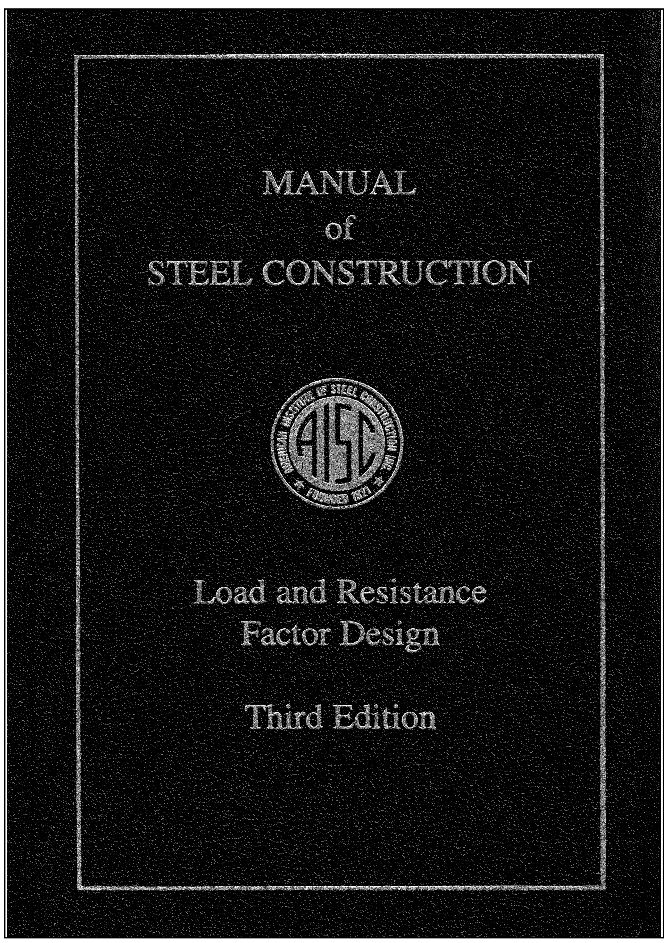
Revisions, January 2003

Manual of Steel Construction
Load and Resistance Factor Design
3rd Edition

The following technical revisions and corrections have been made in the second printing of the Third Edition (January, 2003). To facilitate the incorporation of revisions and corrections, this booklet has been constructed using excerpts from revised pages, with corrections noted. The user may find it convenient in some cases to hand-write corrections; in others, a cut-and-paste approach may be more efficient.



ISBN 1-56424-051-7

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Printed in the United States of America

First Printing: November 2001
Second Printing: January 2003 with revisions

PREFACE

This 3rd Edition *LRFD Manual of Steel Construction* is the twelfth major update of the AISC Manual of Steel Construction, which was first published in 1927. With this revision, member and connection design information has been condensed back into a single volume. It has been reorganized and reformatted to provide practical and efficient access to the information it contains, with a roadmap format to guide the user quickly to the applicable specifications, codes and standards, as well as the applicable provisions in those standards.

The following specifications, codes and standards are included in or with this Manual:

- 1999 *LRFD Specification for Structural Steel Buildings*
- 2000 *LRFD Specification for Steel Hollow Structural Sections*
- 2000 *LRFD Specification for Single-Angle Members*
- 2000 *RCSC Specification for Structural Joints Using ASTM A325 or A490 Bolts*
- 2000 *Code of Standard Practice for Steel Buildings and Bridges*
- AISC Shapes Database V3 CD

The following major improvements have been made in this revision:

- Workable gages for flange fasteners have been reintroduced.
- The revised T, k and k_1 values for W-shapes and the 0.93 wall-thickness reduction factor for HSS have been considered.
- Guidance is provided on the new OSHA safety regulations, stability bracing requirements and proper material specification.
- New information is provided on design drawing information requirements, criteria needed for connection design, mill, fabrication and erection tolerances, façade issues, temperature effects and fire protection requirements with summaries of common UL assemblies.
- Shape information has been updated to the current series.
- Coverage of round HSS has been added.
- Dimensions and properties have been added for double channels back-to-back.
- Tables of surface and box perimeter, weight/area-to-perimeter ratios and surface areas have been expanded to cover all common structural shapes.
- A new section on properly specifying materials, including shapes, plates, fasteners and other products, has been added.
- New information on corrosion protection and seismic design has been added.
- A new section has been added with design aids for tension members, including explicit consideration of net section requirements to ensure connectable member selection.
- Beam selection tables are included for selection based upon I_x, Z_x, I_y, and Z_y.
- Beam charts (ϕM_n vs. L_b) are plotted for both W-shapes and channels.
- New floor plate deflection and bending design aids have been added.
- Additional beam diagrams and formulas have been added.
- A new section has been added with design aids for W-shape beam-columns.
- New bolt length selection tables have been added.
- Bolt entering and tightening clearances have been updated.
- Bolting information has been updated for consistency with the 2000 RCSC Specification.

- Welding information, including the prequalified welded joint tables, has been updated to for consistency with AWS D1.1-2000.
- Information on prying action, Whitmore section and strength of coped beams has been updated.
- Selection tables for shear end-plate connections and single-plate connections have been improved and expanded, including single-plate connections with up to 12 rows of bolts and up to $1^1/_8$-in. diameter.
- New information and examples for flexible moment connections has been added as an update of Disque's historic "type 2 with wind" moment connection design approach.
- Previous limitations on the use of moment end-plate connections have been relaxed.
- Information on the design of anchor rods has been updated, including a new table of minimum dimensions for washers used with anchor rods.
- Composite member tables have been updated to include coverage of both 4 ksi and 5 ksi concrete.
- A cross-reference between U.S. customary and Metric shapes series has been included.

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

By the AISC Committee on Manuals and Textbooks,

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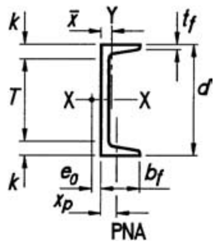


Table 1-5.
C-Shapes
(American Standard Channels)
Dimensions

Shape	Area, A	Depth, d		Web Thickness, t_w		Web $\frac{t_w}{2}$	Flange Width, b_f		Flange Thickness, t_f		Distance k	Distance T	Distance Work-able Gage†
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.
C15×50	14.7	15.0	15	0.716	11/16	3/8	3.72	3 3/4	0.650	5/8	1 7/16	12 1/8	2 1/4
×40	11.8	↓	↓	0.520	1/2	1/4	3.52	3 1/2	↓	↓	↓	↓	2
×33.9	9.95	↓	↓	0.400	3/8	3/16	3.40	3 3/8	↓	↓	↓	↓	2
C12×30	8.81	12.0	12	0.510	1/2	1/4	3.17	3 1/8	0.501	1/2	1 1/8	9 3/4	1 3/4
×25	7.34	↓	↓	0.387	3/8	3/16	3.05	3	↓	↓	↓	↓	↓
×20.7	6.08	↓	↓	0.282	5/16	3/16	2.94	3	↓	↓	↓	↓	↓
C10×30	8.81	10.0	10	0.673	11/16	3/8	3.03	3	0.436	7/16	1	8	1 3/4
×25	7.34	↓	↓	0.526	1/2	1/4	2.89	2 7/8	↓	↓	↓	↓	1 3/4
×20	5.87	↓	↓	0.379	3/8	3/16	2.74	2 3/4	↓	↓	↓	↓	1 1/2
×15.3	4.48	↓	↓	0.240	1/4	1/8	2.60	2 5/8	↓	↓	↓	↓	1 1/2
C9×20	5.87	9.00	9	0.448	7/16	1/4	2.65	2 5/8	0.413	7/16	1	7	1 1/2
×15	4.41	↓	↓	0.285	5/16	3/16	2.49	2 1/2	↓	↓	↓	↓	1 3/8
×13.4	3.94	↓	↓	0.233	1/4	1/8	2.43	2 3/8	↓	↓	↓	↓	1 3/8
C8×18.75	5.51	8.00	8	0.487	1/2	1/4	2.53	2 1/2	0.390	3/8	15/16	6 1/8	1 1/2
×13.75	4.04	↓	↓	0.303	5/16	3/16	2.34	2 3/8	↓	↓	↓	↓	1 3/8
×11.5	3.37	↓	↓	0.220	1/4	1/8	2.26	2 1/4	↓	↓	↓	↓	1 3/8
C7×14.75	4.33	7.00	7	0.419	7/16	1/4	2.30	2 1/4	0.366	3/8	7/8	5 1/4	1 1/4
×12.25	3.60	↓	↓	0.314	5/16	3/16	2.19	2 1/4	↓	↓	↓	↓	↓
×9.8	2.87	↓	↓	0.210	3/16	1/8	2.09	2 1/8	↓	↓	↓	↓	↓
C6×13	3.81	6.00	6	0.437	7/16	1/4	2.16	2 1/8	0.343	5/16	13/16	4 3/8	1 3/8
×10.5	3.08	↓	↓	0.314	5/16	3/16	2.03	2	↓	↓	↓	↓	1 1/8
×8.2	2.39	↓	↓	0.200	3/16	1/8	1.92	1 7/8	↓	↓	↓	↓	1 1/8
C5×9	2.64	5.00	5	0.325	5/16	3/16	1.89	1 7/8	0.320	5/16	3/4	3 1/2	1 1/8
×6.7	1.97	5.00	5	0.190	3/16	1/8	1.75	1 3/4	0.320	5/16	3/4	3 1/2	–
C4×7.25	2.13	4.00	4	0.321	5/16	3/16	1.72	1 3/4	0.296	5/16	3/4	2 1/2	1
×5.4	1.58	↓	↓	0.184	3/16	1/8	1.58	1 5/8	↓	↓	↓	↓	–
Rev. 11/1/02 ×4.5	$\boxed{1.32}$	↓	↓	0.125	1/8	1/16	1.58	1 5/8	↓	↓	↓	↓	–
C3×6	1.76	3.00	3	0.356	3/8	3/16	1.60	1 5/8	0.273	1/4	11/16	1 5/8	–
×5	1.47	↓	↓	0.258	1/4	1/8	1.50	1 1/2	↓	↓	↓	↓	–
×4.1	1.20	↓	↓	0.170	3/16	1/8	1.41	1 3/8	↓	↓	↓	↓	–
Rev. 11/1/02 ×3.5	$\boxed{1.03}$	↓	↓	0.132	1/8	1/16	1.37	1 3/8	↓	↓	↓	↓	–

† See definition of "Workable Gage" in Nomenclature section at the back of this Manual.
– in Workable Gage column indicates that flange is too narrow to allow tabulation of a workable gage.

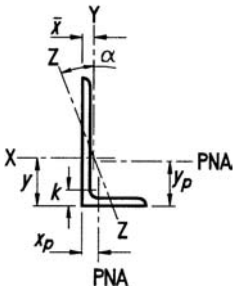
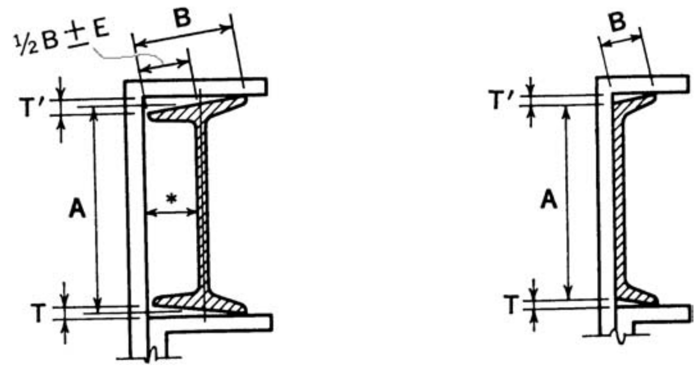


Table 1-7 (cont.). Angles (L-Shapes) Properties

Rev. 11/1/02

Shape	k	Wt.	Area, A	Axis X-X					
				I	S	r	$\bar{y}$	Z	y_p
	in.	lb/ft	in.^{2}	in.^{4}	in.^{3}	in.	in.	in.^{3}	in.
$L5\times3\frac{1}{2}\times\frac{3}{4}$	$1\frac{3}{16}$	19.8	5.82	13.9	4.26	1.55	1.74	7.60	1.12
$\times\frac{5}{8}$		16.8	4.93	12.0	3.63	1.56	1.69	6.50	1.06
$\times\frac{1}{2}$	$\frac{15}{16}$	13.6	4.00	9.96	2.97	1.58	1.65	5.33	0.997
$\times\frac{3}{8}$	$\frac{13}{16}$	10.4	3.05	7.75	2.28	1.59	1.60	4.09	0.933
$\times\frac{5}{16}$	$\frac{3}{4}$	8.72	2.56	6.58	1.92	1.60	1.57	3.45	0.901
$\times\frac{1}{4}$	$\frac{11}{16}$	7.03	2.07	5.36	1.55	1.61	1.55	2.78	0.868
$L5\times3\times\frac{1}{2}$	$\frac{15}{16}$	12.8	3.75	9.43	2.89	1.58	1.74	5.12	1.25
$\times\frac{7}{16}$	$\frac{7}{8}$	11.3	3.31	8.41	2.56	1.59	1.72	4.53	1.21
$\times\frac{3}{8}$	$\frac{13}{16}$	9.74	2.86	7.35	2.22	1.60	1.69	3.93	1.18
$\times\frac{5}{16}$	$\frac{3}{4}$	8.19	2.41	6.24	1.87	1.61	1.67	3.32	1.15
$\times\frac{1}{4}$	$\frac{11}{16}$	6.60	1.94	5.09	1.51	1.62	1.64	2.68	1.12
$L4\times4\times\frac{3}{4}$	$1\frac{1}{8}$	18.5	5.43	7.62	2.79	1.18	1.27	5.02	0.679
$\times\frac{5}{8}$	1	15.7	4.61	6.62	2.38	1.20	1.22	4.28	0.576
$\times\frac{1}{2}$	$\frac{7}{8}$	12.7	3.75	5.52	1.96	1.21	1.18	3.50	0.468
$\times\frac{7}{16}$	$\frac{13}{16}$	11.2	3.30	4.93	1.73	1.22	1.15	3.10	0.413
$\times\frac{3}{8}$	$\frac{3}{4}$	9.72	2.86	4.32	1.5	1.23	1.13	2.69	0.357
$\times\frac{5}{16}$	$\frac{11}{16}$	8.16	2.40	3.67	1.27	1.24	1.11	2.26	0.300
$\times\frac{1}{4}$	$\frac{5}{8}$	6.58	1.93	3.00	1.03	1.25	1.08	1.82	0.242
$L4\times3\frac{1}{2}\times\frac{1}{2}$	$\frac{15}{16}$	11.9	3.50	5.30	1.92	1.23	1.24	3.46	0.497
$\times\frac{3}{8}$	$\frac{13}{16}$	9.10	2.68	4.15	1.48	1.25	1.20	2.66	0.433
$\times\frac{5}{16}$	$\frac{3}{4}$	7.65	2.25	3.53	1.25	1.25	1.17	2.24	0.401
$\times\frac{1}{4}$	$\frac{11}{16}$	6.18	1.82	2.89	1.01	1.26	1.14	1.81	0.368
$L4\times3\times\frac{5}{8}$	$1\frac{1}{16}$	13.6	3.99	6.01	2.28	1.23	1.37	4.08	0.810
$\times\frac{1}{2}$	$\frac{15}{16}$	11.1	3.25	5.02	1.87	1.24	1.32	3.36	0.747
$\times\frac{3}{8}$	$\frac{13}{16}$	8.47	2.49	3.94	1.44	1.26	1.27	2.60	0.683
$\times\frac{5}{16}$	$\frac{3}{4}$	7.12	2.09	3.36	1.22	1.27	1.25	2.19	0.651
$\times\frac{1}{4}$	$\frac{11}{16}$	5.75	1.69	2.75	0.988	1.27	1.22	1.77	0.618
$L3\frac{1}{2}\times3\frac{1}{2}\times\frac{1}{2}$	$\frac{7}{8}$	11.1	3.27	3.63	1.48	1.05	1.05	2.66	0.466
$\times\frac{7}{16}$	$\frac{13}{16}$	9.82	2.89	3.25	1.32	1.06	1.03	2.36	0.412
$\times\frac{3}{8}$	$\frac{3}{4}$	8.51	2.50	2.86	1.15	1.07	1.00	2.06	0.357
$\times\frac{5}{16}$	$\frac{11}{16}$	7.16	2.10	2.44	0.969	1.08	0.979	1.74	0.301
$\times\frac{1}{4}$	$\frac{5}{8}$	5.79	1.70	2.00	0.787	1.09	0.954	1.41	0.243
$L3\frac{1}{2}\times3\times\frac{1}{2}$	$\frac{7}{8}$	10.3	3.02	3.45	1.45	1.07	1.12	2.61	0.480
$\times\frac{7}{16}$	$\frac{13}{16}$	9.09	2.67	3.10	1.29	1.08	1.09	2.32	0.446
$\times\frac{3}{8}$	$\frac{3}{4}$	7.88	2.32	2.73	1.12	1.09	1.07	2.03	0.411
$\times\frac{5}{16}$	$\frac{11}{16}$	6.65	1.95	2.33	0.951	1.09	1.05	1.72	0.375
$\times\frac{1}{4}$	$\frac{5}{8}$	5.38	1.58	1.92	0.773	1.10	1.02	1.39	0.336
$L3\frac{1}{2}\times2\frac{1}{2}\times\frac{1}{2}$	$\frac{7}{8}$	9.41	2.76	3.24	1.41	1.08	1.20	2.52	0.736
$\times\frac{3}{8}$	$\frac{3}{4}$	7.23	2.12	2.56	1.09	1.10	1.15	1.96	0.668
$\times\frac{5}{16}$	$\frac{11}{16}$	6.10	1.79	2.20	0.925	1.11	1.13	1.67	0.633
$\times\frac{1}{4}$	$\frac{5}{8}$	4.94	1.45	1.81	0.753	1.12	1.10	1.36	0.596

Table 1-55.
S-Shapes, M-Shapes, and Channels



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

Permissible Cross-Sectional Variations

Shape	Nominal, Depth, in.	*A*[a] Depth, in. Over	*A*[a] Depth, in. Under	*B* Flange width, in. Over	*B* Flange width, in. Under	*T* + *T′*[b] Flanges out of square, per in. of *B*, in.	*E* Web off Center, in.
S-shapes and M-shapes	3 to 7, incl.	$^3/_{32}$	$^1/_{16}$	$^1/_8$	$^1/_8$	$^1/_{32}$	$^3/_{16}$
	Over 7 to 14, incl.	$^1/_8$	$^3/_{32}$	$^5/_{32}$	$^5/_{32}$		
	Over 14 to 24, incl.	$^3/_{16}$	$^1/_8$	$^3/_{16}$	$^3/_{16}$		
Channels	3 to 7, incl.	$^3/_{32}$	$^1/_{16}$	$^1/_8$	$^1/_8$	$^1/_{32}$	–
	Over 7 to 14, incl.	$^1/_8$	$^3/_{32}$	$^1/_8$	$^5/_{32}$		
	Over 14	$^3/_{16}$	$^1/_8$	$^1/_8$	$^3/_{16}$		

Permissible Variations in Length

Shape	Variations Over Specified Length for Lengths Given[c], in.					
	5 to 10 ft, excl.	10 to 20 ft, excl.	20 to 30 ft, incl.	Over 30 to 40 ft, incl.	Over 40 to 65 ft, incl.	Over 65 ft
All	1	1 $^1/_2$	1 $^3/_4$	2 $^1/_4$	2 $^3/_4$	–

Mill Straightness Tolerances[d]

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Camber	$^1/_8 \text{ in.} \times \frac{\text{(total length, ft)}}{5}$
Sweep	Due to the extreme variations in flexibility of these shapes, permitted variations for sweep are subject to negotiation between the manufacturer and purchaser for the individual sections involved.

Other Permissible Rolling Variations

Area and Weight	± 2.5 percent theoretical or specified amount.
Ends Out of Square	S-shapes, M-shapes and channels $^1/_{64}$ in. per in. of depth.

– indicates that there is no requirement.
[a] *A* is measured at center line of web for beams and at back of web for channels.
[b] *T* + *T′* applies when flanges of channels are toed in or out.
[c] The permitted variation under the specified length is 0 in. for all lengths. There are no requirements for lengths over 65 ft.
[d] The tolerances herein are taken from ASTM A6 and apply to the straightness of members received from the rolling mill, measured as illustrated in Figure 1-1. For tolerance on induced camber and sweep, see Code of Standard Practice Section 6.4.4.

While still formally permitted in the LRFD Specification, the use of other material specifications in steel-to-steel structural bolting applications has become quite uncommon. ASTM A307 bolts are almost as infrequently specified today as are ASTM A501 and A502 rivets.

Twist-Off-Type Tension-Control Bolt Assemblies

As shown in Table 2-3, the preferred material specification for twist-off-type tension-control bolt assemblies is ASTM F1852, which offers a strength level that is equivalent to that of ASTM A325 bolts. When a higher strength is desired, twist-off-type tension-control bolt assemblies can be obtained in a strength level that is equivalent to that of ASTM A490 bolts using the provisions for alternative-design fasteners in RCSC Specification Section 2.8. In either case, Type 1 (medium-carbon steel) is most commonly specified. When atmospheric corrosion resistance is desired, Type 3 can be specified.

Nuts

As shown in Table 2-3, the preferred material specification for heavy-hex nuts is ASTM A563. The appropriate grade and finish is specified per ASTM A563 Table X1.1 according to the bolt or threaded part with which the nut will be used. For steel-to-steel structural bolting applications, the appropriate grade and finish is summarized in RCSC Specification Section 2.4. If its availability can be confirmed prior to specification, ASTM A194 grade 2H nuts are permitted as an alternative as indicated in RCSC Specification Table 2.1.

Washers

As shown in Table 2-3, the preferred material specification for hardened steel washers is ASTM F436. This specification provides for both flat and beveled washers. While standard ASTM F436 washers are sufficient in most applications, there are several specific applications when special washers are required. The special washer requirements in RCSC Specification Section 6 apply when oversized or slotted holes are used in the outer ply of a steel-to-steel structural joint. In anchor rod and other embedment applications, hole sizes are generally larger than those for steel-to-steel structural bolting applications (see Table 14-2 for maximum anchor-rod hole sizes). Accordingly, washers used in such applications are generally larger and may require design consideration for proper force transfer, particularly when the anchorage is subject to tension. See Table 14-2 for anchor-rod washer sizes. Rev. 11/1/02

Compressible-Washer-Type Direct-Tension Indicators

When bolted joints are specified as pretensioned or slip-critical and the direct-tension-indicator pretensioning method is used, ASTM F959 compressible-washer-type direct-tension indicators are specified, as shown in Table 2-3. Type 325 is used with ASTM A325 high-strength bolts and type 490 is used with ASTM A490 high-strength bolts.

Anchor Rods

As shown in Table 2-3, the preferred material specification for anchor rods is ASTM F1554, which covers hooked, headed and threaded and nutted anchor rods in three strength grades: 36, 55 and 105. ASTM F1554 grade 36 is most commonly specified, although grades 55 and 105 are normally available, albeit with potentially longer lead times, when higher strength is required. ASTM F1554 grade 36 or ASTM F1554 grade 55 with weldability supplement S1 and the carbon equivalent formula in ASTM F1554 Section S1.5.2.1 can be specified to allow welded field correction should the anchor rods be placed incorrectly in the field. ASTM F1554 grades 36, 55 and 105 are essentially the anchor-rod equivalents of the generic rod specifications ASTM A36, ASTM A572 grade 55 and A193 grade B7, respectively.

Table 2-10.
Construction Classification,
Restrained and Unrestrained

Wall Bearing	**Single-span and simply supported end spans of multiple bays**[a]	Open-web steel joists or steel beams, supporting concrete slab, precast units, or metal decking	unrestrained
		Concrete slabs, precast units, or metal decking	unrestrained
	Interior spans of multiple bays	Open-web steel joists, steel beams or metal decking, supporting continuous concrete slab	restrained
		Open-web steel joists or steel beams, supporting precast units or metal decking	unrestrained
		Cast-in-place concrete slab systems	restrained
		Precast concrete where the potential thermal expansion is resisted by adjacent construction[b]	restrained
Steel Framing		Steel beams welded, riveted, or bolted to the framing members	restrained
		All types of cast-in-place floor and roof systems (such as beam-and-slabs, flat slabs, pan joists, and waffle slabs) where the floor or roof system is secured to the framing members	restrained
		All types of prefabricated floor or roof systems where the structural members are secured to the framing members and the potential thermal expansion of the floor or roof system is resisted by the framing system or the adjoining floor or roof construction[b]	restrained
Concrete Framing		Beams securely fastened to the framing members	restrained
		All types of cast-in-place floor and roof systems (such as beam-and-slabs, flat slabs, pan joists, and waffle slabs) where the floor system is cast with the framing members	restrained
		Interior and exterior spans of precast systems with cast-in-place joints resulting in restraint equivalent to that which would exist in [concrete framing] [b(i)]	restrained
		All types of prefabricated floor or roof systems where the structural members are secured to such systems and the potential thermal expansion of the floor or roof systems is resisted by the framing system or the adjoining floor or roof construction[b]	restrained
Wood Construction		All types	unrestrained

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[a] Floor and roof system scan be considered restrained when they are tied into walls or without tie beams, the walls being designed and detailed to resist thermal thrust from the floor or roof system.

[b] For example, resistance to potential thermal expansion is considered to be achieved when:
- (i) Continuous structural concrete topping is used,
- (ii) The space between the ends of precast units or between the ends of units and the vertical face of supports is filled with concrete or mortar, or
- (iii) The space between the ends of precast units and the vertical faces of supports, or between the ends of solid or hollow core slab units does not exceed 0.25% of the length for normal weight concrete members of 0.1% of the length for structural light weight concrete members.

From ASTM E119-2000 Table X 3.1. Copyright ASTM. Reprinted with permission.

design strength with $A_e = 0.75A_g = 5.31$ in.^{2} is tabulated as 259 kips.

$$\begin{aligned}\phi_t P_n &= 259 \text{ kips}\left(\frac{A_e}{0.75A_g}\right)\\ &= 259 \text{ kips}\left(\frac{5.11 \text{ in.}^2}{5.31 \text{ in.}^2}\right)\\ &= 249 \text{ kips}\end{aligned}$$

Similarly, for solution b,

$$\begin{aligned}\frac{A_e}{A_g} &= \frac{5.68 \text{ in.}^2}{7.08 \text{ in.}^2}\\ &= 0.802 < 0.923\end{aligned}$$

Therefore, tension rupture controls. For tension rupture, the W8×24 design strength with $A_e = 0.75A_g = 5.31$ in.^{2} is tabulated as 259 kips.

$$\begin{aligned}\phi_t P_n &= 259 \text{ kips}\left(\frac{A_e}{0.75A_g}\right)\\ &= 259 \text{ kips}\left(\frac{5.68 \text{ in.}^2}{5.31 \text{ in.}^2}\right)\\ &= 277 \text{ kips}\end{aligned}$$

Note that end-connection limit-states, such as block shear rupture and bolt bearing strength must also be checked.

EXAMPLE 3.2. Single-angle tension member design.

Given: Determine the design strength of an ASTM A36 L4×4×1/2 with one line of 3/4-in.-diameter bolts in standard holes, two per flange, as illustrated in Figure 3–2. Assume the connection length is 18 in. Also, calculate at what length this tension member would cease to satisfy the slenderness limitation in Single-Angle Specification Section 2.

$F_y = 36$ ksi $A_g = 3.75$ in.^{2} $\boxed{r_z = 0.776 \text{ in.}}$ Rev. 11/1/02

$F_u = 58$ ksi $\bar{y} = 1.18$

Solution: For tension yielding, per Single-Angle Specification Section 2,

$$\begin{aligned}\phi_t P_n &= \phi_t F_y A_g\\ &= 0.9(36 \text{ ksi})(3.75 \text{ in.}^2)\\ &= 122 \text{ kips}\end{aligned}$$

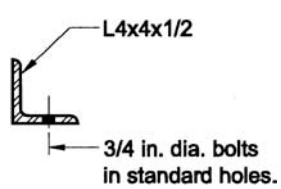

Fig. 3–2. Illustration for Example 3.2.

For tension rupture, per Single-Angle Specification Section 2,

$$
\begin{aligned}
U &= 1 - \frac{\bar{x}}{\ell} \leq 0.9 \\
&= 1 - \frac{1.18 \text{ in.}}{18 \text{ in.}} \leq 0.9 \\
&= 0.934 \leq 0.9 \\
&= 0.9 \\
A_n &= A_g - (d_h + {}^1\!/_{16} \text{ in.})t \\
&= 3.75 \text{ in.}^2 - ({}^{13}\!/_{16} \text{ in.} + {}^1\!/_{16} \text{ in.})({}^1\!/_2 \text{ in.}) \\
&= 3.31 \text{ in.}^2 \\
A_e &= UA_n \\
&= 0.9(3.31 \text{ in.}^2) \\
&= 2.98 \text{ in.}^2 \\
\phi_t P_n &= \phi_t F_u A_e \\
&= 0.75(58 \text{ ksi})(2.98 \text{ in.}^2) \\
&= 130 \text{ kips}
\end{aligned}
$$

Thus, the L4×4×$^1\!/_2$ tension member design strength is controlled by the tension yielding limit-state, where

$$\phi_t P_n = 122 \text{ kips}$$

Per Single-Angle Specification Section B2,

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$$
\begin{aligned}
L_{\max} &= 300\, r_z \\
&\boxed{\begin{aligned} &= \frac{300\,(0.776 \text{ in.})}{12 \text{ in./ft}} \\ &= 19.4 \text{ ft} \end{aligned}}
\end{aligned}
$$

Thus, the L4×4×$^1\!/_2$ tension member satisfies the slenderness requirements up to a 19.4-ft length.

Comments: The preceding calculations can be simplified using Table 3-2. If $A_e/A_g \geq$ 0.745 (see description of Table 3-2), tension yielding will control over tension rupture.

$$
\begin{aligned}
\frac{A_e}{A_g} &= \frac{2.98 \text{ in.}^2}{3.75 \text{ in.}^2} \\
&= 0.795 > 0.745
\end{aligned}
$$

Therefore, tension yielding controls over tension rupture. For tension yielding, the L4×4×$^1\!/_2$ design strength is tabulated as

$$\phi_t P_n = 122 \text{ kips}$$

Note that end-connection limit-states, such as block shear rupture and bolt bearing strength must also be checked.

Solution a: For the column segment between the roof and the floor,

$$\frac{P_u}{A_g} = \frac{250\,\text{kips}}{24.0\,\text{in.}^2}$$
$$= 10.4\,\text{ksi}$$

From Table 4-1, $\tau = 1.00$. That is, the column buckles in the elastic region, so there is no reduction in stiffness for inelasticity.

$$G_{\text{top}} = \tau \frac{\sum \left(\frac{I}{L}\right)_c}{\sum \left(\frac{I}{L}\right)_g}$$

$$\boxed{\begin{aligned} &= (1.00)\frac{\left(\dfrac{881\,\text{in.}^4}{14\,\text{ft}}\right)}{2\left(\dfrac{800\,\text{in.}^4}{35\,\text{ft}}\right)} \\ &= 1.38 \end{aligned}}$$

$$G_{\text{bottom}} = \tau \frac{\sum \left(\frac{I}{L}\right)_c}{\sum \left(\frac{I}{L}\right)_g}$$

$$\boxed{\begin{aligned} &= (1.00)\frac{2\left(\dfrac{881\,\text{in.}^4}{14\,\text{ft}}\right)}{2\left(\dfrac{1{,}360\,\text{in.}^4}{35\,\text{ft}}\right)} \\ &= 1.62 \end{aligned}}$$

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From LRFD Commentary Figure C-C2.2b, K ≈ 1.5.

$$(KL)_{y\,eq} = \frac{(KL)_x}{\dfrac{r_x}{r_y}}$$
$$= \frac{1.5(14\,\text{ft})}{2.44}$$
$$= 8.61\,\text{ft}$$

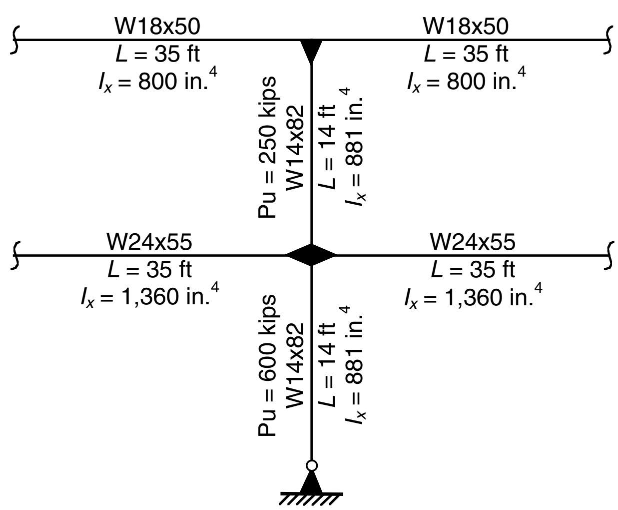

Fig. 4–1. Illustration for Example 4.2.

From Table 4-2,

$$\phi_c P_n \approx \boxed{898\,\text{kips}} > 250\,\text{kips}\ \textbf{o.k.}$$

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For the column segment between the floor and the foundation,

$$\begin{aligned}\frac{P_u}{A_g} &= \frac{600\,\text{kips}}{24.0\,\text{in.}^2}\\ &= 25.0\,\text{ksi}\end{aligned}$$

From Table 4-1, $\tau = 0.85$.

$$G_{\text{top}} = \tau\frac{\sum\left(\frac{l}{L}\right)_c}{\sum\left(\frac{l}{L}\right)_g}$$

$$\boxed{\begin{aligned}&= (0.85)\frac{2\left(\frac{881\,\text{in.}^4}{14\,\text{ft}}\right)}{2\left(\frac{1{,}360\,\text{in.}^4}{35\,\text{ft}}\right)}\\ &= 1.38\end{aligned}}$$

$$G_{\text{bottom}} = 10\,(\text{pinned end})$$

From LRFD Commentary Figure C-C2.2b, K $\approx \boxed{2.0.}$

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$$(KL)_{y\,eq} = \frac{(KL)_x}{\frac{r_x}{r_y}}$$

$$\boxed{\begin{aligned}&= \frac{2.0(14\,\text{ft})}{2.44}\\ &= 11.48\,\text{ft}\end{aligned}}$$

From Table 4-2,

$$\phi_c P_n \approx \boxed{814\,\text{kips}} > 600\,\text{kips}\ \textbf{o.k.}$$

Thus, the W14×82 compression member is adequate.

Solution b: As determined in solution a, for the column segment between the roof and the floor,

$$\phi_c P_n \approx \boxed{898\,\text{kips}}$$

As determined in solution a, for the column segment between the floor and the foundation, $G_{\text{top}} = \boxed{1.38}$ and

$$G_{\text{bottom}} = 1(\text{fixed end})$$

From LRFD Commentary Figure C-C2.2b, K $\approx \boxed{1.4.}$

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$$(KL)_{y\,eq} = \frac{(KL)_x}{\frac{r_x}{r_y}}$$

$$\begin{aligned}&= \boxed{\frac{1.4(14\,\text{ft})}{2.44}}\\ &= \boxed{8.03\,\text{ft}}\end{aligned}$$

From Table 4-2,

$$\phi_c P_n \approx \boxed{913\,\text{kips}}$$

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EXAMPLE 4.3. Single-angle compression member design.

Given: Determine the design strength of an ASTM A36 L4×3$^1/_2$×$^5/_{16}$ with physical length $L = 8$ ft, pinned ends and no bracing along the length of member. Also, calculate at what length this compression member would cease to satisfy the slenderness limitation in Single-Angle Specification Section 4.

$$F_y = 36\,\text{ksi} \qquad A_g = 2.25\,\text{in.}^2$$
$$F_u = 58\,\text{ksi} \qquad r_z = 0.721\,\text{in.}$$

Solution: For L4×3$^1/_2$×$^5/_{16}$,

$$\begin{aligned}\frac{b}{t} &= \frac{4\,\text{in.}}{^5/_{16}\,\text{in.}}\\ &= 12.8\end{aligned}$$

From Single-Angle Specification Section 4,

$$\begin{aligned}\lambda_p &= 0.446\sqrt{\frac{E}{F_y}}\\ &= 0.446\sqrt{\frac{29{,}000\,\text{ksi}}{36\,\text{ksi}}}\\ &= 12.7\end{aligned}$$

$$\begin{aligned}\lambda_r &= 0.910\sqrt{\frac{E}{F_y}}\\ &= 0.910\sqrt{\frac{29{,}000\,\text{ksi}}{36\,\text{ksi}}}\\ &= 25.8\end{aligned}$$

Since $\lambda_p < b/t < \lambda_r$,

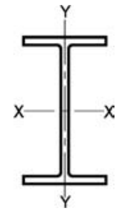
$$\begin{aligned}Q &= 1.34 - 0.761\frac{b}{t}\sqrt{\frac{F_y}{E}}\\ &= 1.34 - 0.761(12.8)\sqrt{\frac{36\,\text{ksi}}{29{,}000\,\text{ksi}}}\\ &= 0.997\end{aligned}$$

From LRFD Commentary Table C-C2.1, $K = 1.0$. From Single-Angle Specification Section 4, using $r = r_z$ (the least radius of gyration for the cross-section),

$$\begin{aligned}\lambda_c &= \frac{Kl}{r\pi}\sqrt{\frac{F_y}{E}}\\ &= \frac{1.0(8\,\text{ft} \times 12\,\text{in./ft})}{(0.721\,\text{in.})\pi}\sqrt{\frac{36\,\text{ksi}}{29{,}000\,\text{ksi}}}\\ &= 1.49\end{aligned}$$

$F_y = 50$ ksi
$\phi_c P_n = 0.85 F_{cr} A_g$

Table 4-2. W-Shapes Design Strength in Axial Compression, $\phi_c P_n$, kips



Shape	W14×										
	808*	730*	665*	605*	550*	500*	455*	426*	398*	370*	342*
Effective length KL (ft) with respect to least radius of gyration r_y											
0	10100	9140	8330	7570	6890	6250	5700	5310	4970	4630	4290
11	9540	8620	7850	7110	6460	5850	5330	4970	4640	4320	4000
12	9440	8530	7760	7030	6390	5780	5260	4900	4580	4260	3950
13	9330	8430	7660	6940	6300	5710	5190	4830	4520	4200	3890
14	9220	8320	7560	6850	6220	5620	5110	4760	4450	4140	3830
15	9100	8200	7450	6750	6120	5540	5030	4680	4380	4070	3760
16	8970	8080	7340	6640	6020	5450	4950	4600	4300	4000	3690
17	8840	7960	7220	6530	5920	5350	4860	4520	4220	3920	3620
18	8700	7820	7100	6420	5810	5250	4770	4430	4140	3840	3550
19	8550	7690	6970	6300	5700	5150	4670	4340	4050	3760	3470
20	8400	7550	6840	6170	5590	5040	4570	4250	3960	3680	3400
22	8090	7250	6560	5910	5350	4820	4370	4050	3780	3500	3230
24	7760	6940	6270	5640	5100	4590	4150	3850	3590	3320	3060
26	7410	6610	5970	5360	4840	4350	3930	3640	3390	3140	2890
28	7060	6280	5660	5080	4570	4100	3700	3430	3190	2950	2710
30	6700	5940	5340	4790	4300	3850	3480	3210	2990	2750	2530
32	6330	5600	5030	4490	4030	3610	3250	3000	2780	2560	2360
34	5970	5250	4710	4200	3760	3360	3020	2780	2580	2380	2180
36	5600	4910	4400	3910	3500	3120	2800	2570	2390	2190	2010
38	5240	4580	4090	3630	3240	2880	2580	2370	2190	2010	1840
40	4880	4250	3780	3350	2990	2650	2370	2170	2010	1840	1680
42	4530	3930	3490	3080	2740	2420	2160	1980	1830	1670	1530
44	4190	3620	3200	2820	2500	2210	1970	1800	1660	1520	1390
46	3860	3310	2930	2580	2290	2020	1800	1650	1520	1390	1270
48	3540	3040	2690	2370	2100	1860	1650	1510	1400	1280	1170
50	3260	2800	2480	2180	1940	1710	1520	1400	1290	1180	1080
Properties											
P_{wo}, kips	5350	4230	3620	3090	2630	2240	1920	1710	1520	1350	1180
P_{wi}, kips/in.	187	154	142	130	119	110	101	94.0	88.5	83.0	77.0
P_{wb}, kips	119000	66100	51700	40100	30800	23900	18800	15100	12700	10400	8320
P_{fb}, kips	7370	6780	5750	4870	4100	3450	2900	2600	2280	1990	1720
L_p, ft	17.0	16.6	16.3	16.1	15.9	15.6	15.5	15.3	15.2	15.1	15.0
L_r, ft	268	242	222	203	188	171	158	147	139	130	121
A_g, in.^{2}	237	215	196	178	162	147	134	125	117	109	101
I_x, in.^{4}	16000	14300	12400	10800	9430	8210	7190	6600	6000	5440	4900
I_y, in.^{4}	5510	4720	4170	3680	3250	2880	2560	2360	2170	1990	1810
r_y, in.	4.82	4.69	4.62	4.55	4.49	4.43	4.38	4.34	4.31	4.27	4.24
Ratio r_x/r_y	1.70	1.74	1.73	1.71	1.70	1.69	1.67	1.67	1.66	1.66	1.65
$P_{ex}(KL)^2/10^4$	458000	409000	355000	309000	270000	235000	206000	189000	172000	156000	140000
$P_{ey}(KL)^2/10^4$	158000	135000	119000	105000	93000	82400	73300	67500	62100	57000	51800

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*ASTM A6 tensile group 4 or 5 shape. Special requirements may apply per LRFD Specification Section A3.1c.

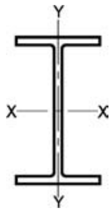

Table 4-2 (cont.). W-Shapes Design Strength in Axial Compression, $\phi_c P_n$, kips

$F_y = 50$ ksi
$\phi_c P_n = 0.85 F_{cr} A_g$

Shape	W14×									
	311*	**283***	**257***	**233***	**211**	**193**	**176**	**159**	**145**	**132**
Effective length KL (ft) with respect to least radius of gyration r_y										
0	3880	3540	3210	2910	2640	2410	2200	1980	1810	1650
11	3610	3290	2980	2700	2440	2230	2030	1830	1670	1510
12	3560	3240	2940	2660	2400	2200	2000	1810	1650	1480
13	3510	3200	2890	2620	2370	2170	1970	1780	1620	1450
14	3460	3140	2850	2570	2330	2130	1940	1740	1590	1430
15	3400	3090	2800	2530	2280	2090	1900	1710	1560	1390
16	3330	3030	2740	2480	2240	2050	1860	1680	1530	1360
17	3270	2970	2690	2430	2190	2010	1820	1640	1500	1330
18	3200	2910	2630	2380	2140	1960	1780	1600	1460	1300
19	3130	2850	2570	2320	2090	1910	1740	1570	1430	1260
20	3060	2780	2510	2270	2040	1870	1700	1530	1390	1220
22	2910	2640	2380	2150	1940	1770	1610	1440	1320	1150
24	2750	2500	2250	2030	1830	1670	1510	1360	1240	1070
26	2590	2350	2120	1910	1710	1560	1420	1270	1160	997
28	2430	2200	1980	1780	1600	1460	1320	1180	1080	920
30	2270	2050	1840	1660	1490	1350	1220	1100	998	844
32	2110	1900	1710	1530	1370	1250	1130	1010	919	769
34	1950	1760	1570	1410	1260	1150	1040	928	842	697
36	1790	1620	1440	1290	1160	1050	946	846	767	627
38	1640	1480	1320	1180	1050	955	859	767	694	563
40	1490	1340	1190	1070	951	863	775	692	626	508
42	1350	1220	1080	967	863	783	703	628	568	461
44	1230	1110	987	881	786	713	641	572	518	420
46	1130	1010	903	806	719	652	586	523	474	384
48	1040	932	829	741	660	599	538	481	435	353
50	956	858	764	682	609	552	496	443	401	325
Properties										
P_{wo}, kips	1010	861	735	621	529	454	396	333	287	263
P_{wi}, kips/in.	70.5	64.5	59.0	53.5	49.0	44.5	41.5	37.3	34.0	32.3
P_{wb}, kips	6390	4900	3730	2780	2150	1610	1310	944	716	611
P_{fb}, kips	1440	1210	1000	832	684	583	483	398	334	298
L_p, ft	14.8	14.7	14.6	14.5	14.4	14.3	14.2	14.1	14.1	13.3
L_r, ft	110	100	91.6	83.4	76	70.1	64.5	58.9	54.7	49.6
A_g, in.^{2}	91.4	83.3	75.6	68.5	62	56.8	51.8	46.7	42.7	38.8
I_x, in.^{4}	4330	3840	3400	3010	2660	2400	2140	1900	1710	1530
I_y, in.^{4}	1610	1440	1290	1150	1030	931	838	748	677	548
r_y, in.	4.20	4.17	4.13	4.10	4.07	4.05	4.02	4.00	3.98	3.76
Ratio r_x/r_y	1.64	1.63	1.62	1.62	1.61	1.60	1.60	1.60	1.59	1.67
$P_{ex}(KL)^2/10^4$	124000	110000	97300	86200	76100	68700	61300	54400	48900	43800
$P_{ey}(KL)^2/10^4$	46100	41200	36900	32900	29500	26600	24000	21400	19400	15700

*ASTM A6 tensile group 4 or 5 shape. Special requirements may apply per LRFD Specification Section A3.1c.

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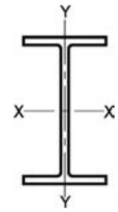
$F_y = 50$ ksi

$\phi_c P_n = 0.85 F_{cr} A_g$

Table 4-2 (cont.). W-Shapes Design Strength in Axial Compression, $\phi_c P_n$, kips



Shape		W14×										
		120	109	99††	90††	82	74	68	61	53	48	43††
Effective length KL (ft) with respect to least radius of gyration r_y	0	1500	1360	1240	1130	1020	927	850	761	663	599	530
	6	1460	1320	1200	1100	959	871	798	714	598	540	482
	7	1450	1310	1190	1080	938	852	781	698	576	520	463
	8	1430	1300	1180	1070	914	830	760	680	552	498	443
	9	1410	1280	1160	1060	888	807	738	660	526	474	422
	10	1390	1260	1150	1040	860	781	714	638	498	449	399
	11	1370	1240	1130	1030	829	753	689	615	469	423	375
	12	1350	1220	1110	1010	797	724	662	591	439	395	350
	13	1320	1200	1090	989	764	694	633	566	409	368	325
	14	1290	1170	1060	969	729	662	604	539	379	340	301
	15	1270	1150	1040	947	694	630	575	513	349	313	276
	16	1240	1120	1020	925	658	598	544	486	319	286	252
	17	1210	1090	991	902	622	565	514	458	290	260	228
	18	1180	1060	965	878	586	532	484	431	263	235	206
	19	1140	1030	938	853	550	499	454	404	236	211	185
	20	1110	1000	911	828	514	467	424	377	213	191	167
	22	1040	943	854	776	445	405	366	325	176	157	138
	24	972	879	796	723	380	345	311	276	148	132	116
	26	902	815	737	670	324	294	265	236	126	113	98.7
	28	832	751	679	616	279	253	229	203	109	97.2	85.1
	30	762	688	621	564	243	221	199	177	94.7	84.7	74.1
	32	694	627	565	512	214	194	175	155	83.2		
	34	628	567	511	463	189	172	155	138			
	36	565	509	458	415	169	153	138	123			
	38	507	457	411	372	151	138	124	110			
	40	457	412	371	336	137	124	112	99.5			
Properties												
P_{wo}, kips		227	192	167	144	185	155	136	116	116	101	85.4
P_{wi}, kips/in.		29.5	26.3	24.3	22.0	25.5	22.5	20.8	18.8	18.5	17.0	15.3
P_{wb}, kips		469	330	260	194	302	207	163	120	115	89.5	64.7
P_{fb}, kips		249	208	171	142	206	173	146	117	123	99.6	79.0
L_p, ft		13.2	13.2	13.5	15.1	8.76	8.76	8.69	8.65	6.78	6.75	6.68
L_r, ft		46.2	43.2	40.6	38.4	29.5	27.9	26.4	24.9	20.1	19.2	18.3
A_g, in.^{2}		35.3	32.0	29.1	26.5	24.0	21.8	20.0	17.9	15.6	14.1	12.6
I_x, in.^{4}		1380	1240	1110	999	881	795	722	640	541	484	428
I_y, in.^{4}		495	447	402	362	148	134	121	107	57.7	51.4	45.2
r_y, in.		3.74	3.73	3.71	3.70	2.48	2.48	2.46	2.45	1.92	1.91	1.89
Ratio r_x/r_y		1.67	1.67	1.66	1.66	2.44	2.44	2.44	2.44	3.07	3.06	3.08
$P_{ex}(KL)^2/10^4$		39500	35500	31800	28600	25200	22800	20700	18300	15500	13900	12300
$P_{ey}(KL)^2/10^4$		14200	12800	11500	10400	4240	3840	3460	3060	1650	1470	1290

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††For W14×99 and W14×90, flange is non-compact. For W14×43, web may be non-compact for combined axial compression and flexure; see AISC LRFD Specification Section B5.

Note: Heavy line indicates Kl/r equal to or greater than 200.

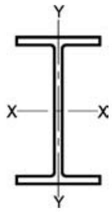

Table 4-2 (cont.). W-Shapes Design Strength in Axial Compression, $\phi_c P_n$, kips

$F_y = 50$ ksi
$\phi_c P_n = 0.85 F_{cr} A_g$

Shape	W12× 336	305	279	252	230	210	190	170	152	136	120
Effective length KL (ft) with respect to least radius of gyration r_y											
0	4200	3810	3480	3150	2880	2630	2370	2130	1900	1700	1500
6	4070	3690	3370	3040	2780	2540	2290	2050	1830	1630	1440
7	4020	3640	3330	3000	2740	2500	2260	2020	1810	1610	1420
8	3970	3590	3280	2960	2710	2470	2220	1990	1780	1590	1400
9	3910	3540	3230	2910	2660	2430	2190	1960	1750	1560	1380
10	3850	3480	3170	2860	2610	2380	2150	1920	1710	1530	1350
11	3780	3420	3110	2810	2560	2330	2100	1880	1680	1490	1320
12	3700	3350	3050	2750	2510	2280	2050	1840	1640	1460	1290
13	3620	3270	2980	2680	2450	2230	2000	1790	1590	1420	1250
14	3540	3190	2910	2610	2380	2170	1950	1740	1550	1380	1220
15	3450	3110	2830	2540	2320	2110	1900	1690	1510	1340	1180
16	3360	3020	2750	2470	2250	2040	1840	1640	1460	1290	1140
17	3260	2940	2670	2390	2180	1980	1780	1580	1410	1250	1100
18	3160	2840	2580	2320	2110	1910	1720	1530	1360	1210	1060
19	3060	2750	2500	2240	2030	1840	1650	1470	1310	1160	1020
20	2960	2660	2410	2160	1960	1780	1590	1420	1260	1110	976
22	2750	2460	2230	1990	1810	1640	1460	1300	1150	1020	892
24	2540	2270	2050	1830	1650	1490	1340	1180	1050	924	808
26	2330	2070	1870	1660	1500	1360	1210	1070	944	831	726
28	2120	1880	1690	1500	1350	1220	1090	959	844	742	646
30	1910	1690	1520	1340	1210	1090	967	852	749	656	569
32	1720	1510	1350	1190	1070	962	853	750	658	577	500
34	1520	1340	1200	1060	951	852	755	664	583	511	443
36	1360	1200	1070	944	848	760	674	593	520	456	395
38	1220	1080	960	847	761	682	605	532	467	409	355
40	1100	970	866	764	687	616	546	480	421	369	320
Properties											
P_{wo}, kips	1580	1340	1170	998	861	738	617	518	435	365	302
P_{wi}, kips/in.	89.0	81.5	76.5	70.0	64.5	59.0	53.0	48.0	43.5	39.5	35.5
P_{wb}, kips	15100	11600	9590	7320	5730	4400	3190	2370	1760	1320	957
P_{fb}, kips	2460	2070	1720	1420	1210	1020	852	684	551	439	347
L_p, ft	12.3	12.1	11.9	11.8	11.7	11.6	11.5	11.4	11.3	11.2	11.1
L_r, ft	131	119	110	99.7	91.9	84.2	76.6	68.9	62.1	55.7	50.0
A_g, in.^{2}	98.8	89.6	81.9	74.0	67.7	61.8	55.8	50.0	44.7	39.9	35.3
I_x, in.^{4}	4060	3550	3110	2720	2420	2140	1890	1650	1430	1240	1070
I_y, in.^{4}	1190	1050	937	828	742	664	589	517	454	398	345
r_y, in.	3.47	3.42	3.38	3.34	3.31	3.28	3.25	3.22	3.19	3.16	3.13
Ratio r_x/r_y	1.85	1.84	1.82	1.81	1.80	1.80	1.79	1.78	1.77	1.77	1.76
$\boxed{P_{ex}(KL)^2/10^4}$	116000	102000	89000	77900	69300	61300	54100	47200	40900	35500	30600
$\boxed{P_{ey}(KL)^2/10^4}$	34100	30100	26800	23700	21200	19000	16900	14800	13000	11400	9870

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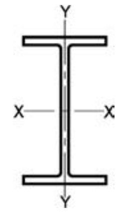
$F_y = 50$ ksi
$\phi_c P_n = 0.85 F_{cr} A_g$

Table 4-2 (cont.). W-Shapes Design Strength in Axial Compression, $\phi_c P_n$, kips



Shape		W12×										
		106	**96**	**87**	**79**	**72**	**65††**	**58**	**53**	**50**	**45**	**40**
Effective length *KL* (ft) with respect to least radius of gyration r_y	**0**	1330	1200	1090	986	897	812	723	663	621	557	497
	6	1280	1150	1050	947	861	779	680	623	562	504	450
	7	1260	1140	1030	933	848	767	666	610	543	486	434
	8	1240	1120	1010	917	834	754	649	594	521	466	416
	9	1210	1100	994	900	818	739	631	577	497	445	396
	10	1190	1070	973	880	800	723	611	559	472	422	376
	11	1160	1050	950	860	781	706	590	539	445	398	354
	12	1130	1020	926	838	761	687	568	518	418	374	332
	13	1100	995	901	814	740	668	545	496	390	349	310
	14	1070	966	874	790	717	647	521	474	363	324	287
	15	1040	935	846	764	694	626	496	451	335	299	265
	16	1000	904	817	738	670	604	471	428	308	274	243
	17	968	871	788	711	645	581	446	404	281	250	222
	18	932	838	758	683	620	558	420	381	255	227	201
	19	895	805	727	655	594	535	395	357	230	204	181
	20	858	771	696	627	569	512	370	334	208	185	163
	22	783	703	634	570	517	464	322	290	172	152	135
	24	708	635	572	514	465	417	276	247	144	128	113
	26	635	569	511	459	415	372	235	210	123	109	96.5
	28	565	505	453	406	367	328	202	181	106	94.1	83.2
	30	497	443	397	355	321	287	176	158	92.3	82.0	72.5
	32	437	390	349	312	282	252	155	139	81.2	72.1	63.7
	34	387	345	309	277	250	223	137	123			
	36	345	308	276	247	223	199	122	110			
	38	310	276	248	221	200	179	110	98.4			
	40	279	249	223	200	181	161	99.2	88.9			
Properties												
P_{wo}, kips		242	206	182	156	137	117	112	101	105	90.5	75.2
P_{wi}, kips/in.		30.5	27.5	25.8	23.5	21.5	19.5	18.0	17.3	18.5	16.8	14.8
P_{wb}, kips		609	445	365	278	213	159	125	110	133	98.6	67.4
P_{fb}, kips		276	228	185	152	126	103	115	93.0	115	93.0	74.6
L_p, ft		11.0	10.9	10.8	10.8	10.7	11.9	8.87	8.76	6.92	6.89	6.85
L_r, ft		44.9	41.4	38.4	35.7	33.6	31.7	27.0	25.6	21.5	20.3	19.2
A_g, in.^{2}		31.2	28.2	25.6	23.2	21.1	19.1	17.0	15.6	14.6	13.1	11.7
I_x, in.^{4}		933	833	740	662	597	533	475	425	391	348	307
I_y, in.^{4}		301	270	241	216	195	174	107	95.8	56.3	50.0	44.1
r_y, in.		3.11	3.09	3.07	3.05	3.04	3.02	2.51	2.48	1.96	1.95	1.94
Ratio r_x/r_y		1.76	1.76	1.75	1.75	1.75	1.75	2.10	2.11	2.64	2.64	2.64
$\boxed{P_{ex}(KL)^2/10^4}$		26700	23800	21200	18900	17100	15300	13600	12200	11200	9960	8790
$\boxed{P_{ey}(KL)^2/10^4}$		8620	7730	6900	6180	5580	4980	3060	2740	1610	1430	1260

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††Flange is non-compact.
Note: Heavy line indicates Kl/r equal to or greater than 200.

