5404 Gurley Avenue
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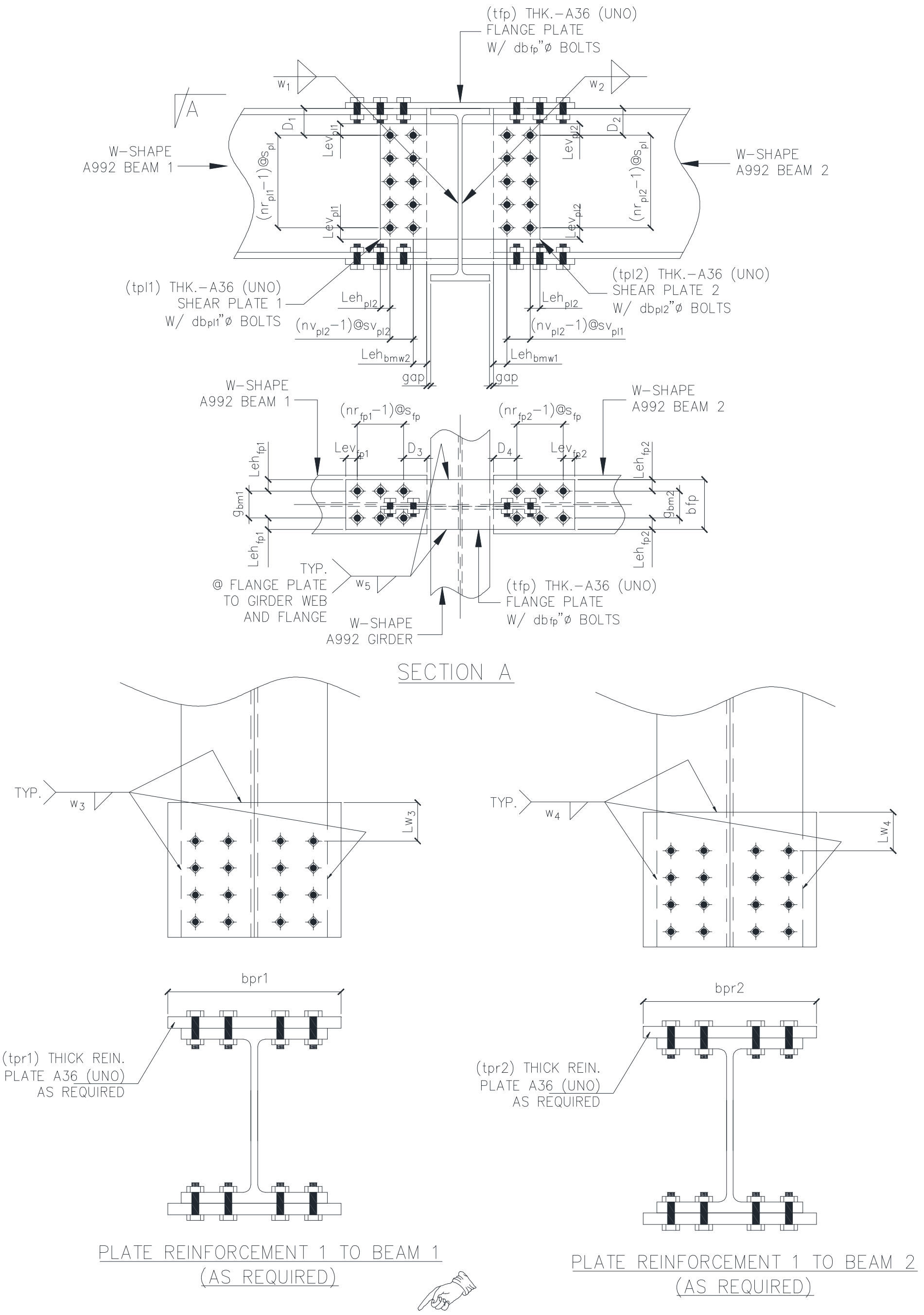
MOMENT CONNECTION: DESIGN OF W-SHAPE BEAMS (BOLTED FLANGE PLATE AND WEB SHEAR PLATE CONNECTION) TO W-SHAPE GIRDER (2-WAY)



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 (ASD - AISC 14th Edition)

GIRDER PROPERTIES (gir): W21X62 - A36

$Fy_{gir} = 36 \cdot ksi$ $d_{gir} = 21 \cdot in$ $tw_{gir} = 0.4 \cdot in$ $k1_{gir} = 0.813 \cdot in$

$Fu_{gir} = 58 \cdot ksi$ $bf_{gir} = 8.24 \cdot in$ $tf_{gir} = 0.615 \cdot in$ $k_{gir} = 1.313 \cdot in$

$E := 29000ksi$

BEAM1 PROPERTIES (bm1): W18X65 - A992

$Fy_{bm1} = 50 \cdot ksi$ $d_{bm1} = 18.4 \cdot in$ $tw_{bm1} = 0.45 \cdot in$ $k1_{bm1} = 0.875 \cdot in$

$Fu_{bm1} = 65 \cdot ksi$ $bf_{bm1} = 7.59 \cdot in$ $tf_{bm1} = 0.75 \cdot in$ $k_{bm1} = 1.438 \cdot in$

Length of Beam, $L_{bm1} := 13ft + 0in$ *Beam Bolt Gage,* $g_{bm1} := 5\frac{1}{2}in$

BEAM2 PROPERTIES (bm2): W18X65 - A992

$Fy_{bm2} = 50 \cdot ksi$ $d_{bm2} = 18.4 \cdot in$ $tw_{bm2} = 0.45 \cdot in$ $k1_{bm2} = 0.875 \cdot in$

$Fu_{bm2} = 65 \cdot ksi$ $bf_{bm2} = 7.59 \cdot in$ $tf_{bm2} = 0.75 \cdot in$ $k_{bm2} = 1.438 \cdot in$

Length of Beam, $L_{bm2} := 13ft + 0in$ *Beam Bolt Gage,* $g_{bm2} := 5\frac{1}{2}in$

FLANGE PLATE (fp): A572-50

$Fy_{fp} = 50 \cdot ksi$ $Fu_{fp} = 65 \cdot ksi$ *Thickness of Plate,* $tfp := 1in$

SHEAR PLATE 1 (pl1): A572-50

$Fy_{pl1} = 50 \cdot ksi$ $Fu_{pl1} = 65 \cdot ksi$ *Thickness of Plate:* $tpl1 := 1in$

SHEAR PLATE 2 (pl2): A572-50

$Fy_{pl2} = 50 \cdot ksi$ $Fu_{pl2} = 65 \cdot ksi$ *Thickness of Plate:* $tpl2 := 1in$

PLATE REINFORCEMENT 1(pr1): A36 (AS REQUIRED)

$Fy_{pr1} = 36 \cdot ksi$ $Fu_{pr1} = 58 \cdot ksi$ *Number of Plate:* $n_{pr1} := 2$

Thickness of Plate: $tpr1 := \frac{1}{2} in$

Width of Plate: $bpr1 = 8.625 \cdot in$

PLATE REINFORCEMENT 2 (pr2): A36 (AS REQUIRED)

$Fy_{pr2} = 36 \cdot ksi$ $Fu_{pr2} = 58 \cdot ksi$ *Number of Plate:* $n_{pr2} := 2$

Thickness of Plate: $tpr2 := \frac{1}{2} in$

Width of Plate: $bpr2 = 8.625 \cdot in$

BOLTS:
For Shear Plate 1 to Beam 1 Connection,

Bolt Diameter, $db_{pl1} = 1 \cdot in$ $Bolt\_Type_{pl1}$ = "A490-SC-SSLT-CLASS_A"

Bolt Shear Strength, $\Lambda rv_{pl1} = 14.464 \cdot kips$ $Conn\_type_{pl1}$ = "Slip Critical-type"

Bolt Tensile Strength, $\Lambda rn_{pl1} = 44.375 \cdot kips$

Beam Web Edge Distance, $Leh_{bmw1} = 1.5 \cdot in$

Plate Vertical Edge Distance, $Lev_{pl1} = 1.5 \cdot in$

Plate Horizontal Edge Distance, $Leh_{pl1} = 1.5 \cdot in$

Bolt Vertical Spacing, $s_{pl1} = 2.75 \cdot in$

Bolt Horizontal Spacing (For Multiple bolt lines), $sv_{pl1} = 3 \cdot in$

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

Hole diameter:

Shear Plate,

$hd_{pl1v} = 1.125 \cdot in$ $hd_{pl1h} = 1.375 \cdot in$

Beam Web,

$hd_{bm1w} = 1.125 \cdot in$

number of bolt rows: $nr_{pl1} := 5$

number of vertical bolt lines: $nv_{pl1} := 4$

total number of bolts: $n_{pl1} := nr_{pl1} \cdot nv_{pl1}$ $n_{pl1} = 20$

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Bolt First Down from top of beam,		$D_1 = 3.75 \cdot in$	

For Flange Plate to Beam Flange 1 Connection:

Bolt Diameter,	$db_{fp1} = 1 \cdot in$	$Bolt\_Type_{fp1}$ = "A490-SC-STD-CLASS_A"	
Bolt Shear Strength,	$\Lambda rv_{fp1} = 14.464 \cdot kips$	$Conn\_type_{fp1}$ = "Slip Critical-type"	
Bolt Tensile Strength,	$\Lambda rn_{fp1} = 44.375 \cdot kips$		
		Hole diameter:	
Flange Plate Vertical Edge Distance,	$Lev_{fp1} = 1.5 \cdot in$	*Flange Plate,*	
Flange Plate Horizontal Edge Distance,	$Leh_{fp1} = 2 \cdot in$	$hd_{fp1} = 1.125 \cdot in$	
Bolt Vertical Spacing,	$s_{fp1} = 3 \cdot in$	*Beam Flange,*	
Bolt Horizontal Spacing (For Multiple bolt lines),	$sv_{fp1} = 3 \cdot in$	$hd_{bm1f} = 1.125 \cdot in$	

number of bolt rows:		$nr_{fp1} := 8$
number of vertical bolt lines:		$nv_{fp1} := 1$
total number of bolts:	$n_{fp1} := 2nr_{fp1} \cdot nv_{fp1}$	$n_{fp1} = 16$
First bolt line to beam flange edge,		$D_3 = 3 \cdot in$

For Shear Plate 2 to Beam 2 Connection,

Bolt Diameter,	$db_{pl2} = 1 \cdot in$	$Bolt\_Type_{pl2}$ = "A490-SC-SSLT-CLASS_A"	
Bolt Shear Strength,	$\Lambda rv_{pl2} = 14.464 \cdot kips$	$Conn\_type_{pl2}$ = "Slip Critical-type"	
Bolt Tensile Strength,	$\Lambda rn_{pl2} = 44.375 \cdot kips$		
Beam Web Edge Distance,	$Leh_{bmw2} = 1.5 \cdot in$	*Hole diameter:*	
		Shear Plate,	
Plate Vertical Edge Distance,	$Lev_{pl2} = 1.5 \cdot in$	$hd_{pl2v} = 1.125 \cdot in$	$hd_{pl2h} = 1.375 \cdot in$
Plate Horizontal Edge Distance,	$Leh_{pl2} = 1.5 \cdot in$	*Beam Web,*	
Bolt Vertical Spacing,	$s_{pl2} = 2.75 \cdot in$	$hd_{bm2w} = 1.125 \cdot in$	

Bolt Horizontal Spacing (For Multiple bolt lines), $sv_{pl2} = 3 \cdot in$

number of bolt rows: $nr_{pl2} := 5$

number of vertical bolt lines: $nv_{pl2} := 4$

total number of bolts: $n_{pl2} := nr_{pl2} \cdot nv_{pl2}$ $n_{pl2} = 20$

Bolt First Down from top of beam, $D_2 = 3.75 \cdot in$

For Flange Plate to Beam Flange 2 Connection:

Bolt Diameter, $db_{fp2} = 1 \cdot in$ $Bolt\_Type_{fp2} =$ "A490-SC-STD-CLASS_A"

Bolt Shear Strength, $\Lambda rv_{fp2} = 14.464 \cdot kips$ $Conn\_type_{fp2} =$ "Slip Critical-type"

Bolt Tensile Strength, $\Lambda rn_{fp2} = 44.375 \cdot kips$

Flange Plate Vertical Edge Distance, $Lev_{fp2} = 1.5 \cdot in$

Flange Plate Horizontal Edge Distance, $Leh_{fp2} = 2 \cdot in$

Bolt Vertical Spacing, $s_{fp2} = 3 \cdot in$

Bolt Horizontal Spacing (For Multiple bolt lines), $sv_{fp2} = 3 \cdot in$

Hole diameter:

Flange Plate,

$hd_{fp2} = 1.125 \cdot in$

Beam Flange,

$hd_{bm2f} = 1.125 \cdot in$

number of bolt rows: $nr_{fp2} := 8$

number of vertical bolt lines: $nv_{fp2} := 1$

total number of bolts: $n_{fp2} := 2nr_{fp2} \cdot nv_{fp2}$ $n_{fp2} = 16$

First bolt line to beam flange edge, $D_4 = 3 \cdot in$

Governing Flange Plate Width,

$$bfp := \max\left[g_{bm1} + 2\left(nv_{fp1} - 1\right) \cdot sv_{fp1} + 2Leh_{fp1}, g_{bm2} + 2\left(nv_{fp2} - 1\right) \cdot sv_{fp2} + 2Leh_{fp2}\right]$$

$bfp = 9.5 \cdot in$

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WELDS: E70xx LH

$Fu_w = 70 \cdot ksi$

Weld Size

Shear Plate 1 to Girder Web,	$w_1 := \frac{1}{4} in$	
Shear Plate 2 to Girder Web,	$w_2 := \frac{1}{4} in$	
Plate Reinforcement 1 to Beam (as req'd),	$w_3 := \frac{1}{4} in$	$Lw_3 := 6in$
Plate Reinforcement 2 to Beam (as req'd),	$w_4 := \frac{1}{4} in$	$Lw_4 := 6in$
Flange Plate to Girder Flange,	$w_5 := \frac{1}{4} in$	

SAFETY AND RESISTANCE FACTORS:

Safety Factor, Ω (ASD)

Resistance Factor, ϕ (LRFD)

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

	safety factor	*resistance factor*	*modification factor*
For tension rupture,	$\Omega_{tr} = 2.00$	$\phi_{tr} = 0.75$	$\Lambda_{tr} = 0.50$
For tension yielding,	$\Omega_{ty} = 1.67$	$\phi_{ty} = 0.9$	$\Lambda_{ty} = 0.60$
For compression,	$\Omega_c = 1.67$	$\phi_c = 0.9$	$\Lambda_c = 0.60$
For member shear (C, WT, L)	$\Omega_v = 1.67$	$\phi_v = 0.9$	$\Lambda_v = 0.60$
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 block shear,	$\Omega_{bs} = 2.00$	$\phi_{bs} = 0.75$	$\Lambda_{bs} = 0.50$
For member/bolt in bearing,	$\Omega_{brg} = 2.00$	$\phi_{brg} = 0.75$	$\Lambda_{brg} = 0.50$

For web crippling,	$\Omega_{cr} = 2.00$	$\phi_{cr} = 0.75$	$\Lambda_{cr} = 0.50$
For member shear yielding (W,S,M,HSS)	$\Omega_{wy} = 1.50$	$\phi_{wy} = 1.00$	$\Lambda_{wy} = 0.67$
For flexural rupture,	$\Omega_{fr} = 2.00$	$\phi_{fr} = 0.75$	$\Lambda_{fr} = 0.50$

APPLIED LOAD:

BEAM 1

% UDL, $UDL := 0.5$

Given Shear Load if any, $V_{giv1} := 0kips$

Beam 1 Shear Load, $V_{bm1} = 102.2 \cdot kips$

Given Axial Load (if any), $P_{giv1} := 0kips$

Beam 1 Axial Load, $P_{bm1} = 0.00 \cdot kips$

% Moment Capacity, $M_{cap1} := 1$

Given Moment Load, $M_{giv1} := 0kips \cdot ft$

Moment Load, $M_{bm1} = 331.84 \cdot kips \cdot ft$ **100% M.cap**

BEAM 2

% UDL, $UDL := 0.5$

Given Shear Load if any, $V_{giv2} := 0kips$

Beam 2 Shear Load, $V_{bm2} = 102.2 \cdot kips$ **50% UDL**

Given Axial Load $P_{giv2} := 0kips$

Beam 2 Axial Load, $P_{bm2} = 0.00 \cdot kips$

% Moment Capacity, $M_{cap2} := 1$

Given Moment Load, $M_{giv2} := 0kips \cdot ft$

Moment Load, $M_{bm2} = 331.84 \cdot kips \cdot ft$ **100% M.cap**

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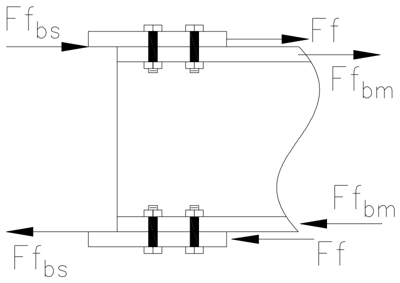
II. CALCULATIONS:

A. BOLTS ON FLANGE PLATE CHECK

1. Forces acting on the Connection



@ Beam Flange,

$$Ff_{bm1} := \frac{P_{bm1}}{2} + \frac{M_{bm1}}{d_{bm1} - tf_{bm1}}$$

$$Ff_{bm1} = 225.611 \cdot kips$$

@ Interface of Beam Flange & Flange Plate,

$$Ff_{bs1} := \frac{P_{bm1}}{2} + \frac{M_{bm1}}{d_{bm1}}$$

$$Ff_{bs1} = 216.415 \cdot kips$$

@ Flange Plate,

$$Ff_1 := \frac{P_{bm1}}{2} + \frac{M_{bm1}}{d_{bm1} + tfp}$$

$$Ff_1 = 205.26 \cdot kips$$

2. Bolt Shear Capacity

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

Shear Capacity per bolt,

$$\Lambda rv_{fp1} = 14.464 \cdot kips$$

Bolt Shear Capacity,

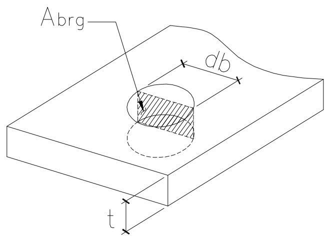
$$Rb_{fp} := n_{fp1} \cdot \Lambda rv_{fp1}$$

$$Rb_{fp} = 231.424 \cdot kips \qquad Ff_{bs1} = 216.415 \cdot kips$$

RESULT = Bolt Shear Capacity > Force Applied, OK

B. BEAM 1 FLANGE CHECK

1. Bolt Bearing on Beam Flange



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

Bearing Area,

$$Abrg_{bm} := db_{fp1} \cdot tf_{bm1}$$

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

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

$$Fbe := \Lambda_{brg} \cdot Fu_{bm1} \cdot \begin{vmatrix} \min\left[1.0 \cdot \left(D_3 - 0.5hd_{bm1f}\right) \cdot tf_{bm1}, 2.0 \cdot Abrg_{bm}\right] & \text{if } hd_{bm1f} \geq hd_{1s} \\ \min\left[1.2 \cdot \left(D_3 - 0.5hd_{bm1f}\right) \cdot tf_{bm1}, 2.4 \cdot Abrg_{bm}\right] & \text{otherwise} \end{vmatrix}$$

$$Fbe = 58.5 \cdot kips$$

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

$$Fbs := \Lambda_{brg} \cdot Fu_{bm1} \cdot \begin{vmatrix} \min\left[1.0 \cdot \left(s_{fp1} - hd_{bm1f}\right) \cdot tf_{bm1}, 2.0 \cdot Abrg_{bm}\right] & \text{if } hd_{bm1f} \geq hd_{1s} \\ \min\left[1.2 \cdot \left(s_{fp1} - hd_{bm1f}\right) \cdot tf_{bm1}, 2.4 \cdot Abrg_{bm}\right] & \text{otherwise} \end{vmatrix}$$

$$Fbs = 54.844 \cdot kips$$

Bolt Bearing Capacity,

$$Rbrg_{bmf} := 2 \cdot nv_{fp1} \cdot \left[\min\left(Fbe, \Lambda rv_{fp1}\right) + \min\left(Fbs, \Lambda rv_{fp1}\right) \cdot \left(nr_{fp1} - 1\right)\right]$$

$$Rbrg_{bmf} = 231.424 \cdot kips \qquad Ff_{bm1} = 225.611 \cdot kips$$

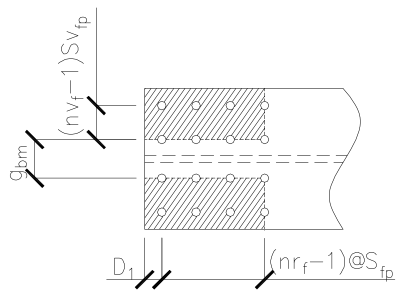
RESULT = Bearing Capacity > Force Applied, OK

2. Block Shear Capacity of Beam Flange

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

Reduction Factor, $U_{bs} := 1.0$ *(Tension Stress is Uniform)*

Gross Shear Area



$$Agv := 2 \cdot \left[\left(nr_{fp1} - 1\right) \cdot s_{fp1} + D_3\right] \cdot tf_{bm1}$$

$$Agv = 36 \cdot in^2$$

Net Tension Area

$$Ant := \left[\left(bf_{bm1} - g_{bm1}\right) - \left(2nv_{fp1} - 1\right) \cdot hd_{bm1f}\right] \cdot tf_{bm1}$$

$Ant = 0.724 \cdot in^2$

Net Shear Area

$Anv := 2 \cdot \left[\left(nr_{fp1} - 1 \right) \cdot s_{fp1} + D_3 - \left(nr_{fp1} - 0.5 \right) \cdot hd_{bm1f} \right] \cdot tf_{bm1}$

$Anv = 23.344 \cdot in^2$

Block Shear Capacity,

$Rbs_{bmf} := \Lambda_{bs} \min \left(0.6 Fu_{bm1} \cdot Anv + U_{bs} \cdot Fu_{bm1} \cdot Ant , 0.6 \cdot Fy_{bm1} \cdot Agv + U_{bs} \cdot Fu_{bm1} \cdot Ant \right)$

$Rbs_{bmf} = 478.725 \cdot kips$ $\quad\quad Ff_{bm1} = 225.611 \cdot kips$

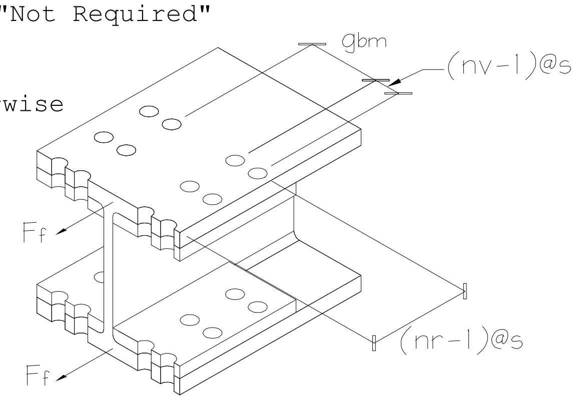
RESULT = Block Shear Capacity > Force Applied, OK

3. Beam Flexural Strength on Reduced Area w/ Reinforcement

(AISC 14th Ed. Specifications Chapter F, Section F13, page 16.1-64)

Effective Flange Thickness,

$$tf_{bmpr} := \begin{cases} tf_{bm1} & \text{if Reinforcement1} = \text{"Not Required"} \\ tf_{bm1} + tpr1 \cdot \left(\dfrac{Fy_{pr1}}{Fy_{bm1}} \right) & \text{otherwise} \end{cases}$$



$tf_{bmpr} = 1.11 \cdot in$

a. Gross Tension Flange Area

$A_{fg} := bf_{bm1} \cdot tf_{bmpr}$

$A_{fg} = 8.425 \cdot in^2$

b. Net Tension Area

$A_{fn} := A_{fg} - \left(2nv_{fp1} \cdot hd_{bm1f} \cdot tf_{bmpr} \right)$

$A_{fn} = 5.927 \cdot in^2$

c. Value of Yt

$$Y_t := \begin{cases} 1.0 & \text{if } \dfrac{Fy_{bm1}}{Fu_{bm1}} \le 0.8 \\ 1.1 & \text{otherwise} \end{cases} \quad\quad \text{(F13-1)}$$

$Y_t = 1$

d. Section Modulus of Section

$$Ix_{bmpr} := 2\left[\frac{bf_{bm1}\cdot tf_{bmpr}{}^{3}}{12} + bf_{bm1}\cdot tf_{bmpr}\cdot\left(\frac{d_{bm1} - tf_{bmpr}}{2}\right)^{2}\right] \ldots$$
$$+ \frac{tw_{bm1}\cdot\left(d_{bm1} - 2tf_{bmpr}\right)^{3}}{12}$$

$$Ix_{bmpr} = 1419.86\cdot in^{4}$$

$$Sx_{bmpr} := \begin{cases} Sx_{bm1} & \text{if } Reinforcement1 = \text{"Not Required"} \\ \dfrac{Ix_{bmpr}}{0.5\left(d_{bm1}\right)} & \text{otherwise} \end{cases}$$

$$Sx_{bmpr} = 154.333\cdot in^{3}$$

e. Revised Flexural Requirement Due to Axial Load

$$M_{rev1} := M_{bm1} + 0.5P_{bm1}\cdot\left(d_{bm1} - tf_{bm1}\right)$$

$$M_{rev1} = 331.836\cdot kips\cdot ft$$

f. Tension Rupture Capacity

Net Moment Capacity,

$$M_n := \frac{\Lambda_b\cdot Fu_{bm1}\cdot A_{fn}\cdot Sx_{bmpr}}{A_{fg}} \qquad (F13\text{-}1)$$

