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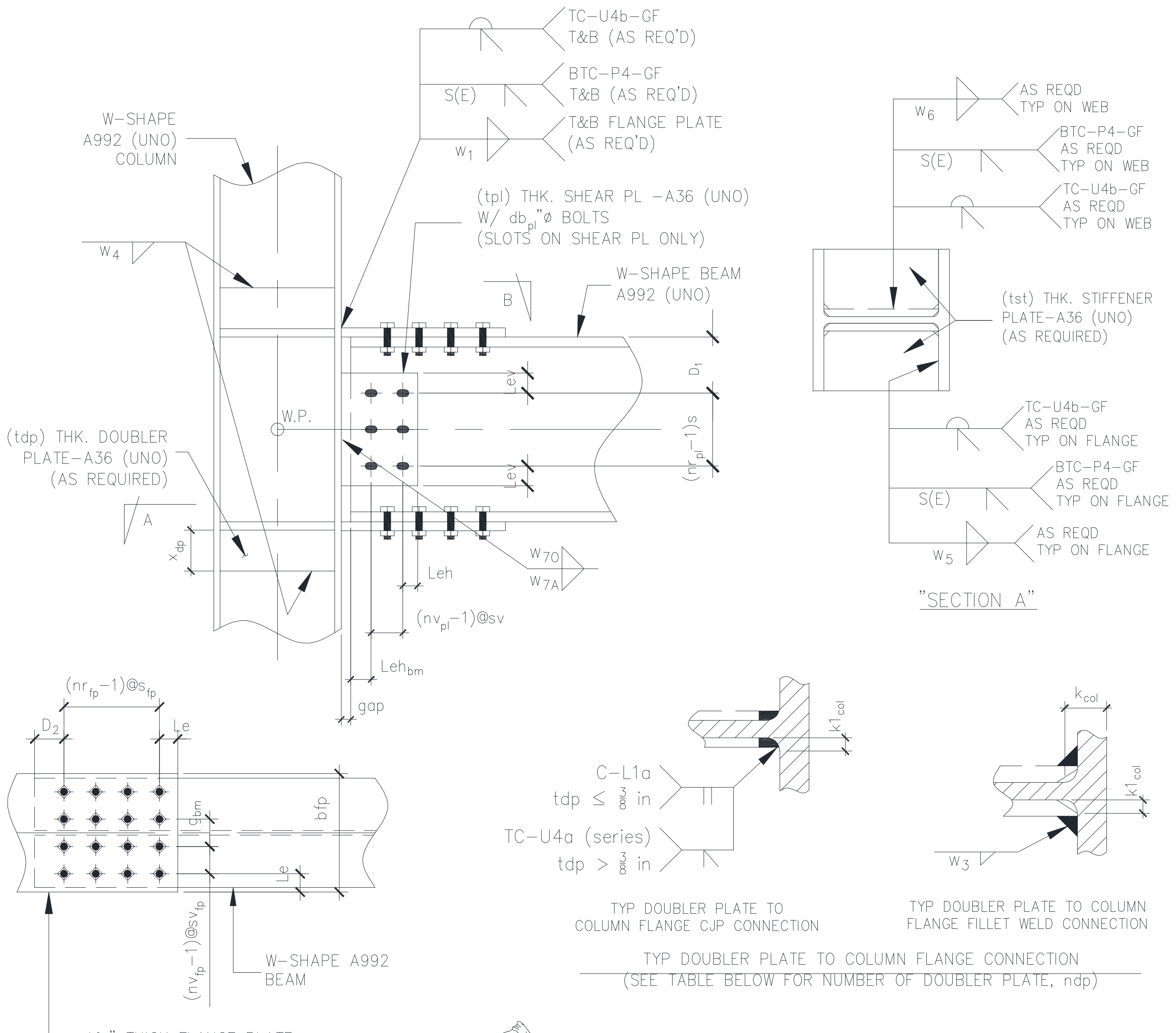
MOMENT CONNECTION: DESIGN OF W-SHAPE MOMENT BEAM TO W-SHAPE COLUMN FLANGE (1WAY) WITH FLANGE PLATE AND SHEAR PLATE CONNECTION



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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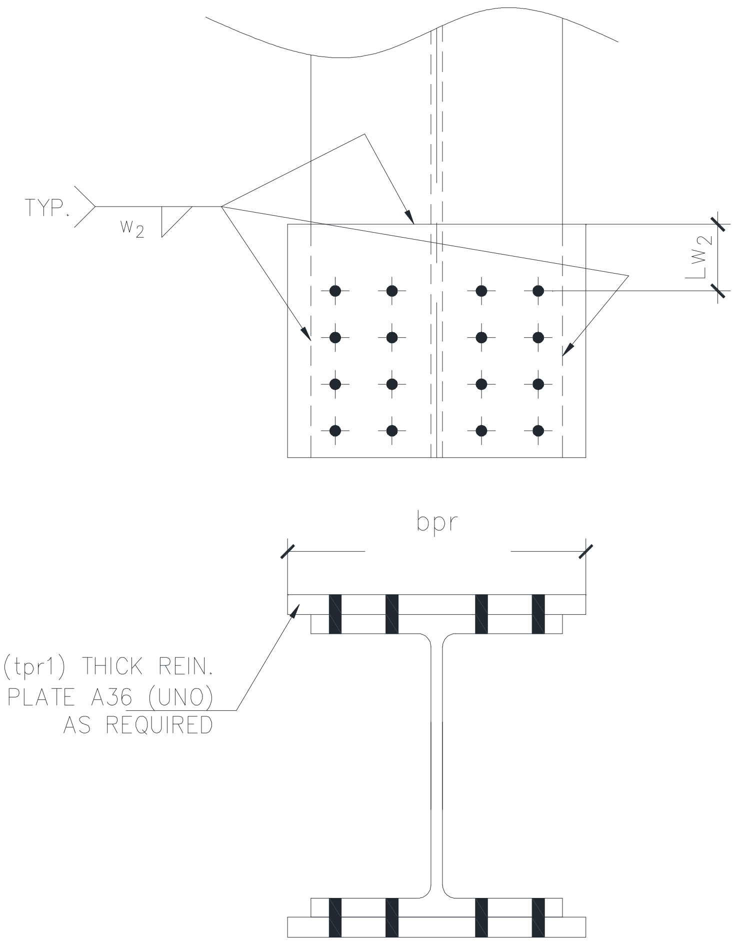
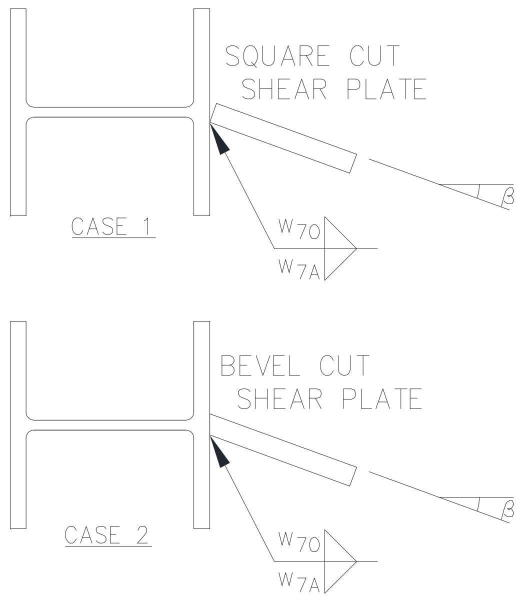


PLATE REINFORCEMENT TO BEAM
(AS REQUIRED)



SHEAR PLATE TO
COLUMN FLANGE WELD

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)

COLUMN PROPERTIES (col): W14X211 - A992

$Fy_{col} = 50 \cdot ksi$ $d_{col} = 15.7 \cdot in$ $tw_{col} = 0.98 \cdot in$ $k1_{col} = 1.688 \cdot in$

$Fu_{col} = 65 \cdot ksi$ $bf_{col} = 15.8 \cdot in$ $tf_{col} = 1.56 \cdot in$ $k_{col} = 2.875 \cdot in$

$E := 29000ksi$

BEAM PROPERTIES (bm): W24X84 - A992

$Fy_{bm} = 50 \cdot ksi$ $d_{bm} = 24.1 \cdot in$ $tw_{bm} = 0.47 \cdot in$ $k1_{bm} = 1.063 \cdot in$

$Fu_{bm} = 65 \cdot ksi$ $bf_{bm} = 9.02 \cdot in$ $tf_{bm} = 0.77 \cdot in$ $k_{bm} = 1.688 \cdot in$

Length of Beam, $L_{bm} := 30ft$ $\beta := 0deg$ *Beam Gage,* $g_{bm} := 5\frac{1}{2}in$

FLANGE PLATE (fp): A572-50

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

SHEAR PLATE (pl): A36

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

PLATE REINFORCEMENT (pr): A36 (AS REQUIRED)

$Fy_{pr} = 36 \cdot ksi$ $tpr := \frac{3}{8}in$ $bpr = 10.5 \cdot in$

$Fu_{pr} = 58 \cdot ksi$ $n_{pr} := 1$

STIFFENER PLATE (st): A572-50 (AS REQUIRED) (Full Depth)

$Fy_{st} = 50 \cdot ksi$ $Lst_{giv} = 12.5 \cdot in$ $tst := tfp$ $clip := 1in$

$Fu_{st} = 65 \cdot ksi$ $bst_{giv} = 7.25 \cdot in$ $nst := 2$

DOUBLER PLATE (dp): A36 (AS REQUIRED) (Extended)

$Fy_{dp} = 36 \cdot ksi$ $Fu_{dp} = 58 \cdot ksi$ $tdp = \frac{3}{8} \cdot in$ $ndp := 1$

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Extension of Doubler Plate (AS REQUIRED),

$$x_{dp} := \mathrm{Ceil}\left(2.5k_{col}, 0.5in\right)$$

$$x_{dp} = 7.5 \cdot in$$

Length of Doubler Plate,

$$Ldp := \mathrm{Ceil}\left(d_{bm} + tfp + 2x_{dp}, \frac{1}{4}in\right)$$

$$Ldp = 40.25 \cdot in$$

BOLTS:

For Shear Plate to Beam Connection:

Bolt Diameter,	$db_{pl} = 1 \cdot in$	$Bolt\_Type_{pl} =$ "A490-SC-SSLT-CLASS_A"
Bolt Shear Strength,	$\Lambda rv_{pl} = 14.464 \cdot kips$	$Conn\_type_{pl} =$ "Slip Critical-type"
Bolt Tensile Strength,	$\Lambda rn_{pl} = 44.375 \cdot kips$	

