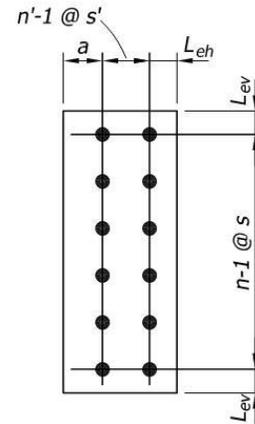


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Single angles and plates	$k_a = 1$
Double angles	$k_a = 2$
Connection material length	L_p
Number of Bolts Rows	n
Number of Bolt Columns	n'
Distance from weld line to first bolt column	a
Vertical bolt spacing	s
Horizontal bolt spacing	s'



Bolt Holes in Calculations

- For all hole related limit states except tear out, the **effective hole diameter** used in calculations is
 $d'_h = d_h + 1/16 \text{ in.} \quad d_b + 1/8$
 The additional 1/16 in. accounts for damage from punching and drilling. $d_b + 1/16$
 - For tear out, the actual hole diameter is used.
- Note: For bearing, the bolt diameter is used. d_b**

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Bolts

From Specification Section J.3.2

High-strength bolts in this Specification are grouped according to material strength as follows:

Group 120	ASTM F3125/F3125M Grades A325, A325M, F1852, and ASTM A354 Grade BC
Group 144	ASTM F3148 Grade 144
Group 150	ASTM F3125/F3125M Grades A490, A490M, F2280, and ASTM A354 Grade BD
Group 200	ASTM F3043 and ASTM F3111

Bolt Group	F _{nv} ksi	F _u Tensile Stress ksi
120	54	120
144	108	144
150	113	150
200	150	200

J3.6 *Single bolt shear* $r_v = F_{nv}A_b$ B-1

$A_b = \frac{\pi d_b^2}{4}$

$d_b =$ nominal unthreaded bolt diameter

Bolt Shear $\phi = 0.75, \Omega = 2.00$

Bolt shear w/o eccentricity $R_n = nr_v$ B-2

Manual *Bolt shear w/ eccentricity* $R_n = Cr_v$ B-3

7-30 Bolt shear coefficient **C** from Manual Table 7-6 thru 7-13

Eccentricity $e_b = a$ for tee connections

Refer to Single Plate and Extended Plate Sections for additional eccentricities.

Bearing and Tearout strength at bolt holes

J3.10 w/o eccentricity on bolts

Bearing = $S_2 d_b t F_u$ (J3-6) *Tearout* = $S_1 l_c t F_u$ (J3-6)

$R_n = k_a(\text{Edge} + (n - 1)\text{Interior})$ $\phi = 0.75, \Omega = 2.00$ B-4a

$\text{Interior} = \text{Min}\left(S_1\left(s - 1.0\left(d_b + \frac{1}{16}\right)\right), S_2 d_b\right) F_u t$

$\text{Edge} = \text{Min}\left(S_1\left(L_{ev} - 0.5\left(d_b + \frac{1}{16}\right)\right), S_2 d_b\right) F_u t$

When deformations at bolt holes are design consideration,

$S_1 = 1.2 \ \& \ S_2 = 2.4$

When deformations at bolt holes are not a design consideration,

$S_1 = 1.5 \ \& \ S_2 = 3.0$, used at SPSC

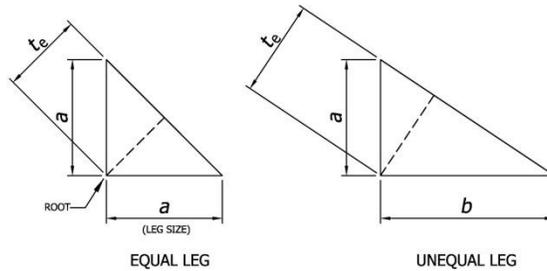
At long slotted holes, $S_1 = 1.0 \ \& \ S_2 = 2.0$

J3.10 w eccentricity on bolts

$R_n = k_a C * \text{min}(\text{Edge}, \text{Bearing})$ $\phi = 0.75, \Omega = 2.00$ B-4b

