REFERENCES	CALCULATIONS	RESULTS
SkyCiv CLOUD STRUCTURAL SOFTWARE	**Column Splice Connection | AISC 360-16 ASD**	
	Lower Column Section Properties: $\mathrm{W14x90}$ - Lower Column Size $d_{lc} = 14\ \mathrm{in}$ - Lower Column Depth $t_{w,lc} = 0.44\ \mathrm{in}$ - Lower Column Web Thickness $b_{f,lc} = 14.5\ \mathrm{in}$ - Lower Column Flange Width $t_{f,lc} = 0.71\ \mathrm{in}$ - Lower Column Flange Thickness $A_{lc} = 26.5\ \mathrm{in}^2$ - Lower Column Area **Lower Column Grade Information:** $\mathrm{A992}$ - Material Grade $F_{y,lc} = 50\ \mathrm{ksi}$ - Lower Column Yield Stress $F_{u,lc} = 65\ \mathrm{ksi}$ - Lower Column Tensile Stress	
	Upper Column Section Properties: $\mathrm{W14x90}$ - Upper Column Size $d_{uc} = 14\ \mathrm{in}$ - Upper Column Depth $t_{w,uc} = 0.44\ \mathrm{in}$ - Upper Column Web Thickness $b_{f,uc} = 14.5\ \mathrm{in}$ - Upper Column Flange Width $t_{f,uc} = 0.71\ \mathrm{in}$ - Upper Column Flange Thickness $A_{uc} = 26.5\ \mathrm{in}^2$ - Upper Column Area **Upper Column Grade Information:** $\mathrm{A992}$ - Material Grade $F_{y,uc} = 50\ \mathrm{ksi}$ - Upper Column Yield Stress $F_{u,uc} = 65\ \mathrm{ksi}$ - Upper Column Tensile Stress	
Make sure columns' contact area is prepared as fit to bear.	**Design Loads:** $V_a = 150\ \mathrm{kip}$ - Shear Load $T_a = 1000\ \mathrm{kip}$ - Tension Load $C_a = 1500\ \mathrm{kip}$ - Compression Load $M_a = 100\ \mathrm{kip{\cdot}ft}$ - Strong-axis Moment Load Compression load designed for bearing $C_{a,bng} = min\left(C_a, \frac{1.8\ F_{y,c}\ A_{bng}}{\Omega}\right) = min\left((1500\ \mathrm{kip}), \frac{1.8\times(50\ \mathrm{ksi})\times(26.125\ \mathrm{in}^2)}{(2)}\right) = 1175.6\ \mathrm{kip}$ $C_{web} = 67.752\ \mathrm{kip}$ - Compression load at web connection. This is proportioned based on web area over the total gross area. Compression load per flange connection $C_{flange} = \frac{(C_a - C_{a,bng} - C_{web})}{2} + \frac{M_a}{d_c - t_{fc}} = \frac{((1500\ \mathrm{kip})-(1175.6\ \mathrm{kip})-(67.752\ \mathrm{kip}))}{2} + \frac{(100\ \mathrm{kipft})}{(14\ \mathrm{in})-(0.71\ \mathrm{in})} = 218.6\ \mathrm{kip}$ $T_{web} = 208.88\ \mathrm{kip}$ - Tension load at web connection. This is proportioned based on web area over the total gross area. Tension load per flange connection $T_{flange} = \frac{(T_a - T_{web})}{2} + \frac{M_a}{d_c - t_{fc}} = \frac{((1000\ \mathrm{kip})-(208.88\ \mathrm{kip}))}{2} + \frac{(100\ \mathrm{kipft})}{(14\ \mathrm{in})-(0.71\ \mathrm{in})} = 485.86\ \mathrm{kip}$ Axial load per flange connection $P_{flange} = max\left(T_{flange}, C_{flange}\right) = max\left((485.86\ \mathrm{kip}), (218.6\ \mathrm{kip})\right) = 485.86\ \mathrm{kip}$ $P_{web} = 208.88$ - Axial Load at Web $R_a = 257.16\ \mathrm{kip}$ - Resultant Load at Web	
	Bolt Information at Lower Column Flange Connection: $3/4\ \mathrm{in}$ - Bolt Size $\mathrm{A325\text{-}SC(A)}$ - Bolting Category $d_b = 0.75\ \mathrm{in}$ - Bolt Diameter $F_{nt} = 90\ \mathrm{ksi}$ - Bolt Nominal Tensile Strength $F_{nv} = 54\ \mathrm{ksi}$ - Bolt Nominal Shear Strength $N_s = 1$ - Number of Slip Planes OVS - Bolt Hole Type at Flange Plate STD - Bolt Hole Type at Column Flange **Bolt Information at Upper Column Flange Connection:** $3/4\ \mathrm{in}$ - Bolt Size $\mathrm{A325\text{-}SC(A)}$ - Bolting Category $d_b = 0.75\ \mathrm{in}$ - Bolt Diameter $F_{nt} = 90\ \mathrm{ksi}$ - Bolt Nominal Tensile Strength $F_{nv} = 54\ \mathrm{ksi}$ - Bolt Nominal Shear Strength $N_s = 1$ - Number of Slip Planes OVS - Bolt Hole Type at Flange Plate STD - Bolt Hole Type at Column Flange **Bolt Information at Lower Column Web Connection:** $3/4\ \mathrm{in}$ - Bolt Size $\mathrm{A325\text{-}SC(A)}$ - Bolting Category $d_b = 0.75\ \mathrm{in}$ - Bolt Diameter $F_{nt} = 90\ \mathrm{ksi}$ - Bolt Nominal Tensile Strength $F_{nv} = 54\ \mathrm{ksi}$ - Bolt Nominal Shear Strength $N_s = 1$ - Number of Slip Planes OVS - Bolt Hole Type at Web Plate STD - Bolt Hole Type at Column Web **Bolt Information at Upper Column Web Connection:** $3/4\ \mathrm{in}$ - Bolt Size $\mathrm{A325\text{-}SC(A)}$ - Bolting Category $d_b = 0.75\ \mathrm{in}$ - Bolt Diameter	

	$F_{nt} = 90\ \mathrm{ksi}$ - Bolt Nominal Tensile Strength $F_{nv} = 54\ \mathrm{ksi}$ - Bolt Nominal Shear Strength $N_s = 1$ - Number of Slip Planes OVS - Bolt Hole Type at Web Plate STD - Bolt Hole Type at Column Web		

REFERENCES	CALCULATIONS	RESULTS

Flange Plate Connection at Lower Column | AISC 360-16 ASD

Flange Plate Geometry:

$b_{fp} = 11$ in - Flange Plate Width

$L_{fp} = 20$ in - Flange Plate Length

$t_{fp} = 1$ in - Flange Plate Thickness

Flange Plate Material Grade:

$F_{yp} = 50$ ksi - Flange Plate Yield Stress

$F_{up} = 65$ ksi - Flange Plate Tensile Stress

Connection Information at Flange Plate:

$n_r = 2$ - Number of Bolt Rows

$g_a = 7$ in - Bolt Gage

$n_c = 3$ - Number of Bolt Columns

$s_c = 3$ in - Bolt Column Spacing

Distances:

$L_{ev,fp} = 2$ in - Flange Plate Vertical Edge Distance

$L_{ev,c} = 2$ in - Column Flange Vertical Edge Distance

$L_{eh,fp} = 2$ in - Flange Plate Horizontal Edge Distance

$L_{eh,c} = 3.75$ in - Column Flange Horizontal Edge Distance

Check No. 1: Connection Detailing Limitations

Detailing Limitations	Limit Value (in)	Actual Value (in)	DCR	Result
Minimum Bolt Gage	4.375	7.000	0.625	**PASS**
Minimum Bolt Column Spacing	2.000	3.000	0.667	**PASS**
Maximum Bolt Column Spacing	6.000	3.000	0.500	**PASS**
Flange Plate Minimum Vertical Edge Distance	1.063	2.000	0.531	**PASS**
Flange Plate Minimum Horizontal Edge Distance	1.063	2.000	0.531	**PASS**
Column Flange Minimum Vertical Edge Distance	1.000	2.000	0.500	**PASS**
Column Flange Minimum Horizontal Edge Distance	1.000	3.750	0.267	**PASS**

Result:

Demand over Capacity Ratio

$DCR = \frac{d}{c} = \frac{(2)}{(3)} = 0.66667$

PASS

Check No. 2: Allowable Capacity of the Bolts in Shear

$\Omega = 1.5$ - Bolt Shear Safety Factor

$\mu = 0.3$ - Mean Slip Coefficient

$D_u = 1.13$

$h_f = 1$ - Filler Factor for SC Bolts

$d_b = 0.75$ in - Bolt Diameter

$T_b = 28$ kip - Minimum Bolt Pretension

$N_s = 1$ - Number of Slip Planes

$n_r = 2$ - Number of Bolt Rows

$n_c = 3$ - Number of Bolt Columns

AISC 360-16 Chapter J3.8 Eq. (J3-4)

$\frac{R_n}{\Omega}$ - Allowable Bolt Shear Capacity

$$\frac{R_n}{\Omega} = \frac{\mu\, D_u\, h_f\, T_b\, N_s\, n_r\, n_c}{\Omega}$$

$$\frac{R_n}{\Omega} = \frac{(0.3) \times (1.13) \times (1) \times (28\text{ kip}) \times (1) \times (2) \times (3)}{(1.5)}$$

$$\frac{R_n}{\Omega} = 37.968\text{ kip}$$

Result:

Demand over Capacity Ratio

$DCR = \frac{P_{flange}}{\frac{R_n}{\Omega}} = \frac{(485.86\text{ kip})}{(37.968\text{ kip})} = 12.796$

FAIL

Check No. 3: Allowable Bolt Bearing Capacity of the Flange Plate

Calculate the bolt bearing capacity of the flange plate.

$\Omega = 2$ - Bolt Bearing Safety Factor

$d_b = 0.75$ in - Bolt Diameter

$t_{fp} = 1$ in - Flange Plate Thickness

$F_{up} = 65$ ksi - Flange Plate Tensile Stress

$n_c = 3$ - Number of Bolt Columns

$n_r = 2$ - Number of Bolt Rows

AISC 360-16 Chapter J3.10 Eq. (J3-6a)

$\frac{R_{n\_bearing}}{\Omega}$ - Allowable Bolt Bearing Capacity of Section

$$\frac{R_{n\_bearing}}{\Omega} = \frac{2.4\, d_b\, t_{fp}\, F_{up}\, n_c\, n_r}{\Omega}$$

$$\frac{R_{n\_bearing}}{\Omega} = \frac{2.4 \times (0.75\text{ in}) \times (1\text{ in}) \times (65\text{ ksi}) \times (3) \times (2)}{(2)}$$

$$\frac{R_{n\_bearing}}{\Omega} = 351\text{ kip}$$

Calculate the clear distance of outer bolts on flange plate.

