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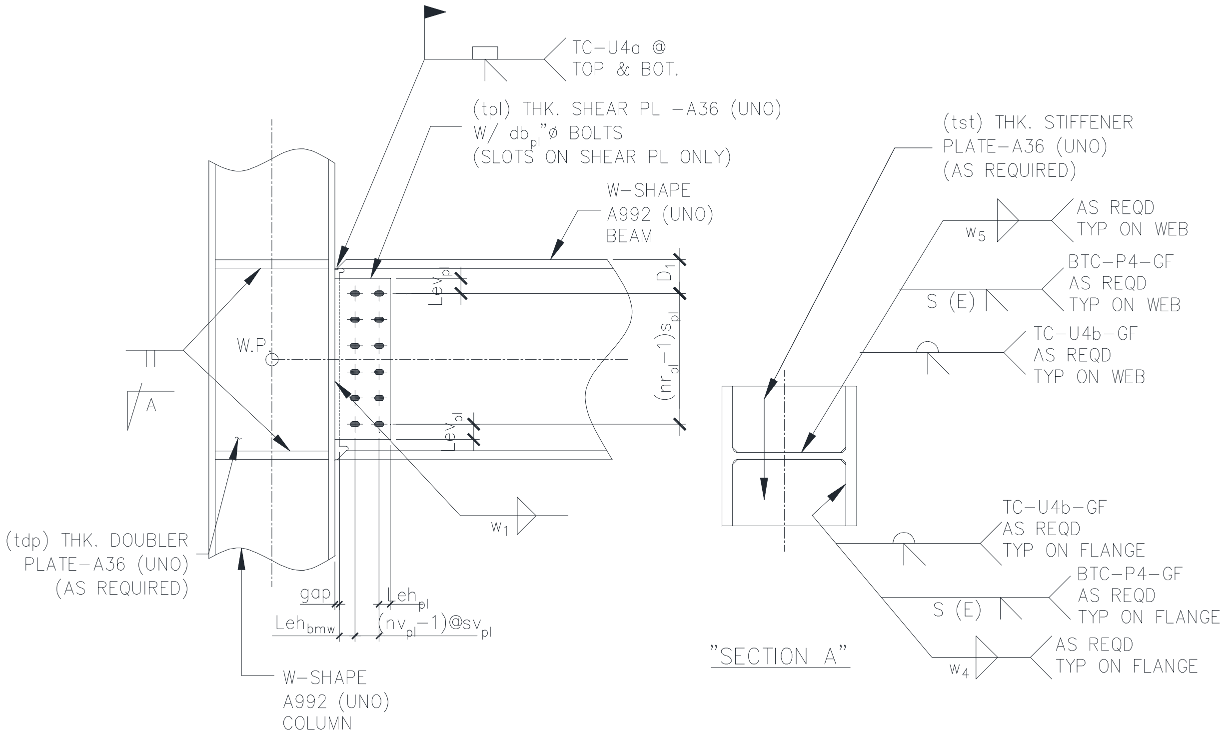
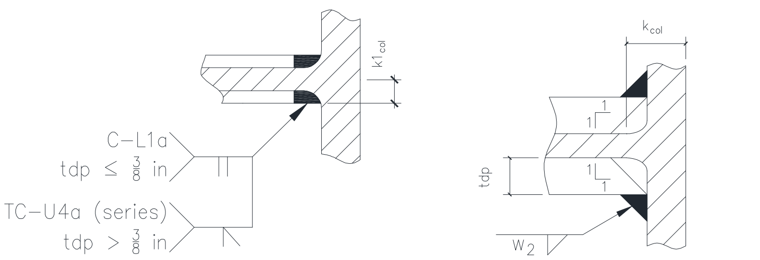
MOMENT CONNECTION: DESIGN OF W-SHAPE MOMENT BEAM TO W-SHAPE COLUMN FLANGE (1WAY) WITH SHEAR PLATE CONNECTION





TYP DOUBLER PLATE TO COLUMN FLANGE CJP CONNECTION

TYP DOUBLER PLATE TO COLUMN FLANGE FILLET WELD CONNECTION

TYP DOUBLER TO COLUMN WEB & FLANGE CONNECTION
(SEE SCHEDULE FOR NUMBER OF DOUBLER PLATE, ndp)

