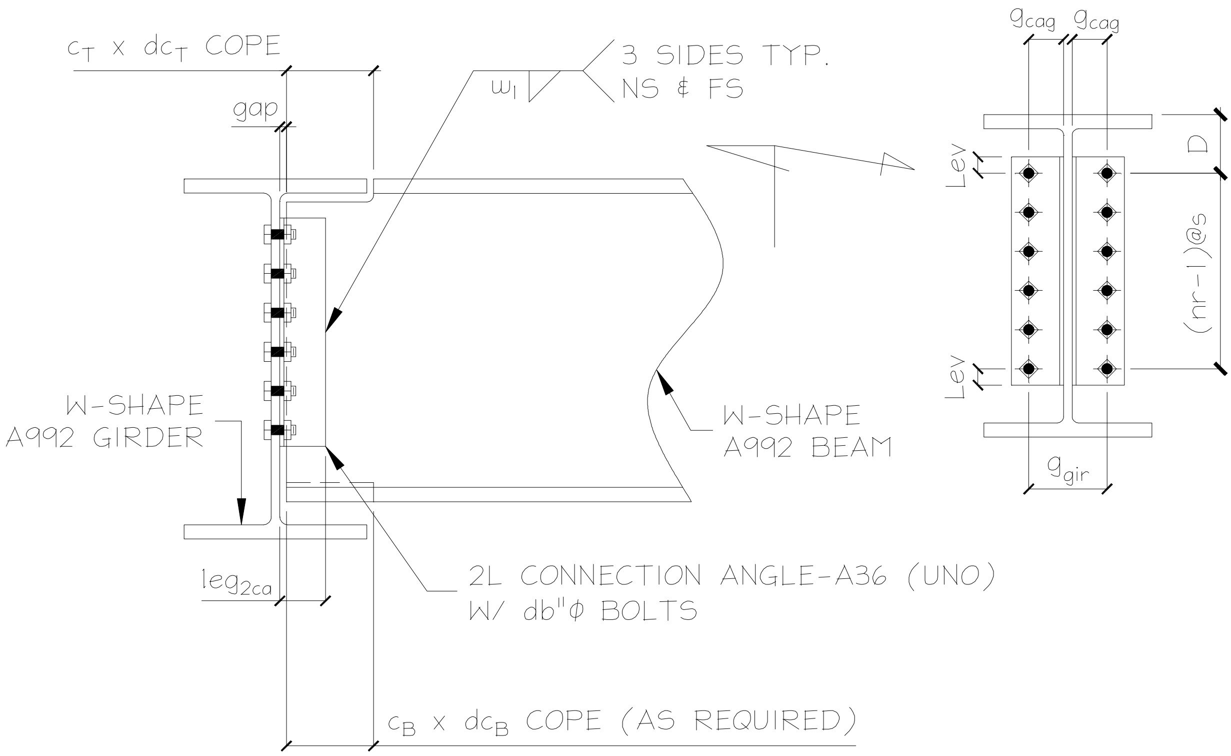
5404 Gurley Avenue
Dallas, TX 75223

SHEAR CONNECTION: DESIGN OF W-SHAPE BEAM TO W-SHAPE GIRDER
CLIP ANGLE CONNECTION (WELDED-BOLTED)



NOTE: (FIGURE ABOVE DOES NOT REPRESENT ACTUAL DESIGN, REFER ON ATTACHED CONNECTION SCHEDULE FOR NUMBER OF BOLTS, COPE DEPTH, AND COPE LENGTH)

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I. DESIGN DATA AND LOAD (ASD - AISC 14th Edition)

GIRDER PROPERTIES (gir): W14X90 - A992

$Fy_{gir} = 50 \cdot ksi$ | $d_{gir} = 14 \cdot in$ | $tw_{gir} = 0.44 \cdot in$ | $k1_{gir} = 1.438 \cdot in$

$Fu_{gir} = 65 \cdot ksi$ | $bf_{gir} = 14.5 \cdot in$ | $tf_{gir} = 0.71 \cdot in$ | $k_{gir} = 2 \cdot in$

$Ag_{gir} = 26.5 \cdot in^2$ | $Sx_{gir} = 143 \cdot in^3$ | $g_{gir} := 5.5in$

BEAM PROPERTIES (bm): W12X50 - A992

$Fy_{bm} = 50 \cdot ksi$ | $d_{bm} = 12.2 \cdot in$ | $tw_{bm} = 0.37 \cdot in$ | $k1_{bm} = 0.938 \cdot in$

$Fu_{bm} = 65 \cdot ksi$ | $bf_{bm} = 8.08 \cdot in$ | $tf_{bm} = 0.64 \cdot in$ | $k_{bm} = 1.5 \cdot in$

$Ag_{bm} = 14.6 \cdot in^2$ | $Sx_{bm} = 64.2 \cdot in^3$ | *Length of beam,* $L_{bm} := 17ft + 0in$

	(TOP)	(BOTTOM)
Depth of Cope:	$dc_T = 1.5 \cdot in$	$dc_B = 0 \cdot in$
Length of Cope:	$c_T = 7.25 \cdot in$	$c_B = 0 \cdot in$

BEAM CONNECTION ANGLE PROPERTIES (ca): 2L4X4X3/8 - A36

$Fy_{ca} = 36 \cdot ksi$ | $leg1_{ca} = 4 \cdot in$ | $t_{ca} = 0.375 \cdot in$

$Fu_{ca} = 58 \cdot ksi$ | $leg2_{ca} = 4 \cdot in$ | *Girder Side Bolt Gage:* $g_{cag} = 2.565 \cdot in$

Number of Connection Angle: $n_{ca} := 3$

BOLTS:

For Connection Angle to Girder Connection:

Bolt Diameter, $db = 0.75 \cdot in$ | Bolt_Type = "A325-N"

Bolt Shear Strength, $\Lambda rv = 11.928 \cdot kips$ | Conn_type = "Bearing-type"

Bolt Tensile Strength, $\Lambda rn = 19.88 \cdot kips$

Clip Angle Vertical Edge Distance, $Lev = 1.25 \cdot in$

Clip Angle Horizontal Edge Distance, $Leh = 1.435 \cdot in$

Hole diameter:

Clip Angle (Girder side), $hd_{cav} = 0.875 \cdot in$ | $hd_{cah} = 1.063 \cdot in$

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Bolt Vertical Spacing, $s = 3 \cdot in$