$L_{ev,fp} = 2$ in - Flange Plate Vertical Edge Distance

$d_h = 0.9375$ in - Vertical Bolt Hole Dimension at Flange Plate

l_{c1} - Clear Distance at First Bolt Row

$$l_{c1} = L_{ev,fp} - \frac{d_h}{2}$$

$$l_{c1} = (2\text{ in}) - \frac{(0.9375\text{ in})}{2}$$

$$l_{c1} = 1.5313\text{ in}$$

Calculate the clear distance of inner bolts on flange plate.
$s_c = 3\text{ in}$ - Bolt Column Spacing
$d_h = 0.9375\text{ in}$ - Vertical Bolt Hole Dimension at Flange Plate
l_{c2} - Clear Distance at Rest of Bolts

$$l_{c2} = s_c - d_h$$

$$l_{c2} = (3\text{ in}) - (0.9375\text{ in})$$

$$l_{c2} = 2.0625\text{ in}$$

Calculate the bolt tear-out capacity of the flange plate.
$l_{c1} = 1.5313\text{ in}$ - Clear Distance at First Bolt Row
$l_{c2} = 2.0625\text{ in}$ - Clear Distance at Rest of Bolts
$t_{fp} = 1\text{ in}$ - Flange Plate Thickness
$F_{up} = 65\text{ ksi}$ - Flange Plate Tensile Stress
$n_c = 3$ - Number of Bolt Columns
$n_r = 2$ - Number of Bolt Rows
$\Omega = 2$ - Bolt Bearing Safety Factor

AISC 360-16 Chapter J3.10 Eq. (J3-6c)

$\frac{R_{n\_tearout}}{\Omega}$ - Allowable Bolt Tear-out Capacity of Section

$$\frac{R_{n\_tearout}}{\Omega} = \frac{1.2\, l_{c1}\, t_{fp}\, F_{up}\, n_r + 1.2\, l_{c2}\, t_{fp}\, F_{up}\, n_r\ (n_c - 1)}{\Omega}$$

$$\frac{R_{n\_tearout}}{\Omega} = \frac{1.2 \times (1.5313\text{ in}) \times (1\text{ in}) \times (65\text{ ksi}) \times (2) + 1.2 \times (2.0625\text{ in}) \times (1\text{ in}) \times (65\text{ ksi}) \times (2) \times ((3) - 1)}{(2)}$$

$$\frac{R_{n\_tearout}}{\Omega} = 441.19\text{ kip}$$

Determine the governing bearing and tear-out capacity of the bolt group on flange plate.

AISC 360-16 Chapter J3.10 Eq. (J3-6a)

$\frac{R_{n\_bearing}}{\Omega} = 351\text{ kip}$ - Allowable Bolt Bearing Capacity of Section

AISC 360-16 Chapter J3.10 Eq. (J3-6c)

$\frac{R_{n\_tearout}}{\Omega} = 441.19\text{ kip}$ - Allowable Bolt Tear-out Capacity of Section

AISC 360-16 Chapter J3.10

$\frac{R_n}{\Omega}$ - Governing Allowable Capacity

$$\frac{R_n}{\Omega} = min\left(\frac{R_{n\_bearing}}{\Omega}, \frac{R_{n\_tearout}}{\Omega}\right)$$

$$\frac{R_n}{\Omega} = min\left((351\text{ kip}), (441.19\text{ kip})\right)$$

$$\frac{R_n}{\Omega} = 351\text{ kip}$$

Result:
Demand over Capacity Ratio
$DCR = \frac{P_{flange}}{\frac{R_n}{\Omega}} = \frac{(485.86\text{ kip})}{(351\text{ kip})} = 1.3842$

FAIL

Check No. 4: Allowable Bolt Bearing Capacity of the Column Flange

Calculate the bolt bearing capacity of the column flange.
$\Omega = 2$ - Bolt Bearing Safety Factor
$d_b = 0.75\text{ in}$ - Bolt Diameter
$t_{f,lc} = 0.71\text{ in}$ - Lower Column Flange Thickness
$F_{u,lc} = 65\text{ ksi}$ - Lower Column Tensile Stress
$n_c = 3$ - Number of Bolt Columns
$n_r = 2$ - Number of Bolt Rows

AISC 360-16 Chapter J3.10 Eq. (J3-6a)

$\frac{R_{n\_bearing}}{\Omega}$ - Allowable Bolt Bearing Capacity of Section

$$\frac{R_{n\_bearing}}{\Omega} = \frac{2.4\, d_b\, t_{f,lc}\, F_{u,lc}\, n_c\, n_r}{\Omega}$$

$$\frac{R_{n\_bearing}}{\Omega} = \frac{2.4 \times (0.75\text{ in}) \times (0.71\text{ in}) \times (65\text{ ksi}) \times (3) \times (2)}{(2)}$$

$$\frac{R_{n\_bearing}}{\Omega} = 249.21\text{ kip}$$

Calculate the clear distance of outer bolts on column flange.
$L_{ev,c} = 2\text{ in}$ - Column Flange Vertical Edge Distance
$d_h = 0.8125\text{ in}$ - Vertical Bolt Hole Dimension at Column Flange
l_{c1} - Clear Distance at First Bolt Row

$$l_{c1} = L_{ev,c} - \frac{d_h}{2}$$

$$l_{c1} = (2 \text{ in}) - \frac{(0.8125 \text{ in})}{2}$$

$$l_{c1} = 1.5938 \text{ in}$$

Calculate the clear distance of inner bolts on column flange.

$s_c = 3$ in - Bolt Column Spacing

$d_h = 0.8125$ in - Vertical Bolt Hole Dimension at Column Flange

l_{c2} - Clear Distance at Rest of Bolts

$$l_{c2} = s_c - d_h$$

$$l_{c2} = (3 \text{ in}) - (0.8125 \text{ in})$$

$$l_{c2} = 2.1875 \text{ in}$$

Calculate the bolt tear-out capacity of the column flange.

$l_{c1} = 1.5938$ in - Clear Distance at First Bolt Row

$l_{c2} = 2.1875$ in - Clear Distance at Rest of Bolts

$t_{f,lc} = 0.71$ in - Lower Column Flange Thickness

$F_{u,lc} = 65$ ksi - Lower Column Tensile Stress

$n_c = 3$ - Number of Bolt Columns

$n_r = 2$ - Number of Bolt Rows

$\Omega = 2$ - Bolt Bearing Safety Factor

AISC 360-16 Chapter J3.10 Eq. (J3-6c) $\frac{R_{n\_tearout}}{\Omega}$ - Allowable Bolt Tear-out Capacity of Section

$$\frac{R_{n\_tearout}}{\Omega} = \frac{1.2\, l_{c1}\, t_{f,lc}\, F_{u,lc}\, n_r + 1.2\, l_{c2}\, t_{f,lc}\, F_{u,lc}\, n_r\, (n_c - 1)}{\Omega}$$

$$\frac{R_{n\_tearout}}{\Omega} = \frac{1.2 \times (1.5938 \text{ in}) \times (0.71 \text{ in}) \times (65 \text{ ksi}) \times (2) + 1.2 \times (2.1875 \text{ in}) \times (0.71 \text{ in}) \times (65 \text{ ksi}) \times (2) \times ((3) - 1)}{(2)}$$

$$\frac{R_{n\_tearout}}{\Omega} = 330.55 \text{ kip}$$

Determine the governing bearing and tear-out capacity of the bolt group on column flange.

AISC 360-16 Chapter J3.10 Eq. (J3-6a) $\frac{R_{n\_bearing}}{\Omega} = 249.21$ kip - Allowable Bolt Bearing Capacity of Section

AISC 360-16 Chapter J3.10 Eq. (J3-6c) $\frac{R_{n\_tearout}}{\Omega} = 330.55$ kip - Allowable Bolt Tear-out Capacity of Section

AISC 360-16 Chapter J3.10 $\frac{R_n}{\Omega}$ - Governing Allowable Capacity

$$\frac{R_n}{\Omega} = min\left(\frac{R_{n\_bearing}}{\Omega}, \frac{R_{n\_tearout}}{\Omega}\right)$$

$$\frac{R_n}{\Omega} = min\left((249.21 \text{ kip}), (330.55 \text{ kip})\right)$$

$$\frac{R_n}{\Omega} = 249.21 \text{ kip}$$

Result:

Demand over Capacity Ratio

$DCR = \frac{P_{flange}}{\frac{R_n}{\Omega}} = \frac{(485.86 \text{ kip})}{(249.21 \text{ kip})} = 1.9496$

FAIL

Check No. 5: Allowable Block Shear Capacity of the Flange Plate

Calculate the net area of the flange plate subject to tension.

$t_{fp} = 1$ in - Flange Plate Thickness

$n_r = 2$ - Number of Bolt Rows

$s_r = 0$ in - Bolt Row Spacing

$g_a = 7$ in - Bolt Gage

$L_{eh,fp} = 2$ in - Flange Plate Horizontal Edge Distance

$d_h = 0.9375$ in - Horizontal Bolt Hole Dimension at Flange Plate

A_{nt} - Net Area Subject to Tension (L-pattern)

$$A_{nt} = t_{fp}\, [(n_r - 2)\, s_r + g_a + L_{eh,fp} - (n_r - 0.5)\, (d_h + 0.0625 \text{ in})]$$

$$A_{nt} = (1 \text{ in}) \times [((2) - 2) \times (0 \text{ in}) + (7 \text{ in}) + (2 \text{ in}) - ((2) - 0.5) \times ((0.9375 \text{ in}) + (0.0625 \text{ in}))]$$

$$A_{nt} = 7.5 \text{ in}^2$$

Calculate the gross area of the flange plate subject to shear.

$t_{fp} = 1$ in - Flange Plate Thickness

$L_{ev,fp} = 2$ in - Flange Plate Vertical Edge Distance

$n_c = 3$ - Number of Bolt Columns

$s_c = 3$ in - Bolt Column Spacing

A_{gv} - Gross Area Subject to Shear (L-pattern)

$$A_{gv} = t_{fp}\, [L_{ev,fp} + (n_c - 1)\, s_c]$$

$$A_{gv} = (1 \text{ in}) \times [(2 \text{ in}) + ((3) - 1) \times (3 \text{ in})]$$

$$A_{gv} = 8 \text{ in}^2$$

Calculate the net area of the flange plate subject to shear.

