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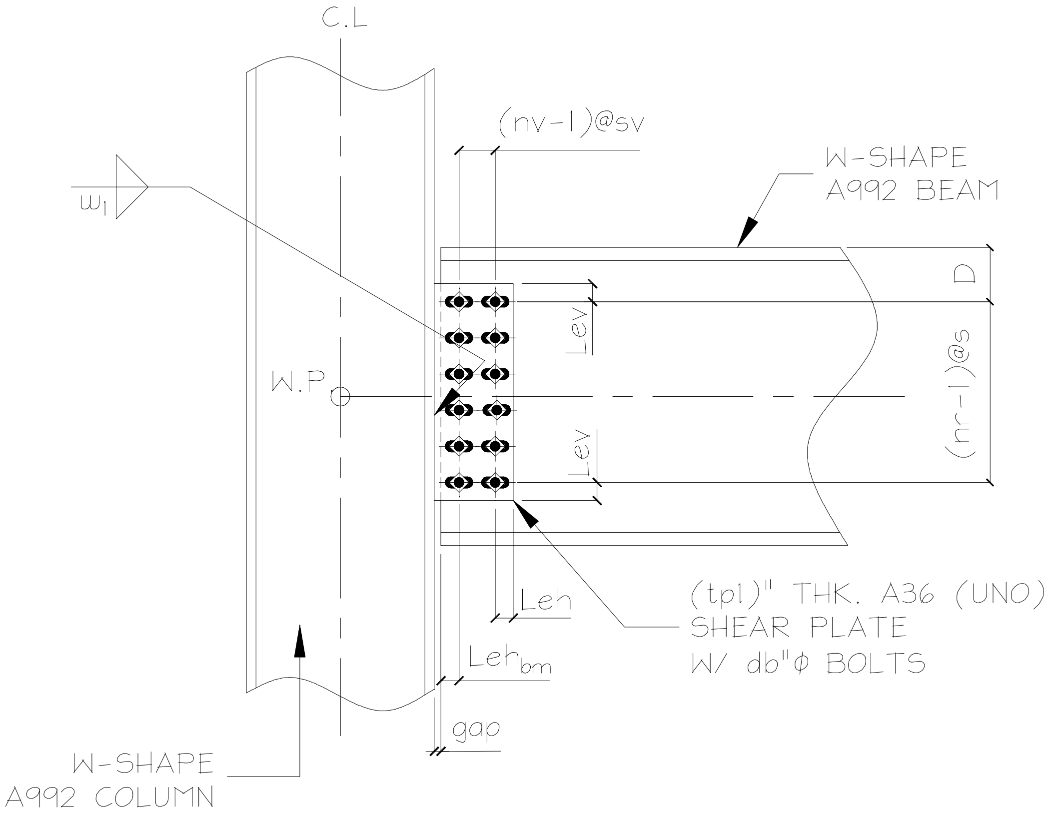
SHEAR CONNECTION: DESIGN OF W-SHAPE BEAM TO W-SHAPE COLUMN FLANGE SHEAR PLATE CONNECTION



NOTE: (FIGURE ABOVE DOES NOT REPRESENT ACTUAL DESIGN, REFER ON ATTACHED CONNECTION SCHEDULE FOR THICKNESS OF PLATE, LENGTH AND SIZE OF WELDS)

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

COLUMN PROPERTIES (col): W18X35 - A992

$Fy_{col} = 50 \cdot ksi$	$d_{col} = 17.7 \cdot in$	$tw_{col} = 0.3 \cdot in$	$k1_{col} = 0.75 \cdot in$
$Fu_{col} = 65 \cdot ksi$	$bf_{col} = 6 \cdot in$	$tf_{col} = 0.425 \cdot in$	$k_{col} = 1.125 \cdot in$
$Ag_{col} = 10.3 \cdot in^2$	$Sx_{col} = 57.6 \cdot in^3$	$E := 29000ksi$	

BEAM PROPERTIES (bm): W21X68 - A992

$Fy_{bm} = 50 \cdot ksi$	$d_{bm} = 21.1 \cdot in$	$tw_{bm} = 0.43 \cdot in$	$k1_{bm} = 0.875 \cdot in$
$Fu_{bm} = 65 \cdot ksi$	$bf_{bm} = 8.27 \cdot in$	$tf_{bm} = 0.685 \cdot in$	$k_{bm} = 1.375 \cdot in$
$Ag_{bm} = 20 \cdot in^2$	$Sx_{bm} = 140 \cdot in^3$	*Length of Beam,*	$L_{bm} := 15ft + 0in$