Bolt Horizontal Spacing (For Multiple bolt lines), $sv = 3 \cdot in$

Bolt First Down from Top of beam, $D = 3 \cdot in$

Gap between edge of beam to edge of support, $gap := \frac{1}{2} in$

Girder,

$hd_{gir} = 0.875 \cdot in$

number of bolt rows: $nr := 3$

number of vertical bolt lines: $nv := 1$

total number of bolts: $n := nr \cdot nv$ $n = 3$

Opposite beam number of bolt rows: $nr_2 := 3$

WELDS: E70xx LH

$Fu_w = 70 \cdot ksi$

Preferred Weld Size

Clip Angle to Beam Web, $w_1 := \frac{1}{4} in$

SAFETY AND RESISTANCE FACTORS:

Safety Factor, Ω (ASD)

Resistance Factor, ϕ (LRFD)

Modification Factor, $\Lambda = \frac{1}{\Omega}$ (IF ASD) $\Lambda = \phi$ (IF LRFD)

	safety factor	*resistance factor*	*modification factor*
For bearing,	$\Omega_{brg} = 2.00$	$\phi_{brg} = 0.75$	$\Lambda_{brg} = 0.50$
For block shear,	$\Omega_{bs} = 2.00$	$\phi_{bs} = 0.75$	$\Lambda_{bs} = 0.50$
For fillet weld (shear),	$\Omega_{vw} = 2.00$	$\phi_{vw} = 0.75$	$\Lambda_{vw} = 0.50$
For flexural local buckling,	$\Omega_b = 1.67$	$\phi_b = 0.90$	$\Lambda_b = 0.60$
For flexural rupture,	$\Omega_{fr} = 2.00$	$\phi_{fr} = 0.75$	$\Lambda_{fr} = 0.50$
For member shear (C, WT, L)	$\Omega_v = 1.67$	$\phi_v = 0.90$	$\Lambda_v = 0.60$

For shear on bolts (Bearing Type),	$\Omega_{vtN} = 2.00$	$\phi_{vtN} = 0.75$	$\Lambda_{vtN} = 0.50$
For shear rupture,	$\Omega_{vr} = 2.00$	$\phi_{vr} = 0.75$	$\Lambda_{vr} = 0.50$
For shear yielding,	$\Omega_{vy} = 1.50$	$\phi_{vy} = 1.00$	$\Lambda_{vy} = 0.67$

APPLIED LOAD:

% UDL, $UDL := 0.5$

Given Load if any, $V_{giv} := 0kips$

Beam Shear Load, $V = 42.3 \cdot kips$ **50% UDL**

Opposite Beam Shear Load (if any), $V_2 := 0kips$

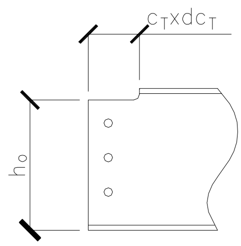
II. CALCULATIONS:

A. BEAM CHECK

1. Coped Beam Capacity

a. Capacity if beam web is single coped at top

(AISC 14th Ed. Manual Part 9, pages 9-6 to 9-7)



Depth of Top Cope,

$dc := dc_T$

$dc = 1.5 \cdot in$

RESULT = depth of cope < half the depth of beam, OK

Length of Cope,

$c := c_T$

$c = 7.25 \cdot in$

RESULT = length of cope < twice the depth of beam, OK

Reduced Beam Depth,

$ho := d_{bm} - dc$

$ho = 10.7 \cdot in$

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Plate Buckling Coefficient,

$$k := \begin{cases} 2.2 \cdot \left(\frac{ho}{c}\right)^{1.65} & \text{if } \frac{c}{ho} \le 1.0 \\ 2.2 \cdot \frac{ho}{c} & \text{otherwise} \end{cases}$$

$k = 4.182$

Plate Buckling Model Adjustment Factor,

$$f := \begin{cases} 2 \cdot \frac{c}{d_{bm}} & \text{if } \frac{c}{d_{bm}} \le 1.0 \\ 1 + \frac{c}{d_{bm}} & \text{otherwise} \end{cases}$$

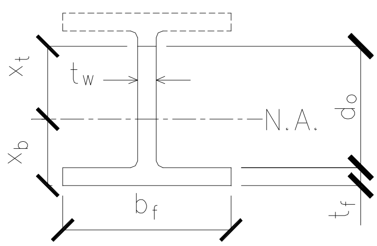
$f = 1.189$

Allowable Flexural Local Buckling Stress/Yielding Stress,

$$Fcr := \min\left[26210 \cdot f \cdot k \cdot \left(\frac{tw_{bm}}{ho}\right)^2 \cdot ksi, Fy_{bm}\right]$$

$Fcr = 50 \cdot ksi$

Location of Neutral Axis on the Reduced Section



$do := ho - tf_{bm}$

$do = 10.06 \cdot in$

$$x_b := \frac{do \cdot tw_{bm} \cdot \left(\frac{do}{2} + tf_{bm}\right) + bf_{bm} \cdot tf_{bm} \cdot \left(\frac{tf_{bm}}{2}\right)}{do \cdot tw_{bm} + bf_{bm} \cdot tf_{bm}}$$

$x_b = 2.559 \cdot in$

$x_t := ho - x_b$

$x_t = 8.141 \cdot in$

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Moment of Inertia,

$$I := \frac{tw_{bm} \cdot do^3}{12} + do \cdot tw_{bm} \cdot \left(\frac{do}{2} - x_t\right)^2 + \frac{bf_{bm} \cdot tf_{bm}^3}{12} + bf_{bm} \cdot tf_{bm} \cdot \left(x_b - \frac{tf_{bm}}{2}\right)^2$$

$$I = 93.517 \cdot in^4$$

Net Section Modulus,

$$Snet := \frac{I}{\max\left(x_t, x_b\right)}$$

$$Snet = 11.487 \cdot in^3$$

Eccentricity,

$$e_{bm} := c + gap$$

$$e_{bm} = 7.75 \cdot in$$

Flexural Local Buckling Capacity/Yielding Capacity,

$$Rbc := \Lambda_b \frac{Fcr \cdot Snet}{e_{bm}}$$

$$Rbc = 44.378 \cdot kips$$

Flexural Rupture Capacity,

$$Rfr := \Lambda_{fr} \frac{Fu_{bm} \cdot Snet}{e_{bm}}$$

$$Rfr = 48.173 \cdot kips$$

Shear Capacity of Reduced Section,

$$Vwg_1 := \Lambda_{vy} \cdot 0.6 \cdot Fy_{bm} \cdot ho \cdot tw_{bm}$$

$$Vwg_1 = 79.18 \cdot kips$$

Coped Beam Capacity,

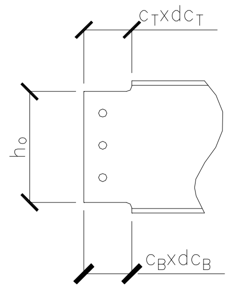
$$Rscb_1 := \min\left(Rbc, Rfr, Vwg_1\right)$$