$t_{fp} = 1$ in - Flange Plate Thickness

$L_{ev,fp} = 2$ in - Flange Plate Vertical Edge Distance

$n_c = 3$ - Number of Bolt Columns
$s_c = 3$ in - Bolt Column Spacing
$d_h = 0.9375$ in - Vertical Bolt Hole Dimension at Flange Plate
A_{nv} - Net Area Subject to Shear (L-pattern)

$$A_{nv} = t_{fp}\ (L_{ev,fp} + (n_c - 1)\ s_c - (n_c - 0.5)\ (d_h + 0.0625\ \text{in}))$$

$$A_{nv} = (1\ \text{in}) \times ((2\ \text{in}) + ((3) - 1) \times (3\ \text{in}) - ((3) - 0.5) \times ((0.9375\ \text{in}) + (0.0625\ \text{in})))$$

$$A_{nv} = 5.5\ \text{in}^2$$

Calculate the allowable block shear capacity of the flange plate.
$\Omega = 2$ - Block Shear Safety Factor
$F_{yp} = 50$ ksi - Flange Plate Yield Stress
$F_{up} = 65$ ksi - Flange Plate Tensile Stress
$U_{bs} = 1$ - Uniformity factor
$A_{gv} = 8\ \text{in}^2$ - Gross Area Subject to Shear (L-pattern)
$A_{nv} = 5.5\ \text{in}^2$ - Net Area Subject to Shear (L-pattern)
$A_{nt} = 7.5\ \text{in}^2$ - Net Area Subject to Tension (L-pattern)

AISC 360-16 Chapter J4.3 Eq. (J4-5)

$\frac{R_n}{\Omega}$ - Allowable Block Shear Capacity of Section

$$\frac{R_n}{\Omega} = \frac{0.6\ F_{up}\ A_{nv} + U_{bs}\ F_{up}\ A_{nt} \le 0.6\ F_{yp}\ A_{gv} + U_{bs}\ F_{up}\ A_{nt}}{\Omega}$$

$$\frac{R_n}{\Omega} = \frac{0.6 \times (65\ \text{ksi}) \times \left(5.5\ \text{in}^2\right) + (1) \times (65\ \text{ksi}) \times \left(7.5\ \text{in}^2\right) \le 0.6 \times (50\ \text{ksi}) \times \left(8\ \text{in}^2\right) + (1) \times (65\ \text{ksi}) \times \left(7.5\ \text{in}^2\right)}{(2)}$$

$$\frac{R_n}{\Omega} = 351\ \text{kip}$$

Result:
Demand over Capacity Ratio
$DCR = \frac{P_{flange}}{\frac{R_n}{\Omega}} = \frac{(485.86\ \text{kip})}{(351\ \text{kip})} = 1.3842$

FAIL

Check No. 6: Allowable Block Shear Capacity of the Column Flange

Calculate the net area of the column flange subject to tension.
$t_{f,lc} = 0.71$ in - Lower Column Flange Thickness
$b_{f,lc} = 14.5$ in - Lower Column Flange Width
$g_a = 7$ in - Bolt Gage
$d_h = 0.8125$ in - Horizontal Bolt Hole Dimension at Column Flange
A_{nt} - Net Area Subject to Tension (2L-pattern)

$$A_{nt} = t_{f,lc}\ [b_{f,lc} - g_a - (d_h + 0.0625\ \text{in})]$$

$$A_{nt} = (0.71\ \text{in}) \times [(14.5\ \text{in}) - (7\ \text{in}) - ((0.8125\ \text{in}) + (0.0625\ \text{in}))]$$

$$A_{nt} = 4.7037\ \text{in}^2$$

Calculate the gross area of the column flange subject to shear.
$t_{f,lc} = 0.71$ in - Lower Column Flange Thickness
$L_{ev,c} = 2$ in - Column Flange Vertical Edge Distance
$n_c = 3$ - Number of Bolt Columns
$s_c = 3$ in - Bolt Column Spacing
A_{gv} - Gross Area Subject to Shear (2L-pattern)

$$A_{gv} = 2\ t_{f,lc}\ [L_{ev,c} + (n_c - 1)\ s_c]$$

$$A_{gv} = 2 \times (0.71\ \text{in}) \times [(2\ \text{in}) + ((3) - 1) \times (3\ \text{in})]$$

$$A_{gv} = 11.36\ \text{in}^2$$

Calculate the net area of the column flange subject to shear.
$t_{f,lc} = 0.71$ in - Lower Column Flange Thickness
$L_{ev,c} = 2$ in - Column Flange Vertical Edge Distance
$n_c = 3$ - Number of Bolt Columns
$s_c = 3$ in - Bolt Column Spacing
$d_h = 0.8125$ in - Vertical Bolt Hole Dimension at Column Flange
A_{nv} - Net Area Subject to Shear (2L-pattern)

$$A_{nv} = 2\ t_{f,lc}\ (L_{ev,c} + (n_c - 1)\ s_c - (n_c - 0.5)\ (d_h + 0.0625\ \text{in}))$$

$$A_{nv} = 2 \times (0.71\ \text{in}) \times ((2\ \text{in}) + ((3) - 1) \times (3\ \text{in}) - ((3) - 0.5) \times ((0.8125\ \text{in}) + (0.0625\ \text{in})))$$

$$A_{nv} = 8.2538\ \text{in}^2$$

Calculate the allowable block shear capacity of the column flange.
$\Omega = 2$ - Block Shear Safety Factor
$F_{y,lc} = 50$ ksi - Lower Column Yield Stress
$F_{u,lc} = 65$ ksi - Lower Column Tensile Stress
$U_{bs} = 1$ - Uniformity factor
$A_{gv} = 11.36\ \text{in}^2$ - Gross Area Subject to Shear (2L-pattern)
$A_{nv} = 8.2538\ \text{in}^2$ - Net Area Subject to Shear (2L-pattern)
$A_{nt} = 4.7037\ \text{in}^2$ - Net Area Subject to Tension (2L-pattern)

AISC 360-16 Chapter J4.3 Eq. (J4-5)

$\frac{R_n}{\Omega}$ - Allowable Block Shear Capacity of Section

$$\frac{R_n}{\Omega} = \frac{0.6\,F_{u,lc}\,A_{nv} + U_{bs}\,F_{u,lc}\,A_{nt} \leq 0.6\,F_{y,lc}\,A_{gv} + U_{bs}\,F_{u,lc}\,A_{nt}}{\Omega}$$

$$\frac{R_n}{\Omega} = \frac{0.6 \times (65\text{ ksi}) \times \left(8.2538\text{ in}^2\right) + (1) \times (65\text{ ksi}) \times \left(4.7037\text{ in}^2\right) \leq 0.6 \times (50\text{ ksi}) \times \left(11.36\text{ in}^2\right) + (1) \times (65\text{ ksi}) \times \left(4.7037\text{ in}^2\right)}{(2)}$$

$$\frac{R_n}{\Omega} = 313.82\text{ kip}$$

Result:
Demand over Capacity Ratio

$DCR = \frac{P_{flange}}{\frac{R_n}{\Omega}} = \frac{(485.86\text{ kip})}{(313.82\text{ kip})} = 1.5482$

FAIL

Check No. 7: Allowable Capacity of the Flange Plate in Tension
Calculate the tensile yielding capacity of the flange plate.

$\Omega = 1.67$ - Tensile Yielding Safety Factor

$F_{yp} = 50\text{ ksi}$ - Flange Plate Yield Stress

$t_{fp} = 1\text{ in}$ - Flange Plate Thickness

$b_{fp} = 11\text{ in}$ - Flange Plate Width

AISC 360-16 Chapter J4.1 Eq. (J4-1)

$\frac{R_{n\_ty}}{\Omega}$ - Allowable Tension Yielding Capacity of Section

$$\frac{R_{n\_ty}}{\Omega} = \frac{F_{yp}\,t_{fp}\,b_{fp}}{\Omega}$$

$$\frac{R_{n\_ty}}{\Omega} = \frac{(50\text{ ksi}) \times (1\text{ in}) \times (11\text{ in})}{(1.67)}$$

$$\frac{R_{n\_ty}}{\Omega} = 329.34\text{ kip}$$

Calculate the tensile rupture capacity of the flange plate.

$\Omega = 2$ - Tensile Rupture Safety Factor

$F_{up} = 65\text{ ksi}$ - Flange Plate Tensile Stress

$t_{fp} = 1\text{ in}$ - Flange Plate Thickness

$b_{fp} = 11\text{ in}$ - Flange Plate Width

$n_r = 2$ - Number of Bolt Rows

$d_h = 0.9375\text{ in}$ - Horizontal Bolt Hole Dimension at Flange Plate

AISC 360-16 Chapter J4.1 Eq. (J4-2)

$\frac{R_{n\_tr}}{\Omega}$ - Allowable Tension Rupture Capacity of Section

$$\frac{R_{n\_tr}}{\Omega} = \frac{F_{up}\,t_{fp}\,[b_{fp} - n_r\,(d_h + 0.0625\text{ in})]}{\Omega}$$

$$\frac{R_{n\_tr}}{\Omega} = \frac{(65\text{ ksi}) \times (1\text{ in}) \times [(11\text{ in}) - (2) \times ((0.9375\text{ in}) + (0.0625\text{ in}))]}{(2)}$$

$$\frac{R_{n\_tr}}{\Omega} = 292.5\text{ kip}$$

Determine the governing tensile capacity of the flange plate.

AISC 360-16 Chapter J4.1 Eq. (J4-1)

$\frac{R_{n\_ty}}{\Omega} = 329.34\text{ kip}$ - Allowable Tension Yielding Capacity of Section

AISC 360-16 Chapter J4.1 Eq. (J4-2)

$\frac{R_{n\_tr}}{\Omega} = 292.5\text{ kip}$ - Allowable Tension Rupture Capacity of Section

AISC 360-16 Chapter J4.1

$\frac{R_n}{\Omega}$ - Governing Allowable Capacity

$$\frac{R_n}{\Omega} = min\left(\frac{R_{n\_ty}}{\Omega}, \frac{R_{n\_tr}}{\Omega}\right)$$

$$\frac{R_n}{\Omega} = min\left((329.34\text{ kip}), (292.5\text{ kip})\right)$$

$$\frac{R_n}{\Omega} = 292.5\text{ kip}$$

Result:
Demand over Capacity Ratio

$DCR = \frac{T_{flange}}{\frac{R_n}{\Omega}} = \frac{(485.86\text{ kip})}{(292.5\text{ kip})} = 1.661$

FAIL

Check No. 8: Allowable Capacity of the Column in Tension
Calculate the tensile rupture capacity of the column.