Welds

Fillet Welds ($F_{EXX} = 70$ ksi)



EFFECTIVE THROAT DIMENSIONS FOR FILLET WELDS

For equal legged Fillet Welds, $A_w = \frac{\sqrt{2}}{2} aL$ Where L = Weld Length

For unequal legged Fillet Welds, $A_w = \frac{ab}{\sqrt{a^2 + b^2}} L$

To allow weld sizes in $D/16$, $A_w = \frac{\sqrt{2} D}{2 \cdot 16} L =$

J2. Welds J2.4. Design Strength

Design Strength = $\phi F_w A_w$.

For Fillet Welds

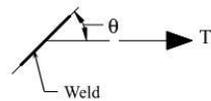
$\phi = 0.75$

$F_w = 0.60 F_{EXX} (1.0 + 0.50 \sin^{1.5} \theta)$

F_{EXX} = electrode strength, ksi

θ = angle of loading measured from the weld longitudinal axis, degrees

= (angle of attack)



J2.4. Fillet Welds Direct shear (no eccentricity)

$$R_n = 0.6 F_{EXX} \frac{\sqrt{2} D}{2 \cdot 16} L (1.0 + 0.5 \sin^{1.5} \theta) = 1.856 DL (1.0 + 0.5 \sin^{1.5} \theta)$$

$$\phi = 0.75, \Omega = 2.00$$

$$\phi R_n = 1.392 DL (1.0 + 0.5 \sin^{1.5} \theta) \quad \text{LRFD} \quad \text{W-1a}$$

$$\frac{R_n}{\Omega} = 0.928 DL (1.0 + 0.5 \sin^{1.5} \theta) \quad \text{ASD} \quad \text{W-1b}$$

Base Metal Effective Weld Sizes:

Shear Rupture $\phi_w 0.60 F_{EXX} \left(\frac{D}{16\sqrt{2}} \right) = \phi_{BM} 0.60 F_u t$

$$D_{eff} = 22.6 \left(\frac{F_u}{F_{EXX}} \right) t = 22.6 \frac{F_u t}{70} = \frac{F_u t}{3.09} \text{ for 70ksi weld}$$

For welds on one side $(D_{eff}) \frac{F_u t}{3.09_{max}}$ W-2

$$t = \frac{3.09 D}{F_u} \text{ for welds on one side of support}$$

When support $t < \frac{3.09 D}{F_u}$, reduce weld capacity by $\frac{F_u t}{3.09 D}$

For weld on both sides $(D_{eff}) \frac{F_u(2t)}{3.09} \frac{F_u t}{6.19_{max}}$ W-3

$$t = \frac{6.19 D}{F_u} \text{ for welds on both sides of support}$$

When support $t < \frac{6.19 D}{F_u}$, reduce weld capacity by $\frac{F_u t}{6.19 D}$

Shear with eccentricity by ultimate analysis

$$R_n = CDL$$

C from AISC Table for single angle

$$e_w = a + e_b$$

C from AISC Table

Shear with eccentricity by vector analysis

Double angle clips at $R_n = \frac{0.6 F_{EXX} \frac{\sqrt{2}}{2} \frac{D}{16} (2L)}{\sqrt{1 + \frac{12.96 e^2}{L^2}}}$

Web Bolt-Weld to support $R_n = 2 * \frac{1.856 DL}{\sqrt{1 + \frac{12.96 e^2}{L^2}}}$ $\phi = 0.75, \Omega = 2.00$ W-5

Where e = width of column leg of angle

Unstiffened seat angle $R_n = \frac{0.6 F_{EXX} \frac{\sqrt{2}}{2} \frac{D}{16} (2L)}{\sqrt{1 + \frac{20.25 e_w^2}{L^2}}}$

$$R_n = 2 * \frac{1.856 DL}{\sqrt{1 + \frac{20.25 e_w^2}{L^2}}} \quad \phi = 0.75, \Omega = 2.00 \quad \text{W-6}$$