$M_n = 352.187\cdot kips\cdot ft$ $\qquad$ $M_{bm1} = 331.836\cdot kips\cdot ft$

RESULT = Net Moment Capacity > Applied Moment, OK

C. PLATE REINFORCEMENT CONNECTION TO BEAM: **Required**

1. Force Acting on Plate Reinforcement

Force on Flange,

$$Ff_{bmi} := \frac{tf_{bm1}}{tf_{bmpr}}\cdot Ff_{bm1}$$

$$Ff_{bmi} = 152.44\cdot kips$$

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Force on Plate Reinforcement,

$F_{pr} := Ff_{bm1} - Ff_{bmi}$

$F_{pr} = 73.171 \cdot kips$

2. Weld Capacity of Plate Reinforcement to Beam

(AISC Specifications 13th Ed, Chapter J, Section J2.2b, Table J2.4, pages 16.1-90 to 16.1-102)

Length of weld,

$Lw_{pr1} := Lw_3$

$Lw_{pr1} = 6 \cdot in$

$Lw_{pr2} := bf_{bm}$

No. of Weld side, $n_{ws} := 1$

Minimum weld size,

$w_{min} = \frac{3}{16} \cdot in$ $w_3 = \frac{1}{4} \cdot in$

RESULT = Preferred Weld Size > Minimum Weld Size, OK

Maximum weld size,

$w_{max} = \frac{7}{16} \cdot in$ $w_3 = \frac{1}{4} \cdot in$

RESULT = Maximum Weld Size > Preferred Weld Size, OK

a. Plate Reinforcement to Beam Flange Using Fillet Weld

For Longitudinal Weld

Shear Strength,

For Beam:

$Rv_{bm} := \Lambda_{vr} \cdot 0.6 \cdot Fu_{bm} \cdot tf_{bm}$

$Rv_{bm} = 14.625 \cdot \frac{kips}{in}$

For Plate Reinforcement:

$Rv_{pr} := \Lambda_{vr} \cdot 0.6 \cdot Fu_{pr} \cdot tpr \cdot n_{ws}$

$Rv_{pr} = 8.7 \cdot \frac{kips}{in}$

For Weld:

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

$$Rv_w = 14.849 \cdot ksi$$

Maximum effective weld size,

$$w_{eff} := \frac{\min(Rv_{bm}, Rv_{pr})}{Rv_w}$$

$$w_{eff} = 0.586 \cdot in$$

Weld Capacity,

$$Rw_{pr1} := 2\Lambda_{vw} \cdot 0.60 \cdot Fu_w \cdot \sin(45 \cdot deg) \cdot n_{ws} \cdot \min(w_{eff}, w_3) \cdot Lw_{pr1}$$

$$Rw_{pr1} = 44.548 \cdot kips$$

For Transverse Weld

Shear Strength,

For Beam:

$$Rv_{bm} := \Lambda_{vr} \cdot 0.6 \cdot Fu_{bm} \cdot tf_{bm}$$

$$Rv_{bm} = 14.625 \cdot \frac{kips}{in}$$

For Plate Reinforcement:

$$Rv_{pr} := \Lambda_{vr} \cdot 0.6 \cdot Fu_{pr} \cdot tpr \cdot n_{ws}$$

$$Rv_{pr} = 8.7 \cdot \frac{kips}{in}$$

For Weld:

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

$$Rv_w = 14.849 \cdot ksi$$

Maximum effective weld size,

$$w_{eff} := \frac{\min(Rv_{bm}, Rv_{pr})}{Rv_w}$$

$w_{eff} = 0.586 \cdot in$

Weld Capacity,

$Rw_{pr2} := \Lambda_{vw} \cdot 0.60 \cdot Fu_w \cdot \sin(45 \cdot deg) \cdot n_{ws} \cdot \min(w_{eff}, w_3) \cdot Lw_{pr2}$

$Rw_{pr2} = 28.176 \cdot kips$

$Rw_{pr} := \max(Rw_{pr1} + Rw_{pr2}, 0.85 \cdot Rw_{pr1} + 1.5 \cdot Rw_{pr2})$

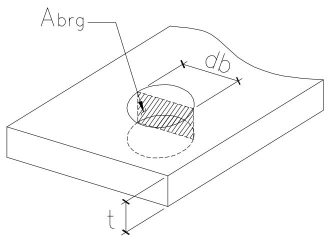
$Rw_{pr} = 80.13 \cdot kips$ $\qquad$ $F_{pr} = 73.171 \cdot kips$

RESULT = Weld Capacity > Force Applied, OK

D. PLATE REINFORCEMENT CHECK: Required

1. Bolt Bearing Capacity of Plate Reinforcement

(AISC 13th Ed. Specifications Chapter J, Section J3.10, page 16.1-111)



Bearing Area,

$Abrg_{pr} := db_{fp} \cdot tpr$

$Abrg_{pr} = 0.5 \cdot in^2$

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

$$Fbe := Fu_{pr} \cdot \begin{cases} \min[1.0 \cdot (Lev_{fp} - 0.5hd_{bmf}) \cdot tpr, 2.0 \cdot Abrg_{pr}] & \text{if } hd_{bmf} \ge hd_{ls} \\ \min[1.2 \cdot (Lev_{fp} - 0.5hd_{bmf}) \cdot tpr, 2.4 \cdot Abrg_{pr}] & \text{otherwise} \end{cases}$$

$Fbe = 32.625 \cdot kips$

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

$$Fbs := Fu_{pr} \cdot \begin{cases} \min[1.0 \cdot (s_{fp} - hd_{bmf}) \cdot tpr, 2.0 \cdot Abrg_{pr}] & \text{if } hd_{bmf} \ge hd_{ls} \\ \min[1.2 \cdot (s_{fp} - hd_{bmf}) \cdot tpr, 2.4 \cdot Abrg_{pr}] & \text{otherwise} \end{cases}$$

$Fbs = 65.25 \cdot kips$

Bolt Bearing Capacity,

$Rbrg_{pr} := 2\Lambda_{brg} nv_{fp} \cdot [Fbe + Fbs(nr_{fp} - 1)]$

$Rbrg_{pr} = 489.375 \cdot kips$ $\qquad$ $F_{pr} = 73.171 \cdot kips$

RESULT = Not Applicable, Reinforcement not Required

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2. Block Shear Capacity of Plate Reinforcement

(AISC 13th Ed. Specifications Chapter J, Section J4.3, pages 16.1-112 to 16.1-113)

Reduction Factor, $U_{bs} := 1.0$ *(Tension Stress is Uniform)*

Gross Shear Area

$$Agv := 2\left[\left(nr_{fp} - 1\right)\cdot s_{fp} + D_2\right]\cdot tpr$$

$$Agv = 24.75\cdot in^2$$

Net Shear Area

$$Anv := 2\cdot\left[\left(nr_{fp} - 1\right)\cdot s_{fp} + D_2 - \left(nr_{fp} - 0.5\right)\cdot hd_{bmf}\right]\cdot tpr$$

$$Anv = 16.312\cdot in^2$$

Net Tension Area

$$Ant := \left[\left(nv_{fp} - 1\right)\cdot sv_{fp} + g_{bm} - \left(2nv_{fp} - 1\right)\cdot hd_{bmf}\right]\cdot tpr$$

$$Ant = 2.187\cdot in^2$$

Block Shear Capacity,

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

$Rbs_{pr} = 330.737\cdot kips$ $F_{pr} = 73.171\cdot kips$

RESULT = Not Applicable, Reinforcement not Required

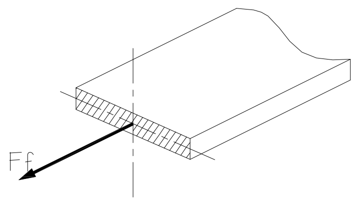
3. Tension Yielding Capacity of Plate Reinforcement

(AISC 13th Ed. Specifications Chapter D, Section D2, page 16.1-26)

Gross Tension Area,

$$Ag_{pr} := bpr\cdot tpr$$

$$Ag_{pr} = 4.313\cdot in^2$$



Tension Yielding Capacity,

$$Rty_{pr} := \Lambda_{ty}\cdot Fy_{pr}\cdot Ag_{pr}$$

$Rty_{pr} = 92.964\cdot kips$ $F_{pr} = 73.171\cdot kips$

RESULT = Not Applicable, Reinforcement not Required

4. Tension Rupture Capacity of Plate Reinforcement

(AISC 13th Ed. Chapter D, Section D2, page 16.1-27)

Net Tension Area,

$$Ant := \left(bpr - 2nv_{fp} \cdot hd_{fp}\right) \cdot tpr$$

$$Ant = 3.187 \cdot in^2$$

Effective Net Tension Area,

$$Ae := \min\left(Ant, 0.85 \cdot Ag_{pr}\right)$$

$$Ae = 3.187 \cdot in^2$$

Tension Rupture Capacity

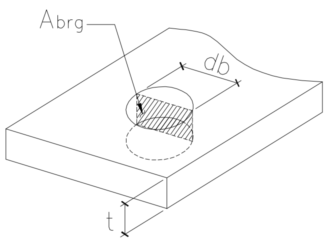
$$Rtr_{pr} := \Lambda_{tr} \cdot Fu_{pr} \cdot Ae$$

$$Rtr_{pr} = 92.437 \cdot kips \qquad F_{pr} = 73.171 \cdot kips$$

RESULT = Not Applicable, Reinforcement not Required

E. FLANGE PLATE CHECK

1. Bolt Bearing Capacity of Flange Plate



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

Bearing Area,

$$Abrg_{fp} := db_{fp1} \cdot tfp$$

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

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

$$Fbe := \Lambda_{brg} \cdot Fu_{fp} \cdot \begin{cases} \min\left[1.0 \cdot \left(Lev_{fp1} - 0.5hd_{fp1}\right) \cdot tfp, 2.0 \cdot Abrg_{fp}\right] & \text{if } hd_{fp1} \ge hd_{ls} \\ \min\left[1.2 \cdot \left(Lev_{fp1} - 0.5hd_{fp1}\right) \cdot tfp, 2.4 \cdot Abrg_{fp}\right] & \text{otherwise} \end{cases}$$

$$Fbe = 36.562 \cdot kips$$

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

$$Fbs := \Lambda_{brg} \cdot Fu_{fp} \cdot \begin{cases} \min\left[1.0 \cdot \left(s_{fp1} - hd_{fp1}\right) \cdot tfp, 2.0 \cdot Abrg_{fp}\right] & \text{if } hd_{fp1} \ge hd_{ls} \\ \min\left[1.2 \cdot \left(s_{fp1} - hd_{fp1}\right) \cdot tfp, 2.4 \cdot Abrg_{fp}\right] & \text{otherwise} \end{cases}$$

$$Fbs = 73.125 \cdot kips$$

Bolt Bearing Capacity,

$$Rbrg_{fp} := 2 \cdot nv_{fp1} \cdot \left[\min\left(Fbe, \Lambda rv_{fp1}\right) + \min\left(Fbs, \Lambda rv_{fp1}\right) \cdot \left(nr_{fp1} - 1\right) \right]$$

$Rbrg_{fp} = 231.424 \cdot kips$ $\qquad$ $Ff_1 = 205.26 \cdot kips$

RESULT = Bearing Capacity > Force Applied, OK

2. Block Shear Capacity of Flange Plate

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

Reduction Factor, $\quad U_{bs} := 1.0 \quad$ *(Tension Stress is Uniform)*

Flange Plate Width,

$bfp = 9.5 \cdot in$

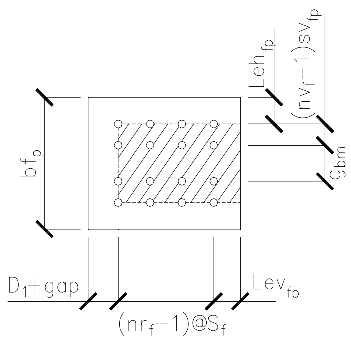
RESULT = NUMBER OF BOLTS, OK

Pattern 1

Gross Shear Area

$$Agv := 2 \cdot \left[\left(nr_{fp1} - 1\right) \cdot s_{fp1} + Lev_{fp1} \right] \cdot tfp$$

$Agv = 45 \cdot in^2$



Net Tension Area

$$Ant := \left[bfp - 2 \cdot Leh_{fp1} - \left(2nv_{fp1} - 1\right) hd_{fp1} \right] \cdot tfp$$

$Ant = 4.375 \cdot in^2$

Net Shear Area

$$Anv := 2 \cdot \left[\left(nr_{fp1} - 1\right) \cdot s_{fp1} + Lev_{fp1} - \left(nr_{fp1} - 0.5\right) hd_{fp1} \right] \cdot tfp$$