Gap between edge of beam to edge of support,	$gap := 1in$	*Hole diameter:*	
		Shear Plate,	
Beam Web Edge Distance,	$Leh_{bmw} = 1.5 \cdot in$	$hd_{plv} = 1.125 \cdot in$	$hd_{plh} = 1.375 \cdot in$
Plate Vertical Edge Distance,	$Lev_{pl} = 1.5 \cdot in$	*Beam Web,*	
Plate Horizontal Edge Distance,	$Leh_{pl} = 1.5 \cdot in$	$hd_{bmw} = 1.125 \cdot in$	
Bolt Vertical Spacing,	$s_{pl} = 3 \cdot in$		
Bolt Horizontal Spacing (For Multiple bolt lines),	$sv_{pl} = 3 \cdot in$		

number of bolt rows: $nr_{pl} := 6$

number of vertical bolt lines: $nv_{pl} := 2$

total number of bolts: $n_{pl} := nr_{pl} \cdot nv_{pl}$ $\quad n_{pl} = 12$

Bolt First Down from top of beam, $D_1 = 4.75 \cdot in$

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For Flange Plate to Beam Flange Connection:

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

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

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

Beam flange bolt gage, $g_{bm} = 5.5 \cdot in$

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

Flange Plate Horizontal Edge Distance, $Leh_{fp} = 2.5 \cdot in$

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

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

<u>Hole diameter:</u>

Flange Plate, $hd_{fp} = 1.125 \cdot in$

Beam Flange, $hd_{bmf} = 1.125 \cdot in$

number of bolt rows: $nr_{fp} := 10$

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

total number of bolts: $n_{fp} := 2nr_{fp} \cdot nv_{fp}$ $n_{fp} = 20$

First bolt line to beam flange edge, $D_2 = 3 \cdot in$

WELDS: E70xx LH

$Fu_w = 70 \cdot ksi$

<u>Weld Size</u>

Flange Plate to Column Flange, $w_1 = \frac{3}{4} \cdot in$

Plate Reinforcement to Beam (as req'd), $w_2 = \frac{5}{16} \cdot in$ $Lw_2 := 4.5in$

Doubler Plate to Column Flange (as req'd), $w_3 = \frac{5}{16} \cdot in$

Doubler Plate to Column Web (as req'd), $w_4 = \frac{5}{16} \cdot in$

Stiffener Plate to Column Flange (as req'd), $w_5 = \frac{3}{4} \cdot in$

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Stiffener Plate to Column Web (as req'd),

$$w_6 = \frac{3}{4} \cdot in$$

Shear Plate to Column Flange,

REFERENCES:

1. DESIGN OF UNSTIFFENED EXTENDED SINGLE-PLATE SHEAR CONNECTIONS by LARRY S. MUIR and CHRISTOPHER M. HEWITT (engineering journal/second quarter/2009/page 70)
2. DESIGN FILLET WELDS FOR SKEWED T-JOINTS-PART 1 by DUANE K. MILLER
3. AWS 2.3.3-considering Z-loss factor

$$Fy := 50ksi$$

$$\theta_1 := 90deg - \beta = 90 \cdot deg$$

$$\theta_2 := 180deg - \theta_1 = 90 \cdot deg$$

$$w7_{case} := \begin{cases} \text{"CASE 1"} & \text{if } tpl \cdot \sin(\beta) \le \frac{3}{16} in \\ \text{"CASE 2"} & \text{otherwise} \end{cases}$$

$$w7_{case} = \text{"CASE 1"}$$

$$w_{7A} := Ceil\left[max\left[\frac{5}{8} tpl , \left(\frac{tpl \cdot Fy \cdot \sqrt{3}}{2 \cdot \sqrt{2} \cdot Fu_w} + \begin{cases} 0 & \text{if } 60deg \le \theta_1 \le 90deg \\ \frac{1}{8} in & \text{if } 45deg \le \theta_1 < 60deg \\ \frac{1}{4} in & \text{if } 30deg \le \theta_1 < 45deg \end{cases} \right) \cdot \frac{\cos(\theta_1 - 90deg)}{\cos\left(\frac{\theta_1}{2}\right)} \right] , \frac{1}{16} in \right]$$

$$w_{7A} = \frac{1}{2} \cdot in$$

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$$w_{10.1} := \frac{\cos\left(\theta_2 - 90deg\right)\cdot tpl\cdot Fy\cdot\sqrt{3}}{2\cdot\sqrt{2}\cdot Fu_w\cdot\cos\left(\frac{\theta_2}{2}\right)}$$

$$w_{10.2} := \begin{cases} 0 & \text{if } tpl\cdot\sin(\beta) < \frac{1}{16}in \\ tpl\cdot\sin(\beta) & \text{otherwise} \end{cases}$$

$$w_{70} := \begin{cases} Ceil\left(w_{10.1} + w_{10.2}, \frac{1}{16}in\right) & \text{if } w7_{case} = \text{"CASE 1"} \\ Ceil\left(w_{10.1}, \frac{1}{16}in\right) & \text{otherwise} \end{cases}$$

$$w_{70} = \frac{1}{2}\cdot in$$

SAFETY AND RESISTANCE FACTORS:

Safety Factor, Ω (ASD)

Resistance Factor, ϕ (LRFD)

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

	safety factor	*resistance factor*	*modification factor*
For tension rupture,	$\Omega_{tr} = 2.00$	$\phi_{tr} = 0.75$	$\Lambda_{tr} = 0.50$
For tension yielding,	$\Omega_{ty} = 1.67$	$\phi_{ty} = 0.90$	$\Lambda_{ty} = 0.60$
For compression,	$\Omega_c = 1.67$	$\phi_c = 0.9$	$\Lambda_c = 0.60$
For shear,	$\Omega_v = 1.67$	$\phi_v = 0.90$	$\Lambda_v = 0.60$
For fillet weld (shear),	$\Omega_{vw} = 2.00$	$\phi_{vw} = 0.75$	$\Lambda_{vw} = 0.50$
For shear rupture,	$\Omega_{vr} = 2.00$	$\phi_{vr} = 0.75$	$\Lambda_{vr} = 0.50$
For shear yielding,	$\Omega_{vy} = 1.50$	$\phi_{vy} = 1.00$	$\Lambda_{vy} = 0.67$
For bearing,	$\Omega_{brg} = 2.00$	$\phi_{brg} = 0.75$	$\Lambda_{brg} = 0.50$
For web compression buckling,	$\Omega_{cb} = 1.67$	$\phi_{cb} = 0.90$	$\Lambda_{cb} = 0.60$
For web crippling,	$\Omega_{cr} = 2.00$	$\phi_{cr} = 0.75$	$\Lambda_{cr} = 0.50$
For web yielding,	$\Omega_{wy} = 1.50$	$\phi_{wy} = 1.00$	$\Lambda_{wy} = 0.67$

For flexural local buckling,	$\Omega_b = 1.67$	$\phi_b = 0.9$	$(\Lambda_b) = 0.60$
For partial penetration weld(shear),	$\Omega_{vwp} = 2.00$	$\phi_{vwp} = 0.75$	$\Lambda_{vwp} = 0.50$
For partial penetration weld(tension),	$\Omega_{twp} = 1.88$	$\phi_{twp} = 0.8$	$\Lambda_{twp} = 0.53$

APPLIED LOADS:

BEAM

Given Shear Load, $V_{giv} := 60kips$

$$V_{bm} := V_{giv} + \frac{2 \cdot \Lambda_b \cdot Fy_{bm} \cdot Zx_{bm}}{L_{bm}}$$

Beam Shear Load, $V_{bm} = 97.259 \cdot kips$

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

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

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

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

Moment Load, $M_{bm} = 558.88 \cdot kips \cdot ft$ **100% M.cap**

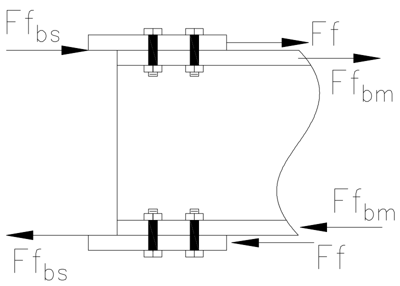
COLUMN

Axial Load, $Pu_{col} := 0kips$

Story Shear, $Vs := 0kips$

II. CALCULATIONS:

A. BOLTS ON FLANGE PLATE CHECK



1. Forces acting on the Connection

@ Beam Flange,

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

$$Ff_{bm} = 287.466 \cdot kips$$

@ Interface of Beam Flange & Flange Plate,

$$Ff_{bs} := \frac{P_{bm}}{2} + \frac{M_{bm}}{d_{bm}}$$

$$Ff_{bs} = 278.282 \cdot kips$$

@ Flange Plate,

$$Ff := \frac{P_{bm}}{2} + \frac{M_{bm}}{d_{bm} + tfp}$$

$$Ff = 267.195 \cdot kips$$

2. Bolt Shear Capacity

(AISC 13th Ed. Specifications Chapter J, Section J3.6, pages 16.1-108 to 16.1-109)

Shear Capacity per bolt,

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

Bolt Shear Capacity,

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

$Rb_{bs} = 289.28 \cdot kips$ $\qquad$ $Ff_{bs} = 278.282 \cdot kips$

RESULT = Bolt Shear Capacity > Force Applied, OK

2. Check for Spacing

(AISC 13th Ed. Specifications Chapter J, Section J3.3 and J3.5, pages 16.1-106 to 16.1-108)

Vertical Spacing,

$$s_{fp} = 3 \cdot in$$

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

$s_{min} = 2.667 \cdot in$

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

$s_{max} = 12.000 \cdot in$

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

Horizontal Spacing,

$sv_{fp} = 3 \cdot in$

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

$sv_{min} = 2.667 \cdot in$

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

$sv_{max} = 12.000 \cdot in$

RESULT = This check is not applicable

3. Check for Edge Distance

(AISC 13th Ed. Specifications Chapter J, Section J3.4 and J3.5, pages 16.1-106 to 16.1-108)

Vertical Edge Distance,

$Lev_{fp} = 1.5 \cdot in$

$D_2 = 3 \cdot in$

$Le_{min} = 1.25 \cdot in$

$Lev_{minfp} = 1.25 \cdot in$

$Lev_{minbm} = 1.25 \cdot in$

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

$Lev_{maxpl} = 6.000 \cdot in$

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

$Lev_{maxbm} = 6.000 \cdot in$

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

Horizontal Edge Distance,

$Leh_{fp} = 2.5 \cdot in$

$Leh_{bmf} := 0.5 \cdot \left[bf_{bm} - \left[g_{bm} + 2 \left(nv_{fp} - 1 \right) sv_{fp} \right] \right]$

