The intent in this Code is that, in the absence of information to the contrary in the *contract documents*, 14 days may be assumed for the purposes of bidding, contracting and scheduling. When additional time is desired, such as when *substantiating connection information* is part of the submittals, the modified allotment should be specified in the *contract documents*. A submittal schedule is commonly used to facilitate the approval process.

If the *approval documents* are approved subject to corrections noted, the *owner's designated representative for design* may or may not require that it be resubmitted for record purposes following correction. If the *approval documents* are not approved, *revisions* must be made and the documents resubmitted until approval is achieved.

4.4.1. Approval, approval subject to corrections noted, and similar approvals of the *approval documents* shall constitute the following:

(a) Confirmation that the *fabricator* has correctly interpreted the *contract documents* in the preparation of those submittals.
(b) Confirmation that the *owner's designated representative* for design has reviewed and approved the *connection* details shown in the *approval documents* and submitted in accordance with Section 3.1.1, if applicable.
(c) Release by the *owner's designated representatives for design* and *construction* for the *fabricator* to begin fabrication using the approved submittals.

Such approval shall not relieve the *fabricator* of the responsibility for either the accuracy of the detailed dimensions in the *approval documents* or the general fit-up of parts that are to be assembled in the field.

The *fabricator* shall determine the fabrication schedule that is necessary to meet the requirements of the contract.

Commentary:
When considering the current language in this Section, the Committee sought language that would parallel the practices of CASE. In CASE Document 962, CASE indicates that when the design of some element of the primary structural system is left to someone other than the *structural engineer of record*, "…such elements, including *connections* designed by others, should be reviewed by the *structural engineer of record*. He [or she] should review such designs and details, accept or reject them and be responsible for their effects on the primary structural system." Historically, this Code has embraced this same concept.

From the inception of this Code, AISC and the industry in general have recognized that only the *owner's designated representative for design* has all the information necessary to evaluate the total impact of *connection* details on the overall structural design of the project. This authority traditionally has been exercised during the approval process for the *approval documents*. The *owner's designated representative for design* has thus retained responsibility for the adequacy and safety of the entire structure since at least the 1927 edition of this Code.

4.4.2. Unless otherwise noted, any additions, deletions or *revisions* that are indicated in responses to *RFIs* or on the approved *approval documents* shall constitute authorization by the *owner* that the additions, deletions or *revisions* are *released for construction*. The *fabricator* and the *erector* shall promptly notify the *owner's designated representative for construction* when any direction or notation in responses to *RFIs* or on the *approval documents* or other information will result in an additional cost and/or a delay. See Sections 3.5 and 9.3.

Commentary:
When the *fabricator* notifies the *owner's designated representative for construction* that a direction or notation in responses to *RFIs* or on the *approval documents* will result in an additional cost or a delay, it is then normally the responsibility of the *owner's designated representative for construction* to subsequently notify the *owner's designated representative for design*.

4.5. Fabrication and/or Erection Documents Not Furnished by the Fabricator

When the *fabrication* and *erection documents* are not prepared by the *fabricator*, but are furnished by others, they shall be delivered to the *fabricator* in a timely manner, or as agreed upon in the *contract documents*. These *fabrication* and *erection documents* shall be prepared, insofar as is practical, in accordance with the shop fabrication and detailing standards of the *fabricator*. The *fabricator* shall not be responsible for the completeness, coordination, or accuracy of *fabrication* and *erection documents* so furnished, nor for the general fit-up of the members that are fabricated from them.

Commentary:
This delivery system of *fabrication* and *erection documents* is discouraged. The preparation of the *fabrication* and *erection documents* is very specific to the needs of the *fabricator* performing the work, and an integral part of the constructability and coordination assurance of the project. If the project team chooses to use this delivery method, the *contract documents* should be very clear as to the managing of this process, including, but not limited to, who and how the following will be handled:

- Standards, format and contents of the *fabrication* and *erection documents*, or representative documents that will be part of the *contract documents*, for the mill order and for fabrication, including field bolts.
- Provisions for proper risk management (errors and omissions or product liability, as applicable).
- Normal "pre-detailing" sequencing, OSHA erection aids, and other Sub Part R requirements incorporated.
- Schedule updates for documents, and impact to overall project schedule and contract, as these dates are impacted.
- *Revision* of *fabrication* and *erection documents* and control of in order to maintain the integrity of all parts of the *fabrication* and *erection documents*.
- Late released items.
- Shop question support, including those that arise on night shifts and weekends.
- Joist, deck and other commodity item question and coordination support.
- Field question support.

4.6. The RFI Process

When *requests for information (RFIs)* are issued, the process shall include the maintenance of a written record of inquiries and responses related to interpretation and implementation of the *contract documents*, including the *clarifications* and/or *revisions* to the *contract documents* that result, if any. *RFIs* shall not be used for the incremental *release for construction* of the *design documents*. When *RFIs* involve discrepancies or *revisions*, see Sections 3.3, 3.5 and 4.4.2.

When a *design model* is used as the *design documents*, the changes and/or *clarifications* made in response to *RFIs* shall be incorporated into the *design model*.

Commentary:
The *RFI* process is most commonly used during the detailing process, but can also be used to forward inquiries by the *erector* or to inform the *owner's designated representative for design* in the event of a *fabricator* or *erector* error and to develop corrective measures to resolve such errors.

The *RFI* process is intended to provide a written record of inquiries and associated responses but not to replace all verbal communication between the parties on the project. *RFIs* should be prepared and responded to in a timely fashion so as not to delay the work of the *steel detailer*, *fabricator* and *erector*. Discussion of the *RFI* issues and possible solutions between the *fabricator*, *erector* and *owner's designated representatives for design* and *construction* often can facilitate timely and practical resolution. Unlike submittals in Section 4.4, *RFI* response time can vary depending on the urgency of the issue, the amount of work required by the *owner's designated representatives for design* and *construction* to develop a complete response, and other circumstances, such as building official approval.

RFIs should be prepared in a standardized format, including *RFI* number and date, identity of the author, reference to a specific location(s) in the *design documents* or *specification* section, the needed response date, a description of a suggested solution (graphic depictions are recommended for more complex issues), and an indication of possible schedule and cost impacts. *RFIs* should be limited to one question each (unless multiple questions are interrelated to the same issue) to facilitate the resolution and minimize response time. Questions and proposed solutions presented in *RFIs* should be clear and complete. *RFI* responses should be equally clear and complete in the depictions of the solutions, and signed and dated by the responding party.

Unless otherwise noted, the *fabricator* and *erector* can assume that a response to an *RFI* constitutes a *release for construction*. However, if the response will result in an increase in cost or a delay in schedule, Section 4.4.2 requires that the *fabricator* and/or *erector* promptly inform the *owner's designated representatives for design* and *construction*.

4.7 Erection Documents

The *erection documents* shall be provided to the *erector* in a timely manner so as to allow the *erector* to properly plan and perform the work.

Commentary:
For planning purposes, this may include release of preliminary *erection documents*, if requested by the *erector*.

SECTION 5. MATERIALS

5.1. Mill Materials

Unless otherwise noted in the *contract documents*, the *fabricator* is permitted to order the materials that are necessary for fabrication when the *fabricator* receives *contract documents* that have been *released for construction*.

Commentary:
The *fabricator* may purchase materials in stock lengths, exact lengths or multiples of exact lengths to suit the dimensions shown in the structural *design documents*. Such purchases will normally be job-specific in nature and may not be suitable for use on other projects or returned for full credit if subsequent design changes make these materials unsuitable for their originally intended use. The *fabricator* should be paid for these materials upon delivery from the mill, subject to appropriate additional payment or credit if subsequent unanticipated modification or reorder is required. Purchasing materials to exact lengths is not considered fabrication.

5.1.1. Unless otherwise specified by means of special testing requirements in the *contract documents*, mill testing shall be limited to those tests that are required for the material in the ASTM specifications indicated in the *contract documents*. Materials ordered to special material requirements shall be marked by the supplier as specified in ASTM A6/A6M Section 12 prior to delivery to the *fabricator's* shop or other point of use. Such material not so marked by the supplier, shall not be used until:

(a) Its identification is established by means of testing in accordance with the applicable ASTM specifications.
(b) A *fabricator's* identification mark, as described in Section 6.1.2 and 6.1.3, has been applied.

5.1.2. When *mill material* does not satisfy ASTM A6/A6M tolerances for camber, profile, flatness or sweep, the *fabricator* shall be permitted to perform corrective procedures, including the use of controlled heating and/or mechanical straightening, subject to the limitations in ANSI/AISC 360.

Commentary:
Mill dimensional tolerances are completely set forth in ASTM A6/A6M. Normal variations in the cross-sectional geometry of *standard structural shapes* must be recognized by the designer, the *fabricator*, the *steel detailer*, and the *erector* (for example, see Figure C-5.1). Such tolerances are mandatory because roll wear, thermal distortions of the hot cross section immediately after leaving the forming rolls and differential cooling distortions that take place on the cooling beds are all unavoidable. Geometric perfection of the cross section is not necessary for either structural or architectural reasons, if the tolerances are recognized and provided for.

ASTM A6/A6M also stipulates tolerances for straightness that are adequate for typical construction. However, these characteristics may be controlled or corrected to closer tolerances during the fabrication process when the added cost is justified by the special requirements for an atypical project.

5.1.3. When variations that exceed ASTM A6/A6M tolerances are discovered or occur after the receipt of *mill material* the *fabricator* shall, at the *fabricator's* option, be permitted to perform the ASTM A6/A6M corrective procedures for mill reconditioning of the surface of *structural steel* shapes and plates.

5.1.4. When special tolerances that are more restrictive than those in ASTM A6/A6M are required for *mill materials*, such special tolerances shall be specified in the *contract documents*. The *fabricator* shall, at the *fabricator's* option, be permitted to order material to ASTM A6/A6M tolerances and subsequently perform the corrective procedures described in Sections 5.1.2 and 5.1.3.

5.2. Stock Materials

5.2.1. If used for structural purposes, materials that are taken from stock by the *fabricator* shall be of a quality that is at least equal to that required in the ASTM specifications indicated in the *contract documents*.

5.2.2. Material test reports shall be accepted as sufficient record of the quality of materials taken from stock by the *fabricator*. The *fabricator* shall review and retain the material test reports that cover such stock materials. However, the *fabricator* need not maintain records that identify individual pieces of stock material against individual material test reports, provided the *fabricator* purchases stock materials that meet the requirements for material grade and quality in the applicable ASTM specifications.

5.2.3. Stock materials that are purchased under no particular specification, under a specification that is less rigorous than the applicable ASTM specifications or without material test reports or other recognized test reports shall not be used without the approval of the *owner's designated representative for design*.

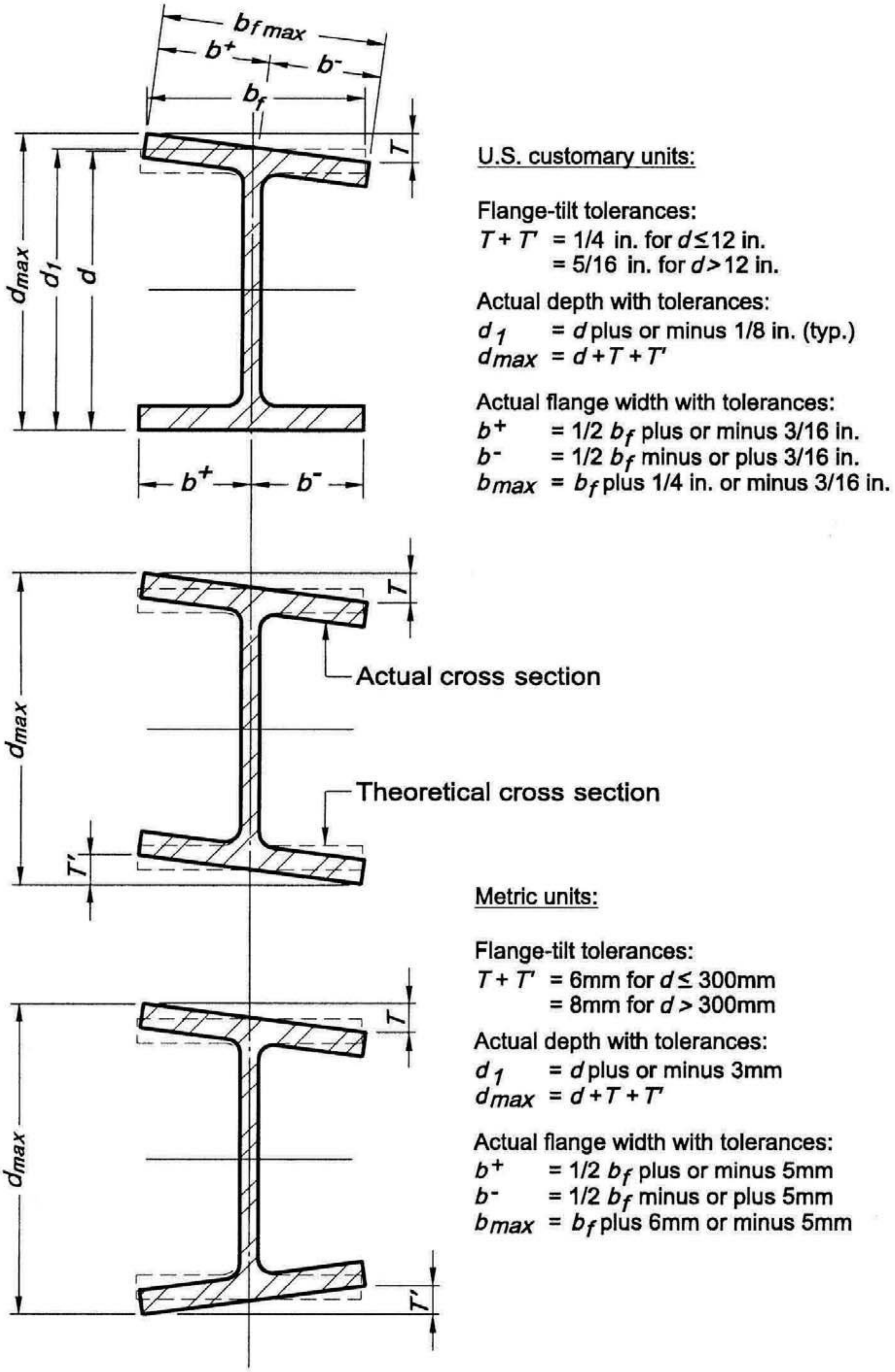


Fig. C-5.1. Mill tolerances on the cross section of a W-shape.

SECTION 6. SHOP FABRICATION AND DELIVERY

6.1. Identification of Material

6.1.1. The *fabricator* shall be able to demonstrate by written procedure and actual practice a method of material identification, visible up to the point of assembling members as follows:

(a) For shop-standard material, identification capability shall include shape designation. Representative material test reports shall be furnished by the *fabricator* if requested to do so by the *owner's designated representative for design*, either in the *contract documents* or in separate written instructions given to the *fabricator* prior to ordering *mill materials*.

(b) For material of grade other than shop-standard material, identification capability shall include shape designation and material grade. Representative material test reports shall be furnished by the *fabricator* if requested to do so by the *owner's designated representative for design*, either in the *contract documents* or in separate written instructions given to the *fabricator* prior to ordering *mill materials*.

(c) For material ordered in accordance with an ASTM supplement or other special material requirements in the *contract documents*, identification capability shall include shape designation, material grade and heat number. The corresponding material test reports shall be furnished by the *fabricator* if requested to do so by the *owner's designated representative for design*, either in the *contract documents* or in separate written instructions given to the *fabricator* prior to ordering *mill materials*.

Unless an alternative system is established in the *fabricator's* written procedures, shop-standard material shall be as follows:

Material	**Shop-Standard Material Grade**
W and WT	ASTM A992/A992M
M, S, MT and ST	ASTM A36/A36M
HP	ASTM A572/A572M Grade 50
L	ASTM A36/A36M
C and MC	ASTM A36/A36M
HSS	ASTM A500/A500M Grade C
Steel Pipe	ASTM A53/A53M Grade B
Plates and Bars	ASTM A36/A36M

Commentary:
The requirements in Section 6.1.1(a) will suffice for most projects. When material is of a strength level that differs from the shop-standard grade, the requirements in Section 6.1.1(b) apply. When special material requirements apply, such as ASTM A6/A6M supplement S5 or S30 for CVN testing or ASTM A6/A6M supplement S8 for ultrasonic testing, the requirements in Section 6.1.1(c) are applicable.

6.1.2. During fabrication, up to the point of assembling members, each piece of material that is ordered to special material requirements shall carry a *fabricator's* identification mark or an original supplier's identification mark. The *fabricator's* identification mark shall be in accordance with the *fabricator's* established material identification system, which shall be on record and available prior to the start of fabrication for the information of the *owner's designated representative for construction*, the building code authority and the *inspector*.

6.1.3. Members that are made of material that is ordered to special material requirements shall not be given the same assembling or erection mark as members made of other material, even if they are of identical dimensions and detail.

6.2. Preparation of Material

6.2.1. The thermal cutting of *structural steel* by hand-guided or mechanically guided means is permitted.

6.2.2. Surfaces that are specified as "finished" in the *contract documents* shall have a roughness height value measured in accordance with ASME B46.1 that is equal to or less than 500 μin. (12.7 μm). The use of any fabricating technique that produces such a finish is permitted.

Commentary:
Most cutting processes, including friction sawing and cold sawing, and milling processes meet a surface roughness limitation of 500 μin. (12.7 μm) per ASME B46.1.

6.3. Fitting and Fastening

6.3.1. Projecting elements of *connection* materials need not be straightened in the connecting plane, subject to the limitations in ANSI/AISC 360.

6.3.2. Backing and runoff tabs shall be used in accordance with AWS D1.1/D1.1M as required to produce sound welds. The *fabricator* or *erector* need not remove backing or runoff tabs unless such removal is specified in the *contract documents*. When the removal of backing is specified in the *contract documents*, such removal shall meet the requirements in AWS D1.1/D1.1M. When the removal of runoff tabs is specified in the *contract documents*, hand flame-cutting close to the edge of the finished member with no further finishing is permitted, unless other finishing is specified in the *contract documents*.

Commentary:
In most cases, the treatment of backing and runoff tabs is left to the discretion of the *owner's designated representative for design*. In some cases, treatment beyond the basic cases described in this Section may be required. As one example, special treatment is required for backing and runoff tabs in beam-to-column moment connections when the requirements in ANSI/AISC 341 must be met. In all cases, the *owner's designated representative for design* should specify the required treatments in the *contract documents*.

6.3.3. Unless otherwise noted in the *fabrication documents*, high-strength bolts for shop-attached *connection* material shall be installed in the shop in accordance with the requirements in ANSI/AISC 360.

6.4. Fabrication Tolerances

The tolerances on *structural steel* fabrication shall be in accordance with the requirements in Section 6.4.1 through 6.4.6.

Commentary:
Fabrication tolerances are stipulated in several specifications and codes, each applicable to a specialized area of construction. Basic fabrication tolerances are stipulated in this Section. For *architecturally exposed structural steel*, see Section 10. Other specifications and codes are also commonly incorporated by reference in the *contract documents*, such as ANSI/AISC 360, the RCSC Specification, AWS D1.1/D1.1M, and the AASHTO Specification.

6.4.1. For members that have both ends finished (see Section 6.2.2) for contact bearing, the variation in the overall length shall be equal to or less than $^1/_{32}$ in. (1 mm). For other members that frame to other *structural steel* elements, the variation in the detailed length shall be as follows:

For members that are equal to or less than 30 ft (9 000 mm) in length, the variation shall be equal to or less than $^1/_{16}$ in. (2 mm).

For members that are greater than 30 ft (9 000 mm) in length, the variation shall be equal to or less than $^1/_8$ in. (3 mm).

6.4.2. For straight and curved structural members, whether of a single *standard structural shape* or built-up, the permitted variation in specified straightness or curvature shall be as listed below. In all cases, completed members shall be free of twists (except as allowed by ASTM standards), bends and open joints. Sharp kinks or sharp bends shall be cause for rejection.

(a) For straight structural members other than compression members, the variation in straightness shall be equal to or less than that specified for structural shapes in the applicable ASTM standards except when a smaller variation is specified in the *contract documents*.

For straight compression members, the variation in straightness shall be equal to or less than $^1/_{1000}$ of the axial length between points that are to be laterally supported.

(b) For curved structural members, the variation in the chord length shall be as defined in Section 6.4.1. The variation in curvature measured at the middle ordinate shall be equal to or less than the permissible variations in straightness as specified in applicable ASTM standards for camber in the strong direction and sweep in the weak direction, inside or outside of the theoretical arc, except when a smaller variation is specified in the *contract documents*. Should no applicable ASTM standard exist, the maximum variation in curvature measured at the middle ordinate shall be plus or minus 1/8 in. (3 mm) times one-fifth the total arc length in ft (times two-thirds the total arc length in m) for members 10 ft (3 m) or greater in length. For members less than 10 ft (3 m) in length, the permissible variation in curvature measured at the middle ordinate shall be plus or minus 1/8 in. (3 mm). The middle ordinate is located between work points as shown in Figure C-6.1.

Commentary:
Curved structural members, as referred to in this section, are defined as those members intended to maintain a specified curvature while in use. This section does not apply to members specified for camber. The location of the arc length is defined by the contract drawings and may be either at the member's inside radius, the outside radius, or the radius between work points.

6.4.3. For beams that are detailed without specified camber, the member shall be fabricated so that, after erection, any incidental camber due to rolling or shop fabrication is upward. For trusses that are detailed without specified camber, the components shall be fabricated so that, after erection, any incidental camber in the truss due to rolling or shop fabrication is upward.

6.4.4. For beams that are specified in the *contract documents* with camber, beams received by the *fabricator* with 75% of the specified camber shall require no further cambering. Otherwise, the variation in camber shall be as follows:

(a) For beams that are equal to or less than 50 ft (15 000 mm) in length, the variation shall be equal to or less than minus zero / plus 1/2 in. (13 mm).

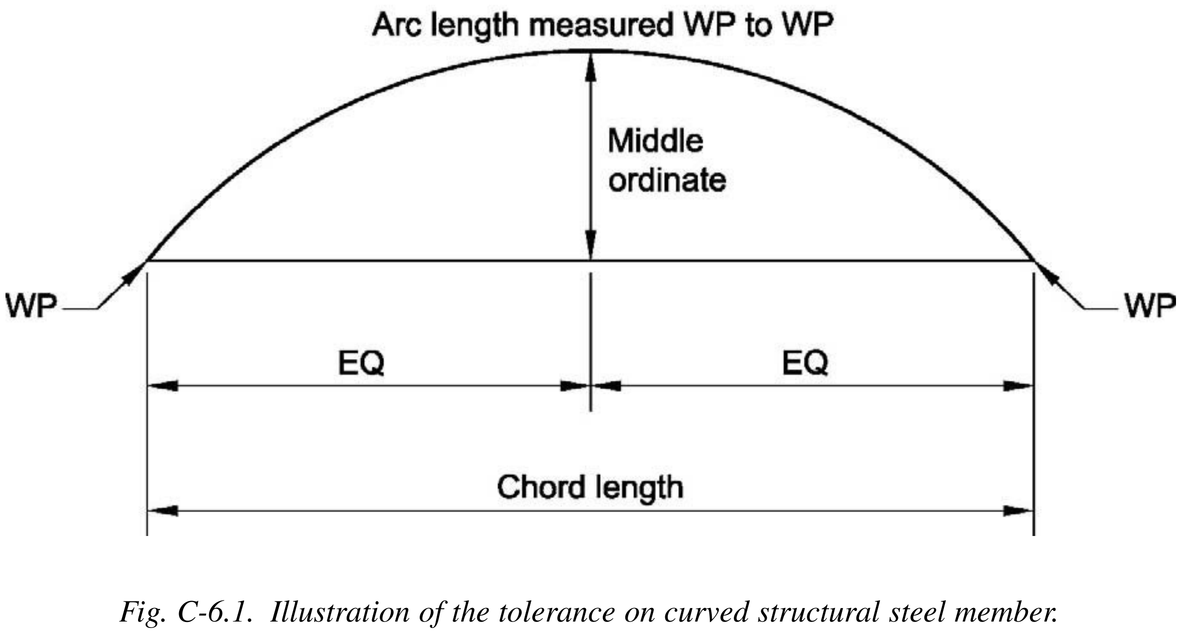


Fig. C-6.1. Illustration of the tolerance on curved structural steel member.

(b) For beams that are greater than 50 ft (15 000 mm) in length, the variation shall be equal to or less than minus zero / plus $^1/_2$ in. plus $^1/_8$ in. for each 10 ft or fraction thereof (13 mm plus 3 mm for each 3 000 mm or fraction thereof) in excess of 50 ft (15 000 mm) in length.

For the purpose of inspection, camber shall be measured in the *fabricator's* shop in the unstressed condition.

Commentary:
There is no known way to inspect beam camber after the beam is received in the field because of factors that include:

(a) The release of stresses in members over time and in varying applications.
(b) The effects of the dead weight of the member.
(c) The restraint caused by the end *connections* in the erected state.
(d) The effects of additional dead load that may ultimately be intended to be applied, if any.

Therefore, inspection of the *fabricator's* work on beam camber must be done in the fabrication shop in the unstressed condition.

6.4.5. For fabricated trusses that are specified in the *contract documents* with camber, the variation in camber at each specified camber point shall be equal to or less than plus or minus $^1/_{800}$ of the distance to that point from the nearest point of support. For the purpose of inspection, camber shall be measured in the *fabricator's* shop in the unstressed condition. For fabricated trusses that are specified in the *contract documents* without indication of camber, the foregoing requirements shall be applied at each panel point of the truss with a zero camber ordinate.

Commentary:
There is no known way to inspect truss camber after the truss is received in the field because of factors that include:

(a) The effects of the dead weight of the member.
(b) The restraint caused by the truss *connections* in the erected state.
(c) The effects of additional dead load that may ultimately be intended to be applied, if any.

Therefore, inspection of the *fabricator's* work on truss camber must be done in the fabrication shop in the unstressed condition. See Figure C-6.2.

6.4.6. When permissible variations in the depths of beams and girders result in abrupt changes in depth at splices, such deviations shall be accounted for as follows:

(a) For splices with bolted joints, the variations in depth shall be taken up with filler plates.
(b) For splices with welded joints, the weld profile shall be adjusted to conform to the variations in depth, the required cross section of weld shall be provided and the slope of the weld surface shall meet the requirements in AWS D1.1/D1.1M.

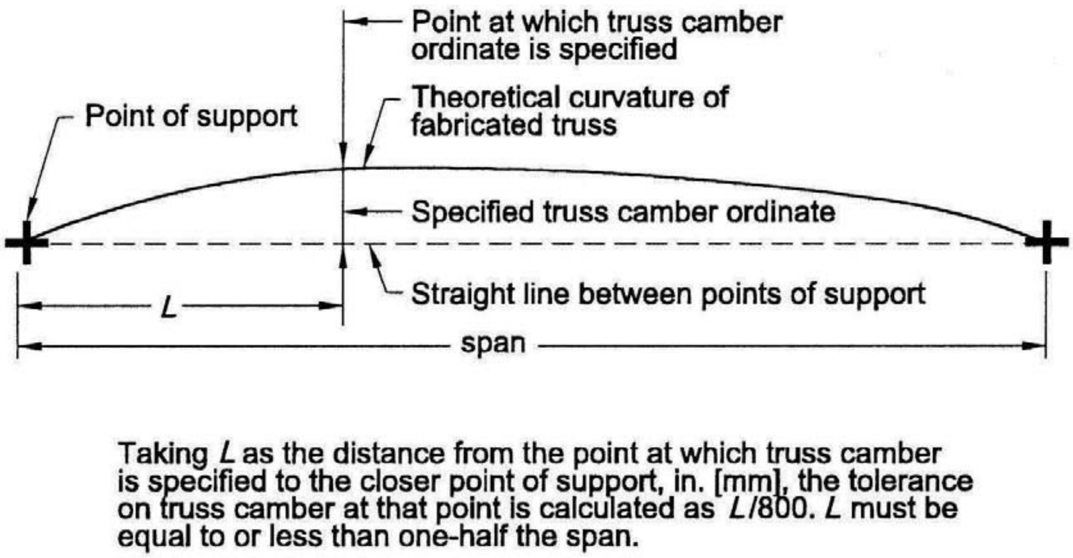


Fig. C-6.2. Illustration of the tolerance on camber for fabricated trusses with specified camber.

6.5. Shop Cleaning and Painting (see also Section 3.1.6)

Structural steel that does not require shop paint shall be cleaned of oil and grease with solvent cleaners, and of dirt and other foreign material by sweeping with a fiber brush or other suitable means. For *structural steel* that is required to be shop painted, the requirements in Sections 6.5.1 through 6.5.4 shall apply.

Commentary:
Extended exposure of unpainted *structural steel* that has been cleaned for the subsequent application of fire protection materials can be detrimental to the fabricated product. Most levels of cleaning require the removal of all loose mill scale, but permit some amount of tightly adhering mill scale. When a piece of *structural steel* that has been cleaned to an acceptable level is left exposed to a normal environment, moisture can penetrate behind the scale, and some "lifting" of the scale by the oxidation process is to be expected. Cleanup of "lifted" mill scale is not the responsibility of the *fabricator*, but is to be assigned by contract requirement to an appropriate contractor.

Section 6.5.4 of this Code is not applicable to weathering steel, for which special cleaning specifications are always required in the *contract documents*.

6.5.1. The *fabricator* is not responsible for deterioration of the shop coat that may result from exposure to ordinary atmospheric conditions or corrosive conditions that are more severe than ordinary atmospheric conditions.

Commentary:
The shop coat of paint is the prime coat of the protective system. It is intended as protection for only a short period of exposure in ordinary atmospheric conditions, and is considered a temporary and provisional coating.

6.5.2. Unless otherwise specified in the *contract documents*, the *fabricator* shall, as a minimum, hand clean the *structural steel* of loose rust, loose mill scale, dirt and other foreign matter, prior to painting, by means of wire brushing or by other methods elected by the *fabricator*, to meet the requirements of SSPC-SP2. If the *fabricator's* workmanship on surface preparation is to be inspected by the inspector, such inspection shall be performed in a timely manner prior to the application of the shop coat.

Commentary:
The selection of a paint system is a design decision involving many factors including:

(a) The *owner's* preference.
(b) The service life of the structure.
(c) The severity of environmental exposure.
(d) The cost of both initial application and future renewals.
(e) The compatibility of the various components that comprise the paint system (surface preparation, shop coat and subsequent coats).