$Anv = 28.125 \cdot in^2$

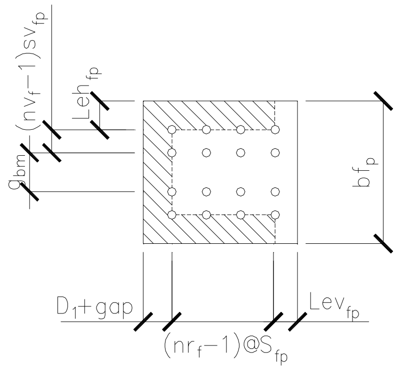
Block Shear Capacity

$$Rbs_{fp1} := \Lambda_{bs} \min\left(0.6 Fu_{fp} Anv + U_{bs} \cdot Fu_{fp} \cdot Ant, 0.6 \cdot Fy_{fp} \cdot Agv + U_{bs} \cdot Fu_{fp} \cdot Ant\right)$$

$Rbs_{fp1} = 690.625 \cdot kips$

Pattern 2

Gross Shear Area



$$Agv := 2\cdot\left(nr_{fp1} - 1\right)\cdot s_{fp1}\cdot tfp$$

$$Agv = 42\cdot in^{2}$$

Net Tension Area

$$Ant := \left(bfp - 2nv_{fp1}\cdot hd_{fp1}\right)\cdot tfp$$

$$Ant = 7.25\cdot in^{2}$$

Net Shear Area

$$Anv := 2\left[\left(nr_{fp1} - 1\right)\cdot s_{fp1} - \left(nr_{fp1} - 1\right)\cdot hd_{fp1}\right]\cdot tfp$$

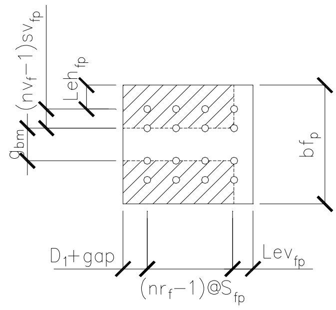
$$Anv = 26.25\cdot in^{2}$$

Block Shear Capacity

$$Rbs_{fp2} := \Lambda_{bs}\min\left(0.6Fu_{fp}\cdot Anv + U_{bs}\cdot Fu_{fp}\cdot Ant\,,\,0.6\cdot Fy_{fp}\cdot Agv + U_{bs}\cdot Fu_{fp}\cdot Ant\right)$$

$$Rbs_{fp2} = 747.5\cdot kips$$

Pattern 3



Gross Shear Area

$$Agv := 2\cdot\left[\left(nr_{fp1} - 1\right)\cdot s_{fp1} + Lev_{fp1}\right]\cdot tfp$$

$$Agv = 45\cdot in^{2}$$

Net Tension Area

$$Ant := \left[bfp - g_{bm1} - \left(2nv_{fp1} - 1\right)hd_{fp1}\right]\cdot tfp$$

$$Ant = 2.875\cdot in^{2}$$

Net Shear Area

$$Anv := 2\cdot\left[\left(nr_{fp1} - 1\right)\cdot s_{fp1} + Lev_{fp1} - \left(nr_{fp1} - 0.5\right)hd_{fp1}\right]\cdot tfp$$

$$Anv = 28.125\cdot in^{2}$$

Block Shear Capacity

$$Rbs_{fp3} := \Lambda_{bs}\min\left(0.6Fu_{fp}Anv + U_{bs}\cdot Fu_{fp}\cdot Ant\,,\,0.6\cdot Fy_{fp}\cdot Agv + U_{bs}\cdot Fu_{fp}\cdot Ant\right)$$

$$Rbs_{fp3} = 641.875\cdot kips$$

Governing Block Shear Capacity

$Rbs_{fp} := \min\left(Rbs_{fp1}, Rbs_{fp2}, Rbs_{fp3}\right)$

$Rbs_{fp} = 641.875 \cdot kips$ $\qquad Ff_1 = 205.26 \cdot kips$

RESULT = Block Shear Capacity > Force Applied, OK

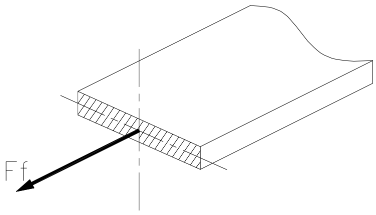
3. Tension Yielding Capacity of Flange Plate

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

Gross Tension Area,

$Ag_{fp} := bfp \cdot tfp$

$Ag_{fp} = 9.5 \cdot in^2$



Tension Yielding Capacity,

$Rty_{fp} := \Lambda_{ty} \cdot Fy_{fp} \cdot Ag_{fp}$

$Rty_{fp} = 284.431 \cdot kips$ $\qquad Ff_1 = 205.26 \cdot kips$

RESULT = Tension Yielding Capacity > Force Applied, OK

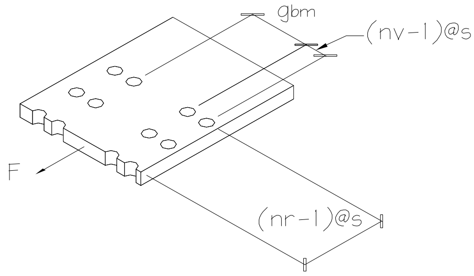
4. Tension Rupture Capacity of Flange Plate

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

Net Tension Area,

$Ant := \left(bfp - 2nv_{fp1} \cdot hd_{fp1}\right) \cdot tfp$

$Ant = 7.25 \cdot in^2$



Effective Net Tension Area,

$Ae := \min\left(Ant, 0.85 \cdot Ag_{fp}\right)$

$Ae = 7.25 \cdot in^2$

Tension Rupture Capacity

$Rtr_{fp} := \Lambda_{tr} \cdot Fu_{fp} \cdot Ae$

$Rtr_{fp} = 235.625 \cdot kips$ $\qquad Ff_1 = 205.26 \cdot kips$

RESULT = Tension Rupture Capacity > Force Applied, OK

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5. Flange Plate Compression Capacity

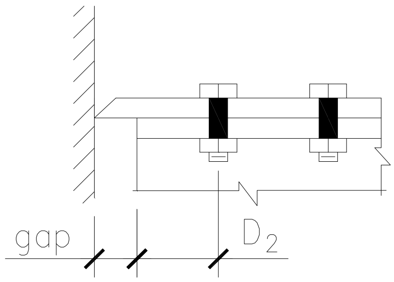
(AISC 14th Ed. Specifications, Chapter J, Section J4.4, pages 16.1-129 to 16.1-130)

Effective Length Factor,

$K = 0.65$ *(Commentary Table C-A7.1)*

Laterally Unbraced Length of Plate,

$L := D_3 + gap$



$L = 3.5 \cdot in$

Radius of Gyration,

$$r := \frac{tfp}{\sqrt{12}}$$

$r = 0.289 \cdot in$

Slenderness Ratio,

$$KLr := \frac{K L}{r} = 7.881$$

Elastic Critical Buckling Stress,

$$F_e := \frac{\pi^2 \cdot E}{KLr^2}$$

$F_e = 4608.437 \cdot ksi$

Compression Buckling Stress,

$$F_{cr} := \begin{cases} Fy_{fp} & \text{if } KLr \le 25 \\ \text{otherwise} & \\ \quad \begin{cases} 0.658^{\frac{Fy_{fp}}{F_e}} \cdot Fy_{fp} & \text{if } KLr \le 4.71 \cdot \sqrt{\dfrac{E}{Fy_{fp}}} \\ 0.877 \cdot F_e & \text{otherwise} \end{cases} \end{cases}$$

$F_{cr} = 50 \cdot ksi$

Compression Capacity,

$$Rcb_{fp} := \Lambda_c \cdot F_{cr} \cdot bfp \cdot tfp$$

$$Rcb_{fp} = 284.431 \cdot kips \qquad Ff_1 = 205.26 \cdot kips$$

RESULT = Compression Capacity > Force Applied, OK

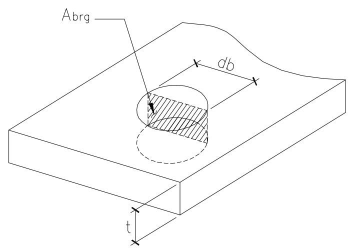
F. FLANGE PLATE TO GIRDER CHECK

(*Please refer to Section* P. **Flange Plate to Girder Check**)

G. BEAM 1 WEB 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,

$$Abrg_{bm} := db_{pl1} \cdot tw_{bm1}$$

$$Abrg_{bm} = 0.45 \cdot in^2$$

Bolt centerline distance from face of support,

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

$$a_b = 10.42 \cdot in$$

Eccentric Load Coefficient,

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

$$C = 7.564$$

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

$$Fbe := Fu_{bm1} \cdot \begin{cases} \min\left[1.0 \cdot \left(D_1 - 0.5 \cdot hd_{bm1w}\right) \cdot tw_{bm1}, 2.0 \cdot Abrg_{bm}\right] & \text{if } hd_{bm1w} \ge hd_{ls} \\ \min\left[1.2 \cdot \left(D_1 - 0.5 \cdot hd_{bm1w}\right) \cdot tw_{bm1}, 2.4 \cdot Abrg_{bm}\right] & \text{otherwise} \end{cases}$$

$$Fbe = 70.2 \cdot kips$$

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

$$Fbs := Fu_{bm1} \cdot \begin{cases} \min\left[1.0 \cdot \left(s_{pl1} - hd_{bm1w}\right) \cdot tw_{bm1}, 2.0 \cdot Abrg_{bm}\right] & \text{if } hd_{bm1w} \ge hd_{ls} \\ \min\left[1.2 \cdot \left(s_{pl1} - hd_{bm1w}\right) \cdot tw_{bm1}, 2.4 \cdot Abrg_{bm}\right] & \text{otherwise} \end{cases}$$

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$Fbs = 57.037 \cdot kips$

Bolt Bearing Capacity,

$$Rbrg_{bm} := \Lambda_{brg} \cdot \frac{C}{nr} \cdot [Fbe + Fbs(nr - 1)]$$

$Rbrg_{bm} = 225.673 \cdot kips$ $\qquad$ $V_{bm1} = 102.2 \cdot kips$

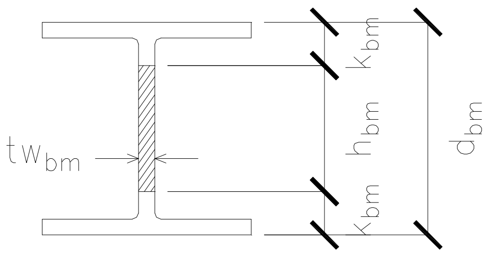
RESULT = Bearing Capacity > Force Applied,OK

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,

$$h := d_{bm1} - 2 \cdot kdes_{bm1}$$



$h = 16.1 \cdot in$

Limiting depth-thickness ratio,

$$h_{tw} := \frac{h}{tw_{bm1}}$$

$h_{tw} = 35.778$

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,

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

$kv = 5$

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Web shear coefficient,

$$Cv := \begin{cases} 1 & \text{if } h_{tw} \le 1.1 \cdot \sqrt{\dfrac{kv \cdot E}{Fy_{bm1}}} \\ \dfrac{1.1 \cdot \sqrt{\dfrac{kv \cdot E}{Fy_{bm1}}}}{h_{tw}} & \text{if } 1.1 \cdot \sqrt{\dfrac{kv \cdot E}{Fy_{bm1}}} < h_{tw} \le 1.37 \cdot \sqrt{\dfrac{kv \cdot E}{Fy_{bm1}}} \\ \dfrac{1.51 \cdot E \cdot kv}{h_{tw}^{2} Fy_{bm1}} & \text{if } 1.37 \cdot \sqrt{\dfrac{kv \cdot E}{Fy_{bm1}}} < h_{tw} \end{cases}$$

(G2-3)

(G2-4)

(G2-5)

$Cv = 1$

Shear Capacity of Section,

$$Rv_{bm} := \Lambda_{vbm} \cdot 0.6 \cdot Fy_{bm1} \cdot d_{bm1} \cdot tw_{bm1} \cdot Cv \quad \textit{(G2-1)}$$

$Rv_{bm} = 165.6 \cdot kips$ $\qquad$ $V_{bm1} = 102.2 \cdot kips$

RESULT = Shear Capacity of Section > Force Applied, OK

H. BEAM 1 TO SHEAR PLATE CHECK

1. Direct Bolt Shear Capacity

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

Shear Capacity per bolt,

$\Lambda rv_{pl1} = 14.464 \cdot kips$

Bolt Shear Capacity,

$Rb_{pl} := C \cdot \Lambda rv_{pl1}$

$Rb_{pl} = 109.406 \cdot kips$ $\qquad$ $V_{bm1} = 102.2 \cdot kips$

RESULT = Bolt Shear Capacity > Force Applied, OK

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I. SHEAR PLATE 1 CHECK

1. Check for Maximum Shear Plate Thickness

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

Exceptions for nv = 1 and nv = 2

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

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

RESULT = Check maximum thickness of plate

Coefficient for Eccentrically Loaded Bolts

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

$C' = 91.406 \cdot in$

Area of Bolts

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

$Ab = 0.785 \cdot in^2$

Length of Plate

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

$Lpl = 14 \cdot in$

Maximum Thickness

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

$tpl_{max} = 1.349 \cdot in$ $\qquad$ $tpl = 1 \cdot in$

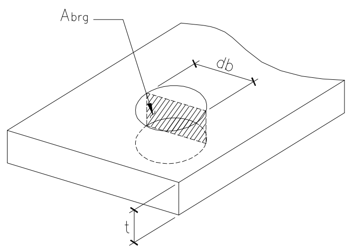
RESULT = Plate Thickness < Max Thickness Permitted,OK

