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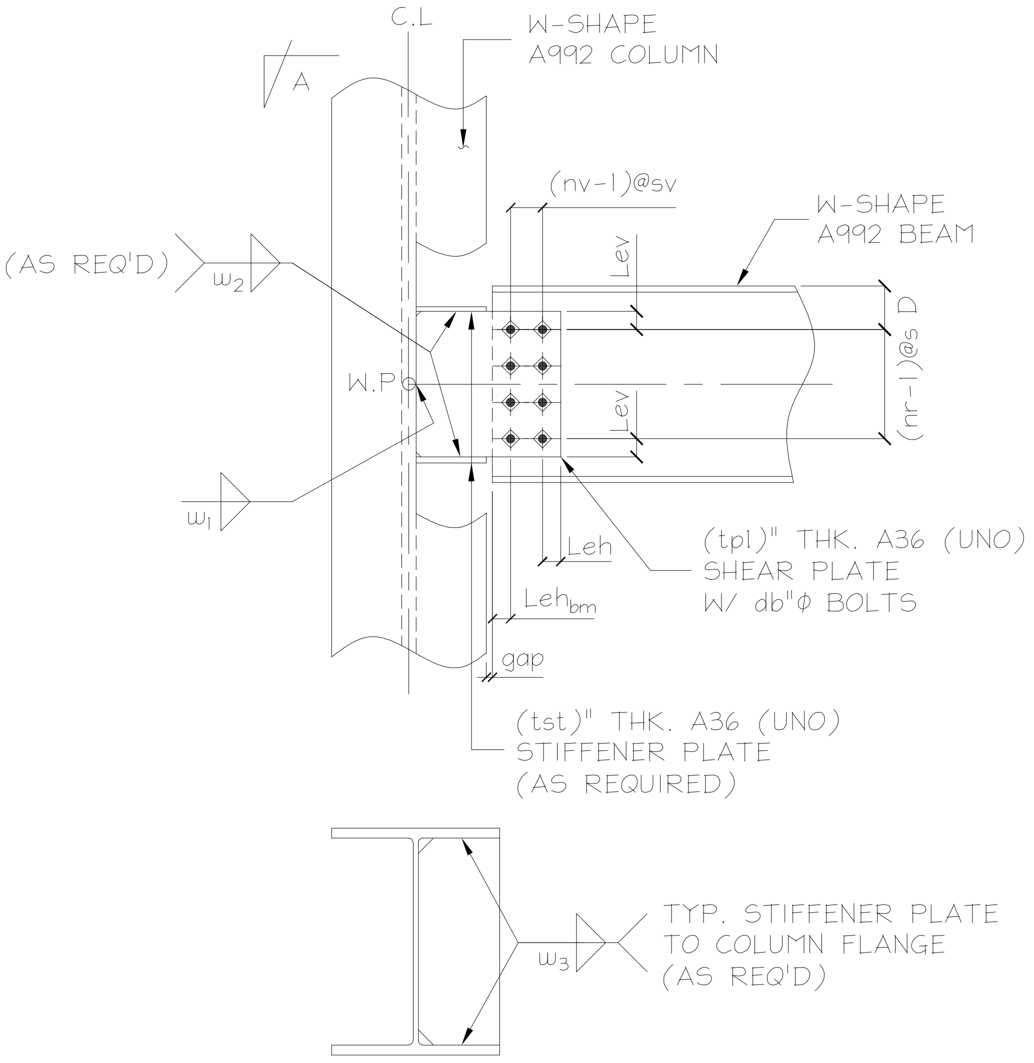
SHEAR CONNECTION: DESIGN OF W-SHAPE BEAM TO W-SHAPE COLUMN WEB SHEAR PLATE CONNECTION (FULL-DEPTH EXTENDED SHEAR PLATE)



SECTION A

NOTE: (FIGURE ABOVE DOES NOT REPRESENT ACTUAL DESIGN, REFER ON ATTACHED BRACE CONNECTION SCHEDULE FOR THICKNESS OF PLATE, LENGTH AND OF SIZE OF WELD, NUMBER OF BOLTS

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

COLUMN PROPERTIES (col): W14X68 - A992

$Fy_{col} = 50 \cdot ksi$ $d_{col} = 14 \cdot in$ $tw_{col} = 0.415 \cdot in$ $k1_{col} = 1.063 \cdot in$

