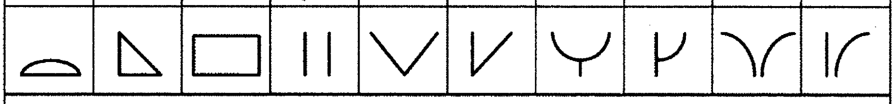
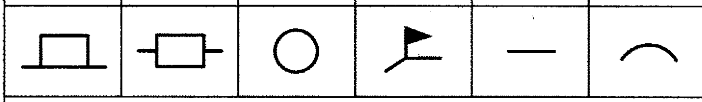
Table 8–2
Prequalified Welded Joints

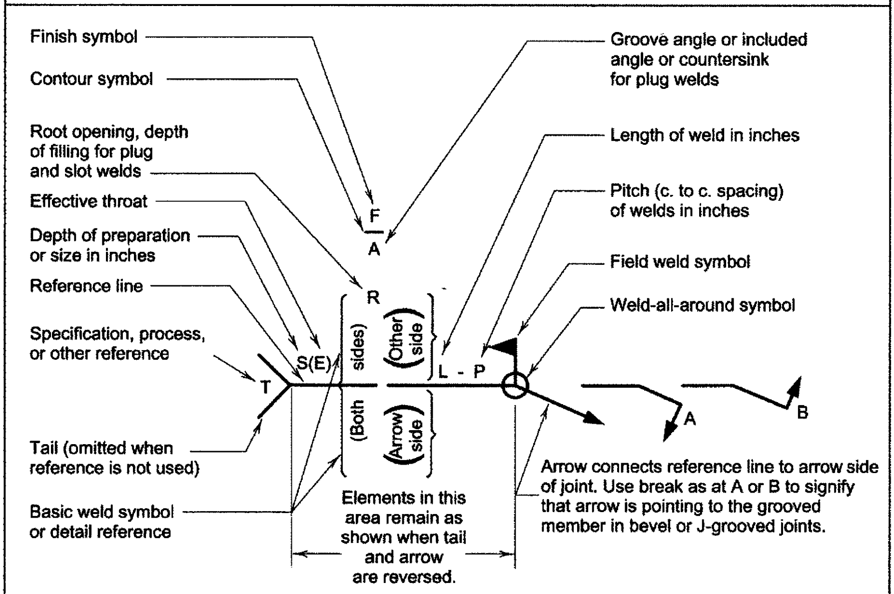
Symbols for Joint Types			
B	butt joint	BC	butt or corner joint
C	corner joint	TC	T- or corner joint
T	T-joint	BTC	butt, T-, or corner joint
Symbols for Base Metal Thickness and Penetration			
L	limited thickness, complete-joint-penetration		
U	unlimited thickness, complete-joint-penetration		
P	partial-joint-penetration		
Symbols for Weld Types			
1	square-groove	6	single-U-groove
2	single-V-groove	7	double-U-groove
3	double-V-groove	8	single-J-groove
4	single-bevel-groove	9	double-J-groove
5	double-bevel-groove	10	flare-bevel-groove
Symbols for Welding Processes if not Shielded Metal Arc Welding (SMAW):			
S	submerged arc welding (SAW)		
G	gas metal arc welding (GMAW)		
F	flux cored arc welding (FCAW)		
Symbols for Welding Positions			
F	flat		
H	horizontal		
V	vertical		
OH	overhead		
Symbols for Joint Designation			
The lower case letters (e.g., a, b, c, d, etc.) are used to differentiate between joints that would otherwise have the same joint designation.			
Symbols for Dimensions			
R	Root opening		
α, β	Groove angles		
f	Root face		
r	J- or U-groove radius		
S, S_1, S_2	PJP groove weld depth of groove		
E, E_1, E_2	PJP groove weld sizes corresponding to S, S_1, S_2, respectively		
Notes to Prequalified Welded Joints			
1	Not prequalified for gas metal arc welding (GMAW) using short circuiting transfer nor GTAW. Refer to AWS D1.1 Annex A.		
2	Joint is welded from one side only.		
3	Cyclic load application limits these joints to the horizontal welding position. Refer to AWS D1.1 Section 2.17.2.		
4	Backgouge root to sound metal before welding second side.		
5	SMAW joints may be used for prequalified GMAW (except GMAW-S) and FCAW.		
6	Minimum effective throat thickness (E) as shown in Specification Table J2.3; S as specified on drawings.		
7	If fillet welds are used in buildings to reinforce groove welds in corner and T-joints, they shall be equal to $^1/_4\,T_1$, but need not exceed $^3/_8$ in. Groove welds in corner and T-joints of cyclically loaded structures shall be reinforced with fillet welds equal to $^1/_4\,T_1$, but need not exceed $^3/_8$ in.		
8	Double-groove welds may have grooves of unequal depth, but the depth of the shallower groove shall be no less than one-fourth of the thickness of the thinner part joined.		
9	Double-groove welds may have grooves of unequal depth, provided these conform to the limitations of Note 6. Also, the effective throat thickness (E) applies individually to each groove.		
10	The orientation of the two members in the joints may vary from 135° to 180° for butt joints, or 45° to 135° for corner joints, or 45° to 90° for T-joints.		
11	For corner joints, the ouside groove preparation may be in either or both members, provided the basic groove configuration is not changed and adequate edge distance is maintained to support the welding operations without excessive edge melting.		
12	Effective throat thickness (E) is based on joints welded flush.		

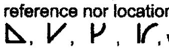
Table 8–2 (continued)
Prequalified Welded Joints

Basic Weld Symbols									
			Groove or Butt						
Back	Fillet	Plug or Slot	Square	V	Bevel	U	J	Flare V	Flare Bevel
									

Supplementary Weld Symbols						
				Contour		For other basic and supplementary weld symbols, see AWS A2.4
Backing	Spacer	Weld All Around	Field Weld	Flush	Convex	
						

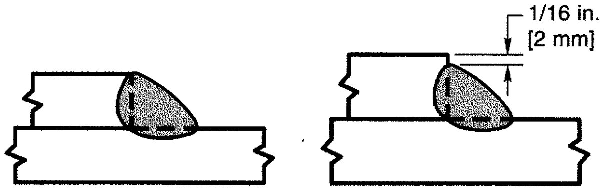
Standard Location of Elements of a Welding Symbol



Note:
Size, weld symbol, length of weld, and spacing must read in that order, from left to right, along the reference line. Neither orientation of reference nor location of the arrow alters this rule.
The perpendicular leg of , weld symbols must be at left.
Arrow and other side welds are of the same size unless otherwise shown. Dimensions of fillet welds must be shown on both the arrow side and the other side symbol.
The point of the field weld symbol must point toward the tail.
Symbols apply between abrupt changes in direction of welding unless governed by the “all around” symbol or otherwise dimensioned.
These symbols do not explicitly provide for the case that frequently occurs in structural work, where duplicate material (such as stiffeners) occurs on the far side of a web or gusset plate. The fabricating industry has adopted this convention: that when the billing of the detail material discloses the existence of a member on the far side as well as on the near side, the welding shown for the near side shall be duplicated on the far side.

FILLET

Table 8–2 (continued)
Prequalified Welded Joints
Fillet Welds



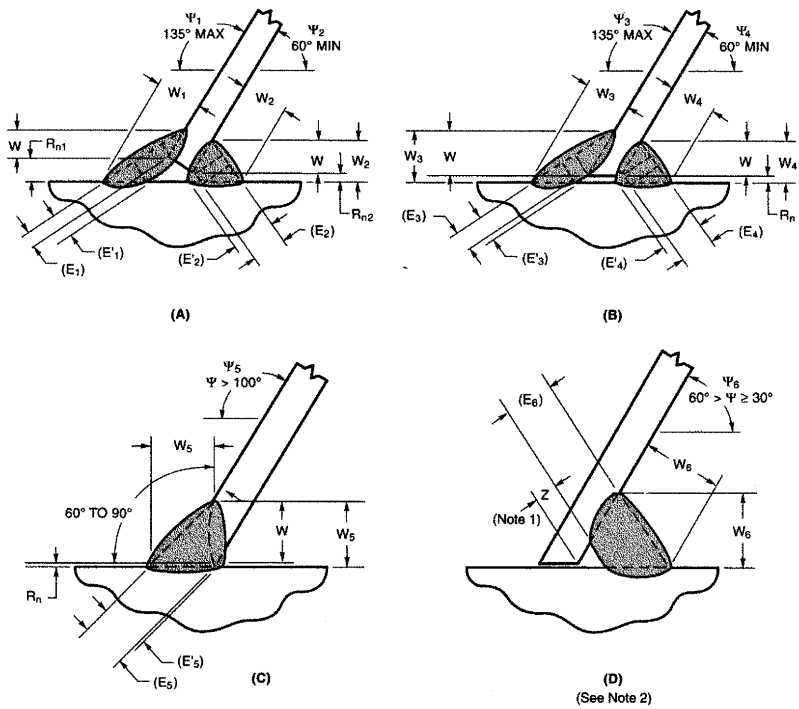
BASE METAL LESS THAN 1/4 in. [6 mm] THICK

(A)

BASE METAL 1/4 in. [6 mm] OR MORE IN THICKNESS

(B)

MAXIMUM DETAILED SIZE OF FILLET WELD ALONG EDGES



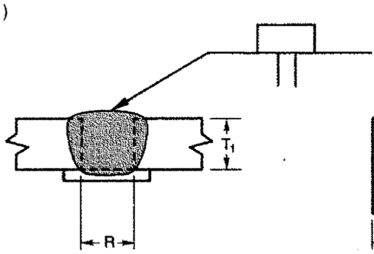
Notes:

1. (E_n), (E'_n) = Effective throat thickness dependant on magnitude of root opening (R_n). Refer to AWS D1.1 Section 5.22.1. Subscript *n* represents 1, 2, 3, 4, or 5.
2. t = thickness of thinner part.
3. Not prequalified for gas metal arc welding (GMAW) using short circuit transfer nor GTAW. Refer to AWS D1.1 Annex A for GMAW-S.
4. Figure D. Apply Z loss dimension of AWS D1.1 Table 2.2 to determine effective throat thickness.
5. Figure D. Not prequlaified for angles under 30°. For welder qualifications see AWS D1.1 Table 4.8.
6. Angles under 60° are permissible, however, if the weld is considered to be a partial-joint-penetration groove weld.

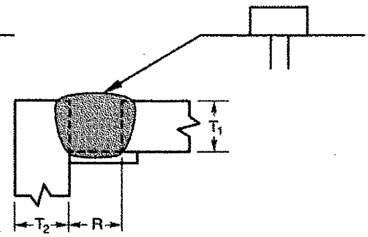
Table 8–2 (continued) Prequalified Welded Joints Complete-Joint-Penetration Groove Welds

CJP

Square-groove weld (1)
Butt joint (B)
Corner joint (C)



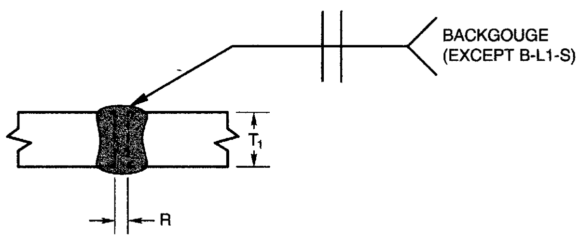
B-L1a



C-L1a

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
				Root Opening	Tolerances				
		T_1	T_2		As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)			
SMAW	B-L1a	$\frac{1}{4}$ max	—	$R = T_1$	$+\frac{1}{16}, -0$	$+\frac{1}{4}, -\frac{1}{16}$	All	—	5, 10
	C-L1a	$\frac{1}{4}$ max	U	$R = T_1$	$+\frac{1}{16}, -0$	$+\frac{1}{4}, -\frac{1}{16}$	All	—	5, 10
FCAW GMAW	B-L1a-GF	$\frac{3}{8}$ max	—	$R = T_1$	$+\frac{1}{16}, -0$	$+\frac{1}{4}, -\frac{1}{16}$	All	Not Required	1, 10

Square-groove weld (1)
Butt joint (B)

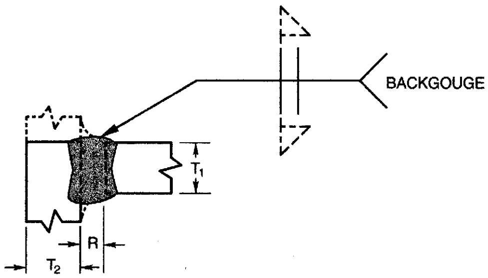


Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
				Root Opening	Tolerances				
		T_1	T_2		As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)			
SMAW	B-L1b	$\frac{1}{4}$ max	—	$R = \frac{T_1}{2}$	$+\frac{1}{16}, -0$	$+\frac{1}{16}, -\frac{1}{8}$	All	—	4, 5, 10
GMAW FCAW	B-L1b-GF	$\frac{3}{8}$ max	—	R = 0 to $\frac{1}{8}$	$+\frac{1}{16}, -0$	$+\frac{1}{16}, -\frac{1}{8}$	All	Not Required	1, 4, 10
SAW	B-L1-S	$\frac{3}{8}$ max	—	R = 0	±0	$+\frac{1}{16}, -0$	F	—	10
SAW	B-L1a-S	$\frac{5}{8}$ max	—	R = 0	±0	$+\frac{1}{16}, -0$	F	—	4, 10

Table 8–2 (continued)
Prequalified Welded Joints
Complete-Joint-Penetration Groove Welds

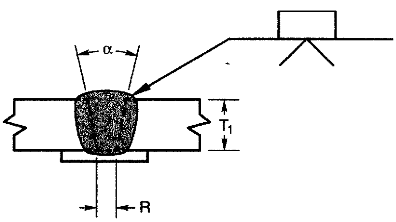
CJP

Square-groove weld (1)
T-joint (T)
Corner joint (C)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
				Root Opening	Tolerances				
		T_1	T_2		As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)			
SMAW	TC-L1b	1/4 max	U	$R = \frac{T_1}{2}$	+1/16, –0	+1/16, –1/8	All	—	4, 5, 7
GMAW FCAW	TC-L1-GF	3/8 max	U	R = 0 to 1/8	+1/16, –0	+1/16, –1/8	All	Not Required	1, 4, 7
SAW	TC-L1-S	3/8 max	U	R = 0	±0	+1/16, –0	F	—	4, 7

Single-V-groove weld (2)
Butt joint (B)



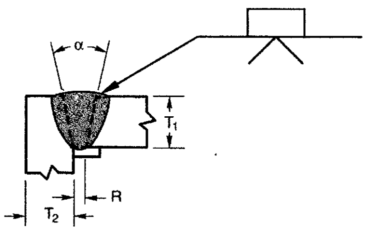
Tolerances	
As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
R = +1/16, –0	+1/4, –1/16
α = +10°, –0°	+10°, –5°

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation		Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Groove Angle			
SMAW	B-U2a	U	—	R = 1/4	α = 45°	All	—	5, 10
				R = 3/8	α = 30°	F, V, OH	—	5, 10
				R = 1/2	α = 20°	F, V, OH	—	5, 10
GMAW FCAW	B-U2a-GF	U	—	R = 3/16	α = 30°	F, V, OH	Required	1, 10
				R = 3/8	α = 30°	F, V, OH	Not req.	1, 10
				R = 1/4	α = 45°	F, V, OH	Not req.	1, 10
SAW	B-L2a-S	2 max	—	R = 1/4	α = 30°	F	—	10
SAW	B-U2-S	U	—	R = 5/8	α = 20°	F	—	10

CJP

Table 8–2 (continued)
Prequalified Welded Joints
Complete-Joint-Penetration Groove Welds

Single-V-groove weld (2)
Corner joint (C)



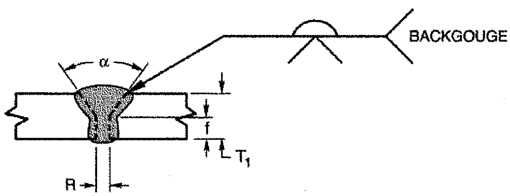
Tolerances	
As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
R = +1/16, −0	+1/4, −1/16
α = +10°, −0°	+10°, −5°

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation		Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Groove Angle			
SMAW	C-U2a	U	U	R = 1/4	α = 45°	All	—	5, 10
				R = 3/8	α = 30°	F, V, OH	—	5, 10
				R = 1/2	α = 20°	F, V, OH	—	5, 10
GMAW FCAW	C-U2a-GF	U	U	R = 3/16	α = 30°	F, V, OH	Required	1
				R = 3/8	α = 30°	F, V, OH	Not req.	1, 10
				R = 1/4	α = 45°	F, V, OH	Not req.	1, 10
SAW	C-L2a-S	2 max	U	R = 1/4	α = 30°	F	—	10
SAW	C-U2-S	U	U	R = 5/8	α = 20°	F	—	10

Table 8–2 (continued)
Prequalified Welded Joints
Complete-Joint-Penetration Groove Welds

CJP

Single-V-groove weld (2)
Butt joint (B)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
					Tolerances				
		T_1	T_2	Root Opening Root Face Groove Angle	As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)			
SMAW	B-U2	U	—	R = 0 to 1/8 f = 0 to 1/8 α = 60°	+1/16, −0 +1/16, −0 + 10°, −0°	+1/16, −1/8 Not Limited +10°, −5°	All	—	4, 5, 10
GMAW FCAW	B-U2-GF	U	—	R = 0 to 1/8 f = 0 to 1/8 α = 60°	+1/16, −0 +1/16, −0 + 10°, −0°	+1/16, −1/8 Not Limited +10°, −5°	All	Not Required	1, 4, 10
SAW	B-L2c-S	Over 1/2 to 1	—	R = 0 f = 1/4 max α = 60°	R = ±0 f = +0, −f α = +10°, −0°	+1/16, −0 ± 1/16 +10°, −5°	F	—	4, 10
		Over 1 to 1 1/2	—	R = 0 f = 1/2 max α = 60°					
		Over 1 1/2 to 2	—	R = 0 f = 5/8 max α = 60°					

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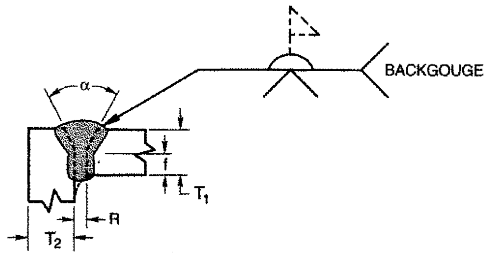
CJP

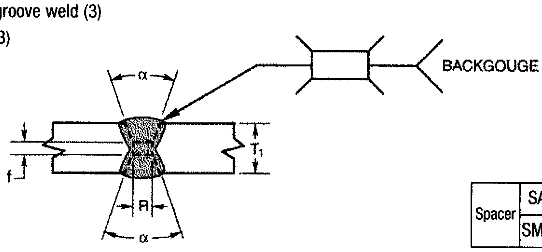
Table 8–2 (continued)
Prequalified Welded Joints
Complete-Joint-Penetration Groove Welds

Single-V-groove weld (2)
Corner joint (C)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
				Root Opening Root Face Groove Angle	Tolerances				
		T_1	T_2		As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)			
SMAW	C-U2	U	U	R = 0 to 1/8 f = 0 to 1/8 α = 60°	+1/16, –0 +1/16, –0 + 10°, –0°	+1/16, –1/8 Not Limited +10°, –5°	All	—	4, 5, 7, 10
GMAW FCAW	C-U2-GF	U	U	R = 0 to 1/8 f = 0 to 1/8 α = 60°	+1/16, –0 +1/16, –0 + 10°, –0°	+1/16, –1/8 Not Limited +10°, –5°	All	Not Required	1, 4, 7, 10
SAW	C-U2b-S	U	U	R = 0 to 1/8 f = 1/4 max α = 60°	±0 +0, –1/4 +10°, –0°	+1/16, –0 ±1/16 +10°, –5°	F	—	4, 7, 10

Double-V-groove weld (3)
Butt joint (B)



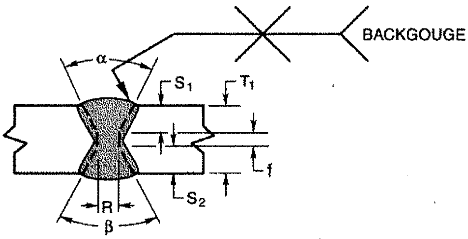
		Tolerances	
		As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
		R = ±0	+1/4, –0
		f = ±0	+1/16, –0
		α = +10°, –0°	+10°, –5°
Spacer	SAW	±0	+1/16, –0
	SMAW	±0	1/8, –0

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Root Face	Groove Angle			
SMAW	B-U3a	U Spacer = 1/8 × R	—	R = 1/4	f = 0 to 1/8	α = 45°	All	—	4, 5, 8, 10
				R = 3/8	f = 0 to 1/8	α = 30°	F, V, OH	—	
				R = 1/2	f = 0 to 1/8	α = 20°	F, V, OH	—	
SAW	B-U3a-S	U Spacer = 1/4 × R	—	R = 5/8	f = 0 to 1/4	α = 20°	F	—	4, 8, 10

Table 8–2 (continued)
Prequalified Welded Joints
Complete-Joint-Penetration Groove Welds

CJP

Double-V-groove weld (3)
Butt joint (B)



For B-U3c-S only		
T_1		S_1
Over	to	
2	$2^1/_2$	$1^3/_8$
$2^1/_2$	3	$1^3/_4$
3	$3^5/_8$	$2^1/_8$
$3^5/_8$	4	$2^3/_8$
4	$4^3/_4$	$2^3/_4$
$4^3/_4$	$5^1/_2$	$3^1/_4$
$5^1/_2$	$6^1/_4$	$3^3/_4$
For $T_1 > 6^1/_4$ or $T_1 \leq 2$ $S_1 = {}^2/_3(T_1 - {}^1/_4)$		

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
					Tolerances				
		T_1	T_2	Root Opening Root Face Groove Angle	As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)			
SMAW	B-U3b	U	—	R = 0 to $^1/_8$	$+^1/_{16}, -0$	$+^1/_{16}, -^1/_8$	All	—	4, 5, 8, 10
GMAW FCAW	B-U3-GF	U	—	f = 0 to $^1/_8$ $\alpha = \beta = 60°$	$+^1/_{16}, -0$ $+10°, -0°$	Not limited $+10°, -5°$	All	Not required	1, 4, 8, 10
SAW	B-U3c-S	U	—	R = 0 f = $^1/_4$ min $\alpha = \beta = 60°$	$+^1/_{16}, -0$ $+^1/_4, -0$ $+10°, -0°$	$+^1/_{16}, -0$ $+^1/_4, -0$ $+10°, -5°$	F	—	4, 8, 10
				To find S_1 see table above: $S_2 = T_1 - (S_1+f)$					

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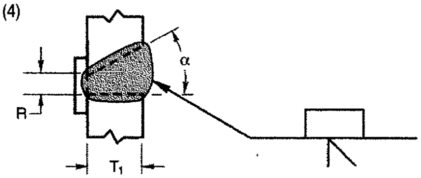
CJP

Table 8–2 (continued)
Prequalified Welded Joints
Complete-Joint-Penetration Groove Welds

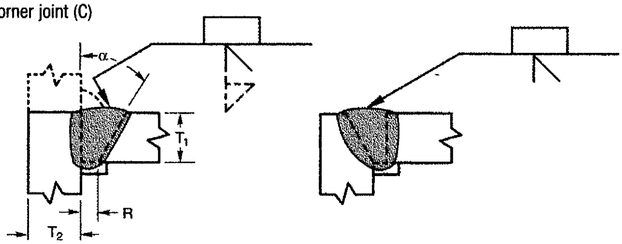
Single-bevel-groove weld (4)
Butt joint (B)



Tolerances	
As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
R = +1/16, –0	+1/4, –1/16
α = +10°, –0°	+10°, –5°

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation		Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Groove Angle			
SMAW	B-U4a	U	—	R = 1/4	α = 45°	All	—	3, 5, 10
				R = 3/8	α = 30°	All	—	3, 5, 10
GMAW FCAW	B-U4a-GF	U	—	R = 3/16	α = 30°	All	Required	1, 3, 10
				R = 1/4	α = 45°	All	Not req.	1, 3, 10
				R = 3/8	α = 30°	F, H	Not req.	1, 3, 10
SAW	B-U4a-S	U	U	R = 3/8	α = 30°	F	—	3, 10
				R = 1/4	α = 45°			

Single-bevel-groove weld (4)
T-joint (T)
Corner joint (C)



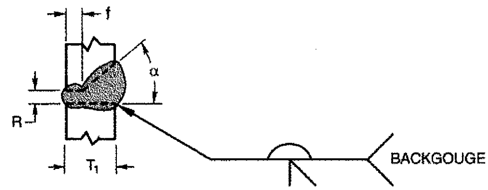
Tolerances	
As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
R = +1/16, –0	+1/4, –1/16
α = +10°, –0°	+10°, –5°

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation		Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Groove Angle			
SMAW	TC-U4a	U	U	R = 1/4	α = 45°	All	—	5, 7, 10, 11
				R = 3/8	α = 30°	F, V, OH	—	5, 7, 10, 11
GMAW FCAW	TC-U4a-GF	U	U	R = 3/16	α = 30°	All	Required	1, 7, 10, 11
				R = 3/8	α = 30°	F	Not req.	1, 7, 10, 11
				R = 1/4	α = 45°	All	Not req.	1, 7, 10, 11
SAW	TC-U4a-S	U	U	R = 3/8	α = 30°	F	—	7, 10, 11
				R = 1/4	α = 45°			

CJP

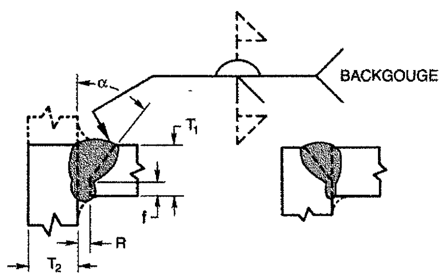
Table 8–2 (continued)
Prequalified Welded Joints
Complete-Joint-Penetration Groove Welds

Single-bevel-groove weld (4)
Butt joint (B)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening Root Face Groove Angle	Tolerances: As Detailed (see 3.13.1)	Tolerances: As Fit-Up (see 3.13.1)			
SMAW	B-U4b	U	—	R = 0 to 1/8 f = 0 to 1/8 α = 45°	+1/16, −0 +1/16, −0 + 10°, −0°	+1/16, −1/8 Not Limited +10°, −5°	All	—	3, 4, 5, 10
GMAW FCAW	B-U4b-GF	U	—				All	Not Required	1, 3, 4, 10
SAW	B-U4b-S	U	U	R = 0 f = 1/4 max α = 60°	±0 +0, −1/8 + 10°, −0°	+1/4, −0 ±1/16 10°, −5°	F	—	3, 4, 10

Single-bevel-groove weld (4)
T-joint (T)
Corner joint (C)



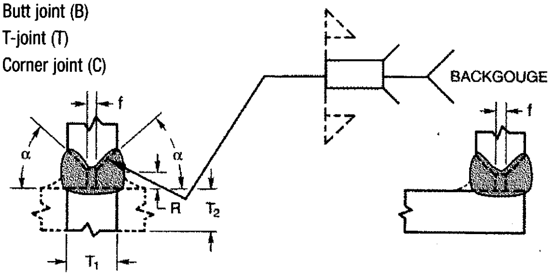
Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening Root Face Groove Angle	Tolerances: As Detailed (see 3.13.1)	Tolerances: As Fit-Up (see 3.13.1)			
SMAW	TC-U4b	U	U	R = 0 to 1/8 f = 0 to 1/8 α = 45°	+1/16, −0 +1/16, −0 + 10°, −0°	+1/16, −1/8 Not Limited +10°, −5°	All	—	4, 5, 7, 10, 11
GMAW FCAW	TC-U4b-GF	U	U				All	Not Required	1, 4, 7, 10, 11
SAW	TC-U4b-S	U	U	R = 0 f = 1/4 max α = 60°	±0 +0, −1/8 + 10°, −0°	+1/4, −0 ±1/16 10°, −5°	F	—	4, 7, 10, 11

Table 8-2 (continued)

CJP

Prequalified Welded Joints
Complete-Joint-Penetration Groove Welds

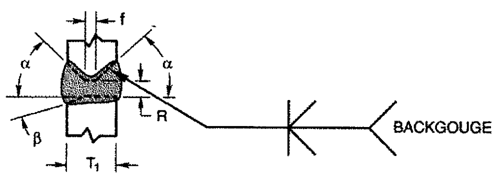
Double-bevel-groove weld (5)
Butt joint (B)
T-joint (T)
Corner joint (C)



	Tolerances	
	As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
	R = ±0	$+^1/_4, -0$
	$f = +^1/_{16}, -0$	$\pm^1/_{16}$
	α = +10°, −0°	+10°, −5°
Spacer	$+^1/_{16}, -0$	$+^1/_8, -0$

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Root Face	Groove Angle			
SMAW	B-U5b	U Spacer = $^1/_8 \times R$	U	$R = ^1/_4$	$f = 0$ to $^1/_8$	α = 45°	All	—	3, 4, 5, 8, 10
	TC-U5a	U Spacer = $^1/_4 \times R$	U	$R = ^1/_4$	$f = 0$ to $^1/_8$	α = 45°	All	—	4, 5, 7, 8, 10, 11
				$R = ^3/_8$	$f = 0$ to $^1/_8$	α = 30°	F, OH	—	4, 5, 7, 8, 10, 11

Double-bevel-groove weld
Butt joint (B)

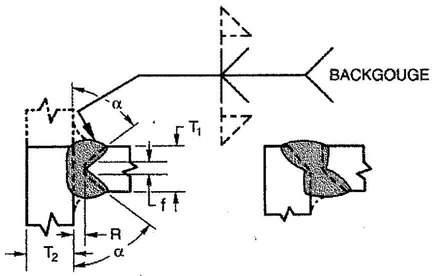


Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
				Root Opening Root Face Groove Angle	Tolerances				
		T_1	T_2		As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)			
SMAW	B-U5a	U	—	$R = 0$ to $^1/_8$ $f = 0$ to $^1/_8$ α = 45° β = 0° to 15°	$+^1/_{16}, -0$ $+^1/_{16}, -0$ $\alpha + \beta \ ^{+10°}_{\ \ 0°}$	$+^1/_{16}, -^1/_8$ Not limited $\alpha + \beta \ ^{+10°}_{-5°}$	All	—	3, 4, 5, 8, 10
GMAW FCAW	B-U5-GF	U	—	$R = 0$ to $^1/_8$ $f = 0$ to $^1/_8$ α = 45° β = 0° to 15°	$+^1/_{16}, -0$ $+^1/_{16}, -0$ α + β= + 10°, −0°	$+^1/_{16}, -^1/_8$ Not limited α + β= + 10°, −5°	All	Not Required	1, 3, 4, 8, 10

CJP

Table 8–2 (continued)
Prequalified Welded Joints
Complete-Joint-Penetration Groove Welds

Double-bevel-groove weld (5)
T-joint (T)
Corner joint (C)

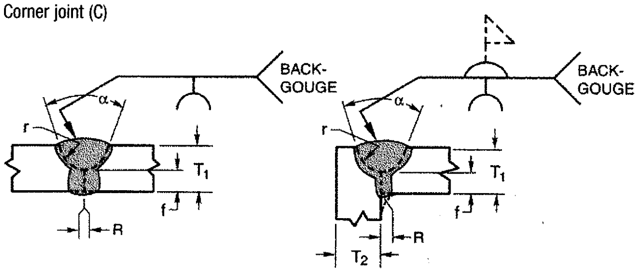


Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening Root Face Groove Angle	Tolerances: As Detailed (see 3.13.1)	Tolerances: As Fit-Up (see 3.13.1)			
SMAW	TC-U5b	U	U	R = 0 to 1/8 f = 0 to 1/8	+1/16, −0 +1/16, −0	+1/16, −1/8 Not limited	All	—	4, 5, 7, 8, 10, 11
GMAW FCAW	TC-U5-GF	U	U	α = 45°	+10°, −0	+10°, −5°	All	Not Required	1, 4, 7, 8, 10, 11
SAW	TC-U5-S	U	U	R = 0 f = 1/4 max α = 60°	± 0 +0, −3/16 +10°, −0°	+1/16, −0 ±1/16 +10°, −5°	F	—	4, 7, 8, 10, 11

Table 8–2 (continued)
Prequalified Welded Joints Complete-Joint-Penetration Groove Welds

CJP

Single-U-groove weld (6)
Butt joint (B)
Corner joint (C)



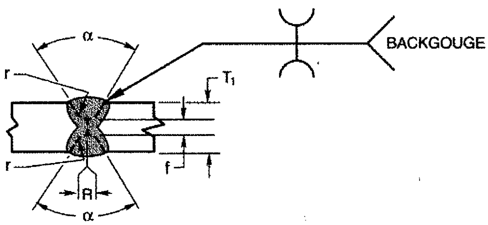
Tolerances	
As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
R = +1/16, −0	+1/16, −1/8
α = +10°, −0°	+10°, −5°
f = ±1/16	Not Limited
r = +1/8, −0	+1/8, −0

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation				Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Groove Angle	Root Face	Bevel Radius			
SMAW	B-U6	U	U	R = 0 to 1/8	α = 45°	f = 1/8	r = 1/4	All	—	4, 5, 10
				R = 0 to 1/8	α = 20°	f = 1/8	r = 1/4	F, OH	—	4, 5, 10
	C-U6	U	U	R = 0 to 1/8	α = 45°	f = 1/8	r = 1/4	All	—	4, 5, 7, 10
				R = 0 to 1/8	α = 20°	f = 1/8	r = 1/4	F, OH	—	4, 5, 7, 10
GMAW FCAW	B-U6-GF	U	U	R = 0 to 1/8	α = 20°	f = 1/8	r = 1/4	All	Not req.	1, 4, 10
	C-U6-GF	U	U	R = 0 to 1/8	α = 20°	f = 1/8	r = 1/4	All	Not req.	1, 4, 7, 10

Table 8–2 (continued)
Prequalified Welded Joints
Complete-Joint-Penetration Groove Welds

CJP

Double-U-groove weld (7)
Butt joint (B)



Tolerances	
As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
For B-U7 and B-U7-GF	
R = +1/16, –0	1/16, –1/8
α = +10°, –0°	+10°, –5°
f = ±1/16, –0	Not Limited
r = +1/4, –0	±1/16
For B-U7-S	
R = ±0	+1/16, –0
f = +0, +1/4	±1/16

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation				Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Groove Angle	Root Face	Bevel Radius			
SMAW	B-U7	U	—	R = 0 to 1/8	α = 45°	f = 1/8	r = 1/4	All	—	4, 5, 8, 10
				R = 0 to 1/8	α = 20°	f = 1/8	r = 1/4	F, OH	—	4, 5, 8, 10
GMAW FCAW	B-U7-GF	U	—	R = 0 to 1/8	α = 20°	f = 1/8	r = 1/4	All	Not req.	1, 4, 10, 8
SAW	B-U7-S	U	—	R = 0	α = 20°	f = 1/4 max	r = 1/4	F	—	4, 8, 10

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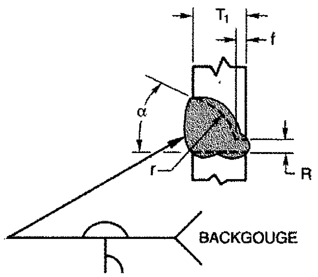
Table 8–2 (continued)
Prequalified Welded Joints Complete-Joint-Penetration Groove Welds

CJP

Single-J-groove weld (8)
Butt joint (B)



Tolerances	
As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
B-U8 and B-U8-GF	
R = +1/16, –0	+1/16, –1/8
α = +10°, –0°	+10°, –5°
f = +1/8, –0	Not Limited
r = +1/4, –0	±1/16
B-U8-S	
R = ±0	+1/4, –0
α = +10°, –0°	+10°, –5°
f = +0, –1/8	±1/16
r = +1/4, –0	±1/16

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation				Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Groove Angle	Root Face	Bevel Radius			
SMAW	B-U8	U	—	R = 0 to 1/8	α = 45°	f = 1/8	r = 3/8	All	—	3, 4, 5, 10
GMAW FCAW	B-U8-GF	U	—	R = 0 to 1/8	α = 30°	f = 1/8	r = 3/8	All	Not req.	1, 3, 4, 10
SAW	B-U8-S	U	U	R = 0	α = 45°	f = 1/4 max	r = 3/8	F	—	3, 4, 10

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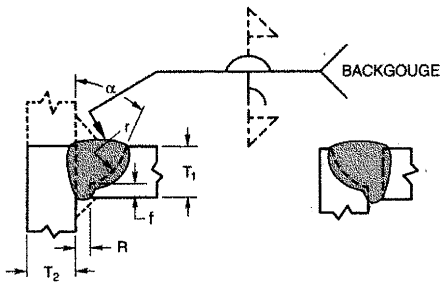
Table 8–2 (continued) Prequalified Welded Joints Complete-Joint-Penetration Groove Welds

CJP

Single-J-groove weld (8)
T-joint (T)
Corner joint (C)



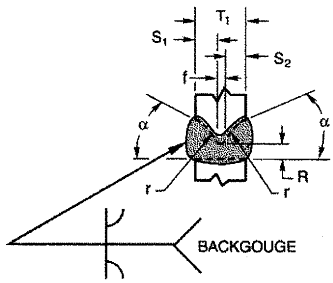
Tolerances	
As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
TC-U8a and TC-U8a-GF	
R = +1/16, −0	1/16, −1/8
α = +10°, −0°	+10°, −5°
f = +1/16, −0	Not Limited
r = +1/4, −0	±1/16
TC-U8a-S	
R = ±0	+1/4, −0
α = +10°, −0°	+10°, −5°
f = +0, −1/8	±1/16
r = +1/4, −0	±1/16

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation				Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Groove Angle	Root Face	Bevel Radius			
SMAW	TC-U8a	U	U	R = 0 to 1/8	α = 45°	f = 1/8	r = 3/8	All	—	4, 5, 7, 10, 11
				R = 0 to 1/8	α = 30°	f = 1/8	r = 3/8	F, OH	—	4, 5, 7, 10, 11
GMAW FCAW	TC-U8a-GF	U	U	R = 0 to 1/8	α = 30°	f = 1/8	r = 3/8	All	Not req.	1, 4, 7, 10, 11
SAW	TC-U8a-S	U	U	R = 0	α = 45°	f = 1/4 max	r = 3/8	F	—	4, 7, 10, 11

Table 8–2 (continued)
Prequalified Welded Joints
Complete-Joint-Penetration Groove Welds

CJP

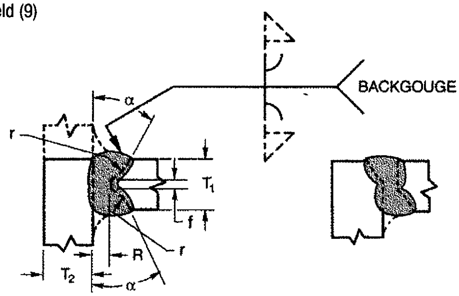
Double-J-groove weld (9)
Butt joint (B)



Tolerances	
As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
R = +1/16, −0	+1/16, −1/8
α = +10°, −0°	+10°, −5°
f = +1/16, −0	Not Limited
r = +1/8, −0	±1/16

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation				Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Groove Angle	Root Face	Bevel Radius			
SMAW	B-U9	U	—	R = 0 to 1/8	α = 45°	f = 1/8	r = 3/8	All	—	3, 4, 5, 8, 10
GMAW FCAW	B-U9-GF	U	—	R = 0 to 1/8	α = 30°	f = 1/8	r = 3/8	All	Not req.	1, 3, 4, 8, 10

Double-J-groove weld (9)
T-joint (T)
Corner joint (C)



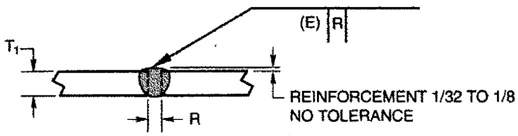
Tolerances	
As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
R = +1/16, −0	+1/16, −1/8
α = +10°, −0°	+10°, −5°
f = +1/16, −0	Not Limited
r = 1/8, −0	±1/16

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation				Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Groove Angle	Root Face	Bevel Radius			
SMAW	TC-U9a	U	U	R = 0 to 1/8	α = 45°	f = 1/8	r = 3/8	All	—	4, 5, 7, 8, 10, 11
				R = 0 to 1/8	α = 30°	f = 1/8	r = 3/8	F, OH	—	4, 5, 7, 8, 11
GMAW FCAW	TC-U9a-GF	U	U	R = 0 to 1/8	α = 30°	f = 1/8	r = 3/8	All	Not req.	1, 4, 7, 8, 10, 11

Table 8–2 (continued)
Prequalified Welded Joints
Partial-Joint-Penetration Groove Welds

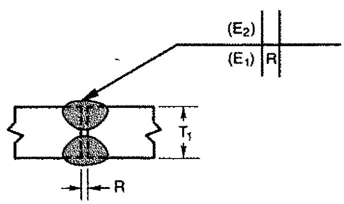
PJP

Square-groove weld (1)
Butt joint (B)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Weld Size (E)	Notes
					Tolerances				
		T_1	T_2	Root Opening	As Detailed (see 3.12.3)	As Fit-Up (see 3.12.3)			
SMAW	B-P1a	$^1/_8$	—	R = 0 to $^1/_{16}$	$+^1/_{16}, -0$	$\pm^1/_{16}$	All	$T_1 - ^1/_{32}$	2, 5
	B-P1c	$^1/_4$ max	—	$R = \frac{T_1}{2}$ min	$+^1/_{16}, -0$	$\pm^1/_{16}$	All	$\frac{T_1}{2}$	2, 5

Square-groove weld (1)
Butt joint (B)



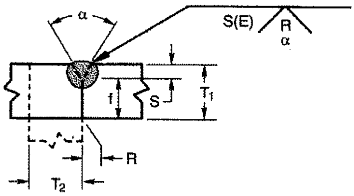
$E_1 + E_2$ must not exceed $\frac{3T_1}{4}$

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Total Weld Size ($E_1 + E_2$)	Notes
					Tolerances				
		T_1	T_2	Root Opening	As Detailed (see 3.12.3)	As Fit-Up (see 3.12.3)			
SMAW	B-P1b	$^1/_4$ max	—	$R = \frac{T_1}{2}$	$+^1/_{16}, -0$	$\pm^1/_{16}$	All	$\frac{3T_1}{4}$	5

Table 8–2 (continued) PJP

Prequalified Welded Joints Partial-Joint-Penetration Groove Welds

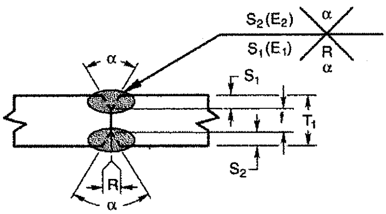
Single-V-groove weld (2)
Butt joint (B)
Corner joint (C)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Weld Size (E)	Notes
		T_1	T_2	Root Opening Root Face Groove Angle	Tolerances: As Detailed (see 3.12.3)	Tolerances: As Fit-Up (see 3.12.3)			
SMAW	BC-P2	1/4 min	U	R = 0 f = 1/32 min α = 60°	−0, +1/16 +U, −0 +10°, −0°	+1/8, −1/16 ±1/16 + 10°, −5°	All	S	2, 5, 6, 10
GMAW FCAW	BC-P2-GF	1/4 min	U	R = 0 f = 1/8 min α = 60°	−0, +1/16 +U, −0 +10°, −0°	+1/8, −1/16 ±1/16 + 10°, −5°	All	S	1, 2, 6, 10
SAW	BC-P2-S	7/16 min	U	R = 0 f = 1/4 min α = 60°	±0 +U, −0 +10°, −0°	+1/16, −0 ±1/16 + 10°, −5°	F	S	2, 6, 10

Table 8–2 (continued) PJP
Prequalified Welded Joints
Partial-Joint-Penetration Groove Welds

Double-V-groove weld (3)
Butt joint (B)



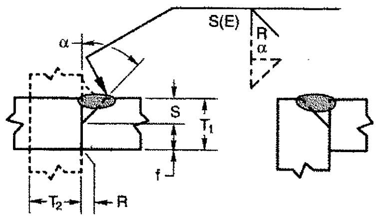
Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Total Weld Size ($E_1 + E_2$)	Notes
				Root Opening Root Face Groove Angle	Tolerances				
		T_1	T_2		As Detailed (see 3.12.3)	As Fit-Up (see 3.12.3)			
SMAW	B-P3	1/2 min	—	R = 0 f = 1/8 min α = 60°	+1/16, −0 +U, −0 +10°, −0°	+1/8, −1/16 ±1/16 + 10°, −5°	All	$S_1 + S_2$	5, 6, 9, 10
GMAW FCAW	B-P3-GF	1/2 min	—	R = 0 f = 1/8 min α = 60°	+1/16, −0 +U, −0 +10°, −0°	+1/8, −1/16 ±1/16 + 10°, −5°	All	$S_1 + S_2$	1, 6, 9, 10
SAW	B-P3-S	3/4 min	—	R = 0 f = 1/4 min α = 60°	±0 +U, −0 +10°, −0°	+1/16, −0 ±1/16 + 10°, −5°	F	$S_1 + S_2$	6, 9, 10

Table 8–2 (continued)

PJP

Prequalified Welded Joints
Partial-Joint-Penetration Groove Welds

Single-bevel-groove weld (4)
Butt joint (B)
T-joint (T)
Corner joint (C)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited) T1	T2	Groove Preparation: Root Opening / Root Face / Groove Angle	Tolerances: As Detailed (see 3.12.3)	Tolerances: As Fit-Up (see 3.12.3)	Allowed Welding Positions	Total Weld Size	Notes
SMAW	BTC-P4	U	U	R = 0 f = 1/8 min α = 45°	+1/16, −0 +U, −0 +10°, −0°	+1/8, −1/16 ±1/16 + 10°, −5°	All	S−1/8	2, 5, 6, 7, 10, 11
GMAW FCAW	BTC-P4-GF	1/4 min	U	R = 0 f = 1/8 min α = 45°	+1/16, −0 +U, −0 +10°, −0°	+1/8, −1/16 ±1/16 + 10°, −5°	F, H V, OH	S S−1/8	1, 2, 6, 7, 10, 11
SAW	TC-P4-S	7/16 min	U	R = 0 f = 1/4 min α = 60°	±0 +U, −0 +10°, −0°	+1/16, −0 ±1/16 + 10°, −5°	F	S	2, 6, 7, 10, 11

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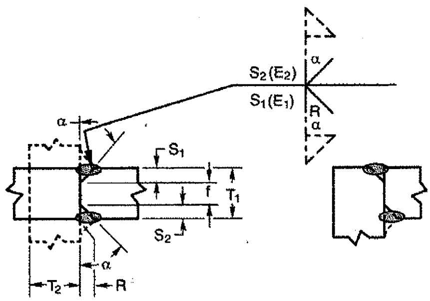
Table 8–2 (continued)
Prequalified Welded Joints
Partial-Joint-Penetration Groove Welds

PJP

Double-bevel-groove weld (5)
Butt joint (B)
T-joint (T)
Corner joint (C)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited) T1	T2	Groove Preparation: Root Opening Root Face Groove Angle	Tolerances: As Detailed (see 3.12.3)	Tolerances: As Fit-Up (see 3.12.3)	Allowed Welding Positions	Total Weld Size ($E_1 + E_2$)	Notes
SMAW	BTC-P5	5/16 min	U	R = 0 f = 1/8 min α = 45°	+1/16, –0 +U, –0 +10°, –0°	+1/8, –1/16 ±1/16 + 10°, –5°	All	$S_1 + S_2$ –1/4	5, 6, 7, 9, 10, 11
GMAW FCAW	BTC-P5-GF	1/2 min	U	R = 0 f = 1/8 min α = 45°	+1/16, –0 +U, –0 +10°, –0°	+1/8, –1/16 ±1/16 + 10°, –5°	F, H	$S_1 + S_2$	1, 6, 7, 9, 10, 11
							V, OH	$S_1 + S_2$ –1/4	
SAW	TC-P5-S	3/4 min	U	R = 0 f = 1/4 min α = 60°	±0 +U, –0 +10°, –0°	+1/16, –0 ±1/16 + 10°, –5°	F	$S_1 + S_2$	6, 7, 9, 10, 11

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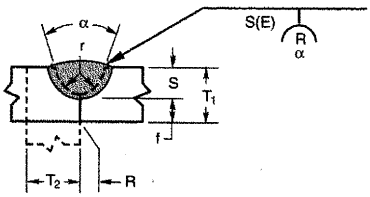
PJP

Table 8–2 (continued)
Prequalified Welded Joints
Partial-Joint-Penetration Groove Welds

Single-U-groove weld (6)
Butt joint (B)
Corner joint (C)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Total Weld Size (E)	Notes
		T_1	T_2	Root Opening Root Face Bevel Radius Groove Angle	Tolerances: As Detailed (see 3.12.3)	Tolerances: As Fit-Up (see 3.12.3)			
SMAW	BC-P6	1/4 min	U	R = 0 f = 1/32 min r = 1/4 α = 45°	+1/16, −0 +U, −0 +1/4, −0 +10°, −0°	+1/8, −1/16 ±1/16 ±1/16 + 10°, −5°	All	S	2, 5, 6, 10
GMAW FCAW	BC-P6-GF	1/4 min	U	R = 0 f = 1/8 min r = 1/4 α = 20°	+1/16, −0 +U, −0 +1/4, −0 +10°, −0°	+1/8, −1/16 ±1/16 ±1/16 + 10°, −5°	All	S	1, 2, 6, 10
SAW	BC-P6-S	7/16 min	U	R = 0 f = 1/4 min r = 1/4 α = 20°	±0 +U, −0 +1/4, −0 +10°, −0°	+1/16, −0° ±1/16 ±1/16 + 10°, −5°	F	S	2, 6, 10

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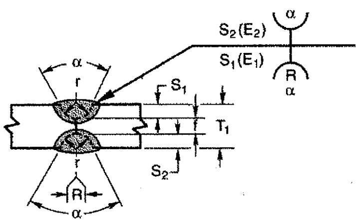
PJP

Table 8–2 (continued)
Prequalified Welded Joints
Partial-Joint-Penetration Groove Welds

Double-U-groove weld (7)
Butt joint (B)

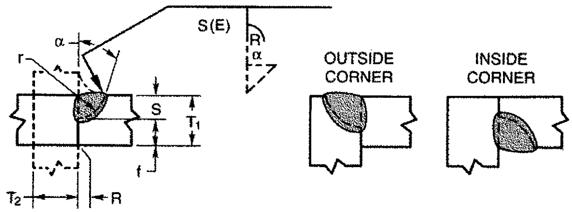


Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Total Weld Size ($E_1 + E_2$)	Notes
		T_1	T_2	Root Opening Root Face Bevel Radius Groove Angle	Tolerances As Detailed (see 3.12.3)	Tolerances As Fit-Up (see 3.12.3)			
SMAW	B-P7	1/2 min	—	R = 0 f = 1/8 min r = 1/4 α = 45°	+1/16, −0 +U, −0 +1/4, −0 +10°, −0°	+1/8, −1/16 ±1/16 ±1/16 + 10°, −5°	All	$S_1 + S_2$	5, 6, 9, 10
GMAW FCAW	B-P7-GF	1/2 min	—	R = 0 f = 1/8 min r = 1/4 α = 20°	+1/16, −0 +U, −0 +1/4, −0 +10°, −0°	+1/8, −1/16 ±1/16 ±1/16 + 10°, −5°	All	$S_1 + S_2$	1, 6, 9, 10
SAW	B-P7-S	3/4 min	—	R = 0 f = 1/4 min r = 1/4 α = 20°	±0 +U, −0 +1/4, −0 +10°, −0°	+1/16, −0° ±1/16 ±1/16 + 10°, −5°	F	$S_1 + S_2$	6, 9,10

Table 8–2 (continued)
Prequalified Welded Joints
Partial-Joint-Penetration Groove Welds

PJP

Single-J-groove weld (8)
Butt joint (B)
T-joint (T)
Corner joint (C)



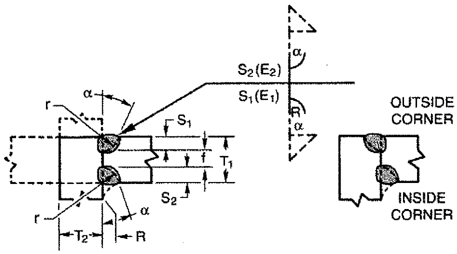
*α_{oc} = Outside corner groove angle.
**α_{ic} = Inside corner groove angle.

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Total Weld Size (E)	Notes
		T_1	T_2	Root Opening Root Face Bevel Radius Groove Angle	Tolerances: As Detailed (see 3.12.3)	Tolerances: As Fit-Up (see 3.12.3)			
SMAW	B-P8	1/4 min	U	R = 0 f = 1/8 min r = 3/8 α = 30°	+1/16, −0 +U, −0 +1/4, −0 +10°, −0°	+1/8, −1/16 ±1/16 ±1/16 +10°, −5°	All	S	5, 6, 7, 10, 11
SMAW	TC-P8	1/4 min	U	R = 0 f = 1/8 min r = 3/8 α_{oc} = 30°* α_{ic} = 45°**	+1/16, −0 +U, −0 +1/4, −0 +10°, −0° +10°, −0°	+1/8, −1/16 ±1/16 ±1/16 +10°, −5° +10°, −5°	All	S	5, 6, 7, 10, 11
GMAW FCAW	B-P8-GF	1/4 min	U	R = 0 f = 1/8 min r = 3/8 α = 30°	+1/16, −0 +U, −0 +1/4, −0 +10°, −0°	+1/8, −1/16 ±1/16 ±1/16 +10°, −5°	All	S	1, 6, 7, 10, 11
GMAW FCAW	TC-P8-GF	1/4 min	U	R = 0 f = 1/8 min r = 3/8 α_{oc} = 30°* α_{ic} = 45°**	+1/16, −0 +U, −0 +1/4, −0 +10°, −0° +10°, −0°	+1/8, −1/16 ±1/16 ±1/16 +10°, −5° +10°, −5°	All	S	1, 6, 7, 10, 11
SAW	B-P8-S	7/16 min	U	R = 0 f = 1/4 min r = 1/2 α = 20°	±0 +U, −0 +1/4, −0 +10°, −0°	+1/16, −0 ±1/16 ±1/16 +10°, −5°	F	S	6, 7, 10, 11
SAW	TC-P8-S	7/16 min	U	R = 0 f = 1/4 min r = 1/2 α_{oc} = 20°* α_{ic} = 45°**	±0 +U, −0 +1/4, −0 +10°, −0° +10°, −0°	+1/16, −0 ±1/16 ±1/16 +10°, −5° +10°, −5°	F	S	6, 7, 10, 11

PJP

Table 8–2 (continued)
Prequalified Welded Joints
Partial-Joint-Penetration Groove Welds

Double-J-groove weld (9)
Butt joint (B)
T-joint (T)
Corner joint (C)



$^{*}\alpha_{oc}$ = Outside corner groove angle.
$^{**}\alpha_{ic}$ = Inside corner groove angle.

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited) T1	T2	Groove Preparation: Root Opening / Root Face / Bevel Radius / Groove Angle	Tolerances: As Detailed (see 3.12.3)	Tolerances: As Fit-Up (see 3.12.3)	Allowed Welding Positions	Total Weld Size ($E_1 + E_2$)	Notes
SMAW	B-P9	1/2 min	U	R = 0 f = 1/8 min r = 3/8 α = 30°	+1/16, –0 +U, –0 +1/4, –0 +10°, –0°	+1/8, –1/16 ±1/16 ±1/16 +10°, –5°	All	$S_1 + S_2$	5, 6, 7, 9, 10, 11
SMAW	TC-P9	1/2 min	U	R = 0 f = 1/8 min r = 3/8 α_{oc} = 30°* α_{ic} = 45°**	+1/16, –0 +U, –0 +1/4, –0 +10°, –0° +10°, –0°	+1/8, –1/16 ±1/16 ±1/16 +10°, –5° +10°, –5°	All	$S_1 + S_2$	5, 6, 7, 9, 10, 11
GMAW FCAW	B-P9-GF	1/2 min	U	R = 0 f = 1/8 min r = 3/8 α = 30°	+1/16, –0 +U, –0 +1/4, –0 +10°, –0°	+1/8, –1/16 ±1/16 ±1/16 +10°, –5°	All	$S_1 + S_2$	1, 6, 7, 9, 10, 11
GMAW FCAW	TC-P9-GF	1/2 min	U	R = 0 f = 1/8 min r = 3/8 α_{oc} = 30°* α_{ic} = 45°**	±0 +U, –0 +1/4, –0 +10°, –0° +10°, –0°	+1/16, –0 ±1/16 ±1/16 +10°, –5° +10°, –5°	All	$S_1 + S_2$	1, 6, 7, 9, 10, 11
SAW	B-P9-S	3/4 min	U	R = 0 f = 1/4 min r = 1/2 α = 20°	±0 +U, –0 +1/4, –0 +10°, –0°	+1/16, –0 ±1/16 ±1/16 +10°, –5°	F	$S_1 + S_2$	6, 7, 9, 10, 11
SAW	TC-P9-S	3/4 min	U	R = 0 f = 1/4 min r = 1/2 α_{oc} = 20°* α_{ic} = 45°**	±0 +U, –0 +1/4, –0 +10°, –0° +10°, –0°	+1/16, –0 ±1/16 ±1/16 +10°, –5° +10°, –5°	F	$S_1 + S_2$	6, 7, 9, 10, 11

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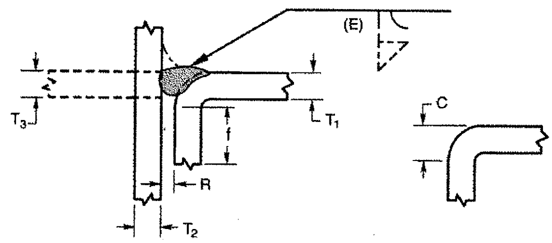
FLARE

Table 8–2 (continued)
Prequalified Welded Joints
Flare-Bevel Groove Welds

Flare-bevel-groove weld (10)
Butt joint (B)
T-joint (T)
Corner joint (C)

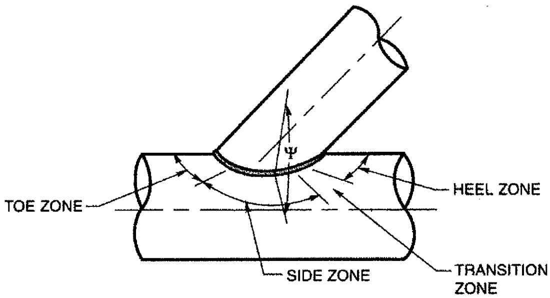


Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)			Groove Preparation			Allowed Welding Positions	Total Weld Size (E)	Notes
		T_1	T_2	T_3	Root Opening Root Face Bend Radius*	Tolerances: As Detailed (see 3.12.3)	Tolerances: As Fit-Up (see 3.12.3)			
SMAW	BTC-P10	3/16 min	U	T_1 min	R = 0 f = 3/16 min $C = \frac{3T_1}{2}$ min	+1/16, –0 +U, –0 +U, –0	+1/8, –1/16 +U, –1/16 +U, –0	All	$\frac{5T_1}{8}$	5, 7, 10, 12
GMAW FCAW	BTC-P10-GF	3/16 min	U	T_1 min	R = 0 f = 3/16 min $C = \frac{3T_1}{2}$ min	+1/16, –0 +U, –0 +U, –0	+1/8, –1/16 +U, –1/16 +U, –0	All	$\frac{5T_1}{8}$	1, 7, 10, 12
SAW	T-P10-S	1/2 min	1/2 min	N/A	R = 0 f = 1/2 min $C = \frac{3T_1}{2}$ min	±0 +U, –0 +U, –0	+1/16, –0° +U, –1/16 +U, –0	F	$\frac{5T_1}{8}$	7, 10, 12

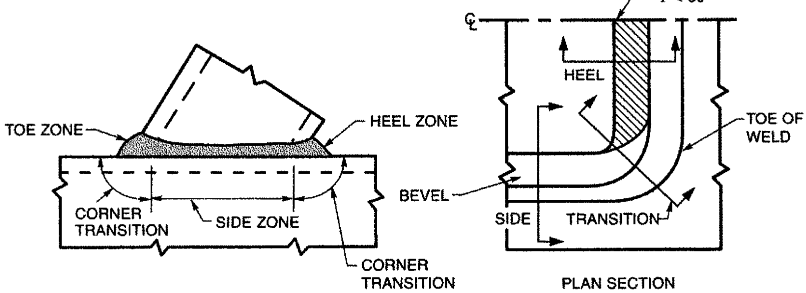
* For cold formed (A500) rectangular tubes, C dimension is not limited. See the following:
Effective Weld Size of Flare-Bevel-Groove Welded Joints. Tests have been performed on cold formed ASTM A 500 material exhibiting a "C" dimension as small as T1 with a nominal radius of 2t. As the radius increases, the "C" dimension also increases. The corner curvature may not be a quadrant of a circle tangent to the sides. The corner dimension, "C," may be less than the radius of the corner.

TUBE

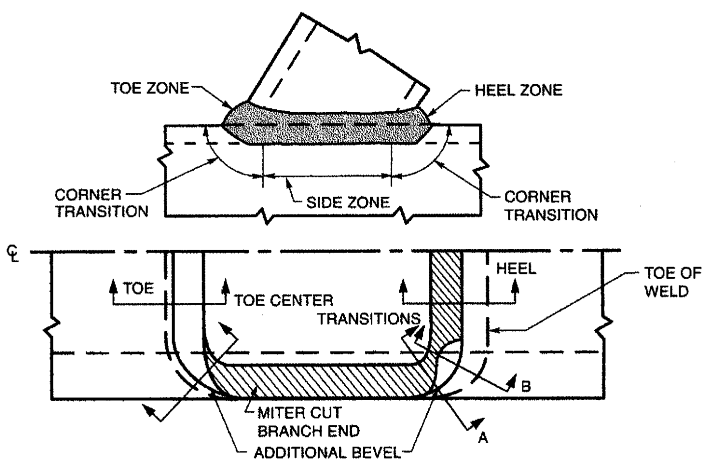
Table 8–2 (continued)
Prequalified Welded Joints
PJP T-, Y-, and K-Tubular Connections



(A) CIRCULAR CONNECTION



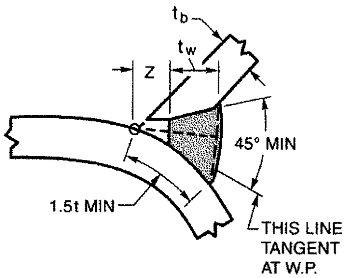
(B) STEPPED BOX CONNECTION



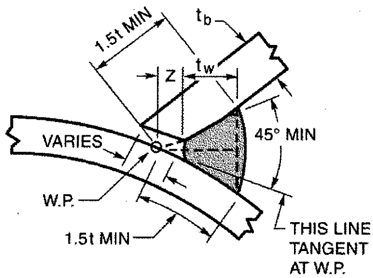
(C) MATCHED BOX CONNECTION

TUBE

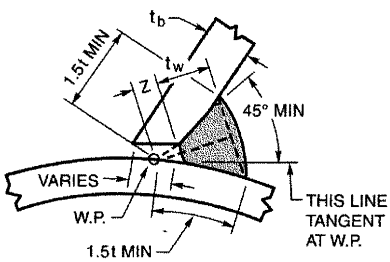
Table 8–2 (continued)
Prequalified Welded Joints
PJP T-, Y-, and K-Tubular Connections



TRANSITION A

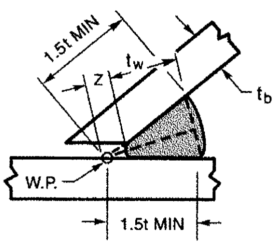


TRANSITION B



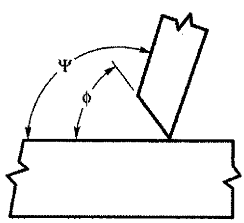
$\Psi = 75°–60°$

TRANSITION OR HEEL



$\Psi = 60°–30°$

HEEL



SKETCH FOR ANGULAR DEFINITION

$150° \geq \Psi \geq 30°$
$90° > \phi \geq 30°$

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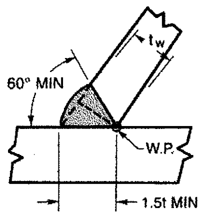
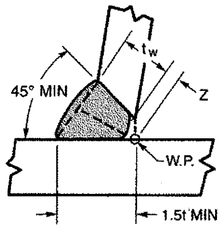
TUBE

Table 8–2 (continued)
Prequalified Welded Joints
PJP T-, Y-, and K-Tubular Connections



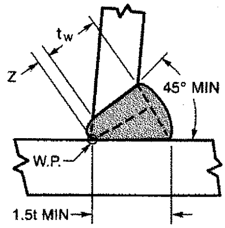
Ψ = 150°–105°

TOE



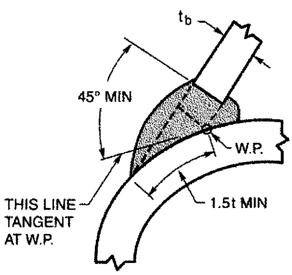
Ψ = 105°–90°

TOE OR HEEL

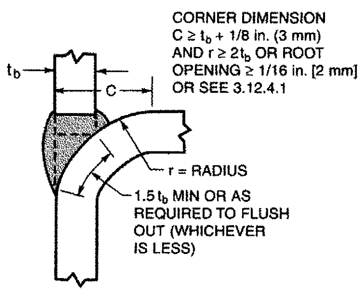


Ψ = 90°–75°

SIDE OR HEEL



TOE CORNER



SIDE MATCHED

General Notes:

- t = thickness of thinner section.
- Bevel to feather edge except in transition and heel zones.
- Root opening: 0 to 3/16 in. [5 mm].
- Not prequalified for under 30°.
- Weld size (effective throat) $t_w \geq t$; Z Loss Dimensions shown in Table 2.8.
- Calculations per 2.24.1.3 shall be done for leg length less than 1.5t, as shown.
- For Box Section, joint preparation for corner transitions shall provide a smooth transition from one detail to another. Welding shall be carried continuously around corners, with corners fully built up and all weld starts and stops within flat faces.
- See Annex B for definition of local dihedral angle, Ψ.
- W.P. = work point.

Table 8–3
Electrode Strength Coefficient, C_1

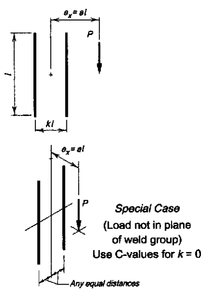
Electrode	F_{EXX} (ksi)	*C1*
E60	60	0.857
E70	70	1.00
E80	80	1.03
E90	90	1.16
E100	100	1.21
E110	110	1.34

Table 8–4
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 0°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



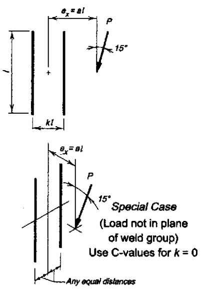
	k															
a	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71
0.100	3.72	3.73	3.72	3.71	3.70	3.69	3.67	3.65	3.63	3.61	3.60	3.56	3.52	3.48	3.45	3.71
0.150	3.67	3.66	3.65	3.64	3.62	3.60	3.58	3.56	3.54	3.52	3.50	3.47	3.43	3.40	3.37	3.34
0.200	3.51	3.51	3.50	3.49	3.47	3.46	3.44	3.43	3.41	3.40	3.38	3.36	3.33	3.30	3.28	3.25
0.250	3.31	3.31	3.31	3.30	3.29	3.28	3.28	3.27	3.26	3.26	3.25	3.23	3.22	3.20	3.18	3.17
0.300	3.09	3.09	3.09	3.10	3.10	3.10	3.11	3.11	3.11	3.11	3.11	3.11	3.11	3.10	3.09	3.08
0.400	2.66	2.66	2.68	2.70	2.73	2.75	2.78	2.80	2.82	2.83	2.85	2.87	2.89	2.90	2.90	2.90
0.500	2.29	2.30	2.32	2.35	2.40	2.44	2.48	2.52	2.55	2.58	2.61	2.65	2.68	2.71	2.73	2.74
0.600	2.00	2.00	2.03	2.07	2.12	2.18	2.23	2.28	2.32	2.36	2.39	2.45	2.50	2.54	2.57	2.59
0.700	1.76	1.76	1.79	1.84	1.90	1.96	2.02	2.07	2.12	2.16	2.21	2.28	2.33	2.38	2.42	2.45
0.800	1.56	1.57	1.60	1.65	1.71	1.77	1.84	1.90	1.95	2.00	2.04	2.12	2.19	2.24	2.29	2.32
0.900	1.41	1.41	1.44	1.49	1.56	1.62	1.69	1.75	1.80	1.85	1.90	1.98	2.05	2.11	2.16	2.21
1.00	1.28	1.28	1.31	1.37	1.43	1.49	1.56	1.62	1.67	1.73	1.77	1.86	1.94	2.00	2.05	2.10
1.20	1.07	1.08	1.11	1.16	1.22	1.28	1.35	1.41	1.46	1.51	1.57	1.66	1.73	1.80	1.86	1.91
1.40	0.927	0.935	0.965	1.01	1.07	1.13	1.19	1.24	1.30	1.35	1.40	1.49	1.57	1.64	1.70	1.76
1.60	0.815	0.821	0.851	0.893	0.944	1.00	1.06	1.11	1.16	1.21	1.26	1.35	1.43	1.50	1.57	1.62
1.80	0.725	0.733	0.760	0.800	0.847	0.899	0.952	1.00	1.05	1.10	1.15	1.24	1.32	1.39	1.45	1.51
2.00	0.655	0.661	0.687	0.723	0.768	0.816	0.867	0.916	0.964	1.01	1.06	1.14	1.22	1.29	1.35	1.41
2.20	0.596	0.603	0.627	0.660	0.701	0.747	0.795	0.841	0.887	0.932	0.975	1.06	1.13	1.20	1.26	1.32
2.40	0.547	0.553	0.575	0.607	0.645	0.688	0.733	0.777	0.821	0.864	0.905	0.984	1.06	1.12	1.18	1.24
2.60	0.505	0.512	0.532	0.561	0.597	0.637	0.680	0.723	0.764	0.805	0.845	0.921	0.991	1.05	1.11	1.17
2.80	0.469	0.476	0.495	0.523	0.556	0.595	0.635	0.675	0.715	0.753	0.792	0.865	0.932	0.995	1.05	1.11
3.00	0.439	0.444	0.463	0.488	0.520	0.556	0.595	0.632	0.671	0.708	0.745	0.815	0.881	0.941	0.997	1.05

Table 8–4 (continued)
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 15°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



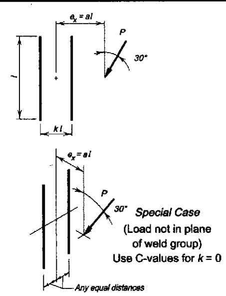
	k															
a	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84
0.100	3.79	3.79	3.78	3.77	3.77	3.76	3.75	3.74	3.73	3.72	3.71	3.69	3.67	3.65	3.64	3.62
0.150	3.68	3.68	3.67	3.66	3.65	3.64	3.63	3.62	3.61	3.61	3.60	3.58	3.57	3.55	3.54	3.53
0.200	3.51	3.50	3.50	3.49	3.49	3.49	3.49	3.48	3.48	3.47	3.47	3.46	3.46	3.45	3.44	3.44
0.250	3.31	3.31	3.31	3.31	3.31	3.31	3.32	3.32	3.33	3.33	3.33	3.34	3.34	3.34	3.34	3.34
0.300	3.09	3.09	3.10	3.11	3.12	3.14	3.15	3.16	3.17	3.18	3.19	3.21	3.23	3.23	3.24	3.25
0.400	2.67	2.67	2.69	2.71	2.75	2.79	2.82	2.85	2.88	2.90	2.93	2.96	3.00	3.02	3.04	3.06
0.500	2.32	2.32	2.34	2.38	2.42	2.47	2.52	2.57	2.61	2.65	2.68	2.74	2.79	2.83	2.86	2.89
0.600	2.02	2.03	2.06	2.10	2.15	2.21	2.27	2.33	2.38	2.42	2.46	2.54	2.60	2.65	2.69	2.73
0.700	1.79	1.79	1.82	1.87	1.93	1.99	2.06	2.12	2.17	2.23	2.27	2.36	2.43	2.49	2.54	2.58
0.800	1.59	1.60	1.63	1.68	1.74	1.81	1.88	1.94	2.00	2.06	2.11	2.20	2.27	2.34	2.40	2.45
0.900	1.44	1.44	1.48	1.53	1.59	1.66	1.73	1.79	1.85	1.91	1.96	2.05	2.14	2.21	2.27	2.32
1.00	1.31	1.31	1.35	1.40	1.46	1.53	1.59	1.66	1.72	1.78	1.83	1.93	2.01	2.09	2.15	2.21
1.20	1.10	1.11	1.14	1.19	1.25	1.32	1.38	1.45	1.51	1.56	1.62	1.72	1.81	1.88	1.95	2.01
1.40	0.952	0.959	0.991	1.04	1.09	1.15	1.22	1.28	1.34	1.39	1.45	1.54	1.63	1.71	1.78	1.85
1.60	0.837	0.844	0.875	0.917	0.969	1.03	1.09	1.15	1.20	1.25	1.31	1.40	1.49	1.57	1.64	1.70
1.80	0.747	0.755	0.783	0.823	0.871	0.924	0.980	1.04	1.09	1.14	1.19	1.28	1.37	1.45	1.52	1.58
2.00	0.675	0.681	0.707	0.744	0.789	0.840	0.892	0.944	0.995	1.04	1.09	1.18	1.27	1.34	1.41	1.47
2.20	0.615	0.621	0.645	0.680	0.721	0.769	0.817	0.867	0.916	0.963	1.01	1.10	1.18	1.25	1.32	1.38
2.40	0.564	0.571	0.593	0.625	0.665	0.709	0.755	0.801	0.848	0.893	0.937	1.02	1.10	1.17	1.24	1.30
2.60	0.521	0.528	0.549	0.579	0.616	0.657	0.701	0.745	0.788	0.832	0.875	0.955	1.03	1.10	1.16	1.22
2.80	0.484	0.491	0.511	0.539	0.573	0.613	0.655	0.696	0.737	0.779	0.819	0.896	0.969	1.04	1.10	1.16
3.00	0.452	0.459	0.477	0.504	0.537	0.573	0.613	0.652	0.692	0.732	0.771	0.845	0.915	0.981	1.04	1.10

Table 8–4 (continued)
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 30°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18
0.100	4.04	4.05	4.05	4.05	4.06	4.07	4.07	4.08	4.08	4.09	4.09	4.09	4.09	4.08	4.08	4.07
0.150	3.83	3.83	3.83	3.83	3.84	3.84	3.84	3.85	3.86	3.87	3.88	3.89	3.91	3.92	3.93	3.93
0.200	3.63	3.63	3.64	3.65	3.65	3.66	3.67	3.68	3.68	3.70	3.71	3.73	3.74	3.76	3.78	3.79
0.250	3.43	3.43	3.43	3.44	3.46	3.48	3.49	3.51	3.52	3.54	3.56	3.58	3.60	3.62	3.64	3.66
0.300	3.21	3.21	3.22	3.24	3.27	3.29	3.32	3.35	3.37	3.39	3.41	3.45	3.48	3.50	3.52	3.54
0.400	2.81	2.81	2.83	2.86	2.90	2.94	2.99	3.03	3.07	3.10	3.14	3.19	3.24	3.28	3.31	3.34
0.500	2.46	2.46	2.48	2.52	2.58	2.63	2.69	2.75	2.80	2.85	2.89	2.96	3.02	3.08	3.12	3.16
0.600	2.17	2.17	2.20	2.24	2.30	2.37	2.44	2.50	2.56	2.61	2.66	2.75	2.83	2.89	2.95	2.99
0.700	1.93	1.93	1.96	2.01	2.08	2.15	2.22	2.29	2.35	2.41	2.47	2.57	2.65	2.72	2.78	2.84
0.800	1.73	1.73	1.77	1.82	1.89	1.96	2.03	2.10	2.17	2.24	2.30	2.40	2.49	2.57	2.64	2.70
0.900	1.56	1.57	1.60	1.66	1.73	1.80	1.87	1.95	2.02	2.08	2.14	2.25	2.34	2.43	2.50	2.57
1.00	1.43	1.43	1.47	1.52	1.59	1.66	1.74	1.81	1.88	1.95	2.01	2.12	2.22	2.30	2.38	2.45
1.20	1.21	1.22	1.25	1.31	1.37	1.44	1.51	1.59	1.65	1.72	1.78	1.89	1.99	2.08	2.16	2.23
1.40	1.05	1.06	1.09	1.14	1.20	1.27	1.34	1.41	1.47	1.53	1.59	1.70	1.81	1.90	1.98	2.05
1.60	0.924	0.932	0.964	1.01	1.07	1.13	1.20	1.26	1.32	1.39	1.44	1.55	1.65	1.74	1.82	1.90
1.80	0.825	0.833	0.864	0.908	0.961	1.02	1.08	1.14	1.20	1.26	1.32	1.42	1.52	1.61	1.69	1.76
2.00	0.745	0.753	0.783	0.823	0.873	0.928	0.987	1.04	1.10	1.15	1.21	1.31	1.41	1.49	1.57	1.65
2.20	0.680	0.688	0.715	0.752	0.799	0.851	0.905	0.960	1.01	1.07	1.12	1.22	1.31	1.39	1.47	1.54
2.40	0.625	0.632	0.657	0.693	0.736	0.785	0.837	0.889	0.940	0.991	1.04	1.13	1.22	1.30	1.38	1.45
2.60	0.577	0.585	0.608	0.641	0.683	0.728	0.777	0.827	0.876	0.924	0.971	1.06	1.15	1.23	1.30	1.37
2.80	0.537	0.544	0.565	0.597	0.636	0.679	0.725	0.772	0.819	0.865	0.911	0.997	1.08	1.16	1.23	1.30
3.00	0.501	0.509	0.529	0.559	0.595	0.636	0.680	0.724	0.769	0.813	0.856	0.940	1.02	1.09	1.16	1.23

Table 8-4 (continued)
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 45°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with

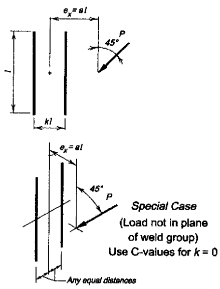
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where

P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



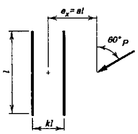
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64
0.100	4.49	4.49	4.50	4.51	4.53	4.55	4.57	4.59	4.61	4.62	4.64	4.66	4.68	4.69	4.69	4.70
0.150	4.18	4.18	4.20	4.23	4.26	4.30	4.34	4.37	4.40	4.43	4.46	4.51	4.55	4.57	4.60	4.62
0.200	3.92	3.92	3.93	3.96	3.99	4.03	4.08	4.13	4.18	4.22	4.26	4.33	4.39	4.44	4.47	4.51
0.250	3.69	3.70	3.71	3.74	3.77	3.81	3.86	3.91	3.95	4.01	4.06	4.14	4.22	4.28	4.33	4.38
0.300	3.48	3.48	3.50	3.53	3.57	3.62	3.67	3.72	3.77	3.81	3.86	3.96	4.05	4.12	4.18	4.24
0.400	3.09	3.09	3.12	3.16	3.21	3.27	3.33	3.39	3.45	3.50	3.55	3.64	3.73	3.82	3.90	3.97
0.500	2.75	2.75	2.78	2.83	2.89	2.96	3.03	3.10	3.17	3.23	3.29	3.39	3.48	3.57	3.65	3.72
0.600	2.46	2.46	2.49	2.55	2.62	2.69	2.77	2.85	2.92	3.00	3.06	3.17	3.27	3.36	3.43	3.51
0.700	2.21	2.22	2.25	2.31	2.38	2.46	2.55	2.63	2.71	2.78	2.85	2.98	3.08	3.17	3.26	3.33
0.800	2.00	2.01	2.05	2.11	2.18	2.27	2.35	2.44	2.52	2.60	2.67	2.80	2.91	3.01	3.09	3.17
0.900	1.83	1.83	1.87	1.93	2.01	2.09	2.18	2.27	2.35	2.43	2.50	2.64	2.75	2.85	2.95	3.03
1.00	1.68	1.69	1.72	1.79	1.86	1.95	2.03	2.12	2.20	2.28	2.36	2.49	2.61	2.72	2.81	2.89
1.20	1.43	1.44	1.48	1.55	1.62	1.70	1.79	1.87	1.95	2.03	2.11	2.24	2.36	2.47	2.57	2.66
1.40	1.25	1.26	1.30	1.36	1.43	1.51	1.59	1.67	1.75	1.83	1.90	2.03	2.15	2.26	2.36	2.45
1.60	1.11	1.12	1.15	1.21	1.28	1.35	1.43	1.51	1.58	1.66	1.72	1.86	1.98	2.09	2.19	2.28
1.80	0.993	1.00	1.04	1.09	1.15	1.22	1.30	1.37	1.44	1.51	1.58	1.71	1.83	1.93	2.03	2.12
2.00	0.900	0.909	0.943	0.991	1.05	1.11	1.18	1.25	1.32	1.39	1.46	1.58	1.69	1.80	1.90	1.99
2.20	0.823	0.832	0.863	0.908	0.963	1.02	1.09	1.16	1.22	1.29	1.35	1.47	1.58	1.68	1.78	1.87
2.40	0.757	0.765	0.795	0.837	0.889	0.947	1.01	1.07	1.13	1.20	1.25	1.37	1.48	1.58	1.67	1.76
2.60	0.701	0.709	0.737	0.777	0.825	0.880	0.937	0.999	1.06	1.12	1.17	1.28	1.39	1.49	1.58	1.66
2.80	0.652	0.661	0.687	0.725	0.771	0.823	0.877	0.935	0.991	1.05	1.10	1.21	1.31	1.40	1.49	1.58
3.00	0.611	0.619	0.643	0.679	0.723	0.772	0.824	0.877	0.932	0.985	1.04	1.14	1.24	1.33	1.42	1.50

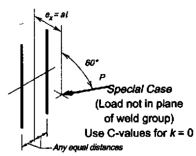
Table 8–4 (continued)
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 60°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x / l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)





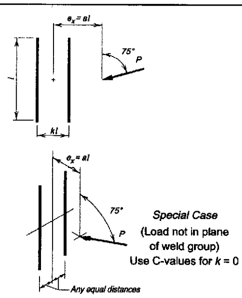
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	5.11	5.11	5.11	5.11	5.11	5.11	5.11	5.11	5.11	5.11	5.11	5.11	5.11	5.11	5.11	5.11
0.100	4.87	4.87	4.90	4.94	4.99	5.03	5.07	5.10	5.12	5.13	5.14	5.15	5.15	5.15	5.15	5.15
0.150	4.61	4.62	4.65	4.71	4.77	4.84	4.91	4.97	5.01	5.05	5.07	5.11	5.12	5.13	5.14	5.14
0.200	4.36	4.37	4.41	4.46	4.54	4.63	4.71	4.79	4.87	4.92	4.97	5.03	5.07	5.09	5.11	5.12
0.250	4.13	4.13	4.17	4.23	4.31	4.40	4.51	4.61	4.70	4.78	4.84	4.94	5.00	5.04	5.07	5.09
0.300	3.93	3.94	3.97	4.02	4.10	4.19	4.30	4.41	4.52	4.62	4.70	4.83	4.91	4.97	5.02	5.04
0.400	3.57	3.58	3.62	3.67	3.75	3.83	3.93	4.03	4.15	4.27	4.39	4.57	4.71	4.81	4.88	4.93
0.500	3.26	3.27	3.31	3.37	3.45	3.54	3.64	3.74	3.83	3.95	4.07	4.29	4.47	4.61	4.72	4.79
0.600	2.98	2.99	3.03	3.10	3.19	3.28	3.38	3.49	3.58	3.69	3.78	4.02	4.22	4.39	4.53	4.63
0.700	2.73	2.74	2.79	2.86	2.95	3.05	3.16	3.26	3.37	3.46	3.57	3.76	3.97	4.16	4.32	4.45
0.800	2.52	2.53	2.57	2.65	2.74	2.85	2.95	3.06	3.17	3.27	3.37	3.55	3.74	3.94	4.11	4.26
0.900	2.33	2.34	2.39	2.47	2.56	2.67	2.77	2.88	2.99	3.09	3.19	3.37	3.54	3.73	3.91	4.07
1.00	2.17	2.18	2.23	2.30	2.40	2.50	2.61	2.72	2.82	2.93	3.03	3.21	3.37	3.54	3.72	3.88
1.20	1.89	1.90	1.95	2.03	2.12	2.22	2.33	2.44	2.54	2.65	2.74	2.93	3.09	3.24	3.39	3.54
1.40	1.67	1.68	1.73	1.81	1.89	1.99	2.10	2.20	2.31	2.41	2.50	2.68	2.85	2.99	3.13	3.27
1.60	1.49	1.51	1.55	1.63	1.71	1.80	1.91	2.01	2.11	2.20	2.30	2.47	2.63	2.78	2.92	3.05
1.80	1.35	1.36	1.41	1.47	1.56	1.65	1.74	1.84	1.93	2.03	2.12	2.29	2.45	2.60	2.73	2.86
2.00	1.23	1.24	1.29	1.35	1.42	1.51	1.60	1.69	1.79	1.88	1.97	2.13	2.29	2.43	2.56	2.69
2.20	1.13	1.14	1.18	1.24	1.31	1.39	1.48	1.57	1.66	1.75	1.83	1.99	2.14	2.28	2.41	2.54
2.40	1.04	1.05	1.09	1.15	1.22	1.29	1.38	1.46	1.55	1.63	1.71	1.87	2.01	2.15	2.28	2.40
2.60	0.968	0.979	1.02	1.07	1.14	1.21	1.29	1.37	1.45	1.53	1.61	1.76	1.90	2.03	2.16	2.28
2.80	0.903	0.915	0.949	1.00	1.06	1.13	1.21	1.28	1.36	1.44	1.51	1.66	1.80	1.93	2.05	2.17
3.00	0.847	0.857	0.891	0.939	0.999	1.07	1.14	1.21	1.28	1.36	1.43	1.57	1.70	1.83	1.95	2.06

Table 8–4 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 75°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



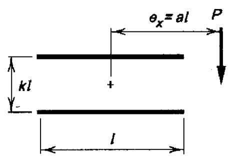
a	*k*: 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44
0.100	5.17	5.20	5.26	5.33	5.39	5.42	5.44	5.45	5.46	5.46	5.46	5.46	5.46	5.45	5.45	5.45
0.150	5.01	5.03	5.10	5.20	5.28	5.35	5.39	5.42	5.43	5.44	5.45	5.45	5.45	5.45	5.45	5.45
0.200	4.85	4.87	4.95	5.06	5.17	5.26	5.32	5.37	5.39	5.42	5.43	5.44	5.45	5.45	5.45	5.45
0.250	4.71	4.72	4.80	4.92	5.04	5.15	5.24	5.30	5.35	5.38	5.40	5.42	5.44	5.44	5.45	5.45
0.300	4.57	4.59	4.65	4.78	4.92	5.04	5.15	5.23	5.29	5.33	5.36	5.40	5.42	5.43	5.44	5.44
0.400	4.32	4.33	4.39	4.51	4.67	4.82	4.96	5.07	5.15	5.22	5.27	5.34	5.38	5.40	5.42	5.43
0.500	4.09	4.11	4.17	4.27	4.43	4.60	4.76	4.89	5.00	5.09	5.16	5.26	5.32	5.36	5.38	5.40
0.600	3.88	3.90	3.96	4.06	4.21	4.38	4.56	4.71	4.85	4.95	5.04	5.17	5.25	5.31	5.34	5.37
0.700	3.69	3.70	3.76	3.87	4.01	4.18	4.36	4.53	4.68	4.81	4.91	5.07	5.17	5.24	5.29	5.33
0.800	3.50	3.52	3.59	3.69	3.83	3.99	4.17	4.35	4.51	4.66	4.78	4.96	5.08	5.17	5.24	5.28
0.900	3.34	3.35	3.42	3.53	3.66	3.81	3.99	4.18	4.35	4.50	4.64	4.85	4.99	5.10	5.18	5.23
1.00	3.18	3.20	3.26	3.37	3.50	3.65	3.83	4.01	4.19	4.35	4.50	4.73	4.90	5.02	5.11	5.18
1.20	2.90	2.91	2.98	3.09	3.22	3.36	3.52	3.70	3.88	4.06	4.22	4.49	4.70	4.85	4.97	5.06
1.40	2.65	2.67	2.74	2.84	2.97	3.11	3.27	3.43	3.61	3.78	3.95	4.25	4.49	4.67	4.81	4.93
1.60	2.44	2.46	2.53	2.63	2.75	2.89	3.04	3.19	3.35	3.53	3.70	4.01	4.27	4.48	4.65	4.78
1.80	2.25	2.27	2.34	2.44	2.56	2.69	2.83	2.98	3.14	3.30	3.47	3.79	4.06	4.29	4.48	4.63
2.00	2.09	2.11	2.18	2.27	2.39	2.51	2.65	2.80	2.95	3.10	3.26	3.58	3.86	4.10	4.31	4.48
2.20	1.95	1.97	2.03	2.12	2.23	2.36	2.49	2.63	2.77	2.92	3.07	3.38	3.66	3.92	4.14	4.32
2.40	1.82	1.84	1.90	1.99	2.10	2.22	2.35	2.48	2.62	2.76	2.90	3.20	3.48	3.74	3.97	4.17
2.60	1.71	1.73	1.79	1.87	1.98	2.09	2.22	2.35	2.48	2.62	2.75	3.04	3.31	3.57	3.80	4.01
2.80	1.61	1.63	1.69	1.77	1.87	1.98	2.10	2.23	2.35	2.49	2.61	2.88	3.16	3.41	3.65	3.86
3.00	1.52	1.54	1.59	1.67	1.77	1.88	1.99	2.12	2.24	2.36	2.49	2.75	3.01	3.26	3.49	3.71

Table 8–5
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 0°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x / l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



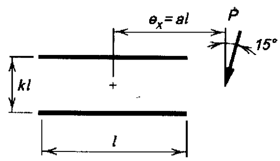
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57
0.100	4.31	4.36	4.48	4.65	4.82	4.98	5.11	5.21	5.29	5.35	5.40	5.45	5.49	5.51	5.52	5.53
0.150	3.90	3.94	4.04	4.20	4.39	4.58	4.75	4.90	5.02	5.12	5.20	5.31	5.38	5.43	5.46	5.48
0.200	3.53	3.57	3.66	3.80	3.99	4.20	4.40	4.57	4.73	4.86	4.97	5.14	5.24	5.32	5.37	5.41
0.250	3.21	3.25	3.34	3.47	3.64	3.84	4.06	4.26	4.43	4.59	4.72	4.93	5.08	5.19	5.27	5.32
0.300	2.93	2.97	3.06	3.18	3.34	3.52	3.74	3.95	4.14	4.32	4.47	4.72	4.91	5.05	5.15	5.22
0.400	2.48	2.51	2.59	2.71	2.85	3.01	3.19	3.40	3.61	3.81	3.99	4.29	4.54	4.73	4.87	4.99
0.500	2.13	2.16	2.24	2.34	2.47	2.62	2.78	2.95	3.14	3.35	3.54	3.88	4.16	4.39	4.58	4.73
0.600	1.86	1.89	1.96	2.05	2.17	2.31	2.45	2.61	2.77	2.95	3.15	3.50	3.81	4.06	4.28	4.46
0.700	1.65	1.67	1.74	1.82	1.93	2.05	2.19	2.33	2.48	2.64	2.81	3.17	3.48	3.75	3.99	4.19
0.800	1.48	1.50	1.56	1.64	1.74	1.85	1.97	2.10	2.24	2.38	2.53	2.87	3.19	3.46	3.71	3.93
0.900	1.34	1.36	1.41	1.49	1.58	1.68	1.79	1.91	2.04	2.17	2.31	2.61	2.92	3.20	3.45	3.68
1.00	1.22	1.24	1.29	1.36	1.44	1.54	1.64	1.75	1.87	1.99	2.12	2.39	2.69	2.97	3.22	3.45
1.20	1.04	1.05	1.09	1.15	1.23	1.31	1.40	1.50	1.60	1.71	1.82	2.05	2.29	2.56	2.81	3.03
1.40	0.899	0.913	0.951	1.00	1.07	1.14	1.22	1.31	1.40	1.49	1.59	1.79	2.00	2.24	2.47	2.69
1.60	0.792	0.805	0.839	0.887	0.945	1.01	1.08	1.16	1.24	1.32	1.41	1.59	1.78	1.98	2.19	2.40
1.80	0.708	0.721	0.751	0.793	0.847	0.907	0.972	1.04	1.11	1.19	1.27	1.43	1.59	1.77	1.96	2.16
2.00	0.641	0.652	0.679	0.717	0.765	0.821	0.881	0.944	1.01	1.08	1.15	1.29	1.45	1.61	1.77	1.95
2.20	0.585	0.595	0.620	0.656	0.699	0.749	0.805	0.863	0.924	0.987	1.05	1.19	1.33	1.47	1.62	1.78
2.40	0.539	0.547	0.569	0.603	0.644	0.689	0.740	0.795	0.851	0.908	0.969	1.09	1.22	1.35	1.49	1.64
2.60	0.499	0.505	0.528	0.559	0.596	0.639	0.685	0.736	0.789	0.843	0.899	1.01	1.13	1.25	1.38	1.51
2.80	0.464	0.471	0.491	0.520	0.555	0.595	0.639	0.685	0.735	0.785	0.837	0.945	1.06	1.17	1.29	1.41
3.00	0.433	0.440	0.459	0.485	0.519	0.556	0.597	0.641	0.688	0.736	0.784	0.885	0.989	1.10	1.21	1.32

Table 8–5 (continued)
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 15°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



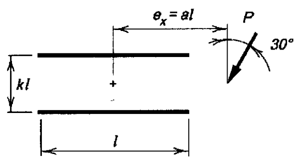
	k															
a	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44
0.100	4.38	4.39	4.45	4.58	4.73	4.88	5.01	5.11	5.19	5.25	5.30	5.36	5.39	5.41	5.42	5.43
0.150	3.96	3.98	4.04	4.14	4.29	4.47	4.64	4.79	4.91	5.01	5.09	5.21	5.28	5.33	5.36	5.38
0.200	3.60	3.62	3.68	3.79	3.92	4.09	4.27	4.45	4.60	4.74	4.85	5.02	5.13	5.21	5.27	5.31
0.250	3.28	3.30	3.37	3.47	3.61	3.76	3.94	4.12	4.29	4.45	4.59	4.81	4.96	5.07	5.15	5.22
0.300	3.00	3.02	3.09	3.20	3.33	3.47	3.64	3.82	4.00	4.17	4.33	4.59	4.78	4.92	5.03	5.11
0.400	2.55	2.57	2.64	2.74	2.87	3.01	3.15	3.31	3.49	3.66	3.83	4.13	4.39	4.58	4.74	4.86
0.500	2.20	2.22	2.28	2.38	2.50	2.63	2.77	2.92	3.07	3.23	3.40	3.71	3.99	4.23	4.42	4.58
0.600	1.92	1.94	2.00	2.10	2.20	2.33	2.46	2.60	2.74	2.89	3.04	3.35	3.63	3.88	4.10	4.29
0.700	1.70	1.72	1.78	1.87	1.97	2.08	2.21	2.34	2.47	2.61	2.74	3.03	3.30	3.56	3.79	4.00
0.800	1.53	1.55	1.60	1.68	1.77	1.89	2.00	2.12	2.25	2.37	2.50	2.76	3.02	3.27	3.51	3.72
0.900	1.38	1.40	1.45	1.53	1.61	1.72	1.83	1.94	2.06	2.18	2.29	2.53	2.78	3.02	3.24	3.46
1.00	1.26	1.28	1.32	1.39	1.48	1.57	1.68	1.78	1.89	2.01	2.12	2.34	2.56	2.79	3.01	3.22
1.20	1.07	1.09	1.13	1.19	1.26	1.35	1.44	1.53	1.63	1.73	1.83	2.03	2.23	2.43	2.63	2.82
1.40	0.929	0.943	0.980	1.03	1.10	1.17	1.26	1.34	1.43	1.52	1.61	1.79	1.97	2.14	2.32	2.50
1.60	0.820	0.832	0.865	0.915	0.975	1.04	1.11	1.19	1.27	1.35	1.43	1.60	1.76	1.92	2.08	2.24
1.80	0.733	0.745	0.775	0.819	0.873	0.935	1.00	1.07	1.14	1.22	1.29	1.44	1.59	1.74	1.88	2.03
2.00	0.663	0.673	0.701	0.741	0.791	0.847	0.908	0.972	1.04	1.11	1.17	1.31	1.45	1.59	1.72	1.85
2.20	0.605	0.615	0.640	0.677	0.723	0.773	0.831	0.889	0.951	1.01	1.08	1.21	1.33	1.46	1.59	1.71
2.40	0.557	0.565	0.589	0.623	0.665	0.712	0.764	0.820	0.876	0.933	0.993	1.11	1.23	1.35	1.47	1.58
2.60	0.516	0.523	0.545	0.577	0.616	0.660	0.708	0.760	0.812	0.867	0.921	1.03	1.15	1.26	1.37	1.47
2.80	0.480	0.487	0.508	0.537	0.573	0.615	0.659	0.707	0.757	0.808	0.859	0.965	1.07	1.17	1.28	1.38
3.00	0.448	0.455	0.475	0.503	0.536	0.575	0.617	0.661	0.709	0.757	0.805	0.904	1.00	1.10	1.20	1.30

Table 8–5 (continued)
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 30°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



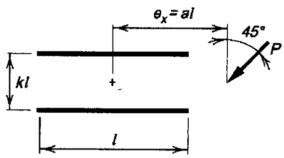
	k														
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	5.10	5.10	5.10	5.10	5.10	5.10	5.10	5.10	5.10	5.10	5.10	5.10	5.10	5.10	5.10	5.10
0.100	4.49	4.50	4.54	4.59	4.66	4.74	4.82	4.89	4.95	4.99	5.03	5.07	5.10	5.12	5.13	5.13
0.150	4.09	4.10	4.13	4.18	4.27	4.36	4.46	4.57	4.67	4.75	4.82	4.92	4.99	5.03	5.06	5.08
0.200	3.76	3.77	3.80	3.86	3.93	4.01	4.11	4.23	4.35	4.46	4.56	4.72	4.83	4.90	4.96	5.00
0.250	3.47	3.47	3.51	3.57	3.64	3.73	3.83	3.92	4.04	4.16	4.28	4.48	4.64	4.75	4.83	4.89
0.300	3.20	3.21	3.25	3.31	3.40	3.49	3.59	3.68	3.78	3.89	4.01	4.24	4.43	4.57	4.68	4.76
0.400	2.75	2.76	2.81	2.88	2.97	3.07	3.17	3.28	3.38	3.48	3.58	3.77	3.99	4.18	4.34	4.47
0.500	2.39	2.41	2.45	2.52	2.62	2.72	2.83	2.94	3.05	3.15	3.25	3.43	3.60	3.80	3.98	4.13
0.600	2.11	2.12	2.17	2.24	2.34	2.44	2.55	2.66	2.77	2.87	2.97	3.15	3.31	3.47	3.65	3.81
0.700	1.87	1.89	1.94	2.01	2.11	2.21	2.31	2.42	2.53	2.63	2.73	2.91	3.07	3.22	3.37	3.52
0.800	1.68	1.70	1.75	1.82	1.91	2.01	2.11	2.22	2.32	2.42	2.52	2.70	2.86	3.01	3.15	3.29
0.900	1.53	1.54	1.59	1.66	1.75	1.84	1.94	2.04	2.15	2.24	2.34	2.51	2.68	2.82	2.96	3.09
1.00	1.40	1.41	1.45	1.52	1.61	1.70	1.79	1.89	1.99	2.09	2.18	2.35	2.51	2.66	2.79	2.92
1.20	1.19	1.20	1.24	1.31	1.38	1.46	1.55	1.64	1.74	1.83	1.91	2.07	2.23	2.37	2.50	2.63
1.40	1.03	1.04	1.08	1.14	1.21	1.28	1.37	1.45	1.53	1.62	1.70	1.85	2.00	2.14	2.26	2.38
1.60	0.912	0.923	0.959	1.01	1.07	1.14	1.22	1.30	1.37	1.45	1.53	1.67	1.81	1.94	2.07	2.18
1.80	0.816	0.827	0.859	0.905	0.964	1.03	1.10	1.17	1.24	1.31	1.38	1.52	1.65	1.78	1.90	2.01
2.00	0.739	0.749	0.779	0.821	0.873	0.933	0.999	1.07	1.13	1.20	1.27	1.39	1.52	1.64	1.75	1.86
2.20	0.675	0.684	0.712	0.751	0.800	0.855	0.915	0.977	1.04	1.10	1.17	1.29	1.41	1.52	1.63	1.73
2.40	0.620	0.629	0.655	0.692	0.737	0.788	0.844	0.901	0.960	1.02	1.08	1.19	1.31	1.41	1.52	1.62
2.60	0.575	0.583	0.607	0.641	0.683	0.731	0.783	0.837	0.892	0.948	1.00	1.11	1.22	1.32	1.42	1.52
2.80	0.535	0.541	0.564	0.597	0.636	0.681	0.729	0.780	0.833	0.885	0.939	1.04	1.14	1.24	1.34	1.43
3.00	0.500	0.507	0.528	0.559	0.596	0.637	0.684	0.731	0.781	0.831	0.881	0.979	1.08	1.17	1.26	1.35

Table 8–5 (continued)
Coefficients C
for Eccentrically Loaded Weld Groups
Angle = 45°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



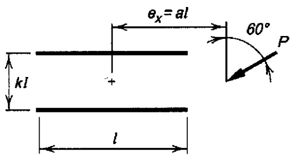
a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64
0.100	4.49	4.49	4.50	4.51	4.53	4.55	4.57	4.59	4.61	4.62	4.64	4.66	4.67	4.69	4.69	4.70
0.150	4.18	4.18	4.20	4.23	4.26	4.30	4.34	4.37	4.40	4.43	4.46	4.51	4.55	4.58	4.60	4.62
0.200	3.92	3.92	3.93	3.96	3.99	4.03	4.08	4.13	4.18	4.22	4.26	4.33	4.39	4.44	4.47	4.51
0.250	3.69	3.70	3.71	3.74	3.77	3.81	3.86	3.91	3.95	4.01	4.06	4.14	4.22	4.28	4.33	4.38
0.300	3.48	3.48	3.50	3.53	3.57	3.62	3.67	3.72	3.77	3.81	3.86	3.96	4.05	4.12	4.18	4.24
0.400	3.09	3.09	3.12	3.16	3.21	3.27	3.33	3.39	3.45	3.50	3.55	3.64	3.73	3.82	3.90	3.97
0.500	2.75	2.75	2.78	2.83	2.89	2.96	3.03	3.10	3.17	3.23	3.29	3.39	3.48	3.57	3.65	3.72
0.600	2.46	2.46	2.49	2.55	2.62	2.69	2.77	2.85	2.92	3.00	3.06	3.17	3.27	3.36	3.43	3.51
0.700	2.21	2.22	2.25	2.31	2.38	2.46	2.55	2.63	2.71	2.78	2.85	2.98	3.08	3.17	3.26	3.33
0.800	2.00	2.01	2.05	2.11	2.18	2.27	2.35	2.44	2.52	2.60	2.67	2.80	2.91	3.01	3.09	3.17
0.900	1.83	1.83	1.87	1.93	2.01	2.09	2.18	2.27	2.35	2.43	2.50	2.64	2.75	2.85	2.95	3.03
1.00	1.68	1.69	1.72	1.79	1.86	1.95	2.03	2.12	2.20	2.28	2.36	2.49	2.61	2.72	2.81	2.89
1.20	1.43	1.44	1.48	1.55	1.62	1.70	1.79	1.87	1.95	2.03	2.11	2.24	2.36	2.47	2.57	2.66
1.40	1.25	1.26	1.30	1.36	1.43	1.51	1.59	1.67	1.75	1.83	1.90	2.03	2.15	2.26	2.36	2.45
1.60	1.11	1.12	1.15	1.21	1.28	1.35	1.43	1.51	1.58	1.66	1.72	1.86	1.98	2.09	2.19	2.28
1.80	0.993	1.00	1.04	1.09	1.15	1.22	1.30	1.37	1.44	1.51	1.58	1.71	1.83	1.93	2.03	2.12
2.00	0.900	0.909	0.943	0.991	1.05	1.11	1.18	1.25	1.32	1.39	1.46	1.58	1.69	1.80	1.90	1.99
2.20	0.823	0.832	0.863	0.908	0.963	1.02	1.09	1.16	1.22	1.29	1.35	1.47	1.58	1.68	1.78	1.87
2.40	0.757	0.765	0.795	0.837	0.889	0.947	1.01	1.07	1.13	1.20	1.25	1.37	1.48	1.58	1.67	1.76
2.60	0.701	0.709	0.737	0.777	0.825	0.880	0.937	0.999	1.06	1.12	1.17	1.28	1.39	1.49	1.58	1.66
2.80	0.652	0.661	0.687	0.725	0.771	0.823	0.877	0.935	0.991	1.05	1.10	1.21	1.31	1.40	1.49	1.58
3.00	0.611	0.619	0.643	0.679	0.723	0.772	0.824	0.877	0.932	0.985	1.04	1.14	1.24	1.33	1.42	1.50

Table 8–5 (continued)
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 60°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x / l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



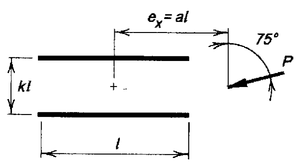
	k															
a	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18
0.100	4.26	4.26	4.26	4.26	4.26	4.25	4.25	4.25	4.24	4.24	4.23	4.22	4.21	4.20	4.19	4.18
0.150	4.13	4.13	4.13	4.13	4.13	4.13	4.14	4.14	4.14	4.14	4.14	4.13	4.13	4.12	4.11	4.10
0.200	3.97	3.97	3.97	3.97	3.98	3.98	3.99	4.00	4.01	4.02	4.02	4.03	4.03	4.03	4.03	4.03
0.250	3.85	3.85	3.85	3.85	3.86	3.86	3.87	3.87	3.88	3.89	3.90	3.92	3.93	3.94	3.94	3.95
0.300	3.74	3.74	3.74	3.75	3.75	3.76	3.76	3.77	3.78	3.78	3.79	3.81	3.83	3.85	3.86	3.86
0.400	3.50	3.51	3.51	3.52	3.53	3.55	3.56	3.57	3.59	3.60	3.61	3.63	3.65	3.67	3.69	3.71
0.500	3.26	3.26	3.27	3.29	3.31	3.33	3.36	3.38	3.41	3.42	3.44	3.48	3.50	3.53	3.55	3.57
0.600	3.02	3.02	3.03	3.05	3.09	3.13	3.16	3.20	3.22	3.25	3.28	3.33	3.37	3.40	3.42	3.45
0.700	2.79	2.79	2.81	2.84	2.88	2.93	2.97	3.02	3.06	3.09	3.12	3.18	3.23	3.27	3.31	3.33
0.800	2.58	2.58	2.61	2.64	2.69	2.75	2.80	2.85	2.90	2.94	2.98	3.05	3.11	3.15	3.19	3.23
0.900	2.39	2.40	2.42	2.46	2.52	2.58	2.64	2.69	2.75	2.80	2.84	2.92	2.98	3.04	3.09	3.13
1.00	2.23	2.23	2.26	2.30	2.36	2.43	2.49	2.55	2.61	2.66	2.71	2.80	2.87	2.93	2.99	3.03
1.20	1.94	1.95	1.98	2.03	2.09	2.16	2.23	2.30	2.37	2.43	2.48	2.58	2.66	2.73	2.79	2.85
1.40	1.71	1.72	1.75	1.81	1.87	1.94	2.02	2.09	2.16	2.23	2.28	2.39	2.48	2.56	2.63	2.68
1.60	1.53	1.54	1.57	1.62	1.69	1.77	1.84	1.91	1.98	2.05	2.11	2.22	2.31	2.40	2.47	2.54
1.80	1.38	1.39	1.42	1.47	1.54	1.61	1.69	1.76	1.83	1.90	1.96	2.07	2.17	2.25	2.33	2.40
2.00	1.25	1.26	1.29	1.35	1.41	1.49	1.56	1.63	1.70	1.76	1.82	1.94	2.04	2.13	2.21	2.28
2.20	1.15	1.16	1.19	1.24	1.31	1.37	1.45	1.52	1.59	1.65	1.71	1.82	1.92	2.01	2.09	2.17
2.40	1.06	1.07	1.10	1.15	1.21	1.28	1.35	1.42	1.48	1.55	1.61	1.72	1.82	1.91	1.99	2.07
2.60	0.981	0.991	1.02	1.07	1.13	1.19	1.26	1.33	1.39	1.46	1.51	1.62	1.72	1.81	1.90	1.97
2.80	0.915	0.924	0.955	1.00	1.06	1.12	1.19	1.25	1.31	1.37	1.43	1.54	1.64	1.73	1.81	1.89
3.00	0.857	0.865	0.896	0.941	0.995	1.06	1.12	1.18	1.24	1.30	1.36	1.46	1.56	1.65	1.73	1.81

Table 8-5 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 75°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84
0.100	3.81	3.81	3.82	3.83	3.83	3.84	3.84	3.85	3.85	3.85	3.85	3.84	3.82	3.81	3.79	3.40
0.150	3.85	3.85	3.86	3.86	3.86	3.86	3.85	3.85	3.84	3.84	3.83	3.81	3.80	3.78	3.76	3.74
0.200	3.84	3.84	3.84	3.83	3.83	3.83	3.82	3.82	3.81	3.80	3.80	3.78	3.76	3.75	3.73	3.71
0.250	3.83	3.83	3.83	3.82	3.82	3.81	3.80	3.80	3.79	3.78	3.77	3.75	3.74	3.72	3.70	3.68
0.300	3.81	3.81	3.81	3.80	3.80	3.79	3.78	3.77	3.77	3.76	3.75	3.73	3.71	3.69	3.67	3.66
0.400	3.78	3.78	3.77	3.76	3.75	3.74	3.73	3.72	3.71	3.70	3.69	3.67	3.66	3.64	3.62	3.61
0.500	3.72	3.72	3.71	3.70	3.69	3.68	3.67	3.66	3.65	3.64	3.64	3.62	3.60	3.59	3.57	3.56
0.600	3.65	3.64	3.64	3.63	3.62	3.61	3.60	3.59	3.58	3.58	3.57	3.56	3.55	3.53	3.52	3.51
0.700	3.55	3.55	3.54	3.54	3.54	3.53	3.52	3.52	3.51	3.51	3.50	3.49	3.49	3.48	3.47	3.46
0.800	3.45	3.45	3.45	3.44	3.44	3.44	3.44	3.44	3.44	3.43	3.43	3.43	3.42	3.42	3.41	3.41
0.900	3.35	3.35	3.35	3.35	3.35	3.35	3.35	3.35	3.35	3.36	3.36	3.36	3.36	3.36	3.36	3.36
1.00	3.23	3.23	3.23	3.24	3.25	3.25	3.26	3.26	3.27	3.28	3.28	3.29	3.30	3.30	3.31	3.30
1.20	3.00	3.00	3.01	3.02	3.04	3.06	3.08	3.09	3.11	3.12	3.13	3.16	3.17	3.19	3.20	3.21
1.40	2.77	2.77	2.78	2.81	2.84	2.87	2.90	2.93	2.95	2.97	2.99	3.02	3.05	3.07	3.09	3.11
1.60	2.57	2.57	2.58	2.61	2.65	2.69	2.73	2.77	2.80	2.83	2.85	2.90	2.93	2.96	2.99	3.01
1.80	2.38	2.38	2.40	2.43	2.48	2.52	2.57	2.62	2.65	2.69	2.72	2.78	2.82	2.86	2.89	2.92
2.00	2.21	2.21	2.23	2.27	2.32	2.37	2.43	2.48	2.52	2.56	2.60	2.66	2.72	2.76	2.80	2.83
2.20	2.05	2.06	2.08	2.13	2.18	2.24	2.30	2.35	2.40	2.44	2.49	2.56	2.62	2.67	2.71	2.74
2.40	1.92	1.92	1.95	1.99	2.05	2.11	2.18	2.23	2.29	2.33	2.38	2.46	2.52	2.58	2.62	2.66
2.60	1.79	1.80	1.83	1.88	1.94	2.00	2.07	2.13	2.18	2.23	2.28	2.36	2.43	2.49	2.54	2.59
2.80	1.69	1.69	1.72	1.77	1.83	1.90	1.96	2.03	2.09	2.14	2.19	2.27	2.35	2.41	2.47	2.51
3.00	1.59	1.59	1.63	1.68	1.74	1.81	1.87	1.94	2.00	2.05	2.10	2.19	2.27	2.33	2.39	2.44

Table 8–6
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 0°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with

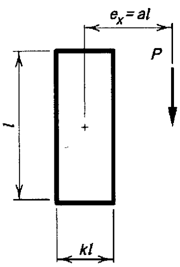
$R_n = CC_1Dl$ ($\phi = 0.75$ $\Omega = 2.00$)

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where

P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x / l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



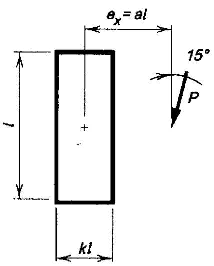
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	3.06	3.62	4.18	4.73	5.29	5.85	6.40	6.96	7.52	8.07	8.63	9.74	10.9	12.0	13.1	14.2
0.100	3.72	4.10	4.56	5.03	5.53	6.04	6.55	7.06	7.58	8.10	8.62	9.66	10.7	11.8	12.8	13.9
0.150	3.67	4.06	4.49	4.94	5.41	5.89	6.37	6.86	7.36	7.86	8.35	9.36	10.4	11.4	12.4	13.4
0.200	3.51	3.93	4.34	4.76	5.21	5.66	6.12	6.59	7.07	7.54	8.03	9.00	10.0	11.0	12.0	13.0
0.250	3.31	3.72	4.13	4.54	4.96	5.39	5.83	6.28	6.74	7.20	7.67	8.61	9.57	10.5	11.5	12.5
0.300	3.09	3.48	3.89	4.29	4.69	5.11	5.53	5.97	6.41	6.85	7.31	8.23	9.17	10.1	11.1	12.1
0.400	2.66	3.01	3.38	3.77	4.16	4.54	4.94	5.35	5.76	6.18	6.61	7.50	8.40	9.33	10.3	11.2
0.500	2.29	2.60	2.94	3.30	3.67	4.04	4.41	4.79	5.18	5.59	6.00	6.84	7.71	8.61	9.52	10.5
0.600	2.00	2.26	2.57	2.90	3.25	3.60	3.95	4.31	4.69	5.07	5.46	6.27	7.11	7.97	8.86	9.76
0.700	1.76	1.99	2.27	2.57	2.90	3.24	3.57	3.91	4.26	4.62	4.99	5.77	6.57	7.41	8.27	9.15
0.800	1.56	1.78	2.02	2.30	2.60	2.93	3.24	3.57	3.90	4.24	4.60	5.33	6.10	6.90	7.73	8.58
0.900	1.41	1.60	1.82	2.08	2.36	2.67	2.97	3.27	3.58	3.91	4.25	4.95	5.68	6.45	7.25	8.07
1.00	1.28	1.45	1.66	1.89	2.16	2.45	2.73	3.02	3.31	3.62	3.94	4.60	5.30	6.04	6.80	7.59
1.20	1.07	1.22	1.40	1.61	1.84	2.09	2.35	2.61	2.87	3.15	3.43	4.03	4.67	5.33	6.04	6.77
1.40	0.927	1.05	1.21	1.40	1.60	1.83	2.06	2.29	2.53	2.77	3.03	3.58	4.15	4.77	5.41	6.09
1.60	0.815	0.925	1.07	1.23	1.42	1.62	1.83	2.04	2.25	2.48	2.71	3.21	3.74	4.30	4.90	5.53
1.80	0.725	0.825	0.952	1.10	1.27	1.45	1.64	1.83	2.03	2.23	2.45	2.90	3.39	3.91	4.47	5.05
2.00	0.655	0.744	0.860	0.995	1.15	1.31	1.49	1.66	1.85	2.03	2.23	2.65	3.10	3.59	4.10	4.65
2.20	0.596	0.679	0.784	0.907	1.05	1.20	1.36	1.52	1.69	1.87	2.05	2.44	2.86	3.31	3.79	4.30
2.40	0.547	0.623	0.720	0.833	0.961	1.10	1.25	1.41	1.56	1.72	1.89	2.25	2.65	3.07	3.52	4.00
2.60	0.505	0.575	0.665	0.771	0.889	1.02	1.16	1.30	1.45	1.60	1.76	2.10	2.47	2.86	3.29	3.73
2.80	0.469	0.535	0.619	0.717	0.828	0.949	1.08	1.21	1.35	1.49	1.64	1.96	2.31	2.68	3.08	3.50
3.00	0.439	0.500	0.579	0.671	0.773	0.888	1.01	1.14	1.27	1.40	1.54	1.84	2.16	2.52	2.89	3.29

Table 8–6 (continued)
Coefficients C
for Eccentrically Loaded Weld Groups
Angle = 15°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



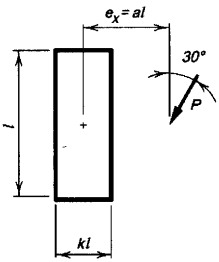
	k														
a	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	3.85	4.39	4.94	5.48	6.03	6.57	7.12	7.66	8.21	8.75	9.30	10.4	11.5	12.6	13.7	14.7
0.100	3.79	4.22	4.69	5.19	5.70	6.21	6.73	7.24	7.77	8.30	8.82	9.88	10.9	12.0	13.0	14.1
0.150	3.68	4.13	4.59	5.05	5.53	6.01	6.49	6.98	7.48	7.97	8.47	9.47	10.5	11.5	12.5	13.6
0.200	3.51	3.95	4.40	4.85	5.31	5.76	6.23	6.69	7.16	7.64	8.12	9.09	10.1	11.1	12.1	13.1
0.250	3.31	3.72	4.15	4.60	5.04	5.48	5.93	6.38	6.83	7.30	7.76	8.70	9.66	10.6	11.6	12.6
0.300	3.09	3.48	3.89	4.33	4.76	5.19	5.62	6.06	6.50	6.95	7.40	8.32	9.26	10.2	11.2	12.2
0.400	2.67	3.01	3.39	3.79	4.20	4.61	5.02	5.44	5.86	6.28	6.71	7.60	8.51	9.43	10.4	11.3
0.500	2.32	2.62	2.95	3.31	3.70	4.10	4.49	4.88	5.28	5.69	6.10	6.95	7.83	8.72	9.64	10.6
0.600	2.02	2.29	2.59	2.92	3.27	3.65	4.03	4.40	4.78	5.17	5.57	6.38	7.23	8.10	8.99	9.90
0.700	1.79	2.02	2.29	2.59	2.92	3.28	3.64	4.00	4.36	4.73	5.11	5.89	6.70	7.54	8.41	9.29
0.800	1.59	1.81	2.05	2.33	2.64	2.97	3.31	3.65	3.99	4.35	4.70	5.45	6.23	7.04	7.87	8.73
0.900	1.44	1.63	1.85	2.11	2.39	2.71	3.03	3.35	3.68	4.01	4.35	5.06	5.81	6.58	7.39	8.21
1.00	1.31	1.48	1.69	1.93	2.20	2.49	2.79	3.10	3.40	3.72	4.04	4.72	5.43	6.17	6.94	7.74
1.20	1.10	1.25	1.43	1.64	1.88	2.13	2.41	2.68	2.95	3.24	3.53	4.14	4.78	5.47	6.18	6.92
1.40	0.952	1.08	1.24	1.43	1.64	1.87	2.11	2.35	2.60	2.85	3.12	3.68	4.27	4.89	5.55	6.24
1.60	0.837	0.951	1.09	1.26	1.45	1.65	1.87	2.10	2.32	2.55	2.79	3.30	3.85	4.42	5.03	5.67
1.80	0.747	0.848	0.977	1.13	1.30	1.48	1.68	1.89	2.09	2.30	2.52	2.99	3.49	4.03	4.59	5.19
2.00	0.675	0.767	0.884	1.02	1.18	1.35	1.53	1.72	1.90	2.10	2.30	2.73	3.20	3.70	4.22	4.78
2.20	0.615	0.699	0.807	0.932	1.07	1.23	1.40	1.57	1.75	1.93	2.11	2.52	2.95	3.41	3.90	4.42
2.40	0.564	0.641	0.741	0.857	0.988	1.13	1.29	1.45	1.61	1.78	1.95	2.33	2.73	3.17	3.63	4.12
2.60	0.521	0.593	0.685	0.793	0.915	1.05	1.19	1.34	1.49	1.65	1.82	2.17	2.55	2.95	3.39	3.85
2.80	0.484	0.552	0.637	0.739	0.851	0.976	1.11	1.25	1.39	1.54	1.70	2.03	2.38	2.77	3.17	3.61
3.00	0.452	0.515	0.596	0.691	0.796	0.912	1.04	1.17	1.31	1.45	1.59	1.90	2.24	2.60	2.99	3.40

Table 8–6 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 30°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x / l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



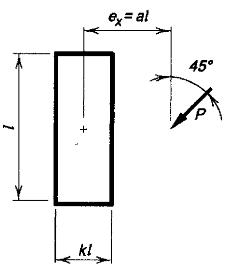
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	4.37	4.89	5.40	5.91	6.43	6.94	7.46	7.97	8.49	9.00	9.52	10.5	11.6	12.6	13.6	14.7
0.100	4.04	4.60	5.13	5.65	6.16	6.67	7.17	7.68	8.19	8.69	9.20	10.2	11.3	12.3	13.3	14.4
0.150	3.83	4.33	4.85	5.36	5.86	6.36	6.86	7.35	7.85	8.35	8.85	9.85	10.9	11.9	12.9	14.0
0.200	3.63	4.09	4.57	5.06	5.55	6.04	6.52	6.99	7.48	7.97	8.46	9.45	10.5	11.5	12.5	13.6
0.250	3.43	3.85	4.29	4.76	5.24	5.72	6.19	6.66	7.12	7.59	8.06	9.03	10.0	11.0	12.1	13.1
0.300	3.21	3.60	4.02	4.47	4.93	5.39	5.87	6.33	6.78	7.24	7.70	8.64	9.61	10.6	11.6	12.6
0.400	2.81	3.15	3.52	3.93	4.35	4.79	5.25	5.71	6.15	6.59	7.03	7.93	8.86	9.81	10.8	11.8
0.500	2.46	2.76	3.10	3.47	3.86	4.27	4.71	5.14	5.58	6.01	6.43	7.31	8.21	9.13	10.1	11.0
0.600	2.17	2.44	2.74	3.08	3.45	3.84	4.24	4.66	5.09	5.50	5.91	6.76	7.63	8.53	9.45	10.4
0.700	1.93	2.17	2.45	2.76	3.10	3.47	3.85	4.25	4.67	5.06	5.45	6.27	7.11	7.98	8.87	9.79
0.800	1.73	1.95	2.21	2.49	2.82	3.16	3.53	3.91	4.29	4.67	5.05	5.83	6.64	7.48	8.35	9.24
0.900	1.56	1.77	2.00	2.27	2.57	2.90	3.24	3.61	3.97	4.33	4.69	5.44	6.22	7.03	7.87	8.73
1.00	1.43	1.61	1.83	2.08	2.37	2.68	3.00	3.34	3.69	4.03	4.37	5.09	5.84	6.62	7.43	8.27
1.20	1.21	1.37	1.56	1.79	2.04	2.31	2.60	2.91	3.22	3.53	3.84	4.50	5.19	5.91	6.67	7.45
1.40	1.05	1.19	1.36	1.56	1.78	2.03	2.29	2.57	2.85	3.13	3.41	4.01	4.65	5.32	6.03	6.76
1.60	0.924	1.05	1.20	1.38	1.58	1.81	2.04	2.29	2.55	2.80	3.07	3.62	4.21	4.83	5.49	6.17
1.80	0.825	0.937	1.08	1.24	1.42	1.63	1.84	2.07	2.30	2.54	2.78	3.29	3.84	4.42	5.03	5.67
2.00	0.745	0.847	0.975	1.13	1.29	1.48	1.67	1.88	2.10	2.31	2.54	3.01	3.52	4.06	4.63	5.24
2.20	0.680	0.772	0.891	1.03	1.18	1.35	1.53	1.73	1.93	2.13	2.34	2.78	3.25	3.76	4.29	4.86
2.40	0.625	0.711	0.819	0.947	1.09	1.25	1.41	1.59	1.78	1.97	2.16	2.58	3.02	3.49	4.00	4.53
2.60	0.577	0.657	0.759	0.877	1.01	1.16	1.31	1.48	1.65	1.83	2.01	2.40	2.82	3.26	3.74	4.24
2.80	0.537	0.611	0.705	0.817	0.941	1.08	1.22	1.38	1.54	1.71	1.88	2.24	2.64	3.06	3.51	3.98
3.00	0.501	0.571	0.660	0.764	0.881	1.01	1.15	1.29	1.45	1.60	1.76	2.11	2.48	2.88	3.30	3.76

Table 8-6 (continued)
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 45°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x/l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



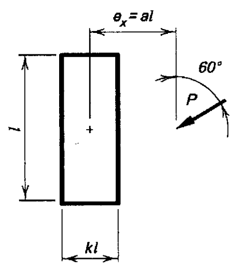
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	4.68	5.15	5.62	6.09	6.55	7.02	7.49	7.96	8.43	8.89	9.36	10.3	11.2	12.2	13.1	14.0
0.100	4.49	4.99	5.48	5.97	6.45	6.94	7.43	7.93	8.42	8.90	9.39	10.4	11.4	12.3	13.3	14.3
0.150	4.18	4.69	5.19	5.67	6.16	6.65	7.15	7.65	8.15	8.65	9.15	10.1	11.1	12.1	13.1	14.1
0.200	3.92	4.39	4.87	5.36	5.84	6.33	6.83	7.33	7.84	8.34	8.85	9.86	10.9	11.9	12.9	13.9
0.250	3.69	4.13	4.58	5.05	5.52	6.01	6.50	7.00	7.51	8.02	8.53	9.54	10.6	11.6	12.6	13.6
0.300	3.48	3.88	4.31	4.76	5.22	5.69	6.18	6.67	7.17	7.68	8.20	9.21	10.2	11.3	12.3	13.3
0.400	3.09	3.45	3.83	4.25	4.68	5.13	5.59	6.07	6.56	7.06	7.56	8.55	9.57	10.6	11.6	12.7
0.500	2.75	3.07	3.42	3.81	4.22	4.65	5.09	5.55	6.03	6.51	7.01	7.95	8.94	10.0	11.0	12.0
0.600	2.46	2.75	3.07	3.43	3.82	4.23	4.66	5.11	5.57	6.04	6.52	7.43	8.38	9.36	10.4	11.4
0.700	2.21	2.48	2.78	3.11	3.48	3.88	4.29	4.72	5.17	5.62	6.08	6.96	7.87	8.82	9.80	10.8
0.800	2.00	2.25	2.53	2.85	3.19	3.57	3.97	4.38	4.81	5.24	5.69	6.54	7.42	8.33	9.29	10.3
0.900	1.83	2.05	2.31	2.61	2.95	3.31	3.68	4.08	4.49	4.91	5.33	6.16	7.00	7.89	8.81	9.76
1.00	1.68	1.88	2.13	2.41	2.73	3.07	3.43	3.81	4.20	4.60	5.01	5.81	6.63	7.48	8.37	9.30
1.20	1.43	1.62	1.84	2.09	2.38	2.69	3.01	3.35	3.71	4.08	4.46	5.20	5.97	6.77	7.59	8.46
1.40	1.25	1.41	1.61	1.84	2.10	2.38	2.68	2.99	3.31	3.65	4.00	4.69	5.41	6.16	6.94	7.75
1.60	1.11	1.25	1.43	1.64	1.88	2.13	2.40	2.69	2.99	3.30	3.62	4.26	4.94	5.64	6.38	7.14
1.80	0.993	1.12	1.29	1.48	1.69	1.93	2.18	2.44	2.72	3.00	3.30	3.90	4.53	5.20	5.89	6.61
2.00	0.900	1.02	1.17	1.35	1.54	1.76	1.99	2.23	2.49	2.75	3.03	3.59	4.18	4.80	5.46	6.14
2.20	0.823	0.932	1.07	1.23	1.42	1.61	1.83	2.05	2.29	2.54	2.79	3.32	3.88	4.47	5.09	5.73
2.40	0.757	0.859	0.988	1.14	1.31	1.49	1.69	1.90	2.12	2.35	2.59	3.09	3.61	4.17	4.75	5.37
2.60	0.701	0.796	0.916	1.06	1.21	1.39	1.57	1.77	1.98	2.19	2.42	2.89	3.38	3.91	4.46	5.04
2.80	0.652	0.741	0.855	0.985	1.13	1.30	1.47	1.65	1.85	2.05	2.27	2.70	3.17	3.67	4.20	4.75
3.00	0.611	0.693	0.800	0.924	1.06	1.21	1.38	1.55	1.74	1.93	2.13	2.54	2.99	3.46	3.97	4.50

Table 8–6 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 60°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ (ϕ = 0.75, Ω = 2.00)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x / l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



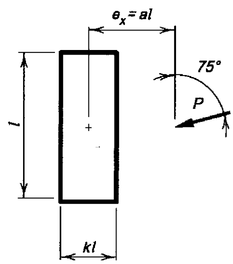
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	5.15	5.58	6.02	6.46	6.89	7.33	7.77	8.20	8.64	9.08	9.52	10.4	11.3	12.1	13.0	13.9
0.100	4.87	5.29	5.74	6.19	6.66	7.13	7.60	8.07	8.53	8.98	9.44	10.3	11.2	12.1	13.0	13.9
0.150	4.61	5.04	5.48	5.93	6.40	6.89	7.37	7.86	8.34	8.82	9.29	10.2	11.1	12.0	12.9	13.9
0.200	4.36	4.79	5.23	5.67	6.14	6.63	7.13	7.62	8.12	8.61	9.10	10.1	11.0	11.9	12.8	13.7
0.250	4.13	4.55	4.99	5.43	5.88	6.37	6.87	7.38	7.89	8.39	8.89	9.87	10.8	11.8	12.7	13.6
0.300	3.93	4.33	4.76	5.19	5.64	6.12	6.62	7.13	7.65	8.16	8.67	9.67	10.6	11.6	12.5	13.5
0.400	3.57	3.94	4.34	4.76	5.20	5.66	6.15	6.66	7.17	7.69	8.21	9.24	10.2	11.2	12.2	13.2
0.500	3.26	3.60	3.98	4.38	4.81	5.27	5.74	6.24	6.75	7.26	7.78	8.81	9.82	10.8	11.8	12.8
0.600	2.98	3.30	3.65	4.05	4.47	4.92	5.38	5.86	6.36	6.87	7.38	8.41	9.43	10.4	11.4	12.4
0.700	2.73	3.03	3.37	3.75	4.16	4.60	5.05	5.52	6.00	6.50	7.00	8.03	9.05	10.1	11.1	12.1
0.800	2.52	2.80	3.12	3.49	3.89	4.31	4.75	5.21	5.68	6.16	6.65	7.66	8.68	9.70	10.7	11.7
0.900	2.33	2.60	2.90	3.25	3.64	4.05	4.47	4.92	5.38	5.85	6.33	7.31	8.32	9.32	10.3	11.3
1.00	2.17	2.42	2.71	3.05	3.42	3.81	4.23	4.66	5.10	5.56	6.02	6.99	7.98	8.96	9.94	10.9
1.20	1.89	2.11	2.38	2.69	3.03	3.40	3.79	4.19	4.61	5.04	5.48	6.40	7.33	8.27	9.24	10.2
1.40	1.67	1.87	2.12	2.41	2.72	3.06	3.42	3.80	4.19	4.60	5.02	5.89	6.76	7.66	8.59	9.54
1.60	1.49	1.68	1.91	2.17	2.46	2.78	3.11	3.47	3.83	4.22	4.61	5.43	6.26	7.11	8.01	8.93
1.80	1.35	1.52	1.73	1.98	2.25	2.54	2.85	3.18	3.53	3.89	4.26	5.04	5.82	6.63	7.48	8.37
2.00	1.23	1.39	1.58	1.81	2.06	2.34	2.63	2.94	3.26	3.60	3.95	4.69	5.43	6.20	7.01	7.86
2.20	1.13	1.27	1.46	1.67	1.91	2.16	2.43	2.73	3.03	3.35	3.68	4.38	5.09	5.82	6.59	7.40
2.40	1.04	1.18	1.35	1.55	1.77	2.01	2.27	2.54	2.83	3.13	3.44	4.10	4.79	5.48	6.21	6.99
2.60	0.968	1.09	1.26	1.44	1.65	1.88	2.12	2.38	2.65	2.93	3.23	3.85	4.51	5.17	5.87	6.61
2.80	0.903	1.02	1.17	1.35	1.55	1.76	1.99	2.23	2.49	2.76	3.04	3.64	4.26	4.90	5.57	6.27
3.00	0.847	0.959	1.10	1.27	1.45	1.66	1.87	2.10	2.35	2.60	2.87	3.44	4.04	4.65	5.29	5.97

Table 8–6 (continued)
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 75°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



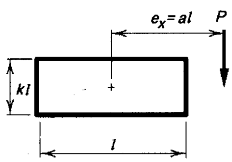
	k															
a	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	5.45	5.84	6.22	6.61	6.99	7.37	7.76	8.14	8.53	8.91	9.30	10.1	10.8	11.6	12.4	13.1
0.100	5.17	5.56	5.98	6.42	6.85	7.27	7.68	8.09	8.48	8.88	9.27	10.0	10.8	11.6	12.4	13.1
0.150	5.01	5.38	5.81	6.26	6.71	7.15	7.58	8.00	8.41	8.81	9.21	10.0	10.8	11.6	12.3	13.1
0.200	4.85	5.22	5.64	6.10	6.56	7.02	7.46	7.90	8.32	8.73	9.14	9.94	10.7	11.5	12.3	13.1
0.250	4.71	5.07	5.48	5.94	6.41	6.88	7.34	7.78	8.22	8.64	9.06	9.88	10.7	11.5	12.3	13.0
0.300	4.57	4.93	5.34	5.79	6.26	6.74	7.21	7.66	8.11	8.54	8.97	9.80	10.6	11.4	12.2	13.0
0.400	4.32	4.68	5.07	5.51	5.99	6.47	6.95	7.42	7.88	8.33	8.76	9.63	10.5	11.3	12.1	12.9
0.500	4.09	4.45	4.83	5.27	5.74	6.23	6.72	7.20	7.67	8.13	8.58	9.44	10.3	11.1	12.0	12.8
0.600	3.88	4.23	4.62	5.04	5.51	5.99	6.49	6.98	7.46	7.93	8.40	9.28	10.1	11.0	11.8	12.6
0.700	3.69	4.02	4.41	4.84	5.29	5.77	6.26	6.76	7.25	7.74	8.21	9.12	10.0	10.8	11.7	12.5
0.800	3.50	3.83	4.21	4.64	5.08	5.55	6.05	6.54	7.04	7.53	8.02	8.95	9.85	10.7	11.5	12.4
0.900	3.34	3.65	4.03	4.45	4.89	5.35	5.84	6.34	6.84	7.33	7.83	8.78	9.70	10.6	11.4	12.3
1.00	3.18	3.49	3.86	4.27	4.71	5.16	5.64	6.13	6.63	7.13	7.63	8.60	9.54	10.4	11.3	12.1
1.20	2.90	3.19	3.54	3.94	4.37	4.82	5.27	5.75	6.25	6.75	7.25	8.24	9.20	10.1	11.0	11.9
1.40	2.65	2.93	3.27	3.65	4.07	4.50	4.95	5.41	5.89	6.38	6.88	7.88	8.86	9.82	10.7	11.7
1.60	2.44	2.70	3.03	3.39	3.79	4.21	4.65	5.10	5.56	6.04	6.53	7.52	8.51	9.49	10.4	11.4
1.80	2.25	2.50	2.81	3.16	3.54	3.95	4.38	4.81	5.26	5.73	6.20	7.18	8.17	9.15	10.1	11.1
2.00	2.09	2.33	2.62	2.96	3.32	3.72	4.13	4.55	4.99	5.44	5.90	6.86	7.84	8.82	9.79	10.8
2.20	1.95	2.17	2.45	2.77	3.13	3.50	3.90	4.31	4.73	5.17	5.62	6.56	7.52	8.50	9.47	10.4
2.40	1.82	2.04	2.31	2.61	2.95	3.31	3.69	4.09	4.50	4.92	5.36	6.27	7.22	8.18	9.15	10.1
2.60	1.71	1.92	2.17	2.46	2.78	3.13	3.50	3.89	4.29	4.70	5.12	6.01	6.93	7.88	8.85	9.81
2.80	1.61	1.81	2.05	2.33	2.64	2.97	3.33	3.70	4.09	4.48	4.90	5.76	6.66	7.59	8.55	9.51
3.00	1.52	1.71	1.94	2.21	2.51	2.83	3.17	3.53	3.90	4.29	4.69	5.52	6.40	7.32	8.26	9.21

Table 8–7
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 0°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



	k														
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	5.57	5.87	6.18	6.49	6.79	7.10	7.41	7.71	8.02	8.32	8.63	9.24	9.86	10.5	11.1	11.7
0.100	4.31	4.67	5.08	5.54	6.01	6.49	6.95	7.40	7.82	8.23	8.62	9.37	10.1	10.8	11.5	12.1
0.150	3.90	4.24	4.65	5.08	5.55	6.03	6.52	7.00	7.47	7.92	8.35	9.18	10.0	10.7	11.4	12.1
0.200	3.53	3.86	4.26	4.69	5.13	5.61	6.10	6.59	7.08	7.56	8.03	8.92	9.76	10.6	11.3	12.1
0.250	3.21	3.53	3.91	4.33	4.77	5.23	5.71	6.20	6.69	7.18	7.67	8.60	9.50	10.3	11.2	12.0
0.300	2.93	3.23	3.60	4.01	4.44	4.88	5.35	5.83	6.32	6.81	7.31	8.27	9.20	10.1	11.0	11.8
0.400	2.48	2.75	3.09	3.46	3.87	4.29	4.73	5.18	5.65	6.13	6.61	7.60	8.57	9.51	10.4	11.3
0.500	2.13	2.38	2.68	3.02	3.40	3.79	4.21	4.63	5.07	5.53	6.00	6.96	7.93	8.90	9.85	10.8
0.600	1.86	2.09	2.36	2.68	3.02	3.38	3.77	4.18	4.59	5.02	5.46	6.38	7.33	8.30	9.26	10.2
0.700	1.65	1.85	2.11	2.39	2.70	3.04	3.41	3.78	4.18	4.58	4.99	5.87	6.79	7.73	8.68	9.64
0.800	1.48	1.67	1.90	2.16	2.45	2.76	3.10	3.45	3.82	4.20	4.60	5.42	6.30	7.21	8.14	9.09
0.900	1.34	1.51	1.72	1.97	2.23	2.53	2.84	3.17	3.51	3.87	4.25	5.03	5.86	6.73	7.64	8.56
1.00	1.22	1.38	1.58	1.80	2.05	2.32	2.61	2.92	3.25	3.59	3.94	4.68	5.47	6.30	7.17	8.07
1.20	1.04	1.17	1.34	1.54	1.76	2.00	2.25	2.53	2.81	3.12	3.43	4.10	4.81	5.57	6.37	7.21
1.40	0.899	1.02	1.17	1.35	1.54	1.75	1.98	2.22	2.48	2.75	3.03	3.63	4.28	4.97	5.71	6.47
1.60	0.792	0.900	1.04	1.19	1.37	1.56	1.76	1.98	2.21	2.45	2.71	3.26	3.85	4.48	5.15	5.85
1.80	0.708	0.805	0.928	1.07	1.23	1.40	1.58	1.78	1.99	2.21	2.45	2.95	3.49	4.07	4.69	5.33
2.00	0.641	0.729	0.840	0.971	1.11	1.27	1.44	1.62	1.81	2.02	2.23	2.69	3.19	3.73	4.30	4.89
2.20	0.585	0.665	0.768	0.887	1.02	1.16	1.32	1.49	1.66	1.85	2.05	2.48	2.94	3.44	3.97	4.51
2.40	0.539	0.612	0.707	0.816	0.940	1.07	1.22	1.37	1.53	1.71	1.89	2.29	2.72	3.18	3.68	4.19
2.60	0.499	0.567	0.655	0.756	0.871	0.996	1.13	1.27	1.43	1.59	1.76	2.13	2.53	2.97	3.42	3.90
2.80	0.464	0.528	0.609	0.705	0.811	0.928	1.05	1.19	1.33	1.48	1.64	1.99	2.37	2.77	3.20	3.65
3.00	0.433	0.493	0.571	0.660	0.759	0.868	0.987	1.11	1.25	1.39	1.54	1.86	2.22	2.60	3.01	3.43

Table 8–7 (continued)
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 15°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with

$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where

P = required force, P_u or P_a, kips

D = number of sixteenths-of-an-inch in the fillet weld size

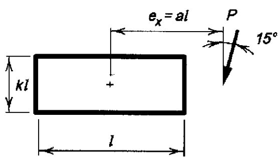
l = characteristic length of weld group, in.

$a = e_x / l$

e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.