S= T-1/8"
E= | S-1/8" if Field Welded (3/8" MIN)
| S if Shop Welded (1/4" MIN.)
(T= thickness of connecting material)

PARTIAL PENETRATION GROOVE WELD
DETAIL (S & E)

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): W14X193 - A992

$Fy_{col} = 50 \cdot ksi$ $d_{col} = 15.5 \cdot in$ $tw_{col} = 0.89 \cdot in$ $k1_{col} = 1.688 \cdot in$

$Fu_{col} = 65 \cdot ksi$ $bf_{col} = 15.7 \cdot in$ $tf_{col} = 1.44 \cdot in$ $k_{col} = 2.75 \cdot in$

$E := 29000ksi$

BEAM PROPERTIES (bm): W18X60 - A992

$Fy_{bm} = 50 \cdot ksi$ $d_{bm} = 18.2 \cdot in$ $tw_{bm} = 0.415 \cdot in$ $k1_{bm} = 0.813 \cdot in$

$Fu_{bm} = 65 \cdot ksi$ $bf_{bm} = 7.56 \cdot in$ $tf_{bm} = 0.695 \cdot in$ $k_{bm} = 1.375 \cdot in$

Length of Beam, $L_{bm} := 20ft$

SHEAR PLATE (pl): A36

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

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

$Fy_{st} = 50 \cdot ksi$ $bst_{giv} = 7.25 \cdot in$ $tst := 1in$ $clip := 1in$

$Fu_{st} = 65 \cdot ksi$ $Lst_{giv} = 12.5 \cdot in$ $nst := 2$

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

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

Length of Doubler Plate,

$$Ldp := Ceil\left(d_{bm} - 2tst, \frac{1}{4} in\right)$$

$$Ldp = 16.25 \cdot in$$

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, $gap := \frac{1}{2} in$

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

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

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

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

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

Hole diameter:

Shear Plate,

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

Beam Web,

$hd_{bmw} = 1.125 \cdot in$

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

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

total number of bolts: $n_{pl} := nr_{pl} \cdot nv_{pl}$ $n_{pl} = 10$

Bolt First Down from top of beam, $D_1 = 3.25 \cdot in$

WELDS: E70xx LH

$Fu_w = 70 \cdot ksi$

Weld Size

Shear Plate to Column, $w_1 := Ceil\left(\frac{5}{8} \cdot tpl, \frac{1}{16} in\right) = \frac{1}{2} \cdot in$

Doubler Plate to Column Flange (As Req'd), $w_2 := Ceil\left(\frac{5}{8} \cdot tdp, \frac{1}{16} in\right) = \frac{1}{4} \cdot in$

Stiffener Plate to Column Flange (As Req'd), $w_4 := Ceil\left(\frac{3}{4} \cdot tst, \frac{1}{16} in\right)$

Stiffener Plate to Column Web (As Req'd), $w_5 := Ceil\left(\frac{3}{4} \cdot tst, \frac{1}{16} in\right)$

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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),	$\Omega_{twp} = 1.88$	$\phi_{twp} = 0.8$	$\Lambda_{twp} = 0.53$

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APPLIED LOADS:

BEAM

% UDL,	$UDL := 0.5$
Given Shear Load (if any),	$V_{giv} := 50 kips$
Beam Shear Load,	$V_{bm} := V_{giv} + \frac{2 \cdot \Lambda_b \cdot Fy_{bm} \cdot Zx_{bm}}{L_{bm}} = 80.689 \cdot kip$
Given Axial Load (if any),	$P_{giv} := 0 kips$
Beam Axial Load,	$P_{bm} = 0.00 \cdot kips$
% Moment Capacity,	$M_{cap} := 1$
Given Moment Load,	$M_{giv} := 0 kips \cdot ft$
Moment Load,	$M_{bm} = 306.89 \cdot kips \cdot ft$ **100% M.cap**

COLUMN

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

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II. CALCULATIONS:

A. BEAM TO COLUMN FLANGE CONNECTION

1. Forces acting on the Connection

@ Beam Flange,

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

$$Ff = 210.376 \cdot kips$$

2. Beam Flange to Column Flange Connection

Length of connection,

$$Lwf := \min\left(bf_{bm}, bf_{col}\right)$$

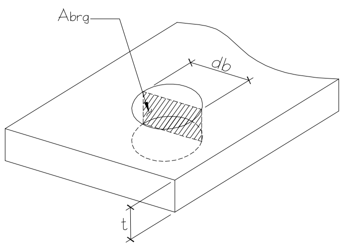
Complete Penetration Groove Weld Capacity,

$$Rw_{cpb} := \Lambda_{ty} \cdot \min\left(Fy_{col}, Fy_{bm}\right) \cdot tf_{bm} \cdot Lwf$$