SHEAR PLATE (pl): A36

$Fy_{pl} = 36 \cdot ksi$	$Fu_{pl} = 58 \cdot ksi$	*Thickness of Plate:*	$tpl := \frac{3}{4} in$

BOLTS:

For Shear Plate to Beam Connection:

Bolt Diameter,	$db = 1 \cdot in$	Bolt_Type = "A490-N"	
Bolt Shear Strength,	$\Lambda rv = 40.055 \cdot kips$	Conn_type = "Bearing-type"	
Bolt Tensile Strength,	$\Lambda rn = 66.562 \cdot kips$	*Hole diameter:*	
Beam Edge Distance,	$Leh_{bm} = 1.5 \cdot in$	*Shear Plate,*	
Plate Vertical Edge Distance,	$Lev = 1.5 \cdot in$	$hd_{plv} = 1.125 \cdot in$	$hd_{plh} = 1.375 \cdot in$
		Beam,	
Plate Horizontal Edge Distance,	$Leh = 1.5 \cdot in$	$hd_{bm} = 1.125 \cdot in$	
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,	$D = 3 \cdot in$		

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

number of bolt rows: $nr := 6$

number of vertical bolt lines: $nv := 2$

total number of bolts: $n := nr \cdot nv$ $n = 12$

WELDS: E70xx LH

$Fu_w = 70 \cdot ksi$

Preferred Weld Size

Shear Plate to Column, $w_1 = \frac{1}{2} \cdot in$

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 tension rupture,	$\Omega_{tr} = 2.00$	$\phi_{tr} = 0.75$	$\Lambda_{tr} = 0.75$
For tension yielding,	$\Omega_{ty} = 1.67$	$\phi_{ty} = 0.9$	$\Lambda_{ty} = 0.90$
For compression,	$\Omega_c = 1.67$	$\phi_c = 0.9$	$\Lambda_c = 0.90$
For member shear (C, WT, L)	$\Omega_v = 1.67$	$\phi_v = 0.9$	$\Lambda_v = 0.90$
For fillet weld (shear),	$\Omega_{vw} = 2.00$	$\phi_{vw} = 0.75$	$\Lambda_{vw} = 0.75$
For shear rupture,	$\Omega_{vr} = 2.00$	$\phi_{vr} = 0.75$	$\Lambda_{vr} = 0.75$
For shear yielding,	$\Omega_{vy} = 1.50$	$\phi_{vy} = 1.00$	$\Lambda_{vy} = 1.00$
For block shear,	$\Omega_{bs} = 2.00$	$\phi_{bs} = 0.75$	$\Lambda_{bs} = 0.75$
For member/bolt in bearing,	$\Omega_{brg} = 2.00$	$\phi_{brg} = 0.75$	$\Lambda_{brg} = 0.75$
For web crippling,	$\Omega_{cr} = 2.00$	$\phi_{cr} = 0.75$	$\Lambda_{cr} = 0.75$
For member shear yielding (W,S,M,HSS)	$\Omega_{wy} = 1.50$	$\phi_{wy} = 1.00$	$\Lambda_{wy} = 1.00$
For flexural rupture,	$\Omega_{fr} = 2.00$	$\phi_{fr} = 0.75$	$\Lambda_{fr} = 0.75$

APPLIED LOAD:

% UDL, $UDL := 0.5$

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

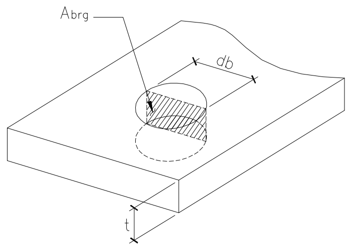
Beam Shear Load, $V = 160 \cdot kips$ **50% UDL**