Because the inspection of shop painting must be concerned with workmanship at each stage of the operation, the *fabricator* provides notice of the schedule of operations and affords the *inspector* access to the work site. Inspection must then be coordinated with that schedule so as to avoid delay of the scheduled operations.

Acceptance of the prepared surface must be made prior to the application of the shop coat because the degree of surface preparation cannot be readily verified after painting. Time delay between surface preparation and the application of the shop coat can result in unacceptable deterioration of a properly prepared surface, necessitating a repetition of surface preparation. This is especially true with blast-cleaned surfaces. Therefore, to avoid potential deterioration of the surface, it is assumed that surface preparation is accepted unless it is inspected and rejected prior to the scheduled application of the shop coat.

The shop coat in any paint system is designed to maximize the wetting and adherence characteristics of the paint, usually at the expense of its weathering capabilities. Deterioration of the shop coat normally begins immediately after exposure to the elements and worsens as the duration of exposure is extended. Consequently, extended exposure of the shop coat will likely lead to its deterioration and may necessitate repair, possibly including the repetition of surface preparation and shop coat application in limited areas. With the introduction of high-performance paint systems, avoiding delay in the application of the shop coat has become more critical. High-performance paint systems generally require a greater degree of surface preparation, as well as early application of weathering protection for the shop coat.

Since the *fabricator* does not control the selection of the paint system, the compatibility of the various components of the total paint system, or the length of exposure of the shop coat, the *fabricator* cannot guarantee the performance of the shop coat or any other part of the system. Instead, the fabricator is responsible only for accomplishing the specified surface preparation and for applying the shop coat (or coats) in accordance with the *contract documents*.

This Section stipulates that the *structural steel* is to be cleaned to meet the requirements in SSPC-SP2. This stipulation is not intended to represent an exclusive cleaning level, but rather the level of surface preparation that will be furnished unless otherwise specified in the *contract documents* if the *structural steel* is to be painted.

6.5.3. Unless otherwise specified in the *contract documents*, paint shall be applied by brushing, spraying, rolling, flow coating, dipping or other suitable means, at the election of the *fabricator*. When the term “shop coat,” “shop paint,” or other equivalent term is used with no paint system specified, the *fabricator’s* standard shop paint shall be applied to a minimum dry-film thickness of one mil (25 μm).

6.5.4. Touch-up of abrasions caused by handling after painting shall be the responsibility of the contractor that performs touch-up in the field or field painting.

Commentary:
Touch-up in the field and field painting are not normally part of the *fabricator’s* or the *erector’s* contract.

6.6. Marking and Shipping of Materials

6.6.1. Unless otherwise specified in the *contract documents*, erection marks shall be applied to the *structural steel* members by painting or other suitable means.

Bolt assemblies and loose bolts, nuts and washers shall be shipped in separate closed containers according to length and diameter, as applicable. Pins and other small parts and packages of bolts, nuts and washers shall be shipped in boxes, crates, kegs or barrels. A list and description of the material shall appear on the outside of each closed container.

Commentary:
In most cases, bolts, nuts and other components in a fastener assembly can be shipped loose in separate containers. However, there are exceptions:

- ASTM F3125/F3125M Grades F1852 and F2280 twist-off-type tension-control bolt assemblies must be assembled and shipped in containers according to grade, length and diameter.
- Galvanized ASTM F3125/F3125M Grade A325 bolts and their corresponding ASTM A563 or A563M nuts must be shipped in the same container according to length and diameter.

See these ASTM standards for the applicable requirements and the RCSC Specification for further explanation.

6.7. Delivery of Materials

6.7.1. Fabricated *structural steel* shall be delivered in a sequence that will permit efficient and economical fabrication and erection, and that is consistent with requirements in the *contract documents*. If the owner or *owner's designated representative for construction* wishes to prescribe or control the sequence of delivery of materials, that entity shall specify the required sequence in the *contract documents*. If the *owner's designated representative for construction* contracts separately for delivery and for erection, the *owner's designated representative for construction* shall coordinate planning between contractors.

6.7.2. *Anchor rods*, washers, nuts and other anchorage or grillage materials that are to be built into concrete or masonry shall be shipped so that they will be available when needed. The *owner's designated representative for construction* shall allow the *fabricator* sufficient time to fabricate and ship such materials before they are needed.

6.7.3. If any shortage is claimed relative to the quantities of materials that are shown in the shipping statements, the *owner's designated representative for construction* or the *erector* shall promptly notify the *fabricator* so that the claim can be investigated.

Commentary:
The quantities of material that are shown in the shipping statement are customarily accepted as correct by the *owner's designated representative for construction*, the *fabricator* and the *erector*.

6.7.4. Unless otherwise specified in the *contract documents*, and subject to the approved *approval documents*, the *fabricator* shall limit the number of field splices to that consistent with minimum project cost.

Commentary:
This Section recognizes that the size and weight of *structural steel* assemblies may be limited by shop capabilities, the permissible weight and clearance dimensions of available transportation or job-site conditions.

6.7.5. If material arrives at its destination in damaged condition, the receiving entity shall promptly notify the *fabricator* and carrier prior to unloading the material, or promptly upon discovery prior to erection.

SECTION 7. ERECTION

7.1. Method of Erection

Fabricated *structural steel* shall be erected using methods and a sequence that will permit efficient and economical performance of erection, and that is consistent with the requirements in the *contract documents*. If the *owner* or *owner's designated representative for construction* wishes to prescribe or control the method and/or sequence of erection, or specifies that certain members cannot be erected in their normal sequence, that entity shall specify the required method and sequence in the *contract documents*. If the *owner's designated representative for construction* contracts separately for fabrication services and for erection services, the *owner's designated representative for construction* shall coordinate planning between contractors.

Commentary:
Design modifications are sometimes requested by the erector to allow or facilitate the erection of the *structural steel* frame. When this is the case, the *erector* should notify the *fabricator* prior to the preparation of the *approval documents* so that the *fabricator* may refer the *erector's* request to the *owner's designated representatives for design* and *construction* for resolution.

7.2. Job-Site Conditions

The *owner's designated representative for construction* shall provide and maintain the following for the *fabricator* and the *erector*:

(a) Adequate access roads into and through the job site for the safe delivery and movement of the material to be erected and of derricks, cranes, trucks and other necessary equipment under their own power.
(b) A firm, properly graded, drained, convenient and adequate space at the job site for the operation of the *erector's* equipment, free from overhead obstructions, such as power lines, telephone lines or similar conditions.
(c) Adequate storage space, when the structure does not occupy the full available job site, to enable the *fabricator* and the *erector* to operate at maximum practical speed.

Otherwise, the *owner's designated representative for construction* shall inform the *fabricator* and the *erector* of the actual job-site conditions and/or special delivery requirements prior to bidding.

7.3. Foundations, Piers and Abutments

The accurate location, strength and suitability of, and access to, all foundations, piers and abutments shall be the responsibility of the *owner's designated representative for construction*.

7.4. Lines and Benchmarks

The *owner's designated representative for construction* shall be responsible for the accurate location of lines and benchmarks at the job site and shall furnish the *erector* with a plan that contains all such information. The *owner's designated representative for construction* shall establish offset lines and reference elevations at each level for the *erector's* use in the positioning of *adjustable items* (see Section 7.13.1.3), if any.

7.5. Installation of Anchor Rods, Foundation Bolts, and Other Embedded Items

7.5.1. *Anchor rods*, foundation bolts, and other embedded items shall be set by the *owner's designated representative for construction* in accordance with *embedment drawings* that have been approved by the *owner's designated representatives for design* and *construction*. The variation in location of these items from the dimensions shown in the approved *embedment drawings* shall be as follows:

(a) The vertical variation in location from the specified top of *anchor rod* location shall be equal to or less than plus or minus 1/2 in. (13 mm).

(b) The horizontal variation in location from the specified position of each *anchor rod* centerline at any location along its projection above the concrete shall be equal to or less than the dimensions given for the *anchor rod* diameters listed as follows:

Anchor Rod Diameter, in. (mm)	Horizontal Variation, in. (mm)
3/4 and 7/8 (19 and 22)	1/4 (6)
1, 1 1/4, 1 1/2 (25, 31, 38)	3/8 (10)
1 3/4, 2, 2 1/2 (44, 50, 63)	1/2 (13)

Commentary:
The tolerances established in this Section have been selected for compatibility with the holes sizes that are recommended for base plates in the AISC *Steel Construction Manual*. If special conditions require more restrictive tolerances, such as for smaller holes, the required tolerances should be stated in the *contract documents*. When the *anchor rods* are set in sleeves, the adjustment provided may be used to satisfy the required *anchor-rod* setting tolerances.

7.5.2. Unless otherwise specified in the *contract documents*, *anchor rods* shall be set with their longitudinal axis perpendicular to the theoretical bearing surface.

7.5.3. Embedded items and *connection* materials that are part of the work of other trades, but that will receive *structural steel*, shall be located and set by the *owner's designated representative for construction* in accordance with an approved *embedment drawing*. The variation in location of these items shall be limited to a magnitude that is consistent with the tolerances that are specified in Section 7.13 for the erection of the *structural steel*.

7.5.4. All work that is performed by the *owner's designated representative for construction* shall be completed so as not to delay or interfere with the work of the *fabricator* and the *erector*. The *owner's designated representative for construction* shall conduct a survey of the as-built locations of *anchor rods*, foundation bolts and other embedded items, and shall verify that all items covered in Section 7.5 meet the corresponding tolerances. When corrective action is necessary, the *owner's designated representative for construction* shall obtain the guidance and approval of the *owner's designated representative for design*.

Commentary:
Few *fabricators* or *erectors* have the capability to provide this survey. Under standard practice, it is the responsibility of others.

7.6. Installation of Bearing Devices

All leveling plates, leveling nuts and washers, and loose base and bearing plates that can be handled without a derrick or crane are set to line and grade by the *owner's designated representative for construction*. Loose base and bearing plates that require handling with a derrick or crane shall be set by the *erector* to lines and grades established by the *owner's designated representative for construction*. The *fabricator* shall clearly scribe loose base and bearing plates with lines or other suitable marks to facilitate proper alignment.

Promptly after the setting of *bearing devices*, the *owner's designated representative for construction* shall check them for line and grade. The variation in elevation relative to the established grade for all *bearing devices* shall be equal to or less than plus or minus $^1/_8$ in. (3 mm). The final location of *bearing devices* shall be the responsibility of the *owner's designated representative for construction*.

Commentary:
The $^1/_8$ in. (3 mm) tolerance on elevation of *bearing devices* relative to established grades is provided to permit some variation in setting *bearing devices*, and to account for the accuracy that is attainable with standard surveying instruments. The use of leveling plates larger than 22 in. by 22 in. (550 mm by 550 mm) is discouraged and grouting is recommended with larger sizes. For the purposes of erection stability, the use of leveling nuts and washers is discouraged when base plates have less than four *anchor rods*.

7.7. Grouting

Grouting shall be the responsibility of the *owner's designated representative for construction*. Leveling plates and loose base and bearing plates shall be promptly grouted after they are set and checked for line and grade. Columns with attached base plates, beams with attached bearing plates and other similar members with attached *bearing devices* that are temporarily supported on leveling nuts and washers, shims or other similar leveling devices, shall be promptly grouted after the *structural steel* frame or portion thereof has been plumbed.

Commentary:
In the majority of structures the vertical load from the column bases is transmitted to the foundations through structural grout. In general, there are three methods by which support is provided for column bases during erection:

(a) Pre-grouted leveling plates or loose base plates.
(b) Shims.
(c) Leveling nuts and washers on the *anchor rods* beneath the column base.

Standard practice provides that loose base plates and leveling plates are to be grouted as they are set. *Bearing devices* that are set on shims or leveling nuts are grouted after plumbing, which means that the weight of the erected *structural steel* frame is supported on the shims or washers, nuts and *anchor rods*. The *erector* must take care to ensure that the load that is transmitted in this temporary condition does not exceed the strength of the shims or washers, nuts and *anchor rods*. These considerations are presented in greater detail in AISC Design Guide 1, *Base Plate and Anchor Rod Design*, and AISC Design Guide 10, *Erection Bracing of Low-Rise Structural Steel Frames*.

7.8. Field Connection Material

7.8.1. The *fabricator* shall provide field *connection* details that are consistent with the requirements in the *contract documents* and that will, in the *fabricator's* opinion, result in economical fabrication and erection.

7.8.2. When the *fabricator* is responsible for erecting the *structural steel*, the *fabricator* shall furnish all materials that are required for both temporary and permanent *connection* of the component parts of the *structural steel* frame.

7.8.3. When the erection of the *structural steel* is not performed by the *fabricator*, the *fabricator* shall furnish the following field *connection* material:

(a) Bolts, nuts and washers in sufficient quantity for all *structural steel*-to-*structural steel* field *connections* that are to be permanently bolted. The *fabricator* shall include an extra 2% plus 3 bolts, subject to a minimum of 5 extra bolts, of each grade, type, diameter, length, and production lot number.
(b) Shims that are shown as necessary for make-up of permanent *structural steel*-to-*structural* steel field *connections*.
(c) Steel backing and run-off tabs that are required for field welding.

7.8.4. The *erector* shall furnish all welding electrodes, fit-up bolts and drift pins used for the erection of the *structural steel*. Non-steel backing, if used, shall be furnished by the erector.

Commentary:
See the Commentary for Section 2.2.

7.9. Loose Material

Unless otherwise specified in the *contract documents*, loose *structural steel* items that are not connected to the *structural steel* frame shall be set by the *owner's designated representative for construction* without assistance from the *erector*.

7.10. Temporary Support of Structural Steel Frames

7.10.1. The *owner's designated representative for design* shall identify the following in the *contract documents*:

(a) The lateral force-resisting system and connecting diaphragm elements that provide for lateral strength and stability in the completed structure.

(b) Any special erection conditions or other considerations that are required by the design concept, such as the use of shores, jacks or loads that must be adjusted as erection progresses to set or maintain camber, position within specified tolerances or prestress.

Commentary:

The intent of Section 7.10.1 of the Code is to alert the *owner's designated representative for construction* and the *erector* of the means for lateral force resistance in the completed structure so that appropriate planning can occur for construction of the building. Examples of a description of the lateral force-resisting system as required in Section 7.10.1(a) are shown in the following.

Example 1 is an all-steel building with a composite metal deck and concrete floor system. All lateral force resistance is provided by welded moment frames in each orthogonal building direction. One suitable description of this lateral force-resisting system is:

All lateral force resistance and stability of the building in the completed structure is provided by moment frames with welded beam to column connections framed in each orthogonal direction (see plan sheets for locations). The composite metal deck and concrete floors serve as horizontal diaphragms that distribute the lateral wind and seismic forces horizontally to the vertical moment frames. The vertical moment frames carry the applied lateral loads to the building foundation.

Example 2 is a steel-framed building with a composite metal deck and concrete floor system. All beam-to-column *connections* are simple *connections* and all lateral force resistance is provided by reinforced concrete shear walls in the building core and in the stairwells. One suitable description of this lateral force-resisting system is:

All lateral force resistance and stability of the building in the completed structure is provided exclusively by cast-in-place reinforced concrete shear walls in the building core and stairwells (see plan sheets for locations). These walls provide all lateral force resistance in each orthogonal building direction. The composite metal deck and concrete floors serve as horizontal diaphragms that distribute the lateral wind and seismic forces horizontally to the concrete shear walls. The concrete shear walls carry the applied lateral loads to the building foundation.

See also Commentary Section 7.10.3.

Section 7.10.1(b) is intended to apply to special requirements inherent in the design concept that could not otherwise be known by the *erector*. Such conditions might include designs that require the use of shores or jacks to impart a load or to obtain a specific elevation or position in a subsequent step of the erection process in a sequentially erected structure or member. These requirements would not be apparent to an *erector*, and must be identified so the *erector* can properly bid, plan and perform the erection.

The *erector* is responsible for installation of all members (including cantilevered members) to the specified plumbness, elevation and alignment within the erection tolerances specified in this Code. The *erector* must provide all temporary supports and devices to maintain elevation or position within these tolerances. These works are part of the means and methods of the *erector* and the *owner's designated representative for design* need not specify these methods or related equipment.

See also the preset requirements for cantilevered members in Section 3.1.

7.10.2. The *owner's designated representative for construction* shall indicate to the *erector*, prior to bidding, the installation schedule for non-*structural steel* elements of the lateral force-resisting system and connecting diaphragm elements identified by the *owner's designated representative for design* in the *contract documents*.

Commentary:
See Commentary Section 7.10.3.

7.10.3. Based upon the information provided in accordance with Sections 7.10.1 and 7.10.2, the *erector* shall determine, furnish and install all temporary supports, such as temporary guys, beams, falsework, cribbing or other elements required for the erection operation. These temporary supports shall be sufficient to secure the bare *structural steel* framing or any portion thereof against loads that are likely to be encountered during erection, including those due to wind and those that result from erection operations.

The *erector* need not consider loads during erection that result from the performance of work by, or the acts of, others, except as specifically identified by the *owner's designated representatives for design* and *construction*, nor those that are unpredictable, such as loads due to hurricane, tornado, earthquake, explosion or collision.

Temporary supports that are required during or after the erection of the *structural steel* frame for the support of loads caused by non-*structural steel* elements, including cladding, interior partitions and other such elements that will induce or transmit loads to the *structural steel* frame during or after erection, shall be the responsibility of others.

Commentary:
Many *structural steel* frames have lateral force-resisting systems that are activated during the erection process. Such lateral force-resisting systems may consist of

welded moment frames, braced frames or, in some instances, columns that cantilever from fixed-base foundations. Such frames are normally braced with temporary guys that, together with the steel deck floor and roof diaphragms or other diaphragm bracing that may be included as part of the design, provide stability during the erection process. The guy cables are also commonly used to plumb the *structural steel* frame. The *erector* normally furnishes and installs the required temporary supports and bracing to secure the bare *structural steel* frame, or portion thereof, during the erection process. When *erection bracing drawings* are required in the *contract documents*, those drawings show this information.

If the *owner's designated representative for construction* determines that steel decking is not installed by the *erector*, temporary diaphragm bracing may be required if a horizontal diaphragm is not available to distribute loads to the vertical and lateral force-resisting system. If the steel deck will not be available as a diaphragm during *structural steel* erection, the *owner's designated representative for construction* must communicate this condition to the *erector* prior to bidding. If such diaphragm bracing is required, it must be furnished and installed by the *erector*.

Sometimes structural systems that are employed by the *owner's designated representative for design* rely upon other elements besides the *structural steel* frame for lateral force resistance. For instance, concrete or masonry shear walls or precast spandrels may be used to provide resistance to vertical and lateral forces in the completed structure. Because these situations may not be obvious to the contractor or the *erector*, it is required in this Code that the *owner's designated representative for design* must identify such situations in the *contract documents*. Similarly, if a structure is designed so that special erection techniques are required, such as jacking to impose certain loads or position during erection, it is required in this Code that such requirements be specifically identified in the *contract documents*.

In some instances, the *owner's designated representative for design* may elect to show erection bracing in the structural *design documents*. When this is the case, the *owner's designated representative for design* should then confirm that the bracing requirements were understood by review and approval of the *erection documents* during the submittal process.

Sometimes during construction of a building, collateral building elements, such as exterior cladding, may be required to be installed on the bare *structural steel* frame prior to completion of the lateral force-resisting system. These elements may increase the potential for lateral loads on the temporary supports. Such temporary supports may also be required to be left in place after the *structural steel* frame has been erected. Special provisions should be made by the *owner's designated representative for construction* for these conditions.

7.10.4. All temporary supports that are required for the erection operation and furnished and installed by the *erector* shall remain the property of the *erector* and shall not be modified, moved or removed without the consent of the *erector*. Temporary supports provided by the *erector* shall remain in place until the portion of the *structural steel* frame that they brace is complete and the lateral force-resisting system and connecting diaphragm elements identified by the *owner's designated representative for design* in accordance with Section 7.10.1 are installed. Temporary supports that are required to be left in place after the completion of *structural steel* erection shall be removed when no longer needed by the *owner's designated representative for construction* and returned to the *erector* in good condition.

7.11. Safety Protection

7.11.1. The *erector* shall provide floor coverings, handrails, walkways and other safety protection for the *erector's* personnel as required by law and the applicable safety regulations. Unless otherwise specified in the *contract documents*, the *erector* is permitted to remove such safety protection from areas where the erection operations are completed.

7.11.2. When safety protection provided by the *erector* is left in an area for the use of other trades after the *structural steel* erection activity is completed, the *owner's designated representative for construction* shall:

(a) Accept responsibility for and maintain this protection.
(b) Indemnify the *fabricator* and the *erector* from damages that may be incurred from the use of this protection by other trades.
(c) Ensure that this protection is adequate for use by other affected trades.
(d) Ensure that this protection complies with applicable safety regulations when being used by other trades.
(e) Remove this protection when it is no longer required and return it to the *erector* in the same condition as it was received.

7.11.3. Safety protection for other trades that are not under the direct employment of the *erector* shall be the responsibility of the owner's *designated representative for construction*.

7.11.4. When permanent steel decking is used for protective flooring and is installed by the *owner's designated representative for construction*, all such work shall be scheduled and performed in a timely manner so as not to interfere with or delay the work of the *fabricator* or the *erector*. The sequence of installation that is used shall meet all safety regulations.

7.11.5. Unless the interaction and safety of activities of others, such as construction by others or the storage of materials that belong to others, are coordinated with the work of the *erector* by the *owner's designated representative for construction*, such activities shall not be permitted until the erection of the *structural steel* frame or portion thereof is completed by the *erector* and accepted by the *owner's designated representative for construction*.

7.12. Structural Steel Frame Tolerances

The accumulation of the mill tolerances and fabrication tolerances shall not cause the erection tolerances to be exceeded.

Commentary:
In editions of this Code previous to the 2005 edition, it was stated that "…variations are deemed to be within the limits of good practice when they do not exceed the cumulative effect of rolling tolerances, fabricating tolerances and erection tolerances." It is recognized in the current provision in this Section that accumulations of mill tolerances and fabrication tolerances generally occur between the locations at which erection tolerances are applied, and not at the same locations.

7.13. Erection Tolerances

Erection tolerances shall be defined relative to member working points and working lines, which shall be defined as follows:

(a) For members other than horizontal members, the member work point shall be the actual center of the member at each end of the shipping piece.
(b) For horizontal members, the working point shall be the actual centerline of the top flange or top surface at each end.
(c) The member working line shall be the straight line that connects the member working points.

The substitution of other working points is permitted for ease of reference, provided they are based upon the preceding definitions.

The tolerances on *structural steel* erection shall be in accordance with the requirements in Sections 7.13.1 through 7.13.3.

Commentary:
The erection tolerances defined in this Section have been developed through long-standing usage as practical criteria for the erection of *structural steel*. Erection tolerances were first defined in the 1924 edition of this Code in Section 7(f), "Plumbing Up." With the changes that took place in the types and use of materials in building construction after World War II, and the increasing demand by *architects* and *owners* for more specific tolerances, AISC adopted new standards for erection tolerances in Section 7(h) of the March 15, 1959 edition of this Code. Experience has proven that those tolerances can be economically obtained.

Differential column shortening may be a consideration in design and construction. In some cases, it may occur due to variability in the accumulation of dead load among different columns (see Figure C-7.1). In other cases, it may be characteristic of the structural system that is employed in the design. Consideration of the effects of differential column shortening may be very important, such as when the slab thickness is reduced, when electrical and other similar fittings mounted on the *structural steel* are intended to be flush with the finished floor, and when there is little clearance between bottoms of beams and the tops of door frames or ductwork.

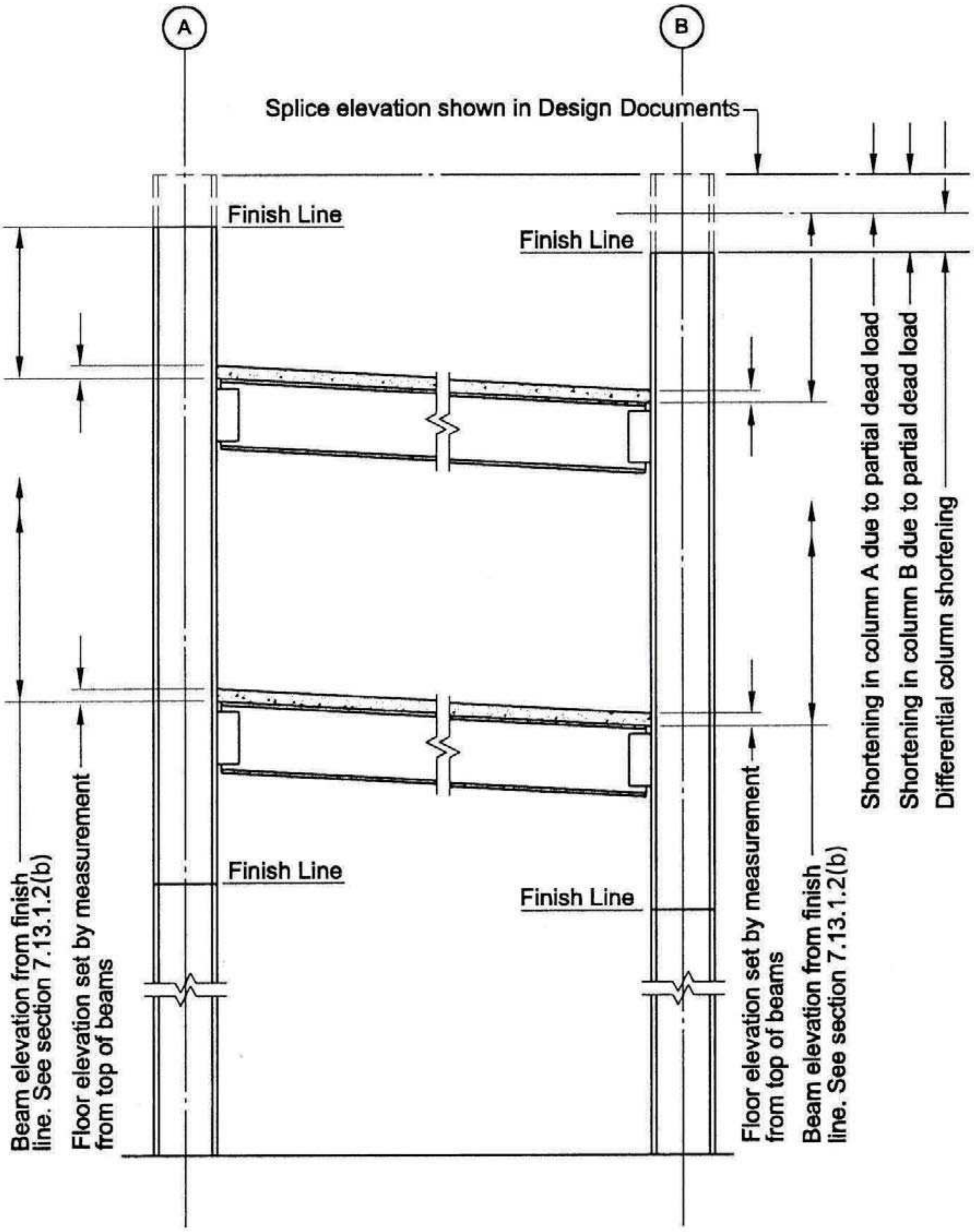


Fig. C-7.1. Effects of differential column shortening.

The effects of the deflection of transfer girders and trusses on the position of columns and hangers supported from them may be a consideration in design and construction. As in the case of differential column shortening, the deflection of these supporting members during and after construction will affect the position and alignment of the framing tributary to these transfer members.

Expansion and contraction in a *structural steel* frame may be a consideration in design and construction. Steel will expand or contract approximately

$^{1}/_{8}$ in. per 100 ft for each change of 15°F (2 mm per 10 000 mm for each change of 15°C) in temperature. This change in length can be assumed to act about the center of rigidity. When anchored to their foundations, end columns will be plumb only when the steel is at normal temperature (see Figure C-7.2). It is therefore necessary to correct field measurements of offsets to the structure from established baselines for the expansion or contraction of the exposed *structural steel* frame. For example, a 200-ft-long (60 000-m-long) building that is plumbed up at 100°F (38°C) should have working points at the tops of the end columns positioned $^{1}/_{2}$ in. (14 mm) further apart than the working points at the corresponding bases in order for the columns to be plumb at 70°F (21°C). Differential temperature effects on column length should also be taken into account in plumbing surveys when tall *structural steel* frames are subjected to sun exposure on one side.

The alignment of lintels, spandrels, wall supports and similar members that are used to connect other building construction units to the *structural steel* frame should have an adjustment of sufficient magnitude to allow for the accumulation of mill tolerances and fabrication tolerances, as well as the erection tolerances. See Figure C-7.3.

7.13.1. The tolerances on position and alignment of member working points and working lines shall be as described in Sections 7.13.1.1 through 7.13.1.3.

7.13.1.1. For an individual column shipping piece, the angular variation of the working line from a plumb line shall be equal to or less than $^{1}/_{500}$ of the distance between working points, subject to the following additional limitations:

(a) For an individual column shipping piece that is adjacent to an elevator shaft, the displacement of member working points shall be equal to or less than 1 in. (25 mm) from the *established column line* in the first 20 stories. Above this level, an increase in the displacement of $^{1}/_{32}$ in. (1 mm) is permitted for each additional story up to a maximum displacement of 2 in. (50 mm) from the *established column line*.

(b) For an exterior individual column shipping piece, the displacement of member working points from the *established column line* in the first 20 stories shall be equal to or less than 1 in. (25 mm) toward and 2 in. (50 mm) away from the building exterior. Above this level, an increase in the displacement of $^{1}/_{16}$ in. (2 mm) is permitted for each additional story up to a maximum displacement of 2 in. (50 mm) toward and 3 in. (75 mm) away from the building exterior.