C = coefficient tabulated below

C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



	k															
a	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	5.45	5.84	6.22	6.61	6.99	7.37	7.76	8.14	8.53	8.91	9.30	10.1	10.8	11.6	12.4	13.1
0.100	4.38	4.75	5.14	5.58	6.06	6.54	7.02	7.48	7.94	8.39	8.82	9.68	10.5	11.3	12.1	12.9
0.150	3.96	4.31	4.70	5.13	5.60	6.09	6.58	7.07	7.55	8.01	8.47	9.35	10.2	11.0	11.9	12.7
0.200	3.60	3.93	4.32	4.75	5.19	5.67	6.16	6.66	7.15	7.64	8.12	9.04	9.93	10.8	11.6	12.4
0.250	3.28	3.60	3.97	4.39	4.83	5.29	5.77	6.27	6.77	7.27	7.76	8.72	9.65	10.5	11.4	12.2
0.300	3.00	3.31	3.67	4.07	4.51	4.95	5.42	5.90	6.40	6.90	7.40	8.39	9.34	10.3	11.1	12.0
0.400	2.55	2.82	3.15	3.53	3.93	4.37	4.81	5.26	5.73	6.22	6.71	7.71	8.70	9.67	10.6	11.5
0.500	2.20	2.44	2.75	3.09	3.47	3.87	4.29	4.72	5.17	5.63	6.10	7.07	8.06	9.04	10.0	11.0
0.600	1.92	2.15	2.42	2.74	3.09	3.46	3.86	4.27	4.69	5.12	5.57	6.50	7.46	8.43	9.41	10.4
0.700	1.70	1.91	2.17	2.46	2.77	3.12	3.49	3.87	4.27	4.68	5.11	5.99	6.91	7.86	8.83	9.79
0.800	1.53	1.72	1.95	2.22	2.52	2.84	3.18	3.54	3.91	4.30	4.70	5.54	6.42	7.34	8.28	9.23
0.900	1.38	1.55	1.77	2.02	2.30	2.60	2.91	3.25	3.60	3.97	4.35	5.14	5.98	6.86	7.77	8.70
1.00	1.26	1.42	1.62	1.86	2.11	2.39	2.69	3.00	3.33	3.68	4.04	4.80	5.59	6.43	7.31	8.19
1.20	1.07	1.21	1.39	1.59	1.81	2.06	2.32	2.60	2.89	3.21	3.53	4.21	4.93	5.70	6.48	7.30
1.40	0.929	1.05	1.21	1.39	1.59	1.81	2.04	2.29	2.55	2.83	3.12	3.74	4.40	5.09	5.80	6.55
1.60	0.820	0.931	1.07	1.23	1.41	1.61	1.82	2.04	2.28	2.53	2.79	3.35	3.95	4.59	5.24	5.93
1.80	0.733	0.833	0.959	1.11	1.27	1.45	1.64	1.84	2.05	2.28	2.52	3.04	3.59	4.18	4.78	5.41
2.00	0.663	0.753	0.868	1.00	1.15	1.31	1.49	1.67	1.87	2.08	2.30	2.77	3.29	3.83	4.39	4.97
2.20	0.605	0.688	0.793	0.916	1.05	1.20	1.36	1.53	1.72	1.91	2.11	2.55	3.03	3.53	4.05	4.60
2.40	0.557	0.633	0.731	0.844	0.971	1.11	1.26	1.42	1.59	1.77	1.95	2.36	2.80	3.27	3.76	4.27
2.60	0.516	0.587	0.677	0.781	0.900	1.03	1.17	1.32	1.47	1.64	1.82	2.20	2.61	3.05	3.50	3.98
2.80	0.480	0.545	0.631	0.728	0.839	0.959	1.09	1.23	1.38	1.53	1.70	2.05	2.44	2.85	3.28	3.73
3.00	0.448	0.511	0.589	0.681	0.785	0.899	1.02	1.15	1.29	1.44	1.59	1.93	2.29	2.67	3.08	3.51

Table 8–7 (continued)
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 30°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with

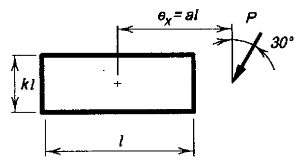
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where

P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	5.15	5.58	6.02	6.46	6.89	7.33	7.77	8.20	8.64	9.08	9.52	10.4	11.3	12.1	13.0	13.9
0.100	4.49	4.93	5.36	5.81	6.28	6.77	7.26	7.75	8.24	8.73	9.20	10.1	11.1	12.0	12.9	13.8
0.150	4.09	4.51	4.94	5.38	5.84	6.32	6.82	7.33	7.84	8.35	8.85	9.83	10.8	11.7	12.7	13.6
0.200	3.76	4.15	4.56	4.98	5.43	5.90	6.39	6.91	7.42	7.94	8.46	9.47	10.5	11.4	12.4	13.3
0.250	3.47	3.83	4.22	4.63	5.07	5.52	6.01	6.51	7.03	7.54	8.06	9.09	10.1	11.1	12.1	13.0
0.300	3.20	3.54	3.91	4.32	4.74	5.19	5.67	6.16	6.67	7.18	7.70	8.73	9.75	10.7	11.7	12.7
0.400	2.75	3.05	3.39	3.77	4.18	4.62	5.07	5.54	6.03	6.52	7.03	8.05	9.08	10.1	11.1	12.1
0.500	2.39	2.66	2.98	3.33	3.72	4.13	4.56	5.02	5.48	5.95	6.43	7.43	8.44	9.45	10.4	11.4
0.600	2.11	2.35	2.64	2.97	3.34	3.73	4.13	4.56	5.00	5.45	5.91	6.87	7.84	8.82	9.80	10.8
0.700	1.87	2.10	2.37	2.67	3.01	3.38	3.76	4.17	4.58	5.01	5.45	6.37	7.29	8.23	9.20	10.2
0.800	1.68	1.89	2.14	2.43	2.75	3.09	3.45	3.83	4.22	4.63	5.05	5.92	6.80	7.70	8.64	9.59
0.900	1.53	1.72	1.95	2.22	2.51	2.83	3.17	3.53	3.90	4.29	4.69	5.52	6.35	7.22	8.12	9.05
1.00	1.40	1.57	1.79	2.04	2.32	2.62	2.94	3.27	3.63	3.99	4.37	5.16	5.96	6.79	7.65	8.55
1.20	1.19	1.34	1.53	1.75	2.00	2.27	2.55	2.85	3.17	3.50	3.84	4.56	5.30	6.05	6.84	7.68
1.40	1.03	1.17	1.34	1.53	1.75	1.99	2.25	2.52	2.80	3.10	3.41	4.07	4.75	5.44	6.17	6.94
1.60	0.912	1.03	1.19	1.36	1.56	1.78	2.01	2.25	2.51	2.78	3.07	3.67	4.30	4.94	5.61	6.32
1.80	0.816	0.925	1.06	1.23	1.40	1.60	1.81	2.03	2.27	2.52	2.78	3.33	3.91	4.51	5.14	5.80
2.00	0.739	0.839	0.964	1.11	1.28	1.45	1.65	1.85	2.07	2.30	2.54	3.05	3.59	4.15	4.73	5.35
2.20	0.675	0.765	0.883	1.02	1.17	1.33	1.51	1.70	1.90	2.11	2.34	2.81	3.31	3.83	4.38	4.96
2.40	0.620	0.704	0.812	0.937	1.08	1.23	1.40	1.57	1.76	1.95	2.16	2.60	3.07	3.56	4.08	4.62
2.60	0.575	0.652	0.753	0.869	1.00	1.14	1.30	1.46	1.63	1.82	2.01	2.43	2.86	3.32	3.81	4.33
2.80	0.535	0.608	0.701	0.811	0.932	1.07	1.21	1.36	1.53	1.70	1.88	2.27	2.68	3.11	3.57	4.06
3.00	0.500	0.568	0.656	0.759	0.872	0.999	1.13	1.28	1.43	1.59	1.76	2.13	2.52	2.93	3.36	3.83

Table 8–7 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 45°

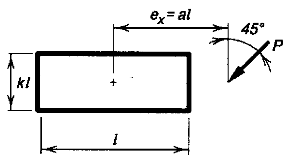
Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with

$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



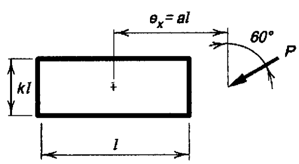
a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	4.68	5.15	5.62	6.09	6.55	7.02	7.49	7.96	8.43	8.89	9.36	10.3	11.2	12.2	13.1	14.0
0.100	4.49	4.99	5.48	5.97	6.45	6.94	7.43	7.93	8.41	8.90	9.39	10.4	11.4	12.3	13.3	14.3
0.150	4.18	4.69	5.19	5.67	6.16	6.65	7.15	7.65	8.15	8.65	9.15	10.1	11.1	12.1	13.1	14.1
0.200	3.92	4.39	4.87	5.36	5.84	6.33	6.83	7.33	7.84	8.34	8.85	9.86	10.9	11.9	12.9	13.9
0.250	3.69	4.13	4.58	5.05	5.52	6.01	6.50	7.00	7.51	8.02	8.53	9.54	10.6	11.6	12.6	13.6
0.300	3.48	3.88	4.31	4.76	5.22	5.69	6.18	6.67	7.17	7.68	8.20	9.21	10.2	11.3	12.3	13.3
0.400	3.09	3.45	3.83	4.25	4.68	5.13	5.59	6.07	6.56	7.06	7.56	8.55	9.57	10.6	11.6	12.7
0.500	2.75	3.07	3.42	3.81	4.22	4.65	5.09	5.55	6.03	6.51	7.01	7.95	8.94	10.0	11.0	12.0
0.600	2.46	2.75	3.07	3.43	3.82	4.23	4.66	5.11	5.57	6.04	6.52	7.43	8.38	9.36	10.4	11.4
0.700	2.21	2.48	2.78	3.11	3.48	3.88	4.29	4.72	5.17	5.62	6.08	6.96	7.87	8.82	9.80	10.8
0.800	2.00	2.25	2.53	2.85	3.19	3.57	3.97	4.38	4.81	5.24	5.69	6.54	7.42	8.33	9.29	10.3
0.900	1.83	2.05	2.31	2.61	2.95	3.31	3.68	4.08	4.49	4.91	5.33	6.16	7.00	7.89	8.81	9.76
1.00	1.68	1.88	2.13	2.41	2.73	3.07	3.43	3.81	4.20	4.60	5.01	5.81	6.63	7.48	8.37	9.30
1.20	1.43	1.62	1.84	2.09	2.38	2.69	3.01	3.35	3.71	4.08	4.46	5.20	5.97	6.77	7.59	8.46
1.40	1.25	1.41	1.61	1.84	2.10	2.38	2.68	2.99	3.31	3.65	4.00	4.69	5.41	6.16	6.94	7.75
1.60	1.11	1.25	1.43	1.64	1.88	2.13	2.40	2.69	2.99	3.30	3.62	4.26	4.94	5.64	6.38	7.14
1.80	0.993	1.12	1.29	1.48	1.69	1.93	2.18	2.44	2.72	3.00	3.30	3.90	4.53	5.20	5.89	6.61
2.00	0.900	1.02	1.17	1.35	1.54	1.76	1.99	2.23	2.49	2.75	3.03	3.59	4.18	4.80	5.46	6.14
2.20	0.823	0.932	1.07	1.23	1.42	1.61	1.83	2.05	2.29	2.54	2.79	3.32	3.88	4.47	5.09	5.73
2.40	0.757	0.859	0.988	1.14	1.31	1.49	1.69	1.90	2.12	2.35	2.59	3.09	3.61	4.17	4.75	5.37
2.60	0.701	0.796	0.916	1.06	1.21	1.39	1.57	1.77	1.98	2.19	2.42	2.89	3.38	3.91	4.46	5.04
2.80	0.652	0.741	0.855	0.985	1.13	1.30	1.47	1.65	1.85	2.05	2.27	2.70	3.17	3.67	4.20	4.75
3.00	0.611	0.693	0.800	0.924	1.06	1.21	1.38	1.55	1.74	1.93	2.13	2.54	2.99	3.46	3.97	4.50

Table 8–7 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 60°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x / l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



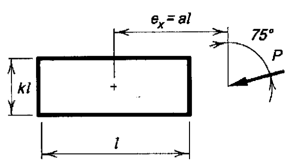
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	4.37	4.89	5.40	5.91	6.43	6.94	7.46	7.97	8.49	9.00	9.52	10.5	11.6	12.6	13.6	14.7
0.100	4.26	4.79	5.31	5.82	6.34	6.86	7.37	7.89	8.40	8.92	9.44	10.5	11.5	12.6	13.6	14.7
0.150	4.13	4.67	5.20	5.71	6.22	6.73	7.25	7.75	8.26	8.77	9.29	10.3	11.3	12.4	13.5	14.5
0.200	3.97	4.51	5.05	5.57	6.08	6.58	7.08	7.59	8.09	8.59	9.10	10.1	11.1	12.2	13.2	14.3
0.250	3.85	4.36	4.87	5.39	5.90	6.40	6.90	7.40	7.89	8.39	8.89	9.90	10.9	11.9	13.0	14.0
0.300	3.74	4.22	4.72	5.22	5.72	6.21	6.70	7.19	7.68	8.17	8.67	9.67	10.7	11.7	12.7	13.7
0.400	3.50	3.94	4.40	4.87	5.36	5.84	6.32	6.78	7.25	7.73	8.21	9.19	10.2	11.2	12.2	13.3
0.500	3.26	3.66	4.09	4.53	5.00	5.47	5.94	6.40	6.86	7.31	7.78	8.72	9.70	10.7	11.7	12.7
0.600	3.02	3.38	3.78	4.21	4.65	5.11	5.57	6.03	6.48	6.93	7.38	8.30	9.25	10.2	11.2	12.2
0.700	2.79	3.13	3.51	3.91	4.33	4.77	5.22	5.68	6.12	6.56	7.00	7.91	8.83	9.78	10.8	11.8
0.800	2.58	2.90	3.25	3.63	4.04	4.46	4.90	5.35	5.79	6.22	6.65	7.54	8.45	9.38	10.3	11.3
0.900	2.39	2.69	3.02	3.38	3.77	4.18	4.61	5.04	5.48	5.90	6.33	7.20	8.09	9.01	10.0	10.9
1.00	2.23	2.50	2.82	3.16	3.53	3.93	4.34	4.77	5.19	5.61	6.02	6.88	7.76	8.66	9.59	10.5
1.20	1.94	2.19	2.47	2.78	3.13	3.49	3.88	4.28	4.69	5.09	5.48	6.30	7.15	8.02	8.91	9.83
1.40	1.71	1.93	2.19	2.47	2.79	3.13	3.50	3.88	4.26	4.64	5.02	5.80	6.61	7.44	8.31	9.19
1.60	1.53	1.73	1.96	2.22	2.52	2.84	3.18	3.54	3.90	4.25	4.61	5.35	6.13	6.93	7.76	8.62
1.80	1.38	1.56	1.77	2.02	2.30	2.60	2.92	3.25	3.59	3.92	4.26	4.97	5.70	6.47	7.27	8.09
2.00	1.25	1.42	1.61	1.85	2.11	2.38	2.69	3.00	3.31	3.63	3.95	4.62	5.33	6.06	6.83	7.63
2.20	1.15	1.30	1.48	1.70	1.94	2.21	2.49	2.78	3.08	3.37	3.68	4.32	4.99	5.69	6.43	7.19
2.40	1.06	1.20	1.37	1.57	1.80	2.05	2.31	2.59	2.87	3.15	3.44	4.05	4.69	5.36	6.07	6.81
2.60	0.981	1.11	1.27	1.46	1.68	1.91	2.16	2.42	2.69	2.95	3.23	3.81	4.42	5.07	5.74	6.45
2.80	0.915	1.04	1.19	1.37	1.57	1.79	2.03	2.27	2.53	2.78	3.04	3.59	4.18	4.79	5.45	6.12
3.00	0.857	0.972	1.12	1.29	1.47	1.68	1.91	2.14	2.38	2.62	2.87	3.40	3.95	4.55	5.18	5.83

Table 8–7 (continued)
Coefficients C
for Eccentrically Loaded Weld Groups
Angle = 75°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



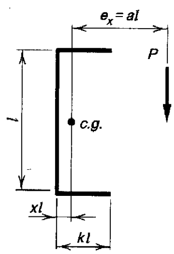
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	3.85	4.39	4.94	5.48	6.03	6.57	7.12	7.66	8.21	8.75	9.30	10.4	11.5	12.6	13.7	14.7
0.100	3.81	4.35	4.90	5.45	5.99	6.54	7.09	7.63	8.18	8.73	9.27	10.4	11.5	12.6	13.6	14.8
0.150	3.85	4.31	4.86	5.41	5.95	6.50	7.04	7.58	8.13	8.67	9.21	10.3	11.4	12.5	13.6	14.7
0.200	3.84	4.26	4.81	5.36	5.90	6.44	6.98	7.52	8.06	8.60	9.14	10.2	11.3	12.4	13.5	14.5
0.250	3.83	4.23	4.75	5.30	5.84	6.38	6.91	7.45	7.99	8.53	9.06	10.1	11.2	12.3	13.3	14.4
0.300	3.81	4.22	4.73	5.24	5.77	6.31	6.84	7.38	7.91	8.44	8.97	10.0	11.1	12.2	13.2	14.3
0.400	3.78	4.21	4.68	5.17	5.67	6.18	6.69	7.21	7.72	8.24	8.76	9.81	10.9	11.9	13.0	14.0
0.500	3.72	4.17	4.63	5.10	5.59	6.07	6.57	7.07	7.57	8.07	8.57	9.59	10.6	11.7	12.7	13.7
0.600	3.65	4.10	4.55	5.01	5.48	5.96	6.44	6.92	7.41	7.90	8.40	9.39	10.4	11.4	12.4	13.5
0.700	3.55	4.00	4.45	4.91	5.36	5.83	6.30	6.77	7.25	7.73	8.21	9.19	10.2	11.2	12.2	13.2
0.800	3.45	3.88	4.33	4.78	5.23	5.68	6.14	6.61	7.07	7.54	8.02	8.98	10.0	10.9	11.9	12.9
0.900	3.35	3.76	4.20	4.65	5.09	5.54	5.98	6.43	6.89	7.36	7.83	8.77	9.73	10.7	11.7	12.7
1.00	3.23	3.64	4.06	4.51	4.94	5.38	5.82	6.26	6.71	7.17	7.63	8.57	9.52	10.5	11.5	12.5
1.20	3.00	3.38	3.78	4.20	4.64	5.06	5.49	5.92	6.36	6.80	7.25	8.16	9.09	10.0	11.0	12.0
1.40	2.77	3.13	3.51	3.91	4.33	4.75	5.17	5.59	6.01	6.44	6.88	7.77	8.69	9.62	10.6	11.5
1.60	2.57	2.89	3.25	3.64	4.05	4.46	4.86	5.27	5.68	6.10	6.53	7.41	8.30	9.22	10.2	11.1
1.80	2.38	2.68	3.02	3.39	3.78	4.18	4.58	4.97	5.38	5.79	6.20	7.06	7.94	8.84	9.77	10.7
2.00	2.21	2.49	2.81	3.16	3.54	3.93	4.32	4.70	5.10	5.49	5.90	6.74	7.60	8.49	9.40	10.3
2.20	2.05	2.32	2.62	2.95	3.31	3.69	4.08	4.45	4.83	5.22	5.62	6.44	7.29	8.16	9.06	10.0
2.40	1.92	2.17	2.45	2.77	3.11	3.48	3.86	4.22	4.59	4.97	5.36	6.16	6.99	7.85	8.73	9.64
2.60	1.79	2.03	2.30	2.60	2.93	3.29	3.65	4.01	4.37	4.74	5.12	5.90	6.72	7.56	8.43	9.31
2.80	1.69	1.91	2.17	2.45	2.77	3.11	3.47	3.82	4.17	4.53	4.90	5.66	6.46	7.28	8.13	9.00
3.00	1.59	1.80	2.04	2.32	2.63	2.95	3.30	3.64	3.98	4.33	4.69	5.43	6.21	7.02	7.85	8.71

Table 8–8
Coefficients *C*
for Eccentrically Loaded Weld Groups
Angle = 0°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



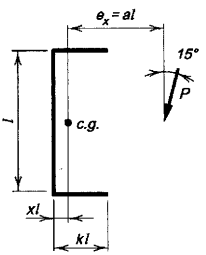
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	1.53	2.09	2.64	3.20	3.76	4.32	4.87	5.43	5.99	6.54	7.10	8.21	9.33	10.4	11.6	12.7
0.100	1.86	2.28	2.78	3.30	3.84	4.37	4.92	5.46	6.01	6.56	7.11	8.21	9.32	10.4	11.5	12.6
0.150	1.83	2.25	2.73	3.23	3.75	4.27	4.80	5.33	5.87	6.40	6.94	8.02	9.12	10.2	11.3	12.4
0.200	1.76	2.18	2.63	3.11	3.60	4.10	4.61	5.13	5.64	6.16	6.68	7.73	8.78	9.83	10.9	12.0
0.250	1.66	2.07	2.51	2.96	3.42	3.90	4.38	4.87	5.37	5.86	6.36	7.37	8.39	9.42	10.5	11.5
0.300	1.55	1.95	2.36	2.79	3.23	3.68	4.14	4.60	5.07	5.55	6.03	7.01	8.00	9.00	10.0	11.0
0.400	1.33	1.69	2.07	2.45	2.84	3.24	3.65	4.07	4.50	4.94	5.39	6.30	7.24	8.19	9.16	10.1
0.500	1.15	1.46	1.79	2.14	2.49	2.85	3.22	3.60	4.00	4.40	4.81	5.67	6.56	7.47	8.40	9.35
0.600	0.997	1.27	1.57	1.88	2.19	2.52	2.85	3.20	3.56	3.94	4.32	5.13	5.97	6.84	7.73	8.65
0.700	0.879	1.12	1.38	1.66	1.95	2.24	2.55	2.87	3.20	3.55	3.91	4.66	5.46	6.29	7.15	8.04
0.800	0.781	0.995	1.23	1.48	1.75	2.02	2.30	2.59	2.90	3.22	3.56	4.27	5.02	5.81	6.64	7.50
0.900	0.703	0.895	1.11	1.33	1.58	1.83	2.09	2.36	2.64	2.94	3.26	3.93	4.64	5.40	6.18	7.00
1.00	0.637	0.812	1.00	1.21	1.44	1.67	1.91	2.16	2.43	2.71	3.01	3.63	4.31	5.02	5.77	6.56
1.20	0.537	0.683	0.844	1.02	1.21	1.42	1.63	1.85	2.08	2.33	2.59	3.15	3.75	4.39	5.07	5.78
1.40	0.464	0.588	0.728	0.881	1.05	1.23	1.41	1.61	1.82	2.04	2.27	2.77	3.31	3.89	4.50	5.15
1.60	0.407	0.516	0.639	0.775	0.923	1.08	1.25	1.43	1.61	1.81	2.02	2.46	2.95	3.47	4.04	4.63
1.80	0.363	0.460	0.569	0.691	0.824	0.969	1.12	1.28	1.45	1.62	1.81	2.22	2.66	3.14	3.65	4.20
2.00	0.327	0.415	0.513	0.623	0.744	0.876	1.01	1.16	1.31	1.47	1.64	2.01	2.42	2.86	3.33	3.85
2.20	0.297	0.377	0.467	0.567	0.677	0.799	0.925	1.06	1.20	1.35	1.50	1.84	2.21	2.62	3.07	3.54
2.40	0.273	0.347	0.428	0.520	0.623	0.735	0.852	0.972	1.10	1.24	1.38	1.70	2.04	2.42	2.83	3.27
2.60	0.252	0.320	0.396	0.480	0.575	0.679	0.788	0.900	1.02	1.15	1.28	1.57	1.90	2.25	2.63	3.05
2.80	0.235	0.297	0.368	0.447	0.535	0.632	0.733	0.837	0.949	1.07	1.19	1.47	1.77	2.10	2.46	2.85
3.00	0.219	0.277	0.343	0.417	0.500	0.591	0.685	0.784	0.888	0.999	1.12	1.37	1.65	1.97	2.31	2.67
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Table 8–8 (continued)
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 15°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



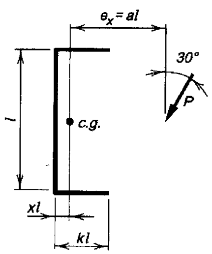
	k															
a	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	1.92	2.47	3.01	3.56	4.10	4.65	5.19	5.74	6.29	6.83	7.38	8.47	9.56	10.6	11.7	12.8
0.100	1.89	2.35	2.87	3.40	3.95	4.50	5.05	5.60	6.14	6.69	7.24	8.33	9.43	10.5	11.6	12.7
0.150	1.84	2.30	2.79	3.29	3.81	4.33	4.86	5.39	5.92	6.45	6.99	8.06	9.13	10.2	11.3	12.4
0.200	1.75	2.21	2.68	3.16	3.65	4.15	4.65	5.16	5.67	6.18	6.69	7.73	8.76	9.81	10.9	11.9
0.250	1.65	2.08	2.54	3.00	3.47	3.94	4.42	4.91	5.39	5.89	6.38	7.38	8.39	9.41	10.4	11.5
0.300	1.55	1.95	2.39	2.82	3.27	3.72	4.18	4.64	5.11	5.58	6.06	7.02	8.01	9.00	10.0	11.0
0.400	1.34	1.69	2.07	2.47	2.87	3.28	3.70	4.12	4.55	4.99	5.43	6.34	7.27	8.22	9.19	10.2
0.500	1.16	1.46	1.80	2.16	2.52	2.89	3.27	3.65	4.05	4.46	4.87	5.73	6.61	7.53	8.46	9.41
0.600	1.01	1.28	1.58	1.89	2.22	2.56	2.90	3.26	3.62	4.00	4.39	5.19	6.04	6.91	7.81	8.73
0.700	0.893	1.13	1.39	1.68	1.97	2.29	2.60	2.93	3.27	3.62	3.98	4.74	5.54	6.37	7.24	8.13
0.800	0.797	1.01	1.25	1.50	1.77	2.06	2.35	2.65	2.96	3.29	3.63	4.35	5.11	5.91	6.74	7.60
0.900	0.719	0.911	1.12	1.35	1.60	1.87	2.14	2.41	2.71	3.01	3.33	4.01	4.73	5.49	6.28	7.11
1.00	0.653	0.828	1.02	1.23	1.46	1.70	1.96	2.21	2.49	2.78	3.08	3.72	4.40	5.12	5.88	6.67
1.20	0.551	0.699	0.861	1.04	1.24	1.45	1.67	1.90	2.14	2.39	2.66	3.23	3.84	4.49	5.18	5.90
1.40	0.476	0.603	0.744	0.900	1.07	1.26	1.46	1.66	1.87	2.10	2.33	2.84	3.39	3.98	4.60	5.26
1.60	0.419	0.531	0.655	0.793	0.944	1.11	1.29	1.47	1.66	1.86	2.08	2.53	3.03	3.57	4.14	4.75
1.80	0.373	0.473	0.584	0.708	0.844	0.995	1.16	1.32	1.49	1.67	1.87	2.28	2.73	3.23	3.75	4.31
2.00	0.337	0.427	0.527	0.639	0.763	0.900	1.05	1.19	1.35	1.52	1.69	2.07	2.49	2.94	3.43	3.95
2.20	0.307	0.388	0.480	0.583	0.696	0.821	0.956	1.09	1.23	1.39	1.55	1.90	2.28	2.70	3.15	3.64
2.40	0.281	0.356	0.440	0.535	0.639	0.755	0.879	1.00	1.14	1.28	1.43	1.75	2.11	2.50	2.92	3.37
2.60	0.260	0.329	0.407	0.493	0.592	0.699	0.813	0.929	1.05	1.19	1.32	1.62	1.96	2.32	2.71	3.14
2.80	0.243	0.307	0.379	0.459	0.549	0.649	0.757	0.865	0.981	1.10	1.23	1.51	1.82	2.16	2.54	2.94
3.00	0.227	0.285	0.353	0.428	0.515	0.608	0.708	0.809	0.917	1.03	1.15	1.42	1.71	2.03	2.38	2.76
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Table 8–8 (continued)
Coefficients *C*
for Eccentrically Loaded Weld Groups
Angle = 30°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x / l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

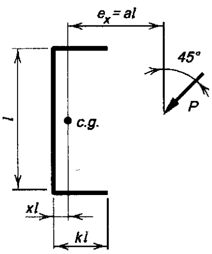


a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.19	2.59	3.00	3.41	3.82	4.22	4.63	5.04	5.45	5.86	6.26	7.08	7.90	8.71	9.53	10.3
0.100	2.02	2.57	3.10	3.62	4.15	4.67	5.19	5.71	6.23	6.76	7.28	8.33	9.37	10.4	11.5	12.5
0.150	1.92	2.43	2.95	3.47	3.98	4.49	5.01	5.52	6.03	6.54	7.06	8.09	9.12	10.2	11.2	12.3
0.200	1.82	2.29	2.79	3.29	3.78	4.27	4.77	5.27	5.77	6.27	6.77	7.78	8.81	9.83	10.9	11.9
0.250	1.71	2.15	2.62	3.10	3.58	4.06	4.53	5.01	5.49	5.97	6.46	7.45	8.45	9.47	10.5	11.5
0.300	1.60	2.01	2.45	2.91	3.37	3.83	4.29	4.75	5.21	5.68	6.15	7.11	8.09	9.09	10.1	11.1
0.400	1.40	1.76	2.14	2.55	2.97	3.40	3.83	4.25	4.69	5.13	5.57	6.48	7.42	8.38	9.36	10.4
0.500	1.23	1.54	1.88	2.24	2.62	3.01	3.41	3.81	4.22	4.63	5.05	5.92	6.81	7.74	8.68	9.65
0.600	1.08	1.36	1.66	1.98	2.33	2.68	3.05	3.43	3.81	4.20	4.60	5.42	6.28	7.17	8.08	9.02
0.700	0.963	1.21	1.48	1.77	2.08	2.41	2.75	3.11	3.46	3.83	4.20	4.99	5.81	6.67	7.56	8.46
0.800	0.863	1.09	1.33	1.60	1.88	2.18	2.50	2.83	3.16	3.51	3.86	4.61	5.40	6.22	7.07	7.94
0.900	0.781	0.985	1.21	1.45	1.71	1.99	2.29	2.59	2.91	3.23	3.57	4.28	5.02	5.81	6.63	7.47
1.00	0.713	0.899	1.10	1.33	1.57	1.83	2.10	2.39	2.69	2.99	3.31	3.98	4.69	5.44	6.22	7.03
1.20	0.605	0.763	0.937	1.13	1.34	1.56	1.81	2.07	2.33	2.60	2.88	3.49	4.13	4.81	5.53	6.28
1.40	0.524	0.661	0.813	0.981	1.17	1.37	1.58	1.81	2.04	2.29	2.55	3.09	3.67	4.29	4.96	5.66
1.60	0.463	0.583	0.717	0.867	1.03	1.21	1.41	1.61	1.82	2.04	2.27	2.77	3.30	3.87	4.48	5.13
1.80	0.413	0.521	0.641	0.776	0.924	1.09	1.27	1.45	1.64	1.84	2.05	2.50	2.99	3.52	4.08	4.69
2.00	0.373	0.471	0.580	0.701	0.836	0.987	1.15	1.32	1.49	1.67	1.87	2.28	2.73	3.22	3.75	4.31
2.20	0.340	0.429	0.529	0.641	0.764	0.903	1.05	1.21	1.36	1.53	1.71	2.09	2.51	2.97	3.45	3.98
2.40	0.312	0.395	0.487	0.589	0.704	0.831	0.969	1.11	1.26	1.41	1.58	1.93	2.32	2.75	3.20	3.70
2.60	0.289	0.364	0.449	0.545	0.652	0.769	0.899	1.03	1.17	1.31	1.46	1.79	2.16	2.55	2.99	3.45
2.80	0.268	0.339	0.419	0.508	0.607	0.717	0.837	0.959	1.09	1.22	1.37	1.67	2.02	2.39	2.80	3.23
3.00	0.251	0.317	0.391	0.475	0.568	0.672	0.784	0.897	1.02	1.14	1.28	1.57	1.89	2.24	2.63	3.04
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Table 8–8 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 45°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$



where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)

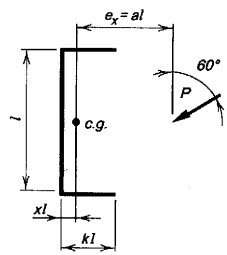
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	2.34	2.81	3.28	3.74	4.21	4.68	5.15	5.62	6.08	6.55	7.02	7.96	8.89	9.83	10.8	11.7
0.100	2.24	2.74	3.24	3.74	4.23	4.73	5.23	5.72	6.22	6.71	7.20	8.19	9.17	10.1	11.1	12.1
0.150	2.09	2.60	3.09	3.58	4.07	4.57	5.06	5.56	6.06	6.55	7.05	8.04	9.03	10.0	11.0	12.0
0.200	1.96	2.44	2.92	3.40	3.88	4.37	4.86	5.36	5.85	6.35	6.84	7.83	8.83	9.82	10.8	11.8
0.250	1.85	2.29	2.75	3.21	3.68	4.16	4.64	5.13	5.62	6.11	6.60	7.58	8.58	9.59	10.6	11.6
0.300	1.74	2.15	2.59	3.03	3.48	3.94	4.41	4.89	5.38	5.86	6.34	7.32	8.31	9.32	10.3	11.3
0.400	1.55	1.91	2.30	2.70	3.12	3.55	3.99	4.44	4.91	5.37	5.83	6.77	7.75	8.75	9.77	10.8
0.500	1.37	1.70	2.05	2.42	2.80	3.20	3.61	4.04	4.48	4.93	5.37	6.27	7.22	8.20	9.20	10.2
0.600	1.23	1.52	1.84	2.18	2.53	2.90	3.29	3.69	4.11	4.54	4.96	5.83	6.72	7.67	8.65	9.64
0.700	1.11	1.37	1.66	1.97	2.30	2.65	3.01	3.39	3.79	4.20	4.61	5.43	6.30	7.20	8.15	9.12
0.800	1.00	1.25	1.51	1.80	2.10	2.43	2.77	3.13	3.51	3.90	4.29	5.08	5.91	6.77	7.69	8.63
0.900	0.913	1.14	1.38	1.65	1.93	2.24	2.56	2.90	3.26	3.64	4.00	4.76	5.56	6.39	7.27	8.18
1.00	0.839	1.05	1.27	1.52	1.79	2.07	2.38	2.70	3.05	3.40	3.75	4.47	5.23	6.04	6.89	7.77
1.20	0.717	0.899	1.10	1.31	1.55	1.80	2.08	2.37	2.68	3.00	3.31	3.97	4.68	5.42	6.21	7.04
1.40	0.625	0.785	0.960	1.15	1.36	1.59	1.84	2.11	2.39	2.67	2.96	3.57	4.22	4.91	5.65	6.41
1.60	0.555	0.696	0.852	1.03	1.21	1.42	1.65	1.89	2.15	2.40	2.67	3.22	3.83	4.47	5.15	5.88
1.80	0.497	0.624	0.765	0.923	1.09	1.29	1.49	1.71	1.95	2.18	2.42	2.94	3.50	4.10	4.74	5.42
2.00	0.451	0.565	0.695	0.837	0.995	1.17	1.36	1.57	1.77	1.99	2.21	2.69	3.21	3.78	4.38	5.02
2.20	0.411	0.517	0.636	0.767	0.913	1.07	1.25	1.44	1.63	1.83	2.04	2.49	2.97	3.50	4.06	4.67
2.40	0.379	0.476	0.585	0.708	0.843	0.993	1.16	1.33	1.51	1.69	1.89	2.30	2.76	3.25	3.79	4.36
2.60	0.351	0.441	0.543	0.656	0.783	0.923	1.08	1.24	1.40	1.57	1.76	2.15	2.58	3.04	3.55	4.09
2.80	0.327	0.411	0.505	0.611	0.731	0.861	1.00	1.16	1.31	1.47	1.64	2.01	2.41	2.86	3.33	3.84
3.00	0.305	0.384	0.473	0.572	0.684	0.808	0.943	1.09	1.23	1.38	1.54	1.89	2.27	2.69	3.14	3.62
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Table 8–8 (continued)
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 60°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x / l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



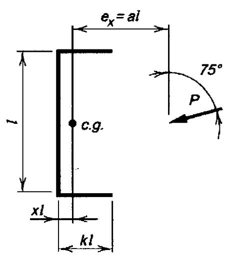
a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.57	3.01	3.45	3.88	4.32	4.76	5.20	5.63	6.07	6.51	6.94	7.82	8.69	9.57	10.4	11.3
0.100	2.43	2.86	3.30	3.75	4.21	4.68	5.14	5.61	6.07	6.53	6.99	7.89	8.78	9.66	10.5	11.4
0.150	2.31	2.74	3.17	3.62	4.07	4.54	5.01	5.49	5.96	6.44	6.91	7.83	8.74	9.63	10.5	11.4
0.200	2.18	2.61	3.04	3.47	3.92	4.39	4.86	5.34	5.83	6.31	6.79	7.73	8.66	9.57	10.5	11.4
0.250	2.07	2.49	2.91	3.33	3.77	4.23	4.70	5.18	5.67	6.16	6.64	7.61	8.55	9.48	10.4	11.3
0.300	1.96	2.37	2.78	3.20	3.63	4.07	4.54	5.02	5.51	5.99	6.49	7.46	8.42	9.36	10.3	11.2
0.400	1.79	2.16	2.54	2.94	3.35	3.77	4.22	4.69	5.17	5.66	6.15	7.14	8.12	9.09	10.0	11.0
0.500	1.63	1.97	2.33	2.71	3.09	3.50	3.93	4.38	4.85	5.33	5.81	6.80	7.79	8.77	9.73	10.7
0.600	1.49	1.81	2.15	2.50	2.87	3.26	3.67	4.10	4.55	5.02	5.50	6.48	7.46	8.42	9.38	10.3
0.700	1.37	1.67	1.99	2.32	2.67	3.04	3.44	3.85	4.29	4.74	5.21	6.16	7.11	8.07	9.03	10.0
0.800	1.26	1.54	1.84	2.16	2.49	2.85	3.23	3.63	4.05	4.48	4.93	5.85	6.78	7.73	8.69	9.65
0.900	1.17	1.43	1.71	2.01	2.33	2.67	3.04	3.43	3.83	4.24	4.68	5.56	6.47	7.40	8.35	9.31
1.00	1.08	1.33	1.60	1.89	2.19	2.52	2.87	3.24	3.63	4.03	4.45	5.30	6.17	7.09	8.02	8.97
1.20	0.945	1.17	1.41	1.67	1.95	2.25	2.57	2.92	3.28	3.65	4.04	4.82	5.65	6.52	7.42	8.34
1.40	0.835	1.03	1.25	1.49	1.75	2.03	2.33	2.65	2.98	3.32	3.68	4.42	5.18	6.01	6.87	7.76
1.60	0.747	0.928	1.13	1.34	1.58	1.84	2.12	2.41	2.72	3.05	3.38	4.07	4.79	5.56	6.38	7.24
1.80	0.675	0.840	1.02	1.22	1.44	1.68	1.94	2.22	2.51	2.81	3.12	3.76	4.44	5.17	5.95	6.76
2.00	0.615	0.767	0.935	1.12	1.32	1.55	1.79	2.05	2.32	2.60	2.89	3.49	4.14	4.83	5.56	6.34
2.20	0.564	0.704	0.860	1.03	1.22	1.43	1.66	1.90	2.15	2.42	2.69	3.26	3.87	4.52	5.22	5.96
2.40	0.521	0.652	0.796	0.957	1.13	1.33	1.54	1.77	2.01	2.26	2.51	3.05	3.63	4.25	4.91	5.61
2.60	0.484	0.605	0.741	0.892	1.06	1.24	1.44	1.66	1.88	2.12	2.36	2.86	3.41	4.01	4.64	5.31
2.80	0.452	0.565	0.692	0.833	0.992	1.17	1.35	1.56	1.77	1.99	2.21	2.70	3.22	3.79	4.39	5.03
3.00	0.423	0.531	0.651	0.783	0.933	1.10	1.27	1.47	1.67	1.88	2.09	2.55	3.05	3.59	4.16	4.78
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Table 8–8 (continued)
Coefficients C
for Eccentrically Loaded Weld Groups
Angle = 75°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



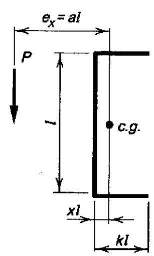
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	2.73	3.11	3.49	3.88	4.26	4.65	5.03	5.42	5.80	6.19	6.57	7.34	8.11	8.88	9.65	10.4
0.100	2.59	2.96	3.35	3.75	4.17	4.58	4.99	5.40	5.80	6.20	6.59	7.37	8.14	8.91	9.68	10.4
0.150	2.50	2.87	3.26	3.67	4.09	4.51	4.94	5.35	5.76	6.17	6.57	7.36	8.13	8.91	9.68	10.4
0.200	2.43	2.79	3.18	3.59	4.01	4.44	4.87	5.29	5.71	6.13	6.53	7.33	8.12	8.90	9.67	10.4
0.250	2.35	2.72	3.10	3.51	3.93	4.36	4.80	5.23	5.65	6.07	6.49	7.30	8.09	8.88	9.65	10.4
0.300	2.28	2.65	3.02	3.43	3.85	4.28	4.72	5.16	5.59	6.02	6.44	7.26	8.06	8.85	9.63	10.4
0.400	2.16	2.52	2.88	3.27	3.69	4.12	4.56	5.01	5.45	5.88	6.31	7.15	7.97	8.78	9.57	10.4
0.500	2.05	2.40	2.75	3.13	3.54	3.97	4.41	4.85	5.30	5.74	6.18	7.03	7.86	8.68	9.48	10.3
0.600	1.94	2.28	2.63	3.00	3.39	3.82	4.26	4.71	5.16	5.61	6.05	6.92	7.77	8.59	9.39	10.2
0.700	1.84	2.17	2.52	2.87	3.26	3.68	4.11	4.56	5.01	5.47	5.92	6.81	7.67	8.50	9.31	10.1
0.800	1.75	2.07	2.41	2.76	3.14	3.54	3.97	4.42	4.87	5.33	5.78	6.68	7.56	8.41	9.24	10.1
0.900	1.67	1.98	2.31	2.65	3.02	3.42	3.84	4.27	4.72	5.18	5.64	6.55	7.45	8.31	9.15	10.0
1.00	1.59	1.89	2.21	2.55	2.91	3.30	3.71	4.14	4.59	5.04	5.50	6.42	7.33	8.21	9.07	9.90
1.20	1.45	1.73	2.04	2.36	2.71	3.08	3.47	3.88	4.32	4.77	5.22	6.14	7.06	7.97	8.85	9.71
1.40	1.33	1.60	1.89	2.19	2.53	2.88	3.25	3.65	4.07	4.50	4.95	5.87	6.79	7.71	8.62	9.51
1.60	1.22	1.48	1.75	2.05	2.36	2.70	3.06	3.44	3.84	4.26	4.70	5.60	6.52	7.44	8.35	9.26
1.80	1.13	1.37	1.63	1.91	2.22	2.54	2.89	3.25	3.63	4.04	4.46	5.34	6.24	7.17	8.09	9.01
2.00	1.05	1.27	1.52	1.79	2.08	2.40	2.73	3.07	3.44	3.84	4.24	5.09	5.99	6.90	7.81	8.73
2.20	0.973	1.19	1.43	1.68	1.96	2.26	2.58	2.92	3.27	3.65	4.04	4.86	5.74	6.62	7.53	8.44
2.40	0.911	1.12	1.34	1.59	1.86	2.14	2.45	2.77	3.11	3.47	3.85	4.65	5.49	6.35	7.24	8.15
2.60	0.855	1.05	1.27	1.50	1.76	2.03	2.33	2.64	2.96	3.31	3.67	4.45	5.26	6.10	6.97	7.87
2.80	0.805	0.992	1.20	1.42	1.67	1.93	2.22	2.51	2.83	3.16	3.51	4.27	5.04	5.86	6.71	7.60
3.00	0.760	0.939	1.13	1.35	1.59	1.84	2.11	2.40	2.71	3.03	3.36	4.09	4.84	5.63	6.47	7.34
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Table 8–9
Coefficients C
for Eccentrically Loaded Weld Groups
Angle = 0°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



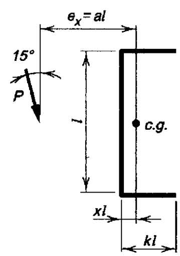
a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	1.53	2.09	2.64	3.20	3.76	4.32	4.87	5.43	5.99	6.54	7.10	8.21	9.33	10.4	11.6	12.7
0.100	1.86	2.30	2.80	3.30	3.81	4.32	4.83	5.34	5.84	6.34	6.84	7.84	8.84	9.83	10.8	11.8
0.150	1.83	2.26	2.73	3.21	3.69	4.18	4.66	5.14	5.62	6.10	6.58	7.54	8.51	9.48	10.5	11.4
0.200	1.76	2.18	2.62	3.07	3.53	3.99	4.45	4.91	5.37	5.83	6.30	7.22	8.16	9.11	10.1	11.0
0.250	1.66	2.06	2.48	2.91	3.35	3.79	4.23	4.67	5.11	5.55	6.00	6.90	7.81	8.73	9.67	10.6
0.300	1.55	1.93	2.33	2.74	3.15	3.57	3.99	4.41	4.84	5.27	5.70	6.57	7.46	8.37	9.28	10.2
0.400	1.33	1.67	2.03	2.39	2.77	3.15	3.53	3.92	4.32	4.71	5.12	5.94	6.79	7.65	8.54	9.44
0.500	1.15	1.44	1.75	2.07	2.41	2.76	3.11	3.47	3.83	4.21	4.59	5.36	6.17	7.00	7.85	8.73
0.600	0.997	1.26	1.52	1.81	2.11	2.43	2.77	3.10	3.44	3.79	4.14	4.88	5.64	6.44	7.26	8.11
0.700	0.879	1.10	1.34	1.60	1.88	2.17	2.48	2.80	3.11	3.44	3.78	4.47	5.20	5.95	6.75	7.56
0.800	0.781	0.980	1.19	1.43	1.69	1.96	2.25	2.55	2.84	3.15	3.46	4.12	4.81	5.53	6.29	7.07
0.900	0.703	0.881	1.08	1.29	1.53	1.78	2.05	2.33	2.61	2.90	3.20	3.81	4.47	5.16	5.88	6.63
1.00	0.637	0.799	0.979	1.18	1.40	1.63	1.88	2.14	2.41	2.69	2.96	3.55	4.17	4.83	5.52	6.24
1.20	0.537	0.673	0.828	1.00	1.19	1.40	1.61	1.84	2.08	2.33	2.58	3.11	3.67	4.27	4.90	5.56
1.40	0.464	0.581	0.716	0.868	1.03	1.22	1.41	1.61	1.83	2.05	2.28	2.76	3.27	3.82	4.40	5.00
1.60	0.407	0.511	0.629	0.765	0.913	1.07	1.25	1.43	1.62	1.83	2.04	2.47	2.95	3.45	3.98	4.55
1.80	0.363	0.455	0.563	0.683	0.817	0.963	1.12	1.28	1.46	1.64	1.84	2.24	2.67	3.14	3.63	4.15
2.00	0.327	0.411	0.508	0.617	0.739	0.872	1.01	1.16	1.32	1.49	1.67	2.04	2.45	2.87	3.33	3.82
2.20	0.297	0.373	0.463	0.563	0.675	0.796	0.925	1.06	1.21	1.36	1.53	1.88	2.25	2.65	3.08	3.54
2.40	0.273	0.343	0.424	0.517	0.620	0.731	0.852	0.980	1.11	1.26	1.41	1.73	2.08	2.46	2.86	3.29
2.60	0.252	0.317	0.392	0.477	0.573	0.677	0.789	0.908	1.03	1.16	1.30	1.61	1.94	2.29	2.67	3.07
2.80	0.235	0.295	0.365	0.444	0.533	0.629	0.733	0.844	0.960	1.08	1.21	1.50	1.81	2.14	2.50	2.88
3.00	0.219	0.276	0.341	0.415	0.499	0.589	0.687	0.791	0.897	1.01	1.13	1.40	1.70	2.01	2.35	2.71
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Table 8–9 (continued)
Coefficients C
for Eccentrically Loaded Weld Groups
Angle = 15°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



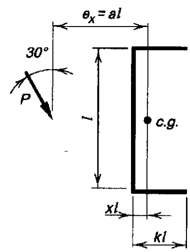
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	1.92	2.47	3.01	3.56	4.10	4.65	5.19	5.74	6.29	6.83	7.38	8.47	9.56	10.6	11.7	12.8
0.100	1.89	2.37	2.87	3.38	3.88	4.38	4.88	5.38	5.87	6.37	6.86	7.85	8.84	9.84	10.8	11.9
0.150	1.84	2.30	2.78	3.26	3.74	4.21	4.69	5.16	5.63	6.10	6.57	7.52	8.47	9.44	10.4	11.4
0.200	1.75	2.19	2.65	3.11	3.56	4.02	4.47	4.92	5.37	5.82	6.27	7.18	8.10	9.04	10.0	10.9
0.250	1.65	2.06	2.49	2.93	3.37	3.80	4.23	4.66	5.09	5.53	5.96	6.84	7.74	8.65	9.58	10.5
0.300	1.55	1.93	2.33	2.74	3.16	3.58	3.99	4.41	4.82	5.24	5.66	6.52	7.39	8.28	9.18	10.1
0.400	1.34	1.67	2.02	2.38	2.75	3.13	3.52	3.92	4.31	4.70	5.10	5.90	6.73	7.59	8.47	9.37
0.500	1.16	1.45	1.75	2.06	2.39	2.74	3.10	3.47	3.84	4.22	4.60	5.37	6.17	7.00	7.85	8.72
0.600	1.01	1.27	1.52	1.80	2.10	2.42	2.75	3.10	3.46	3.82	4.18	4.92	5.69	6.48	7.30	8.14
0.700	0.893	1.12	1.35	1.60	1.87	2.17	2.47	2.80	3.13	3.48	3.83	4.53	5.26	6.02	6.81	7.61
0.800	0.797	0.996	1.21	1.44	1.69	1.96	2.24	2.55	2.86	3.19	3.52	4.19	4.89	5.61	6.36	7.15
0.900	0.719	0.896	1.09	1.31	1.54	1.79	2.05	2.33	2.63	2.93	3.25	3.89	4.55	5.25	5.97	6.72
1.00	0.653	0.815	0.995	1.19	1.41	1.64	1.89	2.15	2.43	2.72	3.02	3.63	4.26	4.92	5.61	6.34
1.20	0.551	0.688	0.844	1.02	1.21	1.41	1.63	1.86	2.10	2.36	2.63	3.18	3.76	4.36	5.00	5.67
1.40	0.476	0.595	0.731	0.884	1.05	1.23	1.43	1.63	1.85	2.08	2.32	2.83	3.35	3.91	4.50	5.12
1.60	0.419	0.524	0.645	0.781	0.931	1.09	1.27	1.45	1.65	1.86	2.08	2.54	3.03	3.54	4.08	4.66
1.80	0.373	0.468	0.576	0.699	0.835	0.983	1.14	1.31	1.49	1.67	1.88	2.30	2.75	3.23	3.73	4.26
2.00	0.337	0.423	0.521	0.632	0.756	0.891	1.03	1.19	1.35	1.53	1.71	2.10	2.52	2.96	3.43	3.93
2.20	0.307	0.385	0.475	0.577	0.691	0.815	0.947	1.09	1.24	1.40	1.57	1.93	2.32	2.73	3.17	3.64
2.40	0.281	0.353	0.436	0.531	0.635	0.749	0.872	1.00	1.14	1.29	1.45	1.79	2.15	2.53	2.95	3.38
2.60	0.260	0.327	0.404	0.491	0.588	0.695	0.808	0.931	1.06	1.20	1.34	1.65	2.00	2.36	2.75	3.16
2.80	0.243	0.304	0.376	0.457	0.547	0.647	0.753	0.867	0.988	1.12	1.25	1.54	1.87	2.21	2.58	2.97
3.00	0.227	0.284	0.351	0.427	0.512	0.604	0.704	0.812	0.925	1.04	1.17	1.44	1.75	2.08	2.42	2.80
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Table 8–9 (continued)
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 30°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x / l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



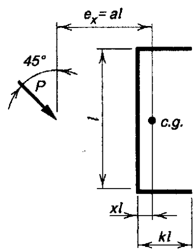
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	2.19	2.59	3.00	3.41	3.82	4.22	4.63	5.04	5.45	5.86	6.26	7.08	7.90	8.71	9.53	10.3
0.100	2.02	2.56	3.06	3.55	4.02	4.50	4.99	5.47	5.95	6.43	6.92	7.90	8.90	9.91	10.9	11.9
0.150	1.92	2.41	2.90	3.37	3.83	4.28	4.74	5.19	5.65	6.12	6.59	7.54	8.51	9.51	10.5	11.5
0.200	1.82	2.27	2.72	3.16	3.60	4.03	4.46	4.89	5.34	5.78	6.23	7.16	8.11	9.09	10.1	11.1
0.250	1.71	2.13	2.55	2.96	3.37	3.78	4.19	4.60	5.02	5.46	5.90	6.79	7.72	8.68	9.66	10.7
0.300	1.60	1.99	2.38	2.77	3.16	3.55	3.94	4.34	4.75	5.18	5.61	6.48	7.38	8.31	9.26	10.2
0.400	1.40	1.74	2.08	2.42	2.78	3.13	3.50	3.89	4.29	4.70	5.12	5.95	6.80	7.69	8.60	9.54
0.500	1.23	1.52	1.82	2.12	2.44	2.78	3.14	3.51	3.89	4.28	4.68	5.49	6.31	7.16	8.04	8.94
0.600	1.08	1.34	1.60	1.88	2.18	2.49	2.83	3.18	3.54	3.92	4.30	5.09	5.87	6.69	7.52	8.40
0.700	0.963	1.19	1.43	1.69	1.96	2.26	2.57	2.90	3.24	3.60	3.97	4.73	5.48	6.26	7.07	7.91
0.800	0.863	1.07	1.29	1.53	1.78	2.06	2.35	2.66	2.98	3.32	3.67	4.40	5.13	5.88	6.66	7.46
0.900	0.781	0.968	1.17	1.39	1.63	1.89	2.16	2.45	2.76	3.08	3.41	4.11	4.81	5.53	6.28	7.06
1.00	0.713	0.884	1.07	1.28	1.51	1.74	2.00	2.27	2.56	2.86	3.18	3.85	4.52	5.22	5.94	6.69
1.20	0.605	0.751	0.916	1.10	1.30	1.51	1.74	1.98	2.24	2.51	2.79	3.40	4.03	4.67	5.34	6.04
1.40	0.524	0.652	0.799	0.961	1.14	1.33	1.53	1.75	1.98	2.23	2.49	3.04	3.62	4.22	4.84	5.49
1.60	0.463	0.576	0.707	0.852	1.01	1.19	1.37	1.57	1.77	2.00	2.23	2.74	3.28	3.83	4.41	5.03
1.80	0.413	0.515	0.633	0.765	0.912	1.07	1.24	1.42	1.61	1.81	2.03	2.49	2.99	3.51	4.05	4.62
2.00	0.373	0.465	0.573	0.695	0.827	0.972	1.13	1.29	1.47	1.65	1.85	2.28	2.75	3.23	3.74	4.28
2.20	0.340	0.425	0.524	0.635	0.757	0.891	1.03	1.19	1.35	1.52	1.71	2.11	2.54	2.99	3.46	3.97
2.40	0.312	0.391	0.481	0.585	0.699	0.821	0.955	1.10	1.25	1.41	1.58	1.95	2.36	2.78	3.23	3.70
2.60	0.289	0.361	0.447	0.541	0.648	0.763	0.887	1.02	1.16	1.31	1.47	1.82	2.20	2.60	3.02	3.47
2.80	0.268	0.336	0.415	0.504	0.604	0.712	0.828	0.952	1.08	1.23	1.37	1.70	2.06	2.44	2.84	3.26
3.00	0.251	0.315	0.388	0.472	0.565	0.667	0.776	0.892	1.02	1.15	1.29	1.60	1.93	2.29	2.67	3.08
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Table 8–9 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 45°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ (ϕ = 0.75, Ω = 2.00)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



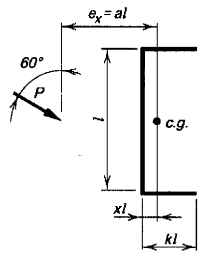
a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.34	2.81	3.28	3.74	4.21	4.68	5.15	5.62	6.08	6.55	7.02	7.96	8.89	9.83	10.8	11.7
0.100	2.24	2.72	3.17	3.61	4.05	4.49	4.95	5.41	5.88	6.35	6.83	7.79	8.75	9.73	10.7	11.7
0.150	2.09	2.57	3.00	3.41	3.82	4.24	4.68	5.13	5.59	6.07	6.55	7.52	8.49	9.47	10.5	11.5
0.200	1.96	2.41	2.83	3.21	3.59	3.99	4.41	4.85	5.30	5.77	6.25	7.21	8.19	9.17	10.2	11.2
0.250	1.85	2.27	2.66	3.02	3.38	3.76	4.16	4.59	5.03	5.49	5.95	6.91	7.89	8.88	9.87	10.9
0.300	1.74	2.13	2.50	2.86	3.20	3.57	3.96	4.37	4.81	5.25	5.70	6.64	7.59	8.56	9.55	10.6
0.400	1.55	1.89	2.22	2.55	2.89	3.24	3.62	4.01	4.42	4.84	5.28	6.18	7.11	8.05	8.99	10.0
0.500	1.37	1.68	1.98	2.28	2.61	2.96	3.31	3.69	4.08	4.49	4.91	5.78	6.69	7.59	8.51	9.45
0.600	1.23	1.50	1.77	2.06	2.37	2.70	3.05	3.41	3.78	4.17	4.57	5.42	6.30	7.18	8.07	8.99
0.700	1.11	1.35	1.60	1.87	2.17	2.48	2.81	3.15	3.51	3.89	4.27	5.09	5.94	6.79	7.67	8.56
0.800	1.00	1.23	1.46	1.72	1.99	2.29	2.60	2.93	3.27	3.63	4.00	4.78	5.61	6.43	7.28	8.16
0.900	0.913	1.12	1.34	1.58	1.84	2.12	2.42	2.73	3.06	3.40	3.76	4.51	5.30	6.10	6.93	7.78
1.00	0.839	1.03	1.24	1.47	1.71	1.97	2.26	2.55	2.87	3.20	3.54	4.26	5.03	5.79	6.59	7.43
1.20	0.717	0.885	1.07	1.28	1.50	1.73	1.98	2.25	2.54	2.84	3.16	3.82	4.53	5.26	6.00	6.78
1.40	0.625	0.775	0.941	1.13	1.33	1.54	1.77	2.01	2.27	2.55	2.84	3.46	4.12	4.80	5.50	6.23
1.60	0.555	0.687	0.839	1.01	1.19	1.38	1.59	1.82	2.06	2.31	2.58	3.15	3.76	4.41	5.06	5.75
1.80	0.497	0.617	0.755	0.908	1.08	1.26	1.45	1.65	1.87	2.11	2.35	2.89	3.46	4.07	4.69	5.33
2.00	0.451	0.560	0.687	0.828	0.981	1.15	1.33	1.52	1.72	1.94	2.17	2.66	3.20	3.77	4.35	4.97
2.20	0.411	0.512	0.628	0.759	0.903	1.06	1.22	1.40	1.59	1.79	2.00	2.47	2.97	3.51	4.06	4.64
2.40	0.379	0.472	0.580	0.701	0.835	0.979	1.13	1.30	1.47	1.66	1.86	2.30	2.77	3.28	3.81	4.35
2.60	0.351	0.437	0.537	0.651	0.776	0.911	1.06	1.21	1.38	1.55	1.74	2.15	2.60	3.08	3.57	4.10
2.80	0.327	0.408	0.501	0.607	0.724	0.852	0.988	1.13	1.29	1.46	1.63	2.02	2.44	2.90	3.37	3.87
3.00	0.305	0.381	0.469	0.569	0.680	0.800	0.929	1.07	1.21	1.37	1.53	1.90	2.30	2.74	3.18	3.66
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Table 8–9 (continued)
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 60°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x / l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



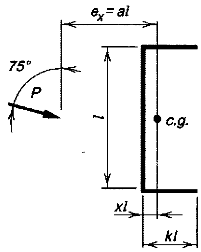
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	2.57	3.01	3.45	3.88	4.32	4.76	5.20	5.63	6.07	6.51	6.94	7.82	8.69	9.57	10.4	11.3
0.100	2.43	2.84	3.23	3.63	4.04	4.47	4.92	5.37	5.82	6.28	6.73	7.64	8.54	9.44	10.3	11.2
0.150	2.31	2.70	3.07	3.44	3.84	4.26	4.70	5.15	5.60	6.06	6.52	7.45	8.36	9.27	10.2	11.1
0.200	2.18	2.58	2.92	3.27	3.65	4.05	4.48	4.92	5.37	5.84	6.30	7.24	8.17	9.09	10.0	10.9
0.250	2.07	2.46	2.79	3.12	3.49	3.89	4.30	4.73	5.17	5.62	6.08	7.01	7.96	8.90	9.83	10.8
0.300	1.96	2.34	2.67	2.99	3.36	3.75	4.15	4.57	5.01	5.45	5.90	6.81	7.74	8.68	9.63	10.6
0.400	1.79	2.13	2.45	2.77	3.12	3.49	3.89	4.30	4.72	5.16	5.60	6.49	7.39	8.30	9.22	10.2
0.500	1.63	1.95	2.25	2.57	2.91	3.26	3.65	4.05	4.46	4.89	5.32	6.21	7.11	8.01	8.92	9.82
0.600	1.49	1.78	2.07	2.39	2.71	3.06	3.43	3.81	4.22	4.64	5.07	5.95	6.85	7.74	8.65	9.55
0.700	1.37	1.64	1.92	2.22	2.54	2.88	3.23	3.60	3.99	4.40	4.82	5.69	6.58	7.49	8.39	9.30
0.800	1.26	1.52	1.78	2.07	2.38	2.71	3.05	3.41	3.79	4.18	4.59	5.45	6.33	7.23	8.14	9.05
0.900	1.17	1.41	1.66	1.94	2.23	2.55	2.88	3.23	3.60	3.98	4.38	5.21	6.08	6.98	7.88	8.79
1.00	1.08	1.31	1.56	1.82	2.10	2.41	2.73	3.07	3.42	3.79	4.18	4.99	5.85	6.73	7.63	8.54
1.20	0.945	1.15	1.37	1.62	1.88	2.16	2.46	2.77	3.10	3.45	3.82	4.59	5.41	6.27	7.14	8.04
1.40	0.835	1.02	1.23	1.45	1.69	1.95	2.23	2.53	2.84	3.16	3.50	4.23	5.01	5.83	6.69	7.55
1.60	0.747	0.917	1.11	1.32	1.54	1.78	2.04	2.31	2.60	2.91	3.23	3.92	4.66	5.44	6.27	7.09
1.80	0.675	0.831	1.01	1.20	1.41	1.63	1.87	2.13	2.40	2.69	3.00	3.64	4.35	5.09	5.87	6.67
2.00	0.615	0.759	0.923	1.10	1.30	1.51	1.73	1.97	2.23	2.50	2.79	3.40	4.06	4.78	5.52	6.28
2.20	0.564	0.699	0.851	1.02	1.20	1.40	1.61	1.84	2.07	2.33	2.60	3.18	3.81	4.49	5.20	5.93
2.40	0.521	0.645	0.788	0.947	1.12	1.31	1.50	1.71	1.94	2.18	2.44	2.99	3.59	4.23	4.91	5.61
2.60	0.484	0.601	0.735	0.883	1.05	1.22	1.41	1.61	1.82	2.05	2.29	2.82	3.39	4.00	4.65	5.31
2.80	0.452	0.561	0.687	0.827	0.981	1.15	1.33	1.51	1.72	1.93	2.16	2.66	3.21	3.79	4.41	5.05
3.00	0.423	0.527	0.645	0.779	0.924	1.08	1.25	1.43	1.62	1.83	2.05	2.52	3.04	3.60	4.20	4.81
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Table 8–9 (continued)
Coefficients C
for Eccentrically Loaded Weld Groups
Angle = 75°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	2.73	3.11	3.49	3.88	4.26	4.65	5.03	5.42	5.80	6.19	6.57	7.34	8.11	8.88	9.65	10.4
0.100	2.59	2.94	3.31	3.69	4.09	4.49	4.91	5.32	5.75	6.13	6.51	7.28	8.05	8.82	9.59	10.4
0.150	2.50	2.85	3.19	3.56	3.95	4.35	4.77	5.18	5.60	6.01	6.42	7.23	8.00	8.77	9.54	10.3
0.200	2.43	2.76	3.09	3.46	3.84	4.24	4.64	5.05	5.47	5.89	6.30	7.13	7.94	8.72	9.50	10.3
0.250	2.35	2.68	3.01	3.37	3.76	4.15	4.55	4.95	5.35	5.76	6.17	7.02	7.83	8.65	9.44	10.2
0.300	2.28	2.61	2.93	3.29	3.68	4.07	4.47	4.87	5.28	5.68	6.08	6.89	7.73	8.56	9.37	10.2
0.400	2.16	2.48	2.80	3.15	3.53	3.93	4.33	4.74	5.15	5.55	5.95	6.75	7.54	8.34	9.17	10.0
0.500	2.05	2.36	2.68	3.02	3.40	3.79	4.19	4.61	5.02	5.43	5.84	6.64	7.43	8.22	9.01	9.80
0.600	1.94	2.25	2.57	2.90	3.27	3.66	4.06	4.47	4.89	5.31	5.72	6.54	7.35	8.14	8.92	9.70
0.700	1.84	2.15	2.46	2.79	3.15	3.53	3.93	4.34	4.76	5.18	5.61	6.44	7.26	8.06	8.85	9.63
0.800	1.75	2.05	2.36	2.69	3.03	3.41	3.80	4.21	4.63	5.06	5.48	6.33	7.16	7.97	8.78	9.57
0.900	1.67	1.95	2.26	2.58	2.93	3.29	3.68	4.09	4.51	4.93	5.36	6.21	7.06	7.89	8.69	9.50
1.00	1.59	1.87	2.17	2.49	2.82	3.18	3.57	3.97	4.38	4.81	5.23	6.09	6.95	7.79	8.62	9.42
1.20	1.45	1.71	2.00	2.31	2.63	2.98	3.35	3.74	4.14	4.56	4.99	5.85	6.72	7.58	8.43	9.27
1.40	1.33	1.58	1.85	2.15	2.46	2.80	3.15	3.52	3.92	4.33	4.74	5.61	6.48	7.36	8.23	9.07
1.60	1.22	1.46	1.72	2.01	2.31	2.63	2.97	3.33	3.71	4.11	4.52	5.37	6.24	7.12	8.00	8.87
1.80	1.13	1.36	1.61	1.88	2.17	2.48	2.80	3.15	3.51	3.90	4.30	5.13	6.00	6.88	7.76	8.65
2.00	1.05	1.26	1.50	1.76	2.04	2.34	2.65	2.99	3.34	3.71	4.10	4.91	5.77	6.64	7.53	8.42
2.20	0.973	1.18	1.41	1.66	1.93	2.21	2.51	2.84	3.18	3.53	3.91	4.70	5.54	6.41	7.29	8.18
2.40	0.911	1.11	1.33	1.57	1.82	2.09	2.39	2.70	3.03	3.37	3.74	4.51	5.32	6.18	7.06	7.94
2.60	0.855	1.04	1.25	1.48	1.73	1.99	2.27	2.57	2.89	3.22	3.57	4.32	5.12	5.96	6.83	7.71
2.80	0.805	0.984	1.19	1.41	1.64	1.89	2.17	2.45	2.76	3.08	3.42	4.15	4.93	5.75	6.61	7.48
3.00	0.760	0.932	1.13	1.34	1.57	1.81	2.07	2.35	2.64	2.95	3.28	3.99	4.75	5.55	6.39	7.25
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Table 8–10
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 0°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with

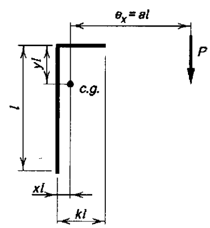
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where

P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



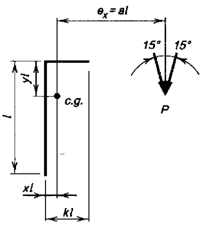
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	1.53	1.81	2.09	2.37	2.64	2.92	3.20	3.48	3.76	4.04	4.32	4.87	5.43	5.99	6.54	7.10
0.100	1.86	2.05	2.28	2.53	2.79	3.05	3.31	3.57	3.84	4.11	4.38	4.93	5.48	6.03	6.58	7.13
0.150	1.83	2.03	2.25	2.49	2.74	2.99	3.24	3.49	3.75	4.01	4.28	4.81	5.35	5.89	6.45	7.01
0.200	1.76	1.97	2.18	2.40	2.63	2.87	3.11	3.36	3.60	3.85	4.11	4.62	5.14	5.66	6.20	6.73
0.250	1.66	1.86	2.07	2.28	2.50	2.73	2.95	3.18	3.42	3.66	3.90	4.40	4.90	5.42	5.94	6.47
0.300	1.55	1.73	1.94	2.15	2.35	2.56	2.78	3.00	3.22	3.45	3.69	4.17	4.66	5.17	5.68	6.20
0.400	1.33	1.49	1.66	1.85	2.05	2.24	2.43	2.63	2.84	3.05	3.27	3.72	4.20	4.69	5.19	5.69
0.500	1.15	1.29	1.43	1.59	1.76	1.95	2.13	2.31	2.50	2.69	2.90	3.33	3.78	4.25	4.74	5.23
0.600	0.997	1.12	1.25	1.38	1.53	1.70	1.87	2.04	2.21	2.39	2.58	2.99	3.42	3.87	4.34	4.82
0.700	0.879	0.985	1.10	1.22	1.35	1.50	1.65	1.81	1.97	2.14	2.32	2.70	3.11	3.55	4.00	4.46
0.800	0.781	0.877	0.976	1.08	1.20	1.33	1.48	1.63	1.78	1.94	2.10	2.46	2.85	3.26	3.70	4.15
0.900	0.703	0.788	0.877	0.975	1.08	1.20	1.33	1.47	1.62	1.76	1.92	2.26	2.62	3.02	3.43	3.86
1.00	0.637	0.715	0.796	0.884	0.981	1.09	1.21	1.34	1.48	1.62	1.76	2.08	2.43	2.80	3.19	3.61
1.20	0.537	0.603	0.671	0.744	0.827	0.920	1.03	1.14	1.26	1.38	1.51	1.79	2.10	2.44	2.79	3.17
1.40	0.464	0.519	0.577	0.641	0.713	0.795	0.887	0.991	1.10	1.21	1.32	1.57	1.85	2.15	2.48	2.83
1.60	0.407	0.456	0.508	0.564	0.627	0.700	0.781	0.873	0.971	1.07	1.17	1.40	1.65	1.92	2.22	2.54
1.80	0.363	0.407	0.452	0.503	0.559	0.624	0.699	0.780	0.869	0.956	1.05	1.25	1.48	1.74	2.01	2.31
2.00	0.327	0.367	0.408	0.453	0.504	0.564	0.631	0.705	0.787	0.865	0.951	1.14	1.35	1.58	1.83	2.11
2.20	0.297	0.333	0.371	0.412	0.459	0.513	0.575	0.644	0.719	0.791	0.869	1.04	1.24	1.45	1.69	1.94
2.40	0.273	0.305	0.340	0.379	0.421	0.472	0.528	0.591	0.660	0.728	0.800	0.959	1.14	1.34	1.56	1.79
2.60	0.252	0.283	0.315	0.349	0.389	0.436	0.488	0.547	0.611	0.673	0.740	0.889	1.06	1.24	1.45	1.67
2.80	0.235	0.263	0.292	0.325	0.361	0.405	0.453	0.508	0.568	0.627	0.689	0.828	0.985	1.16	1.35	1.56
3.00	0.219	0.245	0.272	0.304	0.339	0.379	0.424	0.475	0.531	0.587	0.644	0.775	0.923	1.09	1.27	1.46
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

Table 8–10 (continued)
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 15°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	1.92	2.20	2.47	2.74	3.01	3.29	3.56	3.83	4.10	4.38	4.65	5.19	5.74	6.28	6.83	7.37
0.100	1.89	2.08	2.30	2.54	2.79	3.04	3.30	3.57	3.84	4.12	4.39	4.97	5.56	6.12	6.69	7.26
0.150	1.84	2.04	2.25	2.47	2.70	2.94	3.18	3.43	3.68	3.94	4.19	4.72	5.27	5.83	6.39	6.97
0.200	1.75	1.97	2.17	2.38	2.59	2.81	3.04	3.28	3.51	3.76	4.00	4.51	5.02	5.55	6.09	6.63
0.250	1.65	1.86	2.07	2.26	2.46	2.67	2.88	3.11	3.33	3.56	3.80	4.29	4.79	5.30	5.82	6.35
0.300	1.55	1.74	1.95	2.13	2.32	2.52	2.72	2.93	3.15	3.37	3.60	4.07	4.55	5.06	5.57	6.09
0.400	1.34	1.51	1.69	1.87	2.04	2.22	2.40	2.59	2.79	2.99	3.21	3.65	4.12	4.61	5.10	5.61
0.500	1.16	1.30	1.46	1.63	1.79	1.95	2.11	2.29	2.47	2.66	2.86	3.29	3.73	4.20	4.68	5.18
0.600	1.01	1.14	1.27	1.42	1.57	1.72	1.87	2.03	2.20	2.38	2.57	2.97	3.40	3.85	4.31	4.79
0.700	0.893	1.00	1.12	1.25	1.39	1.53	1.67	1.82	1.98	2.15	2.32	2.70	3.11	3.54	3.99	4.45
0.800	0.797	0.896	0.999	1.11	1.24	1.37	1.51	1.64	1.79	1.95	2.11	2.47	2.86	3.27	3.71	4.16
0.900	0.719	0.807	0.900	1.00	1.11	1.23	1.37	1.50	1.63	1.78	1.93	2.27	2.64	3.03	3.45	3.88
1.00	0.653	0.733	0.817	0.908	1.01	1.12	1.24	1.37	1.50	1.64	1.78	2.10	2.45	2.82	3.22	3.64
1.20	0.551	0.619	0.689	0.765	0.852	0.949	1.05	1.17	1.29	1.41	1.53	1.82	2.13	2.47	2.83	3.22
1.40	0.476	0.535	0.595	0.661	0.736	0.821	0.915	1.02	1.12	1.23	1.35	1.60	1.88	2.19	2.52	2.87
1.60	0.419	0.469	0.523	0.581	0.647	0.723	0.807	0.900	0.993	1.09	1.20	1.42	1.68	1.96	2.26	2.59
1.80	0.373	0.419	0.467	0.519	0.577	0.645	0.721	0.805	0.891	0.980	1.07	1.28	1.52	1.77	2.05	2.35
2.00	0.337	0.377	0.420	0.468	0.521	0.583	0.652	0.728	0.808	0.888	0.975	1.17	1.38	1.62	1.88	2.15
2.20	0.307	0.344	0.383	0.425	0.475	0.531	0.595	0.665	0.739	0.812	0.892	1.07	1.27	1.49	1.73	1.98
2.40	0.281	0.316	0.351	0.391	0.436	0.488	0.547	0.611	0.680	0.748	0.821	0.984	1.17	1.37	1.60	1.84
2.60	0.260	0.292	0.324	0.361	0.403	0.451	0.505	0.565	0.629	0.693	0.761	0.913	1.09	1.28	1.49	1.71
2.80	0.243	0.271	0.301	0.336	0.375	0.419	0.469	0.525	0.585	0.645	0.709	0.852	1.01	1.19	1.39	1.60
3.00	0.227	0.253	0.281	0.313	0.349	0.392	0.439	0.491	0.548	0.604	0.664	0.797	0.948	1.12	1.30	1.50
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

Table 8–10 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 30°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with

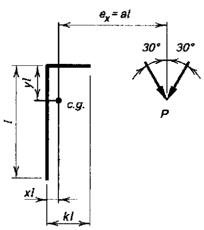
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where

P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x / l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



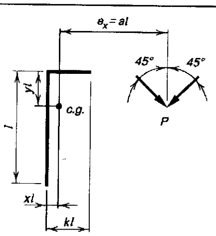
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	2.19	2.44	2.70	2.96	3.21	3.47	3.73	3.99	4.24	4.50	4.76	5.27	5.79	6.30	6.82	7.33
0.100	2.02	2.24	2.47	2.70	2.94	3.18	3.44	3.69	3.95	4.21	4.48	5.02	5.57	6.12	6.67	7.22
0.150	1.92	2.13	2.34	2.55	2.77	3.00	3.23	3.47	3.71	3.96	4.21	4.74	5.27	5.82	6.38	6.93
0.200	1.82	2.02	2.23	2.43	2.64	2.85	3.07	3.29	3.52	3.76	4.00	4.50	5.01	5.55	6.09	6.65
0.250	1.71	1.90	2.11	2.31	2.50	2.70	2.91	3.12	3.34	3.57	3.80	4.28	4.78	5.30	5.83	6.37
0.300	1.60	1.79	1.98	2.18	2.37	2.55	2.75	2.95	3.17	3.38	3.61	4.08	4.57	5.08	5.60	6.13
0.400	1.40	1.57	1.74	1.92	2.10	2.27	2.45	2.64	2.84	3.04	3.26	3.71	4.18	4.67	5.17	5.69
0.500	1.23	1.38	1.53	1.70	1.87	2.02	2.19	2.36	2.55	2.74	2.94	3.37	3.83	4.30	4.79	5.30
0.600	1.08	1.21	1.36	1.51	1.66	1.81	1.96	2.12	2.30	2.48	2.67	3.08	3.52	3.98	4.46	4.95
0.700	0.963	1.08	1.21	1.35	1.49	1.63	1.77	1.92	2.08	2.25	2.43	2.82	3.24	3.69	4.15	4.63
0.800	0.863	0.972	1.09	1.21	1.34	1.47	1.61	1.75	1.90	2.06	2.23	2.60	3.01	3.43	3.89	4.35
0.900	0.781	0.880	0.981	1.09	1.22	1.34	1.47	1.60	1.74	1.90	2.06	2.41	2.80	3.21	3.64	4.09
1.00	0.713	0.801	0.895	0.996	1.11	1.23	1.35	1.47	1.61	1.75	1.91	2.24	2.61	3.00	3.42	3.85
1.20	0.605	0.680	0.757	0.843	0.939	1.04	1.16	1.27	1.39	1.52	1.66	1.96	2.29	2.65	3.04	3.44
1.40	0.524	0.588	0.656	0.729	0.813	0.907	1.01	1.11	1.22	1.34	1.46	1.73	2.04	2.36	2.72	3.09
1.60	0.463	0.519	0.577	0.643	0.716	0.800	0.891	0.989	1.09	1.19	1.30	1.55	1.83	2.13	2.45	2.80
1.80	0.413	0.463	0.516	0.573	0.640	0.715	0.799	0.888	0.977	1.07	1.18	1.40	1.66	1.93	2.23	2.55
2.00	0.373	0.419	0.465	0.519	0.577	0.647	0.723	0.805	0.888	0.976	1.07	1.28	1.51	1.77	2.05	2.35
2.20	0.340	0.381	0.424	0.472	0.527	0.589	0.660	0.736	0.813	0.893	0.980	1.17	1.39	1.63	1.89	2.17
2.40	0.312	0.351	0.389	0.433	0.484	0.541	0.607	0.677	0.749	0.824	0.905	1.08	1.28	1.51	1.75	2.01
2.60	0.289	0.324	0.360	0.401	0.447	0.501	0.561	0.628	0.695	0.764	0.840	1.01	1.19	1.40	1.63	1.88
2.80	0.268	0.301	0.335	0.373	0.416	0.465	0.523	0.584	0.648	0.713	0.784	0.939	1.11	1.31	1.53	1.76
3.00	0.251	0.281	0.313	0.348	0.389	0.436	0.488	0.547	0.607	0.668	0.733	0.880	1.05	1.23	1.43	1.65
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

Table 8–10 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 45°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



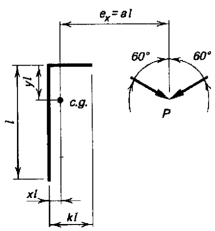
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	2.34	2.57	2.81	3.04	3.28	3.51	3.74	3.98	4.21	4.45	4.68	5.15	5.62	6.08	6.55	7.02
0.100	2.24	2.44	2.65	2.86	3.07	3.29	3.52	3.76	4.00	4.24	4.49	5.01	5.53	6.07	6.60	7.13
0.150	2.09	2.28	2.48	2.68	2.89	3.11	3.33	3.56	3.79	4.03	4.28	4.79	5.32	5.86	6.40	6.95
0.200	1.96	2.14	2.33	2.53	2.74	2.95	3.16	3.38	3.61	3.84	4.08	4.58	5.10	5.64	6.19	6.74
0.250	1.85	2.02	2.21	2.40	2.61	2.81	3.01	3.22	3.44	3.67	3.90	4.39	4.90	5.43	5.98	6.53
0.300	1.74	1.91	2.09	2.27	2.47	2.67	2.87	3.07	3.29	3.51	3.73	4.21	4.71	5.24	5.78	6.33
0.400	1.55	1.70	1.86	2.04	2.23	2.42	2.60	2.79	2.99	3.21	3.43	3.89	4.38	4.89	5.41	5.95
0.500	1.37	1.51	1.67	1.83	2.01	2.19	2.36	2.55	2.74	2.94	3.15	3.59	4.07	4.57	5.08	5.61
0.600	1.23	1.36	1.50	1.65	1.82	1.99	2.15	2.33	2.51	2.70	2.90	3.33	3.79	4.28	4.79	5.31
0.700	1.11	1.23	1.36	1.50	1.65	1.82	1.97	2.13	2.31	2.49	2.68	3.10	3.55	4.02	4.51	5.03
0.800	1.00	1.11	1.24	1.37	1.51	1.67	1.81	1.97	2.13	2.30	2.49	2.89	3.32	3.79	4.27	4.77
0.900	0.913	1.02	1.13	1.26	1.39	1.54	1.67	1.82	1.97	2.14	2.32	2.71	3.12	3.57	4.04	4.53
1.00	0.839	0.936	1.04	1.16	1.29	1.42	1.55	1.69	1.84	2.00	2.17	2.54	2.94	3.37	3.82	4.30
1.20	0.717	0.804	0.899	1.00	1.11	1.23	1.35	1.47	1.61	1.76	1.91	2.25	2.62	3.02	3.45	3.89
1.40	0.625	0.703	0.785	0.873	0.973	1.08	1.19	1.31	1.43	1.56	1.70	2.01	2.36	2.73	3.12	3.54
1.60	0.555	0.623	0.695	0.773	0.861	0.961	1.07	1.17	1.28	1.40	1.53	1.82	2.13	2.48	2.85	3.24
1.80	0.497	0.559	0.621	0.692	0.772	0.863	0.960	1.06	1.16	1.27	1.39	1.65	1.95	2.27	2.61	2.97
2.00	0.451	0.505	0.563	0.627	0.699	0.783	0.872	0.963	1.06	1.16	1.27	1.51	1.79	2.08	2.41	2.75
2.20	0.411	0.461	0.513	0.572	0.639	0.715	0.799	0.884	0.973	1.07	1.17	1.40	1.65	1.93	2.23	2.55
2.40	0.379	0.424	0.473	0.527	0.588	0.657	0.736	0.817	0.900	0.988	1.08	1.29	1.53	1.79	2.07	2.38
2.60	0.351	0.393	0.437	0.487	0.544	0.609	0.681	0.760	0.836	0.919	1.01	1.21	1.43	1.67	1.94	2.23
2.80	0.327	0.365	0.407	0.453	0.507	0.567	0.635	0.709	0.780	0.857	0.943	1.13	1.34	1.57	1.82	2.09
3.00	0.305	0.341	0.381	0.424	0.473	0.531	0.595	0.664	0.732	0.805	0.884	1.06	1.26	1.47	1.72	1.98
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

Table 8–10 (continued)
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 60°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x / l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



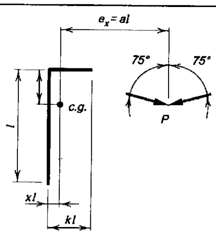
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	3.43	3.72	4.01	4.30	4.60	4.89	5.18	5.47	5.76	6.05	6.34	6.93	7.51	6.07	6.51	6.94
0.100	3.24	3.45	3.67	3.92	4.18	4.46	4.76	5.06	5.38	5.70	6.04	6.72	7.42	6.07	6.56	7.03
0.150	3.07	3.27	3.49	3.73	3.99	4.27	4.57	4.87	5.19	5.51	5.84	6.52	7.23	5.96	6.48	6.97
0.200	2.90	3.10	3.32	3.56	3.82	4.10	4.39	4.70	5.00	5.32	5.64	6.33	7.04	5.83	6.37	6.89
0.250	2.75	2.95	3.16	3.40	3.66	3.94	4.23	4.53	4.83	5.14	5.46	6.14	6.85	5.69	6.24	6.78
0.300	2.62	2.81	3.03	3.26	3.52	3.79	4.08	4.37	4.67	4.97	5.29	5.96	6.66	5.55	6.11	6.66
0.400	2.38	2.57	2.77	3.00	3.25	3.51	3.79	4.08	4.36	4.66	4.97	5.62	6.31	5.28	5.84	6.41
0.500	2.17	2.35	2.55	2.77	3.00	3.26	3.53	3.81	4.08	4.37	4.68	5.32	5.99	5.03	5.58	6.15
0.600	1.99	2.16	2.35	2.56	2.79	3.03	3.30	3.56	3.83	4.11	4.40	5.03	5.70	4.80	5.35	5.91
0.700	1.82	1.99	2.17	2.37	2.59	2.83	3.08	3.34	3.59	3.87	4.15	4.77	5.42	4.59	5.13	5.68
0.800	1.68	1.84	2.01	2.21	2.42	2.65	2.89	3.13	3.38	3.65	3.93	4.52	5.17	4.39	4.91	5.46
0.900	1.55	1.70	1.87	2.06	2.26	2.48	2.72	2.95	3.19	3.44	3.72	4.30	4.93	4.19	4.71	5.25
1.00	1.44	1.59	1.75	1.93	2.12	2.33	2.56	2.78	3.01	3.26	3.52	4.09	4.70	4.01	4.52	5.05
1.20	1.26	1.39	1.54	1.70	1.88	2.08	2.28	2.49	2.70	2.94	3.18	3.71	4.29	3.69	4.17	4.68
1.40	1.11	1.24	1.37	1.52	1.69	1.87	2.06	2.24	2.44	2.66	2.89	3.39	3.94	3.39	3.86	4.34
1.60	0.996	1.11	1.23	1.37	1.52	1.69	1.86	2.04	2.22	2.42	2.64	3.11	3.63	3.13	3.58	4.04
1.80	0.900	1.00	1.12	1.25	1.39	1.54	1.70	1.86	2.04	2.23	2.43	2.87	3.35	2.91	3.33	3.77
2.00	0.820	0.916	1.02	1.14	1.27	1.42	1.56	1.71	1.88	2.05	2.24	2.65	3.11	2.71	3.10	3.52
2.20	0.752	0.841	0.942	1.05	1.17	1.31	1.45	1.58	1.74	1.90	2.08	2.47	2.90	2.53	2.90	3.31
2.40	0.695	0.779	0.869	0.969	1.08	1.21	1.34	1.47	1.62	1.77	1.94	2.31	2.71	2.37	2.73	3.11
2.60	0.645	0.724	0.807	0.900	1.00	1.12	1.25	1.38	1.51	1.66	1.81	2.16	2.54	2.23	2.57	2.93
2.80	0.603	0.676	0.754	0.839	0.937	1.05	1.17	1.29	1.42	1.56	1.70	2.03	2.40	2.10	2.42	2.77
3.00	0.564	0.633	0.706	0.786	0.878	0.983	1.10	1.21	1.33	1.46	1.61	1.92	2.26	1.98	2.29	2.63
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

Table 8–10 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 75°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



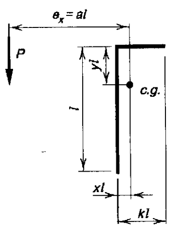
a	*k* = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.73	2.92	3.11	3.30	3.49	3.69	3.88	4.07	4.26	4.46	4.65	5.03	5.42	5.80	6.19	6.57
0.100	2.59	2.68	2.81	2.97	3.15	3.36	3.58	3.82	4.07	4.33	4.58	5.06	5.45	5.84	6.22	6.61
0.150	2.50	2.60	2.73	2.90	3.08	3.29	3.51	3.75	4.00	4.26	4.52	5.02	5.45	5.84	6.23	6.62
0.200	2.43	2.53	2.66	2.83	3.01	3.22	3.44	3.68	3.93	4.19	4.45	4.98	5.45	5.85	6.23	6.62
0.250	2.35	2.46	2.59	2.76	2.94	3.15	3.37	3.61	3.86	4.12	4.39	4.92	5.41	5.85	6.24	6.62
0.300	2.28	2.39	2.53	2.69	2.88	3.08	3.31	3.54	3.79	4.05	4.32	4.86	5.37	5.83	6.24	6.62
0.400	2.16	2.27	2.41	2.57	2.76	2.96	3.18	3.41	3.66	3.92	4.19	4.72	5.27	5.77	6.21	6.62
0.500	2.05	2.16	2.30	2.46	2.64	2.84	3.06	3.29	3.54	3.80	4.06	4.59	5.13	5.66	6.15	6.59
0.600	1.94	2.05	2.19	2.35	2.53	2.73	2.95	3.18	3.42	3.68	3.93	4.46	5.00	5.54	6.06	6.53
0.700	1.84	1.96	2.09	2.25	2.43	2.63	2.84	3.07	3.31	3.56	3.81	4.33	4.87	5.42	5.95	6.45
0.800	1.75	1.87	2.00	2.16	2.33	2.53	2.74	2.97	3.20	3.45	3.69	4.21	4.74	5.30	5.84	6.35
0.900	1.67	1.78	1.91	2.07	2.24	2.43	2.64	2.87	3.10	3.34	3.58	4.09	4.62	5.17	5.72	6.24
1.00	1.59	1.70	1.83	1.99	2.16	2.35	2.55	2.77	3.00	3.23	3.47	3.97	4.50	5.05	5.60	6.13
1.20	1.45	1.56	1.69	1.83	2.00	2.18	2.38	2.59	2.82	3.04	3.27	3.75	4.27	4.81	5.36	5.91
1.40	1.33	1.43	1.56	1.70	1.86	2.04	2.23	2.43	2.64	2.85	3.08	3.55	4.06	4.58	5.13	5.68
1.60	1.22	1.32	1.44	1.58	1.73	1.90	2.09	2.29	2.49	2.69	2.90	3.37	3.86	4.37	4.91	5.46
1.80	1.13	1.23	1.34	1.47	1.62	1.79	1.97	2.16	2.34	2.54	2.75	3.19	3.67	4.17	4.70	5.24
2.00	1.05	1.14	1.25	1.38	1.52	1.68	1.85	2.03	2.21	2.40	2.60	3.03	3.49	3.99	4.49	5.03
2.20	0.973	1.07	1.17	1.30	1.43	1.58	1.75	1.92	2.09	2.27	2.46	2.88	3.33	3.81	4.31	4.83
2.40	0.911	1.00	1.10	1.22	1.35	1.50	1.66	1.82	1.98	2.15	2.34	2.74	3.18	3.64	4.13	4.64
2.60	0.855	0.941	1.04	1.15	1.28	1.42	1.57	1.72	1.88	2.05	2.23	2.61	3.04	3.49	3.96	4.46
2.80	0.805	0.888	0.984	1.09	1.21	1.35	1.49	1.64	1.79	1.95	2.12	2.50	2.91	3.34	3.81	4.29
3.00	0.760	0.841	0.933	1.04	1.15	1.28	1.42	1.56	1.70	1.86	2.02	2.39	2.78	3.21	3.66	4.13
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

Table 8–11
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 0°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x / l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



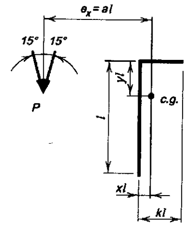
	k															
a	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	1.53	1.81	2.09	2.37	2.64	2.92	3.20	3.48	3.76	4.04	4.32	4.87	5.43	5.99	6.54	7.10
0.100	1.86	2.07	2.32	2.57	2.83	3.08	3.32	3.55	3.77	3.98	4.19	4.60	5.02	5.45	5.89	6.35
0.150	1.83	2.04	2.27	2.51	2.74	2.97	3.18	3.38	3.58	3.78	3.97	4.37	4.79	5.21	5.66	6.11
0.200	1.76	1.96	2.17	2.38	2.59	2.78	2.97	3.17	3.36	3.56	3.75	4.15	4.57	4.99	5.43	5.89
0.250	1.66	1.85	2.03	2.22	2.40	2.58	2.76	2.94	3.14	3.34	3.55	3.95	4.36	4.78	5.22	5.67
0.300	1.55	1.72	1.89	2.06	2.22	2.39	2.56	2.74	2.94	3.14	3.35	3.75	4.16	4.58	5.01	5.46
0.400	1.33	1.48	1.62	1.76	1.90	2.05	2.22	2.40	2.58	2.78	2.98	3.40	3.80	4.21	4.64	5.07
0.500	1.15	1.28	1.40	1.52	1.65	1.79	1.94	2.11	2.29	2.48	2.67	3.08	3.48	3.88	4.30	4.72
0.600	0.997	1.11	1.22	1.33	1.45	1.58	1.72	1.88	2.05	2.23	2.41	2.80	3.20	3.59	3.99	4.41
0.700	0.879	0.977	1.07	1.18	1.29	1.41	1.54	1.68	1.84	2.01	2.19	2.56	2.95	3.33	3.72	4.12
0.800	0.781	0.869	0.959	1.05	1.16	1.27	1.39	1.53	1.67	1.83	2.00	2.35	2.73	3.10	3.47	3.87
0.900	0.703	0.781	0.864	0.953	1.05	1.15	1.27	1.39	1.53	1.68	1.83	2.17	2.53	2.89	3.26	3.63
1.00	0.637	0.709	0.785	0.868	0.956	1.05	1.16	1.28	1.40	1.54	1.69	2.01	2.36	2.71	3.06	3.42
1.20	0.537	0.597	0.663	0.735	0.812	0.899	0.992	1.09	1.21	1.33	1.46	1.75	2.06	2.40	2.72	3.06
1.40	0.464	0.516	0.573	0.635	0.704	0.781	0.864	0.955	1.05	1.17	1.28	1.54	1.83	2.14	2.44	2.76
1.60	0.407	0.453	0.504	0.559	0.621	0.691	0.764	0.845	0.936	1.03	1.14	1.38	1.64	1.92	2.21	2.51
1.80	0.363	0.404	0.449	0.499	0.555	0.617	0.685	0.759	0.840	0.929	1.03	1.24	1.48	1.74	2.02	2.29
2.00	0.327	0.364	0.405	0.451	0.501	0.559	0.620	0.688	0.763	0.844	0.933	1.13	1.35	1.59	1.85	2.11
2.20	0.297	0.332	0.369	0.411	0.457	0.509	0.567	0.629	0.697	0.772	0.855	1.04	1.24	1.47	1.71	1.95
2.40	0.273	0.304	0.339	0.377	0.420	0.468	0.521	0.579	0.643	0.712	0.788	0.957	1.15	1.36	1.58	1.82
2.60	0.252	0.281	0.313	0.348	0.388	0.433	0.483	0.536	0.595	0.660	0.731	0.889	1.07	1.26	1.47	1.69
2.80	0.235	0.261	0.291	0.324	0.361	0.403	0.449	0.500	0.555	0.615	0.681	0.829	0.996	1.18	1.37	1.58
3.00	0.219	0.244	0.272	0.303	0.337	0.376	0.420	0.467	0.519	0.575	0.637	0.776	0.933	1.10	1.28	1.48
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

Table 8–11 (continued)
Coefficients *C*
for Eccentrically Loaded Weld Groups
Angle = 15°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	1.92	2.20	2.47	2.74	3.01	3.29	3.56	3.83	4.10	4.38	4.65	5.19	5.74	6.28	6.83	7.37
0.100	1.89	2.09	2.32	2.55	2.79	3.02	3.25	3.48	3.71	3.93	4.16	4.60	5.04	5.49	5.95	6.42
0.150	1.84	2.05	2.26	2.48	2.70	2.92	3.13	3.35	3.56	3.77	3.97	4.40	4.83	5.27	5.72	6.18
0.200	1.75	1.96	2.17	2.37	2.58	2.78	2.97	3.15	3.34	3.55	3.76	4.18	4.61	5.04	5.49	5.95
0.250	1.65	1.85	2.05	2.24	2.42	2.58	2.76	2.94	3.13	3.33	3.54	3.97	4.39	4.84	5.28	5.74
0.300	1.55	1.73	1.90	2.07	2.23	2.39	2.57	2.75	2.94	3.14	3.35	3.78	4.20	4.64	5.09	5.53
0.400	1.34	1.49	1.63	1.77	1.91	2.07	2.24	2.42	2.60	2.79	2.99	3.42	3.84	4.27	4.71	5.16
0.500	1.16	1.29	1.41	1.53	1.67	1.81	1.97	2.14	2.32	2.50	2.69	3.10	3.52	3.94	4.37	4.81
0.600	1.01	1.13	1.23	1.35	1.47	1.60	1.75	1.91	2.08	2.25	2.44	2.83	3.24	3.65	4.06	4.49
0.700	0.893	0.995	1.09	1.20	1.31	1.43	1.57	1.72	1.88	2.04	2.22	2.59	2.99	3.38	3.79	4.21
0.800	0.797	0.888	0.979	1.08	1.18	1.29	1.42	1.56	1.70	1.87	2.03	2.39	2.77	3.16	3.54	3.95
0.900	0.719	0.800	0.884	0.973	1.07	1.17	1.29	1.42	1.56	1.71	1.87	2.21	2.57	2.95	3.33	3.71
1.00	0.653	0.727	0.805	0.888	0.979	1.08	1.18	1.31	1.44	1.58	1.73	2.05	2.40	2.77	3.12	3.50
1.20	0.551	0.615	0.681	0.753	0.833	0.920	1.01	1.12	1.24	1.36	1.50	1.79	2.11	2.45	2.79	3.13
1.40	0.476	0.531	0.589	0.653	0.724	0.801	0.885	0.979	1.08	1.20	1.32	1.58	1.87	2.19	2.50	2.83
1.60	0.419	0.467	0.519	0.576	0.639	0.709	0.785	0.868	0.961	1.06	1.17	1.41	1.68	1.97	2.27	2.57
1.80	0.373	0.416	0.463	0.515	0.572	0.635	0.704	0.780	0.865	0.957	1.06	1.28	1.52	1.79	2.07	2.36
2.00	0.337	0.376	0.417	0.464	0.517	0.575	0.639	0.708	0.785	0.869	0.961	1.16	1.39	1.64	1.90	2.17
2.20	0.307	0.343	0.381	0.424	0.472	0.524	0.584	0.648	0.717	0.796	0.880	1.07	1.28	1.51	1.76	2.01
2.40	0.281	0.315	0.349	0.389	0.433	0.483	0.537	0.596	0.661	0.733	0.812	0.987	1.18	1.40	1.63	1.87
2.60	0.260	0.291	0.323	0.360	0.401	0.447	0.497	0.553	0.613	0.680	0.753	0.916	1.10	1.30	1.51	1.74
2.80	0.243	0.271	0.300	0.335	0.373	0.416	0.464	0.515	0.571	0.635	0.703	0.855	1.03	1.21	1.41	1.63
3.00	0.227	0.252	0.281	0.312	0.348	0.388	0.433	0.481	0.535	0.593	0.659	0.801	0.963	1.14	1.32	1.53
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

Table 8–11 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 30°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with

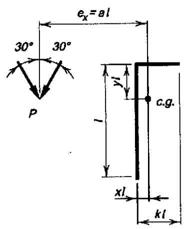
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where

P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



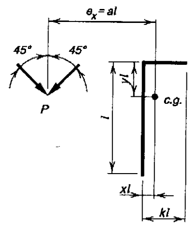
a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.19	2.44	2.70	2.96	3.21	3.47	3.73	3.99	4.24	4.50	4.76	5.27	5.79	6.30	6.82	7.33
0.100	2.02	2.25	2.47	2.70	2.93	3.17	3.41	3.64	3.86	4.08	4.31	4.78	5.27	5.76	6.26	6.77
0.150	1.92	2.12	2.33	2.54	2.76	2.98	3.20	3.42	3.63	3.84	4.07	4.54	5.02	5.51	6.01	6.52
0.200	1.82	2.01	2.21	2.41	2.62	2.83	3.02	3.21	3.41	3.63	3.85	4.33	4.79	5.26	5.74	6.24
0.250	1.71	1.90	2.08	2.27	2.47	2.66	2.83	3.02	3.22	3.43	3.65	4.12	4.57	5.03	5.50	5.99
0.300	1.60	1.78	1.96	2.13	2.32	2.49	2.66	2.84	3.04	3.25	3.47	3.93	4.37	4.82	5.29	5.77
0.400	1.40	1.56	1.71	1.87	2.02	2.18	2.35	2.53	2.72	2.92	3.13	3.58	4.04	4.47	4.92	5.38
0.500	1.23	1.37	1.50	1.63	1.77	1.93	2.09	2.26	2.44	2.64	2.84	3.27	3.73	4.18	4.61	5.06
0.600	1.08	1.21	1.32	1.44	1.57	1.72	1.87	2.04	2.21	2.39	2.59	3.00	3.45	3.90	4.33	4.77
0.700	0.963	1.07	1.18	1.29	1.41	1.54	1.69	1.85	2.01	2.18	2.37	2.77	3.19	3.64	4.07	4.50
0.800	0.863	0.963	1.06	1.17	1.28	1.40	1.53	1.68	1.84	2.01	2.18	2.56	2.97	3.40	3.83	4.25
0.900	0.781	0.872	0.963	1.06	1.16	1.28	1.40	1.54	1.69	1.85	2.02	2.38	2.77	3.19	3.60	4.02
1.00	0.713	0.795	0.879	0.969	1.07	1.17	1.29	1.42	1.56	1.71	1.87	2.22	2.59	2.99	3.39	3.80
1.20	0.605	0.675	0.747	0.827	0.913	1.01	1.11	1.23	1.35	1.49	1.63	1.94	2.29	2.65	3.03	3.42
1.40	0.524	0.584	0.649	0.720	0.796	0.880	0.972	1.07	1.19	1.31	1.44	1.73	2.04	2.38	2.74	3.09
1.60	0.463	0.515	0.573	0.636	0.705	0.780	0.864	0.956	1.06	1.17	1.29	1.55	1.84	2.15	2.49	2.82
1.80	0.413	0.460	0.512	0.569	0.632	0.701	0.777	0.860	0.953	1.05	1.17	1.40	1.67	1.96	2.27	2.59
2.00	0.373	0.416	0.463	0.515	0.572	0.636	0.705	0.781	0.867	0.960	1.06	1.28	1.53	1.80	2.09	2.39
2.20	0.340	0.380	0.423	0.469	0.523	0.581	0.645	0.716	0.793	0.880	0.973	1.18	1.41	1.66	1.93	2.21
2.40	0.312	0.349	0.388	0.432	0.480	0.535	0.595	0.660	0.732	0.812	0.899	1.09	1.30	1.54	1.79	2.06
2.60	0.289	0.323	0.359	0.399	0.445	0.496	0.551	0.612	0.680	0.755	0.835	1.01	1.21	1.43	1.67	1.92
2.80	0.268	0.300	0.333	0.371	0.415	0.461	0.513	0.571	0.633	0.703	0.779	0.947	1.13	1.34	1.56	1.80
3.00	0.251	0.280	0.312	0.347	0.387	0.432	0.480	0.535	0.593	0.659	0.729	0.888	1.07	1.26	1.47	1.69
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

Table 8–11 (continued)
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 45°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



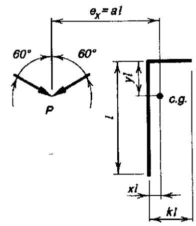
a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.34	2.57	2.81	3.04	3.28	3.51	3.74	3.98	4.21	4.45	4.68	5.15	5.62	6.08	6.55	7.02
0.100	2.24	2.44	2.65	2.87	3.10	3.33	3.56	3.80	4.02	4.26	4.49	4.98	5.47	5.97	6.46	6.95
0.150	2.09	2.28	2.48	2.69	2.91	3.14	3.37	3.58	3.81	4.04	4.28	4.77	5.27	5.77	6.28	6.78
0.200	1.96	2.14	2.32	2.51	2.72	2.94	3.18	3.39	3.61	3.84	4.08	4.57	5.06	5.57	6.08	6.59
0.250	1.85	2.01	2.19	2.37	2.56	2.76	2.98	3.21	3.43	3.66	3.90	4.39	4.88	5.38	5.89	6.40
0.300	1.74	1.90	2.06	2.23	2.41	2.60	2.81	3.03	3.27	3.50	3.73	4.22	4.71	5.21	5.71	6.22
0.400	1.55	1.69	1.83	1.99	2.17	2.36	2.56	2.76	2.97	3.19	3.43	3.89	4.36	4.86	5.37	5.88
0.500	1.37	1.51	1.64	1.79	1.97	2.15	2.32	2.51	2.71	2.92	3.15	3.62	4.07	4.53	5.02	5.52
0.600	1.23	1.35	1.48	1.62	1.79	1.95	2.11	2.29	2.48	2.68	2.90	3.36	3.83	4.28	4.73	5.21
0.700	1.11	1.21	1.34	1.48	1.62	1.77	1.93	2.10	2.28	2.48	2.68	3.13	3.60	4.05	4.49	4.95
0.800	1.00	1.10	1.22	1.35	1.48	1.62	1.77	1.93	2.11	2.29	2.49	2.92	3.37	3.84	4.27	4.72
0.900	0.913	1.01	1.12	1.24	1.35	1.49	1.63	1.79	1.95	2.13	2.32	2.73	3.17	3.63	4.07	4.51
1.00	0.839	0.928	1.03	1.14	1.25	1.37	1.51	1.66	1.82	1.99	2.17	2.56	2.98	3.43	3.88	4.30
1.20	0.717	0.797	0.888	0.980	1.08	1.19	1.31	1.45	1.59	1.75	1.91	2.27	2.66	3.08	3.53	3.94
1.40	0.625	0.697	0.776	0.859	0.949	1.05	1.16	1.28	1.41	1.55	1.70	2.03	2.39	2.79	3.20	3.61
1.60	0.555	0.619	0.688	0.763	0.844	0.933	1.03	1.14	1.26	1.39	1.53	1.83	2.17	2.54	2.93	3.33
1.80	0.497	0.555	0.617	0.684	0.760	0.841	0.932	1.03	1.14	1.26	1.39	1.67	1.98	2.32	2.69	3.07
2.00	0.451	0.503	0.559	0.621	0.691	0.765	0.848	0.940	1.04	1.15	1.27	1.53	1.82	2.14	2.48	2.85
2.20	0.411	0.459	0.511	0.568	0.632	0.701	0.777	0.863	0.957	1.06	1.17	1.41	1.68	1.98	2.30	2.65
2.40	0.379	0.423	0.471	0.523	0.581	0.647	0.717	0.797	0.885	0.981	1.08	1.31	1.56	1.84	2.15	2.47
2.60	0.351	0.391	0.436	0.484	0.540	0.600	0.667	0.740	0.823	0.912	1.01	1.22	1.46	1.72	2.01	2.31
2.80	0.327	0.364	0.405	0.451	0.503	0.560	0.623	0.691	0.768	0.851	0.943	1.14	1.37	1.62	1.88	2.16
3.00	0.305	0.341	0.380	0.423	0.471	0.524	0.583	0.648	0.720	0.799	0.884	1.07	1.29	1.52	1.77	2.04
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

Table 8–11 (continued)
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 60°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \frac{P_u}{\phi C_1 Dl}$ $D_{min} = \frac{P_u}{\phi CC_1 l}$ $l_{min} = \frac{P_u}{\phi CC_1 D}$	$C_{min} = \frac{\Omega P_a}{C_1 Dl}$ $D_{min} = \frac{\Omega P_a}{CC_1 l}$ $l_{min} = \frac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x / l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



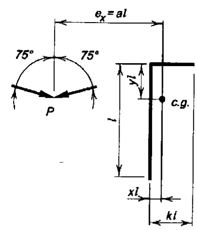
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	2.57	2.79	3.01	3.23	3.45	3.67	3.88	4.10	4.32	4.54	4.76	5.20	5.63	6.07	6.51	6.94
0.100	2.43	2.59	2.76	2.94	3.15	3.37	3.60	3.84	4.08	4.31	4.55	5.02	5.48	5.93	6.38	6.82
0.150	2.31	2.45	2.61	2.79	2.99	3.20	3.43	3.67	3.92	4.17	4.41	4.90	5.37	5.84	6.29	6.74
0.200	2.18	2.32	2.47	2.64	2.83	3.04	3.27	3.51	3.75	4.00	4.26	4.76	5.25	5.73	6.19	6.65
0.250	2.07	2.21	2.35	2.51	2.69	2.91	3.14	3.38	3.62	3.87	4.11	4.61	5.11	5.61	6.09	6.55
0.300	1.96	2.10	2.24	2.40	2.58	2.79	3.01	3.25	3.50	3.74	3.99	4.48	4.97	5.48	5.97	6.44
0.400	1.79	1.92	2.05	2.21	2.39	2.59	2.80	3.03	3.27	3.51	3.77	4.26	4.75	5.24	5.72	6.20
0.500	1.63	1.75	1.88	2.04	2.22	2.42	2.62	2.84	3.07	3.31	3.55	4.06	4.55	5.04	5.52	5.99
0.600	1.49	1.60	1.73	1.89	2.07	2.26	2.46	2.68	2.90	3.13	3.36	3.85	4.36	4.85	5.33	5.81
0.700	1.37	1.48	1.61	1.76	1.93	2.12	2.32	2.53	2.74	2.97	3.19	3.67	4.16	4.67	5.16	5.64
0.800	1.26	1.37	1.49	1.64	1.80	1.99	2.18	2.39	2.60	2.82	3.04	3.51	3.98	4.48	4.98	5.47
0.900	1.17	1.27	1.39	1.53	1.69	1.87	2.06	2.24	2.44	2.66	2.89	3.35	3.82	4.30	4.79	5.29
1.00	1.08	1.18	1.30	1.43	1.59	1.76	1.93	2.11	2.30	2.51	2.73	3.20	3.67	4.13	4.61	5.11
1.20	0.945	1.04	1.14	1.27	1.41	1.56	1.71	1.88	2.05	2.25	2.45	2.89	3.36	3.83	4.29	4.75
1.40	0.835	0.920	1.02	1.14	1.26	1.39	1.53	1.69	1.85	2.03	2.22	2.63	3.08	3.55	3.99	4.45
1.60	0.747	0.825	0.919	1.03	1.13	1.25	1.38	1.53	1.68	1.84	2.02	2.41	2.83	3.28	3.74	4.17
1.80	0.675	0.748	0.835	0.929	1.03	1.14	1.26	1.39	1.53	1.69	1.85	2.21	2.61	3.04	3.48	3.92
2.00	0.615	0.683	0.764	0.848	0.940	1.04	1.15	1.28	1.41	1.55	1.71	2.05	2.42	2.83	3.25	3.69
2.20	0.564	0.628	0.703	0.780	0.865	0.959	1.06	1.18	1.30	1.44	1.58	1.90	2.25	2.64	3.04	3.46
2.40	0.521	0.581	0.649	0.721	0.800	0.888	0.984	1.09	1.21	1.34	1.47	1.77	2.11	2.47	2.85	3.25
2.60	0.484	0.540	0.603	0.669	0.744	0.827	0.916	1.02	1.13	1.25	1.38	1.66	1.97	2.31	2.68	3.06
2.80	0.452	0.505	0.563	0.625	0.695	0.773	0.859	0.953	1.06	1.17	1.29	1.56	1.86	2.18	2.53	2.90
3.00	0.423	0.473	0.527	0.587	0.652	0.727	0.807	0.896	0.995	1.10	1.22	1.47	1.75	2.06	2.39	2.74
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

Table 8–11 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 75°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



a \ k	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.73	2.92	3.11	3.30	3.49	3.69	3.88	4.07	4.26	4.46	4.65	5.03	5.42	5.80	6.19	6.57
0.100	2.59	2.67	2.78	2.93	3.12	3.32	3.53	3.75	3.96	4.17	4.38	4.79	5.24	5.67	6.09	6.51
0.150	2.50	2.59	2.70	2.86	3.05	3.26	3.47	3.70	3.92	4.13	4.34	4.75	5.16	5.60	6.03	6.45
0.200	2.43	2.51	2.63	2.79	2.98	3.19	3.41	3.64	3.87	4.09	4.30	4.71	5.12	5.53	5.97	6.39
0.250	2.35	2.44	2.56	2.72	2.91	3.13	3.35	3.59	3.82	4.04	4.26	4.68	5.08	5.48	5.90	6.34
0.300	2.28	2.37	2.50	2.66	2.85	3.07	3.29	3.53	3.76	4.00	4.22	4.65	5.06	5.45	5.86	6.27
0.400	2.16	2.25	2.38	2.54	2.73	2.94	3.17	3.41	3.66	3.90	4.13	4.58	5.00	5.41	5.80	6.20
0.500	2.05	2.14	2.27	2.43	2.62	2.83	3.06	3.29	3.54	3.79	4.04	4.50	4.94	5.35	5.76	6.15
0.600	1.94	2.04	2.17	2.33	2.52	2.73	2.95	3.18	3.43	3.69	3.93	4.42	4.87	5.30	5.71	6.11
0.700	1.84	1.94	2.07	2.24	2.42	2.63	2.85	3.08	3.32	3.57	3.83	4.33	4.79	5.24	5.66	6.07
0.800	1.75	1.85	1.98	2.15	2.33	2.53	2.75	2.98	3.22	3.47	3.72	4.23	4.71	5.17	5.60	6.02
0.900	1.67	1.77	1.90	2.06	2.24	2.44	2.66	2.88	3.12	3.37	3.61	4.13	4.63	5.10	5.64	5.97
1.00	1.59	1.69	1.82	1.98	2.16	2.35	2.57	2.79	3.03	3.27	3.52	4.03	4.54	5.02	5.47	5.91
1.20	1.45	1.55	1.68	1.83	2.00	2.19	2.40	2.62	2.85	3.09	3.33	3.83	4.35	4.85	5.33	5.78
1.40	1.33	1.42	1.55	1.70	1.86	2.05	2.25	2.46	2.69	2.92	3.15	3.64	4.15	4.67	5.16	5.63
1.60	1.22	1.32	1.44	1.58	1.74	1.92	2.11	2.32	2.53	2.76	2.98	3.45	3.96	4.48	4.99	5.48
1.80	1.13	1.22	1.34	1.47	1.63	1.80	1.99	2.19	2.39	2.50	2.82	3.27	3.76	4.28	4.81	5.31
2.00	1.05	1.14	1.25	1.38	1.53	1.69	1.87	2.06	2.26	2.46	2.66	3.10	3.58	4.08	4.61	5.14
2.20	0.973	1.06	1.17	1.30	1.44	1.60	1.77	1.95	2.14	2.33	2.52	2.95	3.41	3.90	4.42	4.94
2.40	0.911	0.997	1.10	1.22	1.36	1.51	1.67	1.85	2.02	2.21	2.39	2.80	3.25	3.73	4.23	4.75
2.60	0.855	0.939	1.04	1.15	1.28	1.43	1.59	1.76	1.92	2.09	2.27	2.67	3.10	3.57	4.06	4.57
2.80	0.805	0.885	0.981	1.09	1.22	1.36	1.51	1.66	1.82	1.99	2.16	2.55	2.97	3.42	3.90	4.39
3.00	0.760	0.837	0.931	1.04	1.16	1.29	1.43	1.58	1.73	1.90	2.06	2.43	2.84	3.28	3.75	4.23
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

PART 9

DESIGN OF CONNECTING ELEMENTS

SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of connecting 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 bolts and welds, see Parts 7 and 8, respectively. For the design of connections, see Parts 10 through 15. For connecting elements that are part of a seismic force resisting system in which the seismic response modification factor, R, is taken greater than 3, the requirements in the AISC *Seismic Provisions for Structural Steel Buildings* also apply. The AISC *Seismic Provisions for Structural Steel Buildings* is available in Part 6 of the AISC *Seismic Design Manual* from the American Institute of Steel Construction, Inc. at www.aisc.org.

GROSS AREA, EFFECTIVE NET AREA, AND WHITMORE SECTION

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

Gross Area

The gross area, A_g, is determined as specified in AISC Specification Section D3.1, subject to the limitations given below for the Whitmore section.

Effective Net Area

The effective net area, A_e, is determined as specified in AISC Specification Section J4.1, 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 connecting 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 connecting element to less than the full area (Whitmore, 1952). As illustrated in Figure 9–1, the width of the Whitmore section, l_w, 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 connecting element along the line of force. The Whitmore section may spread across the joint between connecting elements, but cannot spread beyond an unconnected edge.

CONNECTING ELEMENTS SUBJECT TO COMBINED LOADING

Connection design has traditionally been based on simple stresses, such as shear, tension, compression, or flexure, not taken in combination. This simplification is adequate because connection elements are usually small or short enough that an interaction-type distribution cannot form. Even a theoretical combination analysis using the von-Mises criterion for plane stress is not any more refined. To illustrate this point, von-Mises criterion is expressed as

$$f_e = \sqrt{f_x^{\,2} - f_x f_y + f_y^{\,2} + 3f_{xy}^{\,2}} \le F_y$$

where

f_x and f_y = normal stresses
f_{xy} = shear stress
F_y = yield stress.

This formulation requires three stresses at any one point. Assuming f_{xy} and f_x are known for any one cut section, f_y on the perpendicular cut section is still undefined and must be assumed, thereby bringing inaccuracy into the formulation. Compounding this dilemma, f_y could be assumed as equal to zero, equal to and having the same sign as f_x, or equal to and having the opposite sign of f_x. Thus, what might appear to be a more sophisticated approach to the analysis and design of a connection does not necessarily add any reliability to the resulting design.

CONNECTING ELEMENTS SUBJECT TO SHEAR

The available strength due to shear yielding and shear rupture, ϕR_n or R_n/Ω, which must equal or exceed the required shear strength, R_u or R_a, respectively, are determined in accordance with AISC Specification Section J4.2

CONNECTING ELEMENTS SUBJECT TO TENSION

The available strength due to tension yielding and tension rupture, ϕR_n or R_n/Ω, which must equal or exceed the required tensile strength, R_u or R_a, respectively, is determined in accordance with AISC Specification Section J4.1.

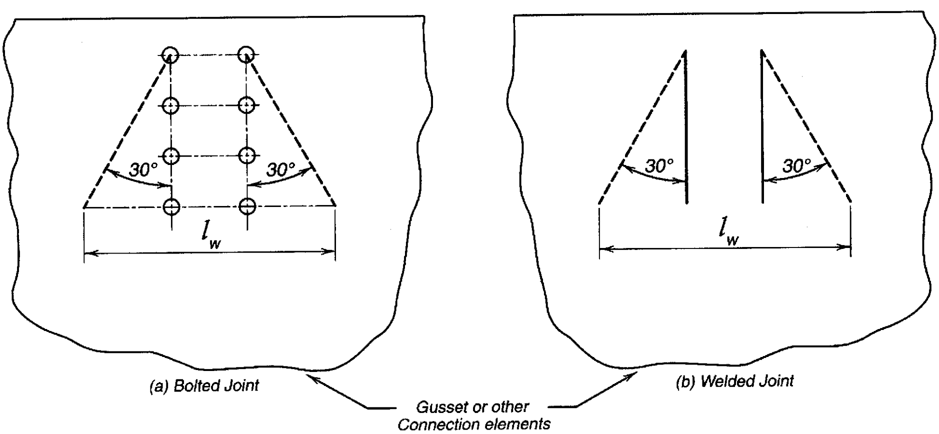


Figure 9–1. Illustration of the width of the Whitmore section.

CONNECTING ELEMENTS SUBJECT TO BLOCK SHEAR RUPTURE

The block shear available rupture strength, ϕR_n or R_n/Ω, which must equal or exceed the required strength, R_u or R_a, respectively, is determined in accordance with AISC Specification Section J4.3. The values tabulated in Table 9–3 can be used to calculate the available block shear rupture strength.

CONNECTING ELEMENT RUPTURE STRENGTH AT WELDS

In many cases, the load path from a weld to the connecting element is such that the strength of the connecting element can be evaluated directly. However, in some cases, the available strength of the connecting element is not directly calculable. For example, while the strength of the beam-web welds for a double-angle connection can be directly calculated, the strength of the beam web at this weld cannot. In cases such as these, it is often convenient to calculate the minimum base-metal thickness that will match the available shear rupture strength of the base metal to the available shear rupture strength of the weld(s).

For fillet welds with $F_{EXX} = 70$ ksi on both sides of the connecting element, the minimum thickness required to match the shear rupture strength of the connecting element to the shear rupture strength of the base metal is

$$t_{min} = \frac{0.6F_{EXX} \times \frac{\sqrt{2}}{2} \times \frac{D}{16} \times 2}{0.6F_u}$$

$$= \frac{6.19D}{F_u}$$

where

D = number of sixteenths of an inch in the weld size

F_u = specified minimum tensile strength of the connecting element, ksi

Similarly, for fillet welds with $F_{EXX} = 70$ ksi on one side of the connection, the minimum thickness required to match the shear rupture strength of the connecting element to the shear rupture strength of the base metal is

$$t_{min} = \frac{3.09D}{F_u}$$

CONNECTING ELEMENTS SUBJECT TO COMPRESSION BUCKLING

When connecting elements are subject to compression, the available strength, ϕP_n or P_n/Ω, which must equal or exceed the required compressive strength, P_u or P_a, respectively, can be determined as given in Specification Section J4.4, when $KL/r < 25$.

CONNECTING ELEMENTS SUBJECT TO FLEXURE

Connection elements are normally short enough and thick enough that flexural effects, if present at all, do not impact the design.

Yielding, Lateral-Torsional Buckling, and Local Buckling

The available flexural strength, ϕM_n or M_n/Ω, which must equal or exceed the required flexural strength, M_u or M_a, respectively, is determined in accordance with AISC *Specification* Chapter F. When connection elements are long enough and thin enough that flexural effects must be considered, these provisions can be applied. User Note F1.1 provides guidance based upon cross-section for which Section in Chapter F is applicable.

Treatment of coped beams is provided below based upon Cheng, et al. (1984). For beam ends with short copes no greater than the length of the connection angle(s), plate, or tee, local web buckling will generally not occur.

Rupture

For beams and rolled girders with bolt holes in the tension flange, see AISC *Specification* Section F13. For coped beams, see the discussion on coped beams below. In other cases for connection elements, the available flexural rupture strength, ϕM_n or M_n/Ω, can be determined as

$$M_n = F_u Z_{net}$$
$$\phi_b = 0.75 \qquad \Omega_b = 2.00$$

Coped Beams

The end reaction for a coped beam may be limited by flexural limit states such as yielding, rupture, local buckling, or lateral-torsional buckling. For a coped beam, the required flexural strength is calculated as

LRFD	ASD
$M_u = R_u e$	$M_a = R_a e$

where

R_u or R_a = beam end reaction (LRFD, ASD), kips

e = distance from the face of the cope to the point of inflection of the beam, in. It is usually assumed that the point of inflection is located at the face of the supporting member and e is as shown in Figure 9–2. However, depending upon the connection type and stiffness and support condition, the point of inflection may move away from the face of the supporting member; when this is the case, a lesser value of e may be justified. The choice of e shown in Figure 9–2 will be conservative.

For a beam coped at the top flange or both the top and bottom flanges, the available flexural rupture strength, ϕM_n or M_n/Ω, can be determined as

$$M_n = F_u S_{net}$$
$$\phi_b = 0.75 \qquad \Omega_b = 2.00$$

The available flexural local buckling strength or a beam coped at the top flange or both the top and bottom flanges must equal or exceed the required strength. The available strength, $\phi_b M_n$ or M_n/Ω_b, is determined as

$$M_n = F_{cr} S_{net}$$
$$\phi_b = 0.90 \qquad \Omega_b = 1.67$$

where

F_{cr} = available buckling stress, determined as given below, ksi
S_{net} = net section modulus, in.^{3} Values of S_{net} are tabulated in Table 9–2

When a beam is coped at the top flange only, the available buckling stress is based upon the classical plate buckling formula with a k-factor corresponding to the condition with three edges simply supported and one free edge. An additional factor, f, is applied to account for stress concentrations at the cope and to correlate the solutions with experimental results (Cheng, et. al., 1984).

The available buckling stress, ϕF_{cr} or F_{cr}/Ω, for a beam coped at the top flange only when $c \le 2d$ and $d_c \le d/2$ (see Figure 9–2) is determined as

$$F_{cr} = \frac{\pi^2 E}{12\left(1-\nu^2\right)}\left(\frac{t_w}{h_o}\right)^2 fk$$

$$= 26{,}210\left(\frac{t_w}{h_o}\right)^2 fk$$

$$\phi = 0.90 \qquad \Omega = 1.67$$

where

E = 29,000 ksi, modulus of elasticity of steel
ν = 0.3, Poisson's ratio
f = plate buckling model adjustment factor

$$= \frac{2c}{d} \text{ when } \frac{c}{d} \le 1.0$$

$$= 1 + \frac{c}{d} \text{ when } \frac{c}{d} > 1.0$$

t_w = beam web thickness
k = plate buckling coefficient

$$= 2.2\left(\frac{h_o}{c}\right)^{1.65} \text{ when } \frac{c}{h_o} \le 1.0$$

$$= \frac{2.2h_o}{c} \text{ when } \frac{c}{h_o} > 1.0$$

$h_o = d - d_c$, reduced beam depth, in. Note that, for convenience, the dimension h_o, 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_o 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 available buckling stress, ϕF_{cr} or F_{cr}/Ω, is based upon a lateral-torsional buckling model with an adjustment factor f_d (Cheng, et al., 1984). The available buckling stress, ϕF_{cr} or F_{cr}/Ω, for a beam coped equally at both flanges with $c \le 2d$ and $d_c \le 0.2d$ (see Figure 9–3) is determined as

$$F_{cr} = 0.62\pi E \frac{t_w^{\,2}}{ch_o} f_d$$

$$\phi = 0.90 \qquad \Omega = 1.67$$

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.

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

$$F_{cr} = F_y Q$$
$$\phi = 0.90 \qquad \Omega = 1.67$$

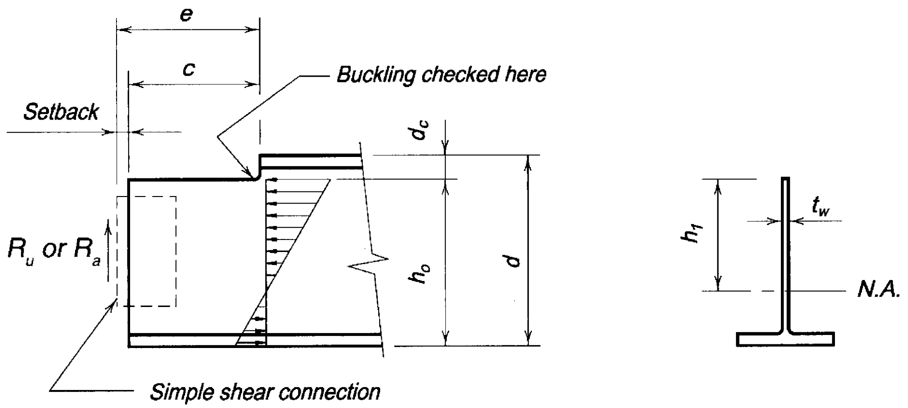


Figure 9–2. Local buckling of beam web coped at top flange only.

where

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

$$\lambda = \frac{h_o\sqrt{F_y}}{10t_w\sqrt{475 + 280\left(\frac{h_o}{c}\right)^2}}$$

where

c = length of plate parallel to the compressive force, in.
h_o = reduced beam depth, in.
t_w = thickness of plate, in.
F_y = yield stress, ksi

BEARING LIMIT STATES

Bearing Strength at Bolt Holes

For available 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 AISC Specification Section J7. The fabrication and erection requirements in AISC Specification Sections M2.6, M2.8, and M4.4 are applicable to connecting elements that transfer load by contact bearing on steel.

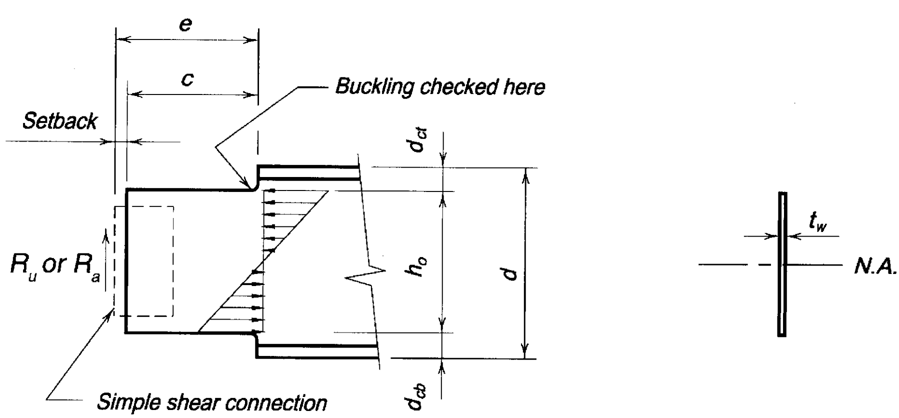


Figure 9–3. Local buckling of beam web coped at both flanges.

Bearing Strength on Concrete or Masonry

The bearing strength of concrete or masonry is determined as given in AISC Specification Section J8. The fabrication and erection requirements in AISC Specification Sections M2.8 and M4.1 are applicable to connecting elements that transfer load by contact bearing on concrete or masonry.

OTHER SPECIFICATION REQUIREMENTS AND DESIGN CONSIDERATIONS

The following other specification requirements and design considerations apply to the design of connecting elements:

Prying Action

Prying action is a phenomenon (in bolted construction only and for tensile bolt forces only) whereby the deformation of a connecting element under a tensile force increases the tensile force in the bolt above that due to the direct tensile force alone. Proper design for prying action includes the selection of bolt diameter and fitting thickness such that there is sufficient stiffness and strength in the connecting element and strength in the bolt. The following discussion of prying action is similar to what has been considered in the past, except that the design basis has been changed to calculate strength in terms of F_u, which provides better correlation with available test data than previous design methods. For the development of the prying action equations presented here, see Thornton (1992) and Swanson (2002).

Consider the tee or angle used in a hanger connection, as shown in Figure 9–4. The thickness required to eliminate prying action, t_{min}, can be determined as

LRFD	ASD
$t_{min} = \sqrt{\dfrac{4.44Tb'}{pF_u}}$	$t_{min} = \sqrt{\dfrac{6.66Tb'}{pF_u}}$

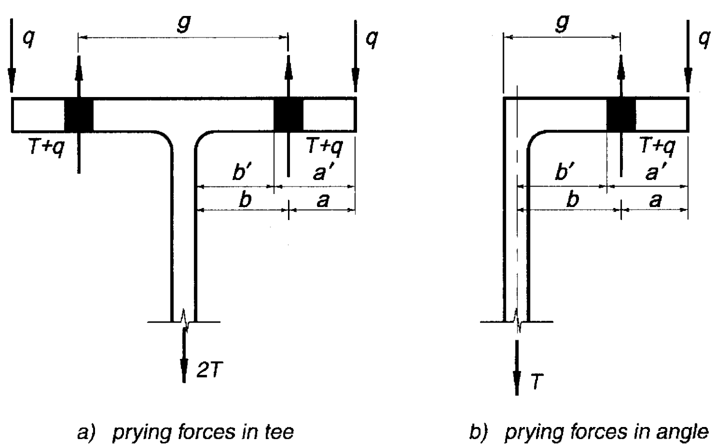


Figure 9–4. Illustration of variables in prying action calculations.

where

T = required strength, r_{ut} or r_{at}, per bolt, kips

$$b' = \left(b - \frac{d_b}{2}\right)$$

b = for a tee-type connecting element, the distance from bolt centerline to the face of the tee stem, in.; for an angle-type connecting element, the distance from bolt centerline to centerline of angle leg, in.

d_b = bolt diameter, in.

p = tributary length per pair of bolts for a tee or angle (perpendicular to the plane of the page in Figure 9–4), which should preferably not exceed the gage between the pair of bolts, g

F_u = specified minimum tensile strength of connecting element, ksi

When the resulting fitting thickness is reasonable, no further check of prying action is necessary. In this solution, the additional force in the bolt due to prying action, q, is essentially zero as long as the bolt force does not exceed T.

Alternatively, it is usually possible to determine a lesser required thickness by designing the connecting element and bolted joint for the actual effects of prying action with q greater than zero. To do so, a preliminary fitting thickness, t, can be selected based upon flexural yielding such that

LRFD	ASD
$T \le \dfrac{F_u t^2 p}{2.22b}$	$T \le \dfrac{F_u t^2 p}{3.33b}$

Table 15–1 can be used to select the preliminary fitting thickness. Subsequently, the thickness required to ensure an acceptable combination of fitting strength and stiffness and bolt strength, t_{min}, can be determined as

LRFD	ASD
$t_{min} = \sqrt{\dfrac{4.44Tb'}{pF_u(1+\delta\alpha')}}$	$t_{min} = \sqrt{\dfrac{6.66Tb'}{pF_u(1+\delta\alpha')}}$

where

$\delta = 1 - \dfrac{d'}{p}$ ratio of the net area at bolt line to gross area at face of the stem or leg of angle

$\alpha' = 1.0$ if $\beta \ge 1$

= the lesser of 1 and $\dfrac{1}{\delta}\left(\dfrac{\beta}{1-\beta}\right)$ if $\beta < 1$

d' = width of the hole along the length of the fitting, in.

$$\beta = \frac{1}{\rho}\left(\frac{B}{T} - 1\right)$$

$$\rho = \frac{b'}{a'}$$

$$a' = \left(a + \frac{d_b}{2}\right) \le \left(1.25b + \frac{d_b}{2}\right)$$

a = distance from the bolt centerline to the edge of the fitting, in.
B = available tension per bolt, ϕr_n or r_n/Ω, kips with $\phi = 0.75$ and $\Omega = 2.00$

If $t_{min} \le t$, the preliminary fitting thickness is satisfactory. Otherwise, a fitting with a thicker flange, or a change in geometry (i.e., b and p) is required.

Although it is not necessary to do so, if desired, the prying force per bolt, q, can be determined as

$$q = B\left[\delta\alpha\rho\left(\frac{t}{t_c}\right)^2\right]$$

$$\alpha = \frac{1}{\delta}\left[\frac{T}{B}\left(\frac{t_c}{t}\right)^2 - 1\right] \ge 0$$

LRFD	ASD
$t_c = \sqrt{\dfrac{4.44Bb'}{pF_u}}$	$t_c = \sqrt{\dfrac{6.66Bb'}{pF_u}}$

t_c = flange or angle thickness required to develop the available strength of the bolt, B, with no prying action, in.

The total force per bolt including the effects of prying action can then be determined as $T + q$.

Alternatively, when the fitting geometry is known, the available tensile strength per bolt, B, determined per AISC Specification Section J3.6, can be multiplied by Q to determine the available tensile strength including the effects of prying action

$$T_{avail} = BQ$$

where

Q = 1 if $\alpha' < 0$, which means that the fitting has sufficient strength and stiffness to develop the full bolt available tensile strength.

$= \left(\dfrac{t}{t_c}\right)^2 (1 + \delta\alpha')$ if $0 \le \alpha' \le 1$, which means that the fitting has sufficient strength to develop the full bolt available tensile strength, but insufficient stiffness to prevent prying action.

$= \left(\dfrac{t}{t_c}\right)^2 (1+\delta)$ if $\alpha' > 1$, which means that the fitting has insufficient strength to develop the full bolt available tensile strength.

$\alpha' = \dfrac{1}{\delta(1+\rho)}\left[\left(\dfrac{t_c}{t}\right)^2 - 1\right]$ = value of α that either maximizes the bolt available tensile strength for a given thickness or minimizes the thickness required for a given bolt available tensile strength.

Rotational Ductility

Simple shear connections provide for the rotational ductility required by AISC Specification Section J1.2 as follows:

1. For double-angle, shear end-plate, single-angle, and tee shear connections, the geometry and thickness of the connecting elements attached to the support (angle legs, plate, or tee flange) are configured so that flexing of those connecting elements accommodates the simple-beam end rotation.
2. For unstiffened and stiffened seated connections, the geometry and thickness of the top or side stability angle is configured so that flexing of that connecting element accommodates the simple-beam end rotation.
3. For single-plate connections, the geometry and thickness of the plate are configured so that the plate will yield, bolt group will rotate, and/or or the bolt holes will elongate at failure prior to the failure of the welds or bolts.

For each of the simple-shear connections in Part 10, except tee shear connections, prescriptive guidance is provided to ensure adequate rotational ductility. Rotational ductility can be ensured for tee shear connections as follows. Note that this approach can also be used to demonstrate adequate rotational ductility in other simple shear connections that flex to accommodate the simple-beam end rotation, but with configurations that differ from those prescribed in Part 10.

When the connecting elements are welded to the support and bolted to the supported beam, weld size, w, with $F_{EXX} = 70$ ksi, must be such that

$$w_{min} = 0.0158\frac{F_y t_f^{\,2}}{b}\left(\frac{b^2}{L^2} + 2\right)$$

but need not exceed $\frac{5}{8}t_s$, where

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

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

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 connecting element delivers a concentrated force to a member or other connecting element, see AISC Specification Section J10 or K1, as appropriate. See also AISC Design Guide No. 13 *Wide-Flange Column Stiffening at Moment Connections: Wind and Seismic Applications*.

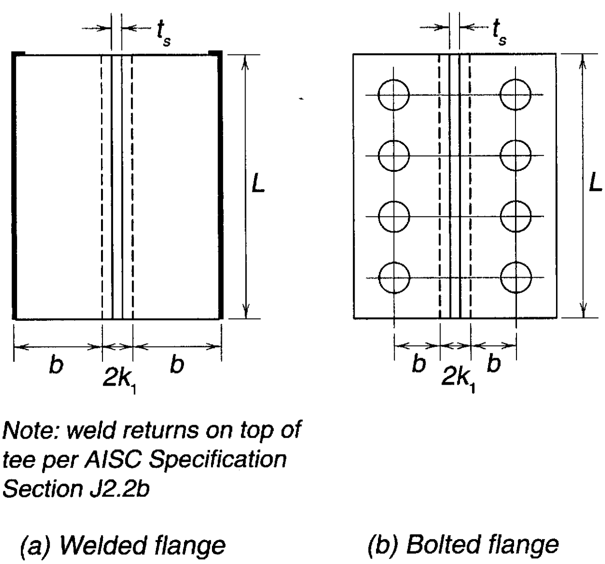


Figure 9–5. Illustration of variables in shear connection ductility checks.

Shims and Fillers

Shims are furnished to the erector for use in filling the spaces allowed for field clearance which might be present at connections such as simple shear connections, PR and FR moment connections, column base plates, and column splices. These shims, illustrated in Figure 9–6, may be either strip shims, with round punched holes, or finger shims, with slots cut through the edge. Whereas strip shims are less expensive to fabricate, finger shims may be laterally inserted and eliminate the need to remove erection bolts or pins already in place.

Finger shims, when inserted fully against the bolt shank, are acceptable for slip-critical connections and are not to be considered as an internal ply with the slotted hole determining the available strength of the connection. This is because less than 25 percent of the contact surface is lost and this is not enough to affect the performance of the joint.

A filler is furnished to occupy spaces which will be present because of dimensional separations between elements of a connection across which load transfer occurs. Examples where fillers might be used are beams framing off center on a column and raised beams.

For the effect of fillers and shims on available joint strength, see RCSC Specification Section 5.1.

Copes, Blocks, and Cuts

When structural members frame together, a minimum clearance of ½ in. should be provided, when possible. In cases where material removal is necessary to provide such a clearance, material may be removed by coping, blocking, or cutting as illustrated in Figure 9–7.

Material removal is costly and should be avoided when possible. In some cases, it may be possible to do so by setting the elevations of the tops of infill beams a sufficient distance below the tops of girders to clear the girder fillet radius. Alternatively, a connection such as that illustrated in Figure 9–8 could be used.

When material removal is necessary, coping is usually the most economical method to remove material. The recommended practices for coping are illustrated in Figure 9–9. The potential notch left by the first cut will occur in waste material and subsequently be removed by the second cut. All re-entrant corners must be shaped notch-free per AWS D1.1 to a radius. An approximate minimum radius to which this corner must be shaped is ½ in.

Copes, blocks, and cuts can significantly reduce the available strengths of members and may require web reinforcement; it may be more economical to use a heavier member than to provide such reinforcement. The available strength of the unreinforced coped member is determined from the limit states of flexural yielding, local buckling, and lateral torsional buckling, if applicable.

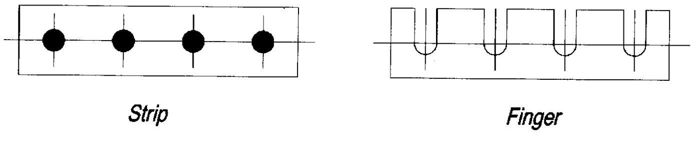


Figure 9–6. Shims.

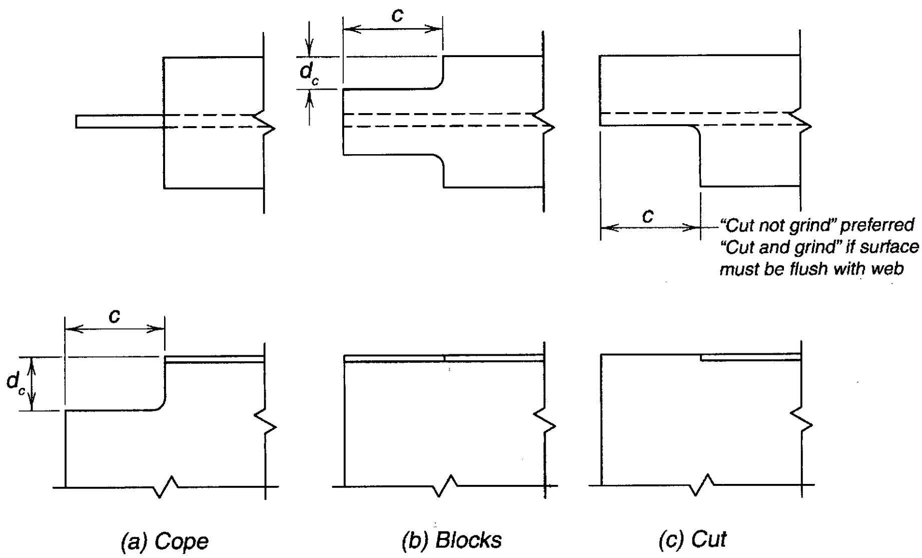


Figure 9–7. Copes, blocks, and cuts.

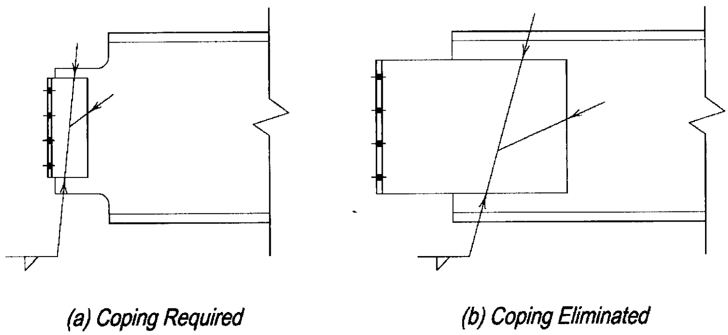


Figure 9–8. Minimizing coping requirements.

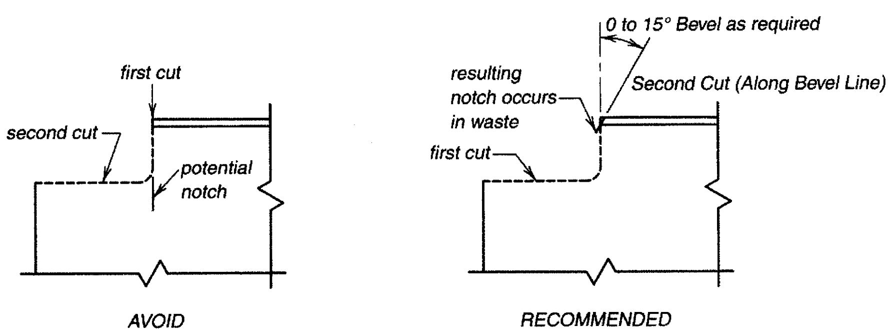


Figure 9–9. Recommending coping practices.

Web Reinforcement of Coped Beams

When the strength of a coped beam is inadequate, either a different beam can be selected to eliminate the need for reinforcement, or reinforcement can be provided to increase the strength. In spite of the increase in material cost, the former solution may be the most economical option due to the appreciable labor cost associated with adding stiffeners and/or doubler plates. When the latter solution is required, some typical reinforcing details are illustrated in Figure 9–10.

The doubler plate illustrated in Figure 9–10a and the longitudinal stiffener illustrated in Figure 9–10b are used with rolled sections where $h/t_w \leq 60$. When a doubler plate is used, the required doubler-plate thickness, $t_{d\ req}$, is determined by substituting the quantity $(t_w + t_{d\ req})$ for t_w in the available strength calculations for flexural yielding and local web buckling. To prevent local crippling of the beam web, the doubler plate must be extended at least a distance d_c (depth of cope) beyond the cope as illustrated in Figure 9–10a. When longitudinal stiffening is used, the stiffening elements must be proportioned to meet the width-thickness ratios specified in AISC Specification Table B4.1. The stiffened cross-section must then be checked for flexural yielding, but local web buckling need not be checked. To prevent local crippling of the beam web, the longitudinal stiffening must be extended a distance d_c beyond the cope as illustrated in Figure 9–10b.

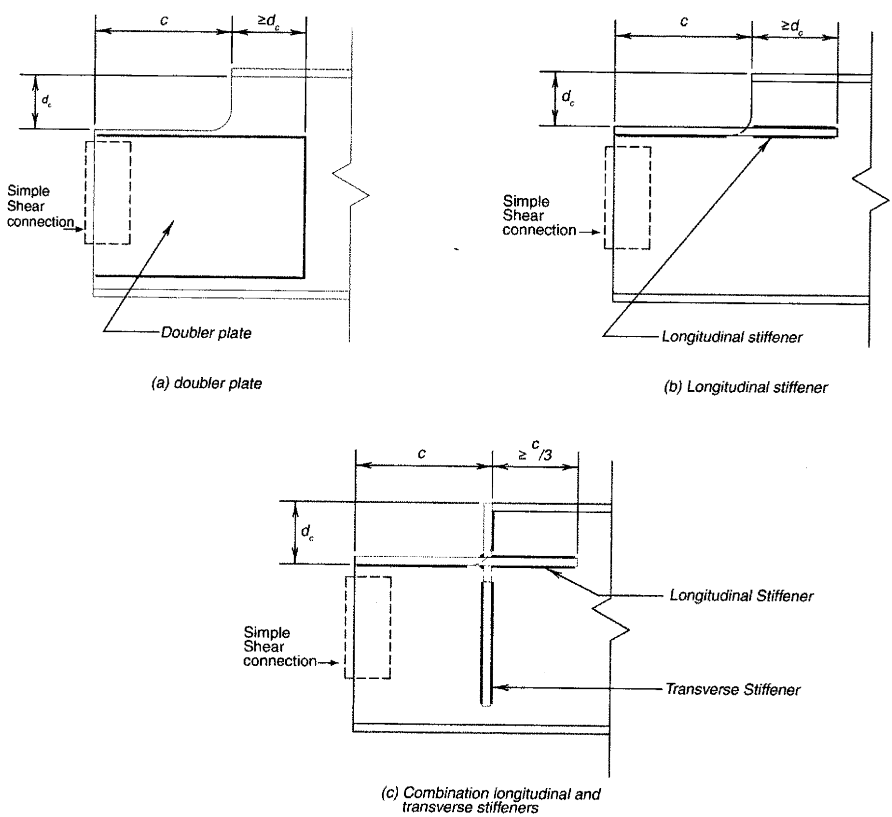


Figure 9–10. Web reinforcement of coped beams.

The combination of longitudinal and transverse stiffeners shown in Figure 9–10c may be required for thin-webbed plate-girders, where $h/t_w > 60$. When longitudinal and transverse stiffening is used, the stiffening elements must be proportioned to meet the width-thickness ratios specified in AISC Specification Table B4.1. The stiffened cross-section must then be checked for flexural yielding, but local web buckling need not be checked. To prevent local crippling of the beam web, longitudinal stiffeners must be extended a distance $c/3$ beyond the cope, as illustrated in Figure 9–10c.

DESIGN TABLES

Table 9–1. Reduction in Area for Holes

Area reduction for standard, oversized, short-slotted, and long-slotted holes in material thicknesses from 3/16 in. to 1 in. are given in Table 9–1. For material thicknesses not listed, the tabular value for 1-in. thickness can be multiplied by the actual thickness.

Table 9–2. Elastic Section Modulus of Coped W-Shapes

Values are given for the gross and net elastic section moduli for coped W-shapes, as illustrated in the table header.

Tables 9–3. Block Shear Rupture

The terms in AISC Specification Equation J4-5 are tabulated in Tables 9–3a, 9–3b, and 9–3c. The indicated values are given per inch of material thickness. Note that when the stress distribution is non-uniform, the tension component from Table 9.3a must be reduced by factor of 0.5 to account for U_{bs}.

Table 9–4. Beam End Bearing Constants

At beam ends, the available strength for local web yielding, ϕR_n or R_n/Ω, is determined per AISC Specification Section J10.2. This can be simplified using the bearing length, N, and the constants R_1 through R_6, as outlined below.

$$R_1 = 2.5kF_{yw}t_w$$
$$R_2 = F_{yw}t_w$$
$$R_3 = 0.40t_w^2\sqrt{\frac{EF_{yw}t_f}{t_w}}$$
$$R_4 = 0.40t_w^2\left(\frac{3}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\sqrt{\frac{EF_{yw}t_f}{t_w}}$$
$$R_5 = 0.40t_w^2\left(1-0.2\left(\frac{t_w}{t_f}\right)^{1.5}\right)\sqrt{\frac{EF_{yw}t_f}{t_w}}$$
$$R_6 = 0.40t_w^2\left(\frac{4}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\sqrt{\frac{EF_{yw}t_f}{t_w}}$$

Local Web Yielding at Beam Ends

At beam ends, the available strength for local web yielding, ϕR_n or R_n/Ω, is determined per AISC Specification Section J10.2 using Equations J10-2 or J10-3, which can be simplified using the constants R_1 and R_2 from Table 9–4 as follows:

When the compressive force to be resisted is applied at a distance, x, from the member end that is less than the depth of the member ($< d$),

LRFD	ASD
$\phi R_n = \phi R_1 + N(\phi R_2)$	$R_n/\Omega = R_1/\Omega + N(R_2/\Omega)$

When the compressive force to be resisted is applied at a distance, x, from the member end that is greater than or equal to the depth of the member ($\geq d$),

LRFD	ASD
$\phi R_n = 2(\phi R_1) + N(\phi R_2)$	$R_n/\Omega = 2(R_1/\Omega) + N(R_2/\Omega)$

Note that, as a minimum, the length of bearing, N, must be equal to k, per AISC Specification Section J10.2

Web Crippling at Beam Ends

At beam ends, the available strength for web crippling, R_n/Ω or ϕR_n, is determined per AISC Specification Section J10.3 using Equations J10-4, J10-5a, or J10-5b, which can be simplified using constants R_3, R_4, R_5, and R_6 from Table 9–4 as follows:

When the compressive force to be resisted is applied at a distance, x, from the member end that is less than one-half of the depth of the member ($< d/2$),

For $N/d \leq 0.2$:

LRFD	ASD
$\phi R_n = \phi R_3 + N(\phi R_4)$	$R_n/\Omega = R_3/\Omega + N(R_4/\Omega)$

For $N/d > 0.2$:

LRFD	ASD
$\phi R_n = \phi R_5 + N(\phi R_6)$	$R_n/\Omega = R_5/\Omega + N(R_6/\Omega)$

When the compressive force to be resisted is applied at a distance, x, from the member end that is greater than or equal to one-half of the depth of the member ($\geq d/2$),

LRFD	ASD
$\phi R_n = 2(\phi R_3) + N(\phi R_4)$	$R_n/\Omega = 2(R_3/\Omega) + N(R_4/\Omega)$

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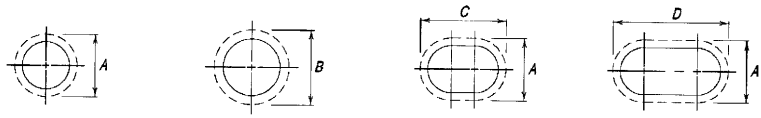
Table 9–1
Reduction in Area for Holes, in.^{2}

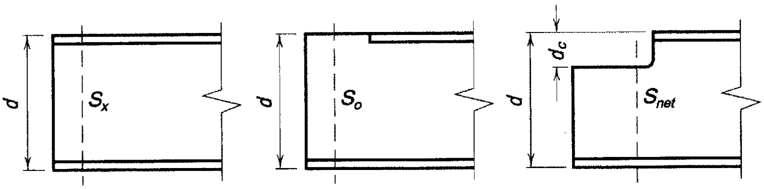


STD Standard Hole | OVS Oversized Hole | SSL Short-Slotted Hole | LSL Long-Slotted Hole

Thickness t, in.	$A \times t$							$B \times t$						
	Bolt Diameter d_b, in.							Bolt Diameter d_b, in.						
	3/4	7/8	1	1 1/8	1 1/4	1 3/8	1 1/2	3/4	7/8	1	1 1/8	1 1/4	1 3/8	1 1/2
3/16	0.164	0.188	0.211	0.234	0.258	0.281	0.305	0.188	0.211	0.246	0.281	0.305	0.328	0.352
1/4	0.219	0.250	0.281	0.313	0.344	0.375	0.406	0.250	0.281	0.328	0.375	0.406	0.438	0.469
5/16	0.273	0.313	0.352	0.391	0.430	0.469	0.508	0.313	0.352	0.410	0.469	0.508	0.547	0.586
3/8	0.328	0.375	0.422	0.469	0.516	0.563	0.609	0.375	0.422	0.492	0.563	0.609	0.656	0.703
7/16	0.383	0.438	0.492	0.547	0.602	0.656	0.711	0.438	0.492	0.574	0.656	0.711	0.766	0.820
1/2	0.438	0.500	0.563	0.625	0.688	0.750	0.813	0.500	0.563	0.656	0.750	0.813	0.875	0.938
9/16	0.492	0.563	0.633	0.703	0.773	0.844	0.914	0.563	0.633	0.738	0.844	0.914	0.984	1.05
5/8	0.547	0.625	0.703	0.781	0.859	0.938	1.02	0.625	0.703	0.820	0.938	1.02	1.09	1.17
11/16	0.602	0.688	0.773	0.859	0.945	1.03	1.12	0.688	0.773	0.902	1.03	1.12	1.20	1.29
3/4	0.656	0.750	0.844	0.938	1.03	1.13	1.22	0.750	0.844	0.984	1.13	1.22	1.31	1.41
13/16	0.711	0.813	0.914	1.02	1.12	1.22	1.32	0.813	0.914	1.07	1.22	1.32	1.42	1.52
7/8	0.766	0.875	0.984	1.09	1.20	1.31	1.42	0.875	0.984	1.15	1.31	1.42	1.53	1.64
15/16	0.820	0.938	1.05	1.17	1.29	1.41	1.52	0.938	1.05	1.23	1.41	1.52	1.64	1.76
1	0.875	1.00	1.13	1.25	1.38	1.50	1.63	1.00	1.13	1.31	1.50	1.63	1.75	1.88

Thickness t, in.	$C \times t$							$D \times t$						
	Bolt Diameter d_b, in.							Bolt Diameter d_b, in.						
	3/4	7/8	1	1 1/8	1 1/4	1 3/8	1 1/2	3/4	7/8	1	1 1/8	1 1/4	1 3/8	1 1/2
3/16	0.199	0.223	0.258	0.293	0.316	0.340	0.363	0.363	0.422	0.480	0.539	0.598	0.656	0.715
1/4	0.266	0.297	0.344	0.391	0.422	0.453	0.484	0.484	0.563	0.641	0.719	0.797	0.875	0.953
5/16	0.332	0.371	0.430	0.488	0.527	0.566	0.605	0.605	0.703	0.801	0.898	0.996	1.09	1.19
3/8	0.398	0.445	0.516	0.586	0.633	0.680	0.727	0.727	0.844	0.961	1.08	1.20	1.31	1.43
7/16	0.465	0.520	0.602	0.684	0.738	0.793	0.848	0.848	0.984	1.12	1.26	1.39	1.53	1.67
1/2	0.531	0.594	0.688	0.781	0.844	0.906	0.969	0.969	1.13	1.28	1.44	1.59	1.75	1.91
9/16	0.598	0.668	0.773	0.879	0.949	1.02	1.09	1.09	1.27	1.44	1.62	1.79	1.97	2.14
5/8	0.664	0.742	0.859	0.977	1.05	1.13	1.21	1.21	1.41	1.60	1.80	1.99	2.19	2.38
11/16	0.730	0.816	0.945	1.07	1.16	1.25	1.33	1.33	1.55	1.76	1.98	2.19	2.41	2.62
3/4	0.797	0.891	1.03	1.17	1.27	1.36	1.45	1.45	1.69	1.92	2.16	2.39	2.63	2.86
13/16	0.863	0.965	1.12	1.27	1.37	1.47	1.57	1.57	1.83	2.08	2.34	2.59	2.84	3.10
7/8	0.930	1.04	1.20	1.37	1.48	1.59	1.70	1.70	1.97	2.24	2.52	2.79	3.06	3.34
15/16	0.996	1.11	1.29	1.46	1.58	1.70	1.82	1.82	2.11	2.40	2.70	2.99	3.28	3.57
1	1.06	1.19	1.38	1.56	1.69	1.81	1.94	1.94	2.25	2.56	2.88	3.19	3.50	3.81

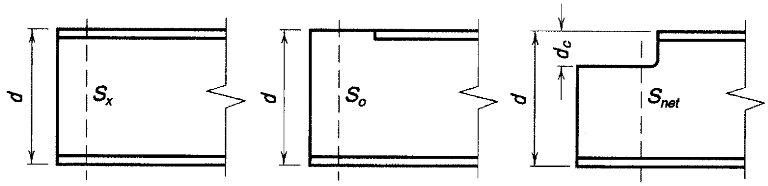
Table 9–2
Elastic Section Moduli for Coped W Shapes



Shape	d in.	t_f in.	S_x in.^{3}	S_o in.^{3}	S_{net}, in.^{3}								
					d_c, in.								
					2	3	4	5	6	7	8	9	10
W44×335	44.0	1.77	1410	494	453	433	413	394	375	357	339	321	304
×290	43.6	1.58	1240	415	380	363	346	330	314	298	283	268	254
×262	43.3	1.42	1110	372	340	325	310	295	281	267	253	240	227
×230	42.9	1.22	971	330	301	288	274	261	249	236	224	212	200
W40×593	43.0	3.23	2340	810	—	—	671	639	607	575	545	515	486
×503	42.1	2.75	1980	671	—	582	554	527	500	473	448	423	398
×431	41.3	2.36	1690	567	—	491	467	444	421	398	376	355	334
×397	41.0	2.20	1560	512	—	444	422	400	379	359	339	319	300
×372	40.6	2.05	1460	480	—	415	394	374	354	335	316	298	280
×362	40.6	2.01	1420	463	—	400	380	361	342	323	305	287	270
×324	40.2	1.81	1280	408	371	352	335	317	300	284	268	252	237
×297	39.8	1.65	1170	374	339	323	306	290	275	259	245	230	216
×277	39.7	1.58	1100	335	304	289	274	260	246	232	219	206	193
×249	39.4	1.42	993	299	271	258	245	232	219	207	195	183	172
×215	39.0	1.22	859	256	231	220	208	197	186	176	166	156	146
×199	38.7	1.07	770	247	224	213	202	191	180	170	160	150	141
W40×392	41.6	2.52	1440	579	—	503	478	454	431	408	386	364	343
×331	40.8	2.13	1210	483	—	419	398	378	358	339	320	302	284
×327	40.8	2.13	1200	470	—	407	387	367	348	329	311	293	276
×294	40.4	1.93	1080	417	379	360	342	325	308	291	275	259	243
×278	40.2	1.81	1020	397	361	344	326	310	293	277	262	246	232
×264	40.0	1.73	971	371	337	321	305	289	274	259	244	230	216
×235	39.7	1.58	875	320	291	276	262	249	235	222	210	197	185
×211	39.4	1.42	786	286	259	246	234	221	209	198	186	175	165
×183	39.0	1.20	675	243	221	210	199	188	178	168	158	149	140
×167	38.6	1.03	600	234	212	201	191	181	171	161	152	143	134
×149	38.2	0.830	513	217	196	186	177	167	158	149	140	132	123

—Indicates that cope depth is less than flange thickness.

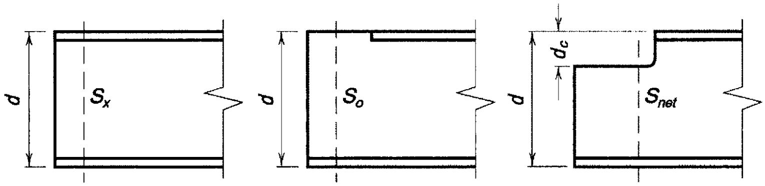
Table 9–2 (continued)
Elastic Section Moduli for Coped W Shapes



Shape	d in.	t_f in.	S_x in.^{3}	S_o in.^{3}	S_{net}, in.^{3}								
					d_c, in.								
					2	3	4	5	6	7	8	9	10
W36×800	42.6	4.29	3040	1040	—	—	—	818	776	735	695	656	618
×652	41.1	3.54	2460	816	—	—	669	635	601	568	536	505	475
×529	39.8	2.91	1990	636	—	547	519	491	464	438	413	388	364
×487	39.3	2.68	1830	581	—	499	473	448	423	399	375	352	330
×441	38.9	2.44	1650	518	—	444	420	398	375	354	332	312	292
×395	38.4	2.20	1490	457	—	391	370	350	330	311	292	274	256
×361	38.0	2.01	1350	412	—	352	333	315	297	279	262	246	230
×330	37.7	1.85	1240	371	335	317	300	283	267	251	235	220	206
×302	37.3	1.68	1130	338	305	289	273	258	243	228	214	200	187
×282	37.1	1.57	1050	314	283	268	253	239	225	211	198	185	173
×262	36.9	1.44	972	294	264	250	236	223	210	197	185	172	161
×247	36.7	1.35	913	277	249	236	223	210	198	185	174	162	151
×231	36.5	1.26	854	260	234	222	209	197	186	174	163	152	142
W36×256	37.4	1.73	895	329	297	281	266	251	237	223	209	196	183
×232	37.1	1.57	809	295	266	251	238	224	211	199	186	174	163
×210	36.7	1.36	719	272	245	232	219	207	195	183	172	161	150
×194	36.5	1.26	664	249	224	212	201	189	178	167	157	146	137
×182	36.3	1.18	623	234	211	199	188	178	167	157	147	137	128
×170	36.2	1.10	581	218	196	185	175	165	155	146	137	128	119
×160	36.0	1.02	542	206	185	175	165	156	147	138	129	120	112
×150	35.9	0.940	504	195	176	166	157	148	139	130	122	114	106
×135	35.6	0.790	439	181	163	154	145	137	129	121	113	105	98.1
W33×387	36.0	2.28	1350	413	—	349	329	310	291	272	254	237	220
×354	35.6	2.09	1240	373	—	315	297	279	262	245	229	213	198
×318	35.2	1.89	1110	330	295	278	262	246	230	216	201	187	173
×291	34.8	1.73	1020	300	268	253	238	223	209	195	182	169	157
×263	34.5	1.57	919	268	239	226	212	199	186	174	162	151	139
×241	34.2	1.40	831	250	223	210	197	185	173	162	150	140	129
×221	33.9	1.28	759	230	205	193	181	170	159	148	138	128	118
×201	33.7	1.15	686	209	186	175	165	154	144	135	125	116	107

—Indicates that cope depth is less than flange thickness.

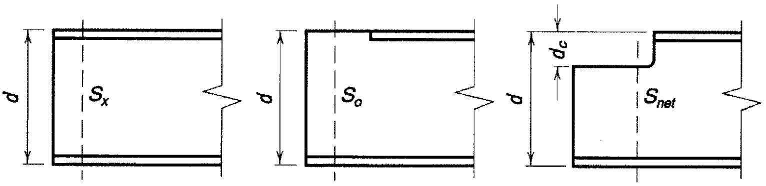
Table 9–2 (continued)
Elastic Section Moduli for Coped W Shapes



Shape	d in.	t_f in.	S_x in.^{3}	S_o in.^{3}	S_{net}, in.^{3} — d_c, in. 2	3	4	5	6	7	8	9	10
W33×169	33.8	1.22	549	191	170	161	151	141	132	124	115	107	98.6
×152	33.5	1.06	487	176	157	148	139	130	122	114	106	97.9	90.5
×141	33.3	0.960	448	165	147	139	130	122	114	106	98.8	91.6	84.6
×130	33.1	0.855	406	155	138	130	122	114	107	99.6	92.5	85.7	79.2
×118	32.9	0.740	359	143	128	120	113	106	98.6	91.9	85.4	79.1	73.0
W30×391	33.2	2.44	1250	378	—	315	295	276	257	239	222	205	188
×357	32.8	2.24	1140	339	—	282	264	246	230	213	197	182	167
×326	32.4	2.05	1040	305	—	254	237	221	206	191	177	163	150
×292	32.0	1.85	930	269	238	223	208	194	180	167	155	142	130
×261	31.6	1.65	829	240	212	198	185	172	160	148	137	126	115
×235	31.3	1.50	748	211	186	174	163	152	141	130	120	110	101
×211	30.9	1.32	665	192	170	159	148	138	128	118	109	99.8	91.2
×191	30.7	1.19	600	174	153	143	133	124	115	106	97.7	89.6	81.8
×173	30.4	1.07	541	158	139	130	121	112	104	96.1	88.4	81.0	73.9
×148	30.7	1.18	436	152	134	125	117	109	101	93.3	86.0	78.9	72.1
×132	30.3	1.00	380	139	123	115	107	99.3	92.1	85.1	78.3	71.8	65.5
×124	30.2	0.930	355	131	115	108	100	93.4	86.5	79.9	73.6	67.4	61.5
×116	30.0	0.850	329	124	109	102	95.3	88.6	82.1	75.8	69.7	63.9	58.2
×108	29.8	0.760	299	118	103	96.5	89.9	83.6	77.4	71.4	65.7	60.1	54.8
×99	29.7	0.670	269	110	96.4	90.0	83.9	77.9	72.1	66.5	61.1	56.0	51.0
×90	29.5	0.610	245	98.7	86.7	80.9	75.4	70.0	64.8	59.7	54.9	50.2	45.7
W27×539	32.5	3.54	1570	509	—	—	394	367	341	316	292	269	247
×368	30.4	2.48	1060	321	—	262	244	226	209	193	177	162	147
×336	30.0	2.28	972	287	—	234	218	202	186	172	157	143	130
×307	29.6	2.09	887	259	—	211	196	181	167	154	141	128	116
×281	29.3	1.93	814	233	203	189	176	162	150	137	126	114	104
×258	29.0	1.77	745	212	185	172	159	147	136	124	114	103	93.3
×235	28.7	1.61	677	193	168	156	145	134	123	113	103	93.2	84.2
×217	28.4	1.50	627	174	152	141	130	120	111	101	92.3	83.7	75.5
×194	28.1	1.34	559	155	134	125	115	106	97.6	89.3	81.3	73.6	66.3
×178	27.8	1.19	505	145	126	117	108	99.7	91.5	83.6	76.1	68.8	61.9
×161	27.6	1.08	458	131	113	105	97.2	89.5	82.0	74.9	68.1	61.5	55.3
×146	27.4	0.975	414	118	102	95.0	87.7	80.7	74.0	67.5	61.3	55.3	49.7

—Indicates that cope depth is less than flange thickness.

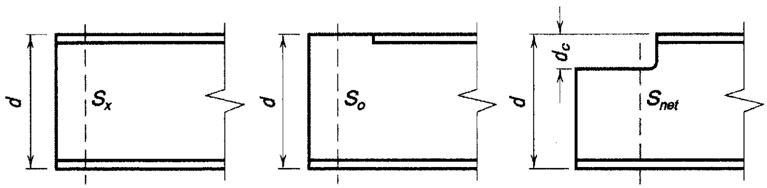
Table 9–2 (continued)
Elastic Section Moduli for Coped W Shapes



Shape	d in.	t_f in.	S_x in.^{3}	S_o in.^{3}	S_{net}, in.^{3}								
					d_c, in.								
					2	3	4	5	6	7	8	9	10
W27×129	27.6	1.10	345	117	101	94.0	86.9	80.1	73.5	67.2	61.1	55.3	49.7
×114	27.3	0.930	299	106	91.6	84.9	78.4	72.2	66.2	60.5	54.9	49.6	44.6
×102	27.1	0.830	267	94.2	81.6	75.6	69.8	64.2	58.9	53.7	48.8	44.0	39.5
×94	26.9	0.745	243	88.0	76.2	70.6	65.1	59.9	54.9	50.1	45.4	41.0	36.8
×84	26.7	0.640	213	80.5	69.7	64.5	59.5	54.7	50.1	45.7	41.4	37.4	33.5
W24×370	28.0	2.72	957	295	—	237	219	201	184	168	153	138	124
×335	27.5	2.48	864	261	—	209	193	177	162	147	133	120	108
×306	27.1	2.28	789	234	—	186	172	157	144	131	118	106	94.9
×279	26.7	2.09	718	210	—	167	154	141	128	116	105	94.3	84.0
×250	26.3	1.89	644	184	158	146	134	123	112	101	91.2	81.7	72.6
×229	26.0	1.73	588	167	143	132	121	111	101	91.0	81.8	73.1	64.9
×207	25.7	1.57	531	149	127	117	107	98.0	89.0	80.4	72.2	64.4	57.0
×192	25.5	1.46	491	136	117	107	98.2	89.5	81.2	73.3	65.8	58.6	51.8
×176	25.2	1.34	450	124	106	97.6	89.4	81.4	73.8	66.5	59.6	53.0	46.8
×162	25.0	1.22	414	115	98.0	90.0	82.3	74.9	67.9	61.1	54.7	48.6	42.8
×146	24.7	1.09	371	104	88.5	81.2	74.2	67.5	61.1	54.9	49.1	43.6	38.3
×131	24.5	0.960	329	94.4	80.3	73.7	67.3	61.1	55.3	49.7	44.3	39.3	34.5
×117	24.3	0.850	291	84.4	71.7	65.7	60.0	54.5	49.2	44.2	39.4	34.8	30.5
×104	24.1	0.750	258	75.4	64.1	58.7	53.5	48.6	43.8	39.3	35.0	30.9	27.1
W24×103	24.5	0.980	245	82.9	70.7	64.9	59.3	53.9	48.8	43.9	39.2	34.8	30.6
×94	24.3	0.875	222	76.2	64.9	59.5	54.3	49.4	44.6	40.1	35.8	31.7	27.9
×84	24.1	0.770	196	68.3	58.0	53.2	48.6	44.1	39.8	35.8	31.9	28.2	24.8
×76	23.9	0.680	176	62.6	53.2	48.7	44.5	40.4	36.4	32.7	29.1	25.8	22.6
×68	23.7	0.585	154	57.5	48.8	44.7	40.8	37.0	33.4	29.9	26.6	23.5	20.6
×62	23.7	0.590	131	56.9	48.3	44.3	40.4	36.7	33.1	29.7	26.5	23.4	20.5
×55	23.6	0.505	114	51.1	43.4	39.7	36.2	32.9	29.7	26.6	23.7	20.9	18.3
W21×201	23.0	1.63	461	125	105	95.2	86.2	77.6	69.4	61.6	54.2	47.3	40.8
×182	22.7	1.48	417	111	93.3	84.8	76.6	68.8	61.4	54.4	47.8	41.6	35.8
×166	22.5	1.36	380	99.3	83.0	75.3	68.0	61.0	54.4	48.1	42.2	36.6	31.4
×147	22.1	1.15	329	91.2	76.1	68.9	62.1	55.7	49.5	43.7	38.2	33.1	28.2
×132	21.8	1.04	295	81.0	67.5	61.1	55.0	49.2	43.7	38.5	33.6	29.0	24.7
×122	21.7	0.960	273	74.1	61.6	55.7	50.2	44.8	39.8	35.0	30.5	26.3	22.4
×111	21.5	0.875	249	67.1	55.7	50.4	45.3	40.4	35.9	31.5	27.4	23.6	20.1
×101	21.4	0.800	227	60.4	50.1	45.3	40.7	36.3	32.1	28.2	24.5	21.1	17.9

—Indicates that cope depth is less than flange thickness.