$Rw_{cpb} = 157.311 \cdot kips$ $\qquad$ $Ff = 210.376 \cdot kips$

RESULT = CJP weld can still fully developed the flange force

B. BEAM WEB CHECK



1. Bolt Bearing Capacity on Beam

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

Bearing Area,

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

$$Abrg_{bm} = 0.415 \cdot in^2$$

Bolt centerline distance from face of support,

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

$$a_b = 3.5 \cdot in$$

Eccentric Load Coefficient,

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

$$C = 7.292$$

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

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

$Fbe = 64.74 \cdot kips$

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

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

$Fbs = 60.694 \cdot kips$

Bolt Bearing Capacity,

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

$Rbrg_{bm} = 224.252 \cdot kips$ $\qquad$ $V_{bm} = 80.689 \cdot kips$

RESULT = Bearing Capacity > Force Applied,OK

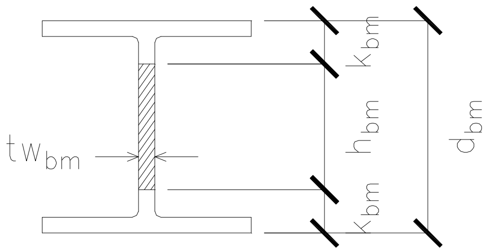
2. Shear Capacity of Beam

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

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

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

$h = 16 \cdot in$



Limiting depth-thickness ratio,

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

$h_{tw} = 38.554$

Clear distance between transverse stiffeners,

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

$a = 0 \cdot in$

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Web plate buckling coefficient,

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

$kv = 5$

Web shear coefficient,

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

$Cv = 1$

Shear Capacity of Section,

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

$Rv_{bm} = 151.06 \cdot kips$ $\qquad V_{bm} = 80.689 \cdot kips$

RESULT = Shear Capacity of Section > Force Applied, OK

C. 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,

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

$Rb = 105.477 \cdot kips$ $\qquad V_{bm} = 80.689 \cdot kips$

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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,

$s_{pl} = 3 \cdot in$

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

$s_{min} = 2.667 \cdot in$

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

$s_{max} = 9.960 \cdot in$

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

Horizontal Spacing,

$sv_{pl} = 3 \cdot in$

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

$sv_{min} = 2.667 \cdot in$

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

$sv_{max} = 9.960 \cdot in$

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,

$Lev_{pl} = 1.5 \cdot in$

$Le_{min} = 1.25 \cdot in$

$C_2 = 0 \cdot in$

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

$Lev_{min} = 1.25 \cdot in$

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

$Lev_{max} = 6.000 \cdot in$

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

Horizontal Edge Distance,

$Leh_{pl} = 1.5 \cdot in$

$Leh_{bmw} = 1.5 \cdot in$

$Le_{min} = 1.25 \cdot in$

$Leh_{minpl} = 1.375 \cdot in$

$Leh_{minbm} = 1.25 \cdot in$

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

$Leh_{maxpl} = 6.000 \cdot in$

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

$Leh_{maxbm} = 4.980 \cdot in$

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

D. 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

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

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

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

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

RESULT = Check maximum thickness of plate

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Coefficient for Eccentrically Loaded Bolts

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

$C' = 38.669 \cdot in$

Area of Bolts

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

$Ab = 0.785 \cdot in^2$

Length of Plate

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

$Lpl = 15 \cdot in$

Maximum Thickness

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

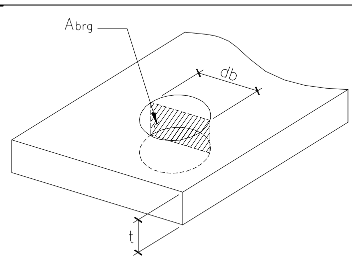
$tpl_{max} = 0.691 \cdot in$ $tpl = 0.75 \cdot in$