Commentary:
The limitations that are described in this Section and illustrated in Figures C-7.4 and C-7.5 make it possible to maintain built-in-place or prefabricated facades in a true vertical plane up to the 20th story, if *connections* that provide for 3 in. (75 mm) of adjustment are used. Above the 20th story, the facade may be maintained within $^{1}/_{16}$ in. (2 mm) per story with a maximum total deviation of 1 in. (25 mm) from a true vertical plane, if *connections* that

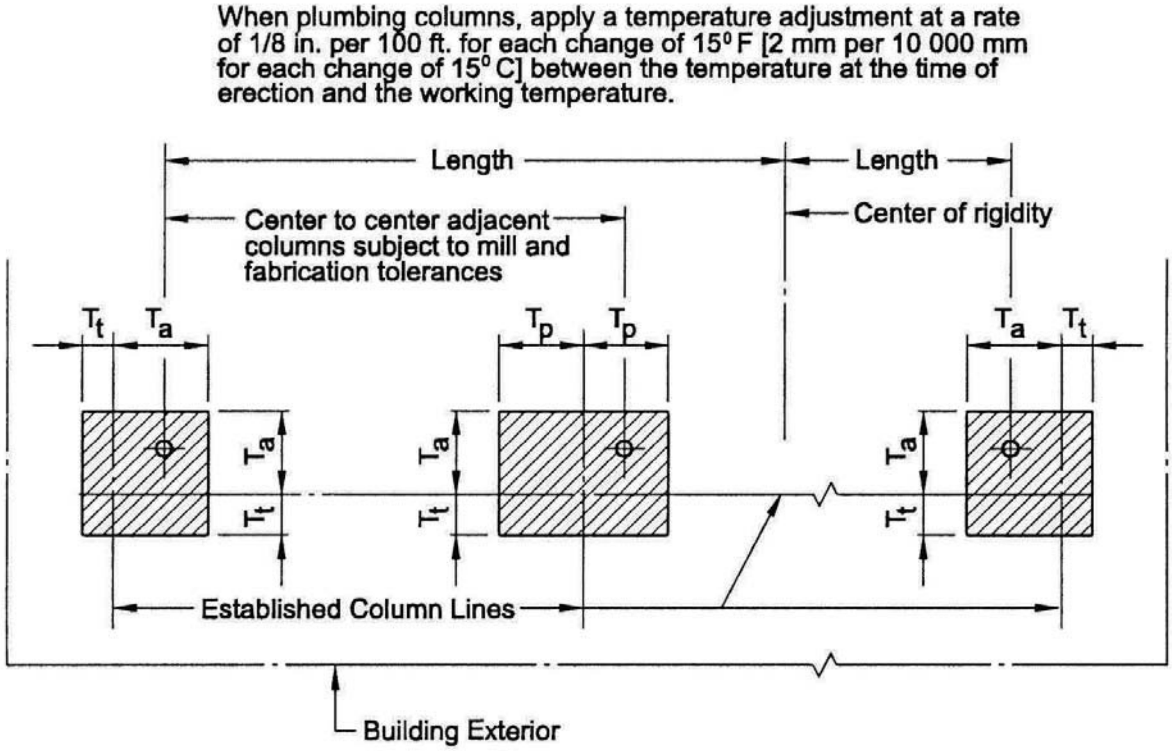


Fig. C-7.2. Tolerances in plan location of column.

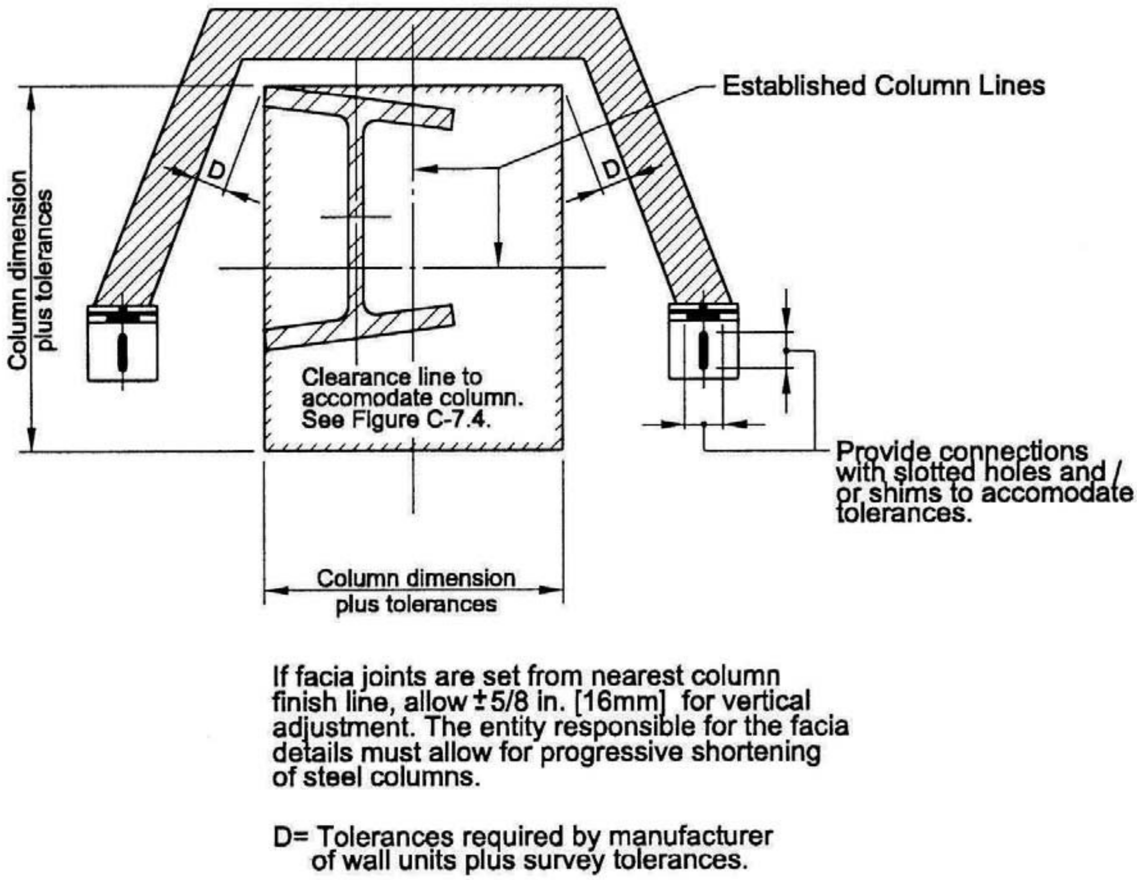


Fig. C-7.3. Clearance required to accommodate fascia.

provide for 3 in. (75 mm) of adjustment are used. *Connections* that permit adjustments of plus 2 in. (50 mm) to minus 3 in. (75 mm)—a total of 5 in. (125 mm)—will be necessary in cases where it is desired to construct the facade to a true vertical plane above the 20th story.

(c) For an exterior individual column shipping piece, the member working points at any splice level for multi-*tier* buildings and at the tops of columns for single-*tier* buildings shall fall within a horizontal envelope, parallel to the exterior *established column line*, that is equal to or less than $1^1/_2$ in. (38 mm) wide for buildings up to 300 ft (90 000 mm) in length. An increase in the width of this horizontal envelope of $^1/_2$ in. (13 mm) is permitted for each additional 100 ft (30 000 mm) in length up to a maximum width of 3 in. (75 mm).

Commentary:
This Section limits the position of exterior column working points at any given splice elevation to a narrow horizontal envelope parallel to the exterior *established column line* (see Figure C–7.6). This envelope is limited to a width of $1^1/_2$ in. (38 mm), normal to the exterior *established column line*, in up to 300 ft (90 000 mm) of building length. The horizontal location of this envelope is not necessarily directly above or below the corresponding envelope at the adjacent splice elevations, but should be within the limitation of the 1 in 500 plumbness tolerance specified for the controlling columns (see Figure C-7.5).

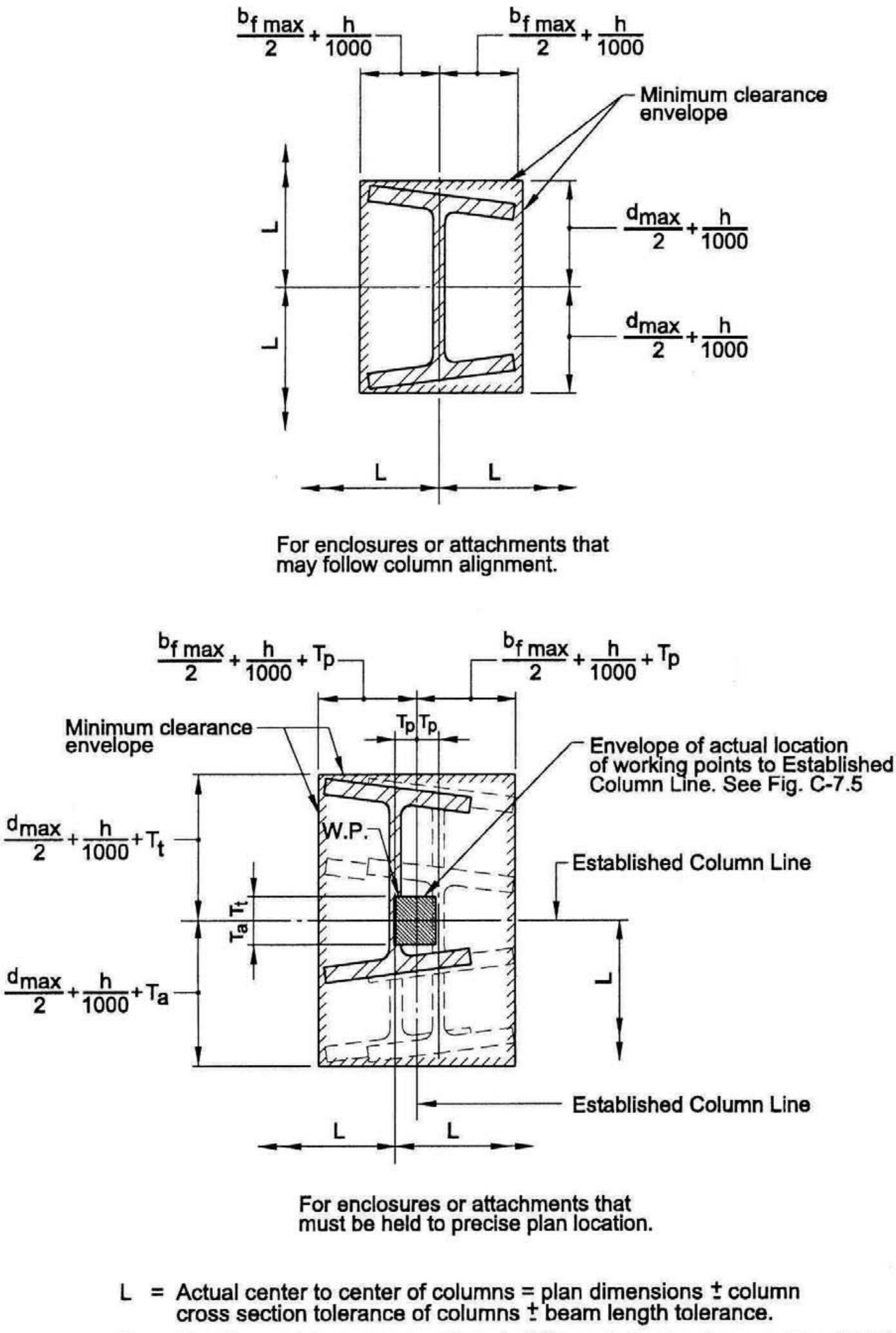
(d) For an exterior column shipping piece, the displacement of member working points from the *established column line* that is nominally parallel to the building exterior shall be equal to or less than 2 in. (50 mm) in the first 20 stories. Above this level, an increase in the displacement of $^1/_{16}$ in. (2 mm) is permitted for each additional story up to a maximum displacement of 3 in. (75 mm) in the direction nominally parallel to the building exterior.

7.13.1.2. For members other than column shipping pieces, the following limitations shall apply:

(a) For a member that consists of an individual, straight shipping piece without field splices, other than a cantilevered member, the variation in alignment shall be acceptable if it is caused solely by variations in column alignment and/or primary supporting member alignment that are within the permissible variations for the fabrication and erection of such members.

(b) For a member that consists of an individual, straight shipping piece that connects to a column, the variation in the distance from the member working point to the upper finished splice line of the column shall be equal to or less than plus $^3/_{16}$ in. (5 mm) and minus $^5/_{16}$ in. (8 mm).

(c) For a member that consists of an individual shipping piece that does not connect to a column, the variation in elevation shall be acceptable if it is caused solely by the variations in the elevations of the supporting members within the permissible variations for the fabrication and erection of those members.



L = Actual center to center of columns = plan dimensions ± column cross section tolerance of columns ± beam length tolerance.

T_a = Plumbness tolerance away from building exterior (varies, see Fig. C-7.5)

T_t = Plumbness tolerance toward building exterior (varies, see Fig. C-7.5)

T_p = Plumbness tolerance parallel to building exterior (= T_a)

Fig. C-7.4. Clearance required to accommodate accumulated column tolerance.

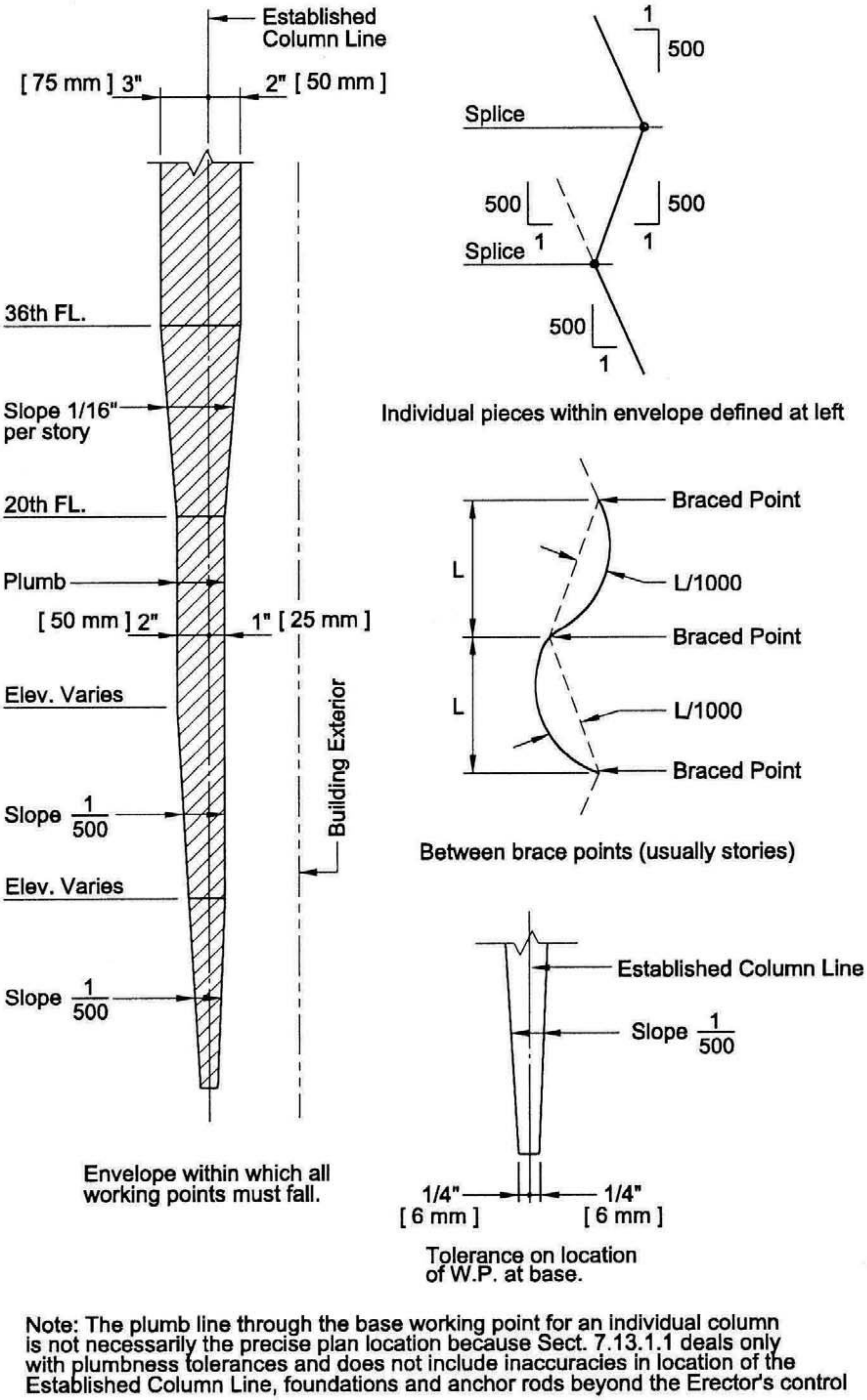


Fig. C-7.5. Exterior column plumbness tolerances normal to building exterior.

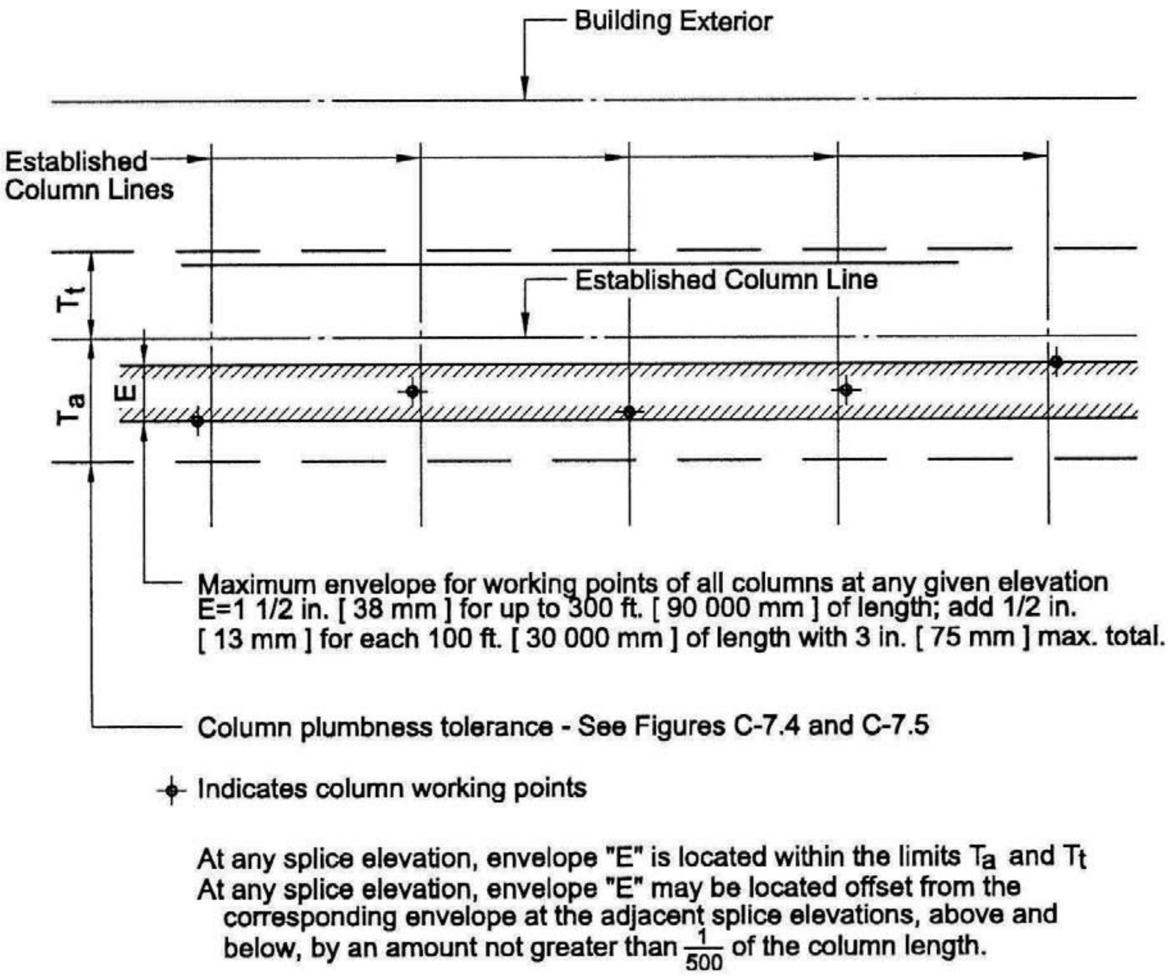


Fig. C-7.6. Tolerances in plan at any splice elevation of exterior columns.

(d) For a member that consists of an individual, straight shipping piece and that is a segment of a field assembled unit containing field splices between points of support, the plumbness, elevation and alignment shall be acceptable if the angular variation, vertically and horizontally, of the working line from a straight line between points of support is equal to or less than $^1/_{500}$ of the distance between working points.

Commentary:
The angular misalignment of the working line of all fabricated shipping pieces relative to the line between support points of the member as a whole in erected position must not exceed 1 in 500. Note that the tolerance is not stated in terms of a linear displacement at any point and is not to be taken as the overall length between supports divided by 500. Typical examples are shown in Figure C-7.7. Numerous conditions within tolerance for these and other cases are possible. The condition described in (d) applies to both plan and elevation tolerances.

(e) For a cantilevered member that consists of an individual, straight shipping piece, the plumbness, elevation and alignment shall be acceptable if the

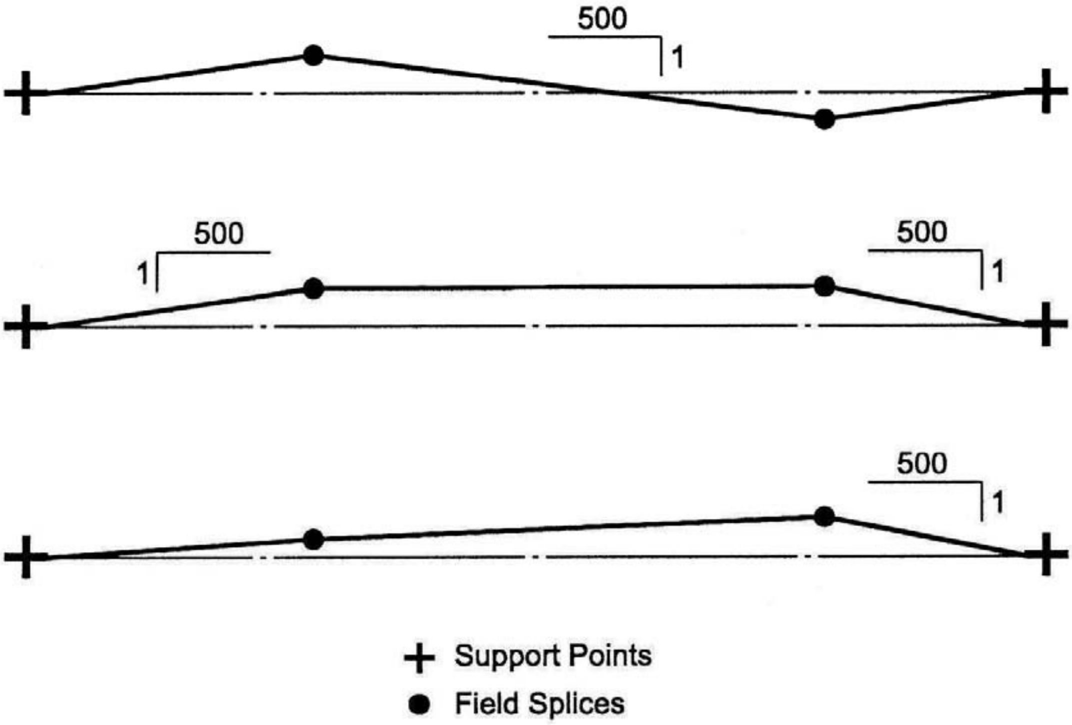


Fig. C-7.7. Alignment tolerances for members with field splices.

angular variation of the working line from a straight line that is extended in the plan direction from the working point at its supported end is equal to or less than $^1/_{500}$ of the distance from the working point at the free end.

Commentary:
This tolerance is evaluated after the fixed end condition is sufficient to stabilize the cantilever and before the temporary support is removed. The preset specified in the *contract documents* should be calculated accordingly. The temporary support cannot be used to induce artificial deflection into the cantilever to meet this tolerance after the fixed end is restrained.

(f) For a member of irregular shape, the plumbness, elevation and alignment shall be acceptable if the fabricated member is within its tolerances and the members that support it are within the tolerances specified in this Code.
(g) For a member that is fully assembled in the field in an unstressed condition, the same tolerances shall apply as if fully assembled in the shop.
(h) For a member that is field-assembled, element-by-element, in place, temporary support shall be used or an alternative erection plan shall be submitted to the *owner's designated representatives for design and construction*. The tolerance in Section 7.13.1.2(d) shall be met in the supported condition with working points taken at the point(s) of temporary support.

Commentary:
Trusses fabricated and erected as a unit or as an assembly of truss segments

> normally have excellent controls on vertical position regardless of fabrication and erection techniques. However, a truss fabricated and erected by assembling individual components in place in the field is potentially more sensitive to deflections of the individual truss components and the partially completed work during erection, particularly the chord members. In such a case, the erection process should follow an erection plan that addresses this issue.

7.13.1.3. For members that are identified as *adjustable items* by the *owner's designated representative for design* in the *contract documents*, the *fabricator* shall provide adjustable *connections* for these members to the supporting *structural steel* frame. Otherwise, the *fabricator* is permitted to provide nonadjustable *connections*. When *adjustable items* are specified, the *owner's designated representative for design* shall indicate the total adjustability that is required for the proper alignment of these supports for other trades. The variation in the position and alignment of *adjustable items* shall be as follows:

(a) The variation in the vertical distance from the upper finished splice line of the nearest column to the support location specified in the structural *design documents* shall be equal to or less than plus or minus $^3/_8$ in. (10 mm).
(b) The variation in the horizontal distance from the established finish line at the particular floor shall be equal to or less than plus or minus $^3/_8$ in. (10 mm).
(c) The variation in vertical and horizontal alignment at the abutting ends of *adjustable items* shall be equal to or less than plus or minus $^3/_{16}$ in. (5 mm).

> **Commentary:**
> When the alignment of lintels, wall supports, curb angles, mullions and similar supporting members for the use of other trades is required to be closer than that permitted by the foregoing tolerances for *structural steel*, the *owner's designated representative for design* must identify such items in the *contract documents* as *adjustable items*.

7.13.2. In the design of steel structures, the *owner's designated representative for design* shall provide for the necessary clearances and adjustments for material furnished by other trades to accommodate the mill tolerances, fabrication tolerances and erection tolerances in this Code for the *structural steel* frame.

> **Commentary:**
> In spite of all efforts to minimize inaccuracies, deviations will still exist; therefore, in addition, the designs of prefabricated wall panels, partition panels, fenestrations, floor-to-ceiling door frames, and similar elements must provide for clearance and details for adjustment as described in Section 7.13.2. Designs must provide for adjustment in the vertical dimension of prefabricated facade

panels that are supported by the *structural steel* frame because the accumulation of shortening of loaded steel columns will result in the unstressed facade supported at each floor level being higher than the *structural steel* framing to which it must be attached. Observations in the field have shown that where a heavy facade is erected to a greater height on one side of a multistory building than on the other, the *structural steel* framing will be pulled out of alignment. Facades should be erected at a relatively uniform rate around the perimeter of the structure.

7.13.3. Prior to placing or applying any other materials, the *owner's designated representative for construction* shall determine that the location of the *structural steel* is acceptable for plumbness, elevation and alignment. The *erector* shall be given either timely notice of acceptance by the *owner's designated representative for construction* or a listing of specific items that are to be corrected in order to obtain acceptance. Such notice shall be rendered promptly upon completion of any part of the work and prior to the start of work by other trades that may be supported, attached or applied to the *structural steel* frame.

7.14. Correction of Errors

The correction of minor misfits by moderate amounts of reaming, grinding, welding or cutting, and the drawing of elements into line with drift pins, shall be considered to be normal erection operations. Errors that cannot be corrected using the foregoing means, or that require major changes in member or *connection* configuration, shall be promptly reported to the *owner's designated representatives for design* and *construction* and the *fabricator* by the *erector*, to enable the responsible entity to either correct the error or approve the most efficient and economical method of correction to be used by others.

Commentary:
As used in this Section, the term "moderate" refers to the amount of reaming, grinding, welding or cutting that must be done on the project as a whole, not the amount that is required at an individual location. It is not intended to address limitations on the amount of material that is removed by reaming at an individual bolt hole, for example, which is limited by the bolt-hole size and tolerance requirements in ANSI/AISC 360 and the RCSC Specification.

7.15. Cuts, Alterations and Holes for Other Trades

Neither the *fabricator* nor the *erector* shall cut, drill or otherwise alter their work, nor the work of other trades, to accommodate other trades, unless such work is clearly specified in the *contract documents*. When such work is so specified, the *owner's designated representatives for design* and *construction* shall furnish complete information as to materials, size, location and number of alterations in a timely manner so as not to delay the preparation of the *approval documents*.

7.16. Handling and Storage

The *erector* shall take reasonable care in the proper handling and storage of the *structural steel* during erection operations to avoid the accumulation of excess dirt and foreign matter. The *erector* shall not be responsible for the removal from the *structural steel* of dust, dirt or other foreign matter that may accumulate during erection as the result of job-site conditions or exposure to the elements. The *erector* shall handle and store all bolts, nuts, washers and related fastening products in accordance with the requirements of the RCSC Specification.

Commentary:
During storage, loading, transport, unloading and erection, blemish marks caused by slings, chains, blocking, tie-downs, etc., occur in varying degrees. Abrasions caused by handling or cartage after painting are to be expected. It must be recognized that any shop-applied coating, no matter how carefully protected, will require touching up in the field. Touching up these blemished areas is the responsibility of the contractor performing the field touch-up or field painting.

The *erector* is responsible for the proper storage and handling of fabricated *structural steel* at the job site during erection. Shop-painted *structural steel* that is stored in the field pending erection should be kept free of the ground and positioned so as to minimize the potential for water retention. The *owner* or *owner's designated representative for construction* is responsible for providing suitable job-site conditions and proper access so that the *fabricator* and the *erector* may perform their work.

Job-site conditions are frequently muddy, sandy, dusty or a combination thereof during the erection period. Under such conditions, it may be impossible to store and handle the *structural steel* in such a way as to completely avoid any accumulation of mud, dirt or sand on the surface of the *structural steel*, even though the *fabricator* and the *erector* manages to proceed with their work.

Repairs of damage to painted surfaces and/or removal of foreign materials due to adverse job-site conditions are outside the scope of responsibility of the *fabricator* and the *erector* when reasonable attempts at proper handling and storage have been made.

7.17. Field Painting

Neither the *fabricator* nor the *erector* is responsible to paint field bolt heads and nuts or field welds, nor to touch up abrasions of the shop coat, nor to perform any other field painting.

7.18. Final Cleaning Up

Upon the completion of erection and before final acceptance, the *erector* shall remove all of the *erector's* falsework, rubbish and temporary buildings.

SECTION 8. QUALITY CONTROL

8.1. General

8.1.1. The *fabricator* shall maintain a quality control program to ensure that the work is performed in accordance with the requirements in this Code, ANSI/AISC 360 and the *contract documents*. The *fabricator* shall have the option to use the AISC Quality Certification Program to establish and administer the quality control program.