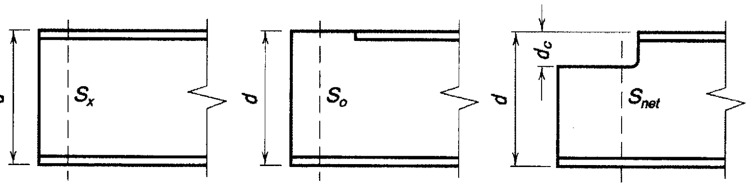
Table 9–2 (continued)
Elastic Section Moduli for Coped W Shapes



Shape	d	t_f	S_x	S_o	S_{net}, in.^{3} d_c, in.								
	in.	in.	in.^{3}	in.^{3}	2	3	4	5	6	7	8	9	10
W21×93	21.6	0.930	192	67.2	56.0	50.7	45.7	40.9	36.3	32.0	27.9	24.1	20.5
×83	21.4	0.835	171	59.0	49.1	44.4	40.0	35.7	31.7	27.9	24.3	20.9	17.8
×73	21.2	0.740	151	51.5	42.7	38.7	34.8	31.0	27.5	24.2	21.0	18.1	15.3
×68	21.1	0.685	140	48.1	39.9	36.1	32.4	29.0	25.6	22.5	19.6	16.8	14.2
×62	21.0	0.615	127	44.1	36.5	33.0	29.7	26.5	23.4	20.5	17.8	15.3	12.9
×55	20.8	0.522	110	40.1	33.2	30.0	26.9	24.0	21.2	18.6	16.1	13.8	11.7
×48	20.6	0.430	93.0	36.2	30.0	27.0	24.2	21.6	19.1	16.7	14.5	12.4	10.4
W21×57	21.1	0.650	111	43.4	36.1	32.6	29.3	26.2	23.2	20.4	17.7	15.2	12.9
×50	20.8	0.535	94.5	39.2	32.5	29.4	26.4	23.6	20.8	18.3	15.9	13.6	11.5
×44	20.7	0.450	81.6	35.2	29.1	26.3	23.6	21.0	18.6	16.3	14.1	12.1	10.2
W18×311	22.3	2.74	624	186	—	140	126	113	100	88.2	77.0	66.5	56.8
×283	21.9	2.50	565	166	—	124	111	99.3	87.8	77.1	67.0	57.6	48.9
×258	21.5	2.30	514	148	—	110	98.3	87.4	77.2	67.5	58.5	50.0	42.3
×234	21.1	2.11	466	130	—	96.1	85.9	76.2	67.1	58.5	50.4	43.0	36.1
×211	20.7	1.91	419	115	94.5	84.8	75.6	66.9	58.7	51.0	43.8	37.1	31.0
×192	20.4	1.75	380	102	83.4	74.7	66.5	58.7	51.4	44.5	38.1	32.1	26.7
×175	20.0	1.59	344	92.1	75.1	67.2	59.7	52.6	45.9	39.6	33.8	28.4	23.5
×158	19.7	1.44	310	81.7	66.4	59.3	52.6	46.2	40.2	34.6	29.4	24.6	
×143	19.5	1.32	282	72.5	58.8	52.4	46.4	40.7	35.4	30.4	25.7	21.5	
×130	19.3	1.20	256	65.2	52.8	47.0	41.5	36.4	31.5	27.0	22.8	19.0	
×119	19.0	1.06	231	61.7	49.8	44.3	39.1	34.2	29.5	25.2	21.2	17.6	
×106	18.7	0.940	204	54.4	43.8	38.9	34.3	29.9	25.8	22.0	18.5	15.2	
×97	18.6	0.870	188	48.9	39.3	34.9	30.7	26.8	23.1	19.6	16.4	13.5	
×86	18.4	0.770	166	43.1	34.6	30.6	26.9	23.4	20.2	17.1	14.3	11.7	
×76	18.2	0.680	146	37.6	30.1	26.7	23.4	20.3	17.5	14.8	12.3	10.1	
W18×71	18.5	0.810	127	42.4	34.1	30.3	26.7	23.3	20.1	17.1	14.3	11.8	
×65	18.4	0.750	117	38.3	30.8	27.3	24.0	20.9	18.0	15.3	12.8	10.5	
×60	18.2	0.695	108	35.0	28.1	24.9	21.9	19.1	16.4	13.9	11.6	9.53	
×55	18.1	0.630	98.3	32.4	26.0	23.0	20.2	17.6	15.1	12.8	10.7	8.72	
×50	18.0	0.570	88.9	29.1	23.4	20.7	18.2	15.8	13.5	11.5	9.54		
W18×46	18.1	0.605	78.8	28.9	23.2	20.6	18.1	15.7	13.5	11.5	9.56	7.81	
×40	17.9	0.525	68.4	24.9	20.0	17.7	15.5	13.5	11.6	9.80	8.16		
×35	17.7	0.425	57.6	22.7	18.2	16.1	14.1	12.3	10.5	8.88	7.37		

—Indicates that cope depth is less than flange thickness.
Note: Values are omitted when cope depth exceeds $d/2$.

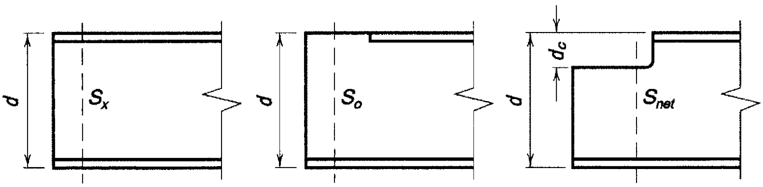
Table 9–2 (continued)
Elastic Section Moduli for Coped W Shapes



Shape	d, in.	t_f, in.	S_x, in.^{3}	S_o, in.^{3}	S_{net}, in.^{3} — d_c, in. 2	3	4	5	6	7	8	9	10
W16×100	17.0	0.985	175	44.4	34.9	30.5	26.4	22.6	19.0	15.7	12.8		
×89	16.8	0.875	155	39.0	30.6	26.7	23.1	19.7	16.5	13.6	11.0		
×77	16.5	0.760	134	33.1	25.9	22.6	19.4	16.5	13.8	11.4	9.13		
×67	16.3	0.665	117	28.3	22.1	19.2	16.5	14.0	11.7	9.58	7.66		
W16×57	16.4	0.715	92.2	29.4	23.0	20.1	17.3	14.8	12.4	10.2	8.17		
×50	16.3	0.630	81.0	25.6	20.0	17.4	15.0	12.7	10.7	8.74	6.99		
×45	16.1	0.565	72.7	22.9	17.9	15.5	13.4	11.3	9.47	7.75	6.19		
×40	16.0	0.505	64.7	20.1	15.6	13.6	11.7	9.89	8.24	6.73	5.35		
×36	15.9	0.430	56.5	18.8	14.6	12.7	10.9	9.21	7.67	6.25			
W16×31	15.9	0.440	47.2	17.1	13.3	11.6	9.96	8.44	7.03	5.73			
×26	15.7	0.345	38.4	14.9	11.6	10.1	8.64	7.31	6.08	4.95			
W14×730	22.4	4.91	1280	365	—	—	—	220	195	172	151	132	114
×665	21.6	4.52	1150	317	—	—	—	187	165	144	126	109	93.3
×605	20.9	4.16	1040	275	—	—	—	158	139	121	105	89.6	76.2
×550	20.2	3.82	931	238	—	—	153	134	117	101	86.9	73.8	62.1
×500	19.6	3.50	838	208	—	—	131	115	99.4	85.3	72.5	60.9	
×455	19.0	3.21	756	182	—	—	113	98.2	84.6	72.1	60.7	50.6	
×426	18.7	3.04	706	164	—	—	101	87.6	75.2	63.8	53.4	44.2	
×398	18.3	2.85	656	150	—	104	91.1	78.7	67.2	56.7	47.2	38.7	
×370	17.9	2.66	607	135	—	93.7	81.4	70.1	59.6	50.0	41.3		
×342	17.5	2.47	558	122	—	83.4	72.3	61.9	52.3	43.6	35.8		
×311	17.1	2.26	506	107	—	72.7	62.7	53.5	44.9	37.2	30.2		
×283	16.7	2.07	459	94.4	—	63.6	54.6	46.3	38.7	31.8	25.6		
×257	16.4	1.89	415	83.1	64.1	55.5	47.4	40.0	33.3	27.1	21.6		
×233	16.0	1.72	375	73.2	56.1	48.4	41.3	34.6	28.6	23.2	18.3		
×211	15.7	1.56	338	64.9	49.5	42.6	36.1	30.2	24.8	19.9			
×193	15.5	1.44	310	57.6	43.8	37.5	31.7	26.4	21.6	17.3			
×176	15.2	1.31	281	52.2	39.5	33.8	28.5	23.6	19.2	15.2			
×159	15.0	1.19	254	45.7	34.5	29.4	24.7	20.4	16.5	13.0			
×145	14.8	1.09	232	40.9	30.7	26.1	21.9	18.0	14.5	11.4			

—Indicates that cope depth is less than flange thickness.
Note: Values are omitted when cope depth exceeds $d/2$.

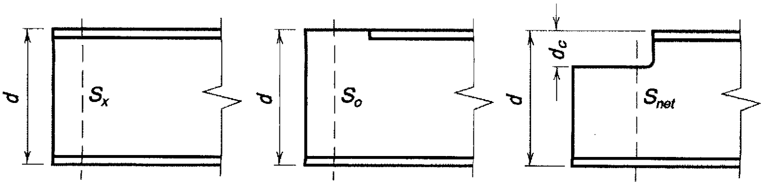
Table 9–2 (continued)
Elastic Section Moduli for Coped W Shapes



Shape	d in.	t_f in.	S_x in.^{3}	S_o in.^{3}	S_{net}, in.^{3}								
					d_c, in.								
					2	3	4	5	6	7	8	9	10
W14×132	14.7	1.03	209	38.1	28.6	24.3	20.3	16.7	13.4	10.5			
×120	14.5	0.940	190	34.2	25.5	21.7	18.1	14.8	11.8	9.20			
×109	14.3	0.860	173	30.0	22.3	18.9	15.7	12.8	10.2	7.91			
×99	14.2	0.780	157	27.2	20.2	17.0	14.2	11.5	9.15	7.04			
×90	14.0	0.710	143	24.3	18.0	15.2	12.6	10.2	8.07	6.18			
W14×82	14.3	0.855	123	28.0	20.9	17.7	14.8	12.1	9.64	7.46			
×74	14.2	0.785	112	24.4	18.2	15.4	12.8	10.4	8.31	6.40			
×68	14.0	0.720	103	22.2	16.5	13.9	11.6	9.41	7.46	5.72			
×61	13.9	0.645	92.1	19.7	14.6	12.3	10.2	8.28	6.54				
W14×53	13.9	0.660	77.8	19.1	14.2	12.0	9.93	8.07	6.39				
×48	13.8	0.595	70.2	17.3	12.8	10.8	8.93	7.23	5.71				
×43	13.7	0.530	62.6	15.3	11.3	9.49	7.84	6.34	4.99				
W14×38	14.1	0.515	54.6	16.0	12.0	10.2	8.48	6.94	5.54	4.28			
×34	14.0	0.455	48.6	14.4	10.8	9.14	7.62	6.22	4.95				
×30	13.8	0.385	42.0	13.2	9.88	8.37	6.96	5.68	4.51				
W14×26	13.9	0.420	35.3	12.3	9.20	7.80	6.50	5.31	4.23				
×22	13.7	0.335	29.0	10.7	7.97	6.75	5.62	4.58	3.64				
W12×336	16.8	2.96	483	123	—	83.1	71.4	60.6	50.8	41.9	34.1		
×305	16.3	2.71	435	108	—	71.4	61.0	51.4	42.7	34.9	28.0		
×279	15.9	2.47	393	96.1	—	63.1	53.5	44.8	36.9	29.8			
×252	15.4	2.25	353	83.7	—	54.2	45.7	38.0	31.0	24.8			
×230	15.1	2.07	321	74.2	—	47.5	39.9	32.9	26.7	21.1			
×210	14.7	1.90	292	65.6	49.0	41.6	34.7	28.5	22.9	17.9			
×190	14.4	1.74	263	57.0	42.3	35.7	29.7	24.2	19.3	14.9			
×170	14.0	1.56	235	49.6	36.5	30.7	25.3	20.5	16.2	12.4			
×152	13.7	1.40	209	43.3	31.6	26.5	21.7	17.5	13.7				
×136	13.4	1.25	186	37.9	27.5	22.9	18.7	14.9	11.6				
×120	13.1	1.11	163	32.8	23.7	19.7	16.0	12.6	9.70				
×106	12.9	0.990	145	27.6	19.8	16.3	13.2	10.4	7.91				
×96	12.7	0.900	131	24.3	17.4	14.3	11.5	9.03	6.83				
×87	12.5	0.810	118	22.2	15.8	13.0	10.4	8.11	6.09				
×79	12.4	0.735	107	19.9	14.1	11.5	9.23	7.16	5.35				
×72	12.3	0.670	97.4	17.9	12.6	10.3	8.24	6.37	4.73				
×65	12.1	0.605	87.9	16.0	11.2	9.16	7.28	5.61	4.14				

—Indicates that cope depth is less than flange thickness.
Note: Values are omitted when cope depth exceeds $d/2$.

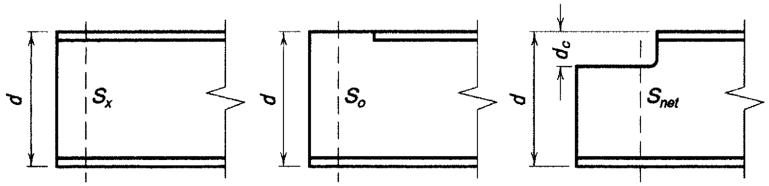
Table 9–2 (continued)
Elastic Section Moduli for Coped W Shapes



Shape	d in.	t_f in.	S_x in.^{3}	S_o in.^{3}	S_{net}, in.^{3} / d_c, in. 2	3	4	5	6	7	8	9	10
W12×58	12.2	0.640	78.0	14.8	10.4	8.52	6.79	5.24	3.88				
×53	12.1	0.575	70.6	13.9	9.75	7.94	6.31	4.85	3.58				
W12×50	12.2	0.640	64.2	14.8	10.4	8.54	6.82	5.27	3.91				
×45	12.1	0.575	57.7	13.1	9.27	7.56	6.02	4.63	3.42				
×40	11.9	0.515	51.5	11.4	8.03	6.54	5.19	3.98					
W12×35	12.5	0.520	45.6	12.3	8.85	7.30	5.89	4.61	3.48				
×30	12.3	0.440	38.6	10.5	7.47	6.15	4.94	3.86	2.90				
×26	12.2	0.380	33.4	9.08	6.47	5.32	4.27	3.32	2.48				
W12×22	12.3	0.425	25.4	9.60	6.89	5.69	4.59	3.59	2.71				
×19	12.2	0.350	21.3	8.39	6.01	4.95	3.98	3.11	2.33				
×16	12.0	0.265	17.1	7.43	5.30	4.36	3.50	2.72					
×14	11.9	0.225	14.9	6.61	4.71	3.86	3.10	2.41					
W10×112	11.4	1.25	126	25.7	17.5	13.9	10.8	8.02					
×100	11.1	1.12	112	22.3	15.0	11.9	9.12	6.72					
×88	10.8	0.990	98.5	19.1	12.8	10.0	7.62	5.54					
×77	10.6	0.870	85.9	16.2	10.7	8.35	6.29	4.52					
×68	10.4	0.770	75.7	13.9	9.13	7.10	5.30	3.77					
×60	10.2	0.680	66.7	12.1	7.88	6.09	4.52	3.18					
×54	10.1	0.615	60.0	10.5	6.78	5.22	3.85	2.69					
×49	10.0	0.560	54.6	9.49	6.13	4.71	3.46	2.40					
W10×45	10.1	0.620	49.1	9.75	6.33	4.88	3.61	2.52					
×39	9.92	0.530	42.1	8.49	5.48	4.20	3.08						
×33	9.73	0.435	35.0	7.49	4.80	3.67	2.67						
W10×30	10.5	0.510	32.4	8.64	5.75	4.51	3.41	2.45					
×26	10.3	0.440	27.9	7.33	4.86	3.80	2.85	2.04					
×22	10.2	0.360	23.2	6.51	4.29	3.34	2.50	1.77					
W10×19	10.2	0.395	18.8	6.52	4.33	3.39	2.55	1.82					
×17	10.1	0.330	16.2	6.01	3.98	3.10	2.33	1.65					
×15	10.0	0.270	13.8	5.53	3.65	2.84	2.12	1.50					
×12	9.87	0.210	10.9	4.43	2.91	2.26	1.68						

Note: Values are omitted when cope depth exceeds $d/2$.

Table 9–2 (continued)
Elastic Section Moduli for Coped W Shapes

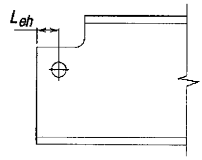


Shape	d in.	t_f in.	S_x in.^{3}	S_o in.^{3}	S_{net}, in.^{3} d_c, in. 2	3	4	5	6	7	8	9	10
W8×67	9.00	0.935	60.4	12.2	7.42	5.44	3.77						
×58	8.75	0.810	52.0	10.4	6.24	4.52	3.08						
×48	8.50	0.685	43.2	7.89	4.63	3.32	2.21						
×40	8.25	0.560	35.5	6.71	3.89	2.74	1.80						
×35	8.12	0.495	31.2	5.66	3.24	2.28	1.47						
×31	8.00	0.435	27.5	5.06	2.88	2.01	1.28						
W8×28	8.06	0.465	24.3	5.04	2.89	2.02	1.30						
×24	7.93	0.400	20.9	4.23	2.40	1.67							
W8×21	8.28	0.400	18.2	4.55	2.67	1.91	1.26						
×18	8.14	0.330	15.2	4.02	2.35	1.66	1.09						
W8×15	8.11	0.315	11.8	4.03	2.36	1.68	1.10						
×13	7.99	0.255	9.91	3.61	2.10	1.49							
×10	7.89	0.205	7.81	2.65	1.54	1.08							

Note: Values are omitted when cope depth exceeds $d/2$.

U_{bs} = 1.0

Table 9–3a
Block Shear Tension Rupture Component
per inch of thickness, kips/in.

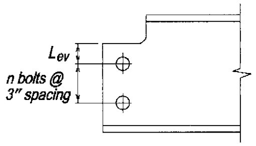


F_u	58 ksi					
L_{eh}, in.	Bolt diameter, d_b, in.					
	3/4		7/8		1	
	$\frac{F_uA_{nt}}{t\Omega}$	$\frac{\phi F_uA_{nt}}{t}$	$\frac{F_uA_{nt}}{t\Omega}$	$\frac{\phi F_uA_{nt}}{t}$	$\frac{F_uA_{nt}}{t\Omega}$	$\frac{\phi F_uA_{nt}}{t}$
	ASD	LRFD	ASD	LRFD	ASD	LRFD
1	16.3	24.5	14.5	21.8	12.7	19.0
1⅛	19.9	29.9	18.1	27.2	16.3	24.5
1¼	23.6	35.3	21.8	32.6	19.9	29.9
1⅜	27.2	40.8	25.4	38.1	23.6	35.3
1½	30.8	46.2	29.0	43.5	27.2	40.8
1⅝	34.4	51.7	32.6	48.9	30.8	46.2
1¾	38.1	57.1	36.3	54.4	34.4	51.7
1⅞	41.7	62.5	39.9	59.8	38.1	57.1
2	45.3	68.0	43.5	65.3	41.7	62.5
2¼	52.6	78.8	50.7	76.1	48.9	73.4
2½	59.8	89.7	58.0	87.0	56.2	84.3
2¾	67.1	101	65.3	97.9	63.4	95.2
3	74.3	111	72.5	109	70.7	106

F_u	65 ksi					
L_{eh}, in.	Bolt diameter, d_b, in.					
	3/4		7/8		1	
	$\frac{F_uA_{nt}}{t\Omega}$	$\frac{\phi F_uA_{nt}}{t}$	$\frac{F_uA_{nt}}{t\Omega}$	$\frac{\phi F_uA_{nt}}{t}$	$\frac{F_uA_{nt}}{t\Omega}$	$\frac{\phi F_uA_{nt}}{t}$
	ASD	LRFD	ASD	LRFD	ASD	LRFD
1	18.3	27.4	16.3	24.4	14.2	21.3
1⅛	22.3	33.5	20.3	30.5	18.3	27.4
1¼	26.4	39.6	24.4	36.6	22.3	33.5
1⅜	30.5	45.7	28.4	42.7	26.4	39.6
1½	34.5	51.8	32.5	48.8	30.5	45.7
1⅝	38.6	57.9	36.6	54.8	34.5	51.8
1¾	42.7	64.0	40.6	60.9	38.6	57.9
1⅞	46.7	70.1	44.7	67.0	42.7	64.0
2	50.8	76.2	48.8	73.1	46.7	70.1
2¼	58.9	88.4	56.9	85.3	54.8	82.3
2½	67.0	101	65.0	97.5	63.0	94.5
2¾	75.2	113	73.1	110	71.1	107
3	83.3	125	81.3	122	79.2	119

ASD	LRFD
Ω = 2.00	ϕ = 0.75

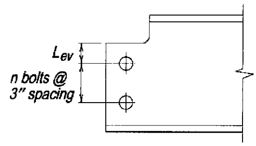
Table 9–3b
Block Shear
Shear Yielding Component
per inch of thickness, kips/in.



L_{ev}, in.	n	F_y, ksi 36 $\frac{0.6F_yA_{gv}}{t\Omega}$ ASD	F_y, ksi 36 $\frac{\phi 0.6F_yA_{gv}}{t}$ LRFD	F_y, ksi 50 $\frac{0.6F_yA_{gv}}{t\Omega}$ ASD	F_y, ksi 50 $\frac{\phi 0.6F_yA_{gv}}{t}$ LRFD	n	F_y, ksi 36 $\frac{0.6F_yA_{gv}}{t\Omega}$ ASD	F_y, ksi 36 $\frac{\phi 0.6F_yA_{gv}}{t}$ LRFD	F_y, ksi 50 $\frac{0.6F_yA_{gv}}{t\Omega}$ ASD	F_y, ksi 50 $\frac{\phi 0.6F_yA_{gv}}{t}$ LRFD
1¼		370	555	514	771		273	409	379	568
1⅜		371	557	516	773		274	411	381	571
1½		373	559	518	776		275	413	383	574
1⅝		374	561	519	779		277	415	384	577
1¾		375	563	521	782		278	417	386	579
1⅞	12	377	565	523	785	9	279	419	388	582
2		378	567	525	788		281	421	390	585
2¼		381	571	529	793		284	425	394	591
2½		383	575	533	799		286	429	398	596
2¾		386	579	536	804		289	433	401	602
3		389	583	540	810		292	437	405	608
1¼		337	506	469	703		240	360	334	501
1⅜		339	508	471	706		242	362	336	503
1½		340	510	473	709		243	364	338	506
1⅝		342	512	474	712		244	367	339	509
1¾		343	514	476	714		246	369	341	512
1⅞	11	344	516	478	717	8	247	371	343	515
2		346	518	480	720		248	373	345	518
2¼		348	522	484	726		251	377	349	523
2½		351	526	488	731		254	381	353	529
2¾		354	531	491	737		257	385	356	534
3		356	535	495	743		259	389	360	540
1¼		305	458	424	636		208	312	289	433
1⅜		306	460	426	638		209	314	291	436
1½		308	462	428	641		211	316	293	439
1⅝		309	464	429	644		212	318	294	442
1¾		310	466	431	647		213	320	296	444
1⅞	10	312	468	433	650	7	215	322	298	447
2		313	470	435	653		216	324	300	450
2¼		316	474	439	658		219	328	304	456
2½		319	478	443	664		221	332	308	461
2¾		321	482	446	669		224	336	311	467
3		324	486	450	675		227	340	315	473

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

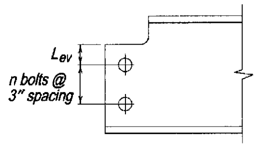
Table 9–3b (continued)
Block Shear
Shear Yielding Component
per inch of thickness, kips/in.



L_{ev}, in.	n	F_y, ksi 36: $\frac{0.6F_yA_{gv}}{t\Omega}$ ASD	36: $\frac{\phi 0.6F_yA_{gv}}{t}$ LRFD	50: $\frac{0.6F_yA_{gv}}{t\Omega}$ ASD	50: $\frac{\phi 0.6F_yA_{gv}}{t}$ LRFD	n	F_y, ksi 36: $\frac{0.6F_yA_{gv}}{t\Omega}$ ASD	36: $\frac{\phi 0.6F_yA_{gv}}{t}$ LRFD	50: $\frac{0.6F_yA_{gv}}{t\Omega}$ ASD	50: $\frac{\phi 0.6F_yA_{gv}}{t}$ LRFD
1¼		175	263	244	366		78.3	117	109	163
1⅜		177	265	246	368		79.6	119	111	166
1½		178	267	248	371		81.0	121	113	169
1⅝		180	269	249	374		82.3	124	114	172
1¾		181	271	251	377		83.7	126	116	174
1⅞	6	182	273	253	380	3	85.0	128	118	177
2		184	275	255	383		86.4	130	120	180
2¼		186	279	259	388		89.1	134	124	186
2½		189	283	263	394		91.8	138	128	191
2¾		192	288	266	399		94.5	142	131	197
3		194	292	270	405		97.2	146	135	203
1¼		143	215	199	298		45.9	68.8	63.8	95.6
1⅜		144	217	201	301		47.2	70.9	65.6	98.4
1½		146	219	203	304		48.6	72.9	67.5	101
1⅝		147	221	204	307		49.9	74.9	69.4	104
1¾		148	223	206	309		51.3	76.9	71.3	107
1⅞	5	150	225	208	312	2	52.7	79.0	73.1	110
2		151	227	210	315		54.0	81.0	75.0	113
2¼		154	231	214	321		56.7	85.0	78.8	118
2½		157	235	218	326		59.4	89.1	82.5	124
2¾		159	239	221	332		62.1	93.1	86.3	129
3		162	243	225	338		64.8	97.2	90.0	135
1¼		111	166	154	231					
1⅜		112	168	156	233					
1½		113	170	158	236					
1⅝		115	172	159	239					
1¾		116	174	161	242					
1⅞	4	117	176	163	245					
2		119	178	165	248					
2¼		121	182	169	253					
2½		124	186	173	259					
2¾		127	190	176	264					
3		130	194	180	270					

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

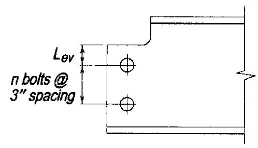
Table 9–3c
Block Shear Shear Rupture Component
per inch of thickness, kips/in.



n	L_{ev}, in.	F_u, ksi											
		58						65					
		¾		⅞		1		¾		⅞		1	
		$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
12	1¼	421	631	396	594	371	556	472	707	444	665	416	623
	1⅜	423	635	398	597	373	560	474	711	446	669	418	627
	1½	425	638	400	600	375	563	477	715	449	673	420	631
	1⅝	427	641	402	604	377	566	479	718	451	676	423	634
	1¾	430	644	405	607	380	569	481	722	453	680	425	638
	1⅞	432	648	407	610	382	573	484	726	456	684	428	642
	2	434	651	409	613	384	576	486	729	458	687	430	645
	2¼	438	657	413	620	388	582	491	737	463	695	435	653
	2½	443	664	418	626	393	589	496	744	468	702	440	660
	2¾	447	670	422	633	397	595	501	751	473	709	445	667
	3	451	677	426	639	401	602	506	759	478	717	450	675
11	1¼	384	576	361	542	338	507	430	645	405	607	379	569
	1⅜	386	579	363	545	340	511	433	649	407	611	381	572
	1½	388	582	365	548	343	514	435	653	410	614	384	576
	1⅝	390	586	368	551	345	517	438	656	412	618	386	580
	1¾	393	589	370	555	347	520	440	660	414	622	389	583
	1⅞	395	592	372	558	349	524	442	664	417	625	391	587
	2	397	595	374	561	351	527	445	667	419	629	394	590
	2¼	401	602	378	568	356	533	450	675	424	636	399	598
	2½	406	608	383	574	360	540	455	682	429	644	403	605
	2¾	410	615	387	581	364	546	459	689	434	651	408	612
	3	414	622	391	587	369	553	464	697	439	658	413	620
10	1¼	347	520	326	489	306	458	389	583	366	548	342	514
	1⅜	349	524	328	493	308	462	391	587	368	552	345	517
	1½	351	527	331	496	310	465	394	590	371	556	347	521
	1⅝	353	530	333	499	312	468	396	594	373	559	350	525
	1¾	356	533	335	502	314	471	399	598	375	563	352	528
	1⅞	358	537	337	506	316	475	401	601	378	567	355	532
	2	360	540	339	509	319	478	403	605	380	570	357	536
	2¼	364	546	344	515	323	484	408	612	385	578	362	543
	2½	369	553	348	522	327	491	413	620	390	585	367	550
	2¾	373	560	352	529	332	498	418	627	395	592	372	558
	3	377	566	357	535	336	504	423	634	400	600	377	565

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

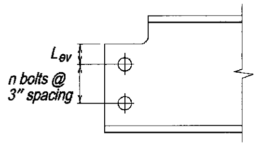
Table 9–3c (continued)
Block Shear
Shear Rupture Component
per inch of thickness, kips/in.



n	L_{ev}, in.	F_u, ksi											
		58						65					
		$^3/_4$		$^7/_8$		1		$^3/_4$		$^7/_8$		1	
		$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
9	1¼	310	465	291	437	273	409	347	521	327	490	306	459
	1⅜	312	468	294	440	275	413	350	525	329	494	308	463
	1½	314	471	296	444	277	416	352	528	332	497	311	466
	1⅝	316	475	298	447	279	419	355	532	334	501	313	470
	1¾	319	478	300	450	282	422	357	536	336	505	316	473
	1⅞	321	481	302	453	284	426	360	539	339	508	318	477
	2	323	484	305	457	286	429	362	543	341	512	321	481
	2¼	327	491	309	463	290	436	367	550	346	519	325	488
	2½	332	498	313	470	295	442	372	558	351	527	330	495
	2¾	336	504	318	476	299	449	377	565	356	534	335	503
	3	340	511	322	483	303	455	381	572	361	541	340	510
8	1¼	273	409	257	385	240	361	306	459	288	431	269	404
	1⅜	275	413	259	388	243	364	308	463	290	435	272	408
	1½	277	416	261	392	245	367	311	466	293	439	274	411
	1⅝	279	419	263	395	247	370	313	470	295	442	277	415
	1¾	282	422	265	398	249	374	316	473	297	446	279	419
	1⅞	284	426	268	401	251	377	318	477	300	450	282	422
	2	286	429	270	405	253	380	321	481	302	453	284	426
	2¼	290	436	274	411	258	387	325	488	307	461	289	433
	2½	295	442	278	418	262	393	330	495	312	468	294	441
	2¾	299	449	283	424	266	400	335	503	317	475	299	448
	3	303	455	287	431	271	406	340	510	322	483	303	455
7	1¼	236	354	222	333	208	312	264	397	249	373	233	349
	1⅜	238	357	224	336	210	315	267	400	251	377	235	353
	1½	240	361	226	339	212	318	269	404	254	380	238	356
	1⅝	243	364	228	343	214	321	272	408	256	384	240	360
	1¾	245	367	231	346	216	325	274	411	258	388	243	364
	1⅞	247	370	233	349	219	328	277	415	261	391	245	367
	2	249	374	235	352	221	331	279	419	263	395	247	371
	2¼	253	380	239	359	225	338	284	426	268	402	252	378
	2½	258	387	244	365	229	344	289	433	273	410	257	386
	2¾	262	393	248	372	234	351	294	441	278	417	262	393
	3	266	400	252	378	238	357	299	448	283	424	267	400

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

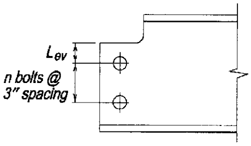
Table 9–3c (continued)
Block Shear
Shear Rupture Component
per inch of thickness, kips/in.



n	L_{ev}, in.	F_u, ksi											
		58						65					
		$^3/_4$		$^7/_8$		1		$^3/_4$		$^7/_8$		1	
		$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
6	1¼	199	299	187	281	175	263	223	335	210	314	196	294
	1⅜	201	302	189	284	177	266	225	338	212	318	199	298
	1½	203	305	191	287	179	269	228	342	215	322	201	302
	1⅝	206	308	194	290	182	272	230	346	217	325	204	305
	1¾	208	312	196	294	184	276	233	349	219	329	206	309
	1⅞	210	315	198	297	186	279	235	353	222	333	208	313
	2	212	318	200	300	188	282	238	356	224	336	211	316
	2¼	216	325	204	307	192	289	243	364	229	344	216	324
	2½	221	331	209	313	197	295	247	371	234	351	221	331
	2¾	225	338	213	320	201	302	252	378	239	358	225	338
	3	229	344	217	326	206	308	257	386	244	366	230	346
5	1¼	162	243	152	228	142	214	182	272	171	256	160	239
	1⅜	164	246	154	232	145	217	184	276	173	260	162	243
	1½	166	250	157	235	147	220	186	280	176	263	165	247
	1⅝	169	253	159	238	149	223	189	283	178	267	167	250
	1¾	171	256	161	241	151	227	191	287	180	271	169	254
	1⅞	173	259	163	245	153	230	194	291	183	274	172	258
	2	175	263	165	248	156	233	196	294	185	278	174	261
	2¼	179	269	170	254	160	240	201	302	190	285	179	269
	2½	184	276	174	261	164	246	206	309	195	293	184	276
	2¾	188	282	178	268	169	253	211	316	200	300	189	283
	3	192	289	183	274	173	259	216	324	205	307	194	291
4	1¼	125	188	117	176	110	165	140	210	132	197	123	185
	1⅜	127	191	120	179	112	168	143	214	134	201	126	188
	1½	129	194	122	183	114	171	145	218	137	205	128	192
	1⅝	132	197	124	186	116	175	147	221	139	208	130	196
	1¾	134	201	126	189	119	178	150	225	141	212	133	199
	1⅞	136	204	128	192	121	181	152	229	144	216	135	203
	2	138	207	131	196	123	184	155	232	146	219	138	207
	2¼	142	214	135	202	127	191	160	239	151	227	143	214
	2½	147	220	139	209	132	197	165	247	156	234	147	221
	2¾	151	227	144	215	136	204	169	254	161	241	152	229
	3	156	233	148	222	140	210	174	261	166	249	157	236

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

Table 9–3c (continued)
Block Shear
Shear Rupture Component
per inch of thickness, kips/in.



n	L_{ev}, in.	F_u, ksi											
		58						65					
		3/4		7/8		1		3/4		7/8		1	
		$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	1¼	88.1	132	82.6	124	77.2	116	98.7	148	92.6	139	86.5	130
	1⅜	90.3	135	84.8	127	79.4	119	101	152	95.1	143	89.0	133
	1½	92.4	139	87.0	131	81.6	122	104	155	97.5	146	91.4	137
	1⅝	94.6	142	89.2	134	83.7	126	106	159	99.9	150	93.8	141
	1¾	96.8	145	91.4	137	85.9	129	108	163	102	154	96.3	144
3	1⅞	99.0	148	93.5	140	88.1	132	111	166	105	157	98.7	148
	2	101	152	95.7	144	90.3	135	113	170	107	161	101	152
	2¼	105	158	100	150	94.6	142	118	177	112	168	106	159
	2½	110	165	104	157	99.0	148	123	185	117	176	111	166
	2¾	114	171	109	163	103	155	128	192	122	183	116	174
	3	119	178	113	170	108	161	133	199	127	190	121	181
	1¼	51.1	76.7	47.8	71.8	44.6	66.9	57.3	85.9	53.6	80.4	50.0	75.0
	1⅜	53.3	79.9	50.0	75.0	46.8	70.1	59.7	89.6	56.1	84.1	52.4	78.6
	1½	55.5	83.2	52.2	78.3	48.9	73.4	62.2	93.2	58.5	87.8	54.8	82.3
	1⅝	57.6	86.5	54.4	81.6	51.1	76.7	64.6	96.9	60.9	91.4	57.3	85.9
	1¾	59.8	89.7	56.6	84.8	53.3	79.9	67.0	101	63.4	95.1	59.7	89.6
2	1⅞	62.0	93.0	58.7	88.1	55.5	83.2	69.5	104	65.8	98.7	62.2	93.2
	2	64.2	96.2	60.9	91.4	57.6	86.5	71.9	108	68.3	102	64.6	96.9
	2¼	68.5	103	65.3	97.9	62.0	93.0	76.8	115	73.1	110	69.5	104
	2½	72.9	109	69.6	104	66.3	99.5	81.7	122	78.0	117	74.3	112
	2¾	77.2	116	73.9	111	70.7	106	86.5	130	82.9	124	79.2	119
	3	81.6	122	78.3	117	75.0	113	91.4	137	87.8	132	84.1	126

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

F_y = 50 ksi

Table 9–4
Beam Bearing Constants

Shape	R_1*		R_2		R_3**		R_4**	
	kips		kips/in.		kips		kips/in.	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	R_1/Ω	ϕR_1	R_2/Ω	ϕR_2	R_3/Ω	ϕR_3	R_4/Ω	ϕR_4
W44×335	218	328	34.2	51.3	332	499	9.99	15.0
×290	170	255	28.8	43.3	243	365	6.81	10.2
×262	144	216	26.2	39.3	199	299	5.70	8.55
×230	119	178	23.7	35.5	159	239	4.94	7.41
W40×593	658	987	59.7	89.5	1040	1550	29.8	44.8
×503	503	755	51.2	76.8	760	1140	22.6	33.8
×431	395	593	44.7	67.0	574	861	17.9	26.8
×397	344	516	40.7	61.0	481	722	14.6	21.8
×372	312	468	38.7	58.0	430	645	13.6	20.4
×362	298	447	37.3	56.0	405	607	12.5	18.7
×324	249	374	33.3	50.0	324	486	9.94	14.9
×297	219	329	31.0	46.5	277	416	8.84	13.3
×277	191	286	27.7	41.5	229	343	6.61	9.91
×249	163	244	25.0	37.5	186	280	5.45	8.18
×215	130	195	21.7	32.5	139	209	4.17	6.26
×199	122	182	21.7	32.5	130	195	4.82	7.23
W40×392	436	655	47.2	70.8	644	965	19.5	29.3
×331	336	504	40.7	61.0	473	710	15.1	22.7
×327	326	488	39.3	59.0	451	676	13.7	20.5
×294	275	412	35.3	53.0	365	548	11.0	16.6
×278	255	383	34.2	51.3	336	504	10.7	16.1
×264	233	349	32.0	48.0	298	447	9.24	13.9
×235	191	286	27.7	41.5	229	343	6.61	9.91
×211	162	243	25.0	37.5	186	279	5.47	8.21
×183	129	193	21.7	32.5	138	207	4.24	6.36
×167	119	179	21.7	32.5	128	192	5.02	7.52
×149	106	158	21.0	31.5	110	165	5.70	8.55
W36×800	1040	1560	79.3	119	1830	2750	53.4	80.0
×652	737	1110	65.7	98.5	1250	1880	38.0	57.0
×529	518	777	53.7	80.5	839	1260	26.0	39.1
×487	454	680	50.0	75.0	724	1090	23.1	34.7
×441	384	576	45.3	68.0	597	895	19.2	28.8
×395	320	480	40.7	61.0	481	722	15.5	23.3
×361	276	414	37.3	56.0	405	607	13.3	19.9
×330	238	357	34.0	51.0	337	506	11.0	16.5
×302	207	311	31.5	47.3	287	430	9.72	14.6
×282	186	279	29.5	44.3	251	377	8.60	12.9
×262	167	251	28.0	42.0	222	334	8.07	12.1
×247	153	230	26.7	40.0	200	300	7.47	11.2
×231	140	210	25.3	38.0	179	269	6.90	10.3

* When compressive force is applied at a distance greater than d from the beam end, this value may be multiplied by two.

** When compressive force is applied at a distance greater than $d/2$ from the beam end, this value may be multiplied by two.

Table 9–4 (continued)
Beam Bearing Constants

F_y = 50 ksi

Nominal Wt.	R_5		R_6		($N = 3^1/_4$)						V_{nx}/Ω_v	$\phi_v V_{nx}$
					$x < d/2$		$d/2 \le x \le d$		$x > d$			
	kips		kips/in.		kips		kips		kips			
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	kips	kips
lb/ft	R_5/Ω	ϕR_5	R_6/Ω	ϕR_6	R_n/Ω	ϕR_n	R_n/Ω	ϕR_n	R_n/Ω	ϕR_n	ASD	LRFD
335	303	455	13.3	20.0	329	495	329	495	547	823	902	1350
290	223	335	9.08	13.6	264	396	264	396	434	651	755	1130
262	183	274	7.60	11.4	218	327	229	344	373	560	680	1020
230	145	218	6.59	9.88	175	263	196	293	315	471	547	823
593	951	1430	39.8	59.7	—	—	—	—	1510	2260	1540	2310
503	697	1050	30.1	45.1	—	—	—	—	1170	1760	1290	1940
431	525	787	23.8	35.7	—	—	—	—	935	1400	1110	1660
397	442	662	19.4	29.1	—	—	—	—	820	1230	999	1500
372	394	590	18.1	27.1	438	657	438	657	750	1120	943	1410
362	371	557	16.6	24.9	419	629	419	629	717	1080	908	1360
324	297	446	13.3	19.9	356	534	357	537	606	911	803	1200
297	254	381	11.8	17.7	306	459	320	480	539	809	741	1110
277	211	317	8.81	13.2	250	375	281	421	472	707	659	988
249	172	258	7.27	10.9	204	307	244	366	407	610	591	886
215	129	193	5.56	8.34	153	229	201	301	305	459	507	760
199	118	177	6.42	9.64	146	218	193	288	291	437	503	754
392	589	884	26.1	39.1	—	—	—	—	1030	1540	1180	1760
331	432	648	20.2	30.3	—	—	—	—	804	1210	995	1490
327	413	620	18.2	27.3	—	—	—	—	780	1170	963	1440
294	335	503	14.7	22.1	390	584	390	584	665	996	856	1280
278	308	461	14.3	21.4	366	550	366	550	621	933	823	1230
264	273	410	12.3	18.5	328	492	337	505	570	854	768	1150
235	211	317	8.81	13.2	250	375	281	421	472	707	659	988
211	172	258	7.30	10.9	204	306	243	365	405	608	591	886
183	127	191	5.66	8.48	152	228	200	299	304	455	507	760
167	115	172	6.69	10.0	144	216	190	285	289	433	502	753
149	95.2	143	7.60	11.4	129	193	174	260	257	386	432	650
800	1680	2520	71.1	107	—	—	—	—	2340	3510	2030	3040
652	1150	1720	50.7	76.0	—	—	—	—	1690	2540	1620	2430
529	770	1160	34.7	52.1	—	—	—	—	1210	1820	1280	1920
487	664	995	30.8	46.3	—	—	—	—	1070	1600	1180	1770
441	547	820	25.6	38.3	—	—	—	—	915	1370	1060	1590
395	442	662	20.7	31.1	452	678	452	678	772	1160	937	1410
361	371	557	17.7	26.6	397	596	397	596	673	1010	851	1280
330	310	465	14.7	22.0	349	523	349	523	587	880	768	1150
302	263	394	13.0	19.4	309	465	309	465	516	776	706	1060
282	230	345	11.5	17.2	279	419	282	423	468	702	657	985
262	203	304	10.8	16.1	248	373	258	388	425	639	619	929
247	182	273	9.96	14.9	224	336	240	360	393	590	587	880
231	162	243	9.20	13.8	201	302	222	334	362	544	555	832

– Indicates that 3.25 in. bearing length is insufficient for end beam reactions since $N < k$.
N = length of bearing.
x = location of concentrated force with respect to the member end.

F_y = 50 ksi

Table 9–4 (continued)
Beam Bearing Constants

Shape	R_1*		R_2		R_3**		R_4**	
	kips		kips/in.		kips		kips/in.	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	R_1/Ω	ϕR_1	R_2/Ω	ϕR_2	R_3/Ω	ϕR_3	R_4/Ω	ϕR_4
W36×256	199	298	32.0	48.0	298	447	9.87	14.8
×232	168	253	29.0	43.5	245	367	8.16	12.2
×210	146	219	27.7	41.5	212	319	8.28	12.4
×194	128	192	25.5	38.3	181	271	7.04	10.6
×182	117	175	24.2	36.3	161	242	6.42	9.63
×170	105	157	22.7	34.0	142	212	5.71	8.56
×160	96.0	144	21.7	32.5	127	191	5.40	8.10
×150	88.1	132	20.8	31.3	115	173	5.23	7.85
×135	77.1	116	20.0	30.0	99.5	149	5.56	8.34
W33×387	322	483	42.0	63.0	514	771	17.6	26.4
×354	278	417	38.7	58.0	435	652	15.2	22.8
×318	232	348	34.7	52.0	351	527	12.2	18.3
×291	201	302	32.0	48.0	298	447	10.6	15.9
×263	171	256	29.0	43.5	245	367	8.78	13.2
×241	151	227	27.7	41.5	215	323	8.63	12.9
×221	133	200	25.8	38.8	186	278	7.77	11.7
×201	115	173	23.8	35.8	156	234	6.82	10.2
W33×169	107	161	22.3	33.5	146	219	5.27	7.90
×152	92.9	139	21.2	31.8	125	188	5.24	7.85
×141	83.7	126	20.2	30.3	111	167	5.00	7.51
×130	75.2	113	19.3	29.0	98.4	148	4.98	7.47
×118	66.0	99.1	18.3	27.5	84.5	127	4.94	7.42
W30×391	366	549	45.3	68.0	597	895	22.4	33.7
×357	313	469	41.3	62.0	498	747	18.7	28.1
×326	270	404	38.0	57.0	420	630	16.1	24.2
×292	224	336	34.0	51.0	337	506	12.9	19.4
×261	189	283	31.0	46.5	277	416	11.1	16.7
×235	158	237	27.7	41.5	223	335	8.80	13.2
×211	136	204	25.8	38.8	188	283	8.27	12.4
×191	117	175	23.7	35.5	157	235	7.11	10.7
×173	101	152	21.8	32.8	132	198	6.26	9.39
W30×148	99.1	149	21.7	32.5	137	206	5.48	8.22
×132	84.6	127	20.5	30.8	116	174	5.54	8.32
×124	77.0	116	19.5	29.3	104	156	5.16	7.73
×116	70.6	106	18.8	28.3	94.3	141	5.11	7.66
×108	64.0	96.1	18.2	27.3	84.5	127	5.16	7.74
×99	57.2	85.8	17.3	26.0	73.9	111	5.11	7.67
×90	49.3	74.0	15.7	23.5	60.6	90.9	4.16	6.25

* When compressive force is applied at a distance greater than *d* from the beam end, this value may be multiplied by two.
** When compressive force is applied at a distance greater than *d*/2 from the beam end, this value may be multiplied by two.

Table 9–4 (continued)
Beam Bearing Constants

F_y = 50 ksi

Nom-inal Wt.	R_5		R_6		($N = 3^1/_4$)						V_{nx}/Ω_v	$\phi_v V_{nx}$
					$x < d/2$		$d/2 \le x \le d$		$x > d$			
	kips		kips/in.		kips		kips		kips			
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	kips	kips
lb/ft	R_5/Ω	ϕR_5	R_6/Ω	ϕR_6	R_n/Ω	ϕR_n	R_n/Ω	ϕR_n	R_n/Ω	ϕR_n	ASD	LRFD
256	273	410	13.2	19.7	303	454	303	454	502	752	719	1080
232	225	337	10.9	16.3	262	394	262	394	430	647	646	969
210	192	288	11.0	16.6	236	354	236	354	382	573	609	914
194	164	246	9.38	14.1	204	305	211	316	339	508	558	837
182	146	219	8.56	12.8	182	273	196	293	313	468	527	790
170	128	192	7.61	11.4	161	240	179	268	284	425	492	738
160	114	172	7.20	10.8	145	217	167	250	263	394	468	702
150	103	154	6.98	10.5	132	199	156	234	244	366	448	672
135	86.3	129	7.41	11.1	118	176	142	214	219	330	383	576
387	472	708	23.5	35.3	459	688	459	688	781	1170	906	1360
354	399	599	20.2	30.4	404	606	404	606	682	1020	825	1240
318	322	484	16.3	24.5	345	517	345	517	577	865	731	1100
291	273	410	14.1	21.2	305	458	305	458	506	760	669	1000
263	225	337	11.7	17.6	265	397	265	397	436	653	601	901
241	196	294	11.5	17.3	241	362	241	362	392	589	567	851
221	168	252	10.4	15.5	211	316	217	326	350	526	526	789
201	141	211	9.09	13.6	178	267	192	289	307	462	482	722
169	134	201	7.02	10.5	163	245	179	270	286	431	453	680
152	113	170	6.98	10.5	142	214	162	242	255	381	425	638
141	99.9	150	6.67	10.0	127	191	149	224	233	350	403	604
130	87.4	131	6.64	9.97	115	172	138	207	213	320	384	576
118	73.7	111	6.59	9.89	101	151	125	188	191	288	325	488
391	547	820	29.9	44.9	513	770	513	770	879	1320	903	1350
357	457	685	25.0	37.5	447	671	447	671	760	1140	813	1220
326	385	577	21.5	32.2	394	589	394	589	664	993	739	1110
292	310	465	17.3	25.9	335	502	335	502	559	838	653	980
261	254	381	14.9	22.3	290	434	290	434	479	717	588	882
235	205	307	11.7	17.6	248	372	248	372	406	609	520	779
211	171	257	11.0	16.5	215	323	220	330	356	534	480	719
191	142	213	9.48	14.2	180	270	194	290	311	465	436	653
173	119	179	8.35	12.5	152	229	172	259	273	411	399	598
148	126	189	7.31	11.0	155	233	170	255	269	404	399	598
132	105	157	7.39	11.1	134	201	151	227	236	354	373	559
124	93.5	140	6.87	10.3	121	181	140	211	217	327	353	529
116	84.1	126	6.81	10.2	111	166	132	198	202	304	339	509
108	74.2	111	6.88	10.3	101	152	123	185	187	281	325	488
99	63.8	95.7	6.82	10.2	90.5	136	113	170	171	256	308	463
90	52.4	78.6	5.55	8.33	74.1	111	100	150	148	222	249	375

– Indicates that 3.25 in. bearing length is insufficient for end beam reactions since $N < k$.
N = length of bearing.
x = location of concentrated force with respect to the member end.

F_y = 50 ksi

Table 9–4 (continued) Beam Bearing Constants

Shape	R_1*		R_2		R_3**		R_4**	
	kips		kips/in.		kips		kips/in.	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	R_1/Ω	ϕR_1	R_2/Ω	ϕR_2	R_3/Ω	ϕR_3	R_4/Ω	ϕR_4
W27×539	**710**	1070	**65.7**	98.5	**1250**	1880	**48.0**	72.0
×368	**376**	564	**46.0**	69.0	**615**	922	**25.2**	37.8
×336	**322**	483	**42.0**	63.0	**514**	771	**21.1**	31.7
×307	**278**	417	**38.7**	58.0	**435**	652	**18.2**	27.3
×281	**240**	360	**35.3**	53.0	**365**	548	**15.2**	22.8
×258	**209**	313	**32.7**	49.0	**311**	466	**13.3**	19.9
×235	**182**	273	**30.3**	45.5	**265**	398	**11.8**	17.7
×217	**158**	237	**27.7**	41.5	**223**	335	**9.69**	14.5
×194	**133**	199	**25.0**	37.5	**181**	272	**8.09**	12.1
×178	**119**	179	**24.2**	36.3	**162**	243	**8.32**	12.5
×161	**103**	154	**22.0**	33.0	**134**	201	**6.97**	10.5
×146	**88.8**	133	**20.2**	30.3	**112**	168	**5.99**	8.99
W27×129	**86.3**	129	**20.3**	30.5	**120**	181	**5.40**	8.09
×114	**72.6**	109	**19.0**	28.5	**99.9**	150	**5.27**	7.91
×102	**61.3**	91.9	**17.2**	25.8	**81.1**	122	**4.39**	6.58
×94	**54.8**	82.3	**16.3**	24.5	**71.3**	107	**4.24**	6.36
×84	**47.5**	71.2	**15.3**	23.0	**60.1**	90.2	**4.11**	6.17
W24×370	**408**	612	**50.7**	76.0	**744**	1120	**33.3**	50.0
×335	**343**	514	**46.0**	69.0	**615**	922	**27.8**	41.7
×306	**292**	438	**42.0**	63.0	**514**	771	**23.4**	35.0
×279	**250**	376	**38.7**	58.0	**435**	652	**20.2**	30.3
×250	**207**	311	**34.7**	52.0	**351**	527	**16.3**	24.5
×229	**178**	268	**32.0**	48.0	**298**	447	**14.2**	21.3
×207	**150**	225	**29.0**	43.5	**245**	367	**11.8**	17.7
×192	**132**	198	**27.0**	40.5	**212**	318	**10.3**	15.5
×176	**115**	173	**25.0**	37.5	**181**	272	**9.01**	13.5
×162	**101**	152	**23.5**	35.3	**157**	236	**8.30**	12.5
×146	**86.1**	129	**21.7**	32.5	**132**	198	**7.36**	11.0
×131	**73.6**	110	**20.2**	30.3	**111**	167	**6.81**	10.2
×117	**61.9**	92.8	**18.3**	27.5	**90.6**	136	**5.83**	8.74
×104	**52.1**	78.1	**16.7**	25.0	**73.7**	111	**5.00**	7.51
W24×103	**67.8**	102	**18.3**	27.5	**97.2**	146	**5.00**	7.50
×94	**59.0**	88.5	**17.2**	25.8	**83.3**	125	**4.64**	6.96
×84	**49.7**	74.6	**15.7**	23.5	**68.1**	102	**4.04**	6.06
×76	**43.3**	64.9	**14.7**	22.0	**58.0**	86.9	**3.78**	5.68
×68	**37.5**	56.3	**13.8**	20.8	**49.2**	73.9	**3.72**	5.58
×62	**39.1**	58.6	**14.3**	21.5	**52.2**	78.2	**4.10**	6.15
×55	**33.1**	49.6	**13.2**	19.8	**42.5**	63.7	**3.74**	5.61

* When compressive force is applied at a distance greater than *d* from the beam end, this value may be multiplied by two.
** When compressive force is applied at a distance greater than *d*/2 from the beam end, this value may be multiplied by two.

Table 9–4 (continued)
Beam Bearing Constants

F_y = 50 ksi

Nom-inal Wt.	R_5		R_6		($N = 3^1/_4$) x < d/2		d/2 ≤ x ≤ d		x > d		V_{nx}/Ω_v	$\phi_v V_{nx}$
	kips		kips/in.		kips		kips		kips			
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	kips	kips
lb/ft	R_5/Ω	ϕR_5	R_6/Ω	ϕR_6	R_n/Ω	ϕR_n	R_n/Ω	ϕR_n	R_n/Ω	ϕR_n	ASD	LRFD
539	1150	1720	64.0	96.0	—	—	—	—	1630	2460	1280	1920
368	564	846	33.6	50.4	—	—	—	—	902	1350	839	1260
336	472	708	28.2	42.3	459	688	459	688	781	1170	756	1130
307	399	599	24.3	36.4	404	606	404	606	682	1020	687	1030
281	335	503	20.3	30.4	355	532	355	532	595	892	621	931
258	285	428	17.7	26.5	315	472	315	472	524	785	568	852
235	243	364	15.7	23.6	280	421	280	421	462	694	522	782
217	205	307	12.9	19.4	248	372	248	372	406	609	472	708
194	166	249	10.8	16.2	207	311	214	321	347	520	422	632
178	147	220	11.1	16.6	189	284	198	297	317	476	403	605
161	121	182	9.29	13.9	157	235	175	261	278	415	364	546
146	101	151	7.99	12.0	131	197	154	231	243	364	331	497
129	110	166	7.19	10.8	138	207	152	228	239	357	337	506
114	90.4	136	7.03	10.5	117	176	134	202	207	311	311	467
102	73.2	110	5.85	8.78	95.4	143	117	176	179	268	279	419
94	63.7	95.5	5.65	8.48	85.1	128	108	162	163	244	264	396
84	52.8	79.2	5.49	8.23	73.5	110	97.2	146	145	217	246	369
370	682	1020	44.4	66.7	573	859	573	859	981	1470	851	1280
335	564	846	37.1	55.6	493	738	493	738	836	1250	760	1140
306	472	708	31.2	46.7	429	643	429	643	721	1080	684	1030
279	399	599	26.9	40.4	376	565	376	565	626	941	620	930
250	322	484	21.8	32.7	320	480	320	480	527	791	548	822
229	273	410	18.9	28.4	282	424	282	424	460	692	500	749
207	225	337	15.7	23.6	244	366	244	366	394	591	447	671
192	195	292	13.8	20.7	220	330	220	330	352	528	413	619
176	166	249	12.0	18.0	196	295	196	295	311	468	379	568
162	144	215	11.1	16.6	177	267	177	267	278	419	353	529
146	120	179	9.81	14.7	156	234	157	235	243	364	322	482
131	99.9	150	9.08	13.6	133	200	139	208	213	318	296	444
117	81.1	122	7.77	11.7	110	164	121	182	183	275	267	400
104	65.7	98.6	6.67	10.0	90.0	135	106	159	158	237	241	361
103	89.1	134	6.67	10.0	113	170	127	191	195	293	270	405
94	75.7	114	6.19	9.28	98.4	148	115	172	174	261	250	376
84	61.6	92.4	5.39	8.08	81.2	122	101	151	150	226	227	340
76	51.9	77.9	5.05	7.57	70.3	105	91.1	136	134	201	210	316
68	43.4	65.0	4.96	7.44	61.3	92.0	82.3	124	120	180	197	295
62	45.7	68.5	5.47	8.20	65.5	98.2	85.6	128	125	187	204	306
55	36.6	54.9	4.99	7.48	54.7	81.9	76.0	114	109	164	167	251

– Indicates that 3.25 in. bearing length is insufficient for end beam reactions since $N < k$.
N = length of bearing.
x = location of concentrated force with respect to the member end.

F_y = 50 ksi

Table 9–4 (continued)
Beam Bearing Constants

Shape	R_1*		R_2		R_3**		R_4**	
	kips		kips/in.		kips		kips/in.	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	R_1/Ω	ϕR_1	R_2/Ω	ϕR_2	R_3/Ω	ϕR_3	R_4/Ω	ϕR_4
W21×201	162	242	30.3	45.5	267	400	14.5	21.8
×182	137	205	27.7	41.5	222	332	12.3	18.4
×166	116	174	25.0	37.5	182	274	9.97	15.0
×147	99.0	149	24.0	36.0	158	237	10.6	15.9
×132	83.1	125	21.7	32.5	128	193	8.78	13.2
×122	73.0	110	20.0	30.0	110	165	7.50	11.2
×111	63.0	94.5	18.3	27.5	91.9	138	6.39	9.58
×101	54.2	81.3	16.7	25.0	76.2	114	5.29	7.93
W21×93	69.1	104	19.3	29.0	103	154	7.01	10.5
×83	57.3	85.9	17.2	25.8	81.3	122	5.52	8.27
×73	47.0	70.5	15.2	22.8	63.6	95.4	4.33	6.49
×68	42.5	63.7	14.3	21.5	56.2	84.3	3.97	5.95
×62	37.2	55.8	13.3	20.0	47.8	71.7	3.58	5.37
×55	31.9	47.9	12.5	18.8	40.0	59.9	3.51	5.26
×48	27.1	40.7	11.7	17.5	32.7	49.1	3.49	5.24
W21×57	38.8	58.2	13.5	20.3	50.0	75.1	3.51	5.26
×50	32.8	49.2	12.7	19.0	41.3	61.9	3.56	5.34
×44	27.7	41.6	11.7	17.5	33.5	50.2	3.33	5.00
W18×311	410	616	50.7	76.0	747	1120	41.5	62.2
×283	350	525	46.7	70.0	631	946	36.3	54.4
×258	288	432	42.7	64.0	529	793	30.7	46.0
×234	243	364	38.7	58.0	437	656	25.4	38.1
×211	204	306	35.3	53.0	363	545	21.8	32.7
×192	172	258	32.0	48.0	300	450	17.9	26.9
×175	148	222	29.7	44.5	255	382	16.0	24.0
×158	124	187	27.0	40.5	211	316	13.5	20.3
×143	105	157	24.3	36.5	173	259	10.9	16.4
×130	89.4	134	22.3	33.5	145	217	9.41	14.1
×119	79.8	120	21.8	32.8	131	197	10.1	15.1
×106	66.0	99.0	19.7	29.5	106	159	8.43	12.6
×97	56.7	85.1	17.8	26.8	87.9	132	6.84	10.3
×86	46.9	70.3	16.0	24.0	70.3	105	5.64	8.46
×76	38.3	57.5	14.2	21.3	55.0	82.5	4.48	6.72
W18×71	50.0	75.0	16.5	24.8	75.5	113	5.86	8.79
×65	43.2	64.8	15.0	22.5	63.0	94.4	4.78	7.18
×60	37.9	56.9	13.8	20.8	53.7	80.5	4.07	6.11
×55	33.5	50.3	13.0	19.5	46.6	69.8	3.76	5.63
×50	28.8	43.1	11.8	17.8	38.5	57.7	3.15	4.73

* When compressive force is applied at a distance greater than *d* from the beam end, this value may be multiplied by two.
** When compressive force is applied at a distance greater than *d*/2 from the beam end, this value may be multiplied by two.

Table 9–4 (continued) Beam Bearing Constants

F_y = 50 ksi

Nominal Wt.	R_5		R_6		($N = 3^1/_4$) $x < d/2$		$d/2 \le x \le d$		$x > d$		V_{nx}/Ω_v	$\phi_v V_{nx}$
	kips		kips/in.		kips		kips		kips			
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	kips	kips
lb/ft	R_5/Ω	ϕR_5	R_6/Ω	ϕR_6	R_n/Ω	ϕR_n	R_n/Ω	ϕR_n	R_n/Ω	ϕR_n	ASD	LRFD
201	245	367	19.3	29.0	260	390	260	390	422	632	419	629
182	203	304	16.4	24.6	227	340	227	340	364	545	377	566
166	167	251	13.3	19.9	197	296	197	296	313	470	337	506
147	142	213	14.2	21.3	177	266	177	266	276	415	318	476
132	116	173	11.7	17.6	154	231	154	231	237	356	284	426
122	98.8	148	10.0	15.0	134	201	138	208	211	318	260	390
111	82.7	124	8.52	12.8	113	169	122	184	185	278	237	355
101	68.6	103	7.05	10.6	93.4	140	108	163	163	244	214	320
93	92.5	139	9.35	14.0	126	188	132	198	201	302	251	376
83	73.5	110	7.35	11.0	99.2	149	113	170	171	256	221	331
73	57.5	86.2	5.77	8.66	77.7	116	96.4	145	143	215	193	290
68	50.6	75.9	5.29	7.94	69.1	104	89.0	134	131	197	182	273
62	42.8	64.2	4.78	7.16	59.4	89.2	80.4	121	118	177	168	252
55	35.1	52.6	4.68	7.02	51.4	77.0	72.5	109	103	154	156	234
48	27.9	41.8	4.66	6.99	44.0	66.1	65.1	97.6	88.1	132	144	217
57	45.1	67.7	4.67	7.01	61.4	92.2	82.7	124	121	182	171	256
50	36.3	54.5	4.74	7.11	52.9	79.3	74.1	111	106	159	158	237
44	28.9	43.3	4.44	6.66	44.3	66.5	65.7	98.5	88.6	133	145	217
311	685	1030	55.3	83.0	575	863	575	863	985	1480	679	1020
283	578	867	48.4	72.6	502	753	502	753	852	1280	612	918
258	485	728	40.9	61.4	427	640	427	640	715	1070	549	824
234	401	602	33.8	50.8	369	553	369	553	612	917	489	733
211	333	500	29.1	43.6	319	478	319	478	523	784	438	657
192	275	413	23.9	35.9	276	414	276	414	448	672	391	586
175	234	350	21.3	32.0	245	367	245	367	393	589	357	535
158	193	289	18.0	27.0	212	319	212	319	336	506	319	479
143	158	238	14.6	21.8	184	276	184	276	289	433	285	427
130	133	199	12.5	18.8	162	243	162	243	251	377	258	387
119	119	178	13.5	20.2	151	227	151	227	230	347	249	373
106	95.3	143	11.2	16.9	130	195	130	195	196	294	221	332
97	79.4	119	9.12	13.7	110	165	115	172	171	257	199	298
86	63.4	95.0	7.52	11.3	88.6	132	98.9	148	146	219	177	265
76	49.6	74.4	5.97	8.96	69.6	104	84.4	127	123	184	155	232
71	68.3	102	7.81	11.7	94.5	142	104	156	154	231	183	274
65	57.1	85.7	6.38	9.57	78.5	118	92.0	138	135	203	165	248
60	48.7	73.1	5.43	8.15	66.9	100	82.8	125	121	181	151	227
55	42.0	63.0	5.01	7.51	58.8	88.1	75.8	114	109	164	141	212
50	34.7	52.0	4.20	6.30	48.7	73.1	67.1	101	95.9	144	128	192

N = length of bearing.
x = location of concentrated force with respect to the member end.

F_y = 50 ksi

Table 9–4 (continued)
Beam Bearing Constants

Shape	R_1*		R_2		R_3**		R_4**	
	kips		kips/in.		kips		kips/in.	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	R_1/Ω	ϕR_1	R_2/Ω	ϕR_2	R_3/Ω	ϕR_3	R_4/Ω	ϕR_4
W18×46	**30.2**	45.3	**12.0**	18.0	**40.5**	60.7	**3.09**	4.63
×40	**24.3**	36.5	**10.5**	15.8	**30.9**	46.3	**2.40**	3.60
×35	**20.7**	31.0	**10.0**	15.0	**25.8**	38.7	**2.59**	3.89
W16×100	**67.6**	101	**19.5**	29.3	**107**	160	**8.65**	13.0
×89	**55.9**	83.8	**17.5**	26.3	**85.7**	129	**7.13**	10.7
×77	**44.1**	66.1	**15.2**	22.8	**64.4**	96.7	**5.42**	8.13
×67	**35.1**	52.7	**13.2**	19.8	**48.8**	73.1	**4.10**	6.15
W16×57	**40.0**	60.0	**14.3**	21.5	**57.4**	86.1	**4.89**	7.33
×50	**32.7**	49.0	**12.7**	19.0	**44.8**	67.2	**3.87**	5.81
×45	**27.8**	41.7	**11.5**	17.3	**36.7**	55.0	**3.26**	4.88
×40	**23.1**	34.6	**10.2**	15.3	**28.8**	43.2	**2.54**	3.80
×36	**20.5**	30.7	**9.83**	14.8	**25.3**	38.0	**2.72**	4.08
W16×31	**19.3**	28.9	**9.17**	13.8	**23.0**	34.6	**2.15**	3.23
×26	**15.6**	23.3	**8.33**	12.5	**17.7**	26.5	**2.09**	3.13
W14×730	**1410**	2110	**102**	154	**2870**	4310	**190**	285
×665	**1210**	1810	**94.3**	142	**2440**	3660	**167**	251
×605	**1030**	1540	**86.5**	130	**2050**	3080	**145**	218
×550	**876**	1310	**79.3**	119	**1730**	2590	**126**	189
×500	**748**	1120	**73.0**	110	**1460**	2190	**111**	166
×455	**639**	959	**67.2**	101	**1230**	1850	**96.8**	145
×426	**568**	851	**62.5**	93.8	**1080**	1620	**84.0**	126
×398	**508**	762	**59.0**	88.5	**957**	1430	**77.0**	115
×370	**449**	674	**55.2**	82.8	**836**	1250	**68.7**	103
×342	**394**	591	**51.3**	77.0	**723**	1090	**60.9**	91.4
×311	**336**	504	**47.0**	70.5	**606**	909	**52.3**	78.5
×283	**287**	430	**43.0**	64.5	**508**	762	**44.8**	67.1
×257	**244**	365	**39.2**	58.8	**422**	633	**37.9**	56.8
×233	**207**	310	**35.7**	53.5	**350**	524	**32.1**	48.1
×211	**176**	264	**32.7**	49.0	**292**	438	**27.7**	41.6
×193	**151**	227	**29.7**	44.5	**243**	364	**22.8**	34.3
×176	**132**	198	**27.7**	41.5	**208**	313	**20.7**	31.1
×159	**111**	167	**24.8**	37.3	**169**	253	**16.8**	25.1
×145	**95.7**	143	**22.7**	34.0	**141**	211	**14.1**	21.2
W14×132	**87.5**	131	**21.5**	32.3	**127**	190	**12.8**	19.3
×120	**75.6**	113	**19.7**	29.5	**106**	159	**10.9**	16.4
×109	**63.8**	95.7	**17.5**	26.3	**85.0**	127	**8.49**	12.7
×99	**55.7**	83.5	**16.2**	24.3	**71.8**	108	**7.46**	11.2
×90	**48.0**	71.9	**14.7**	22.0	**59.2**	88.8	**6.18**	9.27

* When compressive force is applied at a distance greater than *d* from the beam end, this value may be multiplied by two.
** When compressive force is applied at a distance greater than *d*/2 from the beam end, this value may be multiplied by two.

Table 9–4 (continued)
Beam Bearing Constants

F_y = 50 ksi

Nom-inal Wt.	R_5		R_6		($N = 3^1/_4$) $x < d/2$		($N = 3^1/_4$) $d/2 \le x \le d$		($N = 3^1/_4$) $x > d$		V_{nx}/Ω_v	$\phi_v V_{nx}$
	kips		kips/in.		kips		kips		kips		kips	kips
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD		
lb/ft	R_5/Ω	ϕR_5	R_6/Ω	ϕR_6	R_n/Ω	ϕR_n	R_n/Ω	ϕR_n	R_n/Ω	ϕR_n	ASD	LRFD
46	**36.7**	55.1	**4.11**	6.17	**50.5**	75.7	**69.2**	104	**99.4**	149	**130**	195
40	**28.0**	42.0	**3.20**	4.81	**38.7**	58.0	**58.4**	87.9	**77.4**	116	**113**	169
35	**22.7**	34.1	**3.46**	5.19	**34.2**	51.3	**53.2**	79.8	**68.4**	103	**106**	159
100	**97.2**	146	**11.5**	17.3	**131**	196	**131**	196	**199**	297	**199**	298
89	**77.7**	117	**9.51**	14.3	**109**	164	**113**	169	**169**	253	**176**	264
77	**58.5**	87.7	**7.23**	10.8	**82.0**	123	**93.5**	140	**138**	206	**150**	225
67	**44.3**	66.4	**5.47**	8.20	**62.1**	93.1	**78.0**	117	**113**	170	**129**	194
57	**52.1**	78.1	**6.52**	9.78	**73.3**	110	**86.5**	130	**126**	190	**141**	212
50	**40.6**	60.9	**5.16**	7.74	**57.4**	86.1	**74.0**	111	**107**	160	**124**	185
45	**33.2**	49.8	**4.34**	6.51	**47.3**	71.0	**65.2**	97.9	**93.0**	140	**111**	167
40	**26.1**	39.2	**3.38**	5.07	**37.1**	55.7	**56.3**	84.3	**74.1**	111	**97.7**	146
36	**22.4**	33.6	**3.63**	5.44	**34.2**	51.3	**52.4**	78.8	**68.3**	103	**93.6**	140
31	**20.8**	31.1	**2.87**	4.30	**30.1**	45.1	**49.1**	73.8	**60.0**	90.2	**87.3**	131
26	**15.5**	23.3	**2.78**	4.17	**24.5**	36.9	**42.7**	63.9	**49.0**	73.3	**70.5**	106
730	**2590**	3880	**253**	380	—	—	—	—	**3150**	4720	**1380**	2060
665	**2200**	3290	**223**	335	—	—	—	—	**2730**	4080	**1220**	1840
605	**1850**	2780	**193**	290	—	—	—	—	**2340**	3500	**1090**	1630
550	**1560**	2340	**168**	252	—	—	—	—	**2010**	3010	**963**	1450
500	**1320**	1970	**147**	221	—	—	—	—	**1730**	2600	**858**	1290
455	**1110**	1670	**129**	194	—	—	—	—	**1500**	2250	**767**	1150
426	**973**	1460	**112**	168	—	—	—	—	**1340**	2010	**700**	1050
398	**863**	1290	**103**	154	—	—	—	—	**1210**	1810	**647**	971
370	**754**	1130	**91.6**	137	—	—	—	—	**1080**	1620	**593**	890
342	**652**	978	**81.2**	122	**561**	841	**561**	841	**955**	1430	**540**	810
311	**546**	820	**69.8**	105	**489**	733	**489**	733	**825**	1240	**483**	724
283	**458**	687	**59.7**	89.5	**427**	640	**427**	640	**714**	1070	**432**	648
257	**380**	571	**50.5**	75.7	**371**	556	**371**	556	**615**	921	**385**	577
233	**315**	473	**42.8**	64.2	**323**	484	**323**	484	**530**	794	**343**	515
211	**263**	394	**37.0**	55.5	**282**	423	**282**	423	**458**	687	**308**	462
193	**219**	329	**30.5**	45.7	**248**	372	**248**	372	**399**	599	**276**	413
176	**187**	281	**27.6**	41.4	**222**	333	**222**	333	**354**	531	**253**	379
159	**152**	228	**22.3**	33.5	**192**	288	**192**	288	**303**	455	**223**	335
145	**127**	191	**18.8**	28.2	**169**	254	**169**	254	**265**	397	**201**	302
132	**114**	171	**17.1**	25.7	**157**	236	**157**	236	**245**	367	**189**	284
120	**95.3**	143	**14.5**	21.8	**140**	209	**140**	209	**215**	322	**171**	256
109	**76.9**	115	**11.3**	17.0	**114**	170	**121**	181	**184**	277	**150**	226
99	**64.8**	97.2	**9.95**	14.9	**97.1**	146	**108**	162	**164**	246	**137**	206
90	**53.4**	80.2	**8.24**	12.4	**80.2**	121	**95.8**	143	**144**	215	**123**	185

– Indicates that 3.25 in. bearing length is insufficient for end beam reactions since $N < k$.
N = length of bearing.
x = location of concentrated force with respect to the member end.

F_y = 50 ksi

Table 9–4 (continued)
Beam Bearing Constants

Shape	R_1*		R_2		R_3**		R_4**	
	kips		kips/in.		kips		kips/in.	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	R_1/Ω	ϕR_1	R_2/Ω	ϕR_2	R_3/Ω	ϕR_3	R_4/Ω	ϕR_4
W14×82	61.5	92.2	17.0	25.5	81.1	122	7.83	11.7
×74	51.6	77.4	15.0	22.5	64.4	96.6	5.92	8.88
×68	45.3	68.0	13.8	20.8	54.6	81.9	5.11	7.66
×61	38.6	57.9	12.5	18.8	44.4	66.6	4.25	6.38
W14×53	38.6	57.9	12.3	18.5	44.0	66.1	3.98	5.98
×48	33.6	50.4	11.3	17.0	36.8	55.2	3.46	5.19
×43	28.5	42.7	10.2	15.3	29.5	44.3	2.83	4.25
W14×38	23.6	35.5	10.3	15.5	29.8	44.7	2.96	4.45
×34	20.3	30.5	9.50	14.3	24.7	37.1	2.63	3.94
×30	17.7	26.5	9.00	13.5	21.0	31.4	2.67	4.00
W14×26	17.4	26.1	8.50	12.8	20.1	30.1	2.05	3.08
×22	14.1	21.1	7.67	11.5	15.4	23.1	1.91	2.86
W12×336	526	788	59.2	88.8	979	1470	81.3	122
×305	447	671	54.2	81.3	820	1230	70.2	105
×279	391	587	51.0	76.5	716	1070	66.1	99.1
×252	331	497	46.5	69.8	595	893	56.6	84.9
×230	286	429	42.8	64.3	505	757	49.2	73.8
×210	246	368	39.3	59.0	426	638	42.5	63.7
×190	206	309	35.3	53.0	346	519	34.5	51.7
×170	173	259	32.0	48.0	283	424	29.2	43.8
×152	145	217	29.0	43.5	231	347	24.8	37.2
×136	122	182	26.3	39.5	189	284	21.3	31.9
×120	101	151	23.7	35.5	151	227	17.8	26.8
×106	80.7	121	20.3	30.5	114	171	12.9	19.3
×96	68.7	103	18.3	27.5	93.2	140	10.5	15.8
×87	60.4	90.6	17.2	25.8	80.1	120	9.72	14.6
×79	52.2	78.3	15.7	23.5	66.5	99.8	8.24	12.4
×72	45.4	68.2	14.3	21.5	55.6	83.4	7.00	10.5
×65	39.1	58.6	13.0	19.5	45.6	68.4	5.84	8.77
W12×58	37.2	55.8	12.0	18.0	41.6	62.4	4.32	6.48
×53	33.8	50.7	11.5	17.3	37.0	55.5	4.28	6.42
W12×50	35.1	52.7	12.3	18.5	43.4	65.0	4.69	7.04
×45	30.0	45.0	11.2	16.8	35.4	53.1	3.92	5.88
×40	25.0	37.4	9.83	14.8	27.7	41.5	3.02	4.52
W12×35	20.5	30.7	10.0	15.0	28.5	42.8	3.00	4.50
×30	16.0	24.1	8.67	13.0	21.2	31.8	2.34	3.51
×26	13.0	19.6	7.67	11.5	16.4	24.6	1.89	2.84

* When compressive force is applied at a distance greater than d from the beam end, this value may be multiplied by two.
** When compressive force is applied at a distance greater than $d/2$ from the beam end, this value may be multiplied by two.

Table 9–4 (continued)
Beam Bearing Constants
F_y = 50 ksi

Nominal Wt.	R_5		R_6		($N = 3^1/_4$) $x < d/2$		$d/2 \le x \le d$		$x > d$		V_{nx}/Ω_v	$\phi_v V_{nx}$
	kips		kips/in.		kips		kips		kips			
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	kips	kips
lb/ft	R_5/Ω	ϕR_5	R_6/Ω	ϕR_6	R_n/Ω	ϕR_n	R_n/Ω	ϕR_n	R_n/Ω	ϕR_n	ASD	LRFD
82	73.6	110	10.4	15.7	107	161	117	175	178	267	146	219
74	58.8	88.2	7.89	11.8	84.4	127	100	151	152	228	128	191
68	49.9	74.8	6.81	10.2	72.0	108	90.2	136	135	204	117	175
61	40.5	60.7	5.67	8.51	58.9	88.4	79.2	119	116	175	104	156
53	40.3	60.5	5.31	7.97	57.6	86.4	78.6	118	114	171	103	155
48	33.6	50.5	4.61	6.92	48.6	73.0	70.3	106	96.1	144	93.8	141
43	27.0	40.4	3.78	5.66	39.3	58.8	61.6	92.4	77.4	116	83.3	125
38	27.0	40.6	3.95	5.93	39.8	59.9	57.1	85.9	78.8	118	87.4	131
34	22.3	33.4	3.51	5.26	33.7	50.5	51.2	77.0	66.5	99.8	79.7	120
30	18.5	27.8	3.56	5.34	30.1	45.2	47.0	70.4	59.4	88.8	74.7	112
26	18.2	27.3	2.73	4.10	27.1	40.6	45.0	67.7	53.5	80.2	70.9	106
22	13.6	20.4	2.55	3.82	21.9	32.8	39.0	58.5	43.2	64.8	63.2	94.8
336	888	1330	108	163	–	–	–	–	1240	1860	597	896
305	744	1120	93.6	140	–	–	–	–	1070	1610	530	796
279	646	970	88.1	132	557	836	557	836	948	1420	485	728
252	537	806	75.4	113	482	724	482	724	813	1220	430	645
230	455	683	65.6	98.4	425	638	425	638	711	1070	387	580
210	384	576	56.6	84.9	374	560	374	560	620	928	347	521
190	313	470	46.0	69.0	321	481	321	481	527	790	305	457
170	256	383	38.9	58.4	277	415	277	415	450	674	269	404
152	209	313	33.0	49.6	239	358	239	358	384	575	239	358
136	170	255	28.3	42.5	207	310	207	310	329	492	212	318
120	136	204	23.8	35.7	178	266	178	266	279	417	186	279
106	103	155	17.1	25.7	147	220	147	220	227	341	157	236
96	84.3	126	14.0	21.0	128	192	128	192	197	295	140	210
87	72.0	108	13.0	19.4	114	171	116	174	177	265	129	194
79	59.7	89.6	11.0	16.5	95.4	143	103	155	155	233	116	175
72	49.9	74.8	9.33	14.0	80.2	120	91.9	138	137	206	105	158
65	40.9	61.4	7.79	11.7	66.2	99.4	81.3	122	120	181	94.5	142
58	38.1	57.2	5.76	8.64	56.8	85.3	76.2	114	111	167	87.8	132
53	33.6	50.3	5.70	8.56	52.1	78.1	71.2	107	102	153	83.2	125
50	39.5	59.3	6.25	9.38	59.8	89.8	75.1	113	110	166	90.2	135
45	32.3	48.4	5.22	7.83	49.3	73.8	66.4	99.6	96.3	144	80.8	121
40	25.3	37.9	4.02	6.03	38.4	57.5	56.9	85.5	75.0	112	70.4	106
35	26.0	39.1	4.00	6.00	39.0	58.6	53.0	79.5	73.5	110	75.0	113
30	19.3	28.9	3.12	4.68	29.4	44.1	44.2	66.4	57.6	86.4	64.2	96.3
26	14.8	22.3	2.52	3.79	23.0	34.6	37.9	57.0	45.1	67.7	56.2	84.3

– Indicates that 3.25 in. bearing length is insufficient for end beam reactions since $N < k$.
N = length of bearing.
x = location of concentrated force with respect to the member end.

F_y = 50 ksi

Table 9–4 (continued) Beam Bearing Constants

Shape	R_1*		R_2		R_3**		R_4**	
	kips		kips/in.		kips		kips/in.	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	R_1/Ω	ϕR_1	R_2/Ω	ϕR_2	R_3/Ω	ϕR_3	R_4/Ω	ϕR_4
W12×22	15.7	23.6	8.67	13.0	20.8	31.2	2.43	3.64
×19	12.7	19.1	7.83	11.8	16.2	24.3	2.20	3.30
×16	10.4	15.5	7.33	11.0	12.8	19.2	2.42	3.63
×14	8.75	13.1	6.67	10.0	10.2	15.3	2.16	3.24
W10×112	110	165	25.2	37.8	177	265	21.9	32.8
×100	91.8	138	22.7	34.0	143	214	18.3	27.4
×88	75.1	113	20.2	30.3	113	169	14.9	22.4
×77	60.5	90.8	17.7	26.5	86.7	130	11.7	17.5
×68	49.7	74.6	15.7	23.5	68.1	102	9.37	14.1
×60	41.3	61.9	14.0	21.0	54.1	81.1	7.70	11.6
×54	34.4	51.6	12.3	18.5	42.5	63.8	5.90	8.85
×49	30.0	45.1	11.3	17.0	35.7	53.6	5.07	7.61
W10×45	32.7	49.0	11.7	17.5	39.3	58.9	4.95	7.42
×39	27.0	40.6	10.5	15.8	31.0	46.5	4.30	6.44
×33	22.6	33.9	9.67	14.5	24.8	37.2	4.16	6.24
W10×30	20.3	30.4	10.0	15.0	28.3	42.4	3.65	5.48
×26	16.0	24.1	8.67	13.0	21.2	31.8	2.79	4.19
×22	13.2	19.8	8.00	12.0	17.0	25.5	2.73	4.09
W10×19	14.5	21.7	8.33	12.5	18.9	28.4	2.79	4.19
×17	12.6	18.9	8.00	12.0	16.3	24.4	2.99	4.49
×15	10.9	16.4	7.67	11.5	13.8	20.7	3.26	4.88
×12	8.07	12.1	6.33	9.50	9.14	13.7	2.39	3.59
W8×67	63.1	94.7	19.0	28.5	100	150	15.9	23.9
×58	51.2	76.8	17.0	25.5	78.9	118	13.5	20.3
×48	36.0	54.0	13.3	20.0	50.4	75.6	7.94	11.9
×40	28.6	42.9	12.0	18.0	38.9	58.4	7.30	10.9
×35	23.0	34.4	10.3	15.5	29.2	43.9	5.35	8.03
×31	19.7	29.5	9.50	14.3	24.2	36.3	4.81	7.21
W8×28	20.4	30.6	9.50	14.3	25.0	37.5	4.46	6.69
×24	16.2	24.3	8.17	12.3	18.5	27.7	3.35	5.02
W8×21	14.6	21.9	8.33	12.5	19.0	28.6	3.41	5.11
×18	12.1	18.1	7.67	11.5	15.3	22.9	3.27	4.91
W8×15	12.6	18.8	8.17	12.3	16.4	24.6	4.16	6.24
×13	10.6	16.0	7.67	11.5	13.4	20.1	4.31	6.47
×10	7.15	10.7	5.67	8.50	7.64	11.5	2.19	3.29

* When compressive force is applied at a distance greater than *d* from the beam end, this value may be multiplied by two.

** When compressive force is applied at a distance greater than *d*/2 from the beam end, this value may be multiplied by two.

Table 9–4 (continued)
Beam Bearing Constants

F_y = 50 ksi

Nominal Wt.	R_5		R_6		(N = 3¼) $x < d/2$		(N = 3¼) $d/2 \le x \le d$		(N = 3¼) $x > d$		V_{nx}/Ω_v	$\phi_v V_{nx}$
	kips		kips/in.		kips		kips		kips			
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	kips	kips
lb/ft	R_5/Ω	ϕR_5	R_6/Ω	ϕR_6	R_n/Ω	ϕR_n	R_n/Ω	ϕR_n	R_n/Ω	ϕR_n	ASD	LRFD
22	18.8	28.2	3.24	4.85	29.3	44.0	43.9	65.9	57.4	86.1	64.0	96.0
19	14.4	21.7	2.94	4.41	24.0	36.0	38.1	57.5	46.7	70.1	57.2	85.7
16	10.9	16.3	3.23	4.84	21.4	32.0	34.2	51.3	41.3	62.0	52.8	79.1
14	8.51	12.8	2.88	4.31	17.9	26.8	30.4	45.6	34.4	51.7	42.8	64.3
112	160	240	29.2	43.8	192	288	192	288	302	453	172	257
100	129	194	24.4	36.5	166	249	166	249	257	387	151	226
88	102	153	19.9	29.8	141	211	141	211	216	324	131	197
77	78.4	118	15.6	23.3	118	177	118	177	179	268	112	169
68	61.6	92.4	12.5	18.7	101	151	101	151	150	226	97.8	147
60	48.8	73.2	10.3	15.4	82.3	123	86.8	130	128	192	85.8	129
54	38.5	57.8	7.86	11.8	64.0	96.1	74.4	112	109	163	74.7	112
49	32.3	48.5	6.76	10.1	54.3	81.3	66.7	100	96.7	145	68.0	102
45	35.9	53.9	6.60	9.89	57.4	86.0	70.7	106	103	155	70.7	106
39	28.2	42.2	5.73	8.59	46.8	70.1	61.1	92.0	88.1	133	62.5	93.7
33	22.1	33.2	5.55	8.33	40.1	60.3	54.0	81.0	76.6	115	56.4	84.7
30	25.7	38.6	4.87	7.31	41.5	62.4	52.8	79.2	73.1	110	62.8	94.2
26	19.3	28.9	3.73	5.59	31.4	47.1	44.2	66.4	60.2	90.4	53.7	80.6
22	15.1	22.7	3.64	5.46	26.9	40.4	39.2	58.8	51.7	77.6	48.8	73.2
19	17.0	25.5	3.72	5.58	29.1	43.6	41.6	62.3	55.9	84.0	51.2	76.8
17	14.2	21.4	3.99	5.99	27.2	40.9	38.6	57.9	51.2	76.8	48.5	72.8
15	11.6	17.4	4.34	6.51	25.7	38.6	35.8	53.8	46.7	70.2	46.0	69.0
12	7.57	11.4	3.19	4.78	17.9	26.9	28.6	43.0	33.8	50.7	37.5	56.3
67	90.7	136	21.2	31.8	125	187	125	187	188	282	103	154
58	71.1	107	18.0	27.0	106	160	106	160	158	236	89.3	134
48	45.9	68.9	10.6	15.9	79.2	119	79.2	119	115	173	68.0	102
40	34.9	52.4	9.73	14.6	66.5	99.9	67.6	101	96.2	144	59.4	89.1
35	26.3	39.5	7.14	10.7	49.5	74.3	56.5	84.8	79.5	119	50.3	75.5
31	21.6	32.4	6.41	9.61	42.4	63.6	50.6	76.0	70.3	105	45.6	68.4
28	22.6	33.9	5.95	8.93	41.9	62.9	51.3	77.1	71.7	108	45.9	68.9
24	16.7	25.1	4.47	6.70	31.2	46.9	42.8	64.3	58.8	88.0	38.9	58.3
21	17.2	25.7	4.54	6.82	32.0	47.9	41.7	62.5	56.3	84.4	41.4	62.1
18	13.5	20.2	4.36	6.55	27.7	41.5	37.0	55.5	49.1	73.6	37.4	56.2
15	14.1	21.2	5.55	8.32	32.1	48.2	39.2	58.8	51.8	77.6	39.7	59.6
13	11.1	16.7	5.75	8.63	29.8	44.7	35.5	53.4	46.1	69.4	36.8	55.1
10	6.49	9.73	2.93	4.39	16.0	24.0	25.6	38.3	29.5	44.4	26.8	40.2

N = length of bearing.
x = location of concentrated force with respect to the member end.

PART 10

DESIGN OF SIMPLE SHEAR CONNECTIONS

SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of simple shear connections. For the design of flexible moment connections, see Part 11. For the design of fully restrained (FR) moment connections, see Part 12.

FORCE TRANSFER

The required strength (end reaction), R_u or R_a, is determined by analysis as indicated in AISC *Specification* Section B3. Per AISC *Specification* Section J1.2, the ends of members with simple shear connections are normally assumed to be free to rotate under load. While simple shear connections do actually possess some rotational restraint (see curve A in Figure 10–1), this small amount can be neglected and the connection idealized as completely flexible. The simple shear connections shown in this Manual are suitable to accommodate the end rotations required per AISC *Specification* Section J1.2.

Support rotation is acceptably limited for most framing details involving simple shear connections without explicit consideration. The case of a bare spandrel girder supporting infill beams, however, may require consideration to verify that an acceptable level of support rotational stiffness is present. Sumner (2003) showed that a nominal interconnection between the top flange of the girder and the top flange of the framing beam is sufficient to limit support rotation.

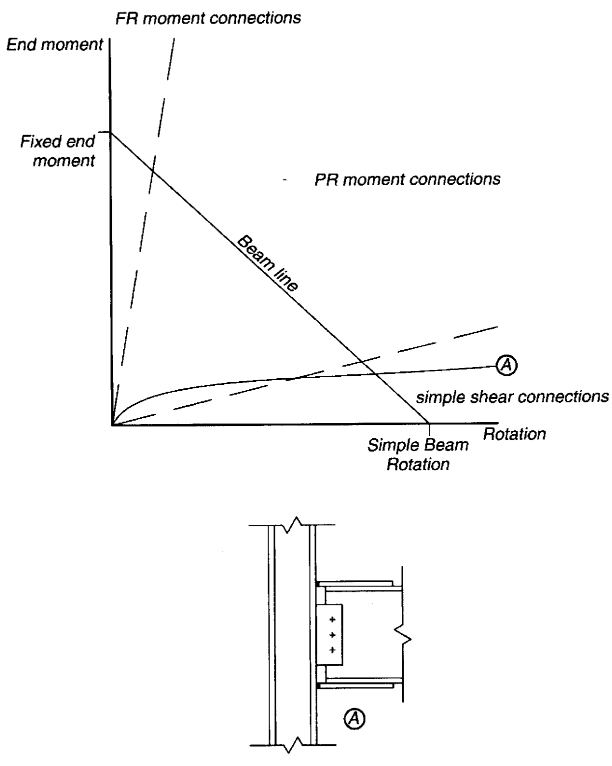


Figure 10–1. Illustration of typical moment rotation curve for simple shear connection.

COMPARING CONNECTION ALTERNATIVES

Two-Sided Connections

Two-sided connections, such as double-angle and shear end-plate connections, offer the following advantages:

1. suitability for use when the end reaction is large;
2. compact connections (usually, the entire connection is contained within the flanges of the supported beam); and,
3. eccentricity perpendicular to the beam axis need not be considered for workable gages (see Table 1–7).

Note that two-sided connections may require additional consideration for erectability, as discussed in "Constructability Considerations" below.

Seated Connections

Unstiffened and stiffened seated connections offer the following advantages:

1. seats can be shop attached to the support, simplifying erection;
2. ample erection clearance is provided;
3. excellent safety during erection since double connections often can be eliminated; and,
4. the bay length of the structure is easily maintained (seated connections may be preferable when maintaining bay length is a concern for repetitive bays of framing).

Note that seated connections can cause erection interference when floors are close, beams are deep, or seats protrude excessively from the column face. The practice of leaning or tilting the columns to erect a column-web connection is difficult, unsafe and should always be avoided.

One-Sided Connections

One-sided connections such as single-plate, single-angle and tee connections offer the following advantages:

1. shop attachment of connection elements to the support, simplifying shop fabrication and erection;
2. reduced material and shop labor requirements;
3. ample erection clearance is provided; and,
4. excellent safety during erection since double connections often can be eliminated.

CONSTRUCTABILITY CONSIDERATIONS

Double Connections

A double connection occurs in field-bolted construction when beams or girders frame opposite each other. Double connections are of concern to OSHA when they occur in the web of a column (see Figure 10–2) or the web of a beam that frames continuously over the top of a column[1] and all field bolts take the same open holes. A positive connection must be

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

made and maintained for the first member to be erected while the second member to be erected is brought into its final position. Conditions requiring the connector to hang one beam temporarily on a partially inserted bolt or drift pin are not allowed by OSHA.

Framing details can be configured using staggered angles or other similar details to provide a means to make a positive connection for the first member while the second member is brought into its final position. Alternatively, a temporary erection seat, as shown in Figure 10–2, can be provided. The erection seat, usually an angle, is sized and attached to the column web to support the dead weight of the member, unless additional loading is indicated in the contract documents. It is located to clear the bottom flange of the supported member by approximately 3/8 in. to accommodate mill, fabrication, and erection tolerances.

The sequence of erection is most important in determining the need for erection seats. If the erection sequence is known, the erection seat is provided on the side needing the support. If the erection sequence is not known, a seat can be provided on both sides of the column web. Temporary erection seats may be reused at other locations after the connection(s) are made, but need not be removed unless they create an interference or removal is required in the contract documents.

See also the discussion under "Special Considerations for Simple Shear Connections."

Accessibility in Column Webs

Because of bolting and welding clearances, double-angle, shear end-plate, single-plate, single-angle, and tee shear connections may not be suitable for connections to the webs of W-shape and similar columns, particularly for W8 columns, unless gages are reduced. Such connections may be impossible for W6, W5, and W4 columns.

There is also an accessibility concern for entering and tightening the field bolts when the connection material is shop-attached to the supporting column web and contained within the column flanges.

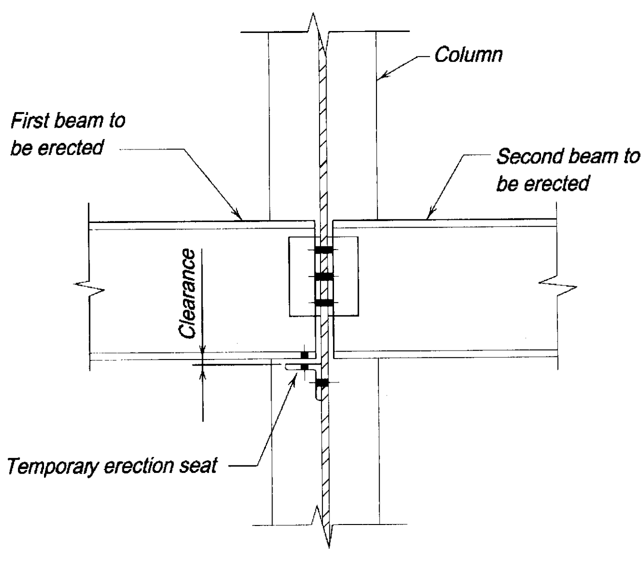


Figure 10–2. Erection seat.

Field-Welded Connections

In field-welded connections, temporary erection bolts are usually provided to support the member until final welding is performed. A minimum of 2 bolts (one bolt in bracing members) must be placed for erection safety per OSHA requirements. Additional erection bolts may be required for loads during erection, to assist in pulling the connection angles up tightly against the web of the supporting beam prior to welding or for other reasons. Temporary erection bolts may be reused at other locations after final welding, but need not be removed unless they create an interference or removal is required in the contract documents.

Riding the Fillet

The detailed dimensions of connection elements 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 10–3.

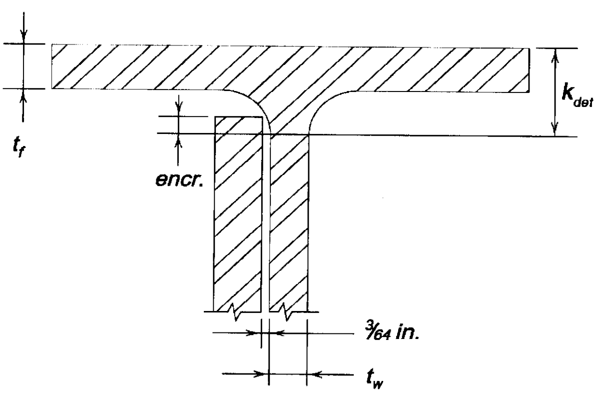
DOUBLE-ANGLE CONNECTIONS

A double-angle connection is made with two angles, one on each side of the web of the beam to be supported, as illustrated in Figure 10–4. These angles may be bolted or welded to the supported beam as well as to the supporting member.

When the angles are welded to the support, adequate flexibility must be provided in the connection. As illustrated in Figure 10–4c, line welds are placed along the toes of the angles with a return at the top per AISC Specification Section J2.2b. Note that welding across the entire top of the angles 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 simple shear connections.

Available Strength

The available strength of a double-angle connection is determined from the applicable limit states for bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a.



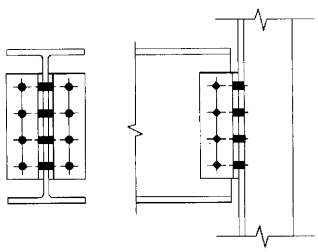
$k_{det} - t_f$	Encr.
in.	in.
5/16	1/8
3/8 to 1/2	3/16
9/16 to 13/16	1/4
7/8 to 1 1/4	5/16
1 5/16 and 1 3/8	3/8

Figure 10–3. Fillet encroachment (riding the fillet).

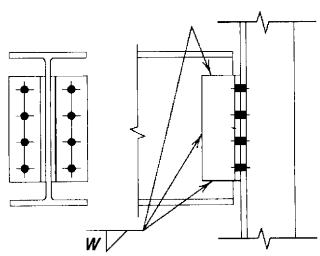
For the workable gages shown in Table 1–7 and standard or short-slotted holes, eccentricity in double-angle connections may be neglected, except in the case of a double vertical row of bolts through the web of the supported beam. Eccentricity should always be considered in the design of welds for double-angle connections.

Recommended Angle Length and Thickness

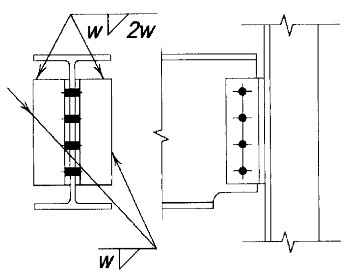
To provide for stability during erection, it is recommended that the minimum angle length be one-half the T-dimension of the beam to be supported. The maximum length of the connection angles must be compatible with the T-dimension of an uncoped beam and the



(a) All-bolted



(b) Bolted/welded, angles welded to supported beam



(c) Bolted/welded, angles welded to support

Figure 10–4. Double-angle connections.

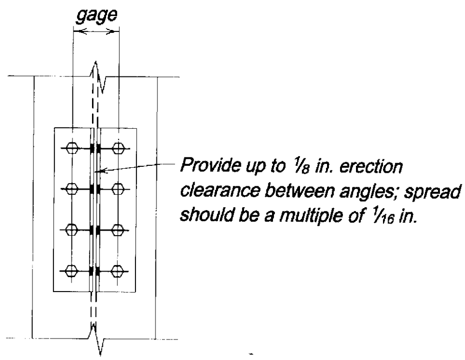
remaining web depth, exclusive of fillets, of a coped beam. Note that the angle may encroach upon the fillet(s), as given in Figure 10–3.

To provide for flexibility, the maximum angle thickness for use with workable gages should be limited to $^5/_8$ in. Alternatively, the shear-connection ductility checks illustrated in Part 9 can be used to justify other combinations of gage and angle thickness.

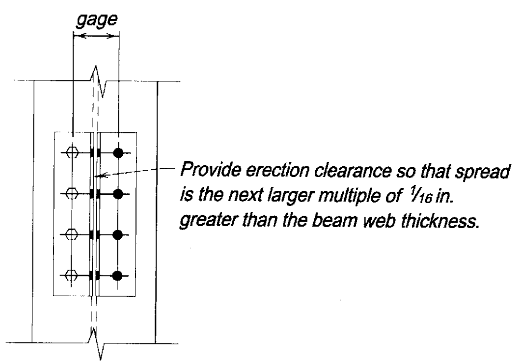
Shop and Field Practices

When framing to a girder web, both angles are usually shop-attached to the web of the supported beam. When framing to a column web, both angles should be shop-attached to the supported beam, when possible, and the associated constructability considerations should be addressed (see the preceding discussion under "Constructability Considerations").

When framing to a column flange, both angles can be shop-attached to the column flange or the supported beam. In the former case, this is a knifed connection, as illustrated in Figure 10–4c, which requires an erection clearance, as illustrated in Figure 10–5a, and that the bottom flange be coped away. Also, provision must be made for possible mill variation in



(a) Both angles shop attached to the column flange (beam knifed into place)



(b) One shop attached to the column flange, other shipped loose

Figure 10–5. Erection clearances for double-angle connections.

the depth of the columns, particularly in fairly long runs (i.e., six or more bays of framing). If both angles are shop-attached to the beam web, the beam length can be shortened to provide for mill overrun with shims furnished at the appropriate intervals to fill the resulting gaps or to provide for mill underrun. If both angles are shop-attached to the column flange, the erected beam is knifed into place and play in the open holes is normally sufficient to provide for the necessary adjustment. Alternatively, short-slotted holes can also be used.

When special requirements preclude the use of any of the foregoing practices, one angle could be shop-attached to the support and the other shipped loose. In this case, the spread between the outstanding legs should equal the decimal beam web thickness plus a clearance that will produce an opening to the next higher $^1/_{16}$-in. increment, as illustrated in Figure 10–5b. Alternatively, short-slotted holes in the support leg of the angle eliminate the need to provide for variations in web thickness. Note that the practice of shipping one angle loose is not desirable because it requires additional material handling as well as added erection costs and difficulty.

Table 10–1. All-Bolted Double-Angle Connections

Table 10–1 is a design aid for all-bolted double-angle connections. Available strengths are tabulated for supported and supporting member material with F_y = 50 ksi and F_u = 65 ksi and angle material with F_y = 36 ksi and F_u = 58 ksi. All values, including slip-critical bolt available strengths, are for comparison with the LRFD load combination for LRFD design and the ASD load combination for ASD design.

Tabulated bolt and angle available strengths consider the limit states of bolt shear, bolt bearing on the angles, shear yielding of the angles, shear rupture of the angles, and block shear rupture of the angles. Values are tabulated for 2 through 12 rows of $^3/_4$-in., $^7/_8$-in, and 1-in. diameter ASTM A325, F1852, and A490 bolts at 3-in. spacing. For calculation purposes, angle edge distances, L_{ev} and L_{eh}, are assumed to be $1^1/_4$ in.

Tabulated beam web available strengths, per in. of web thickness, consider the limit-state of bolt bearing on the beam web. For beams coped at the top flange only, the limit-state of block shear rupture is also considered. Additionally, for beams coped at both the top and bottom flanges, the tabulated values consider the limit-states of shear yielding and shear rupture of the beam web. Values are tabulated for beam web edge distances L_{ev} from $1^1/_4$ in. to 3 in. and for beam end distances, L_{eh}, of $1^1/_2$ in. and $1^3/_4$ in. For calculation purposes, these end distances have been reduced to $1^1/_4$ in. and $1^1/_2$ in., respectively, to account for possible underrun in beam length. For coped members, the limit states of flexural yielding and local buckling must be checked independently per Part 9. When required, web reinforcement of coped members is treated as in Part 9.

Tabulated supporting member available strengths, per in. of flange or web thickness, consider the limit-state of bolt bearing on the support. Note that resistance and safety factors are not noted in these tables, as they vary by limit state.

Table 10–2. Bolted/Welded Double-Angle Connections

Tables 10–2 is a design aid arranged to permit substitution of welds for bolts in connections designed with Tables 10–1. Electrode strength is assumed to be 70 ksi. Holes for erection bolts may be placed as required in angle legs that are to be field-welded.

Welds A may be used in place of bolts through the supported-beam web legs of the double angles or welds B may be used in place of bolts through the support legs of the double angles. Although it is permissible to use welds A and B from Table 10–2 in combination to obtain all-welded connections, it is recommended that such connections be selected from Table 10–3. This table will allow increased flexibility in the selection of angle lengths and connection strengths because Table 10–2 conforms to the bolt spacing and edge distance requirements for the all-bolted double-angle connections of Table 10–1.

Weld available strengths are tabulated for the limit-state of weld shear. Available strengths for welds A are determined by the instantaneous center of rotation method using Table 8–8 with $\theta = 0°$. Available strengths for welds B are determined by the elastic method. With the neutral axis assumed at one-sixth the depth of the angles measured downward and the tops of the angles in compression against each other through the beam web, the available strength, ϕR_n or R_n/Ω, of these welds is determined by

LRFD	ASD
$\phi R_n = 2\times\dfrac{1.392DL}{\sqrt{1+\dfrac{12.96e^2}{L^2}}}$	$\dfrac{R_n}{\Omega} = 2\times\dfrac{0.928DL}{\sqrt{1+\dfrac{12.96e^2}{L^2}}}$

where

D = number of sixteenths-of-an-inch in the weld size.
L = length of the connection angles, in.
e = width of the leg of the connection angle attached to the support, in.

The tabulated minimum thicknesses of the supported beam web for welds A and the support for welds B match the shear rupture strength of these elements with the strength of the weld metal. As derived in Part 9, the minimum supported beam web thickness for welds A (two lines of weld) is

$$t_{min} = \frac{6.19D}{F_u}$$

and the minimum supporting flange or web thickness welds B (one line of weld) is

$$t_{min} = \frac{3.09D}{F_u}$$

When welds B line up on opposite sides of the support, the minimum thickness is the sum of the thicknesses required for each weld. In either case, when less than the minimum material thickness is present, the tabulated weld available strength must be reduced by the ratio of the thickness provided to the minimum thickness.

When Table 10–2 is used, the minimum angle thickness is the weld size plus $^1/_{16}$ in., but not less than the angle thickness determined from Table 10–1. The angle length L must be as tabulated in Table 10–2. In general, 2L4×3$^1/_2$ will accommodate workable gages, with the 4-in. leg attached to the supporting member. The width of web legs in Case I may be optionally reduced from 3$^1/_2$ in. to 3 in. The width of outstanding legs in Case II may be optionally reduced from 4 in. to 3 in. for values of L from 5$^1/_2$ through 17$^1/_2$ in.

Table 10–3. All-Welded Double-Angle Connections

Table 10–3 is a design aid for all-welded double-angle connections. Electrode strength is assumed to be 70 ksi. Holes for erection bolts may be placed as required in angle legs that are to be field-welded.

Weld available strengths are tabulated for the limit-state of weld shear. Available strengths for welds A are determined by the instantaneous center of rotation method using Table 8–8 with $\theta = 0°$. Available strengths for welds B are determined by the elastic method as discussed previously for bolted/welded double-angle connections.

The tabulated minimum thicknesses of the supported beam web for welds A and the support for welds B match the shear rupture strength of these elements with the strength of the weld metal and are determined as discussed previously for Table 10–2. When welds B line up on opposite sides of the support, the minimum thickness is the sum of the thicknesses required for each weld. When less than the minimum material thickness is present, the tabulated weld available strength must be reduced by the ratio of the thickness provided to the minimum thickness.

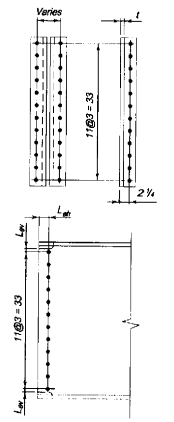
When Table 10–3 is used, the minimum angle thickness must be equal to the weld size plus $^1/_{16}$ in. The angle length, L, must be as tabulated in Table 10–3. 2L4×3 should be used for angle lengths equal to or greater than 18 in. 2L3×3 should be used otherwise.

Beam	F_y = 50 ksi, F_u = 65 ksi
Angle	F_y = 36 ksi, F_u = 58 ksi

Table 10-1
All-Bolted Double-Angle Connections
3/4-in. Bolts

12 Rows

W44



Bolt and Angle Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
A325/ F1852	N	—	197	295	246	369	254	382	254	382
	X	—	197	295	246	369	295	443	318	477
	SC Class A	STD	177	266	177	266	177	266	177	266
		OVS	128	192	128	192	128	192	128	192
		SSLT	151	226	151	226	151	226	151	226
	SC Class B	STD	197	295	246	369	253	380	253	380
		OVS	183	274	183	274	183	274	183	274
		SSLT	195	293	215	323	215	323	215	323
A490	N	—	197	295	246	369	295	443	318	477
	X	—	197	295	246	369	295	443	393	590
	SC Class A	STD	197	295	221	332	221	332	221	332
		OVS	160	240	160	240	160	240	160	240
		SSLT	188	282	188	282	188	282	188	282
	SC Class B	STD	197	295	246	369	295	443	316	475
		OVS	196	294	229	343	229	343	229	343
		SSLT	195	293	244	366	269	403	269	403

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1½		1¾		1½		1¾		1½		1¾	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1¼	498	747	506	759	468	702	476	714	495	743	503	755
	1⅜	500	751	509	763	470	706	479	718	497	746	506	758
	1½	503	754	511	767	473	709	481	722	500	750	508	762
	1⅝	505	758	514	770	475	713	483	725	502	753	510	766
	2	513	769	521	781	483	724	491	736	510	764	518	777
	3	532	798	540	810	502	753	510	765	529	794	537	806
Coped at Both Flanges	1¼	488	731	488	731	458	687	458	687	488	731	488	731
	1⅜	492	739	492	739	463	695	463	695	492	739	492	739
	1½	497	746	497	746	468	702	468	702	497	746	497	746
	1⅝	502	753	502	753	473	709	473	709	502	753	502	753
	2	513	769	517	775	483	724	488	731	510	764	517	775
	3	532	798	540	810	502	753	510	765	529	794	537	806
Uncoped		702	1050	702	1050	702	1050	702	1050	702	1050	702	1050

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	1400	2110

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

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	F_y = 50 ksi F_u = 65 ksi
Angle	F_y = 36 ksi F_u = 58 ksi

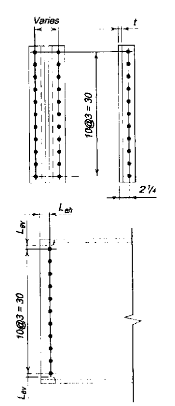
Table 10–1 (continued) All-Bolted Double-Angle Connections

3/4-in. Bolts

Bolt and Angle Available Strength, kips

11 Rows

W44, 40



ASTM Desig.	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
A325/ F1852	N	—	181	271	226	338	233	350	233	350
	X	—	181	271	226	338	271	406	292	437
	SC Class A	STD	162	244	162	244	162	244	162	244
		OVS	117	176	117	176	117	176	117	176
		SSLT	138	207	138	207	138	207	138	207
	SC Class B	STD	181	271	226	338	232	348	232	348
		OVS	168	251	168	251	168	251	168	251
		SSLT	179	269	197	296	197	296	197	296
A490	N	—	181	271	226	338	271	406	292	437
	X	—	181	271	226	338	271	406	361	542
	SC Class A	STD	181	271	203	305	203	305	203	305
		OVS	147	220	147	220	147	220	147	220
		SSLT	173	259	173	259	173	259	173	259
	SC Class B	STD	181	271	226	338	271	406	290	435
		OVS	80	269	210	314	210	314	210	314
		SSLT	179	269	224	336	247	370	247	370

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	457	685	465	697	429	644	437	656	454	680	462	693
	1 3/8	459	689	467	701	431	647	440	659	456	684	464	696
	1 1/2	462	692	470	704	434	651	442	663	458	688	467	700
	1 5/8	464	696	472	708	436	654	444	667	461	691	469	704
	2	471	707	479	719	444	665	452	678	468	702	476	714
	3	491	736	499	748	463	695	471	707	488	732	496	744
Coped at Both Flanges	1 1/4	446	669	446	669	419	629	419	629	446	669	446	669
	1 3/8	451	676	451	676	424	636	424	636	451	676	451	676
	1 1/2	456	684	456	684	429	644	429	644	456	684	456	684
	1 5/8	461	691	461	691	434	651	434	651	461	691	461	691
	2	471	707	475	713	444	665	449	673	468	702	475	713
	3	491	736	499	748	463	695	471	707	488	732	496	744
Uncoped		644	965	644	965	644	965	644	965	644	965	644	965

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	1290	1930

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

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

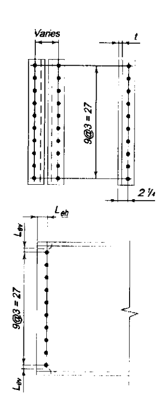
Beam	Angle
F_y = 50 ksi	F_y = 36 ksi
F_u = 65 ksi	F_u = 58 ksi

Table 10-1 (continued)
All-Bolted Double-Angle Connections

¾-in. Bolts

10 Rows

W44, 40, 36



Bolt and Angle Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
A325/ F1852	N	—	164	246	205	308	212	318	212	318
	X	—	164	246	205	308	246	370	265	398
	SC Class A	STD	148	221	148	221	148	221	148	221
		OVS	107	160	107	160	107	160	107	160
		SSLT	126	188	126	188	126	188	126	188
	SC Class B	STD	164	246	205	308	211	316	211	316
		OVS	152	229	152	229	152	229	152	229
		SSLT	163	244	179	269	179	269	179	269
A490	N	—	164	246	205	308	246	370	265	398
	X	—	164	246	205	308	246	370	329	493
	SC Class A	STD	164	246	185	277	185	277	185	277
		OVS	133	200	133	200	133	200	133	200
		SSLT	157	235	157	235	157	235	157	235
	SC Class B	STD	164	246	205	308	246	370	264	396
		OVS	163	245	190	286	190	286	190	286
		SSLT	163	244	204	306	224	336	224	336

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
	L_{eh}*	1½		1¾		1½		1¾		1½		1¾	
L_{ev}, in.		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1¼	415	623	423	635	390	585	398	597	412	618	420	630
	1⅜	418	626	426	639	392	589	401	601	415	622	423	634
	1½	420	630	428	642	395	592	403	605	417	626	425	638
	1⅝	423	634	431	646	397	596	405	608	419	629	428	641
	2	430	645	438	657	405	607	413	619	427	640	435	652
	3	449	674	457	686	424	636	432	648	446	669	454	682
Coped at Both Flanges	1¼	405	607	405	607	380	570	380	570	405	607	405	607
	1⅜	410	614	410	614	385	578	385	578	410	614	410	614
	1½	414	622	414	622	390	585	390	585	414	622	414	622
	1⅝	419	629	419	629	395	592	395	592	419	629	419	629
	2	430	645	434	651	405	607	410	614	427	640	434	651
	3	449	674	457	686	424	636	432	648	446	669	454	682
Uncoped		585	878	585	878	585	878	585	878	585	878	585	878

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	1170	1760

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

* Tabulated values include ¼-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

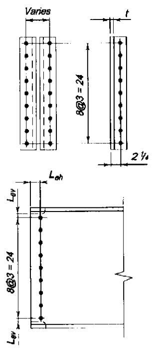
Table 10–1 (continued)
All-Bolted Double-Angle Connections

3/4-in. Bolts

Beam	F_y = 50 ksi, F_u = 65 ksi
Angle	F_y = 36 ksi, F_u = 58 ksi

9 Rows

W44, 40, 36, 33



Bolt and Angle Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
A325/ F1852	N	—	148	222	185	278	191	286	191	286
	X	—	148	222	185	278	222	333	239	358
	SC Class A	STD	133	199	133	199	133	199	133	199
		OVS	96	144	96	144	96	144	96	144
		SSLT	113	169	113	169	113	169	113	169
	SC Class B	STD	148	222	185	278	190	285	190	285
		OVS	137	206	137	206	137	206	137	206
		SSLT	147	220	161	242	161	242	161	242
A490	N	—	148	222	185	278	222	333	239	358
	X	—	148	222	185	278	222	333	296	444
	SC Class A	STD	148	222	166	249	166	249	166	249
		OVS	120	180	120	180	120	180	120	180
		SSLT	141	212	141	212	141	212	141	212
	SC Class B	STD	148	222	185	278	222	333	237	356
		OVS	147	221	171	257	171	257	171	257
		SSLT	147	220	183	275	202	303	202	303

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	374	561	382	573	351	527	359	539	371	556	379	568
	1 3/8	376	564	384	576	353	530	362	542	373	560	381	572
	1 1/2	379	568	387	580	356	534	364	546	376	563	384	576
	1 5/8	381	572	389	584	358	537	366	550	378	567	386	579
	2	388	583	397	595	366	548	374	561	385	578	393	590
	3	408	612	416	624	385	578	393	590	405	607	413	619
Coped at Both Flanges	1 1/4	363	545	363	545	341	512	341	512	363	545	363	545
	1 3/8	368	552	368	552	346	519	346	519	368	552	368	552
	1 1/2	373	559	373	559	351	527	351	527	373	559	373	559
	1 5/8	378	567	378	567	356	534	356	534	378	567	378	567
	2	388	583	392	589	366	548	371	556	385	578	392	589
	3	408	612	416	624	385	578	393	590	405	607	413	619
Uncoped		526	790	526	790	526	790	526	790	526	790	526	790

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	1050	1580

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

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	F_y = 50 ksi, F_u = 65 ksi
Angle	F_y = 36 ksi, F_u = 58 ksi

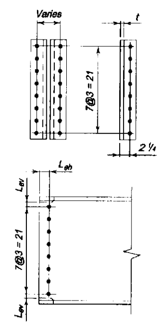
Table 10-1 (continued)
All-Bolted Double-Angle Connections

3/4-in. Bolts

Bolt and Angle Available Strength, kips

8 Rows

W44, 40, 36, 33, 30



ASTM Desig.	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
A325/ F1852	N	—	132	198	165	247	170	254	170	254
	X	—	132	198	165	247	198	297	212	318
	SC Class A	STD	118	177	118	177	118	177	118	177
		OVS	85.3	128	85.3	128	85.3	128	85.3	128
		SSLT	100	151	100	151	100	151	100	151
	SC Class B	STD	132	198	165	247	169	253	169	253
		OVS	122	183	122	183	122	183	122	183
		SSLT	131	196	143	215	143	215	143	215
A490	N	—	132	198	165	247	198	297	212	318
	X	—	132	198	165	247	198	297	264	396
	SC Class A	STD	132	198	148	221	148	221	148	221
		OVS	107	160	107	160	107	160	107	160
		SSLT	126	188	126	188	126	188	126	188
	SC Class B	STD	132	198	165	247	198	297	211	316
		OVS	131	197	152	229	152	229	152	229
		SSLT	131	196	163	245	179	269	179	269

Angle Thickness columns: 1/4, 5/16, 3/8, 1/2.

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type	L_{ev}, in.	STD L_{eh}* 1 1/2 ASD	STD 1 1/2 LRFD	STD 1 3/4 ASD	STD 1 3/4 LRFD	OVS 1 1/2 ASD	OVS 1 1/2 LRFD	OVS 1 3/4 ASD	OVS 1 3/4 LRFD	SSLT 1 1/2 ASD	SSLT 1 1/2 LRFD	SSLT 1 3/4 ASD	SSLT 1 3/4 LRFD
Coped at Top Flange Only	1 1/4	332	498	340	511	312	468	320	480	329	494	337	506
	1 3/8	335	502	343	514	314	472	323	484	332	498	340	510
	1 1/2	337	506	345	518	317	475	325	488	334	501	342	513
	1 5/8	340	509	348	522	319	479	327	491	337	505	345	517
	2	347	520	355	533	327	490	335	502	344	516	352	528
	3	366	550	375	562	346	519	354	531	363	545	372	557
Coped at Both Flanges	1 1/4	322	483	322	483	302	453	302	453	322	483	322	483
	1 3/8	327	490	327	490	307	461	307	461	327	490	327	490
	1 1/2	332	497	332	497	312	468	312	468	332	497	332	497
	1 5/8	336	505	336	505	317	475	317	475	336	505	336	505
	2	347	520	351	527	327	490	332	497	344	516	351	527
	3	366	550	375	562	346	519	354	531	363	545	372	557
Uncoped		468	702	468	702	468	702	468	702	468	702	468	702

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	936	1400

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

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

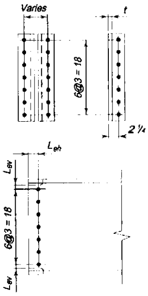
Beam F_y = 50 ksi, F_u = 65 ksi
Angle F_y = 36 ksi, F_u = 58 ksi

Table 10–1 (continued) All-Bolted Double-Angle Connections

3/4-in. Bolts

Bolt and Angle Available Strength, kips

7 Rows
W44, 40, 36, 33, 30, 27, 24



ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
A325/ F1852	N	—	116	174	145	217	148	223	148	223
	X	—	116	174	145	217	174	260	186	278
	SC Class A	STD	103	155	103	155	103	155	103	155
		OVS	74.7	112	74.7	112	74.7	112	74.7	112
		SSLT	87.9	132	87.9	132	87.9	132	87.9	132
	SC Class B	STD	116	174	145	217	148	221	148	221
		OVS	107	160	107	160	107	160	107	160
		SSLT	114	172	126	188	126	188	126	188
A490	N	—	116	174	145	217	174	260	186	278
	X	—	116	174	145	217	174	260	231	347
	SC Class A	STD	116	174	129	194	129	194	129	194
		OVS	93.3	140	93.3	140	93.3	140	93.3	140
		SSLT	110	165	110	165	110	165	110	165
	SC Class B	STD	116	174	145	217	174	260	185	277
		OVS	115	172	133	200	133	200	133	200
		SSLT	114	172	143	214	157	235	157	235

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	291	436	299	449	273	410	281	422	288	432	296	444
	1 3/8	293	440	301	452	275	413	284	425	290	435	298	448
	1 1/2	296	444	304	456	278	417	286	429	293	439	301	451
	1 5/8	298	447	306	459	280	420	288	433	295	443	303	455
	2	306	458	314	470	288	431	296	444	302	454	311	466
	3	325	488	333	500	307	461	315	473	322	483	330	495
Coped at Both Flanges	1 1/4	280	420	280	420	263	395	263	395	280	420	280	420
	1 3/8	285	428	285	428	268	402	268	402	285	428	285	428
	1 1/2	290	435	290	435	273	410	273	410	290	435	290	435
	1 5/8	295	442	295	442	278	417	278	417	295	442	295	442
	2	306	458	310	464	288	431	293	439	302	454	310	464
	3	325	488	333	500	307	461	315	473	322	483	330	495
Uncoped		410	614	410	614	410	614	410	614	410	614	410	614

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	819	1230

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

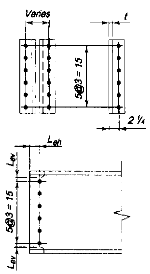
* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	F_y = 50 ksi, F_u = 65 ksi
Angle	F_y = 36 ksi, F_u = 58 ksi

Table 10-1 (continued)
All-Bolted Double-Angle Connections

3/4-in. Bolts

Bolt and Angle Available Strength, kips

6 Rows	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4		5/16		3/8		1/2	
W40, 36, 33, 30, 27, 24, 21				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	A325/ F1852	N	—	99.5	149	124	187	127	191	127	191
		X	—	99.5	149	124	187	149	224	159	239
		SC Class A	STD	88.6	133	88.6	133	88.6	133	88.6	133
			OVS	64	96	64	96	64	96	64	96
			SSLT	75.3	113	75.3	113	75.3	113	75.3	113
		SC Class B	STD	99.5	149	124	187	127	190	127	190
			OVS	91.4	137	91.4	137	91.4	137	91.4	137
			SSLT	98.2	147	108	161	108	161	108	161
	A490	N	—	99.5	149	124	187	149	224	159	239
		X	—	99.5	149	124	187	149	224	199	298
		SC Class A	STD	99.5	149	111	166	111	166	111	166
			OVS	80	120	80	120	80	120	80	120
			SSLT	94.1	141	94.1	141	94.1	141	94.1	141
		SC Class B	STD	99.5	149	124	187	149	224	158	237
			OVS	98.6	148	114	171	114	171	114	171
			SSLT	98.2	147	123	184	134	202	134	202

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	249	374	258	386	234	351	242	363	246	370	255	382
	1 3/8	252	378	260	390	236	355	245	367	249	373	257	385
	1 1/2	254	381	262	394	239	358	247	371	251	377	259	389
	1 5/8	257	385	265	397	241	362	249	374	254	381	262	393
	2	264	396	272	408	249	373	257	385	261	392	269	404
	3	284	425	292	438	268	402	276	414	281	421	289	433
Coped at Both Flanges	1 1/4	239	358	239	358	224	336	224	336	239	358	239	358
	1 3/8	244	366	244	366	229	344	229	344	244	366	244	366
	1 1/2	249	373	249	373	234	351	234	351	249	373	249	373
	1 5/8	254	380	254	380	239	358	239	358	254	380	254	380
	2	264	396	268	402	249	373	254	380	261	392	268	402
	3	284	425	292	438	268	402	276	414	281	421	289	433
Uncoped		351	526	351	526	351	526	351	526	351	526	351	526

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	702	1050

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

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	F_y = 50 ksi F_u = 65 ksi
Angle	F_y = 36 ksi F_u = 58 ksi

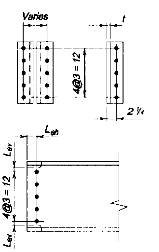
Table 10–1 (continued)
All-Bolted Double-Angle Connections

3/4-in. Bolts

Bolt and Angle Available Strength, kips

5 Rows

W30, 27, 24, 21, 18



ASTM Desig.	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
A325/ F1852	N	—	83.3	125	104	156	106	159	106	159
	X	—	83.3	125	104	156	125	187	133	199
	SC Class A	STD	73.8	111	73.8	111	73.8	111	73.8	111
		OVS	53.3	80.0	53.3	80.0	53.3	80.0	53.3	80.0
		SSLT	62.8	94.1	62.8	94.1	62.8	94.1	62.8	94.1
	SC Class B	STD	83.3	125	104	156	105	158	105	158
		OVS	76.2	114	76.2	114	76.2	114	76.2	114
		SSLT	82.0	123	89.6	134	89.6	134	89.6	134
A490	N	—	83.3	125	104	156	125	187	133	199
	X	—	83.3	125	104	156	125	187	166	249
	SC Class A	STD	83.3	125	92.3	138	92.3	138	92.3	138
		OVS	66.7	100	66.7	100	66.7	100	66.7	100
		SSLT	78.4	118	78.4	118	78.4	118	78.4	118
	SC Class B	STD	83.3	125	104	156	125	187	132	198
		OVS	82.4	124	95.2	143	95.2	143	95.2	143
		SSLT	82.0	123	102	154	112	168	112	168

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type	L_{ev}, in.	STD, L_{eh}* 1 1/2 ASD	STD 1 1/2 LRFD	STD 1 3/4 ASD	STD 1 3/4 LRFD	OVS 1 1/2 ASD	OVS 1 1/2 LRFD	OVS 1 3/4 ASD	OVS 1 3/4 LRFD	SSLT 1 1/2 ASD	SSLT 1 1/2 LRFD	SSLT 1 3/4 ASD	SSLT 1 3/4 LRFD
Coped at Top Flange Only	1 1/4	208	312	216	324	195	293	203	305	205	307	213	320
	1 3/8	210	316	219	328	197	296	206	308	207	311	216	323
	1 1/2	213	319	221	332	200	300	208	312	210	315	218	327
	1 5/8	215	323	223	335	202	303	210	316	212	318	220	331
	2	223	334	231	346	210	314	218	327	220	329	228	342
	3	242	363	250	375	229	344	237	356	239	359	247	371
Coped at Both Flanges	1 1/4	197	296	197	296	185	278	185	278	197	296	197	296
	1 3/8	202	303	202	303	190	285	190	285	202	303	202	303
	1 1/2	207	311	207	311	195	293	195	293	207	311	207	311
	1 5/8	212	318	212	318	200	300	200	300	212	318	212	318
	2	223	334	227	340	210	314	215	322	220	329	227	340
	3	242	363	250	375	229	344	237	356	239	359	247	371
Uncoped		293	439	293	439	293	439	293	439	293	439	293	439

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	585	878

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

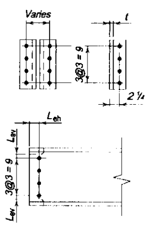
* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	F_y = 50 ksi F_u = 65 ksi
Angle	F_y = 36 ksi F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

3/4-in. Bolts

Bolt and Angle Available Strength, kips

4 Rows W24, 21, 18, 16	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	A325/F1852	N	—	67.1	101	83.9	126	84.8	127	84.8	127
		X	—	67.1	101	83.9	126	101	151	106	159
		SC Class A	STD	59.1	88.6	59.1	88.6	59.1	88.6	59.1	88.6
			OVS	42.7	64.0	42.7	64.0	42.7	64.0	42.7	64.0
			SSLT	50.2	75.3	50.2	75.3	50.2	75.3	50.2	75.3
		SC Class B	STD	67.1	101	83.9	126	84.4	127	84.4	127
			OVS	61.0	91.4	61.0	91.4	61.0	91.4	61.0	91.4
			SSLT	65.8	98.7	71.7	108	71.7	108	71.7	108
	A490	N	—	67.1	101	83.9	126	101	151	106	159
		X	—	67.1	101	83.9	126	101	151	133	199
		SC Class A	STD	67.1	101	73.8	111	73.8	111	73.8	111
			OVS	53.3	80.0	53.3	80.0	53.3	80.0	53.3	80.0
			SSLT	62.8	94.1	62.8	94.1	62.8	94.1	62.8	94.1
		SC Class B	STD	67.1	101	83.9	126	101	151	105	158
			OVS	65.3	97.9	76.2	114	76.2	114	76.2	114
			SSLT	65.8	98.7	82.2	123	89.6	134	89.6	134

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	167	250	175	262	156	234	164	246	164	245	172	257
	1 3/8	169	254	177	266	158	238	167	250	166	249	174	261
	1 1/2	171	257	180	269	161	241	169	254	168	253	177	265
	1 5/8	174	261	182	273	163	245	171	257	171	256	179	268
	2	181	272	189	284	171	256	179	268	178	267	186	279
	3	201	301	209	313	190	285	198	297	198	296	206	309
Coped at Both Flanges	1 1/4	156	234	156	234	146	219	146	219	156	234	156	234
	1 3/8	161	241	161	241	151	227	151	227	161	241	161	241
	1 1/2	166	249	166	249	156	234	156	234	166	249	166	249
	1 5/8	171	256	171	256	161	241	161	241	171	256	171	256
	2	181	272	185	278	171	256	176	263	178	267	185	278
	3	201	301	209	313	190	285	198	297	198	296	206	309
Uncoped		234	351	234	351	234	351	234	351	234	351	234	351

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	468	702

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

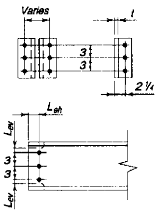
* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	F_y = 50 ksi F_u = 65 ksi
Angle	F_y = 36 ksi F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

3/4-in. Bolts

Bolt and Angle Available Strength, kips



3 Rows W18, 16, 14, 12, 10+ +Ltd. to W10x12, 15, 17, 19, 22, 26, 30	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	A325/ F1852	N	—	50.9	76.4	63.6	95.4	63.6	95.4	63.6	95.4
		X	—	50.9	76.4	63.7	95.5	76.4	115	79.5	119
		SC Class A	STD	44.3	66.4	44.3	66.4	44.3	66.4	44.3	66.4
			OVS	32.0	48.0	32.0	48.0	32.0	48.0	32.0	48.0
			SSLT	37.7	56.5	37.7	56.5	37.7	56.5	37.7	56.5
		SC Class B	STD	50.9	76.4	63.3	94.9	63.3	94.9	63.3	94.9
			OVS	45.7	68.6	45.7	68.6	45.7	68.6	45.7	68.6
			SSLT	49.6	74.4	53.8	80.7	53.8	80.7	53.8	80.7
	A490	N	—	50.9	76.4	63.7	95.5	76.4	115	79.5	119
		X	—	50.9	76.4	63.7	95.5	76.4	115	99.4	149
		SC Class A	STD	50.9	76.4	55.4	83.1	55.4	83.1	55.4	83.1
			OVS	40.0	60.0	40.0	60.0	40.0	60.0	40.0	60.0
			SSLT	47.1	70.6	47.1	70.6	47.1	70.6	47.1	70.6
		SC Class B	STD	50.9	76.4	63.7	95.5	76.4	115	79.1	119
			OVS	47.9	71.8	57.1	85.7	57.1	85.7	57.1	85.7
			SSLT	49.6	74.4	62.0	92.9	67.2	101	67.2	101

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	125	188	133	200	117	176	125	188	122	183	130	195
	1 3/8	128	191	136	204	119	179	128	191	125	187	133	199
	1 1/2	130	195	138	207	122	183	130	195	127	190	135	203
	1 5/8	132	199	141	211	124	186	132	199	129	194	138	206
	2	140	210	148	222	132	197	140	210	137	205	145	217
	3	159	239	167	251	151	227	159	239	156	234	164	246
Coped at Both Flanges	1 1/4	115	172	115	172	107	161	107	161	115	172	115	172
	1 3/8	119	179	119	179	112	168	112	168	119	179	119	179
	1 1/2	124	186	124	186	117	176	117	176	124	186	124	186
	1 5/8	129	194	129	194	122	183	122	183	129	194	129	194
	2	140	210	144	216	132	197	137	205	137	205	144	216
	3	159	239	167	251	151	227	159	239	156	234	164	246
Uncoped		175	263	175	263	175	263	175	263	175	263	175	263

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	351	526

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

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	Angle
F_y = 50 ksi, F_u = 65 ksi	F_y = 36 ksi, F_u = 58 ksi

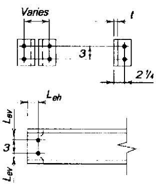
Table 10–1 (continued)
All-Bolted Double-Angle Connections

3/4-in. Bolts

Bolt and Angle Available Strength, kips

2 Rows

W12, 10, 8



ASTM Desig.	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
A325/ F1852	N	—	32.6	48.9	40.8	61.2	42.4	63.6	42.4	63.6
	X	—	32.6	48.9	40.8	61.2	48.9	73.4	53.0	79.5
	SC Class A	STD	29.5	44.3	29.5	44.3	29.5	44.3	29.5	44.3
		OVS	21.3	32.0	21.3	32.0	21.3	32.0	21.3	32.0
		SSLT	25.1	37.7	25.1	37.7	25.1	37.7	25.1	37.7
	SC Class B	STD	32.6	48.9	40.8	61.2	42.2	63.3	42.2	63.3
		OVS	30.5	45.7	30.5	45.7	30.5	45.7	30.5	45.7
		SSLT	32.6	48.9	35.9	53.8	35.9	53.8	35.9	53.8
A490	N	—	32.6	48.9	40.8	61.2	48.9	73.4	53.0	79.5
	X	—	32.6	48.9	40.8	61.2	48.9	73.4	65.3	97.9
	SC Class A	STD	32.6	48.9	36.9	55.4	36.9	55.4	36.9	55.4
		OVS	26.7	40.0	26.7	40.0	26.7	40.0	26.7	40.0
		SSLT	31.4	47.1	31.4	47.1	31.4	47.1	31.4	47.1
	SC Class B	STD	32.6	48.9	40.8	61.2	48.9	73.4	52.7	79.1
		OVS	30.5	45.7	38.1	57.1	38.1	57.1	38.1	57.1
		SSLT	32.6	48.9	40.8	61.2	44.8	67.2	44.8	67.2

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	83.7	126	91.4	137	78.0	117	86.1	129	80.6	121	88.8	133
	1 3/8	86.1	129	94.3	141	80.4	121	88.6	133	83.1	125	91.2	137
	1 1/2	88.6	133	96.7	145	82.9	124	91.0	137	85.5	128	93.6	140
	1 5/8	91.0	137	99.1	149	85.3	128	93.4	140	88.0	132	96.1	144
	2	98.3	147	106	160	92.6	139	101	151	95.3	143	103	155
	3	116	175	117	175	112	168	117	175	113	170	117	175
Coped at Both Flanges	1 1/4	73.1	110	73.1	110	68.3	102	68.3	102	73.1	110	73.1	110
	1 3/8	78.0	117	78.0	117	73.1	110	73.1	110	78.0	117	78.0	117
	1 1/2	82.9	124	82.9	124	78.0	117	78.0	117	82.9	124	82.9	124
	1 5/8	87.8	132	87.8	132	82.9	124	82.9	124	87.8	132	87.8	132
	2	98.3	147	102	154	92.6	139	97.5	146	95.3	143	102	154
	3	116	175	117	175	112	168	117	175	113	170	117	175
Uncoped		117	176	117	176	117	176	117	176	117	176	117	176

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	234	351

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

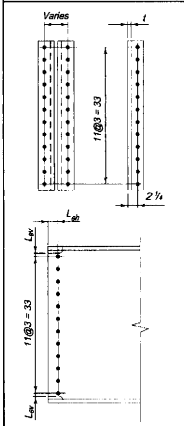
* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	Angle
F_y = 50 ksi, F_u = 65 ksi	F_y = 36 ksi, F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

7/8-in. Bolts

12 Rows — W44



Bolt and Angle Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
A325/ F1852	N	—	196	294	245	367	294	441	346	520
	X	—	196	294	245	367	294	441	392	587
	SC Class A	STD	196	294	245	367	247	370	247	370
		OVS	178	267	178	267	178	267	178	267
		SSLT	194	292	210	315	210	315	210	315
	SC Class B	STD	196	294	245	367	294	441	346	520
		OVS	191	287	239	359	255	382	255	382
		SSLT	194	292	243	365	292	438	300	450
A490	N	—	196	294	245	367	294	441	392	587
	X	—	196	294	245	367	294	441	392	587
	SC Class A	STD	196	294	245	367	294	441	310	465
		OVS	191	287	224	336	224	336	224	336
		SSLT	194	292	243	365	264	395	264	395
	SC Class B	STD	196	294	245	367	294	441	392	587
		OVS	191	287	239	359	287	431	320	480
		SSLT	194	292	243	365	292	438	377	565

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	468	702	476	714	438	657	446	669	465	697	473	710
	1 3/8	470	706	479	718	440	661	449	673	467	701	476	713
	1 1/2	473	709	481	722	443	664	451	676	470	705	478	717
	1 5/8	475	713	483	725	445	668	453	680	472	708	480	721
	2	483	724	491	736	453	679	461	691	480	719	488	732
	3	502	753	510	765	472	708	480	720	499	749	507	761
Coped at Both Flanges	1 1/4	458	687	458	687	429	644	429	644	458	687	458	687
	1 3/8	463	695	463	695	434	651	434	651	463	695	463	695
	1 1/2	468	702	468	702	439	658	439	658	468	702	468	702
	1 5/8	473	709	473	709	444	665	444	665	472	708	473	709
	2	483	724	488	731	453	679	458	687	480	719	488	731
	3	502	753	510	765	472	708	480	720	499	749	507	761
Uncoped		819	1230	819	1230	819	1230	819	1230	819	1230	819	1230

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	1640	2460

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

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	F_y = 50 ksi F_u = 65 ksi
Angle	F_y = 36 ksi F_u = 58 ksi

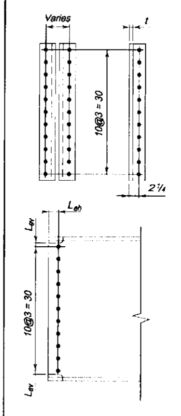
Table 10–1 (continued)
All-Bolted Double-Angle Connections

7/8-in. Bolts

Bolt and Angle Available Strength, kips

11 Rows

W44, 40



ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
A325/ F1852	N	—	180	269	225	337	269	404	317	476
	X	—	180	269	225	337	269	404	359	539
	SC Class A	STD	180	269	225	337	226	339	226	339
		OVS	163	245	163	245	163	245	163	245
		SSLT	178	267	192	288	192	288	192	288
	SC Class B	STD	180	269	225	337	269	404	317	476
		OVS	175	263	219	328	233	350	233	350
		SSLT	178	267	223	334	267	401	275	412
A490	N	—	180	269	225	337	269	404	359	539
	X	—	180	269	225	337	269	404	359	539
	SC Class A	STD	180	269	225	337	269	404	284	426
		OVS	175	263	205	308	205	308	205	308
		SSLT	178	267	223	334	242	362	242	362
	SC Class B	STD	180	269	225	337	269	404	359	539
		OVS	175	263	219	328	263	394	293	440
		SSLT	178	267	223	334	267	401	345	518

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1½		1¾		1½		1¾		1½		1¾	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1¼	429	644	437	656	401	602	410	614	426	639	434	651
	1⅜	431	647	440	659	404	606	412	618	428	643	437	655
	1½	434	651	442	663	406	609	414	622	431	646	439	658
	1⅝	436	654	444	667	409	613	417	625	433	650	441	662
	2	444	665	452	678	416	624	424	636	441	661	449	673
	3	463	695	471	707	436	653	444	665	460	690	468	702
Coped at Both Flanges	1¼	419	629	419	629	392	589	392	589	419	629	419	629
	1⅜	424	636	424	636	397	596	397	596	424	636	424	636
	1½	429	644	429	644	402	603	402	603	429	644	429	644
	1⅝	434	651	434	651	407	611	407	611	433	650	434	651
	2	444	665	449	673	416	624	422	633	441	661	449	673
	3	463	695	471	707	436	653	444	665	460	690	468	702
Uncoped		751	1130	751	1130	751	1130	751	1130	751	1130	751	1130

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	1500	2250

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

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	F_y = 50 ksi, F_u = 65 ksi
Angle	F_y = 36 ksi, F_u = 58 ksi

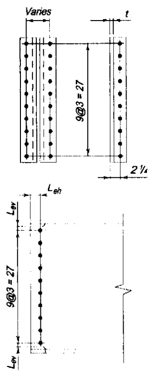
Table 10–1 (continued)
All-Bolted Double-Angle Connections

7/8-in. Bolts

Bolt and Angle Available Strength, kips

10 Rows

W44, 40, 36



ASTM Desig.	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
A325/F1852	N	—	163	245	204	306	245	368	289	433
	X	—	163	245	204	306	245	368	327	490
	SC Class A	STD	163	245	204	306	206	308	206	308
		OVS	149	223	149	223	149	223	149	223
		SSLT	162	243	175	262	175	262	175	262
	SC Class B	STD	163	245	204	306	245	368	289	433
		OVS	159	238	198	298	212	318	212	318
		SSLT	162	243	203	304	243	365	250	375
A490	N	—	163	245	204	306	245	368	327	490
	X	—	163	245	204	306	245	368	327	490
	SC Class A	STD	163	245	204	306	245	368	258	388
		OVS	159	238	187	280	187	280	187	280
		SSLT	162	243	203	304	220	329	220	329
	SC Class B	STD	163	245	204	306	245	368	327	490
		OVS	159	238	198	298	238	357	267	400
		SSLT	162	243	203	304	243	365	314	471

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2 ASD	1 1/2 LRFD	1 3/4 ASD	1 3/4 LRFD	1 1/2 ASD	1 1/2 LRFD	1 3/4 ASD	1 3/4 LRFD	1 1/2 ASD	1 1/2 LRFD	1 3/4 ASD	1 3/4 LRFD
Coped at Top Flange Only	1 1/4	390	585	398	597	365	547	373	559	387	580	395	593
	1 3/8	392	589	401	601	367	551	375	563	389	584	398	596
	1 1/2	395	592	403	605	370	555	378	567	392	588	400	600
	1 5/8	397	596	405	608	372	558	380	570	394	591	402	604
	2	405	607	413	619	379	569	388	581	402	602	410	615
	3	424	636	432	648	399	598	407	611	421	632	429	644
Coped at Both Flanges	1 1/4	380	570	380	570	356	534	356	534	380	570	380	570
	1 3/8	385	578	385	578	361	541	361	541	385	578	385	578
	1 1/2	390	585	390	585	366	548	366	548	390	585	390	585
	1 5/8	395	592	395	592	371	556	371	556	394	591	395	592
	2	405	607	410	614	379	569	385	578	402	602	410	614
	3	424	636	432	648	399	598	407	611	421	632	429	644
Uncoped		683	1020	683	1020	683	1020	683	1020	683	1020	683	1020

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/OVS/SSLT	1370	2050

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

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam		Angle	
F_y = 50 ksi	F_u = 65 ksi	F_y = 36 ksi	F_u = 58 ksi

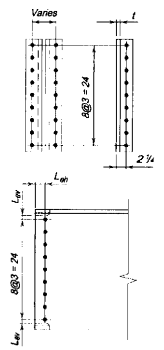
Table 10–1 (continued)
All-Bolted Double-Angle Connections

7/8-in. Bolts

Bolt and Angle Available Strength, kips

9 Rows

W44, 40, 36, 33



ASTM Desig.	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
A325/ F1852	N	—	147	221	184	276	221	331	260	390
	X	—	147	221	184	276	221	331	294	442
	SC Class A	STD	147	221	184	276	185	278	185	278
		OVS	134	201	134	201	134	201	134	201
		SSLT	146	219	157	236	157	236	157	236
	SC Class B	STD	147	221	184	276	221	331	260	390
		OVS	142	214	178	267	191	287	191	287
		SSLT	146	219	182	273	219	328	225	337
A490	N	—	147	221	184	276	221	331	294	442
	X	—	147	221	184	276	221	331	294	442
	SC Class A	STD	147	221	184	276	221	331	233	349
		OVS	142	214	168	252	168	252	168	252
		SSLT	146	219	182	273	198	297	198	297
	SC Class B	STD	147	221	184	276	221	331	294	442
		OVS	142	214	178	267	214	321	240	360
		SSLT	146	219	182	273	219	328	282	424

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	351	527	359	539	328	492	336	505	348	522	356	534
	1 3/8	353	530	362	542	331	496	339	508	350	526	359	538
	1 1/2	356	534	364	546	333	500	341	512	353	529	361	541
	1 5/8	358	537	366	550	336	503	344	516	355	533	363	545
	2	366	548	374	561	343	514	351	527	363	544	371	556
	3	385	578	393	590	362	544	371	556	382	573	390	585
Coped at Both Flanges	1 1/4	341	512	341	512	319	479	319	479	341	512	341	512
	1 3/8	346	519	346	519	324	486	324	486	346	519	346	519
	1 1/2	351	527	351	527	329	494	329	494	351	527	351	527
	1 5/8	356	534	356	534	334	501	334	501	355	533	356	534
	2	366	548	371	556	343	514	349	523	363	544	371	556
	3	385	578	393	590	362	544	371	556	382	573	390	585
Uncoped		614	921	614	921	614	921	614	921	614	921	614	921

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	1230	1840

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

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	Angle
F_y = 50 ksi	F_y = 36 ksi
F_u = 65 ksi	F_u = 58 ksi

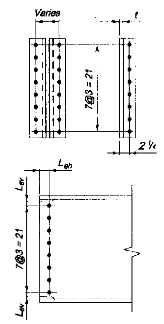
Table 10–1 (continued)
All-Bolted Double-Angle Connections

7/8-in. Bolts

Bolt and Angle Available Strength, kips

8 Rows

W44, 40, 36, 33, 30



ASTM Desig.	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
A325/F1852	N	—	131	197	164	246	197	295	231	346
	X	—	131	197	164	246	197	295	262	393
	SC Class A	STD	131	197	164	246	165	247	165	247
		OVS	119	178	119	178	119	178	119	178
		SSLT	130	194	140	210	140	210	140	210
	SC Class B	STD	131	197	164	246	197	295	231	346
		OVS	126	189	158	237	170	255	170	255
		SSLT	130	194	162	243	194	292	200	300
A490	N	—	131	197	164	246	197	295	262	393
	X	—	131	197	164	246	197	295	262	393
	SC Class A	STD	131	197	164	246	197	295	207	310
		OVS	126	189	149	224	149	224	149	224
		SSLT	130	194	162	243	176	264	176	264
	SC Class B	STD	131	197	164	246	197	295	262	393
		OVS	126	189	158	237	189	284	213	320
		SSLT	130	194	162	243	194	292	251	377

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	312	468	320	480	292	438	300	450	309	463	317	476
	1 3/8	314	472	323	484	294	441	302	453	311	467	320	479
	1 1/2	317	475	325	488	297	445	305	457	314	471	322	483
	1 5/8	319	479	327	491	299	449	307	461	316	474	324	487
	2	327	490	335	502	306	459	314	472	324	485	332	498
	3	346	519	354	531	326	489	334	501	343	515	351	527
Coped at Both Flanges	1 1/4	302	453	302	453	283	424	283	424	302	453	302	453
	1 3/8	307	461	307	461	288	431	288	431	307	461	307	461
	1 1/2	312	468	312	468	293	439	293	439	312	468	312	468
	1 5/8	317	475	317	475	297	446	297	446	316	474	317	475
	2	327	490	332	497	306	459	312	468	324	485	332	497
	3	346	519	354	531	326	489	334	501	343	515	351	527
Uncoped		546	819	546	819	546	819	546	819	546	819	546	819

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	**1090**	**1640**

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

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	Angle
F_y = 50 ksi, F_u = 65 ksi	F_y = 36 ksi, F_u = 58 ksi

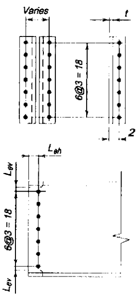
Table 10-1 (continued)
All-Bolted Double-Angle Connections

7/8-in. Bolts

Bolt and Angle Available Strength, kips

7 Rows

W44, 40, 36, 33, 30, 27, 24



ASTM Desig.	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
A325/F1852	N	—	115	172	144	215	172	258	202	303
	X	—	115	172	144	215	172	258	230	344
	SC Class A	STD	115	172	144	215	144	216	144	216
		OVS	104	156	104	156	104	156	104	156
		SSLT	113	170	122	184	122	184	122	184
	SC Class B	STD	115	172	144	215	172	258	202	303
		OVS	110	165	137	206	149	223	149	223
		SSLT	113	170	142	213	170	255	175	262
A490	N	—	115	172	144	215	172	258	230	344
	X	—	115	172	144	215	172	258	230	344
	SC Class A	STD	115	172	144	215	172	258	181	271
		OVS	110	165	131	196	131	196	131	196
		SSLT	113	170	142	213	154	231	154	231
	SC Class B	STD	115	172	144	215	172	258	230	344
		OVS	110	165	137	206	165	247	187	280
		SSLT	113	170	142	213	170	255	220	329

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type: STD, OVS, SSLT; L_{eh}*

L_{ev}, in.		STD L_{eh} 1½ ASD	STD 1½ LRFD	STD 1¾ ASD	STD 1¾ LRFD	OVS 1½ ASD	OVS 1½ LRFD	OVS 1¾ ASD	OVS 1¾ LRFD	SSLT 1½ ASD	SSLT 1½ LRFD	SSLT 1¾ ASD	SSLT 1¾ LRFD
Coped at Top Flange Only	1¼	273	410	281	422	255	383	263	395	270	405	278	417
	1⅜	275	413	284	425	258	386	266	399	272	409	281	421
	1½	278	417	286	429	260	390	268	402	275	412	283	424
	1⅝	280	420	288	433	262	394	271	406	277	416	285	428
	2	288	431	296	444	270	405	278	417	285	427	293	439
	3	307	461	315	473	289	434	297	446	304	456	312	468
Coped at Both Flanges	1¼	263	395	263	395	246	369	246	369	263	395	263	395
	1⅜	268	402	268	402	251	377	251	377	268	402	268	402
	1½	273	410	273	410	256	384	256	384	273	410	273	410
	1⅝	278	417	278	417	261	391	261	391	277	416	278	417
	2	288	431	293	439	270	405	275	413	285	427	293	439
	3	307	461	315	473	289	434	297	446	304	456	312	468
Uncoped		478	717	478	717	478	717	478	717	478	717	478	717

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/OVS/SSLT	956	1430

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

* Tabulated values include ¼-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

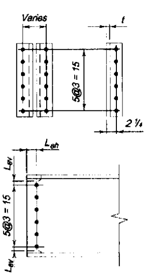
Beam		Angle	
F_y = 50 ksi	F_u = 65 ksi	F_y = 36 ksi	F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

$^{7}/_{8}$-in. Bolts

6 Rows

W40, 36, 33, 30, 27, 24, 21



Bolt and Angle Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness $^{1}/_{4}$ ASD	$^{1}/_{4}$ LRFD	$^{5}/_{16}$ ASD	$^{5}/_{16}$ LRFD	$^{3}/_{8}$ ASD	$^{3}/_{8}$ LRFD	$^{1}/_{2}$ ASD	$^{1}/_{2}$ LRFD
A325/ F1852	N	—	98.6	148	123	185	148	222	173	260
	X	—	98.6	148	123	185	148	222	197	296
	SC Class A	STD	98.6	148	123	185	123	185	123	185
		OVS	89.2	134	89.2	134	89.2	134	89.2	134
		SSLT	97.3	146	105	157	105	157	105	157
	SC Class B	STD	98.6	148	123	185	148	222	173	260
		OVS	93.5	140	117	175	127	191	127	191
		SSLT	97.3	146	122	182	146	219	150	225
A490	N	—	98.6	148	123	185	148	222	197	296
	X	—	98.6	148	123	185	148	222	197	296
	SC Class A	STD	98.6	148	123	185	148	222	155	233
		OVS	93.5	140	112	168	112	168	112	168
		SSLT	97.3	146	122	182	132	198	132	198
	SC Class B	STD	98.6	148	123	185	148	222	197	296
		OVS	93.5	140	117	175	140	210	160	240
		SSLT	97.3	146	122	182	146	219	188	282