$Leh_{bmf} = 1.76 \cdot in$

$Le_{min} = 1.25 \cdot in$

$Leh_{minpl} = 1.25 \cdot in$

$Leh_{minbm} = 1.25 \cdot in$

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

$Leh_{maxpl} = 6.000 \cdot in$

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

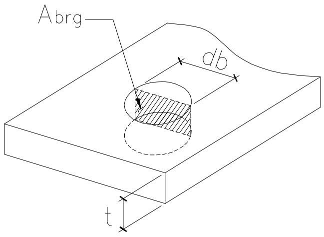
$Leh_{maxbm} = 6.000 \cdot in$

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

B. BEAM FLANGE CHECK

1. Bolt Bearing on Beam Flange

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



Bearing Area,

$Abrg_{bm} := db_{fp} \cdot tf_{bm}$

$Abrg_{bm} = 0.77 \cdot in^2$

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

$$Fbe := Fu_{bm} \cdot \begin{cases} \min\left[1.0 \cdot \left(D_2 - 0.5hd_{bmf}\right) \cdot tf_{bm}, 2.0 \cdot Abrg_{bm}\right] & \text{if } hd_{bmf} \ge hd_{ls} \\ \min\left[1.2 \cdot \left(D_2 - 0.5hd_{bmf}\right) \cdot tf_{bm}, 2.4 \cdot Abrg_{bm}\right] & \text{otherwise} \end{cases}$$

$Fbe = 120.12 \cdot kips$

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

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

$Fbs = 112.612 \cdot kips$

Bolt Bearing Capacity,

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

$Rbrg_{bmf} = 1133.632 \cdot kips$ $\quad Ff_{bm} = 287.466 \cdot kips$

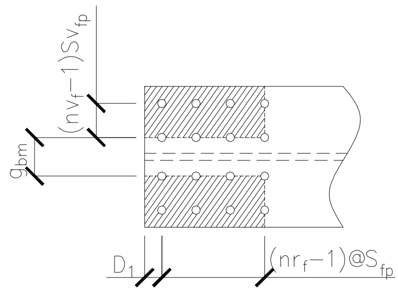
RESULT = Bearing Capacity > Force Applied, OK

2. Block Shear Capacity of Beam Flange

(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



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

$Agv = 46.2 \cdot in^2$

Net Shear Area

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

$Anv = 29.741 \cdot in^2$

Net Tension Area

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

$Ant = 1.844 \cdot in^2$

Block Shear Capacity,

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

$Rbs_{bmf} = 639.889 \cdot kips$ $\quad Ff_{bm} = 287.466 \cdot kips$

RESULT = Block Shear Capacity > Force Applied, OK

3. Beam Flexural Strength on Reduced Area w/ Reinforcement

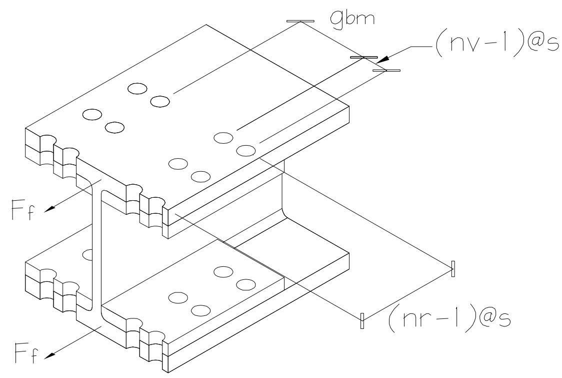
(AISC 13th Ed. Specifications Chapter F, Section F13, page 16.1-61 to 16.1-62)

Effective Flange Thickness,

$$tf_{bmpr} := \begin{cases} tf_{bm} & \text{if Reinforcement = "Not Required"} \\ tf_{bm} + tpr \cdot \left(\dfrac{Fy_{pr}}{Fy_{bm}} \right) & \text{otherwise} \end{cases}$$

$tf_{bmpr} = 1.04 \cdot in$

a. *Gross Tension Flange Area*



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

$$A_{fg} = 9.381 \cdot in^2$$

b. *Net Tension Area*

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

$$A_{fn} = 7.041 \cdot in^2$$

c. *Value of Yt*

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

$$Y_t = 1$$

d. *Section Modulus of Section*

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

$$Ix_{bmpr} = 2914.06 \cdot in^4$$

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

$$Sx_{bmpr} = 241.831 \cdot in^3$$

e. *Revised Flexural Requirement Due to Axial Load*

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

$$M_{rev} = 558.882 \cdot kips \cdot ft$$

f. Tension Rupture Capacity

Net Moment Capacity,

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

$M_n = 588.721 \cdot kips \cdot ft$ $\qquad M_{rev} = 558.882 \cdot kips \cdot ft$

RESULT = Net Moment Capacity > Applied Moment, OK

C. PLATE REINFORCEMENT CONNECTION TO BEAM 1: Required

1. Force Acting on Plate Reinforcement

Force on Flange,

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

$Ff_{bmi} = 212.836 \cdot kips$

Force on Plate Reinforcement,

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

$F_{pr} = 74.631 \cdot kips$

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,

$Lw_{pr1} := Lw_2$

$Lw_{pr1} = 4.5 \cdot in$

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

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

Minimum weld size,

$w_{min} = \frac{3}{16} \cdot in$ $\qquad w_2 = \frac{5}{16} \cdot in$

RESULT = Preferred Weld Size > Minimum Weld Size, OK

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Maximum weld size,

$w_{max} = \frac{5}{16} \cdot in$ $w_2 = \frac{5}{16} \cdot in$

RESULT = Maximum Weld Size > Preferred Weld Size, OK

a. Plate Reinforcement to Beam Flange Using Fillet Weld

For Longitudinal Weld

Shear Strength,

For Beam:

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

$Rv_{bm} = 15.015 \cdot \frac{kips}{in}$

For Plate Reinforcement:

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

$Rv_{pr} = 6.525 \cdot \frac{kips}{in}$

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,

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

$w_{eff} = 0.439 \cdot in$

Weld Capacity,

$Rw_{pr1} := 2\Lambda_{vw} \cdot 0.60 \cdot Fu_w \cdot \sin(45 \cdot deg) \cdot n_{ws} \cdot \min(w_{eff}, w_2) \cdot Lw_{pr1}$

$Rw_{pr1} = 41.763 \cdot kips$

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For Transverse Weld

Shear Strength,

For Beam:

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

$$Rv_{bm} = 15.015 \cdot \frac{kips}{in}$$

For Plate Reinforcement:

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

$$Rv_{pr} = 6.525 \cdot \frac{kips}{in}$$

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,

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

$$w_{eff} = 0.439 \cdot in$$

Weld Capacity,

$$Rw_{pr2} := \Lambda_{vw} \cdot 0.60 \cdot Fu_w \cdot \sin(45 \cdot deg) \cdot n_{ws} \cdot \min(w_{eff}, w_2) \cdot Lw_{pr2}$$

$$Rw_{pr2} = 41.856 \cdot kips$$

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

$Rw_{pr} = 98.283 \cdot kips$ $\qquad$ $F_{pr} = 74.631 \cdot kips$

RESULT = Weld Capacity > Force Applied, OK

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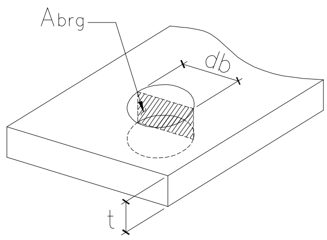
D. PLATE REINFORCEMENT 1 CHECK: Required

1. Bolt Bearing Capacity of Plate Reinforcement

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



Bearing Area,

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

$$Abrg_{pr} = 0.375 \cdot in^2$$

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

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

$$Fbe = 24.469 \cdot kips$$

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

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

$$Fbs = 48.937 \cdot kips$$

Bolt Bearing Capacity,

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

$Rbrg_{pr} = 464.906 \cdot kips$ $\qquad$ $F_{pr} = 74.631 \cdot kips$

RESULT = Not Applicable, Reinforcement not Required

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

Gross Shear Area

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

$$Agv = 22.5 \cdot in^2$$

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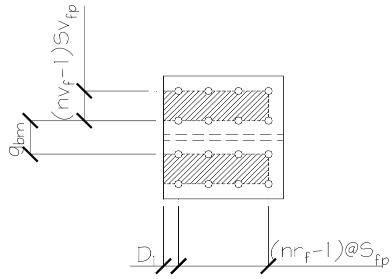
Net Shear Area

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

$$Anv = 14.484 \cdot in^2$$



Net Tension Area

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

$$Ant = 2.062 \cdot in^2$$

Block Shear Capacity,

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

$Rbs_{pr} = 302.812 \cdot kips$ $\qquad$ $F_{pr} = 74.631 \cdot kips$

RESULT = Not Applicable, Reinforcement not Required

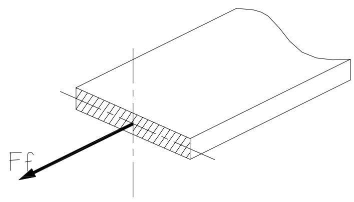
3. Tension Yielding Capacity of Plate Reinforcement

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

Gross Tension Area,

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

$$Ag_{pr} = 3.937 \cdot in^2$$



Tension Yielding Capacity,

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

$Rty_{pr} = 84.88 \cdot kips$ $\qquad$ $F_{pr} = 74.631 \cdot kips$

RESULT = Not Applicable, Reinforcement not Required

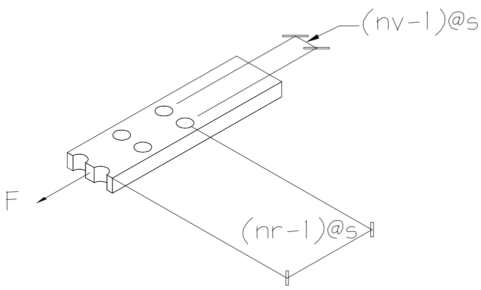
4. Tension Rupture Capacity of Plate Reinforcement

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

Net Tension Area,

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

$$Ant = 3.516 \cdot in^2$$



Effective Net Tension Area,

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

$$Ae = 3.347 \cdot in^2$$

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Tension Rupture Capacity

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

$Rtr_{pr} = 97.059 \cdot kips$ $\quad\quad$ $F_{pr} = 74.631 \cdot kips$

RESULT = Not Applicable, Reinforcement not Required

E. FLANGE PLATE CHECK

1. Bolt Bearing Capacity of Flange Plate

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

Bearing Area,

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

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

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

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

$$Fbe = 73.125 \cdot kips$$

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

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

$$Fbs = 146.25 \cdot kips$$

Bolt Bearing Capacity,

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

$Rbrg_{fp} = 1389.375 \cdot kips$ $\quad\quad$ $Ff = 267.195 \cdot kips$

RESULT = Bearing Capacity > Force Applied, OK

2. Block Shear Capacity of Flange Plate

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

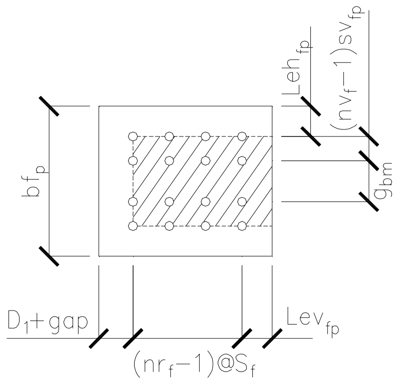
Reduction Factor, $\quad U_{bs} := 1.0 \quad$ *(Tension Stress is Uniform)*