$\Omega = 2$ - Tensile Rupture Safety Factor

$F_{u,lc} = 65\text{ ksi}$ - Lower Column Tensile Stress

$A_{lc} = 26.5\text{ in}^2$ - Lower Column Area

$t_{f,lc} = 0.71\text{ in}$ - Lower Column Flange Thickness

$d_h = 0.8125\text{ in}$ - Horizontal Bolt Hole Dimension at Column Flange

$n_r = 2$ - Number of Bolt Rows

$n_c = 3$ - Number of Bolt Columns

$s_c = 3\text{ in}$ - Bolt Column Spacing

$\bar{y} = 1.1356\text{ in}$ - Centroid of WT section

AISC 360-16 Chapter D3 (Table D3.1 case 2)

$U = 0.81081$ - Shear Lag Factor

AISC 360-16 Chapter J4.1 Eq. (J4-2)

$\frac{R_{n\_tr}}{\Omega}$ - Allowable Tension Rupture Capacity of Section

$$\frac{R_{n\_tr}}{\Omega} = \frac{F_{u,lc}\,U\,[0.5\,A_{lc} - n_r\,(d_h + 0.0625\text{ in})\,t_{f,lc}]}{\Omega}$$

	$$\frac{R_{n\_tr}}{\Omega} = \frac{(65\ \text{ksi}) \times (0.81081) \times \left[0.5 \times \left(26.5\ \text{in}^2\right) - (2) \times ((0.8125\ \text{in}) + (0.0625\ \text{in})) \times (0.71\ \text{in})\right]}{(2)}$$ $$\frac{R_{n\_tr}}{\Omega} = 316.42\ \text{kip}$$ **Result:** Demand over Capacity Ratio $DCR = \frac{T_{flange}}{\frac{R_{n\_tr}}{\Omega}} = \frac{(485.86\ \text{kip})}{(316.42\ \text{kip})} = 1.5355$	FAIL
AISC 360-16 Chapter J4.4 Eq. (J4-6)	**Check No. 9: Allowable Capacity of the Flange Plate in Compression** **Calculate the compression buckling capacity of the flange plate.** $\Omega = 1.67$ - Compression Safety Factor $F_{yp} = 50\ \text{ksi}$ - Flange Plate Yield Stress $E = 29000\ \text{ksi}$ - Modulus for Steel $t_{fp} = 1\ \text{in}$ - Flange Plate Thickness $b_{fp} = 11\ \text{in}$ - Flange Plate Width $K = 1.2$ - Effective Length Factor $L_b = 2\ \text{in}$ - Flange Plate Unbraced Length $\frac{KL}{r} = 8.3138$ - Effective Length Slenderness Ratio Since, $\frac{KL}{r} \leq 25.$ $\frac{R_n}{\Omega}$ - Allowable Compressive Capacity of Section $$\frac{R_n}{\Omega} = \frac{F_{yp}\, t_{fp}\, b_{fp}}{\Omega}$$ $$\frac{R_n}{\Omega} = \frac{(50\ \text{ksi}) \times (1\ \text{in}) \times (11\ \text{in})}{(1.67)}$$ $$\frac{R_n}{\Omega} = 329.34\ \text{kip}$$ **Result:** Demand over Capacity Ratio $DCR = \frac{C_{flange}}{\frac{R_n}{\Omega}} = \frac{(218.6\ \text{kip})}{(329.34\ \text{kip})} = 0.66375$	PASS

REFERENCES	CALCULATIONS	RESULTS

Summary of Checks

Design Checks	Demand	Capacity	DCR	Result
Connection Detailing Limitations	2.000	3.000	0.667	PASS
Allowable Capacity of the Bolts in Shear	485.856	37.968	12.796	FAIL
Allowable Bolt Bearing Capacity of the Flange Plate	485.856	351.000	1.384	FAIL
Allowable Bolt Bearing Capacity of the Column Flange	485.856	249.210	1.950	FAIL
Allowable Block Shear Capacity of the Flange Plate	485.856	351.000	1.384	FAIL
Allowable Block Shear Capacity of the Column Flange	485.856	313.820	1.548	FAIL
Allowable Capacity of the Flange Plate in Tension	485.856	292.500	1.661	FAIL
Allowable Capacity of the Column in Tension	485.856	316.415	1.535	FAIL
Allowable Capacity of the Flange Plate in Compression	218.600	329.341	0.664	PASS

REFERENCES	CALCULATIONS	RESULTS

Flange Plate Connection at Upper Column | AISC 360-16 ASD

Flange Plate Geometry:

$b_{fp} = 11\ \mathrm{in}$ - Flange Plate Width

$L_{fp} = 20\ \mathrm{in}$ - Flange Plate Length

$t_{fp} = 1\ \mathrm{in}$ - Flange Plate Thickness

Flange Plate Material Grade:

$F_{yp} = 50\ \mathrm{ksi}$ - Flange Plate Yield Stress

$F_{up} = 65\ \mathrm{ksi}$ - Flange Plate Tensile Stress

Connection Information at Flange Plate:

$n_r = 2$ - Number of Bolt Rows

$g_a = 7\ \mathrm{in}$ - Bolt Gage

$n_c = 3$ - Number of Bolt Columns

$s_c = 3\ \mathrm{in}$ - Bolt Column Spacing

Distances:

$L_{ev,fp} = 2\ \mathrm{in}$ - Flange Plate Vertical Edge Distance

$L_{ev,c} = 2\ \mathrm{in}$ - Column Flange Vertical Edge Distance

$L_{eh,fp} = 2\ \mathrm{in}$ - Flange Plate Horizontal Edge Distance

$L_{eh,c} = 3.75\ \mathrm{in}$ - Column Flange Horizontal Edge Distance

Check No. 1: Connection Detailing Limitations

Detailing Limitations	Limit Value (in)	Actual Value (in)	DCR	Result
Minimum Bolt Gage	4.375	7.000	0.625	PASS
Minimum Bolt Column Spacing	2.000	3.000	0.667	PASS
Maximum Bolt Column Spacing	6.000	3.000	0.500	PASS
Flange Plate Minimum Vertical Edge Distance	1.063	2.000	0.531	PASS
Flange Plate Minimum Horizontal Edge Distance	1.063	2.000	0.531	PASS
Column Flange Minimum Vertical Edge Distance	1.000	2.000	0.500	PASS
Column Flange Minimum Horizontal Edge Distance	1.000	3.750	0.267	PASS

Result:

Demand over Capacity Ratio

$DCR = \frac{d}{c} = \frac{(2)}{(3)} = 0.66667$

PASS

Check No. 2: Allowable Capacity of the Bolts in Shear

$\Omega = 1.5$ - Bolt Shear Safety Factor

$\mu = 0.3$ - Mean Slip Coefficient

$D_u = 1.13$

$h_f = 1$ - Filler Factor for SC Bolts

$d_b = 0.75\ \mathrm{in}$ - Bolt Diameter

$T_b = 28\ \mathrm{kip}$ - Minimum Bolt Pretension

$N_s = 1$ - Number of Slip Planes

$n_r = 2$ - Number of Bolt Rows

$n_c = 3$ - Number of Bolt Columns

AISC 360-16 Chapter J3.8 Eq. (J3-4)

$\frac{R_n}{\Omega}$ - Allowable Bolt Shear Capacity

$$\frac{R_n}{\Omega} = \frac{\mu\, D_u\, h_f\, T_b\, N_s\, n_r\, n_c}{\Omega}$$

$$\frac{R_n}{\Omega} = \frac{(0.3) \times (1.13) \times (1) \times (28\ \mathrm{kip}) \times (1) \times (2) \times (3)}{(1.5)}$$

$$\frac{R_n}{\Omega} = 37.968\ \mathrm{kip}$$

Result:

Demand over Capacity Ratio

$DCR = \frac{P_{flange}}{\frac{R_n}{\Omega}} = \frac{(485.86\ \mathrm{kip})}{(37.968\ \mathrm{kip})} = 12.796$

FAIL

Check No. 3: Allowable Bolt Bearing Capacity of the Flange Plate

Calculate the bolt bearing capacity of the flange plate.

$\Omega = 2$ - Bolt Bearing Safety Factor

$d_b = 0.75\ \mathrm{in}$ - Bolt Diameter

$t_{fp} = 1\ \mathrm{in}$ - Flange Plate Thickness

$F_{up} = 65\ \mathrm{ksi}$ - Flange Plate Tensile Stress

$n_c = 3$ - Number of Bolt Columns

$n_r = 2$ - Number of Bolt Rows

AISC 360-16 Chapter J3.10 Eq. (J3-6a)

$\frac{R_{n\_bearing}}{\Omega}$ - Allowable Bolt Bearing Capacity of Section

$$\frac{R_{n\_bearing}}{\Omega} = \frac{2.4\, d_b\, t_{fp}\, F_{up}\, n_c\, n_r}{\Omega}$$

$$\frac{R_{n\_bearing}}{\Omega} = \frac{2.4 \times (0.75\ \mathrm{in}) \times (1\ \mathrm{in}) \times (65\ \mathrm{ksi}) \times (3) \times (2)}{(2)}$$

$$\frac{R_{n\_bearing}}{\Omega} = 351\ \mathrm{kip}$$

Calculate the clear distance of outer bolts on flange plate.

$L_{ev,fp} = 2\ \mathrm{in}$ - Flange Plate Vertical Edge Distance

$d_h = 0.9375\ \mathrm{in}$ - Vertical Bolt Hole Dimension at Flange Plate

l_{c1} - Clear Distance at First Bolt Row

$$l_{c1} = L_{ev,fp} - \frac{d_h}{2}$$

$$l_{c1} = (2\text{ in}) - \frac{(0.9375\text{ in})}{2}$$

$$l_{c1} = 1.5313\text{ in}$$

Calculate the clear distance of inner bolts on flange plate.
$s_c = 3\text{ in}$ - Bolt Column Spacing
$d_h = 0.9375\text{ in}$ - Vertical Bolt Hole Dimension at Flange Plate
l_{c2} - Clear Distance at Rest of Bolts

$$l_{c2} = s_c - d_h$$

$$l_{c2} = (3\text{ in}) - (0.9375\text{ in})$$

$$l_{c2} = 2.0625\text{ in}$$

Calculate the bolt tear-out capacity of the flange plate.
$l_{c1} = 1.5313\text{ in}$ - Clear Distance at First Bolt Row
$l_{c2} = 2.0625\text{ in}$ - Clear Distance at Rest of Bolts
$t_{fp} = 1\text{ in}$ - Flange Plate Thickness
$F_{up} = 65\text{ ksi}$ - Flange Plate Tensile Stress
$n_c = 3$ - Number of Bolt Columns
$n_r = 2$ - Number of Bolt Rows
$\Omega = 2$ - Bolt Bearing Safety Factor

AISC 360-16 Chapter J3.10 Eq. (J3-6c)
$\frac{R_{n\_tearout}}{\Omega}$ - Allowable Bolt Tear-out Capacity of Section

$$\frac{R_{n\_tearout}}{\Omega} = \frac{1.2\, l_{c1}\, t_{fp}\, F_{up}\, n_r + 1.2\, l_{c2}\, t_{fp}\, F_{up}\, n_r\, (n_c - 1)}{\Omega}$$

$$\frac{R_{n\_tearout}}{\Omega} = \frac{1.2 \times (1.5313\text{ in}) \times (1\text{ in}) \times (65\text{ ksi}) \times (2) + 1.2 \times (2.0625\text{ in}) \times (1\text{ in}) \times (65\text{ ksi}) \times (2) \times ((3) - 1)}{(2)}$$

$$\frac{R_{n\_tearout}}{\Omega} = 441.19\text{ kip}$$

Determine the governing bearing and tear-out capacity of the bolt group on flange plate.

