AISC Spec or Manual Section | **Connections Formula Number**

Single angles and plates $k_a = 1$

Double angles $k_a = 2$

Connection material length L_p

Number of Bolts in a single line n

Distance from weld line to first bolt column a

Vertical bolt spacing s

Bolts

J3.6 *Single bolt shear* $r_v = F_n A_b$ B-1

$$A_b = \pi d_b^2 / 4$$

$d_b =$ nominal bolt diameter

Bolt Shear ϕ = 0.75, Ω = 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

Bearing strength at bolt holes

J3.10 w/o eccentricity on bolts

$$R_n = k_a(Edge + (n-1)Bearing)$$ ϕ = 0.75, Ω = 2.00 B-4a

$$Bearing = Min\left(S_1\left(s - 1.0\left(d_b + 1/16\right)\right), S_2 d_b\right) F_u t$$

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

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, $S_1 = 1.0 \;\&\; S_2 = 2.0$

AISC Spec or Manual Section			Connections Formula Number
J3.10	w eccentricity on bolts		
	$R_n = k_a C * \min(Edge, Bearing)$	$\phi = 0.75$, $\Omega = 2.00$	B-4b

Welds

Welds (F_{EXX} = 70 ksi)

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

$$R_n = 0.6F_{EXX}\frac{\sqrt{2}}{2}\frac{D}{16}L = 1.856DL$$

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

$$\phi R_n = 1.392DL$$ LRFD — W-1a

$$\frac{R_n}{\Omega} = 0.928DL$$ ASD — W-1b

Base Metal Effective Weld Sizes:

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

$$D_{eff} = 22.6\left(\frac{F_u}{F_{EXX}}\right)t$$

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

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

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

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

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

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

Shear with eccentricity by ultimate analysis

Manual $$R_n = CDL$$ W-4

AISC Spec or Manual Section			Connections Formula Number
Page 8-9		C from AISC Table for single angle	
		$e_w = a + e_b$	
		C from AISC Table	

Shear with eccentricity by vector analysis

$$R_n = 0.6F_{EXX}\frac{\sqrt{2}}{2}\frac{D}{16}(2L)\Big/\sqrt{1+\frac{12.96L_c^2}{L^2}}$$

Double angle clips

$$R_n = \frac{3.712DL}{\sqrt{1+\frac{12.96L_c^2}{L^2}}}$$

$\phi = 0.75$, $\Omega = 2.00$ — W-5

Where L_c = width of column leg of angle

(From Salmon and Johnson)

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

Unstiffened seat angle

$$R_n = \frac{3.712DL}{\sqrt{1+\frac{20.25e_w^2}{L^2}}}$$

$\phi = 0.75$, $\Omega = 2.00$ — W-6

Where L = width of column leg of angle

e_w = eccentricity from column face to shear resultant

(From Salmon and Johnson)

$$R_n = 0.6F_{EXX}\frac{\sqrt{2}}{2}\frac{D}{16}\left(2.4L^2\right)\Big/\sqrt{16e^2+L^2}$$

Stiffened seat

$$R_n = \frac{4.454DL}{\sqrt{1+\frac{10.2W^2}{L^2}}}$$

$\phi = 0.75$, $\Omega = 2.00$ — W-7

Where L = width of column leg of angle

e = 0.8W and

each horizontal leg length = 0.2L

(From Salmon and Johnson)

Connection Material

J4.2(a) *Shear yield strength* $R_n = k_a(0.6F_y)L_p t$ ϕ = 1.00, Ω = 1.50 CM-1

J4.2(b) *Shear rupture strength* $R_n = k_a(0.6F_u)(L_p - n(d_b + 1/8))t$ ϕ = 0.75, Ω = 2.00 CM-2

F11 *Tee Stem Flexural Yielding* $R_n = F_y \dfrac{tL_p^{\,2}}{4a} \le 1.6F_y \dfrac{tL_p^{\,2}}{6a}$ ϕ = 0.90, Ω = 1.67 CM-3

Tee Stem Flexural rupture $R_n = F_u Z_{net} / a$ ϕ = 0.75, Ω = 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

$L_s = L - (n-0.5)(d_b + 1/8)$ CM-5b

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

$R_n = k_a t\left(0.6F_U L_s + U_{bs}F_u L_t\right)$ ϕ = 0.75, Ω = 2.00 CM-5d

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

$U_{bs} = 1.0$ for all except for double row connections

Unstiffed Angle Seat Flexural Yielding

$R_n = F_y \dfrac{Lt_a^{\,2}}{4e} \le (0.6F_y)Lt_a$ ϕ = 0.90, Ω = 1.67 CM-6

$e = l_b/2 + Setback - 1/4 - t_a - r$

where l_b is largest required bearing length from Equations 8b, 8c, 8d and 8e (cannot exceed width of angle leg minus ¾-in.)

