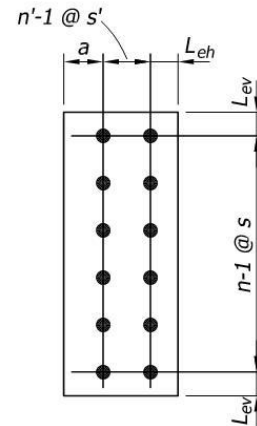


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Formula  
Number**

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

$$d'_h = d_b + 1/16 \text{ in.} \quad d_b + 1/8$$

The additional 1/16 in. accounts for damage from punching and drilling.

- For tear out, the actual hole diameter is used.

**Note:** For bearing, the bolt diameter is used.  $d_b$

## 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*       $R_n = n r_v$       B-2

Manual      *Bolt shear w/ eccentricity*       $R_n = C r_v$       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)

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

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

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

When deformations at bolt holes are design consideration,

$$S_1 = 1.2 \text{ \& } S_2 = 2.4$$

When deformations at bolt holes are not a design consideration,

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

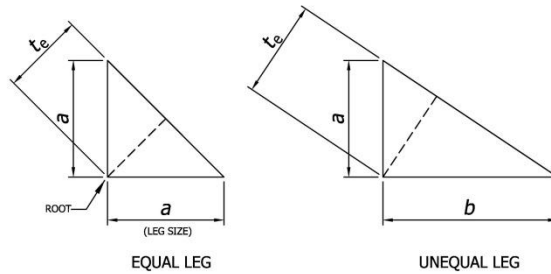
At long slotted holes,  $S_1 = 1.0$  &  $S_2 = 2.0$

J3.10      w eccentricity on bolts

$R_n = k_a C * min(Edge, Bearing)$        $\phi = 0.75, \Omega = 2.00$       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,  $A_w = \frac{\sqrt{2}}{2} \frac{D}{16} L =$

**J2. Welds J2.4. Design Strength**

Design Strength =  $\phi F_w A_w$ .

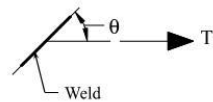
For Fillet Welds

$\phi = 0.75$

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

$F_{EXX}$  = electrode strength, ksi

$\theta$  = angle of loading measured from  
the weld longitudinal axis, degrees  
= (angle of attack)



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

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

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

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

W-1a

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

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

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

Where L = width of column leg of angle

$e_w$  = eccentricity from column face to shear resultant

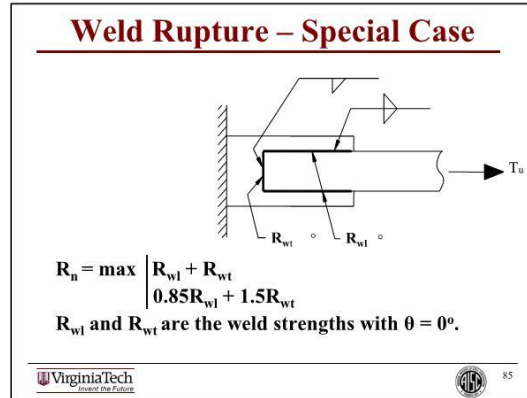
Stiffened seat  $R_n = \frac{0.6 F_{EXX} \frac{\sqrt{2}}{2} \frac{D}{16} (2.4L^2)}{\sqrt{16e^2 + L^2}}$

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

Where L = width of column leg of angle

e = 0.8W and

each horizontal leg length = 0.2L



### 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

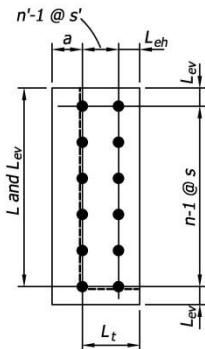
$L_t = L_{eh} - 0.5 \left( d_b + \frac{1}{8} \right)$  (Less 0.25 at beam ends) CM-5c

$R_n = k_a t (0.6 F_u L_s + U_{bs} F_u L_t)$   $\phi = 0.75, \Omega = 2.00$  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  $\frac{3}{4}$ -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

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

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

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

$$l_{b,min} = \frac{R - (\phi\Omega)R_1}{(\phi\Omega)R_2}$$

$$\phi = 1.00, \Omega = 1.50$$

**CM-7**

$$l_{b,min}$$

**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

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

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

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

$$l_{b,min} = \frac{R - (\phi\Omega)R_3}{(\phi\Omega)R_4}$$

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

**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

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

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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*       $R_n = F_y \frac{t_L p^2}{4a} \leq 1.6F_y \frac{t_L p^2}{6a}$        $\phi = 0.90, \Omega = 1.67$       **CM-3**

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

**Stiffened Seated Connections:**

*HSS Wall Yield Line*       $R_n = k \frac{t^2 F_y L}{4e}$        $\phi = 0.90, \Omega = 1.67$       **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, 1<sup>st</sup> Qtr 1965)

*Column Web Yield Line*       $R_n = k \frac{t^2 F_y L}{4e}$        $\phi = 0.90, \Omega = 1.67$       **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, 4<sup>th</sup> 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	$R_n = 6.25 F_y t_f^2$ $P_{fb} = 6.25 F_y t_f^2$	$\phi = 0.90, \Omega = 1.67$	MM-6
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### Web Stiffening

J10.2(b)	Local web yielding	$R_n = (2.5k + l_b) F_y t_w$	$\phi = 1.00, \Omega = 1.50$	
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Where  $l_b$  = Bearing Length

$R_1 = 2.5k F_y t_w$   
 $R_2 = F_y t_w$

Load applied:

$\leq d$  from end

$$(\phi, \Omega) R_n = (\phi, \Omega) R_1 + l_b (\phi, \Omega) R_2 \quad \text{MM-7}$$

$> d$  from end

$$(\phi, \Omega) R_n = 2(\phi, \Omega) R_1 + l_b (\phi, \Omega) R_2 \quad \text{MM-8}$$

J10.3(b)(1)	Web local crippling when $\frac{l_b}{d} \leq 0.2$		$\phi = 0.75, \Omega = 2.00$	
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$$R_n = 0.40 t_w^2 \left[ 1 + 3 \left( \frac{N}{d} \right) \left( \frac{t_w}{t_f} \right)^{1.5} \right] \sqrt{\frac{E F_y t_f}{t_w}} Q_f$$

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

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

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

$$(\phi, \Omega) R_n = (\phi, \Omega) R_3 + l_b (\phi, \Omega) R_4 \quad \text{MM-9}$$

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

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



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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<sub>net</sub> = 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

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

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

$$m = 1$$

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

*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

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

$$t_s = \text{tee stem thickness}$$

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 <sub>b</sub>	Max t <sub>w</sub> or t <sub>p</sub>
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

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$$F_{cr} = F_y Q$$

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

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

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

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

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