$Fu_{col} = 65 \cdot ksi$ $bf_{col} = 10 \cdot in$ $tf_{col} = 0.72 \cdot in$ $k_{col} = 1.563 \cdot in$

$Ag_{col} = 20 \cdot in^2$ $Sx_{col} = 103 \cdot in^3$ $E := 29000ksi$

BEAM PROPERTIES (bm): W18X86 - A992

$Fy_{bm} = 50 \cdot ksi$ $d_{bm} = 18.4 \cdot in$ $tw_{bm} = 0.48 \cdot in$ $k1_{bm} = 1.063 \cdot in$

$Fu_{bm} = 65 \cdot ksi$ $bf_{bm} = 11.1 \cdot in$ $tf_{bm} = 0.77 \cdot in$ $k_{bm} = 1.625 \cdot in$

$Ag_{bm} = 25.3 \cdot in^2$ $Sx_{bm} = 166 \cdot in^3$ *Length of Beam,* $L_{bm} := 15ft + 0in$

SHEAR PLATE (pl): A572-50

$Fy_{pl} = 50 \cdot ksi$ $Fu_{pl} = 65 \cdot ksi$ $tpl := 1in$

STIFFENER PLATE (st): A36 (AS REQUIRED)

$Fy_{st} = 36 \cdot ksi$ $Fu_{st} = 58 \cdot ksi$ $tst := \frac{3}{8}in$

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, $\Lambda rn = 66.562 \cdot kips$

Beam Edge Distance, $Leh_{bm} = 1.5 \cdot in$

Plate Vertical Edge Distance, $Lev = 1.5 \cdot in$

Plate Horizontal Edge Distance, $Leh = 1.5 \cdot in$

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

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

Hole diameter:

Shear Plate,

$hd_{plv} = 1.125 \cdot in$ $hd_{plh} = 1.375 \cdot in$

Beam,

$hd_{bm} = 1.125 \cdot in$

Bolt First Down from Top of beam, $D = 3.25 \cdot in$

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

number of bolt rows: $nr := 5$

number of vertical bolt lines: $nv := 4$

total number of bolts: $n := nr \cdot nv$ $n = 20$

WELDS: E70xx LH

$Fu_w = 70 \cdot ksi$

Preferred Weld Size

Shear Plate to Column Web, $w_1 := Ceil\left(\frac{5}{8} tpl, \frac{1}{16} in\right)$

Shear Plate to Stiffener Plate (As Req'd), $w_2 := \frac{3}{16} in$

Stiffener Plate to Column Flange (As Req'd), $w_3 := \frac{3}{16} in$

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 shear,	$\Omega_v = 1.67$	$\phi_v = 0.9$	$\Lambda_v = 0.90$
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,	$\Omega_{vy} = 1.50$	$\phi_{vy} = 1$	$\Lambda_{vy} = 1.00$

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For bearing,	$\Omega_{brg} = 2.00$	$\phi_{brg} = 0.75$	$\Lambda_{brg} = 0.75$
For web compression buckling,	$\Omega_{cb} = 1.67$	$\phi_{cb} = 0.9$	$\Lambda_{cb} = 0.90$
For web crippling,	$\Omega_{cr} = 2.00$	$\phi_{cr} = 0.75$	$\Lambda_{cr} = 0.75$
For web yielding,	$\Omega_{wy} = 1.50$	$\phi_{wy} = 1$	$\Lambda_{wy} = 1.00$
For flexural local buckling,	$\Omega_b = 1.67$	$\phi_b = 0.9$	$\Lambda_b = 0.90$
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, $V_{giv} := 0kips$

Beam Shear Load, $V = 186 \cdot kips$ **50% UDL**

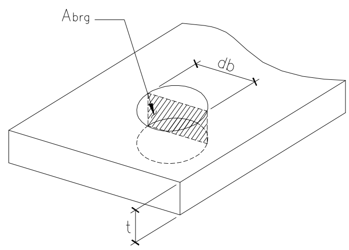
Opposite Beam Shear Load (if any), $V_2 := 0kips$