$Rscb_1 = 44.378 \cdot kips$ $\qquad$ $V = 42.3 \cdot kips$

RESULT = Coped Beam Capacity > Applied Force, OK

b. Capacity if beam web is double coped with same cope length at both flanges

(AISC 14th Ed. Manual Part 9, page 9-8)



Depth of Cope,

Top Cope: $dc_T = 1.5 \cdot in$

Bottom Cope: $dc_B = 0 \cdot in$

Maximum Cope: $dc := \max\left(dc_T, dc_B\right)$

$dc = 1.5 \cdot in$

RESULT = This check is not applicable.

Length of Cope,

Top Cope: $c_T = 7.25 \cdot in$

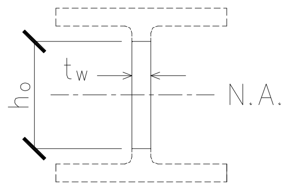
Bottom Cope: $c_B = 0 \cdot in$

Maximum Cope: $c := \max\left(c_T, c_B\right)$

$c = 7.25 \cdot in$

RESULT = This check is not applicable.

Reduced Beam Depth,



$$ho := d_{bm} - dc_T - dc_B$$

$$ho = 10.7 \cdot in$$

Adjustment Factor of Lateral-Torsional Buckling Model,

$$fd := 3.5 - 7.5\left(\frac{dc_T}{d_{bm}}\right)$$

$$fd = 2.578$$

Allowable Flexural Local Buckling Stress/Yielding Stress,

$$Fcr := \min\left(0.62 \cdot \pi \cdot E \cdot fd \cdot \frac{tw_{bm}^{\ 2}}{c \cdot ho}, Fy_{bm}\right)$$

$$Fcr = 50 \cdot ksi$$

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Net Section Modulus,

$$Snet := \frac{tw_{bm} \cdot ho^2}{6}$$

$$Snet = 7.06 \cdot in^3$$

Eccentricity,

$$e_{bm} := c + gap$$

$$e_{bm} = 7.75 \cdot in$$

Flexural Local Buckling Capacity/Yielding Capacity,

$$Rbc := \Lambda_b \frac{Fcr \cdot Snet}{e_{bm}}$$

$$Rbc = 27.275 \cdot kips$$

Flexural Rupture Capacity,

$$Rfr := \Lambda_{fr} \frac{Fu_{bm} \cdot Snet}{e_{bm}}$$

$$Rfr = 29.607 \cdot kips$$

Shear Capacity of Reduced Section,

$$Vwg_2 := \Lambda_{vy} \cdot 0.6 \cdot Fy_{bm} \cdot ho \cdot tw_{bm}$$

$$Vwg_2 = 79.18 \cdot kips$$

Coped Beam Capacity,

$$Rdcb_1 := \min\left(Rbc, Rfr, Vwg_2\right)$$

$Rdcb_1 = 27.275 \cdot kips$ $\quad\quad V = 42.3 \cdot kips$

RESULT = This limit state is not applicable.

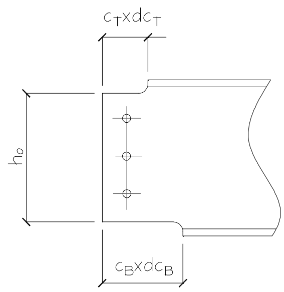
c. Capacity if beam web is double coped with tension flange cope longer than compression flange cope (c.B > c.T)

(AISC 14th Ed. Manual Part 9, page 9-9)

Allowable Flexural Yielding Stress,

$$Fcr := Fy_{bm}$$

$$Fcr = 50 \cdot ksi$$

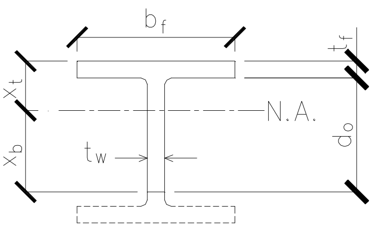


Location of Neutral Axis on the Reduced Section

$$do := d_{bm} - dc_B - tf_{bm}$$

$$do = 11.56 \cdot in$$

$$x_t := \frac{do \cdot tw_{bm} \cdot \left(\frac{do}{2} + tf_{bm}\right) + bf_{bm} \cdot tf_{bm} \cdot \left(\frac{tf_{bm}}{2}\right)}{do \cdot tw_{bm} + bf_{bm} \cdot tf_{bm}}$$



$$x_t = 3.081 \cdot in$$

$$x_b := d_{bm} - dc_B - x_t$$

$$x_b = 9.119 \cdot in$$

Moment of Inertia,

$$I := \frac{tw_{bm} \cdot do^3}{12} + do \cdot tw_{bm} \cdot \left(x_b - \frac{do}{2}\right)^2 + \frac{bf_{bm} \cdot tf_{bm}^{\ 3}}{12} + bf_{bm} \cdot tf_{bm} \cdot \left(x_t - \frac{tf_{bm}}{2}\right)^2$$

$$I = 134.915 \cdot in^4$$

Net Section Modulus at the end of the tension flange cope,

$$Snet := \frac{I}{x_b}$$

$$Snet = 14.796 \cdot in^3$$

Eccentricity,

$$e_{bm} := c_B + gap$$

$$e_{bm} = 0.5 \cdot in$$

Flexural Yielding Capacity,

$$Rbc := \Lambda_b \frac{Fcr \cdot Snet}{e_{bm}}$$

$$Rbc = 885.963 \cdot kips$$

Flexural Rupture Capacity,

$$Rfr := \Lambda_{fr} \frac{Fu_{bm} \cdot Snet}{e_{bm}}$$

$$Rfr = 961.713 \cdot kips$$

Shear Capacity of Reduced Section,

$$Vwg_3 := \Lambda_{vy} \cdot 0.6 \cdot Fy_{bm} \cdot ho \cdot tw_{bm}$$

$$Vwg_3 = 79.18 \cdot kips$$

Coped Beam Capacity,

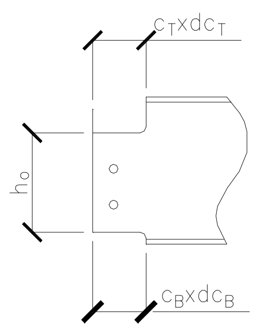
$$Rdcb_2 := \min\left(Rbc, Rfr, Vwg_3\right)$$