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		$1^{1}/_{2}$		$1^{3}/_{4}$		$1^{1}/_{2}$		$1^{3}/_{4}$		$1^{1}/_{2}$		$1^{3}/_{4}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	$1^{1}/_{4}$	234	351	242	363	219	328	227	340	231	346	239	359
	$1^{3}/_{8}$	236	355	245	367	221	332	229	344	233	350	242	362
	$1^{1}/_{2}$	239	358	247	371	223	335	232	347	236	354	244	366
	$1^{5}/_{8}$	241	362	249	374	226	339	234	351	238	357	246	370
	2	249	373	257	385	233	350	241	362	246	368	254	381
	3	268	402	276	414	253	379	261	391	265	398	273	410
Coped at Both Flanges	$1^{1}/_{4}$	224	336	224	336	210	314	210	314	224	336	224	336
	$1^{3}/_{8}$	229	344	229	344	215	322	215	322	229	344	229	344
	$1^{1}/_{2}$	234	351	234	351	219	329	219	329	234	351	234	351
	$1^{5}/_{8}$	239	358	239	358	224	336	224	336	238	357	239	358
	2	249	373	254	380	233	350	239	358	246	368	254	380
	3	268	402	276	414	253	379	261	391	265	398	273	410
Uncoped		409	614	409	614	409	614	409	614	409	614	409	614

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	819	1230

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

* Tabulated values include $^{1}/_{4}$-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

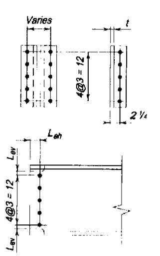
Beam	Angle
F_y = 50 ksi	F_y = 36 ksi
F_u = 65 ksi	F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

7/8-in. Bolts

Bolt and Angle Available Strength, kips

5 Rows	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
W30, 27, 24, 21, 18	A325/F1852	N	—	82.4	124	103	155	124	185	144	216
		X	—	82.4	124	103	155	124	185	165	247
		SC Class A	STD	82.4	124	103	154	103	154	103	154
			OVS	74.3	111	74.3	111	74.3	111	74.3	111
			SSLT	81.1	122	87.4	131	87.4	131	87.4	131
		SC Class B	STD	82.4	124	103	155	124	185	144	216
			OVS	77.2	116	96.5	145	106	159	106	159
			SSLT	81.1	122	101	152	122	182	125	187
	A490	N	—	82.4	124	103	155	124	185	165	247
		X	—	82.4	124	103	155	124	185	165	247
		SC Class A	STD	82.4	124	103	155	124	185	129	194
			OVS	77.2	116	93.3	140	93.3	140	93.3	140
			SSLT	81.1	122	101	152	110	165	110	165
		SC Class B	STD	82.4	124	103	155	124	185	165	247
			OVS	77.2	116	96.5	145	116	174	133	200
			SSLT	81.1	122	101	152	122	182	157	235



Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	195	293	203	305	182	273	190	285	192	288	200	300
	1 3/8	197	296	206	308	184	277	193	289	194	292	203	304
	1 1/2	200	300	208	312	187	280	195	293	197	295	205	307
	1 5/8	202	303	210	316	189	284	197	296	199	299	207	311
	2	210	314	218	327	197	295	205	307	207	310	215	322
	3	229	344	237	356	216	324	224	336	226	339	234	351
Coped at Both Flanges	1 1/4	185	278	185	278	173	260	173	260	185	278	185	278
	1 3/8	190	285	190	285	178	267	178	267	190	285	190	285
	1 1/2	195	293	195	293	183	274	183	274	195	293	195	293
	1 5/8	200	300	200	300	188	282	188	282	199	299	200	300
	2	210	314	215	322	197	295	202	303	207	310	215	322
	3	229	344	237	356	216	324	224	336	226	339	234	351
Uncoped		341	512	341	512	341	512	341	512	341	512	341	512

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	683	1020

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

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	Angle
F_y = 50 ksi	F_y = 36 ksi
F_u = 65 ksi	F_u = 58 ksi

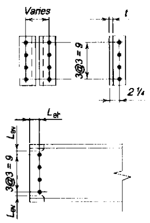
Table 10–1 (continued)
All-Bolted Double-Angle Connections

7/8-in. Bolts

Bolt and Angle Available Strength, kips

4 Rows

W24, 21, 18, 16



ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
A325/ F1852	N	—	65.3	97.9	81.6	122	97.9	147	115	173
	X	—	65.3	97.9	81.6	122	97.9	147	131	196
	SC Class A	STD	65.3	97.9	81.6	122	82.3	123	82.3	123
		OVS	59.4	89.2	59.4	89.2	59.4	89.2	59.4	89.2
		SSLT	64.9	97.3	69.9	105	69.9	105	69.9	105
	SC Class B	STD	65.3	97.9	81.6	122	97.9	147	115	173
		OVS	60.9	91.4	76.1	114	84.9	127	84.9	127
		SSLT	64.9	97.3	81.1	122	97.3	146	99.9	150
A490	N	—	65.3	97.9	81.6	122	97.9	147	131	196
	X	—	65.3	97.9	81.6	122	97.9	147	131	196
	SC Class A	STD	65.3	97.9	81.6	122	97.9	147	103	155
		OVS	60.9	91.4	74.7	112	74.7	112	74.7	112
		SSLT	64.9	97.3	81.1	122	87.9	132	87.9	132
	SC Class B	STD	65.3	97.9	81.6	122	97.9	147	131	196
		OVS	60.9	91.4	76.1	114	91.4	137	107	160
		SSLT	64.9	97.3	81.1	122	97.3	146	126	188

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	156	234	164	246	145	218	154	230	153	229	161	242
	1 3/8	158	238	167	250	148	222	156	234	155	233	164	245
	1 1/2	161	241	169	254	150	225	158	238	158	237	166	249
	1 5/8	163	245	171	257	153	229	161	241	160	240	168	253
	2	171	256	179	268	160	240	168	252	168	251	176	264
	3	190	285	198	297	180	269	188	282	187	281	195	293
Coped at Both Flanges	1 1/4	146	219	146	219	137	205	137	205	146	219	146	219
	1 3/8	151	227	151	227	141	212	141	212	151	227	151	227
	1 1/2	156	234	156	234	146	219	146	219	156	234	156	234
	1 5/8	161	241	161	241	151	227	151	227	160	240	161	241
	2	171	256	176	263	160	240	166	249	168	251	176	263
	3	190	285	198	297	180	269	188	282	187	281	195	293
Uncoped		273	410	273	410	273	410	273	410	273	410	273	410

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	546	819

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

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	Angle
F_y = 50 ksi	F_y = 36 ksi
F_u = 65 ksi	F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

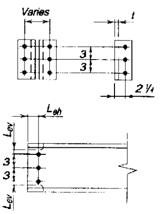
7/8-in. Bolts

Bolt and Angle Available Strength, kips

3 Rows

W18, 16, 14, 12, 10⁺

⁺Ltd. to W10x12, 15, 17, 19, 22, 26, 30



ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
A325/F1852	N	—	47.9	71.8	59.8	89.7	71.8	108	86.6	130
	X	—	47.9	71.8	59.8	89.7	71.8	108	95.7	144
	SC Class A	STD	47.9	71.8	59.8	89.7	61.7	92.5	61.7	92.5
		OVS	44.6	66.9	44.6	66.9	44.6	66.9	44.6	66.9
		SSLT	47.9	71.8	52.4	78.7	52.4	78.7	52.4	78.7
	SC Class B	STD	47.9	71.8	59.8	89.7	71.8	108	86.6	130
		OVS	44.6	66.9	55.7	83.6	63.7	95.5	63.7	95.5
		SSLT	47.9	71.8	59.8	89.7	71.8	108	74.9	112
A490	N	—	47.9	71.8	59.8	89.7	71.8	108	95.7	144
	X	—	47.9	71.8	59.8	89.7	71.8	108	95.7	144
	SC Class A	STD	47.9	71.8	59.8	89.7	71.8	108	77.5	116
		OVS	44.6	66.9	55.7	83.6	56	84	56	84
		SSLT	47.9	71.8	59.8	89.7	65.9	98.8	65.9	98.8
	SC Class B	STD	47.9	71.8	59.8	89.7	71.8	108	95.7	144
		OVS	44.6	66.9	55.7	83.6	66.9	100	80	120
		SSLT	47.9	71.8	59.8	89.7	71.8	108	94.1	141

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	117	176	125	188	109	163	117	176	114	171	122	183
	1 3/8	119	179	128	191	111	167	119	179	116	175	125	187
	1 1/2	122	183	130	195	114	171	122	183	119	178	127	190
	1 5/8	124	186	132	199	116	174	124	186	121	182	129	194
	2	132	197	140	210	124	185	132	197	129	193	137	205
	3	151	227	159	239	143	215	151	227	148	222	156	234
Coped at Both Flanges	1 1/4	107	161	107	161	99.9	150	99.9	150	107	161	107	161
	1 3/8	112	168	112	168	105	157	105	157	112	168	112	168
	1 1/2	117	176	117	176	110	165	110	165	117	176	117	176
	1 5/8	122	183	122	183	115	172	115	172	121	182	122	183
	2	132	197	137	205	124	185	129	194	129	193	137	205
	3	151	227	159	239	143	215	151	227	148	222	156	234
Uncoped		205	307	205	307	205	307	205	307	205	307	205	307

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	409	614

Notes:

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

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	Angle
F_y = 50 ksi, F_u = 65 ksi	F_y = 36 ksi, F_u = 58 ksi

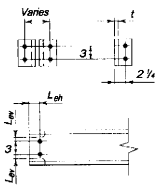
Table 10-1 (continued) All-Bolted Double-Angle Connections

7/8-in. Bolts

Bolt and Angle Available Strength, kips

2 Rows

W12, 10, 8



ASTM Desig.	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
A325/ F1852	N	—	30.5	45.7	38.1	57.1	45.7	68.5	57.7	86.6
	X	—	30.5	45.7	38.1	57.1	45.7	68.5	60.9	91.4
	SC Class A	STD	30.5	45.7	38.1	57.1	41.1	61.7	41.1	61.7
		OVS	28.3	42.4	29.7	44.6	29.7	44.6	29.7	44.6
		SSLT	30.5	45.7	35.0	52.4	35.0	52.4	35.0	52.4
	SC Class B	STD	30.5	45.7	38.1	57.1	45.7	68.5	57.7	86.6
		OVS	28.3	42.4	35.3	53.0	42.4	63.6	42.5	63.7
		SSLT	30.5	45.7	38.1	57.1	45.7	68.5	49.9	74.9
A490	N	—	30.5	45.7	38.1	57.1	45.7	68.5	60.9	91.4
	X	—	30.5	45.7	38.1	57.1	45.7	68.5	60.9	91.4
	SC Class A	STD	30.5	45.7	38.1	57.1	45.7	68.5	51.7	77.5
		OVS	28.3	42.4	35.3	53.0	37.3	56.0	37.3	56.0
		SSLT	30.5	45.7	38.1	57.1	43.9	65.9	43.9	65.9
	SC Class B	STD	30.5	45.7	38.1	57.1	45.7	68.5	60.9	91.4
		OVS	28.3	42.4	35.3	53.0	42.4	63.6	53.3	80.0
		SSLT	30.5	45.7	38.1	57.1	45.7	68.5	60.9	91.4

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2 ASD	1 1/2 LRFD	1 3/4 ASD	1 3/4 LRFD	1 1/2 ASD	1 1/2 LRFD	1 3/4 ASD	1 3/4 LRFD	1 1/2 ASD	1 1/2 LRFD	1 3/4 ASD	1 3/4 LRFD
Coped at Top Flange Only	1 1/4	78.0	117	86.1	129	72.3	108	80.4	121	75.0	112	83.1	125
	1 3/8	80.4	121	88.6	133	74.8	112	82.9	124	77.4	116	85.5	128
	1 1/2	82.9	124	91.0	137	77.2	116	85.3	128	79.8	120	88.0	132
	1 5/8	85.3	128	93.4	140	79.6	119	87.8	132	82.3	123	90.4	136
	2	92.6	139	101	151	86.9	130	95.1	143	89.6	134	97.7	147
	3	112	168	120	180	106	160	115	172	109	164	117	176
Coped at Both Flanges	1 1/4	68.3	102	68.3	102	63.4	95.1	63.4	95.1	68.3	102	68.3	102
	1 3/8	73.1	110	73.1	110	68.3	102	68.3	102	73.1	110	73.1	110
	1 1/2	78.0	117	78.0	117	73.1	110	73.1	110	78.0	117	78.0	117
	1 5/8	82.9	124	82.9	124	78.0	117	78.0	117	82.3	123	82.9	124
	2	92.6	139	97.5	146	86.9	130	92.6	139	89.6	134	97.5	146
	3	112	168	120	180	106	160	115	172	109	164	117	176
Uncoped		137	205	137	205	137	205	137	205	137	205	137	205

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	273	410

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

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

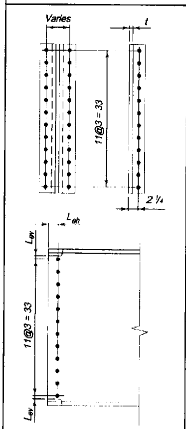
Beam	F_y = 50 ksi, F_u = 65 ksi
Angle	F_y = 36 ksi, F_u = 58 ksi

Table 10-1 (continued)
All-Bolted Double-Angle Connections

1-in. Bolts

12 Rows

W44



Bolt and Angle Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
A325/ F1852	N	—	191	287	239	359	287	431	383	574
	X	—	191	287	239	359	287	431	383	574
	SC Class A	STD	191	287	239	359	287	431	323	484
		OVS	172	258	215	322	233	350	233	350
		SSLT	191	287	239	359	274	411	274	411
	SC Class B	STD	191	287	239	359	287	431	383	574
		OVS	172	258	215	322	258	387	333	500
		SSLT	191	287	239	359	287	431	383	574
A490	N	—	191	287	239	359	287	431	383	574
	X	—	191	287	239	359	287	431	383	574
	SC Class A	STD	191	287	239	359	287	431	383	574
		OVS	172	258	215	322	258	387	293	439
		SSLT	191	287	239	359	287	431	344	516
	SC Class B	STD	191	287	239	359	287	431	383	574
		OVS	172	258	215	322	258	387	344	515
		SSLT	191	287	239	359	287	431	383	574

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
L_{eh}*		1½		1¾		1½		1¾		1½		1¾	
L_{ev}, in.		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1¼	438	657	446	669	393	589	401	601	434	651	442	663
	1⅜	440	661	449	673	395	593	403	605	436	654	444	667
	1½	443	664	451	676	398	597	406	609	439	658	447	670
	1⅝	445	668	453	680	400	600	408	612	441	662	449	674
	2	453	679	461	691	407	611	416	623	449	673	457	685
	3	472	708	480	720	427	640	435	653	468	702	476	714
Coped at Both Flanges	1¼	429	644	429	644	385	578	385	578	429	644	429	644
	1⅜	434	651	434	651	390	585	390	585	434	651	434	651
	1½	439	658	439	658	395	592	395	592	439	658	439	658
	1⅝	444	665	444	665	400	600	400	600	441	662	444	665
	2	453	679	458	687	407	611	414	622	449	673	457	685
	3	472	708	480	720	427	640	435	653	468	702	476	714
Uncoped		909	1360	909	1360	829	1240	829	1240	909	1360	909	1360

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ SSLT	1820	2730
OVS	1660	2490

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

* Tabulated values include ¼-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

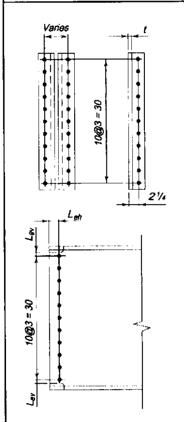
Beam	Angle
F_y = 50 ksi F_u = 65 ksi	F_y = 36 ksi F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

1-in. Bolts

11 Rows

W44, 40



Bolt and Angle Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
A325/ F1852	N	—	175	263	219	328	263	394	350	525
	X	—	175	263	219	328	263	394	350	525
	SC Class A	STD	175	263	219	328	263	394	296	444
		OVS	157	236	196	295	214	321	214	321
		SSLT	175	263	219	328	251	377	251	377
	SC Class B	STD	175	263	219	328	263	394	350	525
		OVS	157	236	196	295	236	354	305	458
		SSLT	175	263	219	328	263	394	350	525
A490	N	—	175	263	219	328	263	394	350	525
	X	—	175	263	219	328	263	394	350	525
	SC Class A	STD	175	263	219	328	263	394	350	525
		OVS	157	236	196	295	236	354	268	402
		SSLT	175	263	219	328	263	394	316	473
	SC Class B	STD	175	263	219	328	263	394	350	525
		OVS	157	236	196	295	236	354	314	471
		SSLT	175	263	219	328	263	394	350	525

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2 ASD	1 1/2 LRFD	1 3/4 ASD	1 3/4 LRFD	1 1/2 ASD	1 1/2 LRFD	1 3/4 ASD	1 3/4 LRFD	1 1/2 ASD	1 1/2 LRFD	1 3/4 ASD	1 3/4 LRFD
Coped at Top Flange Only	1 1/4	401	602	410	614	360	540	368	552	397	596	405	608
	1 3/8	404	606	412	618	362	544	371	556	400	600	408	612
	1 1/2	406	609	414	622	365	547	373	559	402	603	410	615
	1 5/8	409	613	417	625	367	551	375	563	405	607	413	619
	2	416	624	424	636	375	562	383	574	412	618	420	630
	3	436	653	444	665	394	591	402	603	431	647	440	659
Coped at Both Flanges	1 1/4	392	589	392	589	352	528	352	528	392	589	392	589
	1 3/8	397	596	397	596	357	536	357	536	397	596	397	596
	1 1/2	402	603	402	603	362	543	362	543	402	603	402	603
	1 5/8	407	611	407	611	367	550	367	550	405	607	407	611
	2	416	624	422	633	375	562	381	572	412	618	420	630
	3	436	653	444	665	394	591	402	603	431	647	440	659
Uncoped		834	1250	834	1250	761	1140	761	1140	834	1250	834	1250

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ SSLT	1670	2500
OVS	1520	2280

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

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

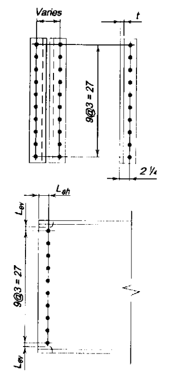
Beam	Angle
F_y = 50 ksi	F_y = 36 ksi
F_u = 65 ksi	F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

1-in. Bolts

10 Rows

W44, 40, 36



Bolt and Angle Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
A325/ F1852	N	—	159	238	198	298	238	357	318	476
	X	—	159	238	198	298	238	357	318	476
	SC Class A	STD	159	238	198	298	238	357	269	403
		OVS	142	214	178	267	194	291	194	291
		SSLT	159	238	198	298	229	343	229	343
	SC Class B	STD	159	238	198	298	238	357	318	476
		OVS	142	214	178	267	214	321	278	416
		SSLT	159	238	198	298	238	357	318	476
A490	N	—	159	238	198	298	238	357	318	476
	X	—	159	238	198	298	238	357	318	476
	SC Class A	STD	159	238	198	298	238	357	318	476
		OVS	142	214	178	267	214	321	244	366
		SSLT	159	238	198	298	238	357	287	430
	SC Class B	STD	159	238	198	298	238	357	318	476
		OVS	142	214	178	267	214	321	285	427
		SSLT	159	238	198	298	238	357	318	476

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
	L_{eh}*	1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
L_{ev}, in.		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	365	547	373	559	327	491	335	503	361	541	369	553
	1 3/8	367	551	375	563	329	494	338	506	363	545	371	557
	1 1/2	370	555	378	567	332	498	340	510	366	548	374	561
	1 5/8	372	558	380	570	334	502	342	514	368	552	376	564
	2	379	569	388	581	342	512	350	525	375	563	384	575
	3	399	598	407	611	361	542	369	554	395	592	403	605
Coped at Both Flanges	1 1/4	356	534	356	534	319	479	319	479	356	534	356	534
	1 3/8	361	541	361	541	324	486	324	486	361	541	361	541
	1 1/2	366	548	366	548	329	494	329	494	366	548	366	548
	1 5/8	371	556	371	556	334	501	334	501	368	552	371	556
	2	379	569	385	578	342	512	349	523	375	563	384	575
	3	399	598	407	611	361	542	369	554	395	592	403	605
Uncoped		758	1140	758	1140	692	1040	692	1040	758	1140	758	1140

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ SSLT	1520	2270
OVS	1380	2080

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

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	Angle
F_y = 50 ksi, F_u = 65 ksi	F_y = 36 ksi, F_u = 58 ksi

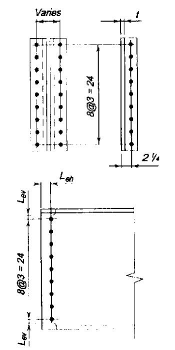
Table 10–1 (continued)
All-Bolted Double-Angle Connections

1-in. Bolts

Bolt and Angle Available Strength, kips

9 Rows

W44, 40, 36, 33



ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
A325/ F1852	N	—	142	214	178	267	214	321	285	427
	X	—	142	214	178	267	214	321	285	427
	SC Class A	STD	142	214	178	267	214	321	242	363
		OVS	128	192	160	240	175	262	175	262
		SSLT	142	214	178	267	206	309	206	309
	SC Class B	STD	142	214	178	267	214	321	285	427
		OVS	128	192	160	240	192	288	250	375
		SSLT	142	214	178	267	214	321	285	427
A490	N	—	142	214	178	267	214	321	285	427
	X	—	142	214	178	267	214	321	285	427
	SC Class A	STD	142	214	178	267	214	321	285	427
		OVS	128	192	160	240	192	288	219	329
		SSLT	142	214	178	267	214	321	258	387
	SC Class B	STD	142	214	178	267	214	321	285	427
		OVS	128	192	160	240	192	288	256	383
		SSLT	142	214	178	267	214	321	285	427

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
L_{eh}*		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
L_{ev}, in.		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	328	492	336	505	294	441	302	453	324	486	332	498
	1 3/8	331	496	339	508	297	445	305	457	327	490	335	502
	1 1/2	333	500	341	512	299	449	307	461	329	494	337	506
	1 5/8	336	503	344	516	301	452	310	464	332	497	340	509
	2	343	514	351	527	309	463	317	475	339	508	347	520
	3	362	544	371	556	328	492	336	505	358	537	366	550
Coped at Both Flanges	1 1/4	319	479	319	479	286	430	286	430	319	479	319	479
	1 3/8	324	486	324	486	291	437	291	437	324	486	324	486
	1 1/2	329	494	329	494	296	444	296	444	329	494	329	494
	1 5/8	334	501	334	501	301	452	301	452	332	497	334	501
	2	343	514	349	523	309	463	316	473	339	508	347	520
	3	362	544	371	556	328	492	336	505	358	537	366	550
Uncoped		683	1020	683	1020	624	936	624	936	683	1020	683	1020

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ SSLT	1370	2050
OVS	1250	1870

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

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	F_y = 50 ksi, F_u = 65 ksi
Angle	F_y = 36 ksi, F_u = 58 ksi

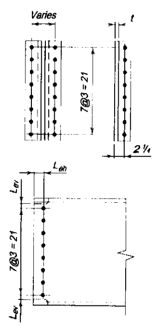
Table 10–1 (continued)
All-Bolted Double-Angle Connections

1-in. Bolts

Bolt and Angle Available Strength, kips

8 Rows

W44, 40, 36, 33, 30



ASTM Desig.	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
A325/ F1852	N	—	126	189	158	237	189	284	252	378
	X	—	126	189	158	237	189	284	252	378
	SC Class A	STD	126	189	158	237	189	284	215	323
		OVS	113	170	141	212	155	233	155	233
		SSLT	126	189	158	237	183	274	183	274
	SC Class B	STD	126	189	158	237	189	284	252	378
		OVS	113	170	141	212	170	254	222	333
		SSLT	126	189	158	237	189	284	252	378
A490	N	—	126	189	158	237	189	284	252	378
	X	—	126	189	158	237	189	284	252	378
	SC Class A	STD	126	189	158	237	189	284	252	378
		OVS	113	170	141	212	170	254	195	293
		SSLT	126	189	158	237	189	284	229	344
	SC Class B	STD	126	189	158	237	189	284	252	378
		OVS	113	170	141	212	170	254	226	339
		SSLT	126	189	158	237	189	284	252	378

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
	L_{eh}*	1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
	L_{ev}, in.	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	292	438	300	450	261	392	269	404	288	431	296	444
	1 3/8	294	441	302	453	264	395	272	408	290	435	298	447
	1 1/2	297	445	305	457	266	399	274	411	293	439	301	451
	1 5/8	299	449	307	461	269	403	277	415	295	442	303	455
	2	306	459	314	472	276	414	284	426	302	453	310	466
	3	326	489	334	501	295	443	303	455	322	483	330	495
Coped at Both Flanges	1 1/4	283	424	283	424	254	380	254	380	283	424	283	424
	1 3/8	288	431	288	431	258	388	258	388	288	431	288	431
	1 1/2	293	439	293	439	263	395	263	395	293	439	293	439
	1 5/8	297	446	297	446	268	402	268	402	295	442	297	446
	2	306	459	312	468	276	414	283	424	302	453	310	466
	3	326	489	334	501	295	443	303	455	322	483	330	495
Uncoped		607	910	607	910	556	834	556	834	607	910	607	910

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ SSLT	1210	1820
OVS	1110	1670

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

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

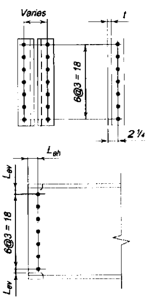
Beam	F_y = 50 ksi, F_u = 65 ksi
Angle	F_y = 36 ksi, F_u = 58 ksi

Table 10-1 (continued)
All-Bolted Double-Angle Connections

1-in. Bolts

7 Rows

W44, 40, 36, 33, 30, 27, 24



Bolt and Angle Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
A325/F1852	N	—	110	165	137	206	165	247	220	330
	X	—	110	165	137	206	165	247	220	330
	SC Class A	STD	110	165	137	206	165	247	188	282
		OVS	98.4	148	123	185	136	204	136	204
		SSLT	110	165	137	206	160	240	160	240
	SC Class B	STD	110	165	137	206	165	247	220	330
		OVS	98.4	148	123	185	148	221	194	291
		SSLT	110	165	137	206	165	247	220	330
A490	N	—	110	165	137	206	165	247	220	330
	X	—	110	165	137	206	165	247	220	330
	SC Class A	STD	110	165	137	206	165	247	220	330
		OVS	98.4	148	123	185	148	221	171	256
		SSLT	110	165	137	206	165	247	201	301
	SC Class B	STD	110	165	137	206	165	247	220	330
		OVS	98.4	148	123	185	148	221	197	295
		SSLT	110	165	137	206	165	247	220	330

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
L_{eh}*		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
L_{ev}, in.		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	255	383	263	395	228	342	236	355	251	377	259	389
	1 3/8	258	386	266	399	231	346	239	358	254	380	262	392
	1 1/2	260	390	268	402	233	350	241	362	256	384	264	396
	1 5/8	262	394	271	406	236	353	244	366	258	388	267	400
	2	270	405	278	417	243	364	251	377	266	399	274	411
	3	289	434	297	446	262	394	271	406	285	428	293	440
Coped at Both Flanges	1 1/4	246	369	246	369	221	331	221	331	246	369	246	369
	1 3/8	251	377	251	377	225	338	225	338	251	377	251	377
	1 1/2	256	384	256	384	230	346	230	346	256	384	256	384
	1 5/8	261	391	261	391	235	353	235	353	258	388	261	391
	2	270	405	275	413	243	364	250	375	266	399	274	411
	3	289	434	297	446	262	394	271	406	285	428	293	440
Uncoped		531	797	531	797	487	731	487	731	531	797	531	797

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/SSLT	**1060**	**1590**
OVS	**975**	**1460**

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

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

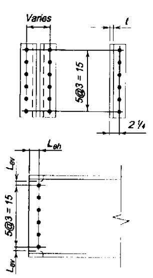
Beam	Angle
F_y = 50 ksi	F_y = 36 ksi
F_u = 65 ksi	F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

1-in. Bolts

6 Rows

W40, 36, 33, 30, 27, 24, 21



Bolt and Angle Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
A325/ F1852	N	—	93.5	140	117	175	140	210	187	281
	X	—	93.5	140	117	175	140	210	187	281
	SC Class A	STD	93.5	140	117	175	140	210	161	242
		OVS	83.7	126	105	157	117	175	117	175
		SSLT	93.5	140	117	175	137	206	137	206
	SC Class B	STD	93.5	140	117	175	140	210	187	281
		OVS	83.7	126	105	157	126	188	167	250
		SSLT	93.5	140	117	175	140	210	187	281
A490	N	—	93.5	140	117	175	140	210	187	281
	X	—	93.5	140	117	175	140	210	187	281
	SC Class A	STD	93.5	140	117	175	140	210	187	281
		OVS	83.7	126	105	157	126	188	146	219
		SSLT	93.5	140	117	175	140	210	172	258
	SC Class B	STD	93.5	140	117	175	140	210	187	281
		OVS	83.7	126	105	157	126	188	167	251
		SSLT	93.5	140	117	175	140	210	187	281

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	219	328	227	340	195	293	204	305	215	322	223	334
	1 3/8	221	332	229	344	198	297	206	309	217	325	225	338
	1 1/2	223	335	232	347	200	300	208	313	219	329	228	341
	1 5/8	226	339	234	351	203	304	211	316	222	333	230	345
	2	233	350	241	362	210	315	218	327	229	344	237	356
	3	253	379	261	391	230	344	238	356	249	373	257	385
Coped at Both Flanges	1 1/4	210	314	210	314	188	282	188	282	210	314	210	314
	1 3/8	215	322	215	322	193	289	193	289	215	322	215	322
	1 1/2	219	329	219	329	197	296	197	296	219	329	219	329
	1 5/8	224	336	224	336	202	303	202	303	222	333	224	336
	2	233	350	239	358	210	315	217	325	229	344	237	356
	3	253	379	261	391	230	344	238	356	249	373	257	385
Uncoped		456	684	456	684	419	629	419	629	456	684	456	684

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ SSLT	912	1370
OVS	839	1260

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

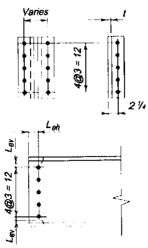
* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	F_y = 50 ksi F_u = 65 ksi	
Angle	F_y = 36 ksi F_u = 58 ksi	

Table 10–1 (continued)
All-Bolted Double-Angle Connections

1-in. Bolts

Bolt and Angle Available Strength, kips

5 Rows W30, 27, 24, 21, 18	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	A325/ F1852	N	—	77.2	116	96.5	145	116	174	154	232
		X	—	77.2	116	96.5	145	116	174	154	232
		SC Class A	STD	77.2	116	96.5	145	116	174	134	202
			OVS	69.1	104	86.3	129	97.2	146	97.2	146
			SSLT	77.2	116	96.5	145	114	171	114	171
		SC Class B	STD	77.2	116	96.5	145	116	174	154	232
			OVS	69.1	104	86.3	129	104	155	138	207
			SSLT	77.2	116	96.5	145	116	174	154	232
	A490	N	—	77.2	116	96.5	145	116	174	154	232
		X	—	77.2	116	96.5	145	116	174	154	232
		SC Class A	STD	77.2	116	96.5	145	116	174	154	232
			OVS	69.1	104	86.3	129	104	155	122	183
			SSLT	77.2	116	96.5	145	116	174	143	215
		SC Class B	STD	77.2	116	96.5	145	116	174	154	232
			OVS	69.1	104	86.3	129	104	155	138	207
			SSLT	77.2	116	96.5	145	116	174	154	232

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	182	273	190	285	163	244	171	256	178	267	186	279
	1 3/8	184	277	193	289	165	247	173	260	180	271	189	283
	1 1/2	187	280	195	293	167	251	176	263	183	274	191	286
	1 5/8	189	284	197	296	170	255	178	267	185	278	193	290
	2	197	295	205	307	177	266	185	278	193	289	201	301
	3	216	324	224	336	197	295	205	307	212	318	220	330
Coped at Both Flanges	1 1/4	173	260	173	260	155	232	155	232	173	260	173	260
	1 3/8	178	267	178	267	160	239	160	239	178	267	178	267
	1 1/2	183	274	183	274	165	247	165	247	183	274	183	274
	1 5/8	188	282	188	282	169	254	169	254	185	278	188	282
	2	197	295	202	303	177	266	184	276	193	289	201	301
	3	216	324	224	336	197	295	205	307	212	318	220	330
Uncoped		380	570	380	570	351	527	351	527	380	570	380	570

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ SSLT	761	1140
OVS	702	1050

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

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam		Angle	
F_y = 50 ksi	F_u = 65 ksi	F_y = 36 ksi	F_u = 58 ksi

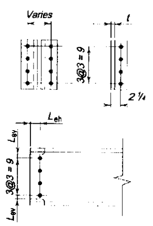
Table 10–1 (continued)
All-Bolted Double-Angle Connections

1-in. Bolts

Bolt and Angle Available Strength, kips

4 Rows

W24, 21, 18, 16



ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
A325/ F1852	N	—	60.9	91.4	76.1	114	91.4	137	122	183
	X	—	60.9	91.4	76.1	114	91.4	137	122	183
	SC Class A	STD	60.9	91.4	76.1	114	91.4	137	108	161
		OVS	54.4	81.6	68.0	102	77.7	117	77.7	117
		SSLT	60.9	91.4	76.1	114	91.4	137	91.4	137
	SC Class B	STD	60.9	91.4	76.1	114	91.4	137	122	183
		OVS	54.4	81.6	68.0	102	81.6	122	109	163
		SSLT	60.9	91.4	76.1	114	91.4	137	122	183
A490	N	—	60.9	91.4	76.1	114	91.4	137	122	183
	X	—	60.9	91.4	76.1	114	91.4	137	122	183
	SC Class A	STD	60.9	91.4	76.1	114	91.4	137	122	183
		OVS	54.4	81.6	68.0	102	81.6	122	97.5	146
		SSLT	60.9	91.4	76.1	114	91.4	137	115	172
	SC Class B	STD	60.9	91.4	76.1	114	91.4	137	122	183
		OVS	54.4	81.6	68.0	102	81.6	122	109	163
		SSLT	60.9	91.4	76.1	114	91.4	137	122	183

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
L_{eh}*		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
L_{ev}, in.		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	145	218	154	230	130	194	138	207	141	212	150	224
	1 3/8	148	222	156	234	132	198	140	210	144	216	152	228
	1 1/2	150	225	158	238	134	202	143	214	146	219	154	232
	1 5/8	153	229	161	241	137	205	145	218	149	223	157	235
	2	160	240	168	252	144	216	152	229	156	234	164	246
	3	180	269	188	282	164	246	172	258	176	263	184	275
Coped at Both Flanges	1 1/4	137	205	137	205	122	183	122	183	137	205	137	205
	1 3/8	141	212	141	212	127	190	127	190	141	212	141	212
	1 1/2	146	219	146	219	132	197	132	197	146	219	146	219
	1 5/8	151	227	151	227	137	205	137	205	149	223	151	227
	2	160	240	166	249	144	216	151	227	156	234	164	246
	3	180	269	188	282	164	246	172	258	176	263	184	275
Uncoped		305	457	305	457	283	424	283	424	305	457	305	457

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ SSLT	609	914
OVS	566	848

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

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam F_y = 50 ksi, F_u = 65 ksi

Angle F_y = 36 ksi, F_u = 58 ksi

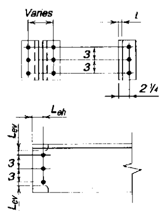
Table 10–1 (continued) All-Bolted Double-Angle Connections

1-in. Bolts

3 Rows

W18, 16, 14, 12, 10⁺

⁺Ltd. to W10x12, 15, 17, 19, 22, 26, 30



Bolt and Angle Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
A325/F1852	N	—	44.6	66.9	55.7	83.6	66.9	100	89.2	134
	X	—	44.6	66.9	55.7	83.6	66.9	100	89.2	134
	SC Class A	STD	44.6	66.9	55.7	83.6	66.9	100	80.7	121
		OVS	39.7	59.5	49.6	74.4	58.3	87.4	58.3	87.4
		SSLT	44.6	66.9	55.7	83.6	66.9	100	68.6	103
	SC Class B	STD	44.6	66.9	55.7	83.6	66.9	100	89.2	134
		OVS	39.7	59.5	49.6	74.4	59.5	89.3	79.4	119
		SSLT	44.6	66.9	55.7	83.6	66.9	100	89.2	134
A490	N	—	44.6	66.9	55.7	83.6	66.9	100	89.2	134
	X	—	44.6	66.9	55.7	83.6	66.9	100	89.2	134
	SC Class A	STD	44.6	66.9	55.7	83.6	66.9	100	89.2	134
		OVS	39.7	59.5	49.6	74.4	59.5	89.3	73.2	110
		SSLT	44.6	66.9	55.7	83.6	66.9	100	86.1	129
	SC Class B	STD	44.6	66.9	55.7	83.6	66.9	100	89.2	134
		OVS	39.7	59.5	49.6	74.4	59.5	89.3	79.4	119
		SSLT	44.6	66.9	55.7	83.6	66.9	100	89.2	134

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type	L_{ev}, in.	STD, L_{eh}* = 1 1/2 ASD	LRFD	STD, 1 3/4 ASD	LRFD	OVS, 1 1/2 ASD	LRFD	OVS, 1 3/4 ASD	LRFD	SSLT, 1 1/2 ASD	LRFD	SSLT, 1 3/4 ASD	LRFD
Coped at Top Flange Only	1 1/4	109	163	117	176	96.7	145	105	157	105	157	113	169
	1 3/8	111	167	119	179	99.1	149	107	161	107	161	115	173
	1 1/2	114	171	122	183	102	152	110	165	110	165	118	177
	1 5/8	116	174	124	186	104	156	112	168	112	168	120	180
	2	124	185	132	197	111	167	119	179	119	179	128	191
	3	143	215	151	227	131	196	139	208	139	208	147	221
Coped at Both Flanges	1 1/4	99.9	150	99.9	150	89	133	89	133	99.9	150	99.9	150
	1 3/8	105	157	105	157	93.8	141	93.8	141	105	157	105	157
	1 1/2	110	165	110	165	98.7	148	98.7	148	110	165	110	165
	1 5/8	115	172	115	172	104	155	104	155	112	168	115	172
	2	124	185	129	194	111	167	118	177	119	179	128	191
	3	143	215	151	227	131	196	139	208	139	208	147	221
Uncoped		229	344	229	344	215	322	215	322	229	344	229	344

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/SSLT	458	687
OVS	429	644

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

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

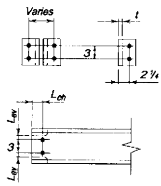
Beam	Angle
F_y = 50 ksi, F_u = 65 ksi	F_y = 36 ksi, F_u = 58 ksi

Table 10–1 (continued) All-Bolted Double-Angle Connections

1-in. Bolts

2 Rows

W12, 10, 8



Bolt and Angle Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
A325/ F1852	N	—	28.3	42.4	35.3	53.0	42.4	63.6	56.6	84.8
	X	—	28.3	42.4	35.3	53.0	42.4	63.6	56.6	84.8
	SC Class A	STD	28.3	42.4	35.3	53.0	42.4	63.6	53.8	80.7
		OVS	25.0	37.5	31.3	46.9	37.5	56.3	38.9	58.3
		SSLT	28.3	42.4	35.3	53.0	42.4	63.6	45.7	68.6
	SC Class B	STD	28.3	42.4	35.3	53.0	42.4	63.6	56.6	84.8
		OVS	25.0	37.5	31.3	46.9	37.5	56.3	50.0	75.0
		SSLT	28.3	42.4	35.3	53.0	42.4	63.6	56.6	84.8
A490	N	—	28.3	42.4	35.3	53.0	42.4	63.6	56.6	84.8
	X	—	28.3	42.4	35.3	53.0	42.4	63.6	56.6	84.8
	SC Class A	STD	28.3	42.4	35.3	53.0	42.4	63.6	56.6	84.8
		OVS	25.0	37.5	31.3	46.9	37.5	56.3	48.8	73.2
		SSLT	28.3	42.4	35.3	53.0	42.4	63.6	56.6	84.8
	SC Class B	STD	28.3	42.4	35.3	53.0	42.4	63.6	56.6	84.8
		OVS	25.0	37.5	31.3	46.9	37.5	56.3	50.0	75.0
		SSLT	28.3	42.4	35.3	53.0	42.4	63.6	56.6	84.8

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2 ASD	1 1/2 LRFD	1 3/4 ASD	1 3/4 LRFD	1 1/2 ASD	1 1/2 LRFD	1 3/4 ASD	1 3/4 LRFD	1 1/2 ASD	1 1/2 LRFD	1 3/4 ASD	1 3/4 LRFD
Coped at Top Flange Only	1 1/4	72.3	108	80.4	121	63.8	95.7	71.9	108	68.3	102	76.4	115
	1 3/8	74.8	112	82.9	124	66.2	99.3	74.3	112	70.7	106	78.8	118
	1 1/2	77.2	116	85.3	128	68.7	103	76.8	115	73.1	110	81.3	122
	1 5/8	79.6	119	87.8	132	71.1	107	79.2	119	75.6	113	83.7	126
	2	86.9	130	95.1	143	78.4	118	86.5	130	82.9	124	91.0	137
	3	106	160	115	172	97.9	147	106	159	102	154	111	166
Coped at Both Flanges	1 1/4	63.4	95.1	63.4	95.1	56.1	84.1	56.1	84.1	63.4	95.1	63.4	95.1
	1 3/8	68.3	102	68.3	102	60.9	91.4	60.9	91.4	68.3	102	68.3	102
	1 1/2	73.1	110	73.1	110	65.8	98.7	65.8	98.7	73.1	110	73.1	110
	1 5/8	78.0	117	78.0	117	70.7	106	70.7	106	75.6	113	78.0	117
	2	86.9	130	92.6	139	78.4	118	85.3	128	82.9	124	91.0	137
	3	106	160	115	172	97.9	147	106	159	102	154	111	166
Uncoped		154	230	154	230	146	219	146	219	154	230	154	230

Support Available Strength per Inch Thickness, kips/in.

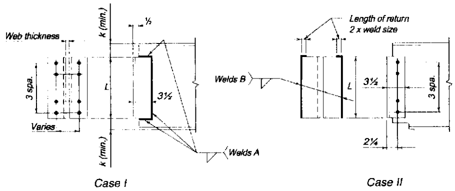
Hole Type	ASD	LRFD
STD/ SSLT	307	461
OVS	293	439

Notes:

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

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

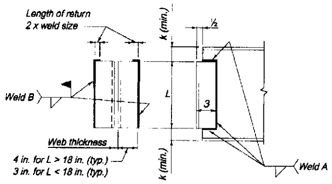
Table 10–2
Bolted/Welded Double-Angle Connections



n	L	Welds A (70 ksi) Weld Size, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	Minimum Web Thickness, in.	Welds B (70 ksi) Weld Size, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	Minimum Support Thickness, in.
12	35 1/2	5/16	391	586	0.476	3/8	366	550	0.286
		1/4	313	469	0.381	5/16	305	458	0.238
		3/16	234	352	0.286	1/4	244	366	0.190
11	32 1/2	5/16	366	548	0.476	3/8	331	496	0.286
		1/4	293	439	0.381	5/16	276	414	0.238
		3/16	219	329	0.286	1/4	221	331	0.190
10	29 1/2	5/16	337	506	0.476	3/8	295	443	0.286
		1/4	270	405	0.381	5/16	246	369	0.238
		3/16	202	303	0.286	1/4	197	295	0.190
9	26 1/2	5/16	309	464	0.476	3/8	259	389	0.286
		1/4	248	371	0.381	5/16	216	324	0.238
		3/16	186	278	0.286	1/4	173	259	0.190
8	23 1/2	5/16	282	422	0.476	3/8	223	335	0.286
		1/4	225	338	0.381	5/16	186	279	0.238
		3/16	169	253	0.286	1/4	149	223	0.190
7	20 1/2	5/16	253	379	0.476	3/8	187	280	0.286
		1/4	202	304	0.381	5/16	156	234	0.238
		3/16	152	228	0.286	1/4	125	187	0.190
6	17 1/2	5/16	223	334	0.476	3/8	150	226	0.286
		1/4	178	267	0.381	5/16	125	188	0.238
		3/16	134	200	0.286	1/4	100	150	0.190
5	14 1/2	5/16	191	287	0.476	3/8	115	172	0.286
		1/4	153	229	0.381	5/16	95.5	143	0.238
		3/16	115	172	0.286	1/4	76.4	115	0.190
4	11 1/2	5/16	158	237	0.476	3/8	79.9	120	0.286
		1/4	126	190	0.381	5/16	66.6	99.9	0.238
		3/16	94.9	142	0.286	1/4	53.3	79.9	0.190
3	8 1/2	5/16	122	184	0.476	3/8	48.1	72.2	0.286
		1/4	97.9	147	0.381	5/16	40.1	60.2	0.238
		3/16	73.4	110	0.286	1/4	32.1	48.1	0.190
2	5 1/2	5/16	83.6	125	0.476	3/8	21.9	32.8	0.286
		1/4	66.9	100	0.381	5/16	18.2	27.3	0.238
		3/16	50.2	75.3	0.286	1/4	14.6	21.9	0.190

ASD	LRFD	Beam	
$\Omega = 2.00$	$\phi = 0.75$	$F_y = 50$ ksi	$F_u = 65$ ksi

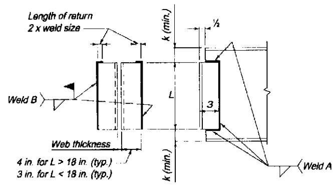
Table 10–3
All-Welded Double-Angle Connections



L	Welds A (70 ksi)				Welds B (70 ksi)			
	Weld Size, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	Minimum Web Thickness, in.	Weld Size, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	Minimum Web Thickness, in.
36	5/16	**395**	592	0.476	3/8	**372**	558	0.286
	1/4	**316**	474	0.381	5/16	**310**	465	0.238
	3/16	**237**	355	0.286	1/4	**248**	372	0.190
34	5/16	**378**	568	0.476	3/8	**349**	523	0.286
	1/4	**303**	454	0.381	5/16	**291**	436	0.238
	3/16	**227**	341	0.286	1/4	**232**	349	0.190
32	5/16	**361**	541	0.476	3/8	**325**	487	0.286
	1/4	**289**	433	0.381	5/16	**271**	406	0.238
	3/16	**217**	325	0.286	1/4	**217**	325	0.190
30	5/16	**342**	513	0.476	3/8	**301**	452	0.286
	1/4	**273**	410	0.381	5/16	**251**	377	0.238
	3/16	**205**	308	0.286	1/4	**201**	301	0.190
28	5/16	**323**	485	0.476	3/8	**277**	416	0.286
	1/4	**259**	388	0.381	5/16	**231**	347	0.238
	3/16	**194**	291	0.286	1/4	**185**	277	0.190
26	5/16	**305**	457	0.476	3/8	**253**	380	0.286
	1/4	**244**	366	0.381	5/16	**211**	317	0.238
	3/16	**183**	274	0.286	1/4	**169**	253	0.190
24	5/16	**286**	429	0.476	3/8	**229**	344	0.286
	1/4	**229**	343	0.381	5/16	**191**	286	0.238
	3/16	**172**	258	0.286	1/4	**153**	229	0.190
22	5/16	**267**	401	0.476	3/8	**205**	308	0.286
	1/4	**214**	321	0.381	5/16	**171**	256	0.238
	3/16	**160**	241	0.286	1/4	**137**	205	0.190
20	5/16	**248**	372	0.476	3/8	**181**	271	0.286
	1/4	**198**	298	0.381	5/16	**151**	226	0.238
	3/16	**149**	223	0.286	1/4	**121**	181	0.190
18	5/16	**228**	341	0.476	3/8	**157**	235	0.286
	1/4	**182**	273	0.381	5/16	**130**	196	0.238
	3/16	**137**	205	0.286	1/4	**104**	157	0.190
16	5/16	**207**	311	0.476	3/8	**148**	222	0.286
	1/4	**166**	249	0.381	5/16	**123**	185	0.238
	3/16	**124**	186	0.286	1/4	**98.5**	148	0.190

ASD	LRFD	Beam	
$\Omega = 2.00$	$\phi = 0.75$	$F_y = 50$ ksi	$F_u = 65$ ksi

Table 10–3 (continued)
All-Welded Double-Angle Connections



L	Welds A (70 ksi)				Welds B (70 ksi)			
	Weld Size, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	Minimum Web Thickness, in.	Weld Size, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	Minimum Web Thickness, in.
14	5/16	186	279	0.476	3/8	123	185	0.286
	1/4	149	223	0.381	5/16	103	154	0.238
	3/16	111	167	0.286	1/4	82.3	123	0.190
12	5/16	164	246	0.476	3/8	99.3	149	0.286
	1/4	131	197	0.381	5/16	82.8	124	0.238
	3/16	98.3	147	0.286	1/4	66.2	99.3	0.190
10	5/16	140	211	0.476	3/8	75.7	113	0.286
	1/4	112	169	0.381	5/16	63.1	94.6	0.238
	3/16	84.3	126	0.286	1/4	50.4	75.7	0.190
9	5/16	128	193	0.476	3/8	64.2	96.3	0.286
	1/4	103	154	0.381	5/16	53.5	80.2	0.238
	3/16	77.1	116	0.286	1/4	42.8	64.2	0.190
8	5/16	116	174	0.476	3/8	53.0	79.5	0.286
	1/4	92.9	139	0.381	5/16	44.2	66.3	0.238
	3/16	69.6	104	0.286	1/4	35.4	53.0	0.190
7	5/16	103	155	0.476	3/8	42.4	63.6	0.286
	1/4	82.5	124	0.381	5/16	35.3	53.0	0.238
	3/16	61.9	92.9	0.286	1/4	28.3	42.4	0.190
6	5/16	90.3	135	0.476	3/8	32.5	48.7	0.286
	1/4	72.3	108	0.381	5/16	27.0	40.6	0.238
	3/16	54.2	81.3	0.286	1/4	21.6	32.5	0.190
5	5/16	77.1	116	0.476	3/8	23.4	35.1	0.286
	1/4	61.7	92.6	0.381	5/16	19.5	29.2	0.238
	3/16	46.3	69.4	0.286	1/4	15.6	23.4	0.190
4	5/16	64.2	96.3	0.476	3/8	15.5	23.2	0.286
	1/4	51.3	77.0	0.381	5/16	12.9	19.3	0.238
	3/16	38.5	57.8	0.286	1/4	10.3	15.5	0.190

ASD	LRFD	Beam	
$\Omega = 2.00$	$\phi = 0.75$	$F_y = 50$ ksi	$F_u = 65$ ksi

SHEAR END-PLATE CONNECTIONS

A shear end-plate connection is made with a plate length less than the supported beam depth, as illustrated in Figure 10–6. The end plate is always shop-welded to the beam web with fillet welds on each side and usually field-bolted to the supporting member. Welds connecting the end plate to the beam web should not be returned across the thickness of the beam web at the top or bottom of the end plate because of the danger of creating a notch in the beam web.

If the end plate is field-welded to the support, adequate flexibility must be provided in the connection. Line welds are placed along the vertical edges of the plate with a return at the top per AISC Specification Section J2.2b. Note that welding across the entire top of the plate 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 simple shear connections.

Design Checks

The available strength of a shear end-plate connection is determined from the applicable limit states for bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). Note that the limit-state of shear rupture of the beam web must be checked along the length of weld connecting the end plate to the beam web. In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a.

Recommended End-Plate Dimensions and Thickness

To provide for stability during erection, it is recommended that the minimum end-plate length be one-half the T-dimension of the beam to be supported. The maximum length of the end plate must be compatible with the clear distance between the flanges of an uncoped beam and the remaining clear distance of a coped beam.

To provide for flexibility, the combination of plate thickness and gage should be consistent with the recommendations given previously for a double-angle connection of similar thickness and gage.

Shop and Field Practices

When framing to a column web, the associated constructability considerations should be addressed (see the preceding discussion under "Constructability Considerations").

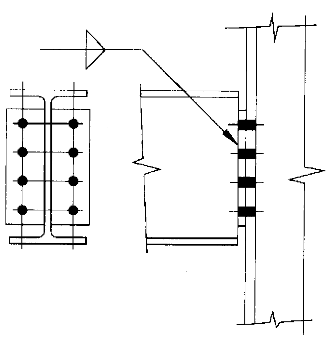


Figure 10–6. Shear end-plate connections.

When framing to a column flange, provision must be made for possible mill variation in the depth of the columns, particularly in fairly long runs (i.e., six or more bays of framing). The beam length can be shortened to provide for mill overrun with shims furnished at the appropriate intervals to fill the resulting gaps or to provide for mill underrun. Shear end-plate connections require close control in cutting the beam to the proper length and in squaring the beam ends such that both end plates are parallel, particularly when beams are cambered.

Table 10–4. Bolted/Welded Shear End-Plate Connections

Table 10–4 is a design aid for shear end-plate connections bolted to the supporting member and welded to the supported beam. Available strengths are tabulated for supported and supporting member material with $F_y = 50$ ksi and $F_u = 65$ ksi and end-plate material with $F_y = 36$ ksi and $F_u = 58$ ksi. Electrode strength is assumed to be 70 ksi. All values, including slip-critical bolt available strengths, are for comparison with the LRFD load combination for LRFD design and the ASD load combination for ASD design.

Tabulated bolt and end-plate available strengths consider the limit-states of bolt shear, bolt bearing on the end plate, shear yielding of the end plate, shear rupture of the end plate, and block shear rupture of the end plate. Values are included for 2 through 12 rows of $^3/_4$-in., $^7/_8$-in., and 1-in. diameter ASTM A325, F1852 and A490 bolts at 3-in. spacing. End-plate edge distances L_{ev} and L_{eh} are assumed to be $1^1/_4$ in.

Tabulated weld available strengths consider the limit-state of weld shear assuming an effective weld length equal to the end-plate length minus twice the weld size. The tabulated minimum beam web thickness matches the shear rupture strength of the web material to the strength of the weld metal. As derived in Part 9, the minimum supported beam web thickness for two lines of weld is

$$t_{min} = \frac{6.19D}{F_u}$$

where D is the number of sixteenths-of-an-inch in the weld size. When less than the minimum material thickness is present, the tabulated weld available strength must be reduced by the ratio of the thickness provided to the minimum thickness.

Tabulated supporting member available strengths, per in. of flange or web thickness, consider the limit-state of bolt bearing.

W44

Table 10–4
Bolted/Welded Shear End-Plate Connections

3/4-in. Bolts
12 Rows

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	197	295	246	369	254	382
	X	—	197	295	246	369	295	443
	SC Class A	STD	177	266	177	266	177	266
		OVS	128	192	128	192	128	192
		SSLT	151	226	151	226	151	226
	SC Class B	STD	197	295	246	369	253	380
		OVS	183	274	183	274	183	274
		SSLT	195	293	215	323	215	323
A490	N	—	197	295	246	369	295	443
	X	—	197	295	246	369	295	443
	SC Class A	STD	197	295	221	332	221	332
		OVS	160	240	160	240	160	240
		SSLT	188	282	188	282	188	282
	SC Class B	STD	197	295	246	369	295	443
		OVS	196	294	229	343	229	343
		SSLT	195	293	244	366	269	403

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
3/16	0.286	196	293	1400	2110
1/4	0.381	260	390		
5/16	0.476	324	486		
3/8	0.571	387	581		

		End-Plate	Beam
STD = Standard holes OVS = Oversized holes SSLT = Short-slotted holes transverse to direction of load	N = Threads included X = Threads excluded SC = Slip critical	F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

3/4-in. Bolts 11 Rows

Table 10–4 (continued) Bolted/Welded Shear End-Plate Connections

W44, 40

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	181	271	226	338	233	350
	X	—	181	271	226	338	271	406
	SC Class A	STD	162	244	162	244	162	244
		OVS	117	176	117	176	117	176
		SSLT	138	207	138	207	138	207
	SC Class B	STD	181	271	226	338	232	348
		OVS	168	251	168	251	168	251
		SSLT	179	269	197	296	197	296
A490	N	—	181	271	226	338	271	406
	X	—	181	271	226	338	271	406
	SC Class A	STD	181	271	203	305	203	305
		OVS	147	220	147	220	147	220
		SSLT	173	259	173	259	173	259
	SC Class B	STD	181	271	226	338	271	406
		OVS	180	269	210	314	210	314
		SSLT	179	269	224	336	247	370

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD
3/16	0.286	179	268
1/4	0.381	238	356
5/16	0.476	296	444
3/8	0.571	354	530

Support Available Strength per Inch Thickness, kips/in.

ASD	LRFD
1290	1930

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

W44, 40, 36

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

3/4-in. Bolts
10 Rows

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	164	246	205	308	212	318
	X	—	164	246	205	308	246	370
	SC Class A	STD	148	221	148	221	148	221
		OVS	107	160	107	160	107	160
		SSLT	126	188	126	188	126	188
	SC Class B	STD	164	246	205	308	211	316
		OVS	152	229	152	229	152	229
		SSLT	163	244	179	269	179	269
A490	N	—	164	246	205	308	246	370
	X	—	164	246	205	308	246	370
	SC Class A	STD	164	246	185	277	185	277
		OVS	133	200	133	200	133	200
		SSLT	157	235	157	235	157	235
	SC Class B	STD	164	246	205	308	246	370
		OVS	163	245	190	286	190	286
		SSLT	163	244	204	306	224	336

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
3/16	0.286	162	243	1170	1760
1/4	0.381	215	323		
5/16	0.476	268	402		
3/8	0.571	320	480		

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

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

3/4-in. Bolts 9 Rows

Table 10–4 (continued) Bolted/Welded Shear End-Plate Connections

W44, 40, 36, 33

Bolt and End-Plate Available Strength, kips								
ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	148	222	185	278	191	286
	X	—	148	222	185	278	222	333
	SC Class A	STD	133	199	133	199	133	199
		OVS	96.0	144	96.0	144	96.0	144
		SSLT	113	169	113	169	113	169
	SC Class B	STD	148	222	185	278	190	285
		OVS	137	206	137	206	137	206
		SSLT	147	220	161	242	161	242
A490	N	—	148	222	185	278	222	333
	X	—	148	222	185	278	222	333
	SC Class A	STD	148	222	166	249	166	249
		OVS	120	180	120	180	120	180
		SSLT	141	212	141	212	141	212
	SC Class B	STD	148	222	185	278	222	333
		OVS	147	221	171	257	171	257
		SSLT	147	220	183	275	202	303

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
3/16	0.286	145	218	1050	1580
1/4	0.381	193	290		
5/16	0.476	240	360		
3/8	0.571	287	430		

		End-Plate	Beam
STD = Standard holes OVS = Oversized holes SSLT = Short-slotted holes transverse to direction of load	N = Threads included X = Threads excluded SC = Slip critical	F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

W44, 40, 36, 33, 30

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

3/4-in. Bolts
8 Rows

Bolt and End-Plate Available Strength, kips								
ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	132	198	165	247	170	254
	X	—	132	198	165	247	198	297
	SC Class A	STD	118	177	118	177	118	177
		OVS	85.3	128	85.3	128	85.3	128
		SSLT	100	151	100	151	100	151
	SC Class B	STD	132	198	165	247	169	253
		OVS	122	183	122	183	122	183
		SSLT	131	196	143	215	143	215
A490	N	—	132	198	165	247	198	297
	X	—	132	198	165	247	198	297
	SC Class A	STD	132	198	148	221	148	221
		OVS	107	160	107	160	107	160
		SSLT	126	188	126	188	126	188
	SC Class B	STD	132	198	165	247	198	297
		OVS	131	197	152	229	152	229
		SSLT	131	196	163	245	179	269

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
3/16	0.286	129	193	936	1400
1/4	0.381	171	256		
5/16	0.476	212	318		
3/8	0.571	253	380		

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

3/4-in. Bolts 7 Rows

Table 10–4 (continued) Bolted/Welded Shear End-Plate Connections

W44, 40, 36, 33, 30, 27, 24

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD
A325/F1852	N	—	116	174	145	217	148	223
	X	—	116	174	145	217	174	260
	SC Class A	STD	103	155	103	155	103	155
		OVS	74.7	112	74.7	112	74.7	112
		SSLT	87.9	132	87.9	132	87.9	132
	SC Class B	STD	116	174	145	217	148	221
		OVS	107	160	107	160	107	160
		SSLT	114	172	126	188	126	188
A490	N	—	116	174	145	217	174	260
	X	—	116	174	145	217	174	260
	SC Class A	STD	116	174	129	194	129	194
		OVS	93.3	140	93.3	140	93.3	140
		SSLT	110	165	110	165	110	165
	SC Class B	STD	116	174	145	217	174	260
		OVS	115	172	133	200	133	200
		SSLT	114	172	143	214	157	235

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD
3/16	0.286	112	168
1/4	0.381	148	223
5/16	0.476	184	277
3/8	0.571	220	330

Support Available Strength per Inch Thickness, kips/in.

ASD	LRFD
819	1230

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

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

W44, 40, 36, 33, 30, 27, 24, 21

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

3/4-in. Bolts
6 Rows

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	99.5	149	124	187	127	191
	X	—	99.5	149	124	187	149	224
	SC Class A	STD	88.6	133	88.6	133	88.6	133
		OVS	64.0	96.0	64.0	96.0	64.0	96.0
		SSLT	75.3	113	75.3	113	75.3	113
	SC Class B	STD	99.5	149	124	187	127	190
		OVS	91.4	137	91.4	137	91.4	137
		SSLT	98.2	147	108	161	108	161
A490	N	—	99.5	149	124	187	149	224
	X	—	99.5	149	124	187	149	224
	SC Class A	STD	99.5	149	111	166	111	166
		OVS	80.0	120	80.0	120	80.0	120
		SSLT	94.1	141	94.1	141	94.1	141
	SC Class B	STD	99.5	149	124	187	149	224
		OVS	98.6	148	114	171	114	171
		SSLT	98.2	147	123	184	134	202

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD
3/16	0.286	95.4	143
1/4	0.381	126	189
5/16	0.476	157	235
3/8	0.571	187	280

Support Available Strength per Inch Thickness, kips/in.

ASD	LRFD
702	1050

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

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

3/4-in. Bolts 5 Rows

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

W30, 27, 24, 21, 18

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	83.3	125	104	156	106	159
	X	—	83.3	125	104	156	125	187
	SC Class A	STD	73.8	111	73.8	111	73.8	111
		OVS	53.3	80.0	53.3	80.0	53.3	80.0
		SSLT	62.8	94.1	62.8	94.1	62.8	94.1
	SC Class B	STD	83.3	125	104	156	105	158
		OVS	76.2	114	76.2	114	76.2	114
		SSLT	82	123	89.6	134	89.6	134
A490	N	—	83.3	125	104	156	125	187
	X	—	83.3	125	104	156	125	187
	SC Class A	STD	83.3	125	92.3	138	92.3	138
		OVS	66.7	100	66.7	100	66.7	100
		SSLT	78.4	118	78.4	118	78.4	118
	SC Class B	STD	83.3	125	104	156	125	187
		OVS	82.4	124	95.2	143	95.2	143
		SSLT	82	123	102	154	112	168

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD
3/16	0.286	78.7	118
1/4	0.381	104	156
5/16	0.476	129	193
3/8	0.571	153	230

Support Available Strength per Inch Thickness, kips/in.

ASD	LRFD
585	878

End-Plate	Beam
F_y = 36 ksi, F_u = 58 ksi	F_y = 50 ksi, F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

W24, 21, 18, 16

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

3/4-in. Bolts
4 Rows

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	67.1	101	83.9	126	84.8	127
	X	—	67.1	101	83.9	126	101	151
	SC Class A	STD	59.1	88.6	59.1	88.6	59.1	88.6
		OVS	42.7	64.0	42.7	64.0	42.7	64.0
		SSLT	50.2	75.3	50.2	75.3	50.2	75.3
	SC Class B	STD	67.1	101	83.9	126	84.4	127
		OVS	61.0	91.4	61.0	91.4	61.0	91.4
		SSLT	65.8	98.7	71.7	108	71.7	108
A490	N	—	67.1	101	83.9	126	101	151
	X	—	67.1	101	83.9	126	101	151
	SC Class A	STD	67.1	101	73.8	111	73.8	111
		OVS	53.3	80.0	53.3	80.0	53.3	80.0
		SSLT	62.8	94.1	62.8	94.1	62.8	94.1
	SC Class B	STD	67.1	101	83.9	126	101	151
		OVS	65.3	97.9	76.2	114	76.2	114
		SSLT	65.8	98.7	82.2	123	89.6	134

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips
		ASD	LRFD
3/16	0.286	61.9	92.9
1/4	0.381	81.7	123
5/16	0.476	101	151
3/8	0.571	120	180

Support Available Strength per Inch Thickness, kips/in.

ASD	LRFD
468	702

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

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

3/4-in. Bolts 3 Rows

Table 10–4 (continued) Bolted/Welded Shear End-Plate Connections

W18, 16, 14, 12, 10*

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	50.9	76.4	63.6	95.4	63.6	95.4
	X	—	50.9	76.4	63.7	95.5	76.4	115.0
	SC Class A	STD	44.3	66.4	44.3	66.4	44.3	66.4
		OVS	32.0	48.0	32.0	48.0	32.0	48.0
		SSLT	37.7	56.5	37.7	56.5	37.7	56.5
	SC Class B	STD	50.9	76.4	63.3	94.9	63.3	94.9
		OVS	45.7	68.6	45.7	68.6	45.7	68.6
		SSLT	49.6	74.4	53.8	80.7	53.8	80.7
A490	N	—	50.9	76.4	63.7	95.5	76.4	115.0
	X	—	50.9	76.4	63.7	95.5	76.4	115.0
	SC Class A	STD	50.9	76.4	55.4	83.1	55.4	83.1
		OVS	40.0	60.0	40.0	60.0	40.0	60.0
		SSLT	47.1	70.6	47.1	70.6	47.1	70.6
	SC Class B	STD	50.9	76.4	63.7	95.5	76.4	115.0
		OVS	47.9	71.8	57.1	85.7	57.1	85.7
		SSLT	49.8	74.4	62.0	92.9	67.2	101.0

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
3/16	0.286	45.2	67.9	351	526
1/4	0.381	59.4	89.1		
5/16	0.476	73.1	110		
3/8	0.571	88.3	129		

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical
*Limited to W10×12, 15,17, 19, 22, 26, 30

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

W12, 10, 8

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

3/4-in. Bolts
2 Rows

Bolt and End-Plate Available Strength, kips									
ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	32.6	48.9	40.8	61.2	42.4	63.6
	X	—	32.6	48.9	40.8	61.2	48.9	73.4
	SC Class A	STD	29.5	44.3	29.5	44.3	29.5	44.3
		OVS	21.3	32.0	21.3	32.0	21.3	32.0
		SSLT	25.1	37.7	25.1	37.7	25.1	37.7
	SC Class B	STD	32.6	48.9	40.8	61.2	42.2	63.3
		OVS	30.5	45.7	30.5	45.7	30.5	45.7
		SSLT	32.6	48.9	35.9	53.8	35.9	53.8
A490	N	—	32.6	48.9	40.8	61.2	48.9	73.4
	X	—	32.6	48.9	40.8	61.2	48.9	73.4
	SC Class A	STD	32.6	48.9	36.9	55.4	36.9	55.4
		OVS	26.7	40.0	26.7	40.0	26.7	40.0
		SSLT	31.4	47.1	31.4	47.1	31.4	47.1
	SC Class B	STD	32.6	48.9	40.8	61.2	48.9	73.4
		OVS	30.5	45.7	38.1	57.1	38.1	57.1
		SSLT	32.6	48.9	40.8	61.2	44.8	67.2

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
3/16	0.286	28.5	42.8	234	351
1/4	0.381	37.1	55.7		
5/16	0.476	45.2	67.9		
3/8	0.571	52.9	79.4		
				End-Plate	Beam
				F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

7/8-in. Bolts 12 Rows

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

W44

Bolt and End-Plate Available Strength, kips								
ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	196	294	245	367	294	441
	X	—	196	294	245	367	294	441
	SC Class A	STD	196	294	245	367	247	370
		OVS	178	267	178	267	178	267
		SSLT	194	292	210	315	210	315
	SC Class B	STD	196	294	245	367	294	441
		OVS	191	287	239	359	255	382
		SSLT	194	292	243	365	292	438
A490	N	—	196	294	245	367	294	441
	X	—	196	294	245	367	294	441
	SC Class A	STD	196	294	245	367	294	441
		OVS	191	287	224	336	224	336
		SSLT	194	292	243	365	264	395
	SC Class B	STD	196	294	245	367	294	441
		OVS	191	287	239	359	287	431
		SSLT	194	292	243	365	292	438

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
3/16	0.286	196	293	1640	2460
1/4	0.381	260	390		
5/16	0.476	324	486		
3/8	0.571	387	581		

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

W44, 40

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

7/8-in. Bolts
11 Rows

Bolt and End-Plate Available Strength, kips								
ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	180	269	225	337	269	404
	X	—	180	269	225	337	269	404
	SC Class A	STD	180	269	225	337	226	339
		OVS	163	245	163	245	163	245
		SSLT	178	267	192	288	192	288
	SC Class B	STD	180	269	225	337	269	404
		OVS	175	263	219	328	233	350
		SSLT	178	267	223	334	267	401
A490	N	—	180	269	225	337	269	404
	X	—	180	269	225	337	269	404
	SC Class A	STD	180	269	225	337	269	404
		OVS	175	263	205	308	205	308
		SSLT	178	267	223	334	242	362
	SC Class B	STD	180	269	225	337	269	404
		OVS	175	263	219	328	263	394
		SSLT	178	267	223	334	267	401

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
3/16	0.286	179	268	1500	2250
1/4	0.381	238	356		
5/16	0.476	296	444		
3/8	0.571	354	530		

	End-Plate	Beam
STD = Standard holes OVS = Oversized holes SSLT = Short-slotted holes transverse to direction of load N = Threads included X = Threads excluded SC = Slip critical	F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

7/8-in. Bolts 10 Rows

Table 10–4 (continued) Bolted/Welded Shear End-Plate Connections

W44, 40, 36

Bolt and End-Plate Available Strength, kips								
ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	163	245	204	306	245	368
	X	—	163	245	204	306	245	368
	SC Class A	STD	163	245	204	306	245	308
		OVS	149	223	149	223	149	223
		SSLT	162	243	175	262	175	262
	SC Class B	STD	163	245	204	306	245	368
		OVS	159	238	198	298	212	318
		SSLT	162	243	203	304	243	365
A490	N	—	163	245	204	306	245	368
	X	—	163	245	204	306	245	368
	SC Class A	STD	163	245	204	306	245	368
		OVS	159	238	187	280	187	280
		SSLT	162	243	203	304	220	329
	SC Class B	STD	163	245	204	306	245	368
		OVS	159	238	198	298	238	357
		SSLT	162	243	203	304	243	365

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
3/16	0.286	162	243	1370	2050
1/4	0.381	215	323		
5/16	0.476	268	402		
3/8	0.571	320	480		
				End-Plate	Beam
				F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

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

W44, 40, 36, 33

Table 10–4 (continued) Bolted/Welded Shear End-Plate Connections

7/8-in. Bolts
9 Rows

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	147	221	184	276	221	331
	X	—	147	221	184	276	221	331
	SC Class A	STD	147	221	184	276	185	278
		OVS	134	201	134	201	134	201
		SSLT	146	219	157	236	157	236
	SC Class B	STD	147	221	184	276	221	331
		OVS	142	214	178	267	191	287
		SSLT	146	219	182	273	219	328
A490	N	—	147	221	184	276	221	331
	X	—	147	221	184	276	221	331
	SC Class A	STD	147	221	184	276	221	331
		OVS	142	214	168	252	168	252
		SSLT	146	219	182	273	198	297
	SC Class B	STD	147	221	184	276	221	331
		OVS	142	214	178	267	214	321
		SSLT	146	219	182	273	219	328

Weld and Beam Web Available Strength, kips / **Support Available Strength per Inch Thickness, kips/in.**

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips	Support ASD	Support LRFD
		ASD	LRFD	ASD	LRFD
3/16	0.286	145	218	1230	1840
1/4	0.381	193	290		
5/16	0.476	240	360		
3/8	0.571	287	430		

End-Plate	Beam
F_y = 36 ksi, F_u = 58 ksi	F_y = 50 ksi, F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

7/8-in. Bolts 8 Rows

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

W44, 40, 36, 33, 30

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	131	197	164	246	197	295
	X	—	131	197	164	246	197	295
	SC Class A	STD	131	197	164	246	165	247
		OVS	119	178	119	178	119	178
		SSLT	130	194	140	210	140	210
	SC Class B	STD	131	197	164	246	197	295
		OVS	126	189	158	237	170	255
		SSLT	130	194	162	243	194	292
A490	N	—	131	197	164	246	197	295
	X	—	131	197	164	246	197	295
	SC Class A	STD	131	197	164	246	197	295
		OVS	126	189	149	224	149	224
		SSLT	130	194	162	243	176	264
	SC Class B	STD	131	197	164	246	197	295
		OVS	126	189	158	237	189	284
		SSLT	130	194	162	243	194	292

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
3/16	0.286	129	193	1090	1640
1/4	0.381	171	256		
5/16	0.476	212	318		
3/8	0.571	253	380		
				End-Plate	**Beam**
				F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

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

W44, 40, 36, 33, 30, 27, 24

Table 10–4 (continued) Bolted/Welded Shear End-Plate Connections

7/8-in. Bolts 7 Rows

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	115	172	144	215	172	258
	X	—	115	172	144	215	172	258
	SC Class A	STD	115	172	144	215	144	216
		OVS	104	156	104	156	104	156
		SSLT	113	170	122	184	122	184
	SC Class B	STD	115	172	144	215	172	258
		OVS	110	165	137	206	149	223
		SSLT	113	170	142	213	170	255
A490	N	—	115	172	144	215	172	258
	X	—	115	172	144	215	172	258
	SC Class A	STD	115	172	144	215	172	258
		OVS	110	165	131	196	131	196
		SSLT	113	170	142	213	154	231
	SC Class B	STD	115	172	144	215	172	258
		OVS	110	165	137	206	165	247
		SSLT	113	170	142	213	170	255

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
3/16	0.286	112	168	956	1430
1/4	0.381	148	223		
5/16	0.476	184	277		
3/8	0.571	220	330		

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

7/8-in. Bolts 6 Rows

Table 10–4 (continued) Bolted/Welded Shear End-Plate Connections

W40, 36, 33, 30, 27, 24, 21

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	98.6	148	123	185	148	222
	X	—	98.6	148	123	185	148	222
	SC Class A	STD	98.6	148	123	185	123	185
		OVS	89.2	134	89.2	134	89.2	134
		SSLT	97.3	146	105	157	105	157
	SC Class B	STD	98.6	148	123	185	148	222
		OVS	93.5	140	117	175	127	191
		SSLT	97.3	146	122	182	146	219
A490	N	—	98.6	148	123	185	148	222
	X	—	98.6	148	123	185	148	222
	SC Class A	STD	98.6	148	123	185	148	222
		OVS	93.5	140	112	168	112	168
		SSLT	97.3	146	122	182	132	198
	SC Class B	STD	98.6	148	123	185	148	222
		OVS	93.5	140	117	175	140	210
		SSLT	97.3	146	122	182	146	219

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips
		ASD	LRFD
3/16	0.286	95.4	143
1/4	0.381	126	189
5/16	0.476	157	235
3/8	0.571	187	280

Support Available Strength per Inch Thickness, kips/in.

ASD	LRFD
819	1230

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

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

W30, 27, 24, 21, 18

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

7/8-in. Bolts
5 Rows

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD
A325/F1852	N	—	82.4	124	103	155	124	185
	X	—	82.4	124	103	155	124	185
	SC Class A	STD	82.4	124	103	154	103	154
		OVS	74.3	111	74.3	111	74.3	111
		SSLT	81.1	122	87.4	131	87.4	131
	SC Class B	STD	82.4	124	103	155	124	185
		OVS	77.2	116	96.5	145	106	159
		SSLT	81.1	122	101	152	122	182
A490	N	—	82.4	124	103	155	124	185
	X	—	82.4	124	103	155	124	185
	SC Class A	STD	82.4	124	103	155	124	185
		OVS	77.2	116	93.3	140	93.3	140
		SSLT	81.1	122	101	152	110	165
	SC Class B	STD	82.4	124	103	155	124	185
		OVS	77.2	116	96.5	145	116	174
		SSLT	81.1	122	101	152	122	182

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD
3/16	0.286	78.7	118
1/4	0.381	104	156
5/16	0.476	193	193
3/8	0.571	153	230

Support Available Strength per Inch Thickness, kips/in.

ASD	LRFD
683	1020

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

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

7/8-in. Bolts 4 Rows

Table 10–4 (continued) Bolted/Welded Shear End-Plate Connections

W24, 21, 18, 16

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD
A325/F1852	N	—	65.3	97.9	81.6	122	97.9	147
	X	—	65.3	97.9	81.6	122	97.9	147
	SC Class A	STD	65.3	97.9	81.6	122	82.3	123
		OVS	59.4	89.2	59.4	89.2	59.4	89.2
		SSLT	64.9	97.3	69.9	105	69.9	105
	SC Class B	STD	65.3	97.9	81.6	122	97.9	147
		OVS	60.9	91.4	76.1	114	84.9	127
		SSLT	64.9	97.3	81.1	122	97.3	146
A490	N	—	65.3	97.9	81.6	122	97.9	147
	X	—	65.3	97.9	81.6	122	97.9	147
	SC Class A	STD	65.3	97.9	81.6	122	97.9	147
		OVS	60.9	91.4	74.7	112	74.7	112
		SSLT	64.9	97.3	81.1	122	87.9	132
	SC Class B	STD	65.3	97.9	81.6	122	97.9	147
		OVS	60.9	91.4	76.1	114	91.4	137
		SSLT	64.9	97.3	81.1	122	97.3	146

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD
3/16	0.286	61.9	92.9
1/4	0.381	81.7	123
5/16	0.476	101	151
3/8	0.571	120	180

Support Available Strength per Inch Thickness, kips/in.

ASD	LRFD
546	819

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

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

W18, 16, 14, 12, 10*

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

7/8-in. Bolts 3 Rows

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	47.9	71.8	59.8	89.7	71.8	108
	X	—	47.9	71.8	59.8	89.7	71.8	108
	SC Class A	STD	47.9	71.8	59.8	89.7	61.7	92.5
		OVS	44.6	66.9	44.6	66.9	44.6	66.9
		SSLT	47.9	71.8	52.4	78.7	52.4	78.7
	SC Class B	STD	47.9	71.8	59.8	89.7	71.8	108
		OVS	44.6	66.9	55.7	83.6	63.7	95.5
		SSLT	47.9	71.8	59.8	89.7	71.8	108
A490	N	—	47.9	71.8	59.8	89.7	71.8	108
	X	—	47.9	71.8	59.8	89.7	71.8	108
	SC Class A	STD	47.9	71.8	59.8	89.7	71.8	108
		OVS	44.6	66.9	55.7	83.6	56.0	84.0
		SSLT	47.9	71.8	59.8	89.7	65.9	98.8
	SC Class B	STD	47.9	71.8	59.8	89.7	71.8	108
		OVS	44.6	66.9	55.7	83.6	66.9	100
		SSLT	47.9	71.8	59.8	89.7	71.8	108

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
3/16	0.286	45.2	67.9	409	614
1/4	0.381	59.4	89.1		
5/16	0.476	73.1	110		
3/8	0.571	86.3	129		

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical
*Limited to W10×12, 15, 17, 19, 22, 26, 30

7/8-in. Bolts 2 Rows

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

W12, 10, 8

Bolt and End-Plate Available Strength, kips								
ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	30.5	45.7	38.1	57.1	45.7	68.5
	X	—	30.5	45.7	38.1	57.1	45.7	68.5
	SC Class A	STD	30.5	45.7	38.1	57.1	41.1	61.7
		OVS	28.3	42.4	29.7	44.6	29.7	44.6
		SSLT	30.5	45.7	35.0	52.4	35.0	52.4
	SC Class B	STD	30.5	45.7	38.1	57.1	45.7	68.5
		OVS	28.3	42.4	35.3	53.0	42.4	63.6
		SSLT	30.5	45.7	38.1	57.1	45.7	68.5
A490	N	—	30.5	45.7	38.1	57.1	45.7	68.5
	X	—	30.5	45.7	38.1	57.1	45.7	68.5
	SC Class A	STD	30.5	45.7	38.1	57.1	45.7	68.5
		OVS	28.3	42.4	35.3	53.0	37.3	56.0
		SSLT	30.5	45.7	38.1	57.1	43.9	65.9
	SC Class B	STD	30.5	45.7	38.1	57.1	45.7	68.5
		OVS	28.3	42.4	35.3	53.0	42.4	63.6
		SSLT	30.5	45.7	38.1	57.1	45.7	68.5

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
3/16	0.286	28.5	42.8	273	409
1/4	0.381	37.1	55.7		
5/16	0.476	45.2	67.9		
3/8	0.571	52.9	79.4		

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

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

W44

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

1-in. Bolts
12 Rows

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	191	287	239	359	287	431
	X	—	191	287	239	359	287	431
	SC Class A	STD	191	287	239	359	287	431
		OVS	172	258	215	322	233	350
		SSLT	191	287	239	359	274	411
	SC Class B	STD	191	287	239	359	287	431
		OVS	172	258	215	322	258	387
		SSLT	191	287	239	359	287	431
A490	N	—	191	287	239	359	287	431
	X	—	191	287	239	359	287	431
	SC Class A	STD	191	287	239	359	287	431
		OVS	172	258	215	322	258	387
		SSLT	191	287	239	359	287	431
	SC Class B	STD	191	287	239	359	287	431
		OVS	172	258	215	322	258	387
		SSLT	191	287	239	359	287	431

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips
		ASD	LRFD
3/16	0.286	196	293
1/4	0.381	260	390
5/16	0.476	324	486
3/8	0.571	387	581

Support Available Strength per Inch Thickness, kips/in.

ASD	LRFD
1820 STD/SSLT	2730 STD/SSLT
1660 OVS	2490 OVS

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

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

1-in. Bolts 11 Rows

Table 10–4 (continued) Bolted/Welded Shear End-Plate Connections

W44, 40

Bolt and End-Plate Available Strength, kips								
ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	175	263	219	328	263	394
	X	—	175	263	219	328	263	394
	SC Class A	STD	175	263	219	328	263	394
		OVS	157	236	196	295	214	321
		SSLT	175	263	219	328	251	377
	SC Class B	STD	175	263	219	328	263	394
		OVS	157	236	196	295	236	354
		SSLT	175	263	219	328	263	394
A490	N	—	175	263	219	328	263	394
	X	—	175	263	219	328	263	394
	SC Class A	STD	175	263	219	328	263	394
		OVS	157	236	196	295	236	354
		SSLT	175	263	219	328	263	394
	SC Class B	STD	175	263	219	328	263	394
		OVS	157	236	196	295	236	354
		SSLT	175	263	219	328	263	394

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
3/16	0.286	179	268	1670 STD/SSLT	2500 STD/SSLT
1/4	0.381	238	356		
5/16	0.476	296	444	1520 OVS	2280 OVS
3/8	0.571	354	530		

		End-Plate	Beam
STD = Standard holes OVS = Oversized holes SSLT = Short-slotted holes transverse to direction of load	N = Threads included X = Threads excluded SC = Slip critical	F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

W44, 40, 36

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

1-in. Bolts 10 Rows

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	159	238	198	298	238	357
	X	—	159	238	198	298	238	357
	SC Class A	STD	159	238	198	298	238	357
		OVS	142	214	178	267	194	291
		SSLT	159	238	198	298	229	343
	SC Class B	STD	159	238	198	298	238	357
		OVS	142	214	178	267	214	321
		SSLT	159	238	198	298	238	357
A490	N	—	159	238	198	298	238	357
	X	—	159	238	198	298	238	357
	SC Class A	STD	159	238	198	298	238	357
		OVS	142	214	178	267	214	321
		SSLT	159	238	198	298	238	357
	SC Class B	STD	159	238	198	298	238	357
		OVS	142	214	178	267	214	321
		SSLT	159	238	198	298	238	357

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD
3/16	0.286	162	243
1/4	0.381	215	323
5/16	0.476	268	402
3/8	0.571	320	480

Support Available Strength per Inch Thickness, kips/in.

ASD	LRFD
1520 STD/SSLT	2270 STD/SSLT
1380 OVS	2080 OVS

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

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

1-in. Bolts 9 Rows

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

W44, 40, 36, 33

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	142	214	178	267	214	321
	X	—	142	214	178	267	214	321
	SC Class A	STD	142	214	178	267	214	321
		OVS	128	192	160	240	175	262
		SSLT	142	214	178	267	206	309
	SC Class B	STD	142	214	178	267	214	321
		OVS	128	192	160	240	192	288
		SSLT	142	214	178	267	214	321
A490	N	—	142	214	178	267	214	321
	X	—	142	214	178	267	214	321
	SC Class A	STD	142	214	178	267	214	321
		OVS	128	192	160	240	192	288
		SSLT	142	214	178	267	214	321
	SC Class B	STD	142	214	178	267	214	321
		OVS	128	192	160	240	192	288
		SSLT	142	214	178	267	214	321

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips
		ASD	LRFD
3/16	0.286	145	218
1/4	0.381	193	290
5/16	0.476	240	360
3/8	0.571	287	430

Support Available Strength per Inch Thickness, kips/in.

ASD		LRFD	
1370	STD/SSLT	2050	STD/SSLT
1250	OVS	1870	OVS

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

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

W44, 40, 36, 33, 30

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

1-in. Bolts 8 Rows

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			$^1/_4$		$^5/_{16}$		$^3/_8$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	126	189	158	237	189	284
	X	—	126	189	158	237	189	284
	SC Class A	STD	126	189	158	237	189	284
		OVS	113	170	141	212	155	233
		SSLT	126	189	158	237	183	274
	SC Class B	STD	126	189	158	237	189	284
		OVS	113	170	141	212	170	254
		SSLT	126	189	158	237	189	284
A490	N	—	126	189	158	237	189	284
	X	—	126	189	158	237	189	284
	SC Class A	STD	126	189	158	237	189	284
		OVS	113	170	141	212	170	254
		SSLT	126	189	158	237	189	284
	SC Class B	STD	126	189	158	237	189	284
		OVS	113	170	141	212	170	254
		SSLT	126	189	158	237	189	284

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD
$^3/_{16}$	0.286	129	193
$^1/_4$	0.381	171	256
$^5/_{16}$	0.476	212	318
$^3/_8$	0.571	253	380

Support Available Strength per Inch Thickness, kips/in.

ASD	LRFD
1210 STD/SSLT	1820 STD/SSLT
1110 OVS	1670 OVS

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

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

1-in. Bolts 7 Rows

Table 10–4 (continued) Bolted/Welded Shear End-Plate Connections

W44, 40, 36, 33, 30, 27, 24

Bolt and End-Plate Available Strength, kips								
ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	110	165	137	206	165	247
	X	—	110	165	137	206	165	247
	SC Class A	STD	110	165	137	206	165	247
		OVS	98.4	148	123	185	136	204
		SSLT	110	165	137	206	160	240
	SC Class B	STD	110	165	137	206	165	247
		OVS	98.4	148	123	185	148	221
		SSLT	110	165	137	206	165	247
A490	N	—	110	165	137	206	165	247
	X	—	110	165	137	206	165	247
	SC Class A	STD	110	165	137	206	165	247
		OVS	98.4	148	123	185	148	221
		SSLT	110	165	137	206	165	247
	SC Class B	STD	110	165	137	206	165	247
		OVS	98.4	148	123	185	148	221
		SSLT	110	165	137	206	165	247

Weld and Beam Web Available Strength, kips			
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD
3/16	0.286	112	168
1/4	0.381	148	223
5/16	0.476	184	277
3/8	0.571	220	330

Support Available Strength per Inch Thickness, kips/in.	
ASD	LRFD
1060 STD/SSLT	1590 STD/SSLT
975 OVS	1460 OVS

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

W40, 36, 33, 30, 27, 24, 21

Table 10–4 (continued) Bolted/Welded Shear End-Plate Connections

1-in. Bolts 6 Rows

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in. $^{1}/_{4}$ ASD	$^{1}/_{4}$ LRFD	$^{5}/_{16}$ ASD	$^{5}/_{16}$ LRFD	$^{3}/_{8}$ ASD	$^{3}/_{8}$ LRFD
A325/F1852	N	—	93.5	140	117	175	140	210
	X	—	93.5	140	117	175	140	210
	SC Class A	STD	93.5	140	117	175	140	210
		OVS	83.7	126	105	157	117	175
		SSLT	93.5	140	117	175	137	206
	SC Class B	STD	93.5	140	117	175	140	210
		OVS	83.7	126	105	157	126	188
		SSLT	93.5	140	117	175	140	210
A490	N	—	93.5	140	117	175	140	210
	X	—	93.5	140	117	175	140	210
	SC Class A	STD	93.5	140	117	175	140	210
		OVS	83.7	126	105	157	126	188
		SSLT	93.5	140	117	175	140	210
	SC Class B	STD	93.5	140	117	175	140	210
		OVS	83.7	126	105	157	126	188
		SSLT	93.5	140	117	175	140	210

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD
$^{3}/_{16}$	0.286	95.4	143
$^{1}/_{4}$	0.381	126	189
$^{5}/_{16}$	0.476	157	235
$^{3}/_{8}$	0.571	187	280

Support Available Strength per Inch Thickness, kips/in.

ASD		LRFD	
912	STD/SSLT	1370	STD/SSLT
839	OVS	1260	OVS

End-Plate	Beam
F_y = 36 ksi, F_u = 58 ksi	F_y = 50 ksi, F_u = 65 ksi

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

1-in. Bolts 5 Rows

Table 10–4 (continued) Bolted/Welded Shear End-Plate Connections

W30, 27, 24, 21, 18

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	77.2	116	96.5	145	116	174
	X	—	77.2	116	96.5	145	116	174
	SC Class A	STD	77.2	116	96.5	145	116	174
		OVS	69.1	104	86.3	129	97.2	146
		SSLT	77.2	116	96.5	145	114	171
	SC Class B	STD	77.2	116	96.5	145	116	174
		OVS	69.1	104	86.3	129	104	155
		SSLT	77.2	116	96.5	145	116	174
A490	N	—	77.2	116	96.5	145	116	174
	X	—	77.2	116	96.5	145	116	174
	SC Class A	STD	77.2	116	96.5	145	116	174
		OVS	69.1	104	86.3	129	104	155
		SSLT	77.2	116	96.5	145	116	174
	SC Class B	STD	77.2	116	96.5	145	116	174
		OVS	69.1	104	86.3	129	104	155
		SSLT	77.2	116	96.5	145	116	174

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips
		ASD	LRFD
3/16	0.286	78.7	118
1/4	0.381	104	156
5/16	0.476	129	193
3/8	0.571	153	230

Support Available Strength per Inch Thickness, kips/in.

ASD		LRFD	
761	STD/SSLT	1140	STD/SSLT
702	OVS	1050	OVS

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

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

W24, 21, 18, 16

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

1-in. Bolts 4 Rows

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	60.9	91.4	76.1	114	91.4	137
	X	—	60.9	91.4	76.1	114	91.4	137
	SC Class A	STD	60.9	91.4	76.1	114	91.4	137
		OVS	54.4	81.6	68.0	102	77.7	117
		SSLT	60.9	91.4	76.1	114	91.4	137
	SC Class B	STD	60.9	91.4	76.1	114	91.4	137
		OVS	54.4	81.6	68.0	102	81.6	122
		SSLT	60.9	91.4	76.1	114	91.4	137
A490	N	—	60.9	91.4	76.1	114	91.4	137
	X	—	60.9	91.4	76.1	114	91.4	137
	SC Class A	STD	60.9	91.4	76.1	114	91.4	137
		OVS	54.4	81.6	68.0	102	81.6	122
		SSLT	60.9	91.4	76.1	114	91.4	137
	SC Class B	STD	60.9	91.4	76.1	114	91.4	137
		OVS	54.4	81.6	68.0	102	81.6	122
		SSLT	60.9	91.4	76.1	114	91.4	137

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips
		ASD	LRFD
3/16	0.286	61.9	92.9
1/4	0.381	81.7	123
5/16	0.476	101	151
3/8	0.571	120	180

Support Available Strength per Inch Thickness, kips/in.

ASD		LRFD	
609	STD/SSLT	914	STD/SSLT
566	OVS	848	OVS

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

1-in. Bolts 3 Rows

Table 10–4 (continued) Bolted/Welded Shear End-Plate Connections

W18, 16, 14, 12, 10*

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			$^1/_4$		$^5/_{16}$		$^3/_8$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	44.6	66.9	55.7	83.6	66.9	100
	X	—	44.6	66.9	55.7	83.6	66.9	100
	SC Class A	STD	44.6	66.9	55.7	83.6	66.9	100
		OVS	39.7	59.5	49.6	74.4	58.3	87.4
		SSLT	44.6	66.9	55.7	83.6	66.9	100
	SC Class B	STD	44.6	66.9	55.7	83.6	66.9	100
		OVS	39.7	59.5	49.6	74.4	59.5	89.3
		SSLT	44.6	66.9	55.7	83.6	66.9	100
A490	N	—	44.6	66.9	55.7	83.6	66.9	100
	X	—	44.6	66.9	55.7	83.6	66.9	100
	SC Class A	STD	44.6	66.9	55.7	83.6	66.9	100
		OVS	39.7	59.5	49.6	74.4	59.5	89.3
		SSLT	44.6	66.9	55.7	83.6	66.9	100
	SC Class B	STD	44.6	66.9	55.7	83.6	66.9	100
		OVS	39.7	59.5	49.6	74.4	59.5	89.3
		SSLT	44.6	66.9	55.7	83.6	66.9	100

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD
$^3/_{16}$	0.286	45.2	67.9
$^1/_4$	0.381	59.4	89.1
$^5/_{16}$	0.476	73.1	110
$^3/_8$	0.571	86.3	129

Support Available Strength per Inch Thickness, kips/in.

ASD		LRFD	
458	STD/SSLT	687	STD/SSLT
429	OVS	644	OVS

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical
*Limited to W10×12, 15, 17, 19, 22, 26, 30

W12, 10, 8

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

1-in. Bolts 2 Rows

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	28.3	42.4	35.3	53.0	42.4	63.6
	X	—	28.3	42.4	35.3	53.0	42.4	63.6
	SC Class A	STD	28.3	42.4	35.3	53.0	42.4	63.6
		OVS	25.0	37.5	31.3	46.9	37.5	56.3
		SSLT	28.3	42.4	35.3	53.0	42.4	63.6
	SC Class B	STD	28.3	42.4	35.3	53.0	42.4	63.6
		OVS	25.0	37.5	31.3	46.9	37.5	56.3
		SSLT	28.3	42.4	35.3	53.0	42.4	63.6
A490	N	—	28.3	42.4	35.3	53.0	42.4	63.6
	X	—	28.3	42.4	35.3	53.0	42.4	63.6
	SC Class A	STD	28.3	42.4	35.3	53.0	42.4	63.6
		OVS	25.3	37.5	31.3	46.9	37.5	56.3
		SSLT	28.3	42.4	35.3	53.0	42.4	63.6
	SC Class B	STD	28.3	42.4	35.3	53.0	42.4	63.6
		OVS	25.0	37.5	31.3	46.9	37.5	56.3
		SSLT	28.3	42.4	35.3	53.0	42.4	63.6

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD
3/16	0.286	28.5	42.8
1/4	0.381	37.1	55.7
5/16	0.476	45.2	67.9
3/8	0.571	52.9	79.4

Support Available Strength per Inch Thickness, kips/in.

ASD		LRFD	
307	STD/SSLT	461	STD/SSLT
293	OVS	439	OVS

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

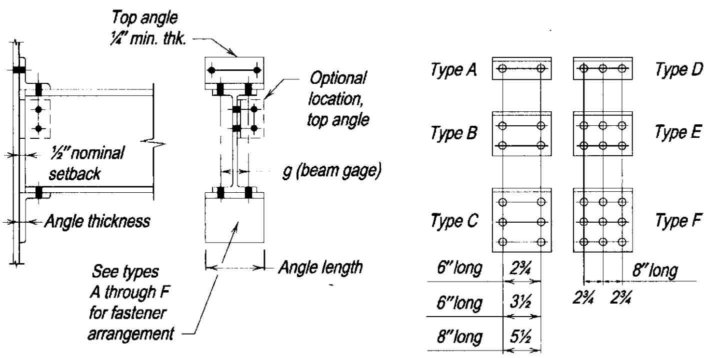
N = Threads included
X = Threads excluded
SC = Slip critical

UNSTIFFENED SEATED CONNECTIONS

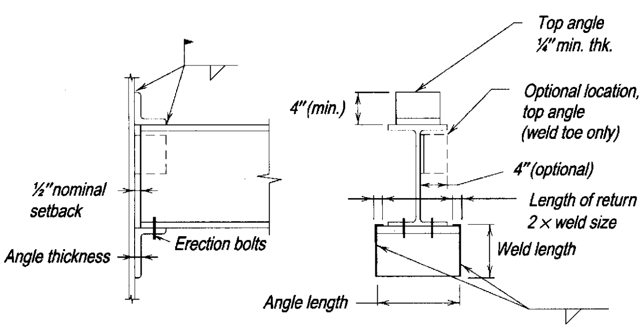
An unstiffened seated connection is made with a seat angle and a top angle, as illustrated in Figure 10–7. These angles may be bolted or welded to the supported beam as well as to the supporting member.

While the seat angle is assumed to carry the entire end reaction of the supported beam, the top angle must be placed as shown or in the optional side location for satisfactory performance and stability (Roeder and Dailey, 1989). The top angle and its connections are not usually sized for any calculated strength requirement. A 1/4-in.-thick angle with a 4-in. vertical leg dimension will generally be adequate. It may be bolted with two bolts through each leg or welded with minimum-size welds to either the supported or the supporting members.

When the top angle is welded to the support and/or the supported beam, adequate flexibility must be provided in the connection. As illustrated in Figure 10–7b, line welds are placed along the toe of each angle leg. Note that welding along the sides of the vertical angle leg must be avoided as it would inhibit the flexibility and, therefore, the necessary end rotation of the connection. The performance of such a connection would not be as intended for unstiffened seated connections.



(a) All-bolted



(b) All-welded

Figure 10–7. Unstiffened seated connections.

Design Checks

The available strength of an unstiffened seated connection is determined from the applicable limit states for bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). Additionally, the strength of the supported beam web must be checked for the limit states of web local yielding and web local crippling. In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a.

The available strength for web local yielding, ϕR_n or R_n/Ω, is determined per AISC *Specification* Section J10.2, which is simplified using the constants in Table 9-4. For further information, see Carter et al. (1997).

Shop and Field Practices

Unstiffened seated connections may be made to the webs and flanges of supporting columns. If adequate clearance exists, unstiffened seated connections may also be made to the webs of supporting girders.

To provide for overrun in beam length, the nominal setback for the beam end is 1/2 in. To provide for underrun in beam length, this setback is assumed to be 3/4 in. for calculation purposes.

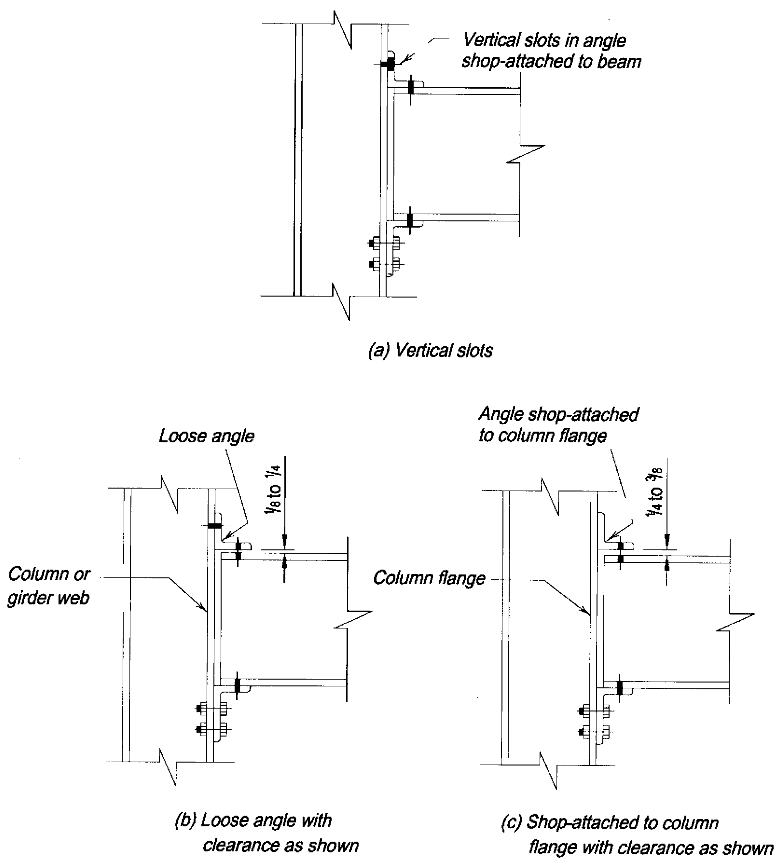


Figure 10–8. Providing for variation in beam depth with seated connections.

The seat angle is preferably shop-attached to the support. Since the bottom flange typically establishes the plane of reference for seated connections, mill variation in beam depth may result in variation in the elevation of the top flange. Such variation is usually of no consequence with concrete slab and metal deck floors, but may be a concern when a grating or steel-plate floor is used. Unless special care is required, the usual mill tolerances for member depth of $^1/_8$ in. to $^1/_4$ in. are ignored. However, when the top angle is shop-attached to the supported beam and field bolted to the support, mill variation in beam depth must be considered. Slotted holes, as illustrated in Figure 10–8a, will accommodate both overrun and underrun in the beam depth and are the preferred method for economy and convenience to both the fabricator and erector. Alternatively, the angle could be shipped loose with clearance provided, as shown in Figure 10–8b. When the top angle is to be field-welded to the support, no provision for mill variation in the beam depth is necessary.

When the top angle is shop-attached to the support, an appropriate erection clearance is provided, as illustrated in Figure 10–8c.

Table 10–5. All-Bolted Unstiffened Seated Connections

Table 10–5 is a design aid for all-bolted unstiffened seats. Seat available strengths are tabulated, assuming a 4-in. outstanding leg, for angle material with F_y = 36 ksi and F_u = 58 ksi and beam material with F_y = 50 ksi and F_u = 65 ksi. All values are for comparison with the LRFD load combination for LRFD design and the ASD load combination for ASD design.

Tabulated seat available strengths consider the limit states of shear yielding and flexural yielding of the outstanding angle leg, and local yielding and crippling of the beam web. A nominal beam setback of $^1/_2$ in. is assumed in these tables. However, this setback is increased to $^3/_4$ in. for calculation purposes in determining the tabulated values to account for the possibility of underrun in beam length.

Bolt available strengths are tabulated for the seat types illustrated in Figure 10–8a with $^3/_4$-in., $^7/_8$-in., and 1-in. diameter ASTM A325, F1852 and A490 bolts. Vertical spacing of bolts and gages in seat angles may be arranged to suit conditions, provided the edge distance and spacing requirements in AISC Specification Section J3 are met. Where thick angles are used, larger entering and tightening clearances may be required in the outstanding angle leg. The suitability of angle sizes and thicknesses for the seat types illustrated in Figure 10–8a is also listed in Table 10–5.

Bolted/Welded Unstiffened Seated Connections

Tables 10–5 and 10–6 may be used in combination to design unstiffened seated connections that are welded to the supporting member and bolted to the supported beam, or bolted to the supporting member and welded to the supported beam.

Table 10–6. All-Welded Unstiffened Seated Connections

Table 10–6 is a design aid for all-welded unstiffened seats (exception: the beam is bolted to the seat). Seat available strengths are tabulated, assuming either a $3^1/_2$-in. or 4-in. outstanding leg (as indicated in the table), for angle material with F_y = 36 ksi and F_u = 58 ksi and beam material with F_y = 50 ksi and F_u = 65 ksi. Electrode strength is assumed to be 70 ksi.

Tabulated seat available strengths consider the limit states of shear yielding and flexural yielding of the outstanding angle leg, and local yielding and crippling of the beam web. A nominal beam setback of 1/2 in. is assumed in these tables. However, this setback is increased to 3/4 in. for calculation purposes in determining the tabulated values to account for the possibility of underrun in beam length.

Weld available strengths are tabulated using the elastic method. The minimum and maximum angle thickness for each case is also tabulated. While these tabular values are based upon 70 ksi electrodes, they may be used for other electrodes, provided the tabular values are adjusted for the electrodes used (e.g., for 60 ksi electrodes, multiply the tabular values by 60/70 = 0.866, etc.) and the welds and base metal meet the required strength level provisions of AISC Specification Section J2. Should combinations of material thickness and weld size selected from Table 10–6 exceed the limits in AISC Specification Section J2.2, the weld size or material thickness should be increased as required.

As can be seen from the following, reduction of the tabulated weld strength is not normally required when unstiffened seats line up on opposite sides of the supporting web. From Salmon and Johnson (1996), the available strength, ϕR_n or R_n/Ω, of the welds to the support is

LRFD	ASD
$\phi R_n = 2\times\dfrac{1.392DL}{\sqrt{1+\dfrac{20.25e^2}{L^2}}}$	$\dfrac{R_n}{\Omega} = 2\times\dfrac{0.928DL}{\sqrt{1+\dfrac{20.25e^2}{L^2}}}$

where

D = number of sixteenths-of-an-inch in the weld size.

L = vertical leg dimension of the seat angle, in.

e = eccentricity of the beam end reaction with respect to the weld lines, in.

The term in the denominator that accounts for the eccentricity e increases the weld size far beyond what is required for shear alone, but with seats on both sides of the supporting member web, the forces due to eccentricity react against each other and have no effect on the web. Furthermore, as illustrated in Figure 10–9, there are actually two shear planes per weld, one at each weld toe and heel for a total of four shear planes. Thus, for an 8-in.-long 7×4×1 seat angle supporting a LRFD required strength of 70 kips or an equivalent ASD required strength of 46.67 kips, the minimum support thickness would be determined as follows

LRFD	ASD
$\dfrac{70\text{ kips}}{0.75\times0.6\times65\text{ ksi}\times7\text{ in.}\times4\text{ planes}}=0.0855\text{ in.}$	$\dfrac{2.0\times46.67\text{ kips}}{0.6\times65\text{ ksi}\times7\text{ in.}\times4\text{ planes}}=0.0855\text{ in.}$

For the identical connection on both sides of the support, the minimum support thickness would be less than 3/16 in. Thus, supporting web thickness is generally not a concern.

L6

Table 10–5
All-Bolted Unstiffened Seated Connections

Angle
F_y = 36 ksi

Outstanding Angle Leg Length Strength, kips

Required Bearing Length N_{req}, in.	Angle Length, in. 6 — Angle Thickness, in. 3/8		1/2		5/8		3/4		1		Min. Angle Leg
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	in.
1/2	18.2	27.3									
9/16	16.2	24.3									
5/8	14.6	21.9	43.1	64.8							
11/16	13.2	19.9	37.0	55.5							
3/4	12.1	18.2	32.3	48.6							
13/16	11.2	16.8	28.7	43.2							
7/8	10.4	15.6	25.9	38.9							
15/16	9.70	14.6	23.5	35.3	54.0	81.0					
1	9.09	13.7	21.6	32.4	50.5	75.9					
1 1/16	8.56	12.9	19.9	29.9	44.9	67.5					
1 1/8	8.08	12.2	18.5	27.8	40.4	60.8					
1 3/16	7.66	11.5	17.2	25.9	36.7	55.2					
1 1/4	7.28	10.9	16.2	24.3	33.7	50.6					
1 5/16	6.93	10.4	15.2	22.9	31.1	46.7	64.7	97.2			3 1/2
1 3/8	6.61	9.94	14.4	21.6	28.9	43.4	58.2	87.5			
1 7/16	6.33	9.51	13.6	20.5	26.9	40.5	52.9	79.5			
1 1/2	6.06	9.11	12.9	19.4	25.3	38.0	48.5	72.9			
1 5/8	5.60	8.41	11.8	17.7	22.5	33.8	41.6	62.5			
1 3/4	5.20	7.81	10.8	16.2	20.2	30.4	36.4	54.7			
1 7/8	4.85	7.29	10.0	15.0	18.4	27.6	32.3	48.6			
2	4.55	6.83	9.24	13.9	16.8	25.3	29.1	43.7	86.2	130	
2 1/8	4.28	6.43	8.62	13.0	15.5	23.4	26.5	39.8	73.9	111	
2 1/4	4.04	6.08	8.08	12.2	14.4	21.7	24.3	36.5	64.7	97.2	
2 3/8	3.83	5.76	7.61	11.4	13.5	20.3	22.4	33.6	57.5	86.4	
2 1/2	3.64	5.47	7.19	10.8	12.6	19.0	20.8	31.2	51.7	77.8	
2 5/8	3.46	5.21	6.81	10.2	11.9	17.9	19.4	29.2	47.0	70.7	
2 3/4	3.31	4.97	6.47	9.72	11.2	16.9	18.2	27.3	43.1	64.8	
2 7/8	3.16	4.75	6.16	9.26	10.6	16.0	17.1	25.7	39.8	59.8	
3	3.03	4.56	5.88	8.84	10.1	15.2	16.2	24.3	37.0	55.5	4
3 1/8	2.91	4.37	5.62	8.45	9.62	14.5	15.3	23.0	34.5	51.8	
3 1/4	2.80	4.21	5.39	8.10	9.19	13.8	14.6	21.9	32.3	48.6	

Bolt Available Strength, kips

Bolt Dia., in.	ASTM Desig.	Thread Cond.	Connection Type from Figure 10-7a: A ASD	A LRFD	B ASD	B LRFD	C ASD	C LRFD
3/4	A325/F1852	N	21.2	31.8	42.4	63.6	63.6	95.4
		X	26.5	39.8	53.0	79.5	79.5	119
	A490	N	26.5	39.8	53.0	79.5	79.5	119
		X	33.1	49.7	66.3	99.4	99.4	149
7/8	A325/F1852	N	28.9	43.3	57.7	86.6	86.6	130
		X	36.1	54.1	72.2	108	108	162
	A490	N	36.1	54.1	72.2	108	108	162
		X	45.1	67.6	90.2	135	135	203
1	A325/F1852	N	37.7	56.5	75.4	113	—	—
		X	47.1	70.7	94.2	141	—	—
	A490	N	47.1	70.7	94.2	141	—	—
		X	58.9	88.4	118	177	—	—

Available Angles

Connection Type	Angle Size	t, in.
A, D	4×3	3/8 – 1/2
	4×3 1/2	3/8 – 1/2
	4×4	3/8 – 3/4
B, E	6×4	3/8 – 3/4
	7×4	3/8 – 3/4
	8×4	1/2 – 1
C, F[b]	8×4	1/2 – 1

[b] Not suitable for use with 1-in. diameter bolts.

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

For tabulated values above the heavy line, shear yielding of the angle leg controls the available strength.

Angle
F_y = 36 ksi

Table 10–5 (continued)
All-Bolted Unstiffened Seated Connections

L8

Outstanding Angle Leg Length Strength, kips

Required Bearing Length N_{req}, in.	Angle Length, in. 8 — Angle Thickness, in. 3/8		1/2		5/8		3/4		1		Min. Angle Leg
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	in.
1/2	24.3	36.5									
9/16	21.6	32.4									
5/8	19.4	29.2	57.5	86.4							
11/16	17.6	26.5	49.3	74.1							
3/4	16.2	24.3	43.1	64.8							
13/16	14.9	22.4	38.3	57.6							
7/8	13.9	20.8	34.5	51.8							
15/16	12.9	19.4	31.4	47.1	72.0	108					
1	12.1	18.2	28.7	43.2	67.4	101					
1 1/16	11.4	17.2	26.5	39.9	59.9	90.0					
1 1/8	10.8	16.2	24.6	37.0	53.9	81.0					
1 3/16	10.2	15.3	23.0	34.6	49.0	73.6					
1 1/4	9.7	14.6	21.6	32.4	44.9	67.5					
1 5/16	9.2	13.9	20.3	30.5	41.5	62.3	86.2	130			3 1/2
1 3/8	8.82	13.3	19.2	28.8	38.5	57.9	77.6	117			
1 7/16	8.44	12.7	18.2	27.3	35.9	54.0	70.5	106			
1 1/2	8.08	12.2	17.2	25.9	33.7	50.6	64.7	97.2			
1 5/8	7.46	11.2	15.7	23.6	29.9	45.0	55.4	83.3			
1 3/4	6.93	10.4	14.4	21.6	26.9	40.5	48.5	72.9			
1 7/8	6.47	9.72	13.3	19.9	24.5	36.8	43.1	64.8			
2	6.06	9.11	12.3	18.5	22.5	33.8	38.8	58.3	115	173	
2 1/8	5.71	8.58	11.5	17.3	20.7	31.2	35.3	53.0	98.5	148	
2 1/4	5.39	8.10	10.8	16.2	19.2	28.9	32.3	48.6	86.2	130	
2 3/8	5.11	7.67	10.1	15.2	18.0	27.0	29.8	44.9	76.6	115	
2 1/2	4.85	7.29	9.58	14.4	16.8	25.3	27.7	41.7	69.0	104	
2 5/8	4.62	6.94	9.08	13.6	15.9	23.8	25.9	38.9	62.7	94.3	
2 3/4	4.41	6.63	8.62	13.0	15.0	22.5	24.3	36.5	57.5	86.4	
2 7/8	4.22	6.34	8.21	12.3	14.2	21.3	22.8	34.3	53.1	79.8	
3	4.04	6.08	7.84	11.8	13.5	20.3	21.6	32.4	49.3	74.1	
3 1/8	3.88	5.83	7.50	11.3	12.8	19.3	20.4	30.7	46.0	69.1	4
3 1/4	3.73	5.61	7.19	10.8	12.2	18.4	19.4	29.2	43.1	64.8	

Bolt Available Strength, kips

Bolt Dia., in.	ASTM Desig.	Thread Cond.	Connection Type from Figure 10-7a — D ASD	D LRFD	E ASD	E LRFD	F ASD	F LRFD
3/4	A325/F1852	N	31.8	47.7	63.6	95.4	95.4	143
		X	39.8	59.6	79.5	119	119	179
	A490	N	39.8	59.6	79.5	119	119	179
		X	49.7	74.6	99.4	149	149	224
7/8	A325/F1852	N	43.3	64.9	86.6	130	130	195
		X	54.1	81.2	108	162	162	244
	A490	N	54.1	81.2	108	162	162	244
		X	67.6	101	135	203	203	304
1	A325/F1852	N	56.5	84.8	113	170	—	—
		X	70.7	106	141	212	—	—
	A490	N	70.7	106	141	212	—	—
		X	88.4	133	177	265	—	—

Available Angles

Connection Type	Angle Size	t, in.
A, D	4×3	3/8 – 1/2
	4×3 1/2	3/8 – 1/2
	4×4	3/8 – 3/4
B, E	6×4	3/8 – 3/4
	7×4	3/8 – 3/4
	8×4	1/2 – 1
C, F[b]	8×4	1/2 – 1

[b] Not suitable for use with 1-in. diameter bolts.

ASD	LRFD	
Ω = 2.00	ϕ = 0.75	For tabulated values above the heavy line, shear yielding of the angle leg controls the available strength.

L6

Table 10-6
All-Welded Unstiffened Seated Connections

Angle
F_y = 36 ksi

Outstanding Angle Leg Length Strength, kips

Required Bearing Length N_{req}, in.	Angle Length, in. 6 — Angle Thickness, in. 3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD	5/8 ASD	5/8 LRFD	3/4 ASD	3/4 LRFD	1 ASD	1 LRFD	Min. Angle Leg in.
1/2	18.2	27.3									
9/16	16.2	24.3									
5/8	14.6	21.9	43.1	64.8							
11/16	13.2	19.9	37.0	55.5							
3/4	12.1	18.2	32.3	48.6							
13/16	11.2	16.8	28.7	43.2							
7/8	10.4	15.6	25.9	38.9							
15/16	9.70	14.6	23.5	35.3	54.0	81.0					
1	9.09	13.7	21.6	32.4	50.5	75.9					
1 1/16	8.56	12.9	19.9	29.9	44.9	67.5					
1 1/8	8.08	12.2	18.5	27.8	40.4	60.8					
1 3/16	7.66	11.5	17.2	25.9	36.7	55.2					
1 1/4	7.28	10.9	16.2	24.3	33.7	50.6					
1 5/16	6.93	10.4	15.2	22.9	31.1	46.7	64.7	97.2			3 1/2
1 3/8	6.61	9.94	14.4	21.6	28.9	43.4	58.2	87.5			
1 7/16	6.33	9.51	13.6	20.5	26.9	40.5	52.9	79.5			
1 1/2	6.06	9.11	12.9	19.4	25.3	38.0	48.5	72.9			
1 5/8	5.60	8.41	11.8	17.7	22.5	33.8	41.6	62.5			
1 3/4	5.20	7.81	10.8	16.2	20.2	30.4	36.4	54.7			
1 7/8	4.85	7.29	9.95	15.0	18.4	27.6	32.3	48.6			
2	4.55	6.83	9.24	13.9	16.8	25.3	29.1	43.7	86.2	130	
2 1/8	4.28	6.43	8.62	13.0	15.5	23.4	26.5	39.8	73.9	111	
2 1/4	4.04	6.08	8.08	12.2	14.4	21.7	24.3	36.5	64.7	97.2	
2 3/8	3.83	5.76	7.61	11.4	13.5	20.3	22.4	33.6	57.5	86.4	
2 1/2	3.64	5.47	7.19	10.8	12.6	19.0	20.8	31.2	51.7	77.8	
2 5/8	3.46	5.21	6.81	10.2	11.9	17.9	19.4	29.2	47.0	70.7	
2 3/4	3.31	4.97	6.47	9.72	11.2	16.9	18.2	27.3	43.1	64.8	
2 7/8	3.16	4.75	6.16	9.26	10.6	16.0	17.1	25.7	39.8	59.8	
3	3.03	4.56	5.88	8.84	10.1	15.2	16.2	24.3	37.0	55.5	4
3 1/8	2.91	4.37	5.62	8.45	9.62	14.5	15.3	23.0	34.5	51.8	
3 1/4	2.80	4.21	5.39	8.10	9.19	13.8	14.6	21.9	32.3	48.6	

Weld (70 ksi) Available Strength, kips

70 ksi Weld Size, in.	Seat Angle Size (long leg vertical) 4 × 3 1/2		5 × 3 1/2	
Design	ASD	LRFD	ASD	LRFD
1/4	11.5	17.2	17.2	25.8
5/16	14.3	21.5	21.5	32.2
3/8	17.2	25.8	25.8	38.7
7/16	20.1	30.1	30.1	45.2
1/2	—	—	34.4	51.6
9/16	—	—	38.7	58.1
5/8	—	—	43.0	64.5
11/16	—	—	47.3	71.0

Available Angle Thickness, in.

	4 × 3 1/2	5 × 3 1/2
Minimum	3/8	3/8
Maximum	1/2	3/4

ASD	LRFD	
Ω = 2.00	ϕ = 0.75	For tabulated values above the heavy line, shear yielding of the angle leg controls the available strength.

Angle
F_y = 36 ksi

Table 10-6 (continued)
All-Welded Unstiffened Seated Connections

L8

Outstanding Angle Leg Length Strength, kips

Required Bearing Length N_{req}, in.	Angle Length, in.										Min. Angle Leg
	8										
	Angle Thickness, in.										
	3/8		1/2		5/8		3/4		1		
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	in.
1/2	24.3	36.5									
9/16	21.6	32.4									
5/8	19.4	29.2	57.5	86.4							
11/16	17.6	26.5	49.3	74.1							
3/4	16.2	24.3	43.1	64.8							
13/16	14.9	22.4	38.3	57.6							
7/8	13.9	20.8	34.5	51.8							
15/16	12.9	19.4	31.4	47.1	72.0	108					
1	12.1	18.2	28.7	43.2	67.4	101					
1 1/16	11.4	17.2	26.5	39.9	59.9	90.0					
1 1/8	10.8	16.2	24.6	37.0	53.9	81.0					
1 3/16	10.2	15.3	23.0	34.6	49.0	73.6					
1 1/4	9.70	14.6	21.6	32.4	44.9	67.5					
1 5/16	9.24	13.9	20.3	30.5	41.5	62.3	86.2	130			3 1/2
1 3/8	8.82	13.3	19.2	28.8	38.5	57.9	77.6	117			
1 7/16	8.44	12.7	18.2	27.3	35.9	54.0	70.5	106			
1 1/2	8.08	12.2	17.2	25.9	33.7	50.6	64.7	97.2			
1 5/8	7.46	11.2	15.7	23.6	29.9	45.0	55.4	83.3			
1 3/4	6.93	10.4	14.4	21.6	26.9	40.5	48.5	72.9			
1 7/8	6.47	9.72	13.3	19.9	24.5	36.8	43.1	64.8			
2	6.06	9.11	12.3	18.5	22.5	33.8	38.8	58.3	115	173	
2 1/8	5.71	8.58	11.5	17.3	20.7	31.2	35.3	53.0	98.5	148	
2 1/4	5.39	8.10	10.8	16.2	19.2	28.9	32.3	48.6	86.2	130	
2 3/8	5.11	7.67	10.1	15.2	18.0	27.0	29.8	44.9	76.6	115	
2 1/2	4.85	7.29	9.58	14.4	16.8	25.3	27.7	41.7	69.0	104	
2 5/8	4.62	6.94	9.08	13.6	15.9	23.8	25.9	38.9	62.7	94.3	
2 3/4	4.41	6.63	8.62	13.0	15.0	22.5	24.3	36.5	57.5	86.4	
2 7/8	4.22	6.34	8.21	12.3	14.2	21.3	22.8	34.3	53.1	79.8	
3	4.04	6.08	7.84	11.8	13.5	20.3	21.6	32.4	49.3	74.1	
3 1/8	3.88	5.83	7.50	11.3	12.8	19.3	20.4	30.7	46.0	69.1	4
3 1/4	3.73	5.61	7.19	10.8	12.2	18.4	19.4	29.2	43.1	64.8	

Weld (70 ksi) Available Strength, kips

70 ksi Weld Size, in.	Seat Angle Size (long leg vertical)					
	6 × 4		7 × 4		8 × 4	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD
1/4	21.8	32.7	28.5	42.7	35.6	53.4
5/16	27.3	40.9	35.6	53.4	44.5	66.7
3/8	32.7	49.1	42.7	64.1	53.4	80.1
7/16	38.2	57.2	49.8	74.7	62.3	93.4
1/2	43.6	65.4	57.0	85.4	71.2	107
9/16	49.1	73.6	64.1	96.1	80.1	120
5/8	54.5	81.8	71.2	107	89.0	133
11/16	60.0	90.0	78.3	117	97.9	147

Available Angle Thickness, in.

	6 × 4	7 × 4	8 × 4
Minimum	3/8	3/8	1/2
Maximum	3/4	3/4	1

ASD	LRFD	
Ω = 2.00	ϕ = 0.75	For tabulated values above the heavy line, shear yielding of the angle leg controls the available strength.

STIFFENED SEATED CONNECTIONS

A stiffened seated connection is made with a seat plate and stiffening element (e.g., a plate, structural tee, or pair of angles) and a top angle, as illustrated in Figure 10–10. The top angle may be bolted or welded to the supported beam as well as to the supporting member and the stiffening element may be bolted or welded to the support. The seat plate should be bolted to the supported beam.

The stiffening element is assumed to carry the entire end reaction of the supported beam applied at a distance equal to $0.8W$. The top angle must be placed as shown or in the optional side location for satisfactory performance and stability (Roeder and Dailey, 1989). The top angle and its connections are not usually sized for any calculated strength requirement. A $^1/_4$-in.-thick angle with a 4-in. vertical leg dimension will generally be adequate. It may be bolted with two bolts through each leg or welded with minimum-size welds to either the supported or the supporting members.

When the top angle is welded to the support and/or the supported beam, adequate flexibility must be provided in the connection. As illustrated in Figure 10–10b, line welds are placed along the toe of each angle leg. Note that welding along the sides of the vertical angle leg must be avoided as it would inhibit the flexibility and, therefore, the necessary end rotation of the connection. The performance of such a connection would not be as intended for simple shear connections.

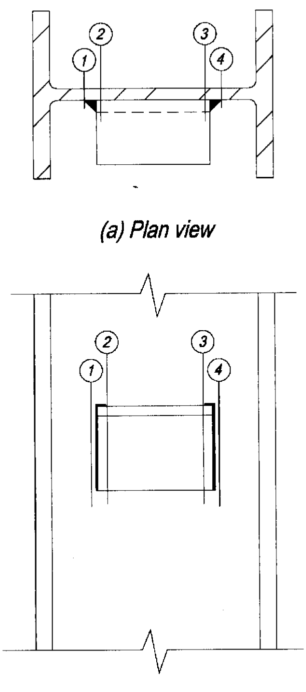


Figure 10–9. Shear planes in column web for unstiffened seated connections.

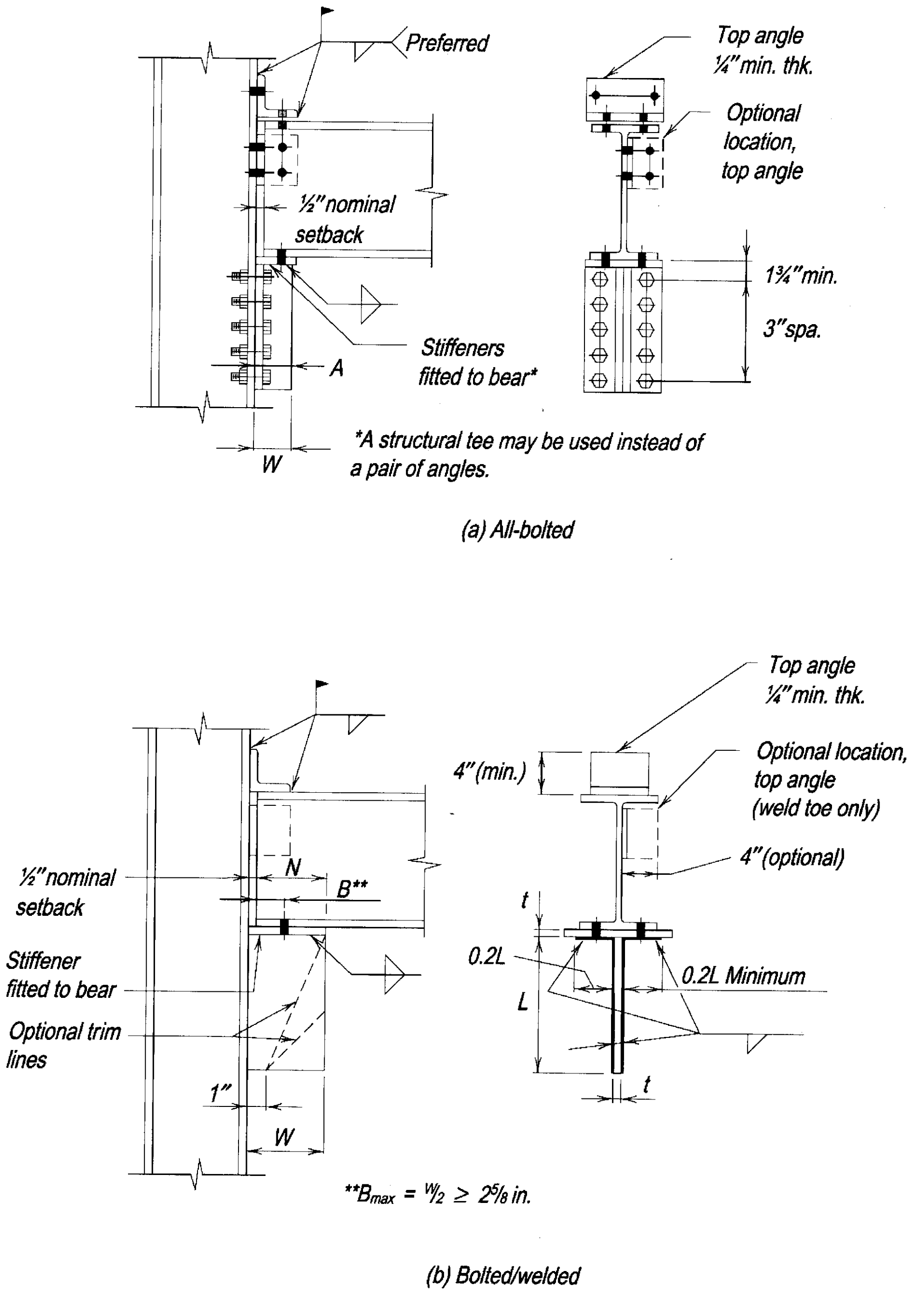


Figure 10–10. Stiffened seated connections.

Design Checks

The available strength of a stiffened seated connection is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). Additionally, the strength of the supported beam web must be checked for the limit states of local web yielding and web crippling. In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a. The available strength for web local yielding, ϕR_n or R_n/Ω, is determined per AISC Specification Section J10.2, which is simplified using the constants in Table 9–4.

When stiffened seated connections such as the one shown in Figure 10–10b are made to one side of a supporting column web, the column web may also need to be investigated for resistance to punching shear. In lieu of a more detailed analysis, Sputo and Ellifritt (1991) showed that punching shear will not be critical if the design parameters below and those summarized graphically in Figure 10–10b are met.

1. This simplified approach is applicable to the following column sections:

W14×43-808	W12×40-336	W10×33-112
W8×24-67	W6×20-25	W5×16-19

2. The supported beam must be bolted to the seat plate with high-strength bolts to account for the prying action caused by rotation of the connection. Welding the beam to the seat plate is not recommended because welds may lack the required strength and ductility. The centerline of the bolts should be located no more than the greater of $W/2$ or $2^5/8$ in. from the column web face.
3. For seated connections where $W = 8$ in. or $W = 9$ in. and $3^1/2$ in. $< B \leq W/2$, or where $W = 7$ in. and 3 in. $< B \leq W/2$ for a W14×43 column, refer to Sputo and Ellifritt (1991). These limitations are summarized at the bottom of Table 10–8.
4. The top angle may be bolted or welded, but must have a minimum $^1/4$-in. thickness.
5. The seat plate should not be welded to the beam flange.

See also Ellifritt and Sputo (1999).

Shop and Field Practices

The comments for unstiffened seated connections are equally applicable to stiffened seated connections.

Table 10–7. All-Bolted Stiffened Seated Connections

Table 10–7 is a design aid for all-bolted stiffened seats. Stiffener available strengths are tabulated for stiffener material with F_y = 36 ksi and F_u = 58 ksi and with F_y = 50 ksi and F_u = 65 ksi.

Tabulated values consider the limit-state of bearing on the stiffening material. The designer must independently check the available strength of the beam web based upon the limit states of local web yielding and local web crippling. A nominal beam setback of $^1/2$ in. is assumed in these tables. However, this setback is increased to $^3/4$ in. for calculation purposes in determining the tabulated values to account for the possibility of underrun in beam length.

Bolt available strengths are tabulated for two vertical rows of from three to seven $^3/4$-in., $^7/8$-in., and 1-in.-diameter ASTM A325, F1852 and A490 high-strength bolts based upon the limit-state of bolt shear. Vertical spacing of bolts and gages in seat angles may be arranged to suit conditions, provided the edge distance and spacing requirements in AISC Specification Section J3 are met.

Table 10–8. Bolted/Welded Stiffened Seated Connections

Table 10–8 is a design aid for stiffened seated connections welded to the support and bolted to the supported beam. Electrode strength is assumed to be 70 ksi.

Weld available strengths are tabulated using the elastic method. While these tabular values are based upon 70 ksi electrodes, they may be used for other electrodes, provided the tabular values are adjusted for the electrodes used (e.g., for 60 ksi electrodes, multiply the tabular values by 60/70 = 0.866, etc.) and the weld and base metal meet the provisions of AISC Specification Section J2.

The thickness of the horizontal seat plate or tee flange should not be less than $^3/_8$ in. If the seat and stiffener are built up from separate plates, the stiffener should be finished to bear under the seat. The welds connecting the two plates should have a strength equal to or greater than the horizontal welds to the support under the seat plate.

The designer must independently check the beam web for web local yielding and web local crippling. The nominal beam setback of $^1/_2$ in. should be assumed to be $^3/_4$ in. for calculation purposes to account for possible underrun in beam length.

The stiffener thickness may be conservatively determined as follows. The minimum stiffener plate thickness, t, for supported beams with unstiffened webs should be the supported beam web thickness, t_w, multiplied by the ratio of F_y of the beam material to F_y of the stiffener material (e.g., F_y beam = 50 ksi, F_y stiffener = 36 ksi, $t = t_w \times 50/36$ minimum). Additionally, the minimum stiffener thickness, t, should be at least $2w$ for stiffener material with F_y = 36 ksi or $1.5w$ for stiffener material with F_y = 50 ksi, where w is the weld size for 70 ksi electrodes.

For 70 ksi electrodes, the minimum column web thickness is

$$t_{min} = \frac{3.09D}{F_u}$$

where

D = the weld size in sixteenths of an inch.

When welds line up on opposite sides of the support, the minimum thickness is the sum of the thicknesses required for each weld. In either case, when less than the minimum material thickness is present, the weld available strength must be reduced by the ratio of the thickness provided to the minimum thickness. As with unstiffened seated connections, the contribution of eccentricity to the required shear yielding strength is negligible. Should combinations of material thickness and weld size selected from Table 10–8 exceed the limits of AISC Specification Section J2, increase the weld size or material thickness as required.

Table 10–7
All-Bolted Stiffened Seated Connections

Stiffener Material		Outstanding Angle Leg Available Strength, kips[a]											
Stiffener Outstanding Leg A, in.[b]		F_y = 36 ksi						F_y = 50 ksi					
		3 1/2		4		5		3 1/2		4		5	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Thickness of Stiffener Outstanding Legs, in.	5/16	55.7	83.5	65.8	98.7	86.1	129	77.3	116	91.4	137	120	179
	3/8	66.8	100	79.0	118	103	155	92.8	139	110	165	143	215
	1/2	89.1	134	105	158	138	207	124	186	146	219	191	287
	5/8	111	167	132	197	172	258	155	232	183	274	239	359
	3/4	134	200	158	237	207	310	186	278	219	329	287	430

Use minimum 3/8-in. thick seat plate wide enough to extend beyond outstanding legs of stiffener.
a See AISC Specification Sect. J7.
b Beam bearing length assumed 3/4 in. less for calculation purposes.

Bolt Diameter, in.	ATSM Desig.	Thread Cond.	Bolt Available Strength, kips — Number of Bolts in One Vertical Row									
			3		4		5		6		7	
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
3/4	A325/F182	N	63.6	95.4	84.8	127	106	159	127	191	148	223
		X	79.5	119	106	159	133	199	159	239	186	278
	A490	N	79.5	119	106	159	133	199	159	239	186	278
		X	99.4	149	133	199	166	249	199	298	232	348
7/8	A325/F182	N	86.6	130	115	173	144	216	173	260	202	303
		X	108	162	144	216	180	271	216	325	253	379
	A490	N	108	162	144	216	180	271	216	325	253	379
		X	135	203	180	271	225	338	271	406	316	474
1	A325/F182	N	113	170	151	226	188	283	226	339	264	396
		X	141	212	188	283	236	353	283	424	330	495
	A490	N	141	212	188	283	236	353	283	424	330	495
		X	177	265	236	353	295	442	353	530	412	619

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$
$\frac{R_n}{\Omega} = \frac{1.8F_yA_{pb}}{2.00}$	$\phi R_n = 0.75 \times 1.8F_yA_{pb}$

Table 10–8
Bolted/Welded Stiffened Seated Connections

L, in.	Width of Seat W, in.											
	4								5			
	70 ksi Weld Size, in.								70 ksi Weld Size, in.			
	1/4		5/16		3/8		7/16		5/16		3/8	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
6	**22.7**	34.0	**28.4**	42.5	**34.0**	51.1	**39.7**	59.6	**23.5**	35.2	**28.2**	42.2
7	**29.9**	44.9	**37.4**	56.1	**44.9**	67.3	**52.4**	78.6	**31.2**	46.9	**37.5**	56.2
8	**37.8**	56.7	**47.2**	70.8	**56.7**	85.0	**66.1**	99.2	**39.8**	59.8	**47.8**	71.7
9	**46.1**	69.2	**57.7**	86.5	**69.2**	104	**80.7**	121	**49.1**	73.7	**59.0**	88.5
10	**54.9**	82.3	**68.6**	103	**82.3**	123	**96.0**	144	**59.0**	88.5	**70.8**	106
11	**63.9**	95.8	**79.8**	120	**95.8**	144	**112**	168	**69.4**	104	**83.3**	125
12	**73.1**	110	**91.4**	137	**110**	165	**128**	192	**80.2**	120	**96.2**	144
13	**82.5**	124	**103**	155	**124**	186	**144**	217	**91.3**	137	**110**	164
14	**92.1**	138	**115**	173	**138**	207	**161**	242	**103**	154	**123**	185
15	**102**	152	**127**	191	**152**	229	**178**	267	**114**	171	**137**	206
16	**111**	167	**139**	209	**167**	250	**195**	292	**126**	189	**151**	227
17	**121**	181	**151**	227	**181**	272	**212**	318	**138**	207	**165**	248
18	**131**	196	**163**	245	**196**	294	**229**	343	**150**	225	**180**	270
19	**140**	211	**175**	263	**211**	316	**246**	369	**162**	243	**194**	291
20	**150**	225	**188**	281	**225**	338	**263**	394	**174**	261	**209**	313
21	**160**	240	**200**	300	**240**	359	**280**	419	**186**	279	**223**	335
22	**169**	254	**212**	318	**254**	381	**296**	445	**198**	297	**238**	357
23	**179**	269	**224**	336	**269**	403	**313**	470	**210**	315	**252**	378
24	**189**	283	**236**	354	**283**	425	**330**	495	**222**	334	**267**	400
25	**198**	297	**248**	372	**297**	446	**347**	520	**235**	352	**281**	422
26	**208**	312	**260**	390	**312**	468	**364**	546	**247**	370	**296**	444
27	**217**	326	**272**	408	**326**	489	**380**	571	**259**	388	**310**	466

Limitations for Connections to Column Webs	
B = 2 5/8 in. max	B = 2 5/8 in. max
W12×40, W14×43 for L ≥ 9 in. limit weld ≤ 1/4 in.	None

Notes:

1. Values shown assume 70 ksi electrodes. For 60 ksi electrodes, multiply tabular values by 0.857, or enter table with 1.17 times the required strength, R_u or R_a. For 80 ksi electrodes, multiply tabular values by 1.14, or enter table with 0.875 times the required strength.
2. Tabulated values are valid for stiffeners with minimum thickness of

$$t_{min} = \frac{F_{y\ beam}}{F_{y\ stiffener}} \times t_w$$

but not less than $2w$ for stiffeners with $F_y = 36$ ksi nor $1.5w$ for stiffeners with $F_y = 50$ ksi. In the above, t_w is the thickness of the unstiffened supported beam web and w is the nominal weld size.

3. Tabulated values may be limited by shear yielding of or bearing on the stiffener; refer to LRFD Specification Sections F2.2 and J8, respectively.

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

Table 10–8 (continued)
Bolted/Welded Stiffened Seated Connections

L, in.	Width of Seat W, in.											
	5				6							
	70 ksi Weld Size, in.				70 ksi Weld Size, in.							
	7/16		1/2		5/16		3/8		7/16		1/2	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
6	**32.8**	49.3	**37.5**	56.3	**19.9**	29.9	**23.9**	35.9	**27.9**	41.9	**31.9**	47.8
7	**43.7**	65.6	**50.0**	75.0	**26.7**	40.1	**32.0**	48.1	**37.4**	56.1	**42.7**	64.1
8	**55.8**	83.7	**63.8**	95.6	**34.3**	51.4	**41.1**	61.7	**48.0**	72.0	**54.8**	82.2
9	**68.8**	103	**78.6**	118	**42.5**	63.8	**51.1**	76.6	**59.6**	89.3	**68.1**	102
10	**82.6**	124	**94.4**	142	**51.4**	77.2	**61.7**	92.6	**72.0**	108	**82.3**	123
11	**97.2**	146	**111**	167	**60.9**	91.3	**73.1**	110	**85.3**	128	**97.4**	146
12	**112**	168	**128**	192	**70.8**	106	**85.0**	127	**99.2**	149	**113**	170
13	**128**	192	**146**	219	**81.2**	122	**97.4**	146	**114**	170	**130**	195
14	**144**	216	**164**	246	**91.9**	138	**110**	165	**129**	193	**147**	220
15	**160**	240	**183**	274	**103**	154	**123**	185	**144**	216	**165**	247
16	**176**	265	**202**	302	**114**	171	**137**	205	**160**	240	**183**	274
17	**193**	290	**221**	331	**126**	188	**151**	226	**176**	264	**201**	301
18	**210**	315	**240**	360	**137**	206	**165**	247	**192**	288	**219**	329
19	**227**	340	**259**	388	**149**	223	**179**	268	**208**	313	**238**	357
20	**244**	365	**278**	417	**161**	241	**193**	289	**225**	337	**257**	386
21	**260**	391	**298**	446	**173**	259	**207**	311	**242**	362	**276**	414
22	**277**	416	**317**	476	**185**	277	**222**	332	**258**	388	**295**	443
23	**294**	442	**336**	505	**197**	295	**236**	354	**275**	413	**315**	472
24	**311**	467	**356**	534	**209**	313	**250**	376	**292**	438	**334**	501
25	**328**	492	**375**	563	**221**	331	**265**	397	**309**	464	**353**	530
26	**345**	518	**395**	592	**233**	349	**280**	419	**326**	489	**373**	559
27	**362**	543	**414**	621	**245**	368	**294**	441	**343**	515	**392**	588

Limitations for Connections to Column Webs

B = 2 5/8 in. max	B = 3 in. max
None	None

Notes:

1. Values shown assume 70 ksi electrodes. For 60 ksi electrodes, multiply tabular values by 0.857, or enter table with 1.17 times the required strength, R_u or R_a. For 80 ksi electrodes, multiply tabular values by 1.14, or enter table with 0.875 times the required strength.
2. Tabulated values are valid for stiffeners with minimum thickness of

$$t_{min} = \frac{F_{y\ beam}}{F_{y\ stiffener}} \times t_w$$

but not less than $2w$ for stiffeners with $F_y = 36$ ksi nor $1.5w$ for stiffeners with $F_y = 50$ ksi. In the above, t_w is the thickness of the unstiffened supported beam web and w is the nominal weld size.
3. Tabulated values may be limited by shear yielding of or bearing on the stiffener; refer to LRFD Specification Sections F2.2 and J8, respectively.

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

Table 10–8 (continued)
Bolted/Welded Stiffened Seated Connections

L, in.	Width of Seat W, in.											
	7								8			
	70 ksi Weld Size, in.								70 ksi Weld Size, in.			
	$^5/_{16}$		$^3/_8$		$^7/_{16}$		$^1/_2$		$^5/_{16}$		$^3/_8$	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
11	54.0	81.0	64.8	97.2	75.6	113	86.4	130	48.4	72.5	58.0	87.1
12	63.1	94.7	75.7	114	88.4	133	101	151	56.7	85.1	68.1	102
13	72.7	109	87.2	131	102	153	116	174	65.6	98.3	78.7	118
14	82.6	124	99.2	149	116	174	132	198	74.8	112	89.8	135
15	93.0	139	112	167	130	195	149	223	84.5	127	101	152
16	104	155	124	186	145	217	166	249	94.4	142	113	170
17	114	172	137	206	160	240	183	275	105	157	126	189
18	126	188	151	226	176	264	201	301	115	173	138	208
19	137	205	164	246	192	287	219	329	126	189	151	227
20	148	223	178	267	208	312	237	356	137	206	165	247
21	160	240	192	288	224	336	256	384	148	222	178	267
22	172	258	206	309	240	361	275	412	160	240	192	287
23	184	275	220	330	257	385	294	440	171	257	205	308
24	195	293	234	352	274	410	313	469	183	274	219	329
25	207	311	249	373	290	435	332	498	195	292	233	350
26	219	329	263	395	307	461	351	526	206	309	248	371
27	231	347	278	417	324	486	370	555	218	327	262	393
28	244	365	292	438	341	511	390	584	230	345	276	414
29	256	383	307	460	358	537	409	613	242	363	291	436
30	268	402	321	482	375	562	428	643	254	381	305	457
31	280	420	336	504	392	588	448	672	266	399	319	479
32	292	438	350	526	409	613	467	701	278	417	334	501

Limitations for Connections to Column Webs	
B = 3$^1/_2$ in. max	B = 3$^1/_2$ in. max
W14 × 43, limit B ≤ 3 in. See item 3 in preceding discussion "Design Checks"	See item 3 in preceding discussion "Design Checks"

Notes:
1. Values shown assume 70 ksi electrodes. For 60 ksi electrodes, multiply tabular values by 0.857, or enter table with 1.17 times the required strength, R_u or R_a. For 80 ksi electrodes, multiply tabular values by 1.14, or enter table with 0.875 times the required strength.
2. Tabulated values are valid for stiffeners with minimum thickness of

$$t_{min} = \frac{F_{y\ beam}}{F_{y\ stiffener}} \times t_w$$

but not less than $2w$ for stiffeners with $F_y = 36$ ksi nor $1.5w$ for stiffeners with $F_y = 50$ ksi. In the above, t_w is the thickness of the unstiffened supported beam web and w is the nominal weld size.
3. Tabulated values may be limited by shear yielding of or bearing on the stiffener; refer to LRFD Specification Sections F2.2 and J8, respectively.

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

Table 10–8 (continued)
Bolted/Welded Stiffened Seated Connections

L, in.	Width of Seat W, in.											
	8				9							
	70 ksi Weld Size, in.				70 ksi Weld Size, in.							
	$^{1}/_{2}$		$^{5}/_{8}$		$^{5}/_{16}$		$^{3}/_{8}$		$^{1}/_{2}$		$^{5}/_{8}$	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
11	**77.4**	116	**96.7**	145	**43.7**	65.6	**52.5**	78.7	**69.9**	105	**87.4**	131
12	**90.8**	136	**113**	170	**51.4**	77.1	**61.7**	92.5	**82.2**	123	**103**	154
13	**105**	157	**131**	197	**59.6**	89.3	**71.5**	107	**95.3**	143	**119**	179
14	**120**	180	**150**	224	**68.2**	102	**81.8**	123	**109**	164	**136**	204
15	**135**	203	**169**	253	**77.2**	116	**92.6**	139	**123**	185	**154**	232
16	**151**	227	**189**	283	**86.5**	130	**104**	156	**138**	208	**173**	260
17	**168**	251	**209**	314	**96.2**	144	**115**	173	**154**	231	**192**	289
18	**184**	277	**231**	346	**106**	159	**127**	191	**170**	255	**212**	319
19	**202**	303	**252**	378	**117**	175	**140**	210	**186**	280	**233**	350
20	**219**	329	**274**	411	**127**	191	**152**	229	**203**	305	**254**	381
21	**237**	356	**297**	445	**138**	207	**165**	248	**220**	331	**276**	413
22	**256**	383	**319**	479	**149**	223	**178**	268	**238**	357	**297**	446
23	**274**	411	**342**	514	**160**	240	**192**	288	**256**	384	**320**	480
24	**292**	439	**366**	548	**171**	257	**205**	308	**274**	411	**342**	513
25	**311**	467	**389**	584	**183**	274	**219**	329	**292**	438	**365**	548
26	**330**	495	**413**	619	**194**	291	**233**	349	**310**	466	**388**	582
27	**349**	524	**436**	655	**206**	308	**247**	370	**329**	494	**411**	617
28	**368**	552	**460**	690	**217**	326	**261**	391	**348**	522	**435**	652
29	**387**	581	**484**	726	**229**	344	**275**	412	**367**	550	**458**	687
30	**407**	610	**508**	762	**241**	362	**289**	434	**386**	578	**482**	723
31	**426**	639	**532**	799	**253**	379	**304**	455	**405**	607	**506**	759
32	**445**	668	**557**	835	**265**	397	**318**	477	**424**	636	**530**	795

Limitations for Connections to Column Webs	
B = 3½ in. max	B = 3½ in. max
See item 3 in preceding discussion "Design Checks"	See item 3 in preceeding discussion "Design Checks"

Notes:

1. Values shown assume 70 ksi electrodes. For 60 ksi electrodes, multiply tabular values by 0.857, or enter table with 1.17 times the required strength, R_u or R_a. For 80 ksi electrodes, multiply tabular values by 1.14, or enter table with 0.875 times the required strength.
2. Tabulated values are valid for stiffeners with minimum thickness of

$$t_{min} = \frac{F_{y\ beam}}{F_{y\ stiffener}} \times t_w$$

but not less than $2w$ for stiffeners with F_y = 36 ksi nor $1.5w$ for stiffeners with F_y = 50 ksi. In the above, t_w is the thickness of the unstiffened supported beam web and w is the nominal weld size.

3. Tabulated values may be limited by shear yielding of or bearing on the stiffener; refer to LRFD Specification Sections F2.2 and J8, respectively.

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

SINGLE-PLATE CONNECTIONS

A single-plate connection is made with a plate, as illustrated in Figure 10–11. The plate is always welded to the support on both sides of the plate and bolted to the supported member.

Design Checks

The available strength of a single-plate connection is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a, respectively.

Single-plate shear connections that satisfy the corresponding dimensional limitations can be designed using the simplified design procedure for the "conventional" configuration. Other single-plate shear connections can be designed using the procedure for the "extended" configuration, which is applicable to any configuration of single-plate shear connections, regardless of connection geometry.

Both the conventional and extended configurations permit the use of ASTM A325, F1852, or A490 bolts. The procedure is valid for bolts that are snug-tightened, pretensioned, or slip-critical. In both the conventional and extended configuration, the design recommendations are equally applicable to plate and beam web material with F_y = 36 ksi or 50 ksi. In both cases, the weld between the single plate and the support should be sized as $^5/_8 t_p$, which will develop the strength of either a 36 ksi or 50 ksi plate.

Conventional Configuration

The following method may be used when the dimensional and other limitations upon which it is based are satisfied.

Dimensional Limitations

1. Only a single vertical row of bolts is permitted. The number of bolts in the connection, n, is limited to 2 to 12.
2. The distance from the bolt line to the weld line, a, must be equal to or less than $3^1/_2$ in.

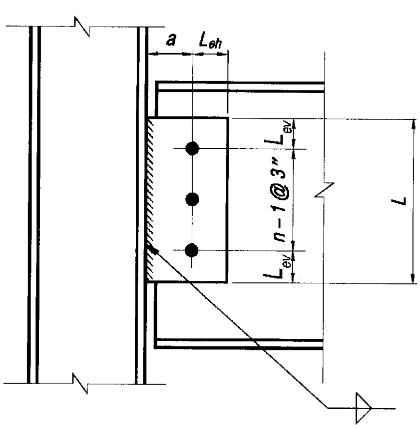


Figure 10–11. Single-plate connection.

3. STD or SSL holes are permitted to be used.
4. The horizontal edge distance, L_{eh}, must be equal to or greater than $2d_b$ for both the plate and the beam web. Note that L_{eh} is measured to the center of the hole or slot.
5. The vertical edge distance, L_{ev}, must satisfy AISC Specification Table J3.4 requirements.
6. Either the plate or the beam web must satisfy $t \le d_b/2 + {}^1/_{16}$ in.

Design Checks

1. The connection must be checked for bolt shear, block shear rupture, and bolt bearing. For STD holes, eccentricity can be ignored when the number of bolts, n, is less than or equal to 9. For connections with 10 to 12 bolts, use $e = n - 4$ and a 1.25 multiplier on the calculated eccentricity coefficient C. For SSL holes, eccentricity can be ignored up to $n = 12$.
2. Check the plate for shear yielding and shear rupture. Plate buckling will not control for the conventional configuration.

Extended Configuration

The following method is useful when the dimensional and other limitations of the conventional method cannot be satisfied. This procedure can be used to determine the strength of single plate shear connections with multiple vertical rows or in the extended configuration, as shown in Figure 10–12.

Dimensional Limitations

1. The number of bolts, n, is not limited.
2. The distance from the bolt line to the weld line, a, is not limited.
3. The use of holes must satisfy AISC Specification Section J3.2 requirements.
4. The horizontal and vertical edge distances, L_{eh} and L_{ev}, must satisfy AISC Specification Table J3.4 requirements.

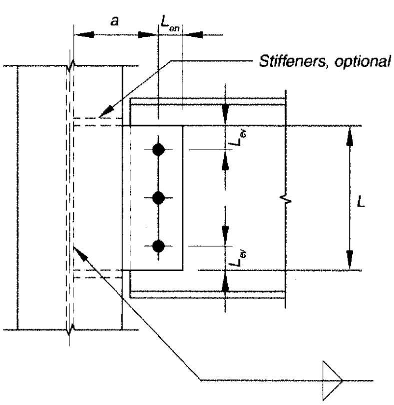


Figure 10–12. Extended single-plate connection.

Design Checks

1. Determine the bolt group required for bolt shear and bolt bearing with eccentricity $e = a$, where a is defined as the distance from the support to the first row of bolts. Exception: alternative considerations of the design eccentricity are acceptable when justified by rational analysis. For example, see Sherman and Ghorbanpoor (2002).
2. Determine the maximum plate thickness permitted such that the plate moment strength does not exceed the moment strength of the bolt group in shear, as follows:

$$t_{max} = \frac{6M_{max}}{F_y d^2}$$

where

$M_{max} = 1.25F_v A_b C'$
$1.25F_v$ = shear strength of an individual bolt from AISC Specification Table J3.2, ksi, multiplied by a factor of 1.25 to remove the 20 percent reduction for uneven for distribution in end-loaded bolt groups (Kulak, 2002). The joint in question is not end-loaded.
A_b = area of an individual bolt, in.^{2}
C' = coefficient from Part 7 for the moment-only case (instantaneous center of rotation at the centroid of the bolt group)
F_y = plate specified yield stress, ksi
d = plate depth, in.

The foregoing check is made at the nominal strength level, since the check is to ensure ductility, not strength.

Exceptions:
a. For a single vertical row of bolts only, the foregoing criterion need not be satisfied if either the beam web or the plate satisfies $t \le d_b/2 + {}^1/_{16}$ and both satisfy $L_{eh} \ge 2d_b$.
b. For a double vertical row of bolts only, the foregoing criterion need not be satisfied if both the beam web and the plate satisfy $t \le d_b/2 + {}^1/_{16}$ and $L_{eh} \ge 2d_b$.
3. Check the plate for shear yielding, shear rupture, and block shear rupture.
4. Check the plate for flexure with the von-Mises shear reduction. That is, check the available flexural yielding strength of the plate, ϕM_n or M_n/Ω, based upon a critical stress, F_{cr}, as follows:

$$F_{cr} = \sqrt{F_y^2 - 3f_v^2}$$

$$M_n = F_{cr}Z$$

$$\phi = 0.90 \qquad \Omega = 1.67$$

5. Check the plate for buckling using the double-coped beam procedure given in Part 9.
6. Ensure that the supported beam is braced at points of support.

The design procedure for extended single-plate shear connections permits the column to be designed for an axial force without eccentricity. In some cases, economy may be gained by considering alternative design procedures that allow the transfer of some moment into the support, for example, 5 percent of the beam fixed-end moment, provided that this moment is also considered in the design of the supporting member.

Recommended Plate Length

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 10–3.