Governing Shear Plate Thickness

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

$$tpl_g = 1 \cdot in$$

2. Bolt Bearing Capacity of Shear Plate



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

Area of Bearing Surface,

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

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

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

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

$$Fbe = 73.125 \cdot kips$$

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

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

$$Fbs = 126.75 \cdot kips$$

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Bolt Bearing Capacity,

$$Rbrg_{pl} := \Lambda_{brg} \cdot \frac{C}{nr} \cdot [Fbe + Fbs(nr - 1)]$$

$Rbrg_{pl} = 438.808 \cdot kips$ $\qquad V = 102.2 \cdot kips$

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,

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

$Lpl = 14 \cdot in$

Check if Length of Plate is acceptable,

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

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

Length = "Plate Length is OK per AISC Requirements"

Gross Shear Capacity,

$$Rvy_{pl} := \Lambda_{vy} \cdot 0.6 \cdot Fy_{pl} \cdot tpl_g \cdot Lpl \qquad (J4\text{-}3)$$

$Rvy_{pl} = 280 \cdot kips$ $\qquad V = 102.2 \cdot kips$

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,

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

$Anv = 8.375 \cdot in^2$

Shear Rupture Capacity,

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

$Rvr_{pl} = 163.313 \cdot kips$ $V = 102.2 \cdot kips$

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)

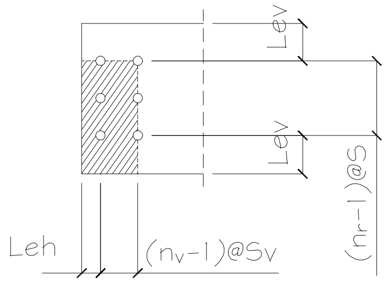
Reduction Factor, $U_{bs} := \begin{cases} 1.0 & \text{if } nv = 1 \\ 0.5 & \text{if } nv > 1 \end{cases}$

(tension stress is uniform)

(tension stress is non-uniform)

$U_{bs} = 0.5$

Gross Shear Area



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

$Agv = 12.5 \cdot in^2$

Net Tension Area

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

$Ant = 5.687 \cdot in^2$

Net Shear Area

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

$Anv = 7.438 \cdot in^2$

Block Shear Capacity of Plate, (J4-5)

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

$Rbs_{pl} = 237.453 \cdot kips$ $V = 102.2 \cdot kips$

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,

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

$$a_b = 5.92 \cdot in$$

Coefficient,

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

$$\lambda = 0.219$$

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

$$Q = 1$$

Allowable Buckling Stress,

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

$$Fcr = 50 \cdot ksi$$

Gross Plastic Section Modulus,

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

$$Zx_{pl} = 49 \cdot in^3$$

Eccentricity,

$$e_{pl} := a_b$$

$$e_{pl} = 5.92 \cdot in$$

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Buckling Capacity,

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

$Rbc_{pl} = 247.815 \cdot kips$ $\qquad$ $V = 102.2 \cdot kips$

RESULT = Local Buckling Capacity will not Control!

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,

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

$Rfc_{pl} = 185.409 \cdot kips$ $\qquad$ $V = 102.2 \cdot kips$

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,

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

$Znet_{pl} = 30.121 \cdot in^3$

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Flexural Rupture Capacity,

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

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

$Rfr_{pl} = 165.361 \cdot kips$ $\qquad V = 102.2 \cdot kips$

RESULT = Flexural Rupture Capacity > Applied Force, OK

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,

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

$V_r := V$

$V_r = 102.2 \cdot kips$

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

$M_r = 1064.924 \cdot kips \cdot in$

Shear yielding,

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

$V_c = 280 \cdot kips$

Flexural yielding,

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

$M_c = 1467.066 \cdot kips \cdot in$

Interaction,

$$\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{M_r}{M_c}\right)^2 = 0.66$$

RESULT = Interaction < 1.0, OK

J. <u>SHEAR PLATE 1 TO GIRDER</u>

1. Weld Capacity of Shear Plate to Girder Web

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

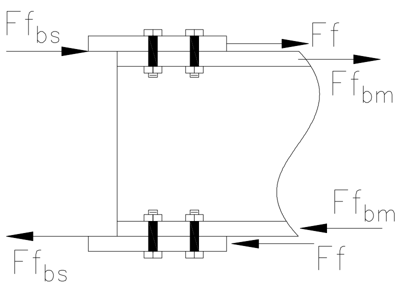
No. of Weld side, $n_{ws} := 2$

Minimum Weld Size,

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

RESULT = Preferred Weld Size > Minimum Weld Size, OK

K. <u>BOLTS ON FLANGE PLATE CHECK</u>



1. Forces acting on the Connection

@ Beam Flange,

$$Ff_{bm2} := \frac{P_{bm2}}{2} + \frac{M_{bm2}}{d_{bm2} - tf_{bm2}}$$

$$Ff_{bm2} = 225.611 \cdot kips$$

@ Interface of Beam Flange & Flange Plate,

$$Ff_{bs2} := \frac{P_{bm2}}{2} + \frac{M_{bm2}}{d_{bm2}}$$

$$Ff_{bs2} = 216.415 \cdot kips$$

@ Flange Plate,

$$Ff_2 := \frac{P_{bm2}}{2} + \frac{M_{bm2}}{d_{bm2} + tfp}$$

$$Ff_2 = 205.26 \cdot kips$$

2. Bolt Shear Capacity

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

Shear Capacity per bolt,

$$\Lambda rv_{fp2} = 14.464 \cdot kips$$

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Bolt Shear Capacity,

$Rb_{fp} := n_{fp2} \cdot \Lambda rv_{fp2}$

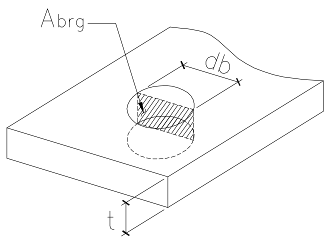
$Rb_{fp} = 231.424 \cdot kips$ $\qquad Ff_{bs2} = 216.415 \cdot kips$

RESULT = Bolt Shear Capacity > Force Applied, OK

L. BEAM FLANGE CHECK

1. Bolt Bearing on Beam Flange

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



Bearing Area,

$Abrg_{bm} := db_{fp2} \cdot tf_{bm2}$

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

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

$$Fbe := \Lambda_{brg} \cdot Fu_{bm2} \cdot \begin{cases} \min\left[1.0 \cdot \left(D_4 - 0.5hd_{bm2f}\right) \cdot tf_{bm2},\ 2.0 \cdot Abrg_{bm}\right] & \text{if } hd_{bm2f} \ge hd_{ls} \\ \min\left[1.2 \cdot \left(D_4 - 0.5hd_{bm2f}\right) \cdot tf_{bm2},\ 2.4 \cdot Abrg_{bm}\right] & \text{otherwise} \end{cases}$$

$Fbe = 58.5 \cdot kips$

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

$$Fbs := \Lambda_{brg} \cdot Fu_{bm2} \cdot \begin{cases} \min\left[1.0 \cdot \left(s_{fp2} - hd_{bm2f}\right) \cdot tf_{bm2},\ 2.0 \cdot Abrg_{bm}\right] & \text{if } hd_{bm2f} \ge hd_{ls} \\ \min\left[1.2 \cdot \left(s_{fp2} - hd_{bm2f}\right) \cdot tf_{bm2},\ 2.4 \cdot Abrg_{bm}\right] & \text{otherwise} \end{cases}$$

$Fbs = 54.844 \cdot kips$

Bolt Bearing Capacity,

$Rbrg_{bmf} := 2 \cdot nv_{fp2} \cdot \left[\min\left(Fbe, \Lambda rv_{fp2}\right) + \min\left(Fbs, \Lambda rv_{fp2}\right) \cdot \left(nr_{fp2} - 1\right)\right]$

$Rbrg_{bmf} = 231.424 \cdot kips$ $\qquad Ff_{bm2} = 225.611 \cdot kips$

RESULT = Bearing Capacity > Force Applied, OK

2. Block Shear Capacity of Beam Flange

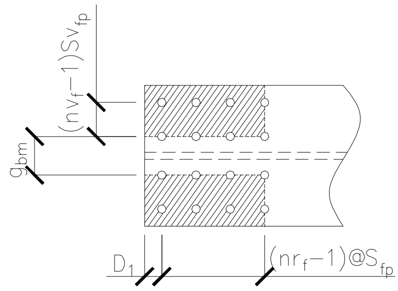
(AISC 14th Ed. Specifications, Chapter J, Section J4.3, page 16.1-129)

Reduction Factor, $U_{bs} := 1.0$ *(Tension Stress is Uniform)*

Gross Shear Area

$$Agv := 2 \cdot \left[\left(nr_{fp2} - 1 \right) \cdot s_{fp2} + D_4 \right] \cdot tf_{bm2}$$

$$Agv = 36 \cdot in^2$$



Net Tension Area

$$Ant := \left[\left(bf_{bm2} - g_{bm2} \right) - \left(2nv_{fp2} - 1 \right) \cdot hd_{bm2f} \right] \cdot tf_{bm2}$$

$$Ant = 0.724 \cdot in^2$$

Net Shear Area

$$Anv := 2 \cdot \left[\left(nr_{fp2} - 1 \right) \cdot s_{fp2} + D_4 - \left(nr_{fp2} - 0.5 \right) \cdot hd_{bm2f} \right] \cdot tf_{bm2}$$

$$Anv = 23.344 \cdot in^2$$

Block Shear Capacity,

$$Rbs_{bmf} := \Lambda_{bs} \min\left(0.6Fu_{bm2} \cdot Anv + U_{bs} \cdot Fu_{bm2} \cdot Ant, 0.6 \cdot Fy_{bm2} \cdot Agv + U_{bs} \cdot Fu_{bm2} \cdot Ant \right)$$

$Rbs_{bmf} = 478.725 \cdot kips$ $Ff_{bm2} = 225.611 \cdot kips$

RESULT = Block Shear Capacity > Force Applied, OK

3. Beam Flexural Strength on Reduced Area w/ Reinforcement

(AISC 14th Ed. Specifications Chapter F, Section F13, page 16.1-64)

Effective Flange Thickness,

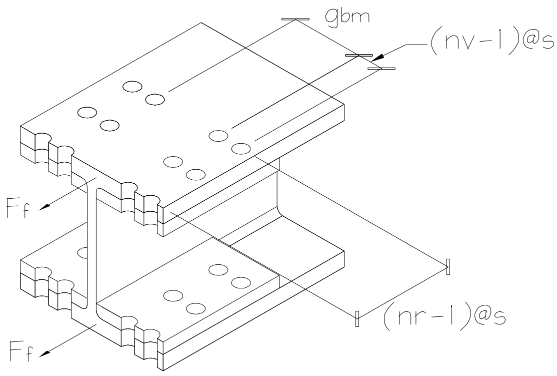
$$tf_{bmpr} := \begin{cases} tf_{bm2} & \text{if Reinforcement2 = "Not Required"} \\ tf_{bm2} + tpr2 \cdot \left(\dfrac{Fy_{pr2}}{Fy_{bm2}} \right) & \text{otherwise} \end{cases}$$

$$tf_{bmpr} = 1.11 \cdot in$$

a. *Gross Tension Flange Area*

$$A_{fg} := bf_{bm2} \cdot tf_{bmpr}$$

$$A_{fg} = 8.425 \cdot in^2$$



b. *Net Tension Area*

$$A_{fn} := A_{fg} - \left(2nv_{fp2} \cdot hd_{bm2f} \cdot tf_{bmpr}\right)$$

$$A_{fn} = 5.927 \cdot in^2$$

c. *Value of Yt*

$$Y_t := \begin{vmatrix} 1.0 & \text{if } \dfrac{Fy_{bm2}}{Fu_{bm2}} \le 0.8 \\ 1.1 & \text{otherwise} \end{vmatrix}$$ (F13-1)

$$Y_t = 1$$

d. *Section Modulus of Section*

$$Ix_{bmpr} := 2\left[\frac{bf_{bm2} \cdot tf_{bmpr}^{\ 3}}{12} + bf_{bm2} \cdot tf_{bmpr} \cdot \left(\frac{d_{bm2} - tf_{bmpr}}{2}\right)^2\right] \dots + \frac{tw_{bm2} \cdot \left(d_{bm2} - 2tf_{bmpr}\right)^3}{12}$$

$$Ix_{bmpr} = 1419.86 \cdot in^4$$

$$Sx_{bmpr} := \begin{vmatrix} Sx_{bm2} & \text{if Reinforcement2} = \text{"Not Required"} \\ \dfrac{Ix_{bmpr}}{0.5\left(d_{bm2}\right)} & \text{otherwise} \end{vmatrix}$$

$$Sx_{bmpr} = 154.333 \cdot in^3$$

e. *Revised Flexural Requirement Due to Axial Load*

$$M_{rev2} := M_{bm2} + 0.5P_{bm2} \cdot \left(d_{bm2} - tf_{bm2}\right)$$

$$M_{rev2} = 331.836 \cdot kips \cdot ft$$

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f. Tension Rupture Capacity

Net Moment Capacity,

$$M_n := \frac{\Lambda_b \cdot Fu_{bm2} \cdot A_{fn} \cdot Sx_{bmpr}}{A_{fg}}$$ *(F13-1)*