RESULT = Use the value of the maximum thickness of plate

Governing Shear Plate Thickness,

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

$tpl_g = \frac{11}{16} \cdot in$



2. Bolt Bearing Capacity of Shear Plate

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

Bearing Area,

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

$$Abrg_{pl} = 0.687 \cdot in^2$$

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

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

$$Fbe = 44.859 \cdot kips$$

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

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

$$Fbs = 89.719 \cdot kips$$

Bolt Bearing Capacity,

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

$$Rbrg_{pl} = 294.419 \cdot kips \qquad V = 80.689 \cdot kips$$

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,

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

$$Lpl = 15 \cdot in$$

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Check if Length of Plate is acceptable,

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

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

Length = "Plate Length is OK per AISC Requirements"

Gross Shear Capacity,

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

$Rvy_{pl} = 148.5 \cdot kips$ $\qquad$ $V = 80.689 \cdot kips$

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,

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

$Anv = 6.445 \cdot in^{2}$

Shear Rupture Capacity,

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

$Rvr_{pl} = 112.148 \cdot kips$ $\qquad$ $V = 80.689 \cdot kips$

RESULT = Shear Rupture Capacity > Force Applied, OK

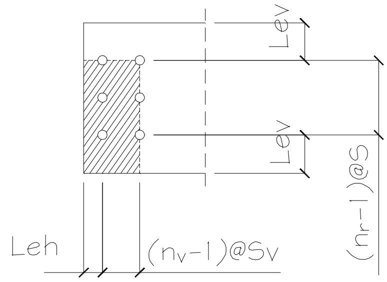
5. Block Shear Capacity of Shear Plate

(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



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

$Agv = 9.281 \cdot in^2$

Net Tension Area

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

$Ant = 1.676 \cdot in^2$

Net Shear Area

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

$Anv = 5.801 \cdot in^2$

Block Shear Capacity of Plate, (J4-5)

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

$Rbs_{pl} = 124.536 \cdot kips$ $V = 80.689 \cdot kips$

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,

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

$$a_b = 2 \cdot in$$

Coefficient,

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

$$\lambda = 0.103$$

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

$$Q = 1$$

Allowable Buckling Stress,

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

$$Fcr = 36 \cdot ksi$$

Gross Plastic Section Modulus,

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

$$Zx_{pl} = 38.672 \cdot in^3$$

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Eccentricity,

$e_{pl} := a_b$

$e_{pl} = 2 \cdot in$

Buckling Capacity,

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

$Rbc_{pl} = 416.823 \cdot kips$ $\quad V = 80.689 \cdot kips$

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,

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

$Rfc_{pl} = 139.64 \cdot kips$ $\quad V = 80.689 \cdot kips$

RESULT = This limit state is not applicable.

8. Flexural Rupture Capacity

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

Net Plastic Section Modulus,

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

$Znet_{pl} = 26.763 \cdot in^3$

Flexural Rupture Capacity,

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

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

$Rfr_{pl} = 388.059 \cdot kips$ $\qquad V = 80.689 \cdot kips$

RESULT = This limit state is not applicable.

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,

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

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

$V_r := V$

$V_r = 80.689 \cdot kips$

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

$M_r = 282.41 \cdot kips \cdot in$

Shear yielding,

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

$V_c = 148.5 \cdot kips$

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Flexural yielding,

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

$M_c = 833.645 \cdot kips \cdot in$

Interaction,

$$\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{M_r}{M_c}\right)^2 = 0.41$$

RESULT = Interaction < 1.0, OK

E. SHEAR PLATE TO COLUMN CHECK

1. Rupture Strength at Weld for Column

Rupture Strength at Weld,

$Rv_{col} := \Lambda_{vr} \cdot 0.6 \cdot Fu_{col} \cdot tf_{col} \cdot 2 \cdot Lpl$

$Rv_{col} = 842.4 \cdot kips$ $V_{bm} = 80.689 \cdot kips$

RESULT = Girder Web Capacity > Force Applied, OK.

