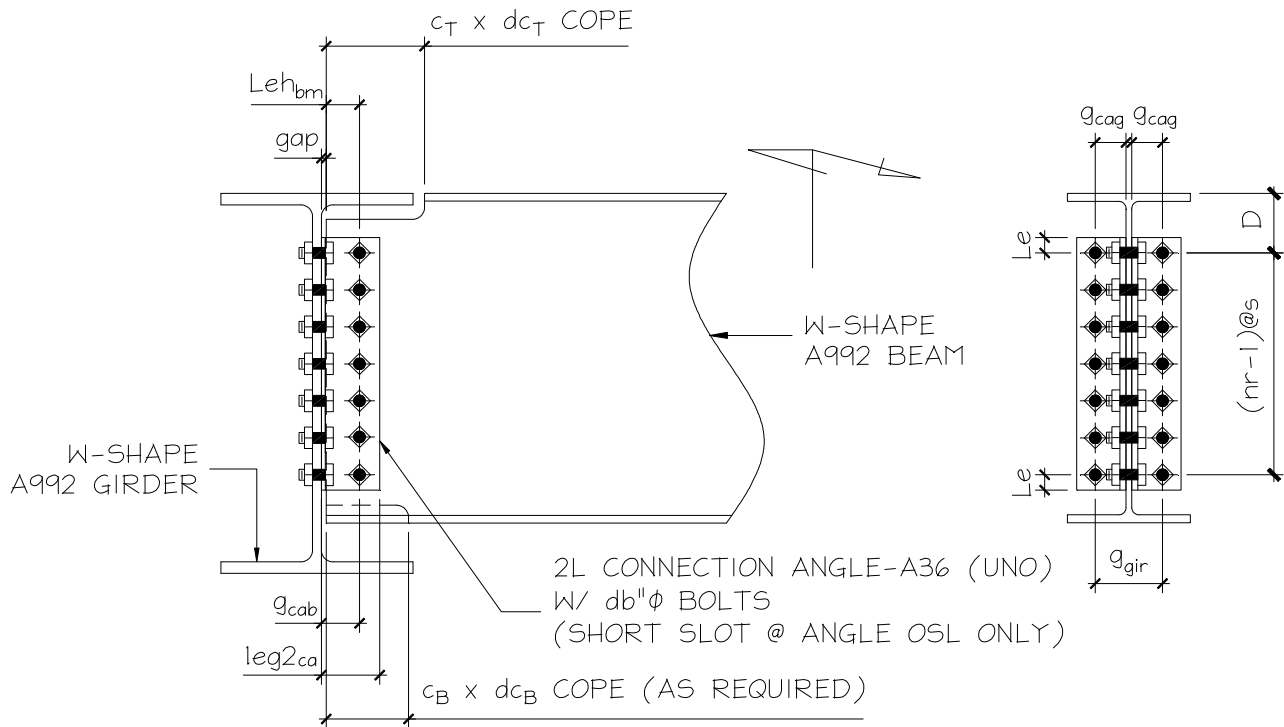




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**SHEAR WITH AXIAL CONNECTION: DESIGN OF W-SHAPE BEAM TO
W-SHAPE GIRDER CLIP ANGLE CONNECTION (BOLTED-BOLTED)**



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): W12X87 - A992

$F_y_{gir} = 50 \cdot \text{ksi}$	$d_{gir} = 12.5 \cdot \text{in}$	$tw_{gir} = 0.515 \cdot \text{in}$	$k1_{gir} = 1.063 \cdot \text{in}$
$F_u_{gir} = 65 \cdot \text{ksi}$	$bf_{gir} = 12.1 \cdot \text{in}$	$tf_{gir} = 0.81 \cdot \text{in}$	$k_{gir} = 1.688 \cdot \text{in}$
$Ag_{gir} = 25.6 \cdot \text{in}^2$	$Sx_{gir} = 118 \cdot \text{in}^3$	$g_{gir} := 5.5 \text{in}$	

BEAM PROPERTIES (bm): W12X45 - A992

$F_y_{bm} = 50 \cdot \text{ksi}$	$d_{bm} = 12.1 \cdot \text{in}$	$tw_{bm} = 0.335 \cdot \text{in}$	$k1_{bm} = 0.938 \cdot \text{in}$
$F_u_{bm} = 65 \cdot \text{ksi}$	$bf_{bm} = 8.05 \cdot \text{in}$	$tf_{bm} = 0.575 \cdot \text{in}$	$k_{bm} = 1.375 \cdot \text{in}$
$Ag_{bm} = 13.1 \cdot \text{in}^2$	$Sx_{bm} = 57.7 \cdot \text{in}^3$	$Lu_{bm} := 20 \text{ft} + 0 \text{in}$	Case = "One-Way"

	(TOP)	(BOTTOM)
Cope Dimensions,	$dc_T = 1.5 \cdot \text{in}$	$dc_B = 1.5 \cdot \text{in}$
	$c_T = 6 \cdot \text{in}$	$c_B = 6 \cdot \text{in}$

CONNECTION ANGLE PROPERTIES (ca):

2L4X4X1/2 - A36

$F_y_{ca} = 36 \cdot \text{ksi}$	$leg1_{ca} = 4 \cdot \text{in}$	$tca = 0.5 \cdot \text{in}$	
$F_u_{ca} = 58 \cdot \text{ksi}$	$leg2_{ca} = 4 \cdot \text{in}$	Beam side bolt gage:	$g_{cab} := 2.5 \text{in}$
Number of Connection Angle:	$n_{ca} := 2$	Girder Side Bolt Gage:	$g_{cag} = 2.583 \cdot \text{in}$

BOLTS:

<i>Bolt Diameter,</i>	$db = 0.75 \cdot \text{in}$	$\text{Bolt_Type} = \text{"A325-N"}$
<i>Bolt Shear Strength,</i>	$\Lambda_{rv} = 11.928 \cdot \text{kips}$	$\text{Conn_type} = \text{"Bearing-type"}$
<i>Bolt Tensile Strength,</i>	$\Lambda_{rn} = 19.88 \cdot \text{kips}$	<u>Hole diameter:</u>
<i>Gap between edge of beam to edge of support,</i>	$\text{gap} := \frac{1}{2} \text{in}$	<i>Clip Angle (Beam side),</i>
<i>Beam Edge Distance,</i>	$\text{Leh}_{bm} = 2 \cdot \text{in}$	$\text{hd}_{bcav} = 0.875 \cdot \text{in}$ $\text{hd}_{bcah} = 0.875 \cdot \text{in}$
<i>Clip Angle Vertical Edge Distance,</i>	$\text{Lev} = 1.25 \cdot \text{in}$	<i>Clip Angle (Girder side),</i>
		$\text{hd}_{gcav} = 0.875 \cdot \text{in}$ $\text{hd}_{gcah} = 0.875 \cdot \text{in}$



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Clip Angle Horizontal Edge Distance,	$Leh_{cag} = 1.417 \cdot \text{in}$	Beam, $hd_{bm} = 0.875 \cdot \text{in}$
	$Leh_{cab} = 1.5 \cdot \text{in}$	Girder, $hd_{gir} = 0.875 \cdot \text{in}$
Bolt Vertical Spacing,	$s = 3 \cdot \text{in}$	
Bolt Horizontal Spacing (For Multiple bolt lines),	$sv = 3 \cdot \text{in}$	
Bolt First Down from Top of beam,	$D = 3 \cdot \text{in}$	

number of bolt rows: $nr := 3$

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

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

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 tension rupture,	$\Omega_{tr} = 2.00$	$\phi_{tr} = 0.75$	$\Lambda_{tr} = 0.50$
For tension yielding,	$\Omega_{ty} = 1.67$	$\phi_{ty} = 0.90$	$\Lambda_{ty} = 0.60$
For compression,	$\Omega_c = 1.67$	$\phi_c = 0.90$	$\Lambda_c = 0.60$
For member shear (C, WT, L)	$\Omega_v = 1.67$	$\phi_v = 0.90$	$\Lambda_v = 0.60$
For fillet weld (shear),	$\Omega_{vw} = 2.00$	$\phi_{vw} = 0.75$	$\Lambda_{vw} = 0.50$
For shear rupture,	$\Omega_{vr} = 2.00$	$\phi_{vr} = 0.75$	$\Lambda_{vr} = 0.50$
For shear yielding,	$\Omega_{vy} = 1.50$	$\phi_{vy} = 1.00$	$\Lambda_{vy} = 0.67$
For member/bolt in bearing,	$\Omega_{brg} = 2.00$	$\phi_{brg} = 0.75$	$\Lambda_{brg} = 0.50$
For web compression buckling,	$\Omega_{cb} = 1.67$	$\phi_{cb} = 0.90$	$\Lambda_{cb} = 0.60$
For web crippling,	$\Omega_{cr} = 2.00$	$\phi_{cr} = 0.75$	$\Lambda_{cr} = 0.50$
For member shear yielding (W,S,M,HSS)	$\Omega_{wy} = 1.50$	$\phi_{wy} = 1.00$	$\Lambda_{wy} = 0.67$



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APPLIED LOADS:

Beam Load

% UDL, UDL := 0.5

Given Load if any, $V_{giv} := 15 \text{ kips}$

Shear Load, $V_{u_{bm}} = 15 \cdot \text{kips}$

Axial Load, $P_{u_{bm}} := 15 \text{ kips}$

Resultant Load, $R_{bm} := \sqrt{V_{u_{bm}}^2 + P_{u_{bm}}^2}$

$R_{bm} = 21.213 \cdot \text{kips}$



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II. CALCULATIONS:

A. BEAM CHECK

1. Rupture Capacity of Beam

a. Shear Rupture Capacity due to Shear Load

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

Net Shear Area

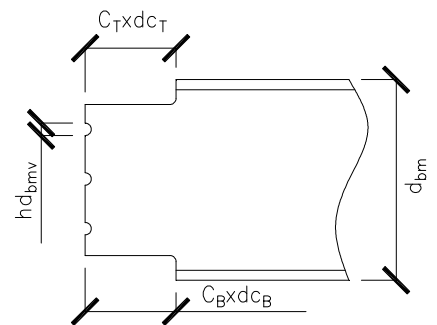
$$A_{nv} := (d_{bm} - dc_T - dc_B - nr \cdot hd_{bm}) \cdot tw_{bm}$$

$$A_{nv} = 2.169 \cdot \text{in}^2$$

Shear Rupture Capacity, (J4-4)

$$R_{vr_{bm}} := \Lambda_{vr} \cdot 0.6 \cdot F_{u_{bm}} \cdot A_{nv}$$

$$R_{vr_{bm}} = 42.298 \cdot \text{kips}$$



b. Tension Rupture Capacity due to Axial Load

(AISC 14th Ed. Specifications Chapter J, Section J4.1, pages 16.1-128 to 16.1-129)

COPE CASE: Coped at top and bottom flanges

Net Tension Area,

$$A_n = 2.169 \cdot \text{in}^2$$

Length of the Connection,

$$L_{con} = 0 \cdot \text{in}$$

Eccentricity of the Connection,

$$e_{con} = NA$$

Reduction Coefficient,

(Table D3.1 page 16.1-28 AISC 14th Ed.)

