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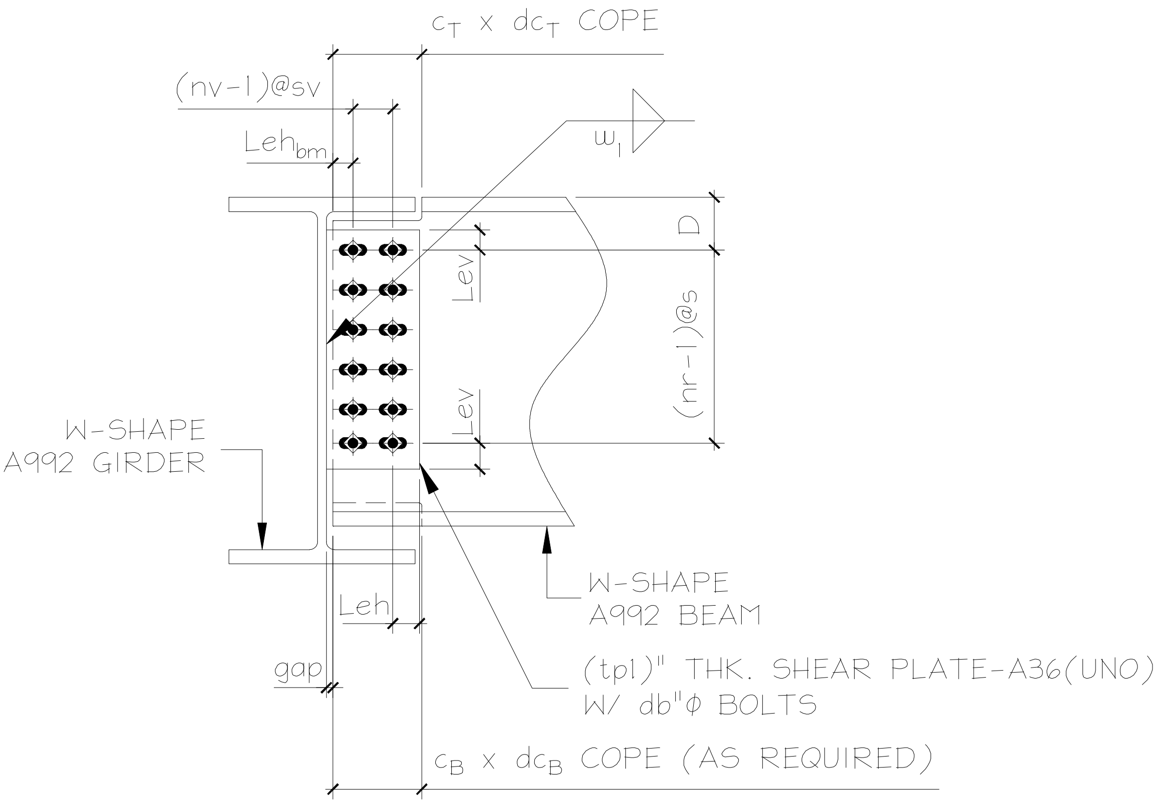
**SHEAR CONNECTION: DESIGN OF W-SHAPE BEAM TO W-SHAPE GIRDER**

**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 ( <u>LRFD</u> - AISC 14th Edition )

**GIRDER PROPERTIES (gir): W18X65 - A992**

$Fy_{gir} = 50 \cdot ksi$ $\quad d_{gir} = 18.4 \cdot in$ $\quad tw_{gir} = 0.45 \cdot in$ $\quad k1_{gir} = 0.875 \cdot in$

$Fu_{gir} = 65 \cdot ksi$ $\quad bf_{gir} = 7.59 \cdot in$ $\quad tf_{gir} = 0.75 \cdot in$ $\quad k_{gir} = 1.438 \cdot in$

$Ag_{gir} = 19.1 \cdot in^2$ $\quad Sx_{gir} = 117 \cdot in^3$ $\quad E := 29000ksi$

**BEAM PROPERTIES (bm): W18X35 - A992**

$Fy_{bm} = 50 \cdot ksi$ $\quad d_{bm} = 17.7 \cdot in$ $\quad tw_{bm} = 0.3 \cdot in$ $\quad k1_{bm} = 0.75 \cdot in$