F. COLUMN LOCAL CHECKS

1. Flange Local Bending

(AISC 14th Ed. Chapter J, Specifications Section J10.1, page 16.1-133)

Flange Force,

$Ff = 210.376 \cdot kips$

Distance of Force to Column End,

$De_{col} := 36in$

Local Bending Capacity,

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

$Rfb = 388.024 \cdot kips$ $Ff = 210.376 \cdot kips$

RESULT = Local Flange Bending Capacity > Force Applied, OK

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2. Web Local Yielding

(AISC 14th Ed. Specifications, Chapter J, Section J10.2, page 16.1-134)

Bearing Length,

$N := tf_{bm}$

$N = 0.695 \cdot in$

Web Yielding capacity,

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

$Rwy = 323.218 \cdot kips$ $\qquad$ $Ff = 210.376 \cdot kips$

RESULT = Web Yielding Capacity > Force Applied, OK

3. Column Web Crippling

(AISC 14th Ed. Specifications, Chapter J, Section J10.3, pages 16.1-134 to 16.1-135)

Web Crippling capacity,

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

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

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

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

$Rwc = 517.019 \cdot kips$ $\qquad$ $Ff = 210.376 \cdot kips$

RESULT = Web Crippling Capacity > Force Applied, OK

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4. Web Panel Zone Shear

(AISC 14th Ed, Chapter J, Specifications Section J10.6, page 16.1-136 to 137)

Force Acting on the Web Panel Zone,

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

$$V_{pz} = 210.376 \cdot kips$$

Column Strength,

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

$$P_c = 1704 \cdot kips$$

Web Panel Zone Shear Capacity,

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

$$Rvz_{col} = 247.814 \cdot kips \qquad V_{pz} = 210.376 \cdot kips$$

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

5. Shear Buckling of Column Web

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

Minimum Thickness of Column Web based on shear buckling,

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

$$tw_{colm} = 0.212 \cdot in \qquad tw_{col} = 0.89 \cdot in$$

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

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

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

$$a = 0\cdot in$$

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

$$kv = 5$$

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

$$Cv = 1$$

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

$$\Lambda_{vcol} = 0.667$$

Shear Buckling Capacity,

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

$Rv_{col} = 275.9\cdot kips$ $\quad V_{pz} = 210.376\cdot kips$

RESULT = Shear Buckling will not control, OK

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G. 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,

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

$$Fst_t = 0 \cdot kips$$

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

$$Fst_c = 0 \cdot kips$$

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

$$Fst = 0 \cdot kips$$

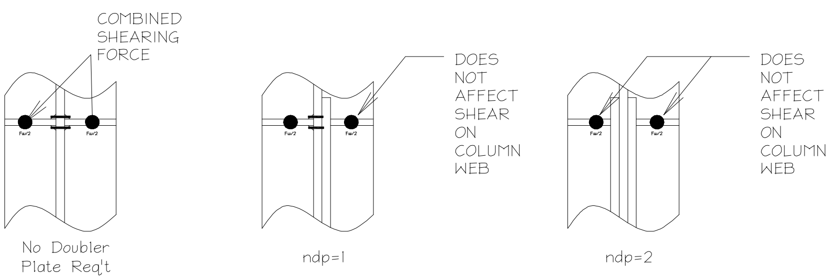
Design Shear Force @ Column Web,

$$Fst_w := Fst$$

$$Fst_w = 0 \cdot kips$$

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

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



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

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

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

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

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

$$twcol_{req3} = 0 \cdot in$$

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

$$twcol_{req4} = 0 \cdot in$$

Required Thickness of Column Web,

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

$twcol_{req} = 0 \cdot in$ $\qquad$ $tw_{col} = 0.89 \cdot in$

RESULT = NOT APPLICABLE, Stiffener Plate Not Required.

H. 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,

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

$$tdp_{req3} = 0.164 \cdot in$$

Due to Forces at Stiffener Plate,

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

$$tdp_{req4} = 0 \cdot in$$

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

$$tdp_{req5} = 0 \cdot in$$

Required Thickness of Doubler Plate,

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

$tdp_{req} = 0.164 \cdot in$ $\qquad$ $tdp = 0 \cdot in$

RESULT = NOT APPLICABLE, Doubler Plate Not Required.

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,

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

$tdp_{minw} = 1.31 \cdot in$ $tdp = 0 \cdot in$

RESULT = NOT APPLICABLE, Doubler Plate Not Required.

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

Minimum weld size,

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

$w_{min} = 0 \cdot in$ $w_2 = \frac{1}{4} \cdot in$

RESULT = NOT APPLICABLE, Doubler Plate Not Required.