Flange Plate Width,

$$bfp := g_{bm} + 2\left(nv_{fp} - 1\right)\cdot sv_{fp} + 2Leh_{fp}$$

$$bfp = 10.5\cdot in$$

RESULT = NUMBER OF BOLTS, OK



Pattern 1

Gross Shear Area

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

$$Agv = 57\cdot in^2$$

Net Shear Area

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

$$Anv = 35.625\cdot in^2$$

Net Tension Area

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

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

Block Shear Capacity

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

$$Rbs_{fp1} = 836.875\cdot kips$$

Pattern 2

Gross Shear Area

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

$$Agv = 54\cdot in^2$$

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Net Shear Area

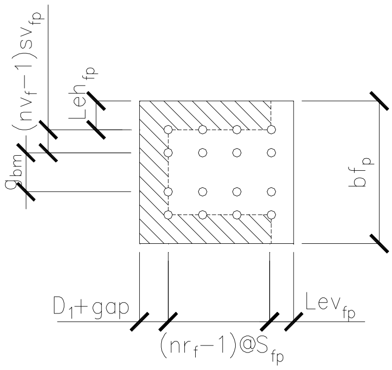
$$Anv := 2\left[\left(nr_{fp} - 1\right) \cdot s_{fp} - \left(nr_{fp} - 1\right) \cdot hd_{fp}\right] \cdot tfp$$

$$Anv = 33.75 \cdot in^2$$

Net Tension Area

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

$$Ant = 8.25 \cdot in^2$$

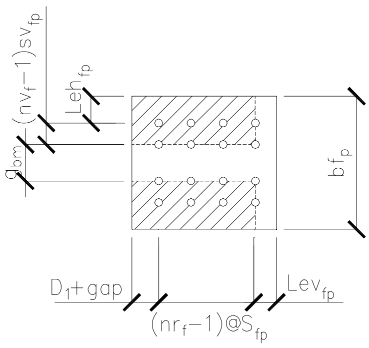


Block Shear Capacity

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

$$Rbs_{fp2} = 926.25 \cdot kips$$

Pattern 3



Gross Shear Area

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

$$Agv = 57 \cdot in^2$$

Net Shear Area

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

$$Anv = 35.625 \cdot in^2$$

Net Tension Area

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

$$Ant = 3.875 \cdot in^2$$

Block Shear Capacity

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

$$Rbs_{fp3} = 820.625 \cdot kips$$

Governing Block Shear Capacity

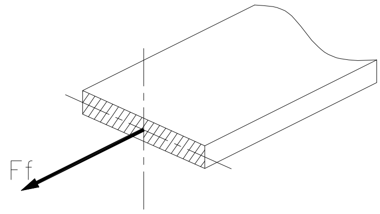
$$Rbs_{fp} := \min\left(Rbs_{fp1}, Rbs_{fp2}, Rbs_{fp3}\right)$$

$$Rbs_{fp} = 820.625 \cdot kips \qquad Ff = 267.195 \cdot kips$$

RESULT = Block Shear Capacity > Force Applied, OK

3. Tension Yielding Capacity of Flange Plate

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



Gross Tension Area,

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

$Ag_{fp} = 10.5 \cdot in^2$

Tension Yielding Capacity,

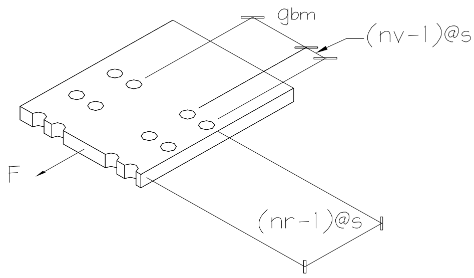
$Rty_{fp} := \Lambda_{ty} \cdot Fy_{fp} \cdot Ag_{fp}$

$Rty_{fp} = 314.371 \cdot kips$ $\qquad Ff = 267.195 \cdot kips$

RESULT = Tension Yielding Capacity > Force Applied, OK

4. Tension Rupture Capacity of Flange Plate

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



Net Tension Area,

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

$Ant = 8.25 \cdot in^2$

Effective Net Tension Area,

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

$Ae = 8.25 \cdot in^2$

Tension Rupture Capacity

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

$Rtr_{fp} = 268.125 \cdot kips$ $\qquad Ff = 267.195 \cdot kips$

RESULT = Tension Rupture Capacity > Force Applied, OK

5. Flange Plate Compression Capacity

(AISC 13th Ed. Specification Chapter J, Section J4.4, page 16.1-113)

(AISC 13th Ed. Specification Chapter E, Section E3, page 16.1-33)

Effective Length Factor,

$K = 0.65$ *(Commentary Table C-C2.2)*

Laterally Unbraced Length of Plate,

$L := D_2 + gap$

$L = 4 \cdot in$

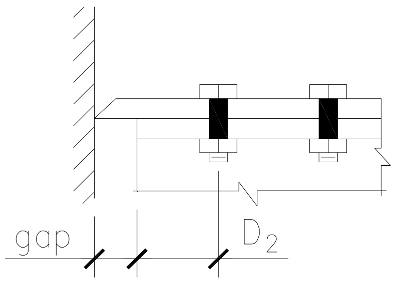
Radius of Gyration,

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

$r = 0.289 \cdot in$

Slenderness Ratio,

$KLr := \dfrac{K\,L}{r} = 9.007$



Elastic Critical Buckling Stress,

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

$F_e = 3528.335 \cdot ksi$

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,

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

$Rcb_{fp} = 314.371 \cdot kips$

$Ff = 267.195 \cdot kips$

RESULT = Compression Capacity > Force Applied, OK

F. WELD CHECK FOR FLANGE PLATE TO COLUMN

1. Forces acting on the Connection

@ Beam Flange,

$Ff = 267.195 \cdot kips$

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2. Weld Check for Flange Plate to Column Flange

a. Flange Plate to Column Flange Connection Using Fillet Weld

Length of weld,

$$Lwf := \min(bfp, bf_{col})$$

$$Lwf = 10.5 \cdot in$$

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

Minimum weld size,

(AISC 13th Ed. Specifications Chapter J, Table J2.4, page 16.1-96

$$w_{min} = \frac{5}{16} \cdot in \qquad w_1 = \frac{3}{4} \cdot in$$

RESULT = Preferred Weld Size > Minimum Weld Size,OK

Shear Strength,

For Column:

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

$$Rv_{col} = 60.84 \cdot \frac{kips}{in}$$

For Flange Plate:

$$Rv_{fp} := \Lambda_{ty} \cdot Fy_{fp} \cdot tfp$$

$$Rv_{fp} = 29.94 \cdot \frac{kips}{in}$$

For Weld:

$$Rv_w := \Lambda_{vw} \cdot 1.5 \cdot 0.6 \cdot Fu_w \cdot \sin(45deg) \cdot n_{ws}$$

$$Rv_w = 44.548 \cdot ksi$$

Maximum effective weld size,

$$w_{eff} := \frac{\min(Rv_{col}, Rv_{fp})}{Rv_w}$$

$$w_{eff} = 0.672 \cdot in$$

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Weld Capacity,

$$Rw_{fp} := \Lambda_{vw} \cdot 1.5 \cdot 0.6 \cdot Fu_{w} \cdot \sin(45deg) \cdot n_{ws} \cdot \min(w_1, w_{eff}) \cdot Lwf$$

$Rw_{fp} = 314.371 \cdot kips$ $\qquad Ff = 267.195 \cdot kips$

RESULT = Weld Capacity > Applied Force, OK

b. *Flange Plate to Column Flange Connection Using Partial Pen*

Check effective thickness of flange,

$tfp \geq \frac{1}{4} in$ $\qquad tfp = 1 \cdot in$

RESULT = Please refer to Fillet weld.

Effective throat thickness,

(AISC 13th Ed. Specifications Chapter J, Table J2.1, page 16.1-94

$$te := tfp - \frac{1}{8} in$$

$te = 0.875 \cdot in$

Partial Penetration Groove Weld Capacity,

$$Rw_{pp} := \min(\Lambda_{ty} Fy_{fp}, \Lambda_{ty} Fy_{col}, \Lambda_{twp} \cdot 0.60 Fu_{w}) \cdot Lwf \cdot te$$

$Rw_{pp} = 205.253 \cdot kips$ $\qquad Ff = 267.195 \cdot kips$

RESULT = Please refer to Fillet weld.

c. *Flange Plate to Column Flange Connection Using Complete Pen*

Complete Penetration Groove Weld Capacity,

$$Rw_{cp} := \Lambda_{ty} \cdot \min(Fy_{col}, Fy_{fp}) \cdot Lwf \cdot tfp$$

$Rw_{cp} = 314.371 \cdot kips$ $\qquad Ff = 267.195 \cdot kips$

RESULT = Please refer to Fillet weld.

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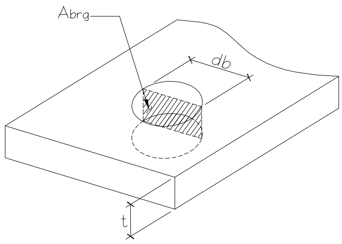
G. BEAM WEB CHECK

1. Bolt Bearing Capacity on Beam



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

Bearing Area,

$$Abrg_{bm} := db_{pl} \cdot tw_{bm}$$

$$Abrg_{bm} = 0.47 \cdot in^2$$

Bolt centerline distance from face of support,

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

$$a_b = 4 \cdot in$$

Eccentric Load Coefficient,

(Table 7-7, AISC 13th Ed.)

$$C = 8.93$$

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

$$Fbe := Fu_{bm} \cdot \left| \begin{array}{ll} \min\left[1.0 \cdot \left(D_1 - 0.5 \cdot hd_{bmw}\right) \cdot tw_{bm}, 2.0 \cdot Abrg_{bm}\right] & \text{if } hd_{bmw} \ge hd_{1s} \\ \min\left[1.2 \cdot \left(D_1 - 0.5 \cdot hd_{bmw}\right) \cdot tw_{bm}, 2.4 \cdot Abrg_{bm}\right] & \text{otherwise} \end{array} \right.$$

$$Fbe = 73.32 \cdot kips$$

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

$$Fbs := Fu_{bm} \cdot \left| \begin{array}{ll} \min\left[1.0 \cdot \left(s_{pl} - hd_{bmw}\right) \cdot tw_{bm}, 2.0 \cdot Abrg_{bm}\right] & \text{if } hd_{bmw} \ge hd_{1s} \\ \min\left[1.2 \cdot \left(s_{pl} - hd_{bmw}\right) \cdot tw_{bm}, 2.4 \cdot Abrg_{bm}\right] & \text{otherwise} \end{array} \right.$$

$$Fbs = 68.737 \cdot kips$$

Bolt Bearing Capacity,

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

$$Rbrg_{bm} = 310.328 \cdot kips \qquad V_{bm} = 97.259 \cdot kips$$

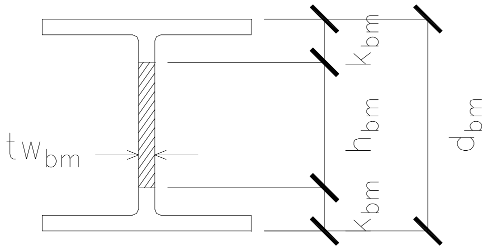
RESULT = Bearing Capacity > Force Applied,OK