r = angle fillet radius taken as $^3/_8$-in.

t_a = angle thickness

J10.2 *Local web yielding* $R_n = (2.5k + l_b)F_{yw}t_w$

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l_b = Bearing Length

Solving for l_b gives

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

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

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

$$l_{b,\min} = \frac{R-(\phi,\Omega)R_1}{(\phi,\Omega)R_2}$$ ϕ = 1.00, Ω = 1.50 CM-5

$$l_{b,\min} = k$$ CM-6

Web crippling when $l_b/d \le 0.2$

J10.3(b)(i) $$R_n = 0.40t_w^2\left[1+3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}}$$

Solving for l_b gives

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

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

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

$$l_{b,\min} = \frac{R-(\phi,\Omega)R_3}{(\phi,\Omega)R_4}$$ ϕ = 0.75, Ω = 2.00 CM-7

Web crippling when $l_b/d > 0.2$

J10.3(b)(ii) $$R_n = 0.40t_w^2\left[1+\left(\frac{4l_b}{d}-0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}}$$

Solving for l_b gives

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

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

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

$$l_{b,\min} = \frac{R - (\phi,\Omega)R_5}{(\phi,\Omega)R_6}$$ $\phi = 0.75,\ \Omega = 2.00$ CM-8

Stiffened Seated Connections:

HSS Wall Yield Line $$R_n = k\frac{t^2F_yL}{4e}$$ $\phi = 0.90,\ \Omega = 1.67$ CM-9

Where B = HSS Connection face width

$$e = 0.8W$$

$$k = f\left[gh + m + n\right]$$

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

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

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

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

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

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

Column Web Yield Line $$R_n = k\frac{t^2F_yL}{4e}$$ $\phi = 0.90,\ \Omega = 1.67$ CM-10

Where T = Clear distance between web fillets

$$e = 0.8W$$

$$k = A\left[CD + E + G\right]$$

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

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

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

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

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

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

Main Material

Beam Web

J4.2(a)	*Shear yield strength*	$R_n = 0.6F_y L_p t$	$\phi = 1.00, \Omega = 1.50$	MM-1
J4.2(b)	*Shear rupture strength*	$R_n = 0.6F_u(L - n(d_b + 1/8))t$	$\phi = 0.75, \Omega = 2.00$	MM-2

Flange Stiffening

J10.1	*Flange local bending*	$R_n = 6.25F_{yf}t^2{}_f$	$\phi = 0.90, \Omega = 1.67$	
		$P_{fb} = 6.25F_{yf}t^2{}_f$		MM-3

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		
		$R_1 = 2.5kF_{yw}t_w$		
		$R_2 = F_{yw}t_w$		
	Load applied:			
	$\leq d$ *from end*	$(\phi,\Omega)R_n = (\phi,\Omega)R_1 + l_b(\phi,\Omega)R_2$		MM-4
	$> d$ *from end*	$(\phi,\Omega)R_n = 2(\phi,\Omega)R_1 + l_b(\phi,\Omega)R_2$		MM-5

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J10.3(b)(i) *Web local crippling when* $l_b/d \le 0.2$ $\phi = 0.75$, $\Omega = 2.00$

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

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

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

$$(\phi,\Omega)R_n = (\phi,\Omega)R_3 + l_b(\phi,\Omega)R_4$$ MM-6

J10.3(b)(ii) *Web local crippling when* $l_b/d > 0.2$ $\phi = 0.75$, $\Omega = 2.00$