$Fu_{bm} = 65 \cdot ksi$ $\quad bf_{bm} = 6 \cdot in$ $\quad tf_{bm} = 0.425 \cdot in$ $\quad k_{bm} = 1.125 \cdot in$

$Ag_{bm} = 10.3 \cdot in^2$ $\quad Sx_{bm} = 57.6 \cdot in^3$ $\quad$ *Length of Beam,* $\quad L_{bm} := 11ft + 0in$

| | *(TOP)* | *(BOTTOM)* |
|---|---|---|
| *Depth of Cope:* | $dc_T = 1.25 \cdot in$ | $dc_B = 1.25 \cdot in$ |
| *Length of Cope:* | $c_T = 3.75 \cdot in$ | $c_B = 3.75 \cdot in$ |

**SHEAR PLATE (pl): A36**

$Fy_{pl} = 36 \cdot ksi$ $\quad Fu_{pl} = 58 \cdot ksi$ $\quad$ *Thickness of Plate:* $\quad tpl := \frac{3}{8} in$

**BOLTS:**

***For Shear Plate to Beam Connection:***

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

*Bolt Shear Strength,* $\quad \Lambda rv = 17.892 \cdot kips$ $\quad$ Conn_type = "Bearing-type"

*Bolt Tensile Strength,* $\quad \Lambda rn = 29.821 \cdot kips$ $\quad$ *<u>Hole diameter:</u>*

*Beam Edge Distance,* $\quad Leh_{bm} = 1.5 \cdot in$ $\quad$ *Shear Plate,*

*Plate Vertical Edge Distance,* $\quad Lev = 1.5 \cdot in$ $\quad hd_{plv} = 0.875 \cdot in$ $\quad hd_{plh} = 1.063 \cdot in$

*Beam,*

*Plate Horizontal Edge Distance,* $\quad Leh = 1.5 \cdot in$ $\quad hd_{bm} = 0.875 \cdot in$

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

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

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

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

*number of bolt rows:* $nr := 5$

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

*total number of bolts:* $n := nr \cdot nv$ $n = 10$

**WELDS: E70xx**

$Fu_w = 70 \cdot ksi$

***Preferred Weld Size***

*Shear Plate to Girder Web,* $w_1 = \frac{1}{4} \cdot in$

**SAFETY AND RESISTANCE FACTORS:**

Safety Factor, $\Omega$ (ASD)

Resistance Factor, $\phi$ (LRFD)

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

| | *safety factor* | *resistance factor* | *modification factor* |
|---|---|---|---|
| For member/bolt in bearing, | $\Omega_{brg} = 2.00$ | $\phi_{brg} = 0.75$ | $\Lambda_{brg} = 0.75$ |
| For block shear, | $\Omega_{bs} = 2.00$ | $\phi_{bs} = 0.75$ | $\Lambda_{bs} = 0.75$ |
| For fillet weld (shear), | $\Omega_{vw} = 2.00$ | $\phi_{vw} = 0.75$ | $\Lambda_{vw} = 0.75$ |
| For flexural local buckling / flexural strength, | $\Omega_b = 1.67$ | $\phi_b = 0.9$ | $\Lambda_b = 0.90$ |
| For flexural rupture, | $\Omega_{fr} = 2.00$ | $\phi_{fr} = 0.75$ | $\Lambda_{fr} = 0.75$ |
| For member shear (C, WT, L) | $\Omega_v = 1.67$ | $\phi_v = 0.90$ | $\Lambda_v = 0.90$ |
| For shear on N-type bolts, | $\Omega_{vtN} = 2.00$ | $\phi_{vtN} = 0.75$ | $\Lambda_{vtN} = 0.75$ |
| For shear rupture, | $\Omega_{vr} = 2.00$ | $\phi_{vr} = 0.75$ | $\Lambda_{vr} = 0.75$ |
| For shear yielding, | $\Omega_{vy} = 1.50$ | $\phi_{vy} = 1.00$ | $\Lambda_{vy} = 1.00$ |

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**APPLIED LOAD:**

*% UDL,* $UDL := 0.5$

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

*Beam Shear Load,* $V = 90.7 \cdot kips$ **50% UDL**

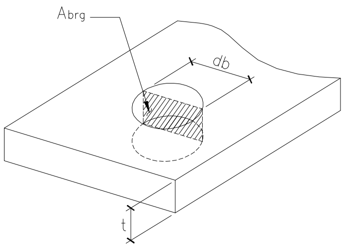
*Opposite Beam Shear Load (if any),* $V_2 := V$