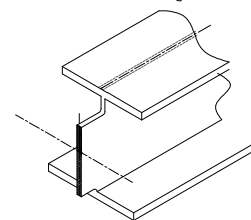
$$U = NA$$

Effective Net Tension Area,

$$A_e := \min(A_n, 0.85A_g)$$

$$A_e = 2.169 \cdot \text{in}^2$$

Coped at top and
bottom flanges



CASE I3



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Tension Rupture Capacity (J4-2)

$$R_{tr_{bm}} := A_{tr} \cdot F_{u_{bm}} \cdot A_e$$

$$R_{tr_{bm}} = 70.497 \cdot \text{kips}$$

Interaction of Rupture Capacities,

$$\left(\frac{V_{u_{bm}}}{R_{vr_{bm}}} \right)^2 + \left(\frac{P_{u_{bm}}}{R_{tr_{bm}}} \right)^2 \leq 1.0$$

$$I_{rc} := \left(\frac{V_{u_{bm}}}{R_{vr_{bm}}} \right)^2 + \left(\frac{P_{u_{bm}}}{R_{tr_{bm}}} \right)^2$$

$$I_{rc} = 0.171$$

RESULT = Rupture Capacity > Force Applied, OK

2. Bolt Bearing Capacity on Beam

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

Bearing Area,

$$A_{brg_{bm}} := d_b \cdot t_{w_{bm}}$$

$$A_{brg_{bm}} = 0.251 \cdot \text{in}^2$$

a. Bolt Bearing due to Shear Load

Bolt centerline distance from face of support,

$$a_b := g_{cab} + 0.5(n_v - 1) \cdot s_v$$

$$a_b = 2.5 \cdot \text{in}$$

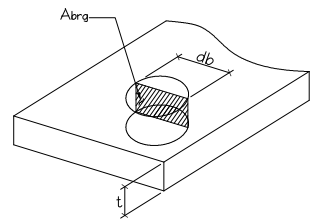
Eccentricity distance of End Reaction from bolt line,

$$e_{b_v} := \begin{cases} 0 \text{ in} & \text{if } a_b \leq 3.0 \text{ in} \wedge n_v = 1 \\ a_b & \text{otherwise} \end{cases}$$

$$e_{b_v} = 0 \cdot \text{in}$$

Load Inclination from vertical,

$$\theta := 0 \text{ deg}$$





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Eccentric Load Coefficient,

(Instantaneous Center of Rotation Method, AISC 14th Ed. Manual Part 7)

$$C_b = 3$$

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

$$F_{be} := \Lambda_{brg} \cdot F_{u_{bm}} \cdot \begin{cases} \begin{bmatrix} 1.0 \cdot (D - d_{cT} - 0.5 \cdot h_{d_{bm}}) \cdot t_{w_{bm}} \\ 1.0 \cdot (L_{eh_{bm}} - 0.5 \cdot h_{d_{bm}}) \cdot t_{w_{bm}} \\ 2.0 \cdot A_{brg_{bm}} \end{bmatrix} & \text{if } h_{d_{bm}} \geq h_{d_{ls}} \\ \begin{bmatrix} 1.2 \cdot (D - d_{cT} - 0.5 \cdot h_{d_{bm}}) \cdot t_{w_{bm}} \\ 1.2 \cdot (L_{eh_{bm}} - 0.5 \cdot h_{d_{bm}}) \cdot t_{w_{bm}} \\ 2.4 \cdot A_{brg_{bm}} \end{bmatrix} & \text{otherwise} \end{cases}$$

$$F_{be} := \begin{cases} \min(F_{be}) & \text{if } e_{b_v} > 0 \text{ in} \\ \min(F_{be_0}, F_{be_2}) & \text{otherwise} \end{cases}$$

$$F_{be} = 13.882 \cdot \text{kips}$$

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

$$F_{bs} := \Lambda_{brg} \cdot F_{u_{bm}} \cdot \begin{cases} \min[1.0 \cdot (s - h_{d_{bm}}) \cdot t_{w_{bm}}, 2.0 \cdot A_{brg_{bm}}] & \text{if } h_{d_{bm}} \geq h_{d_{ls}} \\ \min[1.2 \cdot (s - h_{d_{bm}}) \cdot t_{w_{bm}}, 2.4 \cdot A_{brg_{bm}}] & \text{otherwise} \end{cases}$$

$$F_{bs} = 19.597 \cdot \text{kips}$$

Bolt Bearing Capacity,

$$R_{brg_{bms}} := \begin{cases} C_b \cdot \min(F_{be}, F_{bs}, n_{ca} \cdot \Lambda_{rv}) & \text{if } e_{b_v} > 0 \text{ in} \\ n_v \cdot [\min(F_{be}, n_{ca} \cdot \Lambda_{rv}) + \min(F_{bs}, n_{ca} \cdot \Lambda_{rv}) \cdot (n_r - 1)] & \text{otherwise} \end{cases}$$

$$R_{brg_{bms}} = 53.077 \cdot \text{kips}$$

b. Bolt Bearing due to Axial Load

Bolt vertical centerline distance from beam centerline,

$$a_h := \left| (0.5 \cdot d_{bm}) - [D + 0.5(n_r - 1) \cdot s] \right|$$

$$a_h = 0.05 \cdot \text{in}$$

Eccentricity distance of Axial Load from bolt group centerline,

$$Y_O := a_h$$

$$Y_O = 0.05 \cdot \text{in}$$

Load Inclination from vertical,

$$\theta := 90 \text{deg}$$

Eccentric Load Coefficient,

(Instantaneous Center of Rotation Method, AISC 14th Ed. Manual Part 7)

$$C_a = 2.88$$

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

$$F_{be} := \Lambda_{brg} \cdot F_{u_{bm}} \cdot \begin{cases} \min \left[\begin{array}{l} 1.0 \cdot (D - d_{cT} - 0.5 \cdot h_{d_{bm}}) \cdot t_{w_{bm}} \\ 1.0 \cdot (Le_{h_{bm}} - 0.5 \cdot h_{d_{bm}}) \cdot t_{w_{bm}} \\ 2.0 \cdot A_{brg_{bm}} \end{array} \right] & \text{if } h_{d_{bm}} \geq h_{d_{ls}} \\ \min \left[\begin{array}{l} 1.2 \cdot (D - d_{cT} - 0.5 \cdot h_{d_{bm}}) \cdot t_{w_{bm}} \\ 1.2 \cdot (Le_{h_{bm}} - 0.5 \cdot h_{d_{bm}}) \cdot t_{w_{bm}} \\ 2.4 \cdot A_{brg_{bm}} \end{array} \right] & \text{otherwise} \end{cases}$$

$$F_{be} = 13.882 \cdot \text{kips}$$

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

$$F_{bs} := \Lambda_{brg} \cdot F_{u_{bm}} \cdot \begin{cases} \min[1.0 \cdot (s_v - h_{d_{bm}}) \cdot t_{w_{bm}}, 2.0 \cdot A_{brg_{bm}}] & \text{if } h_{d_{bm}} \geq h_{d_{ls}} \\ \min[1.2 \cdot (s_v - h_{d_{bm}}) \cdot t_{w_{bm}}, 2.4 \cdot A_{brg_{bm}}] & \text{otherwise} \end{cases}$$

$$F_{bs} = 19.597 \cdot \text{kips}$$

Bolt Bearing Capacity,

$$R_{brg_{bma}} := \begin{cases} C_a \cdot \min(F_{be}, F_{bs}, n_{Ca} \cdot \Lambda_{rv}) & \text{if } n_v > 1 \\ C_a \cdot \min(F_{be}, n_{Ca} \cdot \Lambda_{rv}) & \text{otherwise} \end{cases}$$

$$R_{brg_{bma}} = 39.98 \cdot \text{kips}$$



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Interaction of Bolt Bearing Capacities,

$$\left(\frac{V_{u_{bm}}}{R_{brg_{bms}}} \right)^2 + \left(\frac{P_{u_{bm}}}{R_{brg_{bma}}} \right)^2 \leq 1.0$$

$$I_{brg} := \left(\frac{V_{u_{bm}}}{R_{brg_{bms}}} \right)^2 + \left(\frac{P_{u_{bm}}}{R_{brg_{bma}}} \right)^2$$

$$I_{brg} = 0.221$$

RESULT = Bearing Capacity > Force Applied, OK

3. Coped Beam Capacity

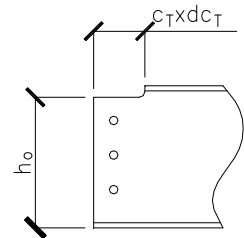
a. Capacity if beam web is single coped at top

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

Depth of Top Cope,

$$dc := dc_T$$

$$dc = 1.5 \cdot \text{in}$$



RESULT = This check is not applicable

Length of Cope,

$$c := c_T$$

$$c = 6 \cdot \text{in}$$

RESULT = This check is not applicable

Reduced Beam Depth,

$$h_o := d_{bm} - dc$$

$$h_o = 10.6 \cdot \text{in}$$

Plate Buckling Coefficient,

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

$$k = 5.626$$



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Plate Buckling Model Adjustment Factor,

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

$$f = 0.992$$

Allowable Flexural Local Buckling Stress/Yielding Stress,

$$F_{cr} := \min \left[26210 \cdot f \cdot k \cdot \left(\frac{t_{w_{bm}}}{h_o} \right)^2 \cdot \text{ksi}, F_{Y_{bm}} \right]$$

$$F_{cr} = 50 \cdot \text{ksi}$$

Location of Neutral Axis on the Reduced Section

$$d_o := h_o - t_{f_{bm}}$$

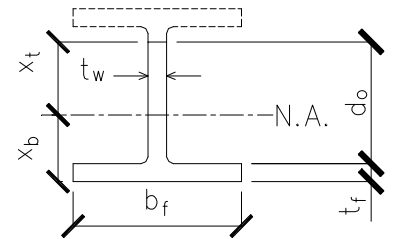
$$d_o = 10.025 \cdot \text{in}$$

$$x_b := \frac{d_o \cdot t_{w_{bm}} \cdot \left(\frac{d_o}{2} + t_{f_{bm}} \right) + b_{f_{bm}} \cdot t_{f_{bm}} \cdot \left(\frac{t_{f_{bm}}}{2} \right)}{d_o \cdot t_{w_{bm}} + b_{f_{bm}} \cdot t_{f_{bm}}}$$

$$x_b = 2.516 \cdot \text{in}$$

$$x_t := h_o - x_b$$

$$x_t = 8.084 \cdot \text{in}$$



Moment of Inertia,

$$I := \frac{t_{w_{bm}} \cdot d_o^3}{12} + d_o \cdot t_{w_{bm}} \cdot \left(\frac{d_o}{2} - x_t \right)^2 + \frac{b_{f_{bm}} \cdot t_{f_{bm}}^3}{12} + b_{f_{bm}} \cdot t_{f_{bm}} \cdot \left(x_b - \frac{t_{f_{bm}}}{2} \right)^2$$

$$I = 82.925 \cdot \text{in}^4$$

Net Section Modulus,

$$S_{net} := \frac{I}{\max(x_t, x_b)}$$



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$$S_{net} = 10.258 \cdot \text{in}^3$$

Eccentricity,

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

$$e_{bm} = 6.5 \cdot \text{in}$$

Flexural Local Buckling Capacity/Yielding Capacity,

$$R_{bc} := \Lambda_b \frac{F_{cr} \cdot S_{net}}{e_{bm}}$$

$$R_{bc} = 47.25 \cdot \text{kips}$$

Flexural Rupture Capacity,

$$R_{fr} := \Lambda_{fr} \frac{F_{u_{bm}} \cdot S_{net}}{e_{bm}}$$

$$R_{fr} = 51.289 \cdot \text{kips}$$

Shear Capacity of Reduced Section,

$$V_{wg1} := \Lambda_{vy} \cdot 0.6 \cdot F_{y_{bm}} \cdot h_o \cdot t_{w_{bm}}$$

$$V_{wg1} = 71.02 \cdot \text{kips}$$

Coped Beam Capacity,

$$R_{scb1} := \min(R_{bc}, R_{fr}, V_{wg1})$$

$$R_{scb1} = 47.25 \cdot \text{kips}$$

$$V_{u_{bm}} = 15 \cdot \text{kips}$$

RESULT = This limit state is not applicable.