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

Solving for l_b gives

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

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

$$(\phi,\Omega)R_n = (\phi,\Omega)R_5 + l_b(\phi,\Omega)R_6$$ MM-7

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

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

$$(\phi,\Omega)R_n = 2\left[(\phi,\Omega)R_3 + l_b(\phi,\Omega)R_4\right]$$ MM-8

<u>Web Compression buckling</u>

J.10.8

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

$$P_{wb} = \frac{24t_w^3\sqrt{EF_{yw}}}{h}$$ $\phi = 0.90$, $\Omega = 1.67$ MM-9

AISC Spec or Manual Section | **Connections Formula Number**

Coped Beam Strength

Coped Section

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

$$R_n = \frac{F_{cr} S_{net}}{e}$$ $\phi = 0.90$, $\Omega = 1.67$ MM-10

Where $S_{net} = I_{tr} / N_{comp}$ for single coped beams and

$\frac{bd^2}{6}$ for double coped beams

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

Top flange coped only

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

$$F_{cr} = 26210 \left(\frac{t_w}{h_o} \right)^2 fK$$ MM-10a

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

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

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

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

$c =$ cope length

$d_c =$ cope depth

$h_o = d - d_c$

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

$$F_{cr} = 0.62 \pi E \left(\frac{t_w^2}{c h_o} \right) f_d \leq F_y$$

$$F_{cr} = 56485 \left(\frac{t_w^2}{c h_o} \right) f_d$$ MM-10b

AISC Spec or Manual Section		Connections Formula Number
	$f_d = 3.5 - 7.5\left(d_c / d \right)$	
All other cope conditions		
	$F_{cr} = F_y Q$	MM-10c
	$Q = 1$ for $\lambda \le 0.7$	
	$Q = 1.34 - 0.486\lambda$ for $0.7 < \lambda \le 1.41$	
	$Q = 1.30 / \lambda^2$ for $\lambda > 1.41$	
	$\lambda = \dfrac{h_o \sqrt{F_y}}{10 t_w \sqrt{475 + 280\left(\dfrac{h_o}{c} \right)^2}}$	
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	$t_{\min} = \dfrac{L_p}{234}\sqrt{\dfrac{F_y}{K}}$	D-1
	$K = \dfrac{6}{\pi^2}\left[(1-\nu) + \dfrac{(\pi bm / a)^2}{6} \right]$	
	Where: $b = L_p / 2$	
	$m = 1$	
	$\nu = 0.3$, Poisson' ratio	
Tee Stem Ductility		
Min. weld size at support	$w_{\min} \ge 0.0155 \dfrac{F_{yc} t_f^2}{b}\left(\dfrac{b^2}{L^2} + 2 \right) \le \dfrac{5}{8} t_s$	D-2
	Where t_f = tee flange thickness	
	b = $(b_f - k_1)/2$ of tee	
	t_s = tee stem thickness	
Max. Tee stem thickness	$t_s = \dfrac{d_b}{2} + \dfrac{1}{16}$	D-3

Single-Plate

HSS Column Checks

K1.2 $$F_y \le 52\ ksi$$

$$F_y / F_u \le 0.8$$

HSS wall cannot be slender $$b/t \le 1.40\sqrt{E/F_y}$$

K1.5 *HSS Punching Shear* $$F_{yp} t_p = F_{u(HSS)} \cdot t_{HSS}$$

$$t_p \le \frac{F_{u(HSS)}}{F_{yp}} t_{HSS}$$ 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 $$w = 5/8\, t_p$$ SP-3

Extended Single-Plate

Maximum plate thickness $$t_{max} = \frac{6M_{max}}{F_y L_p^2}$$ ESP-1

Where $M_{max} = F_v/0.90\, A_b C'$

Exceptions: a. For single bolt column, ignore if

AISC Spec or Manual Section			Connections Formula Number
		either $t_p \; or \; t_w \le \frac{d_b}{2} + \frac{1}{16}$ and both $L_{eh} \ge 2d_b$	
		b. For double bolt column, ignore if both $t_p \; or \; t_w \le \frac{d_b}{2} + \frac{1}{16}$ and both $L_{eh} \ge 2d_b$	
	Shear & flexure interaction (Flexural Yielding)	$R_n = \dfrac{F_y L_p t_p}{\sqrt{2.25 + 16\left(\frac{a}{L_p}\right)^2}}$ $\phi = 0.90,\ \Omega = 1.67$	ESP-2
	Plate buckling	$R_n = \dfrac{F_{cr} S_{net}}{a}$ $\phi = 0.90,\ \Omega = 1.67$	ESP-3
		$F_{cr} = F_y Q$	
		$Q = 1$ for $\lambda \le 0.7$	
		$Q = 1.34 - 0.486\lambda$ for $0.7 < \lambda \le 1.41$	
		$Q = 1.30/\lambda^2$ for $\lambda > 1.41$	
		$\lambda = \dfrac{L_p \sqrt{F_y}}{10 t_p \sqrt{475 + 280\left(\frac{L_p}{a}\right)^2}}$	
	Weld	$w = \frac{5}{8} t_p$	ESP-4
Stabilizer Plate Requirements		If applied load $\le \dfrac{1500 L_p t_p^3}{a^2}$ $\phi = 0.90,\ \Omega = 1.67$ No stabilizer plates required	ESP-5
Connection Torsion Strength		Not applicable if metal deck supports top flange	
		$M_T = R\left(\dfrac{t_w + t_p}{2}\right)$	
		$M_T \le \left[(\phi_v, \Omega_v)0.6F_{yp} - \dfrac{R}{L_p t_p}\right]\dfrac{L_p t_p}{2} + \dfrac{2R(t_w + t_p)b_f}{(\phi_b, \Omega_b)F_{yb} L_s t_w^2}$	ESP-6
		Where L_s = Beam span in inches	

Limit States of Shear Yielding, Shear buckling & 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