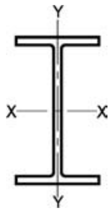

Table 4-2 (cont.). W-Shapes Design Strength in Axial Compression, $\phi_c P_n$, kips

$F_y = 50$ ksi

$\phi_c P_n = 0.85 F_{cr} A_g$

Shape	W10×										
Effective length KL (ft) with respect to least radius of gyration r_y	112	100	88	77	68	60	54	49	45	39	33
0	1400	1250	1100	961	850	748	672	612	565	489	413
6	1330	1180	1040	908	803	706	634	577	515	444	373
7	1300	1160	1020	890	787	692	621	565	497	428	360
8	1270	1140	999	869	769	675	606	551	478	412	345
9	1240	1110	973	847	749	657	590	536	458	393	329
10	1210	1080	945	822	727	638	572	520	436	374	312
11	1170	1040	916	796	703	617	553	502	412	353	294
12	1130	1010	884	768	678	595	533	484	388	332	276
13	1090	970	851	738	652	571	512	464	364	310	257
14	1050	931	817	708	625	547	490	444	339	289	238
15	1010	892	782	677	597	523	468	424	314	267	220
16	961	851	746	645	569	497	445	403	290	246	202
17	915	810	709	612	540	472	422	382	266	225	184
18	870	769	672	580	511	446	399	361	243	205	167
19	824	727	635	547	482	421	376	340	221	185	150
20	778	686	599	515	454	395	353	319	199	167	135
22	688	605	527	452	398	346	309	278	164	138	112
24	601	527	458	392	344	299	266	239	138	116	94.0
26	518	453	393	335	294	255	227	204	118	98.8	80.1
28	447	390	339	289	254	220	196	176	102	85.2	69.1
30	389	340	295	252	221	191	170	153	88.5	74.2	60.2
32	342	299	259	221	194	168	150	134	77.7	65.2	52.9
34	303	265	230	196	172	149	133	119			
36	270	236	205	175	153	133	118	106			
38	242	212	184	157	138	119	106	95.3			
40	219	191	166	141	124	108	95.9	86.0			
Properties											
P_{wo}, kips	330	275	225	182	149	124	104	90.1	98	81.1	67.8
P_{wi}, kips/in.	37.8	34.0	30.3	26.5	23.5	21.0	18.5	17.0	17.5	15.8	14.5
P_{wb}, kips	1430	1040	732	494	344	245	168	130	142	103	80.7
P_{fb}, kips	439	353	276	213	167	130	106	88.2	108	79.0	53.2
L_p, ft	9.47	9.36	9.29	9.18	9.15	9.08	9.04	8.97	7.10	6.99	6.85
L_r, ft	56.5	50.8	45.1	39.9	36.0	32.6	30.2	28.3	24.1	21.9	19.8
A_g, in.^{2}	32.9	29.4	25.9	22.6	20.0	17.6	15.8	14.4	13.3	11.5	9.71
I_x, in.^{4}	716	623	534	455	394	341	303	272	248	209	171
I_y, in.^{4}	236	207	179	154	134	116	103	93.4	53.4	45.0	36.6
r_y, in.	2.68	2.65	2.63	2.60	2.59	2.57	2.56	2.54	2.01	1.98	1.94
Ratio r_x/r_y	1.74	1.74	1.73	1.73	1.71	1.71	1.71	1.71	2.15	2.16	2.16
$\boxed{P_{ex}(KL)^2/10^4}$	20500	17800	15300	13000	11300	9760	8670	7790	7100	5980	4890
$\boxed{P_{ey}(KL)^2/10^4}$	6750	5920	5120	4410	3840	3320	2950	2670	1530	1290	1050

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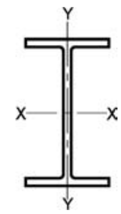
Note: Heavy line indicates Kl/r equal to or greater than 200.

$F_y = 36$ ksi
$\phi_c P_n = 0.85 F_{cr} A_g$

Table 4-3. HP-Shapes Design Strength in Axial Compression, $\phi_c P_n$, kips



Shape	HP14×				HP12×				HP10×		HP8×
Wt/ft	117	102	89††	73††	84	74	63††	53††	57	42††	36
Effective length KL (ft) with respect to least radius of gyration r_y											
0	1050	918	799	655	753	667	563	474	514	379	324
6	1030	898	781	640	729	646	545	459	491	362	302
7	1020	891	775	635	721	639	538	453	483	356	294
8	1010	884	768	629	712	630	531	447	474	349	286
9	1000	875	760	623	701	621	523	440	464	341	276
10	993	865	752	615	690	610	514	432	453	333	266
11	980	854	742	607	677	599	504	424	441	324	255
12	967	842	732	599	663	587	494	415	429	314	243
13	953	830	721	589	649	574	482	406	415	304	232
14	938	816	709	580	634	560	471	396	401	294	219
15	922	802	696	569	618	546	458	385	387	283	207
16	905	788	683	558	601	531	446	374	372	272	195
17	888	772	670	547	584	516	432	363	357	260	182
18	870	756	656	535	567	500	419	351	341	249	170
19	851	740	641	523	548	484	405	339	326	237	158
20	832	723	626	511	530	467	391	327	310	225	146
22	792	687	595	485	492	434	362	303	279	202	123
24	750	650	563	458	454	400	333	278	248	179	104
26	707	613	529	430	416	366	304	253	219	157	88.3
28	664	574	496	402	378	332	275	229	191	136	76.2
30	620	536	462	374	342	300	247	206	166	119	66.4
32	576	498	428	346	307	268	221	183	146	104	58.3
34	533	460	395	319	273	238	196	163	129	92.3	
36	491	423	363	292	243	213	174	145	115	82.3	
38	450	387	332	267	218	191	157	130	103	73.9	
40	411	352	301	241	197	172	141	117	93.4	66.7	
Properties											
P_{wo}, kips	217	174	145	108	170	143	116	88.1	127	84.0	90.1
P_{wi}, kips/in.	29.0	25.4	22.1	18.2	24.7	21.8	18.5	15.7	20.3	14.9	16.0
P_{wb}, kips	1010	677	451	249	729	502	310	187	507	201	308
P_{fb}, kips	131	101	76.6	51.6	95.0	75.4	53.7	38.3	64.6	35.7	40.1
L_p, ft	14.9	14.8	17.2	21.4	12.2	12.2	13.9	16.7	10.2	11.9	8.12
L_r, ft	66.1	58.9	53.0	46.7	54.0	48.9	43.1	38.8	45.6	36.0	35.7
A_g, in.^{2}	34.4	30.0	26.1	21.4	24.6	21.8	18.4	15.5	16.8	12.4	10.6
I_x, in.^{4}	1220	1050	904	729	650	569	472	393	294	210	119
I_y, in.^{4}	443	380	326	261	213	186	153	127	101	71.7	40.3
r_y, in.	3.59	3.56	3.53	3.49	2.94	2.92	2.88	2.86	2.45	2.41	1.95
Ratio r_x/r_y	1.66	1.66	1.67	1.67	1.75	1.75	1.76	1.76	1.71	1.71	1.72
$P_{ex}(KL)^2/10^4$	34900	30100	25900	20900	18600	16300	13500	11200	8410	6010	3410
$P_{ey}(KL)^2/10^4$	12700	10900	9330	7470	6100	5320	4380	3630	2890	2050	1150

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††Flange is non-compact.
Note: Heavy line indicates Kl/r equal to or greater than 200.

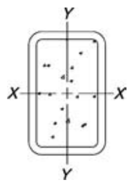

Table 4-12 (cont.). Composite Rectangular (and Square) HSS Design Strength in Axial Compression, $\phi_c P_n$, kips

$F_y = 46$ ksi
$f'_c = 4$ ksi
$\phi_c = 0.85$

Shape		HSS12×12×				HSS12×10×			HSS12×8×			
		5/8	1/2	3/8	5/16	1/2	3/8	5/16	5/8	1/2	3/8	5/16
t_{design}, in.		**0.581**	**0.465**	**0.349**	**0.291**	**0.465**	**0.349**	**0.291**	**0.581**	**0.465**	**0.349**	**0.291**
Wt/ft		**93.1**	**75.9**	**58.0**	**48.8**	**69.1**	**52.9**	**44.6**	**76.1**	**62.3**	**47.8**	**40.3**
Effective length KL (ft) with respect to least radius of gyration, r_{my}	**0**	1340	1170	994	901	1030	874	788	1030	898	754	679
	6	1320	1150	977	884	1010	853	768	996	865	727	654
	7	1310	1140	971	879	998	846	761	983	854	718	645
	8	1300	1130	963	872	988	837	754	968	841	707	636
	9	1290	1120	955	865	977	827	745	950	826	695	625
	10	1280	1110	946	857	964	816	735	932	810	681	613
	11	1270	1100	936	848	950	805	725	912	793	667	600
	12	1250	1090	926	838	936	792	713	890	774	651	586
	13	1240	1080	914	827	920	779	701	867	755	635	571
	14	1220	1060	902	816	903	764	688	843	734	618	555
	15	1200	1050	889	805	885	749	675	818	712	600	539
	16	1180	1030	876	792	867	734	660	792	690	581	522
	17	1160	1010	861	779	847	717	646	765	667	562	505
	18	1140	998	846	766	827	700	630	737	643	542	487
	19	1120	979	831	752	806	683	614	709	619	522	469
	20	1100	961	815	737	785	665	598	681	595	502	451
	21	1080	942	799	722	763	646	581	652	570	482	432
	22	1060	922	782	707	741	628	565	624	546	461	414
	23	1030	901	764	691	719	609	547	595	521	440	395
	24	1010	881	747	675	696	589	530	567	496	420	376
	25	984	860	729	659	673	570	512	538	472	399	358
	26	960	838	711	642	650	550	495	510	448	379	340
	27	935	817	692	625	627	531	477	483	424	359	322
	28	910	795	673	608	604	511	459	456	400	340	304
	29	884	773	655	591	581	491	442	430	378	320	287
	30	859	750	636	574	558	472	424	404	355	302	270
	34	756	661	560	505	468	396	356	314	277	235	211
	38	655	573	485	437	383	324	291	252	222	188	169
	42	559	490	414	372	314	266	238	206	181	154	138
	46	468	411	347	312	262	221	199	172	151	129	115
	50	397	348	294	264	221	187	168	145	128	109	97.4
Properties												
r_{my}, in.		4.62	4.68	4.73	4.76	3.96	4.01	4.04	3.16	3.21	3.27	3.29
r_{mx}/r_{my}		1.00	1.00	1.00	1.00	1.15	1.15	1.15	1.37	1.37	1.37	1.37
$\phi_b M_{nx}$, kip-ft		376	309	239	202	272	211	178	283	235	183	155
$\phi_b M_{ny}$, kip-ft		376	309	239	202	240	186	158	214	178	138	118
$P_{ex}(K_x L_x)^2/10^4$		128	113	95.1	86.0	95.1	80.2	72.4	89.0	77.9	65.5	58.9
$P_{ey}(K_y L_y)^2/10^4$		128	113	95.1	86.0	71.7	60.6	54.8	47.1	41.6	35.0	31.5

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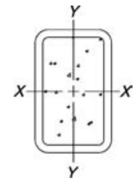
$F_y = 46$ ksi
$f'_c = 4$ ksi
$\phi_c = 0.85$

Table 4-12 (cont.). Composite Rectangular (and Square) HSS Design Strength in Axial Compression, $\phi_c P_n$, kips



Shape		HSS12×6×				HSS10×10×				
		5/8	1/2	3/8	5/16	5/8	1/2	3/8	5/16	1/4
t_{design}, in.		0.581	0.465	0.349	0.291	0.581	0.465	0.349	0.291	0.233
Wt/ft		67.6	55.5	42.7	36.0	76.1	62.3	47.8	40.3	32.6
Effective length KL (ft) with respect to least radius of gyration, r_{my}	0	882	760	634	566	1050	910	766	690	613
	6	826	713	596	533	1020	887	747	673	597
	7	807	697	583	521	1010	878	740	666	592
	8	786	679	568	508	999	869	732	659	585
	9	762	659	551	493	986	859	723	651	578
	10	736	637	534	477	973	847	713	642	570
	11	709	614	515	460	958	834	703	633	562
	12	680	590	495	443	942	821	691	622	552
	13	650	565	474	424	925	806	679	611	542
	14	619	538	452	405	907	791	666	600	532
	15	588	512	430	385	889	775	653	587	521
	16	556	485	408	365	869	758	638	574	509
	17	524	457	385	345	848	740	624	561	497
	18	492	430	363	325	827	722	608	547	485
	19	460	403	340	305	805	703	592	533	472
	20	429	376	318	285	783	683	576	518	459
	21	398	350	296	266	760	664	560	503	446
	22	368	325	275	247	736	644	543	488	432
	23	339	300	254	229	713	623	526	472	418
	24	312	276	234	211	689	603	508	457	404
	25	287	254	216	194	665	582	491	441	390
	26	266	235	200	180	641	561	474	425	376
	27	246	218	185	166	617	540	456	409	362
	28	229	202	172	155	592	519	439	393	348
	29	213	189	160	144	568	499	421	378	334
	30	199	176	150	135	545	478	404	362	320
	34	155	137	117	105	452	398	337	301	266
	38	124	110	93.4	84.0	367	323	274	245	216
	42				68.8	300	265	224	200	177
	46					250	221	187	167	147
	50					212	187	158	141	125
Properties										
r_{my}, in.		2.39	2.44	2.49	2.52	3.80	3.86	3.92	3.94	3.97
r_{mx}/r_{my}		1.73	1.73	1.72	1.71	1.00	1.00	1.00	1.00	1.00
$\phi_b M_{nx}$, kip-ft		237	198	155	131	253	209	163	138	113
$\phi_b M_{ny}$, kip-ft		145	121	95.6	81.4	253	209	163	138	113
$P_{ex}(K_x L_x)^2/10^4$		69.8	61.1	51.3	45.9	68.7	60.4	51.1	45.8	40.4
$P_{ey}(K_y L_y)^2/10^4$		23.3	20.5	17.4	15.7	68.7	60.4	51.1	45.8	40.4

Note: Heavy line indicates Kl/r equal to or greater than 200.

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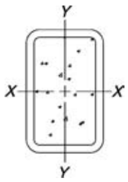

Table 4-12 (cont.). Composite Rectangular (and Square) HSS Design Strength in Axial Compression, $\phi_c P_n$, kips

$F_y = 46$ ksi
$f'_c = 4$ ksi
$\phi_c = 0.85$

Shape	HSS10×8×				HSS10×6×					HSS10×5×		
	1/2	3/8	5/16	1/4	5/8	1/2	3/8	5/16	1/4	3/8	5/16	1/4
t_{design}, in.	0.465	0.349	0.291	0.233	0.581	0.465	0.349	0.291	0.233	0.349	0.291	0.233
Wt/ft	55.5	42.7	36.0	29.2	59.1	48.7	37.6	31.8	25.8	35.1	29.7	24.1
Effective length *KL* (ft) with respect to least radius of gyration, r_{my}												
0	783	657	590	521	764	660	549	490	430	493	439	384
6	753	633	567	502	714	618	515	459	403	451	402	351
7	743	624	560	495	697	604	503	449	394	436	389	340
8	731	614	551	487	678	588	490	437	384	420	375	328
9	718	603	541	478	656	570	475	424	372	403	359	314
10	704	591	530	469	633	550	459	410	360	384	342	300
11	688	578	519	458	609	530	442	395	347	364	325	285
12	671	564	506	447	583	508	424	379	333	344	307	269
13	653	549	493	436	557	485	406	363	318	323	288	253
14	635	534	479	423	529	462	387	346	303	302	269	236
15	615	518	464	410	501	438	367	329	288	281	251	220
16	595	501	449	397	473	414	347	311	272	260	232	204
17	575	484	434	383	445	390	327	293	257	239	213	188
18	554	466	418	369	416	366	307	276	241	219	196	172
19	532	448	402	355	388	342	288	258	226	199	178	157
20	510	430	386	341	361	318	268	241	211	180	161	142
21	488	412	369	326	334	295	249	224	196	164	146	129
22	466	393	353	311	308	273	231	208	181	149	133	118
23	444	375	336	297	283	251	213	192	167	136	122	108
24	423	357	320	282	260	231	196	176	154	125	112	98.8
25	401	338	304	268	239	213	180	162	142	115	103	91.1
26	380	321	288	254	221	197	167	150	131	107	95.5	84.2
27	359	303	272	240	205	182	155	139	121	99.0	88.6	78.1
28	338	286	257	226	191	170	144	129	113	92.1	82.4	72.6
29	318	269	242	213	178	158	134	121	105	85.8	76.8	67.7
30	298	252	227	200	166	148	125	113	98.4	80.2	71.7	63.2
32	262	222	199	176	146	130	110	99.0	86.5	70.5	63.1	55.6
34	232	197	177	156	129	115	97.5	87.7	76.6	62.4	55.9	49.2
36	207	175	157	139	115	103	86.9	78.2	68.3			
38	186	157	141	125	104	92.1	78.0	70.2	61.3			
40	168	142	128	112			70.4	63.4	55.3			
Properties												
r_{my}, in.	3.14	3.19	3.22	3.25	2.34	2.39	2.44	2.47	2.49	2.05	2.07	2.10
r_{mx}/r_{my}	1.19	1.19	1.19	1.18	1.50	1.49	1.49	1.48	1.48	1.72	1.72	1.71
$\phi_b M_{nx}$, kip-ft	179	140	119	96.9	177	148	117	99.4	81.4	105	89.7	73.5
$\phi_b M_{ny}$, kip-ft	154	120	102	83.5	124	104	81.8	69.7	57.3	64.5	55.2	45.5
$P_{ex}(K_x L_x)^2/10^4$	49.3	41.4	37.3	32.8	43.4	38.2	32.3	29.0	25.3	27.6	24.8	21.6
$P_{ey}(K_y L_y)^2/10^4$	34.8	29.4	26.5	23.3	19.3	17.1	14.6	13.1	11.5	9.33	8.41	7.39

Note: Heavy line indicates Kl/r equal to or greater than 200.

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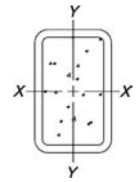
$F_y = 46$ ksi
$f'_c = 4$ ksi
$\phi_c = 0.85$

Table 4-12 (cont.). Composite Rectangular (and Square) HSS Design Strength in Axial Compression, $\phi_c P_n$, kips



Shape		HSS9×7×					HSS9×5×				
		5/8	1/2	3/8	5/16	1/4	5/8	1/2	3/8	5/16	1/4
t_{design}, in.		0.581	0.465	0.349	0.291	0.233	0.581	0.465	0.349	0.291	0.233
Wt/ft		59.1	48.7	37.6	31.8	25.8	50.6	41.9	32.5	27.5	22.4
Effective length KL (ft) with respect to least radius of gyration, r_{my}	0	773	669	558	498	439	547	454	351	297	227
	6	734	636	530	474	417	498	415	322	273	211
	7	720	624	521	466	410	481	401	313	265	205
	8	705	611	510	456	402	463	387	302	256	199
	9	688	597	498	446	392	442	371	290	246	192
	10	669	581	485	434	382	421	353	277	236	184
	11	650	564	472	422	371	398	335	264	225	176
	12	629	547	457	409	360	375	317	250	213	168
	13	606	528	441	395	348	351	297	236	201	159
	14	583	508	425	381	335	327	278	221	189	150
	15	560	488	408	366	322	303	259	207	177	141
	16	535	467	391	350	308	279	239	192	164	132
	17	511	446	374	335	295	256	220	178	152	124
	18	486	425	356	319	281	234	202	164	141	115
	19	460	403	338	303	267	212	184	150	129	106
	20	435	382	321	287	253	191	167	137	118	97.8
	21	411	360	303	272	239	173	151	124	107	89.7
	22	386	339	285	256	225	158	138	113	97.6	81.7
	23	362	318	268	241	212	145	126	104	89.3	74.8
	24	338	298	251	226	198	133	116	95.1	82.1	68.7
	25	315	278	235	211	185	122	107	87.6	75.6	63.3
	26	293	259	219	197	173	113	98.7	81.0	69.9	58.5
	27	271	240	203	182	160	105	91.5	75.1	64.8	54.3
	28	252	223	189	170	149	97.5	85.1	69.9	60.3	50.4
	29	235	208	176	158	139	90.9	79.3	65.1	56.2	47.0
	30	220	194	164	148	130	85.0	74.1	60.9	52.5	43.9
	32	193	171	144	130	114	74.7	65.1	53.5	46.2	38.6
	34	171	151	128	115	101				40.9	34.2
	36	153	135	114	103	90.2					
	38	137	121	102	92.0	81.0					
	40	124	109	92.4	83.1	73.1					
Properties											
r_{my}, in.		2.68	2.73	2.78	2.81	2.84	1.92	1.97	2.03	2.05	2.08
r_{mx}/r_{my}		1.22	1.22	1.22	1.21	1.21	1.60	1.59	1.58	1.58	1.57
$\phi_b M_{nx}$, kip-ft		167	140	110	93.5	76.6	133	112	88.7	75.9	62.4
$\phi_b M_{ny}$, kip-ft		140	117	92.1	78.7	64.5	87.3	74.2	59.0	50.4	41.4
$P_{ex}(K_x L_x)^2/10^4$		37.9	33.6	28.4	25.4	22.3	28.2	25.1	21.1	18.9	16.5
$P_{ey}(K_y L_y)^2/10^4$		25.5	22.5	19.2	17.2	15.2	11.0	9.85	8.41	7.58	6.65

Note: Heavy line indicates Kl/r equal to or greater than 200.

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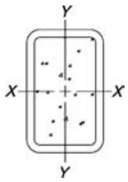

Table 4-12 (cont.). Composite Rectangular (and Square) HSS Design Strength in Axial Compression, $\phi_c P_n$, kips

$F_y = 46$ ksi
$f'_c = 4$ ksi
$\phi_c = 0.85$

Shape		HSS8×8×						HSS8×6×					
		5/8	1/2	3/8	5/16	1/4	3/16	5/8	1/2	3/8	5/16	1/4	3/16
t_{design}, in.		**0.581**	**0.465**	**0.349**	**0.291**	**0.233**	**0.174**	**0.581**	**0.465**	**0.349**	**0.291**	**0.233**	**0.174**
Wt/ft		**59.1**	**48.7**	**37.6**	**31.8**	**25.8**	**19.6**	**50.6**	**41.9**	**32.5**	**27.5**	**22.4**	**17.1**
Effective length *KL* (ft) with respect to least radius of gyration, r_{my}	**0**	776	672	560	501	442	379	642	557	462	413	362	308
	6	744	645	538	482	424	364	598	519	432	386	338	288
	7	733	636	531	475	418	359	583	507	422	377	330	281
	8	720	625	522	467	411	353	566	492	410	366	321	273
	9	706	613	512	458	403	346	547	476	397	355	311	264
	10	691	600	501	449	395	338	527	459	384	343	300	255
	11	675	586	490	438	386	330	505	441	369	329	289	245
	12	657	571	477	427	376	322	483	422	353	316	277	235
	13	638	555	464	415	365	313	459	402	337	301	264	224
	14	619	538	450	403	354	304	435	382	321	286	251	213
	15	598	521	436	390	343	294	411	361	304	271	238	202
	16	577	503	421	377	331	284	387	340	287	256	225	191
	17	556	484	406	364	319	273	362	319	270	241	212	179
	18	534	465	391	350	307	263	338	298	252	226	198	168
	19	511	446	375	336	295	252	314	278	236	211	185	157
	20	489	427	359	321	282	241	291	258	219	196	172	146
	21	466	408	343	307	269	230	268	238	203	181	160	135
	22	444	388	327	293	257	219	246	219	187	167	147	125
	23	421	369	311	278	244	208	225	201	172	154	135	114
	24	399	350	295	264	231	197	207	184	158	141	124	105
	25	377	331	279	250	219	187	190	170	145	130	114	96.9
	26	356	312	264	237	207	176	176	157	134	120	106	89.6
	27	335	294	249	223	195	166	163	146	125	111	98.1	83.0
	28	314	276	234	210	183	156	152	135	116	104	31.2	77.2
	29	294	259	220	197	172	146	141	126	108	96.6	85.1	72.0
	30	274	242	205	184	161	136	132	118	101	90.3	79.5	67.3
	32	241	213	180	162	141	120	116	104	88.8	79.4	69.9	59.1
	34	214	188	160	143	125	106	103	91.9	78.6	70.3	61.9	52.4
	36	191	168	142	128	112	94.7	91.8	82.0	70.1	62.7	55.2	46.7
	38	171	151	128	115	100	85.0		73.6	62.9	56.3	49.5	41.9
	40	154	136	115	103	90.3	76.7				50.8	44.7	37.8
Properties													
r_{my}, in.		2.99	3.04	3.10	3.13	3.15	3.18	2.27	2.32	2.38	2.40	2.43	2.46
r_{mx}/r_{my}		1.00	1.00	1.00	1.00	1.00	1.00	1.26	1.25	1.25	1.25	1.25	1.24
$\phi_b M_{nx}$, kip-ft		154	129	101	86.6	70.7	54.2	125	105	83.1	71.1	58.3	44.9
$\phi_b M_{ny}$, kip-ft		154	129	101	86.6	70.7	54.2	102	85.9	68.3	58.3	48.0	36.9
$P_{ex}(K_x L_x)^2/10^4$		31.9	28.2	23.8	21.4	18.8	16.0	24.4	21.6	18.3	16.4	14.4	12.2
$P_{ey}(K_y L_y)^2/10^4$		31.9	28.2	23.8	21.4	18.8	16.0	15.5	13.8	11.7	10.5	9.27	7.85

Note: Heavy line indicates Kl/r equal to or greater than 200.

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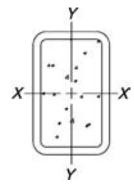
$F_y = 46$ ksi
$f'_c = 4$ ksi
$\phi_c = 0.85$

Table 4-12 (cont.). Composite Rectangular (and Square) HSS Design Strength in Axial Compression, $\phi_c P_n$, kips



Shape	HSS8×4×						HSS7×7×					
	5/8	1/2	3/8	5/16	1/4	3/16	5/8	1/2	3/8	5/16	1/4	3/16
t_{design}, in.	0.581	0.465	0.349	0.291	0.233	0.174	0.581	0.465	0.349	0.291	0.233	0.174
Wt/ft	42.1	35.1	27.4	23.3	19.0	14.5	50.6	41.9	32.5	27.5	22.4	17.1
Effective length KL (ft) with respect to least radius of gyration, r_{my}												
0	513	443	366	324	282	236	645	560	465	416	364	310
6	437	381	317	281	245	206	610	530	441	394	346	294
7	413	361	301	267	222	196	598	520	433	387	339	289
8	387	339	283	252	220	185	585	508	424	379	332	282
9	359	316	265	236	206	173	570	496	413	369	324	275
10	330	292	245	218	191	161	553	482	402	359	315	268
11	301	267	225	201	176	148	536	467	390	349	306	260
12	272	242	206	184	161	136	517	451	377	337	296	251
13	243	218	186	166	146	123	498	434	363	325	285	242
14	216	195	167	149	132	111	477	417	349	313	274	233
15	190	173	149	133	118	99.4	457	399	335	300	263	223
16	167	152	131	118	104	88.0	435	381	320	286	251	213
17	148	134	116	104	92.1	78.0	414	363	305	273	240	203
18	132	120	104	92.9	82.2	69.5	392	344	289	259	228	193
19	118	108	92.9	83.4	73.7	62.4	370	326	274	246	216	182
20	107	97.1	83.9	75.3	66.6	56.3	349	307	259	232	204	172
21	96.7	88.1	76.1	68.3	60.4	51.1	328	289	244	219	192	162
22	88.1	80.3	69.3	62.2	55.0	46.6	307	271	229	206	181	152
23	80.6	73.5	63.4	56.9	50.3	42.6	286	253	214	193	169	142
24	74.0	67.5	58.3	52.3	46.2	39.1	266	236	200	180	158	133
25	68.2	62.2	53.7	48.2	42.6	36.1	247	219	186	167	147	124
26		57.5	49.6	44.5	39.4	33.3	228	203	173	155	136	114
27				41.3	36.5	30.9	211	188	160	144	126	106
28						28.7	197	175	149	134	118	98.6
29							183	163	139	125	110	91.9
30							171	152	130	117	102	85.9
32							151	134	114	102	90.0	75.5
34							133	118	101	90.8	79.7	66.9
36							119	106	90	81.0	71.1	59.7
38							107	94.9	80.8	72.7	63.8	53.5
40							96.3	85.6	72.9	65.6	57.6	48.3
Properties												
r_{my}, in.	1.51	1.56	1.61	1.63	1.66	1.69	2.58	2.63	2.69	2.72	2.75	2.77
r_{mx}/r_{my}	1.75	1.74	1.73	1.73	1.72	1.70	1.00	1.00	1.00	1.00	1.00	1.00
$\phi_b M_{nx}$, kip-ft	94.5	81.1	64.9	55.5	45.9	35.2	114	96.3	76.2	65.2	53.5	41.1
$\phi_b M_{ny}$, kip-ft	57.3	49.3	39.7	34.2	28.3	21.8	114	96.3	76.2	65.2	53.5	41.1
$P_{ex}(K_x L_x)^2/10^4$	17.0	15.2	13.0	11.6	10.1	8.51	20.0	17.8	15.1	13.6	11.9	10.1
$P_{ey}(K_y L_y)^2/10^4$	5.52	5.01	4.33	3.92	3.44	2.90	20.0	17.8	15.1	13.6	11.9	10.1

Note: Heavy line indicates Kl/r equal to or greater than 200.

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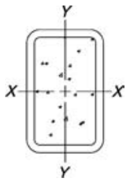

Table 4-12 (cont.). Composite Rectangular (and Square) HSS Design Strength in Axial Compression, $\phi_c P_n$, kips

$F_y = 46$ ksi
$f'_c = 4$ ksi
$\phi_c = 0.85$

Shape		HSS7×5×						HSS7×4×				
		5/8	1/2	3/8	5/16	1/4	3/16	1/2	3/8	5/16	1/4	3/16
t_{design}, in.		0.581	0.465	0.349	0.291	0.233	0.174	0.465	0.349	0.291	0.233	0.174
Wt/ft		42.1	35.1	27.4	23.3	19.0	14.5	31.7	24.9	21.2	17.3	13.2
Effective length KL (ft) with respect to least radius of gyration, r_{my}	0	521	452	374	333	290	245	398	329	292	253	212
	6	469	408	340	303	264	223	340	283	252	219	184
	7	452	394	328	292	255	215	322	268	239	208	175
	8	432	377	315	281	245	207	301	252	225	196	164
	9	411	360	301	268	234	198	280	235	210	184	154
	10	389	341	286	255	223	188	258	217	195	170	143
	11	366	322	270	241	211	178	235	199	179	157	131
	12	342	301	254	227	198	167	213	181	163	143	120
	13	318	281	238	212	186	157	191	163	147	130	108
	14	294	260	221	197	173	146	170	146	132	116	97.3
	15	270	240	204	183	160	135	150	129	118	104	86.7
	16	247	220	188	168	148	125	131	114	104	91.5	76.5
	17	224	201	172	154	135	114	116	101	91.7	81.1	67.8
	18	202	182	157	140	123	104	104	89.9	81.8	72.3	60.5
	19	181	164	141	127	112	94.3	93.2	80.7	73.4	64.9	54.3
	20	164	148	128	114	101	85.1	84.1	72.8	66.3	58.6	49.0
	21	148	134	116	104	91.4	77.2	76.3	66.1	60.1	53.1	44.4
	22	135	122	106	94.5	83.3	70.4	69.5	60.2	54.8	48.4	40.5
	23	124	112	96.5	86.4	76.2	64.4	63.6	55.1	50.1	44.3	37.0
	24	114	102	88.7	79.4	70.0	59.1	58.4	50.6	46.0	40.7	34.0
	25	105	94.5	81.7	73.2	64.5	54.5	53.8	46.6	42.4	37.5	31.3
	26	96.8	87.3	75.5	67.6	59.6	50.4		43.1	39.2	34.7	29.0
	27	89.8	81.0	70.1	62.7	55.3	46.7				32.1	26.9
	28	83.5	75.3	65.1	58.3	51.4	43.4					
	29	77.8	70.2	60.7	54.4	47.9	40.5					
	30	72.7	65.6	56.7	50.8	44.8	37.8					
	32			49.9	44.7	39.3	33.3					
	34						29.5					
	36											
	38											
	40											
Properties												
r_{my}, in.		1.86	1.91	1.97	1.99	2.02	2.05	1.53	1.58	1.61	1.64	1.66
r_{mx}/r_{my}		1.31	1.31	1.30	1.30	1.30	1.29	1.57	1.56	1.55	1.54	1.54
$\phi_b M_{nx}$, kip-ft		88.3	75.6	60.4	51.8	42.8	32.8	64.9	52.1	45.2	37.3	28.7
$\phi_b M_{ny}$, kip-ft		69.7	59.7	47.6	41.1	33.9	26.1	43.5	35.2	30.5	25.3	19.6
$P_{ex}(K_x L_x)^2/10^4$		14.6	13.0	11.1	10.00	8.77	7.37	10.7	9.18	8.26	7.21	6.04
$P_{ey}(K_y L_y)^2/10^4$		8.52	7.66	6.59	5.93	5.20	4.39	4.38	3.80	3.44	3.03	2.54

Note: Heavy line indicates Kl/r equal to or greater than 200.

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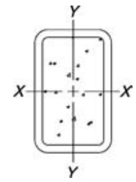
$F_y = 46$ ksi
$f'_c = 4$ ksi
$\phi_c = 0.85$

Table 4-12 (cont.). Composite Rectangular (and Square) HSS Design Strength in Axial Compression, $\phi_c P_n$, kips



Shape	HSS6×6×						HSS6×5×			
	5/8	1/2	3/8	5/16	1/4	3/16	3/8	5/16	1/4	3/16
t_{design}, in.	**0.581**	**0.465**	**0.349**	**0.291**	**0.233**	**0.174**	**0.349**	**0.291**	**0.233**	**0.174**
Wt/ft	**42.1**	**35.1**	**27.4**	**23.3**	**19.0**	**14.5**	**24.9**	**21.2**	**17.3**	**13.2**
Effective length *KL* (ft) with respect to least radius of gyration, r_{my}										
0	524	455	377	336	293	248	335	298	259	218
6	485	422	351	313	273	231	302	269	235	197
7	472	411	342	305	266	225	291	260	226	191
8	457	398	332	296	258	218	279	249	217	183
9	440	385	321	286	250	211	266	238	207	175
10	423	370	308	275	241	203	252	226	197	166
11	404	354	296	264	231	195	238	213	186	157
12	385	338	282	252	221	186	223	200	175	147
13	364	321	268	240	210	177	208	186	163	137
14	344	303	254	228	199	168	193	173	151	128
15	323	286	240	215	188	159	177	160	140	118
16	302	268	225	202	177	149	163	146	128	108
17	281	250	211	189	166	140	148	134	117	99.0
18	261	233	196	176	155	130	134	121	107	90.0
19	241	216	182	164	144	121	121	109	96.2	81.2
20	222	199	169	152	133	112	109	98.6	86.8	73.3
21	203	183	155	140	123	103	98.8	89.4	78.7	66.4
22	185	167	142	128	113	94.9	90.0	81.5	71.7	60.5
23	169	153	130	117	103	86.8	82.3	74.5	65.6	55.4
24	155	140	119	108	94.6	79.7	75.6	68.5	60.3	50.9
25	143	129	110	99.2	87.1	73.5	69.7	63.1	55.5	46.9
26	132	120	102	91.7	80.6	67.9	64.4	58.3	51.4	43.3
27	123	111	94.3	85.0	74.7	63.0	59.7	54.1	47.6	40.2
28	114	103	87.7	79.1	69.5	58.6	55.6	50.3	44.3	37.4
29	106	96.1	81.8	73.7	64.8	54.6	51.8	46.9	41.3	34.8
30	99.3	89.8	76.4	68.9	60.5	51.0	48.4	43.8	38.6	32.6
32	87.3	78.9	67.2	60.5	53.2	44.8	41.1	37.2	32.7	27.7
34	77.3	69.9	59.5	53.6	47.1	39.7				
36	69.0	62.4	53.1	47.8	42.0	35.4				
38			47.6	42.9	37.7	31.8				
40										
Properties										
r_{my}, in.	2.17	2.23	2.28	2.31	2.34	2.37	1.92	1.95	1.98	2.01
r_{mx}/r_{my}	1.00	1.00	1.00	1.00	1.00	1.00	1.16	1.15	1.15	1.15
$\phi_b M_{nx}$, kip-ft	80.0	68.3	54.5	46.9	38.6	29.8	47.6	41.1	34.1	26.3
$\phi_b M_{ny}$, kip-ft	80.0	68.3	54.5	46.9	38.6	29.8	42.1	36.2	30.1	23.2
$P_{ex}(K_x L_x)^2/10^4$	11.6	10.4	8.93	8.03	7.04	5.95	7.54	6.79	5.94	5.00
$P_{ey}(K_y L_y)^2/10^4$	11.6	10.4	8.93	8.03	7.04	5.95	5.67	5.12	4.50	3.78

Note: Heavy line indicates Kl/r equal to or greater than 200.

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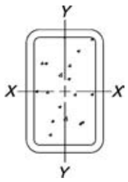

Table 4-12 (cont.). Composite Rectangular (and Square) HSS Design Strength in Axial Compression, $\phi_c P_n$, kips

$F_y = 46$ ksi
$f'_c = 4$ ksi
$\phi_c = 0.85$

Shape	HSS6×4×					HSS6×3×				
	1/2	3/8	5/16	1/4	3/16	1/2	3/8	5/16	1/4	3/16
t_{design}, in.	**0.465**	**0.349**	**0.291**	**0.233**	**0.174**	**0.465**	**0.349**	**0.291**	**0.233**	**0.174**
Wt/ft	**28.3**	**22.3**	**19.0**	**15.6**	**12.0**	**24.9**	**19.7**	**16.9**	**13.9**	**10.7**
Effective length *KL* (ft) with respect to least radius of gyration, r_{my}										
0	353	292	259	225	188	302	249	221	191	158
6	300	250	223	194	162	226	191	170	148	123
7	283	237	211	184	154	204	173	154	135	113
8	264	222	198	173	144	181	155	139	122	102
9	245	206	185	161	135	158	136	122	108	90.6
10	225	190	171	149	125	136	118	107	94.4	79.5
11	204	174	156	137	114	114	101	91.5	81.4	68.9
12	184	157	142	124	104	96.1	85.4	77.3	69.1	58.7
13	165	141	128	112	94.0	81.9	72.8	65.9	58.9	50.0
14	146	126	114	101	84.1	70.6	62.7	56.8	50.8	43.1
15	128	111	101	89.3	74.7	61.5	54.7	49.5	44.2	37.6
16	112	97.6	88.8	78.6	65.7	54.1	48.0	43.5	38.9	33.0
17	99.2	86.4	78.7	69.6	58.2	47.9	42.6	38.5	34.4	29.2
18	88.7	77.1	70.2	62.1	51.9	42.7	38.0	34.4	30.7	26.1
19	79.6	69.2	63.0	55.7	46.6		34.1	30.8	27.6	23.4
20	71.8	62.5	56.9	50.3	42.1				24.9	21.1
21	65.2	56.7	51.6	45.6	38.1					
22	59.4	51.6	47.0	41.6	34.8					
23	54.3	47.2	43.0	38.0	31.8					
24	49.9	43.4	39.5	34.9	29.2					
25	46.0	40.0	36.4	32.2	26.9					
26			33.6	29.8	24.9					
27					23.1					
28										
29										
30										
32										
34										
36										
38										
40										
Properties										
r_{my}, in.	1.50	1.55	1.58	1.61	1.63	1.12	1.17	1.19	1.22	1.25
r_{mx}/r_{my}	1.39	1.38	1.37	1.37	1.37	1.76	1.74	1.74	1.72	1.71
$\phi_b M_{nx}$, kip-ft	50.4	41.1	35.5	29.4	22.8	41.7	34.2	29.7	24.8	19.3
$\phi_b M_{ny}$, kip-ft	38.0	30.8	26.7	22.3	17.3	25.1	20.8	18.2	15.2	11.9
$P_{ex}(K_x L_x)^2/10^4$	7.15	6.17	5.56	4.89	4.10	5.53	4.82	4.37	3.83	3.20
$P_{ey}(K_y L_y)^2/10^4$	3.74	3.25	2.96	2.59	2.19	1.79	1.59	1.45	1.28	1.09

Note: Heavy line indicates Kl/r equal to or greater than 200.

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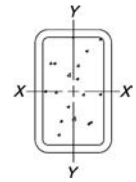
$F_y = 46$ ksi
$f'_c = 4$ ksi
$\phi_c = 0.85$

Table 4-12 (cont.). Composite Rectangular (and Square) HSS Design Strength in Axial Compression, $\phi_c P_n$, kips



Shape		HSS5$^1/_2$×5$^1/_2$×				HSS5×5×					
		$^3/_8$	$^5/_{16}$	$^1/_4$	$^3/_{16}$	$^1/_2$	$^3/_8$	$^5/_{16}$	$^1/_4$	$^3/_{16}$	$^1/_8$
t_{design}, in.		0.349	0.291	0.233	0.174	0.465	0.349	0.291	0.233	0.174	0.116
Wt/ft		24.9	21.2	17.3	13.2	28.3	22.3	19.0	15.6	12.0	8.15
Effective length KL (ft) with respect to least radius of gyration, r_{my}	0	335	298	260	219	355	295	262	227	191	153
	6	308	274	238	201	318	265	236	205	172	138
	7	298	266	231	195	306	255	227	198	166	133
	8	288	256	223	188	292	244	218	189	159	127
	9	276	246	214	180	277	232	207	180	151	121
	10	264	235	205	173	262	219	196	171	143	115
	11	251	224	195	164	245	206	184	161	135	108
	12	237	212	185	155	229	193	172	151	127	101
	13	223	200	174	147	212	179	160	140	118	94.0
	14	209	188	163	137	195	165	148	130	109	87.0
	15	195	175	152	128	178	152	136	119	101	80.0
	16	181	163	142	119	162	138	125	109	92.0	73.2
	17	167	151	131	110	147	125	113	99.5	83.8	66.6
	18	154	139	121	102	131	113	102	90.0	75.8	60.2
	19	141	127	110	93.1	118	101	91.8	80.8	68.1	54.0
	20	128	116	101	84.8	106	91.5	82.9	73.0	61.5	48.8
	21	116	105	91.3	76.9	96.5	83.0	75.2	66.2	55.8	44.2
	22	106	95.6	83.2	70.1	87.9	75.6	68.5	60.3	50.8	40.3
	23	96.8	87.4	76.1	64.1	80.4	69.2	62.7	55.2	46.5	36.9
	24	88.9	80.3	69.9	58.9	73.9	63.6	57.5	50.7	42.7	33.9
	25	81.9	74.0	64.4	54.3	68.1	58.6	53.0	46.7	39.4	31.2
	26	75.7	68.4	59.5	50.2	62.9	54.2	49.0	43.2	36.4	28.9
	27	70.2	63.4	55.2	46.5	58.4	50.2	45.5	40.0	33.7	26.8
	28	65.3	59.0	51.3	43.3	54.3	46.7	42.3	37.2	31.4	24.9
	29	60.9	55.0	47.9	40.3	50.6	43.5	39.4	34.7	29.2	23.2
	30	56.9	51.4	44.7	37.7	47.3	40.7	36.8	32.4	27.3	21.7
	32	50.0	45.2	39.3	33.1		38.1	34.5	28.5	24.0	19.0
	34	44.3	40.0	34.8	29.3						17.9
	36				26.2						
	38										
	40										
Properties											
r_{my}, in.		2.08	2.11	2.13	2.16	1.82	1.87	1.90	1.93	1.96	1.99
r_{mx}/r_{my}		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
$\phi_b M_{nx}$, kip-ft		45.2	39.0	32.2	24.8	45.2	36.6	31.6	26.3	20.3	14.0
$\phi_b M_{ny}$, kip-ft		45.2	39.0	32.2	24.8	45.2	36.6	31.6	26.3	20.3	14.0
$P_{ex}(K_x L_x)^2/10^4$		6.62	5.96	5.23	4.42	5.50	4.76	4.29	3.78	3.19	2.53
$P_{ey}(K_y L_y)^2/10^4$		6.62	5.96	5.23	4.42	5.50	4.76	4.29	3.78	3.19	2.53

Note: Heavy line indicates Kl/r equal to or greater than 200.

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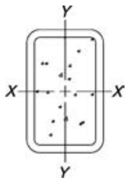

Table 4-12 (cont.). Composite Rectangular (and Square) HSS Design Strength in Axial Compression, $\phi_c P_n$, kips

$F_y = 46$ ksi
$f'_c = 4$ ksi
$\phi_c = 0.85$

Shape		HSS5×4×					HSS5×3×					
		1/2	3/8	5/16	1/4	3/16	1/2	3/8	5/16	1/4	3/16	1/8
t_{design}, in.		**0.465**	**0.349**	**0.291**	**0.233**	**0.174**	**0.465**	**0.349**	**0.291**	**0.233**	**0.174**	**0.116**
Wt/ft		**24.9**	**19.7**	**16.9**	**13.9**	**10.7**	**21.5**	**17.2**	**14.8**	**12.2**	**9.40**	**6.45**
Effective length *KL* (ft) with respect to least radius of gyration, r_{my}	**0**	307	255	226	196	164	259	215	191	165	136	107
	1	306	254	225	195	163	257	214	190	164	135	107
	2	302	251	222	193	161	251	209	185	160	133	104
	3	294	245	218	189	157	240	201	179	154	128	101
	4	285	238	211	183	153	227	190	169	147	122	95.8
	5	273	228	203	176	147	210	177	158	137	114	89.9
	6	259	217	193	168	141	191	162	146	126	105	83.2
	7	244	205	183	159	133	172	147	132	115	96.1	75.9
	8	227	192	171	149	125	151	131	118	103	86.3	68.2
	9	209	178	159	139	116	131	114	104	90.9	76.4	60.5
	10	191	164	146	128	107	112	98.5	90.3	79.0	66.7	52.9
	11	173	149	133	117	98.1	93.4	83.6	77.1	67.7	57.4	45.6
	12	155	135	121	106	89.0	78.4	70.2	64.9	57.1	48.6	38.7
	13	138	120	108	95.2	80.1	66.8	59.8	55.3	48.7	41.4	32.9
	14	121	107	96.0	84.8	71.5	57.6	51.6	47.7	42.0	35.7	28.4
	15	106	93.7	84.4	74.8	63.2	50.2	44.9	41.6	36.6	31.1	24.7
	16	93.1	82.4	74.2	65.8	55.5	44.1	39.5	36.5	32.1	27.3	21.8
	17	82.4	73.0	65.7	58.3	49.2	39.1	35.0	32.4	28.5	24.2	19.3
	18	73.5	65.1	58.6	52.0	43.9	34.9	31.2	28.9	25.4	21.6	17.2
	19	66.0	58.4	52.6	46.6	39.4		28.0	25.9	22.8	19.4	15.4
	20	59.6	52.7	47.5	42.1	35.5					17.5	13.9
	21	54.0	47.8	43.1	38.2	32.2						
	22	49.2	43.6	39.2	34.8	29.4						
	23	45.0	39.9	35.9	31.8	26.9						
	24	41.4	36.6	33.0	29.2	24.7						
	25		33.7	30.4	26.9	22.7						
	26				24.9	21.0						
	27											
	28											
	29											
	30											
Properties												
r_{my}, in.		1.46	1.52	1.54	1.57	1.60	1.09	1.14	1.17	1.19	1.22	1.25
r_{mx}/r_{my}		1.20	1.19	1.19	1.19	1.19	1.51	1.51	1.50	1.50	1.49	1.48
$\phi_b M_{nx}$, kip-ft		37.6	30.9	26.9	22.4	17.4	30.5	25.3	22.1	18.6	14.5	10.1
$\phi_b M_{ny}$, kip-ft		32.3	26.5	23.0	19.2	15.0	21.0	17.6	15.5	13.0	10.2	7.14
$P_{ex}(K_x L_x)^2/10^4$		4.43	3.86	3.50	3.09	2.60	3.37	2.98	2.72	2.39	2.01	1.58
$P_{ey}(K_y L_y)^2/10^4$		3.11	2.72	2.46	2.18	1.84	1.48	1.32	1.21	1.07	0.908	0.718

Note: Heavy line indicates Kl/r equal to or greater than 200.

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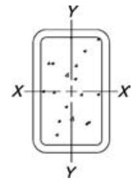
$F_y = 46$ ksi
$f'_c = 4$ ksi
$\phi_c = 0.85$

Table 4-12 (cont.). Composite Rectangular (and Square) HSS Design Strength in Axial Compression, $\phi_c P_n$, kips



Shape	HSS5×2½×			HSS4½×4½×					
	1/4	3/16	1/8	1/2	3/8	5/16	1/4	3/16	1/8
t_{design}, in.	**0.233**	**0.174**	**0.116**	**0.465**	**0.349**	**0.291**	**0.233**	**0.174**	**0.116**
Wt/ft	**11.3**	**8.55**	**6.02**	**24.9**	**19.7**	**16.9**	**13.9**	**10.7**	**7.30**
Effective length KL (ft) with respect to least radius of gyration, r_{my}									
0	149	121	95.7	308	256	227	197	164	131
1	148	120	94.8	293	242	215	187	156	125
2	143	116	92.0	289	240	213	185	154	124
3	136	111	87.6	284	235	209	182	152	121
4	127	103	81.7	276	230	204	177	148	119
5	115	94.4	74.7	267	222	198	172	144	115
6	103	84.6	67.0	268	224	200	174	145	115
7	89.9	74.4	58.9	254	214	190	166	138	110
8	77.0	64.0	50.8	240	202	180	157	131	104
9	64.6	54.1	42.9	225	190	170	148	123	98.1
10	53.0	44.7	35.5	209	177	159	138	115	91.7
11	43.8	36.9	29.3	192	164	147	129	107	85.1
12	36.8	31.0	24.6	176	151	135	119	98.7	78.4
13	31.3	26.4	21.0	159	137	124	109	90.4	71.7
14	27.0	22.8	18.1	144	124	112	98.7	82.2	65.2
15	23.5	19.8	15.8	128	112	101	89.1	74.2	58.8
16	20.7	17.4	13.8	113	99.6	90.5	79.8	66.5	52.6
17		15.5	12.3	100	88.2	80.2	70.9	59.0	46.7
18				89.6	78.7	71.6	63.3	52.6	41.7
19				80.4	70.6	64.2	56.8	47.3	37.4
20				72.5	63.8	58.0	51.2	42.6	33.8
21				65.8	57.8	52.6	46.5	38.7	30.6
22				60.0	52.7	47.9	42.3	35.2	27.9
23				54.9	48.2	43.8	38.7	32.2	25.5
24				50.4	44.3	40.3	35.6	29.6	23.4
25				46.4	40.8	37.1	32.8	27.3	21.6
26				42.9	37.7	34.3	30.3	25.2	20.0
27					35.0	31.8	28.1	23.4	18.5
28						29.6	26.1	21.8	17.2
29								20.3	16.1
30									
Properties									
r_{my}, in.	0.999	1.03	1.05	1.61	1.67	1.70	1.73	1.75	1.78
r_{mx}/r_{my}	1.73	1.72	1.71	1.00	1.00	1.00	1.00	1.00	1.00
$\phi_b M_{nx}$, kip-ft	16.7	12.8	9.14	35.2	28.8	25.1	20.9	16.2	11.3
$\phi_b M_{ny}$, kip-ft	10.2	7.90	5.66	35.2	28.8	25.1	20.9	16.2	11.3
$P_{ex}(K_x L_x)^2/10^4$	2.06	1.70	1.35	3.79	3.31	3.00	2.63	2.22	1.76
$P_{ey}(K_y L_y)^2/10^4$	0.685	0.574	0.460	3.79	3.31	3.00	2.63	2.22	1.76

Note: Heavy line indicates Kl/r equal to or greater than 200.

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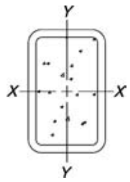

Table 4-12 (cont.). Composite Rectangular (and Square) HSS Design Strength in Axial Compression, $\phi_c P_n$, kips

$F_y = 46$ ksi
$f'_c = 4$ ksi
$\phi_c = 0.85$

Shape	HSS4×4×						HSS4×3×				
	1/2	3/8	5/16	1/4	3/16	1/8	3/8	5/16	1/4	3/16	1/8
t_{design}, in.	**0.465**	**0.349**	**0.291**	**0.233**	**0.174**	**0.116**	**0.349**	**0.291**	**0.233**	**0.174**	**0.116**
Wt/ft	**21.5**	**17.2**	**14.8**	**12.2**	**9.40**	**6.45**	**14.6**	**12.7**	**10.5**	**8.13**	**5.60**
Effective length KL (ft) with respect to least radius of gyration, r_{my}											
0	262	218	194	168	139	110	182	161	140	115	90.3
1	249	207	183	159	132	105	180	160	138	115	89.6
2	246	204	181	157	130	104	176	156	135	112	87.7
3	240	199	177	153	128	101	169	150	130	108	84.4
4	231	193	171	148	124	98.3	159	142	123	102	80.1
5	221	185	164	143	119	94.5	148	132	115	95.8	74.9
6	219	184	164	142	119	93.9	135	121	106	88.3	69.0
7	205	173	154	134	112	88.6	121	109	95.7	80.1	62.6
8	190	161	144	126	105	82.9	107	96.7	85.3	71.6	55.9
9	174	149	133	116	97.1	76.8	93.4	84.4	74.8	63.1	49.3
10	158	136	122	107	89.2	70.6	80.0	72.5	64.6	54.7	42.7
11	142	123	111	96.9	81.2	64.3	67.2	61.2	55.0	46.8	36.5
12	127	111	99.3	87.3	73.3	58.0	56.4	51.4	46.2	39.4	30.8
13	112	98.3	88.5	78.0	65.6	51.9	48.1	43.8	39.4	33.6	26.2
14	97.2	86.5	78.0	69.0	58.1	46.0	41.5	37.8	34.0	29.0	22.6
15	84.6	75.4	68.1	60.4	51.0	40.3	36.1	32.9	29.6	25.2	19.7
16	74.4	66.3	59.9	53.1	44.8	35.4	31.7	28.9	26.0	22.2	17.3
17	65.9	58.7	53.0	47.0	39.7	31.4	28.1	25.6	23.0	19.6	15.3
18	58.8	52.4	47.3	41.9	35.4	28.0	25.1	22.9	20.5	17.5	13.7
19	52.8	47.0	42.5	37.6	31.8	25.1			18.4	15.7	12.3
20	47.6	42.4	38.3	34.0	28.7	22.7					11.1
21	43.2	38.5	34.8	30.8	26.0	20.6					
22	39.3	35.1	31.7	28.1	23.7	18.7					
23	36.0	32.1	29.0	25.7	21.7	17.2					
24		29.5	26.6	23.6	19.9	15.8					
25				21.7	18.3	14.5					
26						13.4					
27											
28											
29											
30											
Properties											
r_{my}, in.	1.41	1.47	1.49	1.52	1.55	1.58	1.11	1.13	1.16	1.19	1.21
r_{mx}/r_{my}	1.00	1.00	1.00	1.00	1.00	1.00	1.25	1.26	1.25	1.25	1.26
$\phi_b M_{nx}$, kip-ft	26.6	22.0	19.3	16.2	12.7	8.83	17.7	15.6	13.1	10.4	7.28
$\phi_b M_{ny}$, kip-ft	26.6	22.0	19.3	16.2	12.7	8.83	14.4	12.7	10.8	8.49	5.97
$P_{ex}(K_x L_x)^2/10^4$	2.46	2.20	1.99	1.76	1.49	1.17	1.66	1.53	1.35	1.14	0.897
$P_{ey}(K_y L_y)^2/10^4$	2.46	2.20	1.99	1.76	1.49	1.17	1.05	0.966	0.861	0.732	0.578

Note: Heavy line indicates Kl/r equal to or greater than 200.