AISC 360-16 Chapter J3.10 Eq. (J3-6a)
$\frac{R_{n\_bearing}}{\Omega} = 351\text{ kip}$ - Allowable Bolt Bearing Capacity of Section

AISC 360-16 Chapter J3.10 Eq. (J3-6c)
$\frac{R_{n\_tearout}}{\Omega} = 441.19\text{ kip}$ - Allowable Bolt Tear-out Capacity of Section

AISC 360-16 Chapter J3.10
$\frac{R_n}{\Omega}$ - Governing Allowable Capacity

$$\frac{R_n}{\Omega} = min\left(\frac{R_{n\_bearing}}{\Omega}, \frac{R_{n\_tearout}}{\Omega}\right)$$

$$\frac{R_n}{\Omega} = min\left((351\text{ kip}), (441.19\text{ kip})\right)$$

$$\frac{R_n}{\Omega} = 351\text{ kip}$$

Result:
Demand over Capacity Ratio
$DCR = \frac{P_{flange}}{\frac{R_n}{\Omega}} = \frac{(485.86\text{ kip})}{(351\text{ kip})} = 1.3842$

FAIL

Check No. 4: Allowable Bolt Bearing Capacity of the Column Flange

Calculate the bolt bearing capacity of the column flange.
$\Omega = 2$ - Bolt Bearing Safety Factor
$d_b = 0.75\text{ in}$ - Bolt Diameter
$t_{f,uc} = 0.71\text{ in}$ - Upper Column Flange Thickness
$F_{u,uc} = 65\text{ ksi}$ - Upper Column Tensile Stress
$n_c = 3$ - Number of Bolt Columns
$n_r = 2$ - Number of Bolt Rows

AISC 360-16 Chapter J3.10 Eq. (J3-6a)
$\frac{R_{n\_bearing}}{\Omega}$ - Allowable Bolt Bearing Capacity of Section

$$\frac{R_{n\_bearing}}{\Omega} = \frac{2.4\, d_b\, t_{f,uc}\, F_{u,uc}\, n_c\, n_r}{\Omega}$$

$$\frac{R_{n\_bearing}}{\Omega} = \frac{2.4 \times (0.75\text{ in}) \times (0.71\text{ in}) \times (65\text{ ksi}) \times (3) \times (2)}{(2)}$$

$$\frac{R_{n\_bearing}}{\Omega} = 249.21\text{ kip}$$

Calculate the clear distance of outer bolts on column flange.
$L_{ev,c} = 2\text{ in}$ - Column Flange Vertical Edge Distance
$d_h = 0.8125\text{ in}$ - Vertical Bolt Hole Dimension at Column Flange
l_{c1} - Clear Distance at First Bolt Row

$$l_{c1} = L_{ev,c} - \frac{d_h}{2}$$

$$l_{c1} = (2\text{ in}) - \frac{(0.8125\text{ in})}{2}$$

$$l_{c1} = 1.5938\text{ in}$$

Calculate the clear distance of inner bolts on column flange.

$s_c = 3\text{ in}$ - Bolt Column Spacing

$d_h = 0.8125\text{ in}$ - Vertical Bolt Hole Dimension at Column Flange

l_{c2} - Clear Distance at Rest of Bolts

$$l_{c2} = s_c - d_h$$

$$l_{c2} = (3\text{ in}) - (0.8125\text{ in})$$

$$l_{c2} = 2.1875\text{ in}$$

Calculate the bolt tear-out capacity of the column flange.

$l_{c1} = 1.5938\text{ in}$ - Clear Distance at First Bolt Row

$l_{c2} = 2.1875\text{ in}$ - Clear Distance at Rest of Bolts

$t_{f,uc} = 0.71\text{ in}$ - Upper Column Flange Thickness

$F_{u,uc} = 65\text{ ksi}$ - Upper Column Tensile Stress

$n_c = 3$ - Number of Bolt Columns

$n_r = 2$ - Number of Bolt Rows

$\Omega = 2$ - Bolt Bearing Safety Factor

AISC 360-16 Chapter J3.10 Eq. (J3-6c)

$\frac{R_{n\_tearout}}{\Omega}$ - Allowable Bolt Tear-out Capacity of Section

$$\frac{R_{n\_tearout}}{\Omega} = \frac{1.2\, l_{c1}\, t_{f,uc}\, F_{u,uc}\, n_r + 1.2\, l_{c2}\, t_{f,uc}\, F_{u,uc}\, n_r\, (n_c - 1)}{\Omega}$$

$$\frac{R_{n\_tearout}}{\Omega} = \frac{1.2 \times (1.5938\text{ in}) \times (0.71\text{ in}) \times (65\text{ ksi}) \times (2) + 1.2 \times (2.1875\text{ in}) \times (0.71\text{ in}) \times (65\text{ ksi}) \times (2) \times ((3) - 1)}{(2)}$$

$$\frac{R_{n\_tearout}}{\Omega} = 330.55\text{ kip}$$

Determine the governing bearing and tear-out capacity of the bolt group on column flange.

AISC 360-16 Chapter J3.10 Eq. (J3-6a)

$\frac{R_{n\_bearing}}{\Omega} = 249.21\text{ kip}$ - Allowable Bolt Bearing Capacity of Section

AISC 360-16 Chapter J3.10 Eq. (J3-6c)

$\frac{R_{n\_tearout}}{\Omega} = 330.55\text{ kip}$ - Allowable Bolt Tear-out Capacity of Section

AISC 360-16 Chapter J3.10

$\frac{R_n}{\Omega}$ - Governing Allowable Capacity

$$\frac{R_n}{\Omega} = min\left(\frac{R_{n\_bearing}}{\Omega}, \frac{R_{n\_tearout}}{\Omega}\right)$$

$$\frac{R_n}{\Omega} = min\left((249.21\text{ kip}), (330.55\text{ kip})\right)$$

$$\frac{R_n}{\Omega} = 249.21\text{ kip}$$

Result:

Demand over Capacity Ratio

$DCR = \frac{P_{flange}}{\frac{R_n}{\Omega}} = \frac{(485.86\text{ kip})}{(249.21\text{ kip})} = 1.9496$

FAIL

Check No. 5: Allowable Block Shear Capacity of the Flange Plate

Calculate the net area of the flange plate subject to tension.

$t_{fp} = 1\text{ in}$ - Flange Plate Thickness

$n_r = 2$ - Number of Bolt Rows

$s_r = 0\text{ in}$ - Bolt Row Spacing

$g_a = 7\text{ in}$ - Bolt Gage

$L_{eh,fp} = 2\text{ in}$ - Flange Plate Horizontal Edge Distance

$d_h = 0.9375\text{ in}$ - Horizontal Bolt Hole Dimension at Flange Plate

A_{nt} - Net Area Subject to Tension (L-pattern)

$$A_{nt} = t_{fp}\, [(n_r - 2)\ s_r + g_a + L_{eh,fp} - (n_r - 0.5)\ (d_h + 0.0625\text{ in})]$$

$$A_{nt} = (1\text{ in}) \times [((2) - 2) \times (0\text{ in}) + (7\text{ in}) + (2\text{ in}) - ((2) - 0.5) \times ((0.9375\text{ in}) + (0.0625\text{ in}))]$$

$$A_{nt} = 7.5\text{ in}^2$$

Calculate the gross area of the flange plate subject to shear.

$t_{fp} = 1\text{ in}$ - Flange Plate Thickness

$L_{ev,fp} = 2\text{ in}$ - Flange Plate Vertical Edge Distance

$n_c = 3$ - Number of Bolt Columns

$s_c = 3\text{ in}$ - Bolt Column Spacing

A_{gv} - Gross Area Subject to Shear (L-pattern)

$$A_{gv} = t_{fp}\, [L_{ev,fp} + (n_c - 1)\ s_c]$$

$$A_{gv} = (1\text{ in}) \times [(2\text{ in}) + ((3) - 1) \times (3\text{ in})]$$

$$A_{gv} = 8\text{ in}^2$$

Calculate the net area of the flange plate subject to shear.