2. Shear Capacity of Beam

(AISC 13th Ed, Specifications Chapter G, Section G2.1, pages 16.1-64 to 16.1-66)

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

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



$$h = 21.56 \cdot in$$

Limiting depth-thickness ratio,

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

$$h_{tw} = 45.872$$

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,

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

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$Cv = 1$

Shear Capacity of Section,

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

$Rv_{bm} = 226.54 \cdot kips$ $V_{bm} = 97.259 \cdot kips$

RESULT = Shear Capacity of Section > Force Applied, OK

H. BEAM TO SHEAR PLATE CHECK

1. Direct Bolt Shear Capacity

(AISC 13th Ed. Chapter J, Section J3.6, pages 16.1-108 to 16.1-109)

Shear Capacity per bolt,

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

Bolt Shear Capacity,

$Rb := C \cdot \Lambda rv_{pl}$

$Rb = 129.166 \cdot kips$ $V_{bm} = 97.259 \cdot kips$

RESULT = Bolt Shear Capacity > Force Applied, OK

2. Check for Spacing

(AISC 13th Ed. Specifications Chapter J, Section J3.3 and J3.5, pages 16.1-106 to 16.1-108)

Vertical Spacing,

$s_{pl} = 3 \cdot in$

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

$s_{min} = 2.667 \cdot in$

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

$s_{max} = 11.280 \cdot in$

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

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Horizontal Spacing,

$sv_{pl} = 3 \cdot in$

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

$sv_{min} = 2.667 \cdot in$

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

$sv_{max} = 11.280 \cdot in$

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

3. Check for Edge Distance

(AISC 13th Ed. Specifications Chapter J, Section J3.4 and J3.5, pages 16.1-106 to 16.1-108)

Vertical Edge Distance,

$Lev_{pl} = 1.5 \cdot in$

$Le_{min} = 1.25 \cdot in$

$C_2 = 0 \cdot in$

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

$Lev_{min} = 1.25 \cdot in$

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

$Lev_{max} = 6.000 \cdot in$

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

Horizontal Edge Distance,

$Leh_{pl} = 1.5 \cdot in$

$Leh_{bmw} = 1.5 \cdot in$

$Le_{min} = 1.25 \cdot in$

$Leh_{minpl} = 1.375 \cdot in$

$Leh_{minbm} = 1.25 \cdot in$

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

$Leh_{maxpl} = 6.000 \cdot in$

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

$Leh_{maxbm} = 5.640 \cdot in$

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

I. SHEAR PLATE CHECK

1. Check for Maximum Shear Plate Thickness

(AISC 13th Ed. Manual Part 10, page 10-103)

Exceptions for nv = 1 and nv = 2

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

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

RESULT = Check maximum thickness of plate

Coefficient for Eccentrically Loaded Bolts

(AISC 13th Ed. Manual Part 7, page 7-19)

$C' = 54.167 \cdot in$

Area of Bolts

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

$Ab = 0.785 \cdot in^2$

Length of Plate

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

$Lpl = 18 \cdot in$

Maximum Thickness

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

$tpl_{max} = 0.672 \cdot in$ $\qquad$ $tpl = 0.75 \cdot in$

RESULT = Use the value of the maximum thickness of plate

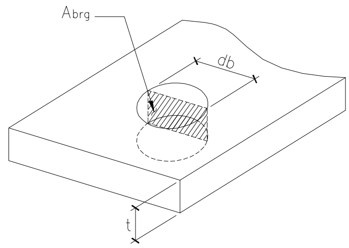
Governing Shear Plate Thickness,

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

$$tpl_g = \frac{5}{8} \cdot in$$

2. Bolt Bearing Capacity of Shear Plate

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



Bearing Area,

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

$$Abrg_{pl} = 0.625 \cdot in^2$$

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

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

$$Fbe = 40.781 \cdot kips$$

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

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

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$Fbs = 81.562 \cdot kips$

Bolt Bearing Capacity,

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

$Rbrg_{pl} = 333.834 \cdot kips$ $\quad\quad V = 97.259 \cdot kips$

RESULT = Bearing Capacity > Force Applied, OK

3. Shear Yielding Capacity of Shear Plate

(AISC 13th Ed, Specifications Chapter J, Section J4.2, page 16.1-112)

Length of Plate,

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

$Lpl = 18 \cdot in$

Check if Length of Plate is acceptable,

(AISC 13th Ed, Manual Part 10, page 10-104)

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

Length = "Plate Length is OK per AISC Requirements"

Gross Shear Capacity,

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

$Rvy_{pl} = 162 \cdot kips$ $\quad\quad V = 97.259 \cdot kips$

RESULT = Shear Yielding Capacity > Force Applied, OK

4. Shear Rupture Capacity of Shear Plate

(AISC 13th Ed, Specifications Chapter J, Section J4.2, page 16.1-112)

Net Area,

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

$Anv = 7.031 \cdot in^2$

Shear Rupture Capacity,

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

$Rvr_{pl} = 122.344 \cdot kips$ $V = 97.259 \cdot kips$

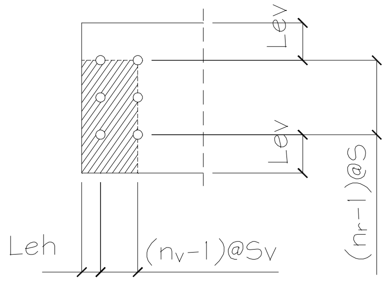
RESULT = Shear Rupture Capacity > Force Applied, OK

5. Block Shear Capacity of Shear Plate

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

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

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

$Agv = 10.312 \cdot in^2$

Net Tension Area

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

$Ant = 1.523 \cdot in^2$

Net Shear Area

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

$Anv = 6.445 \cdot in^2$

Block Shear Capacity of Plate, (J4-5)

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

$Rbs_{pl} = 133.465 \cdot kips$ $V = 97.259 \cdot kips$

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RESULT = Block Shear Capacity > Force Applied, OK

6. Local Buckling Capacity of Shear Plate

(AISC 13th Ed., Manual Part 9, page 9-8 to 9-9)

Distance of bolt line to support,

$$a_b := gap + Leh_{bm}$$

$$a_b = 2.5 \cdot in$$

Coefficient,

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

$$\lambda = 0.141$$

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

$$Q = 1$$

Allowable Buckling Stress,

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

$$Fcr = 36 \cdot ksi$$

Gross Plastic Section Modulus,

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

$$Zx_{pl} = 50.625 \cdot in^3$$

Eccentricity,

$$e_{pl} := a_b$$

$$e_{pl} = 2.5 \cdot in$$

Buckling Capacity,

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

$Rbc_{pl} = 436.527 \cdot kips$ $\quad V = 97.259 \cdot kips$

RESULT = Local Buckling Capacity of plate > Applied Force, OK

7. Flexural Yielding Capacity with von-Mises shear reduction

(AISC 13th 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,

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

$Rfc_{pl} = 151.612 \cdot kips$ $\quad V = 97.259 \cdot kips$

RESULT = Flexural Yielding Capacity > Applied Force, OK

8. Flexural Rupture Capacity

(AISC 13th Ed., Steel Construction Manual Design Examples page IIA-84)

Net Plastic Section Modulus,

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

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

Flexural Rupture Capacity,

(AISC 13th Ed., Manual Part 15, page 15-4)

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

$Rfr_{pl} = 440.437 \cdot kips$ $\quad$ $V = 97.259 \cdot kips$

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,

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

$e_{pl} := gap + Leh_{bmw} + 0.5\,(nv - 1) \cdot sv$

$V_r := V$

$V_r = 97.259 \cdot kips$

$M_r := V_r \cdot e_{pl}$

$M_r = 389.035 \cdot kips \cdot in$

Shear yielding,

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

$V_c = 162 \cdot kips$

Flexural yielding,

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

$M_c = 1091.317 \cdot kips \cdot in$

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Interaction,

$$\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{M_r}{M_c}\right)^2 = 0.488$$

RESULT = Interaction < 1.0, OK

J. SHEAR PLATE TO COLUMN CHECK

1. Rupture Strength at Weld for Column

Rupture Strength at Weld,

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

$Rv_{col} = 1095.12 \cdot kips$ $\quad V_{bm} = 97.259 \cdot kips$

RESULT = Girder Web Capacity > Force Applied, OK.

K. COLUMN LOCAL CHECKS

1. Flange Local Bending

(AISC 13th Ed. Chapter J, Specifications Section J10.2, page 16.1-116)

Flange Force,

$Ff = 267.195 \cdot kips$

Distance of Force to Column End,

$De_{col} := 2in$

Local Bending Capacity,

$$Rfb := \begin{cases} \Lambda_{fb} \cdot 6.25 \cdot tf_{col}^{\,2} \cdot Fy_{col} & \text{if } De_{col} \geq 10 \cdot tf_{col} \\ 0.5 \cdot \Lambda_{fb} \cdot 6.25 \cdot tf_{col}^{\,2} \cdot Fy_{col} & \text{otherwise} \end{cases} \qquad \text{(J10-1)}$$

$Rfb = 227.695 \cdot kips$ $\quad Ff = 267.195 \cdot kips$

RESULT = Provide Stiffener Plate

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2. Web Local Yielding

(AISC 13th Ed. Chapter J, Specifications Section J10.2, page 16.1-116)

Bearing Length,

$N := tfp$

$N = 1 \cdot in$

Web Yielding capacity,

$$Rwy := \Lambda_{wy}\left[\begin{array}{|ll} Fy_{col} \cdot tw_{col} \cdot \left(N + 5 \cdot kdes_{col}\right) & \text{if } De_{col} > d_{col} \\ Fy_{col} \cdot tw_{col} \cdot \left(N + 2.5 \cdot kdes_{col}\right) & \text{otherwise} \end{array}\right] \quad \begin{array}{l} (J10\text{-}2) \\ (J10\text{-}3) \end{array}$$

$Rwy = 209.067 \cdot kips$ $\quad Ff = 267.195 \cdot kips$

RESULT = Provide Stiffener Plate

3. Column Web Crippling

(AISC 13th Ed. Chapter J, Specifications Section J10.3, page 16.1-117)