$Rdcb_2 = 79.18 \cdot kips$ $\qquad$ $V = 42.3 \cdot kips$

RESULT = This limit state is not applicable.

d. <u>Capacity if beam web is deep coped</u> (for dc > 0.2dbm)

(AISC 14th Ed. Manual Part 9, pages 9-8 to 9-9)



Coefficient,

$$\lambda := \frac{ho \cdot \sqrt{Fy_{bm}}}{10 \cdot tw_{bm} \cdot \sqrt{475 + 280 \cdot \left(\frac{ho}{c}\right)^2}} \cdot \sqrt{\frac{1}{ksi}}$$

$$\lambda = 0.621$$

$$Q := \begin{cases} 1 & \text{if } \lambda \le 0.7 \\ 1.34 - 0.486 \cdot \lambda & \text{if } 0.7 < \lambda \le 1.41 \\ \dfrac{1.30}{\lambda^2} & \text{otherwise} \end{cases}$$

$$Q = 1$$

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Allowable Buckling Stress,

$$Fcr := Q \cdot Fy_{bm}$$

$$Fcr = 50 \cdot ksi$$

Net Section Modulus,

$$Snet := \frac{tw_{bm} \cdot ho^2}{6}$$

$$Snet = 7.06 \cdot in^3$$

Eccentricity,

$$e_{bm} := c + gap$$

$$e_{bm} = 7.75 \cdot in$$

Buckling Capacity,

$$Rbc := \Lambda_b \frac{Fcr \cdot Snet}{e_{bm}}$$

$$Rbc = 27.275 \cdot kips$$

Flexural Rupture Capacity,

$$Rfr := \Lambda_{fr} \frac{Fu_{bm} \cdot Snet}{e_{bm}}$$

$$Rfr = 29.607 \cdot kips$$

Shear Capacity of Reduced Section,

$$Vwg_4 := \Lambda_{vy} \cdot 0.6 \cdot Fy_{bm} \cdot ho \cdot tw_{bm}$$

$$Vwg_4 = 79.18 \cdot kips$$

Coped Beam Capacity,

$$Rtdcb := \min\left(Rbc, Rfr, Vwg_4\right)$$

$Rtdcb = 27.275 \cdot kips$ $V = 42.3 \cdot kips$

RESULT = This limit state is not applicable.

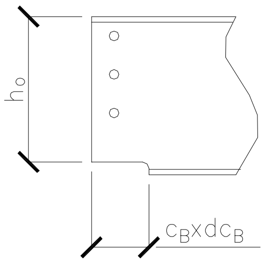
e. *Capacity if beam web is single coped at bottom*

Depth of cope,

$$dc := dc_B$$

$$dc = 0 \cdot in$$

Length of Cope,



$$c := c_B$$

$$c = 0 \cdot in$$

Reduced Beam Depth,

$$ho := d_{bm} - dc_B$$

$$ho = 12.2 \cdot in$$

Allowable Flexural Yielding Stress,

$$Fcr := Fy_{bm}$$

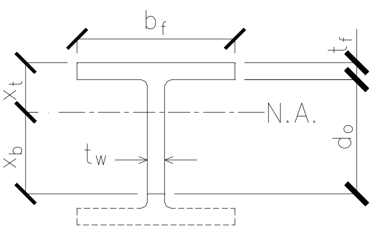
$$Fcr = 50 \cdot ksi$$

Location of Neutral Axis on the Reduced Section

$$do := ho - tf_{bm}$$

$$do = 11.56 \cdot in$$

$$x_t := \frac{do \cdot tw_{bm} \cdot \left(\frac{do}{2} + tf_{bm}\right) + bf_{bm} \cdot tf_{bm} \cdot \left(\frac{tf_{bm}}{2}\right)}{do \cdot tw_{bm} + bf_{bm} \cdot tf_{bm}}$$



$$x_t = 3.081 \cdot in$$

$$x_b := ho - x_t$$

$$x_b = 9.119 \cdot in$$

Moment of Inertia,

$$I := \frac{tw_{bm} \cdot do^3}{12} + do \cdot tw_{bm} \cdot \left(x_b - \frac{do}{2}\right)^2 + \frac{bf_{bm} \cdot tf_{bm}^3}{12} + bf_{bm} \cdot tf_{bm} \cdot \left(x_t - \frac{tf_{bm}}{2}\right)^2$$

$$I = 134.915 \cdot in^4$$

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Net Section Modulus,

$$Snet := \frac{I}{x_b}$$

$$Snet = 14.796 \cdot in^3$$

Eccentricity,

$$e_{bm} := c + gap$$

$$e_{bm} = 0.5 \cdot in$$

Flexural Yielding Capacity,

$$Rbc := \Lambda_b \cdot \frac{Fcr \cdot Snet}{e_{bm}}$$

$$Rbc = 885.963 \cdot kips$$

Flexural Rupture Capacity,

$$Rfr := \Lambda_{fr} \frac{Fu_{bm} \cdot Snet}{e_{bm}}$$

$$Rfr = 961.713 \cdot kips$$

Gross Web Shear,

$$Vwg_5 := \Lambda_{vy} \cdot 0.6 \cdot Fy_{bm} \cdot ho \cdot tw_{bm}$$

$$Vwg_5 = 90.28 \cdot kips$$

Coped Beam Capacity,

$$Rscb_2 := \min\left(Rbc , Rfr , Vwg_5\right)$$

$Rscb_2 = 90.28 \cdot kips$ $\qquad$ $V = 42.3 \cdot kips$

RESULT = This limit state is not applicable.

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f. Capacity if beam is cut-flushed

Reduced Beam Depth,

$ho = 10.7 \cdot in$

Allowable Flexural Local Buckling Stress/Yielding Stress,

$Fcr = 50 \cdot ksi$

Net Section Modulus,

$Snet = 10.481 \cdot in^3$

Eccentricity,

$e_{bm} = 7.75 \cdot in$

Flexural Local Buckling Capacity/Yielding Capacity,

$$Rbc := \Lambda_b \cdot \frac{Fcr \cdot Snet}{e_{bm}}$$

$Rbc = 40.49 \cdot kips$

Flexural Rupture Capacity,

$$Rfr := \Lambda_{fr} \frac{Fu_{bm} \cdot Snet}{e_{bm}}$$

$Rfr = 43.952 \cdot kips$

Gross Web Shear,

$$Vwg_6 := \Lambda_{vy} \cdot 0.6 \cdot Fy_{bm} \cdot ho \cdot tw_{bm}$$

$Vwg_6 = 79.18 \cdot kips$

Coped Beam Capacity,

$$Rcfb := \min\left(Rbc, Rfr, Vwg_6\right)$$

$Rcfb = 40.49 \cdot kips$ $V = 42.3 \cdot kips$

RESULT = This limit state is not applicable.