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,

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

$$Abrg_{bm} = 0.48 \cdot in^2$$

Bolt centerline distance from face of support,

$$a_b := 0.5\left(bf_{col} - tw_{col}\right) + gap + Leh_{bm} + 0.5(nv - 1) \cdot sv$$

$$a_b = 11.292 \cdot in$$

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Eccentric Load Coefficient,

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

$C = 7.417$

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

$$Fbe := \Lambda_{brg} \cdot Fu_{bm} \cdot \begin{cases} \min\left(\begin{bmatrix} 1.0 \cdot (D - 0.5 \cdot hd_{bm}) \cdot tw_{bm} \\ 1.0 \cdot (Leh_{bm} - 0.5 \cdot hd_{bm}) \cdot tw_{bm} \\ 2.0 \cdot Abrg_{bm} \end{bmatrix}\right) & \text{if } hd_{bm} \geq hd_{1s} \\ \min\left(\begin{bmatrix} 1.2 \cdot (D - 0.5 \cdot hd_{bm}) \cdot tw_{bm} \\ 1.2 \cdot (Leh_{bm} - 0.5 \cdot hd_{bm}) \cdot tw_{bm} \\ 2.4 \cdot Abrg_{bm} \end{bmatrix}\right) & \text{otherwise} \end{cases}$$

$Fbe = 26.325 \cdot kips$

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

$$Fbs := \Lambda_{brg} \cdot Fu_{bm} \cdot \begin{cases} \min\left[1.0 \cdot (s - hd_{bm}) \cdot tw_{bm}, 2.0 \cdot Abrg_{bm}\right] & \text{if } hd_{bm} \geq hd_{1s} \\ \min\left[1.2 \cdot (s - hd_{bm}) \cdot tw_{bm}, 2.4 \cdot Abrg_{bm}\right] & \text{otherwise} \end{cases}$$

$Fbs = 52.65 \cdot kips$

Bolt Bearing Capacity,

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

$Rbrg_{bm} = 195.241 \cdot kips$ $V = 186 \cdot kips$

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,

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

$h = 16.06 \cdot in$

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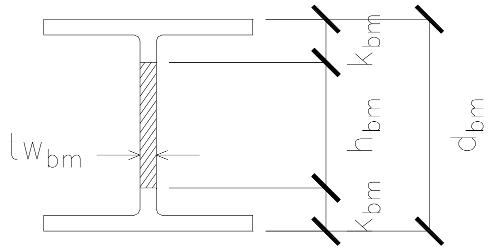
Limiting depth-thickness ratio,



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

$$h_{tw} = 33.458$$

Clear distance between transverse stiffeners,

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

$$a = 0 \cdot in$$

Web plate buckling coefficient,

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

$$kv = 5$$

Web shear coefficient,

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

$$Cv = 1$$

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Shear Capacity of Section,

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

$Rv_{bm} = 264.96 \cdot kips$ $V = 186 \cdot kips$

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,

$\Lambda rv = 40.055 \cdot kips$

Eccentric Bolt Capacity,

$Reb := C \cdot \Lambda rv$

$Reb = 297.072 \cdot kips$ $V = 186 \cdot kips$

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,

$s = 3 \cdot in$

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

$s_{min} = 2.667 \cdot in$

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

$s_{max} = 11.520 \cdot in$

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

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Horizontal Spacing,

$sv = 3 \cdot in$

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

$sv_{min} = 2.667 \cdot in$

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

$sv_{max} = 11.520 \cdot in$

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,

$Leh = 1.5 \cdot in$

$Leh_{bm} = 1.5 \cdot in$

$Le_{min} = 1.25 \cdot in$

$Leh_{minpl} = 1.375 \cdot in$

$Leh_{minbm} = 1.25 \cdot in$

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

$Leh_{maxpl} = 6.000 \cdot in$

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

$Leh_{maxbm} = 5.760 \cdot in$

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

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

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

RESULT = Check maximum thickness of plate

Coefficient for Eccentrically Loaded Bolts

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

$C' = 96.02 \cdot in$

Area of Bolts,

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

$Ab = 0.785 \cdot in^2$

Length of Plate,

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

$Lpl = 15 \cdot in$

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Maximum Thickness,

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

$tpl_{max} = 3.039 \cdot in$ $\qquad\qquad$ $tpl = 1 \cdot in$

RESULT = Plate Thickness < Max Thickness Permitted,OK

Governing Shear Plate Thickness

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

$tpl_g = 1 \cdot in$

2. Check for Stiffener Plate Requirement

(AISC 14th Ed. Manual Part 10, pages 10-105 to 10-106)