II. CALCULATIONS:

A. BEAM CHECK

1. Bolt Bearing Capacity on Beam

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



Bearing Area,

$$Abrg_{bm} := db \cdot tw_{bm}$$

$$Abrg_{bm} = 0.43 \cdot in^2$$

Bolt centerline distance from face of support,

$$a_b := gap + Leh_{bm} + 0.5\,(nv - 1) \cdot sv$$

$$a_b = 3.5 \cdot in$$

Eccentric Load Coefficient,

(Table 7-7, AISC 14th Ed.)

$$C = 9.419$$

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

$$Fbe := \Lambda_{brg} \cdot Fu_{bm} \cdot \left| \begin{array}{ll} \min\left(\begin{bmatrix} 1.0 \cdot \left(D - 0.5 \cdot hd_{bm}\right) \cdot tw_{bm} \\ 1.0 \cdot \left(Leh_{bm} - 0.5 \cdot hd_{bm}\right) \cdot tw_{bm} \\ 2.0 \cdot Abrg_{bm} \end{bmatrix}\right) & \text{if } hd_{bm} \geq hd_{ls} \\ \min\left(\begin{bmatrix} 1.2 \cdot \left(D - 0.5 \cdot hd_{bm}\right) \cdot tw_{bm} \\ 1.2 \cdot \left(Leh_{bm} - 0.5 \cdot hd_{bm}\right) \cdot tw_{bm} \\ 2.4 \cdot Abrg_{bm} \end{bmatrix}\right) & \text{otherwise} \end{array} \right.$$

$$Fbe = 23.583 \cdot kips$$

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

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

$$Fbs = 47.166 \cdot kips$$

Bolt Bearing Capacity,

$$Rbrg_{bm} := C \cdot \min(Fbe, Fbs, \Lambda rv)$$

$$Rbrg_{bm} = 222.117 \cdot kips \qquad V = 160 \cdot kips$$

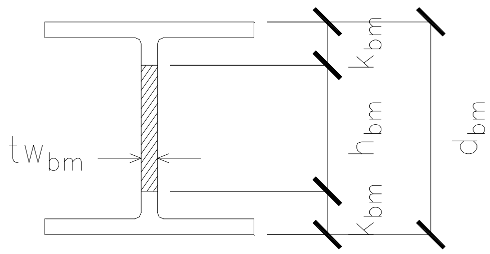
RESULT = Bearing Capacity > Force Applied,OK

2. 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,

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



$$h = 18.72 \cdot in$$

Limiting depth-thickness ratio,

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

$$h_{tw} = 43.535$$

Clear distance between transverse stiffeners,

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

$$a = 0 \cdot in$$

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Web plate buckling coefficient,

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

$kv = 5$

Web shear coefficient,

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

$Cv = 1$

Shear Capacity of Section,

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

$Rv_{bm} = 272.19 \cdot kips$ $\qquad V = 160 \cdot kips$

RESULT = Shear Capacity of Section > Force Applied, OK

B. BEAM TO SHEAR PLATE CHECK

1. Eccentric Bolt Shear Capacity

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

Shear Capacity per bolt,

$\Lambda rv = 40.055 \cdot kips$

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Eccentric Bolt Capacity,

$Reb := C \cdot \Lambda rv$

$Reb = 377.265 \cdot kips$ $V = 160 \cdot kips$

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,

$s = 3 \cdot in$

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

$s_{min} = 2.667 \cdot in$

$s_{max} := \min\left(12in, 24 \cdot \min\left(tw_{bm}, tpl\right)\right)$

$s_{max} = 10.320 \cdot in$

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

Horizontal Spacing,

$sv = 3 \cdot in$

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

$sv_{min} = 2.667 \cdot in$

$sv_{max} := \min\left(12in, 24 \cdot \min\left(tw_{bm}, tpl\right)\right)$

$sv_{max} = 10.320 \cdot in$

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

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,

$Lev = 1.5 \cdot in$

$Le_{min} = 1.25 \cdot in$

$C_2 = 0 \cdot in$

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

$Lev_{min} = 1.25 \cdot in$

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

$Lev_{max} = 6.000 \cdot in$

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

Horizontal Edge Distance,

$Leh = 1.5 \cdot in$

$Leh_{bm} = 1.5 \cdot in$

$Le_{min} = 1.25 \cdot in$

$Leh_{minpl} = 1.375 \cdot in$

$Leh_{minbm} = 1.25 \cdot in$

$Leh_{maxpl} := \min(6in, 12 \cdot tpl)$

$Leh_{maxpl} = 6.000 \cdot in$

$Leh_{maxbm} := \min\left(6in, 12 \cdot tw_{bm}\right)$

$Leh_{maxbm} = 5.160 \cdot in$

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

C. SHEAR PLATE CHECK

1. Check for Maximum Shear Plate Thickness

(AISC 14th Ed. Manual Part 10, page 10-104)