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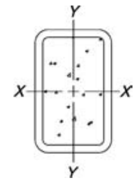
$F_y = 46$ ksi
$f'_c = 4$ ksi
$\phi_c = 0.85$

Table 4-12 (cont.). Composite Rectangular (and Square) HSS Design Strength in Axial Compression, $\phi_c P_n$, kips



Shape	HSS4×2½×			HSS4×2×					HSS3½×3½×				
	5/16	1/4	3/16	3/8	5/16	1/4	3/16	1/8	3/8	5/16	1/4	3/16	1/8
t_{design}, in.	**0.291**	**0.233**	**0.174**	**0.349**	**0.291**	**0.233**	**0.174**	**0.116**	**0.349**	**0.291**	**0.233**	**0.174**	**0.116**
Wt/ft	**11.6**	**9.63**	**7.49**	**12.1**	**10.5**	**8.78**	**6.85**	**4.75**	**14.6**	**12.7**	**10.5**	**8.13**	**5.60**
Effective length *KL* (ft) with respect to least radius of gyration, r_{my}													
0	145	125	103	145	129	111	91.3	70.0	182	162	140	116	91.0
1	143	124	102	142	126	109	89.8	69.0	173	153	133	110	86.5
2	132	120	98.9	134	120	104	85.6	65.8	170	150	130	108	85.0
3	131	113	93.9	122	110	95.4	79.0	60.9	164	146	126	105	82.6
4	121	105	87.2	107	97.0	84.8	70.6	54.6	157	140	121	101	79.3
5	109	95.3	79.3	90.5	82.7	72.9	61.1	47.5	149	133	115	95.8	75.3
6	96.5	84.6	70.7	73.6	68.1	60.6	51.2	40.0	145	130	113	94.3	73.8
7	83.3	73.5	61.7	57.7	54.1	48.7	41.6	32.7	133	120	105	87.4	68.4
8	70.3	62.4	52.7	44.2	41.7	37.9	32.6	25.8	121	109	95.8	80.1	62.6
9	58.0	51.9	44.0	34.9	33.0	30.0	25.8	20.4	109	98.5	86.6	72.6	56.7
10	47.1	42.3	36.0	28.3	26.7	24.3	20.9	16.5	96.4	87.7	77.4	65.0	50.7
11	38.9	34.9	29.8	23.4	22.1	20.1	17.3	13.7	84.3	77.1	68.3	57.5	44.9
12	32.7	29.4	25.0	19.6	18.5	16.9	14.5	11.5	72.8	66.9	59.5	50.3	39.2
13	27.9	25.0	21.3				12.4	9.79	62.1	57.3	51.2	43.4	33.8
14	24.0	21.6	18.4						53.6	49.4	44.1	37.4	29.1
15	20.9	18.8	16.0						46.7	43.0	38.5	32.6	25.4
16		16.5	14.1						41.0	37.8	33.8	28.7	22.3
17									36.3	33.5	29.9	25.4	19.8
18									32.4	29.9	26.7	22.6	17.6
19									29.1	26.8	24.0	20.3	15.8
20									26.2	24.2	21.6	18.3	14.3
21									23.8	22.0	19.6	16.6	13.0
22											17.9	15.2	11.8
23													
24													
25													
26													
27													
28													
29													
30													
Properties													
r_{my}, in.	0.947	0.973	0.999	0.729	0.754	0.779	0.804	0.830	1.26	1.29	1.32	1.35	1.37
r_{mx}/r_{my}	1.46	1.45	1.44	1.77	1.75	1.75	1.73	1.72	1.00	1.00	1.00	1.00	1.00
$\phi_b M_{nx}$, kip-ft	13.7	11.7	9.21	13.2	11.8	10.1	8.07	5.73	16.2	14.3	12.1	9.52	6.66
$\phi_b M_{ny}$, kip-ft	9.83	8.38	6.66	7.97	7.18	6.18	4.93	3.52	16.2	14.3	12.1	9.52	6.66
$P_{ex}(K_x L_x)^2/10^4$	1.29	1.15	0.976	1.14	1.06	0.953	0.809	0.634	1.36	1.25	1.11	0.943	0.743
$P_{ey}(K_y L_y)^2/10^4$	0.609	0.548	0.467	0.368	0.346	0.314	0.270	0.215	1.36	1.25	1.11	0.943	0.743

Note: Heavy line indicates Kl/r equal to or greater than 200.

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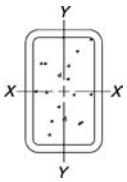

Table 4-12 (cont.). Composite Rectangular (and Square) HSS Design Strength in Axial Compression, $\phi_c P_n$, kips

$F_y = 46$ ksi
$f'_c = 4$ ksi
$\phi_c = 0.85$

Shape		HSS3×3×					HSS2½×2½×			
		3/8	5/16	1/4	3/16	1/8	5/16	1/4	3/16	1/8
t_{design}, in.		**0.349**	**0.291**	**0.233**	**0.174**	**0.116**	**0.291**	**0.233**	**0.174**	**0.116**
Wt/ft		**12.1**	**10.5**	**8.78**	**6.85**	**4.75**	**8.40**	**7.08**	**5.57**	**3.90**
Effective length KL (ft) with respect to least radius of gyration, r_{my}	**0**	148	132	114	94.1	72.9	102	88.8	73.5	56.7
	1	140	124	107	88.8	69.0	96.6	83.6	69.1	53.3
	2	136	121	105	86.7	67.4	92.9	80.6	66.7	51.6
	3	130	116	101	83.3	64.8	87.2	75.9	63.0	48.7
	4	123	109	94.9	78.8	61.4	79.7	69.7	58.1	45.0
	5	113	101	88.2	73.4	57.2	71.1	62.5	52.3	40.7
	6	107	96.3	84.3	70.4	54.9	64.1	57.0	48.1	37.5
	7	95.2	86.0	75.7	63.4	49.5	54.2	48.6	41.3	32.3
	8	83.3	75.5	66.8	56.2	44.0	44.6	40.4	34.6	27.2
	9	71.5	65.1	58.0	49.0	38.4	35.7	32.7	28.3	22.4
	10	60.4	55.2	49.5	42.1	33.1	29.0	26.5	22.9	18.2
	11	50.1	46.0	41.4	35.4	28.0	23.9	21.9	18.9	15.0
	12	42.1	38.6	34.8	29.8	23.5	20.1	18.4	15.9	12.6
	13	35.9	32.9	29.7	25.4	20.0	17.1	15.7	13.6	10.8
	14	30.9	28.4	25.6	21.9	17.3	14.8	13.5	11.7	9.27
	15	26.9	24.7	22.3	19.1	15.0		11.8	10.2	8.08
	16	23.7	21.7	19.6	16.8	13.2				7.10
	17	21.0	19.2	17.4	14.8	11.7				
	18		17.2	15.5	13.2	10.4				
	19				11.9	9.37				
	20									
	21									
	22									
	23									
	24									
	25									
	26									
	27									
	28									
	29									
	30									
Properties										
r_{my}, in.		1.06	1.08	1.11	1.14	1.17	0.880	0.908	0.937	0.965
r_{mx}/r_{my}		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
$\phi_b M_{nx}$, kip-ft		11.2	10.0	8.56	6.80	4.83	6.49	5.62	4.55	3.27
$\phi_b M_{ny}$, kip-ft		11.2	10.0	8.56	6.80	4.83	6.49	5.62	4.55	3.27
$P_{ex}(K_x L_x)^2/10^4$		0.783	0.727	0.653	0.556	0.439	0.377	0.345	0.296	0.236
$P_{ey}(K_y L_y)^2/10^4$		0.783	0.727	0.653	0.556	0.439	0.377	0.345	0.296	0.236

Note: Heavy line indicates Kl/r equal to or greater than 200.

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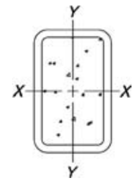
$F_y = 46$ ksi
$f'_c = 5$ ksi
$\phi_c = 0.85$

Table 4-13. Composite Rectangular (and Square) HSS Design Strength in Axial Compression, $\phi_c P_n$, kips



Shape		HSS20×12×		HSS18×12×		HSS16×16×			HSS16×12×		
		5/8	1/2	5/8	1/2	5/8	1/2	3/8	5/8	1/2	3/8
t_{design}, in.		0.581	0.465	0.581	0.465	0.581	0.465	0.349	0.581	0.465	0.349
Wt/ft		127	103	119	96.4	127	103	78.4	110	89.6	68.2
Effective length KL (ft) with respect to least radius of gyration, r_{my}	0	2110	1870	1930	1720	2160	1930	1690	1760	1560	1360
	6	2070	1840	1900	1690	2140	1910	1670	1730	1540	1330
	7	2060	1830	1890	1680	2130	1900	1660	1720	1530	1320
	8	2040	1810	1870	1670	2120	1890	1650	1710	1520	1310
	9	2030	1800	1860	1650	2110	1880	1650	1700	1500	1300
	10	2010	1780	1840	1640	2100	1870	1640	1680	1490	1290
	11	1990	1770	1820	1620	2090	1860	1630	1660	1470	1280
	12	1970	1750	1800	1600	2070	1850	1610	1650	1460	1260
	13	1950	1730	1780	1580	2060	1830	1600	1630	1440	1250
	14	1920	1700	1760	1560	2040	1820	1590	1600	1420	1230
	15	1900	1680	1740	1540	2020	1800	1580	1580	1400	1210
	16	1870	1660	1710	1520	2010	1790	1560	1560	1380	1190
	17	1840	1630	1680	1500	1990	1770	1550	1530	1360	1170
	18	1810	1610	1660	1470	1970	1750	1530	1510	1330	1150
	19	1780	1580	1630	1450	1950	1730	1510	1480	1310	1130
	20	1750	1550	1600	1420	1920	1710	1490	1450	1290	1110
	21	1710	1520	1570	1390	1900	1690	1480	1420	1260	1090
	22	1680	1490	1540	1360	1880	1670	1460	1390	1230	1070
	23	1640	1460	1500	1330	1850	1650	1440	1360	1210	1040
	24	1610	1430	1470	1300	1830	1620	1420	1330	1180	1020
	25	1570	1390	1440	1270	1800	1600	1400	1300	1150	993
	26	1540	1360	1400	1240	1770	1580	1370	1270	1120	968
	27	1500	1330	1370	1210	1750	1550	1350	1240	1100	943
	28	1460	1290	1330	1180	1720	1530	1330	1200	1070	918
	29	1420	1260	1300	1150	1690	1500	1310	1170	1040	892
	30	1380	1230	1260	1120	1660	1480	1290	1140	1010	866
	34	1230	1090	1110	989	1540	1370	1190	1000	889	763
	38	1070	950	972	862	1420	1260	1090	873	772	661
	42	924	817	835	740	1290	1140	990	747	661	564
	46	783	691	704	624	1160	1030	890	628	555	473
	50	662	585	596	528	1040	920	793	532	470	400
Properties											
r_{my}, in.		4.93	4.99	4.87	4.93	6.25	6.31	6.37	4.80	4.86	4.91
r_{mx}/r_{my}		1.49	1.48	1.37	1.37	1.00	1.00	1.00	1.25	1.25	1.25
$\phi_b M_{nx}$, kip-ft		794	649	676	555	690	566	435	569	466	359
$\phi_b M_{ny}$, kip-ft		559	455	511	421	690	566	435	466	383	295
$P_{ex}(K_x L_x)^2/10^4$		473	416	361	318	351	310	266	268	236	202
$P_{ey}(K_y L_y)^2/10^4$		214	189	193	170	351	310	266	172	152	130

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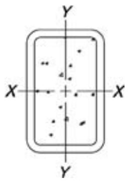

**Table 4-13 (cont.).
Composite Rectangular
(and Square) HSS
Design Strength in Axial
Compression, $\phi_c P_n$, kips**

$F_y = 46$ ksi
$f'_c = 5$ ksi
$\phi_c = 0.85$

Shape	HSS16×8×			HSS14×14×			HSS14×12×		HSS14×10×		
	5/8	1/2	3/8	5/8	1/2	3/8	1/2	3/8	5/8	1/2	3/8
t_{design}, in.	**0.581**	**0.465**	**0.349**	**0.581**	**0.465**	**0.349**	**0.465**	**0.349**	**0.581**	**0.465**	**0.349**
Wt/ft	**93.1**	**75.9**	**58.0**	**110**	**89.6**	**68.2**	**82.7**	**63.1**	**93.1**	**75.9**	**58.0**
Effective length KL (ft) with respect to least radius of gyration, r_{my}											
0	1370	1200	1030	1780	1580	1370	1410	1220	1410	1240	1070
6	1320	1160	992	1750	1560	1350	1390	1200	1380	1210	1050
7	1300	1140	979	1750	1550	1340	1380	1190	1370	1200	1040
8	1280	1130	964	1740	1540	1340	1370	1180	1350	1190	1030
9	1260	1110	947	1730	1530	1330	1360	1170	1340	1180	1010
10	1240	1090	929	1710	1520	1320	1340	1160	1320	1160	1000
11	1210	1060	909	1700	1510	1310	1330	1150	1300	1140	985
12	1180	1040	888	1680	1490	1300	1320	1130	1280	1130	969
13	1150	1010	866	1670	1480	1280	1300	1120	1260	1110	952
14	1120	985	842	1650	1460	1270	1280	1100	1230	1090	935
15	1090	956	818	1630	1450	1250	1260	1090	1210	1070	916
16	1060	927	792	1610	1430	1240	1240	1070	1180	1040	896
17	1020	896	766	1590	1410	1220	1220	1050	1160	1020	876
18	985	865	739	1570	1390	1210	1200	1030	1130	995	854
19	948	833	712	1550	1370	1190	1180	1010	1100	969	833
20	912	801	684	1530	1350	1170	1160	994	1070	944	810
21	874	768	656	1500	1330	1150	1130	974	1040	917	787
22	837	735	628	1480	1310	1130	1110	953	1010	890	764
23	799	703	600	1450	1290	1110	1080	931	979	863	740
24	762	670	572	1430	1270	1090	1060	909	947	835	716
25	725	637	544	1400	1240	1070	1030	886	915	807	692
26	688	605	516	1380	1220	1050	1010	864	883	779	668
27	652	574	489	1350	1190	1030	981	841	852	751	643
28	616	543	463	1320	1170	1010	954	817	820	723	619
29	581	512	436	1290	1140	1090	927	794	788	695	595
30	548	482	411	1260	1120	1070	900	770	756	667	571
34	427	376	320	1150	1010	872	792	676	633	559	477
38	342	301	257	1030	907	780	686	584	517	457	389
42	280	247	210	910	802	688	584	496	423	374	318
46	233	206	175	796	700	600	489	415	353	312	265
50	197	174	148	687	604	515	414	351	298	264	225
Properties											
r_{my}, in.	3.27	3.32	3.37	5.44	5.49	5.55	4.78	4.83	3.98	4.04	4.09
r_{mx}/r_{my}	1.72	1.72	1.71	1.00	1.00	1.00	1.13	1.12	1.30	1.29	1.29
$\phi_b M_{nx}$, kip-ft	445	366	283	521	428	329	383	296	414	341	263
$\phi_b M_{ny}$, kip-ft	273	226	175	521	428	329	345	267	328	271	209
$P_{ex}(K_x L_x)^2/10^4$	189	165	140	222	196	167	169	144	162	143	121
$P_{ey}(K_y L_y)^2/10^4$	63.4	56.0	47.7	222	196	167	133	114	96.3	85.2	72.5

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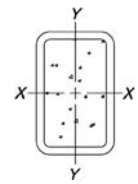
$F_y = 46$ ksi
$f'_c = 5$ ksi
$\phi_c = 0.85$

Table 4-13 (cont.). Composite Rectangular (and Square) HSS Design Strength in Axial Compression, $\phi_c P_n$, kips



Shape		HSS12×12×				HSS12×10×			HSS12×8×			
		5/8	1/2	3/8	5/16	1/2	3/8	5/16	5/8	1/2	3/8	5/16
t_{design}, in.		0.581	0.465	0.349	0.291	0.465	0.349	0.291	0.581	0.465	0.349	0.291
Wt/ft		93.1	75.9	58.0	48.8	69.1	52.9	44.6	76.1	62.3	47.8	40.3
Effective length KL (ft) with respect to least radius of gyration, r_{my}	0	1430	1260	1090	995	1100	950	865	1090	955	814	740
	6	1400	1240	1070	976	1080	926	843	1050	918	783	711
	7	1390	1230	1060	969	1070	917	835	1030	906	772	702
	8	1380	1220	1050	961	1060	907	826	1020	891	760	690
	9	1370	1210	1040	953	1040	896	816	997	875	746	678
	10	1360	1200	1030	943	1030	884	804	976	858	731	664
	11	1340	1180	1020	933	1010	871	792	954	838	715	649
	12	1330	1170	1010	921	997	856	779	931	818	698	633
	13	1310	1150	994	909	979	841	765	906	796	679	616
	14	1290	1140	980	896	960	825	750	880	774	660	598
	15	1270	1120	965	882	941	808	734	853	750	640	580
	16	1250	1100	950	868	920	790	718	825	725	619	561
	17	1230	1080	933	853	899	771	701	796	700	598	541
	18	1210	1060	916	837	876	752	683	766	674	576	521
	19	1180	1040	899	821	853	732	665	736	648	553	501
	20	1160	1020	880	804	830	712	646	706	622	531	480
	21	1140	1000	862	786	806	691	627	675	595	508	459
	22	1110	980	842	769	782	670	608	645	568	485	438
	23	1090	957	823	750	757	648	588	614	541	462	417
	24	1060	934	803	732	732	627	568	584	515	440	397
	25	1030	911	782	713	707	605	548	554	488	417	376
	26	1010	887	761	694	681	583	528	524	462	395	356
	27	979	863	741	675	656	561	508	495	437	373	336
	28	952	839	719	655	631	539	488	466	412	352	317
	29	924	814	698	635	605	517	468	438	387	331	298
	30	896	790	677	616	580	469	448	411	363	311	279
	34	785	692	592	537	483	412	372	320	283	242	217
	38	676	596	508	461	393	334	301	256	226	194	174
	42	573	505	430	389	321	274	247	209	185	158	142
	46	478	422	358	324	268	228	206	175	154	132	119
	50	405	357	303	274	227	193	174	148	131	112	100
Properties												
r_{my}, in.		4.62	4.68	4.73	4.76	3.96	4.01	4.04	3.16	3.21	3.27	3.29
r_{mx}/r_{my}		1.00	1.00	1.00	1.00	1.15	1.15	1.15	1.37	1.37	1.37	1.37
$\phi_b M_{nx}$, kip-ft		376	309	239	202	272	211	178	283	235	183	155
$\phi_b M_{ny}$, kip-ft		376	309	239	202	240	186	158	214	178	138	118
$P_{ex}(K_x L_x)^2/10^4$		131	115	97.9	89.0	97.0	82.4	74.7	90.2	79.2	67.0	60.6
$P_{ey}(K_y L_y)^2/10^4$		131	115	97.9	89.0	73.2	62.2	56.6	47.7	42.4	35.8	32.5

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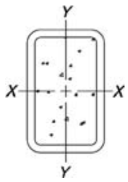

Table 4-13 (cont.). Composite Rectangular (and Square) HSS Design Strength in Axial Compression, $\phi_c P_n$, kips

$F_y = 46$ ksi
$f'_c = 5$ ksi
$\phi_c = 0.85$

Shape	HSS12×6×				HSS10×10×				
	5/8	1/2	3/8	5/16	5/8	1/2	3/8	5/16	1/4
t_{design}, in.	0.581	0.465	0.349	0.291	0.581	0.465	0.349	0.291	0.233
Wt/ft	67.6	55.5	42.7	36.0	76.1	62.3	47.8	40.3	32.6
Effective length *KL* (ft) with respect to least radius of gyration, r_{my}									
0	920	800	677	611	1100	969	828	754	679
6	860	750	635	573	1070	943	806	734	660
7	840	732	620	560	1060	934	799	727	653
8	817	712	604	545	1050	924	790	718	646
9	791	691	585	528	1040	912	780	709	638
10	764	667	566	510	1020	889	769	699	628
11	735	642	545	492	1010	885	757	688	618
12	704	616	523	472	989	870	744	676	607
13	672	589	500	451	971	854	730	663	596
14	639	560	476	429	951	837	715	650	583
15	606	532	452	408	931	819	700	636	571
16	572	502	427	386	910	801	684	621	557
17	538	473	402	363	887	781	667	606	543
18	504	444	378	341	864	761	650	590	529
19	471	415	353	319	841	740	632	574	514
20	438	387	329	298	816	719	614	557	499
21	406	259	306	276	792	698	596	540	483
22	375	332	283	256	766	676	577	522	467
23	344	305	261	236	741	653	558	505	452
24	316	280	240	216	715	631	538	487	435
25	291	258	221	199	689	608	519	470	419
26	269	239	204	184	664	586	500	452	403
27	250	222	189	171	638	563	480	434	387
28	232	206	176	159	612	540	461	416	371
29	216	192	164	148	586	518	442	399	355
30	202	179	153	138	561	496	423	381	339
34	157	140	119	108	463	409	349	314	279
38	126	112	95.6	86.3	373	331	282	253	224
42				70.7	306	271	230	207	183
46					255	226	192	173	153
50					216	191	163	146	129
Properties									
r_{my}, in.	2.39	2.44	2.49	2.52	3.80	3.86	3.92	3.94	3.97
r_{mx}/r_{my}	1.73	1.73	1.72	1.71	1.00	1.00	1.00	1.00	1.00
$\phi_b M_{nx}$, kip-ft	237	198	155	131	253	209	163	138	113
$\phi_b M_{ny}$, kip-ft	145	121	95.6	81.4	253	209	163	138	113
$P_{ex}(K_x L_x)^2/10^4$	70.5	62.0	52.3	47.0	69.7	61.5	52.3	47.2	41.9
$P_{ey}(K_y L_y)^2/10^4$	23.5	20.8	17.7	16.1	69.7	61.5	52.3	47.2	41.9

Note: Heavy line indicates Kl/r equal to or greater than 200.

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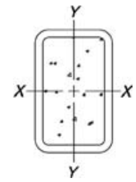
$F_y = 46$ ksi
$f'_c = 5$ ksi
$\phi_c = 0.85$

Table 4-13 (cont.). Composite Rectangular (and Square) HSS Design Strength in Axial Compression, $\phi_c P_n$, kips



Shape	HSS10×8×				HSS10×6×					HSS10×5×		
	1/2	3/8	5/16	1/4	5/8	1/2	3/8	5/16	1/4	3/8	5/16	1/4
t_{design}, in.	**0.465**	**0.349**	**0.291**	**0.233**	**0.581**	**0.465**	**0.349**	**0.291**	**0.233**	**0.349**	**0.291**	**0.233**
Wt/ft	**55.5**	**42.7**	**36.0**	**29.2**	**59.1**	**48.7**	**37.6**	**31.8**	**25.8**	**35.1**	**29.7**	**24.1**
Effective length *KL* (ft) with respect to least radius of gyration, r_{my}												
0	829	706	640	573	795	693	584	527	468	522	469	415
6	797	679	615	550	741	648	547	493	437	476	427	378
7	785	669	606	542	723	632	533	481	427	460	413	365
8	772	658	596	533	703	615	519	468	415	442	397	351
9	758	645	585	523	680	595	503	453	402	423	380	336
10	742	632	572	512	656	574	485	437	388	403	362	320
11	725	617	559	500	630	552	466	421	373	381	342	303
12	707	602	545	487	602	529	447	403	357	359	322	285
13	687	585	530	474	574	504	427	385	341	336	302	267
14	667	568	514	459	545	479	406	366	324	313	282	249
15	646	550	498	445	515	454	384	347	306	291	261	231
16	624	531	481	429	486	428	363	327	289	268	241	213
17	601	512	463	414	456	402	341	308	272	246	221	196
18	578	493	446	397	426	377	320	288	254	225	202	179
19	555	473	427	381	397	351	298	269	237	204	183	162
20	532	453	409	365	368	326	277	250	220	184	165	146
21	508	432	391	348	340	302	257	232	204	167	150	133
22	484	412	373	332	313	279	237	214	188	152	137	121
23	461	392	354	315	287	255	218	197	173	139	125	111
24	437	372	336	299	263	235	200	181	159	128	115	102
25	414	352	318	283	243	216	184	166	146	118	106	93.6
26	391	333	301	267	224	200	170	154	135	109	97.7	86.6
27	369	314	283	251	208	185	158	143	125	101	90.6	80.3
28	347	295	266	236	193	172	147	133	116	93.9	84.3	74.6
29	326	277	250	221	180	161	137	124	109	87.5	78.6	69.6
30	304	259	233	207	168	150	128	116	101	81.8	73.4	65.0
32	267	228	205	182	148	132	112	102	89.2	71.9	64.5	57.1
34	237	202	182	161	131	117	99.6	90.0	79.0	63.7	57.2	50.6
36	211	180	162	144	117	104	88.8	80.2	70.4			
38	190	161	146	129	105	93.6	79.7	72.0	63.2			
40	171	146	131	116			72.0	65.0	57.1			
Properties												
r_{my}, in.	3.14	3.19	3.22	3.25	2.34	2.39	2.44	2.47	2.49	2.05	2.07	2.10
r_{mx}/r_{my}	1.19	1.19	1.19	1.18	1.50	1.49	1.49	1.48	1.48	1.72	1.72	1.71
$\phi_b M_{nx}$, kip-ft	179	140	119	96.9	177	148	117	99.4	81.4	105	89.7	73.5
$\phi_b M_{ny}$, kip-ft	154	120	102	83.5	124	104	81.8	69.7	57.3	64.5	55.2	45.5
$P_{ex}(K_x L_x)^2/10^4$	50.1	42.4	38.3	33.8	43.8	38.6	32.9	29.7	26.0	28.0	25.3	22.2
$P_{ey}(K_y L_y)^2/10^4$	35.4	30.1	27.2	24.1	19.5	17.4	14.8	13.4	11.8	9.48	8.57	7.58

Note: Heavy line indicates Kl/r equal to or greater than 200.

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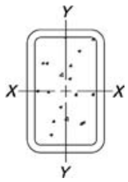

Table 4-13 (cont.). Composite Rectangular (and Square) HSS Design Strength in Axial Compression, $\phi_c P_n$, kips

$F_y = 46$ ksi
$f'_c = 5$ ksi
$\phi_c = 0.85$

Shape	HSS9×7×					HSS9×5×				
	5/8	1/2	3/8	5/16	1/4	5/8	1/2	3/8	5/16	1/4
t_{design}, in.	**0.581**	**0.465**	**0.349**	**0.291**	**0.233**	**0.581**	**0.465**	**0.349**	**0.291**	**0.233**
Wt/ft	**59.1**	**48.7**	**37.6**	**31.8**	**25.8**	**50.6**	**41.9**	**32.5**	**27.5**	**22.4**
Effective length KL (ft) with respect to least radius of gyration, r_{my}										
0	805	704	595	537	479	655	572	479	431	381
6	764	668	565	510	454	592	518	436	392	346
7	749	656	555	501	446	570	500	421	378	335
8	733	642	543	490	436	547	480	405	364	322
9	715	626	530	478	426	521	458	387	348	307
10	695	609	515	465	414	494	435	368	331	292
11	674	591	500	451	402	466	411	348	313	277
12	651	571	483	436	388	436	386	327	294	260
13	628	551	466	421	374	407	360	306	275	244
14	603	530	449	405	360	377	335	285	256	227
15	578	508	430	388	345	347	309	264	237	210
16	552	486	411	371	330	318	284	243	219	194
17	526	463	392	354	314	290	260	223	200	177
18	500	440	373	336	299	263	236	203	183	162
19	473	417	353	319	283	236	213	184	165	147
20	447	394	334	301	267	213	192	166	149	132
21	420	371	315	284	252	193	174	151	135	120
22	395	349	296	267	237	176	159	137	123	109
23	369	327	277	250	222	161	145	126	113	100
24	345	305	259	234	207	148	133	115	104	91.8
25	321	284	241	218	193	136	123	106	95.5	84.6
26	297	263	224	202	179	126	114	98.3	88.3	78.2
27	275	244	208	187	166	117	105	91.2	81.9	72.6
28	256	227	193	174	154	109	98.0	84.8	76.2	67.5
29	239	212	180	162	144	101	91.3	79.0	71.0	62.9
30	223	198	168	152	134	94.7	85.4	73.9	66.3	58.8
32	196	174	148	133	118	83.2	75.0	64.9	58.3	51.7
34	174	154	131	118	104				51.7	45.8
36	155	137	117	105	93.1					
38	139	123	105	94.5	83.6					
40	125	111	94.6	85.3	75.4					
Properties										
r_{my}, in.	2.68	2.73	2.78	2.81	2.84	1.92	1.97	2.03	2.05	2.08
r_{mx}/r_{my}	1.22	1.22	1.22	1.21	1.21	1.60	1.59	1.58	1.58	1.57
$\phi_b M_{nx}$, kip-ft	167	140	110	93.5	76.6	133	112	88.7	75.9	62.4
$\phi_b M_{ny}$, kip-ft	140	117	92.1	78.7	64.5	87.3	74.2	59.0	50.4	41.4
$P_{ex}(K_x L_x)^2/10^4$	38.3	34.0	28.9	26.0	22.9	28.4	25.3	21.5	19.3	16.9
$P_{ey}(K_y L_y)^2/10^4$	25.7	22.8	19.5	17.6	15.6	11.1	9.95	8.53	7.73	6.81

Note: Heavy line indicates Kl/r equal to or greater than 200.

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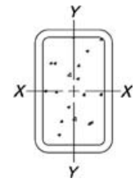
$F_y = 46$ ksi
$f'_c = 5$ ksi
$\phi_c = 0.85$

Table 4-13 (cont.). Composite Rectangular (and Square) HSS Design Strength in Axial Compression, $\phi_c P_n$, kips



Shape	HSS8×8×						HSS8×6×					
	5/8	1/2	3/8	5/16	1/4	3/16	5/8	1/2	3/8	5/16	1/4	3/16
t_{design}, in.	0.581	0.465	0.349	0.291	0.233	0.174	0.581	0.465	0.349	0.291	0.233	0.174
Wt/ft	59.1	48.7	37.6	31.8	25.8	19.6	50.6	41.9	32.5	27.5	22.4	17.1
Effective length KL (ft) with respect to least radius of gyration, r_{my}												
0	809	708	599	541	482	421	666	582	490	442	392	339
6	775	678	574	519	462	403	619	542	457	412	365	315
7	763	668	566	511	455	397	603	528	446	401	356	307
8	750	657	556	502	447	390	585	513	433	390	346	298
9	735	644	545	492	438	382	565	496	419	377	334	289
10	719	629	533	481	429	373	544	478	404	364	322	278
11	701	614	520	470	418	364	521	458	388	349	309	267
12	682	598	506	457	407	354	497	438	371	334	296	255
13	662	581	492	444	395	343	472	417	353	318	282	242
14	641	562	477	430	382	332	447	395	335	301	267	230
15	620	544	461	416	369	321	422	373	317	285	252	217
16	597	524	445	401	356	309	396	351	298	268	238	204
17	574	504	428	386	342	297	370	328	280	252	223	191
18	551	484	411	371	329	285	345	306	261	235	208	178
19	527	464	393	355	314	272	320	285	243	219	194	166
20	503	443	376	339	300	260	296	264	226	203	179	153
21	480	422	358	323	286	247	272	243	208	187	166	141
22	456	401	341	307	272	234	249	223	192	172	152	130
23	432	381	324	292	258	222	228	204	175	157	139	119
24	409	360	306	276	244	210	209	187	161	144	128	109
25	386	340	289	261	230	198	193	173	148	133	118	100
26	363	320	273	246	216	186	178	160	137	123	109	92.8
27	341	301	256	231	203	174	165	148	127	114	101	86.1
28	319	282	240	217	190	163	154	138	118	106	93.9	80.0
29	298	263	225	202	178	152	143	128	110	98.9	87.5	74.6
30	278	246	210	189	166	142	134	120	103	92.5	81.8	69.7
32	245	216	184	166	146	125	118	105	90.6	81.3	71.9	61.3
34	217	192	163	147	129	110	104	93.3	80.2	72.0	63.7	54.3
36	193	171	146	131	115	98.5	92.9	83.2	71.6	64.2	56.8	48.4
38	173	153	131	118	103	88.4		74.7	64.2	57.6	51.0	43.5
40	156	138	118	106	93.3	79.8				52.0	46.0	39.2
Properties												
r_{my}, in.	2.99	3.04	3.10	3.13	3.15	3.18	2.27	2.32	2.38	2.40	2.43	2.46
r_{mx}/r_{my}	1.00	1.00	1.00	1.00	1.00	1.00	1.26	1.25	1.25	1.25	1.25	1.24
$\phi_b M_{nx}$, kip-ft	154	129	101	86.6	70.7	54.2	125	105	83.1	71.1	58.3	44.9
$\phi_b M_{ny}$, kip-ft	154	129	101	86.6	70.7	54.2	102	85.9	68.3	58.3	48.0	36.9
$P_{ex}(K_x L_x)^2/10^4$	32.2	28.6	24.3	21.9	19.4	16.6	24.6	21.9	18.6	16.8	14.8	12.6
$P_{ey}(K_y L_y)^2/10^4$	32.2	28.6	24.3	21.9	19.4	16.6	15.6	13.9	11.9	10.8	9.51	8.12

Note: Heavy line indicates Kl/r equal to or greater than 200.

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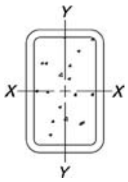

Table 4-13 (cont.). Composite Rectangular (and Square) HSS Design Strength in Axial Compression, $\phi_c P_n$, kips

$F_y = 46$ ksi
$f'_c = 5$ ksi
$\phi_c = 0.85$

Shape		HSS8×4×						HSS7×7×					
		5/8	1/2	3/8	5/16	1/4	3/16	5/8	1/2	3/8	5/16	1/4	3/16
t_{design}, in.		**0.581**	**0.465**	**0.349**	**0.291**	**0.233**	**0.174**	**0.581**	**0.465**	**0.349**	**0.291**	**0.233**	**0.174**
Wt/ft		**42.1**	**35.1**	**27.4**	**23.3**	**19.0**	**14.5**	**50.6**	**41.9**	**32.5**	**27.5**	**22.4**	**17.1**
Effective length KL (ft) with respect to least radius of gyration, r_{my}	**0**	527	459	383	343	301	256	669	586	494	445	395	342
	6	448	448	330	296	260	222	633	554	467	422	374	324
	7	422	372	313	280	247	210	620	543	458	413	367	317
	8	395	349	294	264	232	198	605	531	448	404	358	310
	9	366	324	274	246	217	185	589	517	437	394	349	302
	10	336	299	254	227	201	171	572	502	424	382	339	293
	11	306	273	233	209	184	157	553	486	411	370	329	283
	12	276	247	211	190	168	143	534	469	397	358	317	273
	13	246	222	191	171	152	129	513	451	382	344	305	263
	14	218	198	171	153	136	116	492	433	366	331	293	252
	15	191	175	151	136	121	103	470	414	351	316	280	241
	16	168	154	133	120	106	90.6	447	394	334	302	267	229
	17	149	136	118	106	94.3	80.3	425	375	318	287	254	218
	18	133	121	105	94.7	84.1	71.6	402	355	302	272	241	206
	19	119	109	94.4	85.0	75.5	64.3	379	335	285	257	228	194
	20	108	98.2	85.2	76.7	68.1	58.0	357	316	289	242	214	183
	21	97.5	89.1	77.3	69.6	61.8	52.6	334	296	252	228	201	171
	22	88.9	81.2	70.4	63.4	56.3	47.9	312	277	236	213	189	160
	23	81.3	74.3	64.4	58.0	51.5	43.9	291	259	221	199	176	149
	24	74.7	68.2	59.2	53.3	47.3	40.3	270	240	205	185	164	139
	25	68.8	62.9	54.5	49.1	43.6	37.1	250	223	190	172	152	128
	26		58.1	50.4	45.4	40.3	64.3	231	206	176	159	140	119
	27				42.1	37.4	31.8	214	191	163	147	130	110
	28						29.6	199	177	152	137	121	102
	29							186	165	142	128	113	95.3
	30							173	155	132	119	105	89.1
	32							152	136	116	105	92.6	78.3
	34							135	120	103	93.0	82.1	69.3
	36							120	107	91.9	82.9	73.2	61.9
	38							108	96.3	82.4	74.4	65.7	55.5
	40							97.5	86.9	74.4	67.2	59.3	50.1
Properties													
r_{my}, in.		1.51	1.56	1.61	1.63	1.66	1.69	2.58	2.63	2.69	2.72	2.75	2.77
r_{mx}/r_{my}		1.75	1.74	1.73	1.73	1.72	1.70	1.00	1.00	1.00	1.00	1.00	1.00
$\phi_b M_{nx}$, kip-ft		94.5	81.1	64.9	55.5	45.9	35.2	114	96.3	76.2	65.2	53.5	41.1
$\phi_b M_{ny}$, kip-ft		57.3	49.3	39.7	34.2	28.3	21.8	114	96.3	76.2	65.2	53.5	41.1
$P_{ex}(K_x L_x)^2/10^4$		17.1	15.4	13.1	11.8	10.3	8.74	20.2	18.0	15.4	13.9	12.2	10.4
$P_{ey}(K_y L_y)^2/10^4$		5.54	5.05	4.38	3.98	3.51	2.98	20.2	18.0	15.4	13.9	12.2	10.4

Note: Heavy line indicates Kl/r equal to or greater than 200.

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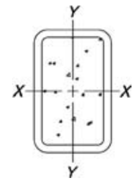
$F_y = 46$ ksi
$f'_c = 5$ ksi
$\phi_c = 0.85$

Table 4-13 (cont.). Composite Rectangular (and Square) HSS Design Strength in Axial Compression, $\phi_c P_n$, kips



Shape		HSS7×5×						HSS7×4×				
		5/8	1/2	3/8	5/16	1/4	3/16	1/2	3/8	5/16	1/4	3/16
t_{design}, in.		**0.581**	**0.465**	**0.349**	**0.291**	**0.233**	**0.174**	**0.465**	**0.349**	**0.291**	**0.233**	**0.174**
Wt/ft		**42.1**	**35.1**	**27.4**	**23.3**	**19.0**	**14.5**	**31.7**	**24.9**	**21.2**	**17.3**	**13.2**
Effective length *KL* (ft) with respect to least radius of gyration, r_{my}	**0**	537	469	394	354	312	267	411	344	308	270	230
	6	483	423	356	320	282	242	350	295	265	232	198
	7	464	408	344	309	272	233	331	279	251	220	187
	8	444	390	330	296	261	224	309	262	235	207	176
	9	422	372	314	282	249	213	287	243	219	193	164
	10	199	352	298	268	236	202	264	224	202	178	151
	11	374	331	281	253	223	191	240	205	185	164	138
	12	350	310	264	237	209	179	217	186	168	149	126
	13	324	288	246	221	195	167	194	167	152	134	113
	14	299	267	228	205	181	155	172	149	135	120	101
	15	275	245	211	189	167	143	151	131	120	106	89.6
	16	250	224	193	173	153	131	133	116	105	93.6	78.7
	17	227	204	176	158	140	119	118	102	93.4	82.9	69.7
	18	204	185	160	144	127	108	105	91.3	83.3	73.9	62.2
	19	183	166	144	129	114	97.4	94.2	81.9	74.8	66.4	55.8
	20	165	149	130	117	103	87.9	85.0	73.9	67.5	59.9	50.4
	21	150	136	118	106	93.7	79.7	77.1	67.1	61.2	54.3	45.7
	22	137	124	107	96.4	85.4	72.6	70.3	61.1	55.8	49.5	41.6
	23	125	113	98.2	88.2	78.1	66.4	64.3	55.9	51.0	45.3	38.1
	24	115	104	90.2	81.0	71.7	61.0	59.0	51.3	46.9	41.6	35.0
	25	106	95.7	83.1	74.7	66.1	56.2	54.4	47.3	43.2	38.3	32.3
	26	97.8	88.4	76.8	69.0	61.1	52.0		43.7	39.9	35.4	29.8
	27	90.6	82.0	71.3	64.0	56.7	48.2				32.9	27.6
	28	84.3	76.3	66.3	59.5	52.7	44.8					
	29	78.6	71.1	61.8	55.5	49.1	41.8					
	30	73.4	66.4	57.7	51.9	45.9	39.1					
	32			50.7	45.6	40.3	34.3					
	34						30.4					
	36											
	38											
	40											
Properties												
r_{my}, in.		1.86	1.91	1.97	1.99	2.02	2.05	1.53	1.58	1.61	1.64	1.66
r_{mx}/r_{my}		1.31	1.31	1.30	1.30	1.30	1.29	1.57	1.56	1.55	1.54	1.54
$\phi_b M_{nx}$, kip-ft		88.3	75.6	60.4	51.8	42.8	32.8	64.9	52.1	45.2	37.3	28.7
$\phi_b M_{ny}$, kip-ft		69.7	59.7	47.6	41.1	33.9	26.1	43.5	35.2	30.5	25.3	19.6
$P_{ex}(K_x L_x)^2/10^4$		14.7	13.2	11.3	10.2	8.96	7.59	10.8	9.28	8.38	7.35	6.20
$P_{ey}(K_y L_y)^2/10^4$		8.57	7.73	6.68	6.03	5.32	4.51	4.41	3.84	3.49	3.08	2.60

Note: Heavy line indicates Kl/r equal to or greater than 200.

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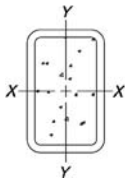

Table 4-13 (cont.). Composite Rectangular (and Square) HSS Design Strength in Axial Compression, $\phi_c P_n$, kips

$F_y = 46$ ksi
$f'_c = 5$ ksi
$\phi_c = 0.85$

Shape	HSS6×6×						HSS6×5×			
	5/8	1/2	3/8	5/16	1/4	3/16	3/8	5/16	1/4	3/16
t_{design}, in.	**0.581**	**0.465**	**0.349**	**0.291**	**0.233**	**0.174**	**0.349**	**0.291**	**0.233**	**0.174**
Wt/ft	**42.1**	**35.1**	**27.4**	**23.3**	**19.0**	**14.5**	**24.9**	**21.2**	**17.3**	**13.2**
Effective length *KL* (ft) with respect to least radius of gyration, r_{my}										
0	541	473	398	357	315	271	351	315	277	237
6	500	438	369	332	293	251	316	284	250	214
7	486	426	359	323	285	244	304	274	241	206
8	470	413	348	313	276	237	291	262	231	197
9	453	398	336	302	267	229	277	249	220	188
10	434	383	323	290	256	220	262	236	208	178
11	415	366	309	278	245	210	247	222	196	167
12	394	349	294	265	234	200	231	208	183	156
13	373	331	280	252	222	190	215	194	171	146
14	352	312	264	238	420	180	198	179	158	135
15	330	294	249	224	198	169	182	165	146	124
16	308	275	233	210	186	158	167	151	133	113
17	287	256	218	196	173	148	151	137	121	103
18	265	238	202	183	161	137	137	124	110	93.2
19	245	220	187	169	149	127	123	111	98.5	83.6
20	225	203	173	156	138	117	111	100	88.9	75.5
21	205	186	159	143	126	107	100	91.2	80.6	68.5
22	187	169	145	131	115	98.0	91.4	83.1	73.4	62.4
23	171	155	132	120	106	89.7	83.7	76.0	67.2	57.1
24	157	142	122	110	97.0	82.3	76.8	69.8	61.7	52.4
25	145	131	112	101	89.4	75.9	70.8	64.3	56.9	48.3
26	134	121	104	93.6	82.7	70.2	65.5	59.5	52.6	44.7
27	124	112	96.0	86.8	76.6	65.1	60.7	55.1	48.8	41.4
28	115	104	89.3	80.7	71.3	60.5	56.5	51.3	45.3	38.5
29	107	97.4	83.2	75.3	66.4	56.4	52.6	47.8	42.3	35.9
30	100	91.0	77.8	70.3	62.1	52.7	49.2	44.7	39.5	33.6
32	88.2	80.0	68.4	61.8	54.6	46.3				
34	78.1	70.8	60.6	54.8	48.3	41.0				
36	69.7	63.2	54.0	48.8	43.1	36.6				
38			48.5	43.8	38.7	32.8				
40										
Properties										
r_{my}, in.	2.17	2.23	2.28	2.31	2.34	2.37	1.92	1.95	1.98	2.01
r_{mx}/r_{my}	1.00	1.00	1.00	1.00	1.00	1.00	1.16	1.15	1.15	1.15
$\phi_b M_{nx}$, kip-ft	80.0	68.3	54.5	46.9	38.6	29.8	47.6	41.1	34.1	26.3
$\phi_b M_{ny}$, kip-ft	80.0	68.3	54.5	46.9	38.6	29.8	42.1	36.2	30.1	23.2
$P_{ex}(K_x L_x)^2/10^4$	11.7	10.5	9.05	8.17	7.20	6.13	7.63	6.90	6.06	5.14
$P_{ey}(K_y L_y)^2/10^4$	11.7	10.5	9.05	8.17	7.20	6.13	5.74	5.20	4.59	3.89

Note: Heavy line indicates Kl/r equal to or greater than 200.

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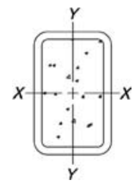
$F_y = 46$ ksi
$f'_c = 5$ ksi
$\phi_c = 0.85$

Table 4-13 (cont.). Composite Rectangular (and Square) HSS Design Strength in Axial Compression, $\phi_c P_n$, kips



Shape	HSS6×4×					HSS6×3×				
	1/2	3/8	5/16	1/4	3/16	1/2	3/8	5/16	1/4	3/16
t_{design}, in.	**0.465**	**0.349**	**0.291**	**0.233**	**0.174**	**0.465**	**0.349**	**0.291**	**0.233**	**0.174**
Wt/ft	**28.3**	**22.3**	**19.0**	**15.6**	**12.0**	**24.9**	**19.7**	**16.9**	**13.9**	**10.7**
Effective length KL (ft) with respect to least radius of gyration, r_{my}										
0	364	305	272	239	203	309	258	230	201	169
1	362	303	271	238	202	306	256	228	199	167
2	357	299	268	235	199	299	250	223	195	164
3	349	293	262	230	195	287	241	215	188	158
4	338	284	254	223	189	271	228	204	179	150
5	324	273	244	214	182	252	213	191	167	141
6	308	260	233	205	174	231	196	176	154	130
7	290	245	220	194	164	207	178	159	140	119
8	271	230	207	182	154	184	158	143	126	107
9	251	213	192	169	143	160	139	126	111	94.4
10	230	196	177	156	132	137	120	109	97.0	82.4
11	208	179	161	142	120	115	103	93.1	83.2	70.9
12	188	161	146	129	109	96.9	86.4	78.4	70.3	60.1
13	167	144	131	116	97.9	82.5	73.6	66.8	59.9	51.2
14	148	128	117	103	87.2	71.2	63.5	57.6	51.7	44.1
15	129	113	103	91.3	76.8	62.0	55.3	50.2	45.0	38.4
16	113	99.0	90.4	80.3	67.5	54.5	48.6	44.1	39.6	33.8
17	100	87.7	80.0	71.1	59.8	48.3	43.0	39.1	35.0	29.9
18	89.6	78.2	71.4	63.4	53.3	43.1	38.4	34.9	31.3	26.7
19	80.4	70.2	64.1	56.9	47.9		34.5	31.3	28.1	24.0
20	72.6	63.3	57.8	51.4	43.2				25.3	21.6
21	65.8	57.4	52.4	46.6	39.2					
22	60.0	52.3	47.8	42.5	35.7					
23	54.9	47.9	43.7	38.8	32.7					
24	50.4	44.1	40.2	35.7	30.0					
25	46.4	40.5	37.0	32.9	27.7					
26			34.2	30.4	25.6					
27					23.7					
28										
29										
30										
Properties										
r_{my}, in.	1.50	1.55	1.58	1.61	1.63	1.12	1.17	1.19	1.22	1.25
r_{mx}/r_{my}	1.39	1.38	1.37	1.37	1.37	1.76	1.74	1.74	1.72	1.71
$\phi_b M_{nx}$, kip-ft	50.4	41.1	35.5	29.4	22.8	41.7	34.2	29.7	24.8	19.3
$\phi_b M_{ny}$, kip-ft	38.0	30.8	26.7	22.3	17.3	25.1	20.8	18.2	15.2	11.9
$P_{ex}(K_x L_x)^2/10^4$	7.20	6.23	5.63	4.97	4.20	5.55	4.86	4.42	3.88	3.27
$P_{ey}(K_y L_y)^2/10^4$	3.77	3.28	3.00	2.64	2.24	1.80	1.60	1.47	1.30	1.11

Note: Heavy line indicates Kl/r equal to or greater than 200.

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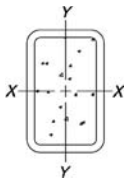

Table 4-13 (cont.). Composite Rectangular (and Square) HSS Design Strength in Axial Compression, $\phi_c P_n$, kips

$F_y = 46$ ksi
$f'_c = 5$ ksi
$\phi_c = 0.85$

Shape		HSS5$^1/_2$×5$^1/_2$×				HSS5×5×					
		$^3/_8$	$^5/_{16}$	$^1/_4$	$^3/_{16}$	$^1/_2$	$^3/_8$	$^5/_{16}$	$^1/_4$	$^3/_{16}$	$^1/_8$
t_{design}, in.		0.349	0.291	0.233	0.174	0.465	0.349	0.291	0.233	0.174	0.116
Wt/ft		24.9	21.2	17.3	13.2	28.3	22.3	19.0	15.6	12.0	8.15
Effective length KL (ft) with respect to least radius of gyration, r_{my}	0	352	316	278	238	367	308	276	242	206	169
	6	322	289	254	217	328	276	248	218	185	152
	7	312	280	246	210	315	265	238	209	178	146
	8	300	270	237	203	301	254	228	200	170	139
	9	288	259	227	194	285	241	216	190	162	132
	10	275	247	217	185	268	227	204	180	153	125
	11	261	235	206	176	251	213	192	169	144	117
	12	246	222	195	166	234	199	179	158	134	109
	13	231	208	183	156	216	184	166	146	124	101
	14	216	195	171	146	199	170	153	135	115	92.8
	15	201	182	159	136	181	155	140	124	105	84.9
	16	186	168	147	125	165	141	128	113	95.9	77.2
	17	172	155	136	116	148	128	116	102	86.8	69.8
	18	157	142	125	106	133	115	104	92.1	78.1	62.8
	19	144	130	114	96.6	119	103	93.5	82.6	70.1	56.1
	20	130	118	103	87.4	108	92.9	84.3	74.6	63.2	50.7
	21	118	107	93.5	79.3	97.5	84.3	76.5	67.6	57.4	46.0
	22	107	97.4	85.2	72.2	88.8	76.8	69.7	61.6	52.3	41.9
	23	98.4	89.1	77.9	66.1	81.3	70.2	63.8	56.4	47.8	38.3
	24	90.3	81.9	71.6	60.7	74.7	64.5	58.6	51.8	43.9	35.2
	25	83.2	75.4	66.0	55.9	68.8	59.4	54.0	47.7	40.5	32.4
	26	77.0	69.8	61.0	51.7	63.6	55.0	49.9	44.1	37.4	30.0
	27	71.4	64.7	56.5	48.0	59.0	51.0	46.3	40.9	34.7	27.8
	28	66.4	60.1	52.6	44.6	54.9	47.4	43.0	38.0	32.3	25.8
	29	61.9	56.1	49.0	41.6	51.1	44.2	40.1	35.5	30.1	24.1
	30	57.8	52.4	45.8	38.9	47.8	41.3	37.5	33.1	28.1	22.5
	32	50.8	46.1	40.3	34.1				29.1	24.7	19.8
	34	45.0	40.8	35.7	30.2						
	36				27.0						
	38										
	40										
Properties											
r_{my}, in.		2.08	2.11	2.13	2.16	1.82	1.87	1.90	1.93	1.96	1.99
r_{mx}/r_{my}		1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
$\phi_b M_{nx}$, kip-ft		45.2	39.0	32.2	24.8	45.2	36.6	31.6	26.3	20.3	14.0
$\phi_b M_{ny}$, kip-ft		45.2	39.0	32.2	24.8	45.2	36.6	31.6	26.3	20.3	14.0
$P_{ex}(K_x L_x)^2/10^4$		6.70	6.05	5.34	4.54	5.54	4.82	4.36	3.85	3.27	2.62
$P_{ey}(K_y L_y)^2/10^4$		6.70	6.05	5.34	4.54	5.54	4.82	4.36	3.85	3.27	2.62

Note: Heavy line indicates Kl/r equal to or greater than 200.

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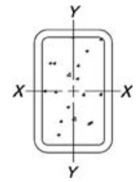
$F_y = 46$ ksi
$f'_c = 5$ ksi
$\phi_c = 0.85$

Table 4-13 (cont.). Composite Rectangular (and Square) HSS Design Strength in Axial Compression, $\phi_c P_n$, kips



Shape		HSS5×4×					HSS5×3×					
		1/2	3/8	5/16	1/4	3/16	1/2	3/8	5/16	1/4	3/16	1/8
t_{design}, in.		0.465	0.349	0.291	0.233	0.174	0.465	0.349	0.291	0.233	0.174	0.116
Wt/ft		24.9	19.7	16.9	13.9	10.7	21.5	17.2	14.8	12.2	9.40	6.45
Effective length KL (ft) with respect to least radius of gyration, r_{my}	0	316	265	237	208	176	265	222	199	173	145	117
	1	315	264	236	207	175	263	221	197	172	144	116
	2	310	260	233	204	173	256	215	193	168	141	113
	3	303	255	228	200	169	246	207	185	162	136	109
	4	293	247	221	194	164	231	196	176	153	129	104
	5	280	237	212	186	157	214	182	164	143	121	96.9
	6	266	225	202	177	150	195	167	151	132	111	89.3
	7	250	212	190	167	142	174	150	136	119	101	81.0
	8	232	198	178	156	132	153	133	121	106	90.2	72.4
	9	214	183	165	145	123	133	116	107	93.6	79.4	63.8
	10	195	168	151	133	113	113	100	92.0	81.0	68.9	55.4
	11	176	153	137	121	103	94.0	84.5	78.3	69.0	58.9	47.3
	12	158	138	124	110	92.9	79.0	71.0	65.8	58.1	49.7.	39.9
	13	140	123	111	98.1	83.2	67.3	60.5	56.1	49.5	42.3	34.0
	14	123	109	97.9	87.0	73.8	58.1	52.1	48.3	42.7	36.5	29.3
	15	107	94.9	85.7	76.3	64.8	50.6	45.4	42.1	37.2	31.8	25.5
	16	93.9	83.4	75.3	67.1	56.9	44.4	39.9	37.0	32.7	27.9	22.4
	17	83.2	73.9	66.7	59.4	50.4	39.4	35.4	32.8	28.9	24.7	19.9
	18	74.2	65.9	59.5	53.0	45.0	35.1	31.5	29.2	25.8	22.1	17.7
	19	66.6	59.2	53.4	47.6	40.4		28.3	26.2	23.2	19.8	15.9
	20	60.1	53.4	48.2	42.9	36.4					17.9	14.4
	21	54.5	48.4	43.7	38.9	33.1						
	22	49.7	44.1	39.9	35.5	30.1						
	23	45.5	40.4	36.5	32.5	27.6						
	24	41.7	37.1	33.5	29.8	25.3						
	25		34.2	30.9	27.5	23.3						
	26				25.4	21.6						
	27											
	28											
	29											
	30											
Properties												
r_{my}, in.		1.46	1.52	1.54	1.57	1.60	1.09	1.14	1.17	1.19	1.22	1.25
r_{mx}/r_{my}		1.20	1.19	1.19	1.19	1.19	1.51	1.51	1.50	1.50	1.49	1.48
$\phi_b M_{nx}$, kip-ft		37.6	30.9	26.9	22.4	17.4	30.5	25.3	22.1	18.6	14.5	10.1
$\phi_b M_{ny}$, kip-ft		32.3	26.5	23.0	19.2	15.0	21.0	17.6	15.5	13.0	10.2	7.14
$P_{ex}(K_x L_x)^2/10^4$		4.46	3.90	3.54	3.14	2.66	3.38	3.00	2.74	2.42	2.05	1.62
$P_{ey}(K_y L_y)^2/10^4$		3.13	2.74	2.49	2.21	1.88	1.48	1.33	1.22	1.09	0.925	0.739

Note: Heavy line indicates Kl/r equal to or greater than 200.