("On the Need for Stiffeners for the Effect of Lap Eccentricity on Extended Single-Plate Connections", William A. Thornton and Patrick J. Fortney)

First bolt line distance from the face of support,

$$a_{b1} := 0.5\left(bf_{col} - tw_{col}\right) + gap + Leh_{bm}$$

$a_{b1} = 6.793 \cdot in$

Available Strength to Resist Lateral Displacement,

$$Rreq_{st} := \Lambda_b 1500 \cdot \pi \cdot \frac{Lpl \cdot tpl_g^{\,3}}{a_{b1}^{\,2}} \cdot ksi \qquad (10\text{-}6)$$

$Rreq_{st} = 1378.845 \cdot kips$ $\qquad\qquad$ $V = 186 \cdot kips$

RESULT = Lateral Displacement Capacity > Force Applied, OK

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Check for Requirement of Stiffener Plates,

$$\eta := \frac{Rreq_{st}}{V}$$

$$\eta = 7.413$$

RESULT = Stiffener Plates are NOT Required

Required Torsional Moment,

$$M_t := V\cdot\left(\frac{tw_{bm} + tpl}{2}\right) \qquad (10\text{-}8)$$

$$M_t = 137.64\cdot kips\cdot in$$

Lateral Bending Capacity of Plate,

$$Rlb_{pl} := \max\left[\left(\Lambda_{vy}\cdot 0.6\cdot Fy_{pl} - \frac{V}{Lpl\cdot tpl_g}\right)\cdot\frac{Lpl\cdot tpl_g{}^2}{2}, 0kips\cdot in\right]$$

$$Rlb_{pl} = 132\cdot kips\cdot in$$

Lateral Bending Capacity of Beam,

$$Rlb_{bm} := \frac{2\cdot V^2\cdot\left(tw_{bm} + tpl_g\right)\cdot bf_{bm}}{\Lambda_b\cdot Fy_{bm}\cdot L_{bm}\cdot tw_{bm}{}^2}$$

$$Rlb_{bm} = 609.078\cdot kips\cdot in$$

Torsional Buckling Capacity,

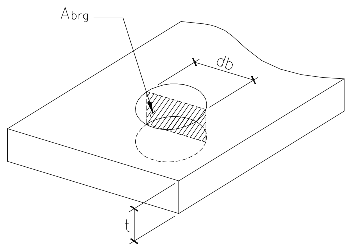
$$Mreq_{st} := \begin{cases} Rlb_{pl} + Rlb_{bm} & \text{if Beam\_type = "Composite"} \\ Rlb_{pl} & \text{otherwise} \end{cases} \qquad (10\text{-}7)$$

$$Mreq_{st} = 741.078\cdot kips\cdot in \qquad M_t = 137.64\cdot kips\cdot in$$

RESULT = Torsional Buckling Capacity > Applied Force, OK

3. 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,

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

$$Abrg_{pl} = 1 \cdot in^2$$

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

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

$$Fbe = 47.531 \cdot kips$$

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

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

$$Fbs = 109.687 \cdot kips$$

Bolt Bearing Capacity,

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

$$Rbrg_{pl} = 297.072 \cdot kips \qquad V = 186 \cdot kips$$

RESULT = Bearing Capacity > Force Applied, OK

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4. Shear Yielding Capacity of Shear Plate

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

Length of Plate,

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

$Lpl = 15\cdot in$

Check if Length of Plate is acceptable,

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

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

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)*

$Rvy_{pl} = 450\cdot kips$ $V = 186\cdot kips$

RESULT = Shear Yielding Capacity > Force Applied, OK

5. Shear Rupture Capacity of Shear Plate

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

Net Area,

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

$Anv = 9.375\cdot in^{2}$

Shear Rupture Capacity,

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

$Rvr_{pl} = 274.219\cdot kips$ $V = 186\cdot kips$

RESULT = Shear Rupture Capacity > Force Applied, OK

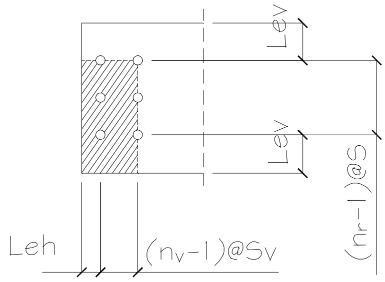
6. Block Shear Capacity of Shear Plate

(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



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

$Agv = 13.5 \cdot in^2$

Net Tension Area

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

$Ant = 5.687 \cdot in^2$

Net Shear Area

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

$Anv = 8.438 \cdot in^2$

Block Shear Capacity of Plate, (J4-5)