Web Crippling capacity,

$$E_{sq} := \sqrt{\frac{E \cdot Fy_{col} \cdot tf_{col}}{tw_{col}}}$$

$$N_1 := 1 + 3 \cdot \left(\frac{N}{d_{col}}\right) \cdot \left(\frac{tw_{col}}{tf_{col}}\right)^{1.5}$$

$$N_2 := 1 + \left(\frac{4N}{d_{col}} - 0.2\right) \cdot \left(\frac{tw_{col}}{tf_{col}}\right)^{1.5}$$

$$Rwc := \Lambda_{cr} \left| \begin{array}{lll} 0.8tw_{col}{}^2 \cdot N_1 \cdot E_{sq} & \text{if } De_{col} \geq \frac{d_{col}}{2} & (J10\text{-}4) \\ 0.4tw_{col}{}^2 \cdot N_1 \cdot E_{sq} & \text{if } De_{col} < \frac{d_{col}}{2} \wedge \frac{N}{d_{col}} \leq 0.2 & (J10\text{-}5a) \\ 0.4tw_{col}{}^2 \cdot N_2 \cdot E_{sq} & \text{if } De_{col} < \frac{d_{col}}{2} \wedge \frac{N}{d_{col}} > 0.2 & (J10\text{-}5b) \end{array} \right.$$

$Rwc = 319.585 \cdot kips$ $\quad Ff = 267.195 \cdot kips$

RESULT = Web Crippling Capacity > Force Applied, OK

4. Web Panel Zone Shear

(AISC 13th Ed, Chapter J, Specifications Section J10.6, page 16.1-119 to 120)

Force Acting on the Web Panel Zone,

$$V_{pz} := \min\left(\frac{M_{bm}}{d_{bm} + tfp}, \frac{2\Lambda_b \cdot Fy_{col} \cdot Zx_{col}}{d_{bm} + tfp}\right) - Vs$$

$$V_{pz} = 267.195 \cdot kips$$

Column Strength,

$$P_c := \begin{cases} Fy_{col} \cdot Ag_{col} & \text{if } Code = \text{"LRFD"} \\ 0.60 \cdot Fy_{col} \cdot Ag_{col} & \text{if } Code = \text{"ASD"} \end{cases}$$

$$P_c = 1860 \cdot kips$$

Web Panel Zone Shear Capacity,

$$Rvz_{col} := \begin{cases} \Lambda_v \cdot 0.6 \cdot Fy_{col} \cdot d_{col} \cdot tw_{col} & \text{if } Pu_{col} \le 0.40 \cdot P_c & (J10-9) \\ \Lambda_v \cdot 0.6 \cdot Fy_{col} \cdot d_{col} \cdot tw_{col} \cdot \left(1.4 - \frac{Pu_{col}}{P_c}\right) & \text{otherwise} & (J10-10) \end{cases}$$

$Rvz_{col} = 276.395 \cdot kips$ $\qquad$ $V_{pz} = 267.195 \cdot kips$

RESULT : Web Panel Zone Shear Capacity > Force Applied, OK

5. Shear Buckling of Column Web

(AISC 13th Ed, Chapter G, Section G.2, page 16.1-65)

Minimum Thickness of Column Web based on shear buckling,

$$tw_{colm} := \frac{d_{col} - 2 \cdot kdes_{col}}{2.24} \cdot \sqrt{\frac{Fy_{col}}{E}} \qquad (G2-1)$$

$tw_{colm} = 0.211 \cdot in$ $\qquad$ $tw_{col} = 0.98 \cdot in$

$$h := d_{col} - 2k_{col}$$

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

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

$$a = 0\cdot in$$

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

$$kv = 5$$

$$Cv := \begin{cases} 1 & \text{if } \dfrac{h}{tw_{col}} \le 1.1\cdot\sqrt{\dfrac{kv\cdot E}{Fy_{col}}} \\ \dfrac{1.1\cdot\sqrt{\dfrac{kv\cdot E}{Fy_{col}}}}{\dfrac{h}{tw_{col}}} & \text{if } 1.1\cdot\sqrt{\dfrac{kv\cdot E}{Fy_{col}}} < \dfrac{h}{tw_{col}} \le 1.37\cdot\sqrt{\dfrac{kv\cdot E}{Fy_{col}}} \\ \dfrac{1.51\cdot E\cdot kv}{\left(\dfrac{h}{tw_{col}}\right)^2 Fy_{col}} & \text{if } 1.37\cdot\sqrt{\dfrac{kv\cdot E}{Fy_{col}}} < \dfrac{h}{tw_{col}} \end{cases}$$

$$Cv = 1$$

$$\Lambda_{vcol} := \begin{cases} \Lambda_{vy} & \text{if } \dfrac{h}{tw_{col}} \le 2.24\cdot\sqrt{\dfrac{E}{Fy_{col}}} \\ \Lambda_{v} & \text{otherwise} \end{cases}$$

$$\Lambda_{vcol} = 0.667$$

$$Rv_{col} := \Lambda_{vcol}\cdot 0.6\cdot Fy_{col}\cdot Cv\cdot tw_{col}\cdot d_{col}$$

$Rv_{col} = 307.72\cdot kips$ $\qquad$ $V_{pz} = 267.195\cdot kips$

RESULT = Shear Buckling will not control, OK

L. REINFORCEMENT DESIGN FORCES

1. Required Strength for Doubler Plate

$$Vu_{dp} := \begin{cases} \max\left(V_{pz} - \min\left(Rvz_{col}, Rv_{col}\right), 0kips\right) & \text{if } tw_{col} < tw_{colm} \\ \max\left(V_{pz} - Rvz_{col}, 0kips\right) & \text{otherwise} \end{cases}$$

$$Vu_{dp} = 0 \cdot kips$$

2. Stiffener Plate Design Force

Stiffener Plate Design Force,

$$Fst_t := \begin{cases} \max\left(Ff - \min\left(Rwy, Rfb\right), 0kips\right) & \text{if } Lwf \geq 0.15 \cdot bf_{col} \\ \max\left(Ff - Rwy, 0kips\right) & \text{otherwise} \end{cases}$$

$$Fst_t = 58.128 \cdot kips$$

$$Fst_c := \max\left(Ff - \min\left(Rwy, Rwc\right), 0kips\right)$$

$$Fst_c = 58.128 \cdot kips$$

$$Fst := \max\left(Fst_t, Fst_c\right)$$

$$Fst = 58.128 \cdot kips$$

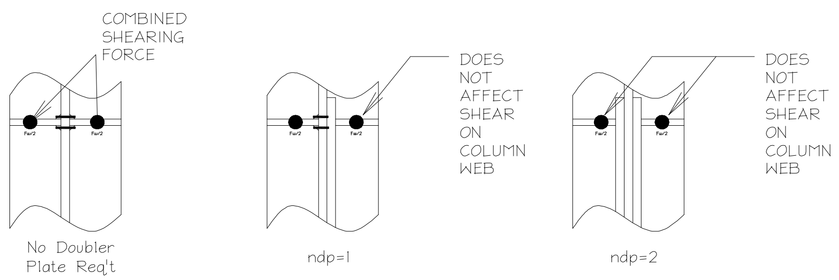
Design Shear Force @ Column Web,

$$Fst_w := Fst$$

$$Fst_w = 58.128 \cdot kips$$

3. Column Web Thickness Check Due to Forces From Stiffener Plates

(Steel Design Guide Series 13, Chapter 4, Section 4.4.2, page 30)



$$twcol_{req1} := \frac{Fst_w}{\Lambda_{vy} \cdot 0.6 \cdot Fy_{col} \cdot (d_{col} - 2 \cdot k_{col}) \cdot 4}$$

$$twcol_{req1} = 0.073 \cdot in$$

$$twcol_{req2} := \frac{Fst_w}{\Lambda_{vy} \cdot 0.6 \cdot Fy_{col} \cdot d_{col} \cdot 2}$$

$$twcol_{req2} = 0.093 \cdot in$$

$$twcol_{req3} := \frac{Fst_w}{\Lambda_{vy} \cdot 0.6 \cdot Fy_{col} \cdot (d_{col} - 2 \cdot k_{col}) \cdot 2}$$

$$twcol_{req3} = 0.146 \cdot in$$

$$twcol_{req4} := \frac{Fst_w}{\Lambda_{vy} \cdot 0.6 \cdot Fy_{col} \cdot d_{col}}$$

$$twcol_{req4} = 0.185 \cdot in$$

Required Thickness of Column Web,

$$twcol_{req} := \begin{cases} \max(twcol_{req3}, twcol_{req4}) & \text{if } Vu_{dp} = 0 \\ \text{otherwise} & \\ \quad \begin{cases} \max(twcol_{req1}, twcol_{req2}) & \text{if } ndp = 1 \\ 0 & \text{otherwise} \end{cases} & \end{cases}$$

$$twcol_{req} = 0.185 \cdot in \qquad tw_{col} = 0.98 \cdot in$$

RESULT = Thickness of Column Web is OK

M. DOUBLER PLATE DESIGN

1. Required Doubler Plate Thickness

Thickness of Doubler Plate,

$$tdp := \begin{cases} 0in & \text{if } Vu_{dp} = 0kips \\ tdp & \text{otherwise} \end{cases}$$

$$tdp = 0 \cdot in$$

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Based on Shear Buckling of Column Web,

$$tdp_{req1} := \begin{cases} \max\left(\dfrac{tw_{colm} - tw_{col}}{ndp}, 0in\right) & \text{if } ndp \geq 1 \\ 0 & \text{otherwise} \end{cases}$$

$$tdp_{req1} = 0 \cdot in$$

Based on Shear Yielding of Doubler Plate,

$$tdp_{req2} := \frac{Vu_{dp}}{\Lambda_{vy} \cdot 0.6 \cdot Fy_{dp} \cdot d_{col} \cdot ndp}$$

$$tdp_{req2} = 0 \cdot in$$

Based on Shear Buckling of Doubler Plate,

$$tdp_{req3} := \frac{\dfrac{\left(d_{col} - 2 \cdot kdes_{col}\right)}{in} \cdot \sqrt{\dfrac{Fy_{dp}}{ksi}}}{418} \cdot in$$

$$tdp_{req3} = 0.163 \cdot in$$

Due to Forces at Stiffener Plate,

$$tdp_{req4} := \frac{Fst_w}{\Lambda_{vy} \cdot 0.6 \cdot Fy_{dp} \cdot \left(Lst_{giv} - 2clip\right) \cdot 4}$$

$$tdp_{req4} = 0.096 \cdot in$$

$$tdp_{req5} := \frac{Fst_w}{\Lambda_{vy} \cdot 0.6 \cdot Fy_{dp} \cdot d_{col} \cdot 2}$$

$$tdp_{req5} = 0.129 \cdot in$$

Required Thickness of Doubler Plate,

$$tdp_{req} := \max\left(tdp_{req1}, tdp_{req2}, tdp_{req3}, tdp_{req4}, tdp_{req5}\right)$$

$tdp_{req} = 0.163 \cdot in$ $\qquad$ $tdp = 0 \cdot in$

RESULT = NOT APPLICABLE, Doubler Plate Not Required.