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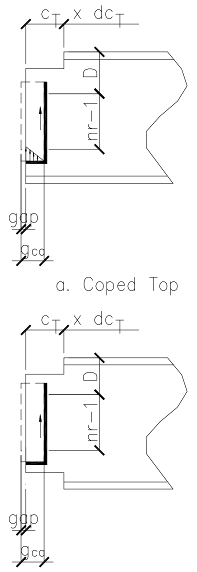
2. Block Shear Capacity of Beam Web

(AISC 14th Ed. Specs Chapter J, Section J4.3, page 16.1-129)

$\left[\text{for}\ dct > 0 \wedge ct \geq \left(leg2_{ca} - gap\right)\right]$

Reduction Factor, $U_{bs} := 1.0$ *(tension stress is uniform)*



a. Coped Top

b. Coped Top & Bottom

Failure Modes on Welded Connection

Gross Shear Area

$$Agv := \left[\left(D - dc_T\right) + (nr - 1)\cdot s + Lev\right]\cdot tw_{bm}$$

$$Agv = 3.237\cdot in^2$$

Net Tension Area

$$Ant := \begin{cases} \left(leg2_{ca} - gap\right)\cdot tw_{bm} & \text{if}\ dc_T > 0 \wedge c_T \geq leg2_{ca} - gap \\ 0 & \text{otherwise} \end{cases}$$

$$Ant = 1.295\cdot in^2$$

Net Shear Area

$$Anv := Agv$$

$$Anv = 3.237\cdot in^2$$

Block Shear Capacity,

$$Rbs_{bm} := \Lambda_{bs} \min\left(0.6\cdot Fu_{bm}\cdot Anv + U_{bs}\cdot Fu_{bm}\cdot Ant\,,\ 0.6\cdot Fy_{bm}\cdot Agv + U_{bs}\cdot Fu_{bm}\cdot Ant\right)$$

$Rbs_{bm} = 90.65\cdot kips$ $V = 42.3\cdot kips$

RESULT = Block Shear Capacity > Force Applied,OK

3. Shear Capacity of Beam

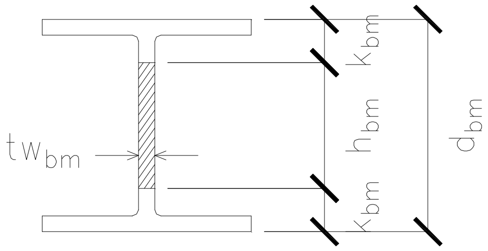
(AISC 14th Ed. Specifications Chapter G, Section G2.1, pages 16.1-67 to 16.1-69)

Clear distance between flanges of beam, less the fillet or corner radii,

$$h := d_{bm} - 2\cdot kdes_{bm}$$

$$h = 9.92\cdot in$$

Limiting depth-thickness ratio,



$$h_{tw} := \frac{h}{tw_{bm}}$$

$$h_{tw} = 26.811$$

Clear distance between transverse stiffeners,

$$a := \begin{vmatrix} 0in & \text{if } h_{tw} < 260 \\ \min\left[3 \cdot h, \left(\frac{260}{h_{tw}}\right)^2 \cdot h\right] & \text{otherwise} \end{vmatrix}$$

$$a = 0 \cdot in$$

Web plate buckling coefficient,

$$kv := \begin{vmatrix} 5 & \text{if } h_{tw} < 260 \\ 5 + \dfrac{5}{\left(\frac{a}{h}\right)^2} & \text{otherwise} \end{vmatrix} \qquad (G2\text{-}6)$$

$$kv = 5$$

Web shear coefficient,

$$Cv := \begin{vmatrix} 1 & \text{if } h_{tw} \le 1.1 \cdot \sqrt{\dfrac{kv \cdot E}{Fy_{bm}}} & (G2\text{-}3) \\ \dfrac{1.1 \cdot \sqrt{\dfrac{kv \cdot E}{Fy_{bm}}}}{h_{tw}} & \text{if } 1.1 \cdot \sqrt{\dfrac{kv \cdot E}{Fy_{bm}}} < h_{tw} \le 1.37 \cdot \sqrt{\dfrac{kv \cdot E}{Fy_{bm}}} & (G2\text{-}4) \\ \dfrac{1.51 \cdot E \cdot kv}{h_{tw}{}^2 Fy_{bm}} & \text{if } 1.37 \cdot \sqrt{\dfrac{kv \cdot E}{Fy_{bm}}} < h_{tw} & (G2\text{-}5) \end{vmatrix}$$

$$Cv = 1$$

Shear Capacity of Section,

$$Rv_{bm} := \Lambda_{vbm} \cdot 0.6 \cdot Fy_{bm} \cdot d_{bm} \cdot tw_{bm} \cdot Cv \qquad (G2\text{-}1)$$

$Rv_{bm} = 90.28 \cdot kips$ $\qquad$ $V = 42.3 \cdot kips$

RESULT = Shear Capacity of Section > Force Applied, OK

B. <u>BEAM TO CLIP ANGLE CONNECTION</u>

1. Eccentric Weld Capacity

(AISC 14th Ed. Specifications Chapter J, pages 16.1-110 to 16.1-117)

(AISC 14th Ed. Manual Part 8, pages 8-9 to 8-15)

No. of Weld side, $n_{ws} := 2$

Minimum weld size,

$w_{min} = \frac{3}{16} \cdot in$ $\qquad$ $w_1 = \frac{1}{4} \cdot in$

RESULT = Preferred Weld Size > Minimum Weld Size,OK

Maximum Weld Size,

$$w_{max} := \begin{vmatrix} t_{ca} - \frac{1}{16} in & \text{if } t_{ca} \ge \frac{1}{4} in \\ t_{ca} & \text{otherwise} \end{vmatrix}$$

$w_{max} = \frac{5}{16} \cdot in$ $\qquad$ $w_1 = \frac{1}{4} \cdot in$

RESULT = Preferred Weld Size < Maximum Weld Size, OK

Shear Strength,

For Beam:

$Rv_b := \Lambda_{vr} \cdot 0.6 \cdot Fu_{bm} \cdot tw_{bm}$

$Rv_b = 7.215 \cdot \frac{kips}{in}$

For Clip Angle:

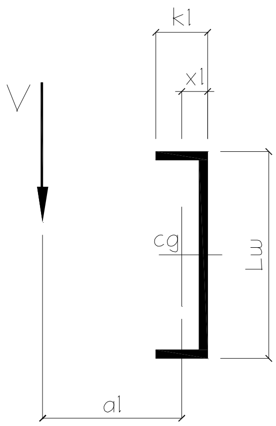
$Rv_{ca} := \Lambda_{vr} \cdot 0.6 \cdot Fu_{ca} \cdot t_{ca} n_{ws}$

$Rv_{ca} = 13.05 \cdot \frac{kips}{in}$

For Weld:

$Rv_w := \Lambda_{vw} \cdot 0.6 \cdot Fu_w \cdot \sin(45deg) \cdot n_{ws}$

$Rv_w = 29.698 \cdot ksi$



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Maximum effective weld size,

$$w_{eff} := \frac{\min(Rv_b, Rv_{ca})}{Rv_w}$$

$$w_{eff} = 0.243 \cdot in$$

Length of weld,

$$Lw := (nr - 1) \cdot s + 2 \cdot Lev$$

$$Lw = 8.5 \cdot in$$

Eccentric load coefficient,

$$kl := leg2_{ca} - gap$$

$$kl = 3.5 \cdot in$$

$$xl := \frac{kl^2}{(2kl + Lw)}$$

$$xl = 0.79 \cdot in$$

$$al := leg2_{ca} - xl$$

$$al = 3.21 \cdot in$$

$$k := \frac{kl}{Lw}$$

$$k = 0.412$$

$$a := \frac{al}{Lw}$$

$$a = 0.378$$

Electrode Strength Coefficient,

$$C_1 = 1.00 \cdot ksi$$

From Table 8-8 in AISC 14th Ed. Chapter 8,

$$C_o = 2.976$$

Eccentric Weld Shear Capacity,

$$Rew_{ca} := \Lambda_{ew} \cdot n_{ws} \cdot C_o \cdot C_1 \cdot \min\left(w_1, w_{eff}\right) \cdot 16 \cdot Lw$$

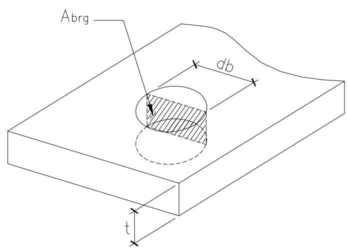
$Rew_{ca} = 98.317 \cdot kips$ $\qquad$ $V = 42.3 \cdot kips$

RESULT = Eccentric Weld Capacity > Force Applied, OK

C. CLIP ANGLE CHECK

1. Bolt Bearing Capacity on Clip Angle

(AISC 14th Ed. Specifications Chapter J, Section J3.10, pages 16.1-127 to 16.1-128)



Bearing Area,

$$Abrg_{ca} := db \cdot t_{ca}$$

$$Abrg_{ca} = 0.281 \cdot in^2$$

Support Side:

Allowable Bearing Strength using edge distance, (J3-6a, J3-6c)

$$Fbe := \Lambda_{brg} \cdot Fu_{ca} \cdot \left| \begin{array}{ll} \min\left[1.0 \cdot \left(Lev - 0.5 \cdot hd_{cav}\right) \cdot t_{ca}, 2.0 \cdot Abrg_{ca}\right] & \text{if } hd_{cah} \geq hd_{ls} \\ \min\left[1.2 \cdot \left(Lev - 0.5 \cdot hd_{cav}\right) \cdot t_{ca}, 2.4 \cdot Abrg_{ca}\right] & \text{otherwise} \end{array} \right.$$

$Fbe = 10.603 \cdot kips$

Allowable Bearing Strength using bolt spacing, (J3-6a, J3-6c)

$$Fbs := \Lambda_{brg} \cdot Fu_{ca} \cdot \left| \begin{array}{ll} \min\left[1.0 \cdot \left(s - hd_{cav}\right) \cdot t_{ca}, 2.0 \cdot Abrg_{ca}\right] & \text{if } hd_{cah} \geq hd_{ls} \\ \min\left[1.2 \cdot \left(s - hd_{cav}\right) \cdot t_{ca}, 2.4 \cdot Abrg_{ca}\right] & \text{otherwise} \end{array} \right.$$

$Fbs = 19.575 \cdot kips$

Bolt Bearing Capacity,

$$Rbrg_{ca} := n_{ca} \cdot nv \cdot \left[\min(Fbe, \Lambda rv) + \min(Fbs, \Lambda rv) \cdot (nr - 1)\right]$$

$Rbrg_{ca} = 103.379 \cdot kips$ $\qquad$ $V = 42.3 \cdot kips$

RESULT = Bearing Capacity > Force Applied, OK

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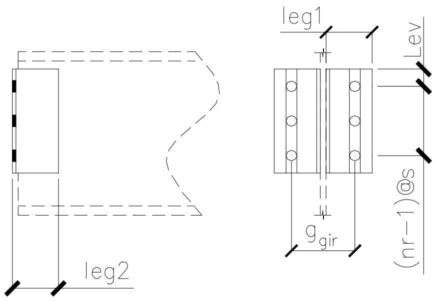
2. Shear Yielding Capacity of Clip Angle



(AISC 14th Ed, Specs. Chapter J, Section J4.2, page 16.1-129)

Length of Angle,

$$Lca := (nr - 1) \cdot s + 2 \cdot Lev$$

$$Lca = 8.5 \cdot in$$

Check if Length of Angle is acceptable per AISC requirements,

(AISC 14th Ed. Manual Part 10, page 10-9)

$$Length := \begin{cases} \text{"Angle Length is OK per AISC Requirements"} & \text{if } Lca > 0.5\left(d_{bm} - 2k_{bm}\right) \\ \text{"Increase Angle Length per AISC Requirements"} & \text{otherwise} \end{cases}$$

Length = "Angle Length is OK per AISC Requirements"

Gross Shear Capacity, *(J4-3)*

$$Rvy_{ca} := \Lambda_{vy} \cdot 0.6 \cdot Fy_{ca} \cdot t_{ca} \cdot Lca \cdot n_{ca}$$