Where L = width of column leg of angle

e_w = eccentricity from column face to shear resultant

Stiffened seat $R_n = \frac{0.6 F_{EXX} \frac{\sqrt{2}}{2} \frac{D}{16} (2.4L^2)}{\sqrt{16e^2 + L^2}}$

$$R_n = \frac{4.454 DL}{\sqrt{1 + \frac{10.2W^2}{L^2}}} \quad \phi = 0.75, \Omega = 2.00 \quad \text{W-7}$$

Where L = width of column leg of angle

e = 0.8W and

each horizontal leg length = 0.2L

Weld Rupture – Special Case

$$R_n = \max \begin{cases} R_{wt} + R_{wt} \\ 0.85R_{wt} + 1.5R_{wt} \end{cases}$$

R_{wt} and R_{wt} are the weld strengths with $\theta = 0^\circ$.

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Connection Material

J4.1(a)	Tensile yield strength	$R_n = F_y A_g = k_a F_y L_p t$	$\phi = 0.90, \Omega = 1.67$	CM-1
J4.1(b)	Tensile rupture strength	$R_n = F_u A_e = k_a F_u \left(L_p - n \left(d_b + \frac{1}{8} \right) \right) t$	$\phi = 0.75, \Omega = 2.00$	CM-2
J4.2(a)	Shear yield strength	$R_n = 0.60 F_y A_{gv} = k_a (0.6 F_y) L_p t$	$\phi = 1.00, \Omega = 1.50$	CM-3
J4.2(b)	Shear rupture strength	$R_n = 0.60 F_u A_{nv} = k_a (0.6 F_u) \left(L_p - n \left(d_b + \frac{1}{8} \right) \right) t$	$\phi = 0.75, \Omega = 2.00$	CM-4

J4.3 Block shear strength

Block shear length	$L = s(n - 1) + L_{ev}$		CM-5
Gross shear area	$A_{gv} = tL$		
Net shear area	$A_{nv} = t(L - (n - 0.5)d_h)$		
Net tension area	$A_{nt} = t(L_{eh} - 0.5d_h)$		

Divide by t gives

L (Vertical length of shear plane) CM-5a

$L_s = L - (n - 0.5) \left(d_b + \frac{1}{8} \right)$ CM-5b

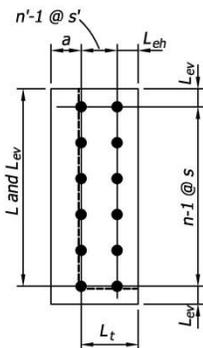
$L_t = L_{eh} - 0.5 \left(d_b + \frac{1}{8} \right)$ (Less 0.25 at beam ends) CM-5c

$R_n = k_a t (0.6 F_u L_s + U_{bs} F_u L_t)$ $\phi = 0.75, \Omega = 2.00$ CM-5d

Where: $0.6 F_u L_s \leq 0.6 F_y L$ and

$U_{bs} = 1.0$ for all except for

multiple column connections



Unstiffed Angle Seat Flexural Yielding

$R_n = F_y \frac{L t a^2}{4e} \leq (0.6 F_y) L t a$ $\phi = 0.90, \Omega = 1.67$ CM-6

$e = \frac{l_b}{2} + \text{Setback} - \frac{1}{4} - t_a - r$

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where l_b is largest required bearing length from Equations CM-7, CM-8, CM-9 & CM-10 (cannot exceed width of angle leg minus $\frac{3}{8}$ -in.)

r = angle fillet radius taken as $\frac{3}{8}$ -in.

t_a = angle thickness

J10.2 *Local web yielding*

$$R_n = (2.5k + l_b)F_{yw}t_w$$

l_b = Bearing Length

Solving for l_b gives

$$l_b = \frac{R_n - 2.5kF_{yw}t_w}{F_{yw}t_w}$$

$$R_1 = 2.5kF_{yw}t_w$$

$$R_2 = l_bF_{yw}t_w \quad \text{Where } l_b=1$$

$$l \frac{R - (\phi, \Omega)R_1}{(\phi, \Omega)R_2} \quad b, \min \quad \phi = 1.00, \Omega = 1.50 \quad \text{CM-7}$$

$$l_{b, \min} \quad \text{CM-8}$$

Web crippling when $\frac{l_b}{d} \leq 0.2$

J10.3(b)(1)

$$R_n = 0.40t_w^2 \left[1 + 3 \left(\frac{l_b}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{EF_{yw}t_f}{t_w}} Q_f$$

Where $Q_f = 1$ for wide-flange members and HSS wall in tension;
 Table K3.2 for all other HSS.