## II. CALCULATIONS:

### A. BEAM WEB CHECK

#### 1. Bolt Bearing Capacity on Beam

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




*Bearing Area,*

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

$$Abrg_{bm} = 0.225 \cdot in^2$$

*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.)*

$$C = 7.292$$

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

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

$$Fbe = 18.647 \cdot kips$$

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

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

$$Fbs = 26.325 \cdot kips$$

*Bolt Bearing Capacity,*

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

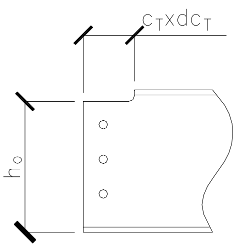
$Rbrg_{bm} = 130.478 \cdot kips$ $\quad\quad$ $V = 90.7 \cdot kips$

**RESULT = Bearing Capacity > Force Applied,OK**

**2. 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,*

$$dc := dc_T$$

$$dc = 1.25 \cdot in$$

**RESULT = This check is not applicable**

*Length of Cope,*

$c := c_T$

$c = 3.75 \cdot in$

**RESULT = This check is not applicable**

*Reduced Beam Depth,*

$ho := d_{bm} - dc$

$ho = 16.45 \cdot in$

*Plate Buckling Coefficient,*

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

$k = 25.232$

*Plate Buckling Model Adjustment Factor,*

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

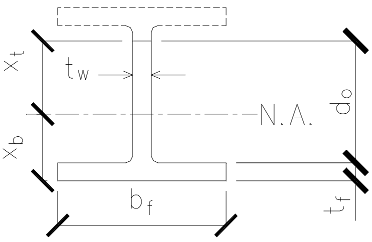
$f = 0.424$

*Allowable Flexural Local Buckling Stress/Yielding Stress,*

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

$Fcr = 50 \cdot ksi$

*Location of Neutral Axis on the Reduced Section*




$do := ho - tf_{bm}$

$do = 16.025 \cdot in$

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

$$x_b = 5.587 \cdot in$$

$$x_t := ho - x_b$$

$$x_t = 10.863 \cdot in$$

*Moment of Inertia,*

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

$$I = 215.639 \cdot in^4$$

*Net Section Modulus,*

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

$$Snet = 19.85 \cdot in^3$$

*Eccentricity,*

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

$$e_{bm} = 4.25 \cdot in$$

*Flexural Local Buckling Capacity/Yielding Capacity,*

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

$$Rbc = 210.182 \cdot kips$$

*Flexural Rupture Capacity,*

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

$$Rfr = 227.697 \cdot kips$$

*Shear Capacity of Reduced Section,*

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

$Vwg_1 = 148.05 \cdot kips$

*Coped Beam Capacity,*

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

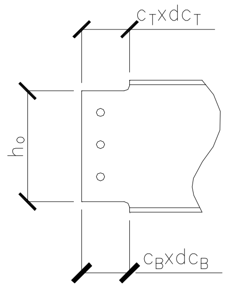
$Rscb_1 = 148.05 \cdot kips$ $\qquad V = 90.7 \cdot kips$

**RESULT = This limit state is not applicable.**

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

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

*Depth of Cope,*




*Top Cope:* $dc_T = 1.25 \cdot in$

*Bottom Cope:* $dc_B = 1.25 \cdot in$

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

$dc = 1.25 \cdot in$

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

*Length of Cope,*

*Top Cope:* $c_T = 3.75 \cdot in$

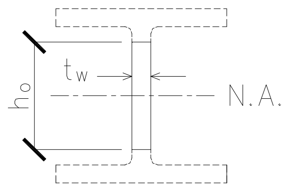
*Bottom Cope:* $c_B = 3.75 \cdot in$

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

$c = 3.75 \cdot in$

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

*Reduced Beam Depth,*




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

$ho = 15.2 \cdot in$

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*Adjustment Factor of Lateral-Torsional Buckling Model,*