Shop and Field Practices

Conventional and extended single-plate connections may be made to the webs of supporting girders and to the flanges of supporting columns. Extended single-plate connections are suitable for connections to the webs of supporting columns when the bolt line is located a sufficient distance beyond the column flanges.

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.

Table 10–9. Single-Plate Connections

Table 10–9 is a design aid for single-plate connections welded to the support and bolted to the supported beam. Available strengths are tabulated for plate material with $F_y = 36$ ksi and $F_u = 58$ ksi.

Tabulated bolt and plate available strengths consider the limit-states of bolt shear, bolt bearing on the plate, shear yielding of the plate, shear rupture of the plate, block shear rupture of the plate, and weld shear. Values are tabulated for two through twelve rows of $^3/_4$-in., $^7/_8$-in., 1-in. and $1^1/_8$-in. diameter ASTM A325, F1852, and A490 bolts at 3-in. spacing. For calculation purposes, plate edge distance, L_{ev}, is in accordance with AISC Specification Section J3.10 and Table J3.4. End distance, L_{eh}, is provided as 2 times the diameter of the bolt, to match tested connections. Weld sizes are tabulated equal to $^5/_8 t_p$.

While the tabular values are based on $a = 3$ in., they may conservatively be used when the distance from the support to the bolt line, a, is between $2^1/_2$ in. and 3 in. The tabulated values are valid for laterally supported beams in steel and composite construction, all types of loading, snug-tightened or pretensioned bolts, and for supported and supporting members of all grades of steel.

Plate
F_y = 36 ksi

Table 10–9a
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

3/4-in. diameter bolts

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
12 (L = 35 1/2)	A325 F1852	N	STD	100	150	118	178	118	178	118	178	—	—	—	—
			SSLT	99.5	149	124	187	127	191	127	191	—	—	—	—
		X	STD	100	150	125	188	148	222	148	222	—	—	—	—
			SSLT	99.5	149	124	187	149	224	159	239	—	—	—	—
	A490	N	STD	100	150	125	188	148	222	148	222	—	—	—	—
			SSLT	99.5	149	124	187	149	224	159	239	—	—	—	—
		X	STD	100	150	125	188	150	225	175	263	—	—	—	—
			SSLT	99.5	149	124	187	149	224	174	261	—	—	—	—
11 (L = 32 1/2)	A325 F1852	N	STD	92.1	138	111	166	111	166	111	166	—	—	—	—
			SSLT	91.4	137	114	171	117	175	117	175	—	—	—	—
		X	STD	92.1	138	115	173	138	207	139	208	—	—	—	—
			SSLT	91.4	137	114	171	137	206	146	219	—	—	—	—
	A490	N	STD	92.1	138	115	173	138	207	139	208	—	—	—	—
			SSLT	91.4	137	114	171	137	206	146	219	—	—	—	—
		X	STD	92.1	138	115	173	138	207	161	242	—	—	—	—
			SSLT	91.4	137	114	171	137	206	160	240	—	—	—	—
10 (L = 29 1/2)	A325 F1852	N	STD	84.0	126	103	155	103	155	103	155	—	—	—	—
			SSLT	83.3	125	104	156	106	159	106	159	—	—	—	—
		X	STD	84.0	126	105	157	126	189	129	194	—	—	—	—
			SSLT	83.3	125	104	156	125	187	133	199	—	—	—	—
	A490	N	STD	84.0	126	105	157	126	189	129	194	—	—	—	—
			SSLT	83.3	125	104	156	125	187	133	199	—	—	—	—
		X	STD	84.0	126	105	157	126	189	147	220	—	—	—	—
			SSLT	83.3	125	104	156	125	187	146	219	—	—	—	—
9 (L = 26 1/2)	A325 F1852	N	STD	75.9	114	94.8	142	95.4	143	95.4	143	—	—	—	—
			SSLT	75.2	113	94.0	141	95.4	143	95.4	143	—	—	—	—
		X	STD	75.9	114	94.8	142	114	171	119	179	—	—	—	—
			SSLT	75.2	113	94.0	141	113	169	119	179	—	—	—	—
	A490	N	STD	75.9	114	94.8	142	114	171	119	179	—	—	—	—
			SSLT	75.2	113	94.0	141	113	169	119	179	—	—	—	—
		X	STD	75.9	114	94.8	142	114	171	133	199	—	—	—	—
			SSLT	75.2	113	94.0	141	113	169	132	197	—	—	—	—
Weld Size				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard Holes
SSLT = Short-slotted holes transverse to direction of load
– indicates that the plate thickness is greater than $d_b/2 + 1/16$ in.
Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included
X = Threads Excluded

Table 10–9a (continued)
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

3/4-in. diameter bolts

Plate F_y = 36 ksi

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
8 (L = 23 1/2)	A325 F1852	N	STD	67.8	102	84.7	127	84.8	127	84.8	127	—	—	—	—
			SSLT	67.1	101	83.9	126	84.8	127	84.8	127	—	—	—	—
		X	STD	67.8	102	84.7	127	102	153	106	159	—	—	—	—
			SSLT	67.1	101	83.9	126	101	151	106	159	—	—	—	—
	A490	N	STD	67.8	102	84.7	127	102	153	106	159	—	—	—	—
			SSLT	67.1	101	83.9	126	101	151	106	159	—	—	—	—
		X	STD	67.8	102	84.7	127	102	153	119	178	—	—	—	—
			SSLT	67.1	101	83.9	126	101	151	117	176	—	—	—	—
7 (L = 20 1/2)	A325 F1852	N	STD	59.7	89.5	74.2	111	74.2	111	74.2	111	—	—	—	—
			SSLT	59.0	88.5	73.7	111	74.2	111	74.2	111	—	—	—	—
		X	STD	59.7	89.5	74.6	112	89.5	134	92.8	139	—	—	—	—
			SSLT	59.0	88.5	73.7	111	88.5	133	92.8	139	—	—	—	—
	A490	N	STD	59.7	89.5	74.6	112	89.5	134	92.8	139	—	—	—	—
			SSLT	59.0	88.5	73.7	111	88.5	133	92.8	139	—	—	—	—
		X	STD	59.7	89.5	74.6	112	89.5	134	104	157	—	—	—	—
			SSLT	59.0	88.5	73.7	111	88.5	133	103	155	—	—	—	—
6 (L = 17 1/2)	A325 F1852	N	STD	51.6	77.4	63.6	95.4	63.6	95.4	63.6	95.4	—	—	—	—
			SSLT	50.9	76.3	63.6	95.4	63.6	95.4	63.6	95.4	—	—	—	—
		X	STD	51.6	77.4	64.5	96.7	77.4	116	79.5	119	—	—	—	—
			SSLT	50.9	76.3	63.6	95.4	76.3	115	79.5	119	—	—	—	—
	A490	N	STD	51.6	77.4	64.5	96.7	77.4	116	79.5	119	—	—	—	—
			SSLT	50.9	76.3	63.6	95.4	76.3	115	79.5	119	—	—	—	—
		X	STD	51.6	77.4	64.5	96.7	77.4	116	90.3	135	—	—	—	—
			SSLT	50.9	76.3	63.6	95.4	76.3	115	89.1	134	—	—	—	—
5 (L = 14 1/2)	A325 F1852	N	STD	43.5	65.2	53.0	79.5	53.0	79.5	53.0	79.5	—	—	—	—
			SSLT	42.8	64.2	53.0	79.5	53.0	79.5	53.0	79.5	—	—	—	—
		X	STD	43.5	65.2	54.3	81.5	65.2	97.8	66.3	99.4	—	—	—	—
			SSLT	42.8	64.2	53.5	80.2	64.2	96.3	66.3	99.4	—	—	—	—
	A490	N	STD	43.5	65.2	54.3	81.5	65.2	97.8	66.3	99.4	—	—	—	—
			SSLT	42.8	64.2	53.5	80.2	64.2	96.3	66.3	99.4	—	—	—	—
		X	STD	43.5	65.2	54.3	81.5	65.2	97.8	76.1	114	—	—	—	—
			SSLT	42.8	64.2	53.5	80.2	64.2	96.3	74.9	112	—	—	—	—
Weld Size				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard Holes

SSLT = Short-slotted holes transverse to direction of load

– indicates that the plate thickness is greater than $d_b/2 + 1/16$ in.

Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included

X = Threads Excluded

Table 10–9a (continued)
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

Plate
F_y = 36 ksi

3/4-in. diameter bolts

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
4 (L = 11 1/2)	A325 F1852	N	STD	34.8	52.2	42.4	63.6	42.4	63.6	42.4	63.6	—	—	—	—
			SSLT	34.7	52.0	42.4	63.6	42.4	63.6	42.4	63.6	—	—	—	—
		X	STD	34.8	52.2	43.5	65.3	52.2	78.3	53.0	79.5	—	—	—	—
			SSLT	34.7	52.0	43.4	65.1	52.0	78.1	53.0	79.5	—	—	—	—
	A490	N	STD	34.8	52.2	43.5	65.3	52.2	78.3	53.0	79.5	—	—	—	—
			SSLT	34.7	52.0	43.4	65.1	52.0	78.1	53.0	79.5	—	—	—	—
		X	STD	34.8	52.2	43.5	65.3	52.2	78.3	60.9	91.4	—	—	—	—
			SSLT	34.7	52.0	43.4	65.1	52.0	78.1	60.7	91.1	—	—	—	—
3 (L = 8 1/2)	A325 F1852	N	STD	25.6	38.3	31.8	47.7	31.8	47.7	31.8	47.7	—	—	—	—
			SSLT	25.6	38.3	31.8	47.7	31.8	47.7	31.8	47.7	—	—	—	—
		X	STD	25.6	38.3	31.9	47.9	38.3	57.5	39.8	59.6	—	—	—	—
			SSLT	25.6	38.3	31.9	47.9	38.3	57.5	39.8	59.6	—	—	—	—
	A490	N	STD	25.6	38.3	31.9	47.9	38.3	57.5	39.8	59.6	—	—	—	—
			SSLT	25.6	38.3	31.9	47.9	38.3	57.5	39.8	59.6	—	—	—	—
		X	STD	25.6	38.3	31.9	47.9	38.3	57.5	44.7	67.1	—	—	—	—
			SSLT	25.6	38.3	31.9	47.9	38.3	57.5	44.7	67.1	—	—	—	—
2 (L = 5 1/2)	A325 F1852	N	STD	16.3	24.5	20.4	30.6	21.2	31.8	21.2	31.8	—	—	—	—
			SSLT	16.3	24.5	20.4	30.6	21.2	31.8	21.2	31.8	—	—	—	—
		X	STD	16.3	24.5	20.4	30.6	24.5	36.7	26.5	39.8	—	—	—	—
			SSLT	16.3	24.5	20.4	30.6	24.5	36.7	26.5	39.8	—	—	—	—
	A490	N	STD	16.3	24.5	20.4	30.6	24.5	36.7	26.5	39.8	—	—	—	—
			SSLT	16.3	24.5	20.4	30.6	24.5	36.7	26.5	39.8	—	—	—	—
		X	STD	16.3	24.5	20.4	30.6	24.5	36.7	28.5	42.8	—	—	—	—
			SSLT	16.3	24.5	20.4	30.6	24.5	36.7	28.5	42.8	—	—	—	—
Weld Size				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard Holes
SSLT = Short-slotted holes transverse to direction of load
– indicates that the plate thickness is greater than $d_b/2$ + 1/16 in.
Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included
X = Threads Excluded

Table 10–9a (continued)
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

7/8-in. diameter bolts

Plate F_y = 36 ksi

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
12 (L = 36)	A325 F1852	N	STD	102	153	128	192	153	230	161	242	161	242	—	—
			SSLT	102	152	127	190	152	228	173	260	173	260	—	—
		X	STD	102	153	128	192	153	230	179	268	201	302	—	—
			SSLT	102	152	127	190	152	228	178	267	203	305	—	—
	A490	N	STD	102	153	128	192	153	230	179	268	201	302	—	—
			SSLT	102	152	127	190	152	228	178	267	203	305	—	—
		X	STD	102	153	128	192	153	230	179	268	204	307	—	—
			SSLT	102	152	127	190	152	228	178	267	203	305	—	—
11 (L = 33)	A325 F1852	N	STD	94.1	141	118	176	141	212	151	226	151	226	—	—
			SSLT	93.4	140	117	175	140	210	159	238	159	238	—	—
		X	STD	94.1	141	118	176	141	212	165	247	188	282	—	—
			SSLT	93.4	140	117	175	140	210	164	245	187	280	—	—
	A490	N	STD	94.1	141	118	176	141	212	165	247	188	282	—	—
			SSLT	93.4	140	117	175	140	210	164	245	187	280	—	—
		X	STD	94.1	141	118	176	141	212	165	247	188	282	—	—
			SSLT	93.4	140	117	175	140	210	164	245	187	280	—	—
10 (L = 30)	A325 F1852	N	STD	86.0	129	108	161	129	194	141	211	141	211	—	—
			SSLT	85.3	128	107	160	128	192	144	216	144	216	—	—
		X	STD	86.0	129	108	161	129	194	151	226	172	258	—	—
			SSLT	85.3	128	107	160	128	192	149	224	171	256	—	—
	A490	N	STD	86.0	129	108	161	129	194	151	226	172	258	—	—
			SSLT	85.3	128	107	160	128	192	149	224	171	256	—	—
		X	STD	86.0	129	108	161	129	194	151	226	172	258	—	—
			SSLT	85.3	128	107	160	128	192	149	224	171	256	—	—
9 (L = 27)	A325 F1852	N	STD	77.9	117	97.4	146	117	175	130	195	130	195	—	—
			SSLT	77.2	116	96.5	145	116	174	130	195	130	195	—	—
		X	STD	77.9	117	97.4	146	117	175	136	205	156	234	—	—
			SSLT	77.2	116	96.5	145	116	174	135	203	154	232	—	—
	A490	N	STD	77.9	117	97.4	146	117	175	136	205	156	234	—	—
			SSLT	77.2	116	96.5	145	116	174	135	203	154	232	—	—
		X	STD	77.9	117	97.4	146	117	175	136	205	156	234	—	—
			SSLT	77.2	116	96.5	145	116	174	135	203	154	232	—	—
Weld Size				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard Holes
SSLT = Short-slotted holes transverse to direction of load
– indicates that the plate thickness is greater than $d_b/2 + 1/16$ in.
Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included
X = Threads Excluded

Table 10–9a (continued)
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

Plate
F_y = 36 ksi

7/8-in. diameter bolts

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
8 (L = 24)	A325 F1852	N	STD	69.6	104	87.0	131	104	157	115	173	115	173	—	—
			SSLT	69.1	104	86.4	130	104	156	115	173	115	173	—	—
		X	STD	69.6	104	87.0	131	104	157	122	183	139	209	—	—
			SSLT	69.1	104	86.4	130	104	156	121	181	138	207	—	—
	A490	N	STD	69.6	104	87.0	131	104	157	122	183	139	209	—	—
			SSLT	69.1	104	86.4	130	104	156	121	181	138	207	—	—
		X	STD	69.6	104	87.0	131	104	157	122	183	139	209	—	—
			SSLT	69.1	104	86.4	130	104	156	121	181	138	207	—	—
7 (L = 21)	A325 F1852	N	STD/ SSLT	60.9	91.4	76.1	114	91.4	137	101	152	101	152	—	—
		X		60.9	91.4	76.1	114	91.4	137	107	160	122	183	—	—
	A490	N		60.9	91.4	76.1	114	91.4	137	107	160	122	183	—	—
		X		60.9	91.4	76.1	114	91.4	137	107	160	122	183	—	—
6 (L = 18)	A325 F1852	N	STD/ SSLT	52.2	78.3	65.3	97.9	78.3	117	86.6	130	86.6	130	—	—
		X		52.2	78.3	65.3	97.9	78.3	117	91.4	137	104	157	—	—
	A490	N		52.2	78.3	65.3	97.9	78.3	117	91.4	137	104	157	—	—
		X		52.2	78.3	65.3	97.9	78.3	117	91.4	137	104	157	—	—
5 (L = 15)	A325 F1852	N	STD/ SSLT	43.5	65.3	54.4	81.6	65.3	97.9	72.2	108	72.2	108	—	—
		X		43.5	65.3	54.4	81.6	65.3	97.9	76.1	114	87.0	131	—	—
	A490	N		43.5	65.3	54.4	81.6	65.3	97.9	76.1	114	87.0	131	—	—
		X		43.5	65.3	54.4	81.6	65.3	97.9	76.1	114	87.0	131	—	—
4 (L = 12)	A325 F1852	N	STD/ SSLT	34.8	52.2	43.5	65.3	52.2	78.3	57.7	86.6	57.7	86.6	—	—
		X		34.8	52.2	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	—	—
	A490	N		34.8	52.2	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	—	—
		X		34.8	52.2	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	—	—
3 (L = 9)	A325 F1852	N	STD/ SSLT	26.1	39.1	32.6	48.9	39.1	58.7	43.3	64.9	43.3	64.9	—	—
		X		26.1	39.1	32.6	48.9	39.1	58.7	45.7	68.5	52.2	78.3	—	—
	A490	N		26.1	39.1	32.6	48.9	39.1	58.7	45.7	68.5	52.2	78.3	—	—
		X		26.1	39.1	32.6	48.9	39.1	58.7	45.7	68.5	52.2	78.3	—	—
2 (L = 6)	A325 F1852	N	STD/ SSLT	17.4	26.1	21.8	32.6	26.1	39.1	28.9	43.3	28.9	43.3	—	—
		X		17.4	26.1	21.8	32.6	26.1	39.1	30.4	45.7	34.8	52.2	—	—
	A490	N		17.4	26.1	21.8	32.6	26.1	39.1	30.4	45.7	34.8	52.2	—	—
		X		17.4	26.1	21.8	32.6	26.1	39.1	30.4	45.7	34.8	52.2	—	—
Weld Size				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard Holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
– indicates that the plate thickness is greater than $d_b/2 + 1/16$ in.
Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included
X = Threads Excluded

1-in. diameter bolts

Table 10–9a (continued)
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

Plate F_y = 36 ksi

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
12 ($L = 36^1/_2$)	A325 F1852	N	STD	100	150	125	188	150	225	175	263	200	300	210	316
			SSLT	100	150	125	188	150	225	175	263	200	300	225	338
		X	STD	100	150	125	188	150	225	175	263	200	300	225	338
			SSLT	100	150	125	188	150	225	175	263	200	300	225	338
	A490	N	STD	100	150	125	188	150	225	175	263	200	300	225	338
			SSLT	100	150	125	188	150	225	175	263	200	300	225	338
		X	STD	100	150	125	188	150	225	175	263	200	300	225	338
			SSLT	100	150	125	188	150	225	175	263	200	300	225	338
11 ($L = 33^1/_2$)	A325 F1852	N	STD	91.9	138	115	172	138	207	161	241	184	276	197	295
			SSLT	91.9	138	115	172	138	207	161	241	184	276	207	310
		X	STD	91.9	138	115	172	138	207	161	241	184	276	207	310
			SSLT	91.9	138	115	172	138	207	161	241	184	276	207	310
	A490	N	STD	91.9	138	115	172	138	207	161	241	184	276	207	310
			SSLT	91.9	138	115	172	138	207	161	241	184	276	207	310
		X	STD	91.9	138	115	172	138	207	161	241	184	276	207	310
			SSLT	91.9	138	115	172	138	207	161	241	184	276	207	310
10 ($L = 30^1/_2$)	A325 F1852	N	STD	83.7	126	105	157	126	188	147	220	167	251	184	275
			SSLT	83.7	126	105	157	126	188	147	220	167	251	188	283
		X	STD	83.7	126	105	157	126	188	147	220	167	251	188	283
			SSLT	83.7	126	105	157	126	188	147	220	167	251	188	283
	A490	N	STD	83.7	126	105	157	126	188	147	220	167	251	188	283
			SSLT	83.7	126	105	157	126	188	147	220	167	251	188	283
		X	STD	83.7	126	105	157	126	188	147	220	167	251	188	283
			SSLT	83.7	126	105	157	126	188	147	220	167	251	188	283
9 ($L = 27^1/_2$)	A325 F1852	N	STD/ SSLT	75.6	113	94.5	142	113	170	132	198	151	227	170	254
		X		75.6	113	94.5	142	113	170	132	198	151	227	170	255
	A490	N		75.6	113	94.5	142	113	170	132	198	151	227	170	255
		X		75.6	113	94.5	142	113	170	132	198	151	227	170	255
8 ($L = 24^1/_2$)	A325 F1852	N	STD/ SSLT	67.4	101	84.3	126	101	152	118	177	135	202	151	226
		X		67.4	101	84.3	126	101	152	118	177	135	202	152	228
	A490	N		67.4	101	84.3	126	101	152	118	177	135	202	152	228
		X		67.4	101	84.3	126	101	152	118	177	135	202	152	228
Weld Size				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard Holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included
X = Threads Excluded

Table 10–9a (continued)
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

Plate
F_y = 36 ksi

1-in. diameter bolts

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
7 (L = 21 1/2)	A325	N	STD/SSLT	59.3	88.9	74.1	111	88.9	133	104	156	119	178	132	198
	F1852	X		59.3	88.9	74.1	111	88.9	133	104	156	119	178	133	200
	A490	N		59.3	88.9	74.1	111	88.9	133	104	156	119	178	133	200
		X		59.3	88.9	74.1	111	88.9	133	104	156	119	178	133	200
6 (L = 18 1/2)	A325	N	STD/SSLT	51.1	76.7	63.9	95.8	76.7	115	89.4	134	102	153	113	170
	F1852	X		51.1	76.7	63.9	95.8	76.7	115	89.4	134	102	153	115	173
	A490	N		51.1	76.7	63.9	95.8	76.7	115	89.4	134	102	153	115	173
		X		51.1	76.7	63.9	95.8	76.7	115	89.4	134	102	153	115	173
5 (L = 15 1/2)	A325	N	STD/SSLT	43.0	64.4	53.7	80.5	64.4	96.7	75.2	113	85.9	129	94.2	141
	F1852	X		43.0	64.4	53.7	80.5	64.4	96.7	75.2	113	85.9	129	96.7	145
	A490	N		43.0	64.4	53.7	80.5	64.4	96.7	75.2	113	85.9	129	96.7	145
		X		43.0	64.4	53.7	80.5	64.4	96.7	75.2	113	85.9	129	96.7	145
4 (L = 12 1/2)	A325	N	STD/SSLT	34.8	52.2	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	75.4	113
	F1852	X		34.8	52.2	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	78.3	117
	A490	N		34.8	52.2	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	78.3	117
		X		34.8	52.2	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	78.3	117
3 (L = 9 1/2)	A325	N	STD/SSLT	26.6	40.0	33.3	50.0	40.0	59.9	46.6	69.9	53.3	79.9	56.5	84.8
	F1852	X		26.6	40.0	33.3	50.0	40.0	59.9	46.6	69.9	53.3	79.9	59.9	89.9
	A490	N		26.6	40.0	33.3	50.0	40.0	59.9	46.6	69.9	53.3	79.9	59.9	89.9
		X		26.6	40.0	33.3	50.0	40.0	59.9	46.6	69.9	53.3	79.9	59.9	89.9
2 (L = 6 1/2)	A325	N	STD/SSLT	18.5	27.7	23.1	34.7	27.7	41.6	32.4	48.5	37.0	55.5	37.7	56.5
	F1852	X		18.5	27.7	23.1	34.7	27.7	41.6	32.4	48.5	37.0	55.5	41.6	62.4
	A490	N		18.5	27.7	23.1	34.7	27.7	41.6	32.4	48.5	37.0	55.5	41.6	62.4
		X		18.5	27.7	23.1	34.7	27.7	41.6	32.4	48.5	37.0	55.5	41.6	62.4
Weld Size				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard Holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included
X = Threads Excluded

$1^1/_8$-in. diameter bolts

Table 10–9a (continued) Single-Plate Connections

Bolt, Weld, and Single-Plate Available Strengths, kips

Plate F_y = 36 ksi

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				$^5/_{16}$		$^3/_8$		$^7/_{16}$		$^1/_2$		$^9/_{16}$		$^5/_8$	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
12 (L = 37)	A325 F1852	N	STD	120	179	144	215	167	251	191	287	215	323	239	359
			SSLT	120	179	144	215	167	251	191	287	215	323	239	359
		X	STD	120	179	144	215	167	251	191	287	215	323	239	359
			SSLT	120	179	144	215	167	251	191	287	215	323	239	359
	A490	N	STD	120	179	144	215	167	251	191	287	215	323	239	359
			SSLT	120	179	144	215	167	251	191	287	215	323	239	359
		X	STD	120	179	144	215	167	251	191	287	215	323	239	359
			SSLT	120	179	144	215	167	251	191	287	215	323	239	359
11 (L = 34)	A325 F1852	N	STD	110	165	132	198	154	231	176	264	198	297	220	330
			SSLT	110	165	132	198	154	231	176	264	198	297	220	330
		X	STD	110	165	132	198	154	231	176	264	198	297	220	330
			SSLT	110	165	132	198	154	231	176	264	198	297	220	330
	A490	N	STD	110	165	132	198	154	231	176	264	198	297	220	330
			SSLT	110	165	132	198	154	231	176	264	198	297	220	330
		X	STD	110	165	132	198	154	231	176	264	198	297	220	330
			SSLT	110	165	132	198	154	231	176	264	198	297	220	330
10 (L = 31)	A325 F1852	N	STD	101	151	121	181	141	211	161	241	181	272	201	302
			SSLT	101	151	121	181	141	211	161	241	181	272	201	302
		X	STD	101	151	121	181	141	211	161	241	181	272	201	302
			SSLT	101	151	121	181	141	211	161	241	181	272	201	302
	A490	N	STD	101	151	121	181	141	211	161	241	181	272	201	302
			SSLT	101	151	121	181	141	211	161	241	181	272	201	302
		X	STD	101	151	121	181	141	211	161	241	181	272	201	302
			SSLT	101	151	121	181	141	211	161	241	181	272	201	302
9 (L = 28)	A325 F1852	N	STD/ SSLT	91.1	137	109	164	128	191	146	219	164	246	182	273
		X		91.1	137	109	164	128	191	146	219	164	246	182	273
	A490	N		91.1	137	109	164	128	191	146	219	164	246	182	273
		X		91.1	137	109	164	128	191	146	219	164	246	182	273
8 (L = 25)	A325 F1852	N	STD/ SSLT	81.6	122	97.9	147	114	171	131	196	147	220	163	245
		X		81.6	122	97.9	147	114	171	131	196	147	220	163	245
	A490	N		81.6	122	97.9	147	114	171	131	196	147	220	163	245
		X		81.6	122	97.9	147	114	171	131	196	147	220	163	245
Weld Size				$^1/_4$		$^1/_4$		$^5/_{16}$		$^5/_{16}$		$^3/_8$		$^7/_{16}$	

STD = Standard Holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included
X = Threads Excluded

Table 10–9a (continued)
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

Plate F_y = 36 ksi

$1\frac{1}{8}$-in. diameter bolts

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				$^5/_{16}$		$^3/_8$		$^7/_{16}$		$^1/_2$		$^9/_{16}$		$^5/_8$	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
7 (*L* = 22)	A325 F1852	N	STD/ SSLT	72.0	108	86.5	130	101	151	115	173	130	195	144	216
		X		72.0	108	86.5	130	101	151	115	173	130	195	144	216
	A490	N		72.0	108	86.5	130	101	151	115	173	130	195	144	216
		X		72.0	108	86.5	130	101	151	115	173	130	195	144	216
6 (*L* = 19)	A325 F1852	N	STD/ SSLT	62.5	93.8	75.0	113	87.5	131	100	150	113	169	125	188
		X		62.5	93.8	75.0	113	87.5	131	100	150	113	169	125	188
	A490	N		62.5	93.8	75.0	113	87.5	131	100	150	113	169	125	188
		X		62.5	93.8	75.0	113	87.5	131	100	150	113	169	125	188
5 (*L* = 16)	A325 F1852	N	STD/ SSLT	53.0	79.5	63.6	95.4	74.2	111	84.8	127	95.4	143	106	159
		X		53.0	79.5	63.6	95.4	74.2	111	84.8	127	95.4	143	106	159
	A490	N		53.0	79.5	63.6	95.4	74.2	111	84.8	127	95.4	143	106	159
		X		53.0	79.5	63.6	95.4	74.2	111	84.8	127	95.4	143	106	159
4 (*L* = 13)	A325 F1852	N	STD/ SSLT	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	78.3	117	87.0	131
		X		43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	78.3	117	87.0	131
	A490	N		43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	78.3	117	87.0	131
		X		43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	78.3	117	87.0	131
3 (*L* = 10)	A325 F1852	N	STD/ SSLT	34.0	51.0	40.8	61.2	47.6	71.4	54.4	81.6	61.2	91.8	68.0	102
		X		34.0	51.0	40.8	61.2	47.6	71.4	54.4	81.6	61.2	91.8	68.0	102
	A490	N		34.0	51.0	40.8	61.2	47.6	71.4	54.4	81.6	61.2	91.8	68.0	102
		X		34.0	51.0	40.8	61.2	47.6	71.4	54.4	81.6	61.2	91.8	68.0	102
2 (*L* = 7)	A325 F1852	N	STD/ SSLT	24.5	36.7	29.4	44.0	34.3	51.4	39.1	58.7	44.0	66.1	47.7	71.6
		X		24.5	36.7	29.4	44.0	34.3	51.4	39.1	58.7	44.0	66.1	48.9	73.4
	A490	N		24.5	36.7	29.4	44.0	34.3	51.4	39.1	58.7	44.0	66.1	48.9	73.4
		X		24.5	36.7	29.4	44.0	34.3	51.4	39.1	58.7	44.0	66.1	48.9	73.4
Weld Size				$^1/_4$		$^1/_4$		$^5/_{16}$		$^5/_{16}$		$^3/_8$		$^7/_{16}$	

STD = Standard Holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included
X = Threads Excluded

Table 10–9b
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

3/4-in. diameter bolts | **Plate F_y = 50 ksi**

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
12 (L = $35^1/_2$)	A325 F1852	N	STD	118	178	118	178	118	178	118	178	—	—	—	—
			SSLT	122	183	127	191	127	191	127	191	—	—	—	—
		X	STD	122	183	148	222	148	222	148	222	—	—	—	—
			SSLT	122	183	152	229	159	239	159	239	—	—	—	—
	A490	N	STD	122	183	148	222	148	222	148	222	—	—	—	—
			SSLT	122	183	152	229	159	239	159	239	—	—	—	—
		X	STD	122	183	152	229	183	274	185	277	—	—	—	—
			SSLT	122	183	152	229	183	274	199	298	—	—	—	—
11 (L = $32^1/_2$)	A325 F1852	N	STD	111	166	111	166	111	166	111	166	—	—	—	—
			SSLT	112	167	117	175	117	175	117	175	—	—	—	—
		X	STD	112	167	139	208	139	208	139	208	—	—	—	—
			SSLT	112	167	139	209	146	219	146	219	—	—	—	—
	A490	N	STD	112	167	139	208	139	208	139	208	—	—	—	—
			SSLT	112	167	139	209	146	219	146	219	—	—	—	—
		X	STD	112	167	139	209	167	251	173	260	—	—	—	—
			SSLT	112	167	139	209	167	251	182	273	—	—	—	—
10 (L = $29^1/_2$)	A325 F1852	N	STD	101	152	103	155	103	155	103	155	—	—	—	—
			SSLT	101	152	106	159	106	159	106	159	—	—	—	—
		X	STD	101	152	126	190	129	194	129	194	—	—	—	—
			SSLT	101	152	126	190	133	199	133	199	—	—	—	—
	A490	N	STD	101	152	126	190	129	194	129	194	—	—	—	—
			SSLT	101	152	126	190	133	199	133	199	—	—	—	—
		X	STD	101	152	126	190	152	228	161	242	—	—	—	—
			SSLT	101	152	126	190	152	228	166	249	—	—	—	—
9 (L = $26^1/_2$)	A325 F1852	N	STD/ SSLT	90.8	136	95.4	143	95.4	143	95.4	143	—	—	—	—
		X		90.8	136	113	170	119	179	119	179	—	—	—	—
	A490	N		90.8	136	113	170	119	179	119	179	—	—	—	—
		X		90.8	136	113	170	136	204	149	224	—	—	—	—
8 (L = $23^1/_2$)	A325 F1852	N	STD/ SSLT	80.4	121	84.8	127	84.8	127	84.8	127	—	—	—	—
		X		80.4	121	101	151	106	159	106	159	—	—	—	—
	A490	N		80.4	121	101	151	106	159	106	159	—	—	—	—
		X		80.4	121	101	151	121	181	133	199	—	—	—	—
Weld Size				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard Holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
– indicates that the plate thickness is greater than $d_b/2 + {}^1/_{16}$ in.
Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included
X = Threads Excluded

Plate
F_y = 50 ksi

Table 10–9b (continued)
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

3/4-in. diameter bolts

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
7 (L = 20 1/2)	A325 F1852	N	STD/SSLT	70.1	105	74.2	111	74.2	111	74.2	111	—	—	—	—
		X		70.1	105	87.6	131	92.8	139	92.8	139	—	—	—	—
	A490	N		70.1	105	87.6	131	92.8	139	92.8	139	—	—	—	—
		X		70.1	105	87.6	131	105	158	116	174	—	—	—	—
6 (L = 17 1/2)	A325 F1852	N	STD/SSLT	59.7	89.6	63.6	95.4	63.6	95.4	63.6	95.4	—	—	—	—
		X		59.7	89.6	74.6	112	79.5	119	79.5	119	—	—	—	—
	A490	N		59.7	89.6	74.6	112	79.5	119	79.5	119	—	—	—	—
		X		59.7	89.6	74.6	112	89.6	134	99.4	149	—	—	—	—
5 (L = 14 1/2)	A325 F1852	N	STD/SSLT	49.4	74.0	53.0	79.5	53.0	79.5	53.0	79.5	—	—	—	—
		X		49.4	74.0	61.7	92.5	66.3	99.4	66.3	99.4	—	—	—	—
	A490	N		49.4	74.0	61.7	92.5	66.3	99.4	66.3	99.4	—	—	—	—
		X		49.4	74.0	61.7	92.5	74.0	111	82.8	124	—	—	—	—
4 (L = 11 1/2)	A325 F1852	N	STD/SSLT	39.0	58.5	42.4	63.6	42.4	63.6	42.4	63.6	—	—	—	—
		X		39.0	58.5	48.8	73.1	53.0	79.5	53.0	79.5	—	—	—	—
	A490	N		39.0	58.5	48.8	73.1	53.0	79.5	53.0	79.5	—	—	—	—
		X		39.0	58.5	48.8	73.1	58.5	87.8	66.3	99.4	—	—	—	—
3 (L = 8 1/2)	A325 F1852	N	STD/SSLT	28.6	43.0	31.8	47.7	31.8	47.7	31.8	47.7	—	—	—	—
		X		28.6	43.0	35.8	53.7	39.8	59.6	39.8	59.6	—	—	—	—
	A490	N		28.6	43.0	35.8	53.7	39.8	59.6	39.8	59.6	—	—	—	—
		X		28.6	43.0	35.8	53.7	43.0	64.4	49.7	74.6	—	—	—	—
2 (L = 5 1/2)	A325 F1852	N	STD/SSLT	18.3	27.4	21.2	31.8	21.2	31.8	21.2	31.8	—	—	—	—
		X		18.3	27.4	22.9	34.3	26.5	39.8	26.5	39.8	—	—	—	—
	A490	N		18.3	27.4	22.9	34.3	26.5	39.8	26.5	39.8	—	—	—	—
		X		18.3	27.4	22.9	34.3	27.4	41.1	32.0	48.0	—	—	—	—
Weld Size				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard Holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
– indicates that the plate thickness is greater than $d_b/2 + 1/16$ in.
Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included
X = Threads Excluded

7/8-in. diameter bolts

Table 10–9b (continued)
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

Plate F_y = 50 ksi

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
12 (*L* = 36)	A325 F1852	N	STD	117	176	146	219	161	242	161	242	161	242	—	—
			SSLT	117	176	146	219	173	260	173	260	173	260	—	—
		X	STD	117	176	146	219	176	263	201	302	201	302	—	—
			SSLT	117	176	146	219	176	263	205	307	216	325	—	—
	A490	N	STD	117	176	146	219	176	263	201	302	201	302	—	—
			SSLT	117	176	146	219	176	263	205	307	216	325	—	—
		X	STD	117	176	146	219	176	263	205	307	234	351	—	—
			SSLT	117	176	146	219	176	263	205	307	234	351	—	—
11 (*L* = 33)	A325 F1852	N	STD	107	161	134	201	151	226	151	226	151	226	—	—
			SSLT	107	161	134	201	159	238	159	238	159	238	—	—
		X	STD	107	161	134	201	161	241	188	282	189	283	—	—
			SSLT	107	161	134	201	161	241	188	282	198	298	—	—
	A490	N	STD	107	161	134	201	161	241	188	282	189	283	—	—
			SSLT	107	161	134	201	161	241	188	282	198	298	—	—
		X	STD	107	161	134	201	161	241	188	282	215	322	—	—
			SSLT	107	161	134	201	161	241	188	282	215	322	—	—
10 (*L* = 30)	A325 F1852	N	STD	97.5	146	122	183	141	211	141	211	141	211	—	—
			SSLT	97.5	146	122	183	144	216	144	216	144	216	—	—
		X	STD	97.5	146	122	183	146	219	171	256	176	263	—	—
			SSLT	97.5	146	122	183	146	219	171	256	180	271	—	—
	A490	N	STD	97.5	146	122	183	146	219	171	256	176	263	—	—
			SSLT	97.5	146	122	183	146	219	171	256	180	271	—	—
		X	STD	97.5	146	122	183	146	219	171	256	195	293	—	—
			SSLT	97.5	146	122	183	146	219	171	256	195	293	—	—
9 (*L* = 27)	A325 F1852	N	STD/ SSLT	87.8	132	110	165	130	195	130	195	130	195	—	—
		X		87.8	132	110	165	132	197	154	230	162	244	—	—
	A490	N		87.8	132	110	165	132	197	154	230	162	244	—	—
		X		87.8	132	110	165	132	197	154	230	176	263	—	—
8 (*L* = 24)	A325 F1852	N	STD/ SSLT	78.0	117	97.5	146	115	173	115	173	115	173	—	—
		X		78.0	117	97.5	146	117	176	137	205	144	216	—	—
	A490	N		78.0	117	97.5	146	117	176	137	205	144	216	—	—
		X		78.0	117	97.5	146	117	176	137	205	156	234	—	—
Weld Size				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard Holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
– indicates that the plate thickness is greater than $d_b/2 + 1/16$ in.
Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included
X = Threads Excluded

Plate
F_y = 50 ksi

Table 10–9b (continued)
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

7/8-in. diameter bolts

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
7 (L = 21)	A325 F1852	N	STD/SSLT	68.3	102	85.3	128	101	152	101	152	101	152	—	—
		X		68.3	102	85.3	128	102	154	119	179	126	189	—	—
	A490	N		68.3	102	85.3	128	102	154	119	179	126	189	—	—
		X		68.3	102	85.3	128	102	154	119	179	137	205	—	—
6 (L = 18)	A325 F1852	N	STD/SSLT	58.5	87.8	73.1	110	86.6	130	86.6	130	86.6	130	—	—
		X		58.5	87.8	73.1	110	87.8	132	102	154	108	162	—	—
	A490	N		58.5	87.8	73.1	110	87.8	132	102	154	108	162	—	—
		X		58.5	87.8	73.1	110	87.8	132	102	154	117	176	—	—
5 (L = 15)	A325 F1852	N	STD/SSLT	48.8	73.1	60.9	91.4	72.2	108	72.2	108	72.2	108	—	—
		X		48.8	73.1	60.9	91.4	73.1	110	85.3	128	90.2	135	—	—
	A490	N		48.8	73.1	60.9	91.4	73.1	110	85.3	128	90.2	135	—	—
		X		48.8	73.1	60.9	91.4	73.1	110	85.3	128	97.5	146	—	—
4 (L = 12)	A325 F1852	N	STD/SSLT	39.0	58.5	48.8	73.1	57.7	86.6	57.7	86.6	57.7	86.6	—	—
		X		39.0	58.5	48.8	73.1	58.5	87.8	68.3	102	72.2	108	—	—
	A490	N		39.0	58.5	48.8	73.1	58.5	87.8	68.3	102	72.2	108	—	—
		X		39.0	58.5	48.8	73.1	58.5	87.8	68.3	102	78.0	117	—	—
3 (L = 9)	A325 F1852	N	STD/SSLT	29.3	43.9	36.6	54.8	43.3	64.9	43.3	64.9	43.3	64.9	—	—
		X		29.3	43.9	36.6	54.8	43.9	65.8	51.2	76.8	54.1	81.2	—	—
	A490	N		29.3	43.9	36.6	54.8	43.9	65.8	51.2	76.8	54.1	81.2	—	—
		X		29.3	43.9	36.6	54.8	43.9	65.8	51.2	76.8	58.5	87.8	—	—
2 (L = 6)	A325 F1852	N	STD/SSLT	19.5	29.3	24.4	36.6	28.9	43.3	28.9	43.3	28.9	43.3	—	—
		X		19.5	29.3	24.4	36.6	29.3	43.9	34.1	51.2	36.1	54.1	—	—
	A490	N		19.5	29.3	24.4	36.6	29.3	43.9	34.1	51.2	36.1	54.1	—	—
		X		19.5	29.3	24.4	36.6	29.3	43.9	34.1	51.2	39.0	58.5	—	—
Weld Size				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard Holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
– indicates that the plate thickness is greater than $d_b/2 + 1/16$ in.
Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included
X = Threads Excluded

Table 10–9b (continued)
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

1-in. diameter bolts

Plate F_y = 50 ksi

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
12 (L = 36 1/2)	A325 F1852	N	STD	112	168	140	210	168	252	196	294	210	316	210	316
			SSLT	112	168	140	210	168	252	196	294	224	336	226	339
		X	STD	112	168	140	210	168	252	196	294	224	336	252	378
			SSLT	112	168	140	210	168	252	196	294	224	336	252	378
	A490	N	STD	112	168	140	210	168	252	196	294	224	336	252	378
			SSLT	112	168	140	210	168	252	196	294	224	336	252	378
		X	STD	112	168	140	210	168	252	196	294	224	336	252	378
			SSLT	112	168	140	210	168	252	196	294	224	336	252	378
11 (L = 33 1/2)	A325 F1852	N	STD	103	154	129	193	154	232	180	270	197	295	197	295
			SSLT	103	154	129	193	154	232	180	270	206	309	207	311
		X	STD	103	154	129	193	154	232	180	270	206	309	232	348
			SSLT	103	154	129	193	154	232	180	270	206	309	232	348
	A490	N	STD	103	154	129	193	154	232	180	270	206	309	232	348
			SSLT	103	154	129	193	154	232	180	270	206	309	232	348
		X	STD	103	154	129	193	154	232	180	270	206	309	232	348
			SSLT	103	154	129	193	154	232	180	270	206	309	232	348
10 (L = 30 1/2)	A325 F1852	N	STD	93.8	141	117	176	141	211	164	246	184	275	184	275
			SSLT	93.8	141	117	176	141	211	164	246	188	282	188	283
		X	STD	93.8	141	117	176	141	211	164	246	188	282	211	317
			SSLT	93.8	141	117	176	141	211	164	246	188	282	211	317
	A490	N	STD	93.8	141	117	176	141	211	164	246	188	282	211	317
			SSLT	93.8	141	117	176	141	211	164	246	188	282	211	317
		X	STD	93.8	141	117	176	141	211	164	246	188	282	211	317
			SSLT	93.8	141	117	176	141	211	164	246	188	282	211	317
9 (L = 27 1/2)	A325 F1852	N	STD/ SSLT	84.7	127	106	159	127	191	148	222	169	254	170	254
		X		84.7	127	106	159	127	191	148	222	169	254	191	286
	A490	N		84.7	127	106	159	127	191	148	222	169	254	191	286
		X		84.7	127	106	159	127	191	148	222	169	254	191	286
8 (L = 24 1/2)	A325 F1852	N	STD/ SSLT	75.6	113	94.5	142	113	170	132	198	151	226	151	226
		X		75.6	113	94.5	142	113	170	132	198	151	227	170	255
	A490	N		75.6	113	94.5	142	113	170	132	198	151	227	170	255
		X		75.6	113	94.5	142	113	170	132	198	151	227	170	255
Weld Size				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard Holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included
X = Threads Excluded

Table 10–9b (continued)
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

Plate F_y = 50 ksi

1-in. diameter bolts

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
7 ($L = 21^1/_2$)	A325	N	STD/SSLT	66.4	99.6	83.0	125	99.6	149	116	174	132	198	132	198
	F1852	X		66.4	99.6	83.0	125	99.6	149	116	174	133	199	149	224
	A490	N		66.4	99.6	83.0	125	99.6	149	116	174	133	199	149	224
		X		66.4	99.6	83.0	125	99.6	149	116	174	133	199	149	224
6 ($L = 18^1/_2$)	A325	N	STD/SSLT	57.3	85.9	71.6	107	85.9	129	100	150	113	170	113	170
	F1852	X		57.3	85.9	71.6	107	85.9	129	100	150	115	172	129	193
	A490	N		57.3	85.9	71.6	107	85.9	129	100	150	115	172	129	193
		X		57.3	85.9	71.6	107	85.9	129	100	150	115	172	129	193
5 ($L = 15^1/_2$)	A325	N	STD/SSLT	48.1	72.2	60.2	90.3	72.2	108	84.2	126	94.2	141	94.2	141
	F1852	X		48.1	72.2	60.2	90.3	72.2	108	84.2	126	96.3	144	108	162
	A490	N		48.1	72.2	60.2	90.3	72.2	108	84.2	126	96.3	144	108	162
		X		48.1	72.2	60.2	90.3	72.2	108	84.2	126	96.3	144	108	162
4 ($L = 12^1/_2$)	A325	N	STD/SSLT	39.0	58.5	48.8	73.1	58.5	87.8	68.3	102	75.4	113	75.4	113
	F1852	X		39.0	58.5	48.8	73.1	58.5	87.8	68.3	102	78.0	117	87.8	132
	A490	N		39.0	58.5	48.8	73.1	58.5	87.8	68.3	102	78.0	117	87.8	132
		X		39.0	58.5	48.8	73.1	58.5	87.8	68.3	102	78.0	117	87.8	132
3 ($L = 9^1/_2$)	A325	N	STD/SSLT	29.9	44.8	37.3	56.0	44.8	67.2	52.3	78.4	56.5	84.8	56.5	84.8
	F1852	X		29.9	44.8	37.3	56.0	44.8	67.2	52.3	78.4	59.7	89.6	67.2	101
	A490	N		29.9	44.8	37.3	56.0	44.8	67.2	52.3	78.4	59.7	89.6	67.2	101
		X		29.9	44.8	37.3	56.0	44.8	67.2	52.3	78.4	59.7	89.6	67.2	101
2 ($L = 6^1/_2$)	A325	N	STD/SSLT	20.7	31.1	25.9	38.8	31.1	46.6	36.3	54.4	37.7	56.5	37.7	56.5
	F1852	X		20.7	31.1	25.9	38.8	31.1	46.6	36.3	54.4	41.4	62.2	46.6	69.9
	A490	N		20.7	31.1	25.9	38.8	31.1	46.6	36.3	54.4	41.4	62.2	46.6	69.9
		X		20.7	31.1	25.9	38.8	31.1	46.6	36.3	54.4	41.4	62.2	46.6	69.9
Weld Size				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard Holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included
X = Threads Excluded

$1\frac{1}{8}$-in. diameter bolts

Table 10–9b (continued)
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

Plate F_y = 50 ksi

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				5/16		3/8		7/16		1/2		9/16		5/8	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
12 (*L* = 37)	A325 F1852	N	STD	134	201	161	241	188	282	215	322	241	362	266	399
			SSLT	134	201	161	241	188	282	215	322	241	362	268	402
		X	STD	134	201	161	241	188	282	215	322	241	362	268	402
			SSLT	134	201	161	241	188	282	215	322	241	362	268	402
	A490	N	STD	134	201	161	241	188	282	215	322	241	362	268	402
			SSLT	134	201	161	241	188	282	215	322	241	362	268	402
		X	STD	134	201	161	241	188	282	215	322	241	362	268	402
			SSLT	134	201	161	241	188	282	215	322	241	362	268	402
11 (*L* = 34)	A325 F1852	N	STD	123	185	148	222	173	259	197	296	222	333	247	370
			SSLT	123	185	148	222	173	259	197	296	222	333	247	370
		X	STD	123	185	148	222	173	259	197	296	222	333	247	370
			SSLT	123	185	148	222	173	259	197	296	222	333	247	370
	A490	N	STD	123	185	148	222	173	259	197	296	222	333	247	370
			SSLT	123	185	148	222	173	259	197	296	222	333	247	370
		X	STD	123	185	148	222	173	259	197	296	222	333	247	370
			SSLT	123	185	148	222	173	259	197	296	222	333	247	370
10 (*L* = 31)	A325 F1852	N	STD	113	169	135	203	158	237	180	271	203	304	225	338
			SSLT	113	169	135	203	158	237	180	271	203	304	225	338
		X	STD	113	169	135	203	158	237	180	271	203	304	225	338
			SSLT	113	169	135	203	158	237	180	271	203	304	225	338
	A490	N	STD	113	169	135	203	158	237	180	271	203	304	225	338
			SSLT	113	169	135	203	158	237	180	271	203	304	225	338
		X	STD	113	169	135	203	158	237	180	271	203	304	225	338
			SSLT	113	169	135	203	158	237	180	271	203	304	225	338
9 (*L* = 28)	A325 F1852	N	STD/ SSLT	102	153	122	184	143	214	163	245	184	276	204	306
		X		102	153	122	184	143	214	163	245	184	276	204	306
	A490	N		102	153	122	184	143	214	163	245	184	276	204	306
		X		102	153	122	184	143	214	163	245	184	276	204	306
8 (*L* = 25)	A325 F1852	N	STD/ SSLT	91.4	137	110	165	128	192	146	219	165	247	183	274
		X		91.4	137	110	165	128	192	146	219	165	247	183	274
	A490	N		91.4	137	110	165	128	192	146	219	165	247	183	274
		X		91.4	137	110	165	128	192	146	219	165	247	183	274
Weld Size				1/4		1/4		5/16		5/16		3/8		7/16	

STD = Standard Holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included
X = Threads Excluded

Table 10–9b (continued) Single-Plate Connections

Plate F_y = 50 ksi

Bolt, Weld, and Single-Plate Available Strengths, kips

1 1/8-in. diameter bolts

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				5/16		3/8		7/16		1/2		9/16		5/8	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
7 (L = 22)	A325	N	STD/ SSLT	80.7	121	96.9	145	113	170	129	194	145	218	161	242
	F1852	X		80.7	121	96.9	145	113	170	129	194	145	218	161	242
	A490	N		80.7	121	96.9	145	113	170	129	194	145	218	161	242
		X		80.7	121	96.9	145	113	170	129	194	145	218	161	242
6 (L = 19)	A325	N	STD/ SSLT	70.1	105	84.1	126	98.1	147	112	168	126	189	140	210
	F1852	X		70.1	105	84.1	126	98.1	147	112	168	126	189	140	210
	A490	N		70.1	105	84.1	126	98.1	147	112	168	126	189	140	210
		X		70.1	105	84.1	126	98.1	147	112	168	126	189	140	210
5 (L = 16)	A325	N	STD/ SSLT	59.4	89.1	71.3	107	83.2	125	95.1	143	107	160	119	178
	F1852	X		59.4	89.1	71.3	107	83.2	125	95.1	143	107	160	119	178
	A490	N		59.4	89.1	71.3	107	83.2	125	95.1	143	107	160	119	178
		X		59.4	89.1	71.3	107	83.2	125	95.1	143	107	160	119	178
4 (L = 13)	A325	N	STD/ SSLT	48.8	73.1	58.5	87.8	68.3	102	78.0	117	87.8	132	95.4	143
	F1852	X		48.8	73.1	58.5	87.8	68.3	102	78.0	117	87.8	132	97.5	146
	A490	N		48.8	73.1	58.5	87.8	68.3	102	78.0	117	87.8	132	97.5	146
		X		48.8	73.1	58.5	87.8	68.3	102	78.0	117	87.8	132	97.5	146
3 (L = 10)	A325	N	STD/ SSLT	38.1	57.1	45.7	68.6	53.3	80.0	60.9	91.4	68.6	103	71.6	107
	F1852	X		38.1	57.1	45.7	68.6	53.3	80.0	60.9	91.4	68.6	103	76.2	114
	A490	N		38.1	57.1	45.7	68.6	53.3	80.0	60.9	91.4	68.6	103	76.2	114
		X		38.1	57.1	45.7	68.6	53.3	80.0	60.9	91.4	68.6	103	76.2	114
2 (L = 7)	A325	N	STD/ SSLT	27.4	41.1	32.9	49.4	38.4	57.6	43.9	65.8	47.7	71.6	47.7	71.6
	F1852	X		27.4	41.1	32.9	49.4	38.4	57.6	43.9	65.8	49.4	74.0	54.8	82.3
	A490	N		27.4	41.1	32.9	49.4	38.4	57.6	43.9	65.8	49.4	74.0	54.8	82.3
		X		27.4	41.1	32.9	49.4	38.4	57.6	43.9	65.8	49.4	74.0	54.8	82.3
Weld Size				1/4		1/4		5/16		5/16		3/8		7/16	

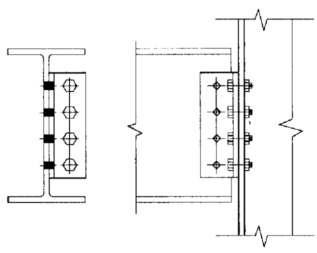
STD = Standard Holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included
X = Threads Excluded

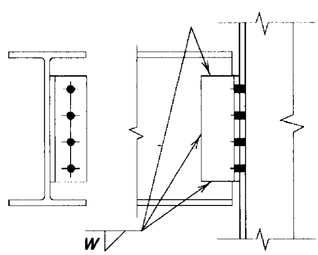
SINGLE-ANGLE CONNECTIONS

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 10–13. This angle is preferably shop-bolted or welded to the supporting member and field-bolted to the supported beam.

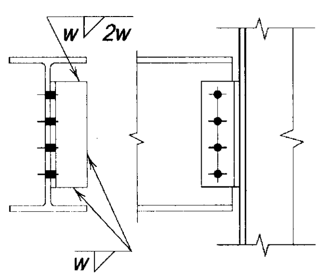
When the angle is welded to the support, adequate flexibility must be provided in the connection. As illustrated in Figure 10–13c, the weld is placed along the toe and across the bottom of the angle with a return at the top per AISC Specification Section J2.2b. Note that welding across the entire top 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 simple shear connections.



(a) All-bolted



(b) Bolted/welded, angle welded to supported beam



Note: weld return on top of angle per Specification Section J2.2b.

(c) Bolted/welded, angle welded to support

Figure 10–13. Single-angle connections.

Design Checks

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

As illustrated in Figure 10–14, 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. (2¾-in. gage plus ¼-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 10–3.

A minimum angle thickness of ⅜-in. for ¾-in. and ⅞-in. diameter bolts, and ½-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.

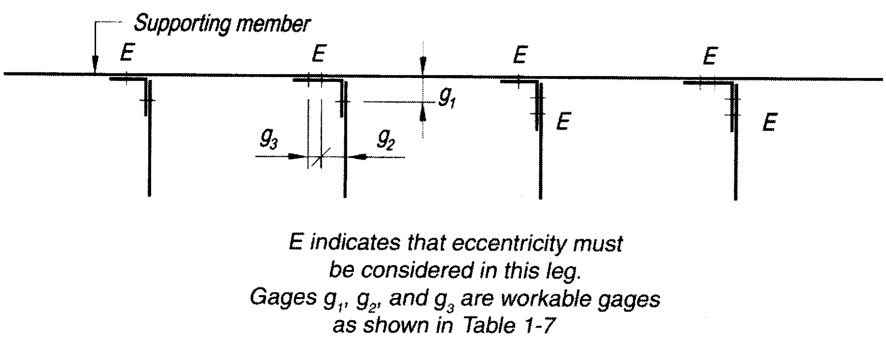


Figure 10–14. Eccentricity in angles.

Table 10–10. All-Bolted Single-Angle Connections

Table 10–10 is a design aid for all-bolted single-angle connections. The tabulated eccentrically loaded bolt group coefficients, C, are useful in determining the available strength, ϕR_n or R_n/Ω, where

$$R_n = C \times r_n$$
$$\phi = 0.75 \qquad \Omega = 2.0$$

In the above equation,

C = coefficient from Table 10–10
r_n = the nominal strength of one bolt in shear or bearing, kips

Table 10–11. Bolted/Welded Single-Angle Connections

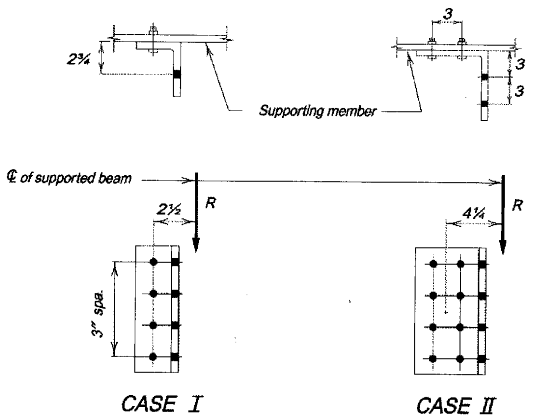
Table 10–11 is a design aid for bolted/welded single-angle connections. Electrode strength is assumed to be 70 ksi. In the rare case where a single-angle connection must be field-welded, erection bolts may be placed in the leg to be field-welded.

Weld available strengths are determined by the instantaneous center of rotation method using Table 8-11 with $\theta = 0°$. The tabulated values assume a half-web thickness of $^1/_4$ in. and may be used conservatively for lesser half-web thicknesses. For half-web thicknesses greater than $^1/_4$ in., the tabulated values should be reduced proportionally to eight percent at a half-web thickness of $^1/_2$ in. The tabulated minimum supporting flange or web thickness is the thickness that matches the strength of the support material to the strength of the weld material. In a manner similar to that illustrated previously for Table 10–2, the minimum material thickness (for one line of weld) may be calculated as:

$$t_{min} = \frac{3.09D}{F_u}$$

where D is the number of sixteenths in the weld size. When welds line up on opposite sides of the support, the minimum thickness is the sum of the thicknesses required for each weld. In either case, when less than the minimum material thickness is present, the tabulated weld available strength should be multiplied by the ratio of the thickness provided to the minimum thickness.

Table 10–10
All-Bolted Single-Angle Connections



Note: standard holes in support leg of angle

Eccentrically Loaded Bolt Group Coefficients, *C*		
Number of Bolts in One Vertical Row, *n*	**Case I**	**Case II**
12	11.4	21.5
11	10.4	19.4
10	9.37	17.3
9	8.34	15.2
8	7.31	13.0
7	6.27	10.9
6	5.22	8.70
5	4.15	6.63
4	3.07	4.70
3	1.99	2.94
2	1.03	1.61
1	—	0.518

$\phi R_n = C \times \phi r_n$ or $R_n/\Omega = C \times r_n/\Omega$

where

C = coefficient from Table above

ϕr_n = design strength of one bolt in shear or bearing, kips/bolt

r_n/Ω = allowable strength of one bolt in shear or bearing, kips/bolt

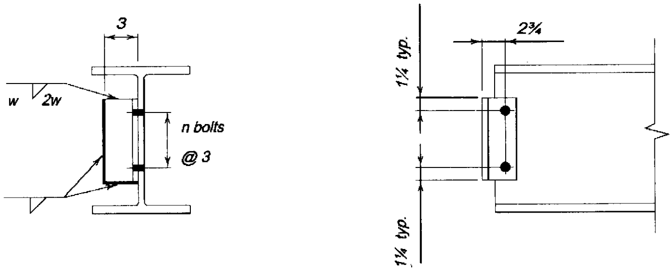
Notes:

For eccentricities less than or equal to those shown above, tabulated values may be used.

For greater eccentricities, coefficient C should be recalculated from Part 7.

Connection may be bearing-type or slip-critical.

Table 10–11
Bolted/Welded Single-Angle Connections



Number of Bolts in One Vertical Row	Bolt and Angle Strength, kips				Angle Size (F_y = 36 ksi)	Angle Length, in.	Weld (70 ksi)			Minimum t_w of Supporting Member with Angles Both Sides of Web, in.
	$^3/_4$ in.		$^7/_8$ in.				Size, w, in.	Available Strength, kips		
	ASD	LRFD	ASD	LRFD				ASD	LRFD	
12	127	191	147	220	L4×3×$^3/_8$	35$^1/_2$	$^5/_{16}$	176	265	0.476
							$^1/_4$	141	212	0.381
							$^3/_{16}$	106	159	0.286
11	117	175	135	202		32$^1/_2$	$^5/_{16}$	164	246	0.476
							$^1/_4$	131	197	0.381
							$^3/_{16}$	98.6	148	0.286
10	106	159	123	184		29$^1/_2$	$^5/_{16}$	151	227	0.476
							$^1/_4$	121	181	0.381
							$^3/_{16}$	90.6	136	0.286
9	95.4	143	110	166		26$^1/_2$	$^5/_{16}$	137	206	0.476
							$^1/_4$	110	165	0.381
							$^3/_{16}$	82.3	123	0.286
8	84.8	127	98.3	147		23$^1/_2$	$^5/_{16}$	123	185	0.476
							$^1/_4$	98.7	148	0.381
							$^3/_{16}$	74	111	0.286
7	74.2	111	86.1	129		20$^1/_2$	$^5/_{16}$	109	164	0.476
							$^1/_4$	87.5	131	0.381
							$^3/_{16}$	65.6	98.4	0.286

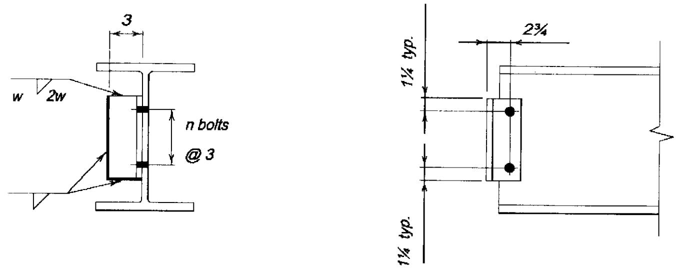
Notes:

Gage in angle leg attached to beam web as well as leg width may be decreased. 3-in. welded leg may not be increased or decreased.

Tabulated weld available strengths are based on a $^1/_4$-in. half web for the supported member. Smaller half webs will result in these values being conservative. For half webs over $^1/_4$ in., weld values must be reduced proportionally to 8% for a $^1/_2$-in. half web or recalculated.

When the beam web thickness of the supporting member is less than the minimum and single-angle connections are back to back, either stagger the angles, or multiply the weld design strength by the ratio of the actual web thickness to the tabulated minimum thickness to determine the reduced weld design strength.

Table 10–11 (continued)
Bolted/Welded Single-Angle Connections



Number of Bolts in One Vertical Row	Bolt and Angle Strength, kips				Angle Size (F_y = 36 ksi)	Angle Length, in.	Weld (70 ksi)			Minimum t_w of Supporting Member with Angles Both Sides of Web, in.
	$^3/_4$ in.		$^7/_8$ in.				Size, w, in.	Available Strength, kips		
	ASD	LRFD	ASD	LRFD				ASD	LRFD	
6	63.6	95.4	74	111	L4×3×$^3/_8$	$17^1/_2$	$^5/_{16}$	94.3	142	0.476
							$^1/_4$	75.5	113	0.381
							$^3/_{16}$	56.6	84.9	0.286
5	53	79.5	61.8	92.7		$14^1/_2$	$^5/_{16}$	79	119	0.476
							$^1/_4$	63.2	94.8	0.381
							$^3/_{16}$	47.4	71.1	0.286
4	42.4	63.6	48.9	73.4		$11^1/_2$	$^5/_{16}$	62.8	94.3	0.476
							$^1/_4$	50.3	75.4	0.381
							$^3/_{16}$	37.7	56.6	0.286
3	31.8	47.7	35.9	53.8		$8^1/_2$	$^5/_{16}$	45.7	68.5	0.476
							$^1/_4$	36.5	54.8	0.381
							$^3/_{16}$	27.4	41.1	0.286
2	21.2	31.8	22.8	34.3		$5^1/_2$	$^5/_{16}$	28.1	42.2	0.476
							$^1/_4$	22.5	33.7	0.381
							$^3/_{16}$	16.9	25.3	0.286

Notes:

Gage in angle leg attached to beam web as well as leg width may be decreased. 3-in. welded leg may not be increased or decreased.

Tabulated weld available strengths are based on a $^1/_4$-in. half web for the supported member. Smaller half webs will result in these values being conservative. For half webs over $^1/_4$ in., weld values must be reduced proportionally to 8% for a $^1/_2$-in. half web or recalculated.

When the beam web thickness of the supporting member is less than the minimum and single-angle connections are back to back, either stagger the angles, or multiply the weld design strength by the ratio of the actual web thickness to the tabulated minimum thickness to determine the reduced weld design strength.

TEE CONNECTIONS

A tee connection is made with a structural tee, as illustrated in Figure 10–15. The tee is preferably shop-bolted or welded to the supporting member and field-bolted to the supported beam.

When the tee is welded to the support, adequate flexibility must be provided in the connection. As illustrated in Figure 10–15b, line welds are placed along the toes of the tee flange with a return at the top per AISC Specification Section J2.2b. Note that welding across the entire top of the tee 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 simple shear connections.

Design Checks

The available strength of a tee connection is determined from the applicable limit-states for bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a.

Eccentricity must be considered when determining the available strength of tee connections. For a flexible support, the bolts or welds attaching the tee flange to the support must be designed for the shear, R_u or R_a. Also, the bolts through the tee stem must be designed for the shear and the eccentric moment, $R_u a$ or $R_a a$, where a is the distance from the face of the support to the centroid of the bolt group through the tee stem.

For a rigid support, the bolts or welds attaching the tee flange to the support must be designed for the shear and the eccentric moment; the bolts through the tee stem must be designed for the shear.

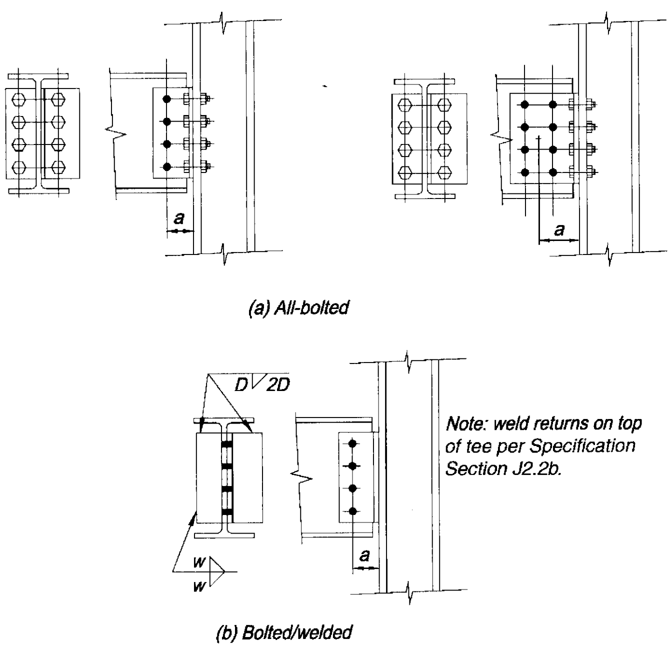


Figure 10–15. Tee connections.

Recommended Tee Length and Flange and Web Thicknesses

To provide for stability during erection, it is recommended that the mimimum tee length be one-half the T-dimension of the beam to be supported. The maximum length of the tee 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 tee may encroach upon the fillet(s) as given in Figure 10–3.

To provide for flexibility, the tee selected should meet the ductility checks illustrated in Part 9. The flange thickness of tees used in simple shear connections should be held to a minimum to permit the flexure necessary to accommodate the end rotation of the beam, unless the tee connection is proportioned to meet the geometric requirements for single-plate connections.

Shop and Field Practices

When framing to a column flange, provision must be made for possible mill variation in the depth of the columns. If the tee is shop-attached to the column flange, play in the open holes usually furnishes the necessary adjustment to compensate for the mill variation. This approach offers the advantage of side erection of the beam. Alternatively, if the tee is shop-attached to the supported beam web, the beam length could be shortened to provide for mill overrun and shims could be furnished at the appropriate intervals to fill the resulting gaps or to provide for mill underrun.

When a single vertical row of bolts is used in a tee stem, a 4-in. or 5-in. stem is required to accommodate the end distance of the supported beam and possible overrun/underrun in beam length. A double vertical row of bolts will require a 7-in. or 8-in. tee stem. There is no maximum limit on L_{eh} for the tee stem.

SHEAR SPLICES

Shear splices are usually made with a single plate, as shown in Figure 10–16a, or two plates, as shown in Figures 10–16b and 10–16c. Although the rotational flexibility required at a shear splice is usually much less than that required at the end of a simple-span beam, when a highly flexible splice is desired, the splice utilizing four framing angles, shown in Figure 10–17, is especially useful. These shear splices may be bolted and/or welded.

The available strength of a shear splice is determined from the applicable limit-states for the bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a.

Eccentricity must be considered in the design of shear splices, with the exception of all-bolted shear splices utilizing four framing angles, as illustrated in Figure 10–17. When the splice is symmetrical, as shown for the bolted splice in Figure 10–16a, each side of the splice is equally restrained regardless of the relative flexibility of the spliced members. Accordingly, as illustrated in Figure 10–18, the eccentricity of the shear to the center of gravity of either bolt group is equal to half the distance between the centroids of the bolt groups. Therefore, each bolt group can be designed for the shear, R_u or R_a, and one-half the eccentric moment, R_ue or R_ae (Kulak and Green, 1990). This approach is also applicable to symmetrical welded splices.

When the splice is not symmetrical, as shown in Figures 10–16b and 10–16c, one side of the splice will possess a higher degree of rigidity. For the splice shown in Figure 10–16b, the right side is more rigid because the stiffness of the weld group exceeds the stiffness of the bolt group, even if the bolts are pretensioned or slip-critical. Also, for the splice shown in Figure 10–16c, the right side is more rigid since there are two vertical rows of bolts while the left side has only one. In these cases, it is conservative to design the side with the higher rigidity for the shear, R_u or R_a, and the full eccentric moment R_ue or R_ae. The side with the lower rigidity can then be designed for the shear only. This approach is applicable regardless of the relative flexibility of the spliced members.

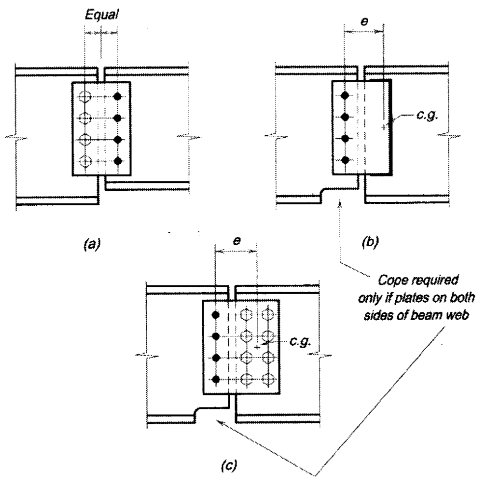


Figure 10–16. Plate-type shear splices.

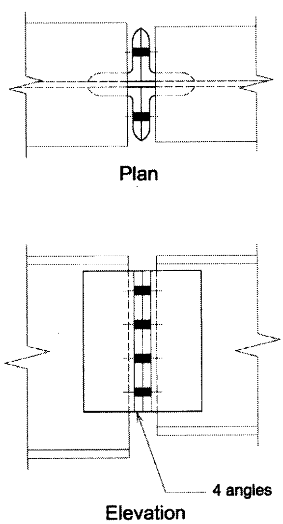


Figure 10–17. Angle-type shear splice.

Some splices, such as those that occur at expansion joints, require special attention and are beyond the scope of this Manual.

SPECIAL CONSIDERATIONS FOR SIMPLE SHEAR CONNECTIONS

Simple Shear Connections Subject to Axial Forces

When simple shear connections are subjected to axial load in addition to the shear, the important limit-states are angle leg bending and prying action. These tend to require that the angle, plate, or flange thickness increase or the gage decrease, or both, and these requirements may compromise the connection's ability to remain flexible enough to accommodate the simple beam end rotation. The shear connection ductility checks derived in Part 9 can be used to ensure that adequate ductility exists.

Simple Shear Connections at Stiffened Column-Web Locations

Stiffeners are obstacles to direct connections to the column web. Figure 10–19a illustrates a seat angle-welded to the toes of the column flanges; Figure 10–19d shows a vertical plate extended beyond the column flanges. Figures 10–19b and 10–19c offer two additional options for framing at locations of diagonal stiffeners; these should be examined carefully as they may create erection problems. Additionally, the deep cope of Figure 10–19c may significantly reduce the available strength of the beam at the end connection. Alternatively, the bottom transverse stiffener could be extended to serve as a seat plate with a bearing stiffener provided to distribute the beam reaction.

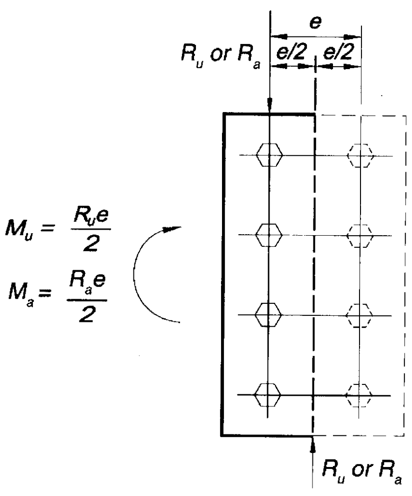


Figure 10–18. Eccentricity in a symmetrical shear splice.

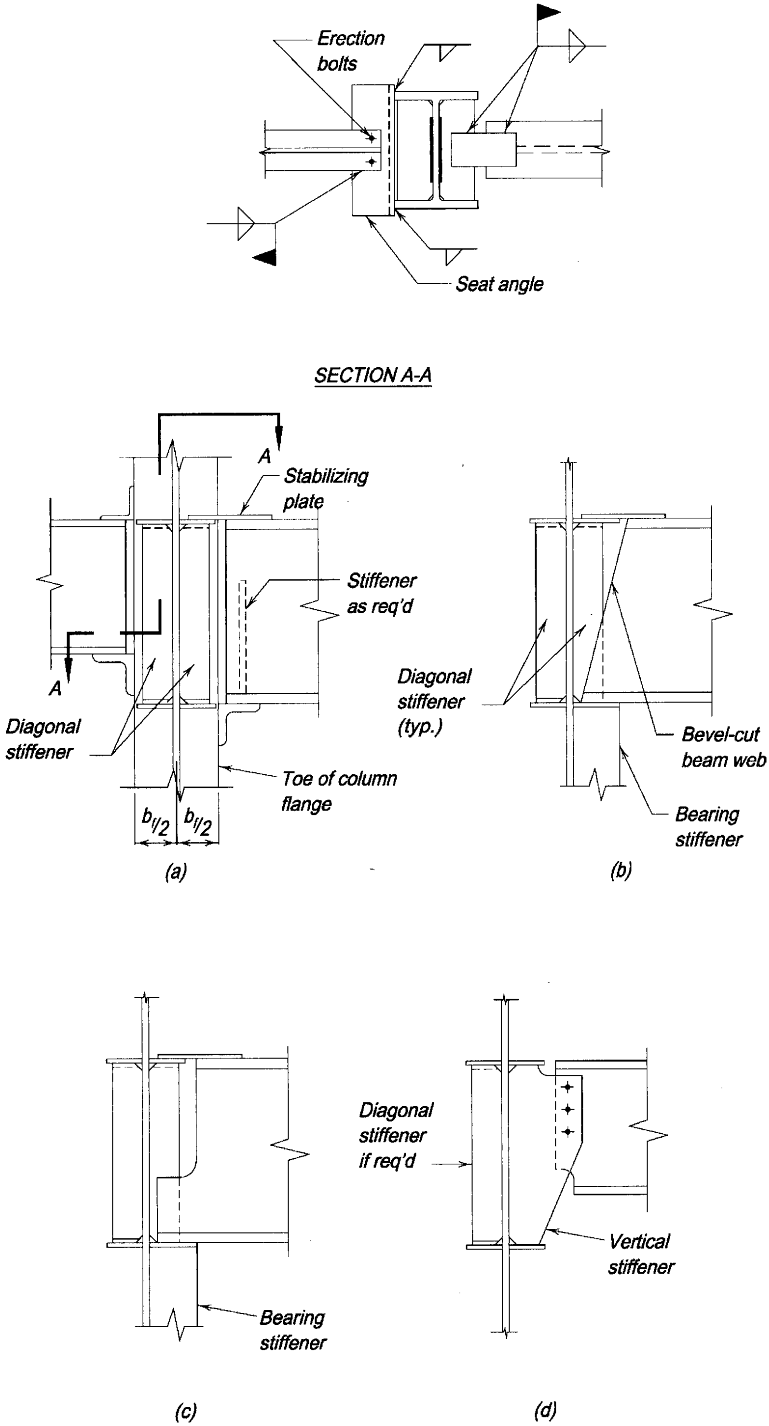


Figure 10–19. Simple shear connections at stiffened column-web locations.

Eccentric Effect of Extended Gages

Consider a simple shear connection to the web of a column that requires transverse stiffeners for two concurrent beam-to-column-flange moment connections. If it were not possible to eliminate the stiffeners by selection of a heavier column section, the field connection would have to be located clear of the column flanges, as shown in Figure 10–20, to provide for access and erectability.

The extension of the connection beyond normal gage lines results in an eccentric moment. While this eccentric moment is usually neglected in a connection framing to a column flange, the resistance of the column to weak-axis bending is typically only 20 to 50 percent of that in the strong axis. Thus the eccentric moment should be considered in this column-web connection, especially if the eccentricity, e, is large. Similarly, eccentricities larger than normal gages may also be a concern in connections to girder webs.

Column-Web Supports

There are two components contributing to the total eccentric moment: (1) the eccentricity of the beam end reaction, Re; and (2) M_{pr}, the partial restraint of the connection. To determine what eccentric moment must be considered in the design, first assume that the column is part of a braced frame for weak-axis bending, is pinned-ended with $K = 1$, and will be concentrically loaded, as illustrated in Figure 10–21. The beam is loaded before the column and will deflect under load as shown in Figure 10–22. Because of the partial restraint of the connection, a couple, M_{pr}, develops between the beam and column and adds to the eccentric couple, Re. Thus, $M_{con} = Re + M_{pr}$.

As the loading of the column begins, the assembly will deflect further in the same direction under load, as indicated in Figure 10–23, until the column load reaches some magnitude P_{sbr} when the rotation of the column will equal the simply supported beam end rotation. At this load, the rotation of the column negates M_{pr} since it also relieves the partial restraint effect of the connection, and $M_{con} = Re$. As the column load is increased above P_{sbr}, the column rotation exceeds the simply supported beam end rotation and a moment M'_{pr} results such that $M_{con} = Re - M'_{pr}$.

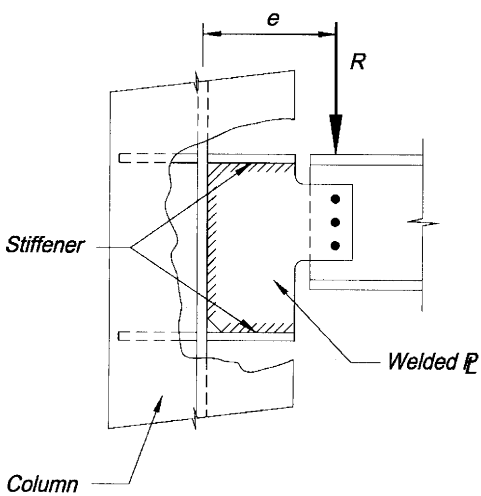


Figure 10–20. Eccentric effect of extended gages.

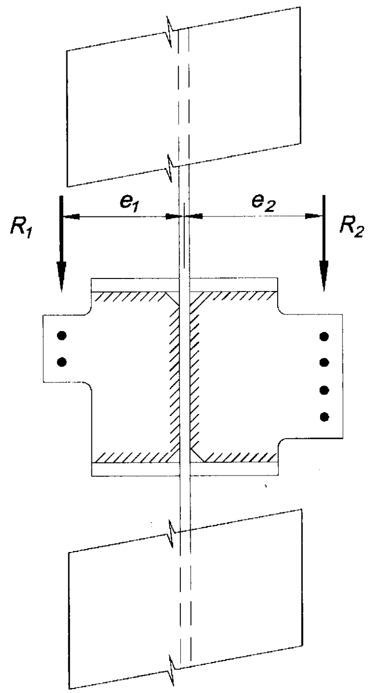


Figure 10–21. Column subject to dual eccentric moments.

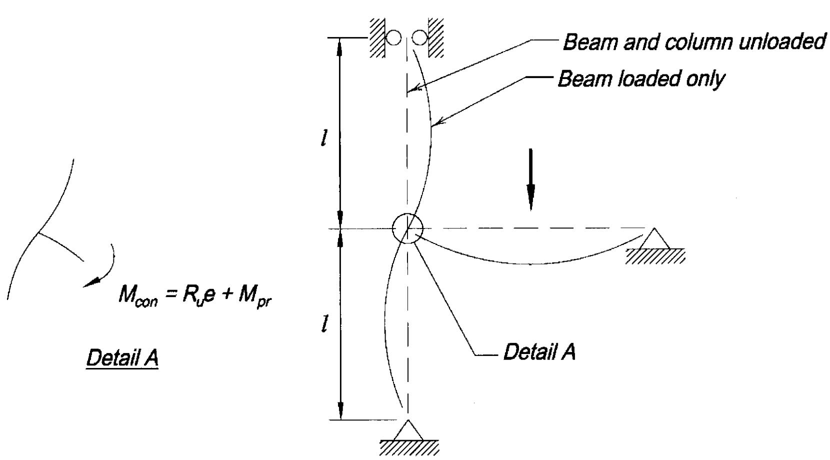


Figure 10–22. Illustration of beam, column, and connection behavior under loading of beam only.

Note that the partial restraint of the connection now actually stabilizes the column and reduces its effective length factor K below the originally assumed value of 1. Thus, since M'_{pr} must be greater than zero, it must also be true that $Re > M_{con}$. It is therefore conservative to design the connection for the shear, R and the eccentric moment, Re.

The welds connecting the plate to the supporting column web should be designed to resist the full shear, R only; the top and bottom plate-to-stiffener welds have minimal strength normal to their length, are not assumed to carry any calculated force, and may be of minimum size in accordance with AISC Specification Section J2.

If simple shear connections frame to both sides of the column web, as illustrated in Figure 10–21, each connection should be designed for its respective shear, R_1, and R_2 and the eccentric moment $|R_2e_2 - R_1e_1|$ may be apportioned between the two simple shear connections as the designer sees fit; the total eccentric moment may be assumed to act on the larger connection, the moment may be divided proportionally among the connections according to the polar moments of inertia of the bolt groups (relative stiffness), or the moment may be divided proportionally between the connections according to the section moduli of the bolt groups (relative moment strength). If provision is made for ductility and stability, it follows from the lower bound theorem of limit states analysis that the distribution which yields the greatest strength is closest to the true strength. Note that the possibility exists that one of the beams may be devoid of live load at the same time that the opposite beam is fully loaded. This condition must be considered by the designer when apportioning the moment.

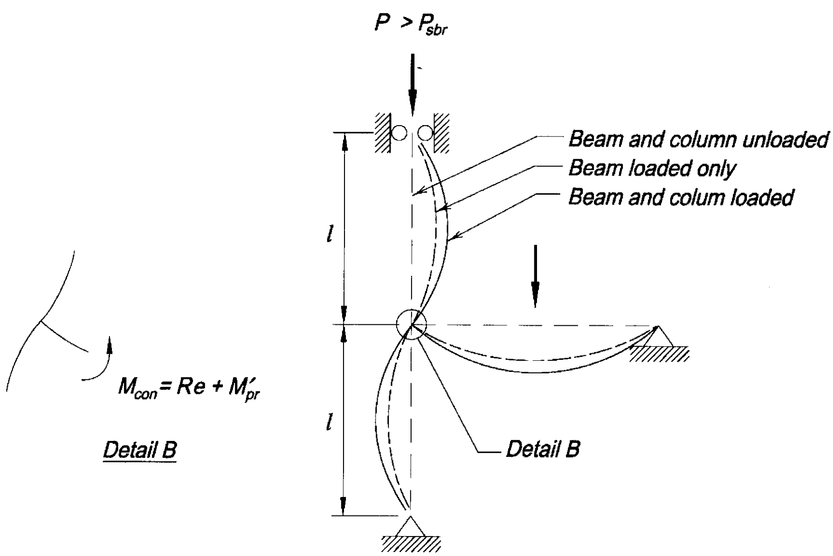


Figure 10–23. Illustration of beam, column, and connection behavior under loading of beam and column.

Girder-Web Supports

The girder-web support of Figure 10–24 usually provides only minimal torsional stiffness or strength. When larger-than-normal gages are used, the end rotation of the supported beam will usually be accommodated through rotation of the girder support. It follows that the bolt group should be designed to resist both the shear, R, and the eccentric moment, Re. The beam end reaction will then be carried through to the center of the supporting girder web.

The welds connecting the plate to the supporting girder web should be designed to resist the shear, R, only; the top and bottom plate-to-girder-flange welds have minimal strength normal to their length, are not assumed to carry any calculated force, and may be of minimum size in accordance with AISC Specification Section J2.

Similarly, for the girder illustrated in Figure 10–25 supporting two eccentric reactions, each connection should be designed for its respective shear R_1 and R_2, and the eccentric moment, $|R_2e_2 - R_1e_1|$, may be apportioned between the two simple shear connections as the designer sees fit.

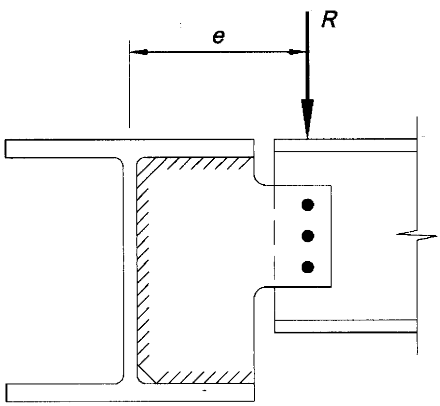


Figure 10–24. Eccentric moment on girder-web support.

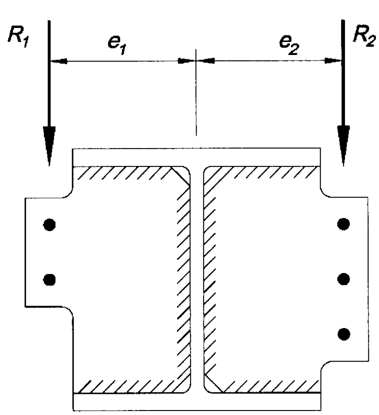


Figure 10–25. Girder-web support subject to dual eccentric moments.

Alternative Treatment of Eccentric Moment

In the foregoing treatment of eccentric moments with column- and girder-web supports, it is possible to design the support (instead of the connection) for the eccentric moment Re. Additionally, when metal deck is used with puddle welds or self-tapping screws, the metal deck tends to reduce relative movement between the two members and thus will tend to carry all or some of the eccentric moment. In these cases, the connection may be designed for the shear, R, only or the shear and a reduced eccentric moment.

Double Connections

When beams frame opposite each other and are welded to the web of the supporting girder or column, there are usually no dimensional constraints imposed on one connection by the presence of the other connection unless erection bolts are common to each connection. When the connections are bolted to the web of the supporting column or girder, however, the close proximity of the connections requires that some or all fasteners be common to both connections. This is known as a double connection. See also the discussion under Erectability Considerations.

Supported Beams of Different Nominal Depths

When beams of different nominal depths frame into a double connection, care must be taken to avoid interference from the bottom flange of the shallower beam with the entering and tightening clearances for the bolts of the connection for the deeper beam. Access to the bolts that will support the deeper beam may be provided by coping or blocking the bottom flange of the shallower beam. Alternatively, stagger may be used to favorably position the bolts around the bottom flange of the shallower beam.

Supported Beams Offset Laterally

Frequently, beams do not frame exactly opposite each other, but are offset slightly, as illustrated in Figure 10–26. Several connection configurations are possible, depending on the offset dimension.

If the offset were equal to the gage on the support, the connection could be designed with all bolts on the same gage lines, as shown in Figure 10–26b, and the angles arranged, as shown in Figure 10–26d. If the offset were less than the gage on the support, staggering the bolts, as shown in Figure 10–26c, would reduce the required gage and the angles could be arranged, as shown in Figure 10–26c. In any case, each bolt transmits an equal share of its beam reaction(s) to the supporting member, with the bolts that are loaded in double shear ultimately carrying twice as much force as those loaded in single shear. Once the geometry of the connection has been determined, the distribution of the forces is patterned after that in the design of a typical connection. For normal gages, eccentricity may be ignored in this type of connection.

Beams Offset From Column Centerline

Framing to the Column Flange from the Strong Axis

As illustrated in Figure 10–27, beam-to-column-flange connections offset from the column centerline may be supported on a typical welded seat, stiffened or unstiffened, provided the welds for the seat can be spaced approximately equally on either side of the beam centerline.

Two such seats offset from the W12×65 column centerline by $2^1/4$ in. and $3^1/2$ in. are shown in Figures 10–27a and 10–27b, respectively. While not shown, top angles should be used with this connection.

Since the entire seat fits within the flange width of the column, the connection of Figure 10–27a is readily selected from the design aids presented previously. However, the larger

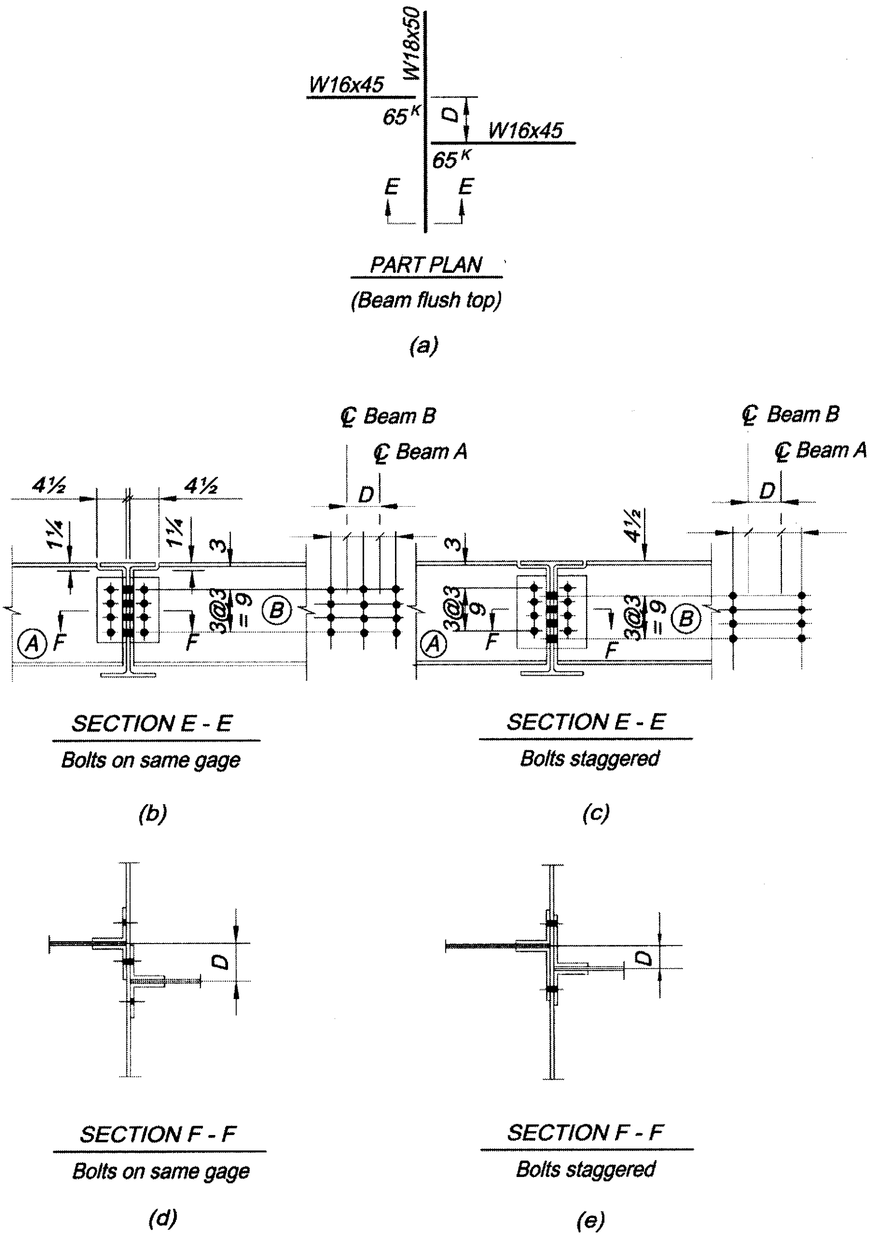
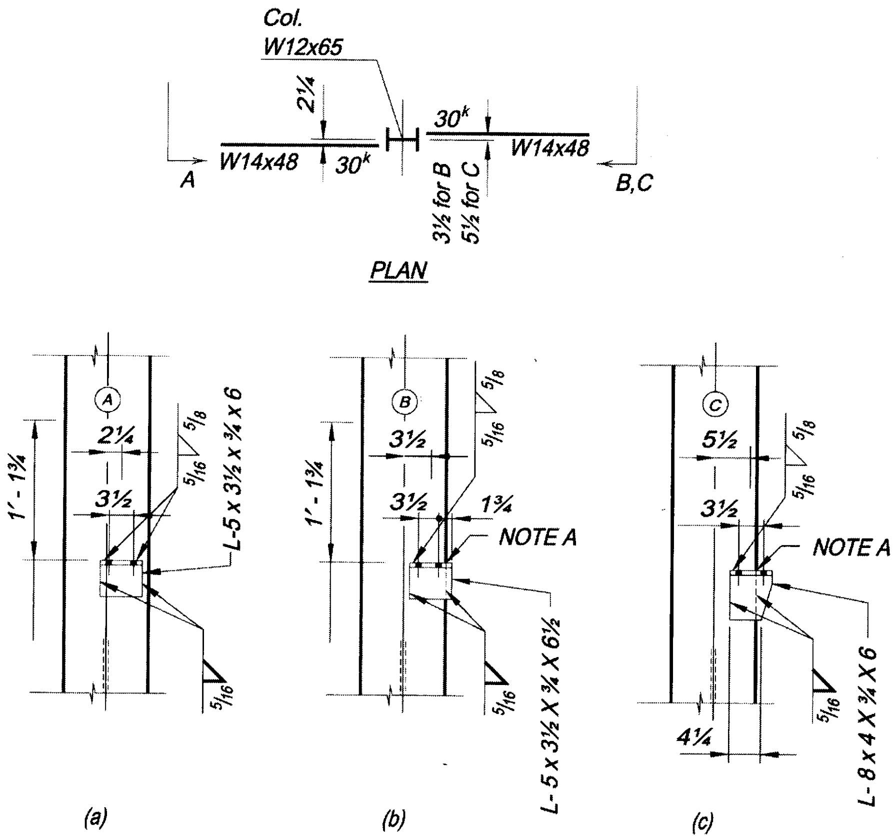


Figure 10–26. Offset beams connected to girder.



NOTE A

End return is omitted because the AWS Code does not permit weld returns to be carried around the corner formed by the column flange toe and seat angle heel.

NOTE B

Beam and top angle not shown for clarity.

Figure 10–27. Offset beams connected column flanges.

beam offsets in Figures 10–27b and 10–27c require that one of the welds be made along the edge of the column flange against the back side of the seat angle. Note that the end return is omitted because weld returns should not be carried around such a corner.

For the beam offset of $5^1/2$ in. shown in Figure 10–27c, the seat angle overhangs the edge of the beam and the horizontal distance between the vertical welds is reduced to $3^1/2$ in.; the center of gravity of the weld group is located $1^1/4$ in. to the left of the beam centerline. The force on each weld may be determined by statics. In this case, the larger force is in the righthand weld and may be determined by summing moments about the lefthand weld. Once the larger force has been determined, the seat should conservatively be designed to carry twice the force in the more highly loaded weld.

Framing to the Column Flange from the Weak Axis

Spandrel beams X and Y in the partial plan shown in Figure 10–28 are offset $4^1/8$ in. from the centerline of column C1, permitting the beam web to be connected directly to the column flange. At column B2, spandrel beam X is offset five inches and requires a $^7/8$-in. filler between the beam web and the column flange. Beams X and Y are both plain-punched beams, with flange cuts on one side, as noted in Figure 10–28a, Section F-F.

In establishing gages, the requirements of other connections to the column at adjacent locations must be considered. While the workable flange gage is $3^1/2$ in. for the W8×28 columns supporting the spandrel beams, for beams Z, the combination of a 4-in. column gage and $1^1/2$-in. stagger of fasteners is used to provide entering and tightening clearance for the field bolts and sufficient edge distance on the column flange, as illustrated in Figure 10–28b. The 4-in. column gage also permits a $1^1/2$-in. edge distance at the ends of the spandrel beams, which will accommodate the normal length tolerance of $\pm^1/4$ in. as specified in "Standard Mill Practice" in Part 1.

The spandrel beams are shown with the notation "Cut and Grind Flush FS" in Sections E-E and F-F. This cut permits the beam web to lie flush against the column flange. The uncut flange on the near side of the spandrel beam contributes to the stiffness of the connection. The $2^1/2$×$^7/8$-in. filler is required between the spandrel beam web and the flange of the column B2 because of the $^7/8$-in. offset. Since the filler in Section E-E, Figure 10–28a, is thicker than $^3/4$ in., it must be fully developed.

In the part plan in Figure 10–29a, the W16×40 beam is offset $6^1/4$ in. from the centerline of column D1. This prevents the web of the W16×40 from being placed flush against the side of the column flange. A plate and filler are used to connect the beam to the column flange, as shown in Figure 10–29b. Such a connection is eccentric and one group of fasteners must be designed for the eccentricity. Lack of space on the inner flange face of the column requires development of the moment induced by the eccentricity in the beam web fasteners.

To minimize the number of field fasteners, the plate in this case is shop-bolted to the beam and field-bolted to the column. A careful check must be made to ensure that the beam can be erected without interference from fittings on the column web. Some fabricators would elect to shop-attach the plate to the column to eliminate possible interference and permit use of plain-punched beams. Additionally, if the column were a heavy section, the fabricator may elect to shop-weld the plate to the column to avoid drilling the thick flanges. The welding of this plate to the column creates a much stiffer connection and the design should be modified to recognize the increased rigidity.

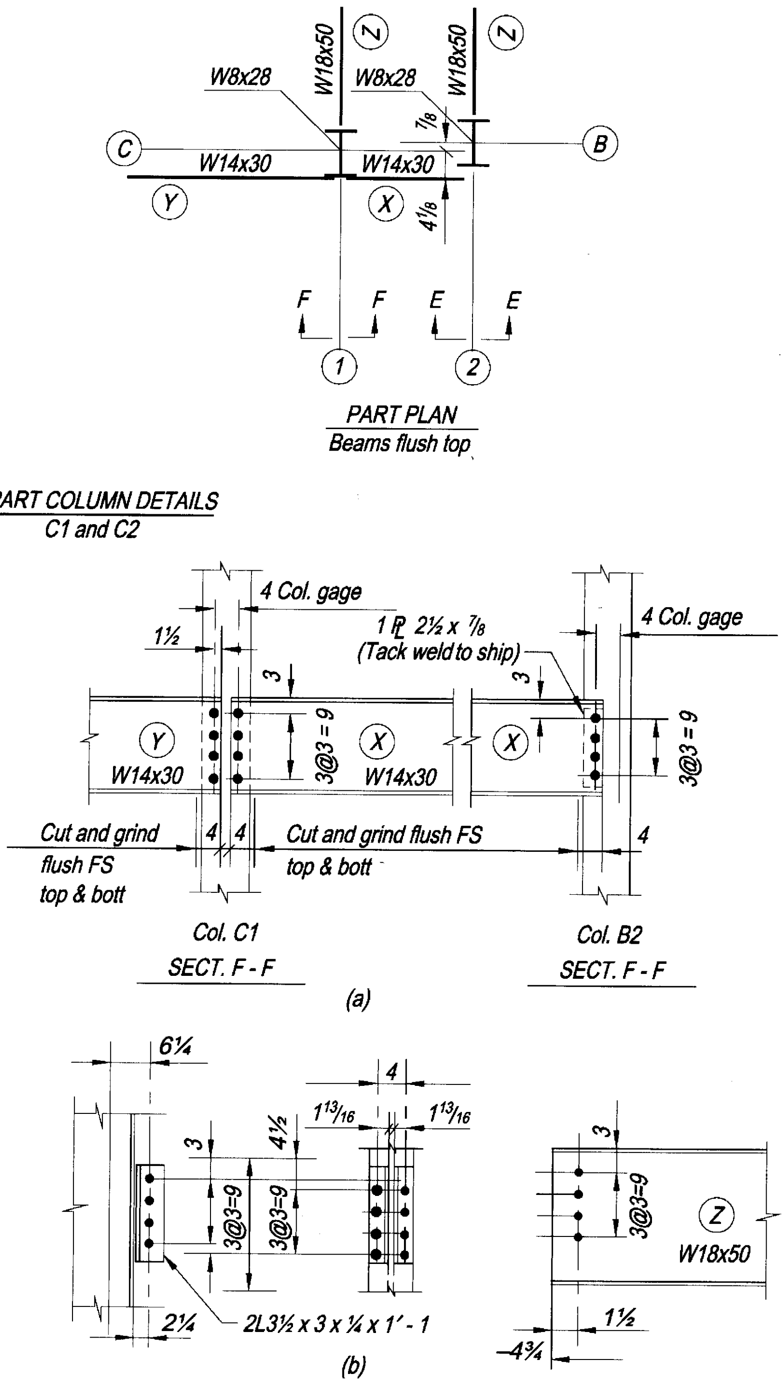


Figure 10–28. Offset beams connected to column.

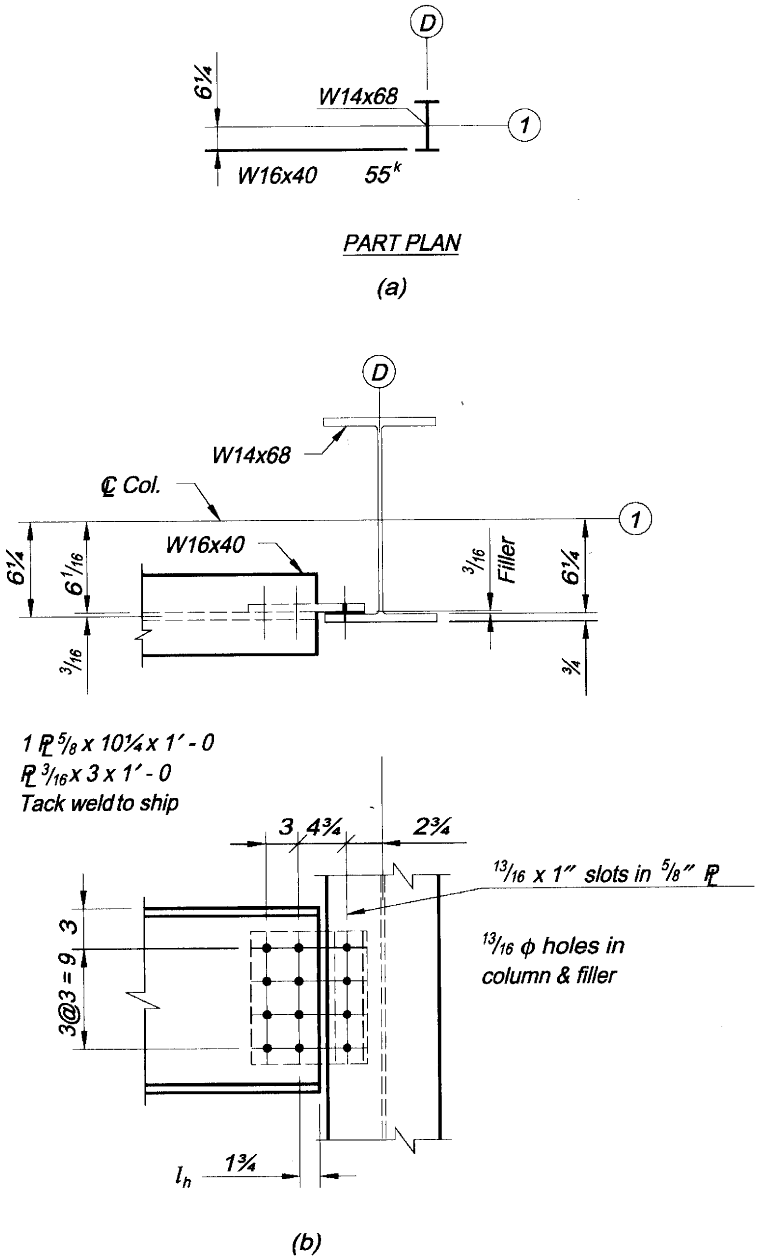


Figure 10–29. Offset beam connected to column.

If the centerline of the W16 were offset $6^1/_{16}$ in. from line 1, it would be possible to cope or cut the flanges flush top and bottom and frame the web directly to the column flange with details similar to those shown in Figure 10–29. This type of framing also provides a connection with more rigidity than normally contemplated in simple construction. A coped connection of this type would create a bending moment at the root of the cope that might require reinforcement of the beam web.

One method frequently adopted to avoid moment transfer to the column because of beam connection rigidity is to use slotted holes and a bearing connection to provide some flexibility. The slotted holes would be provided in the connection plate only and would be in the field connection only. These slotted connections also would accommodate fabrication and erection tolerances.

The type of connection detailed in Figure 10–29 is similar to a coped beam and should be checked for buckling, as illustrated in Part 9. The following differences are apparent and should be recognized in the analysis:

1. The effective length of equivalent "cope" is longer by the amount of end distance to the first bolt gage line.
2. There is an inherent eccentricity due to the beam web and plate thickness. The ordinary web and plate thicknesses normally will not require an analysis for this condition, since the inelastic rotation allowed by the AISC Specification will relieve this secondary moment effect. Two plates may sometimes be required to counter this eccentricity when dimensions are significant.
3. The connection plate can be made of sufficient thickness as required for bending or buckling stresses with a minimum thickness of $^3/_8$ in.

Framing to the Column Web

If the offset of the beam from the centerline of the column web is small enough that the connection may still be centered on or under the supported beam, no special considerations need be made. However, when the offset of the beam is too large to permit the centering of the connection under the beam, as in Figure 10–30, it may be necessary to consider the effect of eccentricity in the fastener group.

The offset of the beam in Figure 10–30 requires that the top and bottom flanges be blocked to provide erection clearance at the column flange. Since only half of each flange, then, remains in which to punch holes, a 6-in. outstanding leg is used for both the seat and top angles of these connections; this permits the use of two field bolts to each of the seat and top angles, which are required by OSHA.

Connections for Raised Beams

When raised beams are connected to column flanges or webs, there is usually no special consideration required. However, when the support is a girder, the differing tops of steel may preclude the use of typical connections. Figure 10–31 shows several typical details commonly used for such cases in bolted construction. Figure 10–32 shows several typical details commonly used in welded construction.

In Figure 10–31a, since the top of the W12×35 is located somewhat less than 12 in. above the top of the W18 supporting beam, a double-angle connection is used. This connection would be designed for the beam reaction and the shop bolts would be governed by double shear or bearing, just as if they were located in a vertical position. However, the field bolts are not required to carry any calculated force under gravity loading.

The maximum permissible distance m depends on the beam reaction, since the web remaining after the bottom cope must provide sufficient area to resist the vertical shear as well as the bending moment which would be critical at the end of the cope. The beam can be reinforced by extending the angles beyond the cope and adding additional shop bolts for development. The angle size and/or thickness can be increased to gain shear area or section modulus, if required. The effect of any eccentricity would be a matter of judgment, but could be neglected for small dimensions.

When this connection is used for flexure or for dynamic or cyclical loading, the web is subjected to high stress concentrations at the end of the cope, and it is good practice to extend the angles, as shown in Figure 10–31a by the dashed lines, to add at least two additional web fasteners.

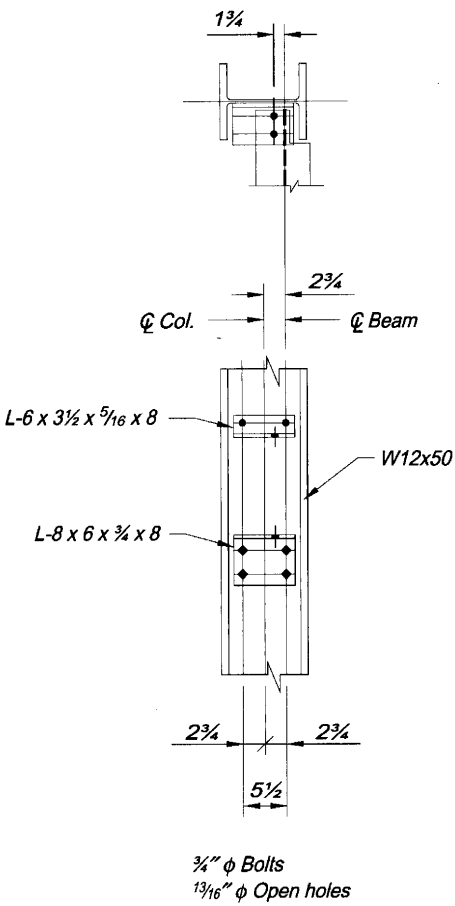


Figure 10–30. Offset beam connected to column web.

Figure 10–31b covers the case where the bottom flange of the W12×35 is located a few inches above the top of the W18. The beam bears directly upon fillers and is connected to the W18 by four field bolts which are not required to transmit a calculated gravity load. If the distance *m* exceeds the thickest plate which can be punched, two or more plates may be used. Even though the fillers in this case need only be $6^1/2$-in. square, the amount of material required increases rapidly as *m* increases. If *m* exceeds 2 or 3 in., another type of detail may be more economical.

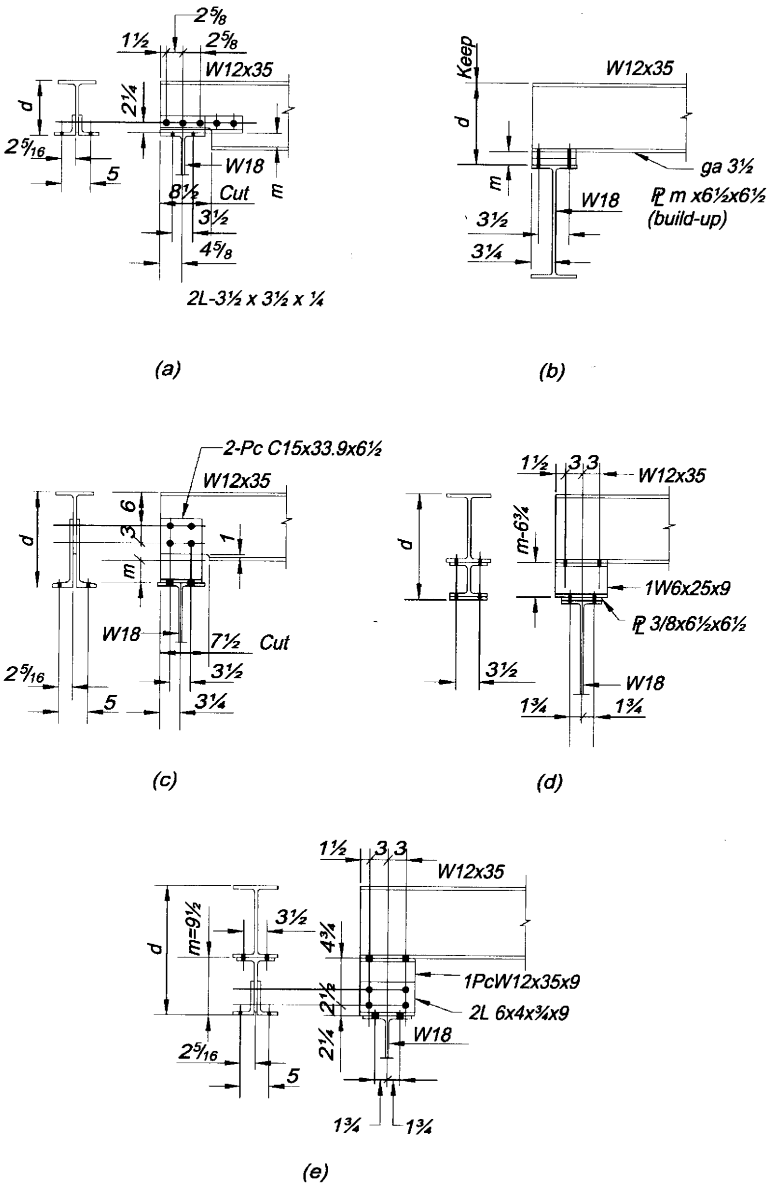


Figure 10–31. Bolted raised-beam connections.

The detail shown in Figure 10–31c is used frequently when m is up to 6 or 7 in. The load on the shop bolts in this case is no greater than that in Figure 10–31a. However, to provide more lateral stiffness, the fittings are cut from a 15-in. channel and are detailed to overlap the beam web sufficiently to permit four shop bolts on two gage lines.

A stool or pedestal, cut from a rolled shape, can be used with or without fillers to provide for the necessary m distance, as in Figure 10–31d. A pair of connection angles and a tee will also serve a similar purpose, as shown in Figure 10–31e. To provide adequate strength to

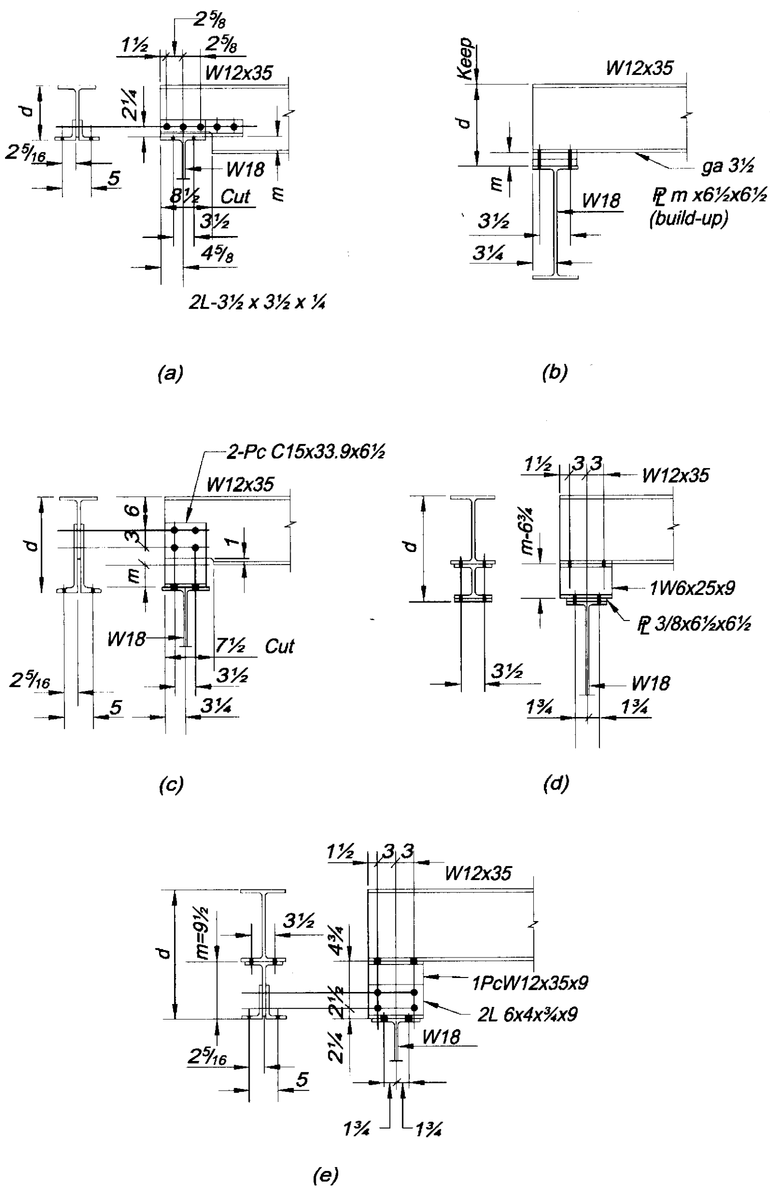


Figure 10–32. Welded raised-beam connections.

carry the beam end reaction and to provide lateral stiffness, the web thickness of the pedestal in each of these cases should be at least as thick as the member being supported.

In Figure 10–32a, welded framing angles are substituted for the bolted angles of Figure 10–32a. In Figure 10–32b, a single horizontal plate is shown replacing the pair of framing angles; this results in a savings in material and the amount of shop-welding. In this case, particular care must be taken in cutting the beam web and positioning the plate at right angles to the beam web. For this reason, if only a few connections of this type are to be made, some fabricators prefer to use the angles, as in Figure 10–32a. If sufficient duplication were available to warrant making a simple jig to position the plate during welding, the solution of Figure 10–32b may be economical.

Figure 10–32c shows a tee centered on the beam web and welded to the bottom flange of the beam. The tee stem thickness should not be less than the beam web thickness. The welded solutions shown in Figures 10–32d and 10–32e are capable of providing good lateral stiffness. The latter two types also permit end rotation as the beam deflects under load. However, if the *m* distance exceeds 3 or 4 in., it is advisable to shop-weld a triangular bracket plate at one end of the beam, as indicated by the dashed lines, to prevent the beam from deflecting along its longitudinal axis.

Other equally satisfactory details may be devised to meet the needs of connections for raised beams. They will vary depending on the size of the supported beam and the distance *m*. When using this type of connection where the load is transmitted through bearing, the provisions of AISC Specification Sections J10.2 and J10.3 must be satisfied for both the supported and supporting members. For the detail of Figure 10–32b, since the rolled fillet has been removed by the cut, the value of *k* would be taken as the thickness of the plate plus the fillet weld size.

AISC Specification Appendix 6 requires stability and restraint against rotation about the beam's longitudinal axis. This provision is most easily accomplished with a floor on top of the supported beam. In the absence of a floor, the top flange may be supported by a strut or bracket attached to the supporting member. When the beam is encased in a wall, this stability may also be provided with wall anchors; refer to "Wall Anchors" in Part 15.

This discussion has considered that the field bolts which attach the beam to the pedestal or support beam are subject to no calculated load. It is important, however, to recognize that when the beam deflects about its neutral axis, a tensile force can be exerted on the outside bolts. The intensity of this tensile force is a function of the dimension *d,* indicated in Figure 10–31, the span length of the supported member, and the beam stiffness. If these forces are large, high-strength bolts should be used and the connection analyzed for the effects of prying action.

Raised-beam connections such as these are used frequently as equipment or machinery supports where it is important to maintain a true and level surface or elevation. When this tolerance becomes important, the dimension *d* should be noted "keep" to advise the fabricator of this importance, as shown in Figure 10–31b. Since the supporting beam is subject to certain camber/deflection tolerances, it also may be appropriate to furnish shim packs between the connection and the supporting member.

Non-Rectangular Simple Shear Connections

It is often necessary to design connections for beams that do not frame into a support orthogonally. Such a beam may be inclined with respect to the supporting member in various directions. Depending upon the relative angular position which a beam assumes, the connection may be classified among three categories: skewed, sloped, or canted. These conditions are illustrated in Figure 10–33 for beam-to-girder web connections; the same descriptions apply to beam-to-column-flange and web connections. Additionally, beams may be oriented in a combination of any or all of these conditions. For any condition of skewed, sloped, or canted framing, the single-plate connection is generally the simplest and most economical of those illustrated in this text.

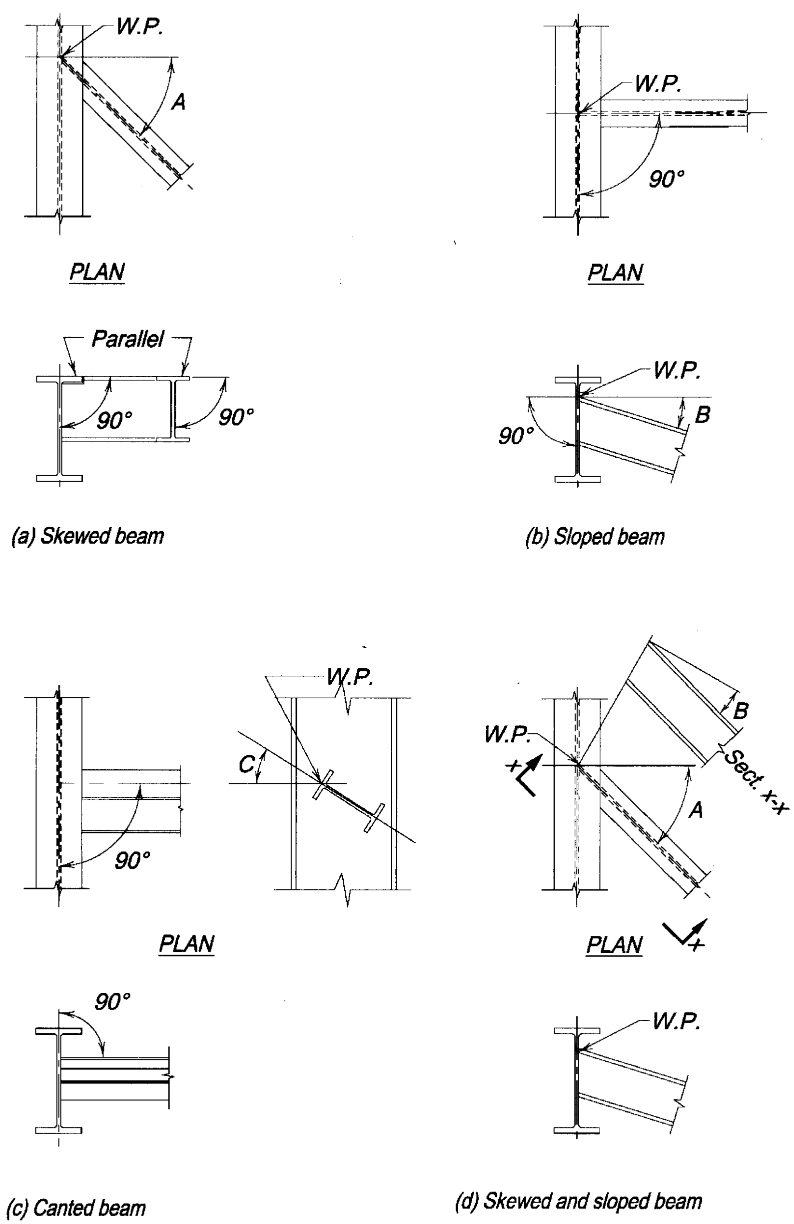


Figure 10–33. Non-rectangular connections.

Skewed Connections

A beam is said to be skewed when its flanges lie in a plane perpendicular to the plane of the face of the supporting member, but its web inclined to the face of the supporting member. The angle of skew A appears in Figure 10–33a and represents the horizontal bevel to which the fittings must be bent or set, or the direction of gage lines on a seated connection.

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 10–34. 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 10–35, 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

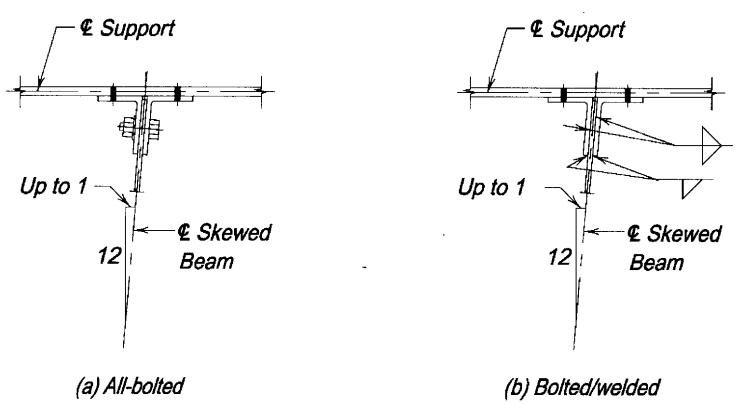


Figure 10–34. Skewed beam connections with bent double angles.

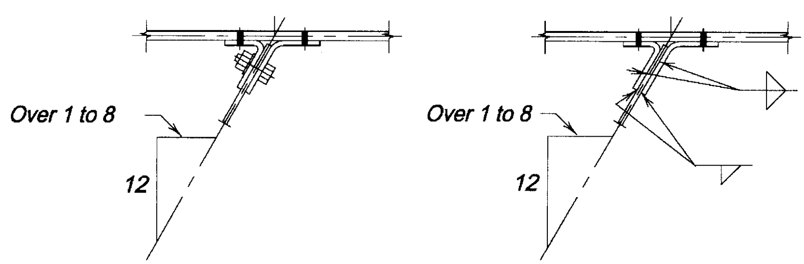


Figure 10–35. Skewed beam connections with double bent plates.

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.

Table 10–12 gives the generally accepted minimum inside-bending radius for plate thickness, *t*, for various grades of steel. Values are for bend lines transverse to the direction of final rolling (Brockenbrough, 1998). When bend lines are parallel to the direction of final rolling, the tabular values may have to be approximately doubled. When bend lines are longer than 36 inches, all radii may have to be increased if problems in bending are encountered.

Before bending, special attention should be given to the condition of plate edges transverse to the bend lines. Flame-cut edges of hardenable steels should be machined or softened by heat treatment. Nicks should be ground out and sharp corners should be rounded.

The strength of bent angles and bent plate connections may be calculated in the same manner as for square framed beams, making due allowances for eccentricity. The load is assumed to be applied at the point where the skewed beam center line intersects the face of the supporting member.

As the angle of skew increases, entering and tightening clearances on the acutely angled side of the connection will require a larger gage on the support. If the gage were to become objectionable, a single bent plate, illustrated in Figure 10–36, may provide a better solution. Note that the single-bent plate may be of the conventional type, or a more compact connection may be developed by "wrapping" the single bent plate, as illustrated in Figure 10–36c.

In all-bolted construction, both the shop and field bolts should be designed for shear and the eccentric moment. A C-shaped weld is preferable to avoid turning the beam during shop fabrication. Single bent plates should be checked for flexural strength.

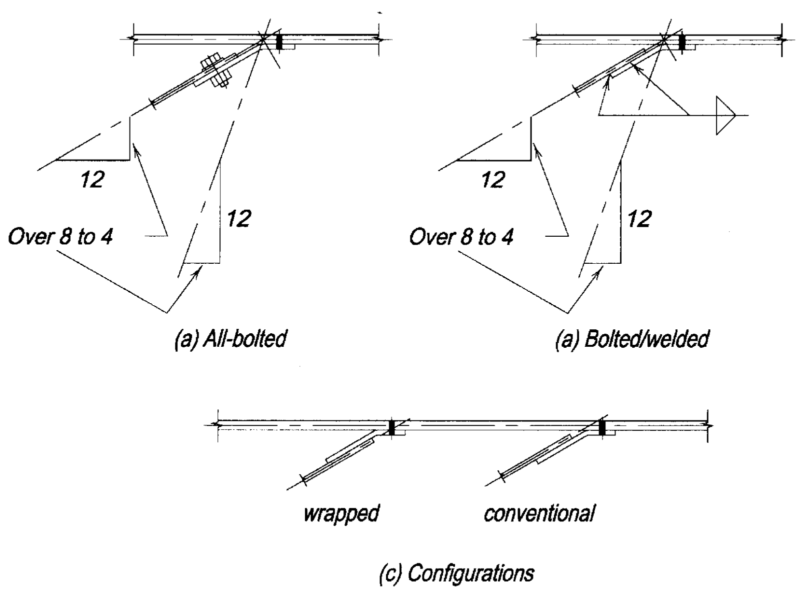


Figure 10–36. Skewed-beam connections with single-bent plates.

Table 10–13 gives clearance dimensions for bent double-angle connections and double- and single-bent plate connections, and specifies beam set-backs and gages. Since these dimensions are based on the maximum material thicknesses and fastener sizes indicated, it is suggested that in cases where many duplicate connections with less than maximum material or fasteners are required, savings can be effected if these dimensions are developed from specific bevels, beam sizes, and fitting thicknesses.

Skewed single-plate and skewed end-plate connections, shown in Figures 10–37 and 10–40, provide a simple, direct connection with a minimum of fittings and multiple punching requirements. When fillet-welded, these connections may be used for skews up to 30° (or a slope of 6⁵⁄16-in-12) provided the root opening formed does not exceed ³⁄16 in. For skew angles greater than 30°, see AWS D1.1, Section 2.2.5.2.

The maximum beam-web thickness which may be supported is a function of the maximum root opening and the angle of skew. If the thickness of the beam web were such that a larger root opening were encountered, the skewed single plate or the web connecting to the skewed end plate may be beveled, as shown in Figures 10–37b and 10–38b. Since no root opening occurs with the bevel, there is no limitation on the thickness of the beam web. However, beveling, especially of the beam web, requires careful finishing and is an expensive procedure which may outweigh its advantages.

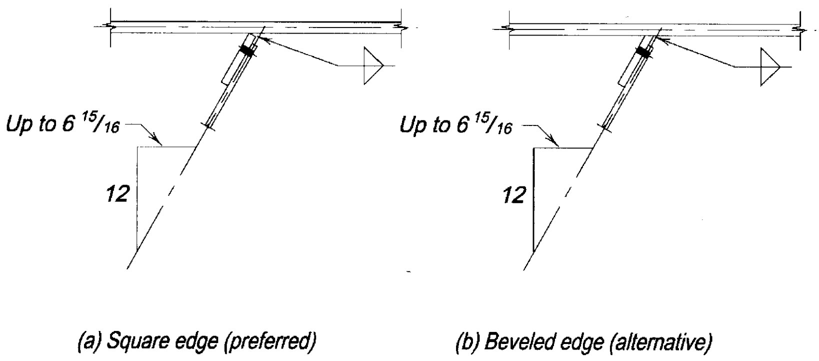


Figure 10–37. Skewed single-plate connections.

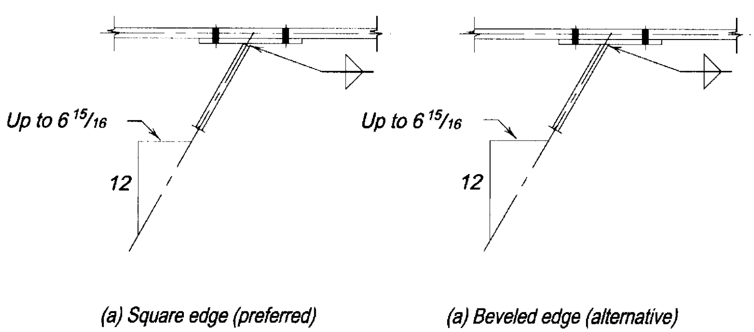


Figure 10–38. Skewed shear end-plate connections.

The design of skewed end-plate connections is similar to that discussed previously in "Shear End-Plate Connections" in this Part. However, when the gage of the bolts is not centered on the beam web, this eccentric loading should be considered. The design of skewed single-plate connections is similar to that discussed previously in "Single-Plate Connections" in this Part.

Table 10–13 specifies gages and the dimension A which is added to the fillet weld size to compensate for the root opening for skewed end-plate connections. This table is based conservatively on a gap of $^1/_8$ in. For beam webs beveled to the appropriate skew, $A = 0$ and the tabulated values do not apply. Table 10–13 also provides similar information for skewed single-plate connections. Additionally, this table provides clearances and dimensions for groove-welded single-plate connections with backing bars for skews greater than 30°; refer to AWS D1.1 for prequalified welds for both types of joints.

When skewed, stiffened seated connections are used, the stiffening element should be located so as to cross the skewed beam centerline well out on the seat. This can be accomplished by shifting the stiffener to the left or right of center to support beams which skew to the left or to the right, respectively. Alternatively, it may be possible to skew the stiffening element.

Sloped Connections

A beam is said to be sloped if the plane of its web is perpendicular to the plane of the face of the supporting member, but its flanges are not perpendicular to this face. The angle of slope B is shown in Figure 10–33b and represents the vertical angle to which the fittings must be set to the web of the sloped beam, or the amount that seat and top angles must be bent.

The design of sloped connections usually can be adapted directly from the rectangular connections covered earlier in this part, with consideration of the geometry of the connection to establish the location of fittings and fasteners. Note that sloped beams often require copes to clear supporting girders, as illustrated in Figure 10–39.

Figure 10–40 shows a sloped beam with double-angle connections, welded to the beam and bolted to the support. The design of this connection is essentially similar to that for rectangular double-angle connections. Alternatively, shear end-plate, tee, single-angle, single-plate, or seated connections could be used. Selection of a particular connection type may be influenced by fabrication economy, erectability, and/or by the types of connections used elsewhere in the structure.

Sloped seated beam connections may utilize either bent angles or plates, depending on the angle of slope. Dimensioning and entering and clearance requirements for sloped seated connections are generally similar to those for skewed connections. The bent seat and top plate shown in Figure 10–41 may be used for smaller bevels.

When the angle of slope is small, it is economical to place transverse holes in the beam web on lines perpendicular to the beam flange; this requires only one stroke of a multiple punch per line. Since non-standard hole arrangements, then, usually occur in the connecting materials (which are single-punched), this requires that sufficient dimensions be provided for the connecting material to contain fasteners with adequate edges and gages, and at the same time fit the angle to the web without encroaching on the flange fillets of the beam. For the end connection of the beam, this was accomplished by using a 6-in. angle leg; a 4-in. or even a 5-in. leg would not have furnished sufficient edge distance at the extreme fastener.

As the angle of slope increases, however, bolts for the end connections cannot conveniently be lined up to permit simultaneous punching of all holes in a transverse row. In this case, the fabricator may choose to disregard beam gage lines and arrange the hole-punching so that ordinary square-framed connection material can be used throughout, as shown in Figure 10–42.

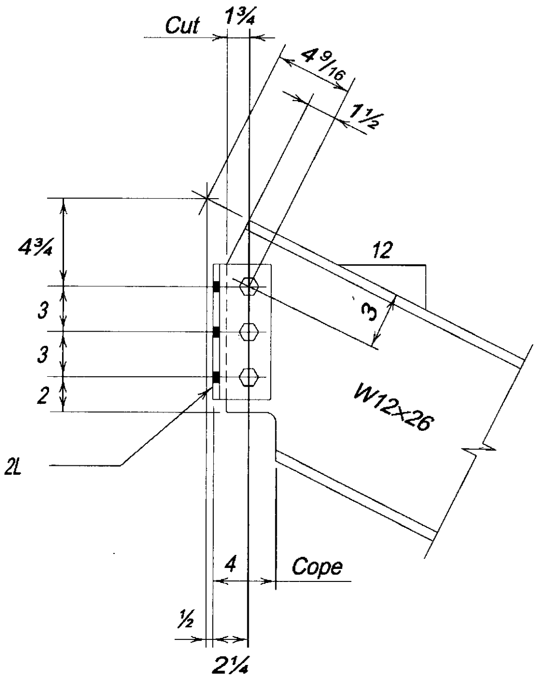


Figure 10–39. Sloped double-angle connection.

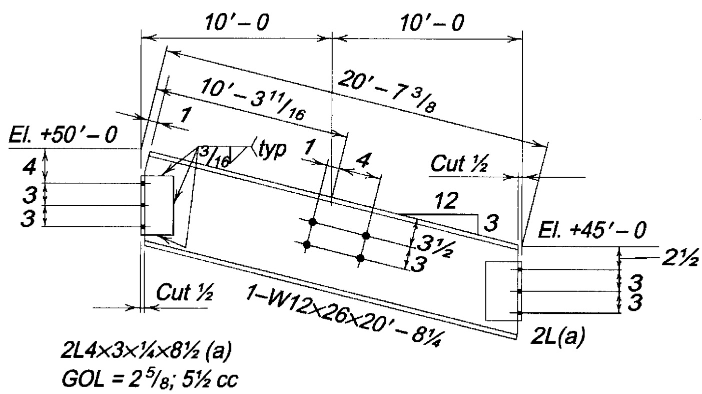


Figure 10–40. Sloped double-angle connection.

Canted Connections

A beam perpendicular to the face of a supporting member, but rotated so that its flanges are tilted with respect to those of the support, is said to be canted. The angle of cant C is shown in Figure 10–33c.

The design of canted connections usually can be adapted directly from the rectangular connections covered earlier in this part. In Figure 10–43, a double-angle connection is used.

Alternatively, shear end-plate, seated, single-angle, single-plate, and tee connections may also be used.

For channel B2, which is supported by a sloping member B1 (not shown), to match the hole pattern in supporting member B1, the holes in the connecting materials must be canted.

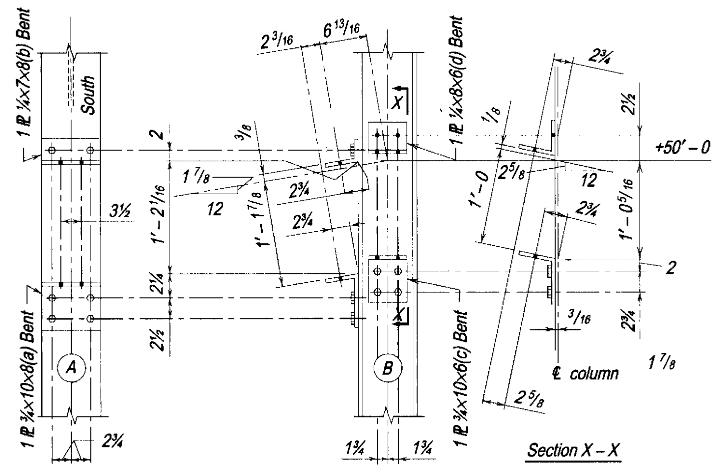


Figure 10–41. Sloped seated connections.

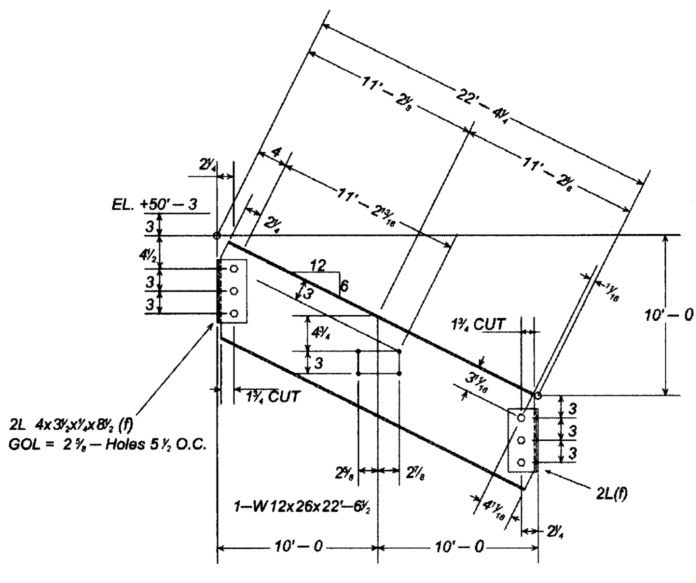


Figure 10–42. Sloped beam with rectangular connections.

As shown in Figure 10–44, the top flange of the channel and the connection angles d^R and d^L are cut to clear the flanges of beam B1. In this detail, with a 3-in-12 angle of cant, 4-in. legs were wide enough to contain the pattern of hole-punching.

Since the multiple punching or drilling of column flanges requires strict adherence to column gage lines, punching is generally skewed in the fittings. When, for some reason, this is not possible, as in Figure 10–45, skewed reference lines are shown on the column to aid in matching connections.

When canted connecting materials are assembled on the beam, particular care must be used in determining the direction of skew for punching the connection angles. An error reversing this skew may permit matching of holes in both members, but the beam will be canted opposite to the intended direction.

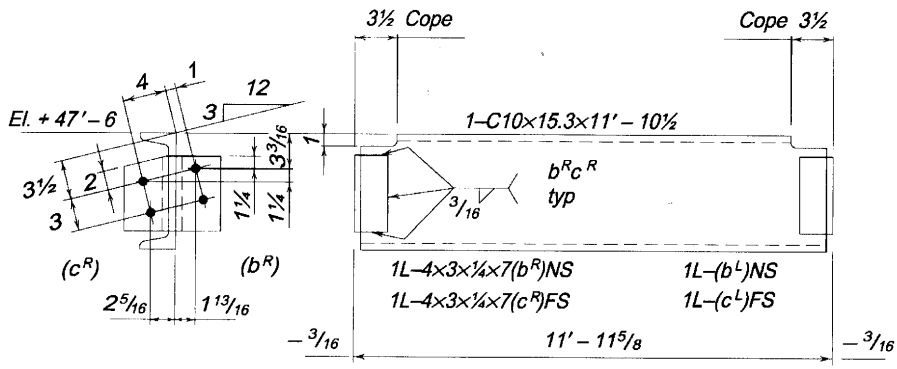


Figure 10–43. Canted double-angle connections.

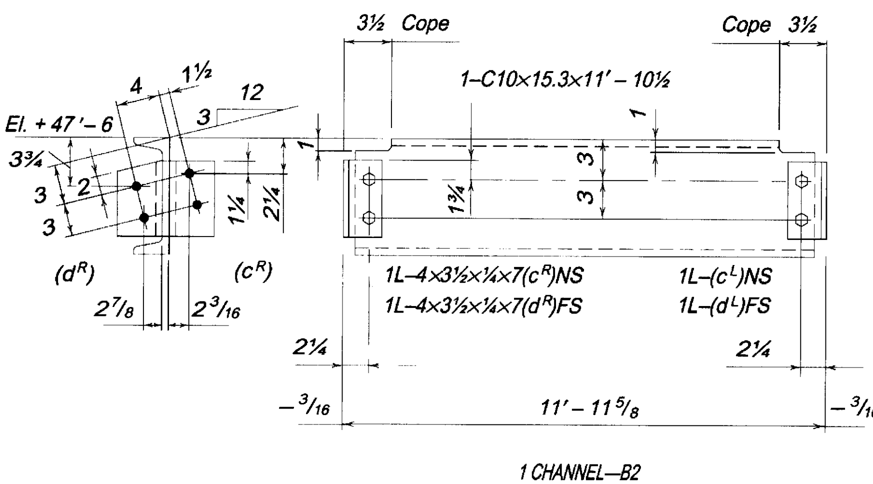


Figure 10–44. Canted connections to a sloping support.

Note the connection angles in Figure 10–45 are shown shop-welded to the beam. This was done to provide tightening clearance for ¾-in. high-strength field bolts in the opposite leg. Had the shop fasteners been bolts, it would have been necessary to stagger the field and shop fasteners and provide longer angles for the increased spacing.

Canted seated beams, shown in Figure 10–46, present few problems other than those in ordinary square-end seated beams. Sufficient width and length of angle leg must be provided to contain the gage line punching or drilling in the column face, as well as the off-center location of the holes matching the punching in the beam flange. The elevation of the top flange centerline and the bevel of the beam flange may be given for reference on the beam detail, although the bevel shown will not affect the fabrication.

Inclines in Two or More Directions (Hip and Valley Framing)

When a beam inclines in two or more directions with respect to the axis of its supporting member, it can be classified as a combination of those inclination directions. For example, the beam of Figure 10–33d is both skewed and sloped. Angle A shows the skew and angle B shows the slope. Note that, since the inclined beam is foreshortened in the elevation, the true angle B appears only in the auxiliary projection, Section X-X. The development of these details is quite complicated and graphical solutions to this compound angle work can be found in any textbook on descriptive geometry. Accurate dimensions may then be determined with basic trigonometry.

DESIGN CONSIDERATIONS FOR SIMPLE SHEAR CONNECTIONS TO HSS COLUMNS

Many of the familiar simple shear connections that are used to connect to wide-flange columns can be used with HSS columns. These include double and single angles, unstiffened and stiffened seats, single plates, and tee connections. One additional connection that is unique for HSS columns is the through-plate; note that this alternative is seldom required structurally and presents a significant economic penalty when a single plate connection would otherwise suffice. Variations in attachments are more limited with HSS columns since the connecting element will typically be shop-welded to the HSS and bolted

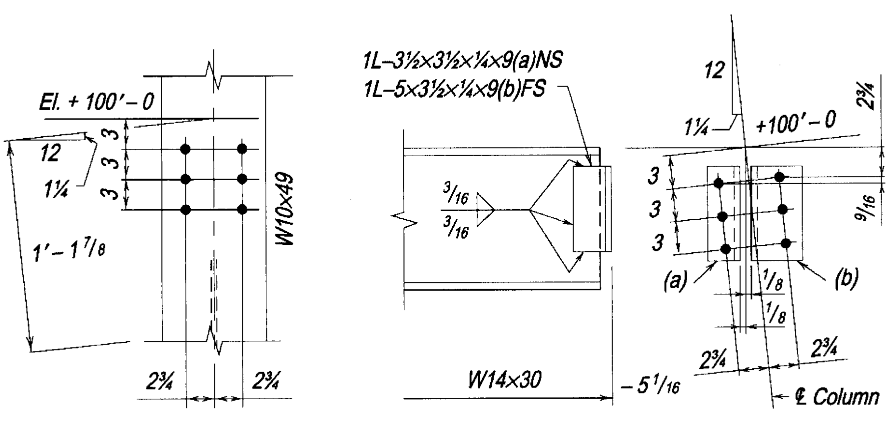


Figure 10–45. Canted connection to column flange.

to the supported beam. Except for seated connections, the bolting will be to the web of a wide-flange or other open profile section. Coping is not required except for bottom-flange copes that facilitate knifed erection with double-angle connections.

Double-Angle Connections to HSS

Table 10–1 is a design aid for double-angle connections. The table shows the compatible sizes of W-beams for the various connection configurations. Based on maximum beam web thickness, maximum weld size, maximum HSS corner radius and 4-in. outstanding angle legs, double-angle connections may be used with any HSS having a width greater than or equal to 12 in. If 3-in. outstanding angle legs are used for connections with six bolts or less, HSS with widths of 10 in. are acceptable for obtaining welds on the flat of the side. For smaller web thicknesses, welds and corner radii, it may be possible to fit the connection on widths of 10 in. if the outstanding angle legs are 4 in. and on widths of 8 in. for outstanding angle legs of 3 in. However, these dimensions must be verified for a particular case. See the tabulated workable flat dimensions for HSS in Part 1.

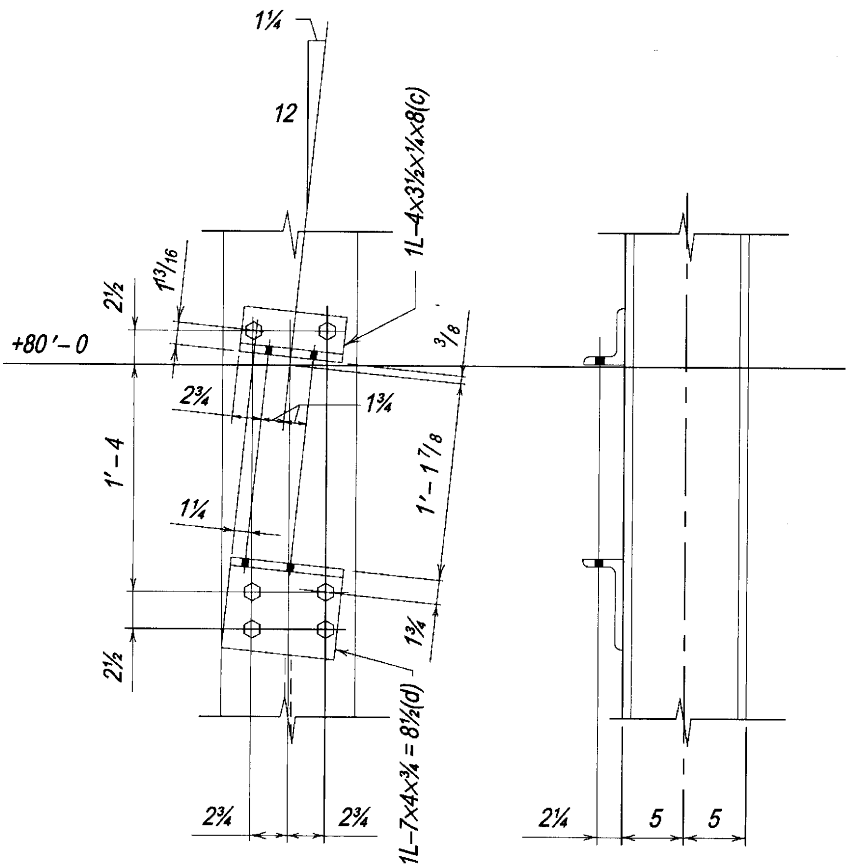


Figure 10–46. Canted seated connections.

Single-Plate Connections to HSS

As long as the HSS wall is not classified as a slender element, the local distortion caused by the single-plate connection will be insignificant in reducing the column strength of the HSS (Sherman, 1996). Therefore, single-plate connections may be used with HSS when $b/t \leq 1.40(E/F_y)^{0.5}$ or 35.1 for $F_y = 46$ ksi. Single-plate connections may also be used with round HSS as long as they are non-slender under axial load ($D/t \leq 0.11E/F_y$).

Unstiffened Seated Connections to HSS

In order to properly attach seat angles to the flat of the HSS, the workable flat must be large enough to accommodate both the width of the seat angle and the welds. Seat widths are usually 6 in. or 8 in., but other widths may also be used. See the tabulated workable flat dimensions for HSS in Part 1.

Table 10–6 may be used for unstiffened seated connections to HSS. The minimum HSS thicknesses are established based on the weld strength. If the HSS thickness is less than the minimum value, the weld strength must be reduced proportionally.

Stiffened Seated Connections to HSS

Tables 10–8 and 10–14 are design aids for stiffened seated connections. Table 10–8 is applicable to all member types, and Table 10–14 presents specific limits for HSS, based on the yield-line mechanism limit state for HSS. Some values for small connection lengths L and large HSS widths B have been reduced to meet the limit-state for a line load with a width of $0.4L$ across the HSS, per AISC Specification Section K1. The strength of the connection is obtained by multiplying the tabulated value for a particular HSS width and stiffener length by the square of the HSS thickness and dividing by the width of the seat. For combinations of B and L that are not listed in Table 10–14, the HSS does not have sufficient flat width to accommodate a weld to the seat that is $0.2L$ on each side of the stiffener. Since the required width also depends on the stiffener thickness and the HSS corner radius, the HSS width must be checked even when the values are tabulated. See the tabulated workable flat dimensions for HSS in Part 1.

The minimum HSS thicknesses associated with the weld strengths of Table 10–8 are given in Table 10–14. If the HSS thickness is less than the minimum tabulated value, the weld strength must be reduced proportionally.

Through-Plate Connections

In the through-plate connection shown in Figure 10–47, the front and rear faces of the HSS are slotted so that the plate can be passed completely through the HSS and welded to both faces. Through-plate connections should be used when the HSS wall is classified as a slender element ($b/t > 1.40(E/F_y)^{0.5}$ or 35.1 for $F_y = 46$ ksi for rectangular HSS; $D/t > 0.11E/F_y$ for round HSS and Pipe) or does not satisfy the punching shear limit-state. A single-plate connection is more economical and should be used if the HSS is neither slender nor inadequate for the punching shear rupture limit-state.

Through-plate connections have the same limit-states as single-plate connections and Table 10–9 may be used to determine the size and number of bolts and the plate thickness. The welds, however, are subject to direct shear and may not have to be as large as those for single-plate connections. For equilibrium of the forces in Figure 10–47, the shear in the welds on the front face should not exceed the strength of the pair of welds. The HSS wall strength can be matched to the weld shear strength to determine the minimum thickness, as illustrated in Part 9. If the thickness of the HSS is less than the minimum, the weld strength must be reduced proportionally. Conservatively, the welds on the rear face may be the same size.

When a connection is made on both sides of the HSS with an extended through-plate, the portion of the plate inside the HSS is subject to a uniform bending moment. For long connections, this portion of the plate may buckle in a lateral-torsional mode prior to yielding, unless H is very small. Using a thicker plate to prevent lateral-torsional buckling would restrict the rotational flexibility of the connection. Therefore, it must be recognized that the plate may buckle and that the moment will be shared with the HSS wall in a complex manner. However, if the HSS would be satisfactory for a single-plate connection, the lateral-torsional buckling limit-state is not a critical concern involving loss of strength.

Single-Angle Connections

For fillet welding on the flat of the HSS side, while keeping the center of the beam web in line with the center of the HSS, single-angle connections must be compatible with one-half the workable flat dimension provided in Part 1. Generally, the following HSS widths and thicknesses will work:

$b = 8$ in. and $t \leq {}^{1}/_{4}$ in.
$b = 9$ in. and $t \leq {}^{3}/_{8}$ in.
$b \geq 10$ in. and any nominal thickness

Alternatively, single angles can be welded to narrow HSS with a flare-bevel weld.

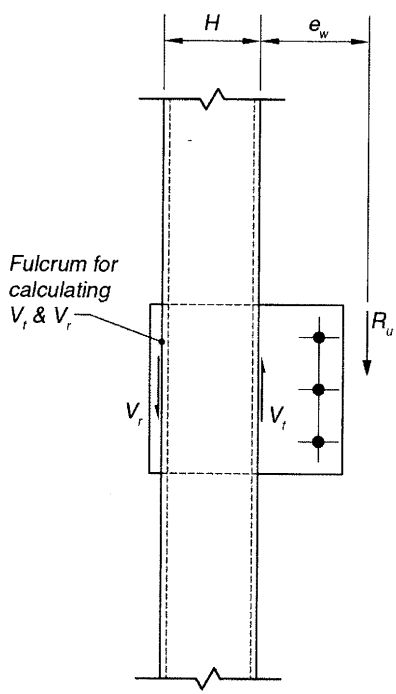


Figure 10–47. Shear forces in a through-plate connection.

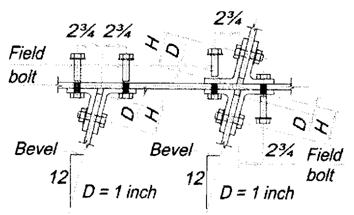
Table 10–12
Minimum Inside Radius for Cold-Bending[1]

ASTM Designation[2]	Thickness t, in.			
	Up to $^3/_4$	Over $^3/_4$ to 1	Over 1 to 2	Over 2
A36, A572-42	$1^1/_2\,t$	$1^1/_2\,t$	$1^1/_2\,t$	$2t$
A242, A529-50, A529-55, A572-50, A588, A992	$1^1/_2\,t$	$1^1/_2\,t$	$2\,t$	$2^1/_2\,t$
A572-55, A852	$1^1/_2\,t$	$1^1/_2\,t$	$2^1/_2\,t$	$3\,t$
A572-60, A572-65	$1^1/_2\,t$	$1^1/_2\,t$	$3\,t$	$1^1/_2\,t$
A514	$1^3/_4\,t$	$2^1/_4\,t$	$4^1/_2\,t$	$5^1/_2\,t$

1 Values are for bend lines perpendicular to direction of final rolling. If bend lines are parallel to final rolling direction, multiply values by 1.5.
2 The grade designation follows the dash; where no grade is shown, all grades and/or classes are included.

Table 10–13
Clearances for All-Bolted Skewed Connections

Values given are for webs up to ¾-in. thick, angles up to ⅝-in. thick, and bent plates up to ½-in. thick. Bolts are either ⅞-in. diameter or 1 in. diameter, as noted. Values will be conservative for material thinner than the maximums listed, or for work with smaller bolts, and may be reduced to suit conditions by calculation or layout. For thicker material or larger bolts, check entering, driving, and tightening clearances and increase D and bolt gages as necessary. All dimensions are in inches. Enter bolts as shown.



Bent angles

Values of *H* for Various Fastener Combinations

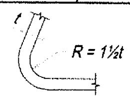
Field Bolts		⅞	1
Shop Bolts		⅞	1
Bevel	Up to 1	4*	4¼*
	Over 1 to 2	4⅛	4⅜
	Over 2 to 3	4⅜	4¾

*For back to back connections, stagger shop and field bolts or increase the 2¾-in. field bolt dimension to 3¼.