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

$Rbs_{pl} = 385.43 \cdot kips$ $V = 186 \cdot kips$

RESULT = Block Shear Capacity > Force Applied, OK

7. Local Buckling Capacity of Shear Plate

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

Distance of bolt line to support,

$a_b := \begin{cases} (gap + Leh_{bm}) & \text{if Stiffeners = "Required"} \\ a_{b1} & \text{otherwise} \end{cases}$

$a_b = 6.793 \cdot in$

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Coefficient,

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

$$\lambda = 0.247$$

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

$$Q = 1$$

Allowable Buckling Stress,

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

$$Fcr = 50 \cdot ksi$$

Gross Plastic Section Modulus,

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

$$Zx_{pl} = 56.25 \cdot in^3$$

Eccentricity,

$$e_{pl} := a_b$$

$$e_{pl} = 6.793 \cdot in$$

Buckling Capacity,

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

$Rbc_{pl} = 372.654 \cdot kips$ $\qquad\qquad$ $V = 186 \cdot kips$

RESULT = Local Buckling Capacity will not Control!

8. 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,

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

$Rfc_{pl} = 287.015 \cdot kips$ $\qquad V = 186 \cdot kips$

RESULT = Flexural Yielding Capacity > Applied Force, OK

9. Flexural Rupture Capacity

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

Net Plastic Section Modulus,

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

$Znet_{pl} = 35.684 \cdot in^3$

Flexural Rupture Capacity,

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

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

$Rfr_{pl} = 256.102 \cdot kips$ $\qquad V = 186 \cdot kips$

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,

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

$$V_r := V$$

$$V_r = 186 \cdot kips$$

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

$$M_r = 2100.405 \cdot kips \cdot in$$

Shear yielding,

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

$$V_c = 450 \cdot kips$$

Flexural yielding,

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

$$M_c = 2531.25 \cdot kips \cdot in$$

Interaction,

$$\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{M_r}{M_c}\right)^2 = 0.859$$

RESULT = Interaction < 1.0, OK

D. SHEAR PLATE TO COLUMN CHECK

1. Weld Check for Shear Plate to Column Web

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

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

Minimum weld size,

$w_{min1} = \frac{5}{8} \cdot in$ $w_1 = \frac{5}{8} \cdot in$

RESULT = Preferred Weld Size = Minimum Weld Size, OK

2. Weld Check for Shear Plate to Stiffener Plate

Minimum weld size,

$w_{min2} = \frac{3}{16} \cdot in$ $w_2 = \frac{3}{16} \cdot in$

RESULT = This check is not applicable

3. Weld Check for Stiffener Plate to Column Flange

Minimum weld size,

$w_{min3} = \frac{3}{16} \cdot in$ $w_3 = \frac{3}{16} \cdot in$

RESULT = This check is not applicable

E. COLUMN CHECK

1. Rupture Strength at Weld for Column Web

(AISC 14th Ed. Manual Part 10, page 10-134)

Length of weld,

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

$Lw = 15 \cdot in$

Length of weld on opposite beam,

$Lw_o := 12in$

Effective Web Thickness,

$$tw_{eff} := tw_{col} \cdot \left(\frac{\frac{V}{Lw}}{\frac{V}{Lw} + \frac{V_2}{Lw_o}} \right)$$

$tw_{eff} = 0.415 \cdot in$

Rupture Strength at Weld,

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

$Rv_{col} = 364.162 \cdot kips$ $V = 186 \cdot kips$

RESULT = Column Web Capacity > Force Applied, OK.