$t_{fp} = 1\text{ in}$ - Flange Plate Thickness

$L_{ev,fp} = 2\text{ in}$ - Flange Plate Vertical Edge Distance

	$n_c = 3$ - Number of Bolt Columns $s_c = 3\text{ in}$ - Bolt Column Spacing $d_h = 0.9375\text{ in}$ - Vertical Bolt Hole Dimension at Flange Plate A_{nv} - Net Area Subject to Shear (L-pattern) $$A_{nv} = t_{fp}\ (L_{ev,fp} + (n_c - 1)\ s_c - (n_c - 0.5)\ (d_h + 0.0625\text{ in}))$$ $$A_{nv} = (1\text{ in}) \times ((2\text{ in}) + ((3) - 1) \times (3\text{ in}) - ((3) - 0.5) \times ((0.9375\text{ in}) + (0.0625\text{ in})))$$ $$A_{nv} = 5.5\text{ in}^2$$ **Calculate the allowable block shear capacity of the flange plate.** $\Omega = 2$ - Block Shear Safety Factor $F_{yp} = 50\text{ ksi}$ - Flange Plate Yield Stress $F_{up} = 65\text{ ksi}$ - Flange Plate Tensile Stress $U_{bs} = 1$ - Uniformity factor $A_{gv} = 8\text{ in}^2$ - Gross Area Subject to Shear (L-pattern) $A_{nv} = 5.5\text{ in}^2$ - Net Area Subject to Shear (L-pattern) $A_{nt} = 7.5\text{ in}^2$ - Net Area Subject to Tension (L-pattern)	
AISC 360-16 Chapter J4.3 Eq. (J4-5)	$\frac{R_n}{\Omega}$ - Allowable Block Shear Capacity of Section $$\frac{R_n}{\Omega} = \frac{0.6\ F_{up}\ A_{nv} + U_{bs}\ F_{up}\ A_{nt} \leq 0.6\ F_{yp}\ A_{gv} + U_{bs}\ F_{up}\ A_{nt}}{\Omega}$$ $$\frac{R_n}{\Omega} = \frac{0.6 \times (65\text{ ksi}) \times \left(5.5\text{ in}^2\right) + (1) \times (65\text{ ksi}) \times \left(7.5\text{ in}^2\right) \leq 0.6 \times (50\text{ ksi}) \times \left(8\text{ in}^2\right) + (1) \times (65\text{ ksi}) \times \left(7.5\text{ in}^2\right)}{(2)}$$ $$\frac{R_n}{\Omega} = 351\text{ kip}$$ **Result:** Demand over Capacity Ratio $DCR = \frac{P_{flange}}{\frac{R_n}{\Omega}} = \frac{(485.86\text{ kip})}{(351\text{ kip})} = 1.3842$	**FAIL**
	Check No. 6: Allowable Block Shear Capacity of the Column Flange **Calculate the net area of the column flange subject to tension.** $t_{f,uc} = 0.71\text{ in}$ - Upper Column Flange Thickness $b_{f,uc} = 14.5\text{ in}$ - Upper Column Flange Width $g_a = 7\text{ in}$ - Bolt Gage $d_h = 0.8125\text{ in}$ - Horizontal Bolt Hole Dimension at Column Flange A_{nt} - Net Area Subject to Tension (2L-pattern) $$A_{nt} = t_{f,uc}\ [b_{f,uc} - g_a - (d_h + 0.0625\text{ in})]$$ $$A_{nt} = (0.71\text{ in}) \times [(14.5\text{ in}) - (7\text{ in}) - ((0.8125\text{ in}) + (0.0625\text{ in}))]$$ $$A_{nt} = 4.7037\text{ in}^2$$ **Calculate the gross area of the column flange subject to shear.** $t_{f,uc} = 0.71\text{ in}$ - Upper Column Flange Thickness $L_{ev,c} = 2\text{ in}$ - Column Flange Vertical Edge Distance $n_c = 3$ - Number of Bolt Columns $s_c = 3\text{ in}$ - Bolt Column Spacing A_{gv} - Gross Area Subject to Shear (2L-pattern) $$A_{gv} = 2\ t_{f,uc}\ [L_{ev,c} + (n_c - 1)\ s_c]$$ $$A_{gv} = 2 \times (0.71\text{ in}) \times [(2\text{ in}) + ((3) - 1) \times (3\text{ in})]$$ $$A_{gv} = 11.36\text{ in}^2$$ **Calculate the net area of the column flange subject to shear.** $t_{f,uc} = 0.71\text{ in}$ - Upper Column Flange Thickness $L_{ev,c} = 2\text{ in}$ - Column Flange Vertical Edge Distance $n_c = 3$ - Number of Bolt Columns $s_c = 3\text{ in}$ - Bolt Column Spacing $d_h = 0.8125\text{ in}$ - Vertical Bolt Hole Dimension at Column Flange A_{nv} - Net Area Subject to Shear (2L-pattern) $$A_{nv} = 2\ t_{f,uc}\ (L_{ev,c} + (n_c - 1)\ s_c - (n_c - 0.5)\ (d_h + 0.0625\text{ in}))$$ $$A_{nv} = 2 \times (0.71\text{ in}) \times ((2\text{ in}) + ((3) - 1) \times (3\text{ in}) - ((3) - 0.5) \times ((0.8125\text{ in}) + (0.0625\text{ in})))$$ $$A_{nv} = 8.2538\text{ in}^2$$ **Calculate the allowable block shear capacity of the column flange.** $\Omega = 2$ - Block Shear Safety Factor $F_{y,uc} = 50\text{ ksi}$ - Upper Column Yield Stress $F_{u,uc} = 65\text{ ksi}$ - Upper Column Tensile Stress $U_{bs} = 1$ - Uniformity factor $A_{gv} = 11.36\text{ in}^2$ - Gross Area Subject to Shear (2L-pattern) $A_{nv} = 8.2538\text{ in}^2$ - Net Area Subject to Shear (2L-pattern) $A_{nt} = 4.7037\text{ in}^2$ - Net Area Subject to Tension (2L-pattern)	
AISC 360-16 Chapter J4.3 Eq. (J4-5)	$\frac{R_n}{\Omega}$ - Allowable Block Shear Capacity of Section	

	$\frac{R_n}{\Omega} = \frac{0.6\,F_{u,uc}\,A_{nv} + U_{bs}\,F_{u,uc}\,A_{nt} \leq 0.6\,F_{y,uc}\,A_{gv} + U_{bs}\,F_{u,uc}\,A_{nt}}{\Omega}$ $\frac{R_n}{\Omega} = \frac{0.6 \times (65\text{ ksi}) \times \left(8.2538\text{ in}^2\right) + (1) \times (65\text{ ksi}) \times \left(4.7037\text{ in}^2\right) \leq 0.6 \times (50\text{ ksi}) \times \left(11.36\text{ in}^2\right) + (1) \times (65\text{ ksi}) \times \left(4.7037\text{ in}^2\right)}{(2)}$ $\frac{R_n}{\Omega} = 313.82\text{ kip}$ **Result:** Demand over Capacity Ratio $DCR = \frac{P_{flange}}{\frac{R_n}{\Omega}} = \frac{(485.86\text{ kip})}{(313.82\text{ kip})} = 1.5482$	FAIL
AISC 360-16 Chapter J4.1 Eq. (J4-1) AISC 360-16 Chapter J4.1 Eq. (J4-2) AISC 360-16 Chapter J4.1 Eq. (J4-1) AISC 360-16 Chapter J4.1 Eq. (J4-2) AISC 360-16 Chapter J4.1	**Check No. 7: Allowable Capacity of the Flange Plate in Tension** **Calculate the tensile yielding capacity of the flange plate.** $\Omega = 1.67$ - Tensile Yielding Safety Factor $F_{yp} = 50\text{ ksi}$ - Flange Plate Yield Stress $t_{fp} = 1\text{ in}$ - Flange Plate Thickness $b_{fp} = 11\text{ in}$ - Flange Plate Width $\frac{R_{n\_ty}}{\Omega}$ - Allowable Tension Yielding Capacity of Section $\frac{R_{n\_ty}}{\Omega} = \frac{F_{yp}\,t_{fp}\,b_{fp}}{\Omega}$ $\frac{R_{n\_ty}}{\Omega} = \frac{(50\text{ ksi}) \times (1\text{ in}) \times (11\text{ in})}{(1.67)}$ $\frac{R_{n\_ty}}{\Omega} = 329.34\text{ kip}$ **Calculate the tensile rupture capacity of the flange plate.** $\Omega = 2$ - Tensile Rupture Safety Factor $F_{up} = 65\text{ ksi}$ - Flange Plate Tensile Stress $t_{fp} = 1\text{ in}$ - Flange Plate Thickness $b_{fp} = 11\text{ in}$ - Flange Plate Width $n_r = 2$ - Number of Bolt Rows $d_h = 0.9375\text{ in}$ - Horizontal Bolt Hole Dimension at Flange Plate $\frac{R_{n\_tr}}{\Omega}$ - Allowable Tension Rupture Capacity of Section $\frac{R_{n\_tr}}{\Omega} = \frac{F_{up}\,t_{fp}\,[b_{fp} - n_r\,(d_h + 0.0625\text{ in})]}{\Omega}$ $\frac{R_{n\_tr}}{\Omega} = \frac{(65\text{ ksi}) \times (1\text{ in}) \times [(11\text{ in}) - (2) \times ((0.9375\text{ in}) + (0.0625\text{ in}))]}{(2)}$ $\frac{R_{n\_tr}}{\Omega} = 292.5\text{ kip}$ **Determine the governing tensile capacity of the flange plate.** $\frac{R_{n\_ty}}{\Omega} = 329.34\text{ kip}$ - Allowable Tension Yielding Capacity of Section $\frac{R_{n\_tr}}{\Omega} = 292.5\text{ kip}$ - Allowable Tension Rupture Capacity of Section $\frac{R_n}{\Omega}$ - Governing Allowable Capacity $\frac{R_n}{\Omega} = min\left(\frac{R_{n\_ty}}{\Omega}, \frac{R_{n\_tr}}{\Omega}\right)$ $\frac{R_n}{\Omega} = min\left((329.34\text{ kip}), (292.5\text{ kip})\right)$ $\frac{R_n}{\Omega} = 292.5\text{ kip}$ **Result:** Demand over Capacity Ratio $DCR = \frac{T_{flange}}{\frac{R_n}{\Omega}} = \frac{(485.86\text{ kip})}{(292.5\text{ kip})} = 1.661$	FAIL
AISC 360-16 Chapter D3 (Table D3.1 case 2) AISC 360-16 Chapter J4.1 Eq. (J4-2)	**Check No. 8: Allowable Capacity of the Column in Tension** **Calculate the tensile rupture capacity of the column.** $\Omega = 2$ - Tensile Rupture Safety Factor $F_{u,uc} = 65\text{ ksi}$ - Upper Column Tensile Stress $A_{uc} = 26.5\text{ in}^2$ - Upper Column Area $t_{f,uc} = 0.71\text{ in}$ - Upper Column Flange Thickness $d_h = 0.8125\text{ in}$ - Horizontal Bolt Hole Dimension at Column Flange $n_r = 2$ - Number of Bolt Rows $n_c = 3$ - Number of Bolt Columns $s_c = 3\text{ in}$ - Bolt Column Spacing $\bar{y} = 1.1356\text{ in}$ - Centroid of WT section $U = 0.81081$ - Shear Lag Factor $\frac{R_{n\_tr}}{\Omega}$ - Allowable Tension Rupture Capacity of Section $\frac{R_{n\_tr}}{\Omega} = \frac{F_{u,uc}\,U\,[0.5\,A_{uc} - n_r\,(d_h + 0.0625\text{ in})\,t_{f,uc}]}{\Omega}$	

	$$\frac{R_{n\_tr}}{\Omega} = \frac{(65\text{ ksi}) \times (0.81081) \times \left[0.5 \times \left(26.5\text{ in}^2\right) - (2) \times ((0.8125\text{ in}) + (0.0625\text{ in})) \times (0.71\text{ in})\right]}{(2)}$$ $$\frac{R_{n\_tr}}{\Omega} = 316.42\text{ kip}$$ **Result:** Demand over Capacity Ratio $DCR = \frac{T_{flange}}{\frac{R_{n\_tr}}{\Omega}} = \frac{(485.86\text{ kip})}{(316.42\text{ kip})} = 1.5355$	**FAIL**
AISC 360-16 Chapter J4.4 Eq. (J4-6)	**Check No. 9: Allowable Capacity of the Flange Plate in Compression** **Calculate the compression buckling capacity of the flange plate.** $\Omega = 1.67$ - Compression Safety Factor $F_{yp} = 50\text{ ksi}$ - Flange Plate Yield Stress $E = 29000\text{ ksi}$ - Modulus for Steel $t_{fp} = 1\text{ in}$ - Flange Plate Thickness $b_{fp} = 11\text{ in}$ - Flange Plate Width $K = 1.2$ - Effective Length Factor $L_b = 2\text{ in}$ - Flange Plate Unbraced Length $\frac{KL}{r} = 8.3138$ - Effective Length Slenderness Ratio Since, $\frac{KL}{r} \leq 25.$ $\frac{R_n}{\Omega}$ - Allowable Compressive Capacity of Section $$\frac{R_n}{\Omega} = \frac{F_{yp}\, t_{fp}\, b_{fp}}{\Omega}$$ $$\frac{R_n}{\Omega} = \frac{(50\text{ ksi}) \times (1\text{ in}) \times (11\text{ in})}{(1.67)}$$ $$\frac{R_n}{\Omega} = 329.34\text{ kip}$$ **Result:** Demand over Capacity Ratio $DCR = \frac{C_{flange}}{\frac{R_n}{\Omega}} = \frac{(218.6\text{ kip})}{(329.34\text{ kip})} = 0.66375$	**PASS**