$M_n = 352.187 \cdot kips \cdot ft$ $M_{rev2} = 331.836 \cdot kips \cdot ft$

RESULT = Net Moment Capacity > Applied Moment, OK

M. PLATE REINFORCEMENT CONNECTION TO BEAM: **Required**

1. Force Acting on Plate Reinforcement

Force on Flange,

$$Ff_{bmi} := \frac{tf_{bm2}}{tf_{bmpr}} \cdot Ff_{bm2}$$

$Ff_{bmi} = 152.44 \cdot kips$

Force on Plate Reinforcement,

$F_{pr} := Ff_{bm2} - Ff_{bmi}$

$F_{pr} = 73.171 \cdot kips$

2. Weld Capacity of Plate Reinforcement to Beam

(AISC Specifications 13th Ed, Chapter J, Section J2.2b, Table J2.4, pages 16.1-90 to 16.1-102)

Length of weld,

$Lw_{pr1} := Lw_4$

$Lw_{pr1} = 6 \cdot in$

$Lw_{pr2} := bf_{bm}$

No. of Weld side, $n_{ws} := 1$

Minimum weld size,

$w_{min} = \frac{3}{16} \cdot in$ $w_4 = \frac{1}{4} \cdot in$

RESULT = Preferred Weld Size > Minimum Weld Size, OK

Maximum weld size,

$w_{max} = \frac{7}{16} \cdot in$ $w_4 = \frac{1}{4} \cdot in$

RESULT = Maximum Weld Size > Preferred Weld Size, OK

a. Plate Reinforcement to Beam Flange Using Fillet Weld

For Longitudinal Weld

Shear Strength,

For Beam:

$$Rv_{bm} := \Lambda_{vr} \cdot 0.6 \cdot Fu_{bm} \cdot tf_{bm}$$

$$Rv_{bm} = 14.625 \cdot \frac{kips}{in}$$

For Plate Reinforcement:

$$Rv_{pr} := \Lambda_{vr} \cdot 0.6 \cdot Fu_{pr} \cdot tpr \cdot n_{ws}$$

$$Rv_{pr} = 8.7 \cdot \frac{kips}{in}$$

For Weld:

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

$$Rv_w = 14.849 \cdot ksi$$

Maximum effective weld size,

$$w_{eff} := \frac{\min(Rv_{bm}, Rv_{pr})}{Rv_w}$$

$$w_{eff} = 0.586 \cdot in$$

Weld Capacity,

$$Rw_{pr1} := 2\Lambda_{vw} \cdot 0.60 \cdot Fu_w \cdot \sin(45 \cdot deg) \cdot n_{ws} \cdot \min(w_{eff}, w_4) \cdot Lw_{pr1}$$

$$Rw_{pr1} = 44.548 \cdot kips$$

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For Transverse Weld

Shear Strength,

For Beam:

$$Rv_{bm} := \Lambda_{vr} \cdot 0.6 \cdot Fu_{bm} \cdot tf_{bm}$$

$$Rv_{bm} = 14.625 \cdot \frac{kips}{in}$$

For Plate Reinforcement:

$$Rv_{pr} := \Lambda_{vr} \cdot 0.6 \cdot Fu_{pr} \cdot tpr \cdot n_{ws}$$

$$Rv_{pr} = 8.7 \cdot \frac{kips}{in}$$

For Weld:

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

$$Rv_w = 14.849 \cdot ksi$$

Maximum effective weld size,

$$w_{eff} := \frac{\min(Rv_{bm}, Rv_{pr})}{Rv_w}$$

$$w_{eff} = 0.586 \cdot in$$

Weld Capacity,

$$Rw_{pr2} := \Lambda_{vw} \cdot 0.60 \cdot Fu_w \cdot \sin(45 \cdot deg) \cdot n_{ws} \cdot \min(w_{eff}, w_4) \cdot Lw_{pr2}$$

$$Rw_{pr2} = 28.176 \cdot kips$$

$$Rw_{pr} := \max(Rw_{pr1} + Rw_{pr2}, 0.85 \cdot Rw_{pr1} + 1.5 \cdot Rw_{pr2})$$

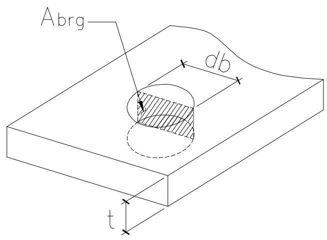
$Rw_{pr} = 80.13 \cdot kips$ $F_{pr} = 73.171 \cdot kips$

RESULT = Weld Capacity > Force Applied, OK

N. PLATE REINFORCEMENT CHECK: Required

1. Bolt Bearing Capacity of Plate Reinforcement

(AISC 13th Ed. Specifications Chapter J, Section J3.10, page 16.1-111)



Bearing Area,

$$Abrg_{pr} := db_{fp} \cdot tpr$$

$$Abrg_{pr} = 0.5 \cdot in^2$$

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

$$Fbe := Fu_{pr} \cdot \begin{cases} \min\left[1.0 \cdot \left(Lev_{fp} - 0.5hd_{bmf}\right) \cdot tpr, 2.0 \cdot Abrg_{pr}\right] & \text{if } hd_{bmf} \geq hd_{ls} \\ \min\left[1.2 \cdot \left(Lev_{fp} - 0.5hd_{bmf}\right) \cdot tpr, 2.4 \cdot Abrg_{pr}\right] & \text{otherwise} \end{cases}$$

$$Fbe = 32.625 \cdot kips$$

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

$$Fbs := Fu_{pr} \cdot \begin{cases} \min\left[1.0 \cdot \left(s_{fp} - hd_{bmf}\right) \cdot tpr, 2.0 \cdot Abrg_{pr}\right] & \text{if } hd_{bmf} \geq hd_{ls} \\ \min\left[1.2 \cdot \left(s_{fp} - hd_{bmf}\right) \cdot tpr, 2.4 \cdot Abrg_{pr}\right] & \text{otherwise} \end{cases}$$

$$Fbs = 65.25 \cdot kips$$

Bolt Bearing Capacity,

$$Rbrg_{pr} := 2\Lambda_{brg} \, nv_{fp} \cdot \left[Fbe + Fbs\left(nr_{fp} - 1\right)\right]$$

$$Rbrg_{pr} = 489.375 \cdot kips \qquad F_{pr} = 73.171 \cdot kips$$

RESULT = Bearing Capacity > Force Applied, OK

2. Block Shear Capacity of Plate Reinforcement

(AISC 13th Ed. Specifications Chapter J, Section J4.3, pages 16.1-112 to 16.1-113)

Reduction Factor, $U_{bs} := 1.0$ *(Tension Stress is Uniform)*

Gross Shear Area

$$Agv := 2\left[\left(nr_{fp} - 1\right) \cdot s_{fp} + D_2\right] \cdot tpr$$

$$Agv = 24.75 \cdot in^2$$

Net Shear Area

$$Anv := 2\cdot\left[\left(nr_{fp} - 1\right)\cdot s_{fp} + D_2 - \left(nr_{fp} - 0.5\right)\cdot hd_{bmf}\right]\cdot tpr$$

$$Anv = 16.312\cdot in^2$$

Net Tension Area

$$Ant := \left[\left(nv_{fp} - 1\right)\cdot sv_{fp} + g_{bm} - \left(2nv_{fp} - 1\right)\cdot hd_{bmf}\right]\cdot tpr$$

$$Ant = 2.187\cdot in^2$$

Block Shear Capacity,

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

$Rbs_{pr} = 330.737\cdot kips$ $\qquad F_{pr} = 73.171\cdot kips$

RESULT = Block Shear Capacity > Force Applied, OK

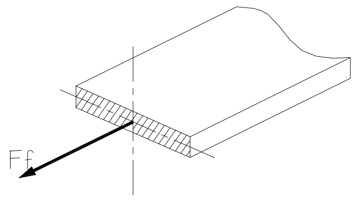
3. Tension Yielding Capacity of Plate Reinforcement

(AISC 13th Ed. Specifications Chapter D, Section D2, page 16.1-26)

Gross Tension Area,

$$Ag_{pr} := bpr\cdot tpr$$

$$Ag_{pr} = 4.313\cdot in^2$$



Tension Yielding Capacity,

$$Rty_{pr} := \Lambda_{ty}\cdot Fy_{pr}\cdot Ag_{pr}$$

$Rty_{pr} = 92.964\cdot kips$ $\qquad F_{pr} = 73.171\cdot kips$

RESULT = Tension Yielding Capacity > Force Applied, OK

4. Tension Rupture Capacity of Plate Reinforcement

(AISC 13th Ed. Chapter D, Section D2, page 16.1-27)

Net Tension Area,

$$Ant := \left(bpr - 2nv_{fp}\cdot hd_{fp}\right)\cdot tpr$$

$$Ant = 3.187\cdot in^2$$

Effective Net Tension Area,

$$Ae := \min\left(Ant\,,\,0.85\cdot Ag_{pr}\right)$$

$$Ae = 3.187\cdot in^2$$

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Tension Rupture Capacity

$Rtr_{pr} := \Lambda_{tr} \cdot Fu_{pr} \cdot Ae$

$Rtr_{pr} = 92.437 \cdot kips$ $\qquad F_{pr} = 73.171 \cdot kips$

RESULT = Tension Rupture Capacity > Force Applied, OK

O. <u>FLANGE PLATE CHECK</u>

1. Bolt Bearing Capacity of Flange Plate

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

Bearing Area,

$Abrg_{fp} := db_{fp2} \cdot tfp$

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

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

$$Fbe := \Lambda_{brg} \cdot Fu_{fp} \cdot \begin{cases} \min\left[1.0 \cdot \left(Lev_{fp2} - 0.5hd_{fp2}\right) \cdot tfp\, , 2.0 \cdot Abrg_{fp}\right] & \text{if } hd_{fp2} \ge hd_{ls} \\ \min\left[1.2 \cdot \left(Lev_{fp2} - 0.5hd_{fp2}\right) \cdot tfp\, , 2.4 \cdot Abrg_{fp}\right] & \text{otherwise} \end{cases}$$

$Fbe = 36.562 \cdot kips$

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

$$Fbs := \Lambda_{brg} \cdot Fu_{fp} \cdot \begin{cases} \min\left[1.0 \cdot \left(s_{fp2} - hd_{fp2}\right) \cdot tfp\, , 2.0 \cdot Abrg_{fp}\right] & \text{if } hd_{fp2} \ge hd_{ls} \\ \min\left[1.2 \cdot \left(s_{fp2} - hd_{fp2}\right) \cdot tfp\, , 2.4 \cdot Abrg_{fp}\right] & \text{otherwise} \end{cases}$$

$Fbs = 73.125 \cdot kips$

Bolt Bearing Capacity,

$$Rbrg_{fp} := 2 \cdot nv_{fp2} \cdot \left[\min\left(Fbe\, , \Lambda rv_{fp2}\right) + \min\left(Fbs\, , \Lambda rv_{fp2}\right) \cdot \left(nr_{fp2} - 1\right)\right]$$

$Rbrg_{fp} = 231.424 \cdot kips$ $\qquad Ff_2 = 205.26 \cdot kips$

RESULT = Bearing Capacity > Force Applied, OK

2. Block Shear Capacity of Flange Plate

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

Reduction Factor, $\qquad U_{bs} := 1.0$ *(Tension Stress is Uniform)*

Flange Plate Width,

$bfp = 9.5 \cdot in$

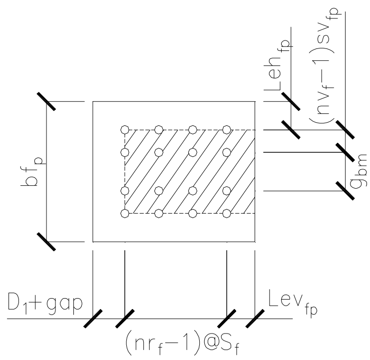
RESULT = NUMBER OF BOLTS, OK

Pattern 1

Gross Shear Area

$$Agv := 2\cdot\left[\left(nr_{fp2} - 1\right)\cdot s_{fp2} + Lev_{fp2}\right]\cdot tfp$$

$$Agv = 45\cdot in^2$$



Net Tension Area

$$Ant := \left[bfp - 2\cdot Leh_{fp2} - \left(2nv_{fp2} - 1\right)hd_{fp2}\right]\cdot tfp$$

$$Ant = 4.375\cdot in^2$$

Net Shear Area

$$Anv := 2\cdot\left[\left(nr_{fp2} - 1\right)\cdot s_{fp2} + Lev_{fp2} - \left(nr_{fp2} - 0.5\right)hd_{fp2}\right]\cdot tfp$$

$$Anv = 28.125\cdot in^2$$

Block Shear Capacity

$$Rbs_{fp1} := \Lambda_{bs}\min\left(0.6Fu_{fp}Anv + U_{bs}\cdot Fu_{fp}\cdot Ant, 0.6\cdot Fy_{fp}\cdot Agv + U_{bs}\cdot Fu_{fp}\cdot Ant\right)$$