> **Commentary:**
> The AISC Quality Certification Program confirms to the construction industry that a certified *structural steel* fabrication shop has the capability by reason of commitment, personnel, organization, experience, procedures, knowledge and equipment to produce fabricated *structural steel* of the required quality for a given category of work. The AISC Quality Certification Program is not intended to involve inspection and/or judgment of product quality on individual projects. Neither is it intended to guarantee the quality of specific fabricated *structural steel* products.

8.1.2. The *erector* shall maintain a quality control program to ensure that the work is performed in accordance with the requirements in this Code, ANSI/AISC 360 and the *contract documents*. The *erector* shall be capable of performing the erection of the *structural steel*, and shall provide the equipment, personnel and management for the scope, magnitude and required quality of each project. The *erector* shall have the option to use the AISC Erector Certification Program to establish and administer the quality control program.

> **Commentary:**
> The AISC Erector Certification Program confirms to the construction industry that a certified *structural steel erector* has the capability by reason of commitment, personnel, organization, experience, procedures, knowledge and equipment to erect fabricated *structural steel* to the required quality for a given category of work. The AISC Erector Certification Program is not intended to involve inspection and/or judgment of product quality on individual projects. Neither is it intended to guarantee the quality of specific erected *structural steel* products.

8.1.3. When the *owner* requires more extensive quality control procedures, or independent inspection by qualified personnel, or requires that the *fabricator* must be certified under the AISC Quality Certification Program and/or requires that the *erector* must be certified under the AISC Erector Certification Program, this shall be clearly stated in the *contract documents*, including a definition of the scope of such inspection.

8.2. Inspection of Mill Material

Material test reports shall constitute sufficient evidence that the mill product satisfies material order requirements. The *fabricator* shall make a visual inspection of material that is received from the mill, but need not perform any material tests unless the *owner's designated representative for design* specifies in the *contract documents* that additional testing is to be performed at the *owner's* expense.

8.3. Nondestructive Testing

When nondestructive testing is required, the process, extent, technique and standards of acceptance shall be clearly specified in the *contract documents*.

8.4. Surface Preparation and Shop Painting Inspection

Inspection of surface preparation and shop painting shall be planned for the acceptance of each operation as the *fabricator* completes it. Inspection of the paint system, including material and thickness, shall be made promptly upon completion of the paint application. When wet-film thickness is to be inspected, it shall be measured during the application.

8.5. Independent Inspection

When inspection by personnel other than those of the *fabricator* and/or *erector* is specified in the contract documents, the requirements in Sections 8.5.1 through 8.5.6 shall be met.

8.5.1. The *fabricator* and the *erector* shall provide the *inspector* with access to all places where the work is being performed. A minimum of 24 hours notification shall be given prior to the commencement of work.

8.5.2. Inspection of shop work by the *inspector* shall be performed in the *fabricator's* shop to the fullest extent possible. Such inspections shall be timely, in-sequence and performed in such a manner as will not disrupt fabrication operations and will permit the repair of nonconforming work prior to any required painting while the material is still in-process in the fabrication shop.

8.5.3. Inspection of field work shall be promptly completed without delaying the progress or correction of the work.

8.5.4. Rejection of material or workmanship that is not in conformance with the *contract documents* shall be permitted at any time during the progress of the work. However, this provision shall not relieve the *owner* or the *inspector* of the obligation for timely, in-sequence inspections.

8.5.5. The *fabricator*, *erector*, and *owner's designated representatives for design* and *construction* shall be informed of deficiencies that are noted by the *inspector* promptly after the inspection. Copies of all reports prepared by the *inspector* shall be promptly given to the *fabricator*, *erector*, and *owner's designated representatives for design* and *construction*. The necessary corrective work shall be performed in a timely manner.

8.5.6. The *inspector* shall not suggest, direct or approve the *fabricator* or *erector* to deviate from the *contract documents* or the approved *approval documents*, or approve such deviation, without the written approval of the *owner's designated representatives for design* and *construction*.

SECTION 9. CONTRACTS

9.1. Types of Contracts

9.1.1. For contracts that stipulate a lump sum price, the work that is required to be performed by the *fabricator* and the *erector* shall be completely defined in the *contract documents*.

9.1.2. For contracts that stipulate a price per pound, the scope of work that is required to be performed by the *fabricator* and the *erector*, the type of materials, the character of fabrication and the conditions of erection shall be based upon the *contract documents*, which shall be representative of the work to be performed.

9.1.3. For contracts that stipulate a price per item, the work that is required to be performed by the *fabricator* and the *erector* shall be based upon the quantity and the character of the items that are described in the *contract documents*.

9.1.4. For contracts that stipulate unit prices for various categories of *structural steel*, the scope of work that is required to be performed by the *fabricator* and the *erector* shall be based upon the quantity, character and complexity of the items in each category as described in the *contract documents*, and shall also be representative of the work to be performed in each category.

9.1.5. When an *allowance* for work is called for in the *contract documents* and the associated work is subsequently defined as to the quantity, complexity and timing of that work after the contract is executed, the contract price for this work shall be adjusted by change order.

Commentary:
Allowances, if used, are not a true definition of the cost of work to be performed. By nature, an *allowance* is only an estimate and placeholder in the bid. Once the actual work is defined, the actual cost can be provided. It must be recognized that the actual cost can be higher or lower than the *allowance*. See Section 9.4.

Allowances required by the *contract documents* or proposed by the bidder should be as thoroughly defined as practicable as to the distinct nature of the work covered by the *allowance*, including whether the *allowance* is to include materials only, fabrication costs and/or erection costs.

9.2. Calculation of Weights

Unless otherwise specified in the contract, for contracts stipulating a price per pound for fabricated *structural steel* that is delivered and/or erected, the quantities of materials for payment shall be determined by the calculation of the gross weight of materials as shown in the *fabrication documents*.

Commentary:
The standard procedure for calculation of weights that is described in this Code meets the need for a universally acceptable system for defining "pay weights" in contracts based upon the weight of delivered and/or erected materials. These

procedures permits the *owner* to easily and accurately evaluate price-per-pound proposals from potential suppliers and enables all parties to a contract to have a clear and common understanding of the basis for payment.

The procedure in this Code affords a simple, readily understood method of calculation that will produce pay weights that are consistent throughout the industry and that may be easily verified by the *owner*. While this procedure does not produce actual weights, it can be used by purchasers and suppliers to define a widely accepted basis for bidding and contracting for *structural steel*. However, any other system can be used as the basis for a contractual agreement. When other systems are used, both the supplier and the purchaser should clearly understand how the alternative procedure is handled.

9.2.1. The unit weight of steel shall be taken as 490 lb/ft^3 (7 850 kg/m^3). The unit weight of other materials shall be in accordance with the manufacturer's published data for the specific product.

9.2.2. The weights of *standard structural shapes*, plates and bars shall be calculated on the basis of *fabrication documents* that show the actual quantities and dimensions of material to be fabricated, as follows:

(a) The weights of all *standard structural shapes* shall be calculated using the nominal weight per ft (mass per m) and the detailed overall length.
(b) The weights of plates and bars shall be calculated using the detailed overall rectangular dimensions.
(c) When parts can be economically cut in multiples from material of larger dimensions, the weight shall be calculated on the basis of the theoretical rectangular dimensions of the material from which the parts are cut.
(d) When parts are cut from *standard structural shapes*, leaving a nonstandard section that is not useable on the same contract, the weight shall be calculated using the nominal weight per ft (mass per m) and the overall length of the *standard structural shapes* from which the parts are cut.
(e) Deductions shall not be made for material that is removed for cuts, copes, clips, blocks, drilling, punching, boring, slot milling, planing or weld joint preparation.

9.2.3. The items for which weights are shown in tables in the AISC *Steel Construction Manual* shall be calculated on the basis of the tabulated weights shown therein.

9.2.4. The weights of items that are not shown in tables in the AISC *Steel Construction Manual* shall be taken from the manufacturer's catalog and the manufacturer's shipping weight shall be used.

Commentary:
Many items that are weighed for payment purposes are not tabulated with weights in the AISC *Steel Construction Manual*. These include, but are not limited to, *anchor rods*, clevises, turnbuckles, sleeve nuts, recessed-pin nuts, cotter pins and similar devices.

9.2.5. The weights of shop or field weld metal and protective coatings shall not be included in the calculated weight for the purposes of payment.

9.3. Revisions to the Contract Documents

Revisions to the *contract documents* shall be confirmed by change order or extra work order. Unless otherwise noted, the issuance of a *revision* to the *contract documents* shall constitute authorization by the *owner* that the *revision* is *released for construction*. The contract price and schedule shall be adjusted in accordance with Sections 9.4 and 9.5.

9.4. Contract Price Adjustment

9.4.1. When the scope of work and responsibilities of the *fabricator* and the *erector* are changed from those previously established in the *contract documents*, an appropriate modification of the contract price shall be made. In computing the contract price adjustment, the *fabricator* and the *erector* shall consider the quantity of work that is added or deleted, the modifications in the character of the work, and the timeliness of the change with respect to the status of material ordering, detailing, fabrication and erection operations.

> **Commentary:**
> The fabrication and erection of *structural steel* is a dynamic process. Typically, material is being acquired at the same time that the *approval documents* are being prepared. Additionally, the fabrication shop will normally fabricate pieces in the order that the *structural steel* is being shipped and erected.
>
> Items that are revised or placed on hold generally upset these relationships and can be very disruptive to the digital modeling/detailing, fabricating and erecting processes. The provisions in Sections 3.5, 4.4.2 and 9.3 are intended to minimize these disruptions so as to allow work to continue. Accordingly, it is required in this Code that the reviewer of requests for contract price adjustments recognize this and allow compensation to the *fabricator* and the *erector* for these inefficiencies and for the materials that are purchased and the detailing, fabrication and erection that has been performed, when affected by the change.

9.4.2. Requests for contract price adjustments shall be presented by the *fabricator* and/or the *erector* in a timely manner and shall be accompanied by a description of the change that is sufficient to permit evaluation and timely approval by the *owner*.

9.4.3. Price-per-pound and price-per-item contracts shall provide for additions or deletions to the quantity of work that are made prior to the time the work is *released for construction*. When changes are made to the character of the work at any time, or when additions and/or deletions are made to the quantity of the work after it is released for detailing, fabrication or erection, the contract price shall be equitably adjusted.

9.5. Scheduling

9.5.1. The contract schedule shall state when the *design documents* will be *released for construction*, if the *design documents* are not available at the time of bidding, and when the job site, foundations, piers and abutments will be ready, free from obstructions and accessible to the *erector*, so that erection can start at the designated time and continue without interference or delay caused by the *owner's designated representative for construction* or other trades.

9.5.2. The *fabricator* and the *erector* shall advise the *owner's designated representatives for design* and *construction*, in a timely manner, of the effect any *revision* has on the contract schedule.

9.5.3. If the fabrication or erection is significantly delayed due to *revisions* to the requirements of the contract, or for other reasons that are the responsibility of others, the *fabricator* and/or *erector* shall be compensated for the additional costs incurred.

9.6. Terms of Payment

The *fabricator* shall be paid for *mill materials* and fabricated product that is stored off the job site. Other terms of payment for the contract shall be outlined in the *contract documents*.

Commentary:
These terms include such items as progress payments for material, fabrication, erection, retainage, performance and payment bonds, and final payment. If a performance or payment bond, paid for by the *owner*, is required by contract, no retainage shall be required.

SECTION 10. ARCHITECTURALLY EXPOSED STRUCTURAL STEEL

10.1. General Requirements

When members are specifically designated as *architecturally exposed structural steel* or *AESS* in the *contract documents*, the requirements in Sections 1 through 9 shall apply as modified in Section 10. Surfaces exposed to view of *AESS* members and components shall be fabricated and erected with the care and dimensional tolerances that are stipulated in Sections 10.2 through 10.6.

Commentary:
The designation of steel as *AESS* adds cost, and that cost is higher as the level of the *AESS* designation increases. However, not all exposed steel must be designated as *AESS*. There are many applications in which the as-produced appearance of fabricated and erected structural steel may be deemed sufficient without any special additional work.

10.1.1. The following categories shall be used when referring to *AESS*:

AESS 1: Basic elements.
AESS 2: Feature elements viewed at a distance greater than 20 ft (6 m).
AESS 3: Feature elements viewed at a distance less than 20 ft (6 m).
AESS 4: Showcase elements with special surface and edge treatment beyond fabrication.
AESS C: Custom elements with characteristics described in the *contract documents*.

Commentary:
The categories are listed in the *AESS* matrix shown in Table 10.1. Each category describes characteristics with successively more detailed—and costly—requirements.

- Basic elements in AESS 1 are those that have workmanship requirements that exceed what would be done in non-AESS construction.
- Feature elements in AESS 2 and 3 exceed the basic requirements, but the intent is to allow the viewer to see the art of metalworking. AESS 2 is achieved primarily through geometry without finish work, and treats things that can be seen at a larger viewing distance, like enhanced treatment of bolts, welds, *connection* and fabrication details, and tolerances for gaps, copes and similar details. AESS 3 is achieved through geometry and basic finish work, and treats things that can be seen at a closer viewing distance or are subject to touch by the viewer, with welds that are generally smooth but visible. AESS 3 involves the use of a mock-up and acceptance is based upon the approved conditions of the mock-up.
- Showcase elements in AESS 4 are those for which the designer intends that the form is the only feature showing in an element. All welds are ground and filled, edges are ground square and true. All surfaces are filled and sanded to a smoothness that doesn't catch on a cloth or glove. Tolerances of fabricated forms are more stringent—generally half of standard tolerance. AESS 4 involves the use of a mock-up and acceptance is based upon the approved conditions of the mock-up.

- Custom elements in AESS C are those with other requirements defined in the *contract documents*.

10.1.2. A mock-up shall be required for AESS 3, 4 and C. If a mock-up is to be used in other *AESS* categories, it shall be specified in the *contract documents*. When required, the nature and extent of the mock-up shall be specified in the *contract documents*. Alternatively, when a mock-up is not practical, the first piece of an element or *connection* can be used to determine acceptability.

Commentary:
Generally, a mock-up is produced and approved in the shop and subsequently placed in the field. The acceptability of the mock-up can be affected by many factors, including distance of view, lighting and finishing. The expectations for the location and conditions of the mock-up at time of approval should be defined in the *contract documents*.

10.2. Contract Documents

The following additional information shall be provided in the *contract documents* when *AESS* is specified:

(a) Specific identification of members or components that are *AESS* using the AESS Categories listed in Section 10.1.2 and Table 10.1.
(b) Fabrication and/or erection tolerances that are to be more restrictive than provided for in this Appendix, if any.
(c) For Category AESS C, the *AESS* matrix included in Table 10.1 shall be used to specify the required treatment of the element.
(d) Any variations from the *AESS* characteristics of Table 10.1.
(e) Any other special requirements for *AESS* members and components, such as the orientation of HSS weld seams and bolt heads.

10.3. Approval Documents

All members designated as *AESS* shall be clearly identified to a Category, either AESS 1, 2, 3, 4 or C, in the *approval documents*. Tack welds, temporary braces, backing and fixtures used in fabrication of *AESS* shall be shown in the *fabrication documents*. Architecturally sensitive *connection* details shall be submitted for approval by the *owner's designated representative for design* prior to completion of the *approval documents*.

Commentary:
Variations, if any, from the AESS Categories listed must be clearly noted. These variations could include machined surfaces, locally abraded surfaces, and forgings. In addition, if distinction is to be made between different surfaces or parts of members, the transition line/plane must be clearly identified/defined on the *approval documents*.

TABLE 10.1
AESS Category Matrix

Id	Category / Characteristics	AESS C Custom Elements	AESS 4 Showcase Elements	AESS 3 Feature Elements in close view	AESS 2 Feature Elements not in close view	AESS 1 Basic Elements	SSS Standard Structural Steel
1.1	Surface preparation to SSPC-SP 6		•	•	•	•	
1.2	Sharp edges ground smooth		•	•	•	•	
1.3	Continuous weld apprearance		•	•	•	•	
1.4	Standard structural bolts		•	•	•	•	
1.5	Weld spatters removed		•	•	•	•	
2.1	Visual samples		•	•	optional		
2.2	One-half standard fabrication tolerances		•	•	•		
2.3	Fabrication marks not apparent		•	•	•		
2.4	Welds uniform and smooth		•	•	•		
3.1	Mill marks removed		•	•			
3.2	Butt and plug welds ground smooth and filled		•	•			
3.3	HSS weld seam oriented for reduced visibility		•	•			
3.4	Cross sectional abutting surface aligned		•	•			
3.5	Joint gap tolerances minimized		•	•			
3.6	All welded connections		optional	optional			
4.1	HSS seam not apparent		•				
4.2	Welds contoured and blended		•				
4.3	Surfaces filed and sanded		•				
4.4	Weld show-through minimized		•				
C.1							
C.2							
C.3							
C.4							
C.5							

User Note:

1.1 Prior to blast cleaning, grease and oil are removed by solvent cleaning to meet SSPC-SP1.
1.2 Rough surfaces are deburred and ground smooth. Sharp edges resulting from flame cutting, grinding and especially shearing are softened.
1.3 Intermittent welds are made continuous, either with additional welding, caulking or body filler. For corrosive enviroments, all joints are seal welded. Seams of hollow structural sections are acceptable as produced.
1.4 All bolt heads in connections are on the same side, as specified, and consistent from one connection to another.
1.5 Weld spatter, slivers, surface discontinuities are removed. Weld projection up to 1/16 in. (2 mm) is acceptable for butt and plug welded joints.

2.1 Visual samples are either a 3-D rendering, a physical sample, a first-off inspection, a scaled mock-up or a full-scale mock-up, as specifed in the *contract documents*.
2.2 These tolerances are one-half of those for standard structural steel as specified in this Code.
2.3 Members markings during the fabrication and erection processes are not visible.

3.1 All mill marks are not visible in the finished product.
3.2 Caulking or body filler is acceptable.
3.3 Seams are oriented away from view or as indicated in the *contract documents*.
3.4 The matching of abutting cross sections is required.
3.5 This characteristic is similar to 2.2 above. A clear distance between abutting members of 1/8 in. (3 mm) is required.
3.6 Hidden bolts may be considered.

4.1 HSS seams are treated so they are not apparent.
4.2 In addidtion to a contoured and blended appearance, welded transitions between members also are contoured and blended.
4.3 The steel surface imperfections are filled and sanded.
4.4 Weld show-through on the back side of a welded element can be minimized by hand grinding the back side surface. The degree of weld-through is a function of weld size and material.

C. Aditional characteristics may be added for custom elements.

10.4. Fabrication

10.4.1. The *fabricator* shall handle the steel with care to avoid marking or distorting the steel members:

(a) Slings shall be nylon-type or chains or wire rope with softeners.
(b) Care shall be taken to minimize damage to any shop paint or coating.
(c) When temporary braces or fixtures are required during fabrication or shipment, or to facilitate erection, care shall be taken to avoid blemishes or unsightly surfaces resulting from the use or removal of such temporary elements.
(d) Tack welds not incorporated into final welds shall be treated consistently with requirements for final welds.
(e) All backing and runoff tabs shall be removed and the welds ground smooth.
(f) All bolt heads in *connections* shall be on the same side, as specified, and consistent from one *connection* to another.

10.4.2. Members fabricated of unfinished, reused, galvanized or weathering steel that are to be *AESS* may still have erection marks, painted marks or other marks on surfaces in the completed structure. Special requirements, if any, shall be specified as Category AESS C.

10.4.3. The permissible tolerances for member depth, width, out of square, and camber and sweep shall be as specified in ASTM A6/A6M and ASTM A500/A500M. The following exceptions apply:

(a) For Categories AESS 3 and 4, the matching of abutting cross sections shall be required.
(b) For Categories AESS 2, 3 and 4, the as-fabricated straightness tolerance shall be one-half of that specified in ASTM A6/A6M and ASTM A500/A500M.

10.4.4. For curved structural members, whether composed of a single *standard structural shape* or built-up, the as-fabricated variation from the theoretical curvature shall be equal to or less than the standard camber and sweep tolerances permitted for straight members in the applicable ASTM standard.

Commentary:
The curvature tolerance for curved *AESS* members is not reduced from that used for curved non-*AESS* members because curved members have no straight line to sight and the resulting deviations are therefore indistinguishable. See also the Commentary to Section 6.4.2.

10.4.5. The tolerance on overall profile dimensions of welded built-up members shall meet the requirements in AWS D1.1/D1.1M. For Categories AESS 2, 3 and 4, the as-fabricated straightness tolerance for the member as a whole shall be one-half of that specified in AWS D1.1/D1.1M.

10.4.6. For Categories AESS 3 and 4, copes, miters and cuts in surfaces exposed to view shall have a gap that is uniform within $^{1}/_{8}$ in. (3 mm), if shown to be an open joint. If instead the joint is shown to be in contact, the contact shall be uniform within $^{1}/_{16}$ in. (2 mm).

10.4.7. For Categories AESS 1, 2 and 3, the surface condition of steel given in ASTM A6/A6M shall be acceptable. For Category AESS 4, surface imperfections shall be filled and sanded to meet the acceptance criteria established with the mock-up required in Section 10.1.2.

10.4.8. For Categories AESS 1, 2 and 3, welds shall meet AWS D1.1/D1.1M requirements, except that weld spatter exposed to view, if any, shall be removed. For Category AESS 4, welds shall be contoured and blended, and spatter exposed to view, if any, shall be removed.

10.4.9. For Categories AESS 1 and 2, weld projection up to $^1/_{16}$ in. (2 mm) is acceptable for butt and plug welded joints. For Categories AESS 3 and 4, welds shall be ground smooth/filled.

10.4.10. For Categories AESS 1, 2 and 3, *weld show-through* shall be acceptable as produced. For Category AESS 4, the *fabricator* shall minimize the *weld show-through*.

Commentary:
Weld show-through is a visual indication of the presence of a weld or welds on the opposite surface from the viewer. It is a function of weld size and material thickness and can't be eliminated in thin material with thick welds. When *weld show-through* is a concern, this should be addressed in the mock-up.

10.4.11. *AESS* shall be prepared to meet the requirement of SSPC-SP 6. Prior to blast cleaning:

(a) Grease or oil, if any is present, shall be removed by solvent cleaning to meet the requirements of SSPC-SP 1.
(b) Weld spatter, slivers and similar surface discontinuities shall be removed.
(c) Sharp corners resulting from shearing, flame cutting or grinding shall be eased.

10.4.12. For Categories AESS 1 and 2, seams of hollow structural sections shall be acceptable as produced. For Category AESS 3, seams shall be oriented as specified in the *contract documents*. For Category AESS 4, seams shall be treated so they are not apparent.

10.5. Delivery of Materials

The *fabricator* shall use special care to avoid bending, twisting or otherwise distorting *AESS*. All tie-downs on loads shall be nylon straps or chains with softeners to avoid damage to edges and surfaces of members. The standard for acceptance of delivered and erected members shall be equivalent to the standard employed at fabrication.

10.6. Erection

The *erector* shall use special care in unloading, handling and erecting *AESS* to avoid marking or distorting the *AESS*. The *erector* shall plan and execute all operations in such a manner that allows the architectural appearance of the structure to be maintained:

(a) Slings shall be nylon-type or chains or wire rope with softeners.
(b) Care shall be taken to minimize damage to any shop paint or coating.
(c) When temporary braces or fixtures are required to facilitate erection, care shall be taken to avoid any blemishes, holes or unsightly surfaces resulting from the use or removal of such temporary elements.
(d) Tack welds not incorporated into final welds shall be ground smooth.
(e) All backing and runoff tabs shall be removed and the welds ground smooth.
(f) All bolt heads in *connections* shall be on the same side, as specified, and consistent from one *connection* to another.
(g) For Category AESS 4, open holes shall be filled with weld metal or body filler and smoothed by grinding or filling to the standards applicable to the shop fabrication of the materials.

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PART 17

MISCELLANEOUS DATA AND MATHEMATICAL INFORMATION

Table 17-1
SI Equivalents of Standard U.S. Shape Profiles
W-Shapes

Shape	SI Equivalent	Shape	SI Equivalent	Shape	SI Equivalent
in. × lb/ft	mm × kg/m	in. × lb/ft	mm × kg/m	in. × lb/ft	mm × kg/m
W44×335	W1100×499	W36×925	W920×1377	W30×391	W760×582
×290	×433	×853	×1269	×357	×531
×262	×390	×802	×1194	×326	×484
×230	×343	×723	×1077	×292	×434
W40×655	W1000×976	×652	×970	×261	×389
×593	×883	×529	×787	×235	×350
×503	×748	×487	×725	×211	×314
×431	×642	×441	×656	×191	×284
×397	×591	×395	×588	×173	×257
×372	×554	×361	×537	W30×148	W760×220
×362	×539	×330	×491	×132	×196
×324	×483	×302	×449	×124	×185
×297	×443	×282	×420	×116	×173
×277	×412	×262	×390	×108	×161
×249	×371	×247	×368	×99	×147
×215	×321	×231	×344	×90	×134
×199	×296	W36×256	W920×381	W27×539	W690×802
W40×392	W1000×584	×232	×345	×368	×548
×331	×494	×210	×313	×336	×500
×327	×486	×194	×289	×307	×457
×294	×438	×182	×271	×281	×419
×278	×415	×170	×253	×258	×384
×264	×393	×160	×238	×235	×350
×235	×350	×150	×223	×217	×323
×211	×314	×135	×201	×194	×289
×183	×272	W33×387	W840×576	×178	×265
×167	×249	×354	×527	×161	×240
×149	×222	×318	×473	×146	×217
		×291	×433	W27×129	W690×192
		×263	×392	×114	×170
		×241	×359	×102	×152
		×221	×329	×94	×140
		×201	×299	×84	×125
		W33×169	W840×251		
		×152	×226		
		×141	×210		
		×130	×193		
		×118	×176		

Table 17-1 (continued)
SI Equivalents of Standard U.S. Shape Profiles
W-Shapes

Shape in. × lb/ft	SI Equivalent mm × kg/m
W24×370	W610×551
×335	×498
×306	×455
×279	×415
×250	×372
×229	×341
×207	×307
×192	×285
×176	×262
×162	×241
×146	×217
×131	×195
×117	×174
×104	×155
W24×103	W610×153
×94	×140
×84	×125
×76	×113
×68	×101
W24×62	W610×92
×55	×82
W21×275	W530×409
×248	×369
×223	×332
×201	×300
×182	×272
×166	×248
×147	×219
×132	×196
×122	×182
×111	×165
×101	×150
W21×93	W530×138
×83	×123
×73	×109
×68	×101
×62	×92
×55	×82
×48	×72

Shape in. × lb/ft	SI Equivalent mm × kg/m
W21×57	W530×85
×50	×74
×44	×66
W18×311	W460×464
×283	×421
×258	×384
×234	×349
×211	×315
×192	×286
×175	×260
×158	×235
×143	×213
×130	×193
×119	×177
×106	×158
×97	×144
×86	×128
×76	×113
W18×71	W460×106
×65	×97
×60	×89
×55	×82
×50	×74
W18×46	W460×68
×40	×60
×35	×52
W16×100	W410×149
×89	×132
×77	×114
×67	×100
W16×57	W410×85
×50	×75
×45	×67
×40	×60
×36	×53
W16×31	W410×46.1
×26	×38.8

Shape in. × lb/ft	SI Equivalent mm × kg/m
W14×873	W360×1299
×808	×1202
×730	×1086
×665	×990
×605	×900
×550	×818
×500	×744
×455	×677
×426	×634
×398	×592
×370	×551
×342	×509
×311	×463
×283	×421
×257	×382
×233	×347
×211	×314
×193	×287
×176	×262
×159	×237
×145	×216
W14×132	W360×196
×120	×179
×109	×162
×99	×147
×90	×134
W14×82	W360×122
×74	×110
×68	×101
×61	×91
W14×53	W360×79
×48	×72
×43	×64
W14×38	W360×58
×34	×51
×30	×44.6
W14×26	W360×39
×22	×32.9

Table 17-1 (continued)
SI Equivalents of Standard U.S. Shape Profiles
W-Shapes

Shape in. × lb/ft	SI Equivalent mm × kg/m
W12×336	W310×500
×305	×454
×279	×415
×252	×375
×230	×342
×210	×313
×190	×283
×170	×253
×152	×226
×136	×202
×120	×179
×106	×158
×96	×143
×87	×129
×79	×117
×72	×107
×65	×97
W12×58	W310×86
×53	×79
W12×50	W310×74
×45	×67
×40	×60
W12×35	W310×52
×30	×44.5
×26	×38.7

Shape in. × lb/ft	SI Equivalent mm × kg/m
W12×22	W310×32.7
×19	×28.3
×16	×23.8
×14	×21.0
W10×112	W250×167
×100	×149
×88	×131
×77	×115
×68	×101
×60	×89
×54	×80
×49	×73
W10×45	W250×67
×39	×58
×33	×49.1
W10×30	W250×44.8
×26	×38.5
×22	×32.7
W10×19	W250×28.4
×17	×25.3
×15	×22.3
×12	×17.9

Shape in. × lb/ft	SI Equivalent mm × kg/m
W8×67	W200×100
×58	×86
×48	×71
×40	×59
×35	×52
×31	×46.1
W8×28	W200×41.7
×24	×35.9
W8×21	W200×31.3
×18	×26.6
W8×15	W200×22.5
×13	×19.3
×10	×15.0
W6×25	W150×37.1
×20	×29.8
×15	×22.5
W6×16	W150×24.0
×12	×18.0
×9	×13.5
×8.5	×13.0
W5×19	W130×28.1
×16	×23.8
W4×13	W100×19.3