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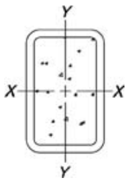

Table 4-13 (cont.). Composite Rectangular (and Square) HSS Design Strength in Axial Compression, $\phi_c P_n$, kips

$F_y = 46$ ksi
$f'_c = 5$ ksi
$\phi_c = 0.85$

Shape	HSS5×2½×			HSS4½×4½×					
	1/4	3/16	1/8	1/2	3/8	5/16	1/4	3/16	1/8
t_{design}, in.	**0.233**	**0.174**	**0.116**	**0.465**	**0.349**	**0.291**	**0.233**	**0.174**	**0.116**
Wt/ft	**11.3**	**8.55**	**6.02**	**24.9**	**19.7**	**16.9**	**13.9**	**10.7**	**7.30**
Effective length *KL* (ft) with respect to least radius of gyration, r_{my}									
0	156	128	104	317	266	238	209	177	144
1	154	127	102	298	250	223	196	167	137
2	149	123	99.3	295	247	221	194	165	135
3	142	117	94.3	289	242	217	190	162	133
4	131	109	87.6	281	236	212	186	158	129
5	119	99.1	79.8	272	228	205	180	153	125
6	106	88.4	71.1	275	232	208	183	155	126
7	92.5	77.3	62.1	261	221	199	174	147	120
8	78.9	66.2	53.1	246	209	188	165	139	113
9	65.8	55.5	44.5	230	196	176	155	131	106
10	53.7	45.5	36.5	213	182	164	145	122	98.7
11	44.4	37.6	30.1	196	168	152	134	113	91.2
12	37.3	31.6	25.3	179	154	140	123	104	83.6
13	31.8	27.0	21.6	162	140	127	112	94.5	76.0
14	27.4	23.2	18.6	146	127	115	102	85.5	68.7
15	23.9	20.2	16.2	130	114	103	91.5	76.8	61.5
16	21.0	17.8	14.2	114	101	92.0	81.7	68.4	54.7
17		15.8	12.6	101	89.4	81.5	72.3	60.6	48.4
18				90.4	79.7	72.7	64.5	54.0	43.2
19				81.1	71.6	65.3	57.9	48.5	38.8
20				73.2	64.6	58.9	52.3	43.8	35.0
21				66.4	58.6	53.4	47.4	39.7	31.7
22				60.5	53.4	48.7	43.2	36.2	28.9
23				55.4	48.8	44.5	39.5	33.1	26.4
24				50.9	44.9	40.9	36.3	30.4	24.3
25				46.9	41.3	37.7	33.4	28.0	22.4
26				43.3	38.2	34.9	30.9	25.9	20.7
					35.4	32.3	28.7	24.0	19.2
28						30.1	26.7	22.3	17.8
29								20.8	16.6
30									
Properties									
r_{my}, in.	0.999	1.03	1.05	1.61	1.67	1.70	1.73	1.75	1.78
r_{mx}/r_{my}	1.73	1.72	1.71	1.00	1.00	1.00	1.00	1.00	1.00
$\phi_b M_{nx}$, kip-ft	16.7	12.8	9.14	35.2	28.8	25.1	20.9	16.2	11.3
$\phi_b M_{ny}$, kip-ft	10.2	7.90	5.66	35.2	28.8	25.1	20.9	16.2	11.3
$P_{ex}(K_x L_x)^2/10^4$	2.08	1.73	1.38	3.81	3.34	3.04	2.68	2.27	1.82
$P_{ey}(K_y L_y)^2/10^4$	0.692	0.584	0.471	3.81	3.34	3.04	2.68	2.27	1.82

Note: Heavy line indicates Kl/r equal to or greater than 200.

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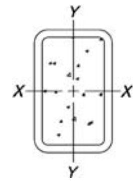
$F_y = 46$ ksi
$f'_c = 5$ ksi
$\phi_c = 0.85$

Table 4-13 (cont.). Composite Rectangular (and Square) HSS Design Strength in Axial Compression, $\phi_c P_n$, kips



Shape		HSS4×4×						HSS4×3×				
		1/2	3/8	5/16	1/4	3/16	1/8	3/8	5/16	1/4	3/16	1/8
t_{design}, in.		**0.465**	**0.349**	**0.291**	**0.233**	**0.174**	**0.116**	**0.349**	**0.291**	**0.233**	**0.174**	**0.116**
Wt/ft		**21.5**	**17.2**	**14.8**	**12.2**	**9.40**	**6.45**	**14.6**	**12.7**	**10.5**	**8.13**	**5.60**
Effective length KL (ft) with respect to least radius of gyration, r_{my}	**0**	269	226	202	177	149	120	187	167	146	122	97.9
	1	253	212	189	166	140	114	185	166	145	122	97.1
	2	249	209	187	163	138	112	181	162	141	119	94.8
	3	243	204	182	160	135	110	173	155	136	114	91.2
	4	235	197	177	155	131	106	164	147	129	108	86.3
	5	224	189	169	148	125	102	152	136	120	101	80.4
	6	223	190	170	149	126	102	138	125	110	92.6	73.7
	7	209	178	160	140	119	95.6	124	112	99.0	83.7	66.5
	8	193	166	149	131	111	89.1	109	99.0	87.9	74.5	59.1
	9	177	153	137	121	102	82.2	95.0	86.1	76.8	65.3	51.7
	10	161	139	125	111	93.5	75.2	81.0	73.7	66.1	56.4	44.5
	11	144	126	113	100	84.8	68.1	67.8	61.9	55.9	47.9	37.7
	12	128	113	102	89.9	76.2	61.1	57.0	52.0	46.9	40.2	31.7
	13	113	99.9	90.2	80.0	67.8	54.3	48.5	44.3	40.0	34.3	27.0
	14	98.0	87.6	79.3	70.5	59.8	47.8	41.9	38.2	34.5	29.5	23.3
	15	85.3	76.3	69.1	61.5	52.2	41.7	36.5	33.3	30.0	25.7	20.3
	16	75.0	67.1	60.7	54.0	45.8	36.6	32.0	29.3	26.4	22.6	17.8
	17	66.4	59.4	53.8	47.9	40.6	32.4	28.4	25.9	23.4	20.0	15.8
	18	59.3	53.0	48.0	42.7	36.2	28.9	25.3	23.1	20.9	17.9	14.1
	19	53.2	47.6	43.1	38.3	32.5	26.0			18.7	16.0	12.6
	20	48.0	42.9	38.9	34.6	29.3	23.4					11.4
	21	43.5	38.9	35.2	31.4	26.6	21.3					
	22	39.7	35.5	32.1	28.6	24.2	19.4					
	23	36.3	32.5	29.4	26.1	22.2	17.7					
	24		29.8	27.0	24.0	20.4	16.3					
	25				22.1	18.8	15.0					
	26						13.9					
	27											
	28											
	29											
	30											
Properties												
r_{my}, in.		1.41	1.47	1.49	1.52	1.55	1.58	1.11	1.13	1.16	1.19	1.21
r_{mx}/r_{my}		1.00	1.00	1.00	1.00	1.00	1.00	1.25	1.26	1.25	1.25	1.26
$\phi_b M_{nx}$, kip-ft		26.6	22.0	19.3	16.2	12.7	8.83	17.7	15.6	13.1	10.4	7.28
$\phi_b M_{ny}$, kip-ft		26.6	22.0	19.3	16.2	12.7	8.83	14.4	12.7	10.8	8.49	5.97
$P_{ex}(K_xL_x)^2/10^4$		2.48	2.21	2.01	1.79	1.52	1.21	1.67	1.54	1.37	1.16	0.920
$P_{ey}(K_yL_y)^2/10^4$		2.48	2.21	2.01	1.79	1.52	1.21	1.06	0.974	0.871	0.744	0.593

Note: Heavy line indicates Kl/r equal to or greater than 200.

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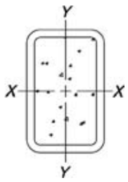

Table 4-13 (cont.).
Composite Rectangular (and Square) HSS
Design Strength in Axial Compression, $\phi_c P_n$, kips

$F_y = 46$ ksi
$f'_c = 5$ ksi
$\phi_c = 0.85$

Shape		HSS4×2½×			HSS4×2×					HSS3½×3½×				
		5/16	1/4	3/16	3/8	5/16	1/4	3/16	1/8	3/8	5/16	1/4	3/16	1/8
t_{design}, in.		**0.291**	**0.233**	**0.174**	**0.349**	**0.291**	**0.233**	**0.174**	**0.116**	**0.349**	**0.291**	**0.233**	**0.174**	**0.116**
Wt/ft		**11.6**	**9.63**	**7.49**	**12.1**	**10.5**	**8.78**	**6.85**	**4.75**	**14.6**	**12.7**	**10.5**	**8.13**	**5.60**
Effective length KL (ft) with respect to least radius of gyration, r_{my}	**0**	150	130	109	148	132	115	95.6	74.9	188	168	147	123	98.8
	1	148	129	108	145	130	113	94.0	73.6	176	157	137	116	93.1
	2	143	125	104	137	123	107	89.5	70.1	173	154	135	114	91.4
	3	135	118	98.7	124	112	98.4	82.4	64.7	167	150	131	110	88.7
	4	124	109	91.4	109	99.1	87.2	73.4	57.7	160	143	126	106	85.0
	5	112	98.5	82.9	91.8	84.3	74.7	63.2	49.9	151	136	119	100	80.5
	6	98.8	87.1	73.6	74.4	69.1	61.9	52.7	41.7	149	134	118	99.2	79.1
	7	85.0	75.4	63.8	58.1	54.7	49.5	42.5	33.8	137	123	109	91.7	73.0
	8	71.5	63.8	54.2	44.5	42.1	38.3	33.1	26.4	124	112	99.2	83.8	66.6
	9	58.7	52.8	45.1	35.2	33.2	30.3	26.2	20.9	111	101	89.3	75.6	59.9
	10	47.6	42.8	36.7	28.5	26.9	24.5	21.2	16.9	98.0	89.6	79.5	67.4	53.3
	11	39.3	35.4	30.3	23.5	22.3	20.3	17.5	14.0	85.5	78.5	69.9	59.4	46.8
	12	33.0	29.8	25.5	19.8	18.7	17.0	14.7	11.8	73.6	67.9	60.7	51.7	40.6
	13	28.1	25.4	21.7				12.5	10.0	62.7	58.0	52.0	44.3	34.8
	14	24.3	21.9	18.7						54.1	50.0	44.8	38.2	30.0
	15	21.1	19.0	16.3						47.1	43.6	39.0	33.3	26.1
	16		16.7	14.3						41.4	38.3	34.3	29.3	23.0
	17									36.7	33.9	30.4	25.9	20.3
	18									32.7	30.2	27.1	23.1	18.1
	19									29.4	27.1	24.3	20.7	16.3
	20									26.5	24.5	22.0	18.7	14.7
	21									24.0	22.2	19.9	17.0	13.3
	22											18.2	15.5	12.1
	23													
	24													
	25													
	26													
	27													
	28													
	29													
	30													
Properties														
r_{my}, in.		0.947	0.973	0.999	0.729	0.754	0.779	0.804	0.830	1.26	1.29	1.32	1.35	1.37
r_{mx}/r_{my}		1.46	1.45	1.44	1.77	1.75	1.75	1.73	1.72	1.00	1.00	1.00	1.00	1.00
$\phi_b M_{nx}$, kip-ft		13.7	11.7	9.21	13.2	11.8	10.1	8.07	5.73	16.2	14.3	12.1	9.52	6.66
$\phi_b M_{ny}$, kip-ft		9.83	8.38	6.66	7.97	7.18	6.18	4.93	3.52	16.2	14.3	12.1	9.52	6.66
$P_{ex}(K_x L_x)^2/10^4$		1.30	1.16	0.990	1.15	1.07	0.960	0.819	0.646	1.37	1.26	1.13	0.959	0.763
$P_{ey}(K_y L_y)^2/10^4$		0.613	0.553	0.474	0.369	0.348	0.317	0.273	0.219	1.37	1.26	1.13	0.959	0.763

Note: Heavy line indicates Kl/r equal to or greater than 200.

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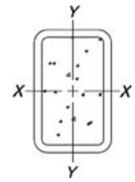
$F_y = 46$ ksi
$f'_c = 5$ ksi
$\phi_c = 0.85$

Table 4-13 (cont.). Composite Rectangular (and Square) HSS Design Strength in Axial Compression, $\phi_c P_n$, kips



Shape	HSS3×3×					HSS2$^1/_2$×2$^1/_2$×			
	$^3/_8$	$^5/_{16}$	$^1/_4$	$^3/_{16}$	$^1/_8$	$^5/_{16}$	$^1/_4$	$^3/_{16}$	$^1/_8$
t_{design}, in.	**0.349**	**0.291**	**0.233**	**0.174**	**0.116**	**0.291**	**0.233**	**0.174**	**0.116**
Wt/ft	**12.1**	**10.5**	**8.78**	**6.85**	**4.75**	**8.40**	**7.08**	**5.57**	**3.90**
Effective length *KL* (ft) with respect to least radius of gyration, r_{my}									
0	151	136	118	99.2	78.5	105	91.8	76.8	60.4
1	142	127	111	92.7	73.6	98.0	85.5	71.5	56.3
2	138	124	108	90.4	71.8	94.3	82.4	69.0	54.4
3	132	118	103	86.8	69.0	88.4	77.4	65.0	51.3
4	124	111	97.5	82.0	65.1	80.8	71.1	59.8	47.2
5	115	103	90.4	76.2	60.5	71.9	63.6	53.8	42.5
6	109	98.7	87.0	73.5	58.2	65.2	58.3	49.7	39.3
7	96.9	87.9	77.9	65.9	52.3	55.0	49.5	42.4	33.7
8	84.6	77.0	68.5	58.2	46.1	45.1	41.0	35.4	28.1
9	72.5	66.2	59.2	50.5	40.1	36.0	33.0	28.7	22.9
10	61.0	55.9	55.9	43.1	34.2	29.2	26.8	23.3	18.6
11	50.5	46.4	46.4	36.1	28.7	24.1	22.1	19.2	15.4
12	42.4	39.0	39.0	30.3	24.1	20.3	18.6	16.2	12.9
13	36.2	33.2	33.2	25.8	20.5	17.3	15.8	13.8	11.0
14	31.2	28.7	28.7	22.3	17.7	14.9	13.7	11.9	9.48
15	27.2	25.0	25.0	19.4	15.4		11.9	10.3	8.26
16	23.9	21.9	21.9	17.1	13.6				7.26
17	21.1	19.4	19.4	15.1	12.0				
18		17.3	17.3	13.5	10.7				
19				12.1	9.62				
20									
21									
22									
23									
24									
25									
26									
27									
28									
29									
30									
Properties									
r_{my}, in.	1.06	1.08	1.11	1.14	1.17	0.880	0.908	0.937	0.965
r_{my}, in.	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
$\phi_b M_{nx}$, kip-ft	11.2	10.00	8.56	6.80	4.83	6.49	5.62	4.55	3.27
$\phi_b M_{ny}$, kip-ft	11.2	10.00	8.56	6.80	4.83	6.49	5.62	4.55	3.27
$P_{ex}(K_x L_x)^2/10^4$	0.787	0.731	0.659	0.564	0.449	0.379	0.347	0.300	0.240
$P_{ey}(K_y L_y)^2/10^4$	0.787	0.731	0.659	0.564	0.449	0.379	0.347	0.300	0.240

Note: Heavy line indicates Kl/r equal to or greater than 200.

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Since $\lambda < \lambda_p$, the flanges are compact. From LRFD Specification Section B5.1, for web compactness

$$\begin{aligned}\lambda_p &= 3.76\sqrt{\frac{E}{F_y}}\\ &= 3.76\sqrt{\frac{29{,}000\,\text{ksi}}{50\,\text{ksi}}}\\ &= 90.6\end{aligned}$$

Since $\lambda < \lambda_p$, the web is compact. Since both the flanges and the web are compact, the W18×40 is compact.

From LRFD Specification Section F1.2a,

$$\begin{aligned}L_p &= 1.76 r_y\sqrt{\frac{E}{F_y}}\\ &= \frac{1.76\,(1.27\,\text{in.})}{12\,\text{in./ft}}\sqrt{\frac{29{,}000\,\text{ksi}}{50\,\text{ksi}}}\\ &= 4.49\,\text{ft}\end{aligned}$$

Since $L_b < L_p$, the beam is braced.

For a beam that is both braced and compact, the flexural yielding limit-state controls the flexural design strength. From LRFD Specification Section F1.1, the flexural yielding design strength $\phi_b M_n$ is

$$\begin{aligned}\phi_b M_n &= \phi_b F_y Z_x\\ &= \frac{0.9(50\,\text{ksi})\left(78.4\,\text{in.}^3\right)}{12\,\text{in./ft}}\\ &= 294\,\text{kip-ft}\end{aligned}$$

From LRFD Specification Section F2, the shear yielding design strength $\phi_v V_n$ is

$$\begin{aligned}\phi_v V_n &= \boxed{\phi_v 0.6 F_y d t_w}\\ &= \boxed{0.9(0.6)(50\,\text{ksi})(17.9\,\text{in.})(0.315\,\text{in.})}\\ &= \boxed{152\,\text{kips}}\end{aligned}$$

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Solution b: As determined in solution a, the beam is compact.

As determined in solution a, $L_p = 4.49$ ft. From LRFD Specification Section F1.2a,

$$\begin{aligned}L_r &= \frac{r_y X_1}{F_L}\sqrt{1+\sqrt{1+X_2 F_L^2}}\\ &= \frac{(1.27\,\text{in.})(1{,}810\,\text{ksi})}{(50\,\text{ksi}-10\,\text{ksi})(12\,\text{in./ft})}\sqrt{1+\sqrt{1+\left(\frac{0.0172}{\text{ksi}^2}\right)(50\,\text{ksi}-10\,\text{ksi})^2}}\\ &= 12.0\,\text{ft}\end{aligned}$$

For a beam that is compact, but unbraced with $L_p < L_b \leq L_r$, either the flexural yielding limit-state or the inelastic lateral-torsional buckling limit-state controls the flexural design strength. For lateral-torsional buckling, the moment gradient effect can be calculated using LRFD Specification Equation F1-3, where

$$C_b = \frac{12.5M_{\max}}{2.5M_{\max} + 3M_A + 4M_B + 3M_C}$$

For the center-span beam segment, using relative values determined from the moment diagram for a uniformly loaded simple-span beam ($M_{\max} = 1.00$, $M_A = 0.972$, $M_B = 1.00$, and $M_C = 0.972$)

$$\begin{aligned} C_b &= \frac{12.5(1.00)}{2.5(1.00) + 3(0.972) + 4(1.00) + 3(0.972)} \\ &= 1.01 \end{aligned}$$

For the end-span beam segments, using relative values determined from the moment diagram for a uniformly loaded simple-span beam ($M_{\max} = 0.889$, $M_A = 0.306$, $M_B = 0.556$, and $M_C = 0.750$)

$$\begin{aligned} C_b &= \frac{12.5(0.889)}{2.5(0.889) + 3(0.306) + 4(0.556) + 3(0.750)} \\ &= 1.46 \end{aligned}$$

Thus, the center span with $C_b = 1.01$ is more critical.

As determined in solution a, $\phi_b M_p = 294$ kip-ft. From LRFD Specification Section F1.2a,

$$\begin{aligned} \phi_b M_r &= \phi_b F_L S_x \\ &= \frac{0.9(50\text{ ksi} - 10\text{ ksi})\left(68.4\text{ in.}^3\right)}{12\text{ in./ft}} \\ &= 205\text{ kip-ft} \end{aligned}$$

and the flexural design strength $\phi_b M_n$ is

$$\begin{aligned} \phi_b M_n &= C_b\left[\phi_b M_p - (\phi_b M_p - \phi_b M_r)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \leq \phi_b M_p \\ &= (1.01)\left[(294\text{ kip-ft}) - (294\text{ kip-ft} - 250\text{ kip-ft})\left(\frac{11.7\text{ ft} - 4.48\text{ ft}}{12.0\text{ ft} - 4.48\text{ ft}}\right)\right] \\ &\leq 294\text{ kip-ft} \\ &= 211\text{ kip-ft} \leq 294\text{ kip-ft} \\ &= 211\text{ kip-ft} \end{aligned}$$

As determined in solution a, the shear yielding design strength $\phi_v V_n$ is

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$$\phi_v V_n = \boxed{152\text{ kips}}$$

Solution c: As determined in solution a, the beam is compact.

As determined in solution b, $L_r = 12.0$ ft

For a beam that is compact, but unbraced with $L_b > L_r$, the elastic lateral-torsional buckling limit-state controls the flexural design strength.

The moment gradient effect can be calculated using LRFD Specification Equation F1-3, where

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C}$$

Using relative values determined from the moment diagram for a uniformly loaded simple-span beam ($M_{max} = 1.00$, $M_A = 0.500$, $M_B = 1.00$, and $M_C = 0.500$)

$$\begin{aligned} C_b &= \frac{12.5(1.00)}{2.5(1.00) + 3(0.750) + 4(1.00) + 3(0.750)} \\ &= 1.14 \end{aligned}$$

From LRFD Specification Section F1.2b, the flexural design strength $\phi_b M_n$ is

$$\begin{aligned} \phi_b M_n &= \phi_b \frac{C_b S_x X_1 \sqrt{2}}{\frac{L_b}{r_y}} \sqrt{1 + \frac{X_1^2 X_2}{2\left(\frac{L_b}{r_y}\right)}} \\ &= 0.9 \frac{(1.14)\left(68.4 \text{ in.}^3\right)(1{,}810 \text{ ksi})\sqrt{2}}{\frac{(35 \text{ ft} \times 12 \text{ in./ft})}{(1.27 \text{ in.})}} \sqrt{1 + \frac{(1{,}810 \text{ ksi})^2 \left(\frac{0.0172}{\text{ksi}^2}\right)}{2\left(\frac{(35 \text{ ft} \times 12 \text{ in./ft})}{(1.27 \text{ in.})}\right)^2}} \\ &= 50.8 \text{ kip-ft} \end{aligned}$$

As determined in solution a, the shear yielding design strength $\phi_v V_n$ is

$$\phi_v V_n = 254 \text{ kips}$$

Comments: The preceding calculations can be simplified using Tables 5-1, 5-3, 5-4 and 5-5. For solution a, from Table 5-3, a W18×40 has $L_p = 4.49$ ft. Thus, for $L_b = 2$ ft,

$$\phi_b M_n = 294 \text{ kip-ft}$$

and

$$\phi_v V_n = 254 \text{ kips}$$

For solution b, from Table 5-3, a W18×40 has $L_p = 4.49$ ft, $\phi_b M_{px} = 294$ kip-ft, $L_r = 12.0$ ft, $\phi_b M_{rx} = 205$ kip-ft and $BF = 11.7$ kips. From solution b, $C_b = 1.46$ for the end span and 1.01 for the center span. Thus, for $L_b = 11.7$ ft, Rev. 11/1/02

$$\begin{aligned} \phi_b M_n &= C_b[\phi_b M_{px} - BF(L_b - L_p)] \le \phi_b M_{px} \\ &= (1.01)\left[(294 \text{ kip-ft}) - 11.7 \text{ kips}\,(11.7 \text{ ft} - 4.48 \text{ ft})\right] \le 294 \text{ kip-ft} \\ &= 211 \text{ kip-ft} \le 294 \text{ kip-ft} \\ &= 211 \text{ kip-ft} \end{aligned}$$

and

$$\phi_v V_n = 254 \text{ kips}$$

$$L'_p = L_p - (L_p - L_r)\left(\frac{\phi_b M_p - \phi_b M'_p}{\phi_b M_p - \phi_b M_r}\right)$$
$$= \left[(5.86\text{ ft}) - (5.86\text{ ft} - 15.4\text{ ft})\left(\frac{401\text{ kip-ft} - 398\text{ kip-ft}}{401\text{ kip-ft} - 279\text{ kip-ft}}\right)\right]$$
$$= 6.09\text{ ft}$$

Since $L_b < L'_p$, the beam is braced as assumed.

From LRFD Specification Section F2, the shear yielding design strength $\phi_v V_n$ is

$$\phi_v V_n = \boxed{\begin{aligned} &\phi_v 0.6 F_y d t_w \\ &0.9(0.6)(50\text{ ksi})(20.6\text{ in.})(0.350\text{ in.}) \\ &195\text{ kips}\end{aligned}}$$

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Solution b: As determined in solution a, the beam has non-compact flanges.

As determined in solution a, $L'_p = 6.09$ ft and $L_r = 15.4$ ft.

For a beam that has non-compact flanges with $L'_p < L_b \leq L_r$, either the flange-local buckling limit-state or the inelastic lateral-torsional buckling limit-state controls the flexural design strength. As determined in solution a, the flexural design strength for flange local buckling is

$$\phi_b M_n = 398\text{ kip-ft}$$

For inelastic lateral-torsional buckling, the moment gradient effect can be calculated using LRFD Specification Equation F1-3, where

$$C_b = \frac{12.5 M_{\max}}{2.5 M_{\max} + 3M_A + 4M_B + 3M_C}$$

For the center-span beam segment, the moment is constant and $C_b = 1$. For the end-span beam segments, using relative values determined from the moment diagram for a simple-span beam with equal concentrated loads at the third points ($M_{\max} = 1.00$, $M_A = 0.250$, $M_B = 0.500$, and $M_C = 0.750$)

$$C_b = \frac{12.5(1.00)}{2.5(1.00) + 3(0.250) + 4(0.500) + 3(0.750)}$$
$$= 1.67$$

Thus, the center span with $C_b = 1$ is more critical.

As determined in solution a, $\phi_b M_p = 401$ kip-ft, $L_p = 5.86$ ft, $\phi_b M_r = 279$ kip-ft, $L_r = 15.4$ ft, $\phi_b M'_p = 398$ kip-ft and $L'_p = 6.09$ ft. From LRFD Specification Appendix F1, the flexural design strength $\phi_b M_n$ is

$$\phi_b M_n = C_b\left[\phi_b M_p - (\phi_b M_p - \phi_b M_r)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \leq \phi_b M'_p$$
$$= (1)\left[(401\text{ kip-ft}) - (401\text{ kip-ft} - 279\text{ kip-ft})\left(\frac{13.3\text{ ft} - 5.86\text{ ft}}{15.4\text{ ft} - 5.86\text{ ft}}\right)\right]$$

$$= 0.9\frac{(1.14)\left(93.0\,\text{in.}^3\right)(1{,}450\,\text{ksi})\sqrt{2}}{\dfrac{(40\,\text{ft}\times 12\,\text{in./ft})}{(1.66\,\text{in.})}}\sqrt{1+\frac{(1{,}450\,\text{ksi})^2\left(\dfrac{0.0436}{\text{ksi}^2}\right)}{2\left(\dfrac{(40\,\text{ft}\times 12\,\text{in./ft})}{(1.66\,\text{in.})}\right)^2}}$$
$$\leq 398\,\text{kip-ft}$$
$$= 70.2\,\text{kip-ft}$$

As determined in solution a, the shear yielding design strength $\phi_v V_n$ is

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$$\phi_v V_n = \boxed{195\ \text{kips}}$$

Comments: The preceding calculations can be simplified using Tables 5-1, 5-3, 5-4 and 5-5. For solution a, from Table 5-3, a W21×48 is indicated as non-compact and provides $\phi_b M_n = 401$ kip-ft, $L'_p = 6.09$ ft, $\phi_b M_r = 279$ kip-ft and $L_r = 15.4$ ft. These values can then be used as illustrated in solution a to determine

$$\phi_b M'_n = 398\,\text{kip-ft}$$

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Also, $\phi_v V_n = \boxed{195\,\text{kips}}$ is given.

For solutions b and c, the values tabulated in Tables 5-1 and 5-3 can be used in a similar fashion as illustrated in solutions b and c.

EXAMPLE 5.3. W-shape flexural member design (selection by moment of inertia for strong-axis bending).

Given: Select an ASTM A992 W-shape flexural member ($F_y = 50$ ksi, $F_u = 65$ ksi) for a required flexural strength M_u of 250 kip-ft, a required shear strength V_u of 40 kips, and a deflection limit of 1 in. For the deflection calculations, assume the load is a uniformly distributed service load of 2 kips/ft and the length of the simple span is 30 ft. For the strength calculations, assume the beam is braced.

Solution: From Table 5-17, Diagram 1, the maximum deflection $\Delta_{\max}$ occurs at mid-span and can be calculated as:

$$\Delta_{\max} = \frac{5wI^4}{384EI}$$

Rearranging and substituting $\Delta_{\max} = 1$ in.,

$$I_{\min} = \frac{5(2\,\text{kips/ft})(30\,\text{ft})^4\times(12\,\text{in./ft})^3}{384\,(29{,}000\,\text{ksi})(1\,\text{in.})}$$
$$= 1{,}260\,\text{in.}^4$$

From Table 5-2, a W24×55 has

$$I_x = 1{,}360\,\text{in.}^4 > 1{,}260\,\text{in.}^4\ \textbf{o.k.}$$

Because the W24×55 is braced (given) and compact, $\phi_b M_n = \phi_b M_{px}$.

From Table 5-2,

$$\phi_b M_{px} = 398\,\text{kip-ft} > 250\,\text{kip-ft}\ \textbf{o.k.}$$
$$\phi_v V_n = \boxed{252\,\text{kips}} > 40\,\text{kips}\ \textbf{o.k.}$$

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Thus, the W24×55 flexural member is **o.k.**

Comments: Note that end connection limit states, such as block shear rupture and bolt bearing strength must also be checked.

EXAMPLE 5.4. W-shape flexural member design (selection using plots of $\phi_b M_p$ vs. L_b for strong-axis bending).

Given: Select an ASTM A992 W-shape flexural member ($F_y = 50$ ksi, $F_u = 65$ ksi) for a required flexural strength M_u of 150 kip-ft, a required shear strength V_u of 20 kips, and a deflection limit of 1 in. For the deflection calculations, assume the load is a uniformly distributed service load of 2 kips/ft and the length of the simple span is 20 ft. For the strength calculations, assume the beam is braced at the ends and midpoint only ($L_b = 10$ ft).

Solution: From Table 5-5, for an unbraced length $M_u = 150$ kip-ft and $L_b = 10$ ft, a W16×31 with $C_b = 1$ has $\phi_b M_{nx} \approx 150$ kip-ft. Since $C_b > 1$, the actual flexural strength will be higher, so the W16×31 is **o.k.** for flexural design strength.

From Table 5-17, Diagram 1, the maximum deflection Δ_{max} occurs at mid-span and can be calculated as:

$$\begin{aligned}\Delta_{max} &= \frac{5wI^4}{384EI} \\ &= \frac{5(2\,\text{kips/ft})(20\,\text{ft})^4 \times (12\,\text{in./ft})^3}{384\,(29{,}000\,\text{ksi})\left(375\,\text{in.}^4\right)} \\ &= 0.662\,\text{in.} < 1\,\text{in.}\ \textbf{o.k.}\end{aligned}$$

From Table 5-2,

$$\phi_v V_n = \boxed{118\,\text{kips}} > 20\,\text{kips}\ \textbf{o.k.}$$

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Thus, the W16×31 flexural member is **o.k.**

Comments: Note that end connection limit states, such as block shear rupture and bolt bearing strength must also be checked.

EXAMPLE 5.5. W-shape flexural member design (determination of maximum end reaction for strong-axis bending).

Given: For an ASTM A992 W21×62, estimate the beam end reaction based upon the maximum simple-span end reaction assuming the beam is subject to its maximum factored uniform load, is braced, and spans

a) 40 ft.
b) 30 ft.
c) 8 ft.

I_x

Table 5-2 (cont.). W-Shapes Selection by I_x

$F_y = 50$ ksi
$\phi_b = 0.90$
$\phi_v = 0.90$

Shape	X-X Axis								Y-Y Axis		
	I_x	Z_x	$\phi_b M_{px}$	$\phi_b M_{rx}$	L_p	L_r	BF	$\phi_v V_n$	I_y	Z_y	$\phi_b M_{py}$
	in.^{4}	in.^{3}	kip-ft	kip-ft	ft	ft	kips	kips	in.^{4}	in.^{3}	kip-ft
W24×55	**1360**	**135**	**506**	**345**	**4.73**	**12.9**	**19.8**	**252**	**29.1**	**13.4**	**46.7**
W21×62	1330	144	540	381	6.25	16.7	15.2	227	57.5	21.7	78.8
W18×76	1330	163	611	438	9.22	24.8	11.1	209	152	42.2	155
W16×89	1310	176	660	468	8.80	27.5	10.3	238	163	48.2	177
W14×109	1240	192	720	519	13.2	43.2	6.69	203	447	92.7	344
W12×136	1240	214	803	558	11.2	55.7	5.49	286	398	98.0	361
W21×57	1170	129	484	333	4.77	13.2	17.8	231	30.7	14.9	52.6
W18×71	1170	146	548	381	6.00	17.8	14.1	247	60.3	24.7	88.9
W21×55	**1140**	**126**	**473**	**330**	**6.11**	**16.1**	**14.3**	**211**	**48.4**	**18.4**	**66.4**
W16×77	1120	151	566	405	8.69	25.3	9.68	203	138	41.2	151
W14×99††	1110	173	646*	471	13.5**	40.6	6.56	186	402	83.6	311
W18×65	1070	133	499	351	5.97	17.1	13.3	224	54.8	22.5	81.0
W12×120	1070	186	698	489	11.1	50.0	5.36	251	345	85.4	315
W14×90††	999	157	576*	429	15.1**	38.4	6.85	166	362	75.6	281
W21×50	**989**	**111**	**416**	**285**	**4.59**	**12.5**	**16.5**	**213**	**24.9**	**12.2**	**43.0**
W18×60	984	123	461	324	5.93	16.6	12.9	204	50.1	20.6	74.8
W16×67	963	131	491	354	8.65	23.8	9.04	174	119	35.5	131
W21×48††	**959**	**107**	**398***	**279**	**6.09****	**15.4**	**13.2**	**195**	**38.7**	**14.9**	**53.5**
W12×106	933	164	615	435	11.0	44.9	5.31	212	301	75.1	277
W18×55	890	112	420	295	5.90	16.1	12.2	191	44.9	18.5	66.9
W14×82	882	139	521	369	8.76	29.6	7.30	197	148	44.8	165
W21×44	**847**	**95.8**	**359**	**246**	**4.45**	**12.0**	**15.0**	**196**	**20.7**	**10.2**	**35.8**
W12×96	833	147	551	393	10.9	41.3	5.20	189	270	67.5	250
W18×50	800	101	379	267	5.83	15.6	11.5	173	40.1	16.6	60.2
W14×74	796	126	473	336	8.76	27.9	7.12	173	134	40.5	150
W16×57	758	105	394	277	5.65	16.6	10.7	190	43.1	18.9	68.1
W12×87	740	132	495	354	10.8	38.4	5.13	174	241	60.4	223
W14×68	722	115	431	309	8.69	26.4	6.91	157	121	36.9	136
W10×112	716	147	551	378	9.47	56.5	3.68	232	236	69.2	255
W18×46	712	90.7	340	236	4.56	12.6	12.9	176	22.5	11.7	41.8
W12×79	662	119	446	321	10.8	35.7	5.03	157	216	54.3	201
W16×50	659	92.0	345	243	5.62	15.7	10.1	167	37.2	16.3	59.1
W14×61	640	102	383	277	8.65	25.0	6.50	141	107	32.8	121
W10×100	623	130	488	336	9.36	50.8	3.66	204	207	61.0	225
W18×40	**612**	**78.4**	**294**	**205**	**4.49**	**12.0**	**11.7**	**152**	**19.1**	**9.95**	**35.7**
W12×72	597	108	405	292	10.7	33.6	4.93	143	195	49.2	182
W16×45	586	82.3	309	218	5.55	15.1	9.45	150	32.8	14.5	52.5
W14×53	541	87.1	327	233	6.78	20.1	7.01	139	57.7	22.0	80.4
W10×88	534	113	424	296	9.29	45.1	3.58	176	179	53.1	196
W12×65††	533	96.8	357*	264	11.9**	31.7	5.01	127	174	44.1	164
W16×40	**518**	**73.0**	**274**	**194**	**5.55**	**14.7**	**8.71**	**132**	**28.9**	**12.7**	**46.4**

†† Indicates flange is non-compact.
* Tabulated value is $\phi_b M'_{px}$ to account for non-compact flange.
** Tabulated value is L'_p to account for non-compact flange.

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$F_y = 50$ ksi
$\phi_b = 0.90$
$\phi_v = 0.90$

Table 5-2 (cont.). W-Shapes Selection by I_x

I_x

Shape	X-X Axis								Y-Y Axis		
	I_x	Z_x	$\phi_b M_{px}$	$\phi_b M_{rx}$	L_p	L_r	BF	$\phi_v V_n$	I_y	Z_y	$\phi_b M_{py}$
	in.^{4}	in.^{3}	kip-ft	kip-ft	ft	ft	kips	kips	in.^{4}	in.^{3}	kip-ft
W18×35	**510**	**66.5**	**249**	**173**	**4.31**	**11.5**	**10.7**	**143**	**15.3**	**8.06**	**28.8**
W14×48	485	78.4	294	211	6.75	19.2	6.70	127	51.4	19.6	72.0
W12×58	475	86.4	324	234	8.87	27.0	4.97	119	107	32.5	120
W10×77	455	97.6	366	258	9.18	39.9	3.53	152	154	45.9	169
W16×36	448	64.0	240	170	5.37	14.1	8.11	127	24.5	10.8	39.4
W14×43	428	69.6	261	188	6.68	18.2	6.31	113	45.2	17.3	63.6
W12×53	425	77.9	292	212	8.76	25.6	4.78	113	95.8	29.1	108
W10×68	394	85.3	320	227	9.15	36.0	3.45	132	134	40.1	149
W12×50	391	71.9	270	193	6.92	21.5	5.30	122	56.3	21.3	78.2
W14×38	383	61.1	229	163	5.47	14.9	7.05	118	26.7	12.1	44.3
W16×31	**375**	**54.0**	**203**	**142**	**4.13**	**11.0**	**8.86**	**118**	**12.4**	**7.03**	**25.3**
W12×45	348	64.2	241	173	6.89	20.3	5.06	109	50.0	19.0	69.8
W10×60	341	74.6	280	200	9.08	32.6	3.39	116	116	35.0	129
W14×34	337	54.2	203	145	5.40	14.3	6.58	108	23.3	10.6	38.9
W12×40	307	57.0	214	155	6.85	19.2	4.79	94.8	44.1	16.8	61.9
W10×54	303	66.6	250	180	9.04	30.2	3.30	101	103	31.3	116
W16×26	**301**	**44.2**	**166**	**115**	**3.96**	**10.4**	**7.83**	**106**	**9.59**	**5.48**	**19.6**
W14×30	288	46.9	176	125	5.26	13.7	6.05	101	19.6	8.97	32.7
W12×35	285	51.2	192	137	5.44	15.2	5.65	101	24.5	11.5	42.0
W10×49	272	60.4	227	164	8.97	28.3	3.24	91.6	93.4	28.3	105
W10×45	248	54.9	206	147	7.10	24.1	3.44	95.4	53.4	20.3	74.8
W14×26	**243**	**39.9**	**150**	**105**	**3.81**	**10.2**	**7.01**	**95.7**	**8.90**	**5.53**	**19.9**
W12×30	238	43.1	162	116	5.37	14.3	5.12	86.3	20.3	9.56	35.1
W10×39	209	46.8	176	126	6.99	21.8	3.32	84.4	45.0	17.2	63.6
W12×26	**204**	**37.2**	**140**	**100**	**5.33**	**13.8**	**4.66**	**75.8**	**17.3**	**8.17**	**30.0**
W14×22	**197**	**32.8**	**123**	**85.8**	**3.67**	**9.65**	**6.22**	**85.1**	**7.00**	**4.37**	**15.8**
W10×33	171	38.8	146	105	6.85	19.8	3.14	76.2	36.6	14.0	51.8
W10×30	170	36.6	137	97.2	4.84	14.6	4.11	85.0	16.7	8.84	32.3
W12×22	**156**	**29.3**	**110**	**76.2**	**3.00**	**8.42**	**6.21**	**86.3**	**4.66**	**3.66**	**13.0**
W10×26	144	31.3	117	83.7	4.80	13.6	3.84	72.3	14.1	7.50	27.5
W12×19	**130**	**24.7**	**92.6**	**63.9**	**2.90**	**7.95**	**5.69**	**77.4**	**3.76**	**2.98**	**10.6**
W10×22	118	26.0	97.5	69.6	4.70	12.7	3.49	66.1	11.4	6.10	22.3
W12×16	**103**	**20.1**	**75.4**	**51.3**	**2.73**	**7.44**	**5.11**	**71.3**	**2.82**	**2.26**	**7.93**
W10×19	96.2	21.6	81.0	56.4	3.09	8.88	4.24	68.8	4.29	3.35	12.0
W12×14	**88.5**	**17.4**	**65.2**	**44.7**	**2.66**	**7.17**	**4.55**	**64.3**	**2.36**	**1.90**	**6.69**
W10×17	81.9	18.7	70.1	48.6	2.98	8.38	3.99	65.4	3.56	2.80	10.0
W10×15	68.9	16.0	60.0	41.4	2.86	7.93	3.67	62.0	2.89	2.30	8.16
W10× 12††	**53.8**	**12.6**	**46.9***	**32.7**	**2.87****	**7.45**	**3.18**	**50.6**	**2.18**	**1.74**	**6.19**

†† Indicates flange is non-compact.

* Tabulated value is $\phi_b M'_{px}$ to account for non-compact flange.
** Tabulated value is L'_p to account for non-compact flange.

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Z_x

Table 5-3 (cont.). W-Shapes Selection by Z_x

$F_y = 50$ ksi
$\phi_b = 0.90$
$\phi_v = 0.90$

Shape	X-X Axis								Y-Y Axis		
	Z_x	I_x	$\phi_b M_{px}$	$\phi_b M_{rx}$	L_p	L_r	BF	$\phi_v V_n$	Z_y	I_y	$\phi_b M_{py}$
	in.^{3}	in.^{4}	kip-ft	kip-ft	ft	ft	kips	kips	in.^{3}	in.^{4}	kip-ft
W27×84	**244**	**2850**	**915**	**639**	**7.31**	**19.3**	**23.0**	**332**	**33.2**	**106**	**119**
W12×152	243	1430	911	627	11.3	62.1	5.59	322	111	454	410
W14×132	234	1530	878	627	13.3	49.7	6.88	256	113	548	419
W18×106	230	1910	863	612	9.40	28.7	13.0	298	60.5	220	222
W24×84	**224**	**2370**	**840**	**588**	**6.89**	**18.6**	**21.5**	**306**	**32.6**	**94.4**	**118**
W21×93	221	2070	829	576	6.50	19.3	19.7	338	34.7	92.9	124
W12×136	214	1240	803	558	11.2	55.7	5.49	286	98.0	398	361
W14×120	212	1380	795	570	13.2	46.3	6.81	231	102	495	380
W18×97	211	1750	791	564	9.36	27.5	12.6	269	55.3	201	203
W24×76	**200**	**2100**	**750**	**528**	**6.78**	**18.0**	**19.8**	**284**	**28.6**	**82.5**	**103**
W16×100	199	1490	746	528	8.87	29.5	10.6	269	54.9	186	201
W21×83	196	1830	735	513	6.46	18.5	18.5	298	30.5	81.4	110
W14×109	192	1240	720	519	13.2	43.2	6.69	203	92.7	447	344
W18×86	186	1530	698	498	9.29	26.0	11.9	238	48.4	175	178
W12×120	186	1070	698	489	11.1	50.0	5.36	251	85.4	345	315
W24×68	**177**	**1830**	**664**	**462**	**6.61**	**17.4**	**18.6**	**266**	**24.5**	**70.4**	**88.3**
W16×89	176	1310	660	468	8.80	27.5	10.3	238	48.2	163	177
W14×99††	173	1110	646*	471	13.5**	40.6	6.56	186	83.6	402	311
W21×73	172	1600	645	453	6.39	17.6	17.1	260	26.6	70.6	95.6
W12×106	164	933	615	435	11.0	44.9	5.31	212	75.1	301	277
W18×76	163	1330	611	438	9.22	24.8	11.1	209	42.2	152	155
W21×68	**160**	**1480**	**600**	**420**	**6.36**	**17.3**	**16.5**	**245**	**24.4**	**64.7**	**88.3**
W14×90††	157	999	576*	429	15.1**	38.4	6.85	166	75.6	362	281
W24×62	**154**	**1560**	**578**	**396**	**4.84**	**13.3**	**21.6**	**275**	**15.8**	**34.5**	**55.1**
W16×77	151	1120	566	405	8.69	25.3	9.68	203	41.2	138	151
W12×96	147	833	551	393	10.9	41.3	5.20	189	67.5	270	250
W10×112	147	716	551	378	9.47	56.5	3.68	232	69.2	236	255
W18×71	146	1170	548	381	6.00	17.8	14.1	247	24.7	60.3	88.9
W21×62	**144**	**1330**	**540**	**381**	**6.25**	**16.7**	**15.2**	**227**	**21.7**	**57.5**	**78.8**
W14×82	139	882	521	369	8.76	29.6	7.30	197	44.8	148	165
W24×55	**135**	**1360**	**506**	**345**	**4.73**	**12.9**	**19.8**	**252**	**13.4**	**29.1**	**46.7**
W18×65	133	1070	499	351	5.97	17.1	13.3	224	22.5	54.8	81.0
W12×87	132	740	495	354	10.8	38.4	5.13	174	60.4	241	223
W16×67	131	963	491	354	8.65	23.8	9.04	174	35.5	119	131
W10×100	130	623	488	336	9.36	50.8	3.66	204	61.0	207	225
W21×57	129	1170	484	333	4.77	13.2	17.8	231	14.9	30.7	52.6

†† Indicates flange is non-compact.

* Tabulated value is $\phi_b M'_{px}$ to account for non-compact flange.
** Tabulated value is L'_p to account for non-compact flange.

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$F_y = 50$ ksi
$\phi_b = 0.90$
$\phi_v = 0.90$

Table 5-3 (cont.). W-Shapes Selection by Z_x

Z_x

Shape	X-X Axis								Y-Y Axis		
	Z_x	I_x	$\phi_b M_{px}$	$\phi_b M_{rx}$	L_p	L_r	BF	$\phi_v V_n$	Z_y	I_y	$\phi_b M_{py}$
	in.^{3}	in.^{4}	kip-ft	kip-ft	ft	ft	kips	kips	in.^{3}	in.^{4}	kip-ft
W21×55	**126**	**1140**	**473**	**330**	**6.11**	**16.1**	**14.3**	**211**	**18.4**	**48.4**	**66.4**
W14×74	126	796	473	336	8.76	27.9	7.12	173	40.5	134	150
W18×60	123	984	461	324	5.93	16.6	12.9	204	20.6	50.1	74.8
W12×79	119	662	446	321	10.8	35.7	5.03	157	54.3	216	201
W14×68	115	722	431	309	8.69	26.4	6.91	157	36.9	121	136
W10×88	113	534	424	296	9.29	45.1	3.58	176	53.1	179	196
W18×55	**112**	**890**	**420**	**295**	**5.90**	**16.1**	**12.2**	**191**	**18.5**	**44.9**	**66.9**
W21×50	**111**	**989**	**416**	**285**	**4.59**	**12.5**	**16.5**	**213**	**12.2**	**24.9**	**43.0**
W12×72	108	597	405	292	10.7	33.6	4.93	143	49.2	195	182
W21×48††	**107**	**959**	**398***	**279**	**6.09****	**15.4**	**13.2**	**195**	**14.9**	**38.7**	**53.5**
W16×57	105	758	394	277	5.65	16.6	10.7	190	18.9	43.1	68.1
W14×61	102	640	383	277	8.65	25.0	6.50	141	32.8	107	121
W18×50	101	800	379	267	5.83	15.6	11.5	173	16.6	40.1	60.2
W10×77	97.6	455	366	258	9.18	39.9	3.53	152	45.9	154	169
W12×65††	96.8	533	357*	264	11.9**	31.7	5.01	127	44.1	174	164
W21×44	**95.8**	**847**	**359**	**246**	**4.45**	**12.0**	**15.0**	**196**	**10.2**	**20.7**	**35.8**
W16×50	92.0	659	345	243	5.62	15.7	10.1	167	16.3	37.2	59.1
W18×46	90.7	712	340	236	4.56	12.6	12.9	176	11.7	22.5	41.8
W14×53	87.1	541	327	233	6.78	20.1	7.01	139	22.0	57.7	80.4
W12×58	86.4	475	324	234	8.87	27.0	4.97	119	32.5	107	120
W10×68	85.3	394	320	227	9.15	36.0	3.45	132	40.1	134	149
W16×45	82.3	586	309	218	5.55	15.1	9.45	150	14.5	32.8	52.5
W18×40	**78.4**	**612**	**294**	**205**	**4.49**	**12.0**	**11.7**	**152**	**9.95**	**19.1**	**35.7**
W14×48	78.4	485	294	211	6.75	19.2	6.70	127	19.6	51.4	72.0
W12×53	77.9	425	292	212	8.76	25.6	4.78	113	29.1	95.8	108
W10×60	74.6	341	280	200	9.08	32.6	3.39	116	35.0	116	129
W16×40	**73.0**	**518**	**274**	**194**	**5.55**	**14.7**	**8.71**	**132**	**12.7**	**28.9**	**46.4**
W12×50	71.9	391	270	193	6.92	21.5	5.30	122	21.3	56.3	78.2
W14×43	69.6	428	261	188	6.68	18.2	6.31	113	17.3	45.2	63.6
W10×54	66.6	303	250	180	9.04	30.2	3.30	101	31.3	103	116
W18×35	**66.5**	**510**	**249**	**173**	**4.31**	**11.5**	**10.7**	**143**	**8.06**	**15.3**	**28.8**
W12×45	64.2	348	241	173	6.89	20.3	5.06	109	19.0	50.0	69.8
W16×36	64.0	448	240	170	5.37	14.1	8.11	127	10.8	24.5	39.4
W14×38	61.1	383	229	163	5.47	14.9	7.05	118	12.1	26.7	44.3
W10×49	60.4	272	227	164	8.97	28.3	3.24	91.6	28.3	93.4	105
W12×40	57.0	307	214	155	6.85	19.2	4.79	94.8	16.8	44.1	61.9
W10×45	54.9	248	206	147	7.10	24.1	3.44	95.4	20.3	53.4	74.8
W14×34	**54.2**	**337**	**203**	**145**	**5.40**	**14.3**	**6.58**	**108**	**10.6**	**23.3**	**38.9**

†† Indicates flange is non-compact.

* Tabulated value is $\phi_b M'_{px}$ to account for non-compact flange.
** Tabulated value is L'_p to account for non-compact flange.

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Z_x

Table 5-3 (cont.). W-Shapes Selection by Z_x

$F_y = 50$ ksi
$\phi_b = 0.90$
$\phi_v = 0.90$

Shape	X-X Axis								Y-Y Axis		
	Z_x	I_x	$\phi_b M_{px}$	$\phi_b M_{rx}$	L_p	L_r	BF	$\phi_v V_n$	Z_y	I_y	$\phi_b M_{py}$
	in.^{3}	in.^{4}	kip-ft	kip-ft	ft	ft	kips	kips	in.^{3}	in.^{4}	kip-ft
W16×31	**54.0**	**375**	**203**	**142**	**4.13**	**11.0**	**8.86**	**118**	**7.03**	**12.4**	**25.3**
W12×35	51.2	285	192	137	5.44	15.2	5.65	101	11.5	24.5	42.0
W14×30	**46.9**	**288**	**176**	**125**	**5.26**	**13.7**	**6.05**	**101**	**8.97**	**19.6**	**32.7**
W10×39	46.8	209	176	126	6.99	21.8	3.32	84.4	17.2	45.0	63.6
W16×26	**44.2**	**301**	**166**	**115**	**3.96**	**10.4**	**7.83**	**106**	**5.48**	**9.59**	**19.6**
W12×30	43.1	238	162	116	5.37	14.3	5.12	86.3	9.56	20.3	35.1
W14×26	**39.9**	**243**	**150**	**105**	**3.81**	**10.2**	**7.01**	**95.7**	**5.53**	**8.90**	**19.9**
W10×33	38.8	171	146	105	6.85	19.8	3.14	76.2	14.0	36.6	51.8
W12×26	**37.2**	**204**	**140**	**100**	**5.33**	**13.8**	**4.66**	**75.8**	**8.17**	**17.3**	**30.0**
W10×30	36.6	170	137	97.2	4.84	14.6	4.11	85.0	8.84	16.7	32.3
W14×22	**32.8**	**197**	**123**	**85.8**	**3.67**	**9.65**	**6.22**	**85.1**	**4.37**	**7.00**	**15.8**
W10×26	31.3	144	117	83.7	4.80	13.6	3.84	72.3	7.50	14.1	27.5
W12×22	**29.3**	**156**	**110**	**76.2**	**3.00**	**8.42**	**6.21**	**86.3**	**3.66**	**4.66**	**13.0**
W10×22	**26.0**	**118**	**97.5**	**69.6**	**4.70**	**12.7**	**3.49**	**66.1**	**6.10**	**11.4**	**22.3**
W12×19	**24.7**	**130**	**92.6**	**63.9**	**2.90**	**7.95**	**5.69**	**77.4**	**2.98**	**3.76**	**10.6**
W10×19	**21.6**	**96.2**	**81.0**	**56.4**	**3.09**	**8.88**	**4.24**	**68.8**	**3.35**	**4.29**	**12.0**
W12×16	**20.1**	**103**	**75.4**	**51.3**	**2.73**	**7.44**	**5.11**	**71.3**	**2.26**	**2.82**	**7.93**
W10×17	18.7	81.9	70.1	48.6	2.98	8.38	3.99	65.4	2.80	3.56	10.0
W12×14	**17.4**	**88.5**	**65.2**	**44.7**	**2.66**	**7.17**	**4.55**	**64.3**	**1.90**	**2.36**	**6.69**
W10×15	16.0	68.9	60.0	41.4	2.86	7.93	3.67	62.0	2.30	2.89	8.16
W10×12††	**12.6**	**53.8**	**46.9***	**32.7**	**2.87****	**7.45**	**3.18**	**50.6**	**1.74**	**2.18**	**6.19**

†† Indicates flange is non-compact.

* Tabulated value is $\phi_b M'_{px}$ to account for non-compact flange.

** Tabulated value is L'_p to account for non-compact flange.

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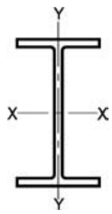
Table 5-4 (cont.). W-Shapes Maximum Total Factored Uniform Load[a]

$F_y = 50$ ksi
$\phi_b = 0.90$
$\phi_v = 0.90$



Shape	W14×									
	68	61	53	48	43	38	34	30	26	22
Span, ft										
5										170
6									191	164
7						236	215	201	171	141
8						229	203	176	150	123
9			278	253	226	204	181	156	133	109
10		281	261	235	209	183	163	141	120	98.4
11	314	278	238	214	190	167	148	128	109	89.5
12	288	255	218	196	174	153	136	117	99.7	82.0
13	265	235	201	181	161	141	125	108	92.1	75.7
14	246	219	187	168	149	131	116	101	85.5	70.3
15	230	204	174	157	139	122	108	93.8	79.8	65.6
16	216	191	163	147	130	115	102	87.9	74.8	61.5
17	203	180	154	138	123	108	95.6	82.8	70.4	57.9
18	192	170	145	131	116	102	90.3	78.2	66.5	54.7
19	182	161	138	124	110	96.5	85.6	74.1	63.0	51.8
20	173	153	131	118	104	91.7	81.3	70.4	59.8	49.2
21	164	146	124	112	99.4	87.3	77.4	67.0	57.0	46.9
22	157	139	119	107	94.9	83.3	73.9	64.0	54.4	44.7
23	150	133	114	102	90.8	79.7	70.7	61.2	52.0	42.8
24	144	128	109	98.0	87.0	76.4	67.8	58.6	49.9	41.0
25	138	122	105	94.1	83.5	73.3	65.0	56.3	47.9	39.4
26	133	118	101	90.5	80.3	70.5	62.5	54.1	46.0	37.8
27	128	113	96.8	87.1	77.3	67.9	60.2	52.1	44.3	36.4
28	123	109	93.3	84.0	74.6	65.5	58.1	50.2	42.7	35.1
29	119	106	90.1	81.1	72.0	63.2	56.1	48.5	41.3	33.9
30	115	102	87.1	78.4	69.6	61.1	54.2	46.9	39.9	32.8
31	111	98.7	84.3	75.9	67.4	59.1	52.5	45.4	38.6	31.7
32	108	95.6	81.7	73.5	65.2	57.3	50.8	44.0	37.4	30.8
33	105	92.7	79.2	71.3	63.3	55.5	49.3	42.6	36.3	29.8
34	101	90.0	76.9	69.2	61.4	53.9	47.8	41.4	35.2	28.9
35	98.6					52.4	46.5			
Beam Properties										
Z_x, in.^{3}	115	102	87.1	78.4	69.6	61.5	54.6	47.3	40.2	33.2
$\phi_b W_c$, kip-ft	3450	3060	2610	2350	2090	1830	1630	1410	1200	984
$\phi_v V_n$, kips	157	141	139	127	113	118	108	101	95.7	85.1
BF, kips	6.91	6.50	7.01	6.70	6.31	7.05	6.58	6.05	7.01	6.22
L_r, ft	26.4	25.0	20.1	19.2	18.2	14.9	14.3	13.7	10.2	9.65
L_p, ft	8.69	8.65	6.78	6.75	6.68	5.47	5.40	5.26	3.81	3.67
$\phi_b M_r$, kip-ft	309	277	233	211	188	163	145	125	105	85.8
$\phi_b M_p$, kip-ft	431	383	327	294	261	229	203	176	150	123

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[a] For beams laterally unsupported, see Table 5-5.
Note: Design strength tabulated above heavy line is limited by design shear strength.