Shear Strength,

For Column:

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

$$Rv_{col} = 28.08 \cdot \frac{kips}{in}$$

For Doubler Plate:

$$Rv_{dp} := \Lambda_{vr} \cdot 0.6 \cdot Fu_{dp} \cdot tdp$$

$$Rv_{dp} = 0 \cdot \frac{kips}{in}$$

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,

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

$$w_{eff} = 0 \cdot in$$

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Length of Weld,

$Lw_{dp} := Ldp$

Weld Capacity,

$Rw_{dp} := \Lambda_{vw} \cdot 0.6 \cdot Fu_w \cdot \sin(45deg) \cdot n_{ws} \cdot \min(w_2, w_{eff}) \cdot Lw_{dp}$

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

RESULT = NOT APPLICABLE, Doubler Plate Not Required.

b. *Doubler Plate to Column Flange Connection Using Complete Pen*

Complete Penetration Groove Weld Capacity,

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

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

RESULT = NOT APPLICABLE, Doubler Plate Not Required.

I. STIFFENER PLATE DESIGN

1. Width of Stiffeners

Maximum Width of Stiffener Plates

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

$bst_{max} = 7.25 \cdot in$

Minimum Width of Stiffener Plates

(AISC 14th Ed, Specifications Chapter J, Section J10.8, page 16.1-138)

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

$bst_{min} = 2.075 \cdot in$

Width of Stiffener Plates

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

$bst = 7.25 \cdot in$

2. Length of Stiffeners

Minimum Length of Stiffener Plates,

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

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

$$Lst_{min} = 7.75 \cdot in$$

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 14th Ed, Chapter J, Section J10.8, page 16.1-138)

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

$$tst_{min} := max\left(\frac{tf_{bm}}{2}, \frac{bst}{16}\right)$$

$tst_{min} = 0.453 \cdot in$ $\qquad$ $tst = 1 \cdot in$

RESULT = NOT APPLICABLE, Stiffener Plate Not Required

4. Tension Capacity of Stiffener Plate

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

Tension Capacity,

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

$Rty_{st} = 374.251 \cdot kips$ $\qquad$ $Fst_t = 0 \cdot kips$

RESULT = NOT APPLICABLE, Stiffener Plate Not Required

5. Bearing Capacity of Stiffener Plate

(AISC 14th Ed. Chapter J, Specifications Section J7, page 16.1-131)

Bearing Capacity,

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

$Rcb_{st} = 562.5 \cdot kips$ $\qquad$ $Fst_c = 0 \cdot kips$

RESULT = NOT APPLICABLE, Stiffener Plate Not Required

6. Shear Yielding Capacity of Stiffener Plate

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

Shear Yielding,

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

$Rvy_{st} = 420 \cdot kips$ $\qquad$ $Fst_w = 0 \cdot kips$

RESULT = NOT APPLICABLE, Stiffener Plate Not Required

7. Weld Check for Stiffener Plate to Column Flange

a. *Forces Acting on the Connection*

$Fst = 0 \cdot kips$

b. *Stiffener Plate to Column Flange Connection 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)

$w_{min} = \frac{5}{16} \cdot in$ $\qquad$ $w_4 = \frac{3}{4} \cdot in$

RESULT = NOT APPLICABLE, Stiffener Plate Not Required

Shear Strength,

For Column:

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

$$Rv_{col} = 56.16 \cdot \frac{kips}{in}$$

For Stiffener Plate:

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

$$Rv_{st} = 32.5 \cdot \frac{kips}{in}$$

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,

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

$$w_{eff} = 0.73 \cdot in$$

Length of Weld,

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

$$Lw_f = 6.25 \cdot in$$

Weld Capacity,

$$Rw_f := \Lambda_{vw} \cdot 1.5 \cdot 0.6 \cdot Fu_w \cdot \sin(45deg) \cdot n_{ws} \cdot \min(w_4, w_{eff}) \cdot Lw_f \cdot nst$$

$Rw_f = 406.25 \cdot kips$ $\qquad$ $Fst = 0 \cdot kips$

RESULT = NOT APPLICABLE, Stiffener Plate Not Required

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

Root Face,

$$f := \frac{1}{8} in$$

End Preparation

$$tfp_S := tst - f$$

$$tfp_S = \frac{7}{8} \cdot in$$

Effective Weld Thickness

$tfp_E := tfp_S - 0$

$tfp_E = \frac{7}{8} \cdot in$ $\qquad$ $tfp_{min} = \frac{5}{16} \cdot in$

RESULT = NOT APPLICABLE, Stiffener Plate Not Required

Partial Penetration Groove Weld Capacity,

$Rw_{pps} := \min\left(\Lambda_{tr} Fu_{st}, \Lambda_{vr} Fu_{col} \cdot n_{ws} \cdot 0.6, \Lambda_{twp} Fu_w \cdot 0.60\right) \cdot Lw_f \cdot tfp_E \cdot nst$