REFERENCES	CALCULATIONS	RESULTS

Summary of Checks

Design Checks	Demand	Capacity	DCR	Result
Connection Detailing Limitations	2.000	3.000	0.667	PASS
Allowable Capacity of the Bolts in Shear	485.856	37.968	12.796	FAIL
Allowable Bolt Bearing Capacity of the Flange Plate	485.856	351.000	1.384	FAIL
Allowable Bolt Bearing Capacity of the Column Flange	485.856	249.210	1.950	FAIL
Allowable Block Shear Capacity of the Flange Plate	485.856	351.000	1.384	FAIL
Allowable Block Shear Capacity of the Column Flange	485.856	313.820	1.548	FAIL
Allowable Capacity of the Flange Plate in Tension	485.856	292.500	1.661	FAIL
Allowable Capacity of the Column in Tension	485.856	316.415	1.535	FAIL
Allowable Capacity of the Flange Plate in Compression	218.600	329.341	0.664	PASS

Web Plate Connection at Lower Column | AISC 360-16 ASD

Web Plate Geometry:

$b_{wp} = 7$ in - Web Plate Width

$L_{wp} = 14$ in - Web Plate Length

$t_{wp} = 1$ in - Web Plate Thickness

Web Plate Material Grade:

$F_{yp} = 50$ ksi - Web Plate Yield Stress

$F_{up} = 65$ ksi - Web Plate Tensile Stress

Connection Information at Web Plate:

$n_r = 2$ - Number of Bolt Rows

$s_r = 3$ in - Bolt Row Spacing

$n_c = 2$ - Number of Bolt Columns

$s_c = 3$ in - Bolt Column Spacing

Distances:

$L_{ev,wp} = 2$ in - Web Plate Vertical Edge Distance

$L_{eh,c} = 2$ in - Column Web Horizontal Edge Distance

$L_{eh,wp} = 2$ in - Web Plate Horizontal Edge Distance

Check No. 1: Connection Detailing Limitations

Detailing Limitations	Limit Value (in)	Actual Value (in)	DCR	Result
Minimum Bolt Row Spacing	2.000	3.000	0.667	PASS
Maximum Bolt Row Spacing	6.000	3.000	0.500	PASS
Minimum Bolt Column Spacing	2.000	3.000	0.667	PASS
Maximum Bolt Column Spacing	6.000	3.000	0.500	PASS
Web Plate Minimum Vertical Edge Distance	1.063	2.000	0.531	PASS
Web Plate Minimum Horizontal Edge Distance	1.063	2.000	0.531	PASS
Minimum Connection Depth	5.000	7.000	0.714	PASS
Maximum Connection Depth	10.000	7.000	0.700	PASS

Result:

Demand over Capacity Ratio

$DCR = \frac{d}{c} = \frac{(5)}{(7)} = 0.71429$

PASS

Check No. 2: Allowable Capacity of the Bolt Group in Shear

$\Omega = 1.5$ - Bolt Shear Safety Factor

$\mu = 0.3$ - Mean Slip Coefficient

$D_u = 1.13$

$h_f = 1$ - Filler Factor for SC Bolts

$d_b = 0.75$ in - Bolt Diameter

$T_b = 28$ kip - Minimum Bolt Pretension

$N_s = 1$ - Number of Slip Planes

$n_r = 2$ - Number of Bolt Rows

$n_c = 2$ - Number of Bolt Columns

AISC 360-16 Chapter J3.8 Eq. (J3-4)

$\frac{R_n}{\Omega}$ - Allowable Bolt Shear Capacity

$$\frac{R_n}{\Omega} = \frac{\mu\, D_u\, h_f\, T_b\, N_s\, n_r\, n_c}{\Omega}$$

$$\frac{R_n}{\Omega} = \frac{(0.3) \times (1.13) \times (1) \times (28 \text{ kip}) \times (1) \times (2) \times (2)}{(1.5)}$$

$$\frac{R_n}{\Omega} = 25.312 \text{ kip}$$

Result:

Demand over Capacity Ratio

$DCR = \frac{R_a}{\frac{R_n}{\Omega}} = \frac{(257.16 \text{ kip})}{(25.312 \text{ kip})} = 10.159$

FAIL

Check No. 3: Allowable Capacity of the Bolt Group in Bearing and Tear-out at Web Plate due to Shear Load

Calculate the bolt bearing capacity of the web plate.

$d_b = 0.75$ in - Bolt Diameter

$t_{wp} = 1$ in - Web Plate Thickness

$F_{up} = 65$ ksi - Web Plate Tensile Stress

$C = 4$ - Bolt Group Coefficient

$\Omega = 2$ - Bolt Bearing Safety Factor

AISC 360-16 Chapter J3.10 Eq. (J3-6a)	$\frac{R_{n\_bearing}}{\Omega}$ - Allowable Bolt Bearing Capacity of Section $$\frac{R_{n\_bearing}}{\Omega} = \frac{2.4\, d_b\, t_{wp}\, F_{up}\, C}{\Omega}$$ $$\frac{R_{n\_bearing}}{\Omega} = \frac{2.4 \times (0.75\ \mathrm{in}) \times (1\ \mathrm{in}) \times (65\ \mathrm{ksi}) \times (4)}{(2)}$$ $$\frac{R_{n\_bearing}}{\Omega} = 234\ \mathrm{kip}$$ **Calculate the clear distance of outer bolts on web plate.** $L_{ev,wp} = 2\ \mathrm{in}$ - Web Plate Vertical Edge Distance $d_h = 0.9375\ \mathrm{in}$ - Vertical Bolt Hole Dimension at Web Plate l_{c1} - Clear Distance at First Bolt Row $$l_{c1} = L_{ev,wp} - \frac{d_h}{2}$$ $$l_{c1} = (2\ \mathrm{in}) - \frac{(0.9375\ \mathrm{in})}{2}$$ $$l_{c1} = 1.5313\ \mathrm{in}$$ **Calculate the clear distance of inner bolts on web plate.** $s_r = 3\ \mathrm{in}$ - Bolt Row Spacing $d_h = 0.9375\ \mathrm{in}$ - Vertical Bolt Hole Dimension at Web Plate l_{c2} - Clear Distance at Rest of Bolts $$l_{c2} = s_r - d_h$$ $$l_{c2} = (3\ \mathrm{in}) - (0.9375\ \mathrm{in})$$ $$l_{c2} = 2.0625\ \mathrm{in}$$ **Calculate the bolt tear-out capacity of the web plate.** $l_{c1} = 1.5313\ \mathrm{in}$ - Clear Distance at First Bolt Row $l_{c2} = 2.0625\ \mathrm{in}$ - Clear Distance at Rest of Bolts $t_{wp} = 1\ \mathrm{in}$ - Web Plate Thickness $F_{up} = 65\ \mathrm{ksi}$ - Web Plate Tensile Stress $C = 4$ - Bolt Group Coefficient $n_r = 2$ - Number of Bolt Rows $\Omega = 2$ - Bolt Bearing Safety Factor		
AISC 360-16 Chapter J3.10 Eq. (J3-6c)	$\frac{R_{n\_tearout}}{\Omega}$ - Allowable Bolt Tear-out Capacity of Section $$\frac{R_{n\_tearout}}{\Omega} = \frac{\left[1.2\, l_{c1}\, t_{wp}\, F_{up} + 1.2\, l_{c2}\, t_{wp}\, F_{up}\, (n_r - 1)\right] \left(\frac{C}{n_r}\right)}{\Omega}$$ $$\frac{R_{n\_tearout}}{\Omega} = \frac{\left[1.2 \times (1.5313\ \mathrm{in}) \times (1\ \mathrm{in}) \times (65\ \mathrm{ksi}) + 1.2 \times (2.0625\ \mathrm{in}) \times (1\ \mathrm{in}) \times (65\ \mathrm{ksi}) \times ((2) - 1)\right] \times \left(\frac{(4)}{(2)}\right)}{(2)}$$ $$\frac{R_{n\_tearout}}{\Omega} = 280.31\ \mathrm{kip}$$ **Determine the governing bearing and tear-out capacity of the bolt group on web plate.**		
AISC 360-16 Chapter J3.10 Eq. (J3-6a)	$\frac{R_{n\_bearing}}{\Omega} = 234\ \mathrm{kip}$ - Allowable bolt bearing capacity of web plate		
AISC 360-16 Chapter J3.10 Eq. (J3-6c)	$\frac{R_{n\_tearout}}{\Omega} = 280.31\ \mathrm{kip}$ - Allowable bolt tear-out capacity of web plate		
AISC 360-16 Chapter J3.10	$\frac{R_n}{\Omega}$ - Governing Allowable Capacity $$\frac{R_n}{\Omega} = min\left(\frac{R_{n\_bearing}}{\Omega}, \frac{R_{n\_tearout}}{\Omega}\right)$$ $$\frac{R_n}{\Omega} = min\left((234\ \mathrm{kip}), (280.31\ \mathrm{kip})\right)$$ $$\frac{R_n}{\Omega} = 234\ \mathrm{kip}$$ **Result:** Demand over Capacity Ratio $DCR = \frac{V_a}{\frac{R_n}{\Omega}} = \frac{(150\ \mathrm{kip})}{(234\ \mathrm{kip})} = 0.64103$		**PASS**
	Check No. 4: Allowable Capacity of the Bolt Group in Bearing and Tear-out at Web Plate due to Axial Load **Calculate the bolt bearing capacity of the web plate.** $d_b = 0.75\ \mathrm{in}$ - Bolt Diameter $t_{wp} = 1\ \mathrm{in}$ - Web Plate Thickness $F_{up} = 65\ \mathrm{ksi}$ - Web Plate Tensile Stress $C = 4$ - Bolt Group Coefficient $\Omega = 2$ - Bolt Bearing Safety Factor		
AISC 360-16 Chapter J3.10 Eq. (J3-6a)	$\frac{R_{n\_bearing}}{\Omega}$ - Allowable Bolt Bearing Capacity of Section $$\frac{R_{n\_bearing}}{\Omega} = \frac{2.4\, d_b\, t_{wp}\, F_{up}\, C}{\Omega}$$		

$$\frac{R_{n\_bearing}}{\Omega} = \frac{2.4 \times (0.75 \text{ in}) \times (1 \text{ in}) \times (65 \text{ ksi}) \times (4)}{(2)}$$

$$\frac{R_{n\_bearing}}{\Omega} = 234 \text{ kip}$$

Calculate the clear distance of outer bolts on web plate.