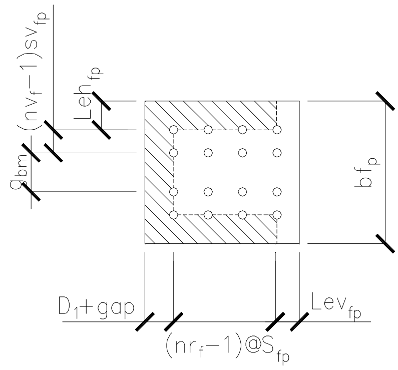
$$Rbs_{fp1} = 690.625\cdot kips$$

Pattern 2

Gross Shear Area

$$Agv := 2\cdot\left(nr_{fp2} - 1\right)\cdot s_{fp2}\cdot tfp$$

$$Agv = 42\cdot in^2$$



Net Tension Area

$$Ant := \left(bfp - 2nv_{fp2}\cdot hd_{fp2}\right)\cdot tfp$$

$$Ant = 7.25\cdot in^2$$

Net Shear Area

$$Anv := 2\left[\left(nr_{fp2} - 1\right)\cdot s_{fp2} - \left(nr_{fp2} - 1\right)\cdot hd_{fp2}\right]\cdot tfp$$

$$Anv = 26.25\cdot in^2$$

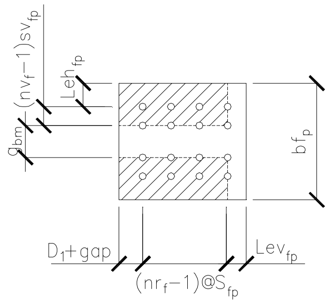
Block Shear Capacity

$$Rbs_{fp2} := \Lambda_{bs}\min\left(0.6Fu_{fp}\cdot Anv + U_{bs}\cdot Fu_{fp}\cdot Ant, 0.6\cdot Fy_{fp}\cdot Agv + U_{bs}\cdot Fu_{fp}\cdot Ant\right)$$

$$Rbs_{fp2} = 747.5\cdot kips$$

Pattern 3

Gross Shear Area



$$Agv := 2\cdot\left[\left(nr_{fp2}-1\right)\cdot s_{fp2} + Lev_{fp2}\right]\cdot tfp$$

$$Agv = 45\cdot in^2$$

Net Tension Area

$$Ant := \left[bfp - g_{bm2} - \left(2nv_{fp2}-1\right)hd_{fp2}\right]\cdot tfp$$

$$Ant = 2.875\cdot in^2$$

Net Shear Area

$$Anv := 2\cdot\left[\left(nr_{fp2}-1\right)\cdot s_{fp2} + Lev_{fp2} - \left(nr_{fp2}-0.5\right)hd_{fp2}\right]\cdot tfp$$

$$Anv = 28.125\cdot in^2$$

Block Shear Capacity

$$Rbs_{fp3} := \Lambda_{bs}\min\left(0.6Fu_{fp}Anv + U_{bs}\cdot Fu_{fp}\cdot Ant, 0.6\cdot Fy_{fp}\cdot Agv + U_{bs}\cdot Fu_{fp}\cdot Ant\right)$$

$$Rbs_{fp3} = 641.875\cdot kips$$

Governing Block Shear Capacity

$$Rbs_{fp} := \min\left(Rbs_{fp1}, Rbs_{fp2}, Rbs_{fp3}\right)$$

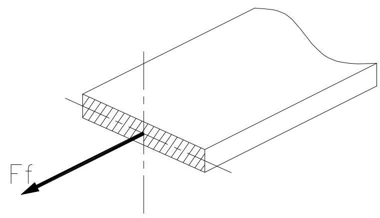
$$Rbs_{fp} = 641.875\cdot kips \qquad Ff_2 = 205.26\cdot kips$$

RESULT = Block Shear Capacity > Force Applied, OK

3. Tension Yielding Capacity of Flange Plate

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

Gross Tension Area,



$$Ag_{fp} := bfp\cdot tfp$$

$$Ag_{fp} = 9.5\cdot in^2$$

Tension Yielding Capacity,

$$Rty_{fp} := \Lambda_{ty}\cdot Fy_{fp}\cdot Ag_{fp}$$

$$Rty_{fp} = 284.431\cdot kips \qquad Ff_2 = 205.26\cdot kips$$

RESULT = Tension Yielding Capacity > Force Applied, OK

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4. Tension Rupture Capacity of Flange Plate

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

Net Tension Area,

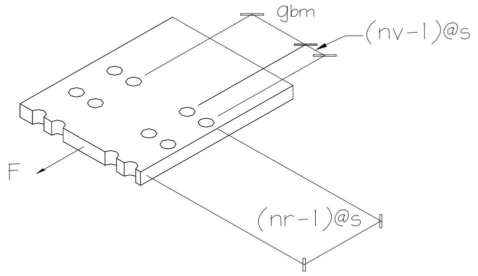
$$Ant := \left(bfp - 2nv_{fp2} \cdot hd_{fp2}\right) \cdot tfp$$

$$Ant = 7.25 \cdot in^2$$

Effective Net Tension Area,

$$Ae := \min\left(Ant , 0.85 \cdot Ag_{fp}\right)$$

$$Ae = 7.25 \cdot in^2$$



Tension Rupture Capacity

$$Rtr_{fp} := \Lambda_{tr} \cdot Fu_{fp} \cdot Ae$$

$$Rtr_{fp} = 235.625 \cdot kips \qquad Ff_2 = 205.26 \cdot kips$$

RESULT = Tension Rupture Capacity > Force Applied, OK

5. Flange Plate Compression Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J4.4, pages 16.1-129 to 16.1-130)

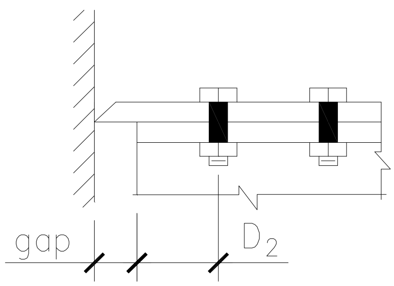
Effective Length Factor,

$K = 0.65$ *(Commentary Table C-A7.1)*

Laterally Unbraced Length of Plate,

$$L := D_4 + gap$$

$$L = 3.5 \cdot in$$



Radius of Gyration,

$$r := \frac{tfp}{\sqrt{12}}$$

$$r = 0.289 \cdot in$$

Slenderness Ratio,

$$KLr := \frac{K\,L}{r} = 7.881$$

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Elastic Critical Buckling Stress,

$$F_e := \frac{\pi^2 \cdot E}{KLr^2}$$

$$F_e = 4608.437 \cdot ksi$$

Compression Buckling Stress,

$$F_{cr} := \begin{cases} Fy_{fp} & \text{if } KLr \le 25 \\ \text{otherwise} & \\ \quad \begin{cases} 0.658^{\frac{Fy_{fp}}{F_e}} \cdot Fy_{fp} & \text{if } KLr \le 4.71 \cdot \sqrt{\frac{E}{Fy_{fp}}} \\ 0.877 \cdot F_e & \text{otherwise} \end{cases} \end{cases}$$

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

Compression Capacity,

$$Rcb_{fp} := \Lambda_c \cdot F_{cr} \cdot bfp \cdot tfp$$

$Rcb_{fp} = 284.431 \cdot kips$ $\quad$ $Ff_2 = 205.26 \cdot kips$

RESULT = Compression Capacity > Force Applied, OK

P. FLANGE PLATE TO GIRDER CHECK

1. Weld of Flange Plate to Girder

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

a. Using Fillet Weld

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

Minimum weld size,

(AISC 14th Ed, Chapter J, Specifications Section J2.2b, Table J2.4)

$w_{min} = \frac{1}{4} \cdot in$ $\quad$ $w_5 = \frac{1}{4} \cdot in$

RESULT = Preferred Weld Size = Minimum Weld Size,OK

Maximum weld size,

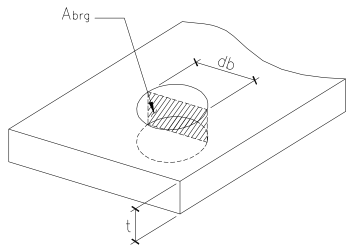
$$w_{max} := \begin{cases} tfp - \frac{1}{16} in & \text{if } tfp \ge \frac{1}{4} in \\ tfp & \text{otherwise} \end{cases}$$

$w_{max} = 0.938 \cdot in$ $\qquad w_5 = \frac{1}{4} \cdot in$

RESULT = Preferred Weld Size < Maximum Weld Size, OK

Q. BEAM 2 WEB 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,

$Abrg_{bm} := db_{pl2} \cdot tw_{bm2}$

$Abrg_{bm} = 0.45 \cdot in^2$

Bolt centerline distance from face of support,

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

$a_b = 10.42 \cdot in$

Eccentric Load Coefficient,

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

$C = 7.564$

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

$$Fbe := Fu_{bm2} \cdot \begin{cases} \min\left[1.0 \cdot \left(D_3 - 0.5 \cdot hd_{bm2w}\right) \cdot tw_{bm2}, 2.0 \cdot Abrg_{bm}\right] & \text{if } hd_{bm2w} \ge hd_{ls} \\ \min\left[1.2 \cdot \left(D_3 - 0.5 \cdot hd_{bm2w}\right) \cdot tw_{bm2}, 2.4 \cdot Abrg_{bm}\right] & \text{otherwise} \end{cases}$$

$Fbe = 70.2 \cdot kips$

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

$$Fbs := Fu_{bm2} \cdot \begin{cases} \min\left[1.0 \cdot \left(s_{pl2} - hd_{bm2w}\right) \cdot tw_{bm2}, 2.0 \cdot Abrg_{bm}\right] & \text{if } hd_{bm2w} \ge hd_{ls} \\ \min\left[1.2 \cdot \left(s_{pl2} - hd_{bm2w}\right) \cdot tw_{bm2}, 2.4 \cdot Abrg_{bm}\right] & \text{otherwise} \end{cases}$$

$Fbs = 57.037 \cdot kips$

Bolt Bearing Capacity,

$$Rbrg_{bm} := \Lambda_{brg} \cdot \frac{C}{nr} \cdot [Fbe + Fbs(nr - 1)]$$

$Rbrg_{bm} = 225.673 \cdot kips$ $V_{bm2} = 102.2 \cdot kips$

RESULT = Bearing Capacity > Force Applied,OK

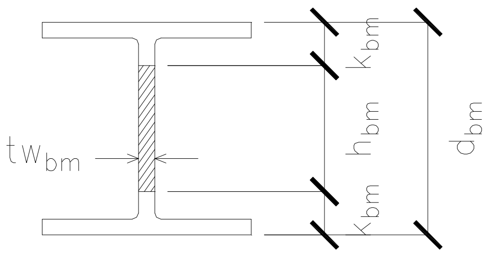
2. Shear Capacity of Beam

(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,

$$h := d_{bm2} - 2 \cdot kdes_{bm2}$$

$h = 16.1 \cdot in$



Limiting depth-thickness ratio,

$$h_{tw} := \frac{h}{tw_{bm2}}$$

$h_{tw} = 35.778$

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,

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

$kv = 5$

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Web shear coefficient,

$$Cv := \begin{cases} 1 & \text{if } h_{tw} \le 1.1 \cdot \sqrt{\dfrac{kv \cdot E}{Fy_{bm2}}} \\ \dfrac{1.1 \cdot \sqrt{\dfrac{kv \cdot E}{Fy_{bm2}}}}{h_{tw}} & \text{if } 1.1 \cdot \sqrt{\dfrac{kv \cdot E}{Fy_{bm2}}} < h_{tw} \le 1.37 \cdot \sqrt{\dfrac{kv \cdot E}{Fy_{bm2}}} \\ \dfrac{1.51 \cdot E \cdot kv}{h_{tw}{}^2 Fy_{bm2}} & \text{if } 1.37 \cdot \sqrt{\dfrac{kv \cdot E}{Fy_{bm2}}} < h_{tw} \end{cases}$$

(G2-3)

(G2-4)

(G2-5)

$Cv = 1$

Shear Capacity of Section,

$Rv_{bm} := \Lambda_{vbm} \cdot 0.6 \cdot Fy_{bm2} \cdot d_{bm2} \cdot tw_{bm2} \cdot Cv$ *(G2-1)*