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b. Capacity if beam web is double coped with same cope length at both flanges
(AISC 14th Ed. Manual Part 9, page 9-8)

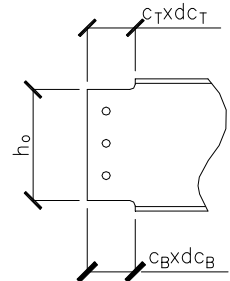
Depth of Cope,

$$\text{Top Cope:} \quad dc_T = 1.5 \cdot \text{in}$$

$$\text{Bottom Cope:} \quad dc_B = 1.5 \cdot \text{in}$$

$$\text{Maximum Cope:} \quad dc := \max(dc_T, dc_B)$$

$$dc = 1.5 \cdot \text{in}$$



RESULT = depth of cope < 0.2 of depth of beam, OK

Length of Cope,

$$\text{Top Cope:} \quad c_T = 6 \cdot \text{in}$$

$$\text{Bottom Cope:} \quad c_B = 6 \cdot \text{in}$$

$$\text{Maximum Cope:} \quad c := \max(c_T, c_B)$$

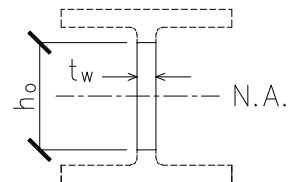
$$c = 6 \cdot \text{in}$$

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

Reduced Beam Depth,

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

$$h_o = 9.1 \cdot \text{in}$$



Adjustment Factor of Lateral-Torsional Buckling Model,

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

$$fd = 2.57$$

Allowable Flexural Local Buckling Stress/Yielding Stress,

$$F_{cr} := \min \left(0.62 \cdot \pi \cdot E \cdot fd \cdot \frac{t_{w_{bm}}^2}{c \cdot h_o}, F_{y_{bm}} \right)$$

$$F_{cr} = 50 \cdot \text{ksi}$$



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Net Section Modulus,

$$S_{net} := \frac{t_{w_{bm}} \cdot h_o^2}{6}$$

$$S_{net} = 4.624 \cdot \text{in}^3$$

Eccentricity,

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

$$e_{bm} = 6.5 \cdot \text{in}$$

Flexural Local Buckling Capacity/Yielding Capacity,

$$R_{bc} := \Lambda_b \frac{F_{cr} \cdot S_{net}}{e_{bm}}$$

$$R_{bc} = 21.297 \cdot \text{kips}$$

Flexural Rupture Capacity,

$$R_{fr} := \Lambda_{fr} \frac{F_{u_{bm}} \cdot S_{net}}{e_{bm}}$$

$$R_{fr} = 23.118 \cdot \text{kips}$$

Shear Capacity of Reduced Section,

$$V_{wg_2} := \Lambda_{vy} \cdot 0.6 \cdot F_{y_{bm}} \cdot h_o \cdot t_{w_{bm}}$$

$$V_{wg_2} = 60.97 \cdot \text{kips}$$

Coped Beam Capacity,

$$R_{dcb_1} := \min(R_{bc}, R_{fr}, V_{wg_2})$$

$$R_{dcb_1} = 21.297 \cdot \text{kips}$$

$$V_{u_{bm}} = 15 \cdot \text{kips}$$

RESULT = Coped Beam Capacity > Applied Force, OK

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)



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Allowable Flexural Yielding Stress,

$$F_{cr} := F_{Y_{bm}}$$

$$F_{cr} = 50 \cdot \text{ksi}$$

Location of Neutral Axis on the Reduced Section

$$d_o := d_{bm} - d_{c_B} - t_{f_{bm}}$$

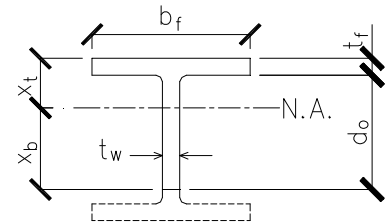
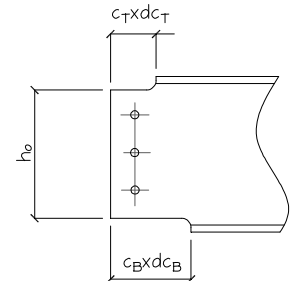
$$d_o = 10.025 \cdot \text{in}$$

$$x_t := \frac{d_o \cdot t_{w_{bm}} \cdot \left(\frac{d_o}{2} + t_{f_{bm}} \right) + b_{f_{bm}} \cdot t_{f_{bm}} \cdot \left(\frac{t_{f_{bm}}}{2} \right)}{d_o \cdot t_{w_{bm}} + b_{f_{bm}} \cdot t_{f_{bm}}}$$

$$x_t = 2.516 \cdot \text{in}$$

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

$$x_b = 8.084 \cdot \text{in}$$



Moment of Inertia,

$$I := \frac{t_{w_{bm}} \cdot d_o^3}{12} + d_o \cdot t_{w_{bm}} \cdot \left(x_b - \frac{d_o}{2} \right)^2 + \frac{b_{f_{bm}} \cdot t_{f_{bm}}^3}{12} + b_{f_{bm}} \cdot t_{f_{bm}} \cdot \left(x_t - \frac{t_{f_{bm}}}{2} \right)^2$$

$$I = 82.925 \cdot \text{in}^4$$

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

$$S_{net} := \frac{I}{x_b}$$

$$S_{net} = 10.258 \cdot \text{in}^3$$

Eccentricity,

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

$$e_{bm} = 6.5 \cdot \text{in}$$



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Flexural Yielding Capacity,

$$R_{bc} := \Lambda_b \frac{F_{cr} \cdot S_{net}}{e_{bm}}$$

$$R_{bc} = 47.25 \cdot \text{kips}$$

Flexural Rupture Capacity,

$$R_{fr} := \Lambda_{fr} \frac{F_{u_{bm}} \cdot S_{net}}{e_{bm}}$$

$$R_{fr} = 51.289 \cdot \text{kips}$$

Shear Capacity of Reduced Section,

$$V_{wg3} := \Lambda_{vY} \cdot 0.6 \cdot F_{Y_{bm}} \cdot h_o \cdot t_{w_{bm}}$$

$$V_{wg3} = 60.97 \cdot \text{kips}$$

Coped Beam Capacity,

$$R_{dcb2} := \min(R_{bc}, R_{fr}, V_{wg3})$$

$$R_{dcb2} = 47.25 \cdot \text{kips}$$

$$V_{u_{bm}} = 15 \cdot \text{kips}$$

RESULT = This limit state is not applicable.

d. Capacity if beam web is deep coped (for $dc > 0.2d_{bm}$)

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

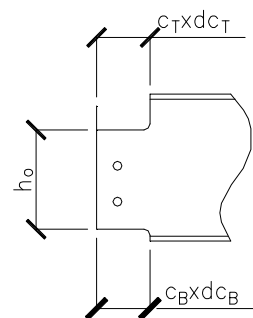
Coefficient,

$$\lambda := \frac{h_o \cdot \sqrt{F_{Y_{bm}}}}{10 \cdot t_{w_{bm}} \cdot \sqrt{475 + 280 \cdot \left(\frac{h_o}{c}\right)^2}} \cdot \sqrt{\frac{1}{\text{ksi}}}$$

$$\lambda = 0.574$$

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

$$Q = 1$$





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Allowable Buckling Stress,

$$F_{cr} := Q \cdot F_{Y_{bm}}$$

$$F_{cr} = 50 \cdot \text{ksi}$$

Net Section Modulus,

$$S_{net} := \frac{t_{w_{bm}} \cdot h_o^2}{6}$$

$$S_{net} = 4.624 \cdot \text{in}^3$$

Eccentricity,

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

$$e_{bm} = 6.5 \cdot \text{in}$$

Buckling Capacity,

$$R_{bc} := \Lambda_b \frac{F_{cr} \cdot S_{net}}{e_{bm}}$$

$$R_{bc} = 21.297 \cdot \text{kips}$$

Flexural Rupture Capacity,

$$R_{fr} := \Lambda_{fr} \frac{F_{u_{bm}} \cdot S_{net}}{e_{bm}}$$

$$R_{fr} = 23.118 \cdot \text{kips}$$

Shear Capacity of Reduced Section,

$$V_{wg_4} := \Lambda_{vY} \cdot 0.6 \cdot F_{Y_{bm}} \cdot h_o \cdot t_{w_{bm}}$$

$$V_{wg_4} = 60.97 \cdot \text{kips}$$

Coped Beam Capacity,

$$R_{tdcb} := \min(R_{bc}, R_{fr}, V_{wg_4})$$

$$R_{tdcb} = 21.297 \cdot \text{kips}$$

$$V_{u_{bm}} = 15 \cdot \text{kips}$$

RESULT = This limit state is not applicable.