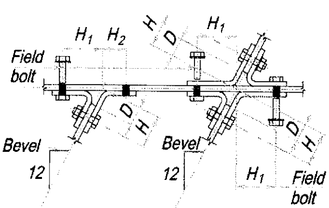
Values of *H*, *H*₁, *H*₂, and *D* for Various Bolt Combinations

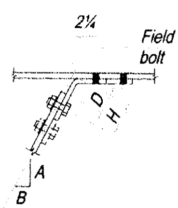
Field Fastener		⅞			1			
Shop Fastener		⅞			1			
Dimension		H	H_1	H_2	H	H_1	H_2	D
Bevel	Over 3 to 4	3¾	3¼	2½	4¼	3¼	2¾	1¼
	Over 4 to 5	3¾	3½	2¼	4½	3½	2½	1¼
	Over 5 to 6	4	3¾	2¼	4¾	3¾	2¼	1½
	Over 6 to 7	4½	4	2¼	5	4	2¼	1½
	Over 7 to 8	4¾	4¼	2¼	5¼	4¼	2¼	1½

Double bent plates



Min. radius of cold bend for A 36 steel up to ½ in. thick. For other bends see Table10-12





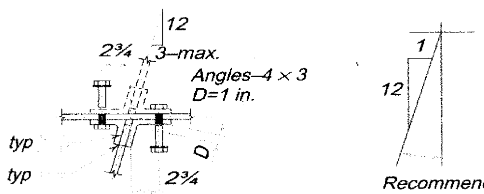
Field bolts—1 in. dia. max.
Shop bolts—1 in. dia. max.

Single bent plates

A	B	Shop Bolts D	Shop Bolts H
12	Over 8 to 9	1½	3
12	Over 9 to 10	1⅝	3⅛
12	Over 10 to 11	1¾	3¼
12	Over 11 to 12	1⅞	3⅜
Under 12 to 11	12	2⅛	3⅝
Under 11 to 10	12	2¼	3¾
Under 10 to 9	12	2½	4
Under 9 to 8	12	2¾	4¼
Under 8 to 7	12	3¼	4¾
Under 7 to 6	12	3¾	5¼
Under 6 to 5	12	4½	6
Under 5 to 4	12	5⅝	7⅛

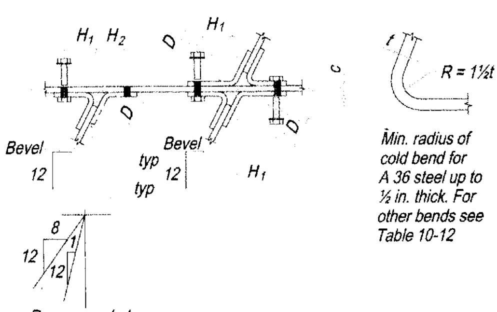
Table 10–13 (continued)
Clearances for Bolted/Welded Skewed Connections

Values given are for webs up to ¾-in. thick, angles up to ⅝-in. thick, and bent plates up to ½-in. thick, with bolts 1 in. diameter maximum. Values will be conservative for thinner material and for work with smaller bolts, and may be reduced to suit conditions by calculation or layout. For thicker material or larger bolts check entering and tightening clearances and increase beam set-back *D* and bolt gages as necessary. Enter bolts as shown. All dimensions are in inches.



Recommended range of skews

Bent angles

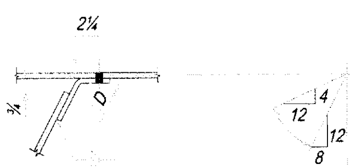


Recommended range of skews

Bevel	*D*	H_1	H_2
Over 3 to 4	*c* + ⅝	3¼	2¾
Over 4 to 5	*c* + 11⁄16	3½	2½
Over 5 to 6	*c* + ¾	3¾	2¼
Over 6 to 7	*c* + 13⁄16	4	2¼
Over 7 to 8	*c* + ⅞	4¼	2¼

$$C = \frac{t_w}{2} + 1/16''$$

Double bent plates



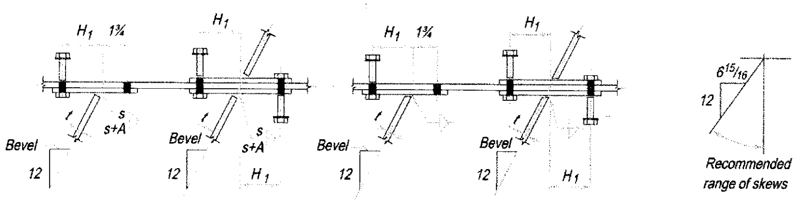
Determine value of D by calculation or layout

Recommended range of skews

Single bent plates

Table 10–13 (continued)
Clearances for Bolted/Welded Skewed Connections

Values given are for material and bolt sizes noted below. See "Shear End-Plate Connections" in Part 9 for proportioning these connections. *S* indicates weld size required for strength, or a size suitable to the thickness of material. When the beam web is cut square, only that portion of the table above the heavy lines is applicable. Dimension *A* is added to the weld size to compensate for the root opening caused by the skew. When the beam web is beveled to the required skew, values of H_1 for the entire table are valid, and A = 0. In either case, where weld strength is critical, increase the weld size to obtain the required throat dimension. Enter bolts as shown. All dimensions are in inches.



Square ends *Beveled ends*

End plates

Bevel	t = 1/4		t = 5/16		t = 3/8		t = 7/16		t = 1/2		t = 5/8		t = 3/4	
	H_1	A	H_1	A	H_1	A	H_1	A	H_1	A	H_1	A	H_1	A
Up to 1⅝	1¾	0	1¾	0	1¾	1/16	1¾	1/16	1¾	1/16	1⅞	⅛	1⅞	⅛
Over 1⅝ to 2⅛	1¾	0	1¾	1/16	1⅞	1/16	1⅞	1/16	1⅞	⅛	2	⅛	2	⅛
Over 2⅛ to 3¼	1⅞	1/16	1⅞	⅛	2	⅛	2	⅛	2	⅛	2⅛	0	2⅛	0
Over 3¼ to 4⅜	2⅛	⅛	2⅛	⅛	2⅛	⅛	2⅛	0	2¼	0	2¼	0	2⅜	0
Over 4⅜ to 5⅝	2¼	⅛	2¼	⅛	2⅜	0	2⅜	0	2⅜	0	2½	0	2½	0
Over 5⅝ to 6 15/16	2½	⅛	2½	0	2½	0	2½	0	2⅝	0	2⅝	0	2¾	0

Bolts: ⅞-in. diameter maximum
End Plate thickness: ⅜-in. maximum
Supporting web thickness: ¾-in. maximum

Use of fillet welds is limited to connections with bevels of 6 15/16 in 12 and less.
For greater bevels consider use of double or single bent plates.

Table 10–13 (continued)
Clearances for Bolted/Welded Skewed Connections

5/16- and 3/8-in. Plate Thickness

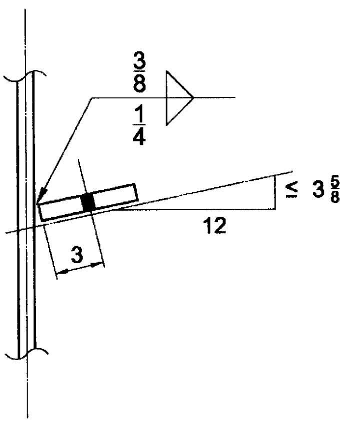
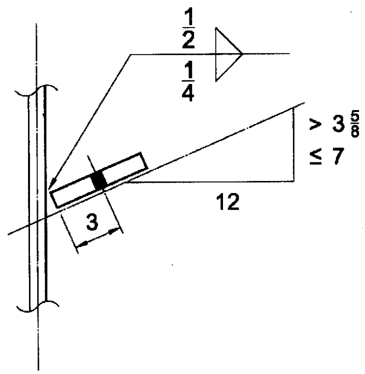
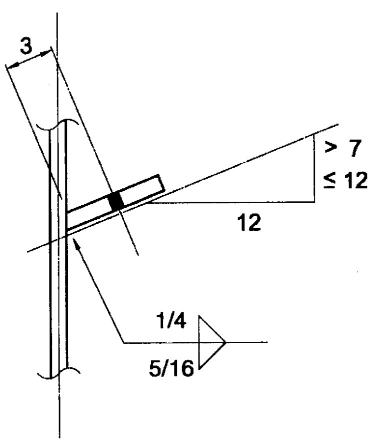
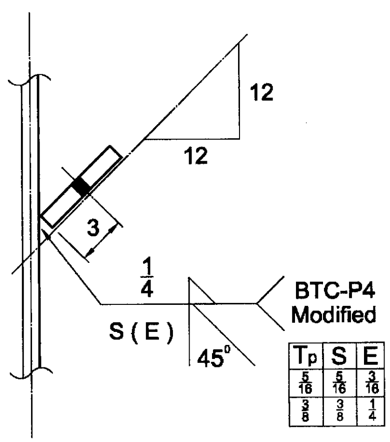
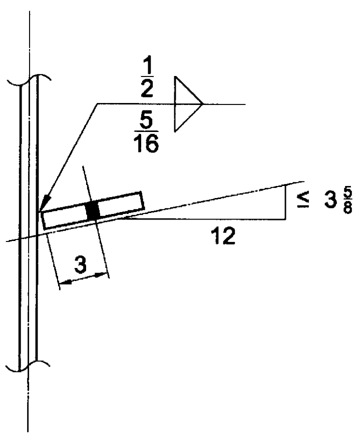
For $\theta \leq 17°$ from Perpendicular	For $17° < \theta \leq 30°$ from Perpendicular
	
For $30° < \theta \leq 45°$ from Perpendicular	**For $\theta = 45°$ from Perpendicular**
	

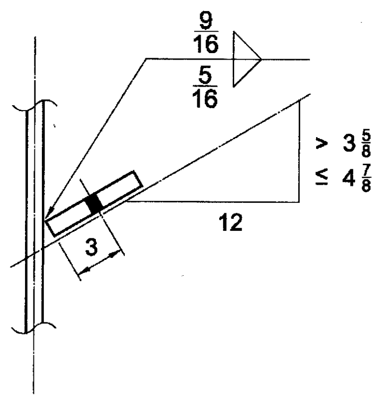
Table 10–13 (continued)
Clearances for Bolted/Welded Skewed Connections

$\frac{1}{2}$-in. Plate Thickness

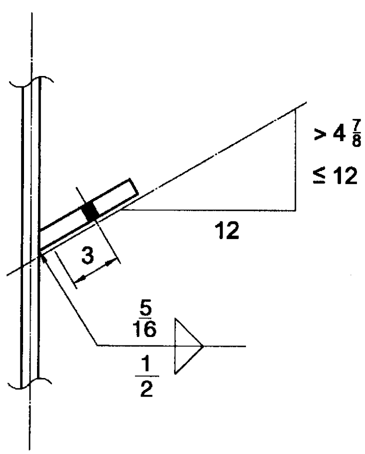
For θ ≤ 17° from Perpendicular



For 17°< θ ≤ 22° from Perpendicular



For 22°< θ ≤ 45° from Perpendicular



For θ = 45° from Perpendicular

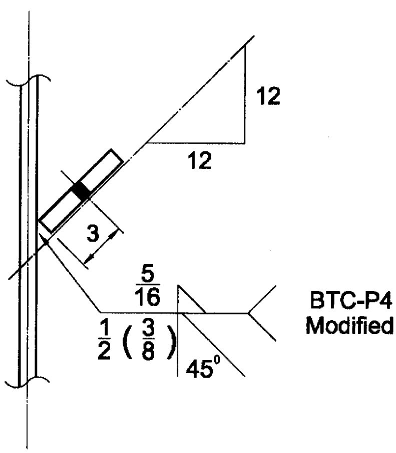


Table 10–14
Required Length and Thickness for Stiffened Seated Connections to HSS

HSS Wall Strength Factor, R_uW/t^2 or R_aW/t^2, kips/in.

L, in.	HSS Width *B*, in.											
	5		5.5		6		7		8		9	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
6	558	839	545	819	536	805	526	791	525	789	528	793
7	687	1030	664	997	646	971	625	940	615	925	612	920
8			798	1200	771	1160	735	1100	714	1070	704	1060
9					911	1370	856	1290	823	1240	804	1210
10					1070	1600	990	1490	942	1420	912	1370
11							1140	1710	1070	1610	1030	1550
12							1300	1960	1210	1820	1160	1740
13									1370	2060	1290	1940
14									1540	2310	1440	2170
15									1720	2580	1600	2410
16											1700	2660
17											1960	2940

Required HSS Thickness

Weld Size, in.	Min. HSS Thickness, in.
1/4	0.224
5/16	0.280
3/8	0.336
7/16	0.392
1/2	0.448
5/8	0.560

Table 10–14 (continued)
Required Length and Thickness for Stiffened Seated Connections to HSS

	HSS Wall Strength Factor, R_uW/t^2 or R_aW/t^2, kips/in.											
	HSS Width *B*, in.											
	10		12		14		16		18		20	
L, in.	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
6	534	802	552	830	561	843	491	737	437	656	393	590
7	614	922	625	940	644	968	667	1000	594	892	535	803
8	700	1050	704	1060	717	1080	736	1110	759	1140	699	1050
9	793	1190	787	1180	794	1190	809	1220	828	1240	851	1280
10	893	1340	876	1320	876	1320	885	1330	901	1350	920	1380
11	1000	1500	971	1460	962	1450	965	1450	976	1470	993	1490
12	1120	1680	1070	1610	1050	1580	1050	1580	1060	1590	1070	1600
13	1240	1870	1180	1770	1150	1730	1140	1710	1140	1710	1150	1720
14	1370	2070	1290	1940	1250	1880	1230	1850	1220	1840	1230	1840
15	1520	2280	1410	2120	1360	2040	1330	1990	1310	1980	1310	1970
16	1670	2510	1540	2320	1470	2210	1430	2150	1410	2120	1400	2100
17	1830	2760	1680	2520	1590	2390	1540	2310	1510	2260	1490	2240
18	2010	3020	1820	2740	1710	2570	1650	2470	1610	2420	1590	2380
19	2190	3300	1970	2970	1840	2770	1760	2650	1710	2580	1680	2530
20	2390	3600	2130	3210	1980	2980	1880	2830	1820	2740	1790	2680
21			2300	3460	2120	3190	2010	3020	1940	2910	1890	2840
22			2480	3730	2280	3420	2140	3220	2060	3090	2000	3010
23			2670	4020	2440	3660	2280	3430	2180	3280	2120	3180
24			2870	4310	2600	3910	2430	3650	2310	3480	2230	3360
25			3080	4630	2780	4170	2580	3880	2450	3680	2360	3540
26					2960	4450	2740	4110	2590	3890	2480	3730
27					3150	4730	2900	4360	2730	4110	2610	3930
28					3350	5030	3070	4620	2880	4330	2750	4130
29					3560	5340	3250	4890	3040	4570	2890	4340
30					3770	5660	3440	5160	3200	4810	3040	4560
31							3630	5450	3370	5070	3190	4790
32							3830	5750	3540	5330	3340	5020

Required HSS Thickness	
Weld Size, in.	Min. HSS Thickness, in.
1/4	0.224
5/16	0.280
3/8	0.336
7/16	0.392
1/2	0.448
5/8	0.560

Table 10–14 (continued)
Required Length and Thickness for Stiffened Seated Connections to HSS

	HSS Wall Strength Factor, R_uW/t^2 or R_aW/t^2, kips/in.											
	HSS Width B, in.											
	22		24		26		28		30		32	
L, in.	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
6	357	536	328	492	302	454	281	421	262	393	246	369
7	486	730	446	669	412	618	382	574	357	535	334	502
8	635	953	582	874	537	807	499	749	466	699	437	656
9	804	1210	737	1110	680	1020	632	948	590	885	553	830
10	943	1420	910	1370	840	1260	780	1170	728	1090	682	1020
11	1010	1520	1030	1560	1020	1530	944	1420	881	1320	826	1240
12	1080	1630	1100	1660	1130	1690	1120	1690	1050	1570	983	1470
13	1160	1740	1180	1770	1200	1800	1220	1830	1230	1850	1150	1730
14	1240	1860	1250	1880	1270	1910	1290	1940	1310	1970	1330	2010
15	1320	1980	1330	2000	1340	2020	1360	2040	1380	2070	1400	2110
16	1400	2100	1410	2120	1420	2130	1430	2160	1450	2180	1470	2210
17	1490	2230	1490	2240	1500	2250	1510	2270	1530	2290	1540	2320
18	1580	2370	1570	2370	1580	2370	1590	2390	1600	2410	1620	2430
19	1670	2510	1660	2500	1660	2500	1670	2510	1680	2520	1690	2540
20	1760	2650	1750	2630	1750	2630	1750	2630	1760	2640	1770	2660
21	1860	2800	1850	2770	1840	2760	1840	2760	1840	2770	1850	2780
22	1960	2950	1940	2920	1930	2900	1920	2890	1920	2890	1930	2900
23	2070	3110	2040	3070	2020	3040	2010	3030	2010	3020	2010	3030
24	2180	3280	2140	3220	2120	3190	2110	3170	2100	3160	2100	3150
25	2290	3450	2250	3380	2220	3340	2200	3310	2190	3290	2190	3290
26	2410	3620	2360	3540	2320	3490	2300	3450	2280	3430	2280	3420
27	2530	3800	2470	3710	2430	3650	2400	3600	2380	3570	2370	3560
28	2650	3990	2590	3890	2540	3810	2500	3760	2480	3720	2460	3700
29	2780	4180	2700	4060	2650	3980	2610	3920	2580	3870	2560	3840
30	2920	4380	2830	4250	2760	4150	2710	4080	2680	4030	2650	3990
31	3050	4590	2950	4440	2880	4330	2820	4250	2780	4180	2760	4140
32	3190	4800	3080	4630	3000	4510	2940	4420	2890	4350	2860	4300

Required HSS Thickness	
Weld Size, in.	Min. HSS Thickness, in.
1/4	0.224
5/16	0.280
3/8	0.336
7/16	0.392
1/2	0.448
5/8	0.560

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PART 11

DESIGN OF FLEXIBLE MOMENT CONNECTIONS

SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of flexible moment connections. For the design of simple shear connections, see Part 10. For the design of fully restrained moment connections, see Part 12. For connections that are part of a seismic force resisting system in which the seismic response modification factor, R, is taken greater than 3, the requirements in the AISC *Seismic Provisions for Structural Steel Buildings* also apply. The AISC *Seismic Provisions for Structural Steel Buildings* is available in Part 6 of the AISC *Seismic Design Manual* from the American Institute of Steel Construction, Inc. at www.aisc.org.

LOAD DETERMINATION

The behavior of PR moment connections is intermediate in degree between the flexibility of simple shear connections and the full rigidity of FR moment connections. Per AISC Specification Section B3.6b, PR moment connections are permitted upon evidence that the connections to be used are capable of furnishing, as a minimum, a predictable percentage of full end restraint. For further information on the use of PR moment connections, see Geschwindner (1991), Nethercot and Chen (1988), Gerstle and Ackroyd (1989), Deierlein et al. (1990), Goverdhan (1984) and Kishi and Chen (1986).

As an alternative, flexible moment connections (FMC) can be used as a simplified and conservative approach to PR moment connection design (Geschwindner and Disque, 2005). Using FMC, any end restraint that the connection may provide to the girder is assumed zero for gravity load because of the uncertainty of that restraint after repeated loading. The beam and its web connections are thus designed as simple, considering only the gravity loads. For lateral loads, the connection is assumed to behave as an FR moment connection for analysis and the full lateral load is carried by the assigned lateral frames. The resulting flexible moment connections are then designed as "fully restrained" for the calculated required strength due to lateral loads only.

Strength

With FMC, the partially restrained connection does not achieve its final moment resisting capacity until it has been subjected to a full cycle of factored maximum specified gravity and lateral loading. This process, termed "Shakedown", is fully described in Rex and Goverdhan (2002) and Geschwindner and Disque (2005). With FMC, this shakedown moment is the plastic moment of the connection. Since the connection is assumed to provide no end restraint to the girder under gravity loads, its full capacity is available to restrain lateral load. Stressing of the connection material may occur under gravity loads but this does not reduce the final strength of the structural connection.

Stability

The stability and second order effects for FMC frames are evaluated by the familiar effective length and amplification factor methods as provided in the AISC Specification Chapter C. The effective length is calculated by the conservative assumption that, under lateral loading, the column is rotationally restrained by only one (leeward) girder, pinned at its far end. The procedure is described in detail in Geschwindner and Disque (2005) and Chen and Lui (1991).

While flexible moment connections (see Figure 11–1) are not true PR moment connections, they do provide a simple, reliable and economical alternative in the design of connections that must resist lateral-load-induced moments. Flexible moment connections usually result in heavier beams and reduced rotational stiffness for columns (higher, more conservative K-factors). Additionally, there are several advantages to their use: (1) simplified analysis; (2) the beams and girders may be designed as simply connected members for gravity loads; and (3) the columns may be designed as axially loaded members with applied moments due to lateral load only. Certain provisions, however, must be met when using this type of moment connection:

1. The lateral frames must resist the lateral moments throughout the entire structure from top to bottom.
2. The beams, columns, and their connections must resist the applied moments for lateral loads.
3. The girders must be capable of carrying the full gravity load as simply supported beams.
4. The connection material must have sufficient inelastic rotation capacity to prevent the welds and/or fasteners from failing due to combined gravity and lateral loading.

Flexible moment connections deliver concentrated forces to the flanges of columns that must be accounted for in the design of the column and column panel-zone per AISC Specification Section J10. Either the column size can be selected with adequate flange and web thicknesses to eliminate the need for column stiffening, or transverse stiffeners and/or web doubler plates can be provided. For further information, refer to AISC Design Guide No. 13 *Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications* (Carter, 1999).

FLANGE-ANGLE FLEXIBLE MOMENT CONNECTIONS

Flange-angle flexible moment connections are made with top and bottom angles and a simple shear connection.

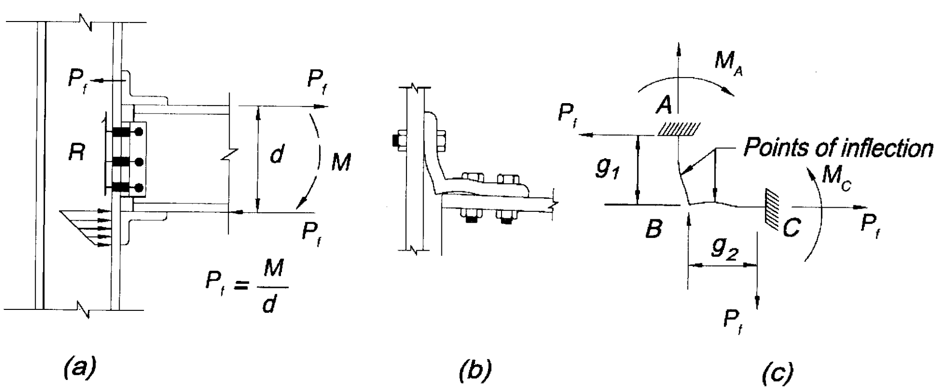


Figure 11–1. Flexible moment connection behavior.

The available strength of a flange-angle flexible moment connection is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8) and connecting elements (see Part 9). In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a.

The tensile force is carried to the angle by the flange bolts, with the angle assumed to deform as illustrated in Figure 11–1. A point of inflection is assumed between the bolt gage line and the face of the connection angle, for use in calculating the local bending moment and the corresponding required angle thickness. The effect of prying action must also be considered.

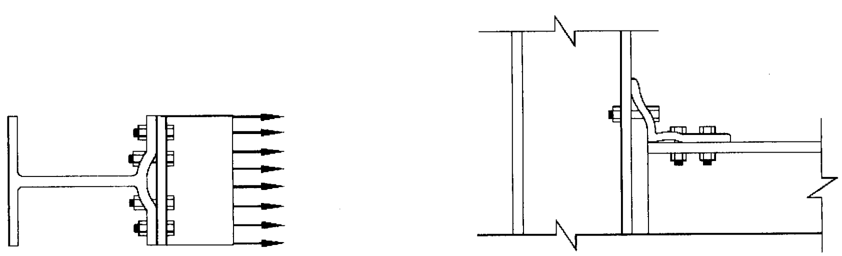
The strength of this type of connection is often limited by the available angle thickness and the maximum number of fasteners that can be placed on a single gage line of the vertical leg of the connection angle at the tension flange. Figure 11–2 illustrates the column flange deformation and shows that only the fasteners closest to the column web are fully effective in transferring forces.

FLANGE-PLATED FLEXIBLE MOMENT CONNECTIONS

Originally proposed by Blodgett (1966), and illustrated in Figure 11–3, a flange-plated flexible moment connection consists of a simple shear connection and top and bottom flange plates that connect the flanges of the supported beam to the supporting column. These flange plates are welded to the supporting column and may be bolted or welded to the flanges of the supported beam. An unwelded length of 1 1/2 times the flange-plate width, b_A, is normally assumed to permit the elongation of the plate necessary for FMC behavior. Other flange-plated details are illustrated in Figures 11–4a and 11–4b.

The available strength of a flange-plated flexible moment connection is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a.

The shop and field practices for flange-plated FR moment connections (see Part 12) are equally applicable to flange-plated flexible moment connections.



(a) (b)

Figure 11–2. Illustration of deformations in flexible moment connections.

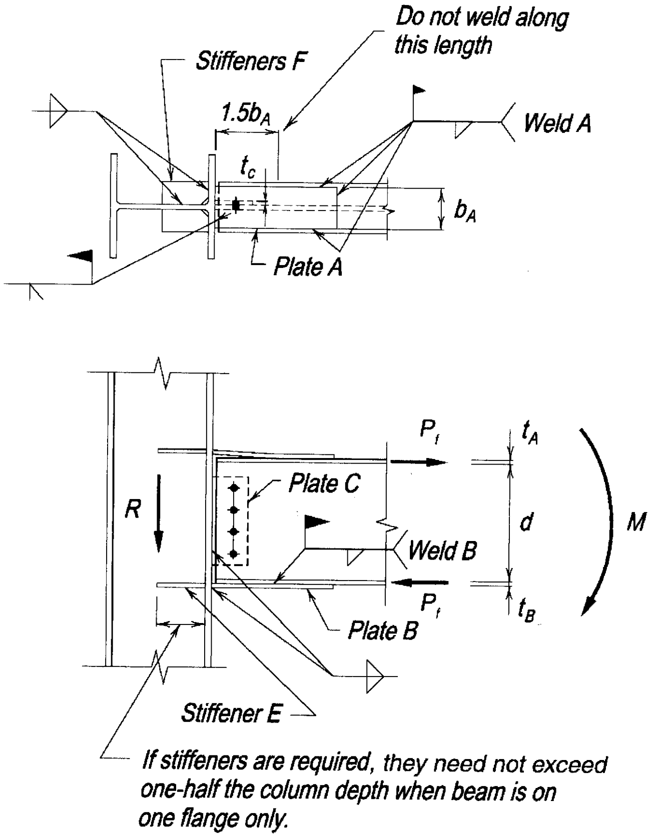


Figure 11–3. Flange-plated flexible moment connection.

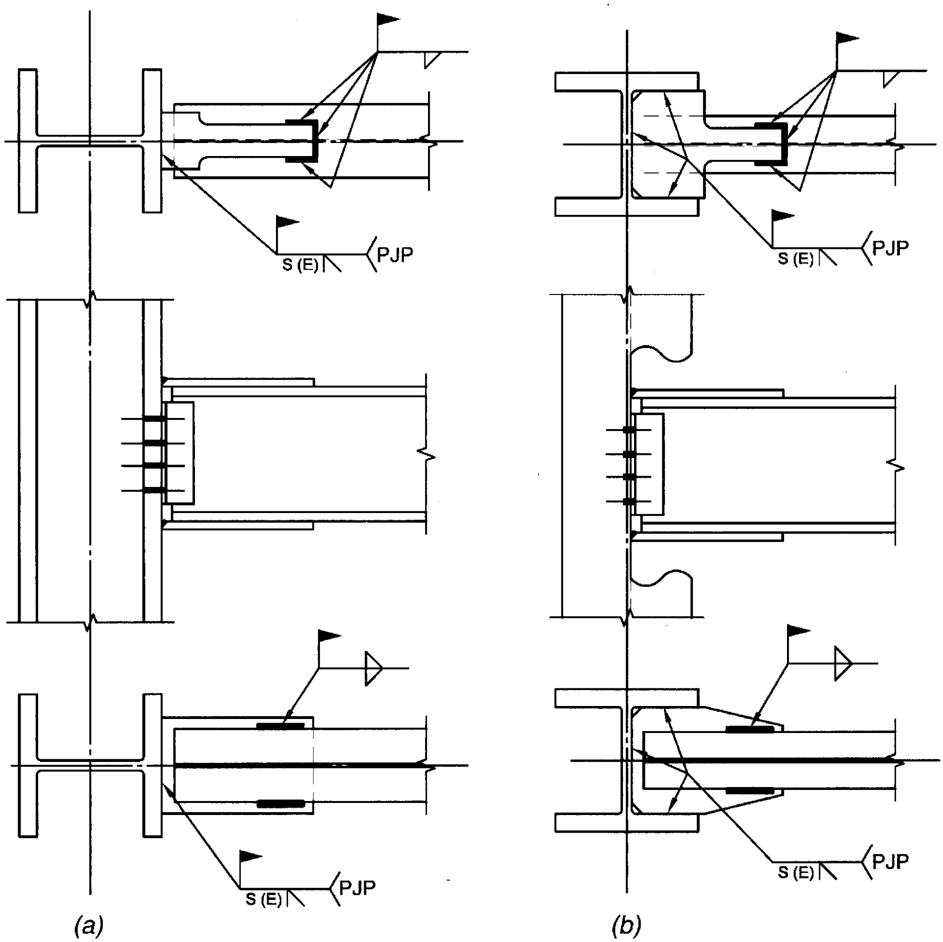


Figure 11–4. Typical flange-plated flexible moment connections.

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PART 12

DESIGN OF FULLY RESTRAINED (FR) MOMENT CONNECTIONS

SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of fully restrained (FR) moment connections. For the design of simple shear connections, see Part 10. For the design of flexible moment connections, see Part 11. For FR moment connections that are part of a seismic force resisting system in which the seismic response modification factor, R, is taken greater than 3, the requirements in the AISC *Seismic Provisions for Structural Steel Buildings* also apply. The AISC *Seismic Provisions for Structural Steel Buildings* is available in Part 6 of the AISC *Seismic Design Manual* from the American Institute of Steel Construction, Inc. at www.aisc.org.

FR MOMENT CONNECTIONS

Load Determination

As defined in AISC Specification Section B3.6b, FR moment connections possess sufficient rigidity to maintain the angles between connected members at the strength limit states, as illustrated in Figure 12–1. While connections considered to be fully restrained seldom actually provide for zero rotation between members, the small amount of rotation present is usually neglected and the connection is idealized as one exhibiting zero end rotation.

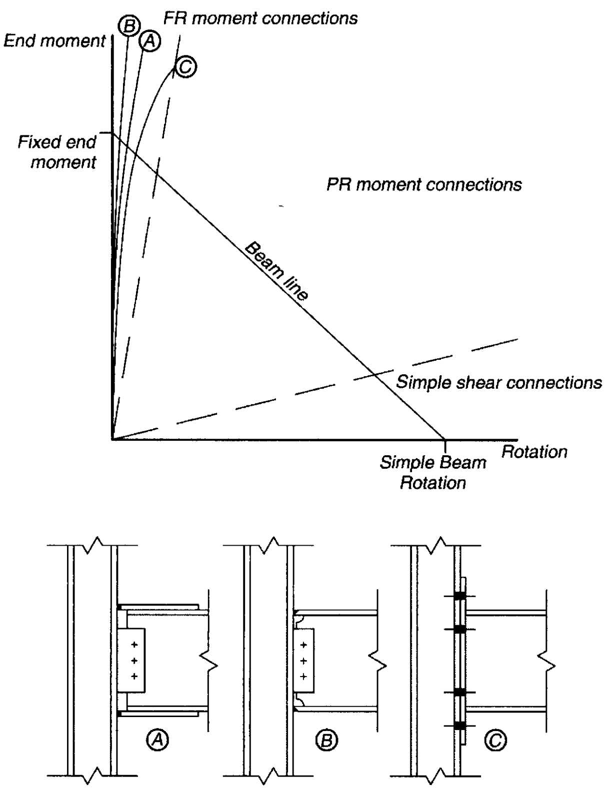


Figure 12–1. FR moment connection behavior.

End connections in FR construction are designed to carry the factored forces and moments, except that some inelastic but self-limiting deformation of a part of the connection is permitted. Huang, et al. (1973) showed that the moment can be resolved into an effective tension-compression couple acting as axial forces at the beam flanges. The flange force, P_{uf} or P_{af}, *is* determined as:

LRFD	ASD
$P_{uf} = \dfrac{M_u}{d_m}$	$P_{af} = \dfrac{M_a}{d_m}$

where

M_u or M_a = required beam end moment, kip-in.

d_m = moment arm between the flange forces, in. (varies for all FR connections and for stiffener design)

Shear is transferred through the beam-web shear connection. Since, by definition, the angle between the beam and column in an FR moment connection remains unchanged under loading, eccentricity can be neglected entirely in the shear connection. Additionally, it is permissible to use bolts in bearing in either standard or slotted holes perpendicular to the line of force. Axial forces, if present, are normally assumed to be distributed uniformly across the beam flange cross-sectional area. However, if the beam-web connection has sufficient stiffness, it can also be assumed to participate in the transfer of beam axial force.

Moment connections deliver concentrated forces to the flanges of columns that must be accounted for in the design of the column and column panel-zone per AISC Specification Section J10. Either the column size can be selected with adequate flange and web thickness to eliminate the need for column stiffening, or transverse stiffeners and/or web doubler plates can be provided. For further information, refer to AISC Design Guide No. 13 *Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications* (Carter, 1999).

Design Checks

The available strength of an FR moment connection is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). The effect of eccentricity in the shear connection can be neglected. Additionally, the strength of the supporting column (and thus the need for stiffening) must be checked. In all cases, the available strength, R_n/Ω or ϕR_n, must equal or exceed the required strength, R_u or R_a.

Temporary Support During Erection

Bolted construction provides a ready means to erect and temporarily connect members by use of the bolt holes. In contrast, FR moment connections in welded construction must be given special attention so that all pieces affecting the alignment of the welded joint may be erected, fitted, and supported until the necessary welds are made. Temporary support can be provided in welded construction by furnishing holes for erection bolts, temporary seats, special lugs, or by other means.

The effects of temporary erection aids on the finished structure should be considered, particularly on members subjected to tension loading or fatigue. They should be permitted to remain in place whenever possible since they seldom are reusable and the cost to remove them can be significant. If left in place, erection aids should be located so as not to cause a stress concentration. If, however, erection aids are to be removed, care should be taken so that the base metal is not damaged.

Temporary supports should be sufficient to carry any loads imposed by the erection process, such as the dead weight of the member, additional construction equipment, or material storage. Additionally, they must be flexible enough to allow plumbing of the structure, particularly in tier buildings.

Welding Considerations for Fully Restrained Moment Connections

Field welding should be arranged for welding in the flat or horizontal position and preference should be given to fillet welds over groove welds, whenever possible. Additionally, the joint detail and welding procedure should be constructed to minimize distortion and the possibility of lamellar tearing.

The typical complete-joint-penetration groove weld in a directly welded flange connection for a rolled beam can be expected to shrink about $^1/_{16}$ in. in the length dimension of the beam when it cools and contracts. Thicker welds, such as for welded plate-girder flanges, will shrink even more—up to $^1/_8$ in. or $^3/_{16}$ in. This amount of shrinkage can cause erection problems in locating and plumbing the columns along lines of continuous beams. A method of calculating weld shrinkage can be found in Lincoln Electric Co. (1973). Unnecessarily thick stiffeners with complete-joint-penetration groove welds should be avoided since the accompanying weld shrinkage may contribute to lamellar tearing.

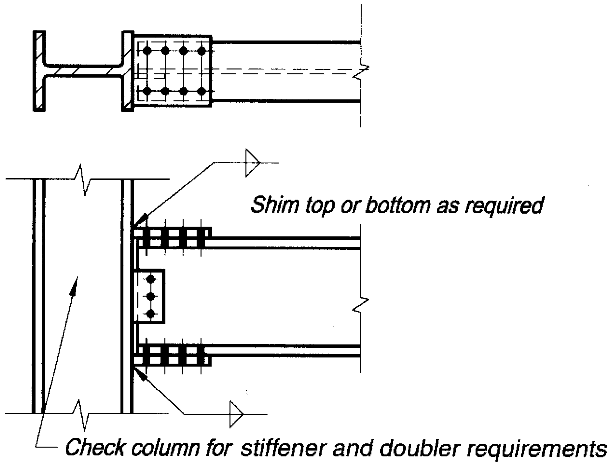
Weld shrinkage can best be controlled by fabricating the beam longer than required by the amount of the anticipated weld shrinkage. Alternatively, the weld-joint root opening can be increased. For further information, refer to AWS D1.1.

FR CONNECTIONS WITH WIDE-FLANGE COLUMNS

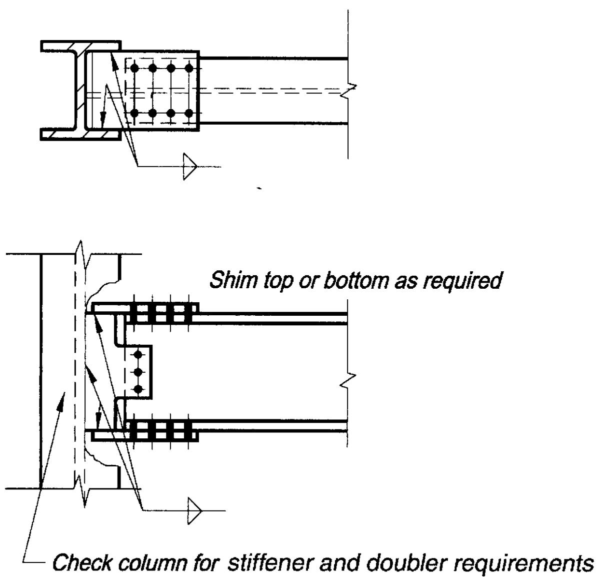
Flange-Plated FR Moment Connections

As illustrated in Figure 12–2, a flange-plated FR moment connection consists of a shear connection and top and bottom flange plates that connect the flanges of the supported beam to the supporting column. These flange plates are welded to the supporting column and may be bolted or welded to the flanges of the supported beam.

In a column-flange connection, the flange plates are usually located with respect to the column web centerline. Because of the column-flange mill tolerance on out-of-squareness with the web, it is desirable to shop-fit long flange plates from the theoretical column-web centerline to assure good field fit-up with the beam. Misalignment on short connections, as illustrated in Figure 12–3, can be accommodated by providing oversized holes in the plates. Since mill tolerances in both the beam and the column may cause significant shop and/or field assembly problems, it may be desirable to ship the flange plates loose for field attachment to the column.

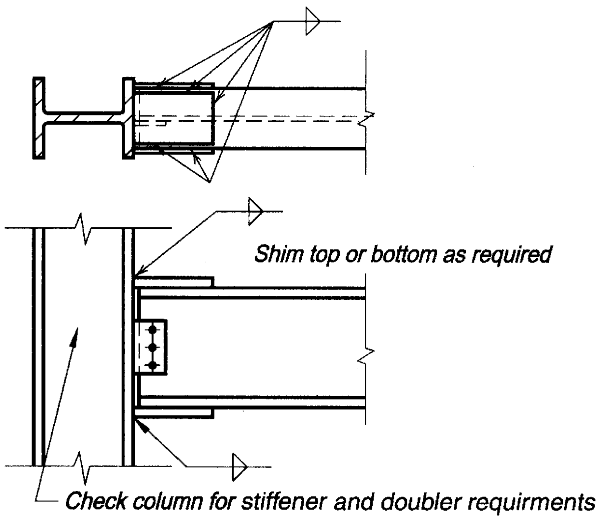


(a) Column flange support, bolted flange plates



(b) Column web support, bolted flange plates

Figure 12–2. Flange-plated FR moment connections.



(c) Column flange support, welded flange plates

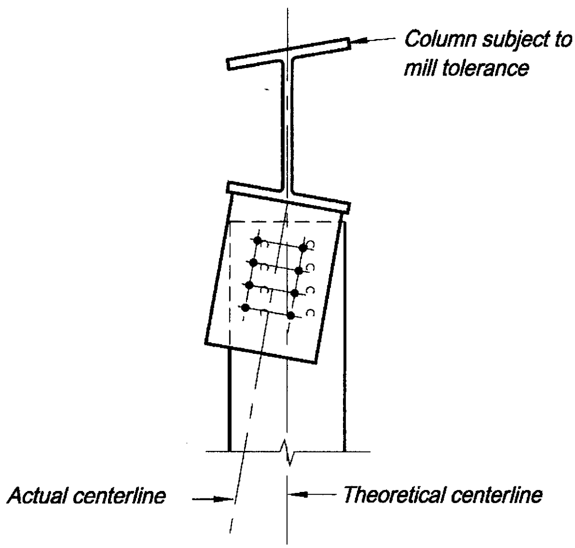


Figure 12–3. Effect of mill tolerances on flange-plated connections.

Directly Welded Flange FR Moment Connections

As illustrated in Figure 12–4, a directly welded flange FR moment connection consists of a shear connection and complete-joint-penetration groove welds, which directly connect the top and bottom flanges of the supported beam to the supporting column. Note, in Figure 12–4b, the stiffener extends beyond the toe of the column flange to eliminate the effects of triaxial stresses.

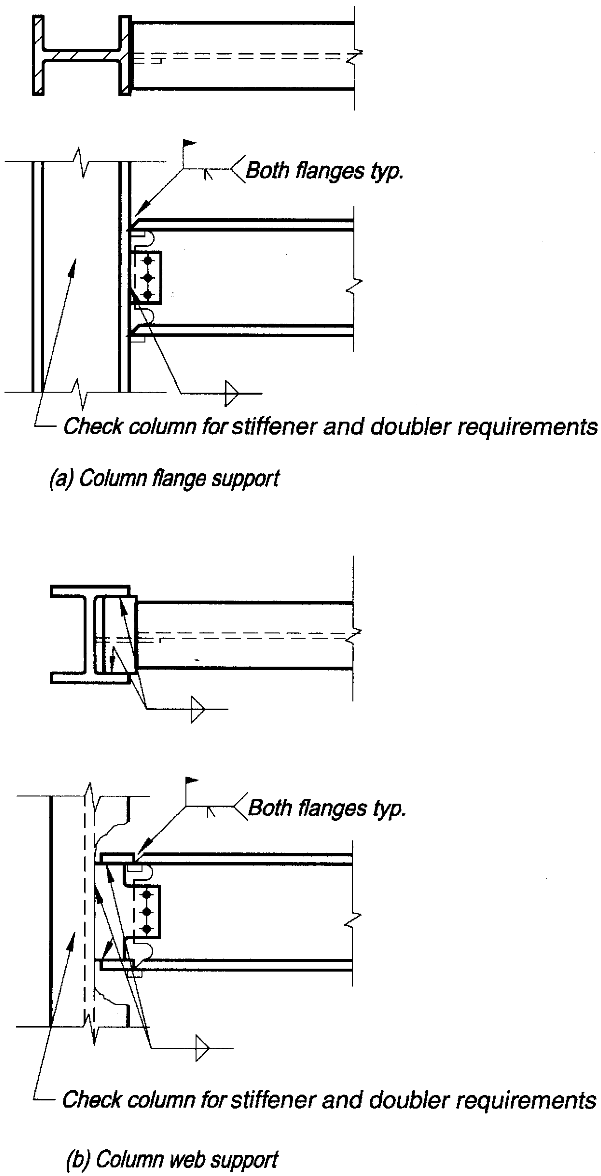


Figure 12–4. Directly welded flange FR moment connections.

Extended End-Plate FR Moment Connections

As illustrated in Figure 12–5, an extended end-plate moment connection consists of a plate of length greater than the beam depth, perpendicular to the longitudinal axis of the supported beam. The end-plate is always welded to the web and flanges of the supported beam and bolted to the supporting member. The principal advantage of extended end-plate moment connections is that all welding is done in the shop. Thus, the erection process is relatively fast and economical.

Figure 12–6 illustrates three commonly used extended end-plate connections. The connections are classified by the number of bolts at the tension flange and by the presence of end-plate to beam flange stiffeners. The four-bolt unstiffened and stiffened extended end-plate connections of Figure 12–6a and 12–6b are generally limited by bolt strength. The connection is compatible for use with nearly one-half of the available beam sections.

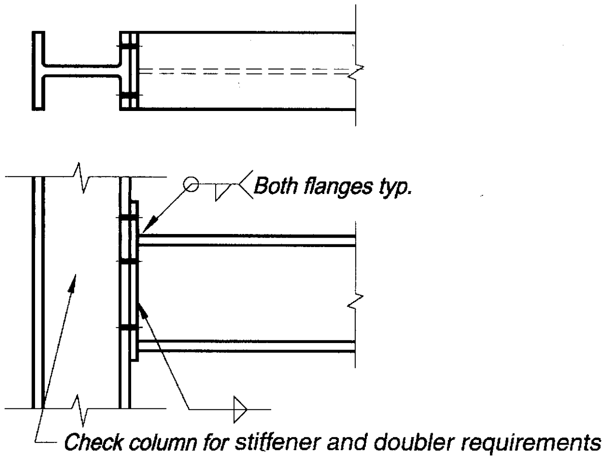


Figure 12–5. Extended end-plate FR moment connection.

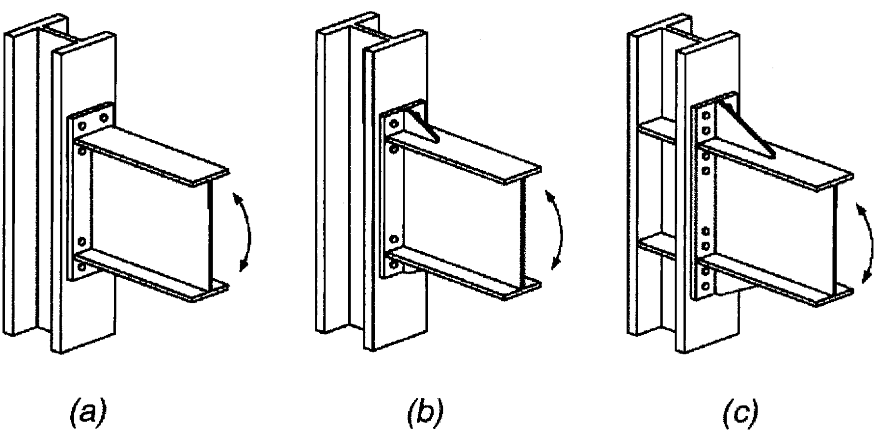


Figure 12–6. Configurations of extended end-plate FR moment connections.

Alternatively, the eight-bolt stiffened extended end-plate connection shown in Figure 12–11c is generally compatible with approximately 90 percent of the available beam sections.

Complete discussion of the design procedures, along with design examples, are found in AISC Design Guide Series No. 4, 2nd Ed., *Extended End-Plate Moment Connections – Seismic and Wind Applications* (Murray and Sumner, 2003). Design procedures and example calculations for nine other end-plate connections, which are commonly used in the metal building industry, are found in AISC Design Guide Series No. 16, *Flush and Extended Multiple Row Moment End-Plate Connections* (Murray and Shoemaker, 2002). Recommended shop and field erection practices, basic design assumptions, and a brief overview of the design procedures follow.

Shop and Field Practices

End-plate moment connections require extra care in shop fabrication and field erection. The fit-up of extended end-plate connections is sensitive to the column flange conditions and may be affected by column flange-to-web squareness, beam camber, or squareness of the beam end. The beam is frequently fabricated short to accommodate the column overrun tolerances with shims furnished to fill any gaps which might result.

As reported by Murray and Meng (1996), use of weld access holes can result in beam flange cracking. If CJP welds are used, the weld cannot be inspected over the web; however, because this location is a relative "soft" spot in the connection, it is of no concern.

Design Assumptions

A summary of the assumptions made in the design guide procedures follows:

1. ASTM A325 or A490 high-strength bolts of diameter not greater than $1^1/_2$ in. must be used.
2. The minimum specified yield stress of the end-plate material must be less than 50 ksi.
3. When the procedures in AISC Design Guide 16 are used, only static loading is permitted (wind, seismic with seismic response modification factor, R, taken equal to or less than 3, snow, and temperature loads are considered static loads).
4. The recommended minimum distance from the face of the beam flange to the nearest bolt centerline (the vertical bolt pitch) is the bolt diameter d_b plus $^1/_2$ in. if the bolt diameter is not greater than 1 in., and $^3/_4$ in. for larger diameter bolts. However, many fabricators prefer to us a standard pitch dimension of 2 in. or $2^1/_2$ in. for all bolt diameters.
5. All of the shear force at a connection is assumed to be resisted by the compression side bolts. End-plate connections need not be designed as slip-critical connections and it is noted that shear is rarely a major concern in the design of moment end-plate connections.
6. The end-plate width effective in resisting the applied moment must be taken as not greater than the beam flange width b_f plus 1 in.
7. The gage of the tension bolts (horizontal distance between vertical bolt lines) must not exceed the beam tension flange width.
8. When CJP welds are used, weld access holes should not be used, and the weld between beam flange-to-web fillets should treated as a PJP weld.
9. For non-seismic connections, when the required resisting moment is less than the

available flexural strength of the beam, the end-plate connection can be designed for required moment but not less than 60 percent of the beam strength.

10. Beam web-to-end-plate welds in the vicinity of the tension bolts should be designed to develop the yield stress of the beam web unless the required moment is less than 60 percent of the beam flexural strength.
11. Only the web-to-end-plate weld between the mid-depth of the beam and the inside face of the beam compression flange or the weld between the inner row of tension bolts plus $2d_b$ and the inside face of the beam compression flange, whichever is smaller, is considered effective in resisting the beam end shear.

Design Procedures

The design procedure in AISC Design Guide 4, 2nd Ed., and AISC Design Guide 16 differ from those in previous AISC design methods. The new procedures are based on yield-line analysis for determining end-plate thickness and modified tee-hanger analysis to determine required bolt strength. The procedures in AISC Design Guide 4, 2nd Ed., are for pretensioned bolts and "thick plates", and result in connections with the smallest possible bolt diameter. For these connections, prying forces are zero. The procedures in AISC Design Guide 16 allow for both "thick plate" and "thin plate" designs. A thin plate design results in the smallest possible end-plate thickness and the maximum bolt prying force. In addition, connections can be designed using either pretensioned or snug-tight bolts.

Column side design procedures are included in AISC Design Guide 4, 2nd Ed. Both Design Guides have complete examples for the various end-plate configurations.

FR MOMENT SPLICES

Beams and girders sometimes are spliced in locations where both shear and moment must be transferred across the splice. Per AISC Specification Section J6, the nominal strength of the smaller section being spliced must be developed in groove-welded butt splices. Other types of beam or girder splices must develop the strength required by the actual forces at the point of the splice.

Location of Moment Splices

A careful analysis is particularly important in continuous structures where a splice may be located at or near the point of inflection. Since this inflection point can and does migrate under service loading, actual forces and moments may differ significantly from those assumed. Furthermore, since loading application and frequency can change in the lifetime of the structure, it is prudent for the designer to specify some minimum strength requirement at the splice. Hart and Milek (1965) propose that splices in fixed-ended beams be located at the one-sixth point of the span and be adequate to resist a moment equal to one-sixth of the flexural strength of the member, as a minimum.

Force Transfer in Moment Splices

Force transfer in moment splices can be assumed to occur in a manner similar to that developed for FR moment connections. That is, the shear, R_u or R_a, is primarily transferred through the beam-web connection and the moment can be resolved into an effective tension-compression couple where the required force at each flange, P_{uf} or P_{af}, is determined by:

LRFD	ASD
$P_{uf} = \dfrac{M_u}{d_m}$	$P_{af} = \dfrac{M_a}{d_m}$

where

M_u or M_a = required moment in the beam at the splice, kip-in.
d_m = moment arm, in. (varies based upon actual connection geometry)

Axial forces, if present, are normally assumed to be distributed uniformly across the beam flange cross-sectional area. However, if the beam-web connection has sufficient stiffness, it can also be assumed to participate in the transfer of beam axial force.

Flange-Plated FR Moment Splices

Moment splices can be designed as shown in Figure 12–7, to utilize flange plates and a web connection. The flange plates and web connection may be bolted or welded.

The splice and spliced beams should be checked in a manner similar to that described previously under "Flange-Plated FR Moment Connections," except that the web connection should be designed as illustrated previously for shear splices in Part 10 without consideration of eccentricity.

Figure 12–7 illustrates two types of splices, bolted and welded. Figure 12–7a illustrates the detail of a bolted flange-plated moment splice. For this case, the flange plates are normally made approximately the same width as the beam flange as shown in Figure 12–7a.

Alternatively, Figure 12–7b illustrates the detail of a welded splice. As shown in Figure 12–7b, the top plate is narrower and the bottom plate is wider than the beam flange, permitting the deposition of weld metal in the downhand or horizontal position without inverting the beam. While this is a benefit in shop fabrication (the beam does not have to be turned over), it is of extreme importance in the field where the weld can be made in the

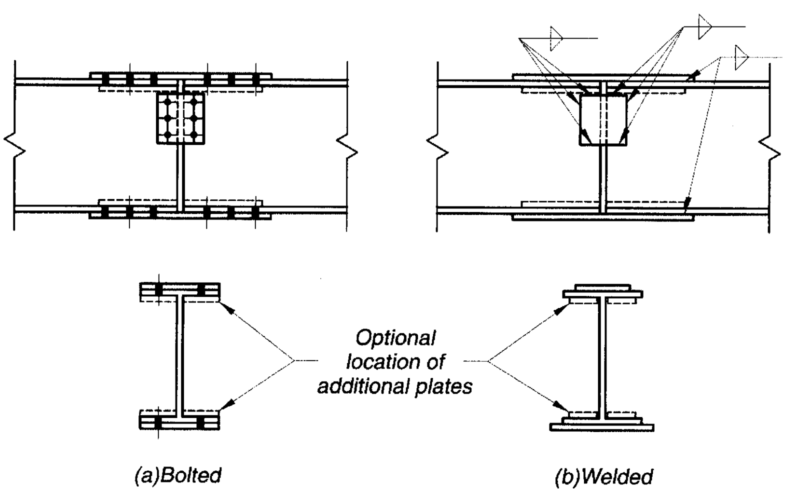


Figure 12–7. Flange-plated moment splice.

horizontal instead of the overhead position, since the beam cannot be turned over. This detail also provides tolerance for field alignment, since the joint gap can be opened or closed. When splices are field-welded, some means for temporary support must be provided as discussed previously in "Temporary Support During Erection".

If the beam or girder flange is thick and the flange forces are large, it may be desirable to place additional plates on the insides of the flanges. In a bolted splice (Figure 12–7a), the bolts are then loaded in double shear and a more compact joint may result. Note that these additional plates must have sufficient area to develop their share of the double-shear bolt load.

In a welded splice (Figure 12–7b), these additional plates must have sufficient area to match the strength of the welds that connect them. Additionally, these plates must be set away from the beam web a distance sufficient to permit deposition of weld metal as shown in Figure 12–8a. This distance is a function of the beam depth and flange width, as well as the welding equipment to be used. A distance of 2 to 2½ in. or more may be required for this access. One alternative is to bevel the bottom edge of the plate to clear the beam fillet and place the plate tight to the beam web with a fillet weld as illustrated in Figure 12–8b. The effects of this bevel on the area of the plate must be considered in determining the required plate width and thickness. Another alternative would be to use unbeveled inclined plates as shown in Figure 12–8b.

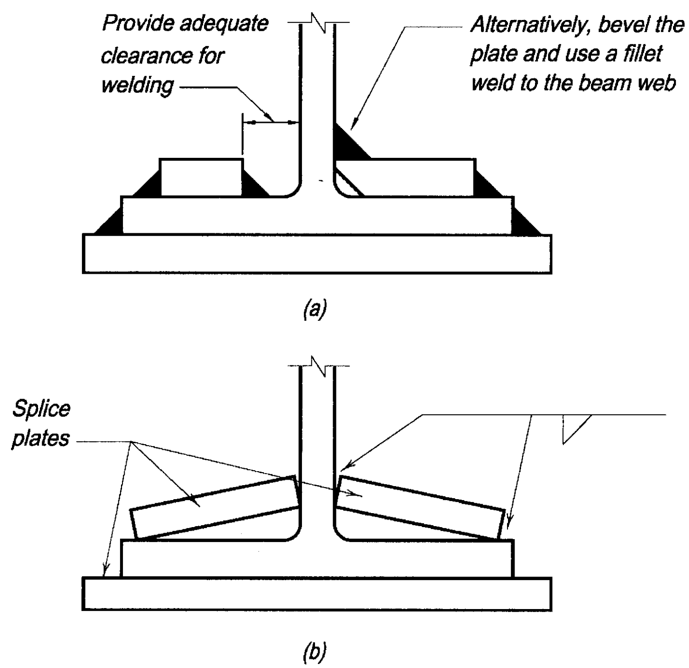


Figure 12–8. Welding clearances for flange-plated moment splices.

Directly Welded Flange FR Moment Splices

Moment splices can be designed, as shown in Figure 12–9, to utilize a complete-joint-penetration groove weld connecting the flanges of the members being spliced. The web connection may then be bolted or welded.

The splice and spliced beams should be checked in a manner similar to that described previously under "Directly Welded Flange FR Moment Connections," except that the web connection should be designed as illustrated previously for shear splices in Part 10.

Although rare in occurrence, some spliced members must be level on top. Where the depths of these spliced members differ, consideration should be given to the use of a flange plate of uniform thickness for the full length of the shallower member. This avoids the fabrication problems created by an inverted transition.

In Figure 12–10, the spliced shapes are different sizes, but of the same shape depth grouping. Because rolled shapes from the same shape depth grouping have the same dimension between the flanges, aligning the inside flange surfaces avoids a more difficult offset transition. Eccentricity resulting from differing flange thicknesses is usually ignored in the design. The web plates normally are aligned to their center lines and the 1 in $2^1/2$ slope is chamfered into the flange or the weld is sloped, depending upon the relative thicknesses.

The groove- (butt-) welded splice preparation shown in Figure 12–9 may be used for either shop or field welding. Alternatively, for shop welding where the beam may be turned over, the joint preparation of the bottom flange could be inverted.

In splices subjected to dynamic or fatigue loading, the backing bar should be removed and the weld should be ground flush when it is normal to the applied stress (AISC, 1977). The access holes should be free of notches and should provide a smooth transition at the juncture of the web and flange.

Extended End-Plate FR Moment Splices

Moment splices can be designed as shown in Figure 12–11 where the tension force is in the bottom flange, to utilize four-bolt unstiffened extended end-plates connecting the members being spliced. If the end-plate and the bolts are designed properly, it is possible to load this type of connection to reach the full plastic moment capacity of the beam, $\phi_b M_p$ or M_p/Ω.

The splice and spliced beams should be checked in a manner similar to that described previously under "Extended End-Plate FR Moment Connections."

The comments for "Extended End-Plate Connections" are equally applicable to extended end-plate moment splices.

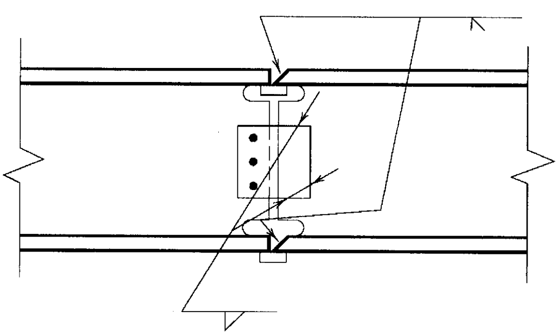


Figure 12–9. Directly welded flange moment splice.

SPECIAL CONSIDERATIONS

FR Moment Connections to Column Webs

It is frequently required that FR moment connections be made to column webs. While the mechanics of analysis and design do not differ from FR moment connection to column flanges, the details of the connection design as well as the ductility considerations required are significantly different.

Recommended Details

When an FR moment connection is made to a column web, it is normal practice to stop the beam short and locate all bolts outside of the column flanges as illustrated in Figure 12–2. This simplifies the erection of the beam and permits the use of an impact wrench to tighten all bolts. It is also preferable to locate welds outside the column flanges to provide adequate clearance.

Ductility Considerations

Driscoll and Beedle (1982) discuss the testing and failure of two FR moment connections to column webs: a directly welded flange connection and a bolted flange-plated connection, shown respectively in Figures 12–12a and 12–12b. Although the connections in these tests were proportioned to be "critical," they were expected to provide inelastic rotations at full plastic load. Failure occurred unexpectedly, however, on the first cycle of loading; brittle fracture occurred in the tension connection plate at the load corresponding to the plastic moment before significant inelastic rotation had occurred.

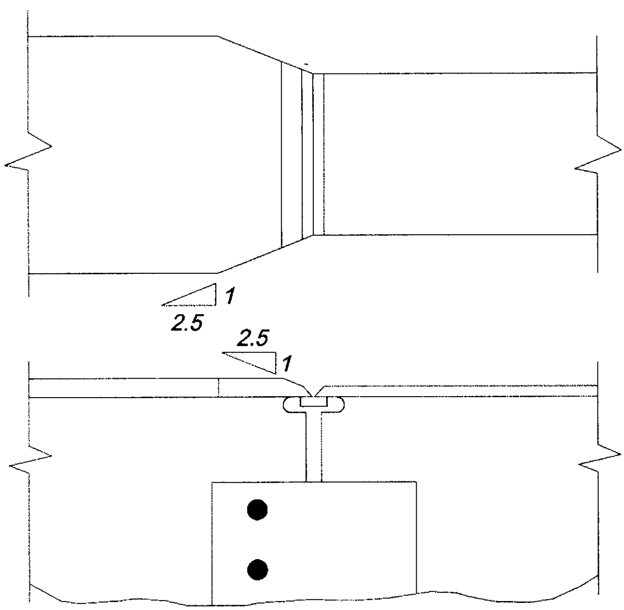


Figure 12–10. Transitions at tension flange for directly welded flange moment splices, when required.

Examination and testing after the unexpected failure revealed that the welds were of proper size and quality and that the plate had normal strength and ductility. The following is quoted, with minor editorial changes relative to figure numbers, directly from Driscoll and Beedle (1982).

"Calculations indicate that the failures occurred due to high strain concentrations. These concentrations are: (1) at the junction of the connection plate and the column flange tip and (2) at the edge of the butt weld joining the beam flange and the connection plate."

"Figure 12–13 illustrates the distribution of longitudinal stress across the width of the connection plate and the concentration of stress in the plate at the column flange tips. It also illustrates the uniform longitudinal stress distribution in the connection plate at

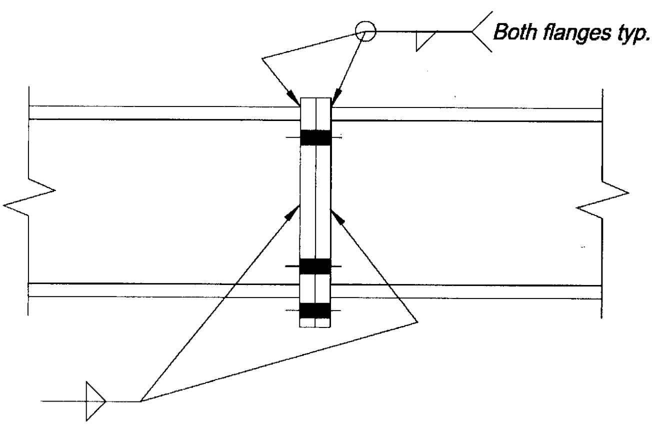


Figure 12–11. Extended end-plate moment splice.

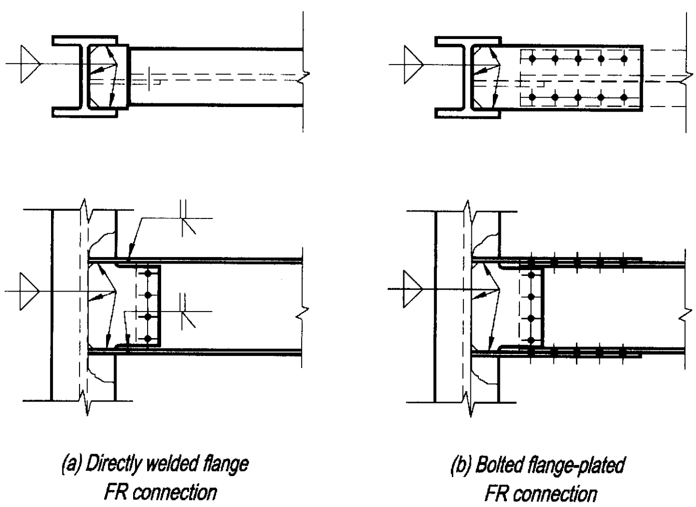


Figure 12–12. Test specimens used by Driscoll and Beedle (1982).

some distance away from the connection. The stress distribution shown represents schematically the values measured during the load tests and those obtained from finite element analysis. (σ_o is a nominal stress in the elastic range.) The results of the analyses are valid up to the loading that causes the combined stress to equal the yield point. Furthermore, the analyses indicate that localized yielding could begin when the applied uniform stress is less than one-third of the yield point. Another contribution of the non-uniformity is the fact that there is no back-up stiffener. This means that the welds to the web near its center are not fully effective."

"The longitudinal stresses in the moment connection plate introduce strains in the transverse and the through-thickness directions (the Poisson effect). Because of the attachment of the connection plate to the column flanges, restraint is introduced; this causes tensile stresses in the transverse and the through-thickness directions. Thus, referring to Figure 12–13, tri-axial tensile stresses are present along Section A-A and they are at their maximum values at the intersections of Sections A-A and C-C. In such a situation, and when the magnitudes of the stresses are sufficiently high, materials that are otherwise ductile may fail by premature brittle fracture."

The results of nine simulated weak-axis FR moment connection tests performed by Driscoll, et. al. (1983) are summarized in Figure 12–14. In these tests, the beam flange was simulated by a plate measuring either 1 in.×10 in. or 1$^1/_8$ in.×9 in. The fracture strength exceeds the yield strength in every case, and sufficient ductility is provided in all cases except for that of Specimen D. Also, if the rolling direction in the first five specimens (A, B, C, D, and E) were parallel to the loading direction, which would more closely approximate an actual beam flange, the ductility ratios for these would be higher. The

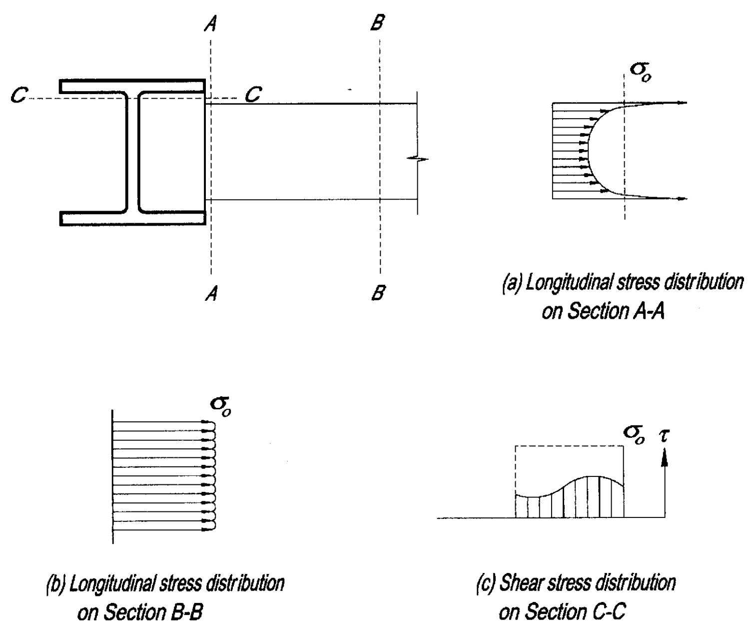


Figure 12–13. Stress distributions in test specimens used by Driscoll and Beedle (1982).

connections with extended connection plates (i.e., projection of 3 in.), with extensions either rectangular or tapered, appeared equally suitable for the static loads of the tests.

Based on the tests, Driscoll, et. al. (1983) report that those specimens with extended connection plates have better toughness and ductility and are preferred in design for seismic loads, even though the other connection types (except D) may be deemed adequate to meet the requirements of many design situations.

In accordance with the preceding discussion, the following suggestions are made regarding the design of this type of connection:

1. For directly welded (butt) flange-to-plate connections, the connection plate should be thicker than the beam flange. This greater area accounts for shear lag and also provides for misalignment tolerances.

   AWS D1.1, Section 3.3.3 restricts the misalignment of abutting parts such as this to 10 percent of the thickness, with $^1/_8$-in. maximum for a part restrained against bending due to eccentricity of alignment. Considering the various tolerances in mill rolling (± $^1/_8$ in. for W-shapes), fabrication, and erection, it is prudent design to call for the connection plate thickness to be increased to accommodate these tolerances and avoid the subsequent problems encountered at erection. An increase of $^1/_8$ in. to $^1/_4$ in. generally is used.

   Frequently, this connection plate also serves as the stiffener for a strong axis FR or PR moment connection. The welds that attach the plate/stiffener to the column flange may then be subjected to combined tensile and shearing or compression and shearing forces. Vector analysis is commonly used to determine weld size and stress.

   It is good practice to use fillet welds whenever possible. Welds should not be made in the column *k*-area for strength.
2. The connection plate should extend at least $^3/_4$ in. beyond the column flange to avoid intersecting welds and to provide for strain elongation of the plate. The extension should also provide adequate room for runout bars when required.
3. Tapering an extended connection plate is only necessary when the connection plate is not welded to the column web (Specimen E, Figure 12–14). Tapering is not necessary if the flange force is always compressive (e.g., at the bottom flange of a cantilevered beam).
4. To provide for increased ductility under seismic loading, a tapered connection plate should extend 3 in. Alternatively, a backup stiffener and an untapered connection plate with 3-in. extension could be used.

Normal and acceptable quality of workmanship for connections involving gravity and wind loading in building construction would tolerate the following:

1. Runoff bars and backing bars may be left for beam with flange thicknesses greater than 2 in. (subject to tensile stress only) where they are welded to columns or used as tension members in a truss.
2. Welds need not be ground, except as required for nondestructive testing.
3. Connection plates that are made thicker or wider for control of tolerances, tensile stress, and shear lag need not be ground or cut to a transition thickness or width to match the beam flange to which they connect.
4. Connection plate edges may be sheared, or plasma- or gas-cut.

5. Intersections and transitions may be made without fillets or radii.
6. Flame-cut edges may have reasonable roughness and notches within AWS tolerances.

If a structure is subjected to loads other than gravity and wind loads, such as seismic, dynamic, or fatigue loading, more stringent control of the quality of fabrication and erection with regard to stress risers, notches, transition geometry, welding, and testing may be necessary; refer to the AISC Seismic Provisions.

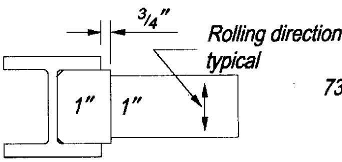
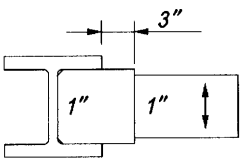
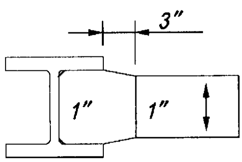
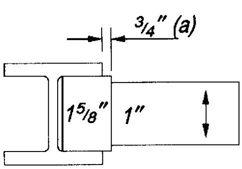
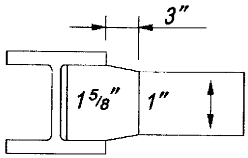
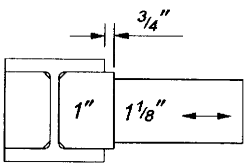
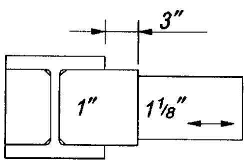
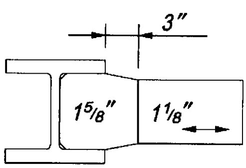
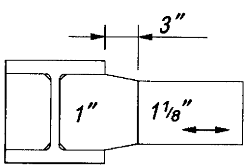
Specimen No.	*Sketch W14x257 (typical)*	*Fracture Load (kips)*	*Fracture Load / Yield Load*	*Ductility Ratio*
A		*730*	*1.38*	*6.3*
B		*824*	*1.55*	*5.3*
C		*756*	*1.43*	*5.43*
D		*570*	*1.11*	*1.71*

Figure 12–14. Results of weak-axis FR moment connection ductility tests performed by Driscoll, et al. (1983).

Specimen No.	*Sketch W14x257 (typical)*	*Fracture Load (kips)*	*Fracture Load / Yield Load*	*Ductility Ratio*
E		*802*	*1.51*	*6.81*
A2		*762*	*1.40*	*17.7*
B2		*795*	*1.46*	*16.5*
E2		*814*	*1.49*	*16.4*[(b)]
C2		*813*	*1.49*	*29.6*

Notes: (a) 3/4" dimension is estimated—no dimension given.

(b) Ductility ratio estimated. Actual value not known due to malfunction in deflection gage.

Figure 12–14. (continued)

FR Moment Connections Across Girder Supports

Frequently, beam-to-girder-web connections must be made continuous across a girder-web support, as with continuous beams and with cantilevered beams at wall, roof-canopy, or building lines. While the same principles of force transfer discussed previously for FR moment connections may be applied, the designer must carefully investigate the relative stiffness of the assembled members being subjected to moment or torsion and provide the fabricator and erector with reliable camber ordinates.

Additionally, the design should still provide some means for final field adjustment to accommodate the accumulated tolerances of mill production, fabrication, and erection; it is very desirable that the details of field connections provide for some adjustment during erection. Figure 12–15 illustrates several details that have been used in this type of connection and the designer may select the desirable components of one or more of the sketches to suit a particular application. Therefore, these components are discussed here as a top flange, bottom flange, and web connection.

Top Flange Connection

As shown in Figure 12–15a, the top flange connection may be directly welded to the top flange of the supporting girder. Figures 12–15b and 12–15c illustrate an independent splice plate that ties the two beams together by use of a longitudinal fillet weld or bolts. This tie plate does not require attachment to the girder flange, although it is sometimes so connected to control noise if the connection is subjected to vibration.

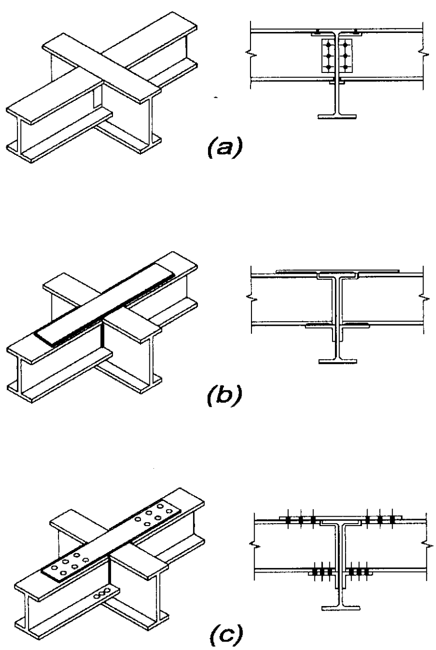


Figure 12–15. FR moment connections across girder-web supports.

Bottom Flange Connection

When the bottom flanges deliver a compressive force only, the flange forces are frequently developed by directly welding these flanges to the girder web as illustrated in Figure 12–15a. Figure 12–15b illustrates the use of an angle or channel below the beam flange to provide for a horizontal fillet weld. The angle or channel should be wider than the beam flange to allow for downhand welding. Figure 12–15c is similar, but uses bolts instead of welds to develop the flange force.

Web Connection

While a single-plate connection is shown in Figure 12–15a and unstiffened seated connections are shown in Figures 12–15b and 12–15c, any of the shear connections in Part 10 may be used. Note that the effect of eccentricity in the shear connection may be neglected.

FR CONNECTIONS WITH HSS

HSS Through-Plate Flange-Plated FR Moment Connections

If the required moment transfer to the column is larger than can be provided by the bolted base plate or cap plate, or if the HSS width is larger than that of the wide flange beam, a through-plate moment connection can be used as illustrated in Figure 12–16. It should be noted that through-plate connections are more difficult to erect than the continuous beam connected framing.

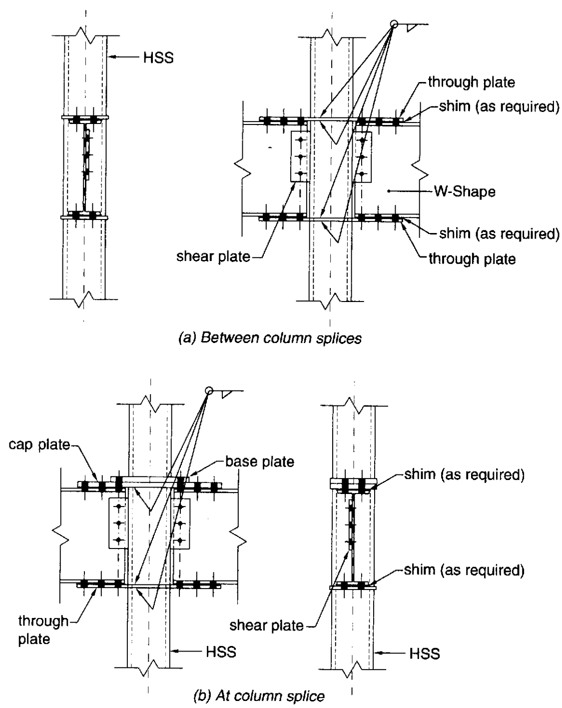


Figure 12–16. Through-plate moment connection.

When moment connections are made using through-plates such as is shown in Figure 12–16, the fabricator must allow adequate clearance between the through-plates and the structural section W-shape so as to allow for the combined effects of mill, fabrication, and erection tolerances. The permissible mill tolerances for W-shape variations in depth and squareness are shown in Table 1–22. Shimming in the field during erection with conventional shims or finger shims is the most commonly used method to fill the gap between the W-shape and the through-plate.

Specific design considerations for through-plate moment connections are as follows:

1. In Figures 12–16a and 12–16b, the column moment transfer into the joint is limited by the fillet weld of the HSS to the through-plates. If necessary, a partial-joint-penetration (PJP) groove weld can be used to improve the connection strength or a complete-joint-penetration (CJP) groove weld with backup bars can be used.
2. In Figure 12–16 an end plate (base plate) is employed to create a splice in the column. Bolt tension with prying on the base plate will determine its thickness and thus limit the moment that can be transferred to the upper HSS.
3. The cap plate, which is also a flange splice plate, should be at least the same thickness as the base plate so that moment transfer between the HSS columns need not rely on load transfer through the beam flanges. The cap plate may need to be thicker than the HSS base plate due to the combined effect of plate bending from the bolted base plate and plate tension or compression from the wide flange moment transfer.
4. The welding of the HSS to the cap and through-plate must be examined for both the HSS normal forces and the shear produced from the moment transfer from the W-shape.

HSS Cut-out Plate Flange-Plated FR Moment Connections

An alternative to interrupting the HSS for the cover or through plate is to use a wider plate with a cut-out to slip around the HSS as illustrated in Figure 12–17. A shear plate can be placed on the front and rear of the HSS faces to provide simple connections for perpendicular beams. The cut-out plate can easily be extended on the near and far sides so that a moment splice is created about both horizontal axes through the joint. The perpendicular framing should ideally be of the same depth for this detail to work well or, in the case of the simple connections, the perpendicular beams could be shallower than the space between the horizontal plates. The cut-out plates are shown as shop-welded; however, they could be field-welded.

For cut-out plate connections, the erection of the beams is more difficult than for continuous beam connections. The beams must be slipped between the two plates and against the single plate connection with shimming being required, unless the upper plate is field-welded in place.

Design Considerations for HSS Directly Welded FR Moment Connections

It may be possible to accomplish the moment transfer to the HSS without having to use a WT splice plate, end-plates, or diaphragm plates. Significant moment transfer can be achieved by attaching the W-shape directly to the face of the HSS either by welding, or by bolting. These connections are capable of developing the available flexural strength of the HSS. The available flexural strength of the W-shape, however, is seldom achieved because of the flexibility of the HSS wall.

The flexural strength for the welded W-shape is based on the strength of the respective flanges in tension and compression acting against the face of the HSS. This flange force can be considered to be the same as that of a plate with the dimensions of the flange.

Several limit states exist for the plate length (flange width) oriented perpendicular to the length of the HSS (Packer and Henderson, 1992).

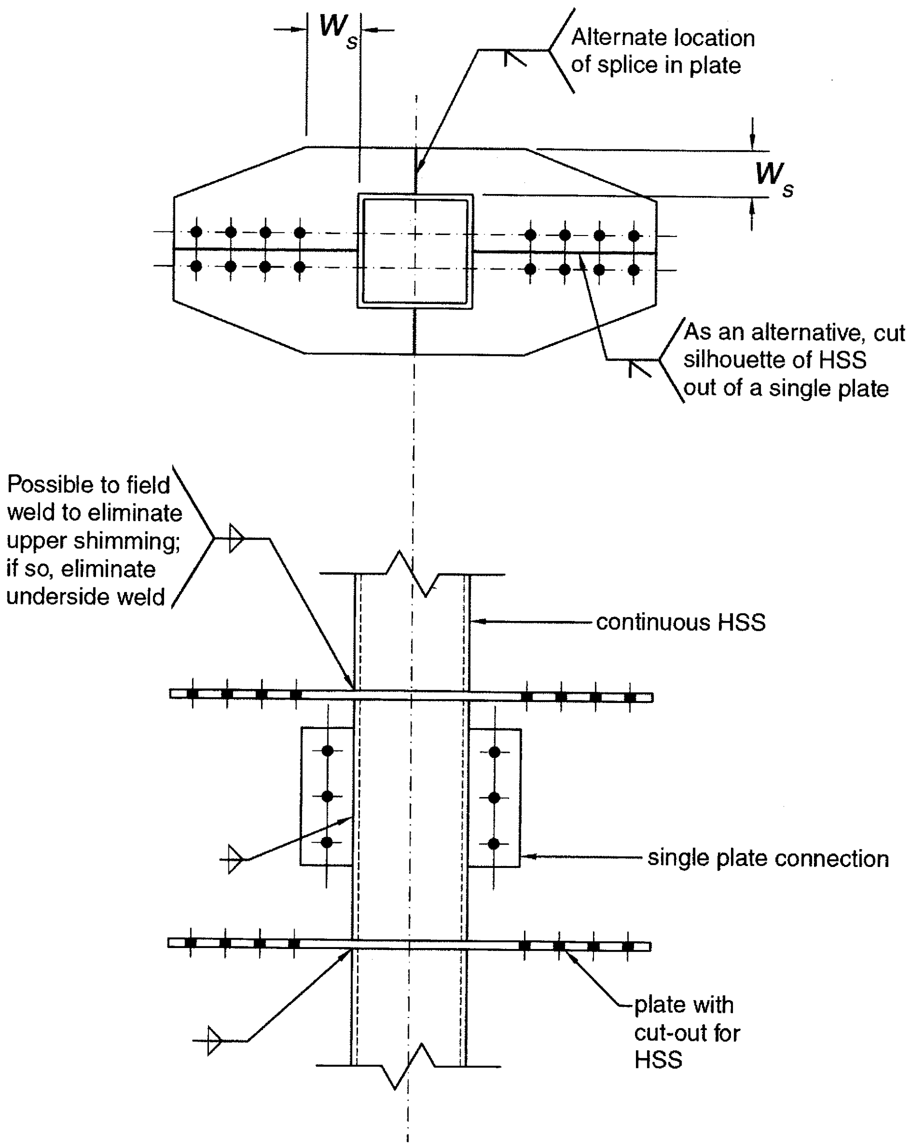


Figure 12–17. Exterior plate moment connection.

Design Considerations for HSS End-Plate FR Moment Connections

HSS end-plates can be bolted to either an end-plate welded to the face of the HSS as in Figure 12–18a or to angles welded to the sides of the HSS as is illustrated in Figure 12–18b. The projection of the end plate beyond the sides of the HSS may interfere with the construction of other building components.

For this connection to be practical, the flange width should be as large as or larger than the HSS width. It is reasonable to consider only two bolts near each flange tip as effective in tension on the beam end plate. For the end plate type connection, the buckling strength of the HSS side wall must be checked.

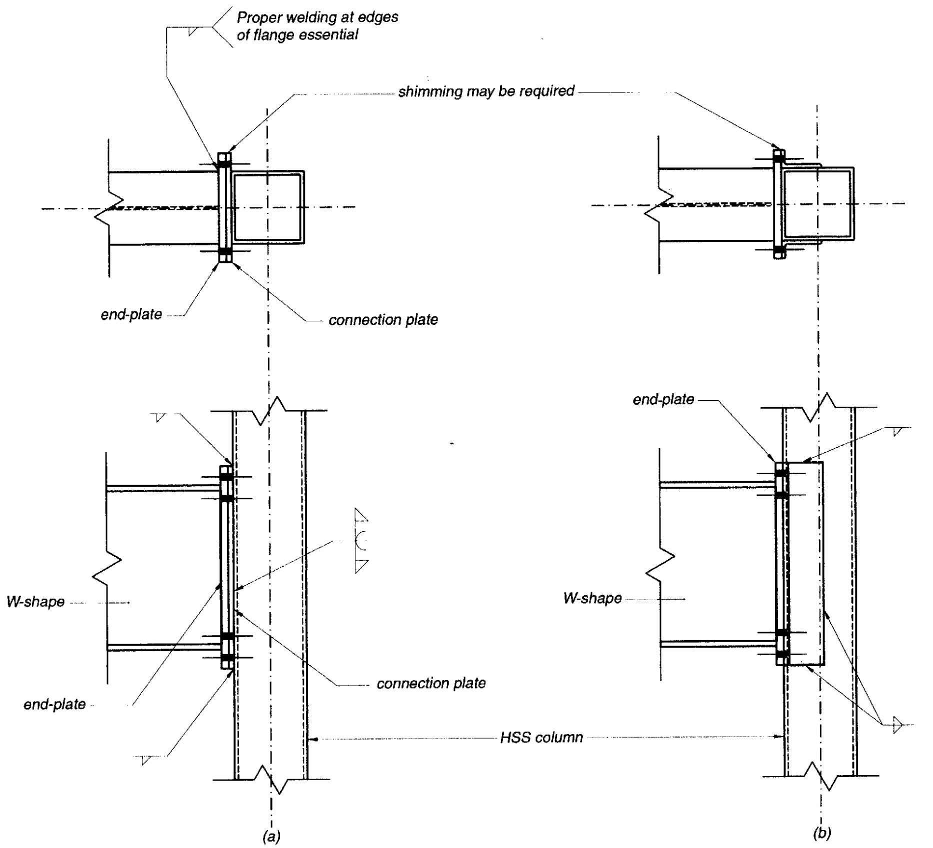


Figure 12–18. End-plate splices to HSS columns.

HSS Columns Above and Below Continuous Beams

Field connection to the flanges of the beam and of continuous beams can be used at joints where there is an HSS above and below a continuous beam. This situation is illustrated in Figures 12–19 and 12–20. If the column load is not high, stiffener plates may be used to transfer the axial load across the beam as shown in Figure 12–19a. If the axial load is higher, it may be necessary to use a split HSS instead of plate stiffeners, as shown in Figure 12–19b. The width of the W-shape must be at least as wide as the HSS and should preferably be wider than the HSS for this detail to be used as shown. It may be necessary to use a rectangular HSS column in order to fit the HSS base plate on the beam flange. The moment transfer to the HSS is limited by the strength of the four bolts, the W-shape flange thickness, and the base and cap plate thicknesses.

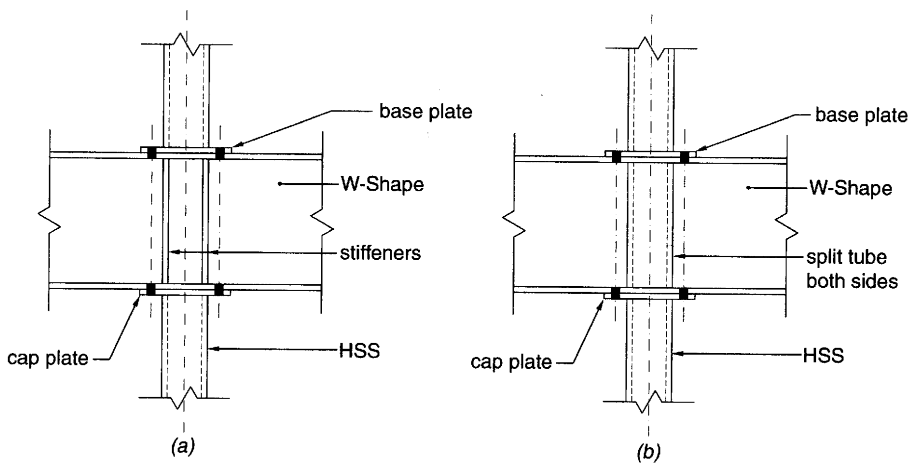


Figure 12–19. HSS columns spliced to continuous beams.

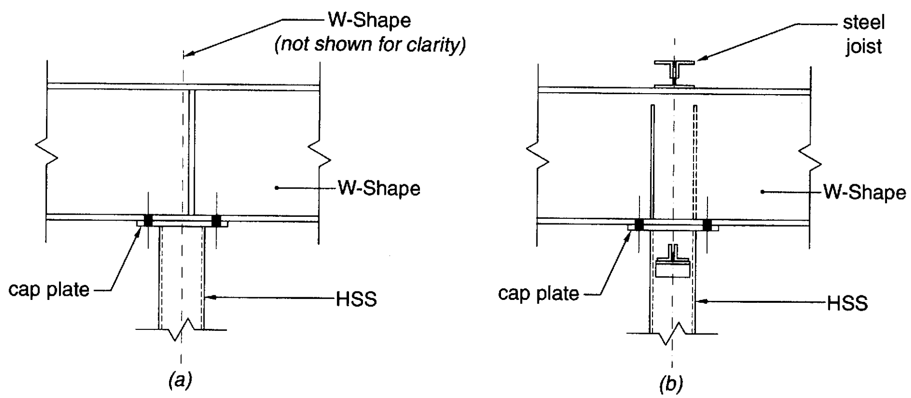


Figure 12–20. Roof beam continuous over HSS column.

HSS Welded Tee Flange Connections

If the primary moment transfer is from a wide flange to an HSS, rather than through the HSS to another wide flange, a number of other connection concepts will work well. One of these is to use structural tee sections to transfer the force from the flanges of the wide flange to the walls of the HSS as is illustrated in Figure 12–21. The tees should be long enough so that a flare bevel-groove (or single J-groove) weld with weld reinforcement can be used to connect the tee to the HSS. An alternative to using the tees to transfer the beam shear would be to use a single plate connection, if a deep enough plate can be fit between the flanges of the tees.

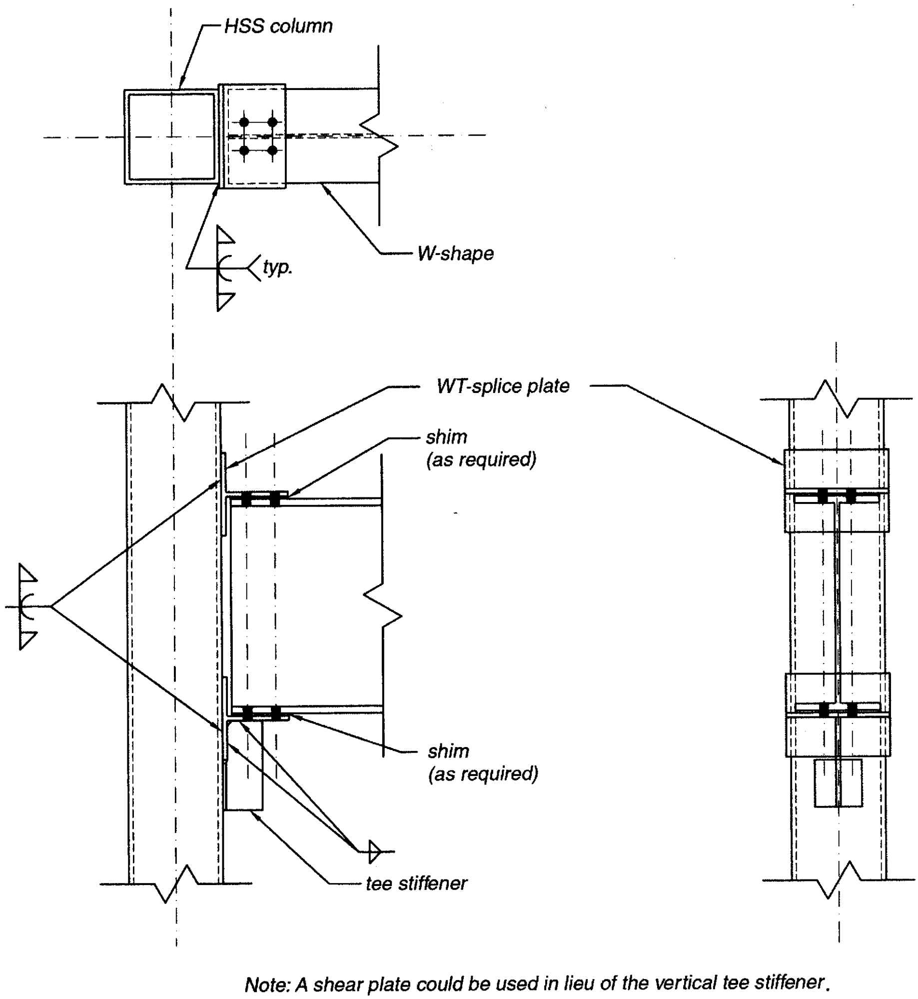
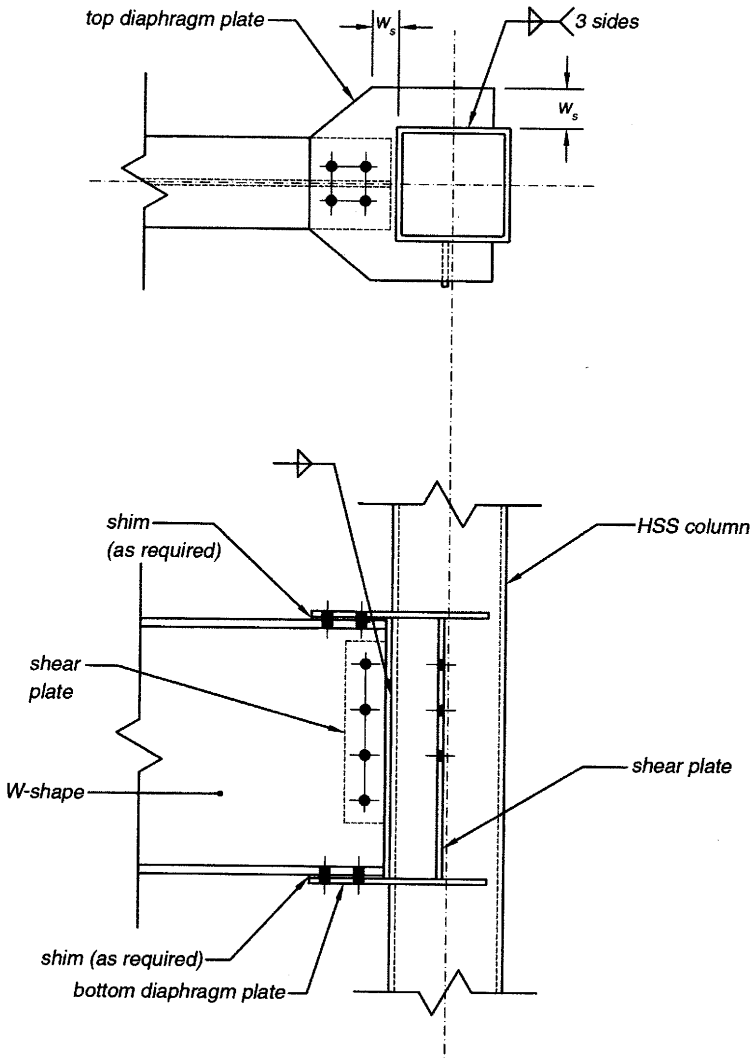


Figure 12–21. Tee splice plates to HSS column.

HSS Diaphragm Plate Connections

If the moment delivered by the W-shape to the HSS cannot be transmitted by other means, then use of diaphragm plates that transfer the flange loads to the sides of the HSS is appropriate. This is illustrated in Figure 12–22. For this moment connection the limit states are those indicated for the cut-out plate connection plus a check of the weld transferring shear from the flange plate to the HSS wall.

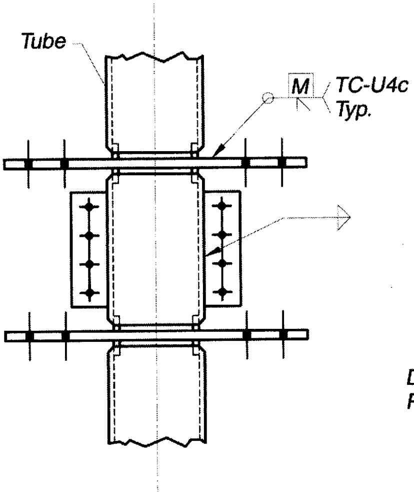


Note: A stiffened seat could also be used in lieu of the shear plate.

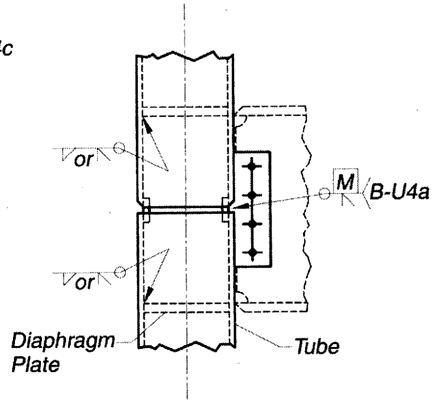
Figure 12–22. Diaphragm plate splice to exterior HSS column.

Suggested Details for HSS to Wide-Flange Moment Connections

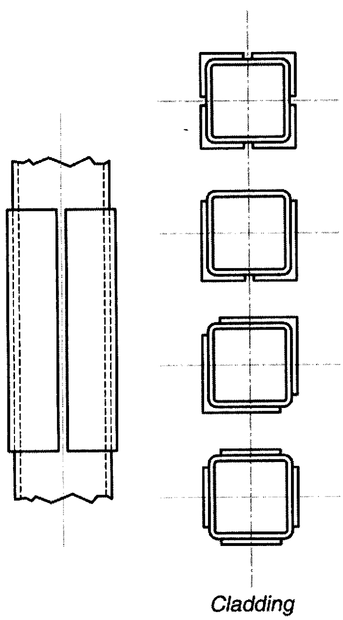
The details shown in Figures 12–23 and 12–24 are suggested details only and are not intended to prohibit the use of other connection details.



Through - Plate Diaphragm



Interior Plate Diaphragm



HSS Column Reinforcement

Figure 12–23. Suggested detail.

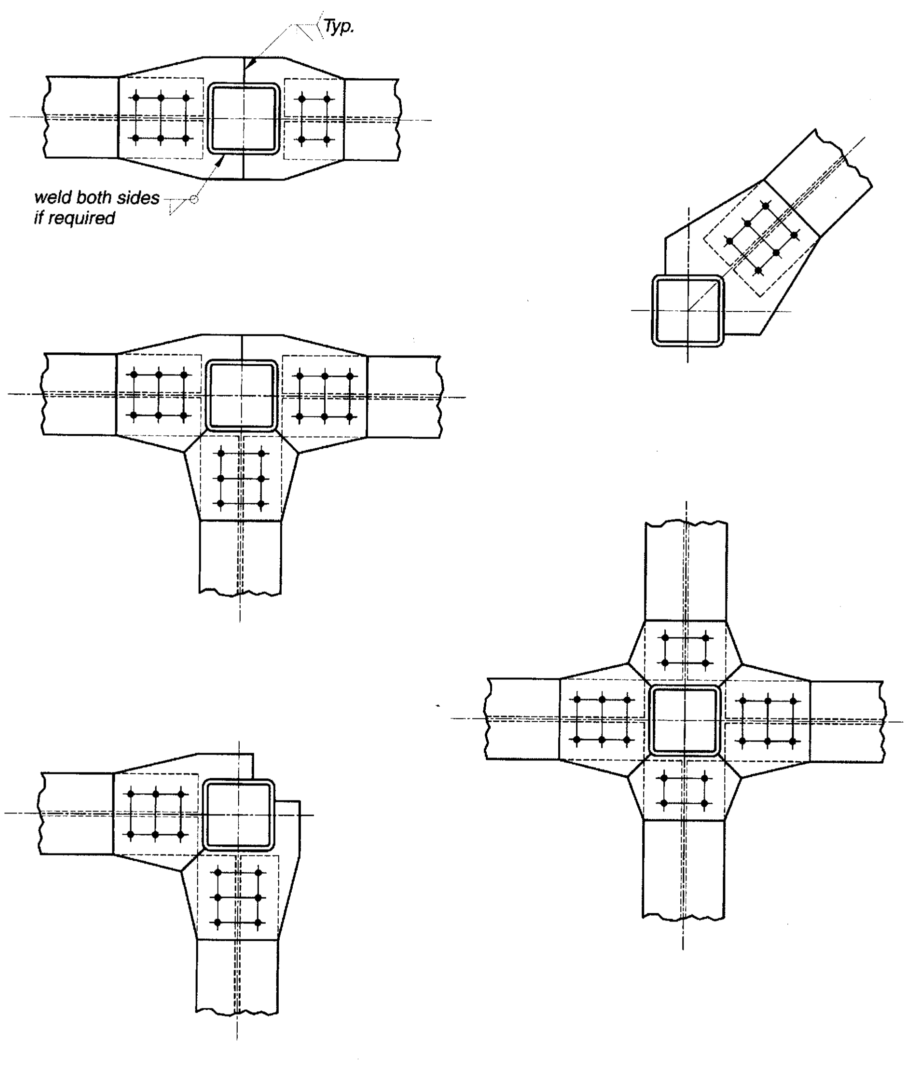


Figure 12–24. Suggested detail.

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PART 13

DESIGN OF BRACING CONNECTIONS AND TRUSS CONNECTIONS

SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of concentric bracing connections and truss connections. For bracing connections and truss connections that are part of a seismic force resisting system in which the seismic response modification factor, R, is taken greater than 3, the requirements in the AISC *Seismic Provisions for Structural Steel Buildings* also apply. The AISC *Seismic Provisions for Structural Steel Buildings* is available in Part 6 of the AISC *Seismic Design Manual* from the American Institute of Steel Construction, Inc. at www.aisc.org.

BRACING CONNECTIONS

Diagonal Bracing Members

Diagonal bracing members can be rods, single angles, channels, double angles, tees, W-shapes, or HSS as required by the loads. Slender diagonal bracing members are relatively flexible and, thus, vibration and sag may be considerations. In slender tension-only bracing composed of light angles, these problems can be minimized with "draw" or pretension created by shortening the fabricated length of the diagonal brace from the theoretical length, L, between member working points. In general, the following deductions will be sufficient to accomplish the required draw: no deduction for $L \leq 10$ ft; deduct $^1/_{16}$ in. for 10 ft $< L \leq 20$ ft; deduct $^1/_8$ in. for $20 < L \leq 35$ ft; and, deduct $^3/_{16}$ in. for $L > 35$ ft. This approach is not applicable to heavier diagonal bracing members, since it is difficult to stretch these members; vibration and sag are not usually design considerations in heavier diagonal bracing members. In any diagonal bracing member, however, it is permissible to deduct an additional $^1/_{32}$ in. when necessary to avoid dimensioning to thirty-seconds of an inch.

When double-angle diagonal bracing members are separated, as at "sandwiched" end connections to gussets, intermittent connections must be provided if the unsupported length of the diagonal brace exceeds the limits specified in AISC Specification Section D4 for tension members or AISC Specification Section E6 for compression members. Note that a minimum of two stitch-fillers are required. These may be either bolted or welded stitch-fillers. Many fabricators prefer ring or rectangular bolted stitch-fillers when the angles require other punching, as at the end connections. In welded construction, a stitch-filler with protruding ends, as shown in Figure 13–1a is preferred because it is easy to fit and weld. The short stitch-filler shown in Figure 13–1b is used if a smooth appearance is desired.

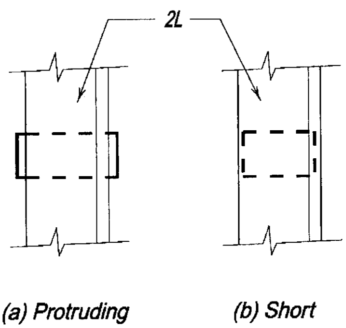


Figure 13–1. Welded stitch fillers.

When a full-length filler is provided, as in corrosive environments, the maximum spacing of stitch bolts should be as specified in AISC 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 available 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.

Required Strength

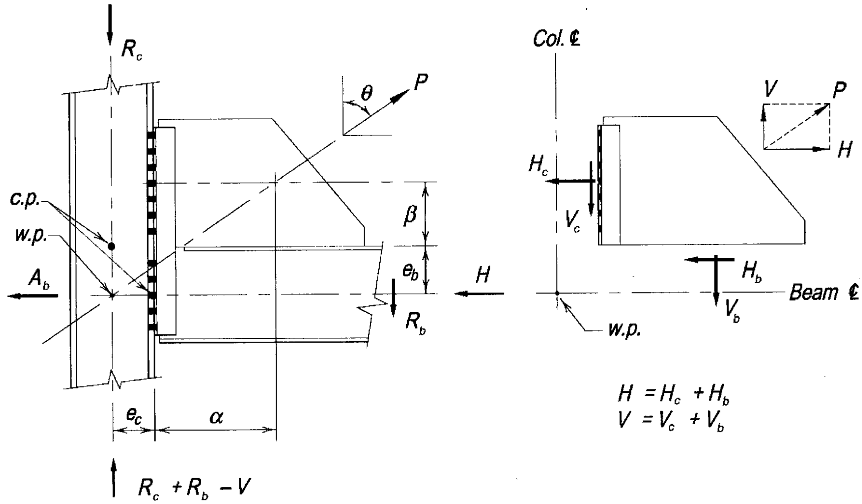
With the control points (c.p.) as illustrated in Figure 13–2 and the working point (w.p.) chosen at the intersection of the centerlines of the beam, column, and diagonal brace as shown in Figure 13–2a, four geometric parameters e_b, e_c, α, and β can be identified, where

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:

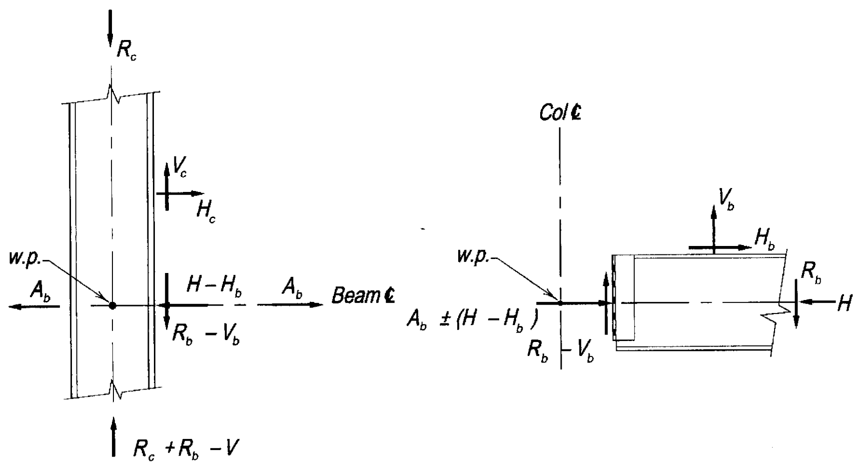
$$\alpha - \beta \tan\theta = e_b \tan\theta - e_c$$

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



(a) Diagonal bracing connection and external forces

(b) Gusset free-body diagram



(c) Column free-body diagram

(d) Beam free-body diagram

R = R_u or R_a, required end reaction of the beam
A_b = A_{ub} or A_{ab}, required transverse force from adjacent bay
H = horizontal component of the required axial force
H_b = H_{ub} or H_{ab}, required shear force on the beam to gusset connection
H_c = H_{uc} or H_{ac}, required axial force on the column to gusset connection
V_b = V_{ub} or V_{ab}, required shear force on the beam to the gusset connection
V_c = V_{uc} or V_{ac}, required shear force on the column to gusset connection
P = P_u or P_a, required axial force
V = vertical component of the required force

Figure 13–2. Force transfer by the uniform force (UF) method, work point (w.p.) and control points (c.p.) as indicated.

and β for which the equation is true, thereby locating the centroids of the gusset-to-beam and gusset-to-column connections.

Once α and β have been determined, the required axial and shear forces for which these connections must be designed can be determined from the following equations:

$$V_c = \frac{\beta}{r}P \qquad H_c = \frac{e_c}{r}P$$

$$H_b = \frac{\alpha}{r}P \qquad V_b = \frac{e_b}{r}P$$

where

$$r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2}$$

The gusset-to-beam connection must be designed for the required shear force, H_b, and the required axial force, V_b, the gusset-to-column connection must be designed for the required shear force, V_c, and the required axial force, H_c, and the beam-to-column connection must be designed for the required shear

$$R - V_b$$

and the required axial force

$$A_b \pm (H - H_b)$$

Note that, while the axial force, P_u or P_a, is shown as a tensile force, it may also be a compressive force; were this the case the signs of the resulting gusset forces would change.

Special Case 1, Modified Working Point Location

As illustrated in Figure 13–3a, the working point in Special Case 1 of the Uniform Force Method is chosen at the corner of the gusset; this may be done to simplify layout or for a column web connection. With this assumption, the terms in the gusset force equations involving e_b and e_c drop out and the interface forces, as shown in Figures 13–3b, 13–3c, and 13–3d, are:

$$H_b = P\sin\theta = H \qquad V_b = 0$$

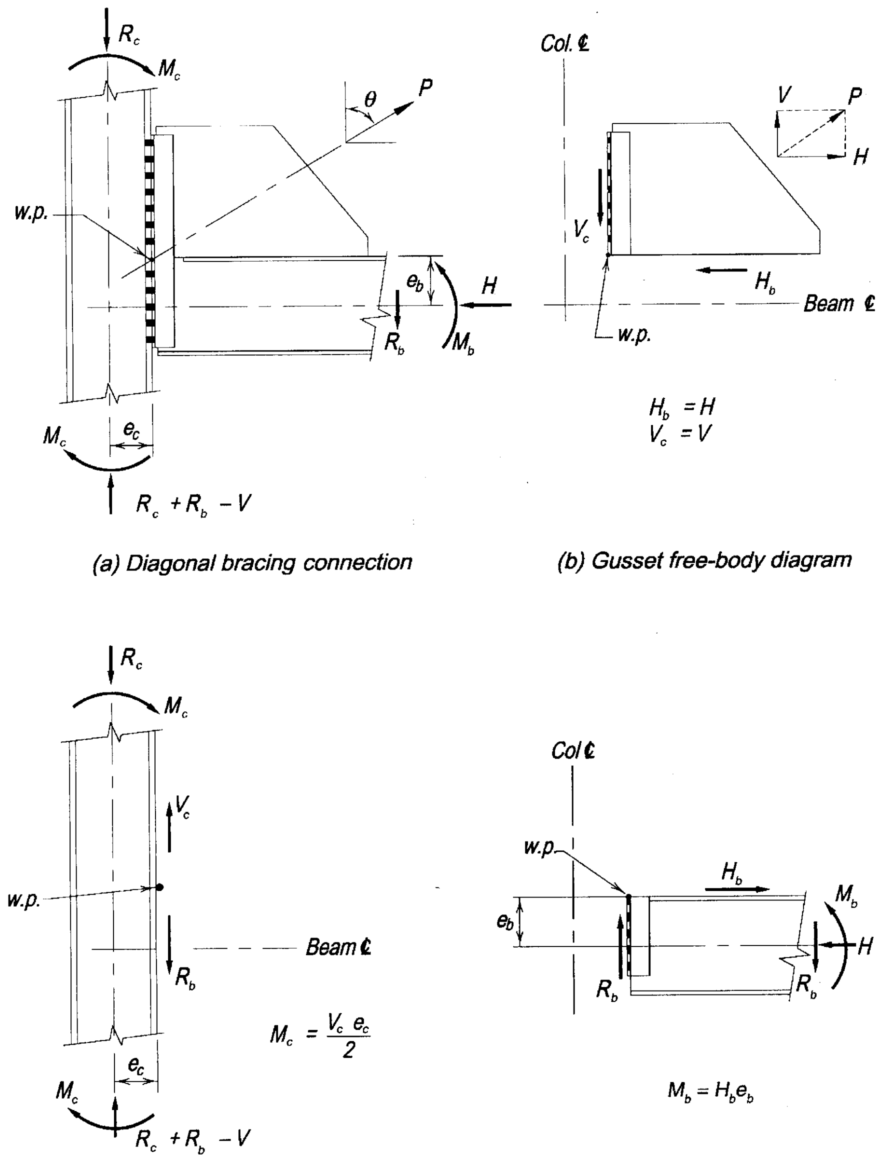
$$V_c = P\cos\theta = V \qquad H_c = 0$$

The gusset-to-beam connection must be designed for the required shear force, H_b, and the gusset-to-column connection must be designed for the required shear force, V_c. Note, however, that the change in working point requires that the beam be designed for the required moment, M_b, where

$$M_b = H_b e_b$$

and the column must be designed for the required moment, M_c. For an intermediate floor, this is determined as:

$$M_c = \frac{V_c e_c}{2}$$



(a) Diagonal bracing connection (b) Gusset free-body diagram

(c) Column free-body diagram (d) Beam free-body diagram

R = R_u or R_a, required end reaction of the beam
A_b = A_{ub} or A_{ab}, required transverse force from adjacent bay
H = horizontal component of the required axial force
H_b = H_{ub} or H_{ab}, required shear force on the beam to gusset connection
H_c = H_{uc} or H_{ac}, required axial force on the column to gusset connection
V_b = V_{ub} or V_{ab}, required shear force on the beam to the gusset connection
V_c = V_{uc} or V_{ac}, required shear force on the column to gusset connection
P = P_u or P_a, required axial force
V = vertical component of the required force

Figure 13–3. Force transfer, UF method special case 1.

An example demonstrating this eccentric special case is presented in AISC (1984). This eccentric case was endorsed by the AISC/ASCE task group (Thornton, 1991) as a reduction of the three recommended methods when the work point is located at the gusset corner. While calculations are somewhat simplified, it should be noted that resolution of the required force P into the shears V_c and H_b may not result in the most economical connection.

Special Case 2, Minimizing Shear in the Beam-to-Column Connection

If the brace force, as illustrated in Figure 13–4a, were compressive instead of tensile and the required beam reaction, R_b, were high, the addition of the extra shear force, V_b, into the beam might exceed the available strength of the beam and require doubler plates or a haunched connection. Alternatively, the vertical force in the gusset-to-beam connection, V_b, can be limited in a manner which is somewhat analogous to using the gusset itself as a haunch.

As illustrated in Figure 13–4b, assume that V_b is reduced by an arbitrary amount, ΔV_b. By statics, the vertical force at the gusset-to-column interface will be increased to $V_c + \Delta V_b$, and a moment M_b will result on the gusset-to-beam connection, where

$$M_b = (\Delta V_b)\alpha$$

If ΔV_b is taken equal to V_b, 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 (Thornton, 1991)

Design by this method may be uneconomical. It is very punishing to the gusset and beam because of the moment M_b 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, in the beam-to-column connection.

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

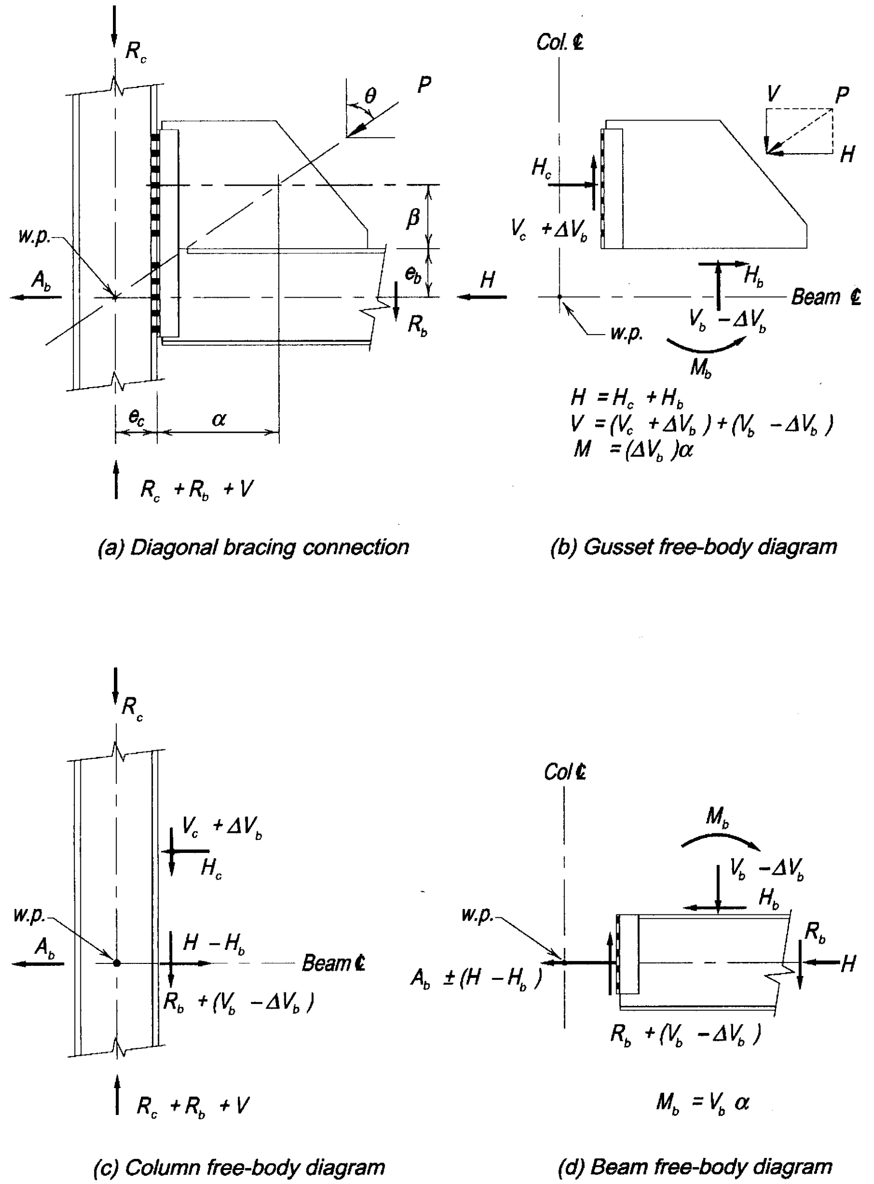
When the connection is to a column web and the brace is shallow (as for large θ) or the beam is deep, it may be more economical to eliminate the gusset-to-column connection entirely and connect the gusset to the beam only. The Uniform Force Method can be applied to this situation by setting β and e_c equal to zero as illustrated in Figure 13–5. Since there is to be no gusset-to-column connection, V_c and H_c also equal zero. Thus, $V_b = V$ and $H_b = H$.

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 required shear force, H_b, and the required axial force, V_b. If $\bar{\alpha} \neq \alpha = e_B \tan\theta$, the gusset-to-beam interface must be designed for the moment, M_b, in addition to H_b and V_b, where

$$M_b = V_b\,(\alpha - \bar{\alpha})$$

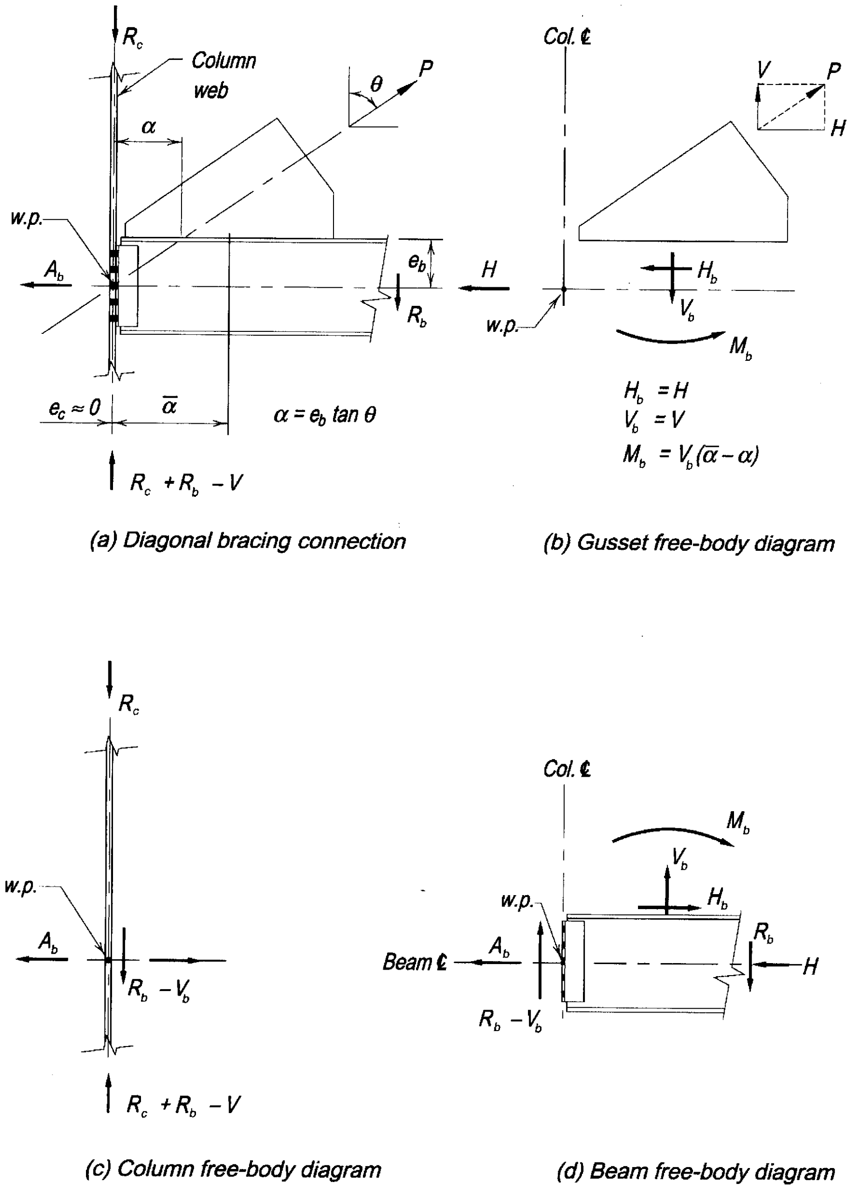
The beam-to-column connection must be designed for the required shear force, $R + V_b$.

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 elimi-



R = R_u or R_a, required end reaction of the beam
A_b = A_{ub} or A_{ab}, required transverse force from adjacent bay
H = horizontal component of the required axial force
H_b = H_{ub} or H_{ab}, required shear force on the beam to gusset connection
H_c = H_{uc} or H_{ac}, required axial force on the column to gusset connection
V_b = V_{ub} or V_{ab}, required shear force on the beam to the gusset connection
V_c = V_{uc} or V_{ac}, required shear force on the column to gusset connection
P = P_u or P_a, required axial force
V = vertical component of the required force

Figure 13–4. Force transfer, UF method special case 2.



R = R_u or R_a, required end reaction of the beam
A_b = A_{ub} or A_{ab}, required transverse force from adjacent bay
H = horizontal component of the required axial force
H_b = H_{ub} or H_{ab}, required shear force on the beam to gusset connection
H_c = H_{uc} or H_{ac}, required axial force on the column to gusset connection
V_b = V_{ub} or V_{ab}, required shear force on the beam to the gusset connection
V_c = V_{uc} or V_{ac}, required shear force on the column to gusset connection
P = P_u or P_a, required axial force
V = vertical component of the required force

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

nated, the beam-to-column connection must be a moment connection designed for the moment, Ve_c, in addition to the shear, V. 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 satisfy the basic relationship

$$\alpha - \beta \tan\theta = e_b \tan\theta - e_c$$

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_b will exist on the gusset-to-beam connection where,

$$M_b = V_b\,(\alpha - \bar{\alpha})$$

Conversely, suppose the gusset-to-column connection were judged to be more rigid. Setting $\alpha = \bar{\alpha}$, the β required for no moment on the gusset-to-column connection may be calculated as

$$\beta = \frac{\bar{\alpha} - K}{\tan\theta}$$

If $\beta \neq \bar{\beta}$, a moment, M_c, will exist on the gusset-to-column connection where,

$$M_c = H_c(\beta - \bar{\beta})$$

If both connections were equally rigid and no obvious allocation of moment could be made, the moment could be distributed based on minimized eccentricities $\alpha - \bar{\alpha}$ and $\beta - \bar{\beta}$ by minimizing the objective function, ξ, where

$$\xi = \left(\frac{\alpha - \bar{\alpha}}{\bar{\alpha}}\right)^2 + \left(\frac{\beta - \bar{\beta}}{\bar{\beta}}\right)^2 - \lambda\left(\alpha - \beta\tan\theta - K\right)$$

In the preceding equation, λ is a Lagrange multiplier.

The values of α and β that minimize ξ are

$$\alpha = \frac{K' \tan\theta + K\left(\dfrac{\bar{\alpha}}{\bar{\beta}}\right)^2}{D}$$

and

$$\beta = \frac{K' - K\tan\theta}{D}$$

where

$$K' = \bar{\alpha}\left(\tan\theta + \frac{\bar{\alpha}}{\bar{\beta}}\right)$$

$$D = \tan^2\theta + \left(\frac{\bar{\alpha}}{\bar{\beta}}\right)^2$$

Available Strength

The available strength of a diagonal bracing connection is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8) and connecting elements (see Part 9). In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a. Note that when the gusset is directly welded to the beam or column, the connection should be designed for the larger of the peak stress and 1.25 times the average stress, but the weld size need not be larger than that required to develop the strength of the gusset. This 25 percent increase is recommended to provide ductility to allow adequate force redistribution in the weld group (Hewitt and Thornton, 2004).

TRUSS CONNECTIONS

Members in Trusses

For light loads, trusses are commonly composed of tees for the top and bottom chords with single-angle or double-angle web members. In welded construction, the single-angle and double-angle web members may, in many cases, be welded to the stem of the tee, thus, eliminating the need for gussets. When single-angle web members are used, all web members should be placed on the same side of the chord; staggering the web members causes a torque on the chord, as illustrated in Figure 13–6.

Double-angle truss members are usually designed to act as a unit. When unequal-leg angles are used, long legs are normally assembled back to back. A simple notation for the angle assembly is LLBB (long legs back-to-back) and SLBB (short legs back-to-back). Alternatively, the notation might be graphical in nature as ┘└ and ─┘└─. For large loads, W-shapes may be used with the web vertical and gussets welded to the flange for the truss connections. Web members may be single angles or double angles, although W-shapes are sometimes used for both chord and web members as shown in Figure 13–7. Heavy shapes in trusses must meet the design and fabrication restrictions and special requirements in AISC Specification Sections A3.1c and A3.1d. With member orientation as shown for the field-welded truss joint in Figure 13–7a, connections usually are made by groove welding flanges to flanges and fillet welding webs directly or indirectly by the use of gussets. Fit-up of joints in this type of construction are very sensitive to dimensional variations in the rolled shapes; fabricators sometimes prefer to use built-up shapes in these cases.

The web connection plate in Figure 13–7a is a typical detail. While the diagonal member could theoretically be cut so that the diagonal web would be extended into the web of the chord for a direct connection, such a detail is difficult to fabricate. Additionally, welding access becomes very limited; note the obvious difficulty of welding the gusset or diagonal directly to the chord web. As illustrated, this weld is usually omitted.

When stiffeners and doubler plates are required for concentrated flange forces, the designer should consider selecting a heavier section to eliminate the need for stiffening. Although this will increase the material cost of the member, the heavier section will likely provide a more economical solution due to the reduction in labor cost associated with the elimination of stiffening (Ricker, 1992 and Thornton, 1992).

Minimum Connection Strength

Truss connections are recommended to be designed for a minimum required strength of 10 kips for LRFD or 6 kips for ASD, as noted in AISC Specification Commentary Section J1.1. Additionally, when trusses are shop assembled or field assembled on the ground for subsequent erection, consideration should be given to loads induced during handling, shipping, and erection.

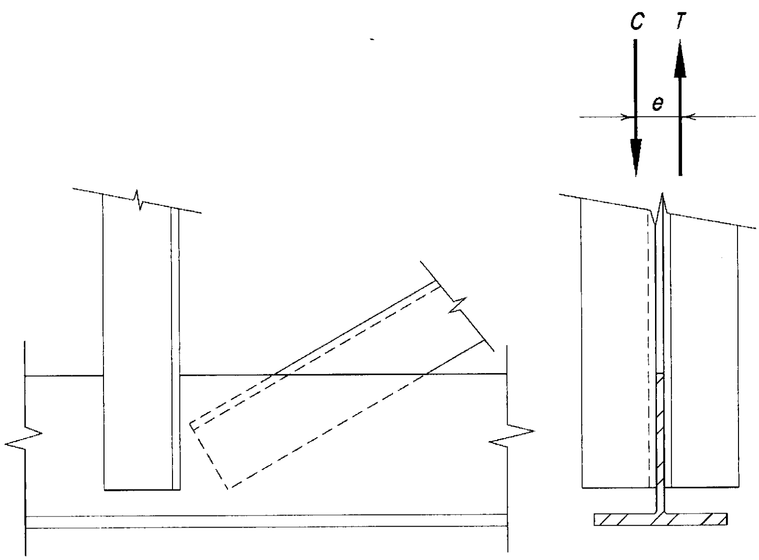


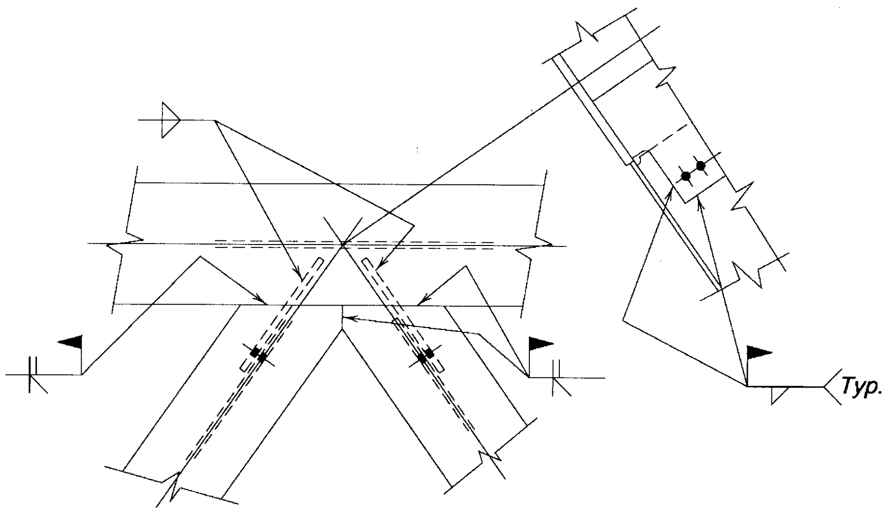
Figure 13–6. Staggered web members result in a torque on the truss chord.

Panel-Point Connections

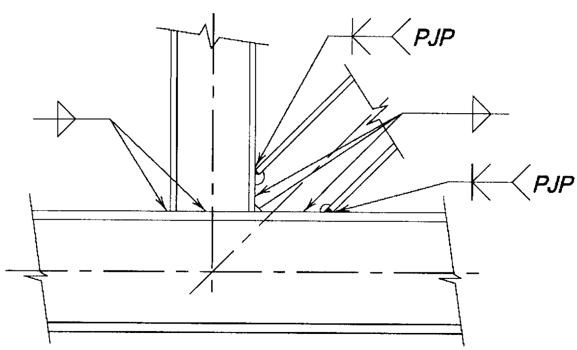
A panel-point connection connects diagonal and/or vertical web members to the chord member of a truss. These web members deliver axial forces, tensile or compressive, to the truss chord. In bolted construction, a gusset is usually required because of bolt spacing and edge distance requirements. In welded construction, it is sometimes possible to eliminate the need for a gusset.

Design Checks

The available strength of a panel-point connection is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8) and connecting elements (see



(a) Shop and field welding



(b) Shop welding

Note: Check vertical and chord for reinforcing requirements

Figure 13–7. Truss panel-point connections for W-shape truss members.

Part 9). In all cases, the available strength, ϕR_n or R_n/Ω, must exceed the required strength, R_u or R_a.

In the panel-point connection of Figure 13–8, the neutral axes of the vertical and diagonal truss members intersect on the neutral axis of the truss chord. As a result, the forces in all members of the truss are axial. It is common practice, however, to modify working lines slightly from the gravity axes to establish repetitive panels and avoid fractional dimensions less than $^1/_8$ in. or to accommodate a larger panel-point connection or a connection for bottom-chord lateral bracing, a purlin, or a sway-frame. This eccentricity and the resulting moment must be considered in the design of the truss chord.

In contrast, for the design of the truss web members, AISC Specification Section J1.7 permits that the center of gravity of the end connection of a statically loaded truss member need not coincide with the gravity axis of the connected member. This is because tests have shown that there is no appreciable difference in the available strength between balanced and unbalanced connections subjected to static loading. Accordingly, the truss web members and their end connections may be designed for the axial load, neglecting the effect of this minor eccentricity.

Shop and Field Practices

In bolted construction, it is convenient to use standard gage lines of the angles as truss working lines; where wider angles with two gage lines are used, the gage line nearest the heel of the angle is the one which is substituted for the gravity axis.

To provide for stiffness in the finished truss, the web members of the truss are extended to near the edge of the fillet of the tee (k-distance). If welded, the required welds are then applied along the heel and toe of each angle, beginning at their ends rather than at the edge of the tee stem.

Support Connections

A truss support connection connects the ends of trusses to supporting members.

Design Checks

The available strength of a support connection is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8) and connecting elements (see Part 9).

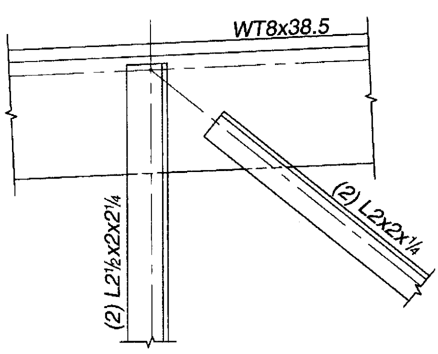


Figure 13–8. Truss panel-point connection.

Additionally, truss support connections produce tensile or compressive single concentrated forces at the beam end; the limit states of the available flange strength in local bending and the limit states of the available web strength in local yielding, crippling, and compression buckling may have to be checked. In all cases, the available strength, ϕR_n or R_n/Ω, must exceed the required strength, R_u or R_a.

At the end of a truss supported by a column, all member axes may not intersect at a common point. When this is the case, an eccentricity results. Typically, it is the neutral axis of the column that does not meet at the working point.

If trusses with similar reactions line up on opposite sides of the column, consideration of eccentricity would not be required since any moment would be transferred through the column and into the other truss. However, if there is little or no load on the opposite side of the column, the resulting eccentricity must be considered.

In Figure 13–9, the truss chord and diagonal intersect at a common working point on the face of the column flange. In this detail, there is no eccentricity in the gusset, gusset-to-column connection, truss chord, or diagonal. However, the column must be designed for the moment due to the eccentricity of the truss reaction from the neutral axis of the column.

For the truss support connection illustrated in Figure 13–10, this eccentricity results in a moment. Assuming the connection between the members is adequate, joint rotation is resisted by the combined flexural strength of the column, the truss top chord, and the truss diagonal. However, the distribution of moment between these members will be proportional to the stiffness of the members. Thus, when the stiffness of the column is much greater than the stiffness of the other elements of the truss support connection, it is good practice to design the column and gusset-to-column connection for the full eccentricity.

Due to its importance, the truss support connection is frequently shown in detail on the design drawing.

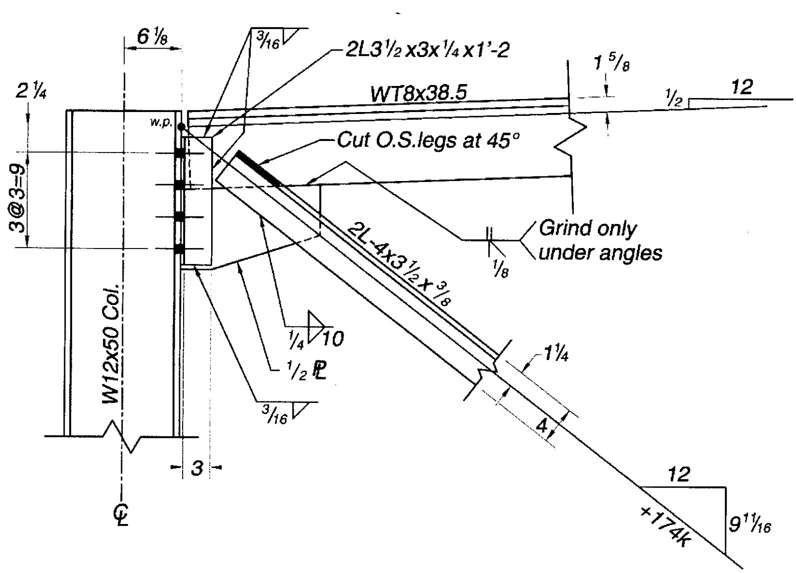


Figure 13–9. Truss support connection, working point (w.p.) on column face.

Shop and Field Practices

When a truss is erected in place and loaded, truss members in tension will lengthen and truss members in compression will shorten. At the support connection, this may cause the tension chord of a "square-ended" truss to encroach on its connection to the supporting column. When the connection is shop-attached to the truss, erection clearance must be provided with shims to fill out whatever space remains after the truss is erected and loaded. In field erected connections, however, provision must be made for the necessary adjustment in the connection.

When the tension chord delivers no calculated force to the connection, adjustment can usually be provided with slotted holes. For short spans with relatively light loads, the comparatively small deflections can be absorbed by the normal hole clearances provided for bolted construction. Slightly greater misalignment can be corrected in the field by reaming the holes. If appreciable deflection is expected, the connection may be welded. Alternatively, bolt holes may be field-drilled, but this is an expensive operation which should be avoided if at all possible.

An approximation of the elongation, Δ, can be determined as

$$\Delta = \frac{Pl}{AE}$$

where

Δ = elongation in inches
P = axial force due to service loads, kips
A = gross area of the truss chord, in.²
l = length, in.

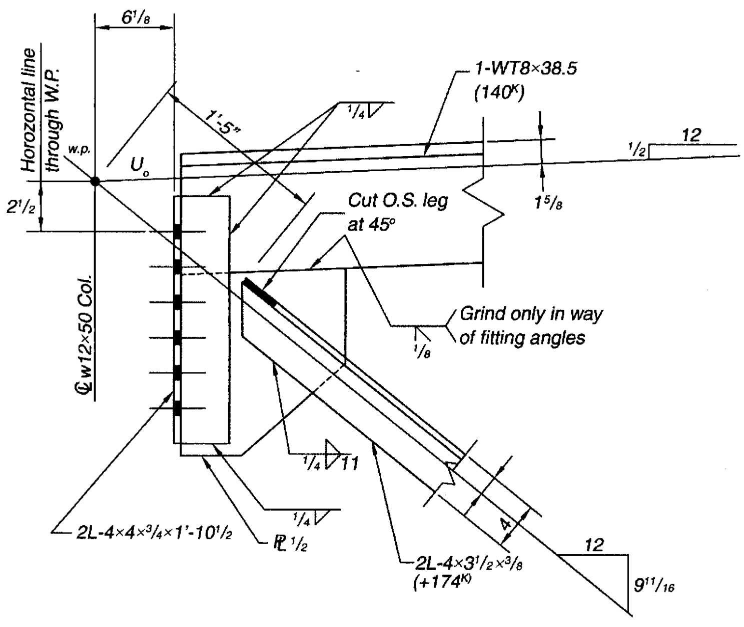


Figure 13–10. Truss-support connection, working point (w.p.) at column centerline.

The total change in length of the truss chord is $\Sigma\Delta_i$, the sum of the changes in the lengths of the individual panel segments of the truss chord. The misalignment at each support connection of the tension chord is one-half the total elongation.

Truss Chord Splices

Truss chord splices are expensive to fabricate and should be avoided whenever possible. In general, chord splices in ordinary building trusses are confined to cases where:

1. the finished truss is too large to be shipped in one piece;
2. the truss chord exceeds the available material length;
3. the reduction in member size of the chord justifies the added cost of a splice; or
4. a sharp change in direction occurs in the working line of the chord and bending does not provide a satisfactory alternative.

Splices at truss chord ends that are finished to bear should be designed in accordance with AISC Specification Section J1.4b.

Design Considerations for HSS to HSS Truss Connections

For the design of HSS-to-HSS truss connections, see AISC Specification Section K2.

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PART 14

DESIGN OF BEAM BEARING PLATES, COLUMN BASE PLATES, ANCHOR RODS, AND COLUMN SPLICES

SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of beam bearing plates, column base plates, anchor rods and column splices. For complete coverage of column base plate connections, see AISC Design Guide 1, *Column Base Plates* (DeWolf, 1990). For bearing plates, column base plates, anchor rods and column splices that are part of a seismic force resisting system in which the seismic response modification factor, R, is taken greater than 3, the requirements in the AISC *Seismic Provisions for Structural Steel Buildings* also apply. The AISC *Seismic Provisions for Structural Steel Buildings* is available in Part 6 of the AISC *Seismic Design Manual* from the American Institute of Steel Construction, Inc. at www.aisc.org.

BEAM BEARING PLATES

A beam bearing plate is made with a plate as illustrated in Figure 14–1.

Force Transfer

The required strength (beam end reaction), R_u or R_a, is distributed from the beam bottom flange to the bearing plate over an area equal to $N \times 2k$, where N is the bearing length (length of contact between the beam bottom flange and the bearing plate), in. The bearing plate is then assumed to distribute the beam end reaction to the concrete or masonry as a uniform bearing pressure by cantilevered bending of the plate. The bearing plate cantilever dimension is taken as

$$n = \frac{B}{2} - k$$

where B is the bearing plate width, in.

In the rare case where a bearing plate is not required, the beam end reaction, R_u or R_a, is assumed to be uniformly distributed from the beam bottom flange to the concrete or masonry as a uniform bearing pressure by cantilevered bending of the beam flanges. The beam-flange cantilever dimension is calculated as for a bearing plate, but using the beam flange width, b_f, in place of B.

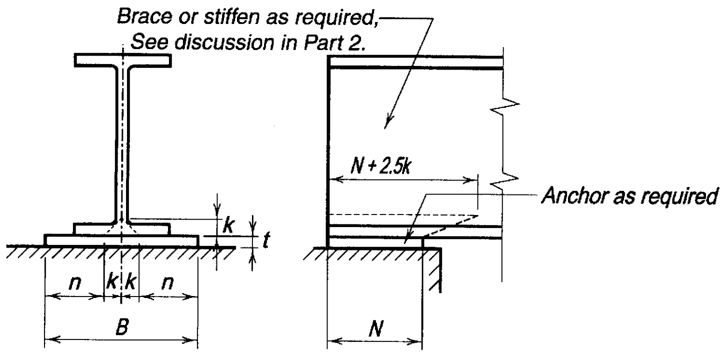


Figure 14–1. Beam bearing plate variables.

Recommended Bearing Plate Dimensions and Thickness

The length of bearing, N, may be established by available wall thickness, clearance requirements, or by the minimum requirements based on local web yielding or web crippling. The selected dimensions of the bearing plate, B and N, should preferably be in full in. Bearing plate thickness should be specified in multiples of $^1/_8$ in. up to $1^1/_4$-in. thickness and in multiples of $^1/_4$ in. thereafter.

Available Strength

The available strength of a beam bearing plate is determined from the applicable limit states from Part 9 (connection elements). In all cases, the available strength, ϕR_n or R_n/Ω, must exceed the required strength, R_u or R_a. The stability of the beam end must also be addressed as discussed in "Design Basis, Stability Bracing, Beam Ends Supported on Bearing Plates" in Part 2.

COLUMN BASE PLATES FOR AXIAL COMPRESSION

A column base plate is made with a plate and a minimum of four anchor rods as illustrated in Figure 14–2. The base plate is often attached to the bottom of column in the shop.

Force Transfer

In Figure 14–3, the required strength (column axial force), P_u or P_a, is distributed from the column end to the column base plate in direct bearing. The column base plate is then assumed to distribute the column axial force to the concrete or masonry as a uniform bearing pressure by cantilevered bending of the plate. The critical base plate cantilever dimension, l, is determined as the larger of m, n, and $\lambda n'$ where

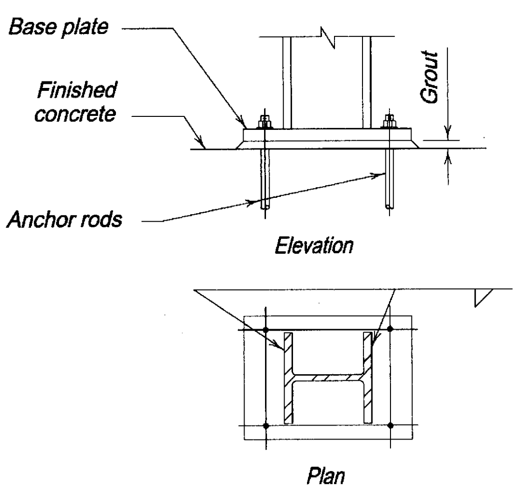


Figure 14–2. Typical column base for axial compressive loads.

$$m = \frac{N - 0.95d}{2}$$

$$n = \frac{B - 0.8b_f}{2}$$

$$n' = \frac{\sqrt{db_f}}{4}$$

$$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} \leq 1$$

LRFD	ASD
$X = \left(\frac{4db_f}{\left(d + b_f\right)^2} \right) \frac{P_u}{\phi_c P_p}$	$X = \left(\frac{4db_f}{\left(d + b_f\right)^2} \right) \frac{\Omega_c P_a}{P_p}$

Note that, because both the term in parentheses and the ratio of the required strength, P_u *or* P_a, to the available strength, $\phi_c P_p$ or P_n/Ω, are always less than or equal to 1, the value of X will always be less than or equal to 1. Note also that λ can always be taken conservatively as 1. For further information, see Thornton (1990) and AISC Design Guide No. 1 *Column Base Plates* (DeWolf and Ricker, 1990).

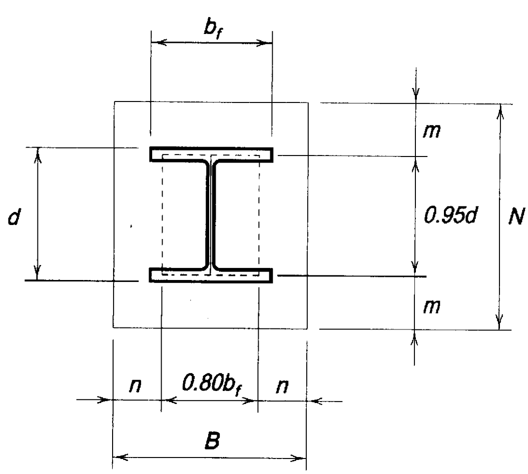


Figure 14–3. Column base-plate design variables.

Recommended Base plate Dimensions and Thickness

The selected dimensions of the base plate B and N should preferably be in full in. Base plate thickness should be specified in multiples of $^{1}/_{8}$ in. up to $1^{1}/_{4}$ in. thickness and in multiples of $^{1}/_{4}$ in. thereafter.

Available Strength

The available strength of an axially loaded column base plate is determined from the applicable limit states in Part 9 (connection elements). From Thornton (1990), the minimum base plate thickness can be calculated as

LRFD	ASD
$t_{min} = l\sqrt{\dfrac{2P_u}{0.9F_yBN}}$	$t_{min} = l\sqrt{\dfrac{3.33P_a}{F_yBN}}$

In all cases, the available strength, ϕR_n or R_n/Ω, must exceed the required strength, R_u or R_a.

Finishing Requirements

Base plate finishing requirements are given in AISC Specification Section M2.8. When finishing is required, the plate material must be ordered thicker than the specified base plate thickness to allow for the material removed in finishing. Finishing allowances are given in Table 14–1 per ASTM A6 flatness tolerances for steel base plates with F_u equal to or less than 60 ksi based upon the width, thickness, and whether one or both sides are to be finished. Finishing allowances for steel base plates with F_u greater than 60 ksi should be increased by 50 percent.

The criteria for fit-up of column splices given in AISC Specification Section M4.4 are also applicable to column base plates.

Holes for Anchor Rods and Grouting

Recommended maximum anchor rod hole sizes are given in Table 14–2. These hole sizes will accommodate reasonable misalignments in the setting of the anchor rods and allow better adjustment of the column base to the correct centerlines. It is normally unnecessary to deduct the area of holes when determining the required base plate area. An adequate washer should be provided for each anchor rod.

When base plates with large areas are used, at least one grout hole should be provided near the center of the base plate through which grout may be placed. This will provide for a more even distribution of the grout and also prevent air pockets. Note that a grout hole may not be required when the grout is dry-packed. Grout holes do not require the same accuracy for size and location as anchor rod holes.

Holes in base plates for anchor rods and grouting often must be flame-cut, because drill sizes and punching capabilities may be limited to smaller diameters. Flame-cut holes may have a slight taper and should be inspected to assure proper clearances for anchor rods.

Grouting and Leveling

High-strength, non-shrink grout is placed between the column base plate and the supporting foundation. When base plates are shipped attached to the column, three methods of column support are:

1. The use of leveling nuts and, in some case, washers on the anchor rods beneath the base plate, as illustrated in Figure 14–4.
2. The use of shim stacks between the base plate and the supporting foundation.
3. The use of a steel leveling plate (normally 1/4 in. thick), set to elevation and grouted prior to the setting of the column. The leveling plate should meet the flatness tolerances specified in ASTM A6. It may be larger than the base plate to accommodate anchor rod placement tolerances and can be used as a setting template for the anchor rods.

For further information on grouting and leveling of column base plates, see AISC Design Guide No. 10 *Erection Bracing of Low-Rise Structural Steel Frames* (Fisher and West, 1997).

When base plates are shipped loose, the base plates are usually grouted after the base plate has been aligned and leveled with one of the preceding methods. For heavy loose base plates, three-point leveling bolts, illustrated in Figure 14–5, are commonly used. These threaded attachments may consist of a nut or an angle and nut welded to the base plate. Leveling bolts must be of sufficient length to compensate for the space provided for grouting. Rounding the point of the leveling bolt will prevent it from "walking" or moving laterally as it is turned. Additionally, a small steel pad under the point reduces friction and prevents damage to the concrete.

Heavy loose base plates should be provided with some means of handling at the erection site. Lifting holes can be provided in the vertical legs of shop-attached connection angles. Lifting lugs can also be used and can remain in place after erection, unless they create an interference or removal is required in the contract documents.

Leveling bolts or nuts should not be used to support the column during erection. If grouting is delayed until after steel erection, the base plate must be shimmed to properly distribute loads to the foundation without overstressing either the base plate or the concrete. This difficulty of supporting columns while leveling and grouting their bases makes it advisable that

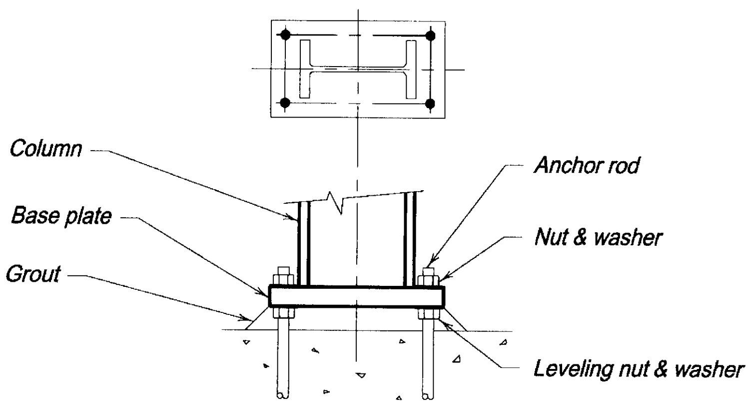


Figure 14–4. Leveling nuts and washers.

footings be finished to near the proper elevation (Ricker, 1989). The top of the rough footing should be set approximately 1 to 2 in. below the bottom of the base plate to provide for adjustment. Alternatively, an angle frame as illustrated in Figure 14–6 could be constructed to the proper elevation and filled with grout prior to erection.

COLUMN BASE PLATES FOR AXIAL TENSION, SHEAR, OR MOMENT

For anchor rod diameters not greater than 1¼ in., angles bolted or welded to the column as shown in Figure 14–7a are generally adequate to transfer uplift forces resulting from axial loads and moments. When greater resistance is required, stiffeners may be used with hori-

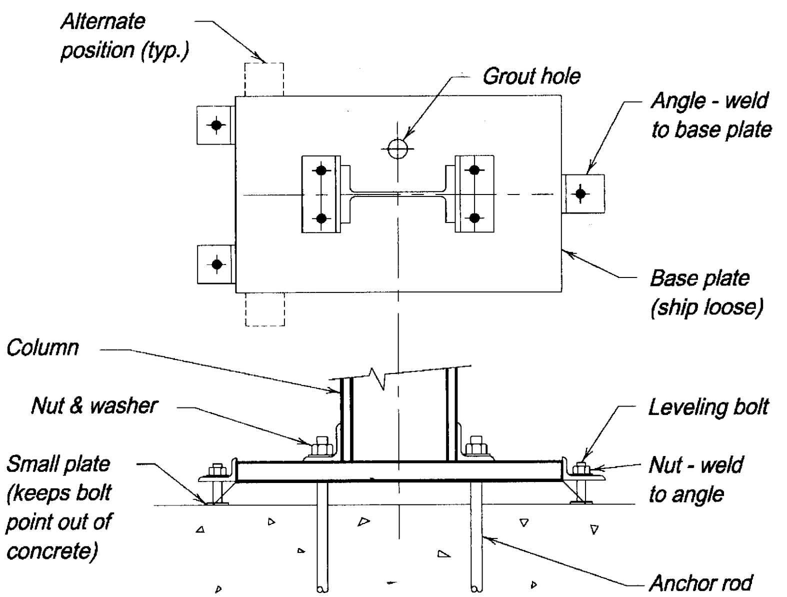


Figure 14–5. Three-point leveling.

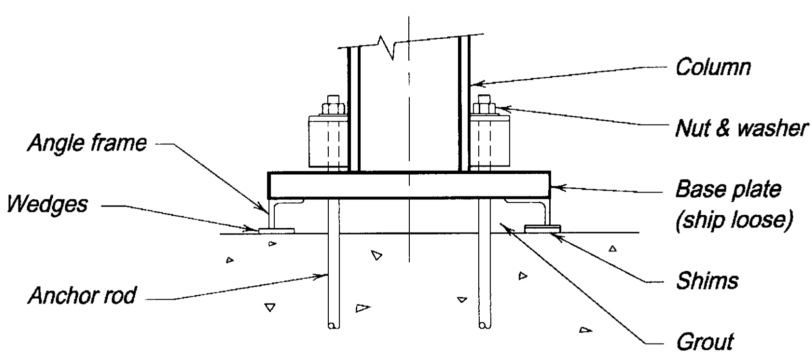


Figure 14–6. Angle-frame leveling.

zontal plates or angles as illustrated in Figure 14–7b. These stiffeners are not usually considered to be part of the column area in bearing on the base plate. The angles preferably should be set back from the column end about $^1/_8$ in. Stiffeners preferably should be set back about 1 in. from the base plate to eliminate a pocket that might prevent drainage and, thus, protect the column and column base plate from corrosion.

For further information, see AISC Design Guide No. 1 *Column Base Plates* (DeWolf and Ricker, 1990).

ANCHOR RODS

Cast-in-place anchor rods, illustrated in Figure 14–8, are generally made from unheaded rod material or headed bolt material. Drilled-in (post-set) anchors can be used for corrective

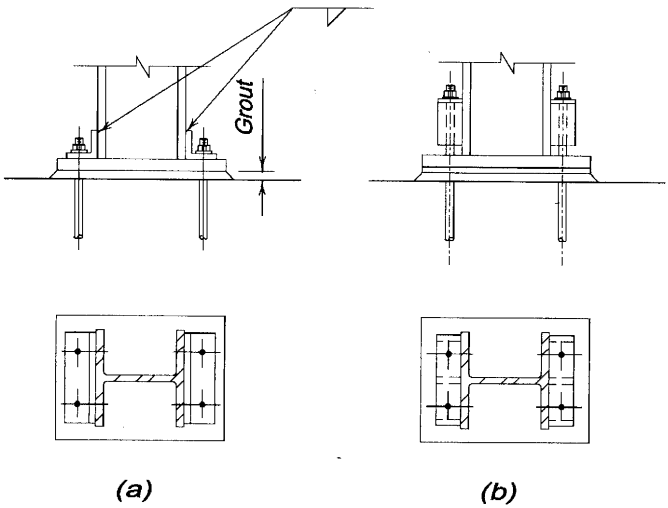


Figure 14–7. Typical column bases for uplift.