$Rv_{bm} = 165.6 \cdot kips$ $V_{bm2} = 102.2 \cdot kips$

RESULT = Shear Capacity of Section > Force Applied, OK

R. BEAM TO SHEAR PLATE CHECK

1. Bolt Shear Capacity

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

Shear Capacity per bolt,

$\Lambda rv_{pl2} = 14.464 \cdot kips$

Bolt Shear Capacity,

$Rb_{pl} := C \cdot \Lambda rv_{pl2}$

$Rb_{pl} = 109.406 \cdot kips$ $V_{bm2} = 102.2 \cdot kips$

RESULT = Bolt Shear Capacity > Force Applied, OK

S. SHEAR PLATE 2 CHECK

1. Check for Maximum Shear Plate Thickness

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

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Exceptions for nv = 1 and nv = 2

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

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

RESULT = Check maximum thickness of plate

Coefficient for Eccentrically Loaded Bolts

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

$C' = 91.406 \cdot in$

Area of Bolts

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

$Ab = 0.785 \cdot in^2$

Length of Plate

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

$Lpl = 14 \cdot in$

Maximum Thickness

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

$tpl_{max} = 1.349 \cdot in$ $\qquad$ $tpl = 1 \cdot in$

RESULT = Plate Thickness < Max Thickness Permitted,OK

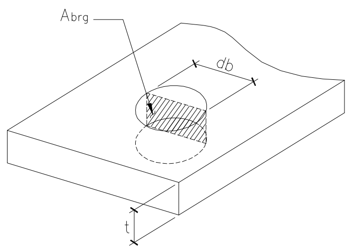
Governing Shear Plate Thickness

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

$$tpl_g = 1 \cdot in$$

2. Bolt Bearing Capacity of Shear Plate

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



Area of Bearing Surface,

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

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

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

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

$$Fbe = 36.562 \cdot kips$$

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

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

$$Fbs = 63.375 \cdot kips$$

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Bolt Bearing Capacity,

$$Rbrg_{pl} := \Lambda_{brg} \cdot \frac{C}{nr} \cdot [Fbe + Fbs(nr - 1)]$$

$Rbrg_{pl} = 219.404 \cdot kips$ $\qquad V = 102.2 \cdot kips$

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,

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

$Lpl = 14 \cdot in$

Check if Length of Plate is acceptable,

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

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

Length = "Plate Length is OK per AISC Requirements"

Gross Shear Capacity,

$$Rvy_{pl} := \Lambda_{vy} \cdot 0.6 \cdot Fy_{pl} \cdot tpl_g \cdot Lpl \qquad (J4\text{-}3)$$

$Rvy_{pl} = 280 \cdot kips$ $\qquad V = 102.2 \cdot kips$

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,

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

$Anv = 8.375 \cdot in^2$

Shear Rupture Capacity,

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

$Rvr_{pl} = 163.313 \cdot kips$ $V = 102.2 \cdot kips$

RESULT = Shear Rupture Capacity > Force Applied, OK

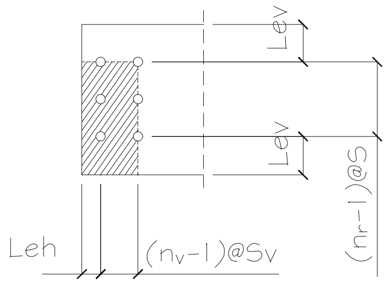
5. Block Shear Capacity of Shear Plate

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

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

$U_{bs} = 0.5$

Gross Shear Area



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

$Agv = 12.5 \cdot in^2$

Net Tension Area

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

$Ant = 5.687 \cdot in^2$

Net Shear Area

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

$Anv = 7.438 \cdot in^2$

Block Shear Capacity of Plate, (J4-5)

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

$Rbs_{pl} = 237.453 \cdot kips$ $V = 102.2 \cdot kips$

RESULT = Block Shear Capacity > Force Applied, OK

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6. Local Buckling Capacity of Shear Plate

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

Distance of bolt line to support,

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

$$a_b = 5.92 \cdot in$$

Coefficient,

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

$$\lambda = 0.219$$

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

$$Q = 1$$

Allowable Buckling Stress,

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

$$Fcr = 50 \cdot ksi$$

Gross Plastic Section Modulus,

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

$$Zx_{pl} = 49 \cdot in^3$$

Eccentricity,

$$e_{pl} := a_b$$

$$e_{pl} = 5.92 \cdot in$$

Buckling Capacity,

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

$Rbc_{pl} = 247.815 \cdot kips$ $\quad\quad V = 102.2 \cdot kips$

RESULT = Local Buckling Capacity will not Control!

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,

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

$Rfc_{pl} = 185.409 \cdot kips$ $\quad\quad V = 102.2 \cdot kips$

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,

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

$Znet_{pl} = 30.121 \cdot in^3$

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Flexural Rupture Capacity,

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

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

$Rfr_{pl} = 165.361 \cdot kips$ $\qquad V = 102.2 \cdot kips$

RESULT = Flexural Rupture Capacity > Applied Force, OK

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,

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

$V_r := V$

$V_r = 102.2 \cdot kips$

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

$M_r = 1064.924 \cdot kips \cdot in$

Shear yielding,

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

$V_c = 280 \cdot kips$

Flexural yielding,

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

$M_c = 1467.066 \cdot kips \cdot in$

Interaction,

$$\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{M_r}{M_c}\right)^2 = 0.66$$

RESULT = Interaction < 1.0, OK

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T. SHEAR PLATE 2 TO GIRDER CHECK

1. Weld of Shear Plate to Girder

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

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

$w_{min} = \frac{3}{16} \cdot in$ $w_2 = \frac{1}{4} \cdot in$

RESULT = Preferred Weld Size > Minimum Weld Size, OK

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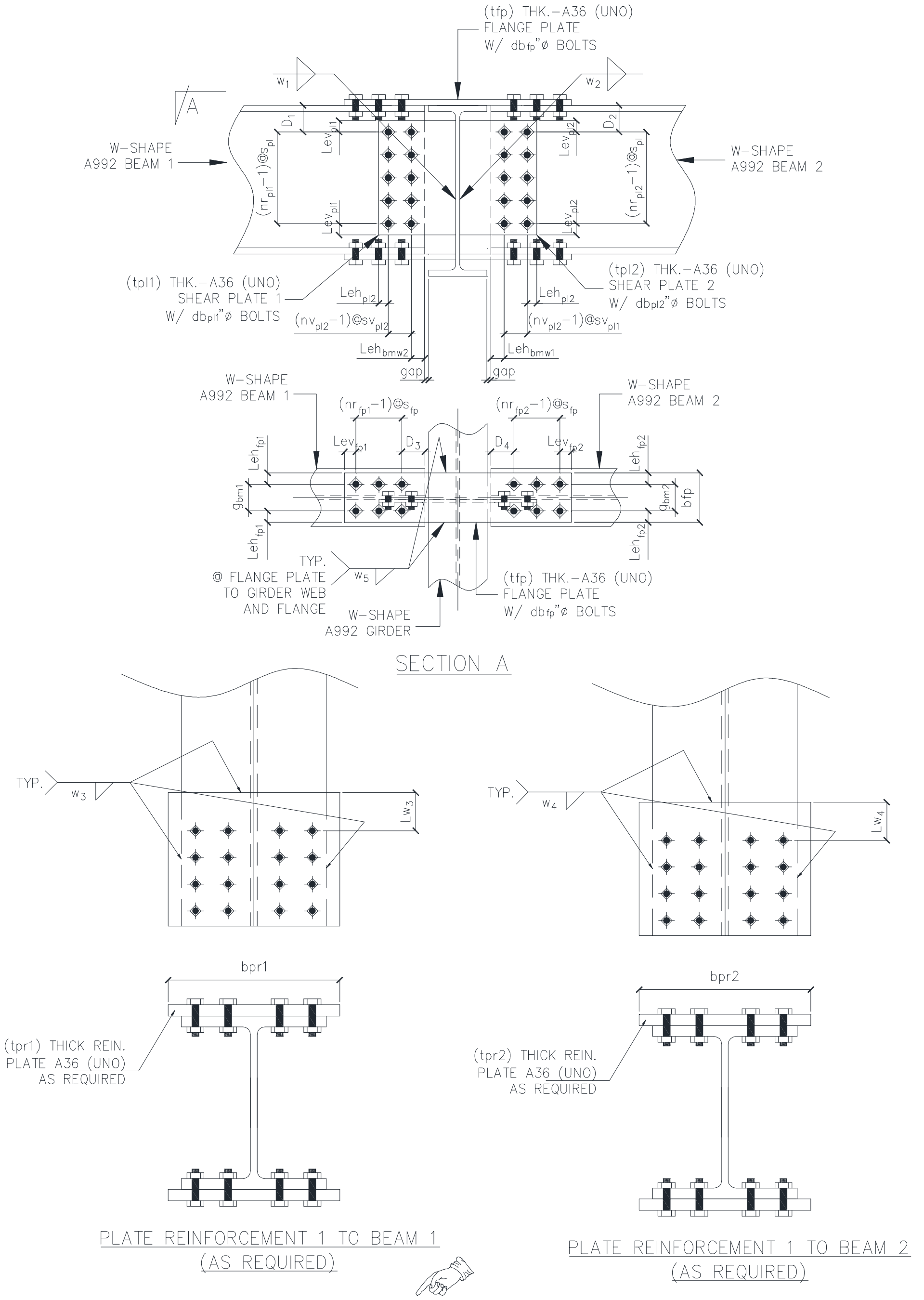
III. DETAILS:

A. SKETCH



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

MOMENT CONNECTION: DESIGN OF W-SHAPE BEAMS (BOLTED FLANGE PLATE AND WEB SHEAR PLATE CONNECTION) TO W-SHAPE GIRDER (2-WAY)

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B. **TABLE: MOMENT CONNECTION SCHEDULE**

Beam 1		Flange Plate			Bolts at Flange Plate						Plate Reinforcement Conn.				
Size	Grade	tfp (in)	bfp (in)	Grade	db_{fp1} (in)	Bolt $Type_{fp1}$	nr_{fp1}	nv_{fp1}	s_{fp1} (in)	sv_{fp1} (in)	tpr1 (in)	Grade	bpr1 (in)	w_3 (in)	Lw_3 (in)
W18X65	A992	1	9 1/2	A572-50	1	A490-SC-STD-CLASS_A STD Hole on BOTH Plies	8	1	3	3	1/2	A36	8 5/8	1/4	6

Girder		Shear Plate 1		Bolts at Shear Plate 1						Beam Shear Load	Beam Axial Load	Beam Moment Load
Size	Grade	tpl1 (in)	Grade	db_{pl1} (in)	Bolt $Type_{pl1}$	nr_{pl1}	nv_{pl1}	s_{pl1} (in)	sv_{pl1} (in)	V_{bm1} (kips)	P_{bm1} (kips)	M_{bm1} (kips-ft)
W21X62	A36	1	A572-50	1	A490-SC-SSLT-CLASS_A SSL on Shear Plate ONLY	5	4	2 3/4	3	102.20	0.00	331.84

Bolt Settings (in)			Edge Distances (in)					gap (in)	Weld Size	
D_1	D_3	g_{bm1}	Lev_{pl1} (in)	Leh_{pl1} (in)	Leh_{bmw1} (in)	Lev_{fp1} (in)	Leh_{fp1} (in)		w_1 (in)	w_5 (in)
3 3/4	3	5 1/2	1 1/2	1 1/2	1 1/2	1 1/2	2	1/2	1/4	1/4

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Beam 2		Flange Plate			Bolts at Flange Plate						Plate Reinforcement Conn.				
Size	Grade	tfp (in)	bfp (in)	Grade	db_{fp2} (in)	Bolt $Type_{fp2}$	nr_{fp2}	nv_{fp2}	s_{fp2} (in)	sv_{fp2} (in)	tpr2 (in)	Grade	bpr2 (in)	w_4 (in)	Lw_4 (in)
W18X65	A992	1	9 1/2	A572-50	1	A490-SC-STD-CLASS_A STD Hole on BOTH Plies	8	1	3	3	1/2	A36	8 5/8	1/4	6

Girder		Shear Plate 2		Bolts at Shear Plate 2						Beam Shear Load V_{bm2} (kips)	Beam Axial Load P_{bm2} (kips)	Beam Moment Load M_{bm2} (kips-ft)
Size	Grade	tpl2 (in)	Grade	db_{pl2} (in)	Bolt $Type_{pl2}$	nr_{pl2}	nv_{pl2}	s_{pl2} (in)	sv_{pl2} (in)			
W21X62	A36	1	A572-50	1	A490-SC-SSLT-CLASS_A SSL on Shear Plate ONLY	5	4	2 3/4	3	102.20	0.00	331.84

Bolt Settings (in)			Edge Distances (in)					gap (in)	Weld Size
D_2	D_4	g_{bm2}	Lev_{pl2} (in)	Leh_{pl2} (in)	Leh_{bmw2} (in)	Lev_{fp2} (in)	Leh_{fp2} (in)		w_2 (in)
3 3/4	3	5 1/2	1 1/2	1 1/2	1 1/2	1 1/2	2	1/2	1/4

IV. REFERENCES

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