e. Capacity if beam web is single coped at bottom

Depth of cope,

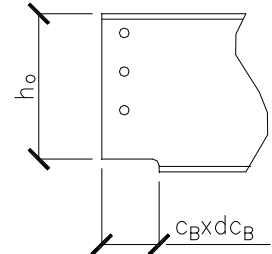
$$dc := dc_B$$

$$dc = 1.5 \cdot \text{in}$$

Length of Cope,

$$c := c_B$$

$$c = 6 \cdot \text{in}$$



Reduced Beam Depth,

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

$$ho = 10.6 \cdot \text{in}$$

Allowable Flexural Yielding Stress,

$$F_{cr} := F_{Y_{bm}}$$

$$F_{cr} = 50 \cdot \text{ksi}$$

Location of Neutral Axis on the Reduced Section

$$do := ho - t_{f_{bm}}$$

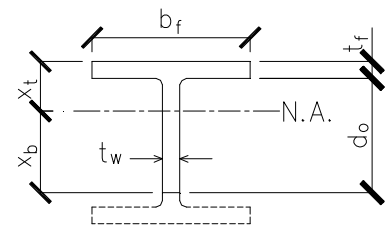
$$do = 10.025 \cdot \text{in}$$

$$x_t := \frac{do \cdot t_{w_{bm}} \cdot \left(\frac{do}{2} + t_{f_{bm}} \right) + b_{f_{bm}} \cdot t_{f_{bm}} \cdot \left(\frac{t_{f_{bm}}}{2} \right)}{do \cdot t_{w_{bm}} + b_{f_{bm}} \cdot t_{f_{bm}}}$$

$$x_t = 2.516 \cdot \text{in}$$

$$x_b := ho - x_t$$

$$x_b = 8.084 \cdot \text{in}$$



Moment of Inertia,

$$I := \frac{t_{w_{bm}} \cdot do^3}{12} + do \cdot t_{w_{bm}} \cdot \left(x_b - \frac{do}{2} \right)^2 + \frac{b_{f_{bm}} \cdot t_{f_{bm}}^3}{12} + b_{f_{bm}} \cdot t_{f_{bm}} \cdot \left(x_t - \frac{t_{f_{bm}}}{2} \right)^2$$

$$I = 82.925 \cdot \text{in}^4$$



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Net Section Modulus,

$$S_{net} := \frac{I}{x_b}$$

$$S_{net} = 10.258 \cdot \text{in}^3$$

Eccentricity,

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

$$e_{bm} = 6.5 \cdot \text{in}$$

Flexural Yielding Capacity,

$$R_{bc} := \Lambda_b \cdot \frac{F_{cr} \cdot S_{net}}{e_{bm}}$$

$$R_{bc} = 47.25 \cdot \text{kips}$$

Flexural Rupture Capacity,

$$R_{fr} := \Lambda_{fr} \cdot \frac{F_{u_{bm}} \cdot S_{net}}{e_{bm}}$$

$$R_{fr} = 51.289 \cdot \text{kips}$$

Gross Web Shear,

$$V_{wg5} := \Lambda_{vy} \cdot 0.6 \cdot F_{Y_{bm}} \cdot h_o \cdot t_{w_{bm}}$$

$$V_{wg5} = 71.02 \cdot \text{kips}$$

Coped Beam Capacity,

$$R_{scb2} := \min(R_{bc}, R_{fr}, V_{wg5})$$

$$R_{scb2} = 47.25 \cdot \text{kips}$$

$$V_{u_{bm}} = 15 \cdot \text{kips}$$

RESULT = This limit state is not applicable.



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f. Capacity if beam is cut-flushed

Reduced Beam Depth,

$$h_o = 10.6 \cdot \text{in}$$

Allowable Flexural Local Buckling Stress/Yielding Stress,

$$F_{cr} = 50 \cdot \text{ksi}$$

Net Section Modulus,

$$S_{net} = 9.355 \cdot \text{in}^3$$

Eccentricity,

$$e_{bm} = 6.5 \cdot \text{in}$$

Flexural Local Buckling Capacity/Yielding Capacity,

$$R_{bc} := \Lambda_b \cdot \frac{F_{cr} \cdot S_{net}}{e_{bm}}$$

$$R_{bc} = 43.092 \cdot \text{kips}$$

Flexural Rupture Capacity,

$$R_{fr} := \Lambda_{fr} \frac{F_{u_{bm}} \cdot S_{net}}{e_{bm}}$$

$$R_{fr} = 46.777 \cdot \text{kips}$$

Gross Web Shear,

$$V_{wg_6} := \Lambda_{vy} \cdot 0.6 \cdot F_{Y_{bm}} \cdot h_o \cdot t_{w_{bm}}$$

$$V_{wg_6} = 71.02 \cdot \text{kips}$$

Coped Beam Capacity,

$$R_{cfb} := \min(R_{bc}, R_{fr}, V_{wg_6})$$

$$R_{cfb} = 43.092 \cdot \text{kips}$$

$$V_{u_{bm}} = 15 \cdot \text{kips}$$

RESULT = This limit state is not applicable.



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4. Block Shear Capacity of Beam

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

a. Block Shear due to Shear Load

(for $dc_T > 0$ and $c_T \geq Le_{h_{bm}} + (nv - 1) \cdot sv$)

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

$$U_{bs} = 1$$

Gross Shear Area

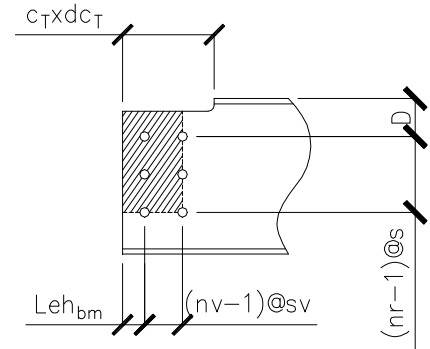
$$Agv := [(nr - 1) \cdot s + (D - dc_T)] \cdot tw_{bm}$$

$$Agv = 2.512 \cdot \text{in}^2$$

Net Tension Area

$$Ant := [Le_{h_{bm}} + (nv - 1) \cdot sv - (nv - 0.5) \cdot hd_{bm}] \cdot tw_{bm}$$

$$Ant = 0.523 \cdot \text{in}^2$$



Net Shear Area

$$Anv := Agv - (nr - 0.5) \cdot hd_{bm} \cdot tw_{bm}$$

$$Anv = 1.78 \cdot \text{in}^2$$

Block Shear Capacity of Beam, (J4-5)

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

$$RbsV_{bm} = 51.716 \cdot \text{kips}$$

b. Block Shear due to Axial Load

(for $dc_T > 0$ and $c_T \geq Le_{h_{bm}} + (nv - 1) \cdot sv$)

Pattern 1

$$\text{Reduction Factor, } U_{bs} := \begin{cases} 1.0 & \text{if } a_h = 0 \cdot \text{in} \quad (\text{tension stress is uniform}) \\ 0.5 & \text{otherwise} \quad (\text{tension stress is non-uniform}) \end{cases}$$

$$U_{bs} = 0.5$$



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Gross Shear Area

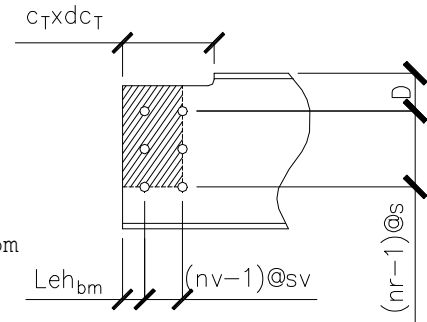
$$A_{gv} := [(n_v - 1) \cdot s_v + L_{eh_{bm}}] \cdot t_{w_{bm}}$$

$$A_{gv} = 0.67 \cdot \text{in}^2$$

Net Tension Area

$$A_{nt} := [(n_r - 1) \cdot s + (D - d_{c_T}) - (n_r - 0.5) \cdot h_{d_{bm}}] \cdot t_{w_{bm}}$$

$$A_{nt} = 1.78 \cdot \text{in}^2$$



Net Shear Area

$$A_{nv} := [L_{eh_{bm}} + (n_v - 1) \cdot s_v - (n_v - 0.5) \cdot h_{d_{bm}}] \cdot t_{w_{bm}}$$

$$A_{nv} = 0.523 \cdot \text{in}^2$$

Block Shear Capacity of Beam, (J4-5)

$$R_{bsA_{bm1}} := \Lambda_{bs} \min(0.6 F_{u_{bm}} \cdot A_{nv} + U_{bs} \cdot F_{u_{bm}} \cdot A_{nt}, 0.6 \cdot F_{y_{bm}} \cdot A_{gv} + U_{bs} \cdot F_{u_{bm}} \cdot A_{nt})$$

$$R_{bsA_{bm1}} = 38.97 \cdot \text{kips}$$

Interaction of Block Shear Capacities,

$$\left(\frac{V_{u_{bm}}}{R_{bsV_{bm}}} \right)^2 + \left(\frac{P_{u_{bm}}}{R_{bsA_{bm1}}} \right)^2 \leq 1.0$$

$$I_{bs} := \left(\frac{V_{u_{bm}}}{R_{bsV_{bm}}} \right)^2 + \left(\frac{P_{u_{bm}}}{R_{bsA_{bm1}}} \right)^2$$

$$I_{bs} = 0.232$$

RESULT = Block Shear Capacity > Force Applied, OK

c. Block Shear due to Axial Load (Pattern 2)

Pattern 2

Reduction Factor, $U_{bs} = 0.5$ (tension stress is non-uniform)



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Gross Shear Area

$$A_{gv} := [(n_v - 1) \cdot s_v + L_{eh_{bm}}] \cdot t_{w_{bm}} \cdot 2$$

$$A_{gv} = 1.34 \cdot \text{in}^2$$

Net Tension Area

$$A_{nt} := [(n_r - 1) \cdot s - (n_r - 1) \cdot h_{d_{bm}}] \cdot t_{w_{bm}}$$

$$A_{nt} = 1.424 \cdot \text{in}^2$$

Net Shear Area

$$A_{nv} := [(n_v - 1) \cdot s_v + L_{eh_{bm}} - (n_v - 0.5) \cdot h_{d_{bm}}] \cdot t_{w_{bm}} \cdot 2$$

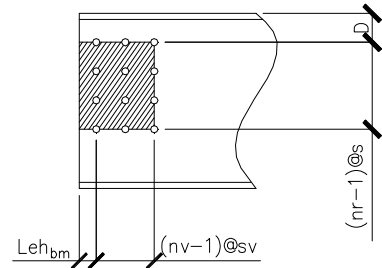
$$A_{nv} = 1.047 \cdot \text{in}^2$$

Block Shear Capacity of Beam, (J4-5)

$$R_{bsA_{bm2}} := \Lambda_{bs} \min(0.6 F_{u_{bm}} \cdot A_{nv} + U_{bs} \cdot F_{u_{bm}} \cdot A_{nt}, 0.6 \cdot F_{y_{bm}} \cdot A_{gv} + U_{bs} \cdot F_{u_{bm}} \cdot A_{nt})$$

$$R_{bsA_{bm2}} = 43.236 \cdot \text{kips}$$

$$P_{u_{bm}} = 15 \cdot \text{kips}$$



RESULT = Block Shear Capacity > Force Applied, OK

5. 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,

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

$$h = 9.94 \cdot \text{in}$$

Limiting depth-thickness ratio,

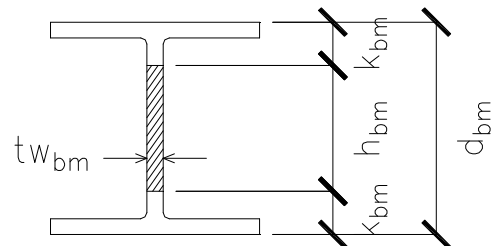
$$h_{tw} := \frac{h}{t_{w_{bm}}}$$

$$h_{tw} = 29.672$$

Clear distance between transverse stiffeners,

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

$$a = 0 \cdot \text{in}$$





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Web plate buckling coefficient,