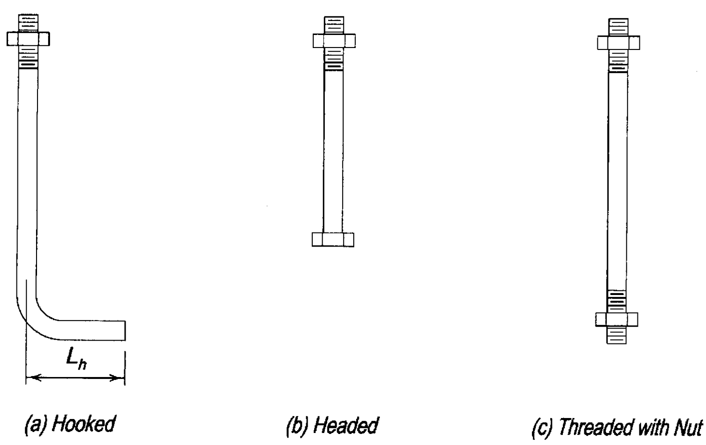


Figure 14–8. Cast-in-place anchor rods.

work or in new work as determined by the owner's designated representative for design and as permitted in the applicable building code. The design of post-set anchors is governed by manufacturer's specifications; see also ACI 349 Appendix B. Post-set anchors that rely upon torque or tension to develop anchorage by wedging action should not be used unless the stability of the column during erection is provided by means other than the post-set anchors.

Minimum Edge Distance and Embedment Length

In general, minimum edge distances, embedment lengths and the design of anchorages into concrete are covered by ACI 318. These provisions include methods to account for edge distance and group action, as does ACI 349. AISC Design Guides 1, 7, and 10 provide additional material on the design of anchor rods in concrete.

In addition to providing the recommended minimum embedment length, anchor rods must extend a distance above the foundation that is sufficient to permit adequate thread engagement of the nut. Adequate thread engagement for anchor rods is identical to the condition described in the RCSC Specification as adequate for steel-to-steel structural joints using high-strength bolts: having the end of the [anchor rod] flush with or outside the face of the nut.

Washer Requirements

Because base plates typically have holes larger than oversized holes to allow for tolerances on the location of the anchor rod, washers are usually furnished from ASTM A36 steel plate. They may be round, square, or rectangular, and generally have holes that are $^1/_{16}$-in. larger than the anchor rod diameter. The thickness must be suitable for the forces to be transferred. Minimum washer sizes are given in Table 14–2.

Hooked Anchor Rods

Hooked anchor rods should be used only for axially loaded members subject to compression only to locate and prevent the displacement or overturning of columns due to erection loads or accidental collisions during erection. Additionally, high-strength steels are not recommended for use in hooked rods since bending with heat may materially affect their strength.

Headed or Threaded and Nutted Anchor Rods

When anchor rods are required for a calculated tensile force, T, a more positive anchorage is formed when headed anchor rods, illustrated in Figure 14–8b, are used. With adequate embedment and edge distance, the limit state is either a tensile failure of the anchor rod or the pull-out of a cone of concrete radiating outward from the head (Marsh and Burdette, 1985) as illustrated in Figure 14–9. Marsh and Burdette (1985) showed that the head of the anchor rod usually provides sufficient anchorage and the use of an additional washer or plate does not add significantly to the anchorage. The nut and threading shown in Figure 14–8c is acceptable in lieu of a bolt head. The nut should be welded to the rod to prevent the rod from turning out when the top nut is tightened.

Anchor Rod Nut Installation

The majority of anchorage applications in buildings do not require special anchor rod nut installation procedures or pretension in the anchor rod. The anchor rod nuts should be "drawn down tight" as columns and bases are erected, per ANSI A10.13 Section 9.6. This condition can be achieved by following the same practices as recommended for snug-tightened installation in steel-to-steel bolted joints in the RCSC Specification. That is, most anchor rod nuts can be installed using the full effort of an ironworker with an ordinary spud wrench.

When, in the judgment of the owner's designated representative for design, the performance of the structure will be compromised by excessive elongation of the anchor rods under tensile loads, pretension may be required. Some examples of applications that may require pretension include structures that cantilever from concrete foundations, moment-resisting column bases with significant tensile forces in the anchor rods, or where load reversal might result in the progressive loosening of the nuts on the anchor rods.

When pretensioning of anchor rods is specified, care must be taken in the design of the column base and the embedment of the anchor rod. The shaft of the anchor rod must be free of bond to the encasing concrete so that the rod is free to elongate as it is pretensioned. Also, loss of pretension due to creep in the concrete must be taken into account. Although the design of pretensioned anchorage devices is beyond the scope of this Manual, it should be noted that pretension should not be specified for anchorage devices that have not been properly designed and configured to be pretensioned.

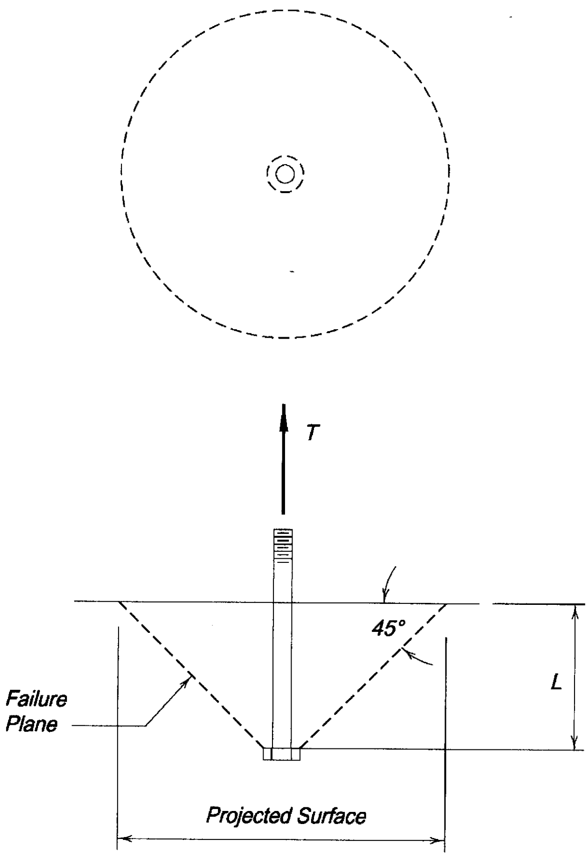


Figure 14–9. Concrete cone subject to pull-out.

COLUMN SPLICES

When the height of a building exceeds the available length of column sections, or when it is economically advantageous to change the column size at a given floor level, it becomes necessary to splice two columns together. Column splices at the final exterior and interior perimeter and at interior openings must be located a minimum of 48 in. above the finished floor to accommodate the attachment of safety cables, except when constructability does not allow. For simplicity and uniformity, other column splices should be located at the same height. Note that column splices placed significantly higher than this are impractical in terms of field assembly.

Fit-Up of Column Splices

From AISC Specification Section M2.6, the ends of columns in a column splice which depend upon contact bearing for the transfer of axial forces must be finished to a common plane by milling, sawing, or other suitable means. In theory, if this were done and the pieces were erected truly plumb, there would be full-contact bearing across the entire surface; this is true in most cases. However, AISC Specification Section M4.4 recognizes that a perfect fit on the entire available surface will not exist in all cases.

A $^1/_{16}$-in. gap is permissible with no requirements for repair or shimming. During erection, at the time of tightening the bolts or depositing the welds, columns will usually be subjected to loads which are significantly less than the design loads. Full-scale tests (Popov and Steven, 1977) which progressed to column failure have demonstrated that subsequent loading to the design loads does not result in distress in the bolts or welds of the splice.

If the gap exceeds $^1/_{16}$ in. but is less than $^1/_4$ in., non-tapered steel shims are required if sufficient contact area does not exist. Mild steel shims are acceptable regardless of the steel grade of the column or bearing material. If required, these shims must be contained, usually with a tack weld, so that they cannot be worked out of the joint.

There is no provision in the AISC Specification for gaps larger than $^1/_4$ in. When such a gap exists, an engineering evaluation should be made of this condition based upon the type of loading transferred by the column splice. Tightly driven tapered shims may be required or the required strength may be developed through flange and web splice plates. Alternatively, the gap may be ground or gouged to a suitable profile and filled with weld metal.

Lifting Devices

As illustrated in Figure 14–10, lifting devices are typically used to facilitate the handling and erection of columns. When flange-plated or web-plated column splices are used for W-shape columns, it is convenient to place lifting holes in these flange plates as illustrated in Figure 14–10a. When butt-plated column splices are used, additional temporary plates with lifting holes may be required as illustrated in Figure 14–10b. W-shape column splices which do not utilize web-plated or butt-plated column splices (i.e., groove-welded column splices) may be provided with a lifting hole in the column web as illustrated in Figure 14–10c. While a hole in the column web reduces the cross-sectional area of the column, this reduction will seldom be critical since the column is sized for the loads at the floor below and the splice is located above the floor. Alternatively, auxiliary plates with lifting holes may be connected to the column so that they do not interfere with the welding. Typical column splices for tubes and box-columns are illustrated in Figure 14–10d. Holes in lifting devices may be drilled, reamed, or flame-cut with a mechanically guided torch. In the latter case, the bearing surface of the hole in the direction of the lift must be smooth.

The lifting device and its attachment to the column must be of sufficient strength to support the weight of the column as it is brought from the horizontal position (as delivered) to the vertical position (as erected); the lifting device and its attachment to the column must be adequate for the tensile forces, shear forces, and moments induced during handling and erection.

A suitable shackle and pin are connected to the lifting device while the column is on the ground. The steel erector usually establishes the size and type of shackle and pin to be used in erection and this information must be transmitted to the fabricator prior to detailing. Except for excessively heavy lifting pieces, it is customary to select a single pin and pinhole diameter to accommodate the majority of structural steel members, whether they

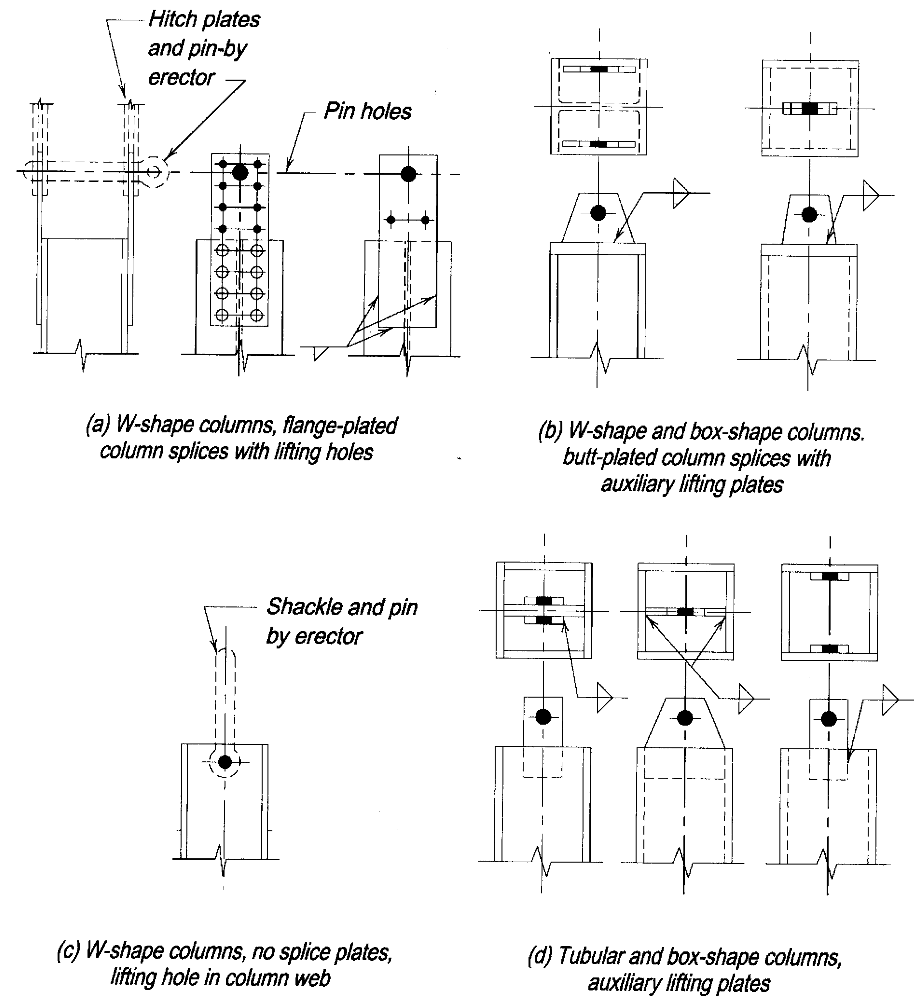


Figure 14–10. Lifting devices for columns.

are columns or other heavy structural steel members. The pin is attached to the lifting hook and a lanyard trails to the ground or floor level. After the column is erected and connected, the pin is removed from the device by means of the lanyard, eliminating the need for an ironworker to climb the column. The shackle pin, as assembled with the column, must be free and clear, so that it may be withdrawn laterally after the column has been landed and stabilized.

The safety of the structure, equipment, and personnel is of utmost importance during the erection period. It is recommended that all welds that are used on the lifting devices and stability devices be inspected very carefully, both in the shop and later in the field, for any damage that may have occurred in handling and shipping. Groove welds frequently are inspected with ultrasonic methods (UT) and fillet welds are inspected with magnetic particle (MT) or liquid dye penetrant (PT) methods.

Column Alignment and Stability During Erection

Column splices should provide for safety and stability during erection when the columns might be subjected to wind, construction, and/or accidental loading prior to the placing of the floor system. The nominal flange-plated, web-plated, and butt-plated column splices developed here consider this type of loading.

In other splices, column alignment and stability during erection are achieved by the addition of temporary lugs for field bolting as illustrated in Figure 14–11. The material thickness, weld size, and bolt diameter required are a function of the loading. A conservative resisting moment arm is normally taken as the distance from the compressive toe or flange face to the gage line of the temporary lug. The overturning moment should be checked about both axes of the column. The recommended minimum plate or angle thickness is $^1/_2$ in.; the recommended minimum weld size is $^5/_{16}$ in.; additionally, high-strength bolts are normally used as stability devices.

Temporary lugs are not normally used as lifting devices. Unless required to be removed in the contract documents, these temporary lugs may remain.

Column alignment is provided with centerpunch marks that are useful in centering the columns in two directions.

Force Transfer in Column Splices

As illustrated in Figure 14–12, for the W-shapes most frequently used as columns, the distance between the inner faces of the flanges is constant throughout any given nominal depth group; as the nominal weight per foot increases for each nominal depth, the flange and web thicknesses increase. From AISC Specification Section J7, the available bearing strength, ϕR_n *or* R_n/Ω, of the contact area of a finished surface is determined with

$$R_n = (1.8 F_y A_{pb})$$

$$\phi = 0.75 \qquad \Omega = 2.0$$

This bearing strength is much greater than the axial strength of the column and will seldom prove critical in the member design. For column splices transferring only axial forces, complete axial force transfer may be achieved through bearing on finished surfaces; bolts or welds are required by AISC Specification Section J1.4 to be sufficient to hold all parts securely in place.

In addition to axial forces, from AISC Specification Section J1.4, column splices must be proportioned to achieve the required strength in tension, due to the combination of dead load and lateral loads. Note that it is not permissible to use forces due to live load to offset the tensile forces from wind or seismic loads.

For dead and wind loads, if the required strength due to the effect of the dead load is greater than the required strength due to the wind load, the splice is not subjected to tension

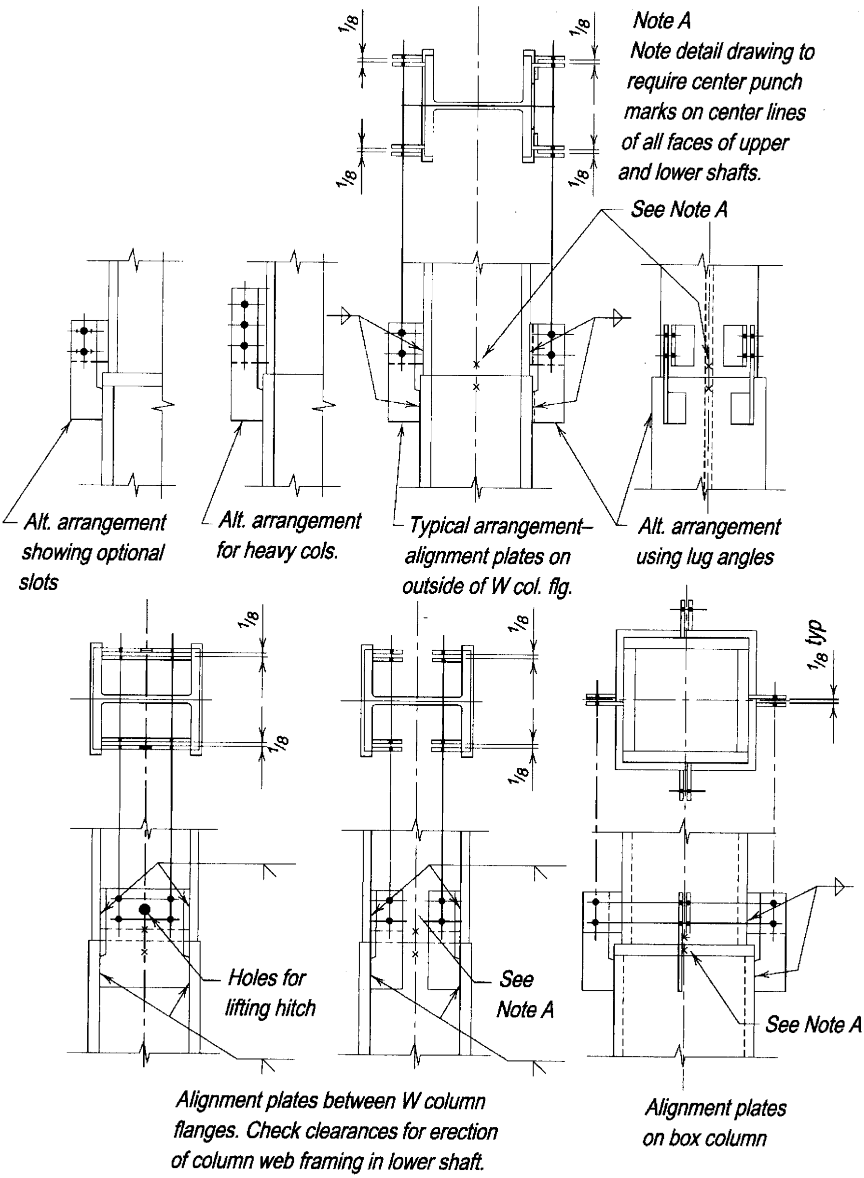


Figure 14–11. Column stability and alignment devices.

and a nominal splice may be selected from those in Table 14–3. When the required strength due to dead load is less than the required strength due to the wind load, the splice will be subjected to tension and the nominal splices from Table 14–3 are acceptable if the available tensile strength of the splice is greater than or equal to the required strength. Otherwise, a splice must be designed with sufficient area and attachment.

When shear from lateral loads is divided among several columns, the force on any single column is relatively small and can usually be resisted by friction on the contact bearing surfaces and/or by the flange plates, web plates, or butt plates. If the required shear strength exceeds the available shear strength of the column splice selected from Table 14–3, a column splice must be designed with sufficient area and attachment.

The column splices shown in Table 14–3 meet the OSHA requirement for 300 lbs located 18 in. from the column face.

Flange-Plated Column Splices

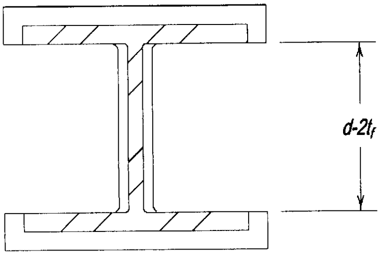
Table 14–3 give typical flange-plated column splice details for W-shape columns. These details are not splice standards, but rather, typical column splices in accordance with AISC Specification provisions and typical erection requirements. Other splice designs may also be developed. It is assumed in all cases that the lower shaft will be the heavier, although not necessarily the deeper, section.

Full-contact bearing is always achieved when lighter sections are centered over heavier sections of the same nominal depth group. If the upper column is not centered on the lower column, or if columns of different nominal depths must bear on each other, some areas of the upper column will not be in contact with the lower column. These areas are hatched in Figure 14–13.

When additional bearing area is not required, unfinished fillers may be used. These fillers are intended for "pack-out" of thickness and are usually set back 1/4 in. or more from the finished column end. Since no force is transferred by these fillers, only nominal attachment to the column is required.

When additional bearing area is required, fillers finished to bear on the larger column may be provided. Such fillers are proportioned to carry bearing loads at the bearing strength calculated from AISC Specification Section J7 and must be connected to the column to transfer this calculated force.

Although flange plates are shown shop-assembled to the lower column, it is equally acceptable to invert this arrangement and place them on the upper column. This will usually require fills of increased thickness to maintain erection clearances.



Column Size	d-$2t_f$ (in.)
W8×24–67	7.13
W10×33–112	8.86
W12×40–336	10.9
W14×43–808	12.6

Figure 14–12. Distance between flanges for typical W-shape columns.

In Table 14–3, Cases I and II are for all-bolted flange-plated column splices for W-shape columns. Bolts in column splices are usually the same size and type as for other bolts on the column. Bolt spacing, end distance, and edge distances resulting from the plate sizes shown permit the use of 3/4-in. and 7/8-in. bolts in the splice details shown. Larger diameter bolts may require an increase in edge or end distances. Refer to AISC Specification Chapter J. The use of high-strength bolts in bearing-type connections is assumed in all field and shop splices. However, when slotted or oversized holes are utilized, or in splices employing undeveloped fillers over 1/4 in. thick, slip-critical connections may be required; refer to AISC Specification Section J6. For ease of erection, field clearances for lap splices fastened by bolts range from 1/8 in. to 3/16 in. under each plate.

Cases IV and V are for all-welded flange-plated column splices for W-shape columns. Splice welds are assumed to be made with E70XX electrodes and are proportioned as required by the AISC Specification provisions. The SAW, GMAW, and FCAW equivalents to E70XX electrodes may be substituted if desired. Field clearance for welded splices are limited to 1/16 in. to control the expense of building up welds to close openings. Note that the fillet weld lengths, *Y*, as compared to the lengths *L*/2, provide 2-in. unwelded distance below and above the column shaft finish line. This provides a degree of flexibility in the splice plates to assist the erector.

Cases VI and VII apply to combination bolted and welded column splices. Since the available strength of the welds will, in most cases, exceed the strength of the bolts, the weld and splice lengths shown may be reduced, if desired, to balance the strength of the fasteners to the upper or lower column, provided that the available strength of the splice is still greater than the required strength of the splice, including erection loading.

Directly Welded Flange Column Splices

Table 14–3 also includes typical directly welded flange column splice details for W-shape and HSS or box-shaped columns. These details are not splice standards, but rather, typical column splices in accordance with AISC Specification provisions and typical erection requirements. Other splice designs may also be developed. It is assumed in all cases that the lower shaft will be the heavier, although not necessarily the deeper, section.

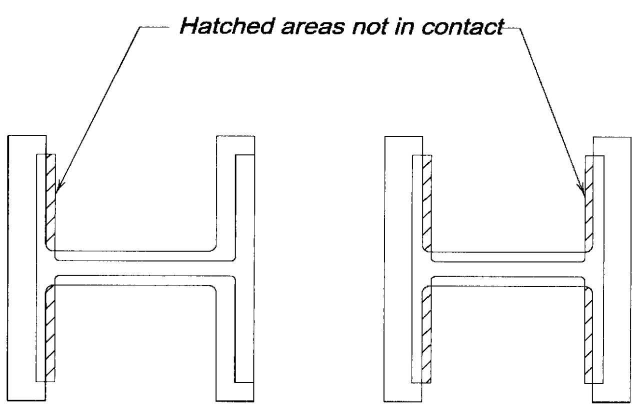


Figure 14–13. Columns not centered or of different nominal depth.

Case VIII applies to W-shape columns spliced with either partial-joint-penetration or complete-joint-penetration groove welds. Case X applies to HSS or box-shaped columns spliced with partial-joint-penetration or complete-joint-penetration groove welds.

Butt-Plated Column Splices

Table 14–3 further includes typical butt-plated column splice details for W-shape and HSS or box-shaped columns. These details are not splice standards, but rather, present typical column splices in accordance with AISC Specification provisions and typical erection requirements. Other splice designs may also be developed. It is assumed in all cases that the lower shaft will be the heavier, although not necessarily the deeper, section.

Butt plates are used frequently on welded splices where the upper and lower columns are of different nominal depths, but may not be economical for bolted splices since fillers cannot be eliminated. Typical butt plates are 1½ in. thick for a W8 over W10 splice, and 2-in. thick for other W-shape combinations such as W10 over W12 and W12 over W14. Butt plates which are subjected to substantial bending stresses, such as required on boxed columns, will require a more careful review and analysis. One common method is to assume forces are transferred through the butt plate on a 45° angle and check the thickness obtained for shear and bearing strength. Finishing requirements for butt plates are specified in AISC Specification Section M2.8.

Case III is a combination flange-plated and butt-plated column splice for W-shape columns. Case IX applies to welded butt-plated column splices for W-shape columns. Case XI applies to welded butt-plated column splices for HSS or box-shaped columns. Case XII applies to welded butt-plated column splices between W-shape and HSS or box-shaped columns.

DESIGN CONSIDERATIONS FOR HSS CAP PLATES

The simplest form of attachment to an HSS is to connect the framing member to the top of an HSS. The cap plate serves as a bearing device to transfer the reactions from the framing member into the HSS. The cap plate may also be used to transfer moment into the HSS column. The moment transfer is through a force couple that consists of both compressive and tensile reactions delivered to the cap plate.

Flexural Strength of the Cap Plate

The available strength of the cap plate, in terms of reaction resistance, is determined as ϕR_n or R_n/Ω with

$$R_n = \frac{Bt_c^{\,2}}{4\left(\dfrac{N_r}{2} + a - \dfrac{H}{2}\right)} F_{yc}$$

$$\phi = 0.90 \qquad \Omega = 1.67$$

where

B = the HSS width, in.
t_c = the cap plate thickness, in.
N_r = the required bearing length for the attached member, in.
a = the distance from the HSS centroid to the end of the attached member, in.

H = the HSS depth, in.
F_{yc} = the specified yield strength of the cap plate, ksi

This equation applies only if the cap plate is subjected to cantilever bending, as shown in Figure 14–14. This occurs when the beam or joist reaction point is outside of the HSS face. If a stiffener is used in the beam and is positioned over the HSS wall, then the equation does not apply, since the cap plate is not subjected to bending. Also if the denominator of the equation results in a negative number, bending of the cap plate can be disregarded.

Compression Yielding and Crippling of the HSS Wall

The available strength of the HSS wall due to compression yielding and compression crippling is determined in accordance with AISC Specification Section K1.6.

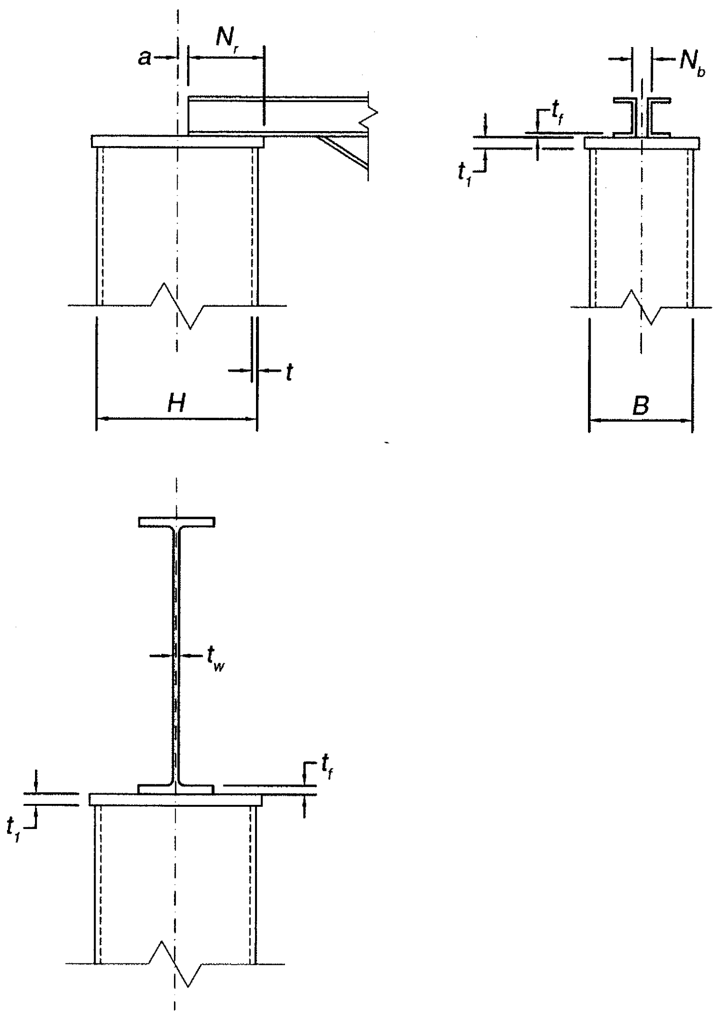


Figure 14–14. Cap plate subject to cantilever bending.

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Table 14–1
Finish Allowances

Size	Thickness (in.)	Add to Fin. One Side (in.)	Add to Fin. Two Sides (in.)
Maximum dimension 24 in. or less	$1\frac{1}{4}$ or less	$\frac{1}{16}$	$\frac{1}{8}$
	over $1\frac{1}{4}$ to 2, incl.	$\frac{1}{8}$	$\frac{1}{4}$
Maximum dimension over 24 in.	$1\frac{1}{4}$ or less	$\frac{1}{8}$	$\frac{1}{4}$
	over $1\frac{1}{4}$ to 2, incl.	$\frac{3}{16}$	$\frac{3}{8}$
56 in. wide or less	over 2 to $7\frac{1}{2}$, incl.	$\frac{1}{4}$	$\frac{3}{8}$
	over $7\frac{1}{2}$ to 10, incl.	$\frac{1}{2}$	$\frac{5}{8}$
	over 10 to 15, incl.	$\frac{3}{4}$	$\frac{7}{8}$
Over 56 in. wide to 72 in. wide	over 2 to 6, incl.	$\frac{1}{4}$	$\frac{3}{8}$
	over 6 to 10, incl.	$\frac{1}{2}$	$\frac{5}{8}$
	over 10 to 15, incl.	$\frac{3}{4}$	$\frac{7}{8}$

Table 14–2
Recommended Maximum Sizes for Anchor-Rod Holes in Base Plates

Anchor Rod Diameter, in.	Max. Hole Diameter, in.	Min. Washer Size, in.	Min. Washer Thickness	Anchor Rod Diameter, in.	Hole Diameter, in.	Min. Washer Size, in.	Min. Washer Thickness
$\frac{3}{4}$	$1\frac{5}{16}$	2	$\frac{1}{4}$	$1\frac{1}{2}$	$2\frac{5}{16}$	$3\frac{1}{2}$	$\frac{1}{2}$
$\frac{7}{8}$	$1\frac{9}{16}$	$2\frac{1}{2}$	$\frac{5}{16}$	$1\frac{3}{4}$	$2\frac{3}{4}$	4	$\frac{5}{8}$
1	$1\frac{13}{16}$	3	$\frac{3}{8}$	2	$3\frac{1}{4}$	5	$\frac{3}{4}$
$1\frac{1}{4}$	$2\frac{1}{16}$	3	$\frac{1}{2}$	$2\frac{1}{2}$	$3\frac{3}{4}$	$5\frac{1}{2}$	$\frac{7}{8}$

Notes: 1. Circular or square washers meeting the washer size are acceptible.
2. Clearance must be considered when choosing an appropriate anchor rod hole location, noting effects such as the position of the rod in the hole with respect to the column, weld size and other interferences.
3. When base plates are less than $1\frac{1}{4}$ in. thick, punching of holes may be an economical option. In this case, $\frac{3}{4}$-in. anchor rods and $1\frac{1}{16}$-in. diameter punched holes may be used with ASTM F844 (USS Standard) washers in place of fabricated plate washers.

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 2½×⅛ 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. or $3/16$ 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.

For lifting devices, see Figure 14-11.

Table 14–3 (continued)
Typical Column Splices
Case I:
All-bolted flange-plated column splices between columns with depth d_u and d_l nominally the same.

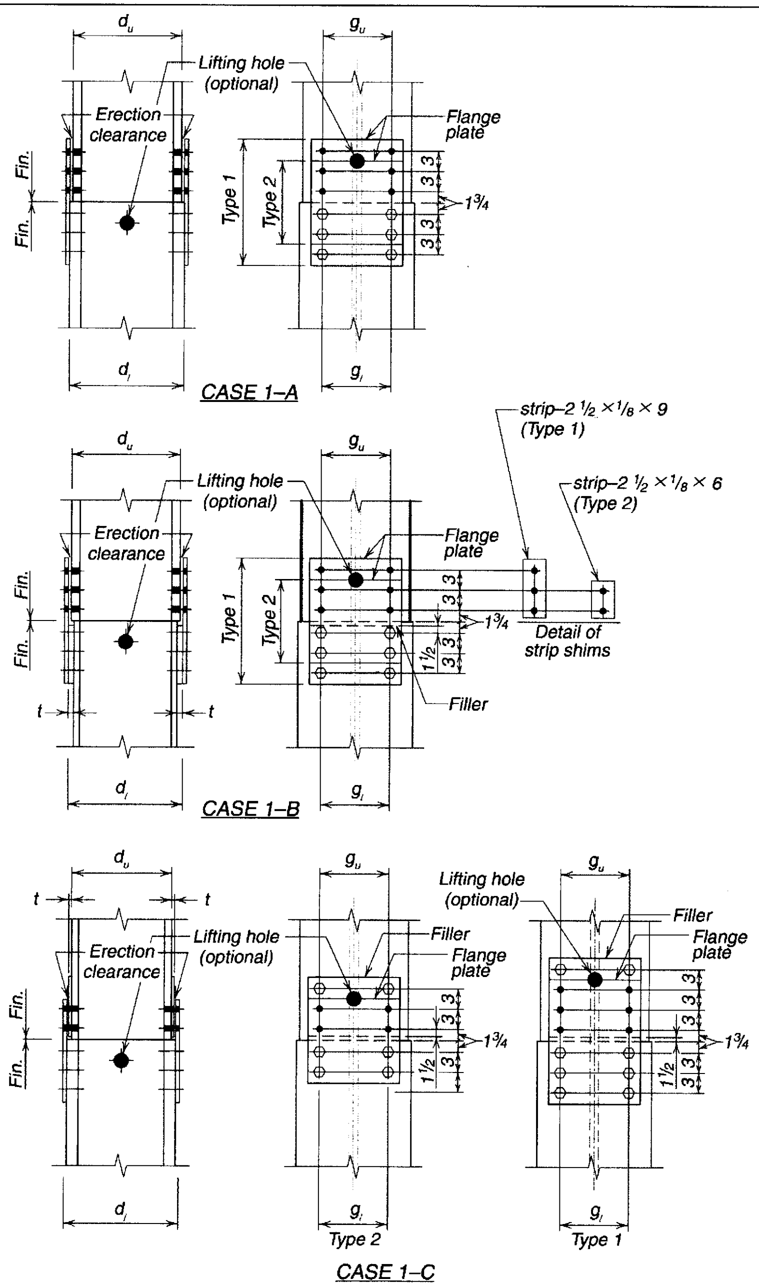


Table 14–3 (continued)

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 ⅛-in. or $^3/_{16}$-in., whichever results in ⅛-in. multiples of filler thickness. Select bolts through fillers (including bolts through flange plates) on each side to develop bearing strength 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 (continued)

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.

For lifting devices, see Figure 14–11.

Table 14–3 (continued)
Typical Column Splices
Case II and III:
All-bolted flange-plated column splices between columns with depth d_u nominally two inches less than depth d_l.

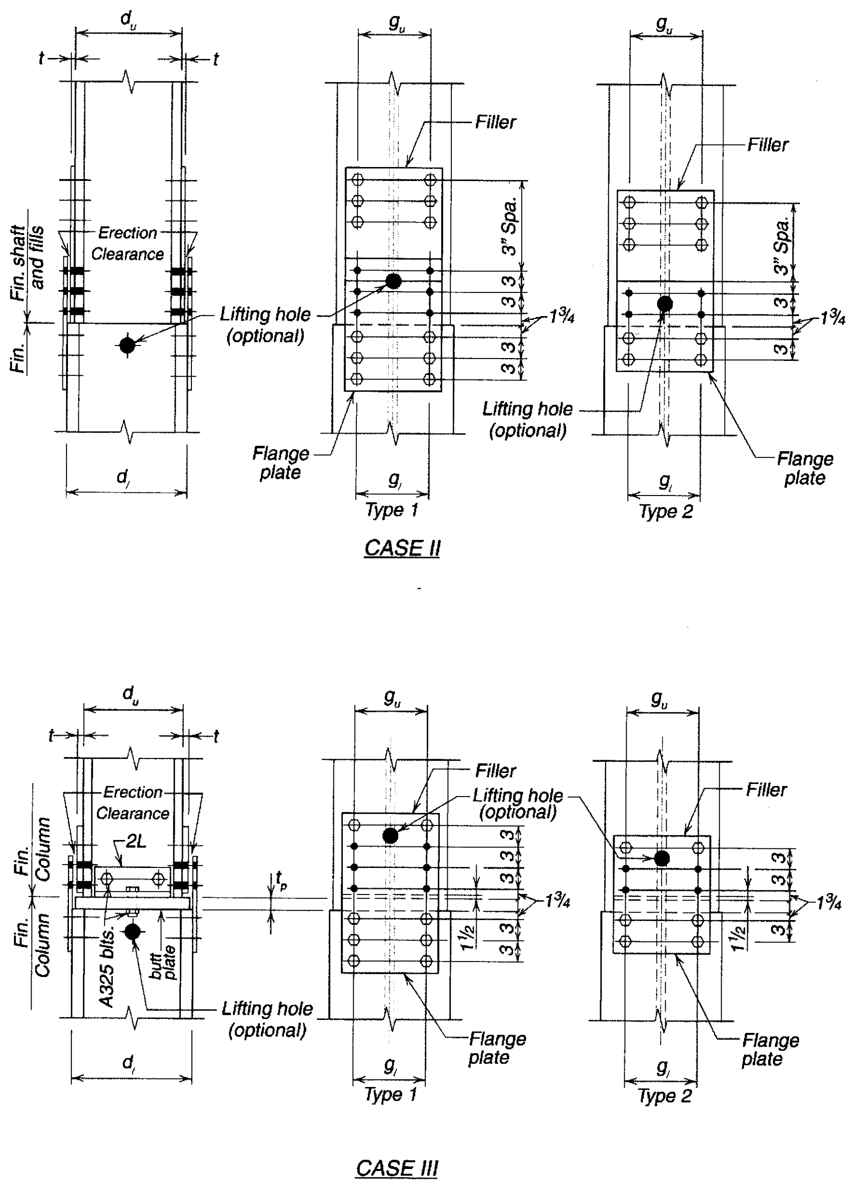


Table 14–3 (continued)
Typical Column Splices
Case IV:
All-welded flange-plated column splices between columns with depths d_u and d_l nominally the same.

Column Size	Flange Plate			Welds			Minimum Space for Welding	
	Width	Thk.	Length L	Size A	Length X	Length Y	M	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.

For lifting devices, see Figure 14–11.

Table 14–3 (continued)
Typical Column Splices
Case IV:
All-bolted flange-plated column splices between columns with depth d_u nominally two inches less than depth d_l.

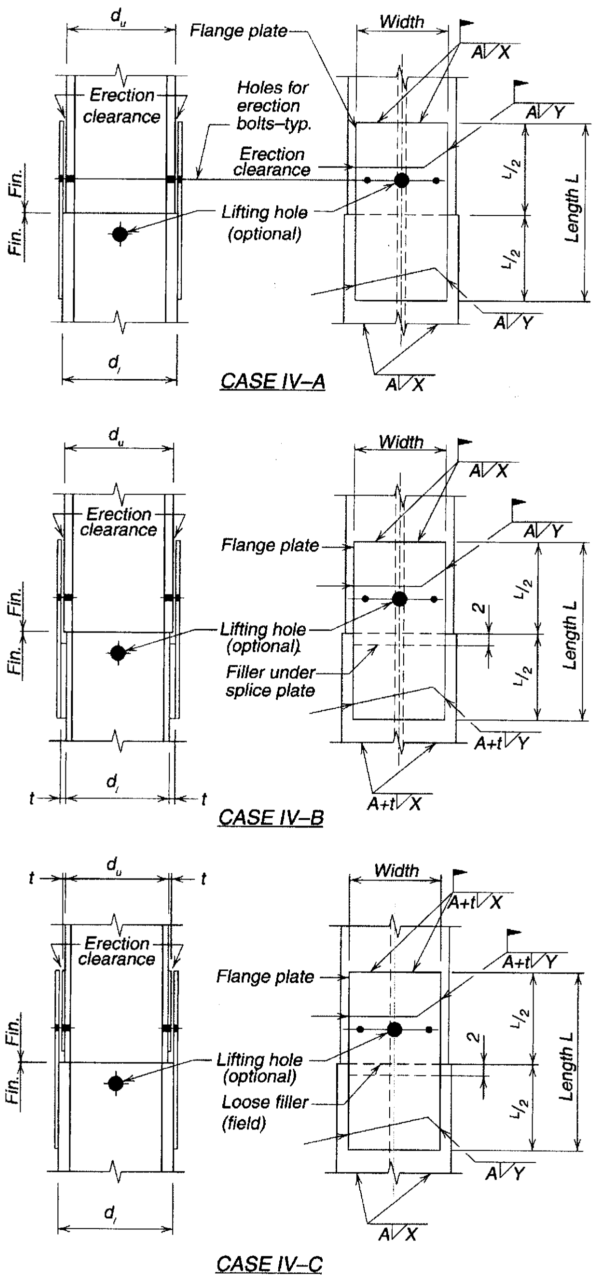
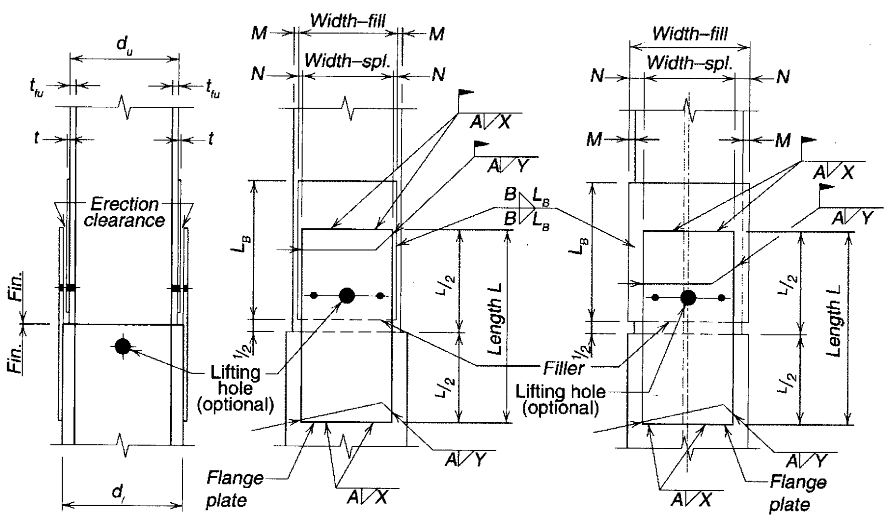


Table 14–3 (continued)
Typical Column Splices
Case IV:
All-welded flange-plated column splices between columns with depths d_u and d_l nominally the same.

Case IV-D: $d_l = (d_u + 5/8$ in.) and over Filler width less than upper column flange width.	Flange plates: Same as Case IV-A, except see Note 1. Fillers (developed on upper column, shop welded to upper column): Select thickness t as $(d_l - d_u) / 2 - 1/16$ in. Select weld size B from AISC Specification; $\leq 5/16$-in. preferred. Select weld length L_B such that $L_B \geq A(X + Y) / B \geq (L / 2 + 1$ in.). Select filler width greater than flange plate width + $2N$ but less than upper column flange width – $2M$. Select filler length as L_B, subject to Note 2.
Case IV-E: $d_l = (d_u + 5/8$ in.) and over Filler width greater than upper column flange width. Use this case only when M or N in Case IV-D are inadequate for welds B and A.	Flange plates: Same as Case IV-A, except see Note 1. Fillers (developed on upper column, shop welded to upper column): Select thickness t as $(d_l - d_u) / 2 - 1/16$ in. Select weld size B from AISC Specification; $\leq 5/16$-in. preferred. Select weld length L_B such that $L_B \geq A(X + Y) / B \geq (L / 2 + 1$ in.). Select filler width as the larger of the flange plate width + $2N$ and the upper column flange width + $2M$, rounded to the next higher $1/4$-in. increment. Select filler length as L_B subject to Note 2.

Table 14–3 (continued)
Typical Column Splices
Case IV:
All-welded flange-plated column splices between columns with depths d_u and d_l nominally the same.



Note 1:
Where welds fasten flange plates to developed fillers, or developed fillers to column flanges (Cases IV-E and V-B), use the table to the right to check minimum fill thickness for balanced fill and weld shear strength.
Assume that an E70XX weld with $A = \frac{1}{2}$, $X = 4$, and $Y = 6$ is to be used at full strength on an A36 fill $\frac{1}{4}$-in. thick. Since this table shows that the minimum fill thickness to develop this $\frac{1}{2}$-in. weld is 0.51 in., the $\frac{1}{4}$-in. fill will be overstressed. A balanced condition is obtained by multiplying the length $(X + Y)$ by the ratio of the minimum to the actual thickness of fill, thus:

Weld A E70XX	Minimum Fill Thickness for Balanced Weld and Plate Shear	
	F_y	
	36	50
$\frac{1}{4}$	0.26	0.19
$\frac{5}{16}$	0.32	0.23
$\frac{3}{8}$	0.38	0.28
$\frac{7}{16}$	0.45	0.33
$\frac{1}{2}$	0.51	0.37

$$(4 + 6) \times \frac{0.51}{0.25} = 20.4$$

use $(X + Y) = 20\frac{1}{2}$-in.

Placing this additional increment of $(X + Y)$ can be done by making weld lengths X continuous across the end of the splice plate and by increasing Y (and therefore the plate Length) if required.

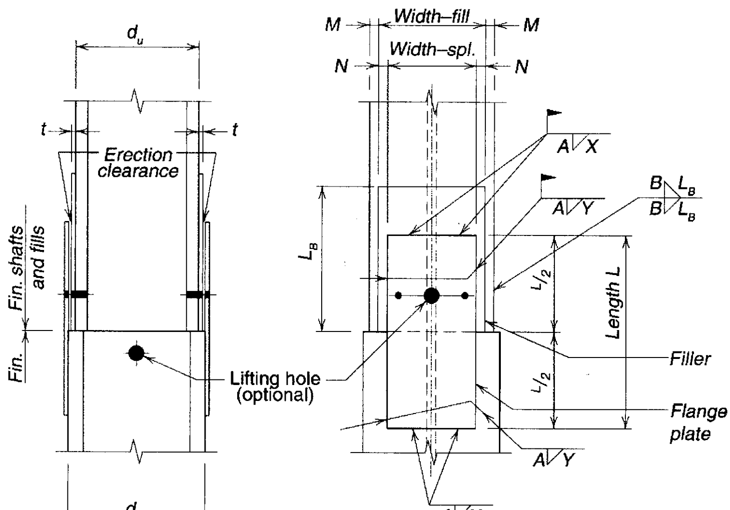
Note 2:
If fill length, based on L_B, is excessive, place weld of size B across one or both ends of fill and reduce L_B accordingly, but not to less than $(L / 2 + 1)$. Omit return welds in Cases IV-E and V-B.

Table 14–3 (continued)
Typical Column Splices
Case V:
All-welded flange-plated column splices between columns with depth d_u nominally two inches less than depth d_l.

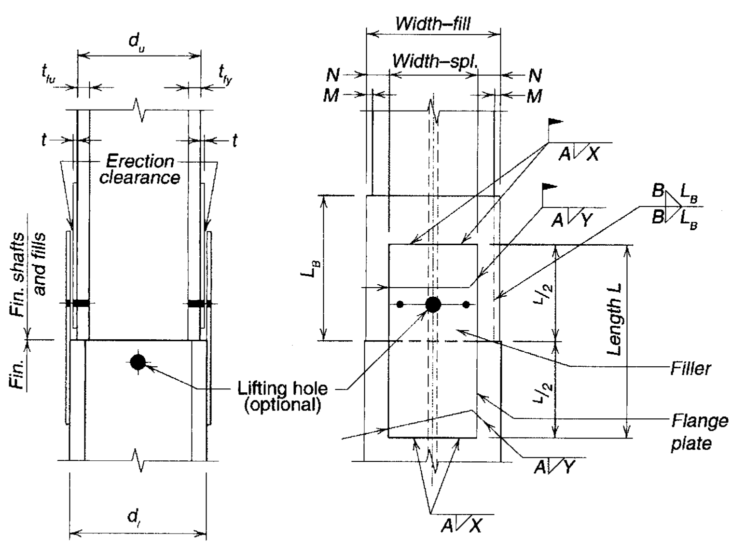
Case V-A: Fillers on upper column developed for bearing on lower column. Filler width less than upper column flange width.	Flange plates: Same as Case IV-A, except see Note 1. Fillers (shop welded to upper column): Select thickness as $(d_l - d_u)/2 - \frac{1}{16}$ in. Select weld size B from AISC Specification; $\leq\frac{5}{16}$ in. preferred. Select weld length L_B to develop bearing strength of the filler but not less than $(L/2 + 1\frac{1}{2}$ in.). Select filler width greater than the flange plate width + $2N$ but less than the upper column flange width − $2M$. See Case IV for M and N.
Case V-B: Same as Case V-A except filler width is greater than upper column flange width. Use this case only when M or N in Case V-A are inadequate for weld A, or when additional filler bearing area is required.	Flange plates: Same as Case IV-A, except see Note 1. Fillers (shop welded to upper column): Select thickness as $(d_l - d_u)/2 - \frac{1}{16}$ in. Select weld size B from AISC Specification; $\leq\frac{5}{16}$ in. preferred. Select weld length L_B to develop bearing strength of the filler but not less than $(L/2 + 1\frac{1}{2}$ in.). Select filler width as the larger of the flange plate width + $2N$ and the upper column flange width + $2M$, rounded to the next higher $\frac{1}{4}$ in. increment. Filler length as L_B, subject to Note 3.

Note 3:
If fill length, based on L_B, is excessive, place weld of size B across end of fill and reduce L_B by one-half of such additional weld length, but not to less than $(L/2 + 1\frac{1}{2})$. Omit return welds in Case V-B.

Table 14–3 (continued)
Typical Column Splices
Case V:
All-welded flange-plated column splices between columns with depth d_u nominally two inches less than depth d_l.



CASE V–A



CASE V–B

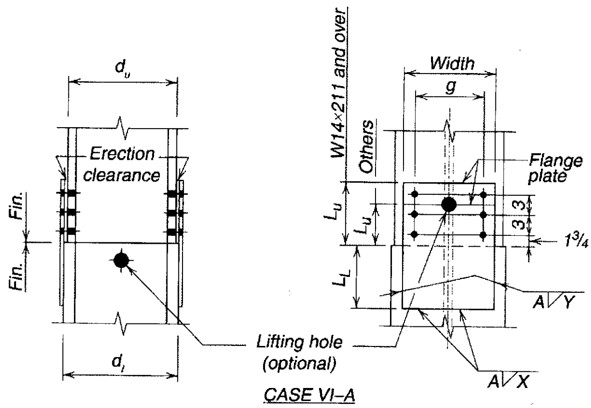
Table 14–3 (continued)
Typical Column Splices
Case VI:
Combination bolted and welded column splices between columns with depths d_u and d_l nominally the same.

Column Size	Flange Plate				Bolts		Welds		
	Width	Thk.	Length		No. of Rows	Gage g	Size A	Length	
			L_U	L_L				X	Y
W14×455 & over	14	5/8	9 1/4	9	3	11 1/2	1/2	5	7
311 to 426	12	5/8	9 1/4	8	3	9 1/2	1/2	4	6
211 to 283	12	1/2	9 1/4	8	3	9 1/2	3/8	4	6
90 to 193	12	3/8	6 1/4	8	2	9 1/2	5/16	4	6
61 to 82	8	3/8	6 1/4	8	2	5 1/2	5/16	3	6
43 to 53	6	5/16	6 1/4	7	2	3 1/2	1/4	2	5
W12×120 to 336	8	1/2	6 1/4	8	2	5 1/2	3/8	3	6
53 to 106	8	3/8	6 1/4	8	2	5 1/2	5/16	3	6
40 to 50	6	5/16	6 1/4	7	2	3 1/2	1/4	2	5
W10×49 to 112	8	3/8	6 1/4	8	2	5 1/2	5/16	3	6
33 to 45	6	5/16	6 1/4	7	2	3 1/2	1/4	2	5
W8×31 to 67	6	3/8	6 1/4	7	2	3 1/2	5/16	2	5
24 & 28	5	5/16	6 1/4	6	2	3 1/2	1/4	2	4

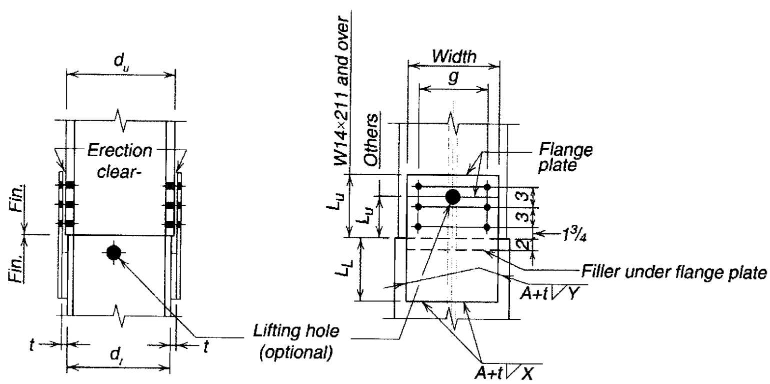
Gages shown may be modified if necessary to accommodate fittings elsewhere on the columns.

Case VI-A: $d_l = (d_u + 1/4$ in.) to $(d_u + 5/8$ in.)	Flange plates: Select flange plate width, bolts, gage and length L_U for upper column; select flange plate thickness, weld size A, weld lengths X and Y, and length L_L for lower column. Total flange plate length is $L_U + L_L$ (see table above). Fillers: None. Shims: Furnish sufficient strip shims 2½×⅛ to obtain 0 to 1/16-in. clearance on each side.
Case VI-B: $d_l = (d_u - 1/4$ in.) to $(d_u + 1/8$ in.)	Flange plates: Same as Case VI-A, except use weld size $A + t$ on lower column. Fillers (shop welded to lower column under flange plate): Select thickness t as ⅛-in. for for $d_l = d_u$ and $d_l = (d_u + 1/8$ in.) or as 3/16-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 $L_L - 2$ in. Shims: Same as Case VI-A.
Case VI-C: $d_l = (d_u + 3/4$ in.) and over	Flange plates: Same as Case VI-A. Fillers (shop welded to upper column): Select thickness t as $(d_l - d_u) / 2$ minus ⅛-in. or 3/16-in., whichever results in ⅛-in. multiples of fill thickness. Select weld size B as minimum size from AISC Specification Section J2. Select weld length as $L_U - 1/4$ in. Select filler width as flange plate width and filler length as $L_U - 1/4$-in. Shims: Same as Case VI-A.

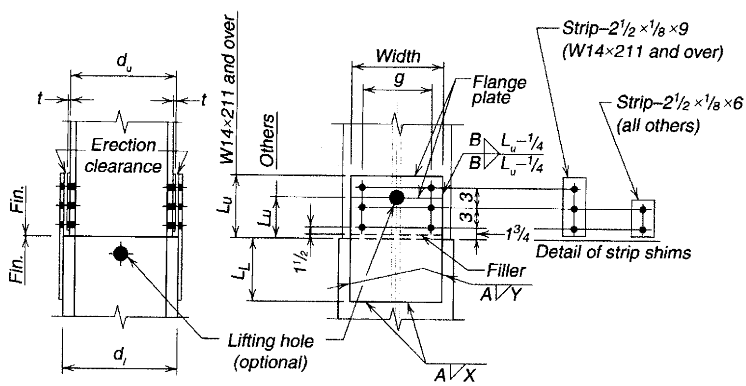
Table 14–3 (continued)
Typical Column Splices
Case VI:
Combination bolted and welded column splices between columns with depths d_u and d_l nominally the same.



CASE VI–A



CASE VI–B



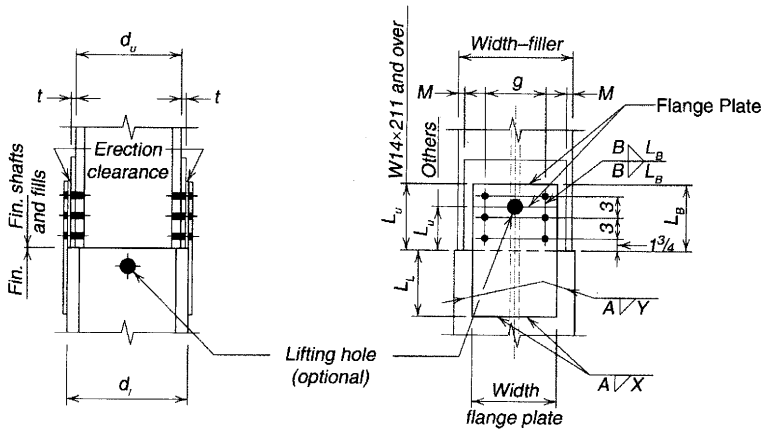
CASE VI–C

Table 14–3 (continued)
Typical Column Splices
Case VII:
Combination bolted and welded flange-plated column splices between columns with depth d_u nominally two inches less than depth d_l. Fillers developed for bearing.

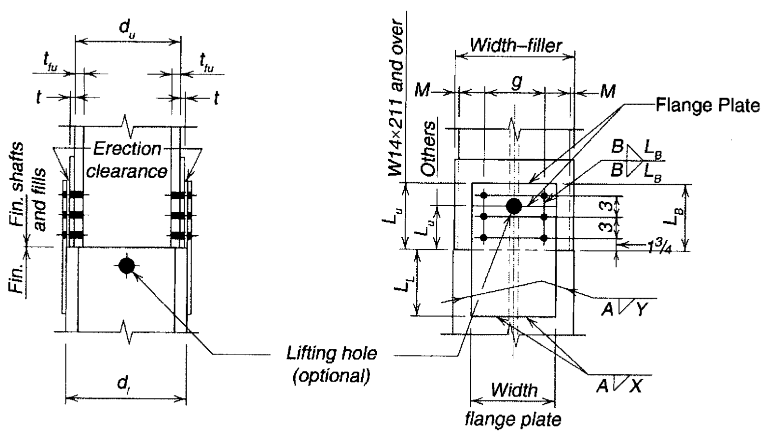
Case VII-A: Fillers of width less than upper column flange width.	Flange plates: Same as Case VI-A. Fillers (shop welded to upper column): Select filler thickness t as $(d_l - d_u)/2$ minus $\frac{1}{8}$-in. or $\frac{3}{16}$-in., whichever results in $\frac{1}{8}$-in. multiples of filler thickness. Select weld size B from AISC Specification; $\leq\frac{5}{16}$-in. preferred. Select weld length L_B to develop bearing strength of filler. Select filler width not less than flange plate width but not greater than upper column flange width $-2M$ (see Case IV). Select filler length as L_B, subject to Note 4.
Case VII-B: Filler of width greater than upper column flange width. Use Case VII-B only when fillers must be widened to provide additional bearing area.	Flange plates: Same as Case VI-A. Fillers (shop welded to upper columns): Same as Case VII-A except select filler width as upper column flange width + $2M$ (see Case IV) rounded to the next larger $\frac{1}{2}$-in. increment.

Note 4:
If fill length based on L_B is excessive, place weld of size B across end of fill and reduce L_B by one-half of such additional weld length, but not less than L_U. Omit return welds, Case VII-B.

Table 14–3 (continued)
Typical Column Splices
Case VII:
Combination bolted and welded flange-plated column splices between columns with depth d_u nominally two inches less than depth d_l. Fillers developed for bearing.



CASE VII–A



CASE VII–B

Table 14–3 (continued)
Typical Column Splices
Case VIII:
Directly welded flange column splices between columns with depths d_u and d_l nominally the same.

These types of splices exhibit versatility. The flanges may be partial-joint-penetration welded as in Cases VIIIA and VIIIB, or complete-joint-penetration welded as in Cases VIIIC, VIIID, and VIIIE The webs may be spliced using the channel(s) as shown in Cases VIIIA, VIIIB, VIIIC, and VIIID, or complete-joint-penetration welded as shown in Case VIIIE. The use of a channel or channels at the web splice provides a higher degree of restraint during the erection phase than does a plate or plates. The use of partial-joint-penetration flange welds provide greater stability during the erection phase than do complete-joint-penetration welds.

The adequacy of any splice arrangement must be confirmed by the user. This is especially true in regions where high winds are prevalent or when the concentrated weight of the fabricated column is significantly off its centerline.When using partial-joint-penetration flange welds, a land width of ¼-in. or greater should be used. The weld sizes are based on the thickness of the thinner column flange, regardless of whether it is the upper or lower column.

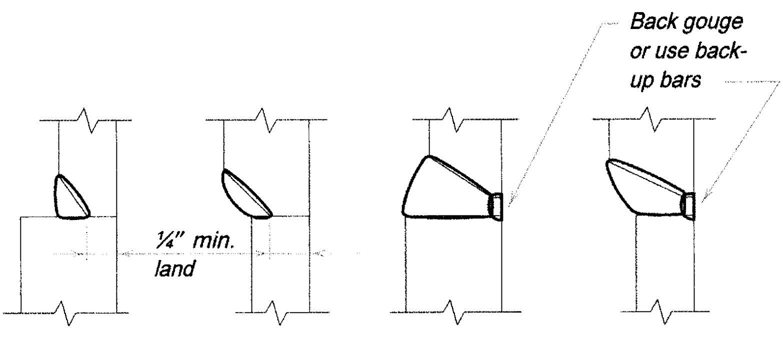
When column flange thicknesses are less than ½-in. it may be more efficient to use flange splice plates as shown in previous cases.

See the table below for minimum effective weld sizes for partial-penetration groove welds.

Partial Penetration Groove Width

[a]Thickness of Column Material T_u	Minimum Effective Weld Size E
[b]Over ½ to ¾, incl.	¼
Over ¾ to 1½, incl.	5⁄16
Over 1½ to 2¼, incl.	⅜
Over 2¼ to 6, incl.	½
Over 6	⅝

[a]Thickness of thinner part joined.
[b]For less than ½, use splice plates.



(a) Partial-joint-penetration groove welds

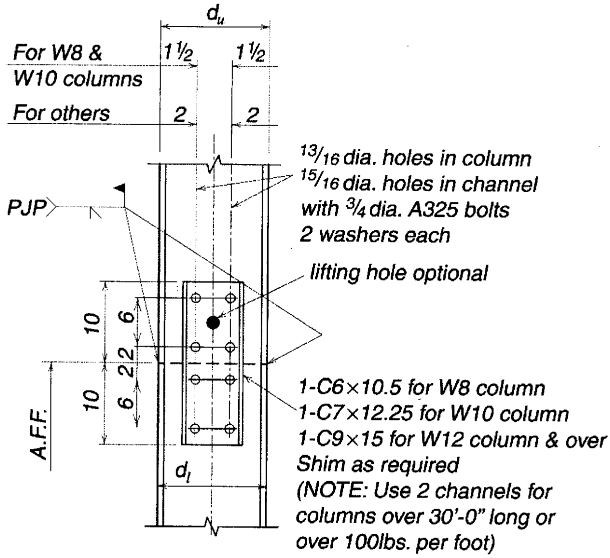
(b) Complete-joint-penetration groove welds

Table 14–3 (continued)

Typical Column Splices

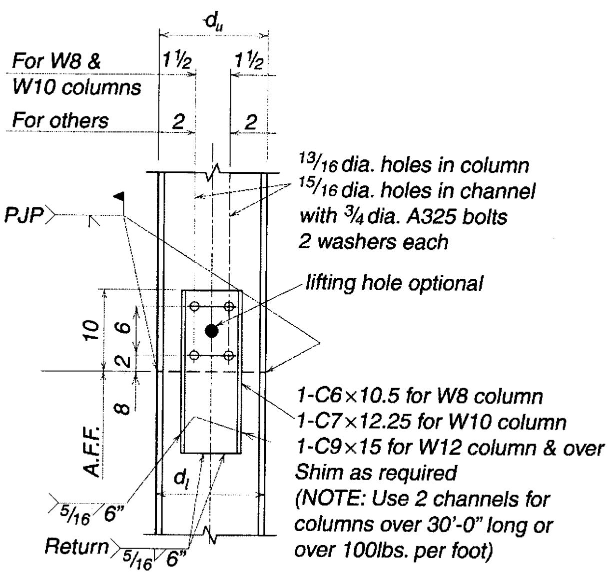
Case VIII:

Directly welded flange column splices between columns with depths d_u and d_l nominally the same.



CASE VIII A

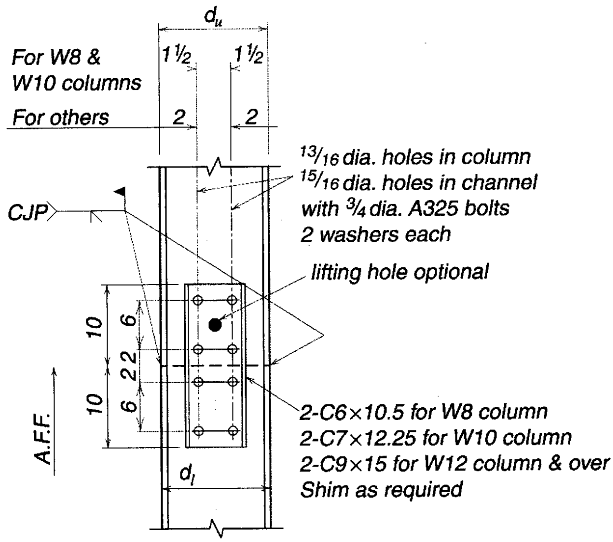
All-bolted web splice, partial-joint-penetration flange welds



CASE VIII B

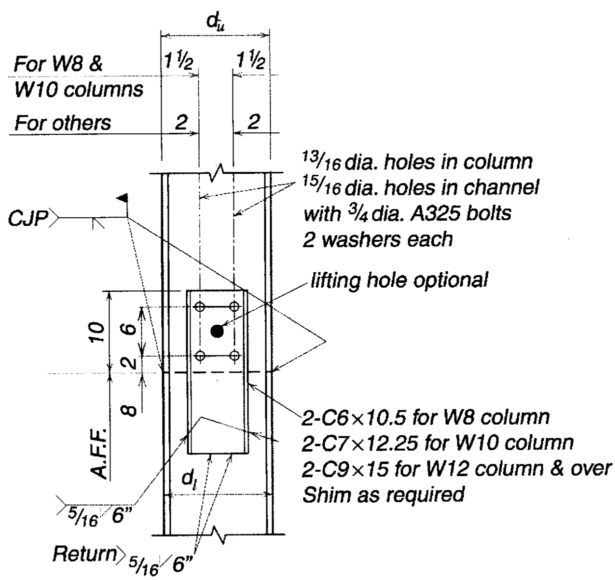
Combination bolted and welded web splice, partial-joint-penetration flange welds

Table 14–3 (continued)
Typical Column Splices
Case VIII:
Directly welded flange column splices between columns with depths d_u and d_l nominally the same.



CASE VIII C

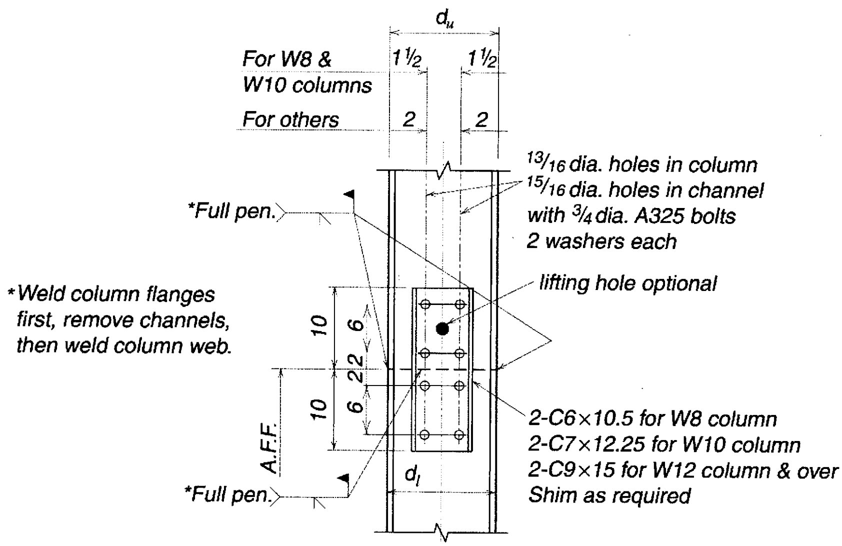
All-bolted web splice, complete-joint-penetration flange welds



CASE VIII D

Combination bolted and welded web splice, complete-joint-penetration flange welds

Table 14–3 (continued)
Typical Column Splices
Case VIII:
Directly welded flange column splices between columns with depths d_u and d_l nominally the same.



CASE VIII E

web splice, complete-joint-penetration flange and web welds

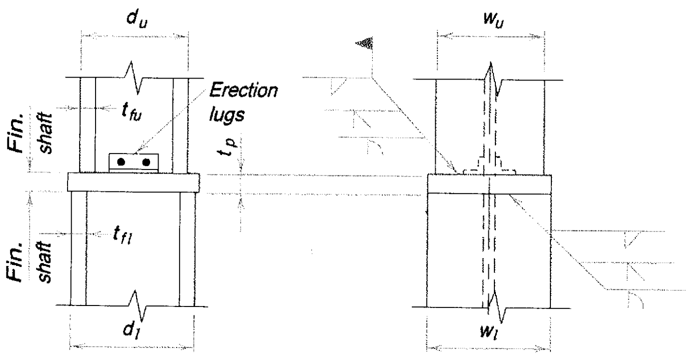
Table 14–3 (continued)
Typical Column Splices
Case IX:
Butt-plated column splices between columns with depth d_u nominally two inches less than depth d_l.

Butt plate: Select a butt plate thickness of 1½-in. for W8 over W10 columns and 2 in. for all other combinations. Select butt plate width and length not less than w_l and d_l assuming the lower is the larger column shaft.

Weld: Select weld to upper column based on the thicker of t_{fu} and t_p. Select weld to lower column based on the thicker of t_{fl} and t_p. The edge preparation required by the groove weld is usually performed on the column shafts. However, special cases such as when the butt plate must be field welded to the lower column require special consideration.

Erection: clip angles, such as those shown in the sketch below, help to locate and stabilize the upper column during the erection phase.

Table 14–3 (continued)
Typical Column Splices
Case IX:
Butt-plated column splices between columns with depth d_u nominally two inches less than depth d_l.

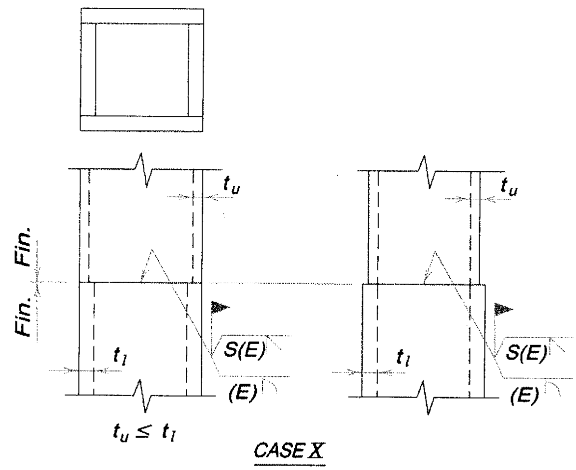


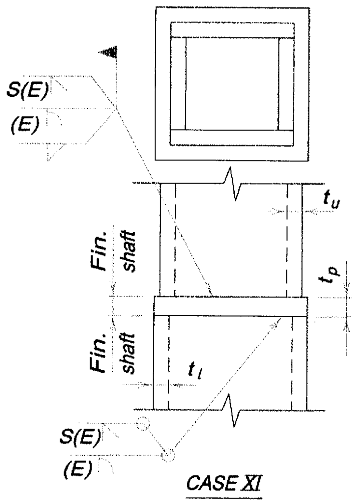
CASE IX

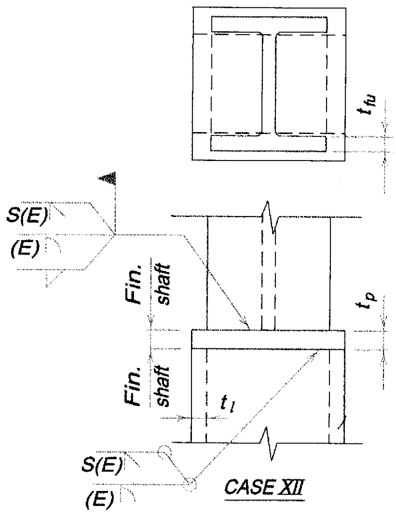
Table 14–3 (continued)
Typical Column Splices
Cases X, XI, XII
Special column splices.

Case X: Directly welded splice between tubular and/or box-shaped columns.	Welds may be either partial-joint- or complete-joint-penetration. The strength of partial-joint-penetration welds is a function of the column wall thickness and appropriate guidelines for minimum land width and effective weld size must be observed. This type of splice usually requires lifting and alignment devices. For lifting devices see Figure 11–21. For alignment devices see Figure 11–22.
Case XI: Butt-plated splices between tubular and/or box-shaped columns.	The butt-plate thickness is selected based on the AISC Specification. Welds may be either partial- or complete-penetration-groove welds, or, if adequate space is provided, fillet welds may be used. Weld strength is based on the thickness of connected material. See comments under Case X above regarding lifting and alignment devices.
Case XII: Butt-plated column splices between W-shape columns and tubular or box-shaped columns.	See comments under Case XI above.

Table 14–3 (continued)
Typical Column Splices
Cases X, XI, XII
Special column splices.







PART 15

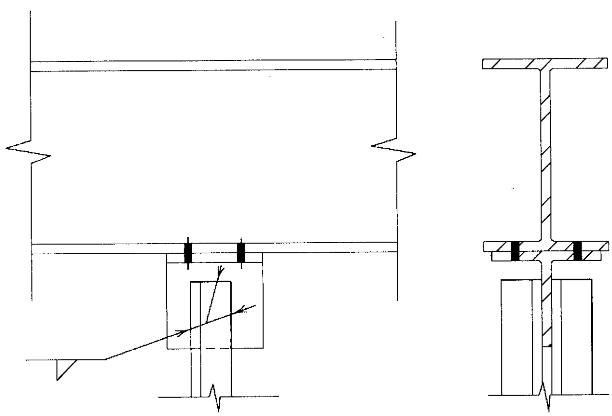
DESIGN OF HANGER CONNECTIONS, BRACKET PLATES, AND CRANE-RAIL CONNECTIONS

SCOPE

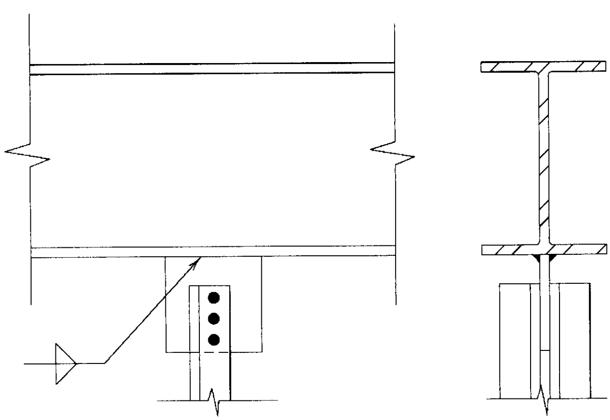
The specification requirements and other design considerations summarized in this Part apply to the design of hanger connections, bracket plates, and crane-rail connections. For the design of similar connections for HSS and pipe, see the AISC Specification Chapter K.

HANGER CONNECTIONS

Hanger connections, illustrated in Figure 15–1, are usually made with a plate, tee, angle, or pair of angles. The available strength of a hanger connection is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). In all cases, the available strength, ϕR_n or R_n/Ω, must exceed the required strength, R_u or R_a.



(a) Tee hanger



(b) Plate hanger

Figure 15–1. Typical hanger connections.

BRACKET PLATES

A bracket plate, illustrated in Figure 15–2, acts as a cantilevered beam. The available strength of a bracket plate is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). Additionally, special checks for flexural yielding, flexural rupture, and local buckling must considered, as follows.

For flexural yielding, the available strength, ϕM_n *or* M_n/Ω, of the bracket plate is determined with

$$M_n = F_y Z$$

$$\phi = 0.90 \qquad \Omega = 1.67$$

where Z is the gross plastic section modulus of the bracket plate. Additionally, triangular-shaped bracket plates should be checked for flexural yielding on the free edge (Salmon and Johnson, 1996). In lieu of a more detailed analysis, the load on the bracket plate can be limited by the available strength, ϕP_n *or* P_n/Ω, with

$$P_n = F_y zbt$$

$$\phi = 0.90 \qquad \Omega = 1.67$$

where

$$z = 1.39 - 2.2\left(\frac{b}{a}\right) + 1.27\left(\frac{b}{a}\right)^2 - 0.25\left(\frac{b}{a}\right)^3$$

b = width of bracket plate as shown in Figure 15–2, in.
a = depth of bracket plate as shown in Figure 15–2, in.
t = thickness of bracket plate, in.

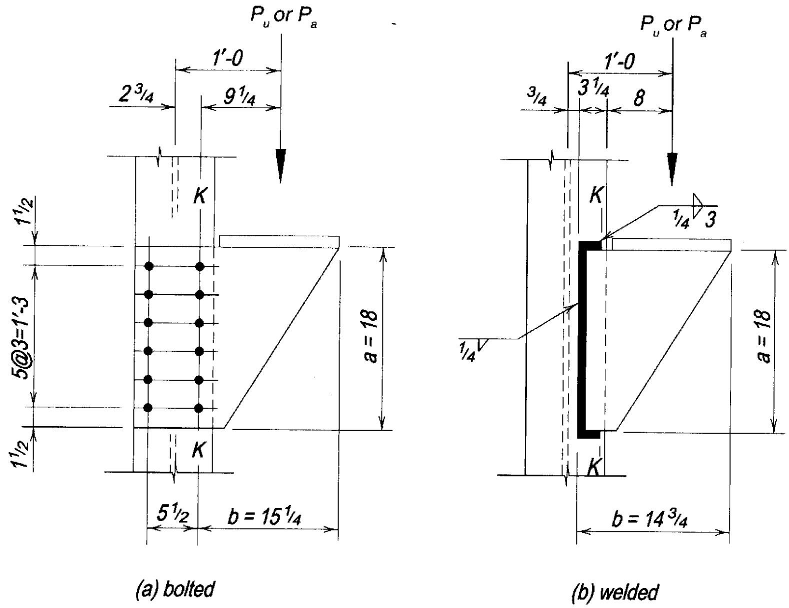


Figure 15–2. Bracket-plate connections.

For flexural rupture, the available strength, ϕM_n or M_n/Ω, of the bracket plate is determined with

$$M_n = F_u Z_{net}$$

$$\phi = 0.75 \qquad \Omega = 2.00$$

where Z_{net} is the net plastic section modulus of the bracket plate. Values of Z_{net} are given in Table 15–2 for various bolt hole diameters and numbers of fasteners at 3-in. spacing.

Local buckling can be prevented in bracket plates if the following width-thickness ratios are satisfied, provided the centroid of the applied load is approximately 0.6b from the line of support (line K in Figure 15–2a) and lateral movement of the outstanding portion of the bracket plate is prevented (Salmon and Johnson, 1996)

$$\frac{b}{t} \le \frac{250}{\sqrt{F_y}} \quad \text{for } 0.5 < \frac{b}{a} \le 1.0$$

$$\frac{b}{t} \le \frac{250}{\sqrt{F_y}}\left(\frac{b}{a}\right) \quad \text{for } 1.0 < \frac{b}{a} \le 2.0.$$

CRANE-RAIL CONNECTIONS

Bolted Splices

It is desirable to use properly installed and maintained bolted splice bars in crane-rail connections rather than welded splice bars, which are frequently subject to failure in service.

Standard rail drilling and joint-bar punching, as furnished by manufacturers of light standard rails for track work, include round holes in rail ends and slotted holes in joint bars to receive standard oval-neck track bolts. Holes in rails are oversized and punching in joint bars is spaced to allow $^1/_{16}$-in. to $^1/_8$-in. clearance between rail ends (see manufacturers' catalogs for spacing and dimensions of holes and slots). Although this construction is satisfactory for track and light crane service, its use in general crane service may lead to high maintenance and joint failure. Welded splices are therefore preferable.

For best service in bolted splices, it is recommended that tight joints be required for all rails for crane service. This will require rail ends to be finished, and the special rail drilling and joint-bar punching tabulated below. Special rail drilling is accepted by some mills, or rails may be ordered blank for shop drilling. End finishing of standard rails can be done at the mill. However, light rails often must be end-finished in the shop or ground at the site prior to erection. In the crane rail range from 104 to 175 lbs per yard, rails and joint bars are manufactured to obtain a tight fit and no further special end finishing, drilling, or punching is required. Because of cumulative tolerance variations in holes, bolt diameters, and rail ends, a slight gap may sometimes occur. It may sometimes be necessary to ream holes through joined bar and rail to permit entry of bolts.

Joint bars for crane service are provided in various sections to match the rails. Joint bars for light and standard rails can be purchased blank for special shop punching to obtain tight joints. See manufacturer data for dimensions, material specifications, and the identification necessary to match the crane-rail section.

Joint-bar bolts, as distinguished from oval-neck track bolts, have straight shanks to the head and are manufactured to ASTM A449 specifications. Nuts are manufactured to ASTM A563 grade B specifications. Alternatively, ASTM A325 bolts and compatible ASTM A563 nuts can be used. Bolt assembly includes an alloy steel spring washer, furnished to American Railway Engineering and Maintenance of Way Association (AREMA) specifications. After installation, bolts should be retightened within 30 days and every three months thereafter.

Welded Splices

When welded splices are specified, consult the manufacturer for recommended rail-end preparation, welding procedure, and method of ordering. Although the joint continuity made possible by this method of splicing is desirable, the careful control required in all stages of the welding operation may be difficult to meet during crane-rail installation. Rails should not be attached to structural supports by welding. Rails with holes for joint bar bolts should not be used in making welded splices.

Hook Bolt Fastenings

Hook bolts (Figure 15–3) are used primarily with light rails when attached to beams that are too narrow for clamps. Rail adjustment to $\pm^1/_2$ in. is inherent in the threaded shank. Hook bolts are paired alternately 3 to 4 in. apart, spaced at about 24 in. on center. The special rail drilling required must be done in the fabricator's shop. Hook bolts are not recommended for use with heavy-duty cycle cranes (Crane Manufacturers Association of America (CMAA) Classes, D, E, and F). It is generally recommended that hook bolts should not be used in runway systems that are longer than 500 ft because the bolts do not allow for longitudinal movement of the rail.

Rail Clip Fastenings

Rail clips are forged or cast devices that are shaped to match specific rail profiles. They are usually bolted to the runway girder flange with one bolt or are sometimes welded. Rail clips have been used satisfactorily with all classes of cranes. However, one drawback is that when

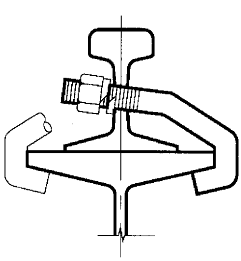


Figure 15–3. Hook bolts.

a single bolt is used, the clip can rotate in response to rail longitudinal movement. This clip rotation can cause cam action that might force the rail out of alignment. Because of this limitation, rail clips should only be used in crane systems subject to infrequent use, and for runways less than 500 ft in length.

Rail Clamp Fastenings

Rail clamps are a common method of attachment for heavy-duty cycle cranes. Rail clamps are detailed to provide two types: tight and floating (see Figure 15–4). Each clamp consists of two plates: an upper clamp plate and a lower filler plate. Dimensions shown are suggested. See manufacturers' catalogs for recommended gages, bolt sizes, and detail dimensions not shown.

The lower plate is flat and nominally matches the height of the toe of the rail flange. The upper plate covers the lower plate and extends over the top of the lower rail flange. In the tight clamp, the upper plate is detailed to fit tightly to the lower tail flange top, thus "clamping" it tightly in place when the fasteners are tightened. In the past, the tight clamp had been illustrated with the filler plates fitted tightly against the rail flange toe. This tight fit-up was rarely achieved in practice and is not considered to be necessary to achieve a tight type clamp. In the floating type clamp, the pieces are detailed to provide a clearance both alongside the rail flange toe and below the upper plate. The floating type does not, in reality, clamp the rail but merely holds the rail within the limits of the clamp clearances. High strength bolts are recommended for both clamp types. Both types should be spaced 3 ft or less apart.

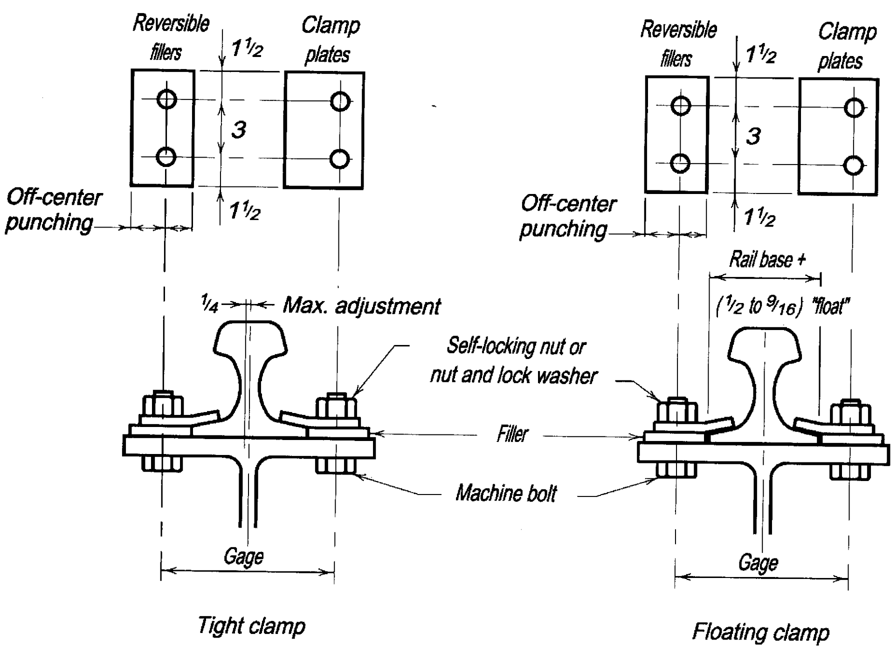


Figure 15–4. Rail clamps.

Patented Rail Clip Fastenings

Each manufacturer's literature presents in detail the desirable aspects of the various designs. In general, patented rail clips are easy to install due to their range of adjustment and provide both limitation of lateral movement and allowance for longitudinal movement. Patented rail clips should be considered as a viable alternative to conventional hook bolts, clips, or clamps. Because of their desirable characteristics, patented rail clips can be used without restriction except as limited by the specific manufacturer's recommendations. Installations using patented rail clips sometimes incorporate pads beneath the rail. When this is done, the lateral float of the rail should be limited as in the case of the tight rail clamps.

DESIGN TABLES

Table 15–1. Preliminary Hanger Connection Selection Table

Values are given for the available tensile strength per in. of fitting length in flexural yielding of a tee fitting flange with $F_y = 36$ ksi and $F_y = 50$ ksi. Table 15–1 can be used to select a trial fitting once the number and size of bolts required is known. The number of bolts required must be selected such that the available tensile strength of one bolt, ϕr_n or r_n/Ω, exceeds the required tensile force per bolt, r_{ut} or r_{at}.

In this table, it is assumed that equal critical moments exist at the face of the tee stem and at the bolt line. From AISC Specification Section F9, the available flexural yielding strength of the tee flange, $\phi_b M_n$ or M_n/Ω_b, is determined with

$$M_n = M_p = F_y Z$$

$$\phi_b = 0.90 \qquad \Omega_b = 1.67$$

In the above equation, the plastic section modulus Z_x per unit length of the tee flange is

$$Z_x = \frac{t^2}{4}$$

where t is the thickness of the angle or tee flange, in. Thus, for a unit length of the tee flange the available flexural strength, $\phi_b M_n$ or M_n/Ω_b, is determined with

$$M_n = \frac{F_y t^2}{4}$$

$$\phi_b = 0.90 \qquad \Omega_b = 1.67$$

and the tensile force on the fitting, $2r_{ut}$ or $2r_{at}$, must be such that

LRFD	ASD
$2r_{ut} \le \dfrac{0.9F_y t^2}{b}$	$2r_{at} \le \dfrac{F_y t^2}{1.67b}$

where b is the distance from bolt centerline to face of the tee stem or center of angle leg, in.

Table 15–2. Net Plastic Section Modulus, Z_{net}

Values of the net plastic section modulus Z_{net} are given in Table 15–2 for various hole diameters and numbers of fasteners spaced 3 in. on center, the usual spacing for these connections.

FORGED STEEL STRUCTURAL HARDWARE

Table 15–3. Dimensions and Weights of Clevises

Dimensions, weights, and available strengths of clevises are listed in Table 15–3.

Table 15–4. Clevis Numbers Compatible with Various Rods and Pins

Compatibility of clevises with various rods and pins is given in Table 15–4.

Table 15–5. Dimensions and Weights of Turnbuckles

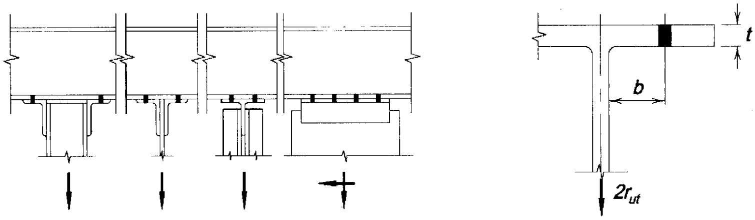
Dimensions, weights, and available strengths of turnbuckles are listed in Table 15–5.

PART 15 REFERENCES

Salmon, C.G. and J.E. Johnson, 1996, *Steel Structures: Design and Behavior*, 4th Edition, Harper Collins, New York, NY.

F_y = 36 ksi

Table 15–1a
Preliminary Hanger Connection Selection Table
Available tensile strength, kips per linear in., limited by flexural yielding of the flange

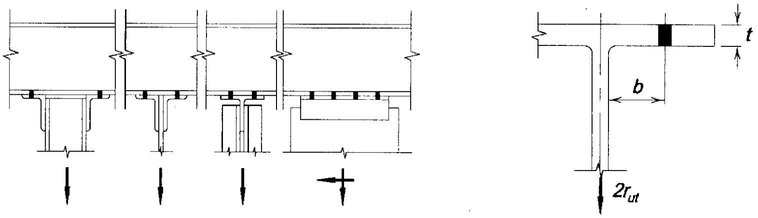


t, in.	b, in.									
	1		$1^1/_4$		$1^1/_2$		$1^3/_4$		2	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
5/16	2.11	3.16	1.68	2.53	1.40	2.11	1.20	1.81	1.05	1.58
3/8	3.03	4.56	2.43	3.65	2.02	3.04	1.73	2.60	1.52	2.28
7/16	4.13	6.20	3.30	4.96	2.75	4.13	2.36	3.54	2.06	3.10
1/2	5.39	8.10	4.31	6.48	3.59	5.40	3.08	4.63	2.69	4.05
9/16	6.82	10.3	5.46	8.20	4.55	6.83	3.90	5.86	3.41	5.13
5/8	8.42	12.7	6.74	10.1	5.61	8.44	4.81	7.23	4.21	6.33
11/16	10.2	15.3	8.15	12.3	6.79	10.2	5.82	8.75	5.09	7.66
3/4	12.1	18.2	9.70	14.6	8.08	12.2	6.93	10.4	6.06	9.11
13/16	14.2	21.4	11.4	17.1	9.49	14.3	8.13	12.2	7.12	10.7
7/8	16.5	24.8	13.2	19.8	11.0	16.5	9.43	14.2	8.25	12.4
15/16	18.9	28.5	15.2	22.8	12.6	19.0	10.8	16.3	9.47	14.2
1	21.6	32.4	17.2	25.9	14.4	21.6	12.3	18.5	10.8	16.2
$1^1/_{16}$	24.3	36.6	19.5	29.3	16.2	24.4	13.9	20.9	12.2	18.3
$1^1/_8$	27.3	41.0	21.8	32.8	18.2	27.3	15.6	23.4	13.6	20.5
$1^3/_{16}$	30.4	45.7	24.3	36.6	20.3	30.5	17.4	26.1	15.2	22.8
$1^1/_4$	33.7	50.6	26.9	40.5	22.5	33.8	19.2	28.9	16.8	25.3

t, in.	$2^1/_4$		$2^1/_2$		$2^3/_4$		3		$3^1/_4$	
5/16	0.936	1.41	0.842	1.27	0.766	1.15	0.702	1.05	0.648	0.974
3/8	1.35	2.02	1.21	1.82	1.10	1.66	1.01	1.52	0.933	1.40
7/16	1.83	2.76	1.65	2.48	1.50	2.26	1.38	2.07	1.27	1.91
1/2	2.40	3.60	2.16	3.24	1.96	2.95	1.80	2.70	1.66	2.49
9/16	3.03	4.56	2.73	4.10	2.48	3.73	2.27	3.42	2.10	3.15
5/8	3.74	5.63	3.37	5.06	3.06	4.60	2.81	4.22	2.59	3.89
11/16	4.53	6.81	4.08	6.13	3.71	5.57	3.40	5.10	3.14	4.71
3/4	5.39	8.10	4.85	7.29	4.41	6.63	4.04	6.08	3.73	5.61
13/16	6.32	9.51	5.69	8.56	5.17	7.78	4.74	7.13	4.38	6.58
7/8	7.34	11.0	6.60	9.92	6.00	9.02	5.50	8.27	5.08	7.63
15/16	8.42	12.7	7.58	11.4	6.89	10.4	6.32	9.49	5.83	8.76
1	9.58	14.4	8.62	13.0	7.84	11.8	7.19	10.8	6.63	9.97
$1^1/_{16}$	10.8	16.3	9.73	14.6	8.85	13.3	8.11	12.2	7.49	11.3
$1^1/_8$	12.1	18.2	10.9	16.4	9.92	14.9	9.09	13.7	8.39	12.6
$1^3/_{16}$	13.5	20.3	12.2	18.3	11.1	16.6	10.1	15.2	9.35	14.1
$1^1/_4$	15.0	22.5	13.5	20.3	12.2	18.4	11.2	16.9	10.4	15.6

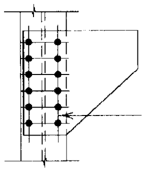
F_y = 50 ksi

Table 15–1b
Preliminary Hanger Connection Selection Table
Available tensile strength, kips per linear in., limited by flexural yielding of the flange



t, in.	b, in.									
	1		1 1/4		1 1/2		1 3/4		2	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
5/16	2.92	4.39	2.34	3.52	1.95	2.93	1.67	2.51	1.46	2.20
3/8	4.21	6.33	3.37	5.06	2.81	4.22	2.41	3.62	2.11	3.16
7/16	5.73	8.61	4.58	6.89	3.82	5.74	3.27	4.92	2.87	4.31
1/2	7.49	11.3	5.99	9.00	4.99	7.50	4.28	6.43	3.74	5.63
9/16	9.47	14.2	7.58	11.4	6.32	9.49	5.41	8.14	4.74	7.12
5/8	11.7	17.6	9.36	14.1	7.80	11.7	6.68	10.0	5.85	8.79
11/16	14.2	21.3	11.3	17.0	9.43	14.2	8.09	12.2	7.08	10.6
3/4	16.8	25.3	13.5	20.3	11.2	16.9	9.62	14.5	8.42	12.7
13/16	19.8	29.7	15.8	23.8	13.2	19.8	11.3	17.0	9.88	14.9
7/8	22.9	34.5	18.3	27.6	15.3	23.0	13.1	19.7	11.5	17.2
15/16	26.3	39.6	21.1	31.6	17.5	26.4	15.0	22.6	13.2	19.8
1	29.9	45.0	24.0	36.0	20.0	30.0	17.1	25.7	15.0	22.5
1 1/16	33.8	50.8	27.0	40.6	22.5	33.9	19.3	29.0	16.9	25.4
1 1/8	37.9	57.0	30.3	45.6	25.3	38.0	21.7	32.5	18.9	28.5
1 3/16	42.2	63.5	33.8	50.8	28.1	42.3	24.1	36.3	21.1	31.7
1 1/4	46.8	70.3	37.4	56.3	31.2	46.9	26.7	40.2	23.4	35.2
	2 1/4		2 1/2		2 3/4		3		3 1/4	
5/16	1.30	1.95	1.17	1.76	1.06	1.60	0.975	1.46	0.900	1.35
3/8	1.87	2.81	1.68	2.53	1.53	2.30	1.40	2.11	1.30	1.95
7/16	2.55	3.83	2.29	3.45	2.08	3.13	1.91	2.87	1.76	2.65
1/2	3.33	5.00	2.99	4.50	2.72	4.09	2.50	3.75	2.30	3.46
9/16	4.21	6.33	3.79	5.70	3.44	5.18	3.16	4.75	2.91	4.38
5/8	5.20	7.81	4.68	7.03	4.25	6.39	3.90	5.86	3.60	5.41
11/16	6.29	9.45	5.66	8.51	5.15	7.73	4.72	7.09	4.35	6.54
3/4	7.49	11.3	6.74	10.1	6.12	9.20	5.61	8.44	5.18	7.79
13/16	8.78	13.2	7.91	11.9	7.19	10.8	6.59	9.90	6.08	9.14
7/8	10.2	15.3	9.17	13.8	8.34	12.5	7.64	11.5	7.05	10.6
15/16	11.7	17.6	10.5	15.8	9.57	14.4	8.77	13.2	8.10	12.2
1	13.3	20.0	12.0	18.0	10.9	16.4	9.98	15.0	9.21	13.8
1 1/16	15.0	22.6	13.5	20.3	12.3	18.5	11.3	16.9	10.4	15.6
1 1/8	16.8	25.3	15.2	22.8	13.8	20.7	12.6	19.0	11.7	17.5
1 3/16	18.8	28.2	16.9	25.4	15.4	23.1	14.1	21.2	13.0	19.5
1 1/4	20.8	31.3	18.7	28.1	17.0	25.6	15.6	23.4	14.4	21.6

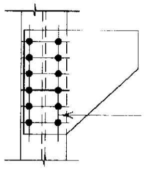
Table 15–2
Net Plastic Section Modulus Z_{net}, in.^{3}



# Bolts in One Vertical Row n	Bracket Plate Depth d, in.	Nominal Bolt Diameter d_b, in.							
		3/4					7/8		
		Bracket Plate Thickness t, in.							
		1/4	3/8	1/2	5/8	3/4	3/8	1/2	5/8
2	6	1.59	2.39	3.19	3.98	4.78	2.25	3.00	3.75
3	9	3.70	5.55	7.40	9.26	11.1	5.25	7.00	8.75
4	12	6.38	9.56	12.8	15.9	19.1	9.00	12.0	15.0
5	15	10.1	15.1	20.2	25.2	30.2	14.3	19.0	23.8
6	18	14.3	21.5	28.7	35.9	43.0	20.3	27.0	33.8
7	21	19.6	29.5	39.3	49.1	58.9	27.8	37.0	46.3
8	24	25.5	38.3	51.0	63.8	76.5	36.0	48.0	60.0
9	27	32.4	48.6	64.8	81.0	97.2	45.8	61.0	76.3
10	30	39.8	59.8	79.7	99.6	120	56.3	75.0	93.8
12	36	57.4	86.1	115	143	172	81.0	108	135
14	42	78.1	117	156	195	234	110	147	184
16	48	102	153	204	255	306	144	192	240
18	54	129	194	258	323	387	182	243	304
20	60	159	239	319	398	478	225	300	375
22	66	193	289	386	482	579	272	363	454
24	72	230	344	459	574	689	324	432	540
26	78	269	404	539	673	808	380	507	634
28	84	312	469	625	781	937	441	588	735
30	90	359	538	717	896	1080	506	675	844
32	96	408	612	816	1020	1220	576	768	960
34	102	461	691	921	1150	1380	650	867	1080
36	108	516	775	1030	1290	1550	729	972	1220

Notes:
The area reduction per hole is assumed to be d_h + 1/16 in.
Bolts spaced 3 in. vertically with 1 1/2 in. edge distance at top and bottom.
Interpolate for intermediate plate thicknesses.

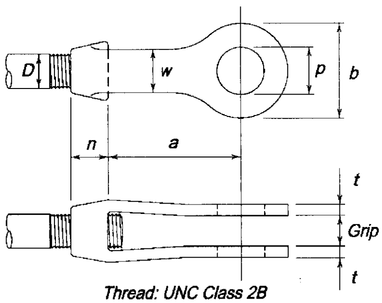
Table 15–2 (continued)
Net Plastic Section Modulus Z_{net}, in.^{3}



# Bolts in One Vertical Row n	Bracket Plate Depth d, in.	Nominal Bolt Diameter d_b, in.						
		7/8		1				
		Bracket Plate Thickness t, in.						
		3/4	7/8	1/2	5/8	3/4	7/8	1
2	6	4.50	5.25	2.81	3.52	4.22	4.92	5.63
3	9	10.5	12.3	6.59	8.24	9.89	11.5	13.2
4	12	18.0	21.0	11.3	14.1	16.9	19.7	22.5
5	15	28.5	33.3	17.8	22.3	26.8	31.2	35.7
6	18	40.5	47.3	25.3	31.6	38.0	44.3	50.6
7	21	55.5	64.8	34.7	43.4	52.1	60.8	69.4
8	24	72.0	84.0	45.0	56.3	67.5	78.8	90.0
9	27	91.5	107	57.2	71.5	85.8	100	114
10	30	113	131	70.3	87.9	105	123	141
12	36	162	189	101	127	152	177	203
14	42	221	257	138	172	207	241	276
16	48	288	336	180	225	270	315	360
18	54	365	425	228	285	342	399	456
20	60	450	525	281	352	422	492	563
22	66	545	635	340	425	510	596	681
24	72	648	756	405	506	608	709	810
26	78	761	887	475	594	713	832	951
28	84	882	1030	551	689	827	965	1100
30	90	1010	1180	633	791	949	1110	1270
32	96	1150	1340	720	900	1080	1260	1440
34	102	1300	1520	813	1020	1220	1420	1630
36	108	1460	1700	911	1140	1370	1590	1820

Notes:
The area reduction per hole is assumed to be d_h + 1/16 in.
Bolts spaced 3 in. vertically with 1 1/2 in. edge distance at top and bottom.
Interpolate for intermediate plate thicknesses.

Table 15–3
Dimensions and Weights of Clevises



Thread: UNC Class 2B

Grip = plate thickness + ¼ in.

Clevis Number	Dimensions, in.							Weight, pounds	Available Strength, kips	
	Max. *D*	Max. *p*	*b*	*n*	*a*	*w*	*t*		ASD	LRFD
2	$\frac{5}{8}$	$\frac{3}{4}$	$1\frac{7}{16}$	$\frac{5}{8}$	$3\frac{9}{16}$	$1\frac{1}{16}$	$\frac{5}{16}$ ($+\frac{1}{32}$, -0)	1	5.83	8.75
$2\frac{1}{2}$	$\frac{7}{8}$	$1\frac{1}{2}$	$2\frac{1}{2}$	1	4	$1\frac{1}{4}$	$\frac{5}{16}$ ($+\frac{1}{32}$, -0)	2.5	12.5	18.8
3	$1\frac{3}{8}$	$1\frac{3}{4}$	3	$1\frac{1}{4}$	$5\frac{1}{16}$	$1\frac{1}{2}$	$\frac{1}{2}$ ($+\frac{1}{16}$, $-\frac{1}{32}$)	4	25.0	37.5
$3\frac{1}{2}$	$1\frac{1}{2}$	2	$3\frac{1}{2}$	$1\frac{1}{2}$	6	$1\frac{3}{4}$	$\frac{1}{2}$ ($+\frac{1}{16}$, $-\frac{1}{16}$)	6	30.0	45.0
4	$1\frac{3}{4}$	$2\frac{1}{4}$	4	$1\frac{3}{4}$	$5\frac{15}{16}$	2	$\frac{1}{2}$ ($+\frac{1}{16}$, $-\frac{1}{16}$)	9	35.0	52.5
5	$2\frac{1}{8}$	$2\frac{1}{2}$	5	$2\frac{1}{4}$	7	$2\frac{1}{2}$	$\frac{5}{8}$ ($+\frac{3}{32}$, -0)	16	62.5	93.8
6	$2\frac{1}{2}$	3	6	$2\frac{3}{4}$	8	3	$\frac{3}{4}$ ($+\frac{3}{32}$, -0)	26	90.0	135
7	3	$3\frac{3}{4}$	7	3	9	$3\frac{1}{2}$	$\frac{7}{8}$ ($+\frac{1}{8}$, $-\frac{1}{16}$)	36	114	171
8	4	$4\frac{1}{4}$	8	4	$10\frac{1}{8}$	4	$1\frac{1}{2}$ ($+\frac{1}{8}$, $-\frac{1}{16}$)	90	225	338

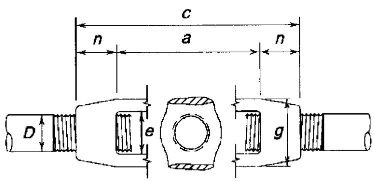
Notes:
Weights and dimensions of clevises are typical; products of all suppliers are essentially similar. User shall verify with the manufacturer that product meets available strength specifications above.
* Tabulated available strengths are based on $\phi = 0.5$, $\Omega = 3.0$. Strength at service load corresponds to a 3:1 safety factor using maximum pin diameter.

Table 15–4
Clevis Numbers Compatible with Various Rods and Pins

Dia. of Tap, in.	Diameter of Pin, in.																	
	$^1/_2$	$^5/_8$	$^3/_4$	$^7/_8$	1	$1^1/_4$	$1^1/_2$	$1^3/_4$	2	$2^1/_4$	$2^1/_2$	$2^3/_4$	3	$3^1/_4$	$3^1/_2$	$3^3/_4$	4	$4^1/_4$
$^3/_8$	2	2	2															
$^1/_2$	2	2	2															
$^5/_8$	2	2	2	$2^1/_2$	$2^1/_2$	$2^1/_2$	$2^1/_2$											
$^3/_4$			$2^1/_2$	$2^1/_2$	$2^1/_2$	$2^1/_2$	$2^1/_2$											
$^7/_8$				$2^1/_2$	$2^1/_2$	$2^1/_2$	$2^1/_2$	3										
1					3	3	3	3										
$1^1/_8$					3	3	3	3	$3^1/_2$									
$1^1/_4$					3	3	3	3	$3^1/_2$									
$1^3/_8$						3	3	$3^1/_2$	$3^1/_2$	4								
$1^1/_2$						$3^1/_2$	$3^1/_2$	4	4	5								
$1^5/_8$						4	4	4	5	5	5							
$1^3/_4$							4	5	5	5	5							
$1^7/_8$							5	5	5	5	5							
2							5	5	5	5	5	6	6					
$2^1/_8$								5	5	6	6	6	6					
$2^1/_4$									6	6	6	6	6	7	7			
$2^3/_8$									6	6	6	6	7	7	7	7		
$2^1/_2$									6	6	6	7	7	7	7	7		
$2^5/_8$											7	7	7	7	7	8		
$2^3/_4$											7	7	7	7	8	8		
$2^7/_8$											7	8	8	8	8	8	8	8
3											7	8	8	8	8	8	8	8
$3^1/_8$												8	8	8	8	8	8	8
$3^1/_4$												8	8	8	8	8	8	8
$3^3/_8$												8	8	8	8	8	8	8
$3^1/_2$													8	8	8	8	8	8
$3^5/_8$													8	8	8	8	8	
$3^3/_4$													8	8	8	8	8	
$3^7/_8$														8	8	8		
4														8	8			

Notes:
Tabular values assume that the net area of the clevis through the pin hole is greater than or equal to 125 percent of the net area of the rod, and is applicable to round rods without upset ends. For other net area ratios, the required clevis size may be calculated by referring to the dimensions tabulated in Table 15–3 and 7–18.

Table 15–5
Dimensions and Weights of Turnbuckles

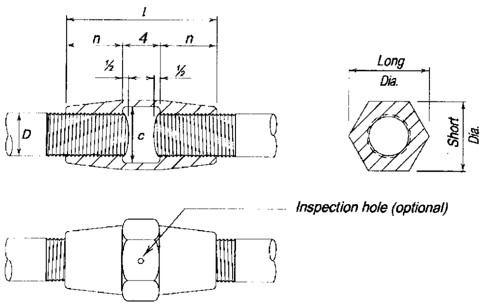


Threads: UNC and 4UN Class 2B

Diameter D, in.	Dimensions, in.					Weight (pounds) for Length a, in.						Available Strength, kips	
	a	n	c	e	g	6	9	12	18	24	26	ASD R_n^*/Ω	LRFD ϕR_n^*
3/8	6	9/16	7 1/8	9/16	1 1/32	0.42						2.00	3.00
1/2	6	25/32	7 9/16	11/16	1 5/16	0.65	0.90	1.20				3.67	5.50
5/8	6	15/16	7 7/8	13/16	1 1/2	0.98	1.35	1.58	2.43			5.83	8.75
3/4	6	1 1/16	8 1/8	15/16	1 23/32	1.45	1.84	2.35	3.06	4.25		8.67	13.0
7/8	6	1 5/16	8 5/8	1 3/32	1 7/8	1.85		3.02	4.20	5.43		12.0	18.0
1	6	1 7/16	8 7/8	1 9/32	2 1/32	2.60		4.02	4.40	6.85	10.0	15.5	23.3
1 1/8	6	1 9/16	9 1/8	1 13/32	2 9/32	4.06		4.70	6.10			19.3	29.0
1 1/4	6	1 9/16	9 1/8	1 9/16	2 17/32	4.00		6.49	7.13	11.3	13.1	25.3	38.0
1 3/8	6	1 13/16	9 5/8	1 11/16	2 3/4	6.15						29.0	43.5
1 1/2	6	1 7/8	9 3/4	1 27/32	3 1/32	6.15		9.70	9.13	16.8	19.4	35.0	52.5
1 5/8	6	2 1/2	11	1 31/32	3 9/32	9.80						40.9	61.3
1 3/4	6	2 1/2	11	2 1/8	3 9/16	9.80		15.3	16.0	19.5		47.2	70.8
1 7/8	6	2 13/16	11 5/8	2 3/8	4	14.0		15.3				62.0	93.0
2	6	2 13/16	11 5/8	2 3/8	4	14.0		15.3		27.5		62.0	93.0
2 1/4	6	3 5/16	12 5/8	2 11/16	4 5/8	19.6		30.9		43.5		80.0	120
2 1/2	6	3 3/4	13 1/2	3	5	23.3		30.9		42.4		100	150
2 3/4	6	4 3/16	14 3/8	3 1/4	5 5/8	31.5				54.0		125	188
3	6	4 5/16	14 5/8	3 5/8	6 1/8	39.5						161	242
3 1/4	6	5 7/16	16 7/8	3 7/8	6 3/4	60.5		79.5				203	305
3 1/2	6	5 7/16	16 7/8	3 7/8	6 3/4	60.5	70.0	79.5				203	305
3 3/4	6	6	18	4 5/8	8 1/2	95.0						280	420
4	6	6	18	4 5/8	8 1/2	95.0						280	420
4 1/4	9	6 3/4	22 1/2	5 1/4	9 3/4		152					390	585
4 1/2	9	6 3/4	22 1/2	5 1/4	9 3/4		152					390	585
4 3/4	9	6 3/4	22 1/2	5 1/4	9 3/4		152					390	585
5	9	7 1/2	24	6	10		200					491	737

Notes:
Weights and dimensions of turnbuckles are typical; products of all suppliers are essentially similar. Users shall verify with the manufacturer that product meets strength specifications above.
* Tabulated available strengths are based on $\phi = 0.5$, $\Omega = 3.0$.

Table 15–6
Dimensions and Weights of Sleeve Nuts

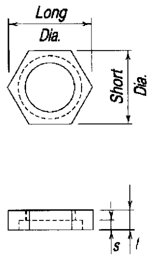


Thread: UNC and 4 UN Class 2B

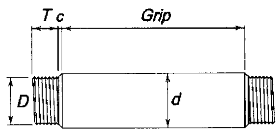
Screw Dia. *D*, in.	Dimensions, in.					Weight, pounds
	Short Dia.	Long Dia.	Length *l*	Nut *n*	Clear *c*	
3/8	11/16	25/32	4	—	—	0.27
7/16	25/32	7/8	4	—	—	0.34
1/2	7/8	1	4	—	—	0.43
9/16	15/16	1 1/16	5	—	—	0.64
5/8	1 1/16	1 7/32	5	—	—	0.93
3/4	1 1/4	1 7/16	5	—	—	1.12
7/8	1 7/16	1 5/8	7	1 7/16	1	1.75
1	1 5/8	1 13/16	7	1 7/16	1 1/8	2.46
1 1/8	1 13/16	2 1/16	7 1/2	1 5/8	1 1/4	3.10
1 1/4	2	2 1/4	7 1/2	1 5/8	1 3/8	4.04
1 3/8	2 3/16	2 1/2	8	1 7/8	1 1/2	4.97
1 1/2	2 3/8	2 11/16	8	1 7/8	1 5/8	6.16
1 5/8	2 9/16	2 15/16	8 1/2	2 1/16	1 3/4	7.36
1 3/4	2 3/4	3 1/8	8 1/2	2 1/16	1 7/8	8.87
1 7/8	2 15/16	3 5/16	9	2 5/16	2	10.4
2	3 1/8	3 1/2	9	2 5/16	2 1/8	12.2
2 1/4	3 1/2	3 15/16	9 1/2	2 1/2	2 3/8	16.2
2 1/2	3 7/8	4 3/8	10	2 3/4	2 5/8	21.1
2 3/4	4 1/4	4 13/16	10 1/2	2 15/16	2 7/8	26.7
3	4 5/8	5 1/4	11	3 3/16	3 1/8	33.2
3 1/4	5	5 5/8	11 1/2	3 3/8	3 3/8	40.6
3 1/2	5 3/8	6	12	3 5/8	3 5/8	49.1
3 3/4	5 3/4	6 3/8	12 1/2	3 13/16	3 7/8	58.6
4	6 1/8	6 7/8	13	4 1/16	4 1/8	69.2
4 1/4	6 1/2	7 1/2	13 1/2	4 3/4	4 3/8	75.0
4 1/2	6 7/8	7 15/16	14	5	4 3/4	90.0
4 3/4	7 1/4	8 3/8	14 1/2	5 1/4	5	98.0
5	7 5/8	8 7/8	15	5 1/2	5 1/4	110
5 1/4	8	9 1/4	15 1/2	5 3/4	5 1/2	122
5 1/2	8 3/8	9 3/4	16	6	5 3/4	142
5 3/4	8 3/4	10 1/8	16 1/2	6 1/4	6	157
6	9 1/8	10 5/8	17	6 1/2	6 1/4	176

Notes:
Weights and dimensions of sleeve nuts are typical; products of all suppliers are essentially similar. User shall verify with the manufacturer that strengths of sleeve nut are greater than the corresponding connecting rod when the same material is used.

Table 15–7
Dimensions and Weights of Recessed-Pin Nuts

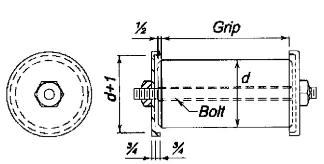


Material: Steel



Thread: 6 UN Class 2A/2B

Pin Dia. *d*, in.	Pin Dimensions, in			Nut Dimensions, in.					Weight, pounds
	Thread		*c*	Thick-ness *t*	Diameter		Recess		
	D	*T*			Short Dia.	Long Dia.	Rough Dia.	*s*	
2, 2 1/4	1 1/2	1	1/8	7/8	3	3 3/8	2 5/8	1/4	1
2 1/2, 2 3/4	2	1 1/8	1/8	1	3 5/8	4 1/8	3 1/8	1/4	2
3, 3 1/4, 3 1/2	2 1/2	1 1/4	1/8	1 1/8	4 3/8	5	3 7/8	3/8	3
3 3/4, 4	3	1 3/8	1/4	1 1/4	4 7/8	5 5/8	4 3/8	3/8	4
4 1/4, 4 1/2, 4 3/4	3 1/2	1 1/2	1/4	1 3/8	5 3/4	6 5/8	5 1/4	1/2	5
5, 5 1/4	4	1 5/8	1/4	1 1/2	6 1/4	7 1/4	5 3/4	1/2	6
5 1/2, 5 3/4, 6	4 1/2	1 3/4	1/4	1 5/8	7	8 1/8	6 1/2	5/8	8
6 1/4, 6 1/2	5	1 7/8	3/8	1 3/4	7 5/8	8 7/8	7	5/8	10
6 3/4, 7	5 1/2	2	3/8	1 7/8	8 1/8	9 3/8	7 1/2	3/4	12
7 1/4, 7 1/2	5 1/2	2	3/8	1 7/8	8 5/8	10	8	3/4	14
7 3/4, 8, 8 1/4	6	2 1/4	3/8	2 1/8	9 3/8	10 7/8	8 3/4	3/4	19
8 1/2, 8 3/4, 9	6	2 1/4	3/8	2 1/8	10 1/4	11 7/8	9 5/8	3/4	24
9 1/4, 9 1/2	6	2 3/8	3/8	2 1/4	11 1/4	13	10 5/8	3/4	32
9 3/4, 10	6	2 3/8	3/8	2 1/4	11 1/4	13	10 5/8	3/4	32

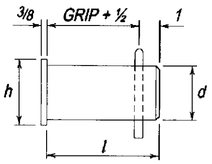


Typical Pin Cap Detail for Pins over 10 in. in dia.
Dimensions shown are approximate

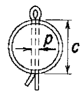
Notes:
Although nuts may be used on all sizes of pins as shown above, a detail similar to that shown at the left is preferable for pin diameters over 10 in. In this detail, the pin is held in place by a recessed cap at each end and secured by a bolt passing completely through the caps and pin. Suitable provisions must be made for attaching pilots and driving nuts.

Table 15–8
Dimensions and Weights of Cotter Pins

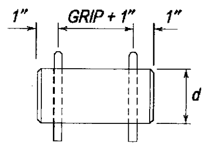
HORIZONTAL OR VERTICAL PIN



l = Length of pin, in.



HORIZONTAL PIN



Pin Diameter d, in.	Pins with Heads		Cotter		
	Head Diameter h, in.	Weight of One pounds	Length c, in.	Diameter p, in.	Weight per 100, pounds
$1^1/_4$	$1^1/_2$	$0.19 + 0.35l$	$^7/_8$	$^1/_4$	2.64
$1^1/_2$	$1^3/_4$	$0.26 + 0.50l$	1	$^1/_4$	3.10
$1^3/_4$	2	$0.33 + 0.68l$	$1^1/_8$	$^1/_4$	3.50
2	$2^3/_8$	$0.47 + 0.89l$	$1^1/_4$	$^3/_8$	9.00
$2^1/_4$	$2^5/_8$	$0.58 + 1.13l$	$1^3/_8$	$^3/_8$	9.40
$2^1/_2$	$2^7/_8$	$0.70 + 1.39l$	$1^1/_2$	$^3/_8$	10.9
$2^3/_4$	$3^1/_8$	$0.82 + 1.68l$	$1^5/_8$	$^3/_8$	11.4
3	$3^1/_2$	$1.02 + 2.00l$	$1^3/_4$	$^1/_2$	28.5
$3^1/_4$	$3^3/_4$	$1.17 + 2.35l$	$1^7/_8$	$^1/_2$	28.5
$3^1/_2$	4	$1.34 + 2.73l$	$1^7/_8$	$^1/_2$	33.8
$3^3/_4$	$4^1/_4$	$1.51 + 3.13l$	$2^1/_4$	$^1/_2$	33.8

ANSI/AISC 360-05
An American National Standard

Specification for Structural Steel Buildings

March 9, 2005

Supersedes the *Load and Resistance Factor Design Specification for Structural Steel Buildings* dated December 27, 1999, the *Specification for Structural Steel Buldings—Allowable Stress Design and Plastic Design* dated June 1, 1989, including Supplement No. 1, the *Specification for Allowable Stress Design of Single-Angle Members* dated June 1, 1989, the *Load and Resistance Factor Design Specification for Single-Angle Members* dated November 10, 2000, and the *Load and Resistance Factor Design Specification for the Design of Steel Hollow Structural Sections* dated November 10, 2000, and all previous versions of these specifications.

Approved by the AISC Committee on Specifications and issued by the AISC Board of Directors

AMERICAN INSTITUTE OF STEEL CONSTRUCTION, INC.
One East Wacker Drive, Suite 700
Chicago, Illinois 60601-1802

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Printed in the United States of America

DEDICATION

Professor Lynn S. Beedle

This edition of the AISC Specification is dedicated to the memory of Dr. Lynn S. Beedle, University Distinguished Professor at Lehigh University. Dr. Beedle served as a faculty member at Lehigh University for 41 years and won a very large number of professional and educational awards, including the 1973 T.R. Higgins Award and the 2003 Geerhard Haaijer Award from AISC. He was a major contributor to several editions of the AISC Specification and a long-time member of the AISC Committee on Specifications. He was instrumental in the development of plastic design methodologies and its implementation into the AISC Specification. He was Director of the Structural Stability Research Council for 25 years, and in that role fostered understanding of various stability problems and helped develop rational design provisions, many of which were adopted in the AISC Specifications. In 1969, he founded the Council on Tall Buildings and Urban Habitat and succeeded in bringing together the disciplines of architecture, structural engineering, construction, environment, sociology and politics, which underlie every major tall building project. He was actively involved in this effort until his death in late 2003 at the age of 85. His contributions to the design and construction of steel buildings will long be remembered by AISC, the steel industry and the structural engineering profession worldwide.

For a more complete discussion of Dr. Beedle's life and accomplishments, see *Catalyst for Skyscraper Revolution: Lynn S. Beedle—A Legend in his Lifetime* by Mir Ali, published by the Council on Tall Buildings and Urban Habitat (2004).

PREFACE

(This Preface is not part of ANSI/AISC 360-05, *Specification for Structural Steel Buildings*, but is included for informational purposes only.)

This Specification has been based upon past successful usage, advances in the state of knowledge, and changes in design practice. The 2005 American Institute of Steel Construction's *Specification for Structural Steel Buildings* for the first time provides an integrated treatment of Allowable Stress Design (ASD) and Load and Resistance Factor Design (LRFD), and thus combines and replaces earlier Specifications that treated the two design methods separately. As indicated in Chapter B of the Specification, designs can be made according to either ASD or LRFD provisions.

This Specification has been developed as a consensus document to provide a uniform practice in the design of steel-framed buildings and other structures. The intention is to provide design criteria for routine use and not to provide specific criteria for infrequently encountered problems, which occur in the full range of structural design.

This Specification is the result of the consensus deliberations of a committee of structural engineers with wide experience and high professional standing, representing a wide geographical distribution throughout the United States. The committee includes approximately equal numbers of engineers in private practice and code agencies, engineers involved in research and teaching, and engineers employed by steel fabricating and producing companies. The contributions and assistance of more than 50 additional professional volunteers working in ten task committees are also hereby acknowledged.

The Symbols, Glossary and Appendices to this Specification are an integral part of the Specification. A non-mandatory Commentary has been prepared to provide background for the Specification provisions and the user is encouraged to consult it. Additionally, non-mandatory User Notes are interspersed throughout the Specification to provide concise and practical guidance in the application of the provisions.

The reader is cautioned that professional judgment must be exercised when data or recommendations in the Specification are applied, as described more fully in the disclaimer notice preceding this Preface.

This Specification was approved by the Committee on Specifications,

James M. Fisher, Chairman
Roger E. Ferch, Vice Chairman
Hansraj G. Ashar
William F. Baker
John M. Barsom
William D. Bast
Reidar Bjorhovde
Roger L. Brockenbrough
Gregory G. Deierlein
Duane S. Ellifritt
Bruce R. Ellingwood
Michael Engelhardt
Shu-Jin Fang
Steven J. Fenves
John W. Fisher
Timothy P. Fraser
Theodore V. Galambos
Louis F. Geschwindner
Lawrence G. Griffis
John L. Gross
Tony C. Hazel
Mark V. Holland
Lawrence A. Kloiber
Roberto T. Leon
Stanley D. Lindsey
James O. Malley
Richard W. Marshall (deceased)
Harry W. Martin
David L. McKenzie
Duane K. Miller
Thomas M. Murray
R. Shankar Nair
Jack E. Petersen
Douglas A. Rees-Evans
Robert E. Shaw, Jr.
Donald R. Sherman
W. Lee Shoemaker
William A. Thornton
Raymond H. R. Tide
Cynthia J. Duncan, Secretary

The Committee gratefully acknowledges the following task committee members for their contribution to this document.

Farid Alfawakhiri
Georges Axmann
Joseph Bohinsky
Bruce Butler
Charles Carter
Robert Dexter (deceased)
Carol Drucker
W. Samuel Easterling
Michael Engestrom
M. Thomas Ferrell
Christopher Foley
Arvind Goverdhan
Jerome Hajjar
Tom Harrington
James Harris
Steven Herth
Todd Helwig
Richard Henige
Christopher Hewitt
Ronald Hiatt
Keith Hjelmstad
Socrates Ioannides
Nestor Iwankiw
Richard Kaehler
Dean Krouse
Barbara Lane
Jay Larson
Michael Lederle
Kevin LeSmith
J. Walter Lewis
Daniel Linzell
LeRoy Lutz
Peter Marshall
Brian Meacham
Saul Mednick
James Milke

Heath Mitchell
Jeffrey Packer
Frederick Palmer
Dhiren Panda
Teoman Pekoz
Carol Pivonka
Clinton Rex
John Ruddy
David Samuelson
Thomas Schlafly
James Swanson
Emile Troup
Chia-Ming Uang
Sriramulu Vinnakota
Robert Weber
Donald White
Robert Wills
Ronald Ziemian
Sergio Zoruba

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SYMBOLS

The section or table number in the right-hand column refers to where the symbol is first used.

Symbol	Definition	Section
A	Column cross-sectional area, in.^{2} (mm^2)	J10.6
A	Total cross-sectional area of member, in.^{2} (mm^2)	E7.2
A_B	Loaded area of concrete, in.^{2} (mm^2)	I2.1
A_{BM}	Cross-sectional area of the base metal, in.^{2} (mm^2)	J2.4
A_b	Nominal unthreaded body area of bolt or threaded part, in.^{2} (mm^2)	J3.6
A_{bi}	Cross-sectional area of the overlapping branch, in.^{2} (mm^2)	K2.3
A_{bj}	Cross-sectional area of the overlapped branch, in.^{2} (mm^2)	K2.3
A_c	Area of concrete, in.^{2} (mm^2)	I2.1
A_c	Area of concrete slab within effective width, in.^{2} (mm^2)	I3.2
A_D	Area of an upset rod based on the major thread diameter, in.^{2} (mm^2)	Table J3.2
A_e	Effective net area, in.^{2} (mm^2)	D2
A_{eff}	Summation of the effective areas of the cross section based on the reduced effective width, b_e, in.^{2} (mm^2)	E7.2
A_{fc}	Area of compression flange	G3.1
A_{fg}	Gross tension flange area, in.^{2} (mm^2)	F13.1
A_{fn}	Net tension flange area, in.^{2} (mm^2)	F13.1
A_{ft}	Area of tension flange, in.^{2} (mm^2)	G3.1
A_g	Gross area of member, in.^{2} (mm^2)	B3.13
A_g	Gross area of section based on design wall thickness, in.^{2} (mm^2)	G6
A_g	Gross area of composite member, in.^{2} (mm^2)	I2.1
A_g	Chord gross area, in.^{2} (mm^2)	K2.2
A_{gv}	Gross area subject to shear, in.^{2} (mm^2)	J4.3
A_n	Net area of member, in.^{2} (mm^2)	B3.13
A_{nt}	Net area subject to tension, in.^{2} (mm^2)	J4.3
A_{nv}	Net area subject to shear, in.^{2} (mm^2)	J4.2
A_{pb}	Projected bearing area, in.^{2} (mm^2)	J7
A_r	Area of adequately developed longitudinal reinforcing steel within the effective width of the concrete slab, in.^{2} (mm^2)	I3.2
A_s	Area of steel cross section, in.^{2} (mm^2)	I2.1
A_{sc}	Cross-sectional area of stud shear connector, in.^{2} (mm^2)	I2.1
A_{sf}	Shear area on the failure path, in.^{2} (mm^2)	D5.1
A_{sr}	Area of continuous reinforcing bars, in.^{2} (mm^2)	I2.1
A_{st}	Stiffener area, in.^{2} (mm^2)	G3.3
A_t	Net tensile area, in.^{2} (mm^2)	App. 3.4
A_w	Web area, the overall depth times the web thickness, dt_w, in.^{2} (mm^2)	G2.1

A_w	Effective area of the weld, in.^{2} (mm^2)	J2.4
A_{wi}	Effective area of weld throat of any ith weld element, in.^{2} (mm^2)	J2.4
A_1	Area of steel concentrically bearing on a concrete support, in.^{2} (mm^2)	J8
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)	J8
B	Overall width of rectangular HSS member, measured 90 degrees to the plane of the connection, in. (mm)	Table D3.1
B	Overall width of rectangular HSS main member, measured 90 degrees to the plane of the connection, in. (mm)	K3.1
B	Factor for lateral-torsional buckling in tees and double angles	F9.2
B_b	Overall width of rectangular HSS branch member, measured 90 degrees to the plane of the connection, in. (mm)	K3.1
B_{bi}	Overall branch width of the overlapping branch	K2.3
B_{bj}	Overall branch width of the overlapped branch	K2.3
B_p	Width of plate, measure 90 degrees to the plane of the connection, in. (mm)	K1.1
B_p	Width of plate, transverse to the axis of the main member, in. (mm)	K2.3
B_1, B_2	Factors used in determining M_u for combined bending and axial forces when first-order analysis is employed	C2.1
C	HSS torsional constant	H3.1
C_b	Lateral-torsional buckling modification factor for nonuniform moment diagrams when both ends of the unsupported segment are braced	F1
C_d	Coefficient relating relative brace stiffness and curvature	App. 6.3.1
C_f	Constant based on stress category, given in Table A-3.1	App. 3.3
C_m	Coefficient assuming no lateral translation of the frame	C2.1
C_p	Ponding flexibility coefficient for primary member in a flat roof	App. 2.1
C_r	Coefficient for web sidesway buckling	J10.4
C_s	Ponding flexibility coefficient for secondary member in a flat roof	App. 2.1
C_v	Web shear coefficient	G2.1
C_w	Warping constant, in.^{6} (mm^6)	E4
D	Nominal dead load	App. 2.2
D	Outside diameter of round HSS member, in. (mm)	Table B4.1
D	Outside diameter, in. (mm)	E7.2
D	Outside diameter of round HSS main member, in. (mm)	K2.1
D	Chord diameter, in. (mm)	K2.2
D_b	Outside diameter of round HSS branch member, in. (mm)	K2.1
D_s	Factor used in Equation G3-3, dependent on the type of transverse stiffeners used in a plate girder	G3.3
D_u	In slip-critical connections, a multiplier that reflects the ratio of the mean installed bolt pretension to the specified minimum bolt pretension	J3.8
E	Modulus of elasticity of steel = 29,000 ksi (200 000 MPa)	Table B4.1
E_c	Modulus of elasticity of concrete = $w_c^{1.5}\sqrt{f_c'}$, ksi ($0.043w_c^{1.5}\sqrt{f_c'}$, MPa)	I2.1

E_{cm}	Modulus of elasticity of concrete at elevated temperature, ksi (MPa)	App. 4.2.3
EI_{eff}	Effective stiffness of composite section, kip-in.^{2} (N-mm^2)	I2.1
E_m	Modulus of elasticity of steel at elevated temperature, ksi (MPa)	App. 4.2.3
E_s	Modulus of elasticity of steel = 29,000 ksi (200 000 MPa)	I2.1
F_a	Available axial stress at the point of consideration, ksi (MPa)	H2
F_{BM}	Nominal strength of the base metal per unit area, ksi (MPa)	J2.4
F_{bw}	Available flexural stress at the point of consideration about the major axis, ksi (MPa)	H2
F_{bz}	Available flexural stress at the point of consideration about the minor axis, ksi (MPa)	H2
F_c	Available stress, ksi (MPa)	K2.2
F_{cr}	Critical stress, ksi (MPa)	E3
F_{cr}	Buckling stress for the section as determined by analysis, ksi (MPa)	F12.2
F_{cry}	Critical stress about the minor axis, ksi (MPa)	E4
F_{crz}	Critical torsional buckling stress, ksi (MPa)	E4
F_e	Elastic critical buckling stress, ksi (MPa)	C1.3
F_{ex}	Elastic flexural buckling stress about the major axis, ksi (MPa)	E4
F_{EXX}	Electrode classification number, ksi (MPa)	J2.4
F_{ey}	Elastic flexural buckling stress about the minor axis, ksi (MPa)	E4
F_{ez}	Elastic torsional buckling stress, ksi (MPa)	E4
F_L	A calculated stress used in the calculation of nominal flexural strength, ksi (MPa)	Table B4.1
F_n	Nominal torsional strength	H3.3
F_n	Nominal tensile stress F_{nt}, or shear stress, F_{nv}, from Table J3.2, ksi (MPa)	J3.6
F_{nt}	Nominal tensile stress from Table J3.2, ksi (MPa)	J3.7
F'_{nt}	Nominal tensile stress modified to include the effects of shearing stress, ksi (MPa)	J3.7
F_{nv}	Nominal shear stress from Table J3.2, ksi (MPa)	J3.7
F_{SR}	Design stress range, ksi (MPa)	App. 3.3
F_{TH}	Threshold fatigue stress range, maximum stress range for indefinite design life from Table A-3.1, ksi (MPa)	App. 3.1
F_u	Specified minimum tensile strength of the type of steel being used, ksi (MPa)	D2
F_u	Specified minimum tensile strength of a stud shear connector, ksi (MPa)	I2.1
F_u	Specified minimum tensile strength of the connected material, ksi (MPa)	J3.10
F_u	Specified minimum tensile strength of HSS material, ksi (MPa)	K1.1
F_{um}	Specified minimum tensile strength of the type of steel being used at elevated temperature, ksi (MPa)	App. 4.2
F_w	Nominal strength of the weld metal per unit area, ksi (MPa)	J2.4
F_{wi}	Nominal stress in any ith weld element, ksi (MPa)	J2.4

F_{wix}	x component of stress F_{wi}, ksi (MPa)	J2.4
F_{wiy}	y component of stress F_{wi}, ksi (MPa)	J2.4
F_y	Specified minimum yield stress of the type of steel being used, ksi (MPa). As used in this Specification, "yield stress" denotes either the specified minimum yield point (for those steels that have a yield point) or specified yield strength (for those steels that do not have a yield point)	Table B4.1
F_y	Specified minimum yield stress of the compression flange, ksi (MPa)	App. 1.3
F_y	Specified minimum yield stress of the column web, ksi (MPa)	J10.6
F_y	Specified minimum yield stress of HSS member material, ksi (MPa)	K1.1
F_y	Specified minimum yield stress of HSS main member material, ksi (MPa)	K2.1
F_{yb}	Specified minimum yield stress of HSS branch member material, ksi (MPa)	K2.1
F_{ybi}	Specified minimum yield stress of the overlapping branch material, ksi (MPa)	K2.3
F_{ybj}	Specified minimum yield stress of the overlapped branch material, ksi (MPa)	K2.3
F_{yf}	Specified minimum yield stress of the flange, ksi (MPa)	J10.1
F_{ym}	Specified minimum yield stress of the type of steel being used at elevated temperature, ksi (MPa)	App. 4.2
F_{yp}	Specified minimum yield stress of plate, ksi (MPa)	K1.1
F_{yr}	Specified minimum yield stress of reinforcing bars, ksi (MPa)	I2.1
F_{yst}	Specified minimum yield stress of the stiffener material, ksi (MPa)	G3.3
F_{yw}	Specified minimum yield stress of the web, ksi (MPa)	J10.2
G	Shear modulus of elasticity of steel = 11,200 ksi (77 200 MPa)	E4
ΣH	Story shear produced by the lateral forces used to compute Δ_H, kips (N)	C2.1
H	Overall height of rectangular HSS member, measured in the plane of the connection, in. (mm)	Table D3.1
H	Overall height of rectangular HSS main member, measured in the plane of the connection, in. (mm)	K2.1
H	Flexural constant	E4
H_b	Overall height of rectangular HSS branch member, measured in the plane of the connection, in. (mm)	K2.1
H_{bi}	Overall depth of the overlapping branch	K2.3
I	Moment of inertia in the place of bending, in.^{4} (mm^4)	C2.1
I	Moment of inertia about the axis of bending, in.^{4} (mm^4)	App. 7.3
I_c	Moment of inertia of the concrete section, in.^{4} (mm^4)	I2.1
I_d	Moment of inertia of the steel deck supported on secondary members, in.^{4} (mm^4)	App. 2.1
I_p	Moment of inertia of primary members, in.^{4} (mm^4)	App. 2.1
I_s	Moment of inertia of secondary members, in.^{4} (mm^4)	App. 2.1

I_s Moment of inertia of steel shape, in.^{4} (mm^4) I2.1
I_{sr} Moment of inertia of reinforcing bars, in.^{4} (mm^4) I2.1
I_x, I_y Moment of inertia about the principal axes, in.^{4} (mm^4) E4
I_y Out-of-plane moment of inertia, in.^{4} (mm^4) . . . . App. 6.2
I_z Minor principal axis moment of inertia, in.^{4} (mm^4) F10.2
I_{yc} Moment of inertia about y-axis referred to the compression flange, or if reverse curvature bending referred to smaller flange, in.^{4} (mm^4) F1
J Torsional constant, in.^{4} (mm^4) E4
K Effective length factor determined in accordance with Chapter C C1.2
K_z Effective length factor for torsional buckling E4
K_1 Effective length factor in the plane of bending, calculated based on the assumption of no lateral translation set equal to 1.0 unless analysis indicates that a smaller value may be used C2.1
K_2 Effective length factor in the plane of bending, calculated based on a sidesway buckling analysis C2.1
L Story height, in. (mm) C2.1
L Length of the member, in. (mm) H3
L Actual length of end-loaded weld, in. (mm) J2.2
L Nominal occupancy live load . . . . App. 4.1.4
L Laterally unbraced length of a member, in. (mm) E2
L Span length, in. (mm) . . . . App. 6.2
L Length of member between work points at truss chord centerlines, in. (mm) E5
L_b Length between points that are either braced against lateral displacement of compression flange or braced against twist of the cross section, in. (mm) F2
L_b Distance between braces, in. (mm) . . . . App. 6.2
L_c Length of channel shear connector, in. (mm) I3.2
L_c Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material, in. (mm) J3.10
L_e Total effective weld length of groove and fillet welds to rectangular HSS, in. (mm) K2.3
L_p Limiting laterally unbraced length for the limit state of yielding in. (mm) F2.2
L_p Column spacing in direction of girder, ft (m) . . . . App. 2
L_{pd} Limiting laterally unbraced length for plastic analysis, in. (mm) . . . . . App. 1.7
L_q Maximum unbraced length for M_r (the required flexural strength), in. (mm) . . . . App. 6.2
L_r Limiting laterally unbraced length for the limit state of inelastic lateral-torsional buckling, in. (mm) F2.2
L_s Column spacing perpendicular to direction of girder, ft (m) . . . . App. 2.1
L_v Distance from maximum to zero shear force, in. (mm) G6

M_A Absolute value of moment at quarter point of the unbraced segment, kip-in. (N-mm) F1
M_a Required flexural strength in chord, using ASD load combinations, kip-in. (N-mm) K2.2
M_B Absolute value of moment at centerline of the unbraced segment, kip-in. (N-mm) F1
M_{br} Required bracing moment, kip-in. (N-mm) . . . . . App. 6.2
M_C Absolute value of moment at three-quarter point of the unbraced segment, kip-in. (N-mm) F1
$M_{c(x,\, y)}$ Available flexural strength determined in accordance with Chapter F, kip-in. (N-mm) H1.1
M_{cx} Available flexural-torsional strength for strong axis flexure determined in accordance with Chapter F, kip-in. (N-mm) H1.3
M_e Elastic lateral-torsional buckling moment, kip-in. (N-mm) F10.2
M_{lt} First-order moment under LRFD or ASD load combinations caused by lateral translation of the frame only, kip-in. (N-mm) C2.1
M_{max} Absolute value of maximum moment in the unbraced segment, kip-in. (N-mm) F1
M_n Nominal flexural strength, kip-in. (N-mm) F1
M_{nt} First-order moment using LRFD or ASD load combinations assuming there is no lateral translation of the frame, kip-in. (N-mm) C2.1
M_p Plastic bending moment, kip-in. (N-mm) . . . . . Table B4.1
M_r Required second-order flexural strength under LRFD or ASD load combinations, kip-in. (N-mm) C2.1
M_r Required flexural strength using LRFD or ASD load combinations, kip-in. (N-mm) H1
M_r Required flexural strength in chord, kip-in. (N-mm) K2.2
$M_{r\text{-}ip}$ Required in-plane flexural strength in branch, kip-in. (N-mm) K3.2
$M_{r\text{-}op}$ Required out-of-plane flexural strength in branch, kip-in. (N-mm) K3.2
M_u Required flexural strength in chord, using LRFD load combinations, kip-in. (N-mm) K2.2
M_y Yield moment about the axis of bending, kip-in. (N-mm) . . . . . Table B4.1
M_1 Smaller moment, calculated from a first-order analysis, at the ends of that portion of the member unbraced in the plane of bending under consideration, kip-in. (N-mm) C2.1
M_2 Larger moment, calculated from a first-order analysis, at the ends of that portion of the member unbraced in the plane of bending under consideration, kip-in. (N-mm) C2.1
N Length of bearing (not less than k for end beam reactions), in. (mm) J10.2
N Bearing length of the load, measured parallel to the axis of the HSS member, (or measured across the width of the HSS in the case of the loaded cap plates), in. (mm) K1.1
N Number of stress range fluctuations in design life . . . . . App. 3.3
N_b Number of bolts carrying the applied tension J3.9

N_i	Additional lateral load	C2.2
N_i	Notional lateral load applied at level i, kips (N)	App. 7.3
N_s	Number of slip planes	J3.8
O_v	Overlap connection coefficient	K2.2
P	Pitch, in. per thread (mm per thread)	App. 3.4
P_{br}	Required brace strength, kips (N)	App. 6.2
P_c	Available axial compressive strength, kips (N)	H1.1
P_c	Available tensile strength, kips (N)	H1.2
P_{co}	Available compressive strength out of the plane of bending, kip (N)	H1.3
P_{e1}, P_{e2}	Elastic critical buckling load for braced and unbraced frame, respectively, kips (N)	C2.1
P_{eL}	Euler buckling load, evaluated in the plane of bending, kips (N)	App. 7.3
$P_{l(t,c)}$	First-order axial force using LRFD or ASD load combinations as a result of lateral translation of the frame only (tension or compression), kips (N)	C2.1
$P_{n(t,c)}$	First-order axial force using LRFD or ASD load combinations, assuming there is no lateral translation of the frame (tension or compression), kips (N)	C2.1
P_n	Nominal axial strength, kips (N)	D2
P_o	Nominal axial compressive strength without consideration of length effects, kips (N)	I2.1
P_p	Nominal bearing strength of concrete, kips (N)	I2.1
P_r	Required second-order axial strength using LRFD or ASD load combinations, kips (N)	C2.1
P_r	Required axial compressive strength using LRFD or ASD load combinations, kips (N)	C2.2
P_r	Required tensile strength using LRFD or ASD load combinations, kips (N)	H1.2
P_r	Required strength, kips (N)	J10.6
P_r	Required axial strength in branch, kips (N)	K3.2d
P_r	Required axial strength in chord, kips (N)	K2.2
P_u	Required axial strength in compression, kips (N)	App. 1.4
P_y	Member yield strength, kips (N)	C2.2
Q	Full reduction factor for slender compression elements	E7
Q_a	Reduction factor for slender stiffened compression elements	E7.2
Q_f	Chord-stress interaction parameter	K2.2
Q_n	Nominal strength of one stud shear connector, kips (N)	I2.1
Q_s	Reduction factor for slender unstiffened compression elements	E7.1
R	Nominal load due to rainwater or snow, exclusive of the ponding contribution, ksi (MPa)	App. 2.2
R	Seismic response modification coefficient	A1.1
R_a	Required strength (ASD)	B3.4
R_{FIL}	Reduction factor for joints using a pair of transverse fillet welds only	App. 3.3

R_g	Coefficient to account for group effect	I3.2
R_m	Factor in Equation C2-6b dependent on type of system	C2.1
R_m	Cross-section monosymmetry parameter	F1
R_n	Nominal strength, specified in Chapters B through K	B3.3
R_n	Nominal slip resistance, kips (N)	J3.8
R_p	Position effect factor for shear studs	I3.2
R_{pc}	Web plastification factor	F4.1
R_{PJP}	Reduction factor for reinforced or nonreinforced transverse partial-joint-penetration (PJP) groove welds	App. 3.3
R_{pt}	Web plastification factor corresponding to the tension flange yielding limit state	F4.4
R_u	Required strength (LRFD)	B3.3
R_{wl}	Total nominal strength of longitudinally loaded fillet welds, as determined in accordance with Table J2.5	J2.4
R_{wt}	Total nominal strength of transversely loaded fillet welds, as determined in accordance with Table J2.5 without the alternate in Section J2.4 (a)	J2.4
S	Elastic section modulus of round HSS, in.^{3} (mm^3)	F8.2
S	Lowest elastic section modulus relative to the axis of bending, in.^{3} (mm^3)	F12
S	Spacing of secondary members, ft (m)	App. 2.1
S	Nominal snow load	App. 4.1.4
S	Chord elastic section modulus, in.^{3} (mm^3)	K2.2
S_c	Elastic section modulus to the toe in compression relative to the axis of bending, in.^{3} (mm^3).	F10.3
S_{eff}	Effective section modulus about major axis, in.^{3} (mm^3)	F7.2
S_{xt}, S_{xc}	Elastic section modulus referred to tension and compression flanges, respectively, in.^{3} (mm^3)	Table B4.1
S_x, S_y	Elastic section modulus taken about the principal axes, in.^{3} (mm^3)	F2.2, F6
S_y	For channels, taken as the minimum section modulus	F6
T	Nominal forces and deformations due to the design-basis fire defined in Section 4.2.1	App. 4.1.4
T_a	Tension force due to ASD load combinations, kips (kN)	J3.9
T_b	Minimum fastener tension given in Table J3.1 or J3.1M, kips (kN)	J3.8
T_c	Available torsional strength, kip-in. (N-mm)	H3.2
T_n	Nominal torsional strength, kip-in. (N-mm)	H3.1
T_r	Required torsional strength, kip-in. (N-mm)	H3.2
T_u	Tension force due to LRFD load combinations, kips (kN)	J3.9
U	Shear lag factor	D3.3
U	Utilization ratio	K2.2
U_{bs}	Reduction coefficient, used in calculating block shear rupture strength	J4.3
U_p	Stress index	App. 2.2
U_s	Stress index	App. 2.2

V Required shear force introduced to column, kips (N) I2.1
V' Required shear force transferred by shear connectors, kips (N) I2.1
V_c Available shear strength, kips (N) G3.3
V_n Nominal shear strength, kips (N). G1
V_r Required shear strength at the location of the stiffener, kips (N) G3.3
V_r Required shear strength using LRFD or ASD load combinations, kips (N) H3.2
Y_i Gravity load from the LRFD load combination or 1.6 times the ASD load combination applied at level i, kips (N) C2.2
Y_t Hole reduction coefficient, kips (N) F13.1
Z Plastic section modulus about the axis of bending, in.^{3} (mm^3) F7.1
Z_b Branch plastic section modulus about the correct axis of bending, in.^{3} (mm^3) K3.3
$Z_{x,y}$ Plastic section modulus about the principal axes, in.^{3} (mm^3) F2, F6.1
a Clear distance between transverse stiffeners, in. (mm) F13.2
a Distance between connectors in a built-up member, in. (mm) E6.1
a Shortest distance from edge of pin hole to edge of member measured parallel to direction of force, in. (mm) D5.1
a Half the length of the nonwelded root face in the direction of the thickness of the tension-loaded plate, in. (mm) . . . . App. 3.3
a_w Ratio of two times the web area in compression due to application of major axis bending moment alone to the area of the compression flange components F4.2
b Outside width of leg in compression, in. (mm) F10.3
b Full width of longest angle leg, in. (mm) E7.1
b Width of unstiffened compression element; for flanges of I-shaped members and tees, the width b is half the full-flange width, b_f; for legs of angles and flanges of channels and zees, the width b is the full nominal dimension; for plates, the width b is the distance from the free edge to the first row of fasteners or line of welds, or the distance between adjacent lines of fasteners or lines of welds; for rectangular HSS, the width b is the clear distance between the webs less the inside corner radius on each side, in. (mm) B4.1, B4.2
b Width of the angle leg resisting the shear force, in. (mm) G4
b_{cf} Width of column flange, in. (mm) J10.6
b_e Reduced effective width, in. (mm) E7.2
b_{eff} Effective edge distance; the distance from the edge of the hole to the edge of the part measured in the direction normal to the applied force, in. (mm) D5.1
b_{eoi} Effective width of the branch face welded to the chord K2.3
b_{eov} Effective width of the branch face welded to the overlapped brace K2.3
b_f Flange width, in. (mm) B4.1
b_{fc} Compression flange width, in. (mm) F4.2
b_{ft} Width of tension flange, in. (mm) G3.1

b_l	Longer leg of angle, in. (mm)	E5
b_s	Shorter leg of angle, in. (mm)	E5
b_s	Stiffener width for one-sided stiffeners, in. (mm)	App. 6.2
d	Nominal fastener diameter, in. (mm)	J3.3
d	Full nominal depth of the section, in. (mm)	B4.1
d	Full nominal depth of tee, in. (mm)	E7.1
d	Depth of rectangular bar, in. (mm)	F11.2
d	Full nominal depth of section, in. (mm)	B4.1
d	Full nominal depth of tee, in. (mm)	E7.1
d	Diameter, in. (mm)	J7
d	Pin diameter, in. (mm)	D5.1
d	Roller diameter, in. (mm)	J7
d_b	Beam depth, in. (mm)	J10.6
d_b	Nominal diameter (body or shank diameter), in. (mm)	App. 3.4
d_c	Column depth, in. (mm)	J10.6
e	Eccentricity in a truss connection, positive being away from the branches, in. (mm)	K2.1
$e_{mid\text{-}ht}$	Distance from the edge of stud shank to the steel deck web, measured at mid-height of the deck rib, and in the load bearing direction of the stud (in other words, in the direction of maximum moment for a simply supported beam), in. (mm)	I3.2
f_a	Required axial stress at the point of consideration using LRFD or ASD load combinations, ksi (MPa)	H2
$f_{b(w,z)}$	Required flexural stress at the point of consideration (major axis, minor axis) using LRFD or ASD load combinations, ksi (MPa)	H2
f'_c	Specified minimum compressive strength of concrete, ksi (MPa)	I1.1
f'_{cm}	Specified minimum compressive strength of concrete at elevated temperatures, ksi (MPa)	App. 4.2
f_o	Stress due to D + R (the nominal dead load + the nominal load due to rainwater or snow exclusive of the ponding contribution), ksi (MPa)	App. 2.2
f_v	Required shear strength per unit area, ksi (MPa)	J3.7
g	Transverse center-to-center spacing (gage) between fastener gage lines, in. (mm)	B3.13
g	Gap between toes of branch members in a gapped K-connection, neglecting the welds, in. (mm)	K2.1
h	Clear distance between flanges less the fillet or corner radius for rolled shapes; for built-up sections, the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used; for tees, the overall depth; for rectangular HSS, the clear distance between the flanges less the inside corner radius on each side, in. (mm)	B4.2
h	Distance between centroids of individual components perpendicular to the member axis of buckling, in. (mm)	E6.1

h_c	Twice the distance from the centroid to the following: the inside face of the compression flange less the fillet or corner radius, for rolled shapes; the nearest line of fasteners at the compression flange or the inside faces of the compression flange when welds are used, for built-up sections, in. (mm)	B4.2
h_o	Distance between flange centroids, in. (mm)	F2.2
h_p	Twice the distance from the plastic neutral axis to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used, in. (mm)	B4.2
h_{sc}	Hole factor	J3.8
j	Factor defined by Equation G2-6 for minimum moment of inertia for a transverse stiffener	G2.2
k	Distance from outer face of flange to the web toe of fillet, in. (mm)	J10.2
k	Outside corner radius of the HSS, which is permitted to be taken as $1.5t$ if unknown, in. (mm)	K1.3
k_c	Coefficient for slender unstiffened elements, in. (mm)	Table B4.1
k_s	Slip-critical combined tension and shear coefficient	J3.9
k_v	Web plate buckling coefficient	G2.1
l	Largest laterally unbraced length along either flange at the point of load, in. (mm)	J10.4
l	Length of bearing, in. (mm)	J7
l	Length of connection in the direction of loading, in. (mm)	Table D3.1
n	Number of nodal braced points within the span	App. 6.2
n	Threads per inch (per mm)	App. 3.4
p	Ratio of element i deformation to its deformation at maximum stress	J2.4
p	Projected length of the overlapping branch on the chord	K2.2
q	Overlap length measured along the connecting face of the chord beneath the two branches	K2.2
r	Governing radius of gyration, in. (mm)	E2
r_{crit}	Distance from instantaneous center of rotation to weld element with minimum Δ_u/r_i ratio, in. (mm)	J2.4
r_i	Minimum radius of gyration of individual component in a built-up member, in. (mm)	E6.1
r_{ib}	Radius of gyration of individual component relative to its centroidal axis parallel to member axis of buckling, in. (mm)	E6.1
$\overline{r_o}$	Polar radius of gyration about the shear center, in. (mm)	E4
r_t	Radius of gyration of the flange components in flexural compression plus one-third of the web area in compression due to application of major axis bending moment alone	F4.2
r_{ts}	Effective radius of gyration used in the determination of L_r for the lateral-torsional buckling limit state for major axis bending of doubly symmetric compact I-shaped members and channels	F2.2
r_x	Radius of gyration about geometric axis parallel to connected leg, in. (mm)	E5

r_y	Radius of gyration about y-axis, in. (mm)	E4
r_z	Radius of gyration for the minor principal axis, in. (mm)	E5
s	Longitudinal center-to-center spacing (pitch) of any two consecutive holes, in. (mm)	B3.13
t	Thickness of element, in. (mm)	B4.2
t	Wall thickness, in. (mm)	E7.2
t	Angle leg thickness, in. (mm)	F10.2
t	Width of rectangular bar parallel to axis of bending, in. (mm)	F11.2
t	Thickness of connected material, in. (mm)	J3.10
t	Thickness of plate, in. (mm)	D5.1
t	Design wall thickness for HSS equal to 0.93 times the nominal wall thickness for ERW HSS and equal to the nominal wall thickness for SAW HSS, in. (mm)	B3.12
t	Total thickness of fillers, in. (mm)	J5
t	Design wall thickness of HSS main member, in. (mm)	K2.1
t_b	Design wall thickness of HSS branch member, in. (mm)	K2.1
t_{bi}	Thickness of the overlapping branch, in. (mm)	K2.3
t_{bj}	Thickness of the overlapped branch, in. (mm)	K2.3
t_{cf}	Thickness of the column flange, in. (mm)	J10.6
t_f	Thickness of the loaded flange, in. (mm)	J10.1
t_f	Flange thickness of channel shear connector, in. (mm)	I3.2
t_{fc}	Compression flange thickness, in. (mm)	F4.2
t_p	Thickness of plate, in. (mm)	K1.1
t_p	Thickness of tension loaded plate, in. (mm)	App. 3.3
t_p	Thickness of the attached transverse plate, in. (mm)	K2.3
t_s	Web stiffener thickness, in. (mm)	App. 6.2
t_w	Web thickness of channel shear connector, in. (mm)	I3.2
t_w	Beam web thickness, in. (mm)	App. 6.3
t_w	Web thickness, in. (mm)	Table B4.1
t_w	Column web thickness, in. (mm)	J10.6
t_w	Thickness of element, in. (mm)	E7.1
w	Width of cover plate, in. (mm)	F13.3
w	Weld leg size, in. (mm)	J2.2
w	Subscript relating symbol to major principal axis bending	
w	Plate width, in. (mm)	Table D3.1
w	Leg size of the reinforcing or contouring fillet, if any, in the direction of the thickness of the tension-loaded plate, in. (mm)	App. 3.3
w_c	Weight of concrete per unit volume ($90 \leq w_c \leq 155$ lbs/ft^3 or $1500 \leq w_c \leq 2500$ kg/m^3)	I2.1
w_r	Average width of concrete rib or haunch, in. (mm)	I3.2
x	Subscript relating symbol to strong axis	
x_o, y_o	Coordinates of the shear center with respect to the centroid, in. (mm)	E4
$\overline{x}$	Connection eccentricity, in. (mm)	Table D3.1

y	Subscript relating symbol to weak axis
z	Subscript relating symbol to minor principal axis bending
α	Factor used in B2 equation C2.1
α	Separation ratio for built-up compression members $= \dfrac{h}{2r_{ib}}$ E6.1
β	Reduction factor given by Equation J2-1 . . . . J2.2
β	Width ratio; the ratio of branch diameter to chord diameter for round HSS; the ratio of overall branch width to chord width for rectangular HSS K2.1
β_T	Brace stiffness requirement excluding web distortion, kip-in./radian (N-mm/radian) . . . . App. 6.2
β_{br}	Required brace stiffness . . . . App. 6.2
β_{eff}	Effective width ratio; the sum of the perimeters of the two branch members in a K-connection divided by eight times the chord width K2.1
β_{eop}	Effective outside punching parameter K2.3
β_{sec}	Web distortional stiffness, including the effect of web transverse stiffeners, if any, kip-in./radian (N-mm/radian) . . . . App. 6.2
β_w	Section property for unequal leg angles, positive for short legs in compression and negative for long legs in compression F10.2
Δ	First-order interstory drift due to the design loads, in. (mm) C2.2
Δ_H	First-order interstory drift due to lateral forces, in. (mm) C2.1
Δ_i	Deformation of weld elements at intermediate stress levels, linearly proportioned to the critical deformation based on distance from the instantaneous center of rotation, r_i, in. (mm) J2.4
Δ_m	Deformation of weld element at maximum stress, in. (mm) J2.4
Δ_u	Deformation of weld element at ultimate stress (fracture), usually in element furthest from instantaneous center of rotation, in. (mm) J2.4
γ	Chord slenderness ratio; the ratio of one-half the diameter to the wall thickness for round HSS; the ratio of one-half the width to wall thickness for rectangular HSS K2.1
ζ	Gap ratio; the ratio of the gap between the branches of a gapped K-connection to the width of the chord for rectangular HSS K2.1
η	Load length parameter, applicable only to rectangular HSS; the ratio of the length of contact of the branch with the chord in the plane of the connection to the chord width K2.1
λ	Slenderness parameter F3
λ_p	Limiting slenderness parameter for compact element B4
λ_{pf}	Limiting slenderness parameter for compact flange F3
λ_{pw}	Limiting slenderness parameter for compact web F4
λ_r	Limiting slenderness parameter for noncompact element B4
λ_{rf}	Limiting slenderness parameter for noncompact flange F3
λ_{rw}	Limiting slenderness parameter for noncompact web F4
μ	Mean slip coefficient for class A or B surfaces, as applicable, or as established by tests J3.8

ϕ	Resistance factor, specified in Chapters B through K	B3.3
ϕ_B	Resistance factor for bearing on concrete	I2.1
ϕ_b	Resistance factor for flexure	F1
ϕ_c	Resistance factor for compression	E1
ϕ_c	Resistance factor for axially loaded composite columns	I2.1b
ϕ_{sf}	Resistance factor for shear on the failure path	D5.1
ϕ_T	Resistance factor for torsion	H3.1
ϕ_t	Resistance factor for tension	D2
ϕ_v	Resistance factor for shear	G1
Ω	Safety factor	B3.4
Ω_B	Safety factor for bearing on concrete	I2.1
Ω_b	Safety factor for flexure	F1
Ω_c	Safety factor for compression	E1
Ω_c	Safety factor for axially loaded composite columns	I2.1b
Ω_{sf}	Safety factor for shear on the failure path	D5.1
Ω_t	Safety factor for torsion	H3.1
Ω_t	Safety factor for tension	D2
Ω_v	Safety factor for shear	G1
ρ_{sr}	Minimum reinforcement ratio for longitudinal reinforcing	I2.1
θ	Angle of loading measured from the weld longitudinal axis, degrees	J2.4
θ	Acute angle between the branch and chord, degrees	K2.1
ε_{cu}	Strain corresponding to compressive strength, f'_c	App. 4.2
τ_b	Parameter for reduced flexural stiffness using the direct analysis method	App. 7.3

GLOSSARY

Terms that appear in this Glossary are *italicized* throughout the Specification, where they first appear within a sub-section.