Table 17-2
SI Equivalents of Standard U.S. Shape Profiles
M-, S- and HP-Shapes

Shape	SI Equivalent	Shape	SI Equivalent	Shape	SI Equivalent
in. × lb/ft	mm × kg/m	in. × lb/ft	mm × kg/m	in. × lb/ft	mm × kg/m
M12.5×12.4	M318×18.5	S24×121	S610×180	HP18×204	HP460×304
×11.6	×17.3	×106	×158	×181	×269
M12×11.8	M310×17.6	S24×100	S610×149	×157	×234
×10.8	×16.1	×90	×134	×135	×202
M12×10	M310×14.9	×80	×119	HP16×183	HP410×272
M10×9	M250×13.4	S20×96	S510×143	×162	×242
×8	×11.9	×86	×128	×141	×211
M10×7.5	M250×11.2	S20×75	S510×112	×121	×181
M8×6.5	M200×9.7	×66	×98	×101	×151
×6.2	×9.2	S18×70	S460×104	×88	×131
M6×4.4	M150×6.6	×54.7	×81.4	HP14×117	HP360×174
×3.7	×5.5	S15×50	S380×74	×102	×152
M5×18.9	M130×28.1	×42.9	×64	×89	×132
M4×6	M100×8.9	S12×50	S310×74	×73	×108
×4.08	×6.1	×40.8	×60.7	HP12×89	HP310×132
×3.45	×5.1	S12×35	S310×52	×84	×125
×3.2	×4.8	×31.8	×47.3	×74	×110
M3×2.9	M75×4.3	S10×35	S250×52	×63	×93
		×25.4	×37.8	×53	×79
		S8×23	S200×34	HP10×57	HP250×85
		×18.4	×27.4	×42	×62
		S6×17.25	S150×25.7	HP8×36	HP200×53
		×12.5	×18.6		
		S5×10	S130×15		
		S4×9.5	S100×14.1		
		×7.7	×11.5		
		S3×7.5	S75×11.2		
		×5.7	×8.5		

Table 17-3
SI Equivalents of Standard U.S. Shape Profiles
Channels

Shape	SI Equivalent	Shape	SI Equivalent
in. × lb/ft	**mm × kg/m**	**in. × lb/ft**	**mm × kg/m**
C15×50	C380×74	MC18×58	MC460×86
×40	×60	×51.9	×77.2
×33.9	×50.4	×45.8	×68.2
C12×30	C310×45	×42.7	×63.5
×25	×37	MC13×50	MC330×74
×20.7	×30.8	×40	×60
C10×30	C250×45	×35	×52
×25	×37	×31.8	×47.3
×20	×30	MC12×50	MC310×74
×15.3	×22.8	×45	×67
C9×20	C230×30	×40	×60
×15	×22	×35	×52
×13.4	×19.9	×31	×46
C8×18.75	C200×27.9	MC12×14.3	MC310×21.3
×13.75	×20.5	MC12×10.6	MC310×15.8
×11.5	×17.1	MC10×41.1	MC250×61.2
C7×14.75	C180×22	×33.6	×50
×12.25	×18.2	×28.5	×42.4
×9.8	×14.6	MC10×25	MC250×37
C6×13	C150×19.3	×22	×33
×10.5	×15.6	MC10×8.4	MC250×12.5
×8.2	×12.2	×6.5	×9.7
C5×9	C130×13	MC9×25.4	MC230×37.8
×6.7	×10.4	×23.9	×35.6
C4×7.25	C100×10.8	MC8×22.8	MC200×33.9
×6.25	×9.3	×21.4	×31.8
×5.4	×8	MC8×20	MC200×29.8
×4.5	×6.7	×18.7	×27.8
C3×6	C75×8.9	MC8×8.5	MC200×12.6
×5	×7.4	MC7×22.7	MC180×33.8
×4.1	×6.1	×19.1	×28.4
×3.5	×5.2	MC6×18	MC150×26.8
		×15.3	×22.8
		MC6×16.3	MC150×24.3
		×15.1	×22.5
		MC6×12	MC150×17.9
		MC6×7	MC150×10.4
		×6.5	×9.7
		MC4×13.8	MC100×20.5
		MC3×7.1	MC75×10.6

Table 17-4
SI Equivalents of Standard U.S. Shape Profiles
Angles

Shape	SI Equivalent
in. × in. × in.	mm × mm × mm
L12×12×1$^{3}/_{8}$	L305×305×34.9
×1$^{1}/_{4}$	×31.8
×1$^{1}/_{8}$	×28.6
×1	×25.4
L10×10×1$^{3}/_{8}$	L254×254×34.9
×1$^{1}/_{4}$	×31.8
×1$^{1}/_{8}$	×28.6
×1	×25.4
×$^{7}/_{8}$	×22.2
×$^{3}/_{4}$	×19.0
L8×8×1$^{1}/_{8}$	L203×203×28.6
×1	×25.4
×$^{7}/_{8}$	×22.2
×$^{3}/_{4}$	×19.0
×$^{5}/_{8}$	×15.9
×$^{9}/_{16}$	×14.3
×$^{1}/_{2}$	×12.7
L8×6×1	L203×152×25.4
×$^{7}/_{8}$	×22.2
×$^{3}/_{4}$	×19.0
×$^{5}/_{8}$	×15.9
×$^{9}/_{16}$	×14.3
×$^{1}/_{2}$	×12.7
×$^{7}/_{16}$	×11.1
L8×4×1	L203×102×25.4
×$^{7}/_{8}$	×22.2
×$^{3}/_{4}$	×19.0
×$^{5}/_{8}$	×15.9
×$^{9}/_{16}$	×14.3
×$^{1}/_{2}$	×12.7
×$^{7}/_{16}$	×11.1
L7×4×$^{3}/_{4}$	L178×102×19.0
×$^{5}/_{8}$	×15.9
×$^{1}/_{2}$	×12.7
×$^{7}/_{16}$	×11.1
×$^{3}/_{8}$	×9.5
L6×6×1	L152×152×25.4
×$^{7}/_{8}$	×22.2
×$^{3}/_{4}$	×19.0
×$^{5}/_{8}$	×15.9
×$^{9}/_{16}$	×14.3
×$^{1}/_{2}$	×12.7
×$^{7}/_{16}$	×11.1
×$^{3}/_{8}$	×9.5
×$^{5}/_{16}$	×7.9
L6×4×$^{7}/_{8}$	L152×102×22.2
×$^{3}/_{4}$	×19.0
×$^{5}/_{8}$	×15.9
×$^{9}/_{16}$	×14.3
×$^{1}/_{2}$	×12.7
×$^{7}/_{16}$	×11.1
×$^{3}/_{8}$	×9.5
×$^{5}/_{16}$	×7.9
L6×3$^{1}/_{2}$×$^{1}/_{2}$	L152×89×12.7
×$^{3}/_{8}$	×9.5
×$^{5}/_{16}$	×7.9
L5×5×$^{7}/_{8}$	L127×127×22.2
×$^{3}/_{4}$	×19.0
×$^{5}/_{8}$	×15.9
×$^{1}/_{2}$	×12.7
×$^{7}/_{16}$	×11.1
×$^{3}/_{8}$	×9.5
×$^{5}/_{16}$	×7.9
L5×3$^{1}/_{2}$×$^{3}/_{4}$	L127×89×19.0
×$^{5}/_{8}$	×15.9
×$^{1}/_{2}$	×12.7
×$^{3}/_{8}$	×9.5
×$^{5}/_{16}$	×7.9
×$^{1}/_{4}$	×6.4
L5×3×$^{1}/_{2}$	L127×76×12.7
×$^{7}/_{16}$	×11.1
×$^{3}/_{8}$	×9.5
×$^{5}/_{16}$	×7.9
×$^{1}/_{4}$	×6.4

Table 17-4 (continued)
SI Equivalents of Standard U.S. Shape Profiles
Angles

Shape	SI Equivalent	Shape	SI Equivalent
in. × in. × in.	mm × mm × mm	in. × in. × in.	mm × mm × mm
L4×4×3/4	L102×102×19.0	L3×2 1/2×1/2	L76×64×12.7
×5/8	×15.9	×7/16	×11.1
×1/2	×12.7	×3/8	×9.5
×7/16	×11.1	×5/16	×7.9
×3/8	×9.5	×1/4	×6.4
×5/16	×7.9	×3/16	×4.8
×1/4	×6.4	L3×2×1/2	L76×51×12.7
L4×3 1/2×1/2	L102×89×12.7	×3/8	×9.5
×3/8	×9.5	×5/16	×7.9
×5/16	×7.9	×1/4	×6.4
×1/4	×6.4	×3/16	×4.8
L4×3×5/8	L102×76×15.9	L2 1/2×2 1/2×1/2	L64×64×12.7
×1/2	×12.7	×3/8	×9.5
×3/8	×9.5	×5/16	×7.9
×5/16	×7.9	×1/4	×6.4
×1/4	×6.4	×3/16	×4.8
L3 1/2×3 1/2×1/2	L89×89×12.7	L2 1/2×2×3/8	L64×51×9.5
×7/16	×11.1	×5/16	×7.9
×3/8	×9.5	×1/4	×6.4
×5/16	×7.9	×3/16	×4.8
×1/4	×6.4	L2 1/2×1 1/2×1/4	L64×38×6.4
L3 1/2×3×1/2	L89×76×12.7	×3/16	×4.8
×7/16	×11.1	L2×2×3/8	L51×51×9.5
×3/8	×9.5	×5/16	×7.9
×5/16	×7.9	×1/4	×6.4
×1/4	×6.4	×3/16	×4.8
L3 1/2×2 1/2×1/2	L89×64×12.7	×1/8	×3.2
×3/8	×9.5		
×5/16	×7.9		
×1/4	×6.4		
L3×3×1/2	L76×76×12.7		
×7/16	×11.1		
×3/8	×9.5		
×5/16	×7.9		
×1/4	×6.4		
×3/16	×4.8		

Table 17-5
SI Equivalents of Standard U.S. Shape Profiles
WT-Shapes

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
WT22×167.5	WT550×249.5
×145	×216.5
×131	×195
×115	×171.5
WT20×327.5	WT500×488
×296.5	×441.5
×251.5	×374
×215.5	×321
×198.5	×295.5
×186	×277
×181	×269.5
×162	×241.5
×148.5	×221.5
×138.5	×206
×124.5	×185.5
×107.5	×160.5
×99.5	×148
WT20×196	WT500×292
×165.5	×247
×163.5	×243
×147	×219
×139	×207.5
×132	×196.5
×117.5	×175
×105.5	×157
×91.5	×136
×83.5	×124.5
×74.5	×111

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
WT18×462.5	WT460×688.5
×426.5	×634.5
×401	×597
×361.5	×538.5
×326	×485
×264.5	×393.5
×243.5	×362.5
×220.5	×328
×197.5	×294
×180.5	×268.5
×165	×245.5
×151	×224.5
×141	×210
×131	×195
×123.5	×184
×115.5	×172.5
WT18×128	WT460×190.5
×116	×172.5
×105	×156.5
×97	×144.5
×91	×135.5
×85	×126.5
×80	×119
×75	×111.5
×67.5	×100.5
WT16.5×193.5	WT420×288
×177	×263.5
×159	×236.5
×145.5	×216.5
×131.5	×196
×120.5	×179.5
×110.5	×164.5
×100.5	×149.5
WT16.5×84.5	WT460×125.5
×76	×113
×70.5	×105
×65	×96.5
×59	×88

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
WT15×195.5	WT380×291
×178.5	×265.5
×163	×242
×146	×217
×130.5	×194.5
×117.5	×175
×105.5	×157
×95.5	×142
×86.5	×128.5
WT15×74	WT380×110
×66	×98
×62	×92.5
×58	×86.5
×54	×80.5
×49.5	×73.5
×45	×67
WT13.5×269.5	WT345×401
×184	×274
×168	×250
×153.5	×228.5
×140.5	×209.5
×129	×192
×117.5	×175
×108.5	×161.5
×97	×144.5
×89	×132.5
×80.5	×120
×73	×108.5
WT13.5×64.5	WT345×96
×57	×85
×51	×76
×47	×70
×42	×62.5

Table 17-5 (continued)
SI Equivalents of Standard U.S. Shape Profiles
WT-Shapes

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
WT12×185	WT305×275.5
×167.5	×249
×153	×227.5
×139.5	×207.5
×125	×186
×114.5	×170.5
×103.5	×153.5
×96	×142.5
×88	×131
×81	×120.5
×73	×108.5
×65.5	×97.5
×58.5	×87
×52	×77.5
WT12×51.5	WT305×76.5
×47	×70
×42	×62.5
×38	×56.5
×34	×50.5
WT12×31	WT12×46
×27.5	×41
WT10.5×137.5	WT265×204.5
×124	×184.5
×111.5	×166
×100.5	×150
×91	×136
×83	×124
×73.5	×109.5
×66	×98
×61	×91
×55.5	×82.5
×50.5	×75
WT10.5×46.5	WT265×69
×41.5	×61.5
×36.5	×54.5
×34	×50.5
×31	×46
×27.5	×41
×24	×36

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
WT10.5×28.5	WT265×42.5
×25	×37
×22	×33
WT9×155.5	WT230×232
×141.5	×210.5
×129	×192
×117	×174.5
×105.5	×157.5
×96	×143
×87.5	×130
×79	×117.5
×71.5	×106.5
×65	×96.5
×59.5	×88.5
×53	×79
×48.5	×72
×43	×64
×38	×56.5
WT9×35.5	WT230×53
×32.5	×48.5
×30	×44.5
×27.5	×41
×25	×37
WT9×23	WT230×34
×20	×30
×17.5	×26
WT8×50	WT205×74.5
×44.5	×66
×38.5	×57
×33.5	×50
WT8×28.5	WT205×42.5
×25	×37.5
×22.5	×33.5
×20	×30
×18	×26.5
WT8×15.5	WT205×23.05
×13	×19.4

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
WT7×436.5	WT180×649.5
×404	×601
×365	×543
×332.5	×495
×302.5	×450
×275	×409
×250	×372
×227.5	×338.5
×213	×317
×199	×296
×185	×275.5
×171	×254.5
×155.5	×231.5
×141.5	×210.5
×128.5	×191
×116.5	×173.5
×105.5	×157
×96.5	×143.5
×88	×131
×79.5	×118.5
×72.5	×108
WT7×66	WT180×98
×60	×89.5
×54.5	×81
×49.5	×73.5
×45	×67
WT7×41	WT180×61
×37	×55
×34	×50.5
×30.5	×45.5
WT7×26.5	WT180×39.5
×24	×36
×21.5	×32
WT7×19	WT180×29
×17	×25.5
×15	×22.3
WT7×13	WT180×19.5
×11	×16.45

Table 17-5 (continued)
SI Equivalents of Standard U.S. Shape Profiles
WT-Shapes

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
WT6×168	WT155×250
×152.5	×227
×139.5	×207.5
×126	×187.5
×115	×171
×105	×156.5
×95	×141.5
×85	×126.5
×76	×113
×68	×101
×60	×89.5
×53	×79
×48	×71.5
×43.5	×64.5
×39.5	×58.5
×36	×53.5
×32.5	×48.5
WT6×29	WT155×43
×26.5	×39.5
WT6×25	WT155×37
×22.5	×33.5
×20	×30
WT6×17.5	WT155×26
×15	×22.25
×13	×19.35

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
WT6×11	WT155×16.35
×9.5	×14.15
×8	×11.9
×7	×10.5
WT5×56	WT125×83.5
×50	×74.5
×44	×65.5
×38.5	×57.5
×34	×50.5
×30	×44.5
×27	×40
×24.5	×36.5
WT5×22.5	WT125×33.5
×19.5	×29
×16.5	×24.55
WT5×15	WT125×22.4
×13	×19.25
×11	×16.35
WT5×9.5	WT125×14.2
×8.5	×12.65
×7.5	×11.15
×6	×8.95

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
WT4×33.5	WT100×50
×29	×43
×24	×35.5
×20	×29.5
×17.5	×26
×15.5	×23.05
WT4×14	WT100×20.85
×12	×17.95
WT4×10.5	WT100×15.65
×9	×13.3
WT4×7.5	WT100×11.25
×6.5	×9.65
×5	×7.5
WT3×12.5	WT75×18.55
×10	×14.9
×7.5	×11.25
WT3×8	WT75×12
×6	×9
×4.5	×6.75
×4.25	×6.5
WT2.5×9.5	WT65×14.05
×8	×11.9
WT2×6.5	WT50×9.65

Table 17-6
SI Equivalents of Standard U.S. Shape Profiles
MT- and ST-Shapes

Shape	SI Equivalent	Shape	SI Equivalent
in. × lb/ft	mm × kg/m	in. × lb/ft	mm × kg/m
MT6.25×6.2	MT159×9.25	ST12×60.5	ST305×90
×5.8	×8.65	×53	×79
MT6×5.9	MT155×8.80	ST12×50	ST305×74.5
×5.4	×8.05	×45	×67
×5	×7.45	×40	×59.5
MT5×4.5	MT125×6.70	ST10×48	ST255×71.5
×4	×5.95	×43	×64
MT5×3.75	MT125×5.60	ST10×37.5	ST255×56
		×33	×49
MT4×3.25	MT100×4.85	ST9×35	ST230×52
×3.1	×4.6	×27.35	×40.7
MT3×2.2	MT75×3.3	ST7.5×25	ST190×37
×1.85	×2.75	×21.45	×32
MT2.5×9.45	MT65×14.05	ST6×25	ST155×37
MT2×3	MT50×4.45	×20.4	×30.35
		ST6×17.5	ST155×26
		×15.9	×23.65
		ST5×17.5	ST125×26
		×12.7	×18.9
		ST4×11.5	ST100×17
		×9.2	×13.7
		ST3×8.6	ST75×12.85
		×6.25	×9.3
		ST2.5×5	ST65×7.5
		ST2×4.75	ST50×7.05
		×3.85	×5.75
		ST1.5×3.75	ST37.5×5.6
		×2.85	×4.25

Table 17-7
SI Equivalents of Standard U.S. Shape Profiles
Rectangular HSS

Shape	SI Equivalent
in. × in. × in.	mm × mm × mm
HSS24×12×3/4	HSS609.6×304.8×19.0
×5/8	×15.9
×1/2	×12.7
HSS20×12×3/4	HSS508×304.8×19.0
×5/8	×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
HSS20×8×5/8	HSS508×203.2×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
HSS20×4×1/2	HSS508×101.6×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
HSS18×6×5/8	HSS457.2×152.4×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
HSS16×12×3/4	HSS406.4×304.8×19.0
×5/8	×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
HSS16×8×5/8	HSS406.4×203.2×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
HSS16×4×5/8	HSS406.4×101.6×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS14×10×5/8	HSS355.6×254×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
HSS14×6×5/8	HSS355.6×152.4×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS14×4×5/8	HSS355.6×101.6×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS12×10×1/2	HSS304.8×254×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
HSS12×8×5/8	HSS304.8×203.2×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS12×6×5/8	HSS304.8×152.4×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS12×4×5/8	HSS304.8×101.6×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8

Table 17-7 (continued)
SI Equivalents of Standard U.S. Shape Profiles
Rectangular HSS

Shape in. × in. × in.	SI Equivalent mm × mm × mm
HSS12×3 1/2×3/8	HSS304.8×88.9×9.5
×5/16	×7.9
HSS12×3×5/16	HSS304.8×76.2×7.9
×1/4	×6.4
×3/16	×4.8
HSS12×2×5/16	HSS304.8×50.8×7.9
×1/4	×6.4
×3/16	×4.8
HSS10×8×5/8	HSS254×203.2×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS10×6×5/8	HSS254×152.4×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS10×5×3/8	HSS254×127×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS10×4×5/8	HSS254×101.6×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS10×3 1/2×1/2	HSS254×88.9×4.8
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2

Shape in. × in. × in.	SI Equivalent mm × mm × mm
HSS10×3×3/8	HSS254×76.2×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS10×2×3/8	HSS254×50.8×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS9×7×5/8	HSS228.6×177.8×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS9×5×5/8	HSS228.6×127×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS9×3×1/2	HSS228.6×76.2×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS8×6×5/8	HSS203.2×152.4×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS8×4×5/8	HSS203.2×101.6×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2

Table 17-7 (continued)
SI Equivalents of Standard U.S. Shape Profiles
Rectangular HSS

Shape	SI Equivalent
in. × in. × in.	mm × mm × mm
HSS8×3×1/2	HSS203.2×76.2×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS8×2×3/8	HSS203.2×50.8×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS7×5×1/2	HSS177.8×127×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS7×4×1/2	HSS177.8×101.6×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS7×3×1/2	HSS177.8×76.2×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS7×2×1/4	HSS177.8×50.8×6.4
×3/16	×4.8
×1/8	×3.2
HSS6×5×1/2	HSS152.4×127×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS6×4×1/2	HSS152.4×101.6×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS6×3×1/2	HSS152.4×76.2×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS6×2×3/8	HSS152.4×50.8×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS5×4×1/2	HSS127×101.6×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS5×3×1/2	HSS127×76.2×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS5×2 1/2×1/4	HSS127×63.5×6.4
×3/16	×4.8
×1/8	×3.2
HSS5×2×3/8	HSS127×50.8×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2

Table 17-7 (continued)
SI Equivalents of Standard U.S. Shape Profiles
Rectangular HSS

Shape	SI Equivalent
in. × in. × in.	mm × mm × mm
HSS4×3×$^{3}/_{8}$	HSS101.6×76.2×9.5
×$^{5}/_{16}$	×7.9
×$^{1}/_{4}$	×6.4
×$^{3}/_{16}$	×4.8
×$^{1}/_{8}$	×3.2
HSS4×2$^{1}/_{2}$×$^{3}/_{8}$	HSS101.6×63.5×9.5
×$^{5}/_{16}$	×7.9
×$^{1}/_{4}$	×6.4
×$^{3}/_{16}$	×4.8
×$^{1}/_{8}$	×3.2
HSS4×2×$^{3}/_{8}$	HSS101.6×50.8×9.5
×$^{5}/_{16}$	×7.9
×$^{1}/_{4}$	×6.4
×$^{3}/_{16}$	×4.8
×$^{1}/_{8}$	×3.2
HSS3$^{1}/_{2}$×2$^{1}/_{2}$×$^{3}/_{8}$	HSS88.9×63.5×9.5
×$^{5}/_{16}$	×7.9
×$^{1}/_{4}$	×6.4
×$^{3}/_{16}$	×4.8
×$^{1}/_{8}$	×3.2
HSS3$^{1}/_{2}$×2×$^{1}/_{4}$	HSS88.9×50.8×6.4
×$^{3}/_{16}$	×4.8
×$^{1}/_{8}$	×3.2
HSS3$^{1}/_{2}$×1$^{1}/_{2}$×$^{1}/_{4}$	HSS88.9×38.1×6.4
×$^{3}/_{16}$	×4.8
×$^{1}/_{8}$	×3.2
HSS3×2$^{1}/_{2}$×$^{5}/_{16}$	HSS76.2×63.5×7.9
×$^{1}/_{4}$	×6.4
×$^{3}/_{16}$	×4.8
×$^{1}/_{8}$	×3.2
HSS3×2×$^{5}/_{16}$	HSS76.2×50.8×7.9
×$^{1}/_{4}$	×6.4
×$^{3}/_{16}$	×4.8
×$^{1}/_{8}$	×3.2
HSS3×1$^{1}/_{2}$×$^{1}/_{4}$	HSS76.2×38.1×6.4
×$^{3}/_{16}$	×4.8
×$^{1}/_{8}$	×3.2
HSS3×1×$^{3}/_{16}$	HSS76.2×25.4×4.8
×$^{1}/_{8}$	×3.2
HSS2$^{1}/_{2}$×2×$^{1}/_{4}$	HSS63.5×50.8×6.4
×$^{3}/_{16}$	×4.8
×$^{1}/_{8}$	×3.2
HSS2$^{1}/_{2}$×1$^{1}/_{2}$×$^{1}/_{4}$	HSS63.5×38.1×6.4
×$^{3}/_{16}$	×4.8
×$^{1}/_{8}$	×3.2
HSS2$^{1}/_{2}$×1×$^{3}/_{16}$	HSS63.5×25.4×4.8
×$^{1}/_{8}$	×3.2
HSS2$^{1}/_{4}$×2×$^{3}/_{16}$	HSS57.2×50.8×4.8
×$^{1}/_{8}$	×3.2
HSS2×1$^{1}/_{2}$×$^{3}/_{16}$	HSS50.8×38.1×4.8
×$^{1}/_{8}$	×3.2
HSS2×1×$^{3}/_{16}$	HSS50.8×25.4×4.8
×$^{1}/_{8}$	×3.2

Table 17-8
SI Equivalents of Standard U.S. Shape Profiles
Square HSS

Shape	SI Equivalent
in. × in. × in.	mm × mm × mm
HSS22×22×7/8	HSS558.8×558.8×22.2
×3/4	×19.0
HSS20×20×7/8	HSS508×508×22.2
×3/4	×19.0
×5/8	×15.9
×1/2	×12.7
HSS18×18×7/8	HSS457.2×457.2×22.2
×3/4	×19.0
×5/8	×15.9
×1/2	×12.7
HSS16×16×7/8	HSS406.4×406.4×22.2
×3/4	×19.0
×5/8	×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
HSS14×14×7/8	HSS355.6×355.6×22.2
×3/4	×19.0
×5/8	×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
HSS12×12×3/4	HSS304.8×304.8×19.0
×5/8	×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS10×10×3/4	HSS254×254×19.0
×5/8	×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS9×9×5/8	HSS228.6×228.6×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS8×8×5/8	HSS203.2×203.2×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS7×7×5/8	HSS177.8×177.8×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS6×6×5/8	HSS152.4×152.4×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS5 1/2×5 1/2×3/8	HSS139.7×139.7×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS5×5×1/2	HSS127×127×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2

Table 17-8 (continued)
SI Equivalents of Standard U.S. Shape Profiles
Square HSS

Shape	SI Equivalent	Shape	SI Equivalent
in. × in. × in.	mm × mm × mm	in. × in. × in.	mm × mm × mm
HSS4 1/2×4 1/2×1/2	HSS114.3×114.3×12.7	HSS3×3×3/8	HSS76.2×76.2×9.5
×3/8	×9.5	×5/16	×7.9
×5/16	×7.9	×1/4	×6.4
×1/4	×6.4	×3/16	×4.8
×3/16	×4.8	×1/8	×3.2
×1/8	×3.2	HSS2 1/2×2 1/2×5/16	HSS63.5×63.5×7.9
HSS4×4×1/2	HSS101.6×101.6×12.7	×1/4	×6.4
×3/8	×9.5	×3/16	×4.8
×5/16	×7.9	×1/8	×3.2
×1/4	×6.4	HSS2 1/4×2 1/4×1/4	HSS57.2×57.2×6.4
×3/16	×4.8	×3/16	×4.8
×1/8	×3.2	×1/8	×3.2
HSS3 1/2×3 1/2×3/8	HSS88.9×88.9×9.5	HSS2×2×1/4	HSS50.8×50.8×6.4
×5/16	×7.9	×3/16	×4.8
×1/4	×6.4	×1/8	×3.2
×3/16	×4.8		
×1/8	×3.2		

Table 17-9
SI Equivalents of Standard U.S. Shape Profiles
Round HSS and Pipes

Shape	SI Equivalent	Shape	SI Equivalent
in. × in. × in.	mm × mm × mm	in. × in. × in.	mm × mm × mm
HSS20.000×0.500	HSS508×12.7	HSS7.000×0.500	HSS177.8×12.7
×0.375	×9.5	×0.375	×9.5
HSS18.000×0.500	HSS457.2×12.7	×0.312	×7.9
×0.375	×9.5	×0.250	×6.4
HSS16.000×0.625	HSS406.4×15.9	×0.188	×4.8
×0.500	×12.7	×0.125	×3.2
×0.438	×11.1	HSS6.875×0.500	HSS174.6×12.7
×0.375	×9.5	×0.375	×9.5
×0.312	×7.9	×0.312	×7.9
×0.250	×6.4	×0.250	×6.4
HSS14.000×0.625	HSS355.6×15.9	×0.188	×4.8
×0.500	×12.7	HSS6.625×0.500	HSS168.3×12.7
×0.375	×9.5	×0.432	×11.0
×0.312	×7.9	×0.375	×9.5
×0.250	×6.4	×0.312	×7.9
HSS12.750×0.500	HSS323.9×12.7	×0.280	×7.1
×0.375	×9.5	×0.250	×6.4
×0.250	×6.4	×0.188	×4.8
HSS10.750×0.500	HSS273.1×12.7	×0.125	×3.2
×0.375	×9.5	HSS6.000×0.500	HSS152.4×12.7
×0.250	×6.4	×0.375	×9.5
HSS10.000×0.625	HSS254×15.9	×0.312	×7.9
×0.500	×12.7	×0.280	×7.1
×0.375	×9.5	×0.250	×6.4
×0.312	×7.9	×0.188	×4.8
×0.250	×6.4	×0.125	×3.2
×0.188	×4.8	HSS5.563×0.500	HSS141.3×12.7
HSS9.625×0.500	HSS244.5×12.7	×0.375	×9.5
×0.375	×9.5	×0.258	×6.6
×0.312	×7.9	×0.188	×4.8
×0.250	×6.4	×0.134	×3.4
×0.188	×4.8	HSS5.500×0.500	HSS139.7×12.7
HSS8.625×0.625	HSS219.1×15.9	×0.375	×9.5
×0.500	×12.7	×0.258	×6.6
×0.375	×9.5	HSS5.000×0.500	HSS127×12.7
×0.322	×8.2	×0.375	×9.5
×0.250	×6.4	×0.312	×7.9
×0.188	×4.8	×0.258	×6.6
HSS7.625×0.375	HSS193.7×9.5	×0.250	×6.4
×0.328	×8.3	×0.188	×4.8
HSS7.500×0.500	HSS190.5×12.7	×0.125	×3.2
×0.375	×9.5	HSS4.500×0.375	HSS114.3×9.5
×0.312	×7.9	×0.337	×8.6
×0.250	×6.4	×0.237	×6.0
×0.188	×4.8	×0.188	×4.8
		×0.125	×3.2

Table 17-9 (continued)
SI Equivalents of Standard U.S. Shape Profiles
Round HSS and Pipes

Shape	SI Equivalent	Shape	SI Equivalent
in. × in. × in.	mm × mm × mm	in. × in. × in.	mm × mm × mm
HSS4.000×0.313	HSS101.6×8.0	HSS2.875×0.250	HSS73×6.4
×0.250	×6.4	×0.203	×5.2
×0.237	×6.0	×0.188	×4.8
×0.226	×5.7	×0.125	×3.2
×0.220	×5.6	HSS2.500×0.250	HSS63.5×6.4
×0.188	×4.8	×0.188	×4.8
×0.125	×3.2	×0.125	×3.2
HSS3.500×0.313	HSS88.9×8.0	HSS2.375×0.250	HSS60.3×6.4
×0.300	×7.6	×0.218	×5.5
×0.250	×6.4	×0.188	×4.8
×0.216	×5.5	×0.154	×3.9
×0.203	×5.2	×0.125	×3.2
×0.188	×4.8	HSS1.900×0.188	HSS48.3×4.8
×0.125	×3.2	×0.145	×3.7
HSS3.000×0.250	HSS76.2×6.4	×0.120	×3.0
×0.216	×5.5	HSS1.660×0.140	HSS42.2×3.6
×0.203	×5.2		
×0.188	×4.8		
×0.152	×3.9		
×0.134	×3.4		
×0.125	×3.2		

Table 17-9 (continued)
SI Equivalents of Standard U.S. Shape Profiles
Round HSS and Pipes

Shape	SI Equivalent	Shape	SI Equivalent
PIPE 26 Std.	PIPE 650 Std.	PIPE 12 xx-Strong	PIPE 300 xx-Strong
PIPE 24 Std.	PIPE 600 Std.	PIPE 10 xx-Strong	PIPE 250 xx-Strong
PIPE 20 Std.	PIPE 500 Std.	PIPE 8 xx-Strong	PIPE 200 xx-Strong
PIPE 18 Std.	PIPE 450 Std.	PIPE 6 xx-Strong	PIPE 150 xx-Strong
PIPE 16 Std.	PIPE 400 Std.	PIPE 5 xx-Strong	PIPE 125 xx-Strong
PIPE 14 Std.	PIPE 350 Std.	PIPE 4 xx-Strong	PIPE 100 xx-Strong
PIPE 12 Std.	PIPE 300 Std.	PIPE 3 xx-Strong	PIPE 80 xx-Strong
PIPE 10 Std.	PIPE 250 Std.	PIPE 2$^1/_2$ xx-Strong	PIPE 65 xx-Strong
PIPE 8 Std.	PIPE 200 Std.	PIPE 2 xx-Strong	PIPE 50 xx-Strong
PIPE 6 Std.	PIPE 150 Std.		
PIPE 5 Std.	PIPE 125 Std.		
PIPE 4 Std.	PIPE 100 Std.		
PIPE 3$^1/_2$ Std.	PIPE 90 Std.		
PIPE 3 Std.	PIPE 80 Std.		
PIPE 2$^1/_2$ Std.	PIPE 65 Std.		
PIPE 2 Std.	PIPE 50 Std.		
PIPE 1$^1/_2$ Std.	PIPE 40 Std.		
PIPE 1$^1/_4$ Std.	PIPE 32 Std.		
PIPE 1 Std.	PIPE 25 Std.		
PIPE $^3/_4$ Std.	PIPE 20 Std.		
PIPE $^1/_2$ Std.	PIPE 15 Std.		
PIPE 26 x-Strong	PIPE 650 x-Strong		
PIPE 24 x-Strong	PIPE 600 x-Strong		
PIPE 20 x-Strong	PIPE 500 x-Strong		
PIPE 18 x-Strong	PIPE 450 x-Strong		
PIPE 16 x-Strong	PIPE 400 x-Strong		
PIPE 14 x-Strong	PIPE 350 x-Strong		
PIPE 12 x-Strong	PIPE 300 x-Strong		
PIPE 10 x-Strong	PIPE 250 x-Strong		
PIPE 8 x-Strong	PIPE 200 x-Strong		
PIPE 6 x-Strong	PIPE 150 x-Strong		
PIPE 5 x-Strong	PIPE 125 x-Strong		
PIPE 4 x-Strong	PIPE 100 x-Strong		
PIPE 3$^1/_2$ x-Strong	PIPE 90 x-Strong		
PIPE 3 x-Strong	PIPE 80 x-Strong		
PIPE 2$^1/_2$ x-Strong	PIPE 65 x-Strong		
PIPE 2 x-Strong	PIPE 50 x-Strong		
PIPE 1$^1/_2$ x-Strong	PIPE 40 x-Strong		
PIPE 1$^1/_4$ x-Strong	PIPE 32 x-Strong		
PIPE 1 x-Strong	PIPE 25 x-Strong		
PIPE $^3/_4$ x-Strong	PIPE 20 x-Strong		
PIPE $^1/_2$ x-Strong	PIPE 15 x-Strong		

Table 17-10
Wire and Sheet Metal Gages
Equivalent thickness, in.