Exceptions for nv = 1 and nv = 2

$tpl \le \frac{db}{2} + \frac{1}{16}$

$Leh_{pl} \ge 2 \cdot db_{pl}$

$tw_{bm} \le \frac{db}{2} + \frac{1}{16}$

$Leh_{bmw} \ge 2 \cdot db_{pl}$

RESULT = Check maximum thickness of plate

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Coefficient for Eccentrically Loaded Bolts

(AISC 14th Ed. Manual Part 7, page 7-19)

$C' = 54.167 \cdot in$

Area of Bolts

$$Ab := \frac{\pi \cdot db^2}{4}$$

$Ab = 0.785 \cdot in^2$

Length of Plate

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

$Lpl = 18 \cdot in$

Maximum Thickness

$$tpl_{max} := \frac{6 \cdot \left(\frac{F_{nv1}}{0.9} \cdot Ab \cdot C' \right)}{Fy_{pl} \cdot Lpl^2} \qquad (10\text{-}3)$$

$tpl_{max} = 1.653 \cdot in$ $\qquad$ $tpl = 0.75 \cdot in$

RESULT = Plate Thickness < Max Thickness Permitted,OK

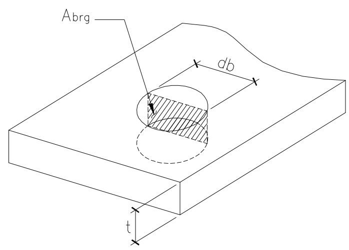
Governing Shear Plate Thickness,

$$tpl_g := \begin{cases} \text{if } Case_{pl} = 1 \\ \quad \begin{cases} tpl & \text{if } tpl < tpl_{max} \\ tpl & \text{if } tpl = tpl_{max} \\ \text{Floor}\left(tpl_{max}, \frac{1}{16} in \right) & \text{otherwise} \end{cases} \\ tpl \quad \text{otherwise} \end{cases}$$

$tpl_g = \frac{3}{4} \cdot in$

2. Bolt Bearing Capacity of Shear Plate

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



Bearing Area,

$$Abrg_{pl} := db \cdot tpl_g$$

$$Abrg_{pl} = 0.75 \cdot in^2$$

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

$$Fbe := \Lambda_{brg} \cdot Fu_{pl} \cdot \left| \begin{array}{ll} \min\left[\begin{bmatrix} 1.0 \cdot (Lev - 0.5hd_{plv}) \cdot tpl_g \\ 1.0 \cdot (Leh - 0.5hd_{plh}) \cdot tpl_g \\ 2.0 \cdot Abrg_{pl} \end{bmatrix}\right] & \text{if } hd_{plh} \geq hd_{ls} \\ \min\left[\begin{bmatrix} 1.2 \cdot (Lev - 0.5hd_{plv}) \cdot tpl_g \\ 1.2 \cdot (Leh - 0.5hd_{plh}) \cdot tpl_g \\ 2.4 \cdot Abrg_{pl} \end{bmatrix}\right] & \text{otherwise} \end{array} \right.$$

$$Fbe = 31.809 \cdot kips$$

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

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

$$Fbs = 73.406 \cdot kips$$

Bolt Bearing Capacity,

$$Rbrg_{pl} := C \cdot \min(Fbe, Fbs, \Lambda rv)$$

$$Rbrg_{pl} = 299.6 \cdot kips \qquad V = 160 \cdot kips$$

RESULT = Bearing Capacity > Force Applied, OK

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3. Shear Yielding Capacity of Shear Plate

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

Length of Plate,

$Lpl := (nr - 1)\,s + 2Lev$

$Lpl = 18 \cdot in$

Check if Length of Plate is acceptable,

(AISC 14th Ed, Manual Part 10, page 10-106)