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

$$fd = 2.97$$

*Allowable Flexural Local Buckling Stress/Yielding Stress,*

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

$$Fcr = 50 \cdot ksi$$

*Net Section Modulus,*

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

$$Snet = 11.552 \cdot in^3$$

*Eccentricity,*

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

$$e_{bm} = 4.25 \cdot in$$

*Flexural Local Buckling Capacity/Yielding Capacity,*

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

$$Rbc = 122.315 \cdot kips$$

*Flexural Rupture Capacity,*

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

$$Rfr = 132.508 \cdot kips$$

*Shear Capacity of Reduced Section,*

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

$$Vwg_2 = 136.8 \cdot kips$$

*Coped Beam Capacity,*

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

$Rdcb_1 = 122.315 \cdot kips$ $\qquad$ $V = 90.7 \cdot kips$

**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)*

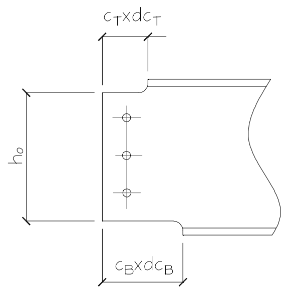
*(AISC 14th Ed. Manual Part 9, page 9-9)*

*Allowable Flexural Yielding Stress,*

$$Fcr := Fy_{bm}$$

$$Fcr = 50 \cdot ksi$$

*Location of Neutral Axis on the Reduced Section*

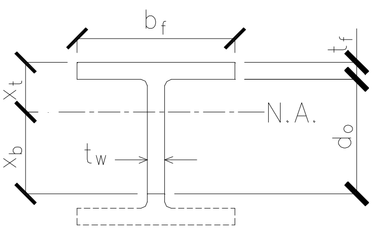



$$do := d_{bm} - dc_B - tf_{bm}$$

$$do = 16.025 \cdot in$$

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

$$x_t = 5.587 \cdot in$$




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

$$x_b = 10.863 \cdot in$$

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*Moment of Inertia,*

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

$$I = 215.639 \cdot in^4$$

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

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

$$Snet = 19.85 \cdot in^3$$

*Eccentricity,*

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

$$e_{bm} = 4.25 \cdot in$$

*Flexural Yielding Capacity,*

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

$$Rbc = 210.182 \cdot kips$$

*Flexural Rupture Capacity,*

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

$$Rfr = 227.697 \cdot kips$$

*Shear Capacity of Reduced Section,*

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

$$Vwg_3 = 136.8 \cdot kips$$

*Coped Beam Capacity,*

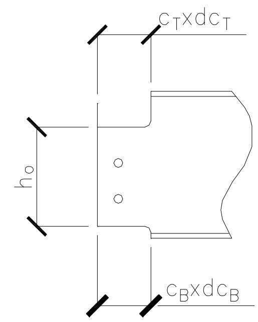
$$Rdcb_2 := \min\left(Rbc, Rfr, Vwg_3\right)$$

$Rdcb_2 = 136.8 \cdot kips$ $\qquad V = 90.7 \cdot kips$

**RESULT = This limit state is not applicable.**

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

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




*Coefficient,*

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

$$\lambda = 0.503$$

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

$$Q = 1$$

*Allowable Buckling Stress,*

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

$$Fcr = 50 \cdot ksi$$

*Net Section Modulus,*

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

$$Snet = 11.552 \cdot in^3$$

*Eccentricity,*

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

$$e_{bm} = 4.25 \cdot in$$

*Buckling Capacity,*

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

$$Rbc = 122.315 \cdot kips$$

*Flexural Rupture Capacity,*

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

$$Rfr = 132.508 \cdot kips$$

*Shear Capacity of Reduced Section,*

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

$$Vwg_4 = 136.8 \cdot kips$$

*Coped Beam Capacity,*

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

$Rtdcb = 122.315 \cdot kips$ $\qquad V = 90.7 \cdot kips$

**RESULT = This limit state is not applicable.**

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

*Depth of cope,*

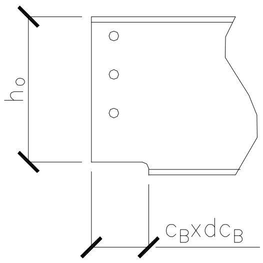
$$dc := dc_B$$

$$dc = 1.25 \cdot in$$

*Length of Cope,*

$$c := c_B$$

$$c = 3.75 \cdot in$$




*Reduced Beam Depth,*

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

$$ho = 16.45 \cdot in$$

*Allowable Flexural Yielding Stress,*

$$Fcr := Fy_{bm}$$

$$Fcr = 50 \cdot ksi$$

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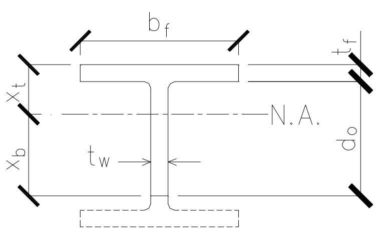
*Location of Neutral Axis on the Reduced Section*