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

$k_v = 5$

Web shear coefficient,

$$C_v := \begin{cases} 1 & \text{if } h_{tw} \leq 1.1 \cdot \sqrt{\frac{k_v \cdot E}{F_{y_{bm}}}} \end{cases} \quad (G2-3)$$

$$\frac{1.1 \cdot \sqrt{\frac{k_v \cdot E}{F_{y_{bm}}}}}{h_{tw}} \quad \text{if } 1.1 \cdot \sqrt{\frac{k_v \cdot E}{F_{y_{bm}}}} < h_{tw} \leq 1.37 \cdot \sqrt{\frac{k_v \cdot E}{F_{y_{bm}}}} \quad (G2-4)$$

$$\frac{1.51 \cdot E \cdot k_v}{h_{tw}^2 F_{y_{bm}}} \quad \text{if } 1.37 \cdot \sqrt{\frac{k_v \cdot E}{F_{y_{bm}}}} < h_{tw} \quad (G2-5)$$

$$C_v = 1$$

Shear Capacity of Section,

$$R_{v_{bm}} := \Lambda_{v_{bm}} \cdot 0.6 \cdot F_{y_{bm}} \cdot d_{bm} \cdot t_{w_{bm}} \cdot C_v \quad (G2-1)$$

$$R_{v_{bm}} = 81.07 \cdot \text{kips}$$

$$V_{u_{bm}} = 15 \cdot \text{kips}$$

RESULT = Shear Capacity of Section > Force Applied, OK

B. BEAM TO CLIP ANGLE CHECK

1. Eccentric Bolt Shear Capacity

(AISC 14th Ed. Manual Part 7, pages 7-6 to 7-12)

Load Inclination from vertical,

$$\theta := \text{atan}\left(\frac{P_{u_{bm}}}{V_{u_{bm}}}\right)$$

$$\theta = 45 \cdot \text{deg}$$

Eccentric Load Coefficient,

(Instantaneous Center of Rotation Method, AISC 14th Ed. Manual Part 7)

$$C = 2.939$$



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Shear Capacity per bolt,

$$\Lambda_{rv} = 11.928 \cdot \text{kips}$$

Eccentric Bolt Capacity,

$$R_{eb} := n_{ca} \cdot C \cdot \Lambda_{rv}$$

$$R_{eb} = 70.122 \cdot \text{kips}$$

$$R_{bm} = 21.213 \cdot \text{kips}$$

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,

$$s = 3 \cdot \text{in}$$

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

$$s_{\min} = 2 \cdot \text{in}$$

$$s_{\max} := \min(12 \text{ in}, 24 \cdot \min(t_{w_{bm}}, t_{ca}, t_{w_{gir}}))$$

$$s_{\max} = 8.040 \cdot \text{in}$$

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

Horizontal Spacing,

$$sv = 3 \cdot \text{in}$$

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

$$sv_{\min} = 2 \cdot \text{in}$$

$$sv_{\max} := \min(12 \text{ in}, 24 \cdot \min(t_{w_{bm}}, t_{ca}, t_{w_{gir}}))$$

$$sv_{\max} = 8.040 \cdot \text{in}$$

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)



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Vertical Edge Distance,

$$Lev = 1.25 \cdot in$$

$$Lev_{bm} := D - dc_T$$

$$Lev_{con} := \begin{pmatrix} Lev \\ Lev_{bm} \end{pmatrix} = \begin{pmatrix} 1.25 \\ 1.5 \end{pmatrix} \cdot in$$

$$Lev_{min} := \begin{pmatrix} Lev_{minca} \\ Lev_{minbm} \end{pmatrix} = \begin{pmatrix} 1 \\ 1 \end{pmatrix} \cdot in$$

$$Lev_{max} := \begin{cases} \min(6in, 12 \cdot tca) & \text{if } \min(Lev_{con}) = Lev \\ \min(6in, 12 \cdot tw_{bm}) & \text{otherwise} \end{cases}$$

$$Lev_{max} = 6 \cdot in$$

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

Horizontal Edge Distance,

$$Leh_{cab} = 1.5 \cdot in$$

$$Leh_{cag} = 1.417 \cdot in$$

$$Leh_{bm} = 2 \cdot in$$

$$Leh_{con} := \begin{pmatrix} Leh_{cab} \\ Leh_{cag} \\ Leh_{bm} \end{pmatrix} = \begin{pmatrix} 1.5 \\ 1.417 \\ 2 \end{pmatrix} \cdot in$$

$$Leh_{min} := \begin{pmatrix} Leh_{mincab} \\ Leh_{mincag} \\ Leh_{minbm} \end{pmatrix} = \begin{pmatrix} 1 \\ 1 \\ 1 \end{pmatrix} \cdot in$$

$$Leh_{max} := \begin{cases} \min(6in, 12 \cdot tw_{bm}) & \text{if } \min(Leh_{con}) = Leh_{bm} \\ \min(6in, 12 \cdot tca) & \text{otherwise} \end{cases}$$

$$Leh_{max} = 6 \cdot in$$

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



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C. CLIP ANGLE CHECK

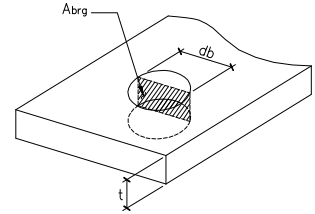
1. Bolt Bearing Capacity on Clip Angle

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

Bearing Area,

$$A_{brg_{ca}} := d_b \cdot t_{ca}$$

$$A_{brg_{ca}} = 0.375 \cdot \text{in}^2$$



Beam Side:

a. Bolt Bearing due to Shear Load

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

$$F_{be} := \Lambda_{brg} \cdot F_{u_{ca}} \cdot \begin{cases} \begin{bmatrix} 1.0 \cdot (Le_v - 0.5 \cdot hd_{bcav}) \cdot t_{ca} \\ 1.0 \cdot (Le_{h_{cab}} - 0.5 \cdot hd_{bcah}) \cdot t_{ca} \\ 2.0 \cdot A_{brg_{ca}} \end{bmatrix} & \text{if } hd_{bcah} \geq hd_{ls} \\ \begin{bmatrix} 1.2 \cdot (Le_v - 0.5 \cdot hd_{bcav}) \cdot t_{ca} \\ 1.2 \cdot (Le_{h_{cab}} - 0.5 \cdot hd_{bcah}) \cdot t_{ca} \\ 2.4 \cdot A_{brg_{ca}} \end{bmatrix} & \text{otherwise} \end{cases}$$

$$F_{be} := \begin{cases} \min(F_{be}) & \text{if } eb_v > 0 \text{ in} \\ \min(F_{be0}, F_{be2}) & \text{otherwise} \end{cases}$$

$$F_{be} = 14.138 \cdot \text{kips}$$

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

$$F_{bs} := \Lambda_{brg} \cdot F_{u_{ca}} \cdot \begin{cases} \min[1.0 \cdot (s - hd_{bcav}) \cdot t_{ca}, 2.0 \cdot A_{brg_{ca}}] & \text{if } hd_{bcah} \geq hd_{ls} \\ \min[1.2 \cdot (s - hd_{bcav}) \cdot t_{ca}, 2.4 \cdot A_{brg_{ca}}] & \text{otherwise} \end{cases}$$

$$F_{bs} = 26.1 \cdot \text{kips}$$

Bolt Bearing Capacity,

$$R_{brg_{cabs}} := n_{ca} \cdot \begin{cases} C_b \cdot \min(F_{be}, F_{bs}, \Lambda_{rv}) & \text{if } e_{b_v} > 0 \text{ in} \\ n_v \cdot [\min(F_{be}, \Lambda_{rv}) + \min(F_{bs}, \Lambda_{rv}) \cdot (n_r - 1)] & \text{otherwise} \end{cases}$$

$$R_{brg_{cabs}} = 71.569 \cdot \text{kips}$$

b. Bolt Bearing due to Axial Load

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

$$F_{be} := \Lambda_{brg} \cdot F_{u_{ca}} \cdot \begin{cases} \min \left[\begin{array}{l} 1.0 \cdot (Le_v - 0.5 \cdot hd_{bcav}) \cdot t_{ca} \\ 1.0 \cdot (Le_{cab} - 0.5 \cdot hd_{bcah}) \cdot t_{ca} \\ 2.0 \cdot A_{brg_{ca}} \end{array} \right] & \text{if } hd_{bcav} \geq hd_{ls} \\ \min \left[\begin{array}{l} 1.2 \cdot (Le_v - 0.5 \cdot hd_{bcav}) \cdot t_{ca} \\ 1.2 \cdot (Le_{cab} - 0.5 \cdot hd_{bcah}) \cdot t_{ca} \\ 2.4 \cdot A_{brg_{ca}} \end{array} \right] & \text{otherwise} \end{cases}$$

$$F_{be} = 14.138 \cdot \text{kips}$$

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

$$F_{bs} := \Lambda_{brg} \cdot F_{u_{ca}} \cdot \begin{cases} \min[1.0 \cdot (s_v - hd_{bcah}) \cdot t_{ca}, 2.0 \cdot A_{brg_{ca}}] & \text{if } hd_{bcav} \geq hd_{ls} \\ \min[1.2 \cdot (s_v - hd_{bcah}) \cdot t_{ca}, 2.4 \cdot A_{brg_{ca}}] & \text{otherwise} \end{cases}$$

$$F_{bs} = 26.1 \cdot \text{kips}$$

Bolt Bearing Capacity,

$$R_{brg_{caba}} := n_{ca} \cdot \begin{cases} C_a \cdot \min(F_{be}, F_{bs}, \Lambda_{rv}) & \text{if } n_v > 1 \\ C_a \cdot \min(F_{be}, \Lambda_{rv}) & \text{otherwise} \end{cases}$$

$$R_{brg_{caba}} = 68.708 \cdot \text{kips}$$

Interaction of Bolt Bearing Capacities,

$$\left(\frac{V_{u_{bm}}}{R_{brg_{cabs}}} \right)^2 + \left(\frac{P_{u_{bm}}}{R_{brg_{caba}}} \right)^2 \leq 1.0$$



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$$I_{brg} := \left(\frac{V_{u_{bm}}}{R_{brg_{cabs}}} \right)^2 + \left(\frac{P_{u_{bm}}}{R_{brg_{caba}}} \right)^2$$

$$I_{brg} = 0.092$$

RESULT = Bearing Capacity > Force Applied, OK

Support Side:

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

$$F_{be} := \Lambda_{brg} \cdot F_{u_{ca}} \cdot \begin{cases} \min[1.0 \cdot (Lev - 0.5 \cdot hd_{gcav}) \cdot t_{ca}, 2.0 \cdot A_{brg_{ca}}] & \text{if } hd_{gcah} \geq hd_{ls} \\ \min[1.2 \cdot (Lev - 0.5 \cdot hd_{gcav}) \cdot t_{ca}, 2.4 \cdot A_{brg_{ca}}] & \text{otherwise} \end{cases}$$

$$F_{be} = 14.138 \cdot \text{kips}$$

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

$$F_{bs} := \Lambda_{brg} \cdot F_{u_{ca}} \cdot \begin{cases} \min[1.0 \cdot (s - hd_{gcav}) \cdot t_{ca}, 2.0 \cdot A_{brg_{ca}}] & \text{if } hd_{gcah} \geq hd_{ls} \\ \min[1.2 \cdot (s - hd_{gcav}) \cdot t_{ca}, 2.4 \cdot A_{brg_{ca}}] & \text{otherwise} \end{cases}$$

$$F_{bs} = 26.1 \cdot \text{kips}$$

Bolt Bearing Capacity,

$$R_{brg_{cag}} := n_{ca} \cdot n_v \cdot [\min(F_{be}, \Lambda_{rv}) + \min(F_{bs}, \Lambda_{rv}) \cdot (nr - 1)]$$

$$R_{brg_{cag}} = 71.569 \cdot \text{kips}$$

$$V_{u_{bm}} = 15 \cdot \text{kips}$$

RESULT = Bearing Capacity > Force Applied, OK

2. Yielding Capacity of Clip Angle

Beam Side:

a. Shear Yielding Capacity due to Shear Load

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

Length of Angle,

$$L_{ca} := (nr - 1) \cdot s + 2 \cdot Lev$$

$$L_{ca} = 8.5 \cdot \text{in}$$

Check if Length of Angle is acceptable,

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



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

Gross Shear Capacity,

$$R_{vy_{ca}} := \Lambda_{vy} \cdot 0.6 \cdot F_{Y_{ca}} \cdot t_{ca} \cdot L_{ca} \cdot n_{ca} \quad (J4-3)$$

$$R_{vy_{ca}} = 122.4 \cdot \text{kips}$$

b. Tension Yielding Capacity due to Axial Load

(AISC 14th Ed. Specifications Chapter J, Section J4.1, page 16.1-128)

Gross Tension Area,

$$A_{g_{ca}} := L_{ca} \cdot t_{ca}$$

$$A_{g_{ca}} = 4.25 \cdot \text{in}^2$$

Gross Tension Capacity,

$$R_{ty_{ca}} := \Lambda_{ty} \cdot F_{Y_{ca}} \cdot A_{g_{ca}} \cdot n_{ca} \quad (J4-1)$$

$$R_{ty_{ca}} = 183.234 \cdot \text{kips}$$

Interaction of Yielding Capacities,

$$\left(\frac{V_{u_{bm}}}{R_{vy_{ca}}} \right)^2 + \left(\frac{P_{u_{bm}}}{R_{ty_{ca}}} \right)^2 \leq 1.0$$

$$I_{yc} := \left(\frac{V_{u_{bm}}}{R_{vy_{ca}}} \right)^2 + \left(\frac{P_{u_{bm}}}{R_{ty_{ca}}} \right)^2$$

$$I_{yc} = 0.022$$

RESULT = Yielding Capacity > Force Applied, OK

3. Rupture Capacity of Clip Angle

Beam Side:

a. Shear Rupture Capacity due to Shear Load

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

Net Shear Area

$$A_{nv} := (L_{ca} - n_r \cdot h_{d_{bcav}}) \cdot t_{ca}$$



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$$A_{nv} = 2.938 \cdot \text{in}^2$$

Shear Rupture Capacity,

$$R_{vr_{ca}} := \Lambda_{vr} \cdot n_{ca} \cdot 0.6 \cdot F_{u_{ca}} \cdot A_{nv} \quad (J4-4)$$

$$R_{vr_{ca}} = 102.225 \cdot \text{kips}$$

b. Tension Rupture Capacity due to Axial Load

(AISC 14th Ed. Specifications Chapter J, Section J4.1, pages 16.1-128 to 16.1-129)

Net Tension Area,

$$A_e := \min \left[(L_{ca} - n_r \cdot h_{d_{bcav}}) \cdot t_{ca}, 0.85 \cdot A_{g_{ca}} \right]$$

$$A_e = 2.938 \cdot \text{in}^2$$

Tension Rupture Capacity,

$$R_{tr_{ca}} := \Lambda_{tr} \cdot F_{u_{ca}} \cdot A_e \cdot n_{ca} \quad (J4-2)$$

$$R_{tr_{ca}} = 170.375 \cdot \text{kips}$$

Interaction of Rupture Capacities,

$$\left(\frac{V_{u_{bm}}}{R_{vr_{ca}}} \right)^2 + \left(\frac{P_{u_{bm}}}{R_{tr_{ca}}} \right)^2 \leq 1.0$$

$$I_{rc} := \left(\frac{V_{u_{bm}}}{R_{vr_{ca}}} \right)^2 + \left(\frac{P_{u_{bm}}}{R_{tr_{ca}}} \right)^2$$

$$I_{rc} = 0.029$$

RESULT = Rupture Capacity > Force Applied, OK

4. Block Shear Capacity of Clip Angle

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

Beam Side:



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a. Block Shear due to Shear Load

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

$$U_{bs} = 1$$

Gross Shear Area

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

$$A_{gv} = 7.25 \cdot \text{in}^2$$

Net Tension Area

$$A_{nt} := n_{ca} \cdot [\text{leg2}_{ca} - g_{cab} - (n_v - 0.5) \cdot h_{d_{bcav}}] \cdot t_{ca}$$

$$A_{nt} = 1.062 \cdot \text{in}^2$$

Net Shear Area

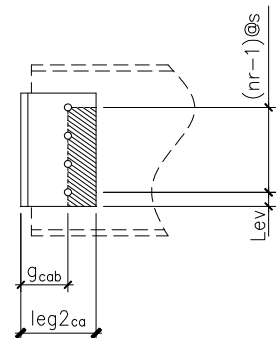
$$A_{nv} := A_{gv} - n_{ca} \cdot (nr - 0.5) \cdot h_{d_{bcav}} \cdot t_{ca}$$

$$A_{nv} = 5.062 \cdot \text{in}^2$$

Block Shear Capacity of Clip Angle, (J4-5)

$$R_{bsV_{cab}} := \Lambda_{bs} \min(0.6 F_{u_{ca}} \cdot A_{nv} + U_{bs} \cdot F_{u_{ca}} \cdot A_{nt}, 0.6 \cdot F_{y_{ca}} \cdot A_{gv} + U_{bs} \cdot F_{u_{ca}} \cdot A_{nt})$$

$$R_{bsV_{cab}} = 109.112 \cdot \text{kips}$$

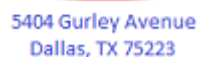


b. Block Shear due to Axial Load

Pattern 1

$$\text{Reduction Factor, } U_{bs} := \begin{cases} 1.0 & \text{if } a_h = 0 \cdot \text{in} & \text{(tension stress is uniform)} \\ 0.5 & \text{otherwise} & \text{(tension stress is non-uniform)} \end{cases}$$

$$U_{bs} = 0.5$$


$$\text{Agv} := n_{\text{ca}} \cdot \lceil (nv - 1) \cdot sv + \text{Leh}_{\text{cab}} \rceil \cdot \text{tca}$$

$$A_{gv} = 1.5 \cdot \text{in}^2$$

$$\text{Ant} := n_{\text{ca}} \cdot \lceil (nr - 1) \cdot s + \text{Lev} - (nr - 0.5) \cdot \text{hd}_{\text{bcav}} \rceil \cdot \text{tca}$$

$$\text{Ant} = 5.062 \cdot \text{in}^2$$

$$\text{Anv} := \text{Agv} - n_{ca} \cdot (nv - 0.5) \cdot \text{hd}_{bcah} \cdot tca$$

$$A_{nv} = 1.062 \cdot \text{in}^2$$

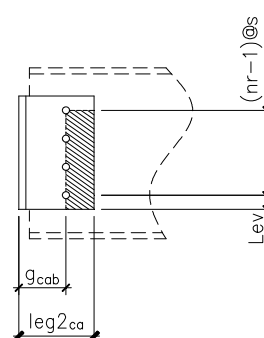
$$\text{RbsA}_{\text{cab1}} := \Lambda_{\text{bs}} \min \left(0.6 \cdot \text{Fu}_{\text{ca}} \cdot \text{Anv} + \text{U}_{\text{bs}} \cdot \text{Fu}_{\text{ca}} \cdot \text{Ant}, 0.6 \cdot \text{Fy}_{\text{ca}} \cdot \text{Agv} + \text{U}_{\text{bs}} \cdot \text{Fu}_{\text{ca}} \cdot \text{Ant} \right)$$

$$RbsA_{Cab1} = 89.606 \cdot kips$$

$$\left(\frac{Vu_{bm}}{RbsV_{cab}}\right)^2 + \left(\frac{Pu_{bm}}{RbsA_{cab1}}\right)^2 \leq 1.0$$

$$I_{bs} := \left(\frac{Vu_{bm}}{RbsV_{cab}} \right)^2 + \left(\frac{Pu_{bm}}{RbsA_{cab1}} \right)^2$$

$$I_{bS} = 0.047$$



RESULT = Block Shear Capacity > Force Applied, OK

c. Block Shear due to Axial Load (Pattern 2)

Pattern 2

Reduction Factor, $U_{bs} = 0.5$

(tension stress is non-uniform)

Gross Shear Area

$$A_{gv} := n_{ca} \cdot [(n_v - 1) \cdot s_v + Le_{h_{cab}}] \cdot t_{ca} \cdot 2$$

$$A_{gv} = 3 \cdot \text{in}^2$$

Net Tension Area

$$A_{nt} := n_{ca} \cdot [(nr - 1) \cdot s - (nr - 1) \cdot hd_{bcav}] \cdot t_{ca}$$

$$A_{nt} = 4.25 \cdot \text{in}^2$$

Net Shear Area

$$A_{nv} := A_{gv} - n_{ca} \cdot (n_v - 0.5) \cdot hd_{bcah} \cdot t_{ca} \cdot 2$$

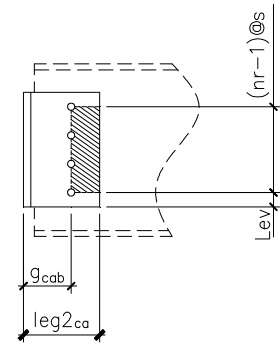
$$A_{nv} = 2.125 \cdot \text{in}^2$$

Block Shear Capacity of Clip Angle, (J4-5)

$$R_{bsA_{cab2}} := \Lambda_{bs} \min(0.6 \cdot F_{u_{ca}} \cdot A_{nv} + U_{bs} \cdot F_{u_{ca}} \cdot A_{nt}, 0.6 \cdot F_{y_{ca}} \cdot A_{gv} + U_{bs} \cdot F_{u_{ca}} \cdot A_{nt})$$

$$R_{bsA_{cab2}} = 94.025 \cdot \text{kips}$$

$$P_{u_{bm}} = 15 \cdot \text{kips}$$



RESULT = Block Shear Capacity > Force Applied, OK

Support Side:

Reduction Factor, $U_{bs} := \begin{cases} 1.0 & \text{if } n_v = 1 \\ 0.5 & \text{if } n_v > 1 \end{cases}$

(tension stress is uniform)