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2. Weld Check for Each Doubler Plate Connection to Column Flange

a. Doubler Plate to Column Flange Connection Using Fillet Weld

Minimum Thickness of Doubler Plate for Fillet Welding,

$$tdp_{minw} := k_{col} - tf_{col}$$

$tdp_{minw} = 1.315 \cdot in$ $\qquad$ $tdp = 0 \cdot in$

RESULT = NOT APPLICABLE, Doubler Plate Not Required.

b. Doubler Plate to Column Flange Connection Using Fillet Weld

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

Minimum weld size,

(AISC Specifications 13th Ed, Chapter J, Section J2.2b, Table J2.4)

$w_{min1} = 0 \cdot in$

(Steel Design Guide Series 13, Chapter 4, Section 4.3.4, page 24)

$$w_{min2} := \max\left(\frac{\Lambda_{vy} \cdot 0.6 \cdot Fy_{dp} \cdot tdp_{req2} \cdot \sqrt{2}}{\Lambda_{vw} \cdot 0.6 \cdot Fu_w}, tdp_{req2} \cdot \sqrt{2}\right)$$

$w_{min2} = 0 \cdot in$

$$w_{min} := \max\left(w_{min1}, w_{min2}\right)$$

$w_{min} = 0 \cdot in$ $\qquad$ $w_3 = \frac{5}{16} \cdot in$

RESULT = NOT APPLICABLE, Doubler Plate Not Required.

c. Doubler Plate to Column Web Connection Using Complete Pen

Complete Penetration Groove Weld Capacity,

$$Rw_{cps} := \Lambda_{vy} \cdot \min\left(Fy_{col}, Fy_{dp}\right) \cdot 0.60 \cdot Ldp \cdot tdp \cdot ndp$$

$Rw_{cps} = 0 \cdot kips$ $\qquad$ $Vu_{dp} = 0 \cdot kips$

RESULT = NOT APPLICABLE, Doubler Plate Not Required.

3. Weld Check for Doubler Plate to Column Web

(AISC Specifications 13th Ed, Chapter J, Section J2.2b, Table J2.4)

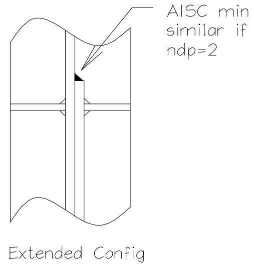
Minimum weld size,

$w_{min} = 0 \cdot in$ $\qquad$ $w_4 = \frac{5}{16} \cdot in$

RESULT = NOT APPLICABLE, Doubler Plate Not Required.

Maximum Weld Size,

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



$w_{max} = 0 \cdot in$ $\qquad$ $w_4 = \frac{5}{16} \cdot in$

RESULT = NOT APPLICABLE, Doubler Plate Not Required.

N. STIFFENER PLATE DESIGN

1. Width of Stiffeners

Maximum Width of Stiffener Plates

$$bst_{max} := Floor\left[0.5 \cdot \left(bf_{col} - tw_{col}\right) - tdp, \frac{1}{4} in\right]$$

$bst_{max} = 7.25 \cdot in$

Minimum Width of Stiffener Plates

(AISC 13th Ed, Specifications Chapter J, Section J10.8, page 16.1-121)

$$bst_{min} := \frac{Lwf}{3} - \left(\frac{tw_{col}}{2} + tdp\right)$$

$bst_{min} = 3.01 \cdot in$

Width of Stiffener Plates

$$bst := min\left(Floor\left(max\left(bst_{min}, bst_{giv}\right), \frac{1}{4} in\right), bst_{max}\right)$$

$bst = 7.25 \cdot in$

2. Length of Stiffeners

Minimum Length of Stiffener Plates,

(Steel Design Guide Series 13, Chapter 4, Section 4.3.3, page 24)

$$Lst_{min} := \frac{d_{col}}{2}$$

$Lst_{min} = 7.85 \cdot in$

Maximum Length of Stiffener Plates

$$Lst_{max} := Floor\left(d_{col} - 2tf_{col}, \frac{1}{4}in\right)$$

$$Lst_{max} = 12.5 \cdot in$$

Length of Stiffener Plates

$$Lst := min\left(Floor\left(max\left(Lst_{min}, Lst_{giv}\right), \frac{1}{4}in\right), Lst_{max}\right)$$

$$Lst = 12.5 \cdot in$$

3. Thickness of Stiffeners

Thickness of Stiffener Plate,

$$tst = 1 \cdot in$$

Minimum Thickness of Stiffener Plate,

(AISC Specifications 13th Ed, Chapter J, Section J10.8, page 16.1-121)

(Steel Design Guide Series 13, Chapter 4, Section 4.3.2, page 23)

$$tst_{min} := max\left(\frac{tfp}{2}, \frac{bst}{15}\right)$$

$tst_{min} = 0.5 \cdot in$ $\qquad$ $tst = 1 \cdot in$

RESULT = Provided Thickness > Minimum Thickness, OK

4. Tension Capacity of Stiffener Plate

(AISC 13th Ed. Chapter J, Specifications Section J4.1, page 16.1-112)

Tension Capacity,

$$Rty_{st} := \Lambda_{ty} \cdot Fy_{st} \cdot 2(bst - clip) \cdot tst$$

$Rty_{st} = 374.251 \cdot kips$ $\qquad$ $Fst_t = 58.128 \cdot kips$

RESULT = Tension Yielding Capacity > Force Applied, OK

5. Bearing Capacity of Stiffener Plate

(AISC 13th Ed. Chapter J, Specifications Section J4.4, page 16.1-113)

Bearing Capacity,

$$Rcb_{st} := 1.8\Lambda_{brg} \cdot Fy_{st} \cdot 2(bst - clip) \cdot tst$$

$Rcb_{st} = 562.5 \cdot kips$ $\qquad$ $Fst_c = 58.128 \cdot kips$

RESULT = Bearing Capacity > Force Applied, OK

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6. Shear Yielding Capacity of Stiffener Plate

(AISC 13th Ed, Chapter J, Specifications Section J4.2, page 16.1-112)

Shear Yielding,

$$Rvy_{st} := \Lambda_{vy} \cdot 0.6 \cdot Fy_{st} \cdot tst \cdot 2 \cdot (Lst - 2clip)$$

$$Rvy_{st} = 420 \cdot kips \qquad Fst_w = 58.128 \cdot kips$$

RESULT = Shear Yielding Capacity > Force Applied, OK

7. Weld Check for Stiffener Plate to Column Flange

a. *Forces acting on each stiffener plate*

$$Fst = 58.128 \cdot kips$$

b. *Stiffener Plate to Column Flange Connection Using Fillet Weld*

Length of weld,

$$Lw_f := \max(bst - clip, 0in)$$

$$Lw_f = 6.25 \cdot in$$

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

Minimum weld size,

(AISC 13th Ed, Chapter J, Specifications Section J2.2b, Table J2.4)

$$w_{min1} = 0.313 \cdot in$$

(Steel Design Guide Series 13, Chapter 4, Section 4.3.4, page 24)

$$w_{min2} := \frac{\Lambda_{ty} \cdot Fy_{st} \cdot tst}{n_{ws} \cdot \Lambda_{vw} \cdot 0.6 \cdot Fu_w \cdot 1.5 \cdot \sin(45deg)}$$

$$w_{min2} = 0.672 \cdot in$$

$$w_{min3} := \frac{Fst}{n_{ws} \cdot \Lambda_{vw} \cdot 0.6 \cdot Fu_w \cdot \sin(45deg) \cdot (bst - clip) \cdot 1.5 \cdot nst}$$

$$w_{min3} = 0.104 \cdot in$$

$$w_{min} := \max(w_{min1}, w_{min2}, w_{min3})$$

$$w_{min} = 0.672 \cdot in \qquad w_5 = \frac{3}{4} \cdot in$$

RESULT = Preferred Fillet Weld Size is OK

c. *Stiffener Plate to Column Flange Connection Using Partial Pen*

Check effective thickness of flange,

$tst \geq \frac{1}{4} in$ $\quad\quad$ $tst = 1 \cdot in$

RESULT = Please refer to Fillet weld.

Effective throat thickness,

(AISC Manual 13th Ed. Part 8, page 8-55, Table 8-2)

$$te := max\left(tst - \frac{1}{8} in, 0in\right)$$

$te = 0.875 \cdot in$

Length of Weld,

$Lw_f = 6.25 \cdot in$

Partial Penetration Groove Weld Capacity,

$$Rw_{pps} := min\left(\Lambda_{ty} Fy_{st}, \Lambda_{ty} Fy_{col}, 0.60 \cdot \Lambda_{twp} Fu_w\right) \cdot Lw_f \cdot te \cdot nst$$

$Rw_{pps} = 244.348 \cdot kips$ $\quad\quad$ $Fst = 58.128 \cdot kips$

RESULT = Please refer to Fillet Weld.

d. *Stiffener Plate to Column Flange Connection Using Complete Pen*

Complete Penetration Groove Weld Capacity,

$$Rw_{cps} := \Lambda_{ty} \cdot min\left(Fy_{col}, Fy_{st}\right) \cdot Lw_f \cdot tst \cdot nst$$

$Rw_{cps} = 374.251 \cdot kips$ $\quad\quad$ $Fst = 58.128 \cdot kips$

RESULT = Please refer to Fillet Weld.

8. Weld Check for Stiffener Plate to Column Web

Length of weld,

$$Lw_w := Lst - 2 \cdot clip$$

$Lw_w = 10.5 \cdot in$

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a. Stiffener Plate to Column Web Connection Using Fillet Weld

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

Minimum weld size,

(AISC Specifications 13th Ed, Chapter J, Section J2.2b, Table J2.4)

$w_{min} = \frac{5}{16} \cdot in$ $\qquad$ $w_6 = \frac{3}{4} \cdot in$

RESULT = Preferred Weld Size > Minimum Weld Size, OK

Shear Strength,

For Column:

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

$$Rv_{col} = 38.22 \cdot \frac{kips}{in}$$

For Stiffener Plate:

$$Rv_{st} := \Lambda_{vr} \cdot 0.6 \cdot Fu_{st} \cdot tst$$

$$Rv_{st} = 19.5 \cdot \frac{kips}{in}$$

For Weld:

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

$$Rv_w = 29.698 \cdot ksi$$

Maximum effective weld size,

$$w_{eff} := \frac{\min(Rv_{col}, Rv_{st})}{Rv_w}$$

$$w_{eff} = 0.657 \cdot in$$

(Steel Design Guide Series 13, Chapter 4, Section 4.3.4, page 24)

$$Rw_w := \Lambda_{vw} \cdot 0.6 \cdot Fu_w \cdot \sin(45deg) \cdot \min(w_{eff}, w_6) \cdot Lw_w \cdot n_{ws} \cdot nst$$

$Rw_w = 409.5 \cdot kips$ $\qquad$ $Fst_w = 58.128 \cdot kips$

RESULT = Weld Capacity > Applied Force, OK

b. Stiffener Plate to Column Web Connection Using Partial Pen

Check effective thickness of flange,

$tst \geq \frac{1}{4} in$ $\qquad tst = 1 \cdot in$

RESULT = Please refer to Fillet weld.