$L_{eh,wp} = 2 \text{ in}$ - Web Plate Horizontal Edge Distance

$d_h = 0.9375 \text{ in}$ - Horizontal Bolt Hole Dimension at Web Plate

l_{c1} - Clear Distance at First Bolt Row

$$l_{c1} = L_{eh,wp} - \frac{d_h}{2}$$

$$l_{c1} = (2 \text{ in}) - \frac{(0.9375 \text{ in})}{2}$$

$$l_{c1} = 1.5313 \text{ in}$$

Calculate the clear distance of inner bolts on web plate.

$s_c = 3 \text{ in}$ - Bolt Column Spacing

$d_h = 0.9375 \text{ in}$ - Horizontal Bolt Hole Dimension at Web Plate

l_{c2} - Clear Distance at Rest of Bolts

$$l_{c2} = s_c - d_h$$

$$l_{c2} = (3 \text{ in}) - (0.9375 \text{ in})$$

$$l_{c2} = 2.0625 \text{ in}$$

Calculate the bolt tear-out capacity of the web plate.

$l_{c1} = 1.5313 \text{ in}$ - Clear Distance at First Bolt Row

$l_{c2} = 2.0625 \text{ in}$ - Clear Distance at Rest of Bolts

$t_{wp} = 1 \text{ in}$ - Web Plate Thickness

$F_{up} = 65 \text{ ksi}$ - Web Plate Tensile Stress

$C = 4$ - Bolt Group Coefficient

$n_c = 2$ - Number of Bolt Columns

$\Omega = 2$ - Bolt Bearing Safety Factor

AISC 360-16 Chapter J3.10 Eq. (J3-6c) — $\frac{R_{n\_tearout}}{\Omega}$ - Allowable Bolt Tear-out Capacity of Section

$$\frac{R_{n\_tearout}}{\Omega} = \frac{[1.2\, l_{c1}\, t_{wp}\, F_{up} + 1.2\, l_{c2}\, t_{wp}\, F_{up}\, (n_c - 1)] \left(\frac{C}{n_c}\right)}{\Omega}$$

$$\frac{R_{n\_tearout}}{\Omega} = \frac{[1.2 \times (1.5313 \text{ in}) \times (1 \text{ in}) \times (65 \text{ ksi}) + 1.2 \times (2.0625 \text{ in}) \times (1 \text{ in}) \times (65 \text{ ksi}) \times ((2) - 1)] \times \left(\frac{(4)}{(2)}\right)}{(2)}$$

$$\frac{R_{n\_tearout}}{\Omega} = 280.31 \text{ kip}$$

Determine the governing bearing and tear-out capacity of the bolt group on web plate.

AISC 360-16 Chapter J3.10 Eq. (J3-6a) — $\frac{R_{n\_bearing}}{\Omega} = 234 \text{ kip}$ - Allowable bolt bearing capacity of web plate

AISC 360-16 Chapter J3.10 Eq. (J3-6c) — $\frac{R_{n\_tearout}}{\Omega} = 280.31 \text{ kip}$ - Allowable bolt tear-out capacity of web plate

AISC 360-16 Chapter J3.10 — $\frac{R_n}{\Omega}$ - Governing Allowable Capacity

$$\frac{R_n}{\Omega} = min\left(\frac{R_{n\_bearing}}{\Omega}, \frac{R_{n\_tearout}}{\Omega}\right)$$

$$\frac{R_n}{\Omega} = min\left((234 \text{ kip}), (280.31 \text{ kip})\right)$$

$$\frac{R_n}{\Omega} = 234 \text{ kip}$$

Result: — PASS

Demand over Capacity Ratio

$DCR = \frac{P_{web}}{\frac{R_n}{\Omega}} = \frac{(208.88)}{(234 \text{ kip})} = 0.000027744$

Check No. 5: Shear and Axial Interaction of Bolt Bearing/Tearout at Web Plate

Calculate the shear and axial interaction of bolt bearing/tearout capacities in the web plate.

$V_a = 150 \text{ kip}$ - Shear Load

$P_{web} = 208.88$ - Axial Load at Web

$Interaction$ - Shear and Axial Interaction

$$Interaction = \sqrt{\left(\frac{V_a}{\frac{R_n}{\Omega}}\right)^2 + \left(\frac{P_{web}}{\frac{R_n}{\Omega}}\right)^2}$$

$$Interaction = \sqrt{\left(\frac{(150 \text{ kip})}{(234 \text{ kip})}\right)^2 + \left(\frac{(208.88)}{(234 \text{ kip})}\right)^2}$$

$$Interaction = 0.64103$$

Result: — PASS

Demand over Capacity Ratio

	$DCR = \frac{Interaction}{1.0} = \frac{(0.64103)}{(1)} = 0.64103$	
AISC 360-16 Chapter J3.10 Eq. (J3-6a) *AISC 360-16 Chapter J3.10 Eq. (J3-6c)* *AISC 360-16 Chapter J3.10 Eq. (J3-6a)* *AISC 360-16 Chapter J3.10 Eq. (J3-6c)* *AISC 360-16 Chapter J3.10*	**Check No. 6: Allowable Capacity of the Bolt Group in Bearing and Tear-out at Column Web due to Shear Load** **Calculate the bolt bearing capacity of the column web.** $d_b = 0.75$ in - Bolt Diameter $t_{w,lc} = 0.44$ in - Lower Column Web Thickness $F_{u,lc} = 65$ ksi - Lower Column Tensile Stress $C = 4$ - Bolt Group Coefficient $\Omega = 2$ - Bolt Bearing Safety Factor $\frac{R_{n\_bearing}}{\Omega}$ - Allowable Bolt Bearing Capacity of Section $$\frac{R_{n\_bearing}}{\Omega} = \frac{2.4\, d_b\, t_{w,lc}\, F_{u,lc}\, C}{\Omega}$$ $$\frac{R_{n\_bearing}}{\Omega} = \frac{2.4 \times (0.75 \text{ in}) \times (0.44 \text{ in}) \times (65 \text{ ksi}) \times (4)}{(2)}$$ $$\frac{R_{n\_bearing}}{\Omega} = 102.96 \text{ kip}$$ **Calculate the clear distance of inner bolts on column web.** $s_r = 3$ in - Bolt Row Spacing $d_h = 0.8125$ in - Vertical Bolt Hole Dimension at Column Web l_{c2} - Clear Distance at Rest of Bolts $$l_{c2} = s_r - d_h$$ $$l_{c2} = (3 \text{ in}) - (0.8125 \text{ in})$$ $$l_{c2} = 2.1875 \text{ in}$$ **Calculate the bolt tear-out capacity of the column web.** $l_{c2} = 2.1875$ in - Clear Distance at Rest of Bolts $t_{w,lc} = 0.44$ in - Lower Column Web Thickness $F_{u,lc} = 65$ ksi - Lower Column Tensile Stress $C = 4$ - Bolt Group Coefficient $\Omega = 2$ - Bolt Bearing Safety Factor $\frac{R_{n\_tearout}}{\Omega}$ - Allowable Bolt Tear-out Capacity of Section $$\frac{R_{n\_tearout}}{\Omega} = \frac{1.2\, l_{c2}\, t_{w,lc}\, F_{u,lc}\, C}{\Omega}$$ $$\frac{R_{n\_tearout}}{\Omega} = \frac{1.2 \times (2.1875 \text{ in}) \times (0.44 \text{ in}) \times (65 \text{ ksi}) \times (4)}{(2)}$$ $$\frac{R_{n\_tearout}}{\Omega} = 150.15 \text{ kip}$$ **Determine the governing bearing and tear-out capacity of the bolt group on column web.** $\frac{R_{n\_bearing}}{\Omega} = 102.96$ kip - Allowable bolt bearing capacity of column web $\frac{R_{n\_tearout}}{\Omega} = 150.15$ kip - Allowable bolt tear-out capacity of column web $\frac{R_n}{\Omega}$ - Governing Allowable Capacity $$\frac{R_n}{\Omega} = min\left(\frac{R_{n\_bearing}}{\Omega}, \frac{R_{n\_tearout}}{\Omega}\right)$$ $$\frac{R_n}{\Omega} = min\left((102.96 \text{ kip}), (150.15 \text{ kip})\right)$$ $$\frac{R_n}{\Omega} = 102.96 \text{ kip}$$ **Result:** Demand over Capacity Ratio $DCR = \frac{V_a}{\frac{R_n}{\Omega}} = \frac{(150 \text{ kip})}{(102.96 \text{ kip})} = 1.4569$	**FAIL**
AISC 360-16 Chapter J3.10 Eq. (J3-6a)	**Check No. 7: Allowable Capacity of the Bolt Group in Bearing and Tear-out at Column Web due to Axial Load** **Calculate the bolt bearing capacity of the column web.** $d_b = 0.75$ in - Bolt Diameter $t_{w,lc} = 0.44$ in - Lower Column Web Thickness $F_{u,lc} = 65$ ksi - Lower Column Tensile Stress $C = 4$ - Bolt Group Coefficient $\Omega = 2$ - Bolt Bearing Safety Factor $\frac{R_{n\_bearing}}{\Omega}$ - Allowable Bolt Bearing Capacity of Section $$\frac{R_{n\_bearing}}{\Omega} = \frac{2.4\, d_b\, t_{w,lc}\, F_{u,lc}\, C}{\Omega}$$ $$\frac{R_{n\_bearing}}{\Omega} = \frac{2.4 \times (0.75 \text{ in}) \times (0.44 \text{ in}) \times (65 \text{ ksi}) \times (4)}{(2)}$$ $$\frac{R_{n\_bearing}}{\Omega} = 102.96 \text{ kip}$$ **Calculate the clear distance of outer bolts on column web.**	

	$L_{eh,c} = 2$ in - Column Web Horizontal Edge Distance $d_h = 0.8125$ in - Horizontal Bolt Hole Dimension at Column Web l_{c1} - Clear Distance at First Bolt Row $$l_{c1} = L_{eh,c} - \frac{d_h}{2}$$ $$l_{c1} = (2\text{ in}) - \frac{(0.8125\text{ in})}{2}$$ $$l_{c1} = 1.5938\text{ in}$$ **Calculate the clear distance of inner bolts on column web.** $s_c = 3$ in - Bolt Column Spacing $d_h = 0.8125$ in - Horizontal Bolt Hole Dimension at Column Web l_{c2} - Clear Distance at Rest of Bolts