$Rw_{pps} = 244.348 \cdot kips$ $\qquad$ $Fst = 0 \cdot kips$

RESULT = NOT APPLICABLE, Stiffener Plate Not Required

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

Complete Penetration Groove Weld Capacity,

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

$Rw_{cps} = 374.251 \cdot kips$ $\qquad$ $Fst = 0 \cdot kips$

RESULT = NOT APPLICABLE, Stiffener Plate Not Required

8. Weld Check for Stiffener Plate to Column Web

a. *Force Acting on the Connection*

$Fst_w = 0 \cdot kips$

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

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

Minimum weld size,

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

$w_{min} = \frac{5}{16} \cdot in$ $\qquad$ $w_5 = \frac{3}{4} \cdot in$

RESULT = NOT APPLICABLE, Stiffener Plate Not Required

Shear Strength,

For Column:

$$Rv_{col} := \Lambda_{vr} \cdot 0.6 \cdot Fu_{col} \cdot \frac{tw_{col}}{2} \cdot n_{ws}$$

$$Rv_{col} = 17.355 \cdot \frac{kips}{in}$$

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,

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

$$w_{eff} = 0.584 \cdot in$$

Length of weld,

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

$$Lw_w = 10.5 \cdot in$$

Weld Capacity,

$$Rw_w := \Lambda_{vw} \cdot 0.6 \cdot Fu_w \cdot \sin(45deg) \cdot \min(w_{eff}, w_5) \cdot Lw_w \cdot n_{ws} \cdot nst$$

$Rw_w = 364.455 \cdot kips$ $\qquad$ $Fst_w = 0 \cdot kips$

RESULT = NOT APPLICABLE, Stiffener Plate Not Required

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

Root Face,

$$f := \frac{1}{8} in$$

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End Preparation

$tfp_S := tst - f$

$tfp_S = \frac{7}{8} \cdot in$

Effective Weld Thickness

$tfp_E := tfp_S - 0$

$tfp_E = \frac{7}{8} \cdot in$ $tfp_{min} = \frac{5}{16} \cdot in$

RESULT = NOT APPLICABLE, Stiffener Plate Not Required

Partial Penetration Groove Weld Capacity,

$Rw_{pps2} := \min\left(\Lambda_{vr} Fu_{st}, \Lambda_{vr} Fu_{col}, \Lambda_{vwp} Fu_w\right) \cdot 0.6 \cdot Lw_w \cdot tfp_E \cdot nst$