(tension stress is non-uniform)

$$U_{bs} = 1$$

Gross Shear Area

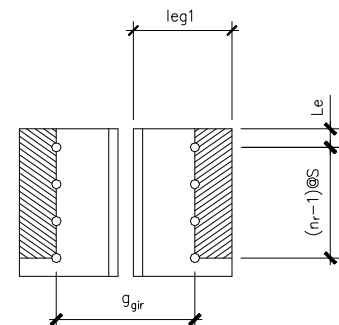
$$A_{gv} := n_{ca} \cdot [(nr - 1) \cdot s + Le_v] \cdot t_{ca}$$

$$A_{gv} = 7.25 \cdot \text{in}^2$$

Net Tension Area

$$A_{nt} := n_{ca} \cdot [leg1_{ca} - g_{cag} - (n_v - 0.5) \cdot hd_{gcah}] \cdot t_{ca}$$

$$A_{nt} = 0.98 \cdot \text{in}^2$$



Net Shear Area

$$A_{nv} := A_{gv} - n_{ca} \cdot (n_r - 0.5) \cdot h_{d_{gcav}} \cdot t_{ca}$$

$$A_{nv} = 5.062 \cdot \text{in}^2$$

Block Shear Capacity of Clip Angle, (J4-5)

$$R_{bs_{cag}} := \Lambda_{bs} \min(0.6 \cdot F_{u_{ca}} \cdot A_{nv} + U_{bs} \cdot F_{u_{ca}} \cdot A_{nt}, 0.6 \cdot F_{y_{ca}} \cdot A_{gv} + U_{bs} \cdot F_{u_{ca}} \cdot A_{nt})$$

$$R_{bs_{cag}} = 106.72 \cdot \text{kips}$$

$$V_{u_{bm}} = 15 \cdot \text{kips}$$

RESULT = Block Shear Capacity > Force Applied, OK

D. CLIP ANGLE TO GIRDER CHECK

1. Bolt Shear Capacity

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

Combined Shear & Tension Capacity per Bolt (J3-3a, J3-3b),

$$\Lambda_{Bv_{gir}} := \begin{cases} \Lambda_{rv} & \text{if } \frac{V_{u_{bm}}}{n_{ca} \cdot n} \leq 0.3 \cdot \Lambda_{rv} \vee \frac{P_{u_{bm}}}{n_{ca} \cdot n} \leq 0.3 \cdot \Lambda_{rn} \\ \min \left[1.3 \cdot \Lambda_{rv} - \frac{\Lambda_{rv}}{\Lambda_{rn}} \left(\frac{P_{u_{bm}}}{n_{ca} \cdot n} \right), \Lambda_{rv} \right] & \text{otherwise} \end{cases}$$

$$\Lambda_{Bv_{gir}} = 11.928 \cdot \text{kips}$$

Shear Capacity per Bolt,

$$\Lambda_{Rv_{gir}} := \begin{cases} \Lambda_{Bv_{gir}} & \text{if Conn_type} = \text{"Bearing-type"} \\ \text{if Conn_type} = \text{"Slip Critical-type"} \\ \left| \begin{array}{ll} \Lambda_{rv} \cdot \left(1 - \frac{1.5 \cdot P_{u_{bm}}}{1.13 \cdot T_b \cdot n_{ca} \cdot n} \right) & \text{if Code} = \text{"ASD"} \\ \Lambda_{rv} \cdot \left(1 - \frac{P_{u_{bm}}}{1.13 \cdot T_b \cdot n_{ca} \cdot n} \right) & \text{if Code} = \text{"LRFD"} \end{array} \right. & \end{cases} \quad \begin{array}{l} (J3-5b) \\ (J3-5a) \end{array}$$

$$\Lambda_{Rv_{gir}} = 11.928 \cdot \text{kips}$$



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Bolt Shear Capacity,

$$Rb_v := n_{ca} \cdot n \cdot \Lambda Rv_{gir}$$

$$Rb_v = 71.569 \cdot \text{kips}$$

$$Vu_{bm} = 15 \cdot \text{kips}$$

RESULT = Bolt Shear Capacity > Force Applied, OK

2. Bolt Tension Capacity with Prying of Angle Leg

(AISC 14th Ed. Specifications Chapter J, Section J3.7, pages 16.1-125 to 16.1-126)

Combined Shear & Tension Capacity per Bolt (J3-3a, J3-3b),

$$\Lambda B_{gir} := \begin{cases} \Lambda Rn & \text{if } \frac{Pu_{bm}}{n_{ca} \cdot n} \leq 0.3 \cdot \Lambda Rn \vee \frac{Vu_{bm}}{n_{ca} \cdot n} \leq 0.3 \cdot \Lambda Rv \\ \min \left[1.3 \cdot \Lambda Rn - \frac{\Lambda Rn}{\Lambda Rv} \left(\frac{Vu_{bm}}{n_{ca} \cdot n} \right), \Lambda Rn \right] & \text{otherwise} \end{cases}$$

$$\Lambda B_{gir} = 19.88 \cdot \text{kips}$$

$$\Lambda Rn_{gir} := \begin{cases} \Lambda Rn & \text{if Conn_type} = \text{"Slip Critical-type"} \\ \Lambda B_{gir} & \text{if Conn_type} = \text{"Bearing-type"} \end{cases}$$

$$\Lambda Rn_{gir} = 19.88 \cdot \text{kips}$$

Tension Strength of Bolt Under Pure Tension,

$$Nb1 := \Lambda Rn$$

$$Nb1 = 19.88 \cdot \text{kips}$$

Distance from bolt centerline to the center of angle leg,

$$b := 0.5(g_{gir} - tw_{bm} - tca)$$

$$b = 2.333 \cdot \text{in}$$

Distance of bolt centerline to edge of angle leg,

$$a := \min[leg1_{ca} - (b + 0.5 \cdot tca), 1.25 \cdot b]$$

$$a = 1.417 \cdot \text{in}$$



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Tributary length of angle,

$$p := \min(s, 2 \cdot b)$$

$$p = 3 \cdot \text{in}$$

$$b' := b - \frac{db}{2}$$

$$b' = 1.957 \cdot \text{in}$$

$$a' := a + \frac{db}{2}$$

$$a' = 1.792 \cdot \text{in}$$

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

$$\rho = 1.092$$

$$\delta := 1 - \frac{db + \frac{1}{16} \text{ in}}{p}$$

$$\delta = 0.729$$

$$t_c := \begin{cases} \sqrt{\frac{4.44 \cdot \Lambda Rn_{gir} \cdot b'}{p \cdot F_{u_{ca}}}} & \text{if Code} = \text{"LRFD"} \\ \sqrt{\frac{6.66 \cdot \Lambda Rn_{gir} \cdot b'}{p \cdot F_{u_{ca}}}} & \text{if Code} = \text{"ASD"} \end{cases}$$

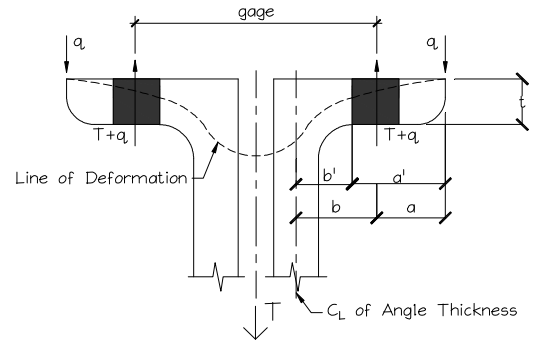
$$t_c = 1.22 \cdot \text{in}$$

$$\alpha' := \frac{1}{\delta \cdot (1 + \rho)} \cdot \left[\left(\frac{t_c}{t_{ca}} \right)^2 - 1 \right]$$

$$\alpha' = 3.25$$

Allowable Tensile Force per Bolt considering Prying Action,

$$Nb2 := \begin{cases} \Lambda Rn_{gir} \cdot \left(\frac{t_{ca}}{t_c} \right)^2 \cdot (1 + \delta) & \text{if } \alpha' > 1.0 \\ \Lambda Rn_{gir} \cdot \left(\frac{t_{ca}}{t_c} \right)^2 \cdot (1 + \delta \cdot \alpha') & \text{if } 0 \leq \alpha' \leq 1.0 \\ \Lambda Rn_{gir} & \text{if } \alpha' < 0 \end{cases}$$





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$$N_{b2} = 5.77 \cdot \text{kips}$$

3. Equivalent Tensile due to Moment Load in the Bolt Group

$$\text{Length of Column Clip Angle,} \quad L_{ca} = 8.5 \cdot \text{in}$$

$$\text{Total Bolt Area,} \quad A_b := \frac{\pi \cdot d_b^2}{4}$$

$$A_b = 0.442 \cdot \text{in}^2$$

$$\text{Effective Column Web Width,} \quad W_{eff} := 8 \cdot t_{ca} - t_{w_{bm}}$$

$$W_{eff} = 3.665 \cdot \text{in}$$

Actual Number of Bolts Under Tension,

$$n_{r_t} = 2$$

Location of Neutral Axis from the bottom of Clip Angle,

$$y_{cg} = 1.921 \cdot \text{in}$$

Distance of Bolts from the Centroid

$$y_1 = 5.329 \cdot \text{in} \quad y_4 = \blacksquare \cdot \text{in}$$

$$y_2 = 2.329 \cdot \text{in}$$

$$y_3 = -0.671 \cdot \text{in}$$



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Moment of Inertia of Bolt Group in Tension

$$I_{x_{ten}} := n_{ca} \cdot A_b \cdot \sum_{i=1}^{nr_t} (y_i)^2$$

$$I_{x_{ten}} = 29.879 \cdot \text{in}^4$$

Outermost Bolt Location in Tension,

$$y_{max} = 5.329 \cdot \text{in}$$

Moment of Inertia of Compression Area,

$$I_{x_{comp}} := \frac{W_{eff} \cdot y_{cg}^3}{3}$$

$$I_{x_{comp}} = 8.666 \cdot \text{in}^4$$

Elastic Section Modulus at Bolt Group in Tension,

$$S_{x_{ten}} := \frac{I_{x_{ten}} + I_{x_{comp}}}{y_{max}}$$

$$S_{x_{ten}} = 7.234 \cdot \text{in}^3$$

Elastic Section Modulus at Compression Area,

$$S_{x_{comp}} := \frac{I_{x_{ten}} + I_{x_{comp}}}{y_{cg}}$$

$$S_{x_{comp}} = 20.061 \cdot \text{in}^3$$

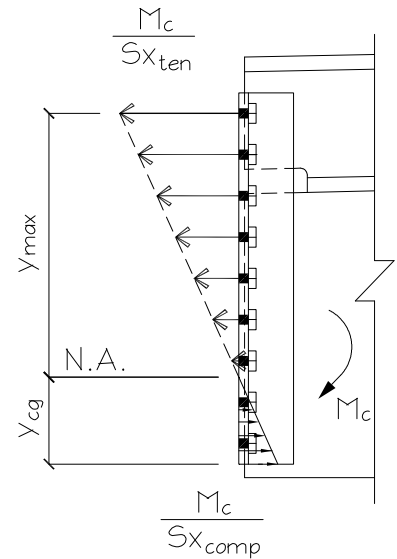
Eccentricity distance of Axial Load from bolt group centerline,

$$y_o := a_h$$

$$y_o = 0.05 \cdot \text{in}$$

Equivalent Tensile Force due to Moment Load,

$$T_M := \frac{P_{ubm} \cdot y_o}{S_{x_{ten}}} \cdot A_b$$





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$$T_M = 0.046 \cdot \text{kips}$$