Solving for l_b gives

$$l_b = \frac{R_n - 0.40t_w^2 \sqrt{\frac{EF_{yw}t_f}{t_w}}}{0.40t_w^2 \left(\frac{3}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \sqrt{\frac{EF_{yw}t_f}{t_w}} Q_f}$$

$$R_3 = 0.40t_w^2 \sqrt{\frac{EF_{yw}t_f}{t_w}} Q_f$$

$$R_4 = 0.40t_w^2 \left(\frac{3}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \sqrt{\frac{EF_{yw}t_f}{t_w}} Q_f$$

$$l \frac{R - (\phi, \Omega)R_3}{(\phi, \Omega)R_4} \quad b, \min \quad \phi = 0.75, \Omega = 2.00 \quad \text{CM-9}$$

Web crippling when $\frac{l_b}{d} > 0.2$

J10.3(b)(2)

$$R_n = 0.40t_w^2 \left[1 + \left(\frac{4l_b}{d} - 0.2 \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{EF_{yw}t_f}{t_w}} Q_f$$

Where $Q_f = 1$ for wide-flange members and HSS wall in tension;
 Table K3.2 for all other HSS.

Solving for l_b gives

$$l_b = \frac{d}{4} \left[\frac{R_n - 0.40t_w^2 \sqrt{\frac{EF_{yw}t_f}{t_w}}}{0.40t_w^2 \left(\frac{t_w}{t_f} \right)^{1.5} \sqrt{\frac{EF_{yw}t_f}{t_w}}} + 0.2 \right]$$

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$$R_5 = 0.40t_w^2 \left(1 - 0.2 \left(\frac{t_w}{t_f} \right)^{1.5} \right) \sqrt{\frac{EF_{yw}t_f}{t_w}}$$

$$R_6 = 0.40t_w^2 \left(\frac{4}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \sqrt{\frac{EF_{yw}t_f}{t_w}}$$

$$l \frac{R - (\phi, \Omega)R_5}{(\phi, \Omega)R_6} \quad \phi = 0.75, \Omega = 2.00 \quad \text{CM-10}$$

F11 Tee Stem Flexural Yielding $R_n = F_y \frac{tL_p^2}{4a} \leq 1.6F_y \frac{tL_p^2}{6a} \quad \phi = 0.90, \Omega = 1.67 \quad \text{CM-3}$

Tee Stem Flexural rupture $R_n = \frac{F_u Z_{net}}{a} \quad \phi = 0.75, \Omega = 2.00 \quad \text{CM-4}$

Stiffened Seated Connections:

HSS Wall Yield Line $R_n = k \frac{t^2 F_y L}{4e} \quad \phi = 0.90, \Omega = 1.67 \quad \text{CM-11}$

Where B = HSS Connection face width

$$e = 0.8W$$

$$k = f[gh + m + n]$$

$$f = \frac{2}{2B - 0.4L - t_s}$$

$$g = \left(1 + \frac{B\sqrt{7}}{4L} \right)$$

$$h = \sqrt{(B - 0.4L - t_s)(7B + 0.4L + t_s)}$$

$$m = \frac{B(B - 0.4L - t_s)}{4L}$$

$$n = 2L + B\sqrt{7}$$

(Abolitz and Warner, AISC Engineering Journal, 1st Qtr 1965)

Column Web Yield Line $R_n = k \frac{t^2 F_y L}{4e} \quad \phi = 0.90, \Omega = 1.67 \quad \text{CM-12}$