$Rw_{pps2} = 358.312 \cdot kips$ $Fst_w = 0 \cdot kips$

RESULT = NOT APPLICABLE, Stiffener Plate Not Required

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

Complete Penetration Groove Weld Capacity,

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

$Rw_{cps2} = 420 \cdot kips$ $Fst_w = 0 \cdot kips$

RESULT = NOT APPLICABLE, Stiffener Plate Not Required

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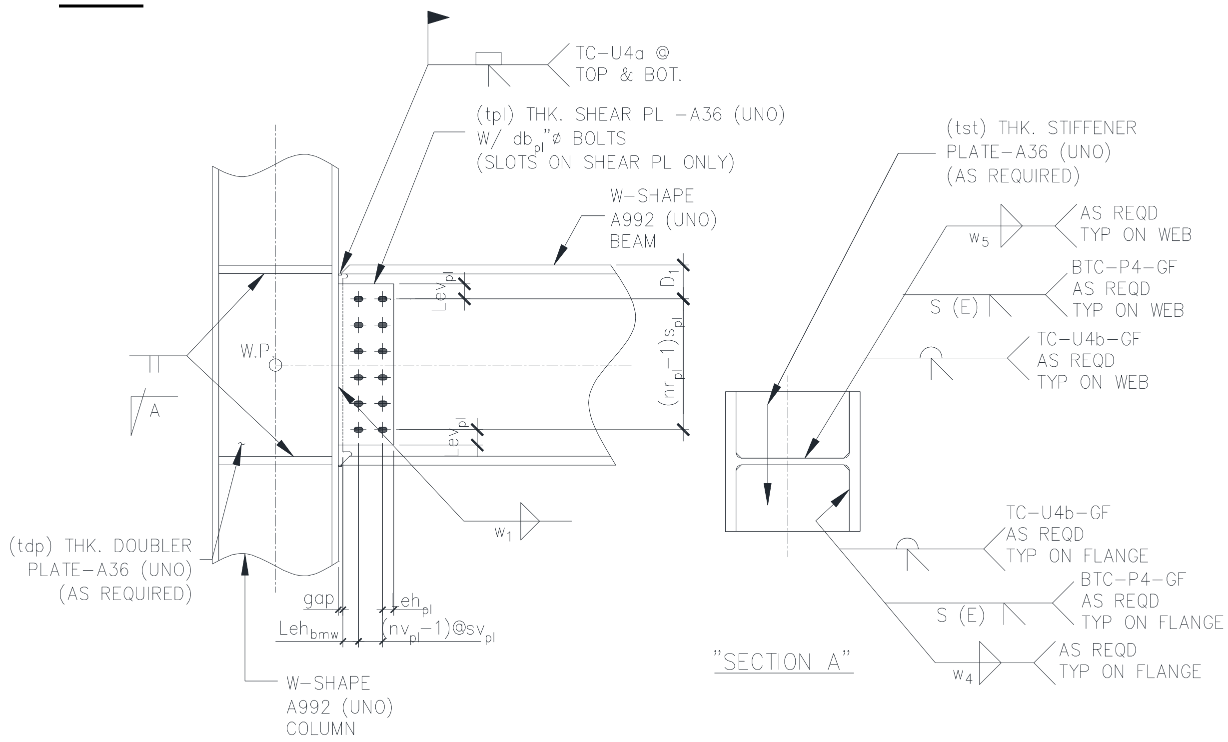
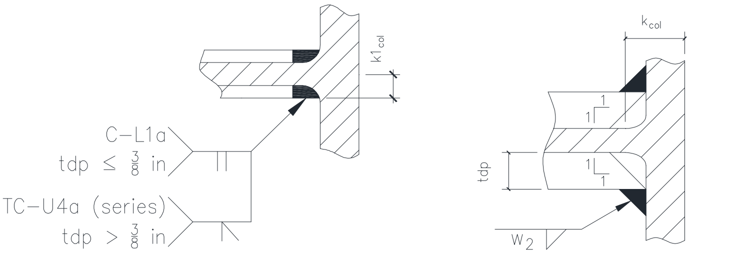
III. DETAILS

A. SKETCH





TYP DOUBLER PLATE TO COLUMN FLANGE CJP CONNECTION

TYP DOUBLER PLATE TO COLUMN FLANGE FILLET WELD CONNECTION

TYP DOUBLER TO COLUMN WEB & FLANGE CONNECTION
(SEE SCHEDULE FOR NUMBER OF DOUBLER PLATE, ndp)

S= T-1/8"
E= S-1/8" if Field Welded (3/8" MIN)
S if Shop Welded (1/4" MIN.)
(T= thickness of connecting material)

PARTIAL PENETRATION GROOVE WELD
DETAIL (S & E)

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 SHEAR PLATE CONNECTION

B. **TABLE: MOMENT CONNECTION SCHEDULE:**

Beam		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					
W18X60	A992	306 39/44	0	80 42/61	0	0

Shear Plate		Weld	Bolts at Shear Plate				Leh_{bmw} (in)	Lev_{pl} (in)	Lev_{pl} (in)	D_1 (in)	gap (in)
tpl (in)	$Grade_{pl}$	w_1 (in)	db_{pl} (in)	Type	nr_{pl}	nv_{pl}					
3/4	A36	1/2	1	A490-SC-SSLT-CLASS_A	5	2	1 1/2	1 1/2	1 1/2	3 1/4	1/2

Column		Doubler Plate Conn.				Stiffener Plate Conn.				Remarks
Size	Grade	tdp (in)	Grade	w_2 (in)	ndp	tst (in)	Grade	w_4 (in)	w_5 (in)	
W14X193	A992	NR	NR	NR	NR	NR	NR	NR	NR	

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