Total Tensile Force on Farthest Bolt,

$$T_B := \max \left(\frac{P_{u_{bm}}}{n_{ca} \cdot n} + T_M, 0.001 \text{kips} \right)$$

$$T_B = 2.546 \cdot \text{kips}$$

Minimum thickness to eliminate prying action,

$$t_{\min} := \begin{cases} \sqrt{\frac{4.44 \cdot T_B \cdot b'}{p \cdot F_{u_{ca}}}} & \text{if Code} = \text{"LRFD"} \\ \sqrt{\frac{6.66 \cdot T_B \cdot b'}{p \cdot F_{u_{ca}}}} & \text{if Code} = \text{"ASD"} \end{cases}$$

$$t_{\min} = 0.437 \cdot \text{in}$$

$$t_{ca} = 0.5 \cdot \text{in}$$

RESULT = Consider Thick Connector Design

Applicability of Prying,

$$\text{Prying}_{ca} := \begin{cases} \text{"Applicable"} & \text{if } t_{ca} < t_{\min} \wedge R_{wb_{gir}} \geq 10 \cdot P_{u_{bm}} \\ \text{"Not Applicable"} & \text{otherwise} \end{cases}$$

$$\text{Prying}_{ca} = \text{"Not Applicable"}$$

Tensile Capacity Per Bolt,

$$N_{b_{ca}} := \begin{cases} \min(N_{b1}, N_{b2}) & \text{if } \text{Prying}_{ca} = \text{"Applicable"} \\ N_{b1} & \text{otherwise} \end{cases}$$

$$N_{b_{ca}} = 19.88 \cdot \text{kips}$$

$$T_B = 2.546 \cdot \text{kips}$$

RESULT = Bolt Tensile Capacity > Force Applied, OK

4. Check Angle Thickness for Prying Action

Minimum thickness to ensure an acceptable combination of fitting strength and stiffness and bolt strength considering the effects of prying action,

$$\rho = 1.092$$



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$$\delta = 0.729$$

$$\beta := \frac{1}{\rho} \cdot \left(\frac{\Lambda Rn_{gir}}{T_B} - 1 \right)$$

$$\beta = 6.235$$

$$\alpha'_t := \begin{cases} 1 & \text{if } \beta \geq 1 \\ \min \left[\frac{1}{\delta} \cdot \left(\frac{\beta}{1 - \beta} \right), 1 \right] & \text{otherwise} \end{cases}$$

$$\alpha'_t = 1$$

$$b' = 1.957 \cdot \text{in}$$

$$tca_{\min} := \begin{cases} \sqrt{\frac{4.44 T_B \cdot b'}{p \cdot F_{u_{ca}} \cdot (1 + \delta \cdot \alpha'_t)}} & \text{if Code} = \text{"LRFD"} \\ \sqrt{\frac{6.66 T_B \cdot b'}{p \cdot F_{u_{ca}} \cdot (1 + \delta \cdot \alpha'_t)}} & \text{if Code} = \text{"ASD"} \end{cases}$$

$$tca_{\min} = 0.332 \cdot \text{in}$$

$$tca = 0.5 \cdot \text{in}$$

RESULT = Prying Limit State Not Applicable

E. GIRDER CHECK

1. Bolt Bearing Capacity on Girder

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

Bearing Area,

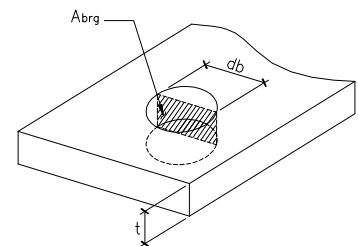
$$A_{brg_{gir}} := d_b \cdot t_{w_{gir}}$$

$$A_{brg_{gir}} = 0.386 \cdot \text{in}^2$$

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

$$F_{be} := \Lambda_{brg} \cdot F_{u_{gir}} \cdot \begin{cases} 2.0 \cdot A_{brg_{gir}} & \text{if } h_{d_{gir}} \geq h_{d_{ls}} \\ 2.4 \cdot A_{brg_{gir}} & \text{otherwise} \end{cases}$$

$$F_{be} = 30.127 \cdot \text{kips}$$



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

$$F_{bs} := \Lambda_{brg} \cdot F_{u_{gir}} \cdot \begin{cases} \min[1.0 \cdot (s - h_{d_{gir}}) \cdot t_{w_{gir}}, 2.0 \cdot A_{brg_{gir}}] & \text{if } h_{d_{gir}} \geq h_{d_{ls}} \\ \min[1.2 \cdot (s - h_{d_{gir}}) \cdot t_{w_{gir}}, 2.4 \cdot A_{brg_{gir}}] & \text{otherwise} \end{cases}$$

$$F_{bs} = 30.127 \cdot \text{kips}$$

Bolt Bearing Capacity,

$$R_{brg_{gir}} := n_{ca} n_v \cdot [\min(F_{be}, \Lambda R_{v_{gir}}) + \min(F_{bs}, \Lambda R_{v_{gir}}) \cdot (n_r - 1)]$$

$$R_{brg_{gir}} = 71.569 \cdot \text{kips}$$

$$V_{u_{bm}} = 15 \cdot \text{kips}$$

RESULT = Bearing Capacity > Force Applied, OK

2. Bending Capacity of Girder Web Using Yield Line Analysis

(Yield Line Theory: (Reference: Yield Line of a Web Connection in Direct Tension by Richard H. Kapp, page 39)

Distance Between Inner Yield Lines, Along the Member

$$L_{yla} := g_{gir}$$

$$L_{yla} = 5.5 \cdot \text{in}$$

Distance Between Inner Yield Lines, Across the Member

$$c := (n_r - 1) \cdot s$$

$$c = 6 \cdot \text{in}$$

Width of Girder Web in Bending,

$$T := d_{gir} - 2 \cdot k_{des_{gir}}$$

$$T = 9.68 \cdot \text{in}$$

Distance from Inner Yield Line to Outer Yield Line, Across the Member

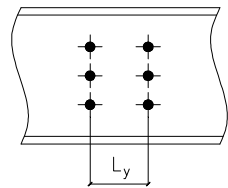
$$b := 0.5 (T - c)$$

$$b = 1.84 \cdot \text{in}$$

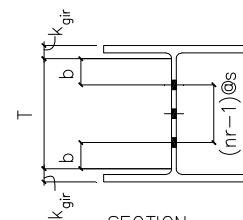
Internal Work for Web Fixed at Flanges,

$$\psi := F_{y_{gir}} \cdot t_{w_{gir}}^2 \quad (\text{Eq. 2})$$

$$\psi = 13.261 \cdot \text{kips}$$



ELEVATION



SECTION



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Bending Capacity of Girder Web,

$$R_{wb_gir} := \Lambda_b \cdot \left(\frac{\psi \cdot L_{yla}}{b} + 4 \cdot \psi \cdot \sqrt{1 + \frac{c}{2 \cdot b}} \right) \quad (Eq. 5)$$

$$R_{wb_gir} = 75.252 \cdot \text{kips}$$

$$P_{u_bm} = 15 \cdot \text{kips}$$

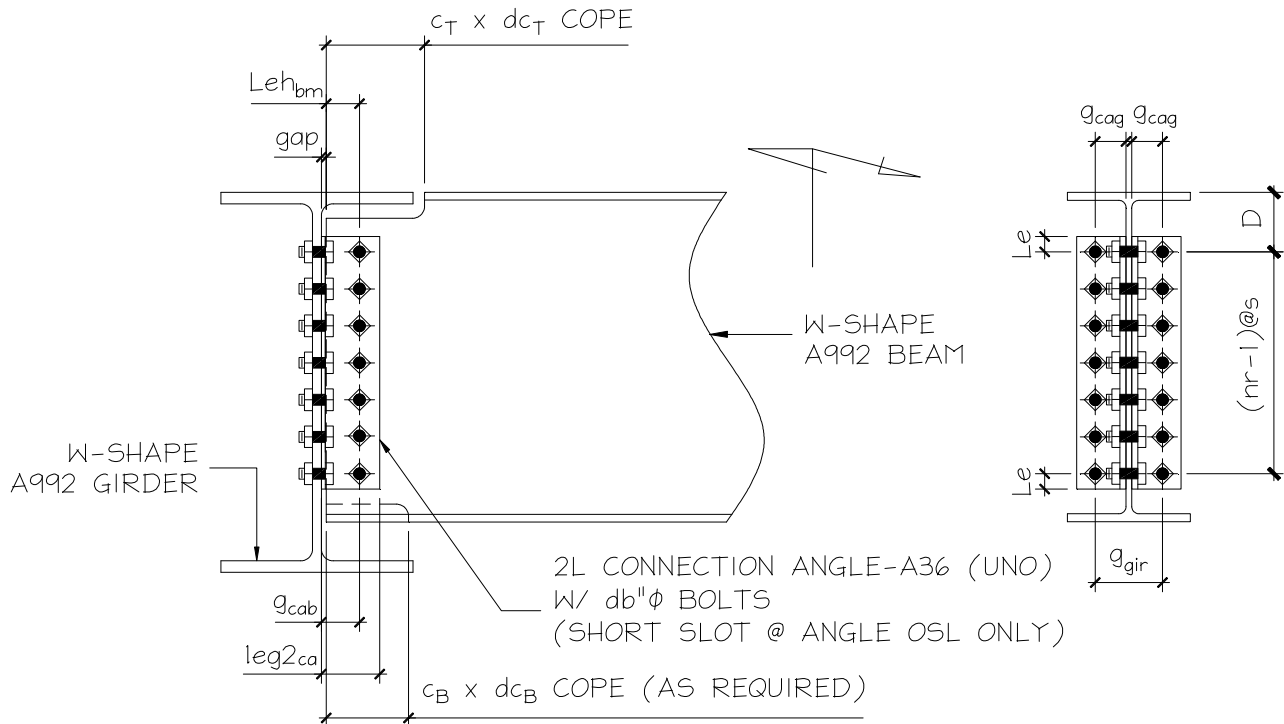
RESULT = Web bending Capacity > Applied Force, OK



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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 WITH AXIAL CONNECTION: DETAIL OF W-SHAPE BEAM TO W-SHAPE GIRDER CLIP ANGLE CONNECTION (BOLTED-BOLTED)



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B. TABLE: SHEAR WITH AXIAL CONNECTION SCHEDULE

Girder	Beam	Connection Angle		g _{gir} (in)	D (in)	Bolts			Beam Shear Load (kips)	Beam Axial Load (kips)
		Size	Grade			db (in)	Bolt Type	nr		
W12X87	W12X45	2L4X4X1/2	A36	5.5	3	3/4	A325-N	3	15.0	15.0

Girder	Beam	gap (in)	Cope Dimensions			
			dc _T (in)	c _T (in)	dc _B (in)	c _B (in)
W12X87	W12X45	1/2	1 1/2	6	1 1/2	6

Note:

1. LLBB- Long Leg Back to Back
2. SLBB - Short Leg Back to Back
3. NR - Not Required

IV. REFERENCES

Steel Construction Manual (14th) - ASD American Institute of Steel Construction, Inc. 2011