Gage No.	U.S. Standard Gage for Uncoated Hot- & Cold-Rolled Sheets[b]	Galvanized Sheet Gage for Hot-Dipped Zinc Coated Sheets[b]	USA Steel Wire Gage	Gage No.	U.S. Standard Gage for Uncoated Hot- & Cold-Rolled Sheets[b]	Galvanized Sheet Gage for Hot-Dipped Zinc Coated Sheets[b]	USA Steel Wire Gage
7/0	–	–	0.490	13	0.0897	0.0934	0.092[a]
6/0	–	–	0.462[a]	14	0.0747	0.0785	0.080
5/0	–	–	0.430[a]	15	0.0673	0.0710	0.072
4/0	–	–	0.394[a]	16	0.0598	0.0635	0.062[a]
3/0	–	–	0.362[a]	17	0.0538	0.0575	0.054
2/0	–	–	0.331	18	0.0478	0.0516	0.048[a]
1/0	–	–	0.306	19	0.0418	0.0456	0.041
1	–	–	0.283	20	0.0359	0.0396	0.035[a]
2	–	–	0.262[a]	21	0.0329	0.0366	–
3	0.2391	–	0.244[a]	22	0.0299	0.0336	–
4	0.2242	–	0.225[a]	23	0.0269	0.0306	–
5	0.2092	–	0.207	24	0.0239	0.0276	–
6	0.1943	–	0.192	25	0.0209	0.0247	–
7	0.1793	–	0.177	26	0.0179	0.0217	–
8	0.1644	0.1681	0.162	27	0.0164	0.0202	–
9	0.1495	0.1532	0.148[a]	28	0.0149	0.0187	–
10	0.1345	0.1382	0.135	29	–	0.0172	–
11	0.1196	0.1233	0.120[a]	30	–	0.0157	–
12	0.1046	0.1084	0.106[a]				

[a] Rounded value. The steel wire gage has been taken from ASTM A510, "Standard Specification for General Requirements for Wire Rods and Coarse Round Wire, Carbon Steel, and Alloy Steel." Sizes originally quoted to four decimal equivalent places have been rounded to three decimal places in accordance with rounding procedures of ASTM E29, "Standard Practice for Using Significant Digits in Test Data to Determine Conformance with Specifications."

[b] The equivalent thicknesses are for information only. The product is commonly specified to decimal thickness (mils), not to gage number.

Table 17-11
Coefficient of Expansion

The coefficient of linear expansion, ε, is the change in length, per unit of length, for a change of one degree of temperature. The coefficient of surface expansion is approximately two times the linear coefficient, and the coefficient of volume expansion, for solids, is approximately three times the linear coefficient.

A bar, free to move, will increase in length with an increase in temperature and will decrease in length with a decrease in temperature. The change in length will be $\varepsilon t l$, where ε is the coefficient of linear expansion, t, the change in temperature, and l, the length. If the ends of a bar are fixed, a change in temperature, t, will cause a change in the unit stress of $E\varepsilon t$, and in the force of $AE\varepsilon t$, where A is the cross-sectional area of the bar and E the modulus of elasticity.

The following table gives the coefficient of linear expansion for 100°, or 100 times the value indicated above.

Example: A piece of mild steel is 40 ft long at 60°F. Find the length at 90°F assuming the ends are free to move.

$$\text{change of length} = \varepsilon t l = \frac{0.00065 \times 30 \times 40}{100} = 0.0078 \text{ ft}$$

The length at 90°F is 40.0078 ft.

Example: A piece of mild steel is 40 ft long and the ends are fixed. If the temperature increases 30°F, what is the resulting change in the unit stress?

$$\text{change in unit stress} = E\varepsilon t = \frac{29{,}000 \times 0.00065 \times 30}{100} = 5.7 \text{ ksi}$$

Coefficients of Expansion for 100 Degrees $= 100\varepsilon$

Materials	Linear Expansion Celsius	Linear Expansion Fahrenheit	Materials	Linear Expansion Celsius	Linear Expansion Fahrenheit
METALS AND ALLOYS			STONE AND MASONRY		
Aluminum, wrought	0.00231	0.00128	Ashlar masonry	0.00063	0.00035
Brass	0.00188	0.00104	Brick masonry	0.00061	0.00034
Bronze	0.00181	0.00101	Cement, portland	0.00126	0.00070
Copper	0.00168	0.00093	Concrete	0.00099	0.00055
Iron, cast, gray	0.00106	0.00059	Granite	0.00080	0.00044
Iron, wrought	0.00120	0.00067	Limestone	0.00076	0.00042
Iron, wire	0.00124	0.00069	Marble	0.00081	0.00045
Lead	0.00286	0.00159	Plaster	0.00166	0.00092
Magnesium, various alloys	0.0029	0.0016	Rubble masonry	0.00063	0.00035
Nickel	0.00126	0.00070	Sandstone	0.00097	0.00054
Steel, mild	0.00117	0.00065	Slate	0.00080	0.00044
Steel, stainless, 18-8	0.00178	0.00099			
Zinc, rolled	0.00311	0.00173			
TIMBER			TIMBER		
Fir (parallel to fiber)	0.00037	0.00021	Fir (perpendicular to fiber)	0.0058	0.0032
Maple (parallel to fiber)	0.00064	0.00036	Maple (perpendicular to fiber)	0.0048	0.0027
Oak (parallel to fiber)	0.00049	0.00027	Oak (perpendicular to fiber)	0.0054	0.0030
Pine (parallel to fiber)	0.00054	0.00030	Pine (perpendicular to fiber)	0.0034	0.0019

Expansion of Water

Maximum Density = 1

°C	Volume	°C	Volume	°C	Volume	°C	Volume	°C	Volume	°C	Volume
0	1.000126	10	1.000257	30	1.004234	50	1.011877	70	1.022384	90	1.035829
4	1.000000	20	1.001732	40	1.007627	60	1.016954	80	1.029003	100	1.043116

Table 17-12
Densities of Common Materials

Substance	Density, lb/ft^3	Substance	Density, lb/ft^3
ASHLAR, MASONRY		River mud	90.0
Granite, syenite, gneiss	143–187	Soil	70.0
Limestone, marble	143–174	Stone riprap	65.0
Sandstone, bluestone	131–150		
		MINERALS	
MORTAR RUBBLE MASONRY		Asbestos	131–174
Granite, syenite, gneiss	137–174	Barytes	280
Limestone, marble	137–162	Basalt	168–199
Sandstone, bluestone	125–137	Bauxite	159
		Borax	106–112
DRY RUBBLE MASONRY		Chalk	112–162
Granite, syenite, gneiss	118–143	Clay, marl	112–162
Limestone, marble	118–131	Dolomite	181
Sandstone, bluestone	112–118	Feldspar, orthoclase	156–162
		Gneiss, serpentine	150–168
BRICK MASONRY		Granite, syenite	156–193
Pressed brick	137–143	Greenstone, trap	174–199
Common brick	112–125	Gypsum, alabaster	143–174
Soft brick	93.5–106	Hornblende	187
		Limestone, marble	156–174
CONCRETE MASONRY		Magnesite	187
Cement, stone, sand	137–150	Phosphate rock, apatite	199
Cement, slag, etc.	118–143	Porphyry	162–181
Cement, cinder, etc.	93.5–106	Pumice, natural	23.1–56.1
		Quartz, flint	156–174
VARIOUS BUILDING MATERIALS		Sandstone, bluestone	137–156
Ashes, cinders	40.0–45.0	Shale, slate	168–181
Cement, portland, loose	90.0	Soapstone, talc	162–174
Cement, portland, set	168–199		
Lime, gypsum, loose	53.0–64.0	**STONE, QUARRIED, PILED**	
Mortar, set	87.2–118	Basalt, granite, gneiss	96.0
Slags, bank slag	67.0–72.0	Limestone, marble, quartz	95.0
Slags, bank screenings	98.0–117	Sandstone	82.0
Slags, machine slag	96.0	Shale	92.0
Slag, slag sand	49.0–55.0	Greenstone, hornblende	107
EARTH, ETC., EXCAVATED		**BITUMINOUS SUBSTANCES**	
Clay, dry	63.0	Asphaltum	68.5–93.5
Clay, damp, plastic	110	Coal, anthracite	87.2–106
Clay and gravel, dry	100	Coal, bituminous	74.8–93.5
Earth, dry, loose	76.0	Coal, lignite	68.5–87.2
Earth, dry, packed	95.0	Coal, peat, turf, dry	40.5–53.0
Earth, moist, loose	78.0	Coal, charcoal, pine	17.4–27.4
Earth, moist, packed	96.0	Coal, charcoal, oak	29.3–35.5
Earth, mud, flowing	108	Coal, coke	62.3–87.2
Earth, mud, packed	115	Graphite	118–143
Riprap, limestone	80.0–85.0	Paraffin	54.2–56.7
Riprap, sandstone	90.0	Petroleum	54.2
Riprap, shale	105	Petroleum, refined	49.2–51.1
Sand, gravel, dry, loose	90.0–105	Petroleum, benzine	45.5–46.7
Sand, gravel, dry, packed	100–120	Petroleum, gasoline	41.1–43.0
Sand, gravel, wet	118–120	Pitch	66.7–71.6
		Tar, bituminous	74.8
EXCAVATIONS IN WATER			
Sand or gravel	60.0	**COAL AND COKE, PILED**	
Sand or gravel and clay	65.0	Coal, anthracite	47.0–58.0
Clay	80.0	Coal, bituminous, lignite	40.0–54.0

Table 17-12 (continued)
Densities of Common Materials

Substance	Density, lb/ft^3	Substance	Density, lb/ft^3
Coal, peat, turf	20.0–26.0	Starch	95.3
Coal, charcoal	10.0–14.0	Sulphur	120–129
Coal, coke	23.0–32.0	Wool	82.2
METALS, ALLOYS, ORES		**TIMBER, U.S. SEASONED**	
Aluminum, cast, hammered	159–171	Moisture content by weight:	
Brass, cast, rolled	523–542	Seasoned timber 15 to 20%	
Bronze, 7.9 to 14% Sn	461–554	Green timber up to 50%	
Bronze, aluminum	480	Ash, white, red	38.6–40.5
Copper, cast, rolled	548–561	Cedar, white, red	19.9–23.7
Copper ore, pyrites	255–268	Chestnut	41.1
Gold, cast, hammered	1200–1210	Cypress	29.9
Iron, cast, pig	449	Fir, Douglas spruce	31.8
Iron, wrought	473–492	Fir, eastern	24.9
Iron, speigel–eisen	467	Elm, white	44.9
Iron, ferro–silicon	417–455	Hemlock	26.2–32.4
Iron ore, hematite	324	Hickory	46.1–52.3
Iron ore, hematite in bank	160–180	Locust	45.5
Iron ore, hematite loose	130–160	Maple, hard	42.4
Iron ore, limonite	224–249	Maple, white	33.0
Iron ore, magnetite	305–324	Oak, chestnut	53.6
Iron slag	156–187	Oak, live	59.2
Lead	710	Oak, red, black	40.5
Lead ore, galena	455–473	Oak, white	46.1
Magnesium, alloys	108–114	Pine, Oregon	31.8
Manganese	449–498	Pine, red	29.9
Manganese, ore, pyrolusite	231–287	Pine, white	25.5
Mercury	847	Pine, yellow, long–leaf	43.6
Monel Metal	548–561	Pine, yellow, short–leaf	38.0
Nickel	554–573	Poplar	29.9
Platinum, cast, hammered	1310–1340	Redwood, California	26.2
Silver, cast, hammered	648–668	Spruce, white, black	24.9–28.7
Steel, rolled	490	Walnut, black	38.0
Tin, cast, hammered	449–467	Walnut, white	25.5
Tin ore, cassiterite	399–436		
Zinc, cast, rolled	430–449	**VARIOUS LIQUIDS**	
Zinc, ore, blende	243–262	Alcohol, 100%	49.2
		Acids, muriatic 40%	74.8
VARIOUS SOLIDS		Acids, nitric 91%	93.5
Cereals, oats, bulk	32.0	Acids, sulphuric 87%	112
Cereals, barley, bulk	39.0	Lye, soda 66%	106
Cereals, corn, rye, bulk	48.0	Oils, vegetable	56.7–58.6
Cereals, wheat, bulk	48.0	Oils, mineral, lubricants	56.1–57.9
Hay and straw, bales	20.0	Water, 4ºC max. density	62.3
Cotton, flax, hemp	91.6–93.5	Water, 100ºC	59.7
Fats	56.1–60.4	Water, ice	54.8–57.3
Flour, loose	24.9–31.2	Water, sea water	63.5–64.2
Flour, pressed	43.6–49.8		
Glass, common	150–162	**GASES**	
Glass, plate or crown	153–169	Air, 0ºC 760 mm	0.0871
Glass, crystal	181–187	Ammonia	0.0478
Leather	53.6–63.5	Carbon dioxide	0.123
Paper	43.6–71.6	Carbon monoxide	0.078
Potatoes, piled	42.0	Gas, illuminating	0.028–0.036
Rubber, caoutchouc	57.3–59.8	Gas, natural	0.038–0.039
Rubber goods	62.3–125	Hydrogen	0.00559
Salt, granulated, piled	48.0	Nitrogen	0.0784
Saltpeter	67.0	Oxygen	0.0892

Table 17-13
Weights of Building Materials

Material	Weight, lb/ft^2	Material	Weight, lb/ft^2
CEILINGS		**PARTITIONS**	
Channel suspended system	1	Wood studs, 2 × 4	
Lathing and plastering	See Partitions	12-16 in. o. c.	2
Acoustical fiber tile	1	Steel studs	
		12-16 in. o. c.	1
FLOORS		Drywall, 1/2 in.	2
Steel deck	See Manufacturer	Drywall, 5/8 in.	2 1/2
Concrete-reinforced, 1 in.		Plaster, 1 in.	
Stone	12 1/2	Cement	10
Structural lightweight	9 1/2	Gypsum	5
Concrete-plain, 1 in.		Lathing	
Stone	12	Metal	1/2
Structural lightweight	9	Gypsum board, 1/2 in.	2
Nonstructural lightweight	3 to 9		
Finishes		**WALLS**	
Terrazzo, 1 in.	13	Brick	
Ceramic or quarry tile, 3/4 in.	10	4 in.	40
Linoleum, 1/4 in.	1	8 in.	80
Mastic, 3/4 in.	9	12 in.	120
Hardwood, 7/8 in.	4	Hollow concrete block	
Softwood, 3/4 in.	2 1/2	(135 pcf-no grout/full grout)	
		4 in.	29/-
ROOFS		6 in.	30/62
Copper	1	8 in.	39/83
Corrugated steel	See Manufacturer	10 in.	47/105
3-ply ready roofing	1	12 in.	54/127
3-ply felt and gravel	5 1/2	Hollow concrete block	
5-ply felt and gravel	6	(125 pcf-no grout/full grout)	
Shingles		4 in.	26/-
Wood	2	6 in.	28/59
Asphalt	3	8 in.	36/81
Clay tile	9 to 14	10 in.	44/102
Slate, 1/4 in.	10	12 in.	50/123
Sheathing		Hollow concrete block	
Wood, 3/4 in.	3	(105 pcf-no grout/full grout)	
Gypsum, 1 in.	4	4 in.	22/-
Insulation, 1 in.		6 in.	24/55
Loose	1/2	8 in.	31/75
Poured	2	10 in.	37/95
Rigid	1 1/2	12 in.	43/115
		Stone, 4 in.	55
		Glass block, 4 in.	18
		Curtain walls	See Manufacturer
		Structural glass, 1 in.	15

For weights of other materials used in building construction, see Table 17-12.
See ASCE/SEI 7, *Minimum Design Loads for Buildings and Other Structures* for additional design dead loads.

Table 17-14
Weights and Measures
United States System

Linear Measure

Inches	Feet	Yards	Rods	Furlongs	Miles
1.0	.08333	.02778	.0050505	.00012626	.00001578
12.0	1.0	.33333	.0606061	.00151515	.00018939
36.0	3.0	1.0	.1818182	.00454545	.00056818
198.0	16.5	5.5	1.0	.025	.003125
7,920.0	660.0	220.0	40.0	1.0	.125
63,360.0	5,280.0	1,760.0	320.0	8.0	1.0

Square and Land Measure

Sq. Inches	Square Feet	Square Yards	Square Rods	Acres	Sq. Miles
1.0	.006944	.000772			
144.0	1.0	.111111			
1,296.0	9.0	1.0	.03306	.000207	
39,204.0	272.25	30.25	1.0	.00625	.0000098
	43,560.0	4,840.0	160.0	1.0	.0015625
		3,097,600.0	102,400.0	640.0	1.0

Avoirdupois Weights

Grains	Drams	Ounces	Pounds	Tons
1.0	.03657	.002286	.000143	.0000000714
27.34375	1.0	.0625	.003906	.00000195
437.5	16.0	1.0	.0625	.00003125
7,000.0	256.0	16.0	1.0	.0005
14,000,000.0	512,000.0	32,000.0	2,000.0	1.0

Dry Measure

Pints	Quarts	Pecks	Cubic Feet	Bushels
1.0	.5	.0625	.01945	.01563
2.0	1.0	.125	.03891	.03125
16.0	8.0	1.0	.31112	.25
51.42627	25.71314	3.21414	1.0	.80354
64.0	32.0	4.0	1.2445	1.0

Liquid Measure

Gills	Pints	Quarts	U.S. Gallons	Cubic Feet
1.0	.25	.125	.03125	.00418
4.0	1.0	.5	.125	.01671
8.0	2.0	1.0	.250	.03342
32.0	8.0	4.0	1.0	.1337
			7.48052	1.0

SI UNITS FOR STRUCTURAL STEEL DESIGN

Although there are seven metric base units in the SI system, only four are currently used by AISC in structural steel design. These base units are listed in Table 17-15.

Table 17-15. Base SI Units for Steel Design

Quantity	Unit	Symbol
length	meter	m
mass	kilogram	kg
time	second	s
temperature	celsius	°C

Similarly, of the numerous decimal prefixes included in the SI system, only three are used in steel design; see Table 17-16.

Table 17-16. SI Prefixes for Steel Design

Prefix	Symbol	Order of Magnitude	Expression
mega	M	10^6	1,000,000 (one million)
kilo	k	10^3	1,000 (one thousand)
milli	m	10^{-3}	0.001 (one thousandth)

In addition, three derived units are applicable to the present conversion. They are shown in Table 17-17.

Table 17-17. Derived SI Units for Steel Design

Quantity	Name	Symbol	Expression
force	newton	N	N = kg x m/s^2
stress	pascal	Pa	Pa = N/m^2
energy	joule	J	J = N x m

Although specified in SI, the pascal is not universally accepted as the unit of stress. Because section properties are expressed in millimeters, it is more convenient to express stress in newtons per square millimeter (1 N/mm^2 = 1 MPa). This is the practice followed in recent international structural design standards. It should be noted that the joule, as the unit of energy, is used to express energy absorption requirements for impact tests. Moments are expressed in terms of N-mm.

A summary of the conversion factors relating traditional U.S. units of measurement to the corresponding SI units is given in Table 17-18.

Table 17-18. Summary of SI Conversion Factors

Multiply	by:	to obtain:
inch (in.)	25.4	millimeters (mm)
foot (ft)	0.3048	meters (m)
pound-mass (lb)	0.4536	kilogram (kg)
pound-force (lbf)	4.448	newton (N)
ksi	6.895	megapascals (MPa), N/mm^2
ft-lbf	1.356	joule (J)
psf	47.88	N/m^2
plf	14.59	N/m

Note that fractions resulting from metric conversion should be rounded to whole millimeters. Common fractions of inches and their metric equivalents are in Table 17-19.

Table 17-19. SI Equivalents of Fractions of an Inch

Fraction, in.	Exact Conversion, mm	Rounded, mm
$^1/_{16}$	1.5875	2
$^1/_8$	3.175	3
$^3/_{16}$	4.7625	5
$^1/_4$	6.35	6
$^5/_{16}$	7.9375	8
$^3/_8$	9.525	10
$^7/_{16}$	11.1125	11
$^1/_2$	12.7	13
$^5/_8$	15.875	16
$^3/_4$	19.05	19
$^7/_8$	22.225	22
1	25.4	25

Bolt diameters are taken directly from the ASTM F3125 rather than converting the diameters of SI bolts dimensioned in inches, since metric bolts are of different physical sizes. The metric bolt designations are in Table 17-20.

Table 17-20. SI Bolt Designation

Designation	Diameter, mm	Diameter, in.
M12	12	0.47
M16	16	0.63
M20	20	0.79
M22	22	0.87
M24	24	0.94
M27	27	1.06
M30	30	1.18
M36	36	1.42

The yield strengths of structural steels are taken from the metric ASTM Specifications. It should be noted that the yield points are slightly different from the traditional values. See Table 17-21. The modulus of elasticity of steel, E, is taken as 200 000 MPa. The shear modulus of elasticity of steel, G, is 77 000 MPa.

Table 17-21. SI Steel Yield Stresses

ASTM Designation	Yield stress, N/mm^2	Yield stress, ksi
A36M	250	36
A572M Gr. 345, A588M	345	50
A514M	690	100

Table 17-22
Weights and Measures
International System of Units (SI)[a]
(Metric practice)

Base Units

Quantity	Unit	Symbol
length	meter	m
mass	kilogram	kg
time	second	s
electric current	ampere	A
thermodynamic temperature	kelvin	K
amount of substance	mole	mol
luminous intensity	candela	cd

Supplementary Units

Quantity	Unit	Symbol
plane angle	radian	rad
solid angle	steradian	sr

Derived Units (with Special Names)

Quantity	Unit	Symbol	Formula
force	newton	N	kg-m/s^2
pressure, stress	pascal	Pa	N/m^2
energy, work, quantity of heat	joule	J	N-m
power	watt	W	J/s

Derived Units (without Special Names)

Quantity	Unit	Formula
area	square meter	m^2
volume	cubic meter	m^3
velocity	meter per second	m/s
acceleration	meter per second squared	m/s^2
specific volume	cubic meter per kilogram	m^3/kg
density	kilogram per cubic meter	kg/m^3

SI Prefixes

Multiplication Factor	Prefix	Symbol
1 000 000 000 000 000 000 = 10^{18}	exa	E
1 000 000 000 000 000 = 10^{15}	peta	P
1 000 000 000 000 = 10^{12}	tera	T
1 000 000 000 = 10^{9}	giga	G
1 000 000 = 10^{6}	mega	M
1 000 = 10^{3}	kilo	k
100 = 10^{2}	hecto[b]	h
10 = 10^{1}	deka[b]	da
0.1 = 10^{-1}	deci[b]	d
0.01 = 10^{-2}	centi[b]	c
0.001 = 10^{-3}	milli	m
0.000 001 = 10^{-6}	micro	μ
0.000 000 001 = 10^{-9}	nano	n
0.000 000 000 001 = 10^{-12}	pico	p
0.000 000 000 000 001 = 10^{-15}	femto	f
0.000 000 000 000 000 001 = 10^{-18}	atto	a

[a] Refer to ASTM IEEE/ASTM SI10 for more complete information on SI.
[b] Use is not recommended.

Table 17-23
SI Conversion Factors[a]

Quantity	Multiply	by	to obtain	
Length	inch	25.400	millimeter	mm
	foot	0.305	meter	m
	yard	0.914	meter	m
	mile (U.S. statute)	1.609	kilometer	km
	millimeter	39.370×10^{-3}	inch	in.
	meter	3.281	foot	ft
	meter	1.094	yard	yd
	kilometer	0.621	mile	mi
Area	square inch	0.645×10^{3}	square millimeter	mm^2
	square foot	0.093	square meter	m^2
	square yard	0.836	square meter	m^2
	square mile (U.S. statute)	2.590	square kilometer	km^2
	acre	4.047×10^{3}	square meter	m^2
	acre	0.405	hectare	
	square millimeter	1.550×10^{-3}	square inch	$in.^2$
	square meter	10.764	square foot	ft^2
	square meter	1.196	square yard	yd^2
	square kilometer	0.386	square mile	mi^2
	square meter	0.247×10^{-3}	acre	
	hectare	2.471	acre	
Volume	cubic inch	16.387×10^{3}	cubic millimeter	mm^3
	cubic foot	28.317×10^{-3}	cubic meter	m^3
	cubic yard	0.765	cubic meter	m^3
	gallon (U.S. liquid)	3.785	liter	l
	quart (U.S. liquid)	0.946	liter	l
	cubic millimeter	61.024×10^{-6}	cubic inch	$in.^3$
	cubic meter	35.315	cubic foot	ft^3
	cubic meter	1.308	cubic yard	yd^3
	liter	0.264	gallon (U.S. liquid)	gal
	liter	1.057	quart (U.S. liquid)	qt
Mass	ounce (avoirdupois)	28.350	gram	g
	pound (avoirdupois)	0.454	kilogram	kg
	short ton	0.907×10^{3}	kilogram	kg
	gram	35.274×10^{-3}	ounce (avoirdupois)	oz av
	kilogram	2.205	pound (avoirdupois)	lb av
	kilogram	1.102×10^{-3}	short ton	

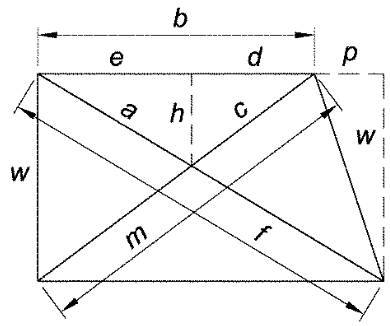
[a] Refer to ASTM IEEE/ASTM SI10 for more complete information on SI.
Note: The conversion factors tabulated herein have been rounded.

Table 17-23 (continued)
SI Conversion Factors[a]

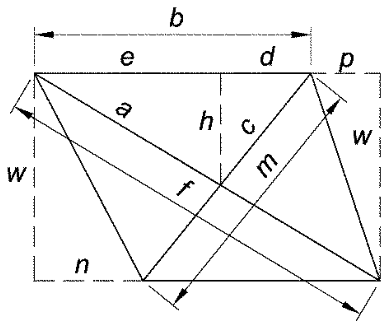
Quantity	Multiply	by	to obtain	
Force	ounce-force	0.278	newton	N
	pound-force	4.448	newton	N
	newton	3.597	ounce-force	
	newton	0.225	pound-force	lbf
Bending Moment	pound-force-inch	0.113	newton-meter	N-m
	pound-force-foot	1.356	newton-meter	N-m
	newton-meter	8.851	pound-force-inch	lbf-in.
	newton-meter	0.738	pound-force-foot	lbf-ft
Pressure, Stress	pound-force per square inch	6.895	kilopascal	kPa
	foot of water (39.2°F)	2.989	kilopascal	kPa
	inch of mercury (32°F)	3.386	kilopascal	kPa
	kilopascal	0.145	pound-force per square inch	lbf/in.^{2}
	kilopascal	0.335	foot of water (39.2°F)	
	kilopascal	0.295	inch of mercury (32°F)	
Energy, Work, Heat	foot-pound-force	1.356	joule	J
	[b]British thermal unit	1.055×10^3	joule	J
	[b]calorie	4.187	joule	J
	kilowatt hour	3.600×10^6	joule	J
	joule	0.738	foot-pound-force	ft-lbf
	joule	0.948×10^{-3}	[b]British thermal unit	Btu
	joule	0.239	[b]calorie	
	joule	0.278×10^{-6}	kilowatt hour	kW-h
Power	foot-pound-force/second	1.356	watt	W
	[b]British thermal unit per hour	0.293	watt	W
	horsepower (550 ft lbf/s)	0.746	kilowatt	kW
	watt	0.738	foot-pound-force/ second	ft-lbf/s
	watt	3.412	[b]British thermal units/ hour	Btu/h
	kilowatt	1.341	horsepower (550 ft-lbf/s)	hp
Angle	degree	17.453×10^{-3}	radian	rad
	radian	57.296	degree	°
Temperature	degree Fahrenheit	$t°C = (t°F - 32)/1.8$	degree Celsius	°F
	degree Celsius	$t°F = 1.8 \times t°C + 32$	degree Fahrenheit	°C

[a] Refer to ASTM IEEE/ASTM SI10 for more complete information on SI.
[b] International Table.
Note: The conversion factors tabulated herein have been rounded.