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

$$do = 16.025 \cdot in$$

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




$$x_t = 5.587 \cdot in$$

$$x_b := ho - x_t$$

$$x_b = 10.863 \cdot in$$

*Moment of Inertia,*

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

$$I = 215.639 \cdot in^4$$

*Net Section Modulus,*

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

$$Snet = 19.85 \cdot in^3$$

*Eccentricity,*

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

$$e_{bm} = 4.25 \cdot in$$

*Flexural Yielding Capacity,*

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

$$Rbc = 210.182 \cdot kips$$

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*Flexural Rupture Capacity,*

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

$Rfr = 227.697 \cdot kips$

*Gross Web Shear,*

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

$Vwg_5 = 148.05 \cdot kips$

*Coped Beam Capacity,*

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

$Rscb_2 = 148.05 \cdot kips$ $\quad$ $V = 90.7 \cdot kips$

**RESULT = This limit state is not applicable.**

f. *Capacity if beam is cut-flushed*

*Reduced Beam Depth,*

$ho = 16.45 \cdot in$

*Allowable Flexural Local Buckling Stress/Yielding Stress,*

$Fcr = 50 \cdot ksi$

*Net Section Modulus,*

$Snet = 17.721 \cdot in^3$

*Eccentricity,*

$e_{bm} = 4.25 \cdot in$

*Flexural Local Buckling Capacity/Yielding Capacity,*

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

$Rbc = 187.638 \cdot kips$

*Flexural Rupture Capacity,*

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

$$Rfr = 203.275 \cdot kips$$

*Gross Web Shear,*

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

$$Vwg_6 = 148.05 \cdot kips$$

*Coped Beam Capacity,*

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

$Rcfb = 148.05 \cdot kips$ $\qquad\qquad$ $V = 90.7 \cdot kips$

**RESULT = This limit state is not applicable.**

**3. Block Shear Capacity of Beam**

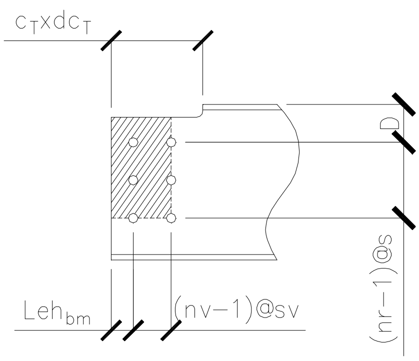
*(AISC 14th Ed. Specifications, Section J4.3, page 16.1-129)*

( *for* $dc_T > 0$ *or* $c_T \ge Leh_{bm} + (nv - 1) \cdot sv$ )

*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*




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

$$Agv = 4.125 \cdot in^2$$

*Net Tension Area*

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

$$Ant = 0.956 \cdot in^2$$

*Net Shear Area*

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

$Anv = 2.944 \cdot in^2$

*Block Shear Capacity of Beam, (J4-5)*

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

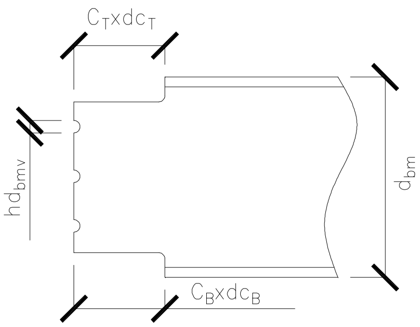
$Rbs_{bm} = 109.413 \cdot kips$ $V = 90.7 \cdot kips$