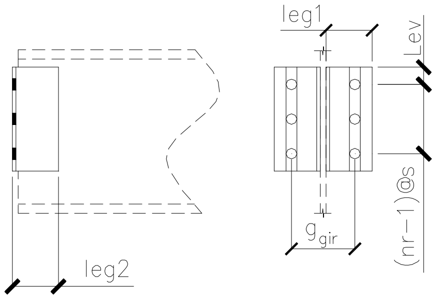
$Rvy_{ca} = 137.7 \cdot kips$ $\qquad V = 42.3 \cdot kips$

RESULT = Shear Yielding Capacity > Force Applied, OK

3. Shear Rupture Capacity of Clip Angle

(AISC 14th Ed, Specifications Chapter J, Section J4.2, page 16.1-129)

<u>*Support Side:*</u>



Net Shear Area

$$Anv := \left(Lca - nr \cdot hd_{cav}\right) \cdot t_{ca}$$

$$Anv = 2.203 \cdot in^2$$

Shear Rupture Capacity *(J4-4)*

$$Rvr_{ca} := \Lambda_{vr} \cdot n_{ca} \cdot 0.6 \cdot Fu_{ca} \cdot Anv$$

$Rvr_{ca} = 115.003 \cdot kips$ $\qquad V = 42.3 \cdot kips$

RESULT = Shear Rupture Capacity > Force Applied, OK

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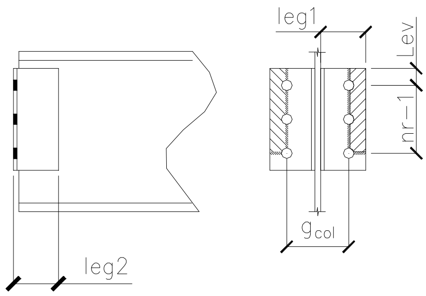
4. Block Shear Capacity of Clip Angle

(AISC 14th Ed. Specs. Chapter J, Section J4.3, page 16.1-129)

Reduction Factor, $U_{bs} := \begin{cases} 1.0 & \text{if } nv = 1 \\ 0.5 & \text{if } nv > 1 \end{cases}$ (tension stress is uniform) (tension stress is non-uniform)

$U_{bs} = 1$

Girder/Support Side:



Gross Shear Area

$Agv := n_{ca} \cdot [(nr - 1) \cdot s + Lev] \cdot t_{ca}$

$Agv = 8.156 \cdot in^2$

Net Tension Area

$Ant := n_{ca} \cdot \left[leg1_{ca} - g_{cag} - (nv - 0.5) \cdot hd_{cah}\right] \cdot t_{ca}$

$Ant = 1.017 \cdot in^2$

Net Shear Area

$Anv := Agv - (nr - 0.5) \cdot hd_{cav} \cdot t_{ca} \cdot n_{ca}$

$Anv = 5.695 \cdot in^2$

Block Shear Capacity on girder side,

$Rbs_{ca} := \Lambda_{bs} \min\left(0.6Fu_{ca} \cdot Anv + U_{bs} \cdot Fu_{ca} \cdot Ant, 0.6 \cdot Fy_{ca} \cdot Agv + U_{bs} \cdot Fu_{ca} \cdot Ant\right)$

$Rbs_{ca} = 117.572 \cdot kips$ $V = 42.3 \cdot kips$

RESULT = Block Shear Capacity > Force Applied,OK

D. CLIP ANGLE TO GIRDER CONNECTION

1. Bolt Shear Capacity

(AISC 14th Ed. Specifications Chapter J, Section J3.6, page 16.1-125)

Shear Capacity per Bolt

$\Lambda rv = 11.928 \cdot kips$

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Bolt Group Shear Capacity,

$Rb_v := n_{ca}\, n \cdot \Lambda rv$

$Rb_v = 107.354 \cdot kips$ $\quad V = 42.3 \cdot kips$

RESULT = Bolt Shear Capacity > Force Applied, OK

2. Check for Spacing

(AISC 14th Ed. Specifications Chapter J, Section J3.3 and J3.5, pages 16.1-122 to 16.1-124)

Vertical Spacing,

$s = 3 \cdot in$

$s_{min} := 2\frac{2}{3} \cdot db$

$s_{min} = 2 \cdot in$

$s_{max} := \min\left(12in, 24 \cdot \min\left(t_{ca}, tw_{gir}\right)\right)$

$s_{max} = 9.000 \cdot in$

RESULT = s > s.min & s < s.max, OK

Horizontal Spacing,

$sv = 3 \cdot in$

$sv_{min} := 2\frac{2}{3} \cdot db$

$sv_{min} = 2 \cdot in$

$sv_{max} := \min\left(12in, 24 \cdot \min\left(t_{ca}, tw_{gir}\right)\right)$

$sv_{max} = 9.000 \cdot in$

RESULT = This check is not applicable

3. Check for Edge Distance

(AISC 14th Ed. Specifications Chapter J, Section J3.4 and J3.5, pages 16.1-122 to 16.1-124)

Vertical Edge Distance,

$Lev = 1.25 \cdot in$

$Le_{min} = 1 \cdot in$

$C_2 = 0 \cdot in$

$Lev_{min} := Le_{min} + C_2$

$Lev_{min} = 1 \cdot in$

$Lev_{max} := \min(6in, 12 \cdot t_{ca})$

$Lev_{max} = 4.500 \cdot in$

RESULT = Lev > Lev_min & Lev < Le_max, OK.

Horizontal Edge Distance,

$Leh = 1.435 \cdot in$

$Le_{min} = 1 \cdot in$

$C_2 = 0.125 \cdot in$

$Leh_{min} := Le_{min} + C_2$

$Leh_{min} = 1.125 \cdot in$

$Leh_{max} := \min(6in, 12 \cdot t_{ca})$

$Leh_{max} = 4.500 \cdot in$

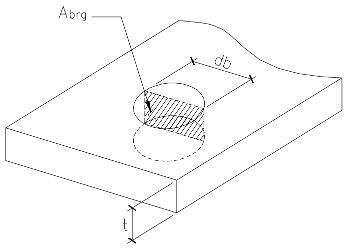
RESULT = Leh > Leh_min & Leh < Le_max, OK.