Effective throat thickness,

(AISC Manual 13th Ed. Part 8, page 8-55, Table 8-2)

$$te := \max\left(tst - \frac{1}{8} in, 0 in\right)$$

$te = 0.875 \cdot in$

Length of Weld,

$Lw_w = 10.5 \cdot in$

Partial Penetration Groove Weld Capacity,

$$Rw_{pps2} := \min\left(\Lambda_{vy} Fy_{st}, \Lambda_{vy} Fy_{col}, \Lambda_{vwp} Fu_w\right) \cdot 0.6 Lw_w \cdot te \cdot nst$$

$Rw_{pps2} = 367.5 \cdot kips$ $\qquad Fst_w = 58.128 \cdot kips$

RESULT = Please refer to Fillet weld.

c. Stiffener Plate to Column Web Connection Using Complete Pen

Complete Penetration Groove Weld Capacity,

$$Rw_{cps2} := \Lambda_{vy} \cdot \min\left(Fy_{col}, Fy_{st}\right) \cdot 0.60 \cdot Lw_w \cdot tst \cdot nst$$

$Rw_{cps2} = 420 \cdot kips$ $\qquad Fst_w = 58.128 \cdot kips$

RESULT = Please refer to Fillet weld.

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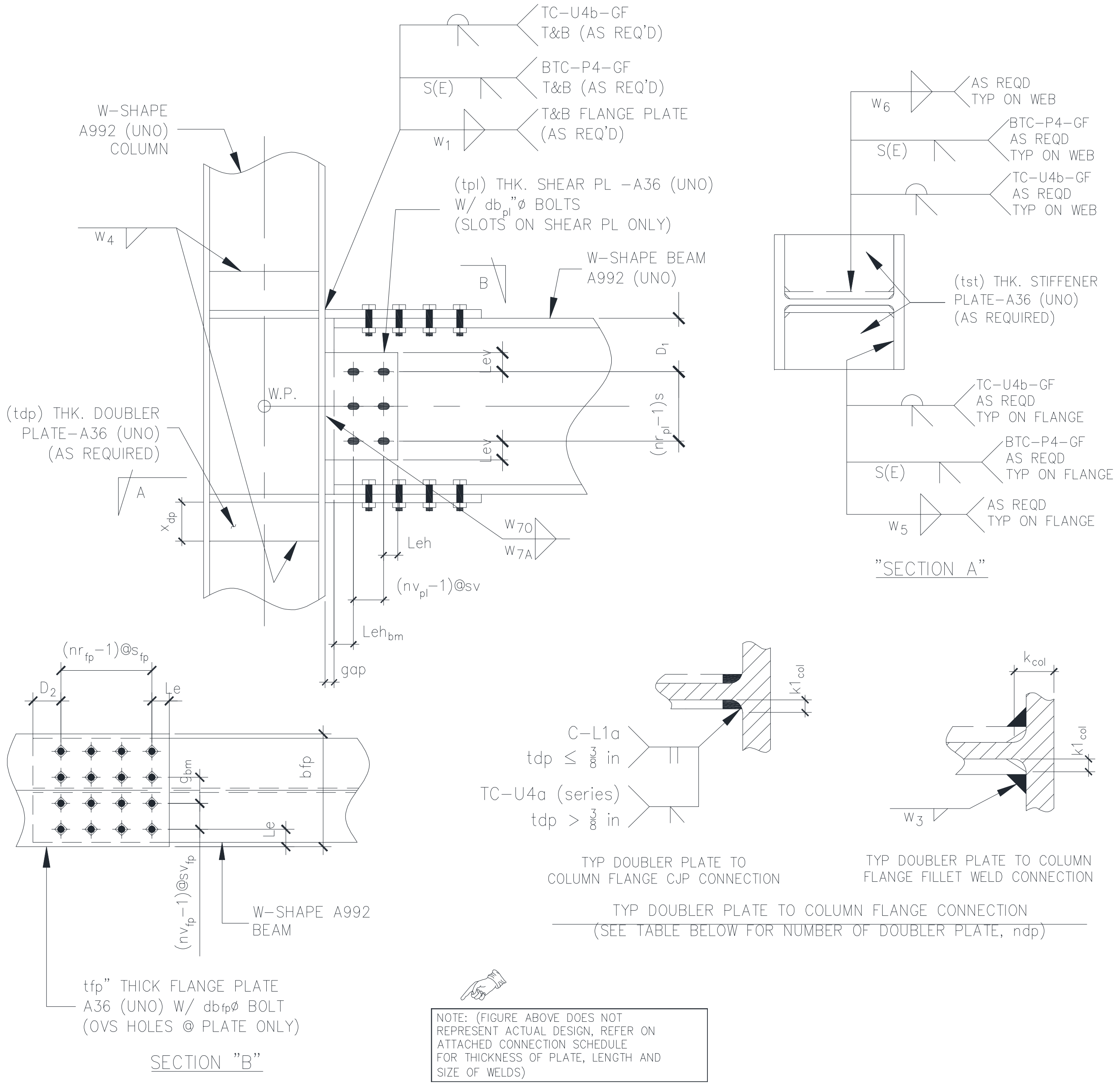
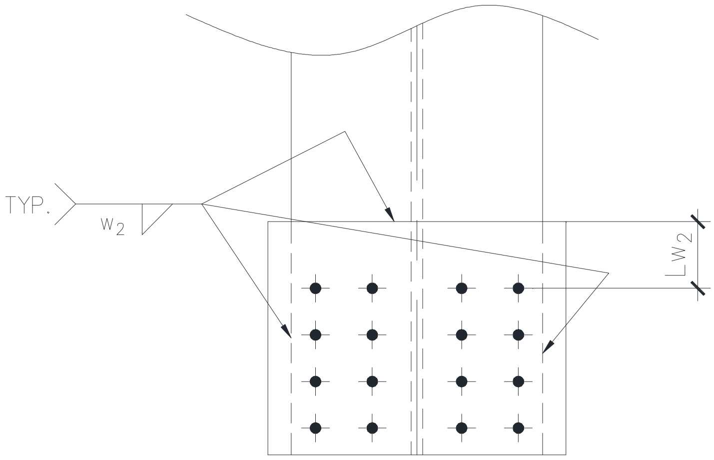
III. DETAILS

A. SKETCH



MIN. BOLT EDGE DISTANCE & SPACING FOR OVS/STD HOLES		
Diam. (db), in	Edge Distance (Le), in	Spacing (s), in
3/4	1 1/2	3
1	1 1/2	3



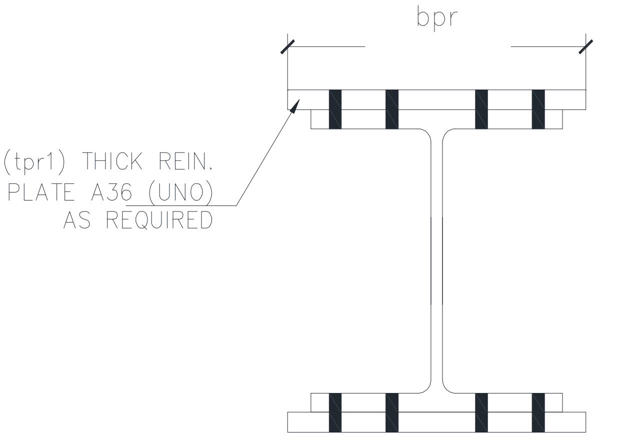
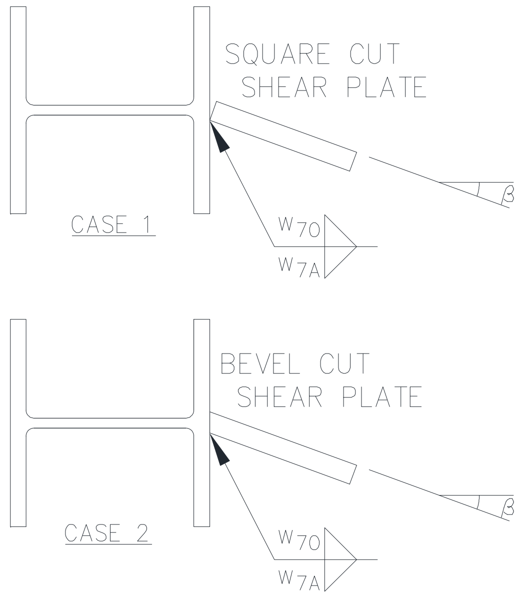


PLATE REINFORCEMENT TO BEAM
(AS REQUIRED)



SHEAR PLATE TO
COLUMN FLANGE WELD

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

MOMENT CONNECTION: DETAIL OF W-SHAPE MOMENT BEAM TO W-SHAPE COLUMN FLANGE (1WAY) WITH FLANGE PLATE AND SHEAR PLATE CONNECTION

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B. TABLE: MOMENT CONNECTION SCHEDULE:

Flange Plate Connection							g_{bm} (in)	D_2 (in)	gap (in)	Plate Reinforcement Conn.				
tfp (in)	Grade	bfp (in)	Bolt Type	db_{fp} (in)	nr_{fp}	nv_{fp}				tpr (in)	Grade	bpr (in)	w_2 (in)	Lw_2 (in)
1	A572-50	10 1/2	A490-SC-STD-CLASS_A	1	10	1	5 1/2	3	1	3/8	A36	10.5	5/16	4.5

D_1 (in)	Shear Plate		Weld			Bolts at Shear Plate				Bolt Spacing		Edge Distance		
	tpl (in)	$Grade_{pl}$	w_{7A}	w_{7O}	CASE	db_{pl} (in)	Type	nr_{pl}	nv_{pl}	s_{pl} (in)	sv_{pl} (in)	Leh_{bmw} (in)	Lev_{pl} (in)	Leh_{pl} (in)
4 3/4	3/4	A36	1/2	1/2	CASE 1	1	A490-SC-SSLT-CLASS_A	6	2	3	3	1 1/2	1 1/2	1 1/2

Beam Moment Conn.			Beam Moment Load, M_{bm} (kip-ft)	Beam Axial Load, P_{bm} (kips)	Beam Shear Load, V_{bm} (kips)	Column Axial Load, Pu_{col} (kips)	Story Shear, Vs (kips)
Size	Grade	w_1 (in)					
W24X84	A992	3/4	558.88224	0	97.258816	0	0

Column		Doubler Plate Conn.						Stiffener Plate Conn.			
Size	Grade	tdp (in)	Grade	x_{dp} (in)	w_3 (in)	w_4 (in)	ndp	tst (in)	Grade	w_5 (in)	w_6 (in)
W14X211	A992	NR	NR	NR	NR	NR	NR	1	A572-50	3/4	3/4

Note:

1. All Welds are E70XX LH
2. NR - Not Required

IV. REFERENCES

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