Where T = Clear distance between web fillets

$$e = 0.8W$$

$$k = A[CD + E + G]$$

$$A = \frac{2}{2T - 0.4L - t_s}$$

$$C = \left(2 + \frac{T\sqrt{3}}{2L} \right)$$

$$D = \sqrt{(T - 0.4L - t_s)(3T + 0.4L + t_s)}$$

$$E = \frac{T(T - 0.4L - t_s)}{2L}$$

$$G = 4L + 2T\sqrt{3}$$

(Ellifritt and Sputo, AISC Engineering Journal, 4th Qtr 1999)

Main Material

Beam Web

J4.1(a)	Tensile yield strength	$R_n = F_y A_g = k_a F_y L_p t$	$\phi = 0.90, \Omega = 1.67$	MM-1
J4.1(b)	Tensile rupture strength	$R_n = F_u A_e = k_a F_u \left(L_p - n \left(d_b + \frac{1}{8} \right) \right) t$	$\phi = 0.75, \Omega = 2.00$	MM-2
J4.2(a)	Shear yield strength	$R_n = 0.60 F_y A_{nv} = 0.6 F_y L_p t$	$\phi = 1.00, \Omega = 1.50$	MM-3
J4.2(b)	Shear rupture strength	$R_n = 0.60 F_u A_e = 0.6 F_u \left(L - n \left(d_b + \frac{1}{8} \right) \right) t$	$\phi = 0.75, \Omega = 2.00$	MM-4
	Beam web strength at weld	$R_n = \frac{0.6 F_u t_w (1.0)}{1.856 (1.5) 2 D (1.0)} WRS$ at double-angles		MM-5

Where WRS = Calculated Weld Rupture Strength

Flange Stiffening

J10.1	Flange local bending	$R_n = 6.25 F_{yf} t_f^2$	$\phi = 0.90, \Omega = 1.67$	
		$P_{fb} = 6.25 F_{yf} t_f^2$		MM-6

Web Stiffening

J10.2(b)	Local web yielding	$R_n = (2.5k + l_b) F_{yw} t_w$	$\phi = 1.00, \Omega = 1.50$	
		Where l_b = Bearing Length		
		$R_1 = 2.5k F_{yw} t_w$		
		$R_2 = F_{yw} t_w$		

Load applied:

$\leq d$ from end $(\phi, \Omega) R_n = (\phi, \Omega) R_1 + l_b (\phi, \Omega) R_2$ **MM-7**

$> d$ from end $(\phi, \Omega) R_n = 2(\phi, \Omega) R_1 + l_b (\phi, \Omega) R_2$ **MM-8**

J10.3(b)(1)	Web local crippling when $\frac{l_b}{d} \leq 0.2$		$\phi = 0.75, \Omega = 2.00$	
-------------	---	--	------------------------------	--

$$R_n = 0.40 t_w^2 \left[1 + 3 \left(\frac{N}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{E F_{yw} t_f}{t_w}} Q_f$$

Where $Q_f = 1$ for wide-flange members and HSS wall in tension;
 Table K3.2 for all other HSS.

$$R_3 = 0.40 t_w^2 \sqrt{\frac{E F_{yw} t_f}{t_w}}$$

$$R_4 = 0.40 t_w^2 \left(\frac{3}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \sqrt{\frac{E F_{yw} t_f}{t_w}}$$

$(\phi, \Omega) R_n = (\phi, \Omega) R_3 + l_b (\phi, \Omega) R_4$ **MM-9**

J10.3(b)(2)	Web local crippling when $\frac{l_b}{d} > 0.2$		$\phi = 0.75, \Omega = 2.00$	
-------------	--	--	------------------------------	--

$$R_n = 0.40 t_w^2 \left[1 + \left(\frac{4N}{d} - 0.2 \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{E F_{yw} t_f}{t_w}}$$