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Table 5-13. Shear Stud Connectors $\boxed{\text{Unreduced}}$ Nominal Shear Strength Q_n, kips[a,b]

Specified Compressive Strength of Concrete f'_c, ksi	Light-Weight Concrete (115 lb/ft^3)				Normal-Weight Concrete (145 lb/ft^3)			
	Nominal Shear Stud Connector Diameter, in.				Nominal Shear Stud Connector Diameter, in.			
	$^1/_2$	$^5/_8$	$^3/_4$	$^7/_8$	$^1/_2$	$^5/_8$	$^3/_4$	$^7/_8$
3	7.86	12.3	17.7	24.1	9.35	14.6	21.0	28.6
3.5	8.82	13.8	19.8	27.0	10.5	16.4	23.6	32.1
4	9.75	15.2	21.9	29.9	11.6	18.1	26.1	35.5
4.5	10.7	16.6	24.0	32.6	11.8	18.4	26.5	36.1
5	11.5	18.0	25.9	35.3	11.8	18.4	26.5	36.1
Minimum Stud Length, in.	**2**	**2$^1/_2$**	**3**	**3$^1/_2$**	**2**	**2$^1/_2$**	**3**	**3$^1/_2$**

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[a] Applicable only to concrete made with ASTM C33 aggregates.

[b] Values do not reflect the strength reduction required for shear connections embedded in a slab on a formed steel deck, outlined in Section I3 of the Specification.

Table 5-15.
Lower Bound Elastic Moment of Inertia I_{LB} for Plastic Composite Sections

I_{LB}

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.											
		in.	kips	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W40×297	TFL	0.000	4370	44200	45100	46100	47100	48200	49200	50300	51400	52500	53700	54800
(23200)	2	0.413	3720	42500	43400	44300	45200	46200	47100	48100	49100	50200	51200	52300
	3	0.825	3070	40500	41300	42100	42900	43800	44700	45600	46500	47400	48300	49300
	4	1.24	2410	38100	38800	39500	40200	40900	41700	42500	43200	44000	44900	45700
	BFL	1.65	1760	35200	35800	36400	36900	37500	38100	38800	39400	40000	40700	41400
	6	4.55	1430	33500	34000	34500	35000	35500	36000	36600	37100	37700	38200	38800
	7	8.15	1090	31600	32000	32300	32800	33200	33600	34000	34500	34900	35400	35800
W40×278	TFL	0.000	4090	40500	41400	42300	43300	44300	45300	46300	47300	48400	49400	50500
(20500)	2	0.453	3550	39100	39900	40800	41700	42600	43500	44400	45400	46400	47400	48400
	3	0.905	3000	37400	38200	39000	39800	40600	41400	42300	43200	44100	45000	45900
	4	1.36	2460	35500	36200	36900	37600	38300	39100	39900	40600	41400	42200	43100
	BFL	1.81	1920	33300	33800	34400	35100	35700	36300	37000	37600	38300	39000	39700
	6	5.69	1470	31100	31500	32000	32500	33100	33600	34100	34700	35200	35800	36400
	7	10.1	1020	28500	28800	29200	29600	30000	30400	30800	31200	31600	32100	32500
W40×277	TFL	0.000	4070	41300	42200	43200	44100	45100	46000	47000	48100	49100	50200	51200
(21900)	2	0.395	3450	39700	40600	41400	42300	43100	44000	45000	45900	46900	47800	48800
	3	0.790	2820	37800	38500	39300	40100	40900	41700	42500	43300	44200	45000	45900
	4	1.19	2200	35500	36200	36800	37500	38100	38800	39500	40300	41000	41700	42500
	BFL	1.58	1570	32700	33200	33700	34300	34800	35300	35900	36500	37000	37600	38200
	6	4.24	1300	31300	31700	32200	32600	33100	33600	34100	34600	35100	35600	36100
	7	7.59	1020	29700	30000	30400	30800	31200	31600	32000	32400	32800	33200	33700
W40×264	TFL	0.000	3880	38200	39000	39900	40800	41700	42700	43700	44600	45600	46600	47700
(19400)	2	0.433	3370	36900	37700	38500	39300	40200	41100	41900	42900	43800	44700	45700
	3	0.865	2850	35300	36000	36800	37500	38300	39100	39900	40800	41600	42500	43400
	4	1.30	2340	33500	34200	34800	35500	36200	36900	37600	38400	39100	39900	40700
	BFL	1.730	1820	31400	31900	32500	33100	33700	34300	34900	35500	36100	36800	37500
	6	5.46	1400	29400	29800	30300	30800	31300	31800	32300	32800	33300	33800	34400
	7	9.90	970	26900	27300	27600	28000	28300	28700	29100	29500	29900	30300	30700
W40×249	TFL	0.000	3670	36900	37700	38500	39300	40200	41100	42000	42900	43800	44800	45700
(19600)	2	0.355	3100	35400	36200	36900	37700	38500	39300	40100	40900	41800	42700	43500
	3	0.710	2540	33700	34400	35100	35700	36500	37200	37900	38700	39400	40200	41000
	4	1.07	1980	31700	32300	32800	33400	34000	34700	35300	35900	36600	37200	37900
	BFL	1.42	1420	29200	29700	30100	30600	31100	31600	32100	32600	33100	33700	34200
	6	4.12	1170	28000	28300	28700	29100	29600	30000	30400	30900	31300	31800	32200
	7	7.48	916	26500	26800	27200	27500	27800	28200	28500	28900	29300	29700	30000
W40×235	TFL	0.000	3450	33900	34600	35400	36200	37000	37900	38700	39600	40500	41400	42300
(17400)	2	0.395	2980	32700	33400	34100	34800	35600	36400	37100	37900	38800	39600	40500
	3	0.790	2510	31300	31900	32600	33200	33900	34600	35300	36100	36800	37600	38300
	4	1.19	2040	29600	30200	30800	31400	32000	32600	33200	33900	34500	35200	35900
	BFL	1.58	1570	27700	28200	28700	29200	29700	30200	30700	31300	31800	32400	33000
	6	5.20	1220	26000	26400	26800	27200	27700	28100	28500	29000	29400	29900	30400
	7	9.46	863	24000	24300	24600	24900	25300	25600	25900	26300	26600	27000	27400

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

Rev. 11/1/02 [b] $\boxed{Y2}$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 5-4c for PNA locations.

[d] Value in parentheses is I_x (in.^{4}) of non-composite steel shape.

Table 5-15 (cont.). Lower Bound Elastic Moment of Inertia I_{LB} for Plastic Composite Sections

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kips	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W40×215	TFL	0.000	3170	31400	32000	32700	33500	34200	35000	35700	36500	37300	38100	39000
(16700)	2	0.305	2690	30200	30800	31400	32100	32800	33500	34200	34900	35600	36400	37100
	3	0.610	2210	28700	29300	29900	30500	31100	31700	32300	33000	33600	34300	35000
	4	0.915	1720	27000	27500	28000	28500	29000	29500	30100	30600	31200	31800	32400
	BFL	1.22	1240	24900	25300	25700	26100	26500	27000	27400	27800	28300	28800	29200
	6	3.85	1020	23800	24200	24500	24900	25200	25600	26000	26300	26700	27100	27500
	7	7.31	793	22600	22800	23100	23400	23700	24000	24300	24600	24900	25300	25600
W40×211	TFL	0.000	3100	30100	30800	31500	32200	32900	33700	34400	35200	36000	36800	37600
(15500)	2	0.355	2680	29000	29700	30300	31000	31600	32300	33000	33800	34500	35200	36000
	3	0.710	2260	27800	28400	29000	29600	30200	30800	31400	32100	32800	33400	34100
	4	1.07	1840	26400	26900	27400	27900	28500	29000	29600	30200	30800	31400	32000
	BFL	1.420	1420	24700	25100	25500	26000	26400	26900	27400	27900	28400	28900	29400
	6	5.04	1100	23100	23500	23900	24200	24600	25000	25400	25800	26200	26600	27100
	7	9.37	775	21300	21600	21900	22200	22500	22800	23100	23400	23700	24000	24300
W40×199	TFL	0.000	2930	28200	28900	29500	30200	30900	31600	32300	33000	33700	34500	35200
(14900)	2	0.268	2500	27200	27800	28400	29000	29600	30200	30900	31500	32200	32900	33600
	3	0.535	2080	26000	26500	27000	27600	28200	28700	29300	29900	30500	31100	31800
	4	0.803	1660	24600	25000	25500	26000	26400	26900	27500	28000	28500	29100	29600
	BFL	1.07	1230	22800	23200	23600	23900	24300	24800	25200	25600	26000	26500	26900
	6	4.23	983	21600	21900	22300	22600	22900	23300	23600	24000	24400	24700	25100
	7	8.10	731	20200	20500	20700	21000	21300	21600	21800	22100	22400	22700	23000
W40×183	TFL	0.000	2690	25700	26300	26900	27500	28200	28800	29400	30100	30800	31500	32200
(13300)	2	0.305	2330	24800	25400	25900	26500	27100	27700	28300	28900	29500	30200	30800
	3	0.610	1970	23800	24300	24800	25300	25900	26400	27000	27500	28100	28700	29300
	4	0.915	1610	22600	23000	23500	24000	24400	24900	25400	25900	26400	26900	27400
	BFL	1.22	1250	21200	21600	21900	22300	22700	23100	23500	24000	24400	24800	25300
	6	4.71	961	19800	20200	20500	20800	21100	21500	21800	22200	22500	22900	23200
	7	9.15	673	18300	18500	18800	19000	19200	19500	19800	20000	20300	20600	20900
W40×167	TFL	0.000	2460	22800	23300	23800	24400	25000	25500	26100	26700	27300	28000	28600
(11600)	2	0.255	2160	22000	22500	23000	23600	24100	24600	25200	25700	26300	26900	27500
	3	0.510	1860	21200	21700	22100	22600	23100	23600	24100	24600	25200	25700	26300
	4	0.765	1560	20300	20700	21100	21500	22000	22400	22900	23300	23800	24300	24800
	BFL	1.02	1260	19200	19500	19900	20300	20600	21000	21400	21800	22300	22700	23100
	6	4.90	936	17800	18000	18300	18600	19000	19300	19600	19900	20300	20600	21000
	7	9.84	615	16100	16300	16500	16700	16900	17200	17400	17700	17900	18100	18400
W40×149	TFL	0.000	2190	19600	20000	20500	21000	21500	22000	22500	23100	23600	24200	24700
(9800)	2	0.208	1950	19000	19400	19900	20300	20800	21300	21800	22300	22800	23300	23900
	3	0.415	1700	18300	18700	19100	19600	20000	20500	20900	21400	21900	22300	22800
	4	0.623	1460	17600	18000	18400	18700	19100	19600	20000	20400	20800	21300	21700
	BFL	0.830	1210	16700	17100	17400	17800	18100	18500	18900	19200	19600	20000	20400
	6	5.15	879	15400	15700	15900	16200	16500	16800	17100	17400	17700	18000	18300
	7	10.4	548	13700	13900	14100	14300	14500	14700	14900	15100	15300	15500	15800

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

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[b] $\boxed{Y2}$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 5-4c for PNA locations.

[d] Value in parentheses is I_x (in.^{4}) of non-composite steel shape.

Table 5-15 (cont.). Lower Bound Elastic Moment of Inertia I_{LB} for Plastic Composite Sections

I_{LB}

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kips	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W36×300	TFL	0.000	4420	38600	39500	40400	41400	42400	43400	44400	45400	46500	47600	48700
(20300)	2	0.420	3710	37000	37800	38700	39500	40400	41400	42300	43200	44200	45200	46200
	3	0.840	3010	35100	35900	36600	37400	38200	39000	39800	40700	41500	42400	43300
	4	1.26	2310	32900	33500	34100	34800	35500	36100	36800	37600	38300	39000	39800
	BFL	1.68	1610	30100	30600	31100	31600	32100	32600	33200	33700	34300	34900	35500
	6	3.99	1360	28900	29300	29800	30200	30700	31200	31600	32100	32600	33100	33700
	7	6.67	1100	27600	28000	28300	28700	29100	29500	29900	30300	30700	31200	31600
W36×280	TFL	0.000	4120	35800	36600	37500	38400	39300	40200	41200	42100	43100	44100	45200
(18900)	2	0.393	3470	34300	35100	35900	36700	37500	38400	39300	40100	41100	42000	42900
	3	0.785	2820	32600	33300	34000	34700	35500	36200	37000	37800	38600	39400	40200
	4	1.18	2170	30600	31100	31700	32300	33000	33600	34300	34900	35600	36300	37000
	BFL	1.57	1510	28000	28400	28900	29400	29800	30300	30800	31400	31900	32400	33000
	6	3.88	1270	26900	27300	27700	28100	28500	28900	29400	29900	30300	30800	31300
	7	6.61	1030	25700	26000	26300	26700	27100	27400	27800	28200	28600	29000	29400
W36×260	TFL	0.000	3830	32800	33600	34400	35200	36100	36900	37800	38700	39600	40600	41500
(17300)	2	0.360	3230	31500	32200	33000	33700	34500	35300	36100	36900	37700	38600	39500
	3	0.720	2630	30000	30600	31200	31900	32600	33300	34000	34700	35500	36200	37000
	4	1.08	2030	28100	28600	29200	29700	30300	30900	31500	32100	32800	33400	34100
	BFL	1.44	1430	25800	26200	26600	27100	27500	28000	28500	28900	29400	29900	30500
	6	3.92	1200	24700	25100	25500	25900	26300	26700	27100	27500	28000	28400	28900
	7	6.77	956	23500	23800	24100	24500	24800	25100	25500	25900	26200	26600	27000
W36×256	TFL	0.000	3770	33000	33700	34600	35400	36200	37100	38000	38900	39800	40700	41700
(16800)	2	0.433	3240	31700	32500	33200	34000	34800	35600	36400	37200	38100	38900	39800
	3	0.865	2710	30300	31000	31600	32300	33000	33800	34500	35300	36000	36800	37600
	4	1.30	2190	28700	29300	29800	30500	31100	31700	32400	33000	33700	34400	35100
	BFL	1.730	1660	26700	27200	27700	28200	28700	29200	29700	30300	30900	31400	32000
	6	5.15	1300	25100	25500	25900	26300	26800	27200	27700	28100	28600	29100	29600
	7	8.88	943	23300	23600	23900	24200	24600	24900	25300	25600	26000	26400	26800
W36×245	TFL	0.000	3610	30600	31300	32100	32900	33600	34400	35300	36100	37000	37800	38700
(16100)	2	0.338	3050	29400	30100	30700	31400	32200	32900	33700	34400	35200	36000	36800
	3	0.675	2490	27900	28500	29200	29800	30400	31100	31700	32400	33100	33900	34600
	4	1.01	1930	26200	26700	27200	27800	28300	28900	29500	30000	30600	31300	31900
	BFL	1.35	1380	24100	24500	24900	25400	25800	26200	26700	27200	27600	28100	28600
	6	3.81	1140	23100	23400	23800	24100	24500	24900	25300	25700	26100	26500	27000
	7	6.78	901	21900	22200	22500	22800	23100	23400	23800	24100	24400	24800	25100
W36×232	TFL	0.000	3410	29400	30100	30800	31600	32300	33100	33900	34700	35500	36400	37200
(15000)	2	0.393	2930	28300	29000	29600	30300	31000	31700	32500	33200	34000	34800	35600
	3	0.785	2460	27100	27700	28300	28900	29500	30200	30800	31500	32200	32900	33600
	4	1.18	1980	25600	26100	26600	27200	27700	28300	28900	29500	30100	30700	31300
	BFL	1.57	1510	23800	24300	24700	25200	25600	26100	26600	27100	27600	28100	28700
	6	5.01	1180	22400	22800	23100	23500	23900	24300	24700	25100	25600	26000	26400
	7	8.77	851	20800	21000	21300	21600	21900	22200	22600	22900	23200	23500	23900

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

Rev. 11/1/02 [b] $\boxed{Y2}$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 5-4c for PNA locations.

[d] Value in parentheses is I_x (in.^{4}) of non-composite steel shape.

Table 5-15 (cont.). Lower Bound Elastic Moment of Inertia I_{LB} for Plastic Composite Sections

I_{LB}

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kips	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W36×230	TFL	0.000	3380	28500	29100	29800	30600	31300	32000	32800	33600	34400	35200	36000
(15000)	2	0.315	2860	27300	28000	28600	29300	29900	30600	31300	32000	32800	33500	34300
	3	0.630	2340	26000	26600	27100	27700	28300	28900	29600	30200	30900	31500	32200
	4	0.945	1820	24400	24900	25400	25900	26400	26900	27500	28000	28600	29100	29700
	BFL	1.26	1300	22500	22900	23200	23600	24000	24500	24900	25300	25800	26200	26700
	6	3.83	1070	21500	21800	22100	22500	22800	23200	23600	23900	24300	24700	25100
	7	6.83	845	20400	20700	20900	21200	21500	21800	22100	22400	22800	23100	23400
W36×210	TFL	0.000	3090	26000	26600	27300	28000	28600	29300	30000	30800	31500	32300	33100
(13200)	2	0.340	2680	25100	25700	26300	26900	27500	28200	28900	29500	30200	30900	31600
	3	0.680	2260	24000	24500	25100	25700	26200	26800	27400	28000	28700	29300	30000
	4	1.02	1850	22800	23300	23700	24200	24800	25300	25800	26400	26900	27500	28100
	BFL	1.36	1430	21300	21700	22100	22500	23000	23400	23900	24300	24800	25300	25800
	6	5.08	1100	19900	20300	20600	20900	21300	21700	22000	22400	22800	23200	23600
	7	9.04	773	18300	18600	18800	19100	19400	19700	19900	20200	20500	20800	21100
W36×194	TFL	0.000	2850	23800	24400	25000	25600	26200	26900	27500	28200	28900	29600	30300
(12100)	2	0.315	2470	23000	23500	24100	24600	25200	25800	26400	27000	27700	28300	29000
	3	0.630	2090	22000	22500	23000	23500	24000	24600	25100	25700	26300	26900	27500
	4	0.945	1710	20900	21300	21800	22200	22700	23200	23700	24200	24700	25200	25700
	BFL	1.260	1330	19500	19900	20300	20700	21100	21500	21900	22300	22800	23200	23700
	6	4.93	1020	18300	18600	18900	19200	19500	19900	20200	20600	20900	21300	21700
	7	8.94	713	16800	17000	17300	17500	17700	18000	18300	18500	18800	19100	19400
W36×182	TFL	0.000	2680	22200	22700	23300	23900	24400	25000	25700	26300	26900	27600	28300
(11300)	2	0.295	2320	21400	21900	22400	23000	23500	24100	24600	25200	25800	26400	27000
	3	0.590	1970	20500	21000	21500	21900	22400	22900	23500	24000	24500	25100	25700
	4	0.885	1610	19500	19900	20300	20700	21200	21600	22100	22600	23000	23500	24000
	BFL	1.18	1250	18200	18600	18900	19300	19700	20000	20400	20800	21200	21700	22100
	6	4.89	961	17000	17300	17600	17900	18200	18600	18900	19200	19600	19900	20200
	7	8.91	670	15700	15900	16100	16300	16600	16800	17000	17300	17600	17800	18100
W36×170	TFL	0.000	2510	20600	21100	21700	22200	22700	23300	23900	24500	25100	25700	26300
(10500)	2	0.275	2180	19900	20400	20900	21400	21900	22400	22900	23500	24000	24600	25200
	3	0.550	1850	19100	19500	20000	20400	20900	21400	21900	22400	22900	23400	23900
	4	0.825	1520	18100	18500	18900	19300	19700	20200	20600	21000	21500	21900	22400
	BFL	1.10	1190	17000	17300	17700	18000	18400	18700	19100	19500	19900	20300	20700
	6	4.78	906	15900	16100	16400	16700	17000	17300	17600	17900	18200	18600	18900
	7	8.89	626	14500	14800	15000	15200	15400	15600	15800	16100	16300	16600	16800
W36×160	TFL	0.000	2350	19200	19600	20100	20600	21100	21700	22200	22700	23300	23900	24400
(9760)	2	0.255	2040	18500	18900	19400	19900	20300	20800	21300	21800	22300	22900	23400
	3	0.510	1740	17800	18200	18600	19000	19400	19900	20300	20800	21300	21800	22300
	4	0.765	1430	16900	17200	17600	18000	18400	18800	19200	19600	20000	20400	20900
	BFL	1.02	1130	15900	16200	16500	16800	17100	17500	17800	18200	18600	18900	19300
	6	4.82	857	14800	15000	15300	15600	15800	16100	16400	16700	17000	17300	17600
	7	8.96	588	13500	13700	13900	14100	14300	14500	14700	15000	15200	15400	15600

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

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[b] $\boxed{Y2}$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 5-4c for PNA locations.

[d] Value in parentheses is I_x (in.^{4}) of non-composite steel shape.

Table 5-15 (cont.). Lower Bound Elastic Moment of Inertia I_{LB} for Plastic Composite Sections

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kips	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W36×150	TFL	0.000	2210	17800	18300	18700	19200	19700	20200	20700	21200	21700	22300	22800
(9040)	2	0.235	1930	17200	17700	18100	18500	19000	19400	19900	20400	20900	21400	21900
	3	0.470	1650	16600	16900	17300	17700	18100	18600	19000	19400	19900	20300	20800
	4	0.705	1360	15700	16100	16400	16800	17200	17500	17900	18300	18700	19100	19500
	BFL	0.94	1080	14800	15100	15400	15700	16000	16400	16700	17000	17400	17700	18100
	6	4.87	817	13800	14000	14300	14500	14800	15100	15300	15600	15900	16200	16500
	7	9.11	553	12600	12700	12900	13100	13300	13500	13700	13900	14100	14300	14500
W36×135	TFL	0.000	1990	15600	16000	16400	16800	17200	17700	18100	18600	19100	19500	20000
(7800)	2	0.198	1750	15100	15500	15800	16200	16600	17100	17500	17900	18300	18800	19200
	3	0.395	1510	14500	14900	15200	15600	16000	16300	16700	17100	17500	17900	18300
	4	0.593	1270	13900	14200	14500	14800	15200	15500	15900	16200	16600	16900	17300
	BFL	0.790	1040	13200	13400	13700	14000	14300	14600	14900	15200	15500	15900	16200
	6	5.02	767	12100	12400	12600	12800	13100	13300	13600	13800	14100	14300	14600
	7	9.53	496	10900	11100	11200	11400	11600	11700	11900	12100	12300	12500	12700
W33×221	TFL	0.000	3260	24600	25200	25900	26500	27200	27900	28600	29300	30100	30800	31600
(12900)	2	0.318	2760	23600	24200	24800	25400	26000	26700	27300	28000	28600	29300	30000
	3	0.635	2260	22500	23000	23500	24100	24600	25200	25800	26400	27000	27600	28200
	4	0.953	1760	21100	21500	22000	22500	22900	23400	23900	24400	24900	25500	26000
	BFL	1.27	1250	19400	19700	20100	20500	20800	21200	21600	22000	22400	22800	23300
	6	3.61	1030	18500	18800	19100	19400	19800	20100	20400	20800	21100	21500	21900
	7	6.43	815	17600	17800	18100	18400	18600	18900	19200	19500	19800	20100	20400
W33×201	TFL	0.000	2960	22100	22700	23300	23900	24500	25100	25700	26400	27100	27700	28400
(11600)	2	0.288	2510	21300	21800	22300	22800	23400	24000	24600	25200	25800	26400	27100
	3	0.575	2060	20200	20700	21200	21700	22200	22700	23200	23700	24300	24800	25400
	4	0.863	1610	19000	19400	19800	20200	20700	21100	21600	22000	22500	23000	23500
	BFL	1.15	1150	17500	17800	18100	18500	18800	19200	19500	19900	20200	20600	21000
	6	3.60	947	16700	17000	17300	17500	17800	18100	18500	18800	19100	19400	19800
	7	6.50	740	15800	16000	16300	16500	16700	17000	17300	17500	17800	18100	18300
W33×169	TFL	0.000	2480	18100	18600	19100	19600	20100	20600	21200	21700	22300	22900	23400
(9290)	2	0.305	2120	17400	17900	18300	18800	19300	19700	20200	20700	21300	21800	22300
	3	0.610	1770	16700	17100	17500	17900	18300	18700	19200	19600	20100	20600	21100
	4	0.915	1420	15700	16100	16400	16800	17200	17600	17900	18300	18800	19200	19600
	BFL	1.22	1070	14600	14900	15200	15500	15800	16100	16500	16800	17100	17500	17800
	6	4.28	845	13800	14000	14300	14500	14800	15100	15300	15600	15900	16200	16500
	7	7.66	619	12800	13000	13200	13400	13600	13800	14000	14300	14500	14700	14900
W33×152	TFL	0.000	2240	16000	16500	16900	17300	17800	18300	18800	19200	19800	20300	20800
(8160)	2	0.265	1930	15400	15800	16200	16700	17100	17500	18000	18400	18900	19400	19900
	3	0.530	1630	14800	15200	15500	15900	16300	16700	17100	17500	17900	18400	18800
	4	0.795	1320	14000	14300	14600	15000	15300	15700	16000	16400	16800	17100	17500
	BFL	1.06	1010	13100	13300	13600	13900	14200	14400	14700	15100	15400	15700	16000
	6	4.38	785	12200	12500	12700	12900	13200	13400	13700	13900	14200	14400	14700
	7	7.93	560	11300	11500	11700	11800	12000	12200	12400	12600	12800	13000	13200

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

Rev. 11/1/02 [b] $\boxed{Y2}$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 5-4c for PNA locations.

[d] Value in parentheses is I_x (in.^{4}) of non-composite steel shape.

Table 5-15 (cont.). Lower Bound Elastic Moment of Inertia I_{LB} for Plastic Composite Sections

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kips	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W33×141	TFL	0.000	2080	14700	15100	15500	15900	16300	16800	17200	17700	18100	18600	19100
(7450)	2	0.240	1800	14200	14500	14900	15300	15700	16100	16500	16900	17400	17800	18200
	3	0.480	1530	13600	13900	14300	14600	15000	15300	15700	16100	16500	16900	17300
	4	0.720	1250	12900	13200	13500	13800	14100	14400	14800	15100	15500	15800	16200
	BFL	0.960	976	12100	12300	12600	12800	13100	13400	13700	14000	14300	14600	14900
	6	4.29	748	11300	11500	11700	11900	12100	12400	12600	12800	13100	13300	13600
	7	8.05	520	10300	10500	10700	10800	11000	11200	11300	11500	11700	11900	12100
W33×130	TFL	0.000	1920	13300	13700	14000	14400	14800	15200	15600	16000	16500	16900	17300
(6710)	2	0.214	1670	12800	13200	13500	13900	14200	14600	15000	15400	15800	16200	16600
	3	0.428	1420	12300	12600	12900	13300	13600	13900	14300	14600	15000	15400	15800
	4	0.641	1180	11700	12000	12300	12600	12900	13200	13500	13800	14100	14500	14800
	BFL	0.855	932	11000	11300	11500	11800	12000	12300	12500	12800	13100	13400	13700
	6	4.39	705	10300	10500	10600	10900	11100	11300	11500	11700	12000	12200	12400
	7	8.30	479	9350	9490	9640	9790	9950	10100	10300	10400	10600	10800	11000
W33×118	TFL	0.000	1740	11800	12100	12500	12800	13200	13500	13900	14300	14700	15100	15500
(5900)	2	0.185	1520	11400	11700	12000	12300	12700	13000	13400	13700	14100	14400	14800
	3	0.370	1310	11000	11300	11500	11800	12100	12500	12800	13100	13400	13800	14100
	4	0.555	1100	10500	10700	11000	11300	11500	11800	12100	12400	12700	13000	13300
	BFL	0.740	884	9890	10100	10300	10600	10800	11000	11300	11500	11800	12100	12300
	6	4.47	659	9150	9330	9510	9700	9890	10100	10300	10500	10700	10900	11200
	7	8.56	434	8260	8390	8530	8660	8800	8950	9090	9250	9400	9560	9720
W30×116	TFL	0.000	1710	9870	10200	10500	10800	11100	11400	11800	12100	12500	12800	13200
(4930)	2	0.213	1490	9530	9810	10100	10400	10700	11000	11300	11600	12000	12300	12600
	3	0.425	1260	9120	9370	9630	9900	10200	10400	10700	11000	11300	11600	12000
	4	0.638	1040	8670	8890	9120	9360	9600	9850	10100	10400	10600	10900	11200
	BFL	0.850	818	8130	8320	8520	8720	8920	9140	9360	9580	9810	10000	10300
	6	3.98	623	7570	7730	7890	8060	8230	8400	8580	8770	8960	9150	9350
	7	7.43	428	6910	7030	7150	7270	7400	7530	7670	7810	7950	8090	8240
W30×108	TFL	0.000	1590	9000	9280	9560	9840	10100	10400	10800	11100	11400	11700	12100
(4470)	2	0.190	1390	8700	8950	9220	9480	9760	10000	10300	10600	10900	11300	11600
	3	0.380	1190	8350	8590	8830	9070	9330	9590	9850	10100	10400	10700	11000
	4	0.570	987	7940	8150	8370	8590	8820	9050	9290	9530	9780	10000	10300
	BFL	0.760	787	7470	7650	7840	8030	8230	8430	8640	8850	9060	9290	9510
	6	4.04	592	6930	7080	7230	7390	7550	7710	7880	8060	8240	8420	8600
	7	7.63	396	6280	6390	6500	6620	6730	6850	6980	7110	7240	7370	7510
W30×99	TFL	0.000	1460	8130	8380	8630	8900	9170	9450	9730	10000	10300	10600	10900
(3990)	2	0.168	1280	7860	8090	8330	8580	8830	9090	9360	9630	9910	10200	10500
	3	0.335	1100	7550	7760	7980	8210	8440	8680	8930	9180	9440	9700	9970
	4	0.503	927	7210	7400	7600	7800	8010	8230	8450	8680	8910	9150	9400
	BFL	0.670	752	6810	6970	7150	7330	7510	7700	7900	8100	8300	8510	8720
	6	4.13	558	6280	6420	6560	6710	6860	7010	7170	7330	7500	7670	7840
	7	7.85	364	5640	5740	5850	5950	6060	6170	6280	6400	6520	6640	6770

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

Rev. 11/1/02 [b] $\boxed{Y2}$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 5-4c for PNA locations.

[d] Value in parentheses is I_x (in.^{4}) of non-composite steel shape.

Table 5-15 (cont.).
Lower Bound Elastic Moment of Inertia I_{LB}
for Plastic Composite Sections

I_{LB}

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kips	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W30×90	TFL	0.000	1320	7310	7540	7770	8010	8250	8500	8760	9020	9290	9570	9850
(3610)	2	0.153	1160	7070	7280	7500	7720	7950	8190	8430	8670	8930	9190	9450
	3	0.305	1000	6800	7000	7200	7400	7610	7830	8050	8280	8510	8750	8990
	4	0.458	844	6500	6670	6850	7040	7230	7430	7630	7830	8040	8260	8480
	BFL	0.610	686	6140	6300	6450	6620	6780	6960	7130	7310	7500	7690	7880
	6	3.95	508	5670	5790	5920	6050	6190	6330	6470	6620	6770	6920	7080
	7	7.73	330	5090	5180	5270	5370	5470	5570	5670	5780	5880	5990	6110
W27×102	TFL	0.000	1500	7250	7480	7730	7980	8240	8510	8780	9060	9350	9650	9950
(3620)	2	0.208	1290	6970	7190	7420	7650	7890	8140	8390	8650	8920	9200	9480
	3	0.415	1090	6670	6870	7080	7290	7510	7730	7960	8200	8450	8700	8950
	4	0.623	878	6300	6470	6650	6840	7030	7230	7430	7640	7850	8070	8300
	BFL	0.830	670	5860	6010	6160	6310	6470	6640	6810	6980	7160	7340	7530
	6	3.40	523	5500	5620	5740	5870	6010	6150	6290	6430	6580	6740	6900
	7	6.27	375	5070	5170	5260	5360	5470	5570	5680	5800	5910	6030	6150
W27×94	TFL	0.000	1390	6580	6800	7020	7260	7490	7740	7990	8250	8520	8790	9070
(3270)	2	0.186	1200	6340	6540	6750	6960	7190	7410	7650	7890	8130	8390	8650
	3	0.373	1010	6060	6240	6430	6630	6830	7030	7250	7460	7690	7920	8160
	4	0.559	827	5740	5900	6070	6250	6420	6610	6800	6990	7190	7390	7600
	BFL	0.745	641	5360	5500	5640	5790	5940	6090	6250	6420	6590	6760	6940
	6	3.38	493	5010	5120	5240	5360	5480	5610	5750	5880	6020	6160	6310
	7	6.38	346	4590	4680	4770	4860	4960	5050	5150	5260	5360	5470	5590
W27×84	TFL	0.000	1240	5770	5970	6160	6370	6580	6800	7030	7260	7490	7740	7990
(2850)	2	0.160	1080	5570	5750	5940	6130	6330	6530	6740	6950	7170	7400	7630
	3	0.320	921	5340	5510	5680	5850	6030	6220	6410	6610	6810	7010	7230
	4	0.480	762	5070	5220	5370	5530	5690	5860	6030	6200	6380	6570	6760
	BFL	0.640	603	4760	4890	5020	5150	5290	5440	5580	5730	5890	6050	6210
	6	3.43	456	4420	4530	4630	4740	4860	4970	5100	5220	5350	5480	5610
	7	6.61	310	4020	4100	4180	4260	4340	4430	4520	4610	4710	4800	4900
W24×76	TFL	0.000	1120	4280	4440	4600	4770	4950	5130	5320	5510	5710	5910	6120
(2100)	2	0.170	967	4120	4270	4420	4580	4740	4910	5080	5260	5440	5630	5830
	3	0.340	814	3930	4070	4210	4350	4500	4650	4810	4970	5140	5310	5490
	4	0.510	662	3720	3840	3960	4090	4220	4350	4490	4630	4780	4930	5090
	BFL	0.680	509	3460	3560	3660	3770	3880	3990	4110	4230	4360	4480	4610
	6	2.99	394	3230	3320	3400	3490	3580	3680	3770	3880	3980	4080	4190
	7	5.59	280	2970	3040	3100	3170	3240	3310	3390	3460	3540	3630	3710
W24×68	TFL	0.000	1010	3760	3900	4050	4200	4360	4520	4690	4860	5040	5220	5410
(1830)	2	0.146	874	3620	3760	3890	4030	4180	4330	4480	4640	4810	4980	5150
	3	0.293	743	3470	3590	3710	3840	3980	4110	4260	4400	4550	4710	4870
	4	0.439	611	3290	3390	3510	3620	3740	3860	3990	4120	4250	4390	4530
	BFL	0.585	480	3080	3170	3260	3360	3460	3570	3670	3790	3900	4020	4140
	6	3.04	366	2860	2930	3010	3090	3180	3260	3350	3450	3540	3640	3740
	7	5.80	251	2600	2660	2720	2780	2840	2900	2970	3040	3110	3180	3260

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

Rev. 11/1/02 [b] $\boxed{Y2}$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 5-4c for PNA locations.

[d] Value in parentheses is I_x (in.^{4}) of non-composite steel shape.

Table 5-15 (cont.). Lower Bound Elastic Moment of Inertia I_{LB} for Plastic Composite Sections

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kips	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W24×62	TFL	0.000	915	3320	3440	3580	3720	3860	4010	4160	4310	4480	4640	4810
(1560)	2	0.148	811	3210	3330	3460	3590	3720	3860	4000	4150	4300	4460	4620
	3	0.295	707	3090	3200	3320	3440	3560	3690	3820	3960	4100	4250	4390
	4	0.443	603	2950	3060	3160	3270	3390	3500	3620	3750	3880	4010	4140
	BFL	0.590	500	2800	2890	2990	3080	3180	3290	3400	3510	3620	3740	3860
	6	3.38	364	2560	2630	2710	2790	2870	2950	3040	3130	3220	3310	3410
	7	6.53	229	2260	2310	2370	2420	2480	2540	2600	2660	2730	2790	2860
W24×55	TFL	0.000	815	2910	3030	3150	3270	3390	3530	3660	3800	3940	4090	4240
(1360)	2	0.126	726	2820	2930	3040	3160	3280	3400	3530	3660	3790	3930	4070
	3	0.253	638	2720	2820	2930	3040	3150	3260	3380	3500	3630	3760	3890
	4	0.379	549	2610	2700	2800	2900	3000	3100	3210	3320	3440	3560	3680
	BFL	0.505	461	2480	2560	2650	2740	2830	2920	3020	3120	3230	3330	3440
	6	3.39	332	2260	2320	2390	2460	2540	2610	2690	2770	2850	2940	3030
	7	6.64	204	1980	2030	2070	2120	2170	2230	2280	2340	2390	2450	2510
W21×62	TFL	0.000	915	2760	2880	3000	3120	3250	3390	3530	3670	3820	3970	4130
(1330)	2	0.154	788	2650	2760	2870	2990	3110	3240	3360	3500	3640	3780	3920
	3	0.308	662	2530	2630	2730	2840	2950	3060	3180	3300	3420	3550	3680
	4	0.461	535	2390	2470	2560	2650	2750	2850	2950	3060	3170	3280	3400
	BFL	0.615	408	2210	2280	2360	2440	2520	2600	2690	2770	2870	2960	3060
	6	2.54	318	2070	2130	2190	2260	2320	2390	2460	2540	2610	2690	2780
	7	4.78	229	1900	1950	2000	2050	2100	2150	2210	2270	2330	2390	2450
W21×57	TFL	0.000	835	2490	2590	2700	2820	2940	3060	3190	3320	3460	3600	3740
(1170)	2	0.163	728	2400	2490	2600	2710	2820	2930	3050	3170	3300	3430	3570
	3	0.325	622	2290	2380	2480	2580	2680	2780	2890	3010	3120	3240	3370
	4	0.488	515	2170	2250	2340	2430	2520	2610	2710	2810	2910	3020	3130
	BFL	0.650	409	2030	2110	2180	2250	2330	2410	2500	2580	2670	2770	2860
	6	2.93	309	1880	1940	2000	2060	2120	2190	2260	2330	2410	2480	2560
	7	5.40	209	1700	1740	1780	1830	1880	1930	1980	2030	2090	2140	2200
W21×55	TFL	0.000	810	2390	2490	2590	2710	2820	2940	3060	3190	3320	3450	3590
(1140)	2	0.131	703	2300	2390	2490	2590	2700	2810	2930	3040	3160	3290	3420
	3	0.261	595	2190	2280	2370	2470	2560	2660	2770	2870	2990	3100	3220
	4	0.392	488	2080	2150	2230	2320	2400	2490	2580	2680	2780	2880	2980
	BFL	0.522	381	1940	2000	2070	2140	2210	2290	2370	2450	2530	2620	2710
	6	2.62	292	1800	1850	1910	1970	2030	2090	2160	2230	2290	2370	2440
	7	5.00	203	1640	1680	1720	1770	1810	1860	1910	1960	2010	2070	2120
W21×50	TFL	0.000	735	2110	2210	2300	2400	2510	2620	2730	2840	2960	3080	3210
(984)	2	0.134	648	2040	2130	2220	2310	2410	2510	2620	2730	2840	2950	3070
	3	0.268	560	1960	2040	2130	2210	2300	2400	2490	2590	2690	2800	2910
	4	0.401	473	1870	1940	2020	2100	2180	2260	2350	2440	2530	2630	2730
	BFL	0.535	386	1760	1830	1890	1960	2030	2110	2180	2260	2350	2430	2520
	6	2.91	285	1620	1670	1720	1780	1840	1900	1960	2020	2090	2160	2230
	7	5.56	184	1440	1470	1510	1550	1590	1640	1680	1730	1780	1820	1880

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

Rev. 11/1/02 [b] $\boxed{Y2}$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 5-4c for PNA locations.

[d] Value in parentheses is I_x (in.[4]) of non-composite steel shape.

Table 5-15 (cont.). Lower Bound Elastic Moment of Inertia I_{LB} for Plastic Composite Sections

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kips	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W21×48	TFL	0.000	705	2030	2110	2210	2300	2400	2500	2610	2720	2830	2950	3070
(959)	2	0.108	617	1950	2040	2120	2210	2300	2400	2500	2600	2710	2820	2930
	3	0.215	530	1870	1950	2030	2110	2200	2280	2380	2470	2570	2670	2770
	4	0.323	442	1780	1850	1920	1990	2070	2150	2230	2320	2400	2490	2590
	BFL	0.430	355	1670	1730	1790	1860	1920	1990	2060	2140	2210	2290	2370
	6	2.71	266	1540	1590	1640	1690	1750	1810	1860	1920	1990	2050	2120
	7	5.26	176	1390	1420	1460	1500	1540	1580	1620	1660	1710	1750	1800
W21×44	TFL	0.000	650	1830	1920	2000	2090	2180	2280	2370	2480	2580	2690	2800
(843)	2	0.113	577	1780	1850	1930	2020	2100	2190	2280	2380	2480	2580	2680
	3	0.225	504	1710	1780	1850	1930	2010	2100	2180	2270	2360	2460	2550
	4	0.338	431	1630	1700	1770	1840	1910	1990	2060	2150	2230	2310	2400
	BFL	0.450	358	1550	1610	1670	1730	1790	1860	1930	2000	2080	2150	2230
	6	2.92	260	1410	1460	1500	1560	1610	1660	1720	1780	1840	1900	1960
	7	5.71	163	1240	1270	1310	1340	1380	1420	1460	1500	1540	1580	1630
W18×60	TFL	0.000	880	2070	2170	2270	2380	2490	2610	2730	2860	2990	3130	3270
(984)	2	0.174	749	1980	2070	2170	2270	2370	2480	2590	2710	2830	2950	3080
	3	0.348	617	1880	1960	2050	2140	2230	2330	2430	2530	2640	2750	2860
	4	0.521	486	1760	1830	1900	1980	2060	2140	2230	2320	2410	2510	2610
	BFL	0.695	355	1610	1660	1720	1790	1850	1920	1990	2060	2140	2220	2300
	6	2.18	287	1520	1570	1620	1670	1730	1780	1840	1910	1970	2040	2110
	7	3.80	220	1420	1460	1500	1540	1590	1640	1680	1730	1790	1840	1900
W18×55	TFL	0.000	810	1880	1970	2070	2170	2270	2380	2490	2600	2720	2850	2980
(890)	2	0.158	691	1800	1880	1970	2060	2160	2260	2360	2470	2580	2690	2810
	3	0.315	573	1710	1790	1860	1950	2030	2120	2210	2310	2410	2510	2620
	4	0.473	454	1600	1670	1730	1810	1880	1960	2040	2120	2210	2300	2390
	BFL	0.630	336	1470	1520	1580	1640	1700	1760	1830	1900	1970	2040	2110
	6	2.15	269	1380	1430	1480	1530	1580	1630	1690	1750	1800	1870	1930
	7	3.86	203	1290	1320	1360	1400	1440	1490	1530	1580	1630	1670	1730
W18×50	TFL	0.000	735	1690	1770	1860	1950	2040	2140	2240	2350	2450	2570	2680
(800)	2	0.143	628	1620	1700	1780	1860	1940	2030	2130	2220	2320	2430	2530
	3	0.285	521	1540	1610	1680	1750	1830	1910	2000	2080	2170	2260	2360
	4	0.428	414	1440	1500	1560	1630	1700	1770	1840	1910	1990	2070	2160
	BFL	0.570	308	1330	1370	1430	1480	1530	1590	1650	1710	1780	1840	1910
	6	2.08	246	1250	1290	1330	1380	1420	1470	1520	1580	1630	1690	1740
	7	3.82	184	1160	1190	1220	1260	1300	1340	1380	1420	1460	1510	1550
W18×46	TFL	0.000	675	1540	1610	1690	1780	1860	1950	2040	2140	2240	2340	2450
(712)	2	0.151	583	1480	1550	1620	1700	1780	1860	1950	2040	2130	2220	2320
	3	0.303	492	1410	1470	1540	1610	1680	1760	1840	1920	2000	2090	2180
	4	0.454	400	1330	1380	1440	1500	1570	1630	1700	1780	1850	1930	2010
	BFL	0.605	308	1230	1280	1330	1380	1430	1490	1550	1610	1670	1730	1800
	6	2.42	239	1140	1180	1220	1270	1310	1360	1410	1460	1510	1570	1620
	7	4.36	169	1040	1070	1100	1140	1170	1210	1250	1280	1320	1370	1410

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

Rev. 11/1/02 [b] $\boxed{Y2}$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 5-4c for PNA locations.

[d] Value in parentheses is I_x (in.^{4}) of non-composite steel shape.

Table 5-15 (cont.). Lower Bound Elastic Moment of Inertia I_{LB} for Plastic Composite Sections

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kips	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W18×40	TFL	0.000	590	1320	1390	1450	1530	1600	1680	1760	1840	1930	2020	2110
(612)	2	0.131	511	1270	1330	1390	1460	1530	1600	1680	1760	1840	1920	2010
	3	0.263	432	1210	1270	1320	1390	1450	1510	1580	1650	1730	1800	1880
	4	0.394	353	1140	1190	1240	1300	1350	1410	1470	1530	1600	1670	1740
	BFL	0.525	274	1060	1100	1150	1190	1240	1290	1340	1390	1450	1510	1560
	6	2.26	211	985	1020	1060	1090	1130	1170	1220	1260	1310	1350	1400
	7	4.27	148	896	922	950	979	1010	1040	1070	1110	1140	1180	1210
W18×35	TFL	0.000	515	1120	1170	1230	1300	1360	1430	1500	1570	1650	1720	1800
(510)	2	0.106	451	1080	1130	1190	1240	1300	1370	1430	1500	1570	1640	1720
	3	0.213	388	1030	1080	1130	1190	1240	1300	1360	1420	1490	1550	1620
	4	0.319	324	978	1020	1070	1120	1170	1220	1270	1330	1390	1450	1510
	BFL	0.425	260	917	955	995	1040	1080	1130	1170	1220	1270	1320	1380
	6	2.37	194	842	873	906	940	975	1010	1050	1090	1130	1170	1220
	7	4.56	129	753	776	800	825	851	878	906	935	965	996	1030
W16×36	TFL	0.000	530	973	1030	1080	1140	1200	1270	1340	1410	1480	1550	1630
(448)	2	0.108	455	933	983	1040	1090	1150	1210	1270	1330	1400	1470	1540
	3	0.215	380	886	931	979	1030	1080	1130	1190	1250	1310	1370	1440
	4	0.323	305	831	871	912	956	1000	1050	1100	1150	1200	1260	1310
	BFL	0.430	229	765	797	831	867	905	944	984	1030	1070	1120	1160
	6	1.82	181	715	743	772	802	833	866	901	936	973	1010	1050
	7	3.46	133	659	680	703	727	752	778	805	833	862	892	923
W16×31	TFL	0.000	457	827	874	923	974	1030	1080	1140	1200	1260	1330	1400
(375)	2	0.110	396	795	838	884	931	981	1030	1090	1140	1200	1260	1320
	3	0.220	335	758	797	838	882	927	974	1020	1070	1130	1180	1240
	4	0.330	274	714	749	786	824	864	906	949	995	1040	1090	1140
	BFL	0.440	213	663	692	723	756	790	825	862	900	940	982	1020
	6	2.00	164	614	639	664	691	720	749	780	812	845	879	914
	7	3.80	114	556	574	594	614	636	658	681	705	730	756	783
W16×26	TFL	0.000	384	674	712	753	796	840	887	935	985	1040	1090	1150
(301)	2	0.0863	337	649	686	724	763	805	849	894	941	990	1040	1090
	3	0.173	289	621	654	689	726	764	804	846	889	934	980	1030
	4	0.259	242	589	619	651	683	718	754	791	830	871	912	956
	BFL	0.345	194	551	577	604	633	663	694	727	760	795	832	869
	6	2.05	145	505	527	549	572	597	622	649	676	705	734	765
	7	4.01	96	450	466	482	499	517	535	555	575	596	617	640
W14×38	TFL	0.000	560	844	896	951	1010	1070	1130	1200	1270	1340	1410	1490
(385)	2	0.129	473	805	853	903	956	1010	1070	1130	1190	1260	1330	1400
	3	0.258	386	759	802	847	894	943	995	1050	1100	1160	1220	1290
	4	0.386	299	704	741	779	819	861	905	951	999	1050	1100	1150
	BFL	0.515	211	636	665	695	726	759	794	830	868	907	948	990
	6	1.38	176	604	629	656	683	712	742	774	807	841	877	914
	7	2.53	140	568	589	611	634	659	684	710	738	766	796	827

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[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.
[b] $\boxed{Y2}$ = distance from top of the steel beam to concrete flange force.
[c] See Figure 5-4c for PNA locations.
[d] Value in parentheses is I_x (in.^{4}) of non-composite steel shape.

Table 5-15 (cont.). Lower Bound Elastic Moment of Inertia I_{LB} for Plastic Composite Sections

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kips	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W14×34	TFL	0.000	500	745	791	840	891	945	1000	1060	1120	1190	1250	1320
(340)	2	0.114	423	711	754	798	845	895	946	1000	1060	1110	1180	1240
	3	0.228	346	671	709	749	791	835	881	929	979	1030	1090	1140
	4	0.341	270	624	656	691	727	764	804	845	888	933	979	1030
	BFL	0.455	193	566	591	618	647	677	708	741	775	811	848	886
	6	1.42	159	535	558	581	606	632	659	687	717	748	780	813
	7	2.61	125	502	521	540	561	582	605	628	653	678	705	732
W14×30	TFL	0.000	443	642	682	725	770	817	866	918	972	1030	1090	1150
(291)	2	0.0963	378	614	651	691	732	775	821	868	918	969	1020	1080
	3	0.193	313	581	615	650	688	727	767	810	855	901	949	999
	4	0.289	248	543	572	603	635	669	704	741	780	820	862	905
	BFL	0.385	183	496	520	545	571	599	627	658	689	722	756	791
	6	1.46	147	466	486	507	530	553	578	604	630	658	687	717
	7	2.80	111	432	448	465	483	502	522	542	564	586	610	634
W14×26	TFL	0.000	385	553	589	626	665	706	749	794	841	890	941	994
(245)	2	0.105	332	530	563	598	634	672	712	754	797	843	890	938
	3	0.210	279	504	534	565	598	633	669	707	746	787	830	874
	4	0.315	226	473	499	527	556	586	618	652	686	722	760	799
	BFL	0.420	173	436	458	481	506	531	558	586	615	645	677	709
	6	1.67	135	405	423	443	463	485	507	530	555	580	607	634
	7	3.18	96.1	368	382	397	413	429	447	465	483	503	523	544
W14×22	TFL	0.000	325	453	483	514	547	581	617	655	694	735	778	822
(199)	2	0.0838	283	436	463	492	523	555	588	624	660	698	738	779
	3	0.168	241	416	441	467	495	525	555	587	621	656	692	730
	4	0.251	199	392	415	438	463	489	517	545	575	606	639	672
	BFL	0.335	157	365	384	404	426	448	472	496	522	548	576	605
	6	1.67	119	335	351	368	386	404	423	444	465	487	509	533
	7	3.32	81.1	301	312	325	338	352	366	381	397	413	430	448
W12×30	TFL	0.000	440	530	567	606	648	691	737	785	835	887	942	998
(238)	2	0.110	368	504	538	573	611	651	692	736	782	829	879	931
	3	0.220	296	473	503	534	567	602	639	678	718	760	804	850
	4	0.330	224	435	460	486	514	544	575	607	641	676	713	751
	BFL	0.440	153	389	408	428	449	472	495	520	546	573	601	631
	6	1.10	131	372	389	407	426	446	467	489	512	536	561	587
	7	1.92	110	355	370	385	402	419	438	457	477	498	520	542
W12×26	TFL	0.000	383	455	487	521	557	594	634	676	719	764	812	861
(204)	2	0.095	321	433	462	493	526	560	596	634	674	715	758	803
	3	0.190	259	407	432	460	489	519	551	585	620	656	694	734
	4	0.285	198	375	397	420	444	470	497	525	555	586	618	652
	BFL	0.380	136	336	352	370	389	409	429	451	474	498	523	548
	6	1.07	116	321	336	351	368	386	404	423	444	465	487	509
	7	1.94	95.6	304	317	331	345	360	376	392	410	428	447	467

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.
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[b] $\boxed{Y2}$ = distance from top of the steel beam to concrete flange force.
[c] See Figure 5-4c for PNA locations.
[d] Value in parentheses is I_x (in.^{4}) of non-composite steel shape.

Table 5-15 (cont.). Lower Bound Elastic Moment of Inertia I_{LB} for Plastic Composite Sections

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kips	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W12×22	TFL	0.000	324	371	398	427	458	490	523	559	596	634	674	716
(156)	2	0.106	281	356	381	408	436	466	497	530	564	600	638	676
	3	0.213	238	338	361	386	412	439	467	497	528	561	595	631
	4	0.319	196	318	339	360	383	408	433	460	487	517	547	578
	BFL	0.425	153	294	312	330	350	370	392	414	438	463	489	515
	6	1.66	117	270	285	300	316	333	351	370	389	410	431	453
	7	3.03	81	242	253	265	277	290	303	317	332	347	363	380
W12×19	TFL	0.000	279	313	336	361	387	414	443	473	505	538	573	608
(130)	2	0.0875	243	300	322	345	369	395	422	450	479	510	542	575
	3	0.175	208	286	306	327	349	373	398	423	450	479	508	539
	4	0.263	173	270	288	307	327	348	370	393	417	442	469	496
	BFL	0.350	138	251	266	283	300	318	337	357	378	400	423	447
	6	1.68	104	229	242	255	270	284	300	317	334	352	370	390
	7	3.14	69.6	203	212	222	233	244	255	267	280	293	307	321
W12×16	TFL	0.000	236	254	273	294	316	339	363	388	415	442	471	501
(103)	2	0.0663	209	245	263	282	303	324	347	371	396	422	449	477
	3	0.133	183	235	252	270	289	309	330	352	375	400	425	451
	4	0.199	156	223	239	255	272	291	310	330	351	373	396	420
	BFL	0.265	130	210	224	239	254	271	288	306	325	344	365	386
	6	1.71	94.3	189	200	212	225	238	251	266	281	297	313	331
	7	3.32	58.9	163	171	179	188	197	207	217	228	239	250	262
W12×14	TFL	0.000	208	220	237	255	274	295	316	338	361	386	411	437
(88.6)	2	0.0563	186	213	229	246	264	283	303	324	346	369	393	418
	3	0.113	163	204	219	235	252	270	288	308	328	350	372	395
	4	0.169	141	195	209	223	239	255	272	290	309	329	349	370
	BFL	0.225	119	184	197	210	224	238	254	270	287	305	323	342
	6	1.68	85.3	165	175	186	197	208	221	234	247	261	276	291
	7	3.35	52	141	148	155	163	171	179	188	198	207	218	228
W10×26	TFL	0.000	381	339	367	397	429	463	499	536	576	617	661	706
(144)	2	0.110	317	321	346	374	403	434	466	500	536	574	613	655
	3	0.220	254	300	322	346	372	399	428	458	490	523	557	594
	4	0.330	190	274	292	312	334	356	380	405	431	459	488	518
	BFL	0.440	127	241	255	270	286	303	321	340	360	381	402	425
	6	0.886	111	232	245	258	273	288	304	321	339	358	377	398
	7	1.49	95.1	222	233	245	258	271	286	301	317	333	351	369
W10×22	TFL	0.000	325	282	306	331	358	387	417	449	483	518	555	593
(118)	2	0.090	273	267	289	313	337	364	391	420	451	483	517	552
	3	0.180	221	251	270	291	312	336	360	386	413	442	472	503
	4	0.270	169	230	246	264	282	302	323	345	368	392	417	443
	BFL	0.360	118	205	218	232	246	261	277	295	312	331	351	371
	6	0.962	99.3	195	206	218	230	244	258	273	289	305	323	341
	7	1.72	81.1	183	193	203	214	225	238	250	264	278	293	308

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

Rev. 11/1/02 [b] $\boxed{Y2}$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 5-4c for PNA locations.

[d] Value in parentheses is I_x (in.^{4}) of non-composite steel shape.