$Length := \left| \begin{array}{ll} \text{"Plate Length is OK per AISC Requirements"} & \text{if } Lpl \geq 0.5\left(d_{bm} - 2k_{bm}\right) \\ \text{"Increase Plate Length per AISC Requirements"} & \text{otherwise} \end{array} \right.$

$Length =$ "Plate Length is OK per AISC Requirements"

Gross Shear Capacity,

$Rvy_{pl} := \Lambda_{vy} \cdot 0.6 \cdot Fy_{pl} \cdot tpl_{g} \cdot Lpl$ *(J4-3)*

$Rvy_{pl} = 291.6 \cdot kips$ $V = 160 \cdot kips$

RESULT = Shear Yielding Capacity > Force Applied, OK

4. Shear Rupture Capacity of Shear Plate

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

Net Area,

$Anv := \left(Lpl - nr \cdot hd_{plv}\right) \cdot tpl_{g}$

$Anv = 8.437 \cdot in^{2}$

Shear Rupture Capacity,

$Rvr_{pl} := \Lambda_{vr} \cdot 0.6 \cdot Fu_{pl} \cdot Anv$ *(J4-4)*

$Rvr_{pl} = 220.219 \cdot kips$ $V = 160 \cdot kips$

RESULT = Shear Rupture Capacity > Force Applied, OK

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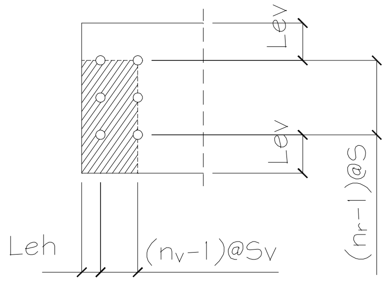
5. Block Shear Capacity of Shear Plate

(AISC 14th Ed. Specifications 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)

$U_{bs} = 0.5$



Gross Shear Area

$Agv := [(nr - 1) \cdot s + Lev] \cdot tpl_g$

$Agv = 12.375 \cdot in^2$

Net Tension Area

$Ant := [Leh + (nv - 1) \cdot sv - (nv - 0.5) \cdot hd_{plh}] \cdot tpl_g$

$Ant = 1.828 \cdot in^2$

Net Shear Area

$Anv := [(nr - 1) \cdot s + Lev - (nr - 0.5) \cdot hd_{plv}] \cdot tpl_g$

$Anv = 7.734 \cdot in^2$

Block Shear Capacity of Plate, (J4-5)

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

$Rbs_{pl} = 240.237 \cdot kips$ $V = 160 \cdot kips$

RESULT = Block Shear Capacity > Force Applied, OK

6. Local Buckling Capacity of Shear Plate

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

Distance of bolt line to support,

$a_b := gap + Leh_{bm}$

$a_b = 2 \cdot in$

Coefficient,

$$\lambda := \frac{Lpl \cdot \sqrt{Fy_{pl}}}{10 \cdot tpl_g \cdot \sqrt{475 + 280\left(\frac{Lpl}{a_b}\right)^2}} \cdot \left(\frac{1}{\sqrt{ksi}}\right)$$

$$\lambda = 0.095$$

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

$$Q = 1$$

Allowable Buckling Stress,

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

$$Fcr = 36 \cdot ksi$$

Gross Plastic Section Modulus,

$$Zx_{pl} := \left(\frac{tpl_g \cdot Lpl^2}{4}\right)$$

$$Zx_{pl} = 60.75 \cdot in^3$$

Eccentricity,

$$e_{pl} := a_b$$

$$e_{pl} = 2 \cdot in$$

Buckling Capacity,

$$Rbc_{pl} := \Lambda_b \cdot \frac{Fcr \cdot Zx_{pl}}{e_{pl}}$$

$Rbc_{pl} = 984.15 \cdot kips$ $\qquad V = 160 \cdot kips$

RESULT = Local Buckling Capacity will not Control!

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7. Flexural Yielding Capacity with von-Mises shear reduction

(AISC 14th Ed., Manual Part 10, page 10-103/Muir, Larry and Hewitt,Christopher, "Design of Unstiffened Extended Single-Plate Shear Connections", Engineering Journal, 2nd Quarter 2009, page 69.)