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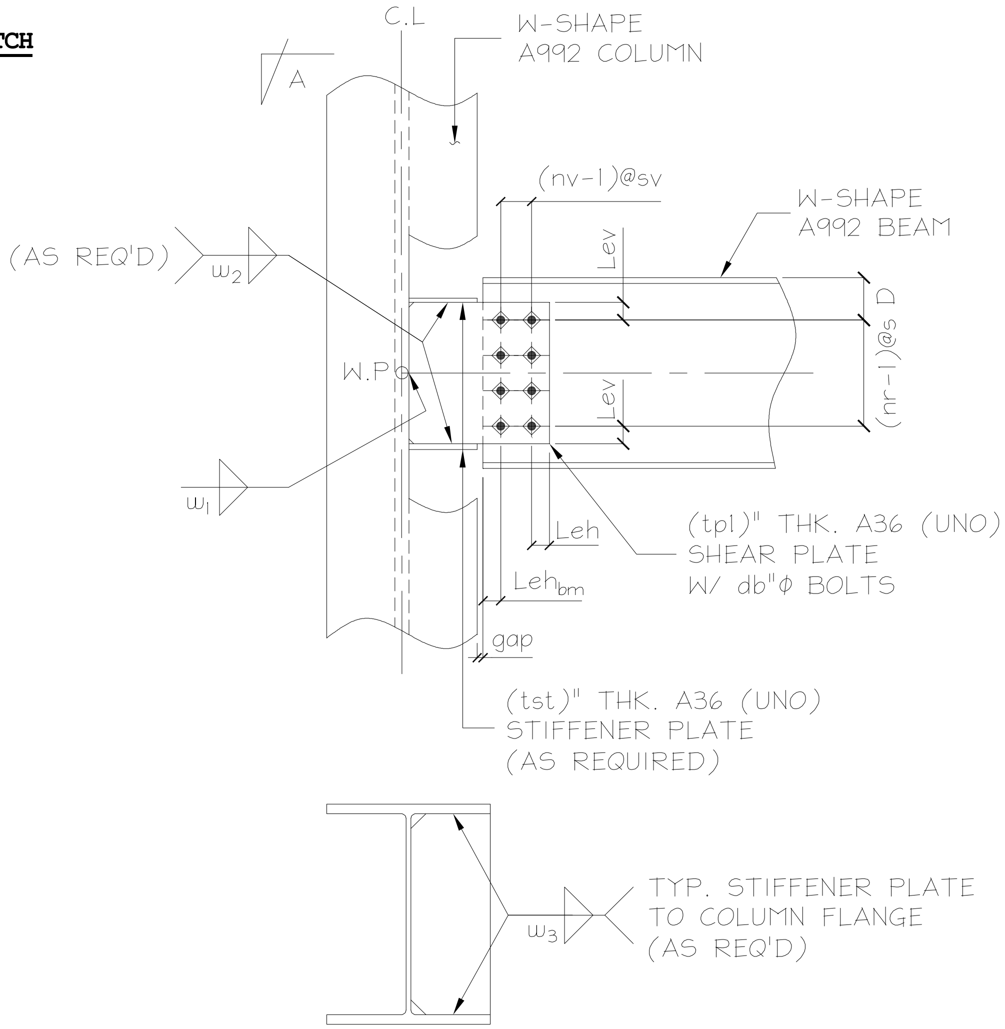
III. DETAILS

A. <u>SKETCH</u>



SECTION A

NOTE: (FIGURE ABOVE DOES NOT REPRESENT ACTUAL DESIGN, REFER ON ATTACHED BRACE CONNECTION SCHEDULE FOR THICKNESS OF PLATE, LENGTH AND OF SIZE OF WELD, NUMBER OF BOLTS

SHEAR CONNECTION: DESIGN OF W-SHAPE BEAM TO W-SHAPE COLUMN WEB SHEAR PLATE CONNECTION (FULL-DEPTH EXTENDED SHEAR PLATE)

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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)
W14X68	A992	3 1/4	1	A572-50	1	A490-N	5	4	3	3	1 1/2	1 1/2

Beam		Stiffener Plate		Edge Distance	gap (in)	Weld Size			Beam Shear Load (kips)	Rcap (kips)	Governing Capacity
Size	Grade	tst (in)	Grade	Leh_{bm} (in)		w_1 (in)	w_2 (in)	w_3 (in)			
W18X86	A992	NR	NR	1 1/2	1/2	5/8	NR	NR	186.00	195.24	Bolt Bearing on Beam Web

IV. REFERENCES

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