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Solving for l_b gives

$$R_5 = 0.40t_w^2 \left(1 - 0.2 \left(\frac{t_w}{t_f} \right)^{1.5} \right) \sqrt{\frac{EF_{yw}t_f}{t_w}}$$

$$R_6 = 0.40t_w^2 \left(\frac{4}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \sqrt{\frac{EF_{yw}t_f}{t_w}}$$

$$(\phi, \Omega)R_n = (\phi, \Omega)R_5 + l_b(\phi, \Omega)R_6 \quad \text{MM-10}$$

J10.3(a) *Web local crippling when load applied > d from end* $\phi = 0.75, \Omega = 2.00$

$$R_n = 0.40t_w^2 \left[1 + 3 \left(\frac{N}{d} \right) \left(\frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{EF_{yw}t_f}{t_w}}$$

$$(\phi, \Omega)R_n = 2[(\phi, \Omega)R_3 + l_b(\phi, \Omega)R_4] \quad \text{MM-11}$$

Web Compression buckling

J.10.8

$$R_n = \frac{24t_w^3 \sqrt{EF_{yw}}}{h} \quad \phi = 0.90, \Omega = 1.67$$

$$P_{wb} = \frac{24t_w^3 \sqrt{EF_{yw}}}{h} \quad \phi = 0.90, \Omega = 1.67 \quad \text{MM-12}$$

Coped Beam Strength

Coped Section

$$M = R_n e = F_{cr} S_{net}$$

$$R_n = \frac{F_{cr} S_{net}}{e} \quad \phi = 0.90, \Omega = 1.67 \quad \text{MM-13}$$

Where

$$S_{net} = \frac{I_{tr}}{N_{comp}} \text{ for single coped beams and } \frac{bd^2}{6} \text{ for double coped beams}$$

$$e = a - L_{eh} + c + 0.25$$

Top flange coped only

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

$$F_{cr} = 26210 \left(\frac{t_w}{h_o} \right)^2 f K \quad \text{MM-12a}$$

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

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

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

$$k = 2.2 \left(\frac{h_o}{c} \right) \text{ when } \frac{c}{h_o} > 1.0$$

c = cope length

d_c = cope depth

$$h_o = d - d_c$$

Both Flanges coped same length and $c \leq 2d, d_c < 0.2d$

$$F_{cr} = 0.62\pi E \left(\frac{t_w^2}{ch_o} \right) f d \leq F_y$$

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$$F_{cr} = 56485 \left(\frac{t_w^2}{c h_o} \right) f_d$$

MM-12b

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

All other cope conditions

$$F_{cr} = F_y Q$$

MM-12c

$$Q = 1 \text{ for } \lambda \leq 0.7$$

$$Q = 1.34 - 0.486\lambda \text{ for } 0.7 < \lambda \leq 1.41$$

$$Q = \frac{1.30}{\lambda^2} \text{ for } \lambda > 1.41$$

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

At cope conditions where tension flange is longer than compression flange

S_{net} = net elastic section modulus at end of tension flange

Ductility

Local buckling

$$t \frac{L_p}{234} \sqrt{\frac{F_y}{K_{min}}}$$

D-1

$$K = \frac{6}{\pi^2} \left[(1 - \nu) + \frac{(\pi b m)^2}{6} \right]$$

Where: $b = \frac{L_p}{2}$

$$m = 1$$

$$\nu = 0.3, \text{ Poisson' ratio}$$

Tee Stem Ductility

Min. weld size at support

$$w \frac{F_{yc} t_f^2}{b} \left(\frac{b^2}{L^2} + 2 \right) \frac{5}{8 s_{min}}$$

D-2

Where t_f = tee flange thickness

$$b = (b_f - k_1) / 2 \text{ of tee}$$

t_s = tee stem thickness

Max. Tee stem thickness

$$t_s = \frac{d_b}{2} + \frac{1}{16}$$

D-3

Single-Plate

HSS Column Checks

K1.2

$$F_y \leq 52 \text{ ksi}$$

$$\frac{F_y}{F_u} \leq 0.8$$

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HSS wall cannot be slender $\frac{b}{t} \leq 1.40 \sqrt{\frac{E}{F_y}}$