Flexural Capacity,

$$Rfc_{pl} := \frac{\Lambda_b \cdot Fy_{pl} \cdot Lpl \cdot tpl_g}{\sqrt{2.25 + 16 \cdot \left(\frac{e_{pl}}{Lpl}\right)^2}}$$

$Rfc_{pl} = 279.586 \cdot kips$ $\qquad V = 160 \cdot kips$

RESULT = Flexural Yielding Capacity > Applied Force, OK

8. Flexural Rupture Capacity

(AISC 14th Ed., Steel Construction Manual Design Examples page IIA-104)

Net Plastic Section Modulus,

$$Znet_{pl} := \begin{cases} \left[\frac{tpl \cdot Lpl^2}{4} - \frac{hd_{plv} \cdot s \cdot tpl \cdot \left(nr^2 - 1\right)}{4} - \frac{tpl \cdot \left(hd_{plv}\right)^2}{4}\right] & \text{if } mod(nr, 2) > 0 \\ \frac{tpl \cdot Lpl^2}{4} - \frac{hd_{plv} \cdot nr^2 \cdot s \cdot tpl}{4} & \text{if } mod(nr, 2) = 0 \end{cases}$$

$Znet_{pl} = 37.969 \cdot in^3$

Flexural Rupture Capacity,

(AISC 14th Ed., Manual Part 15, page 15-4)

$$Rfr_{pl} := \frac{\Lambda_{fr} \cdot Fu_{pl} \cdot Znet_{pl}}{e_{pl}}$$

$Rfr_{pl} = 825.82 \cdot kips$ $\qquad V = 160 \cdot kips$

RESULT = Flexural Rupture Capacity > Applied Force, OK

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9. Interaction of Shear Yielding, Shear Buckling, and Flexural Yielding of Plate

(AISC 14th Ed. Manual Part 10, page 10-104 to 10-105)

From AISC Manual Equation 10-5,

$$\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{M_r}{M_c}\right)^2 \le 1.0$$

$$V_r := V$$

$$V_r = 160 \cdot kips$$

$$M_r := V_r \cdot \left[gap + Leh_{bm} + 0.5\,(nv - 1) \cdot sv\right]$$

$$M_r = 560 \cdot kips \cdot in$$

Shear yielding,

$$V_c := \Lambda_{vy} \cdot 0.6 \cdot Fy_{pl} \cdot tpl_g \cdot Lpl$$

$$V_c = 291.6 \cdot kips$$

Flexural yielding,

$$M_c := \Lambda_b \cdot Fy_{pl} \cdot Zx_{pl}$$

$$M_c = 1968.3 \cdot kips \cdot in$$

Interaction,

$$\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{M_r}{M_c}\right)^2 = 0.382$$

RESULT = Interaction < 1.0, OK

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D. SHEAR PLATE TO COLUMN CHECK

1. Weld Check for Shear Plate to Column Flange

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

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

Minimum weld size,

$w_{min1} = \frac{1}{2} \cdot in$ $w_1 = \frac{1}{2} \cdot in$

RESULT = Preferred Weld Size = Minimum Weld Size, OK

(AISC 14th Ed. Specifications Chapter J, page 16.1-111)

Maximum Weld Size,

$$w_{max} := \begin{cases} tpl - \frac{1}{16} in & \text{if } tpl \geq \frac{1}{4} in \\ tpl & \text{otherwise} \end{cases}$$

$w_{max} = \frac{11}{16} \cdot in$ $w_1 = \frac{1}{2} \cdot in$

RESULT = Maximum Weld > Preferred Weld, OK

E. COLUMN CHECK

1. Rupture Strength at Weld for Column Flange

Length of Weld,

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

$Lw = 18 \cdot in$

Rupture Strength at Weld,

$Rv_{col} := \Lambda_{vr} \cdot 0.6 \cdot Fu_{col} \cdot tf_{col} \cdot n_{ws} \cdot Lw$

$Rv_{col} = 447.525 \cdot kips$ $V = 160 \cdot kips$

RESULT = Column Flange Capacity > Force Applied, OK.