Table 5-15 (cont.). Lower Bound Elastic Moment of Inertia I_{LB} for Plastic Composite Sections

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kips	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W10×19	TFL	0.000	281	238	259	281	304	329	355	383	412	443	474	508
(96.3)	2	0.0988	241	227	246	267	288	311	335	361	388	416	445	476
	3	0.198	202	215	232	251	270	291	313	336	360	386	413	440
	4	0.296	162	200	215	231	248	266	286	306	327	350	373	397
	BFL	0.395	122	182	195	208	222	237	253	270	287	306	325	345
	6	1.25	96.2	169	179	190	202	215	228	243	257	273	289	306
	7	2.29	70.3	153	161	170	179	189	200	211	223	235	248	261
W10×17	TFL	0.000	250	206	224	244	264	286	310	334	360	387	415	445
(81.9)	2	0.0825	216	197	214	232	251	272	293	316	340	365	391	418
	3	0.165	183	187	202	219	236	255	274	295	317	340	364	388
	4	0.248	150	175	189	203	219	235	253	271	290	311	332	354
	BFL	0.330	117	161	173	185	198	212	227	243	259	276	294	313
	6	1.31	89.8	148	157	167	178	190	202	215	229	243	258	274
	7	2.45	62.4	132	139	147	155	164	173	183	193	204	215	227
W10×15	TFL	0.000	221	177	193	210	228	248	268	289	312	336	361	387
(68.9)	2	0.0675	194	170	185	201	218	236	255	275	296	318	342	366
	3	0.135	167	162	176	190	206	223	240	259	278	299	320	342
	4	0.203	140	153	165	178	192	207	223	240	258	276	295	315
	BFL	0.270	113	142	153	164	177	190	204	218	233	250	266	284
	6	1.35	83.8	128	137	147	157	167	178	190	203	216	229	244
	7	2.60	55.1	112	118	125	133	140	148	157	166	175	185	196
W10×12	TFL	0.000	177	139	152	165	180	195	211	229	247	265	285	306
(53.8)	2	0.0525	156	134	145	158	172	186	201	217	234	252	271	290
	3	0.105	135	127	138	150	163	176	190	205	221	237	254	272
	4	0.158	115	121	131	142	153	165	178	191	206	221	236	252
	BFL	0.210	93.8	113	122	131	141	152	163	175	187	200	214	228
	6	1.30	69.0	102	109	116	124	133	142	152	162	173	184	195
	7	2.61	44.3	87.9	93.0	98.4	104	110	117	124	131	139	146	155

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

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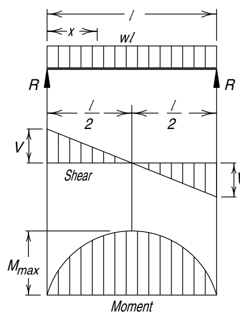
[b] $\boxed{Y2}$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 5-4c for PNA locations.

[d] Value in parentheses is I_x (in.^{4}) of non-composite steel shape.

Table 5-17 (cont.). Shears, Moments, and Deflections

1. SIMPLE BEAM—UNIFORMLY DISTRIBUTED LOAD



Total Equiv. Uniform Load .... $= wl$

$R = V$ .... $= \frac{wl}{2}$

V_x .... $= w\left(\frac{l}{2} - x\right)$

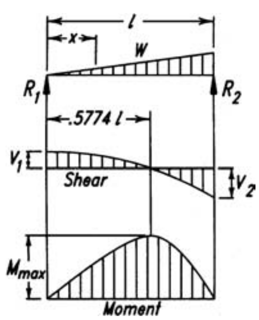
M_{max} (at center)............................ $= \frac{wl^2}{8}$

M_x .... $= \frac{wx}{2}(l - x)$

Δ_{max} (at center)............................ $= \frac{5wl^4}{384EI}$

Δ_x .... $= \frac{wx}{24EI}(l^3 - 2lx^2 + x^3)$

2. SIMPLE BEAM—LOAD INCREASING UNIFORMLY TO ONE END



Total Equiv. Uniform Load .... $= \frac{16W}{9\sqrt{3}} = 1.03W$

$R_1 = V_1$ .... $= \frac{W}{3}$

$R_2 = V_2 = V_{max}$ .... $= \frac{2W}{3}$

V_x .... $= \frac{W}{3} - \frac{Wx^2}{l^2}$

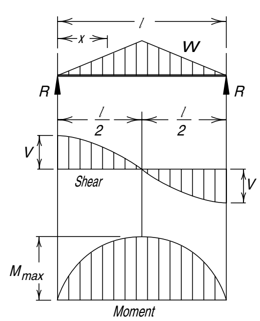
M_{max} (at $x = \frac{l}{\sqrt{3}} = .577l$) .... $= \frac{2Wl}{9\sqrt{3}} = .128Wl$

M_x .... $= \frac{Wx}{3l^2}(l^2 - x^2)$

Δ_{max} (at $x = l\sqrt{1 - \sqrt{\frac{8}{15}}} = .519l$) .... $= \boxed{0.0130\,\frac{Wl^3}{EI}}$ Rev. 11/1/02

Δ_x .... $= \frac{Wx}{180EIl^2}(3x^4 - 10l^2x^2 + 7l^4)$

3. SIMPLE BEAM—LOAD INCREASING UNIFORMLY TO CENTER



Total Equiv. Uniform Load .... $= \frac{4W}{3}$

$R = V$ .... $= \frac{W}{2}$

V_x (when $x < \frac{l}{2}$) .... $= \frac{W}{2l^2}(l^2 - 4x^2)$

M_{max} (at center)............................ $= \frac{Wl}{6}$

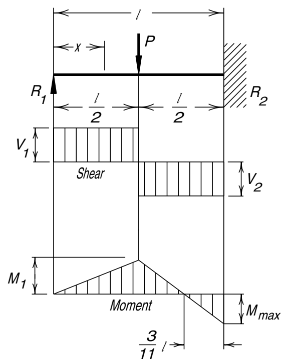
M_x (when $x < \frac{l}{2}$) .... $= Wx\left(\frac{1}{2} - \frac{2x^2}{3l^2}\right)$

Δ_{max} (at center)............................ $= \frac{Wl^3}{60EI}$

Δ_x (when $x < \frac{l}{2}$) .... $= \frac{Wx}{480EIl^2}(5l^2 - 4x^2)^2$

Table 5-17 (cont.). Shears, Moments and Deflections

13. BEAM FIXED AT ONE END, SUPPORTED AT OTHER—CONCENTRATED LOAD AT CENTER



Total Equiv. Uniform Load .... $= \frac{3P}{2}$

$R_1 = V_1$ .... $= \boxed{\frac{5P}{16}}$ Rev. 11/1/02

$R_2 = V_{2\ max}$ .... $= \frac{11P}{16}$

M_{max} (at fixed end) .... $= \frac{3Pl}{16}$

M_1 (at point of load) .... $= \frac{5Pl}{32}$

$M_x \left(\text{when } x < \frac{l}{2}\right)$ .... $= \frac{5Px}{16}$

$M_x \left(\text{when } x > \frac{l}{2}\right)$ .... $= P\left(\frac{l}{2} - \frac{11x}{16}\right)$

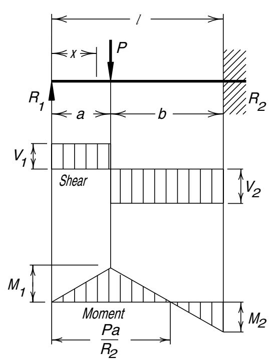
$\Delta_{max} \left(\text{at } x = l\sqrt{\frac{1}{5}} = 0.447l\right)$ .... $= \frac{Pl^3}{48EI\sqrt{5}} = 0.00932\frac{Pl^3}{EI}$

Δ_x (at point of load) .... $= \boxed{\frac{7Pl^3}{768EI}}$ Rev. 11/1/02

$\Delta_x \left(\text{when } x < \frac{l}{2}\right)$ .... $= \frac{Px}{96EI}(3l^2 - 5x^2)$

$\Delta_x \left(\text{when } x > \frac{l}{2}\right)$ .... $= \frac{P}{96EI}(x-l)^2(11x - 2l)$

14. BEAM FIXED AT ONE END, SUPPORTED AT OTHER—CONCENTRATED LOAD AT ANY POINT



$R_1 = V_1$ .... $= \frac{Pb^2}{2l^3}(a + 2l)$

$R_2 = V_2$ .... $= \frac{Pa}{2l^3}(3l^2 - a^2)$

M (at point of load) .... $= R_1 a$

M_2 (at fixed end) .... $= \frac{Pab}{2l^2}(a + l)$

M_x (when $x < a$) .... $= R_1 x$

M_x (when $x > a$) .... $= R_1 x - P(x - a)$

$\Delta_{max} \left(\text{when } a < 0.414l \text{ at } x = l\frac{(l^2 + a^2)}{(3l^2 - a^2)}\right) = \frac{Pa(l^2 + a^2)^3}{3EI(3l^2 - a^2)^2}$

$\Delta_{max} \left(\text{when } a > 0.414l \text{ at } x = l\sqrt{\frac{a}{2l + a}}\right)$ .. $= \frac{Pab^2}{6EI}\sqrt{\frac{a}{2l + a}}$

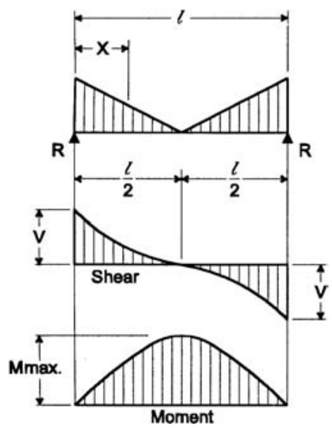
Δ_a (at point of load) .... $= \boxed{\frac{Pa^2b^3}{12EIl^3}(3l + a)}$ Rev. 11/1/02

Δ (when $x < a$) .... $= \frac{Pb^2x}{12EIl^3}(3al^2 - 2lx^2 - ax^2)$

Δ_x (when $x > a$) .... $= \frac{Pa}{12EIl^2}(l - x)^2(3l^2x - a^2x - 2a^2l)$

Table 5-17 (cont.). Shears, Moments and Deflections

34. SIMPLE BEAM—LOAD INCREASING UNIFORMLY FROM CENTER



Total Equiv. Uniform Load .... $= \frac{2W}{3}$

$R = V$ .... $= \frac{W}{2}$

V_x $\left(\text{when } x < \frac{l}{2}\right)$ .... $= \frac{W}{2}\left(\frac{l-2x}{l}\right)^2$

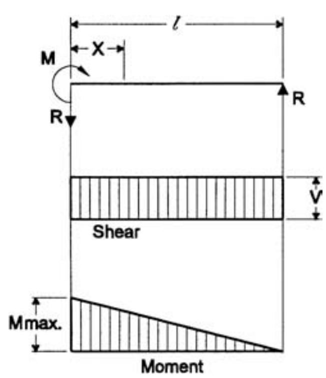
M_{max} (at center) .... $= \frac{Wl}{12}$

M_x $\left(\text{when } x < \frac{l}{2}\right)$ .... $= \frac{W}{2}\left(x - \frac{2x^2}{l} + \frac{4x^3}{3l^2}\right)$

Δ_{max} (at center) .... $= \frac{3Wl^3}{320EI}$

Δ_x $\left(\text{when } x < \frac{l}{2}\right)$ .... $= \frac{W}{12EI}\left(x^3 - \frac{x^4}{l} + \boxed{\frac{2x^5}{5l^2}} - \frac{3l^2x}{8}\right)$ Rev. 11/1/02

35. SIMPLE BEAM—CONCENTRATED MOMENT AT END



Total Equiv. Uniform Load .... $= \frac{8M}{l}$

$R = V$ .... $= \frac{M}{l}$

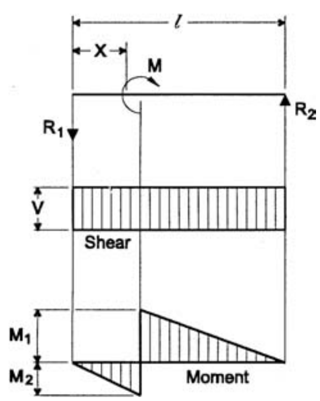
M_{max} .... $= M$

M_x .... $= M\left(1 - \frac{x}{l}\right)$

Δ_{max} (at $x = 0.442l$) .... $= 0.0042\frac{Ml^2}{EI}$

Δ_x .... $= \frac{M}{6EI}\left(3x^2 - \frac{x^3}{l} - 2lx\right)$

36. SIMPLE BEAM—CONCENTRATED MOMENT AT ANY POINT



Total Equiv. Uniform Load .... $= \frac{8M}{l}$

$R = V$ .... $= \frac{M}{l}$

M_x (when $x < a$) .... $= Rx$

M_x (when $x > a$) .... $= M + Rx$

Δ_x (when $x < a$) .... $= \frac{M}{6EI}\left[\left(6a - \frac{3a^2}{l} - 2l\right)x - \frac{x^3}{l}\right]$

Δ_x .... $= \frac{M}{6EI}\left[3(a^2 + x^2) - \frac{x^3}{l} - \left(2l + \frac{3a^2}{l}\right)x\right]$

Therefore, LRFD Specification Equation H1-1a governs. With $M_{uy} = 0$, this equation becomes:

$$\frac{P_u}{\phi_t P_n} + \frac{8}{9}\frac{M_{ux}}{\phi_b M_{nx}} \leq 1.0$$

From Table 5-3, a W10×22 is compact, has $L_p = 4.70$ ft > $L_b = 4$ ft (braced) and, $\phi_b M_{nx} = 97.5$ kip-ft. Thus,

$$\begin{aligned}\frac{P_u}{\phi_t P_n} + \frac{8}{9}\frac{M_{ux}}{\phi_b M_{nx}} &= 0.479 + \frac{8}{9}\frac{55\text{ kip-ft}}{97.5\text{ kip-ft}}\\ &= 0.479 + 0.501\\ &= 0.980 < 1.0 \textbf{ o.k.}\end{aligned}$$

EXAMPLE 6.2. W-Shape subject to combined axial compression and flexure (braced frame).

Given: Check the adequacy of an ASTM A992 W14×176 with $L_x = L_y = 14.0$ ft in a symmetric braced frame subject to the loading $P_u = 1{,}400$ kips, $M_{ux} = 200$ kip-ft, $M_{uy} = 70$ kip-ft. Assume reverse-curvature bending with equal end moments about both axes and no loads along the member.

$F_y = 50$ ksi	$A = 51.8$ in.^{2}	$r_x = 6.43$ in.	Rev. 11/1/02
$F_u = 65$ ksi	$Z_x = 320$ in.^{3}	$r_y = 4.02$ in.	
	$Z_y = 163$ in.^{3}	$\boxed{I_x = 2{,}140 \text{ in.}^4}$	

Solution: For a braced frame, $K = 1.0$ and $K_x L_x = K_y L_y = 14.0$ ft. From Table 4-2,

$$\phi_c P_n = 1{,}940 \text{ kips}$$

From LRFD Specification Section H1.2,

$$\begin{aligned}\frac{P_u}{\phi_c P_n} &= \frac{1{,}400\text{ kips}}{1{,}940\text{ kips}}\\ &= 0.722 > 0.2\end{aligned}$$

Therefore, LRFD Specification Equation H1-1a governs.

From LRFD Specification Section C1.2, for a braced frame, $M_{lt} = 0$. From LRFD Specification Equation C1-1

$$\begin{aligned}M_{ux} &= B_{1x} M_{ntx}, \quad \text{where } M_{ntx} = 200 \text{ kip-ft; and}\\ M_{uy} &= B_{1y} M_{nty}, \quad \text{where } M_{nty} = 70 \text{ kip-ft}\end{aligned}$$

From LRFD Specification Equations C1-2 and C1-3:

$$B_1 = \frac{C_m}{(1 - P_u/P_{e1})} \geq 1$$

where in this case (a braced frame with no transverse loading),

$$C_m = 0.6 - 0.4(M_1/M_2)$$

For reverse curvature bending and equal end moments:

$$M_1/M_2 = +1.0$$
$$C_m = 0.6 - 0.4(1.0) = 0.2$$
$$\boxed{P_{e1} = \frac{\pi^2 EI}{(KL)^2}}$$

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From Table 4-2,

$$\begin{aligned} P_{e1x} &= 61{,}300\,\text{kip-in.}^2 \times 10^4/(14.0\,\text{ft} \times 12\,\text{in./ft})^2 \\ &= 21{,}700\,\text{kips} \end{aligned}$$

$$\begin{aligned} P_{e1y} &= 24{,}000\,\text{kip-in.}^2 \times 10^4/(14.0\,\text{ft} \times 12\,\text{in./ft})^2 \\ &= 8{,}500\,\text{kips} \end{aligned}$$

Thus,

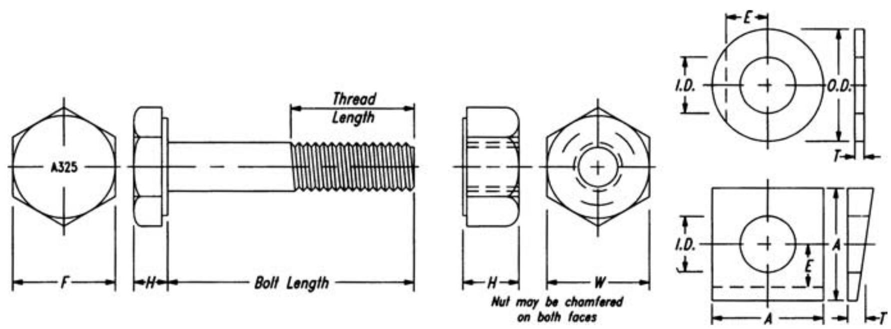
$$\begin{aligned} B_{1x} &= \frac{C_{mx}}{(1 - P_u/P_{e1x})} \geq 1 \\ &= \frac{0.2}{(1 - 1{,}400\,\text{kips}/21{,}700\,\text{kips})} \geq 1 \\ &= 0.214 \geq 1 \\ &= 1 \\ B_{1y} &= \frac{C_{my}}{(1 - P_u/P_{e1y})} \geq 1 \\ &= \frac{0.2}{(1 - 1{,}400\,\text{kips}/8{,}500\,\text{kips})} \geq 1 \\ &= 0.239 \geq 1 \\ &= 1 \end{aligned}$$

$$\begin{aligned} M_{ux} &= 1.0 \times 200\,\text{kip-ft} \\ M_{uy} &= 1.0 \times 70\,\text{kip-ft} \end{aligned}$$

From LRFD Specification Equation F1-4,

$$\begin{aligned} L_p &= 1.76 r_y \sqrt{\frac{E}{F_{yf}}} \\ &= 1.76(4.02\,\text{in.})\sqrt{\frac{29{,}000\,\text{ksi}}{50\,\text{ksi}}}(1\,\text{ft}/12\,\text{in.}) \\ &= 14.2\,\text{ft} \end{aligned}$$

Table 7-1. Dimensions of High-Strength Fasteners, in.



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	Measurement	Nominal Bolt Diameter d_b, in. 1/2	5/8	3/4	7/8	1	1 1/8	1 1/4	1 3/8	1 1/2
A325 and A490 Bolts[a]	Width Across Flats, *F*	7/8	1 1/16	1 1/4	1 7/16	1 5/8	1 13/16	2	2 3/16	2 3/8
	Height, *H*	5/16	25/64	15/32	35/64	39/64	11/16	25/32	27/32	15/16
	Thread Length	1	1 1/4	1 3/8	1 1/2	1 3/4	2	2	2 1/4	2 1/4
	Bolt Length[f] = Grip +− >	11/16	7/8	1	1 1/8	1 1/4	1 1/2	1 5/8	1 3/4	1 7/8
A563 Nuts[b]	Width Across Flats, *W*	7/8	1 1/16	1 1/4	1 7/16	1 5/8	1 13/16	2	2 3/16	2 3/8
	Height, *H*	31/64	39/64	47/64	55/64	63/64	1 7/64	1 7/32	1 11/32	1 15/32
F436 Circular Washers[c]	Nom. Outside Diameter, *OD*	1 1/16	1 5/16	1 15/32	1 3/4	2	2 1/4	2 1/2	2 3/4	3
	Diameter, *ID*	17/32	11/16	13/16	15/16	1 1/8	1 1/4	1 3/8	1 1/2	1 5/8
	Thckns., *T* Min.	0.097	0.122	0.122	0.136	0.136	0.136	0.136	0.136	0.136
	Thckns., *T* Max.	0.177	0.177	0.177	0.177	0.177	0.177	0.177	0.177	0.177
	Min. Edge Distance, *E*[d]	7/16	9/16	21/32	25/32	7/8	1	1 3/32	1 7/32	1 5/16
F436 Square or Rect. Washers[c,e]	Dimension, *A*	1 3/4	1 3/4	1 3/4	1 3/4	1 3/4	2 1/4	2 1/4	2 1/4	2 1/4
	Mean Thckns., *T*	5/16	5/16	5/16	5/16	5/16	5/16	5/16	5/16	5/16
	Taper in Thickness	2:12	2:12	2:12	2:12	2:12	2:12	2:12	2:12	2:12
	Min. Edge Distance, *E*[d]	7/16	9/16	21/32	25/32	7/8	1	1 3/32	1 7/32	1 5/16

[a] Tolerances as specified in ASTM A325 and A490.
[b] Tolerances as specified in ASTM A563.
[c] ASTM F436 Washer Tolerances, in.:

Nominal Outside Diameter	$-1/32; +1/32$
Nominal Diameter of Hole	$-0; +1/32$
Flatness; max. deviation from straight-edge placed on cut side shall not exceed	0.010
Concentricity: center of hole to outside diameter (full indicator runout)	0.030
Burr shall not project above immediately adjacent washer surface more than	0.010

[d] For clipped washers only.
[e] For use with American standard beams (S) and channel (C).
[f] Tabular value does not include thickness of washer(s).

Table 7-13.
Design Bearing Strength at Bolt Holes for Various Edge Distances[a]
kips/in. thickness

Hole Type	Edge Distance L_e, in.	F_u, ksi	Nominal Bolt Diameter d_b, in.							
			$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$
STD SSLT	$1\frac{1}{4}$	58	47.3	44.0	40.8	37.5	34.3	31.0	27.7	24.5
		65	53.0	49.4	45.7	42.0	38.4	34.7	31.1	27.4
	2	58	65.2	78.3	79.9	76.7	73.4	70.1	66.9	63.6
		65	73.1	87.7	89.6	85.9	82.3	78.6	75.0	71.3
SSLP	$1\frac{1}{4}$	58	42.4	39.2	35.9	31.0	26.1	22.8	19.6	16.3
		65	47.5	43.9	40.2	34.7	29.2	25.6	21.9	18.3
	2	58	65.2	78.3	75.0	70.1	65.2	62.0	58.7	55.5
		65	73.1	87.7	84.1	78.6	73.1	69.5	65.8	62.2
OVS	$1\frac{1}{4}$	58	44.0	40.8	37.5	32.6	27.7	24.5	21.2	17.9
		65	49.4	45.7	42.0	36.6	31.1	27.4	23.8	20.1
	2	58	65.2	78.3	76.7	71.8	66.9	63.6	60.4	57.1
		65	73.1	87.7	85.9	80.4	75.0	71.3	67.6	64.0
LSLP	$1\frac{1}{4}$	58	24.5	16.3	8.16	–	–	–	–	–
		65	27.4	18.3	9.14	–	–	–	–	–
	2	58	63.6	55.5	47.3	39.2	31.0	22.8	14.7	6.53
		65	71.3	62.2	53.0	43.9	34.7	25.6	16.5	7.31
LSLT	$1\frac{1}{4}$	58	39.4	36.7	34.0	31.3	28.5	25.8	23.1	20.4
		65	44.2	41.1	38.1	35.0	32.0	28.9	25.9	22.9
	2	58	54.4	65.2	66.6	63.9	61.2	58.5	55.7	53.0
		65	60.9	73.1	74.6	71.6	68.6	65.5	62.5	59.4
STD, SSLT, SSLP, OVS, LSLP	$L_e \geq L_{e\,full}$	58	65.2	78.3	91.3	104	117	130	144	157
		65	73.1	87.7	102	117	132	146	161	175
LSLT	$L_e \geq L_{e\,full}$	58	54.4	65.2	76.1	87.0	97.9	109	120	130
		65	60.9	73.1	85.3	97.5	110	122	134	146
Edge distance for full bearing strength $L_{e\,full}$[b], in.		STD, SSLT, LSLT	$1\frac{5}{8}$	$1\frac{15}{16}$	$2\frac{1}{4}$	$2\frac{9}{16}$	$2\frac{7}{8}$	$3\frac{3}{16}$	$3\frac{1}{16}$	$3\frac{13}{16}$
		OVS	$1\frac{11}{16}$	2	$2\frac{5}{16}$	$2\frac{5}{8}$	3	$3\frac{5}{16}$	$3\frac{5}{8}$	$3\frac{15}{16}$
		SSLP	$1\frac{11}{16}$	2	$2\frac{5}{16}$	$2\frac{11}{16}$	3	$3\frac{5}{16}$	$3\frac{5}{8}$	$3\frac{15}{16}$
		LSLP	$2\frac{1}{16}$	$2\frac{7}{16}$	$2\frac{7}{8}$	$3\frac{1}{4}$	$3\frac{11}{16}$	$4\frac{1}{16}$	$4\frac{1}{2}$	$4\frac{7}{8}$

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LSLP = Long-Slotted Hole oriented parallel to the line of force.
LSLT = Long-Slotted Hole oriented transverse to the line of force.
OVS = Oversized Hole.
SSLP = Short-Slotted Hole oriented parallel to the line of force.
SSLT = Short-Slotted Hole oriented transverse to the line of force.
STD = Standard Hole.

–indicates edge distance is inadequate for this hole size.
[a] Edge distance indicated is from the center of the hole or slot to the edge of the material in the line of force. Hole deformation is considered. When hole deformation is not a consideration, see LRFD Specification Section J3.10.
[b] Decimal value has been rounded to the nearest sixteenth of an inch.

$\phi = 0.75$

Table 7-14.
Design Tensile Strength of Bolts, kips

ASTM Desig.	ϕF_t (ksi)	Nominal Bolt Diameter d_b, in.							
		$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$
		Nominal Bolt Area, in.^{2}							
		0.307	0.442	0.601	0.785	0.994	1.23	1.48	1.77
A325 & F1852	67.5	20.7	29.8	40.6	53.0	67.1	82.8	100	119
A490	84.8	26.0	37.4	51.0	66.6	84.2	104	126	150
A307	33.8	10.4	14.9	20.3	26.5	33.5	41.4	50.1	59.6

$\phi = 1.0$ STD
$\phi = 0.85$ OVS & SSL
$\phi = 0.7$ LSLT
$\phi = 0.6$ LSLP

Table 7-16. Slip-Critical Connections Design Resistance to Shear at Service Loads Using Service Loads, ϕR_n, kips[1] (Class A Faying Surface, $\mu = 0.33$)[2]

ASTM Desig.	Hole Type	Loading	Nominal Bolt Diameter d_b, in.							
			5/8	3/4	7/8	1	1 1/8	1 1/4	1 3/8	1 1/2
			Nominal Bolt Area, in.[2]							
			0.307	0.442	0.601	0.785	0.994	1.23	1.48	1.77
A325 F1852	STD	S	5.22	7.51	10.2	13.4	16.9	20.9	25.2	30.0
		D	10.4	15.0	20.4	26.7	33.8	41.7	50.5	60.1
	OVS SSL	S	4.60	6.63	9.02	11.8	14.9	18.4	22.3	26.5
		D	9.20	13.3	18.0	23.6	29.8	36.8	44.5	53.0
	LSLT	S	3.68	5.30	7.22	9.42	11.9	14.7	17.8	21.2
		D	7.36	10.6	14.4	18.8	23.9	29.5	35.6	42.4
	LSLP	S	3.07	4.42	6.01	7.85	9.94	12.3	14.8	17.7
		D	6.14	8.84	12.0	15.7	19.9	24.5	29.7	35.3
A490	STD	S	6.44	9.28	12.6	16.5	20.9	25.8	31.2	37.1
		D	12.9	18.6	25.3	33.0	41.7	51.5	62.4	74.2
	OVS SSL	S	5.52	7.95	10.8	14.1	17.9	22.1	26.7	31.8
		D	11.0	15.9	21.6	28.3	35.8	44.2	53.5	63.6
	LSLT	S	4.60	6.63	9.02	11.8	14.9	18.4	22.3	26.5
		D	9.20	13.3	18.0	23.6	29.8	36.8	44.5	53.0
	LSLP	S	3.99	5.74	7.82	10.2	12.9	16.0	19.3	23.0
		D	7.98	11.5	15.6	20.4	25.8	31.9	38.6	45.9

STD = Standard Hole.
OVS = Oversized Hole.
SSL = Short-Slotted Hole.
LSLP = Long-Slotted Hole parallel to line of force.
LSLT = Long-Slotted Hole transverse to line of force.
S = Single Shear.
D = Double Shear.

[1] For design slip resistance using factored loads, refer to Table 7-15.
[2] For Class B faying surfaces, multiply the tabled design resistance by 1.52.
For Class C faying surfaces, multiply the tabled design resistance by 1.06.

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SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of connection elements (angles, plates, tees, gussets, etc.) used to transfer load from one structural member to another, as well as the affected elements of the connected members (beam webs, beam flanges, column webs, column flanges, etc.). For design considerations for bolted and welded joints, see Parts 7 and 8, respectively. For the design of connections, see Parts 10 through 15. For the design of connection elements for HSS and steel pipe, see the AISC *Hollow Structural Sections Connections Manual*. For connection elements that are part of a seismic-force-resisting system in which the seismic response modification factor R is taken greater than 3, see the Seismic Provisions, which are available from AISC at www.aisc.org.

LOAD DETERMINATION

The required strength(s) for connection elements are determined using force-transfer models such as those described in Parts 10 through 15 to distribute the member end reactions, which are determined by analysis as indicated in LRFD Specification Section A5.

GROSS AREA, NET AREA AND WHITMORE SECTION

In the determination of the design strength of connection elements, the gross area A_g is of interest for yielding limit states and the net area A_n is of interest for rupture limit states. In either case, the Whitmore section may limit the effective width to less than the overall dimension of a connection element.

Gross Area

The gross area A_g is determined as specified in LRFD Specification Section B1, subject to the limitations given below for the Whitmore section.

Net Area

The net area A_n is determined as specified in LRFD Specification Section B2, except as limited in LRFD Specification Section J5.2(b) and subject to the limitations given below for the Whitmore section. The reduction in area for bolt holes can be determined using Table 9-1.

Whitmore Section (Effective Width)

When connection elements are large in comparison to the bolted or welded joints within them, the Whitmore section may limit the gross and net areas of the connection element to less than the full area (Whitmore, 1952). As illustrated in Figure 9-1, the width of the Whitmore section is determined at the end of the joint by spreading the force from the start of the joint 30 degrees to each side in the connection element along the line of force. The Whitmore section may spread across the joint between connection elements (see Figure 13-11), but cannot spread beyond an unconnected edge. Rev. 11/1/02

YIELDING LIMIT STATES

Connection Elements Subject to Shear

The shear yielding design strength ϕR_n, which must equal or exceed the required strength (factored shear) R_u, is determined in accordance with LRFD Specification Section J5.3 using Equation J5-3.

$$
\begin{aligned}
f &= \text{plate buckling model adjustment factor}\\
&= \frac{2c}{d} \quad \text{when} \quad \frac{c}{d} \le 1.0\\
&= 1 + \frac{c}{d} \quad \text{when} \quad \frac{c}{d} > 1.0\\
k &= \text{plate buckling coefficient}\\
&= 2.2\left(\frac{h_o}{c}\right)^{1.65} \quad \text{when} \quad \boxed{\frac{c}{h_o} \le 1.0}\\
&= \frac{2.2h_o}{c} \quad \text{when} \quad \boxed{\frac{c}{h_o} > 1.0}
\end{aligned}
$$

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$h_o = d - d_c$, reduced beam depth, in. Note that, for convenience, the dimension h_0 as illustrated in Figure 9-2 is used in these calculations instead of the more correct dimension h_1 to eliminate the detailed calculation required to locate the neutral axis of the coped beam. Alternatively, the dimension h_1 may be substituted for h_0 in the local buckling calculations.

c = cope length as illustrated in Figure 9-2, in.

d = beam depth, in.

d_c = cope depth as illustrated in Figure 9-2, in.

When a beam is coped at both flanges, the design buckling stress ϕF_{cr} is based upon a web buckling model with an adjustment factor f_d (Cheng et al., 1984). The design buckling stress ϕF_{cr} for a beam coped at both flanges when $c \le 2d$ and $d_c \le 0.2d$ (see Figure 9-3) is:

$$
\begin{aligned}
\phi F_{cr} &= \phi 0.62\pi E \frac{{t_w}^2}{ch_o} f_d\\
&= 50{,}840 \frac{{t_w}^2}{ch_o} f_d
\end{aligned}
$$

where

$$f_d = 3.5 - 7.5\left(\frac{d_c}{d}\right)$$

d_c = cope depth at the compression flange, in.

and all other variables are as defined previously.

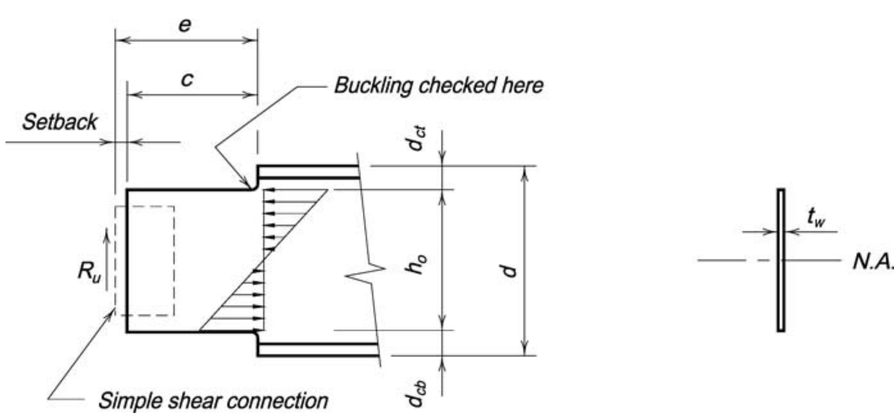

Fig. 9–3. Local buckling of beam web coped at both flanges.

When a beam is coped at both flanges and $d_c > 0.2d$, a conservative procedure also based upon the aforementioned classical plate buckling equation can be used. Including both elastic and inelastic buckling, the design buckling stress ϕF_{cr} is

$$\phi F_{cr} = 0.9 F_y Q$$

where

$$\begin{aligned} Q &= 1 \text{ for } \lambda \le 0.7 \\ &= (1.34 - 0.486\lambda) \text{ for } 0.7 < \lambda \le 1.41 \\ &= (1.30/\lambda^2) \text{ for } \lambda > 1.41 \end{aligned}$$

$$\boxed{\lambda = \frac{\sqrt{F_y}}{167}\frac{1}{\sqrt{K}}\frac{h_o}{2t_w}}$$

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K = plate buckling coefficient, which depends upon the plate aspect ratio. If the length is denoted by c and the half depth of the beam web remaining is denoted by $h_o/2$, then K is related to the aspect ratio $2c/h_o$ as given in the following table:

$2c/h_o$	K
0.25	16
0.3	13
0.4	10
0.5	6
0.6	4.5
0.75	2.5
1	1.3
1.5	0.8
2	0.6
3	0.5
≥ 4	0.425

When $2c/h_o$ is equal to or greater than 4, K has the constant value of 0.425 and this approach can be reduced to that for single angles given in LRFD Specification Section B5.3a(a).

BEARING LIMIT STATES

Bearing Strength at Bolt Holes

For design bearing strength at bolt holes, see Part 7.

Steel-on-Steel Bearing Strength (Other Than at Bolt Holes)

Bearing strength for applications other than at bolt holes is determined as given in LRFD Specification Section J8. The fabrication and erection requirements in LRFD Specification Sections M2.6, M2.8, and M4.4 are applicable to connection elements that transfer load by contact bearing on steel.

Bearing Strength on Concrete or Masonry

The bearing strength of concrete or masonry is determined as given in LRFD Specification Section J9. The fabrication and erection requirements in LRFD Specification Sections M2.8 and M4.1 are applicable to connection elements that transfer load by contact bearing on concrete or masonry.

but need not exceed $^3/_4\, t_s$, where

t_f = thickness of the tee flange, in.
t_s = thickness of the tee stem, in.
b = flexible width in connection element as illustrated in Figure 9-5, in.
L = depth of connection element as illustrated in Figure 9-5, in.

For a tee bolted to the support and bolted or welded to the supported beam, the minimum diameter for bolts through the tee flange for ductility must be such that

$$\boxed{d_{b\ \min} = 0.163\, t_f \sqrt{\frac{F_y}{b}\left(\frac{b^2}{L^2} + 2\right)}}$$

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but need not exceed $0.69\sqrt{t_s}$. Additionally, to provide for rotational ductility when the tee stem is bolted to the supported beam, the maximum tee stem thickness should be such that

$$t_{s\ \max} = \frac{d_b}{2} + {}^1/_{16}\ \text{in.}$$

When the tee stem is welded to the supported beam, there is no perceived ductility problem for this weld.

Concentrated Forces

If the connection element delivers a concentrated force to a member or other connection element, see LRFD Specification Section K1 or HSS Specification Section 8, as appropriate. See also AISC Design Guide No. 13 *Wide-Flange Column Stiffening at Moment Connections: Wind and Seismic Applications*.

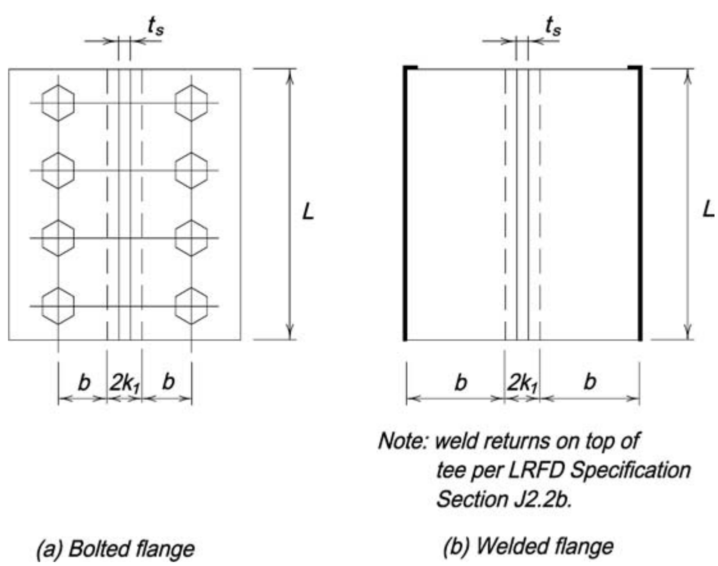

Fig. 9–5. Illustration of variables in shear connection ductility checks.

$F_y = 50$ ksi
$\phi = 1.00$
$\phi_r = 0.75$
$\phi_v = 0.90$

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Table 9-5. Beam End Bearing Constants

Shape	ϕR_1 kips	ϕR_2 kips/in.	$\phi_r R_3$ kips	$\phi_r R_4$ kips/in.	$\phi_r R_5$ kips	$\phi_r R_6$ kips/in.	$\phi R (N = 3^1/_4)$ kips	$\phi_v V_n$ kips
W44×335	327	51.0	495	14.8	452	19.7	500	1210
×290	258	43.5	368	10.4	338	13.8	402	1020
×262	218	39.5	302	8.69	277	11.6	331	924
×230	178	35.5	239	7.41	218	9.88	263	822
W40×593	990	89.5	1550	44.8	1430	59.7	1300	2080
×503	758	77.0	1150	34.1	1050	45.4	1020	1750
×431	593	67.0	861	26.8	787	35.7	825	1490
×397	516	61.0	722	21.8	662	29.1	732	1350
×372	468	58.0	646	20.3	591	27.1	669	1270
×362	447	56.0	607	18.7	557	24.9	637	1230
×324	374	50.0	486	14.9	446	19.9	534	1090
×297	329	46.5	416	13.3	381	17.7	459	999
×277	286	41.5	343	9.88	317	13.2	375	890
×249	244	37.5	280	8.17	258	10.9	306	798
×215	195	32.5	209	6.26	193	8.34	229	684
×199	183	32.5	196	7.19	177	9.58	219	679
W40×392	657	71.0	970	29.6	888	39.5	907	1590
×331	505	61.0	710	22.6	649	30.2	713	1340
×327	488	59.0	676	20.5	620	27.3	690	1300
×278	381	51.0	501	15.8	458	21.1	552	1110
×264	349	48.0	447	13.9	410	18.5	492	1040
×235	286	41.5	343	9.88	317	13.2	375	890
×211	244	37.5	280	8.17	258	10.9	306	798
×183	195	32.5	209	6.26	193	8.34	229	684
×167	180	32.5	191	7.56	172	10.1	216	677
×149	158	31.5	165	8.55	143	11.4	192	650
W36×798	1550	119	2750	81.1	2520	108	2040	2700
×650	1110	98.5	1880	57.8	1720	77.1	1510	2150
×527	777	80.5	1260	39.6	1160	52.9	1100	1700
×439	576	68.0	895	29.2	820	38.9	848	1410
×393	480	61.0	722	23.7	662	31.6	722	1250
×359	414	56.0	607	20.3	557	27.0	637	1130
×328	357	51.0	506	16.8	465	22.3	561	1020
×300	311	47.3	430	14.8	394	19.8	478	936
×280	279	44.3	377	13.1	345	17.5	419	872
×260	251	42.0	334	12.3	304	16.4	374	823
×245	230	40.0	300	11.4	273	15.2	337	780
×230	210	38.0	269	10.5	243	14.0	303	737
W36×256	298	48.0	447	14.8	410	19.8	471	969
×232	259	43.5	367	12.3	337	16.3	407	871
×210	219	41.5	319	12.4	288	16.6	359	822
×194	192	38.3	271	10.5	246	14.1	306	754
×182	175	36.3	242	9.64	219	12.9	274	711
×170	157	34.0	212	8.56	192	11.4	240	665
×160	143	32.5	191	8.11	172	10.8	218	632
×150	132	31.3	173	7.84	154	10.5	199	606
×135	116	30.0	149	8.32	129	11.1	176	577
W33×387	483	63.0	771	26.4	708	35.2	707	1220
×354	418	58.0	652	22.7	599	30.3	614	1110
×318	349	52.0	527	18.3	484	24.4	527	988
×291	302	48.0	447	15.9	410	21.2	471	902
×263	257	43.5	367	13.2	337	17.6	406	810
×241	227	41.5	323	12.9	294	17.3	365	766
×221	200	38.8	278	11.7	251	15.6	316	709
×201	174	35.8	234	10.2	211	13.6	267	651

$F_y = 50$ ksi
$\phi = 1.00$
$\phi_r = 0.75$ Rev.
$\phi_v = 0.90$ 11/1/02

Table 9-5 (cont.). Beam End Bearing Constants

Shape	ϕR_1 kips	ϕR_2 kips/in.	$\phi_r R_3$ kips	$\phi_r R_4$ kips/in.	$\phi_r R_5$ kips	$\phi_r R_6$ kips/in.	$\phi R(N = 3^1/_4)$ kips	$\phi_v V_n$ kips
W33×169	161	33.5	219	7.90	201	10.5	245	611
×152	140	31.8	188	7.81	171	10.4	214	574
×141	125	30.3	167	7.51	150	10.0	191	544
×130	113	29.0	148	7.47	131	9.96	172	518
×118	99.2	27.5	127	7.41	111	9.87	151	489
W30×391	549	68.0	895	33.7	820	44.9	795	1220
×357	469	62.0	747	28.1	685	37.5	686	1100
×326	405	57.0	630	24.2	577	32.2	604	997
×292	337	51.0	506	19.4	465	25.9	516	881
×261	284	46.5	416	16.7	381	22.3	449	793
×235	237	41.5	335	13.2	307	17.6	377	701
×211	203	38.8	283	12.4	258	16.5	323	647
×191	175	35.5	236	10.6	214	14.2	270	589
×173	152	32.8	198	9.36	179	12.5	229	538
W30×148	149	32.5	206	8.22	189	11.0	232	539
×132	127	30.8	174	8.32	157	11.1	201	503
×124	116	29.3	156	7.73	140	10.3	181	477
×116	106	28.3	141	7.67	126	10.2	166	458
×108	96.1	27.3	127	7.75	111	10.3	152	439
×99	86.2	26.0	111	7.66	95.7	10.2	136	417
×90	74.0	23.5	90.9	6.25	78.6	8.34	111	374
W27×539	1060	98.5	1880	72.0	1720	96.0	1410	1730
×368	564	69.0	922	37.8	846	50.4	806	1130
×336	483	63.0	771	31.7	708	42.3	707	1020
×307	418	58.0	652	27.3	599	36.5	623	927
×281	361	53.0	548	22.8	503	30.4	545	839
×258	313	49.0	466	19.9	428	26.5	488	767
×235	273	45.5	398	17.7	364	23.6	432	705
×217	237	41.5	335	14.5	307	19.4	381	636
×194	200	37.5	272	12.1	249	16.2	311	569
×178	187	36.3	243	12.5	220	16.6	284	544
×161	154	33.0	201	10.5	182	13.9	235	492
×146	133	30.3	168	8.98	151	12.0	197	448
W27×129	130	30.5	181	8.10	166	10.8	207	455
×114	109	28.5	150	7.91	136	10.5	176	420
×102	92.3	25.8	122	6.58	110	8.77	143	377
×94	82.0	24.5	107	6.36	95.5	8.48	128	356
×84	71.3	23.0	90.2	6.17	79.2	8.23	110	332
W24×370	612	76.0	1120	50.0	1020	66.6	936	1150
×335	514	69.0	922	41.8	846	55.7	806	1020
×306	438	63.0	771	35.1	708	46.8	707	922
×279	376	58.0	652	30.3	599	40.4	623	836
×250	311	52.0	527	24.5	484	32.7	535	739
×229	268	48.0	447	21.3	410	28.4	471	674
×207	225	43.5	367	17.7	337	23.6	413	604
×192	198	40.5	318	15.5	292	20.6	368	558
×176	173	37.5	272	13.5	249	18.1	316	510
×162	151	35.3	236	12.5	215	16.6	277	476
×146	130	32.5	198	11.1	179	14.7	234	433
×131	111	30.3	167	10.2	150	13.6	200	400
×117	92.6	27.5	136	8.73	122	11.6	164	361
×104	78.5	25.0	111	7.49	98.6	9.99	135	325

$F_y = 50$ ksi
$\phi = 1.00$
$\phi_r = 0.75$ Rev.
$\phi_v = 0.90$ 11/1/02

Table 9-5 (cont.). Beam End Bearing Constants

Shape	ϕR_1 kips	ϕR_2 kips/in.	$\phi_r R_3$ kips	$\phi_r R_4$ kips/in.	$\phi_r R_5$ kips	$\phi_r R_6$ kips/in.	$\phi R(N = 3^1/_4)$ kips	$\phi_v V_n$ kips
W24×103	102	27.5	146	7.51	134	10.0	170	364
×94	89.1	25.8	125	6.96	114	9.28	148	338
×84	74.6	23.5	102	6.06	92.4	8.08	122	306
×76	64.9	22.0	86.9	5.68	77.9	7.57	105	284
×68	56.5	20.8	73.9	5.59	65.0	7.45	92.0	266
W24×62	63.9	21.5	78.2	6.16	68.5	8.22	98.3	275
×55	54.8	19.8	63.7	5.60	54.9	7.47	81.9	252
W21×201	242	45.5	400	21.8	367	29.0	432	565
×182	205	41.5	332	18.4	304	24.6	381	509
×166	174	37.5	274	14.9	251	19.9	322	456
×147	149	36.0	237	15.9	213	21.2	288	430
×132	125	32.5	192	13.3	173	17.7	235	383
×122	110	30.0	165	11.2	148	15.0	201	352
×111	94.6	27.5	138	9.58	124	12.8	169	319
×101	80.9	25.0	114	7.91	103	10.6	140	289
W21×93	104	29.0	154	10.5	139	14.0	188	338
×83	86.3	25.8	122	8.28	110	11.0	149	298
×73	70.6	22.8	95.4	6.51	86.2	8.68	117	260
×68	64.0	21.5	84.3	5.96	75.9	7.95	104	245
×62	56.0	20.0	71.7	5.37	64.2	7.16	89.1	227
×55	47.8	18.8	59.9	5.26	52.6	7.02	77.0	211
×48	40.7	17.5	49.1	5.25	41.8	6.99	66.1	195
W21×57	58.2	20.3	75.1	5.25	67.7	7.00	92.1	231
×50	49.4	19.0	61.9	5.34	54.5	7.13	79.3	213
×44	41.5	17.5	50.2	4.99	43.3	6.65	66.4	196
W18×175	221	44.5	382	24.0	350	32.0	416	481
×158	181	40.5	316	20.3	289	27.1	366	431
×143	157	36.5	259	16.4	238	21.8	312	384
×130	134	33.5	217	14.1	199	18.8	263	349
×119	120	32.8	197	15.1	178	20.2	246	336
×106	99.1	29.5	159	12.7	143	16.9	200	298
×97	84.9	26.8	132	10.3	119	13.7	165	269
×86	70.2	24.0	105	8.46	95.0	11.3	133	238
×76	57.4	21.3	82.5	6.72	74.4	8.96	104	209
W18×71	74.9	24.8	113	8.77	102	11.7	142	247
×65	64.7	22.5	94.4	7.16	85.7	9.54	118	224
×60	57.0	20.8	80.5	6.12	73.1	8.16	100	204
×55	50.2	19.5	69.8	5.64	63.0	7.52	88.2	191
×50	43.2	17.8	57.7	4.73	52.0	6.30	73.0	173
W18×46	45.5	18.0	60.7	4.62	55.1	6.16	75.7	176
×40	36.5	15.8	46.3	3.60	42.0	4.81	58.0	152
×35	31.0	15.0	38.7	3.89	34.1	5.19	51.3	143
W16×100	123	29.3	160	13.0	146	17.3	203	269
×89	104	26.3	129	10.7	117	14.2	163	238
×77	83.6	22.8	96.7	8.14	87.7	10.9	123	203
×67	67.6	19.8	73.1	6.16	66.4	8.22	93.2	174
W16×57	60.2	21.5	86.1	7.35	78.1	9.80	110	190
×50	48.9	19.0	67.2	5.79	60.9	7.72	86.0	167
×45	41.7	17.3	55.0	4.89	49.8	6.52	71.0	150
×40	34.6	15.3	43.2	3.81	39.2	5.07	55.7	132
×36	30.7	14.8	38.0	4.07	33.6	5.43	51.3	127

$F_y = 50$ ksi
$\phi = 1.00$
$\phi_r = 0.75$
$\phi_v = 0.90$

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Table 9-5 (cont.). Beam End Bearing Constants

Shape	ϕR_1 kips	ϕR_2 kips/in.	$\phi_r R_3$ kips	$\phi_r R_4$ kips/in.	$\phi_r R_5$ kips	$\phi_r R_6$ kips/in.	$\phi R(N = 3^1/_4)$ kips	$\phi_v V_n$ kips
W16×31	29.0	13.8	34.6	3.22	31.1	4.30	45.1	118
×26	23.3	12.5	26.5	3.13	23.3	4.17	36.8	106
W14×808	2680	187	5910	486	5170	648	3620	2300
×730	2110	154	4310	285	3880	380	2870	1860
×665	1820	142	3660	252	3290	335	2520	1650
×605	1550	130	3090	219	2780	292	2190	1470
×550	1310	119	2590	189	2340	252	1910	1300
×500	1130	110	2190	166	1970	221	1670	1160
×455	965	101	1860	146	1670	195	1460	1040
×426	850	94.0	1620	127	1470	169	1320	949
×398	761	88.5	1440	115	1300	154	1200	875
×370	676	83.0	1260	104	1140	139	1090	802
×342	591	77.0	1090	91.6	978	122	972	728
×311	504	70.5	909	78.6	820	105	857	651
×283	430	64.5	762	67.3	687	89.7	754	582
×257	367	59.0	637	57.4	574	76.6	662	523
×233	310	53.5	524	48.2	473	64.3	575	462
×211	264	49.0	438	41.6	394	55.5	511	415
×193	227	44.5	364	34.2	329	45.6	451	372
×176	198	41.5	313	31.1	281	41.5	407	341
×159	167	37.3	253	25.1	228	33.5	337	302
×145	144	34.0	211	21.1	191	28.2	282	272
W14×132	131	32.3	190	19.2	171	25.6	254	256
×120	114	29.5	159	16.3	143	21.8	214	231
×109	96.1	26.3	127	12.8	115	17.0	171	203
×99	83.6	24.3	108	11.2	97.2	14.9	146	186
×90	72.1	22.0	88.8	9.29	80.2	12.4	120	166
W14×82	92.8	25.5	122	11.8	110	15.7	161	197
×74	77.6	22.5	96.6	8.86	88.2	11.8	127	173
×68	68.0	20.8	81.9	7.68	74.8	10.2	108	157
×61	58.1	18.8	66.6	6.37	60.7	8.50	88.3	141
W14×53	57.8	18.5	66.1	5.98	60.5	7.98	86.4	139
×48	50.6	17.0	55.2	5.19	50.5	6.92	73.0	127
×43	42.7	15.3	44.3	4.23	40.4	5.65	58.8	113
W14×38	30.2	15.5	44.7	4.45	40.6	5.93	59.8	118
×34	25.6	14.3	37.1	3.94	33.4	5.25	50.5	108
×30	22.0	13.5	31.4	4.01	27.8	5.35	45.2	101
W14×26	26.2	12.8	30.1	3.08	27.3	4.10	40.6	95.7
×22	21.1	11.5	23.1	2.87	20.4	3.83	32.9	85.1

$F_y = 50$ ksi
$\phi = 1.00$
$\phi_r = 0.75$
$\phi_v = 0.90$
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Table 9-5 (cont.). Beam End Bearing Constants

Shape	ϕR_1 kips	ϕR_2 kips/in.	$\phi_r R_3$ kips	$\phi_r R_4$ kips/in.	$\phi_r R_5$ kips	$\phi_r R_6$ kips/in.	$\phi R(N = 3^1/_4)$ kips	$\phi_v V_n$ kips
W12×336	790	89.0	1480	123	1340	164	1150	807
×305	673	81.5	1240	106	1120	142	1000	717
×279	587	76.5	1070	98.8	970	132	894	657
×252	499	70.0	898	85.8	809	114	774	582
×230	431	64.5	762	74.4	687	99.2	683	526
×210	369	59.0	638	63.8	576	85.0	607	468
×190	309	53.0	520	51.5	471	68.7	520	412
×170	260	48.0	424	43.9	383	58.5	449	363
×152	217	43.5	347	37.2	313	49.6	393	322
×136	183	39.5	284	31.9	255	42.5	338	286
×120	151	35.5	228	26.7	204	35.6	293	251
×106	121	30.5	171	19.3	155	25.7	238	212
×96	103	27.5	140	15.8	126	21.0	195	189
×87	91.1	25.8	120	14.6	108	19.5	171	174
×79	78.2	23.5	99.8	12.3	89.6	16.5	143	157
×72	68.3	21.5	83.4	10.5	74.8	13.9	120	143
×65	58.5	19.5	68.4	8.78	61.4	11.7	99.4	127
W12×58	55.8	18.0	62.4	6.48	57.2	8.63	85.2	119
×53	50.5	17.3	55.5	6.40	50.3	8.53	78.1	113
W12×50	52.7	18.5	65.0	7.03	59.3	9.37	89.8	122
×45	45.2	16.8	53.1	5.86	48.4	7.81	73.8	109
×40	37.6	14.8	41.5	4.54	37.9	6.05	57.6	94.8
W12×35	30.7	15.0	42.8	4.50	39.1	6.00	58.6	101
×30	24.1	13.0	31.8	3.52	28.9	4.69	44.1	86.3
×26	19.5	11.5	24.6	2.84	22.3	3.79	34.6	75.8
W12×22	25.1	13.0	31.2	3.64	28.2	4.86	44.0	86.3
×19	20.4	11.8	24.3	3.29	21.7	4.39	35.9	77.4
×16	16.8	11.0	19.2	3.63	16.3	4.84	32.0	71.3
×14	14.2	10.0	15.3	3.24	12.8	4.32	26.8	64.3
W10×112	165	37.8	265	32.7	240	43.6	306	232
×100	138	34.0	214	27.4	194	36.5	265	204
×88	113	30.3	169	22.4	153	29.9	226	176
×77	91.2	26.5	130	17.5	118	23.3	190	152
×68	74.7	23.5	102	14.1	92.4	18.7	153	132
×60	62.0	21.0	81.1	11.6	73.2	15.4	123	116
×54	51.8	18.5	63.8	8.84	57.8	11.8	96.1	101
×49	45.0	17.0	53.6	7.62	48.5	10.2	81.6	91.6
W10×45	49.0	17.5	58.9	7.42	53.9	9.89	86.1	95.4
×39	40.6	15.8	46.5	6.44	42.2	8.59	70.2	84.4
×33	33.9	14.5	37.2	6.24	33.2	8.33	60.2	76.2
W10×30	30.4	15.0	42.4	5.46	38.6	7.29	62.2	85.0
×26	24.0	13.0	31.8	4.20	28.9	5.60	47.1	72.3
×22	19.8	12.0	25.5	4.08	22.7	5.44	40.4	66.1
W10×19	21.7	12.5	28.4	4.20	25.5	5.60	43.7	68.8
×17	18.9	12.0	24.4	4.49	21.4	5.99	40.9	65.4
×15	16.4	11.5	20.7	4.89	17.4	6.52	38.6	62.0
×12	12.1	9.50	13.7	3.59	11.4	4.78	26.9	50.6

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End Plate	Beam	1-in.-Diameter Bolts
$F_y = 36$ ksi	$F_y = 50$ ksi	12 Rows
$F_u = 58$ ksi	$F_u = 65$ ksi	W44

Table 10-4 (cont.). Bolted/Welded Shear End-Plate Connections

Bolt and End-Plate Design Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness $1/4$	End-Plate Thickness $5/16$	End-Plate Thickness $3/8$
A325/ F1852	N	–	287	359	431
	X	–	287	359	431
	SC Class A	STD	287	359	431
		OVS	258	322	387
		SSLT	287	359	388
	SC Class B	STD	287	359	431
		OVS	258	322	387
		SSLT	287	359	431
A490	N	–	287	359	431
	X	–	287	359	431
	SC Class A	STD	287	359	431
		OVS	258	322	387
		SSLT	287	359	431
	SC Class B	STD	287	359	431
		OVS	258	322	387
		SSLT	287	359	431

Weld (70 ksi) and Beam Web Design Strength, kips

70 ksi Weld Size, in.	ϕR_n, kips	Minimum Beam Web Thickness, in.
$3/16$	293	0.286
$1/4$	390	0.381
$5/16$	486	0.476
$3/8$	581	0.571

Support Design Strength per Inch Thickness, kips/in.