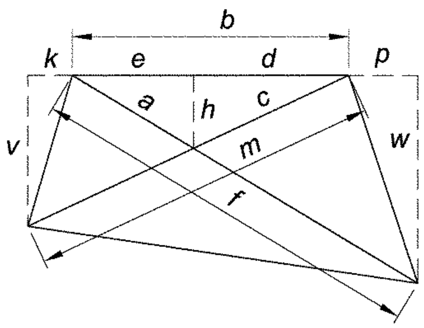
Table 17-24
Bracing Formulas



Given	To Find	Formula
bpw	f	$\sqrt{(b+p)^2+w^2}$
bw	m	$\sqrt{b^2+w^2}$
bp	d	$b^2 \div (2b+p)$
bp	e	$b(b+p) \div (2b+p)$
bfp	a	$bf \div (2b+p)$
bmp	c	$bm \div (2b+p)$
bpw	h	$bw \div (2b+p)$
afw	h	$aw \div f$
cmw	h	$cw \div m$



Given	To Find	Formula
bpw	f	$\sqrt{(b+p)^2+w^2}$
bnw	m	$\sqrt{(b-n)^2+w^2}$
bnp	d	$b(b-n) \div (2b+p-n)$
bnp	e	$b(b+p) \div (2b+p-n)$
$bfnp$	a	$bf \div (2b+p-n)$
$bmnp$	c	$bm \div (2b+p-n)$
$bnpw$	h	$bw \div (2b+p-n)$
afw	h	$aw \div f$
cmw	h	$cw \div m$

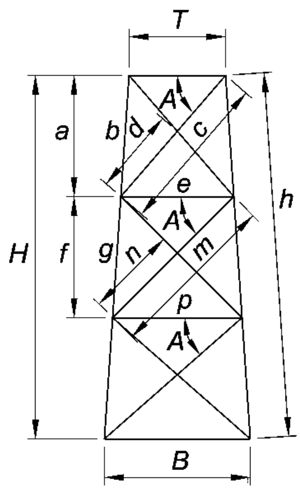


Given	To Find	Formula
bpw	f	$\sqrt{(b+p)^2+w^2}$
bkv	m	$\sqrt{(b+k)^2+v^2}$
$bkpvw$	d	$bw(b+k) \div [v(b+p)+w(b+k)]$
$bkpvw$	e	$bv(b+p) \div [v(b+p)+w(b+k)]$
$bfkpvw$	a	$fbv \div [v(b+p)+w(b+k)]$
$bkmpvw$	c	$bmw \div [v(b+p)+w(b+k)]$
$bkpvw$	h	$bvw \div [v(b+p)+w(b+k)]$
afw	h	$aw \div f$
cmv	h	$cv \div m$

Parallel Bracing

$k = (\log B - \log T) \div$ no. of panels. Constant k plus the logarithm of any line equals the log of the corresponding line in the next panel below.

$a = TH \div (T + e + p)$
$b = Th \div (T + e + p)$
$c = \sqrt{(^1\!/_2\,T + {}^1\!/_2\,e)^2 + a^2}$
$d = ce \div (T + e)$

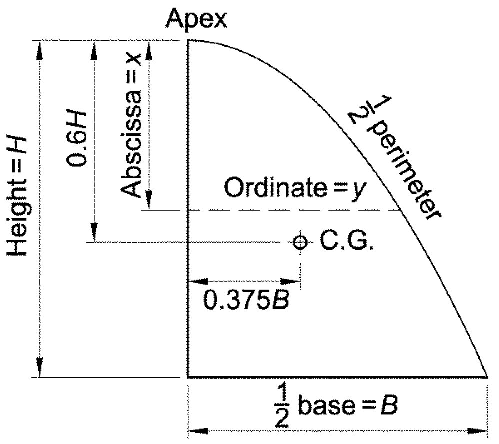


$\log e = k + \log T$
$\log f = k + \log a$
$\log g = k + \log b$
$\log m = k + \log c$
$\log n = k + \log d$
$\log p = k + \log e$

The above method can be used for any number of panels. In the formulas for a and b the sum in parenthesis, which in the case shown is $(T + e + p)$, is always composed of all the horizontal distances except the base.

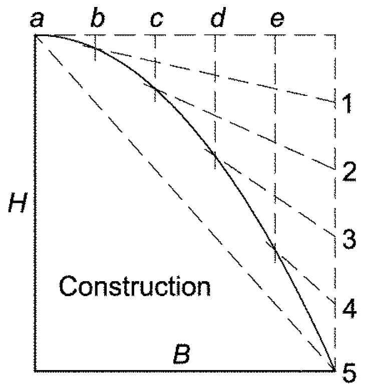
Table 17-25
Properties of the Parabola and Ellipse

Parabola



Parameter, $P = \dfrac{B^2}{H}$ $x = \dfrac{y^2}{P}$

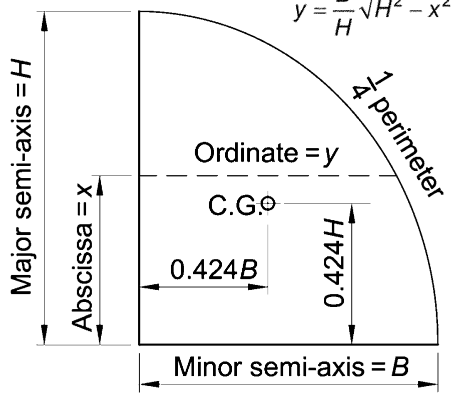
Area, $A = \dfrac{2}{3}HB$ $y = \sqrt{xP}$



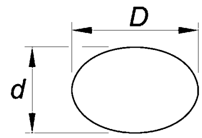
Ellipse

$$\frac{x^2}{H^2} + \frac{y^2}{B^2} = 1 \qquad x = \frac{H}{B}\sqrt{B^2 - y^2}$$

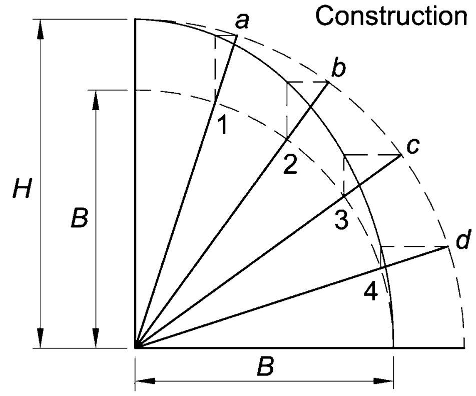
$$y = \frac{B}{H}\sqrt{H^2 - x^2}$$



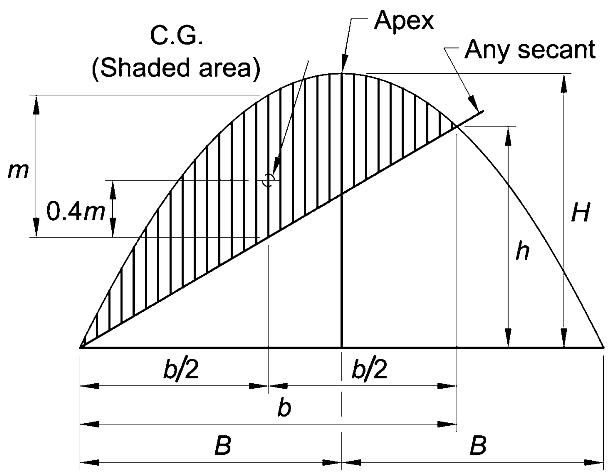
Approximate $\frac{1}{4}$ perimeter $= \dfrac{\pi}{4}\sqrt{2\left(H^2 + B^2\right)}$



Area $= 0.7854Dd$



Area Between Parabolic Curve and Secant



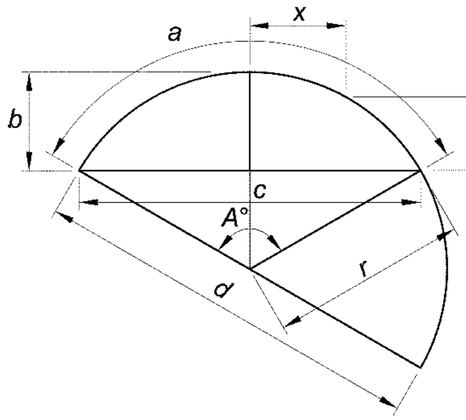
Shaded area $= \dfrac{2}{3}bm = \dfrac{Hb^3}{6B^2}$

$$h = Hb\left(\frac{2B - b}{B^2}\right)$$

$$m = \frac{Hb^2}{4B^2}$$

Length b may vary from 0 to $2B$

Table 17-26
Properties of the Circle



Circumference = 6.28318 r = 3.14159d
Diameter = 0.31831 circumference
Area = 3.14159r^2

$$\text{Arc } a = \frac{\pi r A^\circ}{180^\circ} = 0.017453 r A^\circ$$

$$\text{Angle } A^\circ = \frac{180^\circ a}{\pi r} = 57.29578\frac{a}{r}$$

$$\text{Angle } A^\circ = 2\sin^{-1}(c/2r)$$

$$\text{Angle } A^\circ = 4\tan^{-1}(2b/c)$$

$$\text{Radius } r = \frac{4b^2 + c^2}{8b}$$

$$\text{Chord } c = 2\sqrt{2br - b^2} = 2r\sin\frac{A}{2}$$

$$\text{Rise } b = r - \frac{1}{2}\sqrt{4r^2 - c^2} = \frac{c}{2}\tan\frac{A}{4}$$

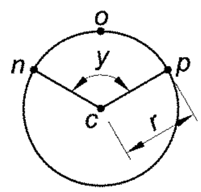
$$= 2r\sin^2\frac{A}{4} = r + y - \sqrt{r^2 - x^2}$$

$$y = b - r + \sqrt{r^2 - x^2}$$

$$x = \sqrt{r^2 - (r + y - b)^2}$$

Diameter of circle of equal periphery as square	= 1.27324 side of square
Side of square of equal periphery as circle	= 0.78540 diameter of circle
Diameter of circle circumscribed about square	= 1.41421 side of square
Side of square inscribed in circle	= 0.70711 diameter of circle

Circular Sector



r = radius of circle

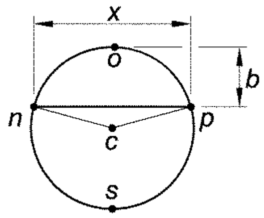
y = angle ncp in degrees

Area of Sector $ncpo$ = ½ (length of arc $nop \times r$)

$$= \text{Area of circle} \times \frac{y}{360}$$

$$= 0.0087266 \times r^2 \times y$$

Circular Segment



r = radius of circle
x = chord
b = rise

Area of segment nop = Area of sector ncpo − Area of triangle ncp

$$= \frac{(\text{Length of arc } nop \times r) - x(r - b)}{2}$$

r = radius of circle

y = angle ncp in degrees

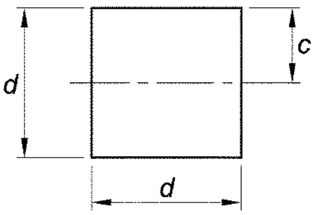
Area of sector $ncpo$ = ½ (length of arc $nop \times r$)

$$= \text{Area of circle} \times \frac{y}{360}$$

$$= 0.0087266 \times r^2 \times y$$

Table 17-27
Properties of Geometric Sections

Square
Axis of moments through center



$A = d^2$

$c = \dfrac{d}{2}$

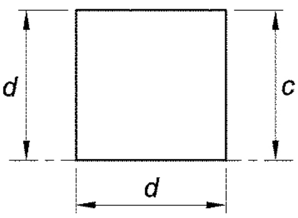
$I = \dfrac{d^4}{12}$

$S = \dfrac{d^3}{6}$

$r = \dfrac{d}{\sqrt{12}} = .288675\ d$

$Z = \dfrac{d^3}{4}$

Square
Axis of moments on base



$A = d^2$

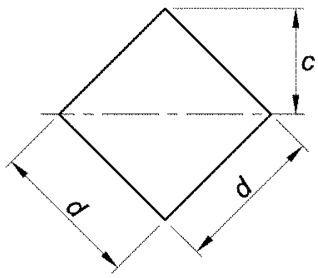
$c = d$

$I = \dfrac{d^4}{3}$

$S = \dfrac{d^3}{3}$

$r = \dfrac{d}{\sqrt{3}} = .577350\ d$

Square
Axis of moments on diagonal



$A = d^2$

$c = \dfrac{d}{\sqrt{2}} = .707107\ d$

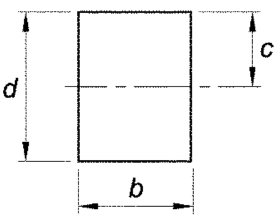
$I = \dfrac{d^4}{12}$

$S = \dfrac{d^3}{6\sqrt{2}} = .117851\ d^3$

$r = \dfrac{d}{\sqrt{12}} = .288675\ d$

$Z = \dfrac{2c^3}{3} = \dfrac{d^3}{3\sqrt{2}} = .235702\ d^3$

Rectangle
Axis of moments through center



$A = bd$

$c = \dfrac{d}{2}$

$I = \dfrac{bd^3}{12}$

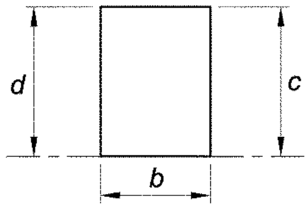
$S = \dfrac{bd^2}{6}$

$r = \dfrac{d}{\sqrt{12}} = .288675\ d$

$Z = \dfrac{bd^2}{4}$

Table 17-27 (continued)
Properties of Geometric Sections

Rectangle
Axis of moments on base



$A = bd$

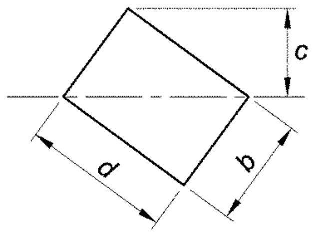
$c = d$

$I = \dfrac{bd^3}{3}$

$S = \dfrac{bd^2}{3}$

$r = \dfrac{d}{\sqrt{3}} = .577350\,d$

Rectangle
Axis of moments on diagonal



$A = bd$

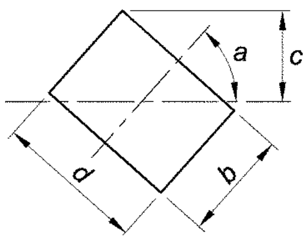
$c = \dfrac{bd}{\sqrt{b^2 + d^2}}$

$I = \dfrac{b^3 d^3}{6(b^2 + d^2)}$

$S = \dfrac{b^2 d^2}{6\sqrt{b^2 + d^2}}$

$r = \dfrac{bd}{\sqrt{6(b^2 + d^2)}}$

Rectangle
Axis of moments any line through center of gravity



$A = bd$

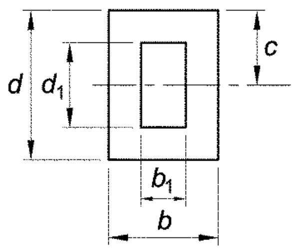
$c = \dfrac{b \sin a + d \cos a}{2}$

$I = \dfrac{bd\,(b^2 \sin^2 a + d^2 \cos^2 a)}{12}$

$S = \dfrac{bd\,(b^2 \sin^2 a + d^2 \cos^2 a)}{6\,(b \sin a + d \cos a)}$

$r = \sqrt{\dfrac{b^2 \sin^2 a + d^2 \cos^2 a}{12}}$

Hollow Rectangle
Axis of moments through center



$A = bd - b_1 d_1$

$c = \dfrac{d}{2}$

$I = \dfrac{bd^3 - b_1 d_1^3}{12}$

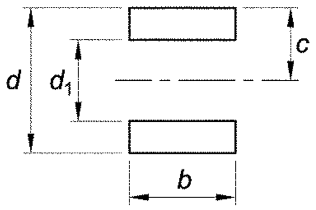
$S = \dfrac{bd^3 - b_1 d_1^3}{6d}$

$r = \sqrt{\dfrac{bd^3 - b_1 d_1^3}{12A}}$

$Z = \dfrac{bd^2}{4} - \dfrac{b_1 d_1^2}{4}$

Table 17-27 (continued)
Properties of Geometric Sections

Equal Rectangles
Axis of moments through
center of gravity



$A = b(d - d_1)$

$c = \dfrac{d}{2}$

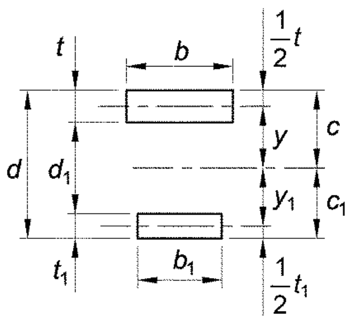
$I = \dfrac{b(d^3 - d_1^3)}{12}$

$S = \dfrac{b(d^3 - d_1^3)}{6d}$

$r = \sqrt{\dfrac{d^3 - d_1^3}{12(d - d_1)}}$

$Z = \dfrac{b}{4}(d^2 - d_1^2)$

Unequal Rectangles
Axis of moments through
center of gravity



$A = bt + b_1t_1$

$c = \dfrac{\frac{1}{2}bt^2 + b_1t_1(d - \frac{1}{2}t_1)}{A}$

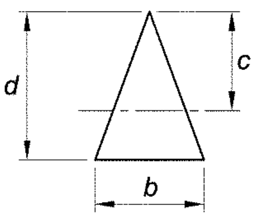
$I = \dfrac{bt^3}{12} + bty^2 + \dfrac{b_1t_1^3}{12} + b_1t_1y_1^2$

$S = \dfrac{I}{c} \qquad S_1 = \dfrac{I}{c_1}$

$r = \sqrt{\dfrac{I}{A}}$

$Z = bty + b_1t_1y_1$

Triangle
Axis of moments through
center of gravity



$A = \dfrac{bd}{2}$

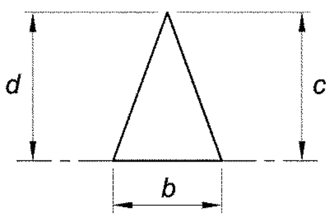
$c = \dfrac{2d}{3}$

$I = \dfrac{bd^3}{36}$

$S = \dfrac{bd^2}{24}$

$r = \dfrac{d}{\sqrt{18}} = .235702\,d$

Triangle
Axis of moments on base



$A = \dfrac{bd}{2}$

$c = d$

$I = \dfrac{bd^3}{12}$

$S = \dfrac{bd^2}{12}$

$r = \dfrac{d}{\sqrt{6}} = .408248\,d$

Table 17-27 (continued)
Properties of Geometric Sections

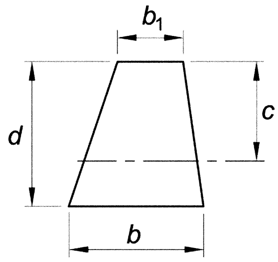
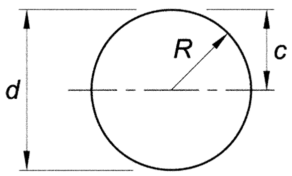
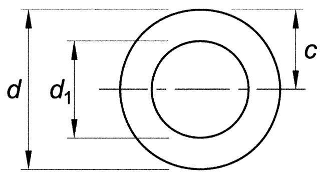
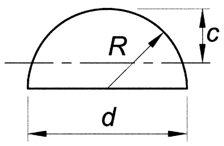
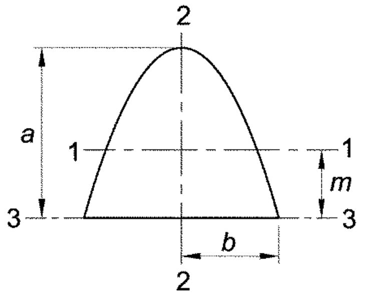
Trapezoid Axis of moments through center of gravity  	$A = \dfrac{d(b + b_1)}{2}$ $c = \dfrac{d(2b + b_1)}{3(b + b_1)}$ $I = \dfrac{d^3(b^2 + 4bb_1 + b_1^2)}{36(b + b_1)}$ $S = \dfrac{d^2(b^2 + 4bb_1 + b_1^2)}{12(2b + b_1)}$ $r = \dfrac{d}{6(b + b_1)}\sqrt{2(b^2 + 4bb_1 + b_1^2)}$
Circle Axis of moments through center  	$A = \dfrac{\pi d^2}{4} = \pi R^2 = .785398\ d^2 = 3.141593\ R^2$ $c = \dfrac{d}{2} = R$ $I = \dfrac{\pi d^4}{64} = \dfrac{\pi R^4}{4} = .049087\ d^4 = .785398\ R^4$ $S = \dfrac{\pi d^3}{32} = \dfrac{\pi R^3}{4} = .098175\ d^3 = .785398\ R^3$ $r = \dfrac{d}{4} = \dfrac{R}{2}$ $Z = \dfrac{d^3}{6}$
Hollow Circle Axis of moments through center  	$A = \dfrac{\pi(d^2 - d_1^2)}{4} = .785398\ (d^2 - d_1^2)$ $c = \dfrac{d}{2}$ $I = \dfrac{\pi(d^4 - d_1^4)}{64} = .049087\ (d^4 - d_1^4)$ $S = \dfrac{\pi(d^4 - d_1^4)}{32d} = .098175\ \dfrac{d^4 - d_1^4}{d}$ $r = \dfrac{\sqrt{d^2 + d_1^2}}{4}$ $Z = \dfrac{d^3}{6} - \dfrac{d_1^3}{6}$
Half Circle Axis of moments through center of gravity  	$A = \dfrac{\pi R^2}{2} = 1.570796\ R^2$ $c = R\left(1 - \dfrac{4}{3\pi}\right) = .575587\ R$ $I = R^4\left(\dfrac{\pi}{8} - \dfrac{8}{9\pi}\right) = .109757\ R^4$ $S = \dfrac{R^3}{24}\dfrac{(9\pi^2 - 64)}{(3\pi - 4)} = .190687\ R^3$ $r = R\dfrac{\sqrt{9\pi^2 - 64}}{6\pi} = .264336\ R$

Table 17-27 (continued)
Properties of Geometric Sections

Parabola



$$A = \frac{4}{3} ab$$

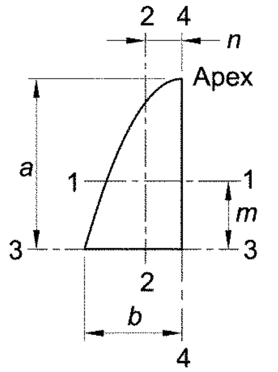
$$m = \frac{2}{5} a$$

$$I_1 = \frac{16}{175} a^3 b$$

$$I_2 = \frac{4}{15} ab^3$$

$$I_3 = \frac{32}{105} a^3 b$$

Half Parabola



$$A = \frac{2}{3} ab$$

$$m = \frac{2}{5} a$$

$$n = \frac{3}{8} b$$

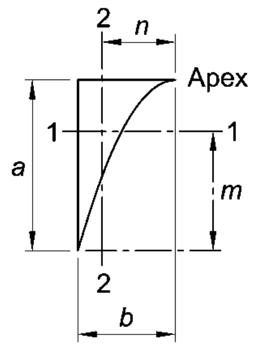
$$I_1 = \frac{8}{175} a^3 b$$

$$I_2 = \frac{19}{480} ab^3$$

$$I_3 = \frac{16}{105} a^3 b$$

$$I_4 = \frac{2}{15} ab^3$$

Complement of
Half Parabola



$$A = \frac{1}{3} ab$$

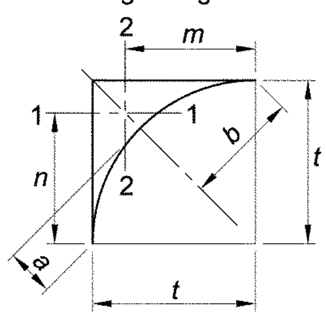
$$m = \frac{7}{10} a$$

$$n = \frac{3}{4} b$$

$$I_1 = \frac{37}{2{,}100} a^3 b$$

$$I_2 = \frac{1}{80} ab^3$$

Parabolic Fillet in
Right Angle



$$a = \frac{t}{2\sqrt{2}}$$

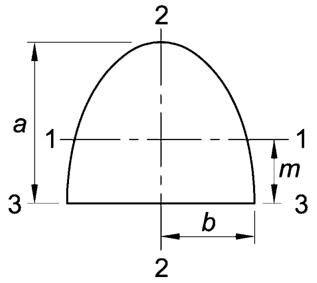
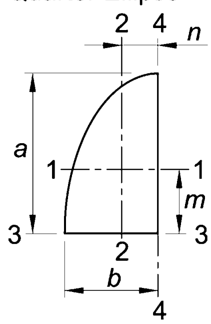
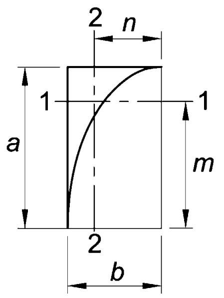
$$b = \frac{t}{\sqrt{2}}$$

$$A = \frac{1}{6} t^2$$

$$m = n = \frac{4}{5} t$$

$$I_1 = I_2 = \frac{11}{2{,}100} t^4$$

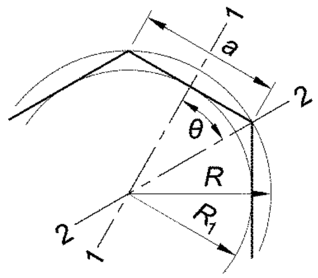
Table 17-27 (continued)
Properties of Geometric Sections

Half Ellipse* 	$A = \frac{1}{2}\pi ab$ $m = \frac{4a}{3\pi}$ $I_1 = a^3 b\left(\frac{\pi}{8} - \frac{8}{9\pi}\right)$ $I_2 = \frac{1}{8}\pi ab^3$ $I_3 = \frac{1}{8}\pi a^3 b$
Quarter Ellipse* 	$A = \frac{1}{4}\pi ab$ $m = \frac{4a}{3\pi}$ $n = \frac{4b}{3\pi}$ $I_1 = a^3 b\left(\frac{\pi}{16} - \frac{4}{9\pi}\right)$ $I_2 = ab^3\left(\frac{\pi}{16} - \frac{4}{9\pi}\right)$ $I_3 = \frac{1}{16}\pi a^3 b$ $I_4 = \frac{1}{16}\pi ab^3$
Elliptic Complement* 	$A = ab\left(1 - \frac{\pi}{4}\right)$ $m = \frac{a}{6\left(1 - \frac{\pi}{4}\right)}$ $n = \frac{b}{6\left(1 - \frac{\pi}{4}\right)}$ $I_1 = a^3 b\left(\frac{1}{3} - \frac{\pi}{16} - \frac{1}{36\left(1 - \frac{\pi}{4}\right)}\right)$ $I_2 = ab^3\left(\frac{1}{3} - \frac{\pi}{16} - \frac{1}{36\left(1 - \frac{\pi}{4}\right)}\right)$

*To obtain properties of half circle, quarter circle, and circular complement, substitute $a = b = R$.

Table 17-27 (continued)
Properties of Geometric Sections

Regular Polygon
Axis of moments
through center



n = Number of sides

$$\theta = \frac{180°}{n}$$

$$a = 2\sqrt{R^2 - R_1^2}$$

$$R = \frac{a}{2\sin\theta}$$

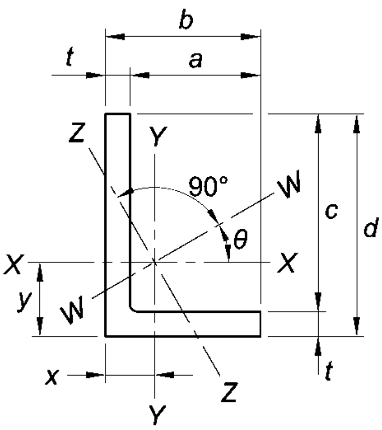
$$R_1 = \frac{a}{2\tan\theta}$$

$$A = \frac{1}{4} na^2 \cot\theta = \frac{1}{2} nR^2 \sin 2\theta = nR_1^2 \tan\theta$$

$$I_1 = I_2 = \frac{A(6R^2 - a^2)}{24} = \frac{A(12R_1^2 + a^2)}{48}$$

$$r_1 = r_2 = \sqrt{\frac{6R^2 - a^2}{24}} = \sqrt{\frac{12R_1^2 + a^2}{48}}$$

Angle
Axis of moments through
center of gravity



$$\tan 2\theta = \frac{2K}{I_Y - I_X}$$

$$A = t(b + c), \quad x = \frac{b^2 + ct}{2(b + c)}, \quad y = \frac{d^2 + at}{2(b + c)}$$

K = Product of Inertia about X-X and Y-Y axes

$$= \pm \frac{abcdt}{4(b + c)}$$

$$I_X = \frac{1}{3}\left[t(d - y)^3 + by^3 - a(y - t)^3\right]$$

$$I_Y = \frac{1}{3}\left[t(b - x)^3 + dx^3 - c(x - t)^3\right]$$

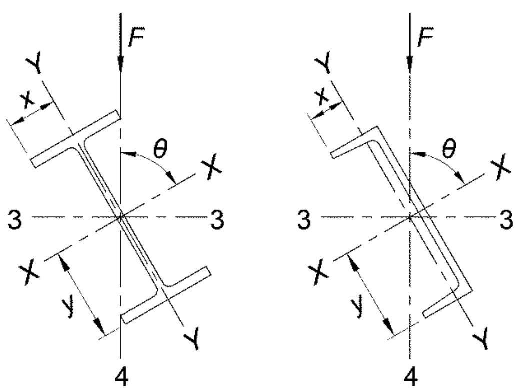
$$I_Z = I_X \sin^2\theta + I_Y \cos^2\theta + K \sin 2\theta$$

$$I_W = I_X \cos^2\theta + I_Y \sin^2\theta - K \sin 2\theta$$

K is negative when heel of angle, with respect to center of gravity, is in 1st or 3rd quadrant, positive when in 2nd or 4th quadrant.