**RESULT = This limit state is not applicable.**

**4. Shear Rupture Capacity of Beam**

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

*Net Shear Area*




$Anv := \left(d_{bm} - dc_T - dc_B - nr \cdot hd_{bm}\right) \cdot tw_{bm}$

$Anv = 3.248 \cdot in^2$

*Shear Rupture Capacity (J4-4)*

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

$Rvr_{bm} = 94.989 \cdot kips$ $V = 90.7 \cdot kips$

**RESULT = This limit state is not applicable.**

**5. Shear Capacity of Beam**

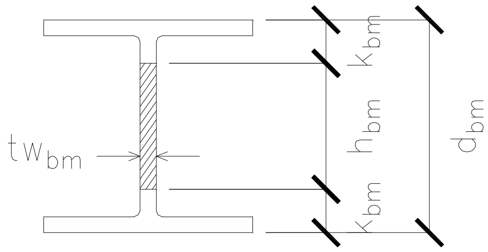
*(AISC 14th Ed. Specifications Chapter G, Section G2.1, pages 16.1-67 to 16.1-69)*

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

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

$h = 16.046 \cdot in$

*Limiting depth-thickness ratio,*




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

$$h_{tw} = 53.487$$

*Clear distance between transverse stiffeners,*

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

$$a = 0 \cdot in$$

*Web plate buckling coefficient,*

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

$$kv = 5$$

*Web shear coefficient,*

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

$$Cv = 1$$

*Shear Capacity of Section,*

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

$Rv_{bm} = 159.3 \cdot kips$ $V = 90.7 \cdot kips$

**RESULT = Shear Capacity of Section > Force Applied, OK**

**B. <u>BEAM TO SHEAR PLATE CHECK</u>**

**1. Eccentric Bolt Shear Capacity**

*(AISC 14th Ed. Specifications Chapter 7, pages 7-6 to 7-12)*

*Shear Capacity per bolt,*

$\Lambda rv = 17.892 \cdot kips$

*Eccentric Bolt Capacity,*

$Reb := C \cdot \Lambda rv$

$Reb = 130.478 \cdot kips$ $V = 90.7 \cdot kips$

**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)*

*<u>Vertical Spacing,</u>*

$s = 3 \cdot in$

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

$s_{min} = 2 \cdot in$

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

$s_{max} = 7.200 \cdot in$

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

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*Horizontal Spacing,*

$sv = 3 \cdot in$

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

$sv_{min} = 2 \cdot in$

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

$sv_{max} = 7.200 \cdot in$

**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,*

$Lev = 1.5 \cdot in$

$Lev_{bm} := D - dc_T$

$Lev_{bm} = 1.75 \cdot in$

$Le_{min} = 1 \cdot in$

$Lev_{minpl} = 1 \cdot in$

$Lev_{minbm} = 1 \cdot in$

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

$Lev_{maxpl} = 4.500 \cdot in$

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

$Lev_{maxbm} = 3.600 \cdot in$

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

*Horizontal Edge Distance,*

$Leh = 1.5 \cdot in$

$Leh_{bm} = 1.5 \cdot in$

$Le_{min} = 1 \cdot in$

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$Leh_{minpl} = 1.125 \cdot in$

$Leh_{minbm} = 1 \cdot in$

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

$Leh_{maxpl} = 4.500 \cdot in$

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

$Leh_{maxbm} = 3.600 \cdot in$

**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*

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

$Leh \ge 2 \cdot db$  $Leh_{bm} \ge 2 \cdot db$

**RESULT = No need to check maximum thickness of plate**

*Coefficient for Eccentrically Loaded Bolts*

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

$C' = 38.669 \cdot in$

*Area of Bolts*

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

$Ab = 0.442 \cdot in^2$

*Length of Plate*

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

$Lpl = 15 \cdot in$

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*Maximum Thickness*

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

$tpl_{max} = 0.759 \cdot in$ $\qquad$ $tpl = 0.375 \cdot in$

**RESULT = Max Thickness need not be checked**

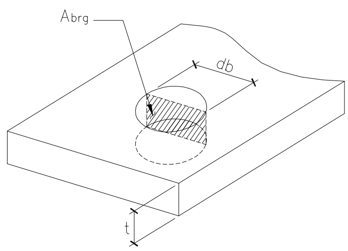
*Governing Shear Plate Thickness*

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

$$tpl_g = \frac{3}{8} \cdot in$$

**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,*

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

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

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

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

$Fbe = 18.963 \cdot kips$

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

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

$Fbs = 29.362 \cdot kips$

*Bolt Bearing Capacity,*

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

$Rbrg_{pl} = 130.478 \cdot kips$ $V = 90.7 \cdot kips$

**RESULT = Bearing Capacity > Force Applied, OK**

**3. Shear Yielding Capacity of Shear Plate**

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

*Length of Plate,*

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

$Lpl = 15 \cdot in$

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*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 $Lpl \geq 0.5(d_{bm} - 2k_{bm})$
"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)*