E. GIRDER CHECK

1. Bolt Bearing Capacity on Girder

(AISC 14th Ed. Specifications Chapter J, Section J3.10, pages 16.1-127 to 16.1-128)

Total force acting per bolt,



$$V_{bm1} := \frac{V}{nr \cdot n_{ca}}$$

$V_{bm1} = 4.7 \cdot kips$

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$$V_{bm2} := \begin{cases} \dfrac{V_2}{nr_2 \cdot n_{ca}} & \text{if } nr_2 \neq 0 \\ 0 & \text{otherwise} \end{cases}$$

$$V_{bm2} = 0 \cdot kips$$

$$V_{bm12} := V_{bm1} + V_{bm2}$$

$$V_{bm12} = 4.7 \cdot kips$$

Effective Thickness of Web,

$$tw_{eff} := tw_{gir} \cdot \frac{V_{bm1}}{V_{bm12}}$$

$$tw_{eff} = 0.44 \cdot in$$

Bearing Area,

$$Abrg_{gir} := db \cdot tw_{eff}$$

$$Abrg_{gir} = 0.33 \cdot in^2$$

Allowable Bearing Strength per bolt using edge distance,(J3-6a,J3-6c)

$$Fbe := \Lambda_{brg} \cdot Fu_{gir} \cdot \begin{cases} 2.0 \cdot Abrg_{gir} & \text{if } hd_{gir} \geq hd_{ls} \\ 2.4 \cdot Abrg_{gir} & \text{otherwise} \end{cases}$$

$$Fbe = 25.74 \cdot kips$$

Allowable Bearing Strength per bolt using bolt spacing,(J3-6a,J3-6c)

$$Fbs := \Lambda_{brg} \cdot Fu_{gir} \cdot \begin{cases} \min\left[1.0 \cdot \left(s - hd_{gir}\right) \cdot tw_{eff}, 2.0 \cdot Abrg_{gir}\right] & \text{if } hd_{gir} \geq hd_{ls} \\ \min\left[1.2 \cdot \left(s - hd_{gir}\right) \cdot tw_{eff}, 2.4 \cdot Abrg_{gir}\right] & \text{otherwise} \end{cases}$$

$$Fbs = 2.145\, ft \cdot \frac{kips}{in}$$

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Bolt Bearing Capacity,

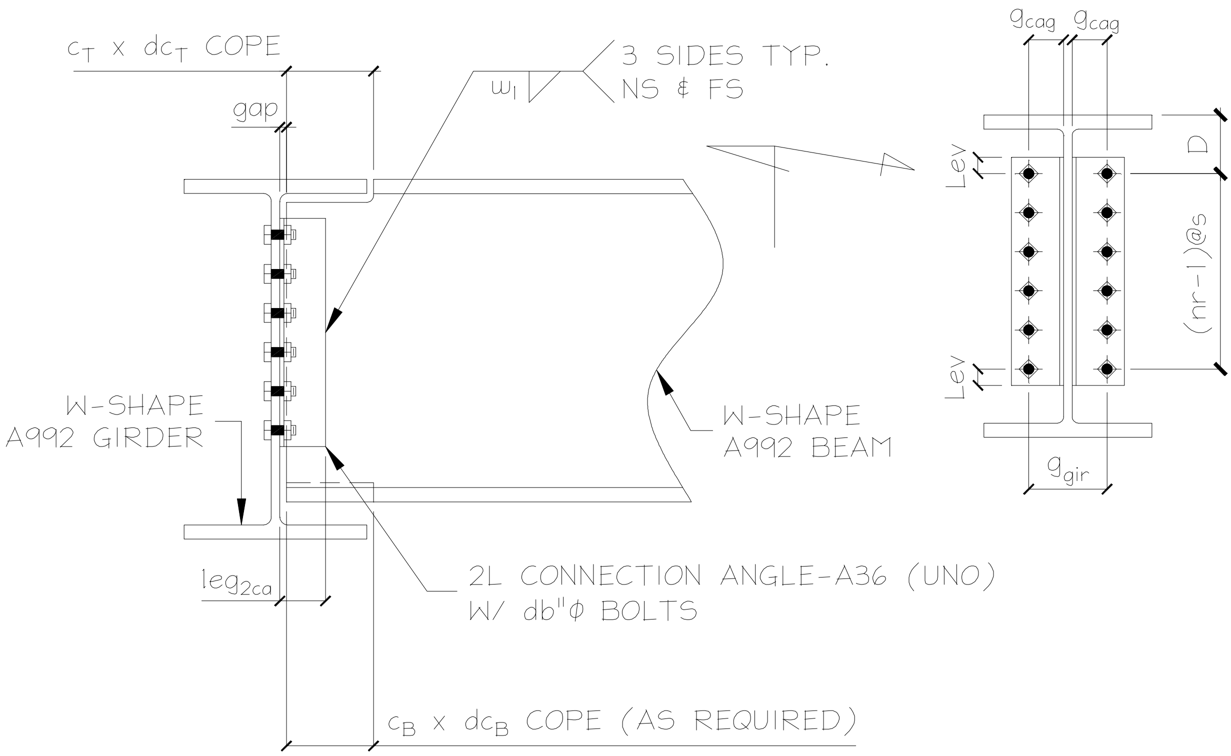
$$Rbrg_{gir} := n_{ca}\, nv \cdot [\min(Fbe, \Lambda rv) + \min(Fbs, \Lambda rv) \cdot (nr - 1)]$$

$Rbrg_{gir} = 107.354 \cdot kips$ $\qquad V = 42.3 \cdot kips$

RESULT = Bearing Capacity > Force Applied,OK

III. DETAILS

A. SKETCH



NOTE: (FIGURE ABOVE DOES NOT REPRESENT ACTUAL DESIGN, REFER ON ATTACHED CONNECTION SCHEDULE FOR NUMBER OF BOLTS, COPE DEPTH, AND COPE LENGTH)

SHEAR CONNECTION: DETAIL OF W-SHAPE BEAM TO W-SHAPE GIRDER
CLIP ANGLE CONNECTION (WELDED-BOLTED)

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B. TABLE: SHEAR CONNECTION SCHEDULE

Girder		Beam		D (in)	Cope Dimensions				g_{gir} (in)	gap (in)	Beam Shear Load (kips)	Rcap (kips)	Governing Capacity
Size	Grade	Size	Grade		dc_T (in)	c_T (in)	dc_B (in)	c_B (in)					
W14X90	A992	W12X50	A992	3	1 1/2	7 1/4	-	-	5 1/2	1/2	42.3	44.4	Flexural Cope Buckling

Connection Angle				Bolt Type	Bolts		s (in)	Weld Size w_1 (in)	Lev (in)
Size	Grade	g_{cag} (in)	$leg2_{ca}$ (in)		db (in)	nr			
2L4X4X3/8	A36	2 9/16	4	A325-N	3/4	3	3	1/4	1 1/4

IV. REFERENCES

Steel Construction Manual (14th)- ASD American Institute of Steel Construction, Inc. 2010