Hole Type	kips/in.
STD/SSLT	2730
OVS	2490

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes oriented transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

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End Plate	Beam	1-in.-Diameter Bolts
$F_y = 36$ ksi	$F_y = 50$ ksi	10 Rows
$F_u = 58$ ksi	$F_u = 65$ ksi	W44,40, 36

Table 10-4 (cont.). Bolted/Welded Shear End-Plate Connections

Bolt and End-Plate Design Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness 1/4	5/16	3/8
A325/ F1852	N	–	238	298	357
	X	–	238	298	357
	SC Class A	STD	238	298	357
		OVS	214	267	321
		SSLT	238	298	323
	SC Class B	STD	238	298	357
		OVS	214	267	321
		SSLT	238	298	357
A490	N	–	238	298	357
	X	–	238	298	357
	SC Class A	STD	238	298	357
		OVS	214	267	321
		SSLT	238	298	357
	SC Class B	STD	238	298	357
		OVS	214	267	321
		SSLT	238	298	357

Weld (70 ksi) and Beam Web Design Strength, kips

70 ksi Weld Size, in.	ϕR_n, kips	Minimum Beam Web Thickness, in.
3/16	243	0.286
1/4	323	0.381
5/16	402	0.476
3/8	480	0.571

Support Design Strength per Inch Thickness, kips/in.

STD/SSLT	2270
OVS	2080

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes oriented transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

W16×50, ASTM A992

$t_w = 0.380$ in. $d = 16.3$ in. $t_f = 0.630$ in. $k = 1.03$ in.
$F_y = 50$ ksi, $F_u = 65$ ksi

W14×90, ASTM A992

$t_w = 0.440$ in.
$F_y = 50$ ksi, $F_u = 65$ ksi

Use $^7/_8$-in.-diameter A325-N bolts in standard holes. Assume ASTM A36 angle material with $F_y = 36$ ksi and $F_u = 58$ ksi.

Solution: *Design seat angle and bolts*

For local web yielding,

$$N_{\min} = \frac{R_u - \phi R_1}{\phi R_2} \geq k$$
$$\boxed{= \frac{55\text{ kips} - 48.9\text{ kips}}{19\text{ kips/in.}} \geq 1.03\text{ in.} \quad = 0.321\text{ in.} \geq 1.03\text{ in.}}$$
$$= 1.03\text{ in.}$$

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For web crippling,

When $\dfrac{N}{d} \leq 0.2$

$$N_{\min} = \frac{R_u - \phi_r R_3}{\phi_r R_4}$$
$$\boxed{= \frac{55\text{ kips} - 67.2\text{ kips}}{5.79\text{ kips/in.}}}$$

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which results in a negative quantity.

When $\dfrac{N}{d} > 0.2$

$$N_{\min} = \frac{R_u - \phi_r R_5}{\phi_r R_6}$$
$$\boxed{= \frac{55\text{ kips} - 60.9\text{ kips}}{7.72\text{ kips/in.}}}$$

which results in a negative quantity.

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Thus, $N_{\min} = 1.03$ in.

From Table 10-5, an 8-in. angle length with a $^3/_4$-in. thickness and a $3^1/_2$-in. minimum outstanding leg will provide

$\phi R_n = 117$ kips > 55 kips **o.k.**

Try L6×4×$^3/_4$, 8-in. long with $5^1/_2$-in. bolt gage.

For $^7/_8$-in.-diameter A325-N bolts, connection type B (four bolts) provides

$\phi R_n = 86.6$ kips > 55 kips **o.k.**

The table indicates a 6×4×$^3/_4$ is available (4-in. OSL) connect the seat and top angles to the column flange. Assume ASTM A36 angle material with $F_y = 36\,\text{ksi}$ and $F_u = 58\,\text{ksi}$.

Solution: *Design seat angle and welds*

For local web yielding,

$$N_{\min} = \frac{R_u - \phi R_1}{\phi R_2} \geq k$$
$$\boxed{\begin{aligned} &= \frac{55\,\text{kips} - 56\,\text{kips}}{20.0\,\text{kips/in.}} \geq 1.12\,\text{in.} \\ &= \text{-}0.05\,\text{in.} \geq 1.12\,\text{in.} \end{aligned}}$$
$$= 1.12\,\text{in.}$$

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For web crippling,

When $\dfrac{N}{d} \leq 0.2$

$$N_{\min} = \frac{R_u - \phi_r R_3}{\phi_r R_4}$$
$$\boxed{= \frac{55\,\text{kips} - 71.7\,\text{kips}}{5.37\,\text{kips/in.}}}$$

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which results in a negative quantity.

When $\dfrac{N}{d} > 0.2$

$$N_{\min} = \frac{R_u - \phi_r R_5}{\phi_r R_6}$$
$$\boxed{= \frac{55\,\text{kips} - 64.2\,\text{kips}}{7.02\,\text{kips/in.}}}$$

which results in a negative quantity.

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Thus, $N_{\min} = 1.12\,\text{in.}$

From Table 10-5, an 8-in.-angle length with a $^3/_4$-in. thickness and a $3^1/_2$-in. minimum outstanding leg will provide

$$\phi R_n = 117\,\text{kips} > 55\,\text{kips} \qquad \textbf{o.k.}$$

Tabular values include checks of local yielding and web crippling strengths of beam web. **o.k.**

Try L8×4×$^3/_4$, 8 in. long with $^5/_{16}$-in. fillet welds.

For weld strength, from Table 10-6,

$$\phi R_n = 66.8\,\text{kips} > 55\,\text{kips} \qquad \textbf{o.k.}$$

Use two $^3/_4$-in.-diameter A325-N bolts to connect the beam to the seat angle.

Select top angle, bolts, and welds

For local web yielding, use constants ϕR_1 and ϕR_2 from Table 9-5. Rev. 11/1/02

$$W_{\min} = \frac{R_u - \phi R_1}{\phi R_2} + \text{ setback}$$

$$\boxed{\begin{aligned} &= \frac{125\,\text{kips} - 64\,\text{kips}}{21.5\,\text{kips/in.}} + {}^3\!/_4\text{-in.} \\ &= 3.59\,\text{in.} \end{aligned}}$$

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The minimum stiffener width W for web crippling controls. Thus, use $W = 7$ in.

Check assumption

$$\begin{aligned} \frac{N}{d} &= \frac{6.71\,\text{in.} - {}^1\!/_2\,\text{in.}}{21.1\,\text{in.}} \\ &= 0.294 > 0.2 \qquad \textbf{o.k.} \end{aligned}$$

Determine stiffener length L and stiffener to column flange weld size

From Table 10-8, a stiffener with $L = 15$ in. and ${}^5\!/_{16}$-in. weld size provides

$$\phi R_n = 139\,\text{kips} > 125\,\text{kips} \qquad \textbf{o.k.}$$

Determine weld requirements for seat plate

Using ${}^5\!/_{16}$-in. fillet welds the minimum length of seat-plate-to-column-flange weld on each side of the stiffener is $0.2(L) = 3$ in. Use three inches of weld on each side of the stiffener. This also establishes the minimum weld between the seat plate and stiffener; use three inches of ${}^5\!/_{16}$-in. weld on both sides of the stiffener.

Determine seat plate dimensions

To accommodate two ${}^3\!/_4$ in. diameter A325-N bolts on a $5{}^1\!/_2$-in. gage connecting the beam flange to the seat plate, a width of eight inches is adequate. This is greater than the width required to accommodate the seat-plate-to-column-flange welds.

Use PL ${}^3\!/_8$ in.$\times$7 in.$\times$8 in. for the seat plate.

Determine stiffener plate thickness

To develop the stiffener-to-seat-plate welds, the minimum stiffener thickness is

$$\begin{aligned} t_{\min} &= 2({}^5\!/_{16}\,\text{in.}) \\ &= {}^5\!/_8\,\text{in.} \end{aligned}$$

For a stiffener with $F_y = 36$ ksi and beam with $F_y = 50$ ksi, the minimum stiffener thickness is

$$\begin{aligned} t_{\min} &= 1.4 t_w \\ &= 1.4(0.430\,\text{in.}) \\ &= 0.602\,\text{in.} \end{aligned}$$

connections. For a rigid support with standard holes:

$$e_b = |(n - 1) - a|$$

For a rigid support with short-slotted holes

$$e_b = \left|\frac{2n}{3} - a\right|$$

When the support condition is intermediate between flexible and rigid or cannot be readily classified as flexible or rigid, the larger value of e_b may conservatively be taken from the above equations.

For any combination of support condition and hole type, the 70 ksi electrode weld size should be equal to three-quarters of the plate thickness t_p for plate material with $F_y = 36$ ksi and $F_u = 58$ ksi. This weld size ensures that the plate will yield prior to weld fracture.

The foregoing procedure is valid for single-plate connections with $2^1/_2$ in. $\leq a \leq 3\ ^1/_2$ in. Single-plate connections with geometries and configurations other than those described above can be used based upon rational analysis.

Recommended Plate Length and Thickness

To provide for stability during erection, it is recommended that the minimum plate length be one-half the T-dimension of the beam to be supported. The maximum length of the plate must be compatible with the T-dimension of an uncoped beam and the remaining web depth, exclusive of fillets, of a coped beam. Note that the plate may encroach upon the fillet(s) as given in Figure 10-4.

To prevent local buckling of the plate, the minimum plate thickness should be such that,

$$\boxed{t_{p\,\min} = \frac{L}{234}\sqrt{\frac{Fy}{K}} \geq {}^1/_4 \text{ in.}}$$

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where K is the plate buckling coefficient tabulated in Part 9 for local buckling of beams coped at both the top and bottom flanges. To use the table in Part 9, calculate the plate aspect ratio as $\boxed{2a/L.}$ Rev. 11/1/02

To provide for rotational ductility in the single plate, the maximum plate thickness should be such that

$$t_{p\,\max} = \frac{d_b}{2} + {}^1/_{16} \text{ in.} \geq t_{p\,\min}$$

where d_b is the bolt diameter, in. This ensures that bearing deformations will occur in the bolt holes prior to bolt shear.

Shop and Field Practices

Single-plate connections may be made to the webs of supporting girders and to the flanges of supporting columns. Because of bolting clearances, field-bolted single-plate connections may not be suitable for connections to the webs of supporting columns unless provision is made to extend the plate to locate the bolt line a sufficient distance beyond the column flanges. Such extension may require stiffening of the plate and the column web.

With the plate shop attached to the support, side erection of the beam is permitted. Play in the open holes usually compensates for mill variation in column flange supports and other field adjustments. Thus, slotted holes are not normally required.

SINGLE-ANGLE CONNECTIONS

A single-angle connection is made with an angle on one side of the web of the beam to be supported, as illustrated in Figure 10-17. This angle is preferably shop bolted or welded to the supporting member and field bolted to the supported beam.

When the angle is welded to the support, adequate flexibility must be provided in the connection. As illustrated in Figure 10-17c, the weld is placed along the toe and across the bottom of the angle with a return at the top per LRFD Specification Section J2.2b. Note that welding across the entire top of the angle must be avoided as it would inhibit the flexibility and, therefore, the necessary end rotation of the connection. The performance of the resulting connection would not be as intended for single-angle connections.

Design Checks

The design strength of a single-angle connection is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8) and connection elements (see Part 9). In all cases, the design strength ϕR_n must equal or exceed the required strength R_u. Rev. 11/1/02

As illustrated in Figure 10-18, the effect of eccentricity should always be considered in the angle leg attached to the support. Additionally, eccentricity should be considered in the case of a double vertical row of bolts through the web of the supported beam or if the eccentricity exceeds 3 in. ($2^3/_4$-in gage plus $^1/_4$-in. half web). Eccentricity should always be considered in the design of welds for single-angle connections.

Recommended Angle Length and Thickness

To provide for stability during erection, it is recommended that the mimimum angle length be one-half the T-dimension of the beam to be supported. The maximum length of the connection angles must be compatible with the T-dimension of an uncoped beam and the remaining web depth, exclusive of fillets, of a coped beam. Note that the angle may encroach upon the fillet(s) as given in Figure 10-4.

A minimum angle thickness of $^3/_8$ in. for $^3/_4$-in. and $^7/_8$-in.-diameter bolts, and $^1/_2$ in. for 1-in.-diameter bolts should be used. A 4×3 angle is normally selected for a single angle welded to the support with the 3-in. leg being the welded leg.

Shop and Field Practices

Single-angle connections may be readily made to the webs of supporting girders and to the flanges of supporting columns. When framing to a column flange, provision must be made for possible mill variation in the depth of the columns. Since the angle is usually shop attached to the column flange, play in the open holes or horizontal slots in the angle leg may be used to provide the necessary adjustment to compensate for the mill variation. Attaching the angle to the column flange offers the advantage of side erection of the beam. The same is true for a girder web or truss support. Additionally, proper bay dimensions may be maintained without the need for shims. This advantage is lost in the case that the angle is shop attached to the supported beam web.

EXAMPLE 10.13. All-bolted single-angle connection (beam-to-girder web).

Given: Design an all-bolted single-angle connection (case I) for a W18×35 beam to W21×62 girder-web connection.

$$R_u = 40\,\text{kips}$$

The angle of skew A appears in Figure 10-38a and represents the horizontal bevel to which the fittings must be bent or set, or the direction of gage lines on a seated connection.

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When the skew angle is less than 5° (1-in-12 slope), a pair of double angles can be bent inward or outward to make the connection as shown in Figure 10-39. While bent angle sections are usually drawn as bending in a straight line from the heel, rolled angles will tend to bend about the root of the fillet (dimension *k* in Manual Part 1). This produces a significant jog in the leg alignment, which is magnified by the amount of bend. Above this angle of skew, it becomes impractical to bend rolled angles.

For skews approximately greater than 5° (1-in.-12 slope), a pair of bent plates, shown in Figure 10-40, may be a more practical solution. Bent plates are not subject to the deformation problem described for bent angles, but the radius and direction of the bend must be considered to avoid cracking during the cold-bending operation.

Bent plates exhibit better ductility when bent perpendicular to the rolling direction and are, therefore, less likely to crack. Whenever possible, bent connection plates should be billed with the width dimension parallel to the bend line. The length of the plate is measured on its mid-thickness, without regard to the radius of the bend. While this will provide a plate that is slightly longer than necessary, this will be corrected when the bend is laid out to the proper radius prior to fabrication.

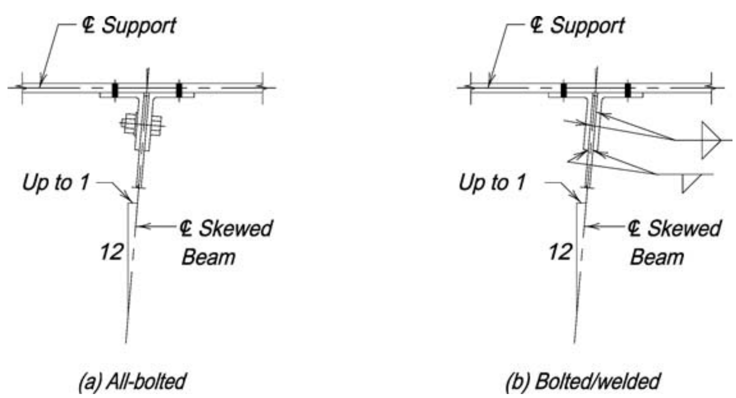

Fig. 10–39. Skewed beam connections with bent double angles.

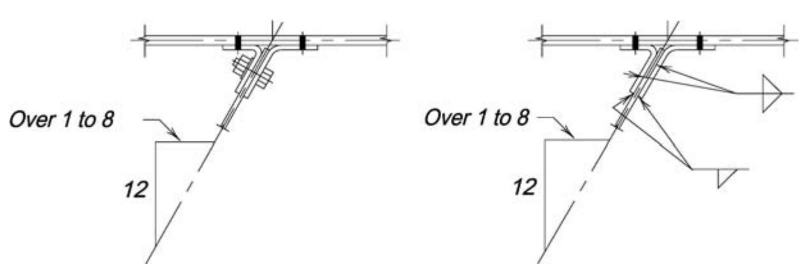

Fig. 10–40. Skewed beam connections with double bent plates.

Try a $\frac{5}{16}$-in. fillet weld. The minimum length of weld l_{min} is:

$$l_{min} = \frac{P_{uf}}{1.392D} = \frac{150 \text{ kips}}{1.392(5 \text{ sixteenths})} = 21.6 \text{ in.}$$

Use 8 in. of weld along each side and $6\frac{1}{4}$ in. of weld along the end of the flange plate.

Select tension flange plate dimensions

To provide for an 8-in. weld length and an unwelded length of $1\frac{1}{2}$ times the plate width, use PL $\frac{3}{4}$ in. × $6\frac{1}{4}$ in. × $17\frac{1}{2}$ in.

Determine required weld size for fillet welds to supporting column flange.

$$D_{min} = \frac{P_{uf}}{1.5 \times 2 \times 1.392\,l} = \frac{144 \text{ kips}}{1.5 \times 2 \times 1.392(6\frac{1}{4} \text{ in.})} = 5.52 \rightarrow 6 \text{ sixteenths}$$

Use $\frac{3}{8}$-in. fillet welds.

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Design the compression flange plate and connection

The compression flange plate should have approximately the same area as the tension flange plate (4.69 in.^{2}). Assume a shelf dimension of $\frac{5}{8}$ in. The plate width, then, is 7.495 in. + 2($\frac{5}{8}$ in.) = 8.745 in. To approximately balance the flange-plate areas, try a $\frac{5}{8}$ in. × $8\frac{3}{4}$ in. compression flange plate.

Check design compressive strength of flange plate

Assuming $K = 0.65$ and $l = \frac{3}{4}$ in. ($\frac{1}{2}$-in. setback plus $\frac{1}{4}$-in. tolerance).

$$\frac{Kl}{r} = \frac{0.65(\frac{3}{4} \text{ in.})}{\sqrt{\dfrac{(8\frac{3}{4} \text{ in.})(\frac{5}{8} \text{ in.})^3/12}{(8\frac{3}{4} \text{ in.})(\frac{5}{8} \text{ in.})}}} = 2.70$$

From LRFD Specification Table 3-36 with $\frac{Kl}{r} = 2.70$,

$$\phi_c F_{cr} = 30.59 \text{ ksi}$$

and the design compressive strength of the flange-plate is

$$\begin{aligned}\phi R_n &= \phi_c F_{cr} A \\ &= (30.59\text{ ksi})(8\tfrac{3}{4}\text{ in.} \times \tfrac{5}{8}\text{ in.}) \\ &= 167\text{ kips} > 144\text{ kips} \qquad \textbf{o.k.}\end{aligned}$$

Determine required weld size and length for fillet welds to beam flange

As before for the tension flange plate, with $^5/_{16}$-in. fillet welds, use 8 in. along each side and 6 $^1/_4$ in. along the end of the compression flange plate.

Select compression flange plate dimensions

Use PL $^5/_8$ in. × 8 $^3/_4$ in. × 1′-0.

Determine required weld size for fillet welds to supporting column flange

$$\boxed{\begin{aligned}D_{min} &= \frac{P_{uf}}{1.5 \times 2 \times 1.392\, l} \\ &= \frac{144\text{ kips}}{1.5 \times 2 \times 1.392(8\tfrac{3}{4}\text{ in.})} \\ &= 3.94 \rightarrow 4\text{ sixteenths}\end{aligned}}$$

Use $^1/_4$-in. fillet welds.

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Comment: The column section should be checked for stiffening requirements. A check of the applicable limit states from LRFD Specification Section K1 (refer to "Column Stiffening at Moment Connections in this Part) will show the W14×109 column in the above example is adequate without stiffening.

PART 11 REFERENCES

Ackroyd, M.H., 1987, "Simplified Frame Design of Type PR Construction," *Engineering Journal*, Vol. 24, No. 4, (4th Qtr.), pp. 141–146, AISC, Chicago, IL.

Blodgett, O.W., 1966, *Design of Welded Structures*, James F. Lincoln Arc Welding Foundation, Cleveland, OH.

Carter, C.J., 1999, AISC Design Guide No. 13 *Wide-Flange Column Stiffening at Moment Connections: Wind and Seismic Applications*, AISC, Chicago, IL.

Deierlein, G.G., S.H. Hsieh, and Y.J. Shen, 1990, "Computer-Aided Design of Steel Structures with Flexible Connections," *Proceedings of the 1990 National Steel Construction Conference*, pp. 9.1–9.21, AISC, Chicago, IL.

Disque, R.O., 1975, "Directional Moment Connections—A Proposed Design Method for Unbraced Steel Frames," *Engineering Journal*, Vol. 12, No. 1, (1st Qtr.), pp. 14–18, AISC, Chicago, IL.

Use $^3\!/_8$ in. fillet welds.

Design the compression flange plate and connection.

Check design compressive strength of flange plate assuming $K = 0.65$ and $l = 2$ in. ($1^1\!/_2$ in. edge distance plus $^1\!/_2$ in. setback)

$$\frac{Kl}{r} = \frac{0.65(2\text{ in.})}{\sqrt{\dfrac{(7^1\!/_4\text{ in.})(^3\!/_4\text{ in.})^3\!/_{12}}{(7\,^1\!/_4\text{ in.})(^3\!/_4\text{ in.})}}}$$
$$= 6.00$$

From LRFD Specification Table 3-36 with $\dfrac{Kl}{r} = 6.00$,

$$\phi F_{cr} = 30.54\text{ ksi}$$

and the design compressive strength of the flange plate is

$$\begin{aligned}\phi R_n &= \phi_c F_{cr} A \\ &= (30.54\text{ ksi})\left(7^1\!/_4\text{ in.} \times {}^3\!/_4\text{ in.}\right) \\ &= 167\text{ kips}\end{aligned}$$

Since the design strength equals the required strength, the flange plate is adequate.

The compression flange plate will be identical to the tension flange plate: a $^3\!/_4$ in.×7 $^1\!/_4$ in. plate with eight bolts in two rows of four bolts on a 4 in. gage and $^3\!/_8$ in. fillet welds to the supporting column flange.

Comment: The column must be checked for stiffening requirements. For further information, see AISC Design Guide No. 13 *Wide-Flange Column Stiffening at Moment Connections—Wind and Seismic Applications* (Carter, 1999).

EXAMPLE 12.2. Welded flange-plated FR moment connection (beam-to-column flange).

Given: Design a welded flange-plated FR moment connection for a W18×50 beam to W14×99 column flange connection. For structural members, $F_y = 50$ ksi; for connecting material $F_y = 36$ ksi. Use 70 ksi electrodes and ASTM A325-N bolts.

$$\begin{aligned}R_u &= 45.0\text{ kips} \\ M_u &= 250\ \boxed{\text{ft-kips}}\end{aligned}$$

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W18×50, ASTM A992

$d = 18.0$ in.	$b_f = 7.50$ in.	$Z_x = 101$ in.^{3}
$t_w = 0.355$ in.	$t_f = 0.570$ in.	

W14×99, ASTM A992

$d = 14.2$ in. $\quad b_f = 14.6$ in. $\quad k = 2\frac{1}{16}$ in.
$t_w = 0.485$ in. $\quad t_f = 0.780$ in. $\quad T = 10$ in.

Solution:

Check beam design flexural strength:

$$
\begin{aligned}
Z_{req} &= \frac{M_u \times 12 \text{ in./ft}}{0.9F_y} \\
&= \frac{(250 \text{ ft-kips})(12 \text{ in./ft})}{0.9(50 \text{ ksi})} \\
&= 66.7 \text{ in.}^3 \\
Z_x &= 101 \text{ in.}^3
\end{aligned}
$$

Since $Z_x > Z_{req}$, the beam design flexural strength is **o.k.**

Design the single-plate web connection.

From Example 12-1, a three-bolt, $\frac{5}{16}$ in.-thick single plate with two $\frac{5}{16}$ in. fillet welds will be adequate.

Design the tension flange plate and connection.

Calculate the flange force P_{uf}.

$$
\begin{aligned}
P_{uf} &= \frac{M_u \times 12 \text{ in./ft}}{d} \\
&= \frac{(250 \text{ ft-kips})(12 \text{ in./ft})}{18.0 \text{ in.}} \\
&= 167 \text{ kips}
\end{aligned}
$$

Determine tension flange-plate dimensions.

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From Figure 12-4, assume a shelf dimension of $\frac{5}{8}$ in. on both sides of the plate. The plate width, then, is 7.495 in.$-2(\frac{5}{8}$ in.$) = 6.245$. Try a 1 in.$\times 6\frac{1}{4}$-in. flange plate.

Check tension yielding of the flange plate:

$$
\begin{aligned}
\phi R_n &= \phi F_y A_g \\
&= 0.9 \times 36 \text{ ksi} \times 6\tfrac{1}{4} \text{ in.} \times 1 \text{ in.} \\
&= 202.5 \text{ kips} \quad \textbf{o.k.}
\end{aligned}
$$

Determine required weld size and length for fillet welds to beam flange. Try a $\frac{5}{16}$ in. fillet weld. The minimum length of weld l_{min} is:

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$$
\begin{aligned}
l_{min} &= \frac{P_{uf}}{2\text{x}1.392D} \\
&= \frac{167 \text{ kips}}{2\text{x}1.392(5 \text{ sixteenths})} \\
&= 12.0 \text{ in.}
\end{aligned}
$$

W14×99, ASTM A992

$d = 14.2$ in. $\quad b_f = 14.6$ in. $\quad k = 2^1/_{16}$ in.
$k_1 = 1^7/_{16}$ in. $\quad t_w = 0.485$ in. $\quad t_f = 0.780$ in.
$T = 10$ in.

Solution: *Check beam design flexural strength.*

From Example 12.2, the beam design flexural strength is **o.k.**

Design the bolts (a minimum of four bolts is required at the tension flange; a minimum of two bolts is required at the compression flange).

Calculate the flange force P_{uf}.

$$\begin{aligned} P_{uf} &= \frac{M_u \times 12 \text{ in./ft}}{(d - t_f)} \\ &= \frac{(250 \text{ ft-kips})(12 \text{ in./ft})}{18.0 \text{ in.} - 0.570 \text{ in.}} \\ &= 172 \text{ kips} \end{aligned}$$

Determine number of 1-in. diameter A325-SC bolts required for tension (Note that pretensioned bearing-type bolts would also be acceptable). From Table 7-14

$$\begin{aligned} n_{\min} &= \frac{P_{uf}}{\phi r_n} \\ &= \frac{172 \text{ kips}}{53.0 \text{ kips/bolt}} \\ &= 3.25 \rightarrow 4 \text{ bolts} \end{aligned}$$

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Determine number of 1-in. diameter A325-SC bolts required for slip resistance. From Table 7-15

$$\begin{aligned} n_{\min} &= \frac{R_u}{\phi r_n} \\ &= \frac{45 \text{ kips}}{19.0 \text{ kips/bolt}} \\ &= 2.37 \rightarrow 3 \text{ bolts} \end{aligned}$$

Minimum of four bolts at tension flange and two bolts at compression flange controls. Try six 1-in. diameter A325-SC bolts (N for bolt shear check).

Check bolt shear:

From Table 7-10 for six 1-in. diameter A325-N bolts:

$$\begin{aligned} \phi R_n &= 6 \times 28.3 \text{ kips/bolt} \\ &= 170 \text{ kips} > 45.0 \text{ kips} \end{aligned}$$ **o.k.**

Try $^3/_4$ in.-thick end plate.

Try a $3/4$ in.×$8\frac{1}{2}$ in. end plate.

Check shear yielding of the end plate.

From LRFD Specification Section J5.3:

$$\begin{aligned}\phi R_n &= 2 \times \phi(0.60 F_y A_g)\\ &= 2 \times 0.9\left(0.6 \times 36\text{ ksi} \times 8\tfrac{1}{2}\text{ in.} \times \tfrac{3}{4}\text{ in.}\right)\\ &= 248\text{ kips} > 172\text{ kips} \qquad \textbf{o.k.}\end{aligned}$$

Determine required fillet weld for beam-web-to-end-plate connection.

From LRFD Specification Table J2.4, the minimum size is $1/4$ in. Determine size required to develop web flexural strength near tension bolts:

$$\begin{aligned}D_{\min} &= \frac{0.9 F_y t_w}{2 \times 1.392}\\ &= \frac{0.9 \times 50\text{ ksi} \times 0.355\text{ in.}}{2 \times 1.392}\\ &= 5.74 \rightarrow 6\text{ sixteenths}\end{aligned}$$

Use $3/8$ in. fillet weld on both sides of the beam web from the inside face of the beam flange to the centerline of the inside bolt holes plus two bolt diameters.

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Determine size required for the factored shear R_u. R_u is resisted by weld between the mid-depth of the beam and the inside face of the compression flange or between the inner row of tension bolts plus two bolt diameters, whichever is smaller. By inspection the former governs for this example.

$$\begin{aligned}l &= \frac{d}{2} - t_f\\ &= \frac{18.0\text{ in.}}{2} - 0.570\text{ in.}\\ &= 8.43\text{ in.}\\ D_{\min} &= \frac{R_u}{2 \times 1.392\, l}\\ &= \frac{45\text{ kips}}{2 \times 1.392(8.43\text{ in.})}\\ &= 1.92 \rightarrow 5\text{ sixteenths (minimum size)}\end{aligned}$$

Use $1/4$ in. fillet weld on both sides of the beam web below the tension-bolt region.

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Determine required fillet weld size for beam flange to end-plate connection.

$$\begin{aligned}l &= 2(b_f + t_f) - t_w\\ &= 2(7.50\text{ in.} + 0.570\text{ in.}) - 0.355\text{ in.}\\ &= 15.8\text{ in.}\end{aligned}$$

When a full-length filler is provided, as in corrosive environments, the maximum spacing of stitch bolts should be as specified in LRFD Specification Section J3.5. Alternatively, the edges of the filler may be seal welded.

Force Transfer in Diagonal Bracing Connections

There has been some controversy as to which of several available analysis methods provides the best means for the safe and economical design and analysis of diagonal bracing connections. To resolve this situation, starting in 1981, AISC sponsored extensive computer studies of this connection by Richard (1986). Associated with Richard's work, full scale tests were performed by Bjorhovde and Chakrabarti (1985), Gross and Cheok (1988), and Gross (1990). Also, AISC and ASCE formed a task group to recommend a design method for this connection. In 1990, this task group recommended three methods for further study; refer to Appendix A of Thornton (1991).

Using the results of the aforementioned full scale tests, Thornton (1991) showed that these three methods yield safe designs, and that of the three methods, the Uniform Force Method (see Model 3 of Thornton, 1991) best predicts both the design strength and critical limit state of the connection. Furthermore, Thornton (1992) showed that the Uniform Force Method yields the most economical design through comparison of actual designs by the different methods and through consideration of the efficiency of force transmission. For the above reasons, and also because it is the most versatile method, the Uniform Force Method has been adopted for use in this book.

The Uniform Force Method

The essence of the Uniform Force Method is to select the geometry of the connection so that moments do not exist on the three connection interfaces; i.e., gusset-to-beam, gusset-to-column, and beam-to-column. In the absence of moment, these connections may then be designed for shear and/or tension only, hence the origin of the name Uniform Force Method.

With the control points as illustrated in Figure 13-2 and the working point chosen at the intersection of the centerlines of the beam, column, and diagonal brace as shown in Figure 13-2a, four geometric parameters e_b, e_c, α, and β can be identified,

where

e_b = one-half the depth of the beam, in.
e_c = one-half the depth of the column, in. Note that, for a column web support, $e_c \approx 0$
α = distance from the face of the column flange or web to the centroid of the gusset-to-beam connection, in.
β = distance from the face of the beam flange to the centroid of the gusset-to-column connection, in.

For the force distribution shown in the free-body diagrams of Figures 13-2b, 13-2c, and 13-2d to remain free of moments on the connection interfaces, the following expression must be satisfied.

$$\alpha - \beta \tan\theta = e_b \tan\theta - e_c \tag{13-1}$$

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Since the variables on the right of the equal sign (e_b, e_c, and θ) are all defined by the members being connected and the geometry of the structure, the designer may select values of α and β for which the equation is true, thereby locating the centroids of the gusset-to-beam and gusset-to-column connections.

If ΔV_{ub} is taken equal to V_{ub}, none of the vertical component of the brace force is transmitted to the beam; the resulting procedure is that presented by AISC (1984) for concentric gravity axes, extended to connections to column flanges. This method was also recommended by the AISC/ASCE task group.

Design by this method may be uneconomical. It is very punishing to the gusset and beam because of the moment M_{ub} induced on the gusset-to-beam connection. This moment will require a larger connection and a thicker gusset. Additionally, the limit state of local web yielding may limit the strength of the beam. This special case interrupts the natural flow of forces assumed in the Uniform Force Method and thus is best used when the beam-to-column interface is already highly loaded, independently of the brace, by a high shear R_u in the beam-to-column connection.

Special Case 3, No Gusset-to-Column Web Connection

When the connection is to a column web and the brace is shallow (as for large θ) or the beam is deep, it may be more economical to eliminate the gusset-to-column connection entirely and connect the gusset to the beam only. The Uniform Force Method can be applied to this situation by setting β and e_c equal to zero as illustrated in Figure 13-5. Since there is to be no gusset-to-column connection, V_{uc} and H_{uc} also equal zero. Thus, $V_{ub} = V_u$ and $H_{ub} = H_u$.

If $\bar{\alpha} = \alpha = e_b \tan\theta$, there is no moment on the gusset-to-beam interface and the gusset-to-beam connection can be designed for the factored shear force H_{ub} and the factored axial force V_{ub}. If $\bar{\alpha} \neq \alpha = e_B \tan\theta$, the gusset-to-beam interface must be designed for the moment M_{ub} in addition to H_{ub} and V_{ub},

where

$$M_{ub} = V_{ub}(\alpha - \bar{\alpha})$$

The beam-to-column connection must be designed for the factored shear force $R_u + V_{ub}$.

Note that, since the connection is to a column web, e_c is zero and hence H_c is also zero. For a connection to a column flange, if the gusset-to-column-flange connection is eliminated, the beam-to-column connection must be a moment connection designed for the moment $V_u e_c$ in addition to the shear V_u. Thus, uniform forces on all interfaces are no longer possible.

Analysis of Existing Diagonal Bracing Connections

A combination of α and β which provides for no moments on the three interfaces can usually be achieved when a connection is being designed. However, when analyzing an existing connection or when other constraints exist on gusset dimensions, the values of α and β may not
Rev. 11/1/02 satisfy Equation 13-1. When this happens, uniform interface forces will not satisfy equilibrium and moments will exist on one or both gusset edges or at the beam-to-column interface.

To illustrate this point, consider an existing design where the actual centroids of the gusset-to-beam and gusset-to-column connections are at $\bar{\alpha}$ and $\bar{\beta}$, respectively. If the connection at one edge of the gusset is more rigid than the other, it is logical to assume that the more rigid edge takes all of the moment necessary for equilibrium. For instance, the gusset of Figure 13-2 is shown welded to the beam and bolted with double angles to the column. For this configuration, the gusset-to-beam connection will be much more rigid than the gusset-to-column connection.

Take α and β as the ideal centroids of the gusset-to-beam and gusset-to-column connections, respectively. Setting $\beta = \bar{\beta}$, the α required for no moment on the gusset-to-beam connection may be calculated as:

$$\alpha = K + \bar{\beta}\tan\theta$$

where

$$K = e_b \tan\theta - e_c$$

If $\alpha \neq \bar{\alpha}$, a moment M_{ub} will exist on the gusset-to-beam connection, where

$$M_{ub} = V_{ub}(\alpha - \bar{\alpha})$$

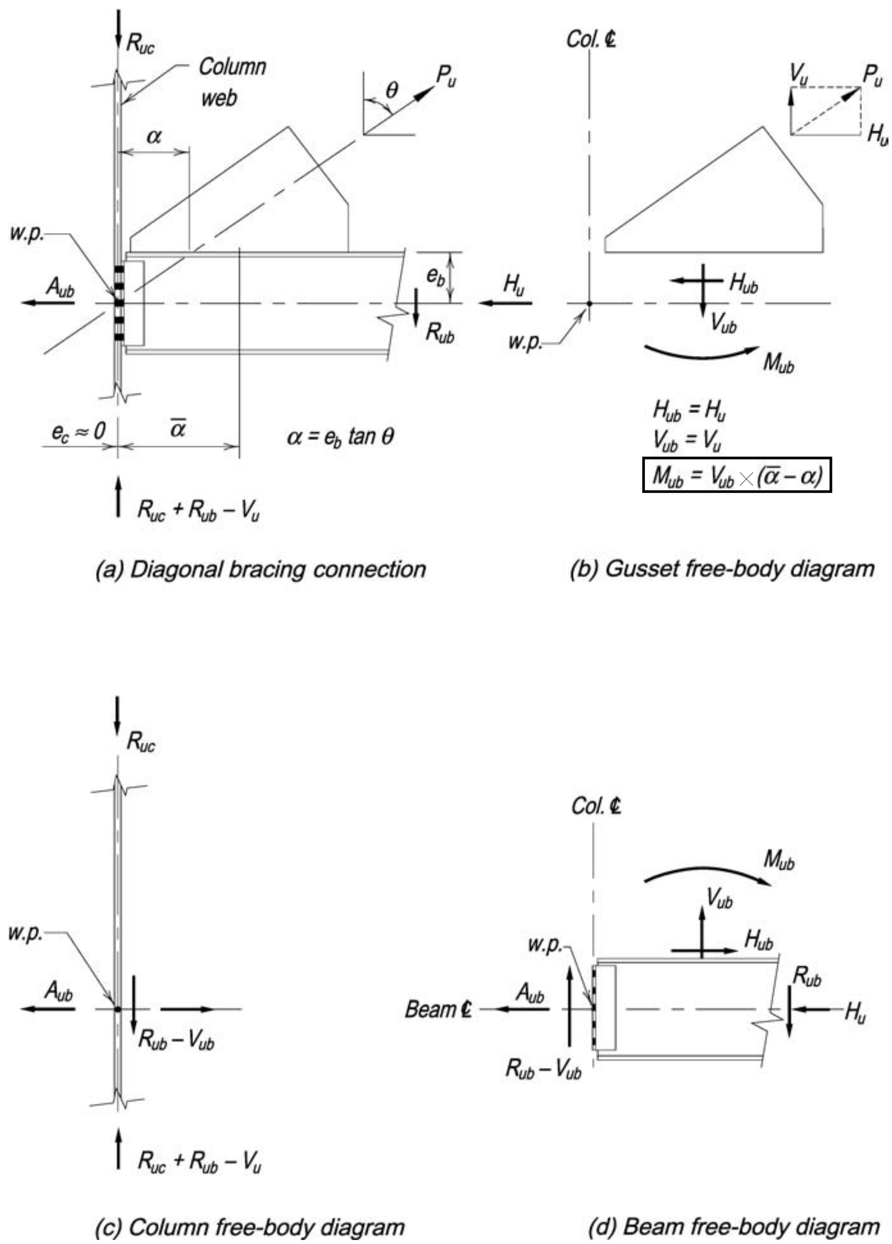

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Fig. 13–5. Force transfer, UF method special case 3.

Check tension yielding of the angles

$$\begin{aligned}\phi R_n &= \phi F_y A_g \\ &= 0.90(36 \text{ ksi})(10.9 \text{ in.}^2) \\ &= 353 \text{ kips} > 259 \text{ kips} \qquad \textbf{o.k.}\end{aligned}$$

Check tension rupture of the angles.

From LRFD Specification Sections B3.2,

$$\boxed{\begin{aligned}U &= 1 - \frac{\bar{x}}{l} \le 0.9 \\ &= 1 - \frac{1.27 \text{ in.}}{15 \text{ in.}} \le 0.9\end{aligned}}$$

$$\begin{aligned}&= 0.92 \rightarrow 0.9 \\ UA_n &= 0.9(10.9 \text{ in.}^2 - 2 \times 0.75 \text{ in.} \times 1 \text{ in.}) \\ &= 8.46 \text{ in.}^2\end{aligned}$$

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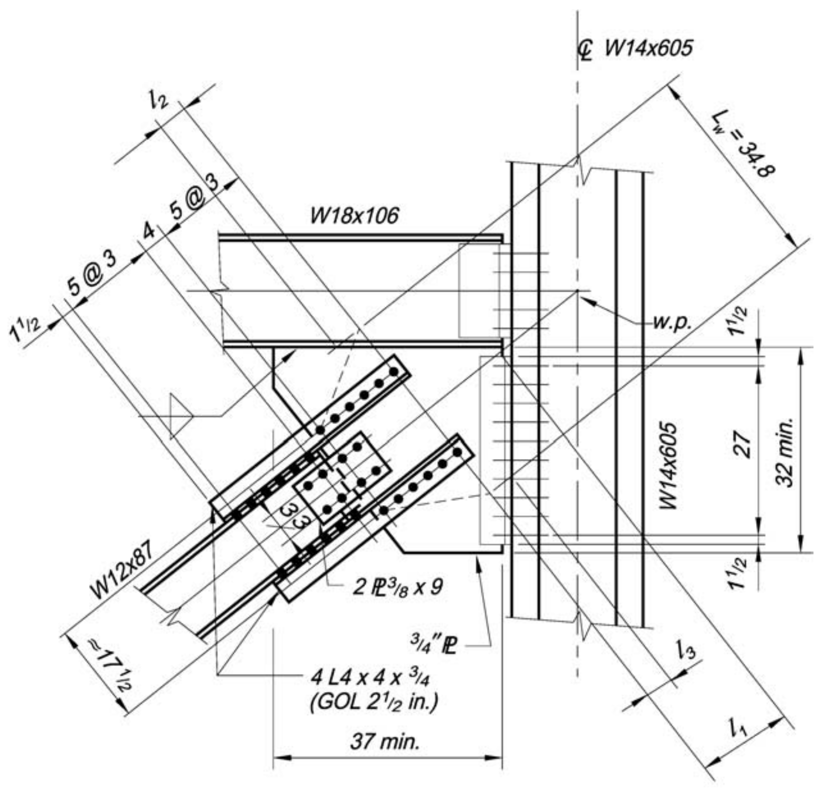

(b) connection

Fig. 13–11. (Continued)

With $n = 5$, $L_{ev} = 1^1/_4$ in., $L_{eh} = 3^1/_4$ in., $0.6F_u A_{nv} > F_u A_{nt}$. Thus

$$\begin{aligned}\phi R_n &= \phi[0.6F_u A_{nv} + F_y A_{gt}] \times 2 \text{ blocks} \\ &\leq \phi[0.6F_u A_{nv} + F_u A_{nt}] \times 2 \text{ blocks}\end{aligned}$$

By inspection, the first part of this equation governs, and

$$\begin{aligned}\phi R_n &= 0.75\big[0.6(58 \text{ ksi})\big(13^1/_4 \text{ in.} - 5 \times 1 \text{ in.}\big)(1 \text{ in.}) \\ &\quad + (36 \text{ ksi})\big(3^1/_4 \text{ in.}\big)(1 \text{ in.})\big](2) \\ &= 606 \text{ kips} > 168 \text{ kips} \qquad \textbf{o.k.}\end{aligned}$$

Check column flange.

By inspection, the 4.16 in.-thick column flange has adequate flexural strength, stiffeners, and bearing strength.

Comments: Were the brace in compression, the buckling strength of the gusset would have to be checked, where

$$\phi R_n = \phi_c F_{cr} A_w$$

In the above equation, $\phi_c F_{cr}$ may be determined from $\frac{kl_1}{r}$ with LRFD Specification Table 3-36, where l_1 is the perpendicular distance from the Whitmore section to the interior corner of the gusset. Alternatively, the average value of Rev. 11/1/02

$$\frac{l_1 + l_2 + l_3}{3}$$

may be substituted (AISC, 1984), where these quantities are illustrated in Figure 13-11. Note that, for this example, l_2 is negative since part of the Whitmore section is in the beam web.

The effective length factor K has been established as 0.5 by full scale tests on bracing connections (Gross, 1990). It assumes that the gusset is supported on both edges as is the case in Figure 13-11. In cases where the gusset is supported on one edge only, such as that illustrated in Figure 13-12d (and possibly Figure 13-12a), the brace can more readily move out-of-plane and a sidesway mode of buckling can occur in the gusset. For this case, K should be taken as 1.2.

EXAMPLE 13.2. Bracing connection design.

Given: Refer to Figure 13-12. Each of the four designs shown for the diagonal bracing connection between the W14×68 brace, W24×55 beam, and W14×211 column web have been developed using the Uniform Force Method (the General Case and Special Cases 1, 2, and 3) for the load case of $1.2D + 1.3W$. Refer to AISC (1992) for the unfactored loads and complete designs. For the given values of α and β, determine the interface forces on the gusset-to-column and gusset-to-beam connections for Rev. 11/1/02

W18×50, ASTM A992

$d = 18.0$ in. $\quad b_f = 7.50$ in. $\quad k = 0.972$ in.
$t_w = 0.355$ in. $\quad t_f = 0.570$ in. $\quad k_1 = {}^{13}/_{16}$ in.

Solution a: $N = 10$ in.

Check local web yielding

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From Table $\boxed{\text{9-5,}}$

$$N_{req} = \frac{R_u - \phi R_1}{\phi R_2}$$

$$\boxed{\begin{aligned} &= \frac{85 \text{ kips} - 43.1 \text{ kips}}{17.8 \text{ kips/in.}} \\ &= 2.35 \text{ in.} < 10 \text{ in.} \qquad \textbf{o.k.} \end{aligned}}$$

Check web crippling

$$\frac{N}{d} = \frac{10 \text{ in.}}{18.0 \text{ in.}} = 0.556$$

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Since $\frac{N}{d} > 0.2$, from Table $\boxed{\text{9-5,}}$

$$N_{req} = \frac{R_u - \phi R_5}{\phi R_6} = \frac{85 \text{ kips} - 51.9 \text{ kips}}{6.29 \text{ kips/in.}} = 5.26 \text{ in.} < 10 \text{ in.} \qquad \textbf{o.k.}$$

Check bearing strength of concrete

$$\begin{aligned} \phi_c P_p &= \phi_c(0.85 f_c')A_1 \\ &= 0.60\,(0.85 \times 3 \text{ ksi})(7.50 \text{ in.} \times 10 \text{ in.}) \\ &= 115 \text{ kips} > 85 \text{ kips} \qquad \textbf{o.k.} \end{aligned}$$

Check beam flange thickness

$$\begin{aligned} n &= \frac{b_f}{2} - k \\ &= \frac{7.50}{2} - 0.972 \text{ in.} \\ &= 2.78 \text{ in.} \end{aligned}$$

$$\begin{aligned} t_{req} &= \sqrt{\frac{2.22 R_u n^2}{A_1 F_y}} \\ &= \sqrt{\frac{2.22(85 \text{ kips})(2.78 \text{ in.})^2}{(7.50 \text{ in.} \times 10 \text{ in.})(50 \text{ ksi})}} \\ &= 0.624 \text{ in.} > 0.570 \text{ in.} \qquad \textbf{n.g.} \end{aligned}$$

Table 14-3.
Typical Column Splices

Case I:
All-bolted flange-plated column splices between columns with depth d_u and d_l nominally the same.

Column Size	Gage g_u or g_l	Flange Plates			
		Type	Width	Thk.	Length
W14×455 to 730	13½	1	16	¾	1′ 6½
257 to 426	11½	1	14	⅝	1′ 6½
145 to 233	11½	1	14	½	1′ 6½
90 to 132	11½	2	14	⅜	1′ 0½
43 to 82	5½	2	8	⅜	1′ 0½
W12×120 to 336	5½	2	8	⅝	1′ 0½
40 to 106	5½	2	8	⅜	1′ 0½
W10×33 to 112	5½	2	8	⅜	1′ 0½
W8×31 to 67	5½	2	8	⅜	1′ 0½
24 & 28	3½	2	6	⅜	1′ 0½

Gages shown may be modified if necessary to accommodate fittings elsewhere on the column.

Case I-A: $d_l = (d_u + 1/4$ in.) to $(d_u + 5/8$ in.)	Flange plates: Select g_u for upper column; select g_l and flange plate dimensions for lower columns (see table above). Fillers: None. Shims: Furnish sufficient strip shims 2½×1⅛ to provide 0 to 1/16-in. clearance each side.
Case I-B: $d_l = (d_u - 1/4$ in.) to $(d_u + 1/8$ in.)	Flange plates: Same as Case I-A. Fillers (shop bolted under flange plates): Select thickness as ⅛-in. for $d_l = d_u$ and $d_l = (d_u + 1/8$ in.) or as ¼-in. for $d_l = (d_u - 1/8$ in.) and $d_l = (d_u - 1/4$ in.) Select width to match flange plate and length as 0′ 9 for Type 1 or 0′ 6 for Type 2. Shims: Same as Case I-A.
Case I-C: $d_l = (d_u + 3/4$ in.) and over.	Flange plates: Same as Case I-A. Fillers (shop bolted to upper column): Select thickness as $(d_l - d_u)$ / 2 minus ⅛-in., whichever results in ⅛-in. multiples of filler thickness. Select width to match flange plate, but not greater than upper column flange width. Select length as 1′ 0 for Type 1 or 0′ 9 for Type 2. Shims: Same as Case I-A.

Rev. 11/1/02 For lifting devices, see Figure 14-11.

Table 14-3 (cont.).
Typical Column Splices

Case II:
All-bolted flange-plated column splices between columns with depth d_u nominally two inches less than depth d_l.

Fillers on upper column developed for bearing on lower column.	Flange plates: Same as Case I-A. Fillers (shop bolted to upper column): Select thickness as (d_l – d_u) / 2 minus 1⁄8-in. or 3⁄16-in., whichever results in 1⁄8-in. multiples of filler thickness. Select bolts through fillers (including bolts through flange plates) on each side to develop bearing stregnth of the filler. Select width to match flange plate, but not greater than upper column flange width unless required for bearing strength. Select length as required to accommodate required number of bolts. Shims: Same as Case I-A.

Table 14-3 (cont.). Rev. 11/1/02
Typical Column splices

Case III:
All-bolted flange-plated and butt-plated column splices between columns with depth d_u nominally two inches less than depth d_l.

Fillers on upper column developed for bearing on lower column.

Column Size	Gage g_u or g_l	Flange Plates Type	Width	Thk.	Length
W14×455 to 730	13½	1	16	¾	1′ 8½
257 to 426	11½	1	14	⅝	1′ 8½
145 to 233	11½	1	14	½	1′ 8½
90 to 132	11½	2	14	⅜	1′ 2½
43 to 82	5½	2	8	⅜	1′ 2½
W12×120 to 336	5½	2	8	⅝	1′ 2½
40 to 106	5½	2	8	⅜	1′ 2½
W10×33 to 112	5½	2	8	⅜	1′ 2½
W8×31 to 67	5½	2	8	⅜	1′ 2
24 & 28	3½	2	8	⅜	1′ 2

Gages shown may be modified if necessary to accommodate fittings elsewhere on the column.

Flange plates: Select g_u for upper column, select g_l and flange plate dimensions for lower column (see table above).
Fillers (shop bolted to upper column): Same as Case I-C.
Shims: Same as Case I-A.
Butt plate: Select thickness as 1½-in. for W8 upper column or two inches for others. Select width the same as upper column and length as d_l – ¼ in.

Rev. 11/1/02 For lifting devices, see Figure 14-11.

Table 14-3 (cont.). Typical Column Splices

Case IV:
All-welded flange-plated column splices between columns with depths d_u and d_l nominally the same.

Column Size	Flange Plate Width	Flange Plate Thk.	Flange Plate Length L	Welds Size A	Welds Length X	Welds Length Y	Minimum Space for Welding M	Minimum Space for Welding N
W14×455 & over	14	5/8	1′-6	1/2	5	7	13/16	11/16
311 to 426	12	5/8	1′-4	1/2	4	6	13/16	11/16
211 to 283	12	1/2	1′-4	3/8	4	6	11/16	9/16
90 to 193	12	3/8	1′-4	5/16	4	6	5/8	1/2
61 to 82	8	3/8	1′-4	5/16	3	6	5/8	1/2
43 to 53	6	5/16	1′-2	1/4	2	5	9/16	7/16
W12×120 to 336	8	1/2	1′-4	3/8	3	6	11/16	9/16
53 to 106	8	3/8	1′-4	5/16	3	6	5/8	1/2
40 to 50	6	5/16	1′-2	1/4	2	5	9/16	7/16
W10×49 to 112	8	3/8	1′-4	5/16	3	6	5/8	1/2
33 to 45	6	5/16	1′-2	1/4	2	5	9/16	7/16
W8×31 to 67	6	3/8	1′-2	5/16	2	5	5/8	1/2
24 & 28	5	5/16	1′-0	1/4	2	4	9/16	7/16

Case IV-A: $d_l = (d_u + 1/8)$	Flange plates: Select flange-plate width and length and weld lengths for upper (lighter) column; select flange-plate thickness and weld size for lower (heavier) column. Fillers: None.
Case IV-B: $d_l = (d_u - 1/4$ in.) to d_u	Flange plates: Same as Case IV-A, except use weld size $A + t$ on lower column. Fillers (undeveloped on lower column, shop welded under flange plates): Select thickness t as $(d_l - d_u) / 2 + 1/16$ in. Select width to match flange plate and length as $L / 2 - 2$ in.
Case IV-C: $d_l = (d_u + 1/4$ in.) to $(d_u + 1/2$ in.)	Flange plates: Same as Case IV-A, except use weld size $A + t$ on upper column. Fillers (undeveloped on upper column, shipped loose): Select thickness t as $(d_l - d_u) / 2 - 1/16$ in. Select width to match flange plate and length as $L / 2 - 2$ in.

Rev. 11/1/02 For lifting devices, see Figure 14-11.

Cold-Formed Welded and Seamless High-Strength, Low-Alloy Structural Tubing with Improved Atmospheric Corrosion Resistance, ASTM A847

and Quenched-and-Tempered Alloy Structural Steel Plates for Bridges, ASTM A709/A709M

Rev. 9/4/01

Quenched and Tempered Low-Alloy Structural Steel Plate with 70 ksi (485 MPa) Minimum Yield Strength to 4 in. (100 mm) Thick, ASTM A852/A852M

High-Strength Low-Alloy Steel Shapes of Structural Quality, Produced by Quenching and Self-Tempering Process (QST), ASTM A913/A913M

Steel for Structural Shapes for Use in Building Framing, ASTM A992/A992M

Certified mill test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM A6/A6M, Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling or A568/A568M, Standard Specification for Steel, Sheet, Carbon, and High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, General Requirements for, as applicable, shall constitute sufficient evidence of conformity with one of the above ASTM standards. If requested, the fabricator shall provide an affidavit stating that the structural steel furnished meets the requirements of the grade specified.

1b. Unidentified Steel

Unidentified steel, if surface conditions are acceptable according to criteria contained in ASTM A6/A6M, is permitted to be used for unimportant members or details, where the precise physical properties and weldability of the steel would not affect the strength of the structure.

1c. Heavy Shapes

For ASTM A6/A6M Group 4 and 5 rolled shapes to be used as members subject to primary tensile stresses due to tension or flexure, toughness need not be specified if splices are made by bolting. If such members are spliced using complete-joint-penetration groove welds, the steel shall be specified in the contract documents to be supplied with Charpy V-notch (CVN) impact testing in accordance with ASTM A6/A6M, Supplementary Requirement S5. The impact test shall meet a minimum average value of 20 ft-lbs. (27 J) absorbed energy at +70°F (+21°C) and shall be conducted in accordance with ASTM A673/A673M, with the following exceptions:

(1) The center longitudinal axis of the specimens shall be located as near as practical to midway between the inner flange surface and the center of the flange thickness at the intersection with the web mid-thickness.