10-10 HSS Punching Shear $F_{yp}t_p = F_{u(HSS)} \cdot t_{HSS}$
 $t_p \leq \frac{F_{u(HSS)}}{F_{yp}} t_{HSS}$

SP-1

Eccentricity & Rotational Ductility

n	Hole Type	e _b	Max t _w or t _p
2-5	SSLT	$\frac{a}{2}$	None
	STD	$\frac{a}{2}$	$\frac{d_b}{2} + \frac{1}{16}$
6-12	SSLT	$\frac{a}{2}$	$\frac{d_b}{2} + \frac{1}{16}$
	STD	a	$\frac{d_b}{2} - \frac{1}{16}$

SP-2

Weld $w = \frac{5}{8}t_p$

SP-3

Extended Single-Plate

Maximum plate thickness $t \leq \frac{6M_{max}}{F_y L_p^2}$

ESP-1

Where $M \leq \frac{F_v}{0.90} b_{max}$

- Exceptions:
- For single bolt column, ignore if either t_p or $t_w \leq \frac{d_b}{2} + \frac{1}{16}$ and both $L_{eh} \geq 2d_b$
 - For double bolt column, ignore if both t_p or $t_w \leq \frac{d_b}{2} + \frac{1}{16}$ and both $L_{eh} \geq 2d_b$

Shear & flexure interaction (Flexural Yielding) $R_n = \frac{F_y L_p t_p}{\sqrt{2.25 + 16 \left(\frac{a}{L_p}\right)^2}}$ $\phi = 0.90, \Omega = 1.67$

ESP-2

Plate buckling $R_n = \frac{F_{cr} S_{net}}{a}$ $\phi = 0.90, \Omega = 1.67$

ESP-3

AISC
Spec or Manual
Section

Connections
Formula
Number

$$F_{cr} = F_y Q$$

$$Q = 1 \text{ for } \lambda \leq 0.7$$

$$Q = 1.34 - 0.486\lambda \text{ for } 0.7 < \lambda \leq 1.41$$

$$Q = \frac{1.30}{\lambda^2} \text{ for } \lambda > 1.41$$

$$\lambda = \frac{L_p \sqrt{F_y}}{10 t_p \sqrt{475 + 280 \left(\frac{L_p}{a}\right)^2}}$$

Weld

$$w = \frac{5}{8} t_p$$

ESP-4

Stabilizer Plate Requirements

$$\text{If applied load } \leq \frac{1500 L_p t_p^3}{a^2} \quad \phi = 0.90, \Omega = 1.67$$

ESP-5

No stabilizer plates required

Connection Torsion Strength

Not applicable if metal deck supports top flange

$$M_T = R \left(\frac{t_w + t_p}{2} \right)$$

$$M_T \leq \left[(\phi_v, \Omega_v) 0.6 F_{yp} - \frac{R}{L_p t_p} \right] \frac{L_p t_p}{2} + \frac{2R(t_w + t_p) b_f}{(\phi_b, \Omega_b) F_{yb} L_s t_w^2}$$

ESP-6

Where L_s = Beam span in inches

Limit States of Shear Yielding & Flexural Yielding

$$\left(\frac{V_r}{(\phi, \Omega) V_n} \right)^2 + \left(\frac{M_r}{(\phi, \Omega) M_n} \right)^2 \leq 1$$

ESP-7

Flexural LTB

F.11.2.(a)

$$\text{If } \frac{L_b d}{t^2} \leq \frac{0.08E}{F_y}, \text{ then LTB does not apply}$$

F11.2.(b)

$$\text{If } \frac{0.08E}{F_y} < \frac{L_b d}{t^2} < \frac{1.9E}{F_y},$$

$$M_n = C_b \left[1.52 - 0.274 \left(\frac{L_b d}{t^2} \right) \frac{F_y}{E} \right] M_y \leq M_p$$

Where $C_b = 1.84$

F11.2.(c)

$$\text{If } \frac{L_b d}{t^2} > \frac{1.9E}{F_y},$$

$$M_n = F_{cr} S_x \leq M_p$$

$$R_n = \frac{M_n}{a}$$

ESP-8