Note that this is an idealized angle configuration and it differs from that provided by producers with dimensions given in Table 1-7.

Beams and Channels
Transverse force oblique
through center of gravity



$$I_3 = I_X \sin^2\theta + I_Y \cos^2\theta$$

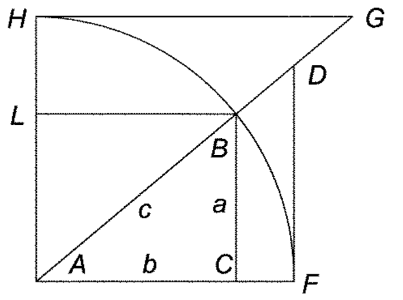
$$I_4 = I_X \cos^2\theta + I_Y \sin^2\theta$$

$$f_b = M\left(\frac{y}{I_X}\sin\theta + \frac{x}{I_Y}\cos\theta\right)$$

where M is bending moment due to force F.

Table 17-28
Trigonometric Formulas

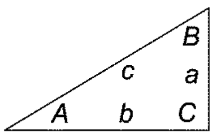
Trigonometric Functions



Radius $AF = 1$

$$= \sin^2 A + \cos^2 A = \sin A \operatorname{cosec} A$$
$$= \cos A \sec A = \tan A \cot A$$
$$\sin A = \frac{\cos A}{\cot A} = \frac{1}{\operatorname{cosec} A} = \cos A \tan A = \sqrt{1 - \cos^2 A} = BC$$
$$\cos A = \frac{\sin A}{\tan A} = \frac{1}{\sec A} = \sin A \cot A = \sqrt{1 - \sin^2 A} = AC$$
$$\tan A = \frac{\sin A}{\cos A} = \frac{1}{\cot A} = \sin A \sec A = FD$$
$$\cot A = \frac{\cos A}{\sin A} = \frac{1}{\tan A} = \cos A \operatorname{cosec} A = HG$$
$$\sec A = \frac{\tan A}{\sin A} = \frac{1}{\cos A} = AD$$
$$\operatorname{cosec} A = \frac{\cot A}{\cos A} = \frac{1}{\sin A} = AG$$

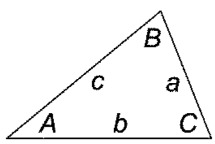
Right Angled Triangles



$$a^2 = c^2 - b^2$$
$$b^2 = c^2 - a^2$$
$$c^2 = a^2 + b^2$$

Known	Required A	Required B	Required a	Required b	Required c	Required Area
a, b	$\tan A = \frac{a}{b}$	$\tan B = \frac{b}{a}$			$\sqrt{a^2 + b^2}$	$\frac{ab}{2}$
a, c	$\sin A = \frac{a}{c}$	$\cos B = \frac{a}{c}$		$\sqrt{c^2 - a^2}$		$\frac{a\sqrt{c^2 - a^2}}{2}$
A, a		$90° - A$		$a \cot A$	$\frac{a}{\sin A}$	$\frac{a^2 \cot A}{2}$
A, b		$90° - A$	$b \tan A$		$\frac{b}{\cos A}$	$\frac{b^2 \tan A}{2}$
A, c		$90° - A$	$c \sin A$	$c \cos A$		$\frac{c^2 \sin 2A}{4}$

Oblique Angled Triangles



$$s = \frac{a + b + c}{2}$$
$$K = \sqrt{\frac{(s-a)(s-b)(s-c)}{s}}$$

$$a^2 = b^2 + c^2 - 2bc \cos A$$
$$b^2 = a^2 + c^2 - 2ac \cos B$$
$$c^2 = a^2 + b^2 - 2ab \cos C$$

Known	Required A	Required B	Required C	Required b	Required c	Required Area
a, b, c	$\tan \frac{1}{2} A = \frac{K}{s - a}$	$\tan \frac{1}{2} B = \frac{K}{s - b}$	$\tan \frac{1}{2} C = \frac{K}{s - c}$			$\sqrt{s(s-a)(s-b)(s-c)}$
a, A, B			$180° - (A + B)$	$\frac{a \sin B}{\sin A}$	$\frac{a \sin C}{\sin A}$	
a, b, A		$\sin B = \frac{b \sin A}{a}$			$\frac{b \sin C}{\sin B}$	
a, b, C	$\tan A = \frac{a \sin C}{b - a \cos C}$				$\sqrt{a^2 + b^2 - 2ab \cos C}$	$\frac{ab \sin C}{2}$

GENERAL NOMENCLATURE

The following definitions apply, as these variables are used in the AISC *Steel Construction Manual*. Additional nomenclature used in both the *Manual* and the AISC *Specification* can be found in the AISC *Specification for Structural Steel Buildings,* in Part 16 of this Manual.

A	Cross-sectional area, in.^{2}
A	Gross area of the truss chord, in.^{2}
A	Minimum side dimension for square or rectangular beveled washer, in.
A_b	Nominal unthreaded body area of bolt, in.^{2}
A_b	Required transverse force from adjacent bay, kips
A_c	Area of concrete, in.^{2}
A_e	Effective net area, in.^{2}
A_f	Flange area, in.^{2}
A_g	Gross cross-sectional area of the shear plate, in.^{2}
A_g	Total cross-sectional area of member, in.^{2}
A_{gt}	Gross area subject to tension, in.^{2}
A_{gv}	Gross area subject to shear, in.^{2}
A_{nt}	Net area subject to tension, in.^{2}
A_{nv}	Net area subject to shear, in.^{2}
A_{pb}	Projected area in bearing, in.^{2}
A_s	Cross-sectional area of the steel section, in.^{2}
A_{sr}	Area of continuous reinforcing bars, in.^{2}
A_{srs}	Area of all continuous reinforcing bars at the centerline, in.^{2}
A_w	Area of web, $(d - 2k)t_w$, in.
A_w	Area of web, the overall depth times the web thickness, dt_w, in.^{2}
A_{wei}	Effective weld area, in.^{2}
B	Bearing plate width, in.
B	HSS width, in.
B	Base plate width, in.
B_b	Overall width of rectangular HSS branch member or plate, measured 90° to the plane of the connection, in.
B_c	Available tension per bolt based on the limit state of tension only or the combined limit states of tension and shear rupture, kips
B_e	Effective width of rectangular HSS branch member or plate in.
C	Coefficient for eccentrically loaded bolt and weld groups
C	HSS torsional constant, in.^{3}
C	Width across points of square or hex bolt head or nut, or maximum diameter of countersunk bolt head, in.
C_b	Lateral-torsional buckling modification factor for nonuniform moment diagrams when both ends of the segment are braced
C_w	Warping constant, in.^{6}
C_1	Loading constant used in deflection calculations
C_1	Clearance for tightening, in.

C_1 Electrode strength coefficient where, for E70 electrodes, $C_1 = 1.00$
C_2 Clearance for entering, in.
C_3 Clearance for fillet based on one standard hardened washer, in.
C' Coefficient for eccentrically loaded bolt groups subjected to moment only
D Required number of sixteenths of an inch in the weld size on each side of the connecting element
D Weld size in sixteenths of an inch
E Minimum effective throat thickness for partial-joint-penetration groove weld, in.
E Modulus of elasticity of steel = 29,000 ksi
E_T Tangent modulus, ksi
ENA Elastic neutral axis
F Clearance for tightening staggered bolts, in.
F Width across flats of bolt head, in.
F_c Available stress in main member, ksi
F_{cr} Critical stress, ksi
F_{cr} Flexural local buckling stress, ksi
F_e' Euler stress for a prismatic member divided by safety factor, ksi
F_{EXX} Filler metal classification strength, ksi
F_{nwi} Nominal stress in the *i*th weld element, ksi
F_{nt} Nominal tensile strength from AISC *Specification* Table J3.2, ksi
F_{nv} Nominal shear strength from AISC *Specification* Table J3.2, ksi
F_p Nominal bearing stress on fastener, ksi
F_u Specified minimum tensile strength, ksi
F_y Specified minimum yield strength, ksi
F_{yb} F_y of a beam, ksi
F_{yc} F_y of a cap plate, ksi
F_{yf} Specified minimum yield stress of the flange, ksi
F_{yw} Specified minimum yield stress of the web, ksi
G Ratio of the total column stiffness framing into a joint to that of the stiffening members framing into the same joint
H Flexural constant
H Horizontal component of the required axial force, kips
H_b Required shear force on the gusset-to-beam connection, kips
H_c Required axial force on the gusset-to-column connection, kips
H_1 Height of bolt head, in.
H_2 Maximum bolt shank extension based on one standard hardened washer, in.
I Moment of inertia, in.^{4}
I Moment of inertia of the combined cross section, in.^{4}
I_c Moment of inertia of column section about axis perpendicular to plane of buckling, in.^{4}
I_g Moment of inertia of girder about axis perpendicular to plane of buckling, in.^{4}
I_p Polar moment of inertia of bolt and weld groups, ($I_p = I_x + I_y$), in.^{4} per in.^{2}
I_x Combined moment of inertia of the bolt group and compression block about the neutral axis, in.^{4}
I_x, I_y Moment of inertia about the principal axes, in.^{4}
IC Instantaneous center of rotation for bolt and weld groups
J Torsional constant, in.^{4}

K	Effective length factor
K_{dep}	Fillet depth, $(k - t_f)$, in.
L	Distance over which the load is delivered, measured along the longer dimension of the plate element, in.
L	Length of connection in the direction of loading, in.
L	Length over which the load is delivered, measured parallel to the supported edges, in.
L	Span length, in.
L	Total length of beam between reaction points, ft
L_b	Length between points that are either braced against lateral displacement of compression flange or braced against twist of the cross section, in.
L_c	Effective length of member, in.
L_{cx}	Effective length of member for buckling about x-axis, in.
L_{cy}	Effective length of member for buckling about y-axis, in.
L_{cz}	Effective length of member for buckling about longitudinal axis, in.
L_p'	Limiting laterally unbraced length for the maximum design flexural strength for noncompact shapes, uniform moment case ($C_b = 1.0$), in. or ft, as indicated
L_p	Limiting laterally unbraced length for the limit state of yielding, in.
L_r	Limiting laterally unbraced length for the limit state of inelastic lateral-torsional buckling, in.
M	Applied moment, kip-in.
M	Maximum service-load moment, kip-ft
M	Beam bending moment, kip-in. or kip-ft, as indicated
M_a	Required beam end moment using ASD load combinations, kip-in.
M_a	Required flexural strength using ASD load combinations, kip-in. or kip-ft, as indicated
M_a	Required moment in the beam at the splice using ASD load combinations, kip-in.
M_b	Required beam moment, kip-in. or kip-ft, as indicated
M_c	Available flexural strength, kip-in.
M_c	Required column moment, kip-in. or kip-ft, as indicated
M_{cy}	Available flexural strength about the y-axis based on the LRFD or ASD load combinations, kip-in.
M_{lt}	First-order moment using LRFD or ASD load combinations, due to lateral translation of the structure only, kip-in.
M_{max}	Maximum moment, kip-in.
M_{nt}	First-order moment using LRFD or ASD load combinations, with the structure restrained against lateral translation, kip-in.
M_p	Plastic bending moment, kip-in.
M_p'	Maximum available flexural strength for noncompact shapes, when $L_b \le L_p'$, kip-in. or kip-ft, as indicated
M_{px}	Plastic bending moment about the x-axis, kip-ft
M_r	Limiting buckling moment, M_{cr}, when $\lambda = \lambda_r$ and $C_b = 1.0$, kip-in. or kip-ft, as indicated
M_r	Required second-order flexural strength using LRFD or ASD load combinations, kip-in.
M_r	Required flexural strength, kip-in.
M_{ro}	Required moment of the HSS using ASD or LRFD load combinations, kip-in.
M_{rx}, M_{ry}	Required flexural strength, kip-in.

M_u	Required beam end moment using LRFD load combinations, kip-in.
M_u	Required flexural strength using LRFD load combinations, kip-in. or kip-ft, as indicated
M_u	Required moment in the beam at the splice using LRFD load combinations, kip-in.
M_y	Flexural yield moment, kip-in.
N	Applied normal force, kips
N	Length of base plate, in.
N_b	Number of bolts in a joint
P	Axial force due to service loads, kips
P	Bolt stagger, in.
P	Concentrated load, kips
P	Required force, kips
P_a	Required axial strength (tension or compression) using ASD load combinations, kips
P_{af}	Beam flange force, tensile or compressive, using ASD load combinations, kips
P_c	Available axial strength, kips
P_e	Elastic Euler buckling load, kips
P_{ex}, P_{ey}	Elastic Euler buckling load about the x- and y-axis, kips
P_{fb}	Resistance to flange local bending per AISC *Specification* Equation J10-1 (used to check need for column web stiffeners), kips
P_{lt}	First-order axial force using LRFD or ASD load combinations, due to lateral translation of the structure only, kips
P_{nt}	First-order axial force using LRFD or ASD load combinations, with the structure restrained against lateral translation, kips
P_r	Required axial strength, kips
P_r	Required second-order axial strength using LRFD or ASD load combinations, kips
P_{ro}	Required strength of the HSS using LRFD or ASD load combinations, kips
P_u	Required axial strength (tension or compression) using LRFD load combinations, kips
P_{uf}	Beam flange force, tensile or compressive, using LRFD load combinations, kips
P_{wb}	Resistance to web compression buckling per AISC *Specification* Equation J10-8 (used to check need for column web stiffening), kips
P_{wi}	A factor consisting of terms from the second portion of AISC *Specification* Equation J10-2 (used in a column web stiffener check for web local yielding), kip/in.
P_{wo}	A factor consisting of the first portion of AISC *Specification* Equation J10-2 (used in a column web stiffener check for web local yielding), kips
PNA	Plastic neutral axis
Q	First moment of the channel area about the neutral axis of the combined cross section, in.^{3}
Q_n	Nominal shear strength of one steel headed stud anchor, kips
R	Nominal reaction, kips
R	Nominal shear strength of one bolt at a deformation Δ, kips
R_a	Beam end reaction based on ASD load combinations, kips
R_a	Required strength determined by analysis for the ASD load combinations, kips
R_b	Required end reaction of the beam, kips
R_c	Required column axial load above the connection, kips
R_M	Coefficient to account for influence of P-δ on P-Δ

R_{max}	Upper limit for corner radius of HSS, in.
R_{min}	Lower limit for corner radius of HSS, in.
R_n	Nominal strength determined according to the AISC *Specification* provisions, kips
R_u	Beam end reaction based on LRFD load combinations, kips
R_u	Required strength determined by analysis for the LRFD load combinations, kips
R_{ult}	Ultimate shear strength of one bolt, kips
R_1	Beam bearing constant for web local yielding, see Part 9, kips
R_1	Left end beam reaction, kips
R_2	Beam bearing constant for web local yielding, see Part 9, kip/in.
R_2	Right end or intermediate beam reaction, kips
R_3	Beam bearing constant for web local crippling, see Part 9, kips
R_3	Right end beam reaction, kips
R_4	Beam bearing constant for web local crippling, see Part 9, kip/in.
R_5	Beam bearing constant for web local crippling, see Part 9, kips
R_6	Beam bearing constant for web local crippling, see Part 9, kip/in.
S	Elastic section modulus about the axis of bending, in.^{3}
S	Spacing, in. or ft, as indicated
S	Groove depth for partial-joint-penetration groove welds, in.
S_{net}	Elastic section modulus at the cope, in.^{3}
S_{net}	Net elastic section modulus, in.^{3}
S_x	Minimum elastic section modulus taken about the x-axis, in.^{3}
S_1, S_2	Elastic section modulus about the x-axis referred to the designated edge of member, in.^{3}
T	Distance between web toes of fillets at top and at bottom of web, in. = $d - 2k$
T	Tension force due to service loads, kips
T	Required strength, kips
T	Thickness of flat circular washer or mean thickness of square or rectangular beveled washer, in.
T	Width of element, in.
T_a	Required tension force per bolt using ASD load combinations, kips
T_c	Available tensile strength including the effects of prying action, kips
T_u	Required tension force per bolt using LRFD load combinations, kips
U	Shear lag coefficient
V	Maximum vertical shear for any condition of symmetrical loading, kips
V	Shear force, kips
V	Vertical component of the required force, kips
V	Vertical shear, kips
V'	Horizontal shear strength at the steel-concrete interface, kips
V_a	Required shear strength using ASD load combinations, kips
V_b	Shear force component, kips
V_b	Required axial force on the gusset-to-beam connection, kips
V_c	Required shear force on the gusset-to-column connection, kips
V_c	Available shear strength, kips
V_{nx}	Nominal strong-axis shear strength, kips
V_r	Required shear strength, kips
V_u	Required shear strength using LRFD load combinations, kips
W	Dimension of the stiffening element parallel to the beam web, in.

W	Width across flats of nut, in.
W_a	Total factored uniformly distributed load using ASD load combinations, kips
W_c	Uniform load constant for beams, kip-ft
W_u	Total factored uniformly distributed load using LRFD load combinations, kips
Y	Fillet weld length, in.
Y_{con}	Distance from top of steel beam to top of concrete, in.
$Y1$	Distance from top of steel beam to the plastic neutral axis, in.
$Y2$	Distance from top of steel beam to the concrete flange force in a composite beam, in.
Z	Gross plastic section modulus, in.^{3}
Z_{net}	Plastic section modulus at the cope, in.^{3}
Z_{net}	Net plastic section modulus, in.^{3}
Z_{pl}	Plastic section modulus of the shear plate, in.^{3}
Z_s	Full x-axis plastic section modulus of steel shape, in.^{3}
Z_{sE}	Plastic section modulus of the steel shape about the y-axis, in.^{3}
Z_x	Plastic section modulus about the x-axis, in.^{3}
Z_y	Plastic section modulus about the y-axis, in.^{3}
a	Depth of bracket plate, in.
a	Distance from bolt centerline to edge of fitting subjected to prying action, but not greater than $1.25b$, in.
a	Distance from the support to the bolt line in a single plate connection, in.
a	Distance from the support to the first line of bolts, in.
a	Distance from the weld line to the bolt line closest to the support, in.
a	Distance measured along width of the plate element from one edge of connected element to the nearest support, in.
a	Effective concrete flange thickness of a composite beam, in.
a'	Length of free edge of bracket plate, in.
a'	Weld length, in.
b	For a tee-type connecting element, the distance from bolt centerline to the face of the tee stem, in.; for angle-type connecting element, the distance from bolt centerline to centerline of angle leg, in.
b	Distance measured along width of the plate element from one edge of connected element to the farthest support, in.
b	Effective concrete flange width in a composite beam, in.
b	Flexible width in connecting element, in.
b	Measured distance along beam which may be greater or less than a, in.
b	Width of the flat wall of square or rectangular HSS, or the width of the longer leg for angles, or width of the back-to-back legs of long legs back-to-back double angles, or the width of outstanding legs of short legs back-to-back double angles, in.
b_A	Flange-plate width, in.
b_{eff}	Effective width, in.
b_f	Width of flange, in.
b_f	Connection element width, in.
c	Cope length, in.
c	Distance over which the load is delivered, measured along the shorter dimension of the plate element, in.

c	Radial distance from center of gravity to center of bolt most remote from center of gravity, in.
c	Radial distance from center of gravity to point in weld group most remote from center of gravity, in.
c_b	Length of bottom cope, in.
c_{eff}	Effective width of the attached element accounting for uneven stress distribution, in.
c_t	Length of top cope, in.
c_x, c_y	Horizontal and vertical components of the diagonal distance c, in.
c.g.	Center of gravity
d	Beam depth, in.
d	Bolt diameter, in.
d	Depth of compression block, in.
d	Fastener diameter, in.
d	Overall depth of member, or width of shorter leg for angles, or width of the outstanding legs of long legs back-to-back double angles, or the width of the back-to-back legs of short legs back-to-back double angles, in.
d_b	Nominal bolt diameter, in.
d_c	Cope depth, in.
d_{ct}	Cope depth at the compression flange, in.
d_h	Hole diameter, in.
d_m	Moment arm between resultant tensile force and resultant compressive force, in.
d'	Width of the hole along the length of the fitting, in.
d'_h	Hole diameter + $^1/_{16}$ in., in.
e	Eccentricity, in.
e	Base of natural logarithm = 2.718...
e	Distance from the face of the supporting member to the face of the cope, unless a lower value can be justified, in.
e	Distance of the beam end reaction with respect to the weld lines, in.
e	Width of the leg of the connection angle attached to the support, in.
e_b	One-half the depth of the beam, in.
e_c	One-half the depth of the column, in.
e_o	Horizontal distance from designated member edge to member shear center, in.
e_x	Horizontal component of eccentricity of P with respect to centroid of weld group, in.
e_x	Horizontal distance from the centroid of the bolt group to the line of action P, in.
f	Computed compressive stress in the stiffened element, ksi
f	Plate buckling adjustment factor for beams coped at top flange only
f_a	Computed axial stress, ksi
f_a	Shear per linear inch of weld due to the applied normal force, kip/in.
f_b	Maximum bending stress, ksi
f_b	Shear per linear inch of weld due to the applied moment, kip/in.
f_v	Shear per linear inch of weld due to the applied shear, kip/in.
f_w	Total design stress, kip/in.
h	Clear distance between flanges less the fillet or corner radius for rolled shapes, in.
h	Depth of the flat wall or square or rectangular HSS, in.
h	Hook length for hooked anchor rods, in.
h_o	Distance between flange centroids, in.

h_o	Reduced beam depth of coped beam, in.
k	Plate buckling coefficient for beams coped at top flange only
k	Distance from outer face of flange to the web toe of fillet, in.
k_c	Coefficient for slender unstiffened elements
k_{des}	Distance from outer face of flange to the web toe of fillet used for design, in.
k_{det}	Distance from outer face of flange to the web toe of fillet used for detailing, in.
k_1	Distance from web center line to flange toe of fillet, in.
k_1	Modified plate buckling coefficient
l	Critical base plate cantilever dimension, in.
l	Depth of plate, in.
l	Depth of connecting element, in.
l	Length of weld, in.
l	Span length, in.
l	Total length of beam between reaction points, in.
l	Vertical leg dimension of the seat angle, in.
l_a	Length of weld over which the applied normal force is distributed, in.
l_b	Length of bearing, in.
$l_{b,req}$	Required bearing length, in.
l_{eh}	Horizontal edge distance, in.
l_{ev}	Vertical edge distance, in.
l_i	Distance from the center of gravity of the bolt group to the *i*th bolt, in.
l_{max}	Distance from the center of gravity of the bolt group to the center of the farthest bolt, in.
l_p	Length of the single-plate shear connection, in.
l_w	Length of each weld, in.
m	Cantilever dimension for base plate, in.
n	Cantilever dimension for bearing plate, in.
n	Number of bolts in a vertical row
n	Number of bolts in the connection
n	Number of bolt rows
n	Number of fasteners
n'	Number of bolts above the neutral axis (in tension)
p	Tributary length used in determining prying action, in.
p_i	Ratio of element i deformation to its deformation at maximum stress
q	Horizontal shear, kip/in.
q_r	Prying force per bolt, kips
r	Radius of gyration, in.
r_a	Required shear strength per bolt using ASD load combinations, kip/bolt
r_{at}	Required tensile strength per bolt or per inch of weld using ASD load combinations (force per bolt or per inch of weld due to a tensile force), kips
r_{av}	Required shear strength per bolt or per inch of weld using ASD load combinations (force per bolt or per inch of weld due to a shear force), kips
r_{cr}	Distance from instantaneous center of rotation to weld element with minimum Δ_{ui}/r_i ratio, in.
r_{ma}	Required shear force on the bolt most remote from the center of gravity, due to moment using ASD load combinations, kips

r_{mu}	Required shear force on the bolt most remote from the center of gravity, due to moment using LRFD load combinations, kips
r_n	Nominal strength of one bolt, kips
$\bar{r}_o$	Polar radius of gyration about the shear center, in.
r_p	Required shear strength per bolt due to a concentric force, kips/bolt
r_{pa}	Shear per linear inch of weld due to the concentric force using ASD load combinations, kip/in.
r_{pu}	Shear per linear inch of weld due to the concentric force using LRFD load combinations, kip/in.
r_{ts}	Effective radius of gyration, in.
r_u	Required shear strength per bolt using LRFD load combinations, kip/bolt
r_{ut}	Required tensile strength per bolt or per inch of weld using LRFD load combinations (force per bolt or per inch of weld due to a tensile force), kips
r_{uv}	Required shear strength per bolt or per inch of weld using LRFD load combinations (force per bolt or per inch of weld due to a shear force), kips
r_x, r_y	Radius of gyration about x and y axes respectively, in.
r_z	Radius of gyration about the minor principal axis, in.
s	Separation between double angles or channels back-to-back, in.
s	Vertical bolt row spacing, in.
t	Change in temperature, degrees Fahrenheit or Celsius, as indicated
t	Design wall thickness of HSS member, in.
t	Initial temperature in °F
t	Thickness of angle leg, in.
t	Thickness of bracket plate, in.
t_b	Thickness of beam flange or connection plate delivering concentrated force, in.
t_c	Flange or angle thickness required to develop design tensile strength of bolts with no prying action, in.
t_{des}	Design thickness of an HSS wall, in.
$t_{d\ req}$	Required doubler-plate thickness, in.
t_f	Lesser connection element thickness, in.
t_f	Thickness of flange, in.
t_{min}	Minimum base metal thickness required to match the shear rupture strength of the weld, in.
t_{nom}	Nominal thickness of an HSS wall, in.
t_p	Plate thickness, in.
t_{sw}	Thickness of the tee stem, or supported beam web, in.
t_w	Web thickness, in.
w	Uniformly distributed load per unit of length, kip/in.
w	Weld leg size, in.
w_{min}	Minimum weld size, in.
x	Horizontal distance from the support to the location of applied bearing force, in.
$\bar{x}$	Horizontal distance from the designated edge of member to center of gravity, in.
x_p	Horizontal distance from the designated edge of member to its plastic neutral axis, in.
y	Distance from line X-X to the c.g. of the bolt group above the neutral axis, in.
$\bar{y}$	Vertical distance from the designated edge of member to center of gravity, in.

y_p	Vertical distance from the designated edge of member to its plastic neutral axis, in.
Δ	Deflection, in.
Δ	Deformation, in.
Δ	Elongation, in.
Δ	Total deformation, including shear, bearing and bending deformation in the bolt and bearing deformation of the connection elements, in.
Δ_i	Deformation of the *i*th weld element at an intermediate stress level, linearly proportioned to the critical deformation based on distance from the instantaneous center of rotation, r_i, in.
Δ_{max}	Maximum deflection, in.
Δ_{max}	Maximum deformation, in.
Δ_{max}	Maximum deformation on the bolt farthest from the center of gravity = 0.34 in.
Δ_{ucr}	Deformation of the weld element with minimum ratio Δ_{ui}/r_i at ultimate stress (rupture), usually in the element furthest from the instantaneous center of rotation, in.
Δ_{ui}	Deformation of the *i*th weld element at ultimate stress (rupture), in.
Δ_x	Deflection at any point x distance from left reaction, in.
Δ_α	Deflection at point of load, in.
Ω	Safety factor given by the AISC *Specification* for a particular limit state
Ω_b	Safety factor for flexure
Ω_c	Safety factor for compression
Ω_t	Safety factor for tension
Ω_v	Safety factor for shear
α	Distance from the face of the column flange or web to the centroid of the gusset-to-beam connection for uniform force method, in.
α	Ratio of the moment at the bolt line to the moment at the face of the tee stem, or at the center of the unconnected angle leg thickness
$\bar{\alpha}$	Actual distance from face of column flange or web to centroid of gusset-to-beam connection for uniform force method, in.
α'	Value of a used for prying action that either maximizes the bolt available tensile strength for a given thickness or minimizes the thickness required for a given bolt available tensile strength
β	Distance from the face of the beam flange to the centroid of the gusset-to-column connection for uniform force method, in.
$\bar{\beta}$	Actual distance from face of beam flange to centroid of gusset-to-column connection for uniform force method, in.
δ	Deflection, in.
δ	Ratio of the net length at the bolt line to the gross length at the face of the stem or leg of angle
ε	Coefficient of linear expansion, with units as indicated
τ_b	Stiffness reduction factor, for use with the alignment charts (AISC *Specification* Figures C-C2.3 and C-C2.4) in the determination of effective length factors, K, for columns
θ	Angle between the line of action of the required force and the weld longitudinal axis, degrees
θ_1	Angle between the longitudinal axis of *i*th weld element and the direction of the resultant force acting on the element, degrees

λ	Width-to-thickness ratio for the element as defined in AISC *Specification* Section B4.1
λ_p	Limiting width-to-thickness parameter for compact element
λ_{pf}	Limiting width-to-thickness parameter for compact flange
λ_r	Limiting width-to-thickness parameter for noncompact element
λ_{rf}	Limiting width-to-thickness parameter for noncompact flange
ϕ	Resistance factor given by the AISC *Specification* for a particular limit state
ϕ_b	Resistance factor for flexure
ϕ_c	Resistance factor for compression
ϕ_t	Resistance factor for tension
ϕ_v	Resistance factor for shear
ϕR_n	Design strength from AISC *Specification*; must equal or exceed required strength using LRFD load combinations, R_u
ϕr_n	Design strength per bolt or per inch of weld from AISC *Specification*; must equal or exceed required strength per bolt or per inch of weld using LRFD load combinations, r_u
R_n/Ω	Allowable strength from AISC *Specification*; must equal or exceed required strength using ASD load combinations, R_a
r_n/Ω	Allowable strength per bolt or per inch of weld from AISC *Specification*; must equal or exceed required strength per bolt or per inch of weld using ASD load combinations, r_a
σ_o	Nominal stress
τ_b	Stiffness reduction parameter

INDEX

The following list of terms provides reference to items found in the AISC *Steel Construction Manual*, as well as selected supporting references. The locations of supporting references have been abbreviated as follows:

"DG#" is used for items found in AISC's Design Guide series.
"SDM" is used for items found in the AISC *Seismic Design Manual.*
"DSC" is used for items found in AISC's *Detailing for Steel Construction.*
"AISC *Design Examples*" indicates that information can be found in the *Design Examples* posted on the AISC web site at **www.aisc.org.**