(2) Tests shall be conducted by the producer on material selected from a location representing the top of each ingot or part of an ingot used to produce the product represented by these tests.

For plates exceeding two-in. (50 mm) thick used for built-up cross-sections with bolted splices and subject to primary tensile stresses due to tension or flexure, material toughness need not be specified. If such cross-sections are spliced using complete-joint-penetration welds, the steel shall be specified in the contract documents to be supplied with Charpy V-notch testing in accordance with ASTM A6/A6M, Supplementary Requirement S5. The impact test shall be conducted by the producer in accordance with ASTM A673/A673M, Frequency P, and shall meet a minimum average value of 20 ft-lbs. (27 J) absorbed energy at +70°F (+21°C).

The above supplementary requirements also apply when complete-joint-penetra-

TABLE B5.1 (cont.)
Limiting Width-Thickness Ratios for Compression Elements

	Description of Element	Width Thick-ness Ratio	Limiting Width-Thickness Ratios: λ_p (compact)	Limiting Width-Thickness Ratios: λ_r (noncompact)
Stiffened Elements	Flanges of rectangular box and hollow structural sections of uniform thickness subject to bending or compression; flange cover plates and dia-phragm plates between lines of fasteners or welds for uniform compression for plastic analysis	b/t	 $1.12\sqrt{E/F_y}$ $0.939\sqrt{E/F_y}$	 $1.40\sqrt{E/F_y}$ -
	Unsupported width of cover plates perforated with a suc-cession of access holes [b]	b/t	NA	$1.86\sqrt{E/F_y}$
	Webs in flexural compression [a]	h/t_w	$3.76\sqrt{E/F_y}$ [c], [g]	$5.70\sqrt{E/F_y}$ [h]
	Webs in combined flexural and axial compression	h/t_w	for $P_u/\phi_b P_y \le 0.125$ [c],[g] $3.76\sqrt{\frac{E}{F_y}}\left(1-\frac{2.75P_u}{\phi_b P_y}\right)$ for $P_u/\phi_b P_y > 0.125$ [c][g] $1.12\sqrt{\frac{E}{F_y}}\left(2.33-\frac{P_u}{\phi_b P_y}\right)$ $\ge 1.49\sqrt{\frac{E}{F_y}}$	[h] $5.70\sqrt{\frac{E}{F_y}}\left(1-0.74\frac{P_u}{\boxed{\phi_b P_y}}\right)$ Rev. 9/4/01
	All other uniformly compres-sed stiffened elements, i.e., supported along two edges	b/t h/t_w	NA	$1.49\sqrt{E/F_y}$
	Circular hollow sections In axial compression In flexure	D/t	[d] NA $0.07E/F_y$	 $0.11E/F_y$ $0.31E/F_y$

[a] For hybrid beams, use the yield strength of the flange F_{yf} instead of F_y.

[b] Assumes net area of plate at widest hole.

[c] ~~Assumes an inelastic rotation capacity of 3 radians. For structures in zones of high seismicity, a greater rotation capacity may be required.~~

[d] For plastic design use $0.045E/F_y$.

[e] F_L = smaller of $(F_{yf} - F_r)$ or F_{yw}, ksi (MPa)
F_r = compressive residual stress in flange
= 10 ksi (69 MPa) for rolled shapes
= 16.5 ksi (114 MPa) for welded shapes

[f] $k_c = \frac{4}{\sqrt{h/t_w}}$ and $0.35 \le k_c \le 0.763$

[g] For members with unequal flanges, use h_p instead of h when comparing to λ_p.

[h] For members with unequal flanges, see Appendix B5.1.

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Assumes an inelastic ductility ratio (ratio of strain at fracture to strain at yield) of 3. When the seismic response modification factor ***R*** is taken greater than 3, a greater rotation capacity may be required.

2. Doubly and Singly Symmetric Members in Flexure and Compression

The interaction of flexure and compression in symmetric shapes shall be limited by Equations H1-1a and H1-1b

where

P_u = required compressive strength, kips (N)
P_n = nominal compressive strength determined in accordance with Section E2, kips (N)
ϕ = ϕ_c = resistance factor for compression = 0.85 (see Section E2)
ϕ_b = resistance factor for flexure = 0.90

H2. UNSYMMETRIC MEMBERS AND MEMBERS UNDER TORSION AND COMBINED TORSION, FLEXURE, SHEAR, AND/OR AXIAL FORCE

The design strength, $\phi\boxed{F_n,}$ of the member shall equal or exceed the required strength expressed in terms of the normal stress f_{un} or the shear stress f_{uv}, determined by elastic analysis for the factored loads:

(a) For the limit state of yielding under normal stress:

$$f_{un} \le \phi \boxed{F_n} \qquad \text{(H2-1)}$$

$$\phi = 0.90$$

$$\boxed{F_n = F_y}$$

(b) For the limit state of yielding under shear stress:

$$f_{uv} \le 0.6\phi\boxed{F_n} \qquad \text{(H2-2)}$$

$$\phi = 0.90$$

$$\boxed{F_n = F_y}$$

(c) For the limit state of buckling:

$$f_{un} \text{ or } f_{uv} \le \phi_c \boxed{F_n}, \text{ as applicable} \qquad \text{(H2-3)}$$

$$\phi_c = 0.85$$

$$\boxed{F_n = F_{cr}}$$

Some constrained local yielding is permitted adjacent to areas which remain elastic.

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H3. ALTERNATIVE INTERACTION EQUATIONS FOR MEMBERS UNDER COMBINED STRESS

See Appendix H3.

TABLE J2.1
Effective Throat Thickness of Partial-Joint-Penetration Groove Welds

Welding Process	Welding Position	Included Angle at Root of Groove	Effective Throat Thickness
Shielded metal arc Submerged arc	All	J or U joint	Depth of chamfer
Gas metal arc		Bevel or V joint ≥ 60°	
Flux-cored arc		Bevel or V joint < 60° but ≥ 45°	Depth of chamfer Minus ⅛-in. (3 mm)

TABLE J2.2
Effective Throat Thickness of Flare Groove Welds

Type of Weld	Radius (R) of Bar or Bend	Effective Throat Thickness
Flare bevel groove	All	$\frac{5}{16}R$
Flare V-groove	All	$\boxed{\frac{1}{2}R \text{ [a]}}$

[a] Use $\frac{3}{8}R$ for Gas Metal Arc Welding (except short circuiting transfer process) when $R \geq 1$ in. (25 mm)

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TABLE J2.3
Minimum Effective Throat Thickness of Partial-Joint-Penetration Groove Welds

Material Thickness of Thicker Part Joined, in. (mm)	Minimum Effective Throat Thickness [a], in. (mm)
To ¼ (6) inclusive	⅛ (3)
Over ¼ (6) to ½ (13)	3/16 (5)
Over ½ (13) to ¾ (19)	¼ (6)
Over ¾ (19) to 1½ (38)	5/16 (8)
Over 1½ (38) to 2¼ (57)	⅜ (10)
Over 2¼ (57) to 6 (150)	½ (13)
Over 6 (150)	⅝ (16)

[a] See Table J2.1

exception, particular care shall be taken to provide sufficient preheat for soundness of the weld.

2. Fillet Welds

2a. Effective Area

The effective area of fillet welds shall be as defined in AWS D1.1 Section 2.4.3 and 2.11. The effective throat thickness of a fillet weld shall be the shortest distance from the root of the joint to the face of the diagrammatic weld, except that for fillet welds made by the submerged arc process, the effective throat thickness shall be

TABLE J3.5
Nominal Tension Stress (F_t), ksi (MPa)
Fasteners in Bearing-type Connections

Description of Fasteners	Threads Included in the Shear Plane	Threads Excluded from the Shear Plane
A307 bolts	$59 - 2.5\ f_v \leq 45$ $\boxed{(407 - 2.5\ f_v \leq 310)}$	
A325 bolts A325M bolts	$117 - 2.5\ f_v \leq 90$ $(807 - 2.5\ f_v \leq 621)$	$117 - 2.0\ f_v \leq 90$ $(807 - 2.0\ f_v \leq 621)$
A490 bolts A490M bolts	$147 - 2.5\ f_v \leq 113$ $(1010 - 2.5\ f_v \leq 779)$	$147 - 2.0\ f_v \leq 113$ $(1010 - 2.0\ f_v \leq 779)$
Threaded parts A449 bolts over 1½ diameter	$0.98F_u - 2.5\ f_v \leq 0.75F_u$	$0.98F_u - 2.0\ f_v \leq 0.75F_u$
A502 Gr. 1 rivets	$59 - 2.4\ f_v \leq 45$ $(407 - 2.4\ f_v \leq 310)$	
A502 Gr. 2 rivets	$78 - 2.4\ f_v \leq 60$ $(538 - 2.4\ f_v \leq 414)$	

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TABLE J3.6
Values of Edge Distance Increment C_2, in.

Nominal Diameter of Fastener (in.)	Oversized Holes	Slotted Holes		
		Long Axis Perpendicular to Edge		Long Axis Parallel to Edge
		Short Slots	Long Slots [a]	
≤ ⅞	1/16	⅛	¾d	0
1	⅛	⅛		
≥ 1⅛	⅛	3/16		

[a] When length of slot is less than maximum allowable (see Table J3.5), C_2 are permitted to be reduced by one-half the difference between the maximum and actual slot lengths.

TABLE J3.6M
Values of Edge Distance Increment C_2, mm

Nominal Diameter of Fastener (mm)	Oversized Holes	Slotted Holes		
		Long Axis Perpendicular to Edge		Long Axis Parallel to Edge
		Short Slots	Long Slots [a]	
≤ 22	2	3	0.75d	0
24	3	3		
≥ 27	3	5		

[a] When length of slot is less than maximum allowable (see Table J3.5), C_2 are permitted to be reduced by one-half the difference between the maximum and actual slot lengths.

If $d \leq 25$ in. (635 mm),

$$R_n = 1.2(F_y - 13)ld / 20 \tag{J8-2}$$

$$(\text{Metric: } R_n = 1.2(F_y - 90)\, ld / 20) \tag{J8-2M}$$

If $d > 25$ in. (635 mm),

$$R_n = 6.0(F_y - 13)l\sqrt{d} / 20 \tag{J8-3}$$

$$\boxed{(\text{Metric: } R_n = 30.2(F_y - 90)l\sqrt{d}\ / 20)} \tag{J8-3M}$$

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where

d = diameter, in. (mm)
l = length of bearing, in. (mm)

J9. COLUMN BASES AND BEARING ON CONCRETE

Proper provision shall be made to transfer the column loads and moments to the footings and foundations.

In the absence of code regulations, design bearing loads on concrete may be taken as $\phi_c P_p$:

(a) On the full area of a concrete support

$$P_p = 0.85 f_c' A_1 \tag{J9-1}$$

(b) On less than the full area of a concrete support

$$P_p = 0.85 f_c' A_1 \sqrt{A_2 / A_1} \tag{J9-2}$$

where

ϕ_c = 0.60
A_1 = area of steel concentrically bearing on a concrete support, in.^{2} (mm^2)
A_2 = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, in.^{2} (mm^2)
$\sqrt{A_2 / A_1} \leq 2$

J10. ANCHOR RODS AND EMBEDMENTS

Steel anchor rods and embedments shall be proportioned to develop the factored load combinations stipulated in Section A4. If the load factors and combinations stipulated in Section A4 are used to design concrete structural elements, the provisions of ACI 318 shall be used with appropriate ϕ factors as given in ACI 318, Appendix C.

3a. Unstiffened Compression Elements

The design strength of unstiffened compression elements whose width-thickness ratio exceeds the applicable limit λ_r as stipulated in Section B5.1 shall be subject to a reduction factor Q_s. The value of Q_s shall be determined by Equations A-B5-3 through A-B5-10, as applicable. When such elements comprise the compression flange of a flexural member, the design flexural strength, in ksi, shall be computed using $\phi_b F_y Q_s$, where $\phi_b = 0.90$. The design strength of axially loaded compression members shall be modified by the appropriate reduction factor Q, as provided in Appendix B5.3d.

(a) For single angles:

when $0.45\sqrt{E/F_y} < b/t < 0.91\sqrt{E/F_y}$:

$$Q_s = 1.340 - 0.76(b/t)\sqrt{F_y/E} \quad \text{(A-B5-3)}$$

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when $b/t \boxed{\geq} 0.91\sqrt{E/F_y}$:

$$Q_s = 0.53E/\left[F_y\left(b/t\right)^2\right] \quad \text{(A-B5-4)}$$

(b) For flanges, angles, and plates projecting from rolled beams or columns or other compression members:

when $0.56\sqrt{E/F_y} < b/t < 1.03\sqrt{E/F_y}$:

$$Q_s = 1.415 - 0.74(b/t)\sqrt{F_y/E} \quad \text{(A-B5-5)}$$

when $b/t \geq 1.03\sqrt{E/F_y}$:

$$Q_s = 0.69E/\left[F_y\left(b/t\right)^2\right] \quad \text{(A-B5-6)}$$

(c) For flanges, angles and plates projecting from built-up columns or other compression members:

when $0.64\sqrt{E/(F_y/k_c)} < b/t < 1.17\sqrt{E/(F_y/k_c)}$:

$$Q_s = 1.415 - 0.65(b/t)\sqrt{(F_y/k_cE)} \quad \text{(A-B5-7)}$$

when $b/t \geq 1.17\sqrt{E/(F_y/k_c)}$:

$$Q_s = 0.90Ek_c/\left[F_y\left(b/t\right)^2\right] \quad \text{(A-B5-8)}$$

The coefficient, k_c, shall be computed as follows:

(a) For I-shaped sections:

$$k_c = \frac{4}{\sqrt{h/t_w}},\ 0.35 \le k_c \le 0.763$$

where

h = depth of web, in. (mm)
t_w = thickness of web, in. (mm)

(b) For other sections:

$k_c = 0.763$

(d) For stems of tees:

when $0.75\sqrt{E/F_y} < \boxed{d/t} < 1.03\sqrt{E/F_y}$:

$$Q_s = 1.908 - 1.22\boxed{(d/t)}\sqrt{F_y/E} \qquad \text{(A-B5-9)}$$

when $\boxed{d/t} \ge 1.03\sqrt{E/F_y}$:

$$Q_s = 0.69\boxed{E}/\left[F_y\ \boxed{(d/t)}^2\right] \qquad \text{(A-B5-10)}$$

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where

d = width of unstiffened compression element as defined in Section B5.1, in. (mm)
t = thickness of unstiffened element, in. (mm)

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3b. Stiffened Compression Elements

When the width-thickness ratio of uniformly compressed stiffened elements (except perforated cover plates) exceeds the limit λ_r stipulated in Section B5.1, a reduced effective width b_e shall be used in computing the design properties of the section containing the element.

(a) For flanges of square and rectangular sections of uniform thickness:

when $\dfrac{b}{t} \ge 1.40\sqrt{\dfrac{E}{f}}$:

$$b_e = 1.91t\sqrt{\frac{E}{f}}\left[1 - \frac{0.38}{(b/t)}\sqrt{\frac{E}{f}}\right] \qquad \text{(A-B5-11)}$$

otherwise $b_e = b$.

(b) For other uniformly compressed elements:

when $\dfrac{b}{t} \ge 1.49\sqrt{\dfrac{E}{f}}$:

$$b_e = 1.91t\sqrt{\frac{E}{f}}\left[1 - \frac{0.34}{(b/t)}\sqrt{\frac{E}{f}}\right] \qquad \text{(A-B5-12)}$$

otherwise $b_e = b$.

where

TABLE A-F1.1
Nominal Strength Parameters

Shape	Plastic Moment M_p	Limit State of Buckling	Limiting Buckling Moment M_r
Channels and doubly and singly symmetric I-shaped beams (including hybrid beams) bent about major axis [a]	$F_y Z_x$ [b]	LTB doubly symmetric members and channels	$F_L S_x$
		LTB singly symmetric members	$F_L S_{xc} \le F_{yt} S_{xt}$
		FLB	$F_L S_x$
		WLB	$R_e F_{yf} S_x$
Channels and doubly and singly symmetric I-shaped members bent about minor axis [a]	$F_y Z_y$	FLB	$F_y S_y$

NOTE: LTB applies only for strong axis bending.
[a] Excluding double angles and tees.
[b] Computed from fully plastic stress distribution for hybrid sections.

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[c] $X_1 = \dfrac{\pi}{S_x}\sqrt{\dfrac{EGJA}{2}}$ $\boxed{X_2 = 4\dfrac{C_w}{I_y}\left(\dfrac{S_x}{GJ}\right)^2}$

[d] $\lambda_r = \dfrac{X_1}{F_L}\sqrt{1+\sqrt{1+X_2 F_L^2}}$

[e] $F_{cr} = \dfrac{M_{cr}}{S_{xc}}$, where $M_{cr} = \dfrac{2EC_b}{L_b}\sqrt{I_y J}\left[B_1 + \sqrt{\left(1+B_2+B_1^2\right)}\right] \le M_p$

where

$B_1 = 2.25\left[2(I_{yc}/I_y)-1\right](h/L_b)\sqrt{(I_y/J)}$

$B_2 = 25(1-I_{yc}/I_y)(I_{yc}/J)(h/L_b)^2$

$C_b = 1.0$ if $I_{yc}/I_y < 0.1$ or $I_{yc}/I_y > 0.9$

$$F_{w\gamma} = \frac{5.9E}{(h_w L / r_{To})^2} \tag{A-F3-7}$$

where

h_s = factor equal to $1.0 + \boxed{0.023\gamma}\sqrt{Ld_o / A_f}$ Rev. 11/1/01

h_w = factor equal to $1.0 + 0.00385\gamma\sqrt{L / r_{To}}$

r_{To} = radius of gyration of a section at the smaller end, considering only the compression flange plus one-third of the compression web area, taken about an axis in the plane of the web, in. (mm)

A_f = area of the compression flange, in.^{2} (mm^2)

and where B is determined as follows:

(a) When the maximum moment M_2 in three adjacent segments of approximately equal unbraced length is located within the central segment and M_1 is the larger moment at one end of the three-segment portion of a member:

$$B = 1.0 + 0.37\left(1.0 + \frac{M_1}{M_2}\right) + 0.50\gamma\left(1.0 + \frac{M_1}{M_2}\right) \geq 1.0 \tag{A-F3-8}$$

(b) When the largest computed bending stress f_{b2} occurs at the larger end of two adjacent segments of approximately equal unbraced lengths and f_{b1} is the computed bending stress at the smaller end of the two-segment portion of a member:

$$B = 1.0 + 0.58\left(1.0 + \frac{f_{b1}}{f_{b2}}\right) - 0.70\gamma\left(1.0 + \frac{f_{b1}}{f_{b2}}\right) \geq 1.0 \tag{A-F3-9}$$

(c) When the largest computed bending stress f_{b2} occurs at the smaller end of two adjacent segments of approximately equal unbraced length and f_{b1} is the computed bending stress at the larger end of the two-segment portion of a member:

$$B = 1.0 + 0.55\left(1.0 + \frac{f_{b1}}{f_{b2}}\right) + 2.20\gamma\left(1.0 + \frac{f_{b1}}{f_{b2}}\right) \geq 1.0 \tag{A-F3-10}$$

In the foregoing, $\gamma = (d_L - d_o) / d_o$ is calculated for the unbraced length that contains the maximum computed bending stress. M_1 / M_2 is considered as negative when producing single curvature. In the rare case where M_1 / M_2 is positive, it is recommended that it be taken as zero. f_{b1} / f_{b2} is considered as negative when producing single curvature. If a point of contraflexure occurs in one of two adjacent unbraced segments, f_{b1} / f_{b2} is considered as positive. The ratio $f_{b1} / f_{b2} \neq 0$.

(d) When the computed bending stress at the smaller end of a tapered member or segment thereof is equal to zero:

$$V_n = 0.6F_{yw}A_w\left(C_v + \frac{1-C_v}{1.15\sqrt{1+(a/h)^2}}\right) \quad \text{(A-G3-2)}$$

Also see Appendix G4 and G5.

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Tension field action is not permitted for end-panels in non-hybrid plate girders, all panels in hybrid and web-tapered plate girders, and when a/h exceeds 3.0 or $\boxed{[260/(h/t_w)]^2}$. For these cases, the nominal strength is:

$$V_n = 0.6F_{yw}A_wC_v \quad \text{(A-G3-3)}$$

The web plate buckling coefficient k_v is given as

$$k_v = 5 + \frac{5}{(a/h)^2} \quad \text{(A-G3-4)}$$

except that k_v shall be taken as 5.0 if a/h exceeds 3.0 or $[260/(h/t_w)]^2$.

The shear coefficient C_v is determined as follows:

(a) For $1.10\sqrt{\dfrac{k_vE}{F_{yw}}} \le \dfrac{h}{t_w} \le 1.37\sqrt{\dfrac{k_vE}{F_{yw}}}$:

$$C_v = \frac{1.10\sqrt{k_vE/F_{yw}}}{h/t_w} \quad \text{(A-G3-5)}$$

(b) For $\dfrac{h}{t_w} > 1.37\sqrt{\dfrac{k_vE}{F_{yw}}}$:

$$C_v = \frac{1.51k_vE}{(h/t_w)^2F_{yw}} \quad \text{(A-G3-6)}$$

G4. TRANSVERSE STIFFENERS

Transverse stiffeners are not required in plate girders where $h/t_w \le 2.45\sqrt{E/F_{yw}}$, or where the required shear V_u, as determined by structural analysis for the factored loads, is less than or equal to $0.6\phi_vF_{yw}A_wC_v$, where C_v is determined for $k_v = 5$ and $\phi_v = 0.90$. Stiffeners may be required in certain portions of a plate girder to develop the required shear or to satisfy the limitations given in Appendix G1. Transverse stiffeners shall satisfy the requirements of Appendix F2.3.

When designing for tension field action, the stiffener area A_{st} shall not be less than

C_m = coefficient applied to the bending term in the interaction equation for prismatic members and dependent on column curvature caused by applied moments, see Section C1.

$$M'_{px} = 1.2M_{px}\left[1-(P_u/P_y)\right] \le M_{px} \quad \text{(A-H3-5)}$$

$$M'_{py} = 1.2M_{py}\left[1-(P_u/P_y)^2\right] \le M_{py} \quad \text{(A-H3-6)}$$

$$M'_{nx} = M_{nx}\left(1-\frac{P_u}{\phi_c P_n}\right)\left(1-\frac{P_u}{P_{ex}}\right) \quad \text{(A-H3-7)}$$

$$M'_{ny} = M_{ny}\left(1-\frac{P_u}{\phi_c P_n}\right)\left(1-\frac{P_u}{P_{ey}}\right) \quad \text{(A-H3-8)}$$

(b) For box-section members:

$$\zeta = 1.7 - \frac{P_u/P_y}{\ln(P_u/P_y)} \quad \text{(A-H3-9)}$$

$$\eta = 1.7 - \frac{P_u/P_y}{\ln(P_u/P_y)} - a\lambda_x\left(\frac{P_u}{P_y}\right)^b > 1.1 \quad \text{(A-H3-10)}$$

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For $P_u/P_y \le \boxed{0.4}$, $a = 0.06$, and $b = 1.0$;

For $P_u/P_y > \boxed{0.4}$, $a = 0.15$, and $b = 2.0$:

$$M'_{px} = 1.2M_{px}\left[1-P_u/P_y\right] \le M_{px} \quad \text{(A-H3-11a)}$$

$$M'_{py} = 1.2M_{py}\left[1-P_u/P_y\right] \le M_{py} \quad \text{(A-H3-11b)}$$

$$M'_{nx} = M_{nx}\left(1-\frac{P_u}{\phi_c P_n}\right)\left(1-\frac{P_u}{P_{ex}}\frac{1.25}{(B/H)^{1/3}}\right) \quad \text{(A-H3-12)}$$

$$M'_{ny} = M_{ny}\left(1-\frac{P_u}{\phi_c P_n}\right)\left(1-\frac{P_u}{P_{ey}}\frac{1.25}{(B/H)^{1/2}}\right) \quad \text{(A-H3-13)}$$

where

P_n = nominal compressive strength determined in accordance with Section E2, kips (N)
P_u = required axial strength, kips (N)
P_y = compressive yield strength A_gF_y, kips (N)
ϕ_b = resistance factor for flexure = 0.90
ϕ_c = resistance factor for compression = 0.85
P_e = Euler buckling strength A_gF_y/λ_c^2, where λ_c is the column slenderness parameter defined by Equation E2-4, kips (N)
M_u = required flexural strength, kip-in. (N-mm)

TABLE A-J3.1
Nominal Tension Stress (F_t), ksi (MPa)
Fasteners in Bearing-type Connections

Description of Fasteners	Threads Included in the Shear Plane	Threads Excluded from the Shear Plane
A307 bolts	$\sqrt{45^2 - 6.25f_v^2}$	
(Metric)	$\left(\sqrt{310^2 - 6.25f_v^2}\right)$	
A325 bolts	$\sqrt{90^2 - 6.25f_v^2}$	$\sqrt{90^2 - 4.00f_v^2}$
(A325M bolts)	$\left(\sqrt{621^2 - 6.25f_v^2}\right)$	$\left(\sqrt{621^2 - 4.00f_v^2}\right)$
A490 bolts	$\sqrt{113^2 - 6.31f_v^2}$	$\sqrt{113^2 - 4.04f_v^2}$
Rev. 9/4/01 (A490M bolts)	$\boxed{\left(\sqrt{779^2 - 6.31f_v^2}\right)}$	$\boxed{\left(\sqrt{779^2 - 4.04f_v^2}\right)}$
Threaded parts A449 bolts over 1½ in. (38 mm)	$\sqrt{(0.75F_u)^2 - 6.25f_v^2}$	$\sqrt{(0.75F_u)^2 - 4.00f_v^2}$
A502 Gr. 1 rivets	$\sqrt{45^2 - 5.76f_v^2}$	
(Metric)	$\left(\sqrt{310^2 - 5.76f_v^2}\right)$	
A502 Gr. 2 rivets	$\sqrt{60^2 - 5.86f_v^2}$	
(Metric)	$\left(\sqrt{414^2 - 5.86f_v^2}\right)$	

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TABLE A-J3.2
Slip-Critical Resistance to Shear at Service Loads, F_v, ksi (MPa), of High-Strength Bolts[a]

Type of Bolt	Resistance to Shear at Service Loads, ksi (MPa)			
	Standard Size Holes	Oversized and Short-slotted Holes	Long-slotted Holes: Perpendicular to Line of Force	Long-slotted Holes: Parallel to Line of Force
A325 (A325M)	17 (117)	15 (103)	12 (83)	10 (69)
A490 (A490M)	21 (145)	18 (124)	15 (103)	13 (90)

[a] For each shear plane.

proportioned to the critical deformation based on distance from the instantaneous center of rotation, r_i, in. (mm)

$= r_i \Delta_u / r_{crit}$

Δ_u = $1.087(\theta + 6)^{-0.65} w \leq 0.17w$, deformation of weld element at ultimate stress (fracture), usually in element furthest from instantaneous center of rotation, in. (mm)

w = leg size of the fillet weld, in. (mm)

r_{crit} = distance from instantaneous center of rotation to weld element with minimum Δ_u / r_i ratio, in. (mm)

J3. BOLTS AND THREADED PARTS

7. Combined Tension and Shear in Bearing-Type Connections

As an alternative to the use of the equations in Table J3.5, the use of the equations in Table A-J3.1 is permitted.

8. High-Strength Bolts in Slip-Critical Connections

8b. Slip-Critical Connections Designed at Service Loads

The design resistance to shear per bolt $\phi F_v A_b$ for use at service loads shall equal or exceed the shear per bolt due to service loads,

where

ϕ = 1.0 for standard, oversized, and short-slotted holes and long-slotted holes when the long slot is perpendicular or parallel to the line of force

F_v = ~~nominal~~ slip-critical shear resistance tabulated in Table A-J3.2, ksi (MPa). The values for F_v in Table A-J3.2 are based on Class A surfaces with slip coefficient $\mu = 0.33$. When specified by the designer, the ~~nominal~~ slip resistance for connections having special faying surface conditions is permitted to be adjusted to the applicable values in the RCSC Load and Resistance Factor Design Specification. Rev. 9/4/01

When the loading combination includes wind loads in addition to dead and live loads, the total shear on the bolt due to combined load effects, at service load, may be multiplied by 0.75.

9. Combined Tension and Shear in Slip-Critical Connections

9b. Slip-Critical Connections Designed at Service Loads

When a slip-critical connection is subjected to an applied tension T that reduces the net clamping force, the slip resistance per bolt, $\phi F_v A_b$, according to Appendix J3.8b shall be multiplied by the following factor:

$$1 - \frac{T}{0.8 T_b N_b}$$

where

T_b = minimum fastener tension from Table J3.1, kips (N)

N_b = number of bolts carrying service-load tension T

APPENDIX K

CONCENTRATED FORCES, PONDING, AND FATIGUE

Appendix K2 provides an alternative determination of roof stiffness. Appendix K3 pertains to the design of members and connections subject to high cyclic loading (fatigue).

K2. PONDING

The provisions of this Appendix are permitted to be used when a more exact determination of flat roof framing stiffness is needed than that given by the provision of Section K2 that $C_p + 0.9C_s \leq 0.25$.

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For any combination of primary and secondary framing, the stress index is computed as

$$U_p = \left[\frac{F_y - f_o}{f_o}\right]_p \quad \text{for the primary member} \tag{A-K2-1}$$

$$U_s = \left[\frac{F_y - f_o}{f_o}\right]_s \quad \text{for the secondary member} \tag{A-K2-2}$$

where

f_o = the stress due to $1.2D + 1.2R$ (D = nominal dead load, R = nominal load due to rain water or ice exclusive of the ponding contribution),* ksi (MPa)

Enter Figure A-K2.1 at the level of the computed stress index U_p determined for the primary beam; move horizontally to the computed C_s value of the secondary beams and then downward to the abscissa scale. The combined stiffness of the primary and secondary framing is sufficient to prevent ponding if the flexibility constant read from this latter scale is more than the value of C_p computed for the given primary member; if not, a stiffer primary or secondary beam, or combination of both, is required. In the above,

$$C_p = \frac{32 L_s L_p^4}{10^7 I_p}$$

$$\left(\text{Metric: } C_p = \frac{504 L_s L_p^4}{I_p}\right)$$

$$C_s = \frac{32 S L_s^4}{10^7 I_s}$$

*Depending upon geographic location, this loading should include such amount of snow as might also be present, although ponding failures have occurred more frequently during torrential summer rains when the rate of precipitation exceeded the rate of drainage runoff and the resulting hydraulic gradient over large roof areas caused substantial accumulation of water some distance from the eaves. A load factor of 1.2 shall be used for loads resulting from these phenomena.

center of the connected leg, the effects of eccentricity shall be ignored. If the center of gravity of the connecting welds lies outside this zone, the total stresses, including those due to joint eccentricity, shall be included in the calculation of stress range.

3. Design Stress Range

The range of stress at service loads shall not exceed the stress range computed as follows.

(a) For stress categories A, B, B′, C, D, E and E′ the design stress range, F_{SR}, shall be determined by Equation A-K3.1 or A-K3.1M.

$$F_{SR} = \left(\frac{C_f}{N}\right)^{0.333} \geq F_{TH} \qquad \text{(A-K3.1)}$$

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$$\left(\text{Metric: } F_{SR} = \left(\frac{C_f \times \boxed{329}}{N}\right)^{0.333} \geq F_{TH}\right) \qquad \text{(A-K3.1M)}$$

where

F_{SR} = Design stress range, ksi (MPa)
C_f = Constant from Table A-K3.1 for the category
N = Number of stress range fluctuations in design life
= Number of stress range fluctuations per day × 365 × years of design life
F_{TH} = Threshold fatigue stress range, maximum stress range for indefinite design life from Table A-K3.1, ksi (MPa)

(b) For stress category F, the design stress range, F_{SR}, shall be determined by Equation A-K3.2 or A-K3.2M.

$$F_{SR} = \left(\frac{C_f}{N}\right)^{0.167} \geq F_{TH} \qquad \text{(A-K3.2)}$$

$$\left(\text{Metric: } F_{SR} = \left(\frac{C_f \times 11 \times 10^4}{N}\right)^{0.167} \geq F_{TH}\right) \qquad \text{(A-K3.2M)}$$

(c) For tension-loaded plate elements connected at their end by cruciform, T- or corner details with complete-joint-penetration groove welds or partial-joint-penetration groove welds, fillet welds, or combinations of the preceding, transverse to the direction of stress, the design stress range on the cross section of the tension-loaded plate element at the toe of the weld shall be determined as follows:

Based upon crack initiation from the toe of the weld on the tension loaded plate element the design stress range, F_{SR}, shall be determined by Equation A-K3.1 or A-K3.1M, for Category C which is equal to

$$F_{SR} = \left(\frac{44 \times 10^8}{N}\right)^{0.333} \geq 10$$

$$\left(\text{Metric: } F_{SR} = \left(\frac{14.4 \times 10^{11}}{N}\right)^{0.333} \geq 68.9\right)$$

Based upon crack initiation from the root of the weld the design stress range, F_{SR}, on the tension loaded plate element using transverse partial-joint-penetration groove welds, with or without reinforcing or contouring fillet welds, the design stress range on the cross section at the toe of the weld shall be determined by Equation A-K3.3 or A-K3.3M, Category C′ as follows:

$$F_{SR} = R_{PJP}\left(\frac{44 \times 10^{8}}{N}\right)^{0.333} \quad \text{(A-K3.3)}$$

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$$\boxed{\left(\text{Metric: } F_{SR} = R_{PJP}\left(\frac{14.4 \times 10^{11}}{N}\right)^{0.333}\right)} \quad \text{(A-K3.3M)}$$

where:

R_{PJP} = reduction factor for reinforced or non-reinforced transverse partial-joint-penetration (PJP) joints. Use Category C if R_{PJP} = 1.0.

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$$= \left(\frac{0.65 - 0.59\left(\frac{2a}{t_p}\right) + 0.72\left(\frac{w}{t_p}\right)}{t_p^{0.167}}\right) \leq 1.0 \quad \boxed{= \left(\frac{1.12 - 1.01\left(\frac{2a}{t_p}\right) + 1.24\left(\frac{w}{t_p}\right)}{t_p^{0.167}}\right) \leq 1.0 \text{ (Metric)}}$$

$2a$ = the length of the non-welded root face in the direction of the thickness of the tension-loaded plate, in. (mm)

w = the leg size of the reinforcing or contouring fillet, if any, in the direction of the thickness of the tension-loaded plate, in. (mm)

t_p = thickness of tension loaded plate, in. (mm)

Based upon crack initiation from the roots of a pair of transverse fillet welds on opposite sides of the tension loaded plate element the design stress range, F_{SR}, on the cross section at the toe of the welds shall be determined by Equation A-K3.4 or A-K3.4M, Category C″ as follows:

$$F_{SR} = R_{FIL}\left(\frac{44 \times 10^{8}}{N}\right)^{0.333} \quad \text{(A-K3.4)}$$

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$$\boxed{\left(\text{Metric: } F_{SR} = R_{FIL}\left(\frac{14.4 \times 10^{11}}{N}\right)^{0.333}\right)} \quad \text{(A-K3.4M)}$$

where

R_{FIL} = reduction factor for joints using a pair of transverse fillet welds only. Use Category C if R_{FIL} = 1.0.

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$$= \left(\frac{0.06 + 0.72(w / t_p)}{t_p^{0.167}}\right) \leq 1.0 \quad \boxed{= \left(\frac{0.10 + 1.24(w / t_p)}{t_p^{0.167}}\right) \leq 1.0 \text{ (Metric)}}$$

TABLE A-K3.1 (Cont'd)
Fatigue Design Parameters

Description	Stress Category	Constant C_f	Threshold F_{TH} Ksi (MPa)	Potential Crack Initiation Point
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)				
6.4 Base metal subject to longitudinal stress at transverse members, with or without transverse stress, attached by fillet or partial penetration groove welds parallel to direction of stress when the detail embodies a transition radius, *R*, with weld termination ground smooth:				In weld termination or from the toe of the weld extending into member
$R > 2$ in. (50 mm)	D	22×10^8	7 (48)	
$R \leq 2$ in. (50 mm)	E	11×10^8	4.5 (31)	
SECTION 7 – BASE METAL AT SHORT ATTACHMENTS[1]				
Rev. 11/1/02 7.1 Base metal subject to longitudinal loading at details attached by fillet welds parallel or transverse to the direction of stress where the detail embodies no transition radius and with detail length in direction of stress, *a*, and attachment height normal to surface of member, *b*:				In the member at the end of the weld
$a < 2$ in. (50 mm)	C	44×10^8	10 (69)	
2 in. (50 mm) $\leq a \leq 12b$ or 4 in (100 mm)	D	22×10^8	7 (48)	
$a > 12b$ or 4 in. (100 mm) when b is ≤ 1 in. (25 mm)	E	11×10^8	4.5 (31)	
$a > 12b$ or 4 in. (100 mm) when b is > 1 in. (25 mm)	E′	3.9×10^8	2.6 (18)	
7.2 Base metal subject to longitudinal stress at details attached by fillet or partial joint penetration groove welds, with or without transverse load on detail, when the detail embodies a transition radius, *R*, with weld termination ground smooth:				In weld termination extending into member
$R > 2$ in. (50 mm)	D	22×10^8	7 (48)	
$R \leq 2$ in. (50 mm)	E	11×10^8	4.5 (31)	

[1] "Attachment" as used herein, is defined as any steel detail welded to a member which, by its mere presence and independent of its loading, causes a discontinuity in the stress flow in the member and thus reduces the fatigue resistance.

ticular structure. Rather, the decision as to which welding process and which filler metal is to be utilized is usually left with the fabricator or erector. To ensure that the proper filler metals are used, codes restrict the usage of certain filler materials, or impose qualification testing to prove the suitability of the specific electrode.

A4. LOADS AND LOAD COMBINATIONS

The load factors and load combinations are developed in Ellingwood, MacGregor, Galambos, and Cornell (1982) based on the recommended minimum loads given in ASCE 7 (ASCE, 1998).

The load factors and load combinations recognize that when several loads act in combination with the dead load (e.g., dead plus live plus wind), only one of these takes on its maximum lifetime value, while the other load is at its "arbitrary point-in-time value" (i.e., at a value which can be expected to be on the structure at any time). For example, under dead, live, and wind loads the following combinations are appropriate:

$$\gamma_D D + \gamma_L L \qquad \text{(C-A4-1)}$$

$$\gamma_D D + \gamma_{L_a} L_a + \gamma_W W \qquad \text{(C-A4-2)}$$

$$\gamma_D D + \gamma_L L + \gamma_{W_a} W_a \qquad \text{(C-A4-3)}$$

where γ is the appropriate load factor as designated by the subscript symbol. Subscript a refers to an "arbitrary point-in-time" value.

The mean value of arbitrary point-in-time live load L_a is on the order of 0.24 to 0.4 times the mean maximum lifetime live load L for many occupancies, but its dispersion is far greater. The arbitrary point-in-time wind load W_a, acting in conjunction with the maximum lifetime live load, is the maximum daily wind. It turns out that $\gamma_{W_a} W_a$ is a negligible quantity so only two load combinations remain:

$$1.2D + 1.6L \qquad \text{(C-A4-4)}$$

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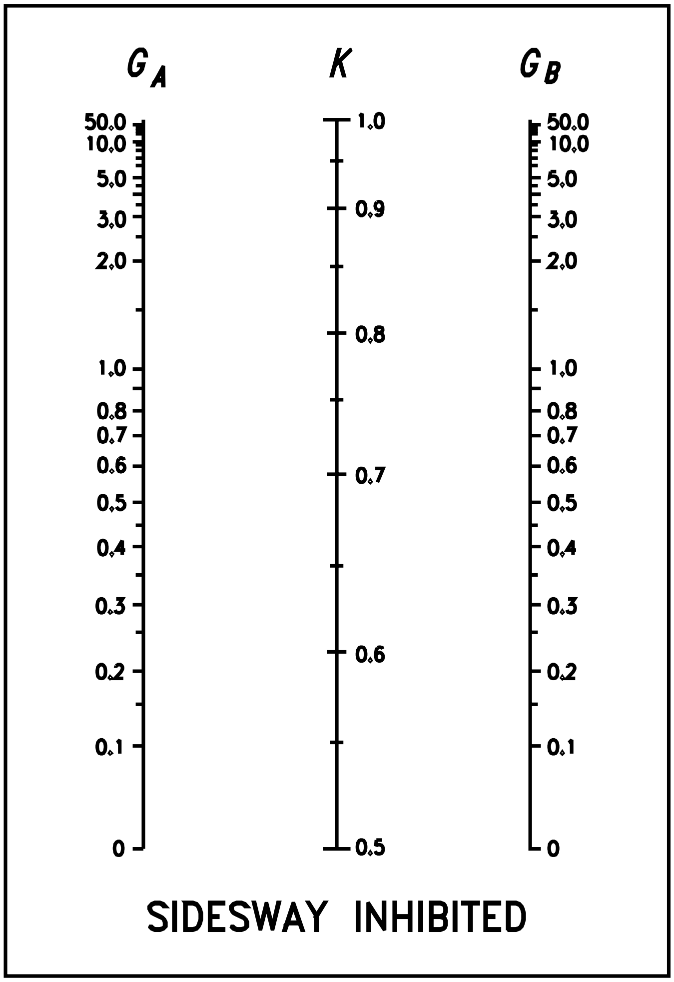
$$1.2D + 0.5L + \boxed{1.6}W \qquad \text{(C-A4-5)}$$

The load factor 0.5 assigned to L in the second formula reflects the statistical properties of L_a, but to avoid having to calculate yet another load, it is reduced so it can be combined with the maximum lifetime wind load.

The nominal loads D, L, W, E, and S are the code loads or the loads given in ASCE 7. The latest edition of the ASCE 7 Standard on structural loads released in 1998 has adopted, in most aspects, the seismic design provisions from NEHRP (1997), as has the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 1997 and 1999). The reader is referred to the commentaries to these documents for an expanded discussion on seismic loads, load factors, and seismic design of steel buildings.



Notes for Fig. C-C2.2a and b: The subscripts A and B refer to the joints at the two ends of the column section being considered. G is defined as

$$G = \frac{\Sigma(I_c \ / \ L_c)}{\Sigma(I_g \ / \ L_g)}$$

in which Σ indicates a summation of all members rigidly connected to that joint and lying on the plane in which buckling of the column is being considered. I_c *is the moment of inertia and* L_c *the unsupported length of a column section, and* I_g *is the moment of inertia and* L_g *the unsupported length of a girder or other restraining member.* I_c *and* I_g *are taken about axes perpendicular to the plane of buckling being considered.*

For column ends supported by but not rigidly connected to a footing or foundation, G is theoretically infinity, but, unless actually designed as a true friction-free pin, may be taken as "10" for practical designs. If the column end is rigidly attached to a properly designed footing, G may be taken as 1.0. Smaller values may be used if justified by analysis. Rev. 9/4/01

Fig. C-C2.2a. Alignment chart for effective length of columns in continuous frames –Sidesway Inhibited.

ond equation). Therefore, the requirement that the nominal compressive strength P_n be based on the effective length KL in the general equation is continued in the LRFD Specification as it has been in the AISC ASD Specification since 1961. It is not intended that these provisions be applicable to limit nonlinear secondary flexure that might be encountered in large amplitude earthquake stability design (ATC, 1978).

The defined term M_u is the maximum moment in a member. In the calculation of this moment, inclusion of beneficial second order effects of tension is optional. But consideration of detrimental second order effects of axial compression and translation of gravity loads is required. Provisions for calculation of these effects are given in Chapter C.

The interaction equations in Appendix H3 have been recommended for biaxially loaded H and wide flange shapes in Galambos (1998) and Springfield (1975). These equations which can be used only in braced frames represent a considerable liberalization over the provisions given in Section H1; it is, therefore, also necessary to check yielding under service loads, using the appropriate load and resistance factors for the serviceability limit state in Equation H1-1a or H1-1b with $\boxed{M_{nx}} = S_xF_y$ and $\boxed{M_{ny}} = S_yF_y$. Appendix H3 also provides interaction equations for rectangular box-shaped beam-columns. These equations are taken from Zhou and Chen (1985). Rev. 11/1/02

H2. UNSYMMETRIC MEMBERS AND MEMBERS UNDER TORSION AND COMBINED TORSION, FLEXURE, SHEAR, AND/OR AXIAL FORCE

This section deals with types of cross sections and loadings not covered in Section H1, especially where torsion is a consideration. For such cases it is recommended to perform an elastic analysis based on the theoretical numerical methods available from the literature for the determination of the maximum normal and shear stresses, or for the elastic buckling stresses. In the buckling calculations an equivalent slenderness parameter is determined for use in Equation E2-2 or E2-3, as follows:

$$\lambda_e = \sqrt{F_y / F_e}$$

where F_e is the elastic buckling stress determined from a stability analysis. This procedure is similar to that of Appendix E3.

For the analysis of members with open sections under torsion refer to Seaburg and Carter (1997). Rev. 11/1/02

$$S_{eff} = S_s + \sqrt{\left(\Sigma Q_n / C_f\right)}\left(S_{tr} - S_s\right) \qquad \text{(C-I3-7)}$$

where

S_s = section modulus for the structural steel section, referred to the tension flange, in.^{3} (mm^3)

S_{tr} = section modulus for the fully composite uncracked transformed section, referred to the tension flange of the steel section, in.^{3} (mm^3)

Equations C-I3-6 and C-I3-7 should not be used for ratios $\Sigma Q_n / C_f$ less than 0.25. This restriction is to prevent excessive slip, as well as substantial loss in beam stiffness. Studies indicate that Equations C-I3-6 and C-I3-7 adequately reflect the reduction in beam stiffness and strength, respectively, when fewer connectors are used than required for full composite action (Grant, Fisher, and Slutter, 1977).

It is not practical to make accurate deflection calculations of composite flexural sections in the design office. Careful comparisons to short-term deflection tests indicate that the effective moment of inertia, I_{eff}, is 15 to 30 percent lower than that calculated based on linear elastic theory. Therefore, for realistic deflection calculations, I_{eff} should be taken as 0.80 I_{eff} or 0.75 I_{eff}. As an alternative, it has been shown that one may use lower bound moment of inertia, I_{lb}, as defined below:

$$I_{lb} = I_x + A_s(Y_{ENA} - d_3)^2 + (\Sigma Q_n / F_y)(2d_3 + d_1 - Y_{ENA})^2 \qquad \text{(C-I3-8)}$$

where

d_1 = distance from the centroid of the longitudinal slab reinforcement to the top of the steel section, in. (mm)

d_3 = distance from P_{yc} to the top of the steel section, in. (mm)

I_{lb} = lower bound moment of inertia, in.^{3} (mm^3)

$\boxed{Y_{ENA} = [(A_s d_3 + (\Sigma Q_n/F_y)\ (2d_3 + d_1))/(A_s + (\Sigma Q_n/F_y))]}$ Rev. 11/1/02

Calculations for long-term deformations due to creep and shrinkage may also be carried out. Because the basic properties of the concrete are not known to the designer, simplified models such as those proposed by Viest, Fountain, and Singleton (1958), Branson (1964), Chien and Ritchie (1984), and Viest, Colaco, Furlong, Griffis, Leon, and Wyllie (1997) can be used.

Negative Flexural Design Strength. The flexural strength in the negative moment region is the strength of the steel beam alone or the plastic strength of the composite section made up of the longitudinal slab reinforcement and the steel section.

Plastic Stress Distribution for Negative Moment. When an adequately braced compact steel section and adequately developed longitudinal reinforcing bars act compositely in the negative moment region, the nominal flexural strength is determined from the plastic stress distributions as shown in Figure C-I3.2. The tensile force T in the reinforcing bars is the smaller of:

$$T = A_r F_{yr} \qquad \text{(C-I3-9)}$$

$$T = \Sigma Q_n \qquad \text{(C-I3-10)}$$

There are practical cases in the design of structures where slip of the connection is desirable in order to allow for expansion and contraction of a joint in a controlled manner. Regardless of whether force transfer is required in the directions normal to the slip direction, the nuts should be hand-tightened with a spud wrench and then backed off one-quarter turn. Furthermore, it is advisable to deform the bolt threads or use a locking nut or jamb nut to insure that the nut does not back off under service conditions. Thread deformation is commonly accomplished with a cold chisel and hammer applied at one location. Note that tack-welding of the nut to the bolt threads is discouraged.

2. Size and Use of Holes

To provide some latitude for adjustment in plumbing up a frame during erection, three types of enlarged holes are permitted, subject to the approval of the designer. The nominal maximum sizes of these holes are given in Table J3.3 or J3.3M. The use of these enlarged holes is restricted to connections assembled with bolts and is subject to the provisions of Sections J3.3 and J3.4.

3. Minimum Spacing

The *maximum* factored strength R_n at a bolt or rivet hole in bearing requires that the distance between the centerline of the first fastener and the edge of a plate toward which the force is directed should not be less than $1\frac{1}{2}d$ where d is the fastener diameter (Kulak et al., 1987). By similar reasoning the distance measured in the line of force, from the centerline of any fastener to the nearest edge of an adjacent hole, should not be less than $3d$, to ensure maximum design strength in bearing. Plotting of numerous test results indicates that the critical bearing strength is directly proportional to the above defined distances up to a maximum value of $3d$, above which no additional bearing strength is achieved (Kulak et al., 1987). ~~Table J3.7 lists the increments that must be added to adjust the spacing upward to compensate for an increase in hole dimension parallel to the line of force.~~ Section J3.10 gives the bearing strength criteria as a function of spacing.

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4. Minimum Edge Distance

Critical bearing stress is a function of the material tensile strength, the spacing of

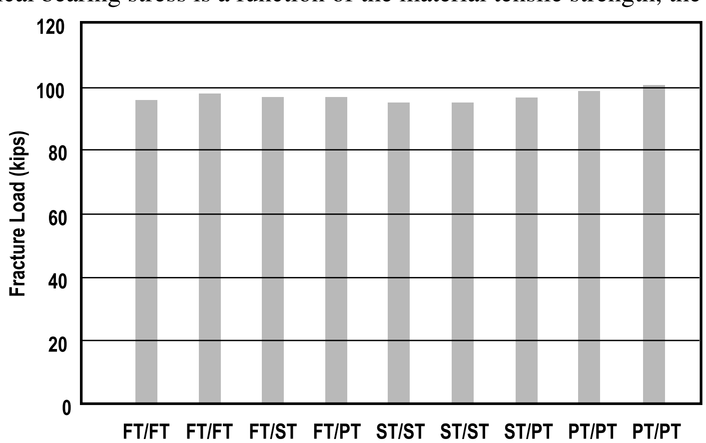

Fig. C-J3.2. Johnson (1996) tests, $3\frac{1}{4}$-in.-long, $\frac{3}{4}$-in.-diameter ASTM A325 bolts.

F_{crft}, F_{cry}, F_{crz}	Flexural-torsional buckling stresses for double-angle and tee-shaped compression members, ksi
F_e	Elastic buckling stress, ksi
F_{ex}	Elastic flexural buckling stress about the major axis, ksi
F_{ey}	Elastic flexural buckling stress about the minor axis, ksi
F_{ez}	Elastic torsional buckling stress, ksi
F_{my}	Modified yield stress for the design of composite columns, ksi
F_n	Nominal shear rupture strength, ksi
F_n, F_{nt}	Nominal strength of bolt, ksi
F_p	Nominal bearing stress on fastener, ksi
F_r	Compressive residual stress in flange [10 ksi for rolled shapes; 16.5 ksi for welded built-up shapes]
$F_{s\gamma}$	Stress for tapered members defined by LRFD Specification Equation A-F3-6, ksi
F_t	Nominal tensile strength of bolt from LRFD Specification Table J3.2, ksi
F_u	Specified minimum tensile strength of the type of steel being used, ksi
F_v	Nominal shear strength of bolt from LRFD Specification Table J3.2, ksi
F_w	Nominal strength of the weld electrode material, ksi
$F_{w\gamma}$	Stress for tapered members defined by Equation A-F3-7, ksi
F_y	Specified minimum yield stress of the type of steel being used, ksi. As used in the LRFD Specification, "yield stress" denotes either the specified minimum yield point (for steels that have a yield point) or specified yield strength (for steels that do not have a yield point)
F_y'''	The theoretical maximum yield stress (ksi) based on the web depth-thickness ratio (h / t_w) above which the web of a column is considered a $\boxed{\text{slender element}}$ (See LRFD Specification Table B5.1) $= \left(\frac{253}{h / t_w}\right)^2$ Note: In the tables, — indicates $F_y''' > 65$ ksi. Rev. 11/1/02
F_{yb}	F_y of a beam, ksi
F_{yc}	F_y of a column, ksi
F_{yf}	Specified minimum yield stress of the flange, ksi
F_{yr}	Specified minimum yield stress of reinforcing bars, ksi
$F_{y\,st}$	Specified minimum yield stress of the stiffener material, ksi
F_{yw}	Specified minimum yield stress of the web, ksi
G	Shear modulus of elasticity of steel (G = 11,200 ksi)
G	Ratio of the total column stiffness framing into a joint to that of the stiffening members framing into the same joint
H	Horizontal force, kips
H	Flexural constant
H	Average story height
H	Height of bolt head or nut, in.
H	Theoretical thread height, in. (see Table 7-4)
H_s	Length of shear stud connector after welding, in.
H_1	Height of bolt head, in. (see Tables 7-3)
H_2	Maximum bolt shank extension based on one standard hardened washer, in. (see Tables 7-3)
I	Moment of inertia, in.^{4}
I_{LB}	Lower bound moment of inertia for composite section, in.^{4}
I_c	Moment of inertia of column section about axis perpendicular to plane of buckling, in.^{4}
I_d	Moment of inertia of the steel deck supported on secondary members, in.^{4}
I_g	Moment of inertia of girder about axis perpendicular to plane of buckling, in.^{4}
I_p	Moment of inertia of primary member, in.^{4}

S_{eff}	Effective section modulus about major axis, in.^{3}
S_{net}	Net elastic section modulus, in.^{3}
S_w	Warping statical moment at a point on the cross section, in.^{4}
S_x	Elastic section modulus about major axis, in.^{3}
S_x'	Elastic section modulus of larger end of tapered member about its major axis, in.^{3}
S_{xt}, S_{xc}	Elastic section modulus referred to tension and compression flanges, respectively, in.^{3}
SRF	Stiffness reduction factors (Table 4-1), for use with the alignment charts (LRFD Specification Figure C-C2.2) in the determination of effective length factors *K* for columns
T	Distance between web toes of fillets at top and at bottom of web, in. = d - 2k
T	Tension force due to service loads, kips
T	Thickness of flat circular washer or mean thickness of square or rectangular beveled washer, in.
T	Unfactored tensile force on slip-critical connections designed at service loads, kips
T_b, T_m	Specified pretension load in high-strength bolt (LRFD Specification Table J3.1), kips
T_{stl}	Tensile force in steel in a composite beam, kips
T_{Tot}	Sum of tensile forces in a composite beam, kips
T_u	Required tensile strength due to factored loads, kips
U	Reduction coefficient, used in calculating effective net area
V	Shear force, kips
V_b	Shear force component, kips
V_h	Total horizontal force transferred by the shear connections, kips
V_n	Nominal shear strength, kips
V_u	Required shear strength, kips
W	Wind load
W	Uniformly distributed load, kips
W	Weight, lbs or kips, as indicated
W	Width across flats of nut, in.
W_c	Uniform load constant for beams, kip-ft
W_{no}	Normalized warping function at a point at the flange edge, in.^{2}
W_u	Total factored uniformly distributed load, kips
Workable Gage	Gage for fasteners in flange (Part 1) that provides for entering and tightening clearances and edge distance and spacing requirements, in. When the listed value is shaded, the actual size, combination and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility. Other gages that provide for entering and tightening clearances and edge distance and spacing requirements can also be used.
X_1	Beam buckling factor defined by LRFD Specification Equation F1-8
X_2	Beam buckling factor defined by LRFD Specification Equation F1-9
Y_{ENA}	Distance from bottom of steel beam to elastic neutral axis, in.
Y_{con}	Distance from top of steel beam to top of concrete, in.
$\boxed{Y1}$	Distance from top of steel beam to the plastic neutral axis, in.
$\boxed{Y2}$	Distance from top of steel beam to the concrete flange force in a composite beam, in.
Z	Plastic section modulus, in.^{3}
Z'	Additional plastic section modulus corresponding to $^1/_{16}$-inch increase in web thickness for built-up wide flange section, in.^{3}
Z_e	Effective plastic section modulus, in.^{3}
a	Clear distance between transverse stiffeners, in.
a	Distance between connectors in a built-up member, in.

Errata 11/1/02