$Rvy_{pl} = 121.5 \cdot kips$ $V = 90.7 \cdot kips$

**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,*

$Anv := (Lpl - nr \cdot hd_{plv}) \cdot tpl_g$

$Anv = 3.984 \cdot in^2$

*Shear Rupture Capacity,*

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

$Rvr_{pl} = 103.992 \cdot kips$ $V = 90.7 \cdot kips$

**RESULT = Shear Rupture Capacity > Force Applied, OK**

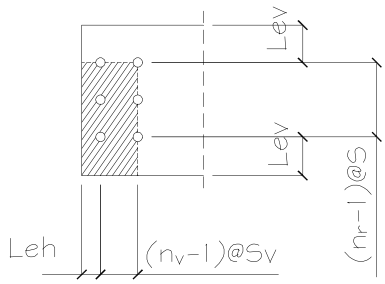
**5. Block Shear Capacity of Shear Plate**

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

*Reduction Factor,* $U_{bs} :=$ | 1.0 if $nv = 1$ (tension stress is uniform)
0.5 if $nv > 1$ (tension stress is non-uniform)

$U_{bs} = 0.5$

*Gross Shear Area*




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

$$Agv = 5.062\cdot in^2$$

*Net Tension Area*

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

$$Ant = 1.09\cdot in^2$$

*Net Shear Area*

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

$$Anv = 3.586\cdot in^2$$

*Block Shear Capacity of Plate, (J4-5)*

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

$Rbs_{pl} = 105.717\cdot kips$ $\quad\quad$ $V = 90.7\cdot kips$

**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,*

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

$$\lambda = 0.188$$

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

$Q = 1$

*Allowable Buckling Stress,*

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

$Fcr = 36 \cdot ksi$

*Gross Plastic Section Modulus,*

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

$Zx_{pl} = 21.094 \cdot in^3$

*Eccentricity,*

$e_{pl} := a_b$

$e_{pl} = 2 \cdot in$

*Buckling Capacity,*

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

$Rbc_{pl} = 341.719 \cdot kips$ $V = 90.7 \cdot kips$

**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,*

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

$Rfc_{pl} = 114.479 \cdot kips$ $\qquad\qquad$ $V = 90.7 \cdot kips$

**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,*

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

$Znet_{pl} = 15.116 \cdot in^3$

*Flexural Rupture Capacity,*

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

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

$Rfr_{pl} = 328.767 \cdot kips$ $\qquad\qquad$ $V = 90.7 \cdot kips$

**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,*

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

$$V_r := V$$

$$V_r = 90.7 \cdot \text{kips}$$

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

$$M_r = 317.45 \cdot \text{kips} \cdot \text{in}$$

*Shear yielding,*

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

$$V_c = 121.5 \cdot \text{kips}$$

*Flexural yielding,*

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

$$M_c = 683.437 \cdot \text{kips} \cdot \text{in}$$

*Interaction,*

$$\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{M_r}{M_c}\right)^2 = 0.773$$

**RESULT = Interaction < 1.0, OK**

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## D. SHEAR PLATE TO GIRDER CHECK