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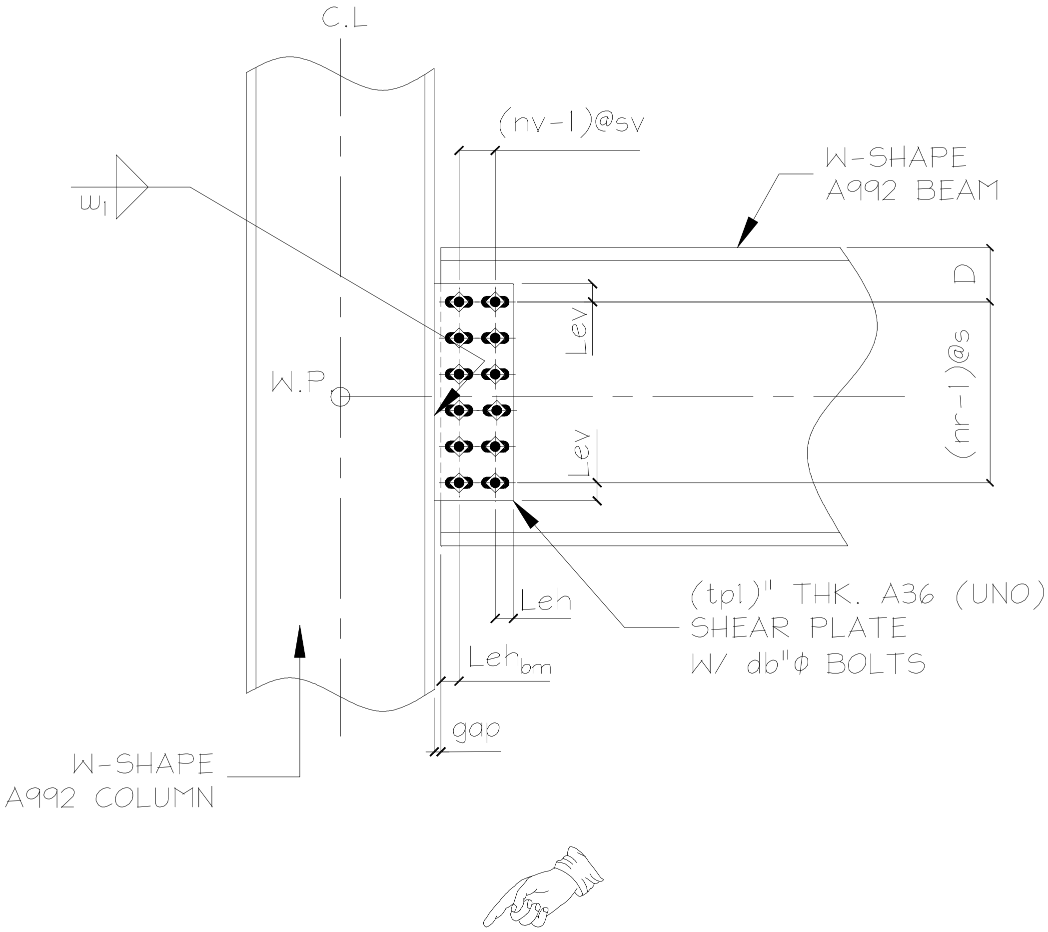
III. DETAILS:

A. SKETCH



NOTE: (FIGURE ABOVE DOES NOT REPRESENT ACTUAL DESIGN, REFER ON ATTACHED CONNECTION SCHEDULE FOR THICKNESS OF PLATE, LENGTH AND SIZE OF WELDS)

SHEAR CONNECTION: DETAIL OF W-SHAPE BEAM TO W-SHAPE COLUMN FLANGE SHEAR PLATE CONNECTION

B. TABLE: SHEAR CONNECTION SCHEDULE

Column		D (in)	Shear Plate		Bolts at Shear Plate				Bolt Spacing		Edge Distance	
Size	Grade		tpl (in)	Grade	db (in)	Type	nr	nv	s (in)	sv (in)	Lev (in)	Leh (in)
W18X35	A992	3	3/4	A36	1	A490-N	6	2	3	3	1 1/2	1 1/2

Beam		Edge Distance	gap (in)	Weld Size	Beam Shear Load (kips)	Rcap (kips)	Governing Capacity
Size	Grade	Leh_{bm} (in)		w_1 (in)			
W21X68	A992	1 1/2	1/2	1/2	160.00	220.22	Shear Rupture of Plate

IV. REFERENCES

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