Notes:

(1) Terms designated with † are common AISI-AISC terms that are coordinated between the two standards developers.
(2) Terms designated with * are usually qualified by the type of *load effect,* for example, *nominal tensile strength*, *available compressive strength*, *design flexural strength*.
(3) Terms designated with ** are usually qualified by the type of component, for example, web *local buckling*, flange *local bending*.

Allowable strength †. *Nominal strength* divided by the *safety factor*, R_n / Ω.

Allowable stress. Allowable strength divided by the appropriate section property, such as section modulus or cross-section area.

Amplification factor. Multiplier of the results of *first-order analysis* to reflect *second-order* effects.

Applicable building code†. Building code under which the structure is designed.

ASD (Allowable Strength Design)†. Method of proportioning *structural components* such that the *allowable strength* equals or exceeds the *required strength* of the component under the action of the *ASD load combinations.*

ASD load combination†. Load combination in the *applicable building code* intended for allowable strength design (*allowable stress design*).

Authority having jurisdiction. Organization, political subdivision, office or individual charged with the responsibility of administering and enforcing the provisions of the *applicable building code*.

*Available strength**†. *Design strength* or *allowable strength,* as appropriate.

*Available stress**. *Design stress* or *allowable stress*, as appropriate.

Average rib width. Average width of the rib of a corrugation in a *formed steel deck*.

Batten plate. Plate rigidly connected to two parallel components of a built-up *column* or *beam* designed to transmit shear between the components.

Beam. Structural member that has the primary function of resisting bending moments.

Beam-column. Structural member that resists both axial force and bending moment.

Bearing. In a bolted connection, *limit state* of shear forces transmitted by the bolt to the connection elements.

Bearing (local compressive yielding). Limit state of *local* compressive *yielding* due to the action of a member bearing against another member or surface.

Bearing-type connection. Bolted *connection* where shear forces are transmitted by the bolt bearing against the connection elements.

Block shear rupture. In a connection, *limit state* of tension fracture along one path and shear yielding or shear fracture along another path.

Braced frame†. An essentially vertical truss system that provides resistance to lateral forces and provides stability for the *structural system*.

Branch face. Wall of *HSS branch member.*

Branch member. For *HSS connections*, member that terminates at a *chord member* or *main member*.

Buckling. Limit state of sudden change in the geometry of a structure or any of its elements under a critical loading condition.

Buckling strength. Nominal strength for *buckling* or *instability limit states*.

Built-up member, cross-section, section, shape. Member, cross-section, section or shape fabricated from structural steel elements that are welded or bolted together.

Camber. Curvature fabricated into a beam or truss so as to compensate for deflection induced by loads.

Charpy V-Notch impact test. Standard dynamic test measuring notch toughness of a specimen.

Chord member. For *HSS*, primary member that extends through a truss *connection*.

Cladding. Exterior covering of structure.

Cold-formed steel structural member†. Shape manufactured by press-braking blanks sheared from sheets, cut lengths of coils or plates, or by roll forming cold- or hot-rolled coils or sheets; both forming operations being performed at ambient room temperature, that is, without manifest addition of heat such as would be required for hot forming.

Column. Structural member that has the primary function of resisting axial force.

Combined system. Structure comprised of two or more lateral load-resisting systems of different type.

Compact section. Section capable of developing a fully plastic stress distribution and possessing a *rotation capacity* of approximately three before the onset of local buckling.

Complete-joint-penetration groove weld (CJP). Groove weld in which weld metal extends through the joint thickness, except as permitted for *HSS connections*.

Composite. Condition in which steel and concrete elements and members work as a unit in the distribution of internal forces.

Concrete crushing. Limit state of compressive failure in concrete having reached the ultimate strain.

Concrete haunch. Section of solid concrete that results from stopping the deck on each side of the girder in a *composite* floor system constructed using a *formed steel deck.*

Concrete-encased beam. Beam totally encased in concrete cast integrally with the slab.

Connection†. Combination of structural elements and *joints* used to transmit forces between two or more members.

Cope. Cutout made in a structural member to remove a flange and conform to the shape of an intersecting member.

Cover plate. Plate welded or bolted to the flange of a member to increase cross-sectional area, section modulus or moment of inertia.

Cross connection. HSS connection in which forces in *branch members* or connecting elements transverse to the *main member* are primarily equilibrated by forces in other branch members or connecting elements on the opposite side of the main member.

*Design load**†. Applied *load* determined in accordance with either *LRFD load combinations* or *ASD load combinations*, whichever is applicable.

*Design strength**†. *Resistance factor* multiplied by the *nominal strength*, ϕR_n.

Design stress range. Magnitude of change in stress due to the repeated application and removal of service live *loads*. For locations subject to stress reversal it is the algebraic difference of the peak stresses.

*Design stress**. *Design strength* divided by the appropriate section property, such as section modulus or cross section area.

Design wall thickness. *HSS* wall thickness assumed in the determination of section properties.

Diagonal bracing. Inclined structural member carrying primarily axial force in a *braced frame.*

Diagonal stiffener. Web stiffener at column panel zone oriented diagonally to the flanges, on one or both sides of the web.

Diaphragm plate. Plate possessing in-plane shear stiffness and strength, used to transfer forces to the supporting elements.

Diaphragm†. Roof, floor or other membrane or bracing system that transfers in-plane forces to the lateral force resisting system.

Direct analysis method. Design method for stability that captures the effects of residual stresses and initial out-of-plumbness of frames by reducing stiffness and applying *notional loads* in a *second-order analysis.*

Direct bond interaction. Mechanism by which force is transferred between steel and concrete in a *composite* section by bond stress.

Distortional failure. Limit state of an *HSS* truss connection based on distortion of a rectangular *HSS* chord member into a rhomboidal shape.

Distortional stiffness. Out-of-plane flexural stiffness of web.

Double curvature. Deformed shape of a beam with one or more inflection points within the span.

Double-concentrated forces. Two equal and opposite forces that form a couple on the same side of the loaded member.

Doubler. Plate added to, and parallel with, a *beam* or *column* web to increase resistance to concentrated forces.

Drift. Lateral deflection of structure.

Effective length factor, K. Ratio between the *effective length* and the unbraced length of the member.

Effective length. Length of an otherwise identical *column* with the same strength when analyzed with pinned end conditions.

Effective net area. Net area modified to account for the effect of shear lag.

Effective section modulus. Section modulus reduced to account for buckling of slender compression elements.

Effective width. Reduced width of a plate or slab with an assumed uniform stress distribution which produces the same effect on the behavior of a structural member as the actual plate or slab width with its nonuniform stress distribution.

Elastic analysis. Structural analysis based on the assumption that the structure returns to its original geometry on removal of the *load*.

Encased composite column. Composite column consisting of a *structural concrete column* and one or more embedded steel shapes.

End panel. Web panel with an adjacent panel on one side only.

End return. Length of *fillet weld* that continues around a corner in the same plane.

Engineer of record. Licensed professional responsible for sealing the contract documents.

Expansion rocker. Support with curved surface on which a member bears that can tilt to accommodate expansion.

Expansion roller. Round steel bar on which a member bears that can roll to accommodate expansion.

Eyebar. Pin-connected tension member of uniform thickness, with forged or thermally cut head of greater width than the body, proportioned to provide approximately equal strength in the head and body.

Factored load†. Product of a *load factor* and the *nominal load.*

Fastener. Generic term for bolts, rivets, or other connecting devices.

Fatigue. Limit state of crack initiation and growth resulting from repeated application of live *loads*.

Faying surface. Contact surface of connection elements transmitting a shear force.

Filled composite column. Composite column consisting of a shell of *HSS* or *pipe* filled with *structural concrete*.

Filler metal. Metal or alloy to be added in making a welded joint.

Filler. Plate used to build up the thickness of one component.

Fillet weld reinforcement. Fillet welds added to *groove welds*.

Fillet weld. Weld of generally triangular cross section made between intersecting surfaces of elements.

First-order analysis. Structural analysis in which equilibrium conditions are formulated on the undeformed structure; *second-order effects* are neglected.

Fitted bearing stiffener. Stiffener used at a support or concentrated *load* that fits tightly against one or both flanges of a *beam* so as to transmit load through bearing.

Flare bevel groove weld. Weld in a groove formed by a member with a curved surface in contact with a planar member.

Flare V-groove weld. Weld in a groove formed by two members with curved surfaces.

Flat width. Nominal width of rectangular *HSS* minus twice the outside corner radius. In absence of knowledge of the corner radius, the flat width may be taken as the total section width minus three times the thickness.

Flexural buckling. Buckling mode in which a compression member deflects laterally without twist or change in cross-sectional shape.

Flexural-torsional buckling†. Buckling mode in which a compression member bends and twists simultaneously without change in cross-sectional shape.

Force. Resultant of distribution of stress over a prescribed area.

Formed section. See *cold-formed steel structural member.*

Formed steel deck. In composite construction, *steel* cold formed into a decking profile used as a permanent concrete form.

Fully restrained moment connection. Connection capable of transferring moment with negligible rotation between connected members.

Gage. Transverse center-to-center spacing of *fasteners*.

Gap connection. HSS truss *connection* with a gap or space on the *chord* face between intersecting *branch members.*

General collapse. *Limit state* of *chord plastification* of opposing sides of a round *HSS chord member* at a cross-connection.

Geometric axis. Axis parallel to web, flange or angle leg.

Girder filler. Narrow piece of *sheet steel* used as a fill between the edge of a deck sheet and the flange of a girder in a *composite* floor system constructed using a *formed steel deck*.

Girder. See *Beam*.

Girt†. Horizontal structural member that supports wall panels and is primarily subjected to bending under horizontal *loads*, such as wind load.

Gouge. Relatively smooth surface groove or cavity resulting from plastic deformation or removal of material.

Gravity axis. Axis through the center of gravity of a member along its length.

Gravity frame. Portion of the framing system not included in the *lateral load resisting system*.

Gravity load. *Load*, such as that produced by dead and live loads, acting in the downward direction.

Grip (of bolt). Thickness of material through which a bolt passes.

Groove weld. Weld in a groove between connection elements. See also AWS D1.1.

Gusset plate. Plate element connecting truss members or a strut or brace to a *beam* or *column*.

Horizontal shear. Force at the interface between steel and concrete surfaces in a *composite beam*.

HSS. Square, rectangular or round hollow structural steel section produced in accordance with a *pipe* or tubing product specification.

User Note: A pipe can be designed using the same design rules for round *HSS* sections as long as it conforms to ASTM A53 Class B and the appropriate parameters are used in the design.

Inelastic analysis. *Structural analysis* that takes into account inelastic material behavior, including *plastic analysis*.

In-plane instability. *Limit state* of a *beam-column* bent about its major axis while *lateral buckling* or *lateral-torsional buckling* is prevented by *lateral bracing*.

Instability. *Limit state* reached in the loading of a structural component, frame or structure in which a slight disturbance in the *loads* or geometry produces large displacements.

Joint eccentricity. For *HSS* truss *connection*, perpendicular distance from *chord member* center of gravity to intersection of *branch member* work points.

Joint†. Area where two or more ends, surfaces, or edges are attached. Categorized by type of *fastener* or weld used and method of force transfer.

K-connection. HSS connection in which forces in *branch members* or connecting elements transverse to the *main member* are primarily equilibriated by forces in other branch members or connecting elements on the same side of the main member.

Lacing. Plate, angle or other steel shape, in a lattice configuration, that connects two steel shapes together.

Lap joint. Joint between two overlapping connection elements in parallel planes.

Lateral bracing. Diagonal bracing, *shear walls* or equivalent means for providing in-plane lateral stability.

Lateral load resisting system. Structural system designed to resist lateral loads and provide stability for the structure as a whole.

Lateral load. Load, such as that produced by wind or earthquake effects, acting in a lateral direction.

Lateral-torsional buckling. Buckling mode of a flexural member involving deflection normal to the plane of bending occurring simultaneously with twist about the shear center of the cross-section.

Leaning column. Column designed to carry *gravity loads* only, with *connections* that are not intended to provide resistance to *lateral loads.*

Length effects. Consideration of the reduction in strength of a member based on its *unbraced length*.

Limit state. Condition in which a structure or component becomes unfit for service and is judged either to be no longer useful for its intended function (*serviceability limit state*) or to have reached its ultimate load-carrying capacity (*strength limit state*).

Load†. Force or other action that results from the weight of building materials, occupants and their possessions, environmental effects, differential movement, or restrained dimensional changes.

Load effect†. Forces, stresses and deformations produced in a *structural component* by the applied *loads*.

Load factor†. Factor that accounts for deviations of the *nominal load* from the actual *load*, for uncertainties in the analysis that transforms the load into a *load effect* and for the probability that more than one extreme load will occur simultaneously.

*Local bending***. *Limit state* of large deformation of a flange under a concentrated tensile force.

*Local buckling***. *Limit state* of buckling of a compression element within a cross section.

*Local crippling***. *Limit state* of local failure of web plate in the immediate vicinity of a concentrated *load* or reaction.

*Local yielding**. Yielding* that occurs in a local area of an element.

LRFD (Load and Resistance Factor Design)†. Method of proportioning *structural components* such that the *design strength* equals or exceeds the *required strength* of the component under the action of the *LRFD load combinations*.

LRFD load combination†. Load combination in the *applicable building code* intended for strength design (*load and resistance factor design).*

Main member. For *HSS* connections, *chord member*, column or other HSS member to which *branch members* or other connecting elements are attached.

Mechanism. Structural system that includes a sufficient number of real hinges, plastic hinges or both, so as to be able to articulate in one or more rigid body modes.

Mill scale. Oxide surface coating on steel formed by the hot rolling process.

Milled surface. Surface that has been machined flat by a mechanically guided tool to a flat, smooth condition.

Moment connection. Connection that transmits bending moment between connected members.

Moment frame†. Framing system that provides resistance to lateral loads and provides stability to the *structural system*, primarily by shear and flexure of the framing members and their connections.

Net area. Gross area reduced to account for removed material.

Nodal brace. Brace that prevents lateral movement or twist independently of other braces at adjacent brace points (see *relative brace*).

Nominal dimension. Designated or theoretical dimension, as in the tables of section properties.

Nominal load†. Magnitude of the *load* specified by the *applicable building code*.

Nominal rib height. Height of *formed steel deck* measured from the underside of the lowest point to the top of the highest point.

*Nominal strength**†. Strength of a structure or component (without the *resistance factor* or *safety factor* applied) to resist *load effects*, as determined in accordance with this *Specification.*

Noncompact section. Section that can develop the *yield stress* in its compression elements before *local buckling* occurs, but cannot develop a *rotation capacity* of three.

Nondestructive testing. Inspection procedure wherein no material is destroyed and integrity of the material or component is not affected.

Notch toughness. Energy absorbed at a specified temperature as measured in the Charpy V-Notch test.

Notional load. Virtual *load* applied in a *structural analysis* to account for destabilizing effects that are not otherwise accounted for in the design provisions.

Out-of-plane buckling. Limit state of a beam-column bent about its major axis while lateral buckling or *lateral-torsional buckling* is not prevented by lateral bracing.

Overlap connection. HSS truss *connection* in which intersecting *branch members* overlap.

Panel zone. Web area of beam-to-column connection delineated by the extension of beam and column flanges through the connection, transmitting moment through a shear panel.

Partial-joint-penetration groove weld (PJP). Groove weld in which the penetration is intentionally less than the complete thickness of the connected element.

Partially restrained moment connection. Connection capable of transferring moment with rotation between connected members that is not negligible.

Percent elongation. Measure of ductility, determined in a tensile test as the maximum elongation of the gage length divided by the original gage length.

Permanent load†. *Load* in which variations over time are rare or of small magnitude. All other *loads* are *variable loads*.

Pipe. See *HSS*.

Pitch. Longitudinal center-to-center spacing of fasteners. Center-to-center spacing of bolt threads along axis of bolt.

Plastic analysis. Structural analysis based on the assumption of rigid-plastic behavior, in other words, that equilibrium is satisfied throughout the structure and the stress is at or below the yield stress.

Plastic hinge. Yielded zone that forms in a structural member when the *plastic moment* is attained. The member is assumed to rotate further as if hinged, except that such rotation is restrained by the *plastic moment*.

Plastic moment. Theoretical resisting moment developed within a fully yielded cross section.

Plastic stress distribution method. Method for determining the stresses in a composite member assuming that the steel section and the concrete in the cross section are fully plastic.

Plastification. In an HSS connection, *limit state* based on an out-of-plane flexural yield line *mechanism* in the *chord* at a *branch member* connection.

Plate girder. Built-up *beam*.

Plug weld. Weld made in a circular hole in one element of a joint fusing that element to another element.

Ponding. Retention of water due solely to the deflection of flat roof framing.

Post-buckling strength. Load or force that can be carried by an element, member, or frame after initial buckling has occurred.

Pretensioned joint. Joint with high-strength bolts tightened to the specified minimum pretension.

Properly developed. Reinforcing bars detailed to yield in a ductile manner before crushing of the concrete occurs. Bars meeting the provisions of ACI 318 insofar as development length, spacing and cover shall be deemed to be properly developed.

Prying action. Amplification of the tension force in a bolt caused by leverage between the point of applied load, the bolt and the reaction of the connected elements.

Punching load. Component of *branch member* force perpendicular to a *chord*.

Purlin†. Horizontal structural member that supports roof deck and is primarily subjected to bending under vertical loads such as snow, wind or dead loads.

P-δ effect. Effect of *loads* acting on the deflected shape of a member between joints or nodes.

P-Δ effect. Effect of *loads* acting on the displaced location of joints or nodes in a structure. In tiered building structures, this is the effect of *loads* acting on the laterally displaced location of floors and roofs.

Quality assurance. System of shop and field activities and controls implemented by the owner or his/her designated representative to provide confidence to the owner and the building authority that quality requirements are implemented.

Quality control. System of shop and field controls implemented by the fabricator and erector to ensure that contract and company fabrication and erection requirements are met.

Rational engineering analysis†. *Analysis* based on theory that is appropriate for the situation, relevant test data if available, and sound engineering judgment.

Reentrant. In a *cope* or weld access hole, a cut at an abrupt change in direction in which the exposed surface is concave.

Relative brace. Brace that controls the relative movement of two adjacent brace points along the length of a *beam* or *column* or the relative lateral displacement of two stories in a frame (see *nodal brace*).

*Required strength**†. Forces, stresses and deformations acting on the *structural component*, determined by either *structural analysis*, for the *LRFD* or *ASD load combinations*, as appropriate, or as specified by this *Specification* or Standard.

Resistance factor, φ†. Factor that accounts for unavoidable deviations of the *nominal strength* from the actual strength and for the manner and consequences of failure.

Reverse curvature. See *double curvature.*

Root of joint. Portion of a *joint* to be welded where the members are closest to each other.

Rotation capacity. Incremental angular rotation that a given shape can accept prior to excessive load shedding, defined as the ratio of the inelastic rotation attained to the idealized elastic rotation at first yield.

Rupture strength. In a *connection*, strength limited by tension or shear rupture.

Safety factor, Ω†. Factor that accounts for deviations of the actual strength from the nominal strength, deviations of the actual *load* from the *nominal load*, uncertainties in the analysis that transforms the load into a *load effect*, and for the manner and consequences of failure.

Second-order analysis. Structural analysis in which equilibrium conditions are formulated on the deformed structure; *second-order effects* (both P-δ and P-Δ, unless specified otherwise) are included.

Second-order effect. Effect of *loads* acting on the deformed configuration of a structure; includes P-δ *effect* and P-Δ *effect*.

Seismic response modification coefficient. Factor that reduces seismic *load effects* to strength level.

Service load combination. Load combination under which serviceability limit states are evaluated.

Service load†. *Load* under which *serviceability limit states* are evaluated.

Serviceability limit state. Limiting condition affecting the ability of a structure to preserve its appearance, maintainability, durability or the comfort of its occupants or function of machinery, under normal usage.

Shear buckling. Buckling mode in which a plate element, such as the web of a beam, deforms under pure shear applied in the plane of the plate.

Shear connector. Headed stud, channel, plate or other shape welded to a steel member and embedded in concrete of a *composite member* to transmit shear forces at the interface between the two materials.

Shear connector strength. Limit state of reaching the strength of a *shear connector,* as governed by the connector bearing against the concrete in the slab or by the *tensile strength* of the connector.

Shear rupture. Limit state *of rupture (fracture) due to shear.*

Shear wall†. Wall that provides resistance to lateral loads in the plane of the wall and provides stability for the structural system.

Shear yielding. Yielding that occurs due to shear.

Shear yielding (punching). In an HSS connection, *limit state* based on out-of-plane shear strength of the *chord* wall to which *branch members* are attached.

Sheet steel. In a composite floor system, steel used for closure plates or miscellaneous trimming in a *formed steel deck*.

Shim. Thin layer of material used to fill a space between faying or bearing surfaces.

Sidesway buckling. *Limit state* of lateral buckling of the tension flange opposite the location of a concentrated compression force.

Sidewall crippling. *Limit state* of *web crippling* of the sidewalls of a *chord member* at a *HSS* truss *connection*.

Sidewall crushing. *Limit state* based on bearing strength of *chord member* sidewall in *HSS* truss *connection*.

Simple connection. Connection that transmits negligible bending moment between connected members.

Single-concentrated force. Tensile or compressive force applied normal to the flange of a member.

Single curvature. Deformed shape of a beam with no inflection point within the span.

Slender-element section. Cross section possessing plate components of sufficient slenderness such that *local buckling* in the elastic range will occur.

Slip. In a bolted connection, *limit state* of relative motion of connected parts prior to the attainment of the *available strength* of the connection.

Slip-critical connection. Bolted *connection* designed to resist movement by friction on the faying surface of the connection under the clamping forces of the bolts.

Slot weld. Weld made in an elongated hole fusing an element to another element.

Snug-tightened joint. Joint with the connected plies in firm contact as specified in Chapter J.

Specified minimum tensile strength. Lower limit of *tensile strength* specified for a material as defined by ASTM.

Specified minimum yield stress†. Lower limit of *yield stress* specified for a material as defined by ASTM.

Splice. Connection between two structural elements joined at their ends to form a single, longer element.

Stability. Condition reached in the loading of a structural component, frame or structure in which a slight disturbance in the *loads* or geometry does not produce large displacements.

Stiffened element. Flat compression element with adjoining out-of-plane elements along both edges parallel to the direction of loading.

Stiffener. Structural element, usually an angle or plate, attached to a *member* to distribute *load*, transfer shear or prevent buckling.

Stiffness. Resistance to deformation of a member or structure, measured by the ratio of the applied force (or moment) to the corresponding displacement (or rotation).

Strain compatibility method. Method for determining the stresses in a composite member considering the stress-strain relationships of each material and its location with respect to the neutral axis of the cross section.

Strength limit state. Limiting condition affecting the safety of the structure, in which the ultimate load-carrying capacity is reached.

Stress. Force per unit area caused by axial force, moment, shear or torsion.

Stress concentration. Localized stress considerably higher than average (even in uniformly loaded cross sections of uniform thickness) due to abrupt changes in geometry or localized loading.

Strong axis. Major principal centroidal axis of a cross section.

Structural analysis†. Determination of *load effects* on members and *connections* based on principles of structural mechanics.

Structural component†. Member, connector, connecting element or assemblage.

Structural steel. Steel elements as defined in Section 2.1 of the AISC *Code of Standard Practice for Steel Buildings and Bridges*.

Structural system. An assemblage of load-carrying components that are joined together to provide interaction or interdependence.

T-connection. HSS connection in which the *branch member* or connecting element is perpendicular to the *main member* and in which forces transverse to the main member are primarily equilibriated by shear in the main member.

Tensile rupture. Limit state of rupture (fracture) due to tension.

Tensile strength (of material)†. Maximum tensile stress that a material is capable of sustaining as defined by ASTM.

Tensile strength (of member). Maximum tension force that a member is capable of sustaining.

Tensile yielding. Yielding *that occurs due to tension.*

Tension and shear rupture. In a bolt, *limit state* of rupture (fracture) due to simultaneous tension and shear *force*.

Tension field action. Behavior of a panel under shear in which diagonal tensile forces develop in the web and compressive forces develop in the *transverse stiffeners* in a manner similar to a Pratt truss.

Thermally cut. Cut with gas, plasma or laser.

Tie plate. Plate element used to join two parallel components of a *built-up column*, girder or strut rigidly connected to the parallel components and designed to transmit shear between them.

Toe of fillet. Junction of a fillet weld face and base metal. Tangent point of a rolled section fillet.

Torsional bracing. Bracing resisting twist of a *beam* or *column*.

Torsional buckling. Buckling mode in which a compression member twists about its shear center axis.

Torsional yielding. Yielding that occurs due to torsion.

Transverse reinforcement. Steel reinforcement in the form of closed ties or welded wire fabric providing confinement for the concrete surrounding the steel shape core in an *encased concrete composite column*.

Transverse stiffener. Web *stiffener* oriented perpendicular to the flanges, attached to the web.

Tubing. See *HSS*.

Turn-of-nut method. Procedure whereby the specified pretension in high-strength bolts is controlled by rotating the fastener component a predetermined amount after the bolt has been snug tightened.

Unbraced length. Distance between braced points of a member, measured between the centers of gravity of the bracing members.

Uneven load distribution. In an *HSS connection*, condition in which the load is not distributed through the cross section of connected elements in a manner that can be readily determined.

Unframed end. The end of a member not restrained against rotation by stiffeners or connection elements.

Unstiffened element. Flat compression element with an adjoining out-of-plane element along one edge parallel to the direction of loading.

Variable load†. *Load* not classified as *permanent load*.

Vertical bracing system. System of *shear walls*, *braced frames* or both, extending through one or more floors of a building.

Weak axis. Minor principal centroidal axis of a cross section.

Weathering steel. High-strength, low-alloy steel that, with suitable precautions, can be used in normal atmospheric exposures (not marine) without protective paint coating.

Web buckling. Limit state of lateral instability of a web.

Web compression buckling. *Limit state* of out-of-plane compression buckling of the web due to a concentrated compression force.

Web sidesway buckling. *Limit state* of lateral buckling of the tension flange opposite the location of a concentrated compression force.

Weld metal. Portion of a fusion weld that has been completely melted during welding. Weld metal has elements of filler metal and base metal melted in the weld thermal cycle.

Weld root. See *root of joint*.

Y-connection. HSS connection in which the *branch member* or connecting element is not perpendicular to the *main member* and in which forces transverse to the main member are primarily equilibriated by shear in the main member.

Yield moment. In a member subjected to bending, the moment at which the extreme outer fiber first attains the *yield stress.*

Yield point†. First stress in a material at which an increase in strain occurs without an increase in stress as defined by ASTM.

Yield strength†. *Stress* at which a material exhibits a specified limiting deviation from the proportionality of stress to strain as defined by ASTM.

Yield stress†. Generic term to denote either *yield point* or *yield strength*, as appropriate for the material.

Yielding. Limit state of inelastic deformation that occurs after the *yield stress* is reached.

Yielding (plastic moment). Yielding throughout the cross section of a member as the bending moment reaches the *plastic moment.*

Yielding (yield moment). Yielding at the extreme fiber on the cross section of a member when the bending moment reaches the *yield moment.*

CHAPTER A

GENERAL PROVISIONS

This chapter states the scope of the Specification, summarizes referenced specification, code, and standard documents, and provides requirements for materials and contract documents.

The chapter is organized as follows:

A1. Scope
A2. Referenced Specifications, Codes and Standards
A3. Material
A4. Structural Design Drawings and Specifications

A1. SCOPE

The *Specification for Structural Steel Buildings*, hereafter referred to as the Specification, shall apply to the design of the *structural steel* system, where the steel elements are defined in the AISC *Code of Standard Practice for Steel Buildings and Bridges*, Section 2.1.

This Specification includes the Symbols, the Glossary, Chapters A through M, and Appendices 1 through 7. The Commentary and the User Notes interspersed throughout are not part of the Specification.

User Note: User notes are intended to provide concise and practical guidance in the application of the provisions.

This Specification sets forth criteria for the design, fabrication, and erection of *structural steel* buildings and other structures, where other structures are defined as those structures designed, fabricated, and erected in a manner similar to buildings, with building-like vertical and lateral load resisting elements. Where conditions are not covered by the Specification, designs are permitted to be based on tests or analysis, subject to the approval of the *authority having jurisdiction*. Alternate methods of analysis and design shall be permitted, provided such alternate methods or criteria are acceptable to the authority having jurisdiction.

User Note: For the design of structural members, other than hollow structural sections (*HSS*), that are cold-formed to shapes, with elements not more than 1 in. (25 mm) in thickness, the provisions in the AISI *North American Specification for the Design of Cold-Formed Steel Structural Members* are recommended.

1. Low-Seismic Applications

When the *seismic response modification coefficient*, *R*, (as specified in the *applicable building code*) is taken equal to or less than 3, the design, fabrication, and erection of structural-steel-framed buildings and other structures shall comply with this Specification.

2. High-Seismic Applications

When the *seismic response modification coefficient*, *R*, (as specified in the *applicable building code*) is taken greater than 3, the design, fabrication and erection of structural-steel-framed buildings and other structures shall comply with the requirements in the *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341), in addition to the provisions of this Specification.

3. Nuclear Applications

The design of nuclear structures shall comply with the requirements of the *Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures in Nuclear Facilities* (ANSI/AISC N690) including Supplement No. 2 or the *Load and Resistance Factor Design Specification for Steel Safety-Related Structures for Nuclear Facilities* (ANSI/AISC N690L), in addition to the provisions of this Specification.

A2. REFERENCED SPECIFICATIONS, CODES AND STANDARDS

The following specifications, codes and standards are referenced in this Specification:

<u>ACI International (ACI)</u>

ACI 318-02 *Building Code Requirements for Structural Concrete and Commentary*

ACI 318M-02 *Metric Building Code Requirements for Structural Concrete and Commentary*

<u>American Institute of Steel Construction, Inc. (AISC)</u>

AISC 303-05 *Code of Standard Practice for Steel Buildings and Bridges*

ANSI/AISC 341-05 *Seismic Provisions for Structural Steel Buildings*

ANSI/AISC N690-1994(R2004) *Specification for the Design, Fabrication and Erection of Steel Safety-Related Structures for Nuclear Facilities,* including Supplement No. 2

ANSI/AISC N690L-03 *Load and Resistance Factor Design Specification for Steel Safety-Related Structures for Nuclear Facilities*

<u>American Society of Civil Engineers (ASCE)</u>

SEI/ASCE 7-02 *Minimum Design Loads for Buildings and Other Structures*

ASCE/SFPE 29-99 *Standard Calculation Methods for Structural Fire Protection*

<u>American Society of Mechanical Engineers (ASME)</u>

ASME B18.2.6-96 *Fasteners for Use in Structural Applications*

ASME B46.1-95 *Surface Texture, Surface Roughness, Waviness, and Lay*

ASTM International (ASTM)

A6/A6M-04a *Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling*

A36/A36M-04 *Standard Specification for Carbon Structural Steel*

A53/A53M-02 *Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless*

A193/A193M-04a *Standard Specification for Alloy-Steel and Stainless Steel Bolting Materials for High-Temperature Service*

A194/A194M-04 *Standard Specification for Carbon and Alloy Steel Nuts for Bolts for High Pressure or High-Temperature Service, or Both*

A216/A216M-93(2003) *Standard Specification for Steel Castings, Carbon, Suitable for Fusion Welding, for High Temperature Service*

A242/A242M-04 *Standard Specification for High-Strength Low-Alloy Structural Steel*

A283/A283M-03 *Standard Specification for Low and Intermediate Tensile Strength Carbon Steel Plates*

A307-03 *Standard Specification for Carbon Steel Bolts and Studs, 60,000 PSI Tensile Strength*

A325-04 *Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength*

A325M-04 *Standard Specification for High-Strength Bolts for Structural Steel Joints (Metric)*

A354-03a *Standard Specification for Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners*

A370-03a *Standard Test Methods and Definitions for Mechanical Testing of Steel Products*

A449-04 *Standard Specification for Quenched and Tempered Steel Bolts and Studs*

A490-04 *Standard Specification for Heat-Treated Steel Structural Bolts, 150 ksi Minimum Tensile Strength*

A490M-04 *Standard Specification for High-Strength Steel Bolts, Classes 10.9 and 10.9.3, for Structural Steel Joints (Metric)*

A500-03a *Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes*

A501-01 *Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing*

A502-03 *Standard Specification for Steel Structural Rivets*

A514/A514M-00a *Standard Specification for High-Yield Strength, Quenched and Tempered Alloy Steel Plate, Suitable for Welding*

A529/A529M-04 *Standard Specification for High-Strength Carbon-Manganese Steel of Structural Quality*

A563-04 *Standard Specification for Carbon and Alloy Steel Nuts*

A563M-03 *Standard Specification for Carbon and Alloy Steel Nuts [Metric]*

A568/A568M-03 *Standard Specification for Steel, Sheet, Carbon, and High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, General Requirements for*

A572/A572M-04 *Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel*

A588/A588M-04 *Standard Specification for High-Strength Low-Alloy Structural Steel with 50 ksi [345 MPa] Minimum Yield Point to 4 in. [100 mm] Thick*

A606-04 *Standard Specification for Steel, Sheet and Strip, High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance*

A618/A618M-04 *Standard Specification for Hot-Formed Welded and Seamless High-Strength Low-Alloy Structural Tubing*

A673/A673M-04 *Standard Specification for Sampling Procedure for Impact Testing of Structural Steel*

A668/A668M-04 *Standard Specification for Steel Forgings, Carbon and Alloy, for General Industrial Use*

A709/A709M-04 *Standard Specification for Carbon and High-Strength Low-Alloy Structural Steel Shapes, Plates, and Bars and Quenched-and-Tempered Alloy Structural Steel Plates for Bridges*

A751-01 *Standard Test Methods, Practices, and Terminology for Chemical Analysis of Steel Products*

A847-99a(2003) *Standard Specification for Cold-Formed Welded and Seamless High-Strength, Low-Alloy Structural Tubing with Improved Atmospheric Corrosion Resistance*

A852/A852M-03 *Standard Specification for Quenched and Tempered Low-Alloy Structural Steel Plate with 70 ksi [485 MPa] Minimum Yield Strength to 4 in. [100 mm] Thick*

A913/A913M-04 *Standard Specification for High-Strength Low-Alloy Steel Shapes of Structural Quality, Produced by Quenching and Self-Tempering Process (QST)*

A992/A992M-04 *Standard Specification for Steel for Structural Shapes for Use in Building Framing*

User Note: ASTM A992 is the most commonly referenced specification for W shapes.

A1011/A1011M-04 *Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability*

C33-03 *Standard Specification for Concrete Aggregates*

C330-04 *Standard Specification for Lightweight Aggregates for Structural Concrete*

E119-00a *Standard Test Methods for Fire Tests of Building Construction and Materials*

E709-01 *Standard Guide for Magnetic Particle Examination*

F436-03 *Standard Specification for Hardened Steel Washers*

F959-02 *Standard Specification for Compressible-Washer-Type Direct Tension Indicators for Use with Structural Fasteners*

F1554-99 *Standard Specification for Anchor Bolts, Steel, 36, 55, and 105 ksi Yield Strength*

User Note: ASTM F1554 is the most commonly referenced specification for anchor rods. Grade and weldability must be specified.

F1852-04 *Standard Specification for "Twist-Off" Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength*

American Welding Society (AWS)

AWS D1.1/D1.1M-2004 *Structural Welding Code–Steel*

AWS A5.1-2004 *Specification for Carbon Steel Electrodes for Shielded Metal Arc Welding*

AWS A5.5-96 *Specification for Low-Alloy Steel Electrodes for Shielded Metal Arc Welding*

AWS A5.17/A5.17M-97 *Specification for Carbon Steel Electrodes and Fluxes for Submerged Arc Welding*

AWS A5.18:2001 *Specification for Carbon Steel Electrodes and Rods for Gas Shielded Arc Welding*

AWS A5.20-95 *Specification for Carbon Steel Electrodes for Flux Cored Arc Welding*

AWS A5.23/A5.23M-97 *Specification for Low-Alloy Steel Electrodes and Fluxes for Submerged Arc Welding*

AWS A5.25/A5.25M-97 *Specification for Carbon and Low-Alloy Steel Electrodes and Fluxes for Electroslag Welding*

AWS A5.26/A5.26M-97 *Specification for Carbon and Low-Alloy Steel Electrodes for Electrogas Welding*

AWS A5.28-96 *Specification for Low-Alloy Steel Electrodes and Rods for Gas Shielded Arc Welding*

AWS A5.29:1998 *Specification for Low-Alloy Steel Electrodes for Flux Cored Arc Welding*

Research Council on Structural Connections (RCSC)

Specification for Structural Joints Using ASTM A325 or A490 Bolts, 2004

A3. MATERIAL

1. Structural Steel Materials

Material test reports or reports of tests made by the fabricator or a testing laboratory shall constitute sufficient evidence of conformity with one of the above listed ASTM standards. For hot-rolled structural shapes, plates, and bars, such tests shall be made in accordance with ASTM A6/A6M; for sheets, such tests shall be made in accordance with ASTM A568/A568M; for *tubing* and *pipe*, such tests shall be made in accordance with the requirements of the applicable ASTM standards listed above for those product forms. If requested, the fabricator shall provide an affidavit stating that the *structural steel* furnished meets the requirements of the grade specified.

1a. ASTM Designations

Structural steel material conforming to one of the following ASTM specifications is approved for use under this Specification:

(1) Hot-rolled structural shapes

ASTM A36/A36M
ASTM A529/A529M
ASTM A572/A572M
ASTM A588/A588M
ASTM A709/A709M
ASTM A913/A913M
ASTM A992/ A992M

(2) Structural tubing

ASTM A500
ASTM A501
ASTM A618
ASTM A847

(3) Pipe

ASTM A53/A53M, Gr. B

(4) Plates

ASTM A36/A36M
ASTM A242/A242M
ASTM A283/A283M
ASTM A514/A514M
ASTM A529/A529M
ASTM A572/A572M
ASTM A588/A588M
ASTM A709/A709M
ASTM A852/A852M
ASTM A1011/A1011M

(5) Bars

ASTM A36/A36M
ASTM A529/A529M
ASTM A572/A572M
ASTM A709/A709M

(6) Sheets

ASTM A606
A1011/A1011M SS, HSLAS, AND HSLAS-F

1b. Unidentified Steel

Unidentified steel free of injurious defects 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. Rolled Heavy Shapes

ASTM A6/A6M hot-rolled shapes with a flange thickness exceeding 2 in. (50 mm), used as members subject to primary (computed) tensile *force*s due to tension or flexure and spliced using *complete-joint-penetration groove weld*s that fuse through the thickness of the member, shall be specified as follows. The contract documents shall require that such shapes be supplied with *Charpy V-Notch (CVN) impact test* results in accordance with ASTM A6/A6M, Supplementary Requirement S30, *Charpy V-Notch Impact Test for Structural Shapes – Alternate Core Location*. The impact test shall meet a minimum average value of 20 ft-lbs (27 J) absorbed energy at +70 °F (+21 °C).

The above requirements do not apply if the *splice*s and *connection*s are made by bolting. The above requirements do not apply to hot-rolled shapes with a flange thickness exceeding 2 in. (50 mm) that have shapes with flange or web elements less than 2 in. (50 mm) thick welded with *complete-joint-penetration groove weld*s to the face of the shapes with thicker elements.

> **User Note:** Additional requirements for joints in heavy rolled members are given in Sections J1.5, J1.6, J2.7, and M2.2.

1d. Built-Up Heavy Shapes

Built-up cross-sections consisting of plates with a thickness exceeding 2 in. (50 mm), used as members subject to primary (computed) tensile *force*s due to tension or flexure and spliced or connected to other members using *complete-joint-penetration groove weld*s that fuse through the thickness of the plates, shall be specified as follows. The contract documents shall require that the steel be supplied with *Charpy V-Notch impact test* results in accordance with ASTM A6/A6M, Supplementary Requirement S5, *Charpy V-Notch Impact Test.* The impact test shall be conducted 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 requirements also apply to built-up cross-sections consisting of plates exceeding 2 in. (50 mm) that are welded with complete-joint-penetration groove welds to the face of other sections.

> **User Note:** Additional requirements for joints in heavy built-up members are given in Sections J1.5, J1.6, J2.7, and M2.2.

2. Steel Castings and Forgings

Cast steel shall conform to ASTM A216/A216M, Gr. WCB with Supplementary Requirement S11. Steel forgings shall conform to ASTM A668/A668M. Test reports produced in accordance with the above reference standards shall constitute sufficient evidence of conformity with such standards.

3. Bolts, Washers and Nuts

Bolt, washer, and nut material conforming to one of the following ASTM specifications is approved for use under this Specification:

(1) Bolts:

ASTM A307
ASTM A325
ASTM A325M
ASTM A449
ASTM A490
ASTM A490M
ASTM F1852

(2) Nuts:

ASTM A194/A194M
ASTM A563
ASTM A563M

(3) Washers:

ASTM F436
ASTM F436M

(4) Compressible-Washer-Type Direct Tension Indicators:

ASTM F959
ASTM F959M

Manufacturer's certification shall constitute sufficient evidence of conformity with the standards.

4. Anchor Rods and Threaded Rods

Anchor rod and threaded rod material conforming to one of the following ASTM specifications is approved for use under this Specification:

ASTM A36/A36M
ASTM A193/A193M
ASTM A354
ASTM A449
ASTM A572/A572M
ASTM A588/A588M
ASTM F1554

User Note: ASTM F1554 is the preferred material specification for anchor rods.

A449 material is acceptable for high-strength anchor rods and threaded rods of any diameter.

Threads on anchor rods and threaded rods shall conform to the Unified Standard Series of ASME B18.2.6 and shall have Class 2A tolerances.

Manufacturer's certification shall constitute sufficient evidence of conformity with the standards.

5. Filler Metal and Flux for Welding

Filler metals and fluxes shall conform to one of the following specifications of the American Welding Society:

AWS A5.1
AWS A5.5
AWS A5.17/A5.17M
AWS A5.18
AWS A5.20
AWS A5.23/A5.23M
AWS A5.25/A5.25M
AWS A5.26/A5.26M
AWS A5.28
AWS A5.29
AWS A5.32/A5.32M

Manufacturer's certification shall constitute sufficient evidence of conformity with the standards. Filler metals and fluxes that are suitable for the intended application shall be selected.

6. Stud Shear Connectors

Steel stud *shear connector*s shall conform to the requirements of *Structural Welding Code–Steel*, AWS D1.1.

> **User Note:** Studs are made from cold drawn bar, either semi-killed or killed aluminum or silicon deoxidized, conforming to the requirements of ASTM A29/A29M-04, Standard Specification for Steel Bars, Carbon and Alloy, Hot-Wrought, General Requirements for.

Manufacturer's certification shall constitute sufficient evidence of conformity with AWS D1.1.

A4. STRUCTURAL DESIGN DRAWINGS AND SPECIFICATIONS

The design drawings and specifications shall meet the requirements in the *Code of Standard Practice for Steel Buildings and Bridges*, except for deviations specifically identified in the design drawings and/or specifications.

CHAPTER B

DESIGN REQUIREMENTS

The general requirements for the analysis and design of steel structures that are applicable to all chapters of the specification are given in this chapter.

The chapter is organized as follows:

B1. GENERAL PROVISIONS

The design of members and *connection*s shall be consistent with the intended behavior of the framing system and the assumptions made in the *structural analysis*. Unless restricted by the *applicable building code*, *lateral load* resistance and *stability* may be provided by any combination of members and connections.

B2. LOADS AND LOAD COMBINATIONS

The loads and load combinations shall be as stipulated by the *applicable building code*. In the absence of a building code, the loads and load combinations shall be those stipulated in SEI/ASCE 7. For design purposes, the *nominal load*s shall be taken as the *load*s stipulated by the applicable building code.

User Note: For LRFD designs, the load combinations in SEI/ASCE 7, Section 2.3 apply. For ASD designs, the load combinations in SEI/ASCE 7, Section 2.4 apply.

B3. DESIGN BASIS

Designs shall be made according to the provisions for *Load and Resistance Factor Design* (LRFD) or to the provisions for *Allowable Strength Design* (ASD).

1. Required Strength

The *required strength* of structural members and *connection*s shall be determined by *structural analysis* for the appropriate load combinations as stipulated in Section B2.

Design by *elastic*, *inelastic* or *plastic analysis* is permitted. Provisions for inelastic and plastic analysis are as stipulated in Appendix 1, Inelastic Analysis and Design. The provisions for moment redistribution in continuous beams in Appendix 1, Section 1.3 are permitted for elastic analysis only.

2. Limit States

Design shall be based on the principle that no applicable strength or serviceability *limit state* shall be exceeded when the structure is subjected to all appropriate load combinations.

3. Design for Strength Using Load and Resistance Factor Design (LRFD)

Design according to the provisions for *Load and Resistance Factor Design* (LRFD) satisfies the requirements of this Specification when the *design strength* of each *structural component* equals or exceeds the *required strength* determined on the basis of the *LRFD load combinations*. All provisions of this Specification, except for those in Section B3.4, shall apply.

Design shall be performed in accordance with Equation B3-1:

$$R_u \leq \phi R_n \tag{B3-1}$$

where

R_u = required strength (LRFD)
R_n = *nominal strength*, specified in Chapters B through K
ϕ = *resistance factor*, specified in Chapters B through K
ϕR_n = design strength

4. Design for Strength Using Allowable Strength Design (ASD)

Design according to the provisions for *Allowable Strength Design* (ASD) satisfies the requirements of this Specification when the *allowable strength* of each *structural component* equals or exceeds the *required strength* determined on the basis of the *ASD load combinations*. All provisions of this Specification, except those of Section B3.3, shall apply.

Design shall be performed in accordance with Equation B3-2:

$$R_a \leq R_n/\Omega \tag{B3-2}$$

where

R_a = required strength (ASD)
R_n = *nominal strength*, specified in Chapters B through K
Ω = safety factor, specified in Chapters B through K
R_n/Ω = allowable strength

5. Design for Stability

Stability of the structure and its elements shall be determined in accordance with Chapter C.

6. Design of Connections

Connection elements shall be designed in accordance with the provisions of Chapters J and K. The *forces* and deformations used in design shall be consistent with the intended performance of the connection and the assumptions used in the *structural analysis*.

User Note: Section 3.1.2 of the *Code of Standard Practice* addresses communication of necessary information for the design of connections.

6a. Simple Connections

A simple connection transmits a negligible moment across the connection. In the analysis of the structure, simple connections may be assumed to allow unrestrained relative rotation between the framing elements being connected. A simple connection shall have sufficient rotation capacity to accommodate the required rotation determined by the analysis of the structure. Inelastic rotation of the connection is permitted.

6b. Moment Connections

A moment connection transmits moment across the connection. Two types of moment connections, FR and PR, are permitted, as specified below.

(a) *Fully-Restrained (FR) Moment Connections*
A fully-restrained (FR) moment connection transfers moment with a negligible rotation between the connected members. In the analysis of the structure, the connection may be assumed to allow no relative rotation. An FR connection shall have sufficient strength and stiffness to maintain the angle between the connected members at the strength limit states.

(b) *Partially-Restrained (PR) Moment Connections*
Partially-restrained (PR) moment connections transfer moments, but the rotation between connected members is not negligible. In the analysis of the structure, the force-deformation response characteristics of the connection shall be included. The response characteristics of a PR connection shall be documented in the technical literature or established by analytical or experimental means. The component elements of a PR connection shall have sufficient strength, stiffness, and deformation capacity at the strength limit states.

7. Design for Serviceability

The overall structure and the individual members, *connection*s, and connectors shall be checked for serviceability. Performance requirements for serviceability design are given in Chapter L.

8. Design for Ponding

The roof system shall be investigated through *structural analysis* to assure adequate strength and *stability* under *ponding* conditions, unless the roof surface is provided with a slope of $^1/_4$ in. per ft (20 mm per meter) or greater toward points of free drainage or an adequate system of drainage is provided to prevent the accumulation of water.

See Appendix 2, Design for Ponding, for methods of checking ponding.

9. Design for Fatigue

Fatigue shall be considered in accordance with Appendix 3, Design for Fatigue, for members and their *connection*s subject to repeated *loading*. Fatigue need not be considered for seismic effects or for the effects of wind loading on normal building *lateral load resisting systems* and building enclosure components.

10. Design for Fire Conditions

Two methods of design for fire conditions are provided in Appendix 4, Structural Design for Fire Conditions: Qualification Testing and Engineering Analysis. Compliance with the fire protection requirements in the *applicable building code* shall be deemed to satisfy the requirements of this section and Appendix 4.

Nothing in this section is intended to create or imply a contractual requirement for the *engineer of record* responsible for the structural design or any other member of the design team.

> **User Note:** Design by qualification testing is the prescriptive method specified in most building codes. Traditionally, on most projects where the architect is the prime professional, the architect has been the responsible party to specify and coordinate fire protection requirements. Design by Engineering Analysis is a new engineering approach to fire protection. Designation of the person(s) responsible for designing for fire conditions is a contractual matter to be addressed on each project.

11. Design for Corrosion Effects

Where corrosion may impair the strength or serviceability of a structure, *structural components* shall be designed to tolerate corrosion or shall be protected against corrosion.

12. Design Wall Thickness for HSS

The *design wall thickness*, t, shall be used in calculations involving the wall thickness of hollow structural sections (*HSS*). The design wall thickness, t, shall be taken equal to 0.93 times the nominal wall thickness for electric-resistance-welded (ERW) HSS and equal to the nominal thickness for submerged-arc-welded (SAW) HSS.

13. Gross and Net Area Determination

a. Gross Area

The gross area, A_g, of a member is the total cross-sectional area.

b. Net Area

The *net area*, A_n, of a member is the sum of the products of the thickness and the net width of each element computed as follows:

In computing net area for tension and shear, the width of a bolt hole shall be taken as $^1/_{16}$ in. (2 mm) greater than the *nominal dimension* of the hole.

For a chain of holes extending across a part in any diagonal or zigzag line, the net width of the part shall be obtained by deducting from the gross width the sum of the diameters or slot dimensions as provided in Section J3.2, of all holes in the chain, and adding, for each *gage* space in the chain, the quantity $s^2/4g$

where

s = longitudinal center-to-center spacing (*pitch*) of any two consecutive holes, in. (mm)

g = transverse center-to-center spacing (gage) between *fastener* gage lines, in. (mm)

For angles, the gage for holes in opposite adjacent legs shall be the sum of the gages from the back of the angles less the thickness.

For slotted *HSS* welded to a *gusset plate*, the net area, A_n, is the gross area minus the product of the thickness and the total width of material that is removed to form the slot.

In determining the net area across plug or *slot weld*s, the *weld metal* shall not be considered as adding to the net area.

User Note: Section J4.1(b) limits A_n to a maximum of $0.85A_g$ for splice plates with holes.

B4. CLASSIFICATION OF SECTIONS FOR LOCAL BUCKLING

Sections are classified as *compact*, *noncompact*, or *slender-element section*s. For a section to qualify as compact its flanges must be continuously connected to the web or webs and the width-thickness ratios of its compression elements must not exceed the limiting width-thickness ratios λ_p from Table B4.1. If the width-thickness ratio of one or more compression elements exceeds λ_p, but does not exceed λ_r from Table B4.1, the section is noncompact. If the width-thickness ratio of any element exceeds λ_r, the section is referred to as a *slender-element section*.

1. Unstiffened Elements

For *unstiffened elements* supported along only one edge parallel to the direction of the compression *force*, the width shall be taken as follows:

(a) For flanges of I-shaped members and tees, the width b is one-half the full-flange width, b_f.
(b) For legs of angles and flanges of channels and zees, the width b is the full *nominal dimension*.
(c) For plates, the width b is the distance from the free edge to the first row of *fasteners* or line of welds.
(d) For stems of tees, d is taken as the full nominal depth of the section.

User Note: Refer to Table B4.1 for the graphic representation of unstiffened element dimensions.

2. Stiffened Elements

For *stiffened elements* supported along two edges parallel to the direction of the compression *force*, the width shall be taken as follows:

(a) For webs of rolled or *formed sections*, h is the clear distance between flanges less the fillet or corner radius at each flange; h_c is twice the distance from the centroid to the inside face of the compression flange less the fillet or corner radius.
(b) For webs of built-up sections, h is the distance between adjacent lines of *fasteners* or the clear distance between flanges when welds are used, and h_c is twice the distance from the centroid to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used; h_p is twice the distance from the plastic neutral axis to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used.
(c) For flange or *diaphragm plates* in built-up sections, the width b is the distance between adjacent lines of fasteners or lines of welds.
(d) For flanges of rectangular hollow structural sections (*HSS*), the width b is the clear distance between webs less the inside corner radius on each side. For webs of rectangular HSS, h is the clear distance between the flanges less the inside corner radius on each side. If the corner radius is not known, b and h shall be taken as the corresponding outside dimension minus three times the thickness. The thickness, t, shall be taken as the *design wall thickness*, per Section B3.12.

User Note: Refer to Table B4.1 for the graphic representation of stiffened element dimensions.

For tapered flanges of rolled sections, the thickness is the nominal value halfway between the free edge and the corresponding face of the web.

TABLE B4.1
Limiting Width-Thickness Ratios for Compression Elements

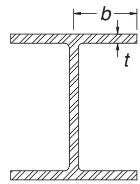
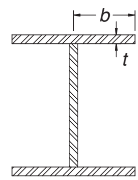
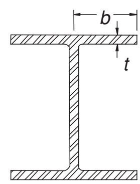
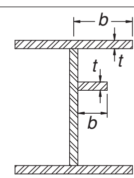
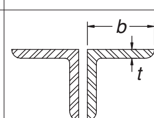
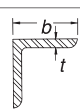
	Case	Description of Element	Width Thick-ness Ratio	Limiting Width-Thickness Ratios λ_p (compact)	Limiting Width-Thickness Ratios λ_r (noncompact)	Example
Unstiffened Elements	1	Flexure in flanges of rolled I-shaped sections and channels	b/t	$0.38\sqrt{E/F_y}$	$1.0\sqrt{E/F_y}$	
Unstiffened Elements	2	Flexure in flanges of doubly and singly symmetric I-shaped built-up sections	b/t	$0.38\sqrt{E/F_y}$	$0.95\sqrt{k_c E/F_L}$ [a],[b]	
Unstiffened Elements	3	Uniform compression in flanges of rolled I-shaped sections, plates projecting from rolled I-shaped sections; outstanding legs of pairs of angles in continuous contact and flanges of channels	b/t	NA	$0.56\sqrt{E/F_y}$	
Unstiffened Elements	4	Uniform compression in flanges of built-up I-shaped sections and plates or angle legs projecting from built-up I-shaped sections	b/t	NA	$0.64\sqrt{k_c E/F_y}$ [a]	
Unstiffened Elements	5	Uniform compression in legs of single angles, legs of double angles with separators, and all other unstiffened elements	b/t	NA	$0.45\sqrt{E/F_y}$	
Unstiffened Elements	6	Flexure in legs of single angles	b/t	$0.54\sqrt{E/F_y}$	$0.91\sqrt{E/F_y}$	

TABLE B4.1 (cont.)
Limiting Width-Thickness Ratios for Compression Elements

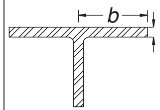
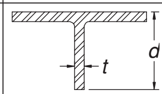
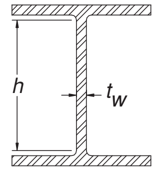
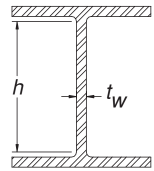
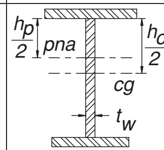
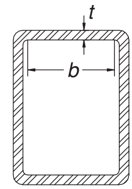
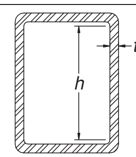
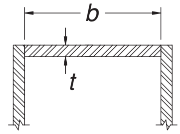
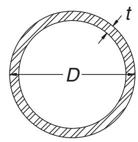
	Case	Description of Element	Width Thick-ness Ratio	Limiting Width-Thickness Ratios λ_p (compact)	λ_r (noncompact)	Example
	7	Flexure in flanges of tees	b/t	$0.38\sqrt{E/F_y}$	$1.0\sqrt{E/F_y}$	
	8	Uniform compression in stems of tees	d/t	NA	$0.75\sqrt{E/F_y}$	
Stiffened Elements	9	Flexure in webs of doubly symmetric I-shaped sections and channels	h/t_w	$3.76\sqrt{E/F_y}$	$5.70\sqrt{E/F_y}$	
Stiffened Elements	10	Uniform compression in webs of doubly symmetric I-shaped sections	h/t_w	NA	$1.49\sqrt{E/F_y}$	
Stiffened Elements	11	Flexure in webs of singly-symmetric I-shaped sections	h_c/t_w	$\dfrac{\dfrac{h_c}{h_p}\sqrt{\dfrac{E}{F_y}}}{\left(0.54\dfrac{M_p}{M_y}-0.09\right)^2} \le \lambda_r$	$5.70\sqrt{E/F_y}$	
Stiffened Elements	12	Uniform compression in flanges of rectangular box and hollow structural sections of uniform thickness subject to bending or compression; flange cover plates and diaphragm plates between lines of fasteners or welds	b/t	$1.12\sqrt{E/F_y}$	$1.40\sqrt{E/F_y}$	
Stiffened Elements	13	Flexure in webs of rectangular HSS	h/t	$2.42\sqrt{E/F_y}$	$5.70\sqrt{E/F_y}$	

TABLE B4.1 (cont.)
Limiting Width-Thickness Ratios for Compression Elements

Case	Description of Element	Width Thick-ness Ratio	Limiting Width-Thickness Ratios		Example
			λ_p (compact)	λ_r (noncompact)	
14	Uniform compression in all other stiffened elements	b/t	NA	$1.49\sqrt{E/F_y}$	
15	Circular hollow sections In uniform compression In flexure	 D/t D/t	 NA $0.07\,E/F_y$	 $0.11\,E/F_y$ $0.31\,E/F_y$	

[a] $k_c = \frac{4}{\sqrt{h/t_w}}$, but shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes. (See Cases 2 and 4)

[b] $F_L = 0.7F_y$ for minor-axis bending, major axis bending of slender-web built-up I-shaped members, and major axis bending of compact and noncompact web built-up I-shaped members with $S_{xt}/S_{xc} \geq 0.7$; $F_L = F_y S_{xt}/S_{xc} \geq 0.5F_y$ for major-axis bending of compact and noncompact web built-up I-shaped members with $S_{xt}/S_{xc} < 0.7$. (See Case 2)

B5. FABRICATION, ERECTION AND QUALITY CONTROL

Shop drawings, fabrication, shop painting, erection, and *quality control* shall meet the requirements stipulated in Chapter M, Fabrication, Erection, and Quality Control.

B6. EVALUATION OF EXISTING STRUCTURES

Provisions for the evaluation of existing structures are presented in Appendix 5, Evaluation of Existing Structures.

CHAPTER C

STABILITY ANALYSIS AND DESIGN

This chapter addresses general requirements for the stability analysis and design of members and frames.

The chapter is organized as follows:

C1. STABILITY DESIGN REQUIREMENTS

1. General Requirements

Stability shall be provided for the structure as a whole and for each of its elements. Any method that considers the influence of *second-order effects* (including P-Δ and P-δ *effects*), flexural, shear and axial deformations, geometric imperfections, and member stiffness reduction due to residual stresses on the stability of the structure and its elements is permitted. The methods prescribed in this chapter and Appendix 7, Direct Analysis Method, satisfy these requirements. All component and *connection* deformations that contribute to the lateral displacements shall be considered in the stability analysis.

In structures designed by elastic analysis, individual member stability and stability of the structure as a whole are provided jointly by:

(1) Calculation of the *required strengths* for members, connections and other elements using one of the methods specified in Section C2.2, and
(2) Satisfaction of the member and connection design requirements in this specification based upon those required strengths.

In structures designed by inelastic analysis, the provisions of Appendix 1, Inelastic Analysis and Design, shall be satisfied.

2. Member Stability Design Requirements

Individual member stability is provided by satisfying the provisions of Chapters E, F, G, H and I.

User Note: Local buckling of cross section components can be avoided by the use of compact sections defined in Section B4.

Where elements are designed to function as braces to define the unbraced length of columns and beams, the bracing system shall have sufficient stiffness and strength to control member movement at the braced points. Methods of satisfying this requirement are provided in Appendix 6, Stability Bracing for Columns and Beams.

3. System Stability Design Requirements

Lateral stability shall be provided by *moment frames*, *braced frames*, *shear walls*, and/or other equivalent *lateral load resisting systems*. The overturning effects of *drift* and the destabilizing influence of *gravity loads* shall be considered. *Force* transfer and *load* sharing between elements of the framing systems shall be considered. Braced-frame and shear-wall systems, moment frames, gravity framing systems, and *combined systems* shall satisfy the following specific requirements:

3a. Braced-Frame and Shear-Wall Systems

In structures where lateral stability is provided solely by diagonal bracing, shear walls, or equivalent means, the *effective length factor*, K, for compression members shall be taken as 1.0, unless *structural analysis* indicates that a smaller value is appropriate. In braced-frame systems, it is permitted to design the columns, beams, and diagonal members as a vertically cantilevered, simply connected truss.

> **User Note:** Knee-braced frames function as moment-frame systems and should be treated as indicated in Section C1.3b. Eccentrically braced frame systems function as combined systems and should be treated as indicated in Section C1.3d.

3b. Moment-Frame Systems

In frames where lateral stability is provided by the flexural stiffness of connected beams and columns, the effective length factor K or elastic critical buckling stress, F_e, for *columns* and *beam-columns* shall be determined as specified in Section C2.

3c. Gravity Framing Systems

Columns in gravity framing systems shall be designed based on their actual length ($K = 1.0$) unless analysis shows that a smaller value may be used. The lateral stability of gravity framing systems shall be provided by moment frames, braced frames, shear walls, and/or other equivalent lateral load resisting systems. *P-Δ effects* due to load on the gravity columns shall be transferred to the lateral load resisting systems and shall be considered in the calculation of the required strengths of the lateral load resisting systems.

3d. Combined Systems

The analysis and design of members, *connections* and other elements in combined systems of moment frames, braced frames, and/or shear walls and gravity frames shall meet the requirements of their respective systems.

C2. CALCULATION OF REQUIRED STRENGTHS

Except as permitted in Section C2.2b, *required strengths* shall be determined using a *second-order analysis* as specified in Section C2.1. Design by either second-order or *first-order analysis* shall meet the requirements specified in Section C2.2.

1. Methods of Second-Order Analysis

Second-order analysis shall conform to the requirements in this Section.

1a. General Second-Order Elastic Analysis

Any second-order elastic analysis method that considers both P-Δ and P-δ *effects* may be used.

The Amplified First-Order Elastic Analysis Method defined in Section C2.1b is an accepted method for second-order elastic analysis of braced, moment, and combined framing systems.

1b. Second-Order Analysis by Amplified First-Order Elastic Analysis

User Note: A method is provided in this section to account for second-order effects in frames by amplifying the axial forces and moments in members and connections from a first-order analysis.

The following is an approximate second-order analysis procedure for calculating the required flexural and axial strengths in members of *lateral load resisting systems*. The required second-order flexural strength, M_r, and axial strength, P_r, shall be determined as follows:

$$M_r = B_1 M_{nt} + B_2 M_{lt} \tag{C2-1a}$$

$$P_r = P_{nt} + B_2 P_{lt} \tag{C2-1b}$$

where

$$B_1 = \frac{C_m}{1 - \alpha P_r / P_{e1}} \geq 1 \tag{C2-2}$$

For members subjected to axial compression, B_1 may be calculated based on the first-order estimate $P_r = P_{nt} + P_{lt}$.

User Note: B_1 is an amplifier to account for second order effects caused by displacements between brace points (P-δ) and B_2 is an amplifier to account for second order effects caused by displacements of braced points (P-Δ).

For members in which $B_1 \leq 1.05$, it is conservative to amplify the sum of the non-sway and sway moments (as obtained, for instance, by a first-order elastic analysis) by the B_2 amplifier, in other words, $M_r = B_2(M_{nt} + M_{lt})$.

$$B_2 = \frac{1}{1 - \dfrac{\alpha \Sigma P_{nt}}{\Sigma P_{e2}}} \geq 1 \tag{C2-3}$$

User Note: Note that the B_2 amplifier (Equation C2-3) can be estimated in preliminary design by using a maximum lateral drift limit corresponding to the story shear ΣH in Equation C2-6b.

and

$$\alpha = 1.00 \text{ (LRFD)} \quad \alpha = 1.60 \text{ (ASD)}$$

M_r = required second-order flexural strength using LRFD or ASD load combinations, kip-in. (N-mm)

M_{nt} = first-order moment using LRFD or ASD load combinations, assuming there is no lateral translation of the frame, kip-in. (N-mm)

M_{lt} = first-order moment using LRFD or ASD load combinations caused by lateral translation of the frame only, kip-in. (N-mm)

P_r = required second-order axial strength using LRFD or ASD load combinations, kips (N)

P_{nt} = first-order axial force using LRFD or ASD load combinations, assuming there is no lateral translation of the frame, kips (N)

ΣP_{nt} = total vertical load supported by the story using LRFD or ASD load combinations, including gravity column loads, kips (N)

P_{lt} = first-order axial force using LRFD or ASD load combinations caused by lateral translation of the frame only, kips (N)

C_m = a coefficient assuming no lateral translation of the frame whose value shall be taken as follows:

(i) For beam-columns not subject to transverse loading between supports in the plane of bending,

$$C_m = 0.6 - 0.4(M_1/M_2) \tag{C2-4}$$

where M_1 and M_2, calculated from a first-order analysis, are the smaller and larger moments, respectively, at the ends of that portion of the member unbraced in the plane of bending under consideration. M_1/M_2 is positive when the member is bent in reverse curvature, negative when bent in single curvature.

(ii) For beam-columns subjected to transverse loading between supports, the value of C_m shall be determined either by analysis or conservatively taken as 1.0 for all cases.

P_{e1} = elastic critical buckling resistance of the member in the plane of bending, calculated based on the assumption of zero sidesway, kips (N)

$$P_{e1} = \frac{\pi^2 EI}{(K_1 L)^2} \tag{C2-5}$$

ΣP_{e2} = elastic critical buckling resistance for the story determined by sidesway buckling analysis, kips (N)

For moment frames, where sidesway buckling effective length factors K_2 are determined for the columns, it is permitted to calculate the elastic story sidesway buckling resistance as

$$\Sigma P_{e2} = \Sigma \frac{\pi^2 EI}{(K_2 L)^2} \tag{C2-6a}$$

For all types of lateral load resisting systems, it is permitted to use

$$\Sigma P_{e2} = R_M \frac{\Sigma HL}{\Delta_H} \tag{C2-6b}$$

where

E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)
R_M = 1.0 for braced-frame systems;
= 0.85 for moment-frame and combined systems, unless a larger value is justified by analysis
I = moment of inertia in the plane of bending, in.^{4} (mm^4)
L = story height, in. (mm)
K_1 = effective length factor in the plane of bending, calculated based on the assumption of no lateral translation, set equal to 1.0 unless analysis indicates that a smaller value may be used
K_2 = effective length factor in the plane of bending, calculated based on a sidesway buckling analysis

User Note: Methods for calculation of K_2 are discussed in the Commentary.

Δ_H = first-order interstory drift due to lateral forces, in. (mm). Where Δ_H varies over the plan area of the structure, Δ_H shall be the average drift weighted in proportion to vertical load or, alternatively, the maximum drift.
ΣH = story shear produced by the lateral forces used to compute Δ_H, kips (N)

2. Design Requirements

These requirements apply to all types of braced, moment, and combined framing systems. Where the ratio of second-order drift to first-order drift is equal to or less than 1.5, the *required strengths* of members, *connections* and other elements shall be determined by one of the methods specified in Sections C2.2a or C2.2b, or by the *Direct Analysis Method* of Appendix 7. Where the ratio of second-order drift to first-order drift is greater than 1.5, the required strengths shall be determined by the Direct Analysis Method of Appendix 7.

User Note: The ratio of second-order drift to first-order drift can be represented by B_2, as calculated using Equation C2-3. Alternatively, the ratio can be calculated by comparing the results of a second-order analysis to the results of a first-order analysis, where the analyses are conducted either under LRFD load combinations directly or under ASD load combinations with a 1.6 factor applied to the ASD gravity loads.

For the methods specified in Sections 2.2a or 2.2b:

(1) Analyses shall be conducted according to the design and loading requirements specified in either Section B3.3 (LRFD) or Section B3.4 (ASD).
(2) The structure shall be analyzed using the nominal geometry and the nominal elastic stiffness for all elements.

2a. Design by Second-Order Analysis

Where required strengths are determined by a *second-order analysis*:

(1) The provisions of Section C2.1 shall be satisfied.

(2) For design by ASD, analyses shall be carried out under 1.6 times the *ASD load combinations* and the results shall be divided by 1.6 to obtain the required strengths.

User Note: The amplified first order analysis method of Section C2.1b incorporates the 1.6 multiplier directly in the B_1 and B_2 amplifiers, such that no other modification is needed.

(3) All gravity-only load combinations shall include a minimum lateral load applied at each level of the structure of $0.002Y_i$, where Y_i is the *design gravity load* applied at level i, kips (N). This minimum *lateral load* shall be considered independently in two orthogonal directions.

User Note: The minimum lateral load of $0.002Y_i$, in conjunction with the other design-analysis constraints listed in this section, limits the error that would otherwise be caused by neglecting initial out-of-plumbness and member stiffness reduction due to residual stresses in the analysis.

(4) Where the ratio of second-order drift to first-order drift is less than or equal to 1.1, members are permitted to be designed using $K = 1.0$. Otherwise, *columns* and *beam-columns* in *moment frames* shall be designed using a K factor or column buckling stress, F_e, determined from a sidesway buckling analysis of the structure. Stiffness reduction adjustment due to column inelasticity is permitted in the determination of the K factor. For *braced frames*, K for compression members shall be taken as 1.0, unless structural analysis indicates a smaller value may be used.

2b. Design by First-Order Analysis

Required strengths are permitted to be determined by a first-order analysis, with all members designed using $K = 1.0$, provided that

(1) The required compressive strengths of all members whose flexural stiffnesses are considered to contribute to the lateral stability of the structure satisfy the following limitation:

$$\alpha P_r \le 0.5 P_y \qquad \text{(C2-7)}$$

where

$$\alpha = 1.0 \text{ (LRFD)} \quad \alpha = 1.6 \text{ (ASD)}$$

P_r = required axial compressive strength under LRFD or ASD load combinations, kips (N)

P_y = member yield strength ($= AF_y$), kips (N)

(2) All load combinations include an additional lateral load, N_i, applied in combination with other loads at each level of the structure, where

$$N_i = 2.1(\Delta/L)Y_i \ge 0.0042Y_i \qquad \text{(C2-8)}$$

Y_i = gravity load from the LRFD load combination or 1.6 times the ASD load combination applied at level i, kips (N)

Δ/L = the maximum ratio of Δ to L for all stories in the structure
Δ = first-order interstory drift due to the design loads, in. (mm). Where Δ varies over the plan area of the structure, Δ shall be the average drift weighted in proportion to vertical load or, alternatively, the maximum drift.
L = story height, in. (mm)

User Note: The drift Δ is calculated under LRFD load combinations directly or under ASD load combinations with a 1.6 factor applied to the ASD gravity loads.

This additional lateral load shall be considered independently in two orthogonal directions.

(3) The non-sway amplification of beam-column moments is considered by applying the B_1 amplifier of Section C2.1 to the total member moments.

CHAPTER D

DESIGN OF MEMBERS FOR TENSION

This chapter applies to members subject to axial tension caused by static *force*s acting through the centroidal axis.

The chapter is organized as follows:

D1. Slenderness Limitations
D2. Tensile Strength
D3. Area Determination
D4. Built-Up Members
D5. Pin-Connected Members
D6. Eyebars

User Note: For cases not included in this chapter the following sections apply:

• B3.9	Members subject to fatigue.
• Chapter H	Members subject to combined axial tension and flexure.
• J3.	Threaded rods.
• J4.1	Connecting elements in tension.
• J4.3	Block shear rupture strength at end connections of tension members.

D1. SLENDERNESS LIMITATIONS

There is no maximum slenderness limit for design of members in tension.

User Note: For members designed on the basis of tension, the slenderness ratio L/r preferably should not exceed 300. This suggestion does not apply to rods or hangers in tension.

D2. TENSILE STRENGTH

The *design tensile strength*, $\phi_t P_n$, and the *allowable tensile strength*, P_n/Ω_t, of tension members, shall be the lower value obtained according to the *limit state*s of *tensile yielding* in the gross section and *tensile rupture* in the net section.

(a) For tensile yielding in the gross section:

$$P_n = F_y A_g \qquad \text{(D2-1)}$$

$$\phi_t = 0.90 \text{ (LRFD)} \qquad \Omega_t = 1.67 \text{ (ASD)}$$

(b) For tensile rupture in the net section:

$$P_n = F_u A_e \tag{D2-2}$$

$$\phi_t = 0.75 \text{ (LRFD)} \qquad \Omega_t = 2.00 \text{ (ASD)}$$

where

A_e = *effective net area*, in.^{2} (mm^2)
A_g = gross area of member, in.^{2} (mm^2)
F_y = *specified minimum yield stress* of the type of steel being used, ksi (MPa)
F_u = *specified minimum tensile strength* of the type of steel being used, ksi (MPa)

When members without holes are fully connected by welds, the effective net area used in Equation D2-2 shall be as defined in Section D3. When holes are present in a member with welded end *connection*s, or at the welded connection in the case of plug or *slot weld*s, the effective net area through the holes shall be used in Equation D2-2.

D3. AREA DETERMINATION

1. Gross Area

The gross area, A_g, of a member is the total cross-sectional area.

2. Net Area

The *net area*, A_n, of a member is the sum of the products of the thickness and the net width of each element computed as follows:

In computing net area for tension and shear, the width of a bolt hole shall be taken as $^1/_{16}$ in. (2 mm) greater than the *nominal dimension* of the hole.

For a chain of holes extending across a part in any diagonal or zigzag line, the net width of the part shall be obtained by deducting from the gross width the sum of the diameters or slot dimensions as provided in Section J3.2, of all holes in the chain, and adding, for each *gage* space in the chain, the quantity $s^2/4g$

where

s = longitudinal center-to-center spacing (*pitch*) of any two consecutive holes, in. (mm)
g = transverse center-to-center spacing (gage) between *fastener* gage lines, in. (mm)

For angles, the gage for holes in opposite adjacent legs shall be the sum of the gages from the back of the angles less the thickness.

For slotted *HSS* welded to a *gusset plate*, the net area, A_n, is the gross area minus the product of the thickness and the total width of material that is removed to form the slot.

In determining the net area across plug or *slot welds*, the *weld metal* shall not be considered as adding to the net area.

User Note: Section J4.1(b) limits A_n to a maximum of $0.85A_g$ for splice plates with holes.

3. Effective Net Area

The effective area of tension members shall be determined as follows:

$$A_e = A_n U \tag{D3-1}$$

where U, the shear lag factor, is determined as shown in Table D3.1.

Members such as single angles, double angles and WT sections shall have *connections* proportioned such that U is equal to or greater than 0.60. Alternatively, a lesser value of U is permitted if these tension members are designed for the effect of eccentricity in accordance with H1.2 or H2.

D4. BUILT-UP MEMBERS

For limitations on the longitudinal spacing of connectors between elements in continuous contact consisting of a plate and a shape or two plates, see Section J3.5.

Either perforated *cover plates* or *tie plates* without *lacing* are permitted to be used on the open sides of built-up tension members. Tie plates shall have a length not less than two-thirds the distance between the lines of welds or *fasteners* connecting them to the components of the member. The thickness of such tie plates shall not be less than one-fiftieth of the distance between these lines. The longitudinal spacing of intermittent welds or fasteners at tie plates shall not exceed 6 in. (150 mm).

User Note: The longitudinal spacing of connectors between components should preferably limit the slenderness ratio in any component between the connectors to 300.

D5. PIN-CONNECTED MEMBERS

1. Tensile Strength

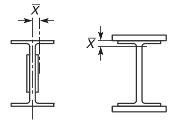
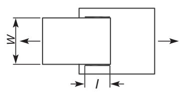
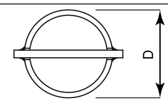
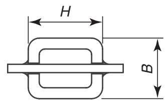
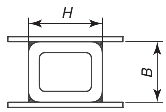
The *design tensile strength*, $\phi_t P_n$, and the *allowable tensile strength*, P_n/Ω_t, of pin-connected members, shall be the lower value obtained according to the *limit states* of *tensile rupture*, *shear rupture*, *bearing,* and *yielding*.

(a) For tensile rupture on the net effective area:

$$P_n = 2tb_{eff}F_u \tag{D5-1}$$

$$\phi_t = 0.75\,(\text{LRFD}) \qquad \Omega_t = 2.00\,(\text{ASD})$$

TABLE D3.1
Shear Lag Factors for Connections to Tension Members

Case	Description of Element		Shear Lag Factor, U	Example
1	All tension members where the tension load is transmitted directly to each of cross-sectional elements by fasteners or welds. (except as in Cases 3, 4, 5 and 6)		$U = 1.0$	—
2	All tension members, except plates and HSS, where the tension load is transmitted to some but not all of the cross-sectional elements by fasteners or longitudinal welds (Alternatively, for W, M, S and HP, Case 7 may be used.)		$U = 1 - \bar{x}/l$	
3	All tension members where the tension load is transmitted by transverse welds to some but not all of the cross-sectional elements.		$U = 1.0$ and A_n = area of the directly connected elements	—
4	Plates where the tension load is transmitted by longitudinal welds only.		$l \geq 2w \ldots U = 1.0$ $2w > l \geq 1.5w \ldots U = 0.87$ $1.5w > l \geq w \ldots U = 0.75$	
5	Round HSS with a single concentric gusset plate		$l \geq 1.3D \ldots U = 1.0$ $D \leq l < 1.3D \ldots U = 1 - \bar{x}/l$ $\bar{x} = D/\pi$	
6	Rectangular HSS	with a single concentric gusset plate	$l \geq H \ldots U = 1 - \bar{x}/l$ $\bar{x} = \dfrac{B^2 + 2BH}{4(B+H)}$	
		with two side gusset plates	$l \geq H \ldots U = 1 - \bar{x}/l$ $\bar{x} = \dfrac{B^2}{4(B+H)}$	
7	W, M, S or HP Shapes or Tees cut from these shapes. (If U is calculated per Case 2, the larger value is permitted to be used)	with flange connected with 3 or more fasteners per line in direction of loading	$b_f \geq 2/3d \ldots U = 0.90$ $b_f < 2/3d \ldots U = 0.85$	—
		with web connected with 4 or more fasteners in the direction of loading	$U = 0.70$	—
8	Single angles (If U is calculated per Case 2, the larger value is permitted to be used)	with 4 or more fasteners per line in direction of loading	$U = 0.80$	—
		with 2 or 3 fasteners per line in the direction of loading	$U = 0.60$	—

l = length of connection, in. (mm); w = plate width, in. (mm); $\bar{x}$ = connection eccentricity, in. (mm); B = overall width of rectangular HSS member, measured 90 degrees to the plane of the connection, in. (mm); H = overall height of rectangular HSS member, measured in the plane of the connection, in. (mm)

(b) For shear rupture on the effective area:

$$P_n = 0.6F_u A_{sf} \tag{D5-2}$$

$$\phi_{sf} = 0.75\,(\text{LRFD}) \qquad \Omega_{sf} = 2.00\,(\text{ASD})$$

where

$A_{sf} = 2t(a + d/2)$, in.^{2}(mm^2)

a = shortest distance from edge of the pin hole to the edge of the member measured parallel to the direction of the *force*, in. (mm)

$b_{eff} = 2t + 0.63$, in. ($= 2t + 16$, mm) but not more than the actual distance from the edge of the hole to the edge of the part measured in the direction normal to the applied force

d = pin diameter, in. (mm)

t = thickness of plate, in. (mm)

(c) For bearing on the projected area of the pin, see Section J7.

(d) For yielding on the gross section, use Equation D2-1.

2. Dimensional Requirements

The pin hole shall be located midway between the edges of the member in the direction normal to the applied *force*. When the pin is expected to provide for relative movement between connected parts while under full *load*, the diameter of the pin hole shall not be more than $^1/_{32}$ in. (1 mm) greater than the diameter of the pin.

The width of the plate at the pin hole shall not be less than $2b_{eff} + d$ and the minimum extension, a, beyond the bearing end of the pin hole, parallel to the axis of the member, shall not be less than $1.33 \times b_{eff}$.

The corners beyond the pin hole are permitted to be cut at 45° to the axis of the member, provided the *net area* beyond the pin hole, on a plane perpendicular to the cut, is not less than that required beyond the pin hole parallel to the axis of the member.

D6. EYEBARS

1. Tensile Strength

The *available tensile strength* of *eyebar*s shall be determined in accordance with Section D2, with A_g taken as the cross-sectional area of the body.

For calculation purposes, the width of the body of the eyebars shall not exceed eight times its thickness.

2. Dimensional Requirements

*Eyebar*s shall be of uniform thickness, without reinforcement at the pin holes, and have circular heads with the periphery concentric with the pin hole.

The radius of transition between the circular head and the eyebar body shall not be less than the head diameter.

The pin diameter shall not be less than seven-eighths times the eyebar body width, and the pin hole diameter shall not be more than $^1/_{32}$ in. (1 mm) greater than the pin diameter.

For steels having F_y greater than 70 ksi (485 MPa), the hole diameter shall not exceed five times the plate thickness, and the width of the eyebar body shall be reduced accordingly.

A thickness of less than $^1/_2$ in. (13 mm) is permissible only if external nuts are provided to tighten pin plates and *filler* plates into snug contact. The width from the hole edge to the plate edge perpendicular to the direction of applied *load* shall be greater than two-thirds and, for the purpose of calculation, not more than three-fourths times the eyebar body width.

CHAPTER E

DESIGN OF MEMBERS FOR COMPRESSION

This chapter addresses members subject to axial compression through the centroidal axis.

The chapter is organized as follows:

E1. General Provisions
E2. Slenderness Limitations and Effective Length
E3. Compressive Strength for Flexural Buckling of Members without Slender Elements
E4. Compressive Strength for Torsional and Flexural-Torsional Buckling of Members without Slender Elements
E5. Single Angle Compression Members
E6. Built-Up Members
E7. Members with Slender Elements

User Note: For members not included in this chapter the following sections apply:
- H1. – H3. Members subject to combined axial compression and flexure.
- H4. Members subject to axial compression and torsion.
- J4.4 Compressive strength of connecting elements.
- I2. Composite axial members.

E1. GENERAL PROVISIONS

The *design compressive strength*, $\phi_c P_n$, and the *allowable compressive strength*, P_n/Ω_c, are determined as follows:

The *nominal compressive strength*, P_n, shall be the lowest value obtained according to the *limit states* of *flexural buckling*, *torsional buckling* and *flexural-torsional buckling*.

(a) For doubly symmetric and singly symmetric members the limit state of flexural buckling is applicable.
(b) For singly symmetric and unsymmetric members, and certain doubly symmetric members, such as cruciform or built-up *columns*, the limit states of torsional or flexural-torsional buckling are also applicable.

$$\phi_c = 0.90 \text{ (LRFD)} \quad \Omega_c = 1.67 \text{ (ASD)}$$

E2. SLENDERNESS LIMITATIONS AND EFFECTIVE LENGTH

The effective length factor, K, for calculation of column slenderness, KL/r, shall be determined in accordance with Chapter C,

where

L = laterally unbraced length of the member, in. (mm)
r = governing radius of gyration, in. (mm)
K = the *effective length factor* determined in accordance with Section C2

User Note: For members designed on the basis of compression, the slenderness ratio KL/r preferably should not exceed 200.

E3. COMPRESSIVE STRENGTH FOR FLEXURAL BUCKLING OF MEMBERS WITHOUT SLENDER ELEMENTS

This section applies to compression members with *compact* and *noncompact sections*, as defined in Section B4, for uniformly compressed elements.

User Note: When the torsional unbraced length is larger than the lateral unbraced length, this section may control the design of wide flange and similarly shaped columns.

The *nominal compressive strength*, P_n, shall be determined based on the *limit state* of *flexural buckling*.

$$P_n = F_{cr} A_g \tag{E3-1}$$

The *flexural buckling stress*, F_{cr}, is determined as follows:

(a) When $\dfrac{KL}{r} \leq 4.71\sqrt{\dfrac{E}{F_y}}$ (or $F_e \geq 0.44F_y$)

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y \tag{E3-2}$$

(b) When $\dfrac{KL}{r} > 4.71\sqrt{\dfrac{E}{F_y}}$ (or $F_e < 0.44F_y$)

$$F_{cr} = 0.877F_e \tag{E3-3}$$

where

F_e = elastic critical buckling stress determined according to Equation E3-4, Section E4, or the provisions of Section C2, as applicable, ksi (MPa)

$$F_e = \frac{\pi^2 E}{\left(\dfrac{KL}{r}\right)^2} \tag{E3-4}$$

User Note: The two equations for calculating the limits and applicability of Sections E3(a) and E3(b), one based on KL/r and one based on F_e, provide the same result.

E4. COMPRESSIVE STRENGTH FOR TORSIONAL AND FLEXURAL-TORSIONAL BUCKLING OF MEMBERS WITHOUT SLENDER ELEMENTS

This section applies to singly symmetric and unsymmetric members, and certain doubly symmetric members, such as cruciform or built-up *column*s with *compact* and *noncompact sections*, as defined in Section B4 for uniformly compressed elements. These provisions are not required for single angles, which are covered in Section E5.

The *nominal compressive strength*, P_n, shall be determined based on the *limit state*s of *flexural-torsional* and *torsional buckling*, as follows:

$$P_n = F_{cr} A_g \tag{E4-1}$$

(a) For double-angle and tee-shaped compression members:

$$F_{cr} = \left(\frac{F_{cry} + F_{crz}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{cry}F_{crz}H}{\left(F_{cry} + F_{crz}\right)^2}}\right] \tag{E4-2}$$

where F_{cry} is taken as F_{cr} from Equation E3-2 or E3-3, for *flexural buckling* about the y-axis of symmetry and $\frac{KL}{r} = \frac{KL}{r_y}$, and

$$F_{crz} = \frac{GJ}{A_g \bar{r}_o^2} \tag{E4-3}$$

(b) For all other cases, F_{cr} shall be determined according to Equation E3-2 or E3-3, using the torsional or flexural-torsional elastic buckling *stress*, F_e, determined as follows:

(i) For doubly symmetric members:

$$F_e = \left[\frac{\pi^2 E C_w}{(K_z L)^2} + GJ\right]\frac{1}{I_x + I_y} \tag{E4-4}$$

(ii) For singly symmetric members where y is the axis of symmetry:

$$F_e = \left(\frac{F_{ey} + F_{ez}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{ey}F_{ez}H}{\left(F_{ey} + F_{ez}\right)^2}}\right] \tag{E4-5}$$

(iii) For unsymmetric members, F_e is the lowest root of the cubic equation:

$$(F_e - F_{ex})(F_e - F_{ey})(F_e - F_{ez}) - F_e^2(F_e - F_{ey})\left(\frac{x_o}{\bar{r}_o}\right)^2 - F_e^2(F_e - F_{ex})\left(\frac{y_o}{\bar{r}_o}\right)^2 = 0 \tag{E4-6}$$

where

A_g = gross area of member, in.^{2} (mm^2)
C_w = warping constant, in.^{6} (mm^6)

$$\bar{r}_o^2 = x_o^2 + y_o^2 + \frac{I_x + I_y}{A_g} \quad \text{(E4-7)}$$

$$H = 1 - \frac{x_o^2 + y_o^2}{\bar{r}_o^2} \quad \text{(E4-8)}$$

$$F_{ex} = \frac{\pi^2 E}{\left(\frac{K_x L}{r_x}\right)^2} \quad \text{(E4-9)}$$

$$F_{ey} = \frac{\pi^2 E}{\left(\frac{K_y L}{r_y}\right)^2} \quad \text{(E4-10)}$$

$$F_{ez} = \left(\frac{\pi^2 E C_w}{(K_z L)^2} + GJ\right)\frac{1}{A_g \bar{r}_o^2} \quad \text{(E4-11)}$$

G = shear modulus of elasticity of steel = 11,200 ksi (77 200 MPa)
I_x, I_y = moment of inertia about the principal axes, in.^{4} (mm^4)
J = torsional constant, in.^{4} (mm^4)
K_z = *effective length factor* for torsional buckling
x_o, y_o = coordinates of shear center with respect to the centroid, in. (mm)
$\bar{r}_o$ = polar radius of gyration about the shear center, in. (mm)
r_y = radius of gyration about y-axis, in. (mm)

User Note: For doubly symmetric I-shaped sections, C_w may be taken as $I_y h_o^2/4$, where h_o is the distance between flange centroids, in lieu of a more precise analysis. For tees and double angles, omit term with C_w when computing F_{ez} and take x_o as 0.

E5. SINGLE ANGLE COMPRESSION MEMBERS

The *nominal compressive strength*, P_n, of single angle members shall be determined in accordance with Section E3 or Section E7, as appropriate, for axially loaded members, as well as those subject to the slenderness modification of Section E5(a) or E5(b), provided the members meet the criteria imposed.

The effects of eccentricity on single angle members are permitted to be neglected when the members are evaluated as axially loaded compression members using one of the effective slenderness ratios specified below, provided that: (1) members are loaded at the ends in compression through the same one leg; (2) members are attached by welding or by minimum two-bolt *connections*; and (3) there are no intermediate transverse *loads*.

(a) For equal-leg angles or unequal-leg angles connected through the longer leg that are individual members or are web members of planar trusses with adjacent web members attached to the same side of the *gusset plate* or chord:

(i) When $0 \le \frac{L}{r_x} \le 80$:

$$\frac{KL}{r} = 72 + 0.75\frac{L}{r_x} \tag{E5-1}$$

(ii) When $\frac{L}{r_x} > 80$:

$$\frac{KL}{r} = 32 + 1.25\frac{L}{r_x} \le 200 \tag{E5-2}$$

For unequal-leg angles with leg length ratios less than 1.7 and connected through the shorter leg, KL/r from Equations E5-1 and E5-2 shall be increased by adding $4[(b_l/b_s)^2 - 1]$, but KL/r of the members shall not be less than $0.95L/r_z$.

(b) For equal-leg angles or unequal-leg angles connected through the longer leg that are web members of box or space trusses with adjacent web members attached to the same side of the *gusset plate* or chord:

(i) When $0 \le \frac{L}{r_x} \le 75$:

$$\frac{KL}{r} = 60 + 0.8\frac{L}{r_x} \tag{E5-3}$$

(ii) When $\frac{L}{r_x} > 75$:

$$\frac{KL}{r} = 45 + \frac{L}{r_x} \le 200 \tag{E5-4}$$

For unequal-leg angles with leg length ratios less than 1.7 and connected through the shorter leg, KL/r from Equations E5-3 and E5-4 shall be increased by adding $6[(b_l/b_s)^2 - 1]$, but KL/r of the member shall not be less than $0.82L/r_z$,

where

L = length of member between work points at truss chord centerlines, in. (mm)

b_l = longer leg of angle, in. (mm)

b_s = shorter leg of angle, in. (mm)

r_x = radius of gyration about *geometric axis* parallel to connected leg, in. (mm)

r_z = radius of gyration for the minor principal axis, in. (mm)

(c) Single angle members with different end conditions from those described in Section E5(a) or (b), with leg length ratios greater than 1.7, or with transverse loading shall be evaluated for combined axial *load* and flexure using the provisions of Chapter H. End connection to different legs on each end or to both legs, the use of single bolts or the attachment of adjacent web members to opposite sides of the *gusset plate* or chord shall constitute different end conditions requiring the use of Chapter H provisions.

E6. BUILT-UP MEMBERS

1. Compressive Strength

(a) The *nominal compressive strength* of *built-up members* composed of two or more shapes that are interconnected by bolts or welds shall be determined in accordance with Sections E3, E4, or E7 subject to the following modification. In lieu of more accurate analysis, if the buckling mode involves relative deformations that produce shear *forces* in the connectors between individual shapes, KL/r is replaced by $(KL/r)_m$ determined as follows:

(i) For intermediate connectors that are snug-tight bolted:

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{a}{r_i}\right)^2} \qquad \text{(E6-1)}$$

(ii) For intermediate connectors that are welded or pretensioned bolted:

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + 0.82\frac{\alpha^2}{(1+\alpha^2)}\left(\frac{a}{r_{ib}}\right)^2} \qquad \text{(E6-2)}$$

where

$\left(\frac{KL}{r}\right)_m$ = modified *column* slenderness of *built-up member*

$\left(\frac{KL}{r}\right)_o$ = column slenderness of built-up member acting as a unit in the buckling direction being considered

a = distance between connectors, in. (mm)

r_i = minimum radius of gyration of individual component, in. (mm)

r_{ib} = radius of gyration of individual component relative to its centroidal axis parallel to member axis of buckling, in. (mm)

α = separation ratio = $h/2r_{ib}$

h = distance between centroids of individual components perpendicular to the member axis of buckling, in. (mm)

(b) The nominal compressive strength of built-up members composed of two or more shapes or plates with at least one open side interconnected by perforated *cover plates* or *lacing* with *tie plates* shall be determined in accordance with Sections E3, E4, or E7 subject to the modification given in Section E6.1(a).

2. Dimensional Requirements

Individual components of compression members composed of two or more shapes shall be connected to one another at intervals, a, such that the effective slenderness ratio Ka/r_i of each of the component shapes, between the *fasteners*, does not exceed three-fourths times the governing slenderness ratio of the *built-up member*. The least radius of gyration, r_i, shall be used in computing the slenderness ratio of each component part. The end *connection* shall be welded or pretensioned bolted with Class A or B *faying surfaces*.

User Note: It is acceptable to design a bolted end *connection* of a built-up compression member for the full compressive *load* with bolts in shear and bolt values based on bearing values; however, the bolts must be pretensioned. The requirement for Class A or B faying surfaces is not intended for the resistance of the axial force in the built-up member, but rather to prevent relative movement between the components at the end as the built-up member takes a curved shape.

At the ends of built-up compression members bearing on base plates or *milled surfaces*, all components in contact with one another shall be connected by a weld having a length not less than the maximum width of the member or by bolts spaced longitudinally not more than four diameters apart for a distance equal to $1^1/2$ times the maximum width of the member.

Along the length of built-up compression members between the end connections required above, longitudinal spacing for intermittent welds or bolts shall be adequate to provide for the transfer of the required *force*s. For limitations on the longitudinal spacing of fasteners between elements in continuous contact consisting of a plate and a shape or two plates, see Section J3.5. Where a component of a built-up compression member consists of an outside plate, the maximum spacing shall not exceed the thickness of the thinner outside plate times $0.75\sqrt{E/F_y}$, nor 12 in. (305 mm), when intermittent welds are provided along the edges of the components or when fasteners are provided on all *gage* lines at each section. When fasteners are staggered, the maximum spacing on each gage line shall not exceed the thickness of the thinner outside plate times $1.12\sqrt{E/F_y}$ nor 18 in. (460 mm).

Open sides of compression members built up from plates or shapes shall be provided with continuous *cover plates* perforated with a succession of access holes. The unsupported width of such plates at access holes, as defined in Section B4, is assumed to contribute to the *available strength* provided the following requirements are met:

(1) The width-thickness ratio shall conform to the limitations of Section B4.

User Note: It is conservative to use the limiting width/thickness ratio for Case 14 in Table B4.1 with the width, b, taken as the transverse distance between the nearest lines of fasteners. The net area of the plate is taken at the widest hole. In lieu of this approach, the limiting width thickness ratio may be determined through analysis.

(2) The ratio of length (in direction of *stress*) to width of hole shall not exceed two.
(3) The clear distance between holes in the direction of stress shall be not less than the transverse distance between nearest lines of connecting fasteners or welds.
(4) The periphery of the holes at all points shall have a minimum radius of $1^1/2$ in. (38 mm).

As an alternative to perforated cover plates, *lacing* with *tie plate*s is permitted at each end and at intermediate points if the lacing is interrupted. Tie plates shall be as near the ends as practicable. In members providing *available strength*, the end tie plates shall have a length of not less than the distance between the lines of fasteners or welds connecting them to the components of the member. Intermediate tie plates shall have a length not less than one-half of this distance. The thickness of tie plates shall be not less than one-fiftieth of the distance between lines of welds or fasteners connecting them to the segments of the members. In welded construction, the welding on each line connecting a tie plate shall total not less than one-third the length of the plate. In bolted construction, the spacing in the direction of stress in tie plates shall be not more than six diameters and the tie plates shall be connected to each segment by at least three fasteners.

Lacing, including flat bars, angles, channels, or other shapes employed as lacing, shall be so spaced that the L/r ratio of the flange included between their connections shall not exceed three-fourths times the governing slenderness ratio for the member as a whole. Lacing shall be proportioned to provide a shearing strength normal to the axis of the member equal to 2 percent of the *available compressive strength* of the member. The L/r ratio for lacing bars arranged in single systems shall not exceed 140. For double lacing this ratio shall not exceed 200. Double lacing bars shall be joined at the intersections. For lacing bars in compression, l is permitted to be taken as the unsupported length of the *lacing* bar between welds or fasteners connecting it to the components of the *built-up member* for single lacing, and 70 percent of that distance for double lacing.

User Note: The inclination of lacing bars to the axis of the member shall preferably be not less than 60° for single lacing and 45° for double lacing. When the distance between the lines of welds or fasteners in the flanges is more than 15 in. (380 mm), the lacing shall preferably be double or be made of angles.

For additional spacing requirements, see Section J3.5.

E7. MEMBERS WITH SLENDER ELEMENTS

This section applies to compression members with slender sections, as defined in Section B4 for uniformly compressed elements.

The *nominal compressive strength*, P_n, shall be determined based on the *limit states* of *flexural*, *torsional* and *flexural-torsional buckling*.

$$P_n = F_{cr} A_g \tag{E7-1}$$

(a) When $\frac{KL}{r} \leq 4.71\sqrt{\frac{E}{QF_y}}$ (or $F_e \geq 0.44QF_y$)

$$F_{cr} = Q\left[0.658^{\frac{QF_y}{F_e}}\right]F_y \tag{E7-2}$$

(b) When $\frac{KL}{r} > 4.71\sqrt{\frac{E}{QF_y}}$ (or $F_e < 0.44QF_y$)

$$F_{cr} = 0.877F_e \tag{E7-3}$$

where

F_e = elastic critical buckling stress, calculated using Equations E3-4 and E4-4 for doubly symmetric members, Equations E3-4 and E4-5 for singly symmetric members, and Equation E4-6 for unsymmetric members, except for single angles where F_e is calculated using Equation E3-4.

Q = 1.0 for members with *compact* and *noncompact sections*, as defined in Section B4, for uniformly compressed elements

= Q_sQ_a for members with *slender-element section*s, as defined in Section B4, for uniformly compressed elements.

> **User Note:** For cross sections composed of only unstiffened slender elements, $Q = Q_s$ ($Q_a = 1.0$). For cross sections composed of only stiffened slender elements, $Q = Q_a$ ($Q_s = 1.0$). For cross sections composed of both stiffened and unstiffened slender elements, $Q = Q_sQ_a$.

1. Slender Unstiffened Elements, Q_s

The reduction factor Q_s for slender *unstiffened elements* is defined as follows:

(a) For flanges, angles, and plates projecting from rolled *column*s or other compression members:

(i) When $\frac{b}{t} \leq 0.56\sqrt{\frac{E}{F_y}}$

$$Q_s = 1.0 \tag{E7-4}$$

(ii) When $0.56\sqrt{E/F_y} < b/t < 1.03\sqrt{E/F_y}$

$$Q_s = 1.415 - 0.74\left(\frac{b}{t}\right)\sqrt{\frac{F_y}{E}} \tag{E7-5}$$

(iii) When $b/t \geq 1.03\sqrt{E/F_y}$

$$Q_s = \frac{0.69E}{F_y\left(\frac{b}{t}\right)^2} \tag{E7-6}$$

(b) For flanges, angles, and plates projecting from built-up columns or other compression members:

(i) When $\frac{b}{t} \leq 0.64\sqrt{\frac{Ek_c}{F_y}}$

$$Q_s = 1.0 \tag{E7-7}$$

(ii) When $0.64\sqrt{\frac{Ek_c}{F_y}} < b/t \leq 1.17\sqrt{\frac{Ek_c}{F_y}}$

$$Q_s = 1.415 - 0.65\left(\frac{b}{t}\right)\sqrt{\frac{F_y}{Ek_c}} \tag{E7-8}$$

(iii) When $b/t > 1.17\sqrt{\frac{Ek_c}{F_y}}$

$$Q_s = \frac{0.90Ek_c}{F_y\left(\frac{b}{t}\right)^2} \tag{E7-9}$$

where

$k_c = \frac{4}{\sqrt{h/t_w}}$, and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes

(c) For single angles

(i) When $\frac{b}{t} \leq 0.45\sqrt{\frac{E}{F_y}}$

$$Q_s = 1.0 \tag{E7-10}$$

(ii) When $0.45\sqrt{E/F_y} < b/t \leq 0.91\sqrt{E/F_y}$

$$Q_s = 1.34 - 0.76\left(\frac{b}{t}\right)\sqrt{\frac{F_y}{E}} \tag{E7-11}$$

(iii) When $b/t > 0.91\sqrt{E/F_y}$

$$Q_s = \frac{0.53E}{F_y\left(\frac{b}{t}\right)^2} \tag{E7-12}$$

where

b = full width of longest angle leg, in. (mm)

(d) For stems of tees

(i) When $\frac{d}{t} \leq 0.75\sqrt{\frac{E}{F_y}}$

$$Q_s = 1.0 \tag{E7-13}$$

(ii) When $0.75\sqrt{\dfrac{E}{F_y}} < d/t \le 1.03\sqrt{\dfrac{E}{F_y}}$

$$Q_s = 1.908 - 1.22\left(\frac{d}{t}\right)\sqrt{\frac{F_y}{E}} \tag{E7-14}$$

(iii) When $d/t > 1.03\sqrt{\dfrac{E}{F_y}}$

$$Q_s = \frac{0.69E}{F_y\left(\dfrac{d}{t}\right)^2} \tag{E7-15}$$

where

b = width of unstiffened compression element, as defined in Section B4, in. (mm)
d = the full nominal depth of tee, in. (mm)
t = thickness of element, in. (mm)

2. Slender Stiffened Elements, Q_a

The reduction factor, Q_a, for slender *stiffened elements* is defined as follows:

$$Q_a = \frac{A_{eff}}{A} \tag{E7-16}$$

where

A = total cross-sectional area of member, in.^{2} (mm^2)
A_{eff} = summation of the effective areas of the cross section based on the reduced *effective width*, b_e, in.^{2} (mm^2)

The reduced effective width, b_e, is determined as follows:

(a) For uniformly compressed slender elements, with $\dfrac{b}{t} \ge 1.49\sqrt{\dfrac{E}{f}}$, except flanges of square and rectangular sections of uniform thickness:

$$b_e = 1.92t\sqrt{\frac{E}{f}}\left[1 - \frac{0.34}{(b/t)}\sqrt{\frac{E}{f}}\right] \le b \tag{E7-17}$$

where

f is taken as F_{cr} with F_{cr} calculated based on $Q = 1.0$.

(b) For flanges of square and rectangular *slender-element sections* of uniform thickness with $\dfrac{b}{t} \ge 1.40\sqrt{\dfrac{E}{f}}$:

$$b_e = 1.92t\sqrt{\frac{E}{f}}\left[1 - \frac{0.38}{(b/t)}\sqrt{\frac{E}{f}}\right] \le b \tag{E7-18}$$

where

$f = P_n / A_{eff}$

User Note: In lieu of calculating $f = P_n/A_{eff}$, which requires iteration, f may be taken equal to F_y. This will result in a slightly conservative estimate of column capacity.

(c) For axially-loaded circular sections:

When $0.11\dfrac{E}{F_y} < \dfrac{D}{t} < 0.45\dfrac{E}{F_y}$

$$Q = Q_a = \frac{0.038E}{F_y(D/t)} + \frac{2}{3} \tag{E7-19}$$

where

D = outside diameter, in. (mm)
t = wall thickness, in. (mm)

CHAPTER F

DESIGN OF MEMBERS FOR FLEXURE

This chapter applies to members subject to simple bending about one principal axis. For simple bending, the member is loaded in a plane parallel to a principal axis that passes through the shear center or is restrained against twisting at *load* points and supports.

The chapter is organized as follows:

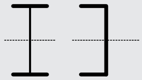
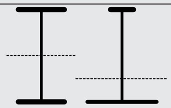
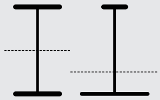
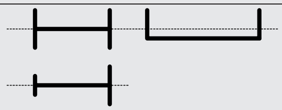
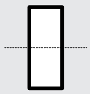
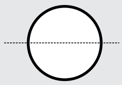
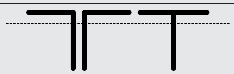
- F1. General Provisions
- F2. Doubly Symmetric Compact I-Shaped Members and Channels Bent about Their Major Axis
- F3. Doubly Symmetric I-Shaped Members with Compact Webs and Noncompact or Slender Flanges Bent about Their Major Axis
- F4. Other I-Shaped Members with Compact or Noncompact Webs Bent about Their Major Axis
- F5. Doubly Symmetric and Singly Symmetric I-Shaped Members with Slender Webs Bent about Their Major Axis
- F6. I-Shaped Members and Channels Bent about Their Minor Axis
- F7. Square and Rectangular HSS and Box-Shaped Members
- F8. Round HSS
- F9. Tees and Double Angles Loaded in the Plane of Symmetry
- F10. Single Angles
- F11. Rectangular Bars and Rounds
- F12. Unsymmetrical Shapes
- F13. Proportions of Beams and Girders

User Note: For members not included in this chapter the following sections apply:

- H1–H3. Members subject to biaxial flexure or to combined flexure and axial force.
- H4. Members subject to flexure and torsion.
- Appendix 3. Members subject to fatigue.
- Chapter G. Design provisions for shear.

For guidance in determining the appropriate sections of this chapter to apply, Table User Note F1.1 may be used.

TABLE User Note F1.1
Selection Table for the Application of Chapter F Sections

Section in Chapter F	Cross Section	Flange Slenderness	Web Slenderness	Limit States
F2		C	C	Y, LTB
F3		NC, S	C	LTB, FLB
F4		C, NC, S	C, NC	Y, LTB, FLB, TFY
F5		C, NC, S	S	Y, LTB, FLB, TFY
F6		C, NC, S	N/A	Y, FLB
F7		C, NC, S	C, NC	Y, FLB, WLB
F8		N/A	N/A	Y, LB
F9		C, NC, S	N/A	Y, LTB, FLB
F10		N/A	N/A	Y, LTB, LLB
F11		N/A	N/A	Y, LTB
F12	Unsymmetrical shapes	N/A	N/A	All limit states

Y = yielding, LTB = lateral-torsional buckling, FLB = flange local buckling, WLB = web local buckling, TFY = tension flange yielding, LLB = leg local buckling, LB = local buckling, C = compact, NC = noncompact, S = slender

F1. GENERAL PROVISIONS

The *design flexural strength*, $\phi_b M_n$, and the *allowable flexural strength*, M_n/Ω_b, shall be determined as follows:

(1) For all provisions in this chapter

$$\phi_b = 0.90\,(\text{LRFD}) \qquad \Omega_b = 1.67\,(\text{ASD})$$

and the *nominal flexural strength*, M_n, shall be determined according to Sections F2 through F12.

(2) The provisions in this chapter are based on the assumption that points of support for *beams* and girders are restrained against rotation about their longitudinal axis.

The following terms are common to the equations in this chapter except where noted:

C_b = *lateral-torsional buckling* modification factor for nonuniform moment diagrams when both ends of the unsupported segment are braced

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} R_m \le 3.0 \qquad \text{(F1-1)}$$

where

M_{max} = absolute value of maximum moment in the unbraced segment, kip-in. (N-mm)

M_A = absolute value of moment at quarter point of the unbraced segment, kip-in. (N-mm)

M_B = absolute value of moment at centerline of the unbraced segment, kip-in. (N-mm)

M_C = absolute value of moment at three-quarter point of the unbraced segment, kip-in. (N-mm)

R_m = cross-section monosymmetry parameter
= 1.0, doubly symmetric members
= 1.0, singly symmetric members subjected to *single curvature* bending
= $0.5 + 2\left(\frac{I_{yc}}{I_y}\right)^2$, singly symmetric members subjected to *reverse curvature* bending

I_y = moment of inertia about the principal y-axis, in.^{4} (mm^4)

I_{yc} = moment of inertia about y-axis referred to the compression flange, or if reverse curvature bending, referred to the smaller flange, in.^{4} (mm^4)

In singly symmetric members subjected to *reverse curvature* bending, the *lateral-torsional buckling* strength shall be checked for both flanges. The available flexural strength shall be greater than or equal to the maximum required moment causing compression within the flange under consideration.

C_b is permitted to be conservatively taken as 1.0 for all cases. For cantilevers or overhangs where the free end is unbraced, $C_b = 1.0$.

User Note: For doubly symmetric members with no transverse loading between brace points, Equation F1-1 reduces to 2.27 for the case of equal end moments of opposite sign and to 1.67 when one end moment equals zero.

F2. DOUBLY SYMMETRIC COMPACT I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MAJOR AXIS

This section applies to doubly symmetric I-shaped members and channels bent about their major axis, having compact webs and compact flanges as defined in Section B4.

User Note: All current ASTM A6 W, S, M, C and MC shapes except W21×48, W14×99, W14×90, W12×65, W10×12, W8×31, W8×10, W6×15, W6×9, W6×8.5, and M4×6 have compact flanges for $F_y \leq 50$ ksi (345 MPa); all current ASTM A6 W, S, M, HP, C and MC shapes have compact webs at $F_y \leq 65$ ksi (450 MPa).

The *nominal flexural strength*, M_n, shall be the lower value obtained according to the *limit states* of *yielding* (*plastic moment*) and *lateral-torsional buckling*.

1. Yielding

$$M_n = M_p = F_y Z_x \tag{F2-1}$$

where

F_y = *specified minimum yield stress* of the type of steel being used, ksi (MPa)
Z_x = plastic section modulus about the x-axis, in.^{3} (mm^3)

2. Lateral-Torsional Buckling

(a) When $L_b \leq L_p$, the limit state of lateral-torsional buckling does not apply.
(b) When $L_p < L_b \leq L_r$

$$M_n = C_b \left[M_p - (M_p - 0.7F_y S_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \leq M_p \tag{F2-2}$$

(c) When $L_b > L_r$

$$M_n = F_{cr} S_x \leq M_p \tag{F2-3}$$

where

L_b = length between points that are either braced against lateral displacement of compression flange or braced against twist of the cross section, in. (mm)

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2}\sqrt{1 + 0.078\frac{Jc}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2} \tag{F2-4}$$

and where

E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)
J = torsional constant, in.^{4} (mm^4)
S_x = elastic section modulus taken about the x-axis, in.^{3} (mm^3)

User Note: The square root term in Equation F2-4 may be conservatively taken equal to 1.0.

The limiting lengths L_p and L_r are determined as follows:

$$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} \tag{F2-5}$$

$$L_r = 1.95 r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{S_x h_o}} \sqrt{1 + \sqrt{1 + 6.76 \left(\frac{0.7F_y}{E} \frac{S_x h_o}{Jc} \right)^2}} \tag{F2-6}$$

where

$$r_{ts}^2 = \frac{\sqrt{I_y C_w}}{S_x} \tag{F2-7}$$

and

For a doubly symmetric I-shape: $c = 1$ (F2-8a)

For a channel: $c = \frac{h_o}{2}\sqrt{\frac{I_y}{C_w}}$ (F2-8b)

where

h_o = distance between the flange centroids, in. (mm)

User Note: If the square root term in Equation F2-4 is conservatively taken equal to 1, Equation F2-6 becomes

$$L_r = \pi r_{ts} \sqrt{\frac{E}{0.7F_y}}$$

Note that this approximation can be extremely conservative.

For doubly symmetric I-shapes with rectangular flanges, $C_w = \frac{I_y h_o^2}{4}$ and thus Equation F2-7 becomes

$$r_{ts}^2 = \frac{I_y h_o}{2S_x}$$

r_{ts} may be approximated accurately and conservatively as the radius of gyration of the compression flange plus one-sixth of the web:

$$r_{ts} = \frac{b_f}{\sqrt{12\left(1 + \frac{1}{6}\frac{h t_w}{b_f t_f}\right)}}$$

F3. DOUBLY SYMMETRIC I-SHAPED MEMBERS WITH COMPACT WEBS AND NONCOMPACT OR SLENDER FLANGES BENT ABOUT THEIR MAJOR AXIS

This section applies to doubly symmetric I-shaped members bent about their major axis having compact webs and noncompact or slender flanges as defined in Section B4.

User Note: The following shapes have noncompact flanges for $F_y = 50$ ksi (345 MPa): W21×48, W14×99, W14×90, W12×65, W10×12, W8×31, W8×10, W6×15, W6×9, W6 × 8.5, and M4×6. All other ASTM A6 W, S, M, and HP shapes have compact flanges for $F_y \leq 50$ ksi (345 MPa).

The nominal flexural strength, M_n, shall be the lower value obtained according to the *limit states* of *lateral-torsional buckling* and compression flange *local buckling*.

1. Lateral-Torsional Buckling

For *lateral-torsional buckling*, the provisions of Section F2.2 shall apply.

2. Compression Flange Local Buckling

(a) For sections with noncompact flanges

$$M_n = \left[M_p - (M_p - 0.7F_yS_x)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] \tag{F3-1}$$

(b) For sections with slender flanges

$$M_n = \frac{0.9Ek_cS_x}{\lambda^2} \tag{F3-2}$$

where

$\lambda = \dfrac{b_f}{2t_f}$

$\lambda_{pf} = \lambda_p$ is the limiting slenderness for a compact flange, Table B4.1

$\lambda_{rf} = \lambda_r$ is the limiting slenderness for a noncompact flange, Table B4.1

$k_c = \dfrac{4}{\sqrt{h/t_w}}$ and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes

F4. OTHER I-SHAPED MEMBERS WITH COMPACT OR NONCOMPACT WEBS BENT ABOUT THEIR MAJOR AXIS

This section applies to: (a) doubly symmetric I-shaped members bent about their major axis with noncompact webs; and (b) singly symmetric I-shaped members with webs attached to the mid-width of the flanges, bent about their major axis, with compact or noncompact webs, as defined in Section B4.

User Note: I-shaped members for which this section is applicable may be designed conservatively using Section F5.

The nominal flexural strength, M_n, shall be the lowest value obtained according to the *limit states* of compression flange *yielding*, *lateral-torsional buckling*, compression flange *local buckling* and tension flange yielding.

1. Compression Flange Yielding

$$M_n = R_{pc} M_{yc} = R_{pc} F_y S_{xc} \tag{F4-1}$$

2. Lateral-Torsional Buckling

(a) When $L_b \le L_p$, the *limit state* of *lateral-torsional buckling* does not apply.
(b) When $L_p < L_b \le L_r$

$$M_n = C_b \left[R_{pc} M_{yc} - \left(R_{pc} M_{yc} - F_L S_{xc} \right) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \le R_{pc} M_{yc} \tag{F4-2}$$

(c) When $L_b > L_r$

$$M_n = F_{cr} S_{xc} \le R_{pc} M_{yc} \tag{F4-3}$$

where

$$M_{yc} = F_y S_{xc} \tag{F4-4}$$

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\dfrac{L_b}{r_t} \right)^2} \sqrt{1 + 0.078 \frac{J}{S_{xc} h_o} \left(\frac{L_b}{r_t} \right)^2} \tag{F4-5}$$

For $\dfrac{I_{yc}}{I_y} \le 0.23$, J shall be taken as zero.

The stress, F_L, is determined as follows:

(i) For $\dfrac{S_{xt}}{S_{xc}} \ge 0.7$

$$F_L = 0.7 F_y \tag{F4-6a}$$

(ii) For $\dfrac{S_{xt}}{S_{xc}} < 0.7$

$$F_L = F_y \frac{S_{xt}}{S_{xc}} \ge 0.5 F_y \tag{F4-6b}$$

The limiting laterally unbraced length for the limit state of *yielding*, L_p, is

$$L_p = 1.1 r_t \sqrt{\frac{E}{F_y}} \tag{F4-7}$$

The limiting unbraced length for the limit state of inelastic lateral-torsional buckling, L_r, is

$$L_r = 1.95 r_t \frac{E}{F_L} \sqrt{\frac{J}{S_{xc} h_o}} \sqrt{1 + \sqrt{1 + 6.76 \left(\frac{F_L}{E} \frac{S_{xc} h_o}{J} \right)^2}} \tag{F4-8}$$

The web *plastification* factor, R_{pc}, is determined as follows:

(i) For $\dfrac{h_c}{t_w} \le \lambda_{pw}$

$$R_{pc} = \frac{M_p}{M_{yc}} \tag{F4-9a}$$

(ii) For $\dfrac{h_c}{t_w} > \lambda_{pw}$

$$R_{pc} = \left[\frac{M_p}{M_{yc}} - \left(\frac{M_p}{M_{yc}} - 1\right)\left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right)\right] \le \frac{M_p}{M_{yc}} \tag{F4-9b}$$

where

M_p $= Z_x F_y \le 1.6 S_{xc} F_y$
S_{xc}, S_{xt} = elastic section modulus referred to tension and compression flanges, respectively, in.^{3} (mm^3)
λ $= \dfrac{h_c}{t_w}$
λ_{pw} $= \lambda_p$, the limiting slenderness for a compact web, Table B4.1
λ_{rw} $= \lambda_r$, the limiting slenderness for a noncompact web, Table B4.1

The effective radius of gyration for lateral-torsional buckling, r_t, is determined as follows:

(i) For I-shapes with a rectangular compression flange:

$$r_t = \frac{b_{fc}}{\sqrt{12\left(\dfrac{h_o}{d} + \dfrac{1}{6}a_w \dfrac{h^2}{h_o d}\right)}} \tag{F4-10}$$

where

$$a_w = \frac{h_c t_w}{b_{fc} t_{fc}} \tag{F4-11}$$

b_{fc} = compression flange width, in. (mm)
t_{fc} = compression flange thickness, in. (mm)

(ii) For I-shapes with channel caps or *cover plates* attached to the compression flange:

r_t = radius of gyration of the flange components in flexural compression plus one-third of the web area in compression due to application of major axis bending moment alone, in. (mm)
a_w = the ratio of two times the web area in compression due to application of major axis bending moment alone to the area of the compression flange components

User Note: For I-shapes with a rectangular compression flange, r_t may be approximated accurately and conservatively as the radius of gyration of the compression flange plus one-third of the compression portion of the web; in other words,

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1 + \frac{1}{6}a_w\right)}}$$

3. Compression Flange Local Buckling

(a) For sections with compact flanges, the *limit state* of *local buckling* does not apply.

(b) For sections with noncompact flanges

$$M_n = \left[R_{pc}M_{yc} - (R_{pc}M_{yc} - F_L S_{xc})\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] \tag{F4-12}$$

(c) For sections with slender flanges

$$M_n = \frac{0.9Ek_c S_{xc}}{\lambda^2} \tag{F4-13}$$

where

F_L is defined in Equations F4-6a and F4-6b

R_{pc} is the web *plastification* factor, determined by Equations F4-9

$k_c = \dfrac{4}{\sqrt{h/t_w}}$ and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes

$\lambda = \dfrac{b_{fc}}{2t_{fc}}$

$\lambda_{pf} = \lambda_p$, the limiting slenderness for a compact flange, Table B4.1

$\lambda_{rf} = \lambda_r$, the limiting slenderness for a noncompact flange, Table B4.1

4. Tension Flange Yielding

(a) When $S_{xt} \geq S_{xc}$, the *limit state* of tension flange *yielding* does not apply.

(b) When $S_{xt} < S_{xc}$

$$M_n = R_{pt}M_{yt} \tag{F4-14}$$

where

$$M_{yt} = F_y S_{xt}$$

The web *plastification* factor corresponding to the tension flange yielding limit state, R_{pt}, is determined as follows:

(i) For $\dfrac{h_c}{t_w} \leq \lambda_{pw}$

$$R_{pt} = \frac{M_p}{M_{yt}} \tag{F4-15a}$$

(ii) For $\frac{h_c}{t_w} > \lambda_{pw}$

$$R_{pt} = \left[\frac{M_p}{M_{yt}} - \left(\frac{M_p}{M_{yt}} - 1\right)\left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right)\right] \le \frac{M_p}{M_{yt}} \qquad \text{(F4-15b)}$$

where

$\lambda \quad = \frac{h_c}{t_w}$

$\lambda_{pw} = \lambda_p$, the limiting slenderness for a compact web, defined in Table B4.1

$\lambda_{rw} = \lambda_r$, the limiting slenderness for a noncompact web, defined in Table B4.1

F5. DOUBLY SYMMETRIC AND SINGLY SYMMETRIC I-SHAPED MEMBERS WITH SLENDER WEBS BENT ABOUT THEIR MAJOR AXIS

This section applies to doubly symmetric and singly symmetric I-shaped members with slender webs attached to the mid-width of the flanges, bent about their major axis, as defined in Section B4.

The nominal flexural strength, M_n, shall be the lowest value obtained according to the *limit states* of compression flange *yielding*, *lateral-torsional buckling*, compression flange *local buckling* and tension flange yielding.

1. Compression Flange Yielding

$$M_n = R_{pg} F_y S_{xc} \qquad \text{(F5-1)}$$

2. Lateral-Torsional Buckling

$$M_n = R_{pg} F_{cr} S_{xc} \qquad \text{(F5-2)}$$

(a) When $L_b \le L_p$, the *limit state* of *lateral-torsional buckling* does not apply.

(b) When $L_p < L_b \le L_r$

$$F_{cr} = C_b \left[F_y - (0.3F_y)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le F_y \qquad \text{(F5-3)}$$

(c) When $L_b > L_r$

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_t}\right)^2} \le F_y \qquad \text{(F5-4)}$$

where

L_p is defined by Equation F4-7

$$L_r = \pi r_t \sqrt{\frac{E}{0.7F_y}} \qquad \text{(F5-5)}$$

R_{pg} is the bending strength reduction factor:

$$R_{pg} = 1 - \frac{a_w}{1200 + 300a_w}\left(\frac{h_c}{t_w} - 5.7\sqrt{\frac{E}{F_y}}\right) \leq 1.0 \qquad \text{(F5-6)}$$

a_w is defined by Equation F4-11 but shall not exceed 10

and

r_t is the effective radius of gyration for lateral buckling as defined in Section F4.

3. Compression Flange Local Buckling

$$M_n = R_{pg} F_{cr} S_{xc} \qquad \text{(F5-7)}$$

(a) For sections with compact flanges, the *limit state* of compression flange *local buckling* does not apply.

(b) For sections with noncompact flanges

$$F_{cr} = \left[F_y - (0.3F_y)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] \qquad \text{(F5-8)}$$

(c) For sections with slender flange sections

$$F_{cr} = \frac{0.9Ek_c}{\left(\frac{b_f}{2t_f}\right)^2} \qquad \text{(F5-9)}$$

where

$k_c = \dfrac{4}{\sqrt{h/t_w}}$ and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes

$\lambda = \dfrac{b_{fc}}{2t_{fc}}$

$\lambda_{pf} = \lambda_p$, the limiting slenderness for a compact flange, Table B4.1

$\lambda_{rf} = \lambda_r$, the limiting slenderness for a noncompact flange, Table B4.1

4. Tension Flange Yielding

(a) When $S_{xt} \geq S_{xc}$, the *limit state* of tension flange *yielding* does not apply.

(b) When $S_{xt} < S_{xc}$

$$M_n = F_y S_{xt} \qquad \text{(F5-10)}$$

F6. I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MINOR AXIS

This section applies to I-shaped members and channels bent about their minor axis.

The nominal flexural strength, M_n, shall be the lower value obtained according to the *limit states* of *yielding* (*plastic moment*) and flange *local buckling*.

1. Yielding

$$M_n = M_p = F_y Z_y \leq 1.6F_y S_y \qquad \text{(F6-1)}$$

2. Flange Local Buckling

(a) For sections with compact flanges the limit state of yielding shall apply.

> **User Note:** All current ASTM A6 W, S, M, C and MC shapes except W21×48, W14×99, W14×90, W12×65, W10×12, W8×31, W8×10, W6×15, W6×9, W6×8.5, and M4×6 have compact flanges at $F_y \leq 50$ ksi (345 MPa).

(b) For sections with noncompact flanges

$$M_n = \left[M_p - (M_p - 0.7F_yS_y)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] \tag{F6-2}$$

(c) For sections with slender flanges

$$M_n = F_{cr}S_y \tag{F6-3}$$

where

$$F_{cr} = \frac{0.69E}{\left(\frac{b_f}{2t_f}\right)^2} \tag{F6-4}$$

$\lambda = \dfrac{b}{t}$

$\lambda_{pf} = \lambda_p$, the limiting slenderness for a compact flange, Table B4.1

$\lambda_{rf} = \lambda_r$, the limiting slenderness for a noncompact flange, Table B4.1

S_y for a channel shall be taken as the minimum section modulus

F7. SQUARE AND RECTANGULAR HSS AND BOX-SHAPED MEMBERS

This section applies to square and rectangular *HSS*, and doubly symmetric box-shaped members bent about either axis, having compact or noncompact webs and compact, noncompact or slender flanges as defined in Section B4.

The nominal flexural strength, M_n, shall be the lowest value obtained according to the *limit states* of *yielding (plastic moment)*, flange *local buckling* and web *local buckling* under pure flexure.

1. Yielding

$$M_n = M_p = F_yZ \tag{F7-1}$$

where

Z = plastic section modulus about the axis of bending, in.^{3} (mm^3)

2. Flange Local Buckling

(a) For *compact sections*, the *limit state* of flange *local buckling* does not apply.
(b) For sections with noncompact flanges

$$M_n = M_p - (M_p - F_yS)\left(3.57\frac{b}{t}\sqrt{\frac{F_y}{E}} - 4.0\right) \leq M_p \tag{F7-2}$$

(c) For sections with slender flanges

$$M_n = F_y S_{eff} \tag{F7-3}$$

where

S_{eff} is the *effective section modulus* determined with the *effective width* of the compression flange taken as:

$$b_e = 1.92t\sqrt{\frac{E}{F_y}}\left[1 - \frac{0.38}{b/t}\sqrt{\frac{E}{F_y}}\right] \le b \tag{F7-4}$$

3. Web Local Buckling

(a) For *compact sections*, the *limit state* of web *local buckling* does not apply.
(b) For sections with noncompact webs

$$M_n = M_p - (M_p - F_y S_x)\left(0.305\frac{h}{t_w}\sqrt{\frac{F_y}{E}} - 0.738\right) \le M_p \tag{F7-5}$$

F8. ROUND HSS

This section applies to round *HSS* having D/t ratios of less than $\dfrac{0.45E}{F_y}$.

The nominal flexural strength, M_n, shall be the lower value obtained according to the *limit states* of *yielding (plastic moment)* and *local buckling*.

1. Yielding

$$M_n = M_p = F_y Z \tag{F8-1}$$

2. Local Buckling

(a) For *compact sections*, the *limit state* of flange *local buckling* does not apply.
(b) For noncompact sections

$$M_n = \left(\frac{0.021E}{\frac{D}{t}} + F_y\right) S \tag{F8-2}$$

(c) For sections with slender walls

$$M_n = F_{cr} S \tag{F8-3}$$

where

$$F_{cr} = \frac{0.33E}{\frac{D}{t}} \tag{F8-4}$$

S = elastic section modulus, in.^{3} (mm^3)

F9. TEES AND DOUBLE ANGLES LOADED IN THE PLANE OF SYMMETRY

This section applies to tees and double angles loaded in the plane of symmetry.

The nominal flexural strength, M_n, shall be the lowest value obtained according to the *limit states* of *yielding (plastic moment)*, *lateral-torsional buckling* and flange *local buckling*.

1. Yielding

$$M_n = M_p \tag{F9-1}$$

where

$$M_p = F_y Z_x \leq 1.6 M_y \quad \text{for stems in tension} \tag{F9-2}$$

$$\leq M_y \quad \text{for stems in compression} \tag{F9-3}$$

2. Lateral-Torsional Buckling

$$M_n = M_{cr} = \frac{\pi\sqrt{EI_y GJ}}{L_b}\left[B + \sqrt{1+B^2}\right] \tag{F9-4}$$

where

$$B = \pm\, 2.3\left(\frac{d}{L_b}\right)\sqrt{\frac{I_y}{J}} \tag{F9-5}$$

The plus sign for B applies when the stem is in tension and the minus sign applies when the stem is in compression. If the tip of the stem is in compression anywhere along the *unbraced length*, the negative value of B shall be used.

3. Flange Local Buckling of Tees

$$M_n = F_{cr} S_{xc} \tag{F9-6}$$

S_{xc} is the elastic section modulus referred to the compression flange.
F_{cr} is determined as follows:

(a) For *compact sections*, the *limit state* of flange *local buckling* does not apply.
(b) For noncompact sections

$$F_{cr} = F_y\left(1.19 - 0.50\left(\frac{b_f}{2t_f}\right)\sqrt{\frac{F_y}{E}}\right) \tag{F9-7}$$

(c) For slender sections

$$F_{cr} = \frac{0.69E}{\left(\frac{b_f}{2t_f}\right)^2} \tag{F9-8}$$

F10. SINGLE ANGLES

This section applies to single angles with and without continuous lateral restraint along their length.

Single angles with continuous lateral-torsional restraint along the length shall be permitted to be designed on the basis of *geometric axis* (x, y) bending. Single angles without continuous lateral-torsional restraint along the length shall be designed using the provisions for *principal axis* bending except where the provision for bending about a geometric axis is permitted.

> **User Note:** For geometric axis design, use section properties computed about the x- and y-axis of the angle, parallel and perpendicular to the legs. For principal axis design use section properties computed about the major and minor principal axes of the angle.

The *nominal flexural strength*, M_n, shall be the lowest value obtained according to the *limit states* of *yielding (plastic moment)*, *lateral-torsional buckling* and leg *local buckling*.

1. Yielding

$$M_n = 1.5 M_y \tag{F10-1}$$

where

M_y = *yield moment* about the axis of bending, kip-in. (N-mm)

2. Lateral-Torsional Buckling

For single angles without continuous lateral-torsional restraint along the length

(a) When $M_e \leq M_y$

$$M_n = \left(0.92 - \frac{0.17 M_e}{M_y}\right) M_e \tag{F10-2}$$

(b) When $M_e > M_y$

$$M_n = \left(1.92 - 1.17\sqrt{\frac{M_y}{M_e}}\right) M_y \leq 1.5 M_y \tag{F10-3}$$

where

M_e, the elastic *lateral-torsional buckling* moment, is determined as follows:

(i) For bending about one of the *geometric axes* of an equal-leg angle with no lateral-torsional restraint

(a) With maximum compression at the toe

$$M_e = \frac{0.66 E b^4 t C_b}{L^2}\left(\sqrt{1 + 0.78\left(\frac{Lt}{b^2}\right)^2} - 1\right) \tag{F10-4a}$$

(b) With maximum tension at the toe

$$M_e = \frac{0.66 E b^4 t C_b}{L^2}\left(\sqrt{1 + 0.78\left(\frac{Lt}{b^2}\right)^2} + 1\right) \tag{F10-4b}$$

M_y shall be taken as 0.80 times the *yield moment* calculated using the geometric section modulus.

User Note: M_n may be taken as M_y for single angles with their vertical leg toe in compression, and having a span-to-depth ratio less than or equal to $\frac{1.64E}{F_y}\sqrt{\left(\frac{t}{b}\right)^2 - 1.4\frac{F_y}{E}}$.

(ii) For bending about one of the geometric axes of an equal-leg angle with lateral-torsional restraint at the point of maximum moment only

M_e shall be taken as 1.25 times M_e computed using Equation F10-4a or F10-4b.

M_y shall be taken as the yield moment calculated using the geometric section modulus.

(iii) For bending about the major principal axis of equal-leg angles:

$$M_e = \frac{0.46Eb^2t^2C_b}{L} \tag{F10-5}$$

(iv) For bending about the major principal axis of unequal-leg angles:

$$M_e = \frac{4.9EI_zC_b}{L^2}\left(\sqrt{\beta_w^2 + 0.052\left(\frac{Lt}{r_z}\right)^2} + \beta_w\right) \tag{F10-6}$$

where

C_b is computed using Equation F1-1 with a maximum value of 1.5.
L = laterally *unbraced length* of a member, in. (mm)
I_z = minor principal axis moment of inertia, in.^{4} (mm^4)
r_z = radius of gyration for the minor principal axis, in. (mm)
t = angle leg thickness, in. (mm)
β_w = a section property for unequal leg angles, positive for short legs in compression and negative for long legs in compression. If the long leg is in compression anywhere along the unbraced length of the member, the negative value of β_w shall be used.

User Note: The equation for β_w and values for common angle sizes are listed in the Commentary.

3. Leg Local Buckling

The *limit state* of leg *local buckling* applies when the toe of the leg is in compression.

(a) For *compact sections,* the limit state of leg local buckling does not apply.
(b) For sections with noncompact legs

$$M_n = F_yS_c\left(2.43 - 1.72\left(\frac{b}{t}\right)\sqrt{\frac{F_y}{E}}\right) \tag{F10-7}$$

(c) For sections with slender legs

$$M_n = F_{cr} S_c \tag{F10-8}$$

where

$$F_{cr} = \frac{0.71E}{\left(\frac{b}{t}\right)^2} \tag{F10-9}$$

b = outside width of leg in compression, in. (mm)

S_c = elastic section modulus to the toe in compression relative to the axis of bending, in.^{3} (mm^3). For bending about one of the *geometric axes* of an equal-leg angle with no lateral-torsional restraint, S_c shall be 0.80 of the geometric axis section modulus.

F11. RECTANGULAR BARS AND ROUNDS

This section applies to rectangular bars bent about either *geometric axis* and rounds.

The *nominal flexural strength*, M_n, shall be the lower value obtained according to the *limit states* of *yielding* (*plastic moment*) and *lateral-torsional buckling*, as required.

1. Yielding

For rectangular bars with $\frac{L_b d}{t^2} \leq \frac{0.08E}{F_y}$ bent about their major axis, rectangular bars bent about their minor axis, and rounds:

$$M_n = M_p = F_y Z \leq 1.6 M_y \tag{F11-1}$$

2. Lateral-Torsional Buckling

(a) For rectangular bars with $\frac{0.08E}{F_y} < \frac{L_b d}{t^2} \leq \frac{1.9E}{F_y}$ bent about their major axis:

$$M_n = C_b \left[1.52 - 0.274\left(\frac{L_b d}{t^2}\right)\frac{F_y}{E}\right] M_y \leq M_p \tag{F11-2}$$

(b) For rectangular bars with $\frac{L_b d}{t^2} > \frac{1.9E}{F_y}$ bent about their major axis:

$$M_n = F_{cr} S_x \leq M_p \tag{F11-3}$$

where

$$F_{cr} = \frac{1.9EC_b}{\frac{L_b d}{t^2}} \tag{F11-4}$$

t = width of rectangular bar parallel to axis of bending, in. (mm)

d = depth of rectangular bar, in. (mm)

L_b = length between points that are either braced against lateral displacement of the compression region or braced against twist of the cross section, in. (mm)

(c) For rounds and rectangular bars bent about their minor axis, the *limit state* of *lateral-torsional buckling* need not be considered.

F12. UNSYMMETRICAL SHAPES

This section applies to all unsymmetrical shapes, except single angles.

The *nominal flexural strength*, M_n, shall be the lowest value obtained according to the *limit states* of *yielding (yield moment), lateral-torsional buckling* and *local buckling* where

$$M_n = F_n S \tag{F12-1}$$

where

S = lowest elastic section modulus relative to the axis of bending, in.^{3} (mm^3)

1. Yielding

$$F_n = F_y \tag{F12-2}$$

2. Lateral-Torsional Buckling

$$F_n = F_{cr} \leq F_y \tag{F12-3}$$

where

F_{cr} = buckling *stress* for the section as determined by analysis, ksi (MPa)

User Note: In the case of Z-shaped members, it is recommended that F_{cr} be taken as $0.5F_{cr}$ of a channel with the same flange and web properties.

3. Local Buckling

$$F_n = F_{cr} \leq F_y \tag{F12-4}$$

where

F_{cr} = buckling *stress* for the section as determined by analysis, ksi (MPa)

F13. PROPORTIONS OF BEAMS AND GIRDERS

1. Hole Reductions

This section applies to rolled or built-up shapes, and cover-plated *beams* with holes, proportioned on the basis of flexural strength of the gross section.

In addition to the *limit states* specified in other sections of this Chapter, the *nominal flexural strength*, M_n, shall be limited according to the limit state of *tensile rupture* of the tension flange.

(a) For $F_u A_{fn} \geq Y_t F_y A_{fg}$, the limit state of tensile rupture does not apply.

(b) For $F_u A_{fn} < Y_t F_y A_{fg}$, the nominal flexural strength, M_n, at the location of the holes in the tension flange shall not be taken greater than:

$$M_n = \frac{F_u A_{fn}}{A_{fg}} S_x \quad \text{(F13-1)}$$

where

A_{fg} = gross tension flange area, calculated in accordance with the provisions of Section D3.1, in.^{2} (mm^2)

A_{fn} = net tension flange area, calculated in accordance with the provisions of Section D3.2, in.^{2} (mm^2)

Y_t = 1.0 for $F_y/F_u \leq 0.8$
= 1.1 otherwise

2. Proportioning Limits for I-Shaped Members

Singly symmetric I-shaped members shall satisfy the following limit:

$$0.1 \leq \frac{I_{yc}}{I_y} \leq 0.9 \quad \text{(F13-2)}$$

I-shaped members with slender webs shall also satisfy the following limits:

(a) For $\frac{a}{h} \leq 1.5$

$$\left(\frac{h}{t_w}\right)_{\max} = 11.7\sqrt{\frac{E}{F_y}} \quad \text{(F13-3)}$$

(b) For $\frac{a}{h} > 1.5$

$$\left(\frac{h}{t_w}\right)_{\max} = \frac{0.42E}{F_y} \quad \text{(F13-4)}$$

where

a = clear distance between *transverse stiffeners*, in. (mm)

In unstiffened girders h/t_w shall not exceed 260. The ratio of the web area to the compression flange area shall not exceed 10.

3. Cover Plates

Flanges of welded *beams* or girders may be varied in thickness or width by splicing a series of plates or by the use of *cover plates*.

The total cross-sectional area of cover plates of bolted girders shall not exceed 70 percent of the total flange area.

High-strength bolts or welds connecting flange to web, or cover plate to flange, shall be proportioned to resist the total *horizontal shear* resulting from the bending *force*s on the girder. The longitudinal distribution of these bolts or intermittent welds shall be in proportion to the intensity of the shear.

However, the longitudinal spacing shall not exceed the maximum permitted for compression or tension members in Section E6 or D4, respectively. Bolts or welds connecting flange to web shall also be proportioned to transmit to the web any *loads* applied directly to the flange, unless provision is made to transmit such loads by direct bearing.

Partial-length cover plates shall be extended beyond the theoretical cutoff point and the extended portion shall be attached to the beam or girder by high-strength bolts in a slip-critical *connection* or *fillet welds*. The attachment shall be adequate, at the applicable strength given in Sections J2.2, J3.8, or B3.9 to develop the cover plate's portion of the flexural strength in the beam or girder at the theoretical cutoff point.

For welded cover plates, the welds connecting the cover plate termination to the beam or girder shall have continuous welds along both edges of the cover plate in the length a', defined below, and shall be adequate to develop the cover plate's portion of the strength of the beam or girder at the distance a' from the end of the cover plate.

(a) When there is a continuous weld equal to or larger than three-fourths of the plate thickness across the end of the plate

$$a' = w \tag{F13-5}$$

where

w = width of cover plate, in. (mm)

(b) When there is a continuous weld smaller than three-fourths of the plate thickness across the end of the plate

$$a' = 1.5w \tag{F13-6}$$

(c) When there is no weld across the end of the plate

$$a' = 2w \tag{F13-7}$$

4. Built-Up Beams

Where two or more *beams* or channels are used side-by-side to form a flexural member, they shall be connected together in compliance with Section E6.2. When concentrated *loads* are carried from one beam to another, or distributed between the beams, *diaphragms* having sufficient *stiffness* to distribute the load shall be welded or bolted between the beams.

CHAPTER G

DESIGN OF MEMBERS FOR SHEAR

This chapter addresses webs of singly or doubly symmetric members subject to shear in the plane of the web, single angles and *HSS* sections, and shear in the weak direction of singly or doubly symmetric shapes.

The chapter is organized as follows:

G1. General Provisions
G2. Members with Unstiffened or Stiffened Webs
G3. Tension Field Action
G4. Single Angles
G5. Rectangular HSS and Box Members
G6. Round HSS
G7. Weak Axis Shear in Singly and Doubly Symmetric Shapes
G8. Beams and Girders with Web Openings

User Note: For applications not included in this chapter, the following sections apply:

- H3.3 Unsymmetric sections.
- J4.2 Shear strength of connecting elements.
- J10.6 Web panel zone shear.

G1. GENERAL PROVISIONS

Two methods of calculating shear strength are presented below. The method presented in Section G2 does not utilize the post *buckling strength* of the member (*tension field action*). The method presented in Section G3 utilizes tension field action.

The *design shear strength*, $\phi_v V_n$, and the *allowable shear strength*, V_n/Ω_v, shall be determined as follows.

For all provisions in this chapter except Section G2.1a:

$$\phi_v = 0.90 \text{ (LRFD)} \qquad \Omega_v = 1.67 \text{ (ASD)}$$

G2. MEMBERS WITH UNSTIFFENED OR STIFFENED WEBS

1. Nominal Shear Strength

This section applies to webs of singly or doubly symmetric members and channels subject to shear in the plane of the web.

The *nominal shear strength*, V_n, of unstiffened or stiffened webs, according to the *limit states* of *shear yielding* and *shear buckling*, is

$$V_n = 0.6F_y A_w C_v \tag{G2-1}$$

(a) For webs of rolled I-shaped members with $h/t_w \leq 2.24\sqrt{E/F_y}$:

$$\phi_v = 1.00\,(\text{LRFD}) \qquad \Omega_v = 1.50\,(\text{ASD})$$

and

$$C_v = 1.0 \tag{G2-2}$$

User Note: All current ASTM A6 W, S and HP shapes except W44×230, W40×149, W36×135, W33×118, W30×90, W24×55, W16×26 and W12×14 meet the criteria stated in Section G2.1(a) for $F_y \leq 50$ ksi (345 MPa).

(b) For webs of all other doubly symmetric shapes and singly symmetric shapes and channels, except round *HSS*, the web shear coefficient, C_v, is determined as follows:

(i) For $h/t_w \leq 1.10\sqrt{k_v E/F_y}$

$$C_v = 1.0 \tag{G2-3}$$

(ii) For $1.10\sqrt{k_v E/F_y} < h/t_w \leq 1.37\sqrt{k_v E/F_y}$

$$C_v = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} \tag{G2-4}$$

(iii) For $h/t_w > 1.37\sqrt{k_v E/F_y}$

$$C_v = \frac{1.51Ek_v}{(h/t_w)^2 F_y} \tag{G2-5}$$

where

A_w = the overall depth times the web thickness, dt_w, in.^{2} (mm^2)

The web plate buckling coefficient, k_v, is determined as follows:

(i) For unstiffened webs with $h/t_w < 260$, $k_v = 5$ except for the stem of tee shapes where $k_v = 1.2$.

(ii) For stiffened webs,

$$k_v = 5 + \frac{5}{(a/h)^2}$$

$$= 5 \text{ when } a/h > 3.0 \text{ or } a/h > \left[\frac{260}{(h/t_w)}\right]^2$$

where

a = clear distance between transverse *stiffener*s, in. (mm)

h = for rolled shapes, the clear distance between flanges less the fillet or corner radii, in. (mm)

= for built-up welded sections, the clear distance between flanges, in. (mm)
= for built-up bolted sections, the distance between *fastener* lines, in. (mm)
= for tees, the overall depth, in. (mm)

User Note: For all ASTM A6 W, S, M and HP shapes except M12.5×12.4, M12.5×11.6, M12×11.8, M12×10.8, M12×10, M10×8, and M10×7.5, when $F_y \le 50$ ksi (345 MPa), $C_v = 1.0$.

2. Transverse Stiffeners

Transverse *stiffener*s are not required where $h/t_w \le 2.46\sqrt{E/F_y}$, or where the required shear strength is less than or equal to the available shear strength provided in accordance with Section G2.1 for $k_v = 5$.

Transverse stiffeners used to develop the available web shear strength, as provided in Section G2.1, shall have a moment of inertia about an axis in the web center for *stiffener* pairs or about the face in contact with the web plate for single stiffeners, which shall not be less than $at_w^3 j$, where

$$j = \frac{2.5}{(a/h)^2} - 2 \ge 0.5 \qquad \text{(G2-6)}$$

Transverse stiffeners are permitted to be stopped short of the tension flange, provided bearing is not needed to transmit a concentrated load or reaction. The weld by which transverse stiffeners are attached to the web shall be terminated not less than four times nor more than six times the web thickness from the near toe to the web-to-flange weld. When single stiffeners are used, they shall be attached to the compression flange, if it consists of a rectangular plate, to resist any uplift tendency due to torsion in the flange. When *lateral bracing* is attached to a stiffener, or a pair of stiffeners, these, in turn, shall be connected to the compression flange to transmit 1 percent of the total flange *force*, unless the flange is composed only of angles.

Bolts connecting stiffeners to the girder web shall be spaced not more than 12 in. (305 mm) on center. If intermittent *fillet weld*s are used, the clear distance between welds shall not be more than 16 times the web thickness nor more than 10 in. (250 mm).

G3. TENSION FIELD ACTION

1. Limits on the Use of Tension Field Action

Consideration of *tension field action* is permitted for flanged members when the web plate is supported on all four sides by flanges or *stiffener*s. Consideration of tension field action is not permitted for:

(a) *end panels* in all members with transverse stiffeners;
(b) members when a/h exceeds 3.0 or $[260/(h/t_w)]^2$;
(c) $2A_w/(A_{fc} + A_{ft}) > 2.5$; or
(d) h/b_{fc} or $h/b_{ft} > 6.0$

where

A_{fc} = area of compression flange, in.^{2} (mm^2)
A_{ft} = area of tension flange, in.^{2} (mm^2)
b_{fc} = width of compression flange, in. (mm)
b_{ft} = width of tension flange, in. (mm)

In these cases, the nominal shear strength, V_n, shall be determined according to the provisions of Section G2.

2. Nominal Shear Strength with Tension Field Action

When *tension field action* is permitted according to Section G3.1, the nominal shear strength, V_n, with tension field action, according to the *limit state* of tension field *yielding*, shall be

(a) For $h/t_w \le 1.10\sqrt{k_v E/F_y}$

$$V_n = 0.6F_y A_w \qquad \text{(G3-1)}$$

(b) For $h/t_w > 1.10\sqrt{k_v E/F_y}$

$$V_n = 0.6F_y A_w \left(C_v + \frac{1 - C_v}{1.15\sqrt{1 + (a/h)^2}} \right) \qquad \text{(G3-2)}$$

where

k_v and C_v are as defined in Section G2.1.

3. Transverse Stiffeners

Transverse *stiffeners* subject to *tension field action* shall meet the requirements of Section G2.2 and the following limitations:

(1) $(b/t)_{st} \le 0.56\sqrt{\dfrac{E}{F_{yst}}}$

(2) $$A_{st} > \frac{F_y}{F_{yst}} \left[0.15D_s h t_w (1 - C_v)\frac{V_r}{V_c} - 18t_w^2 \right] \ge 0 \qquad \text{(G3-3)}$$

where

$(b/t)_{st}$ = the width-thickness ratio of the stiffener
F_{yst} = *specified minimum yield stress* of the stiffener material, ksi (MPa)
C_v = coefficient defined in Section G2.1
D_s = 1.0 for stiffeners in pairs
= 1.8 for single angle stiffeners
= 2.4 for single plate stiffeners
V_r = *required shear strength* at the location of the stiffener, kips (N)
V_c = *available shear strength*; $\phi_v V_n$ (LRFD) or V_n/Ω_v (ASD) with V_n as defined in Section G3.2, kips (N)

G4. SINGLE ANGLES

The nominal shear strength, V_n, of a single angle leg shall be determined using Equation G2-1 with $C_v = 1.0$, $A_w = bt$ where b = width of the leg resisting the shear *force*, in. (mm) and $k_v = 1.2$.

G5. RECTANGULAR HSS AND BOX MEMBERS

The *nominal shear strength*, V_n, of rectangular *HSS* and box members shall be determined using the provisions of Section G2.1 with $A_w = 2ht$ where h for the width resisting the shear *force* shall be taken as the clear distance between the flanges less the inside corner radius on each side and $t_w = t$ and $k_v = 5$. If the corner radius is not known, h shall be taken as the corresponding outside dimension minus three times the thickness.

G6. ROUND HSS

The *nominal shear strength*, V_n, of round *HSS*, according to the *limit states* of *shear yielding* and *shear buckling*, is

$$V_n = F_{cr} A_g / 2 \tag{G6-1}$$

where

F_{cr} shall be the larger of

$$F_{cr} = \frac{1.60E}{\sqrt{\frac{L_v}{D}}\left(\frac{D}{t}\right)^{\frac{5}{4}}} \tag{G6-2a}$$

and

$$F_{cr} = \frac{0.78E}{\left(\frac{D}{t}\right)^{\frac{3}{2}}} \tag{G6-2b}$$

but shall not exceed $0.6F_y$

A_g = gross area of section based on design wall thickness, in.^{2} (mm^2)

D = outside diameter, in. (mm)

L_v = the distance from maximum to zero shear *force*, in. (mm)

t = *design wall thickness*, equal to 0.93 times the nominal wall thickness for ERW *HSS* and equal to the nominal thickness for SAW HSS, in. (mm)

User Note: The shear buckling equations, Equations G6-2a and G6-2b, will control for *D*/*t* over 100, high strength steels, and long lengths. If the shear strength for standard sections is desired, shear yielding will usually control.

G7. WEAK AXIS SHEAR IN SINGLY AND DOUBLY SYMMETRIC SHAPES

For singly and doubly symmetric shapes loaded in the *weak axis* without torsion, the nominal shear strength, V_n, for each shear resisting element shall be determined using Equation G2-1 and Section G2.1(b) with $A_w = b_f t_f$ and $k_v = 1.2$.