### 1. Weld Check for Shear Plate to Girder Web

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

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

*Minimum weld size,*

$w_{min1} = \frac{1}{4} \cdot in$ $\quad$ $w_1 = \frac{1}{4} \cdot in$

**RESULT = Preferred Weld Size = Minimum Weld Size, OK**

*(AISC 14th Ed. Specifications Chapter J, page 16.1-11)*

*Maximum Weld Size,*

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

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

**RESULT = Maximum Weld > Preferred Weld, OK**

## E. GIRDER CHECK

### 1. Rupture Strength at Weld for Girder Web

*Length of weld,*

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

$Lw = 15 \cdot in$

*Length of weld on opposite beam,*

$Lw_o := 12 in$

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*Effective Web Thickness,*

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

$$tw_{eff} = 0.2 \cdot in$$

*No. of Weld side,*

$$n_{ws} := 2$$

*Rupture Strength at Weld,*

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

$Rv_{gir} = 175.5 \cdot kips$ $\quad$ $V = 90.7 \cdot kips$

**RESULT = Girder Web Capacity > Force Applied, OK.**

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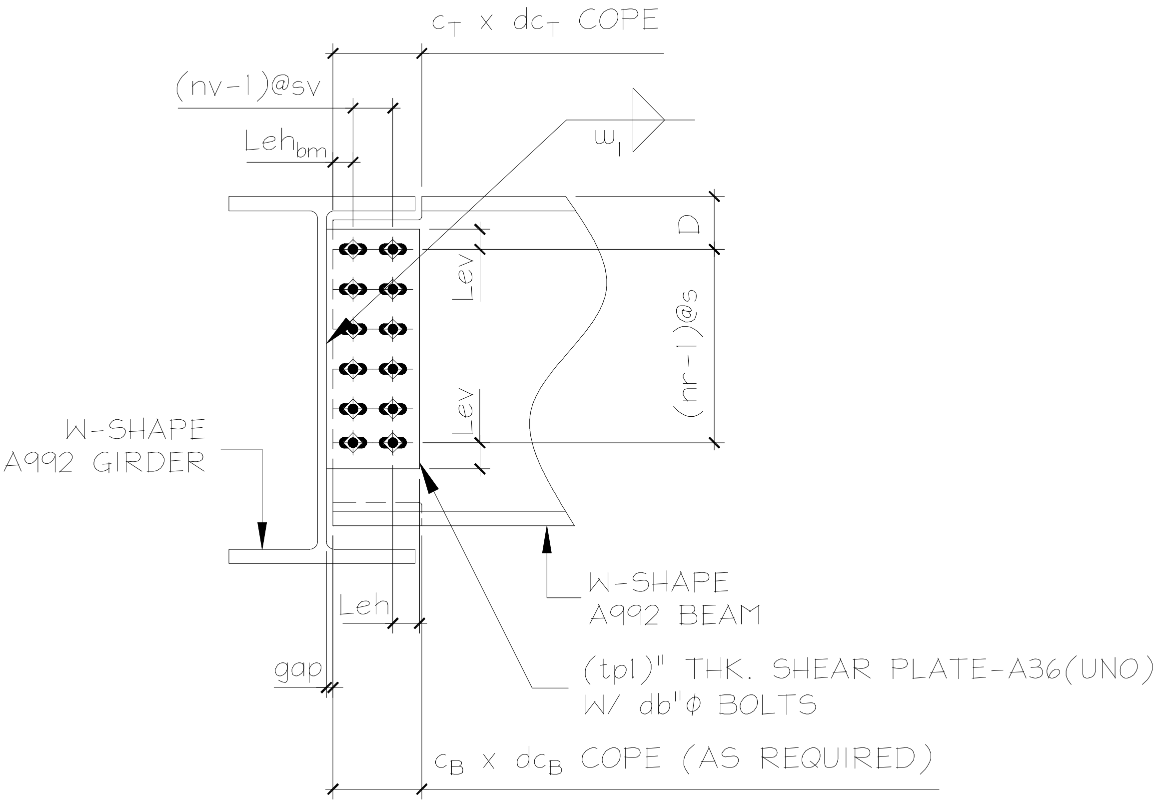
## III. DETAILS:

### 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: DESIGN OF W-SHAPE BEAM TO W-SHAPE GIRDER SHEAR PLATE CONNECTION**

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B. **TABLE: SHEAR CONNECTION SCHEDULE**

| Girder | | 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) |
| W18X65 | A992 | 3 | 3/8 | A36 | 3/4 | A325-N | 5 | 2 | 3 | 3 | 1 1/2 | 1 1/2 |

| Beam | | Cope Dimensions (in) | | | | Edge Distance | gap (in) | Weld Size | Beam Shear Load (kips) | Rcap (kips) | Governing Capacity |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Size | Grade | $dc_T$ | $c_T$ | $dc_B$ | $c_B$ | $Leh_{bm}$ (in) | | $w_1$ (in) | | | |
| W18X35 | A992 | 1 1/4 | 3 3/4 | 1 1/4 | 3 3/4 | 1 1/2 | 1/2 | 1/4 | 90.70 | 94.99 | Shear Rupture of Beam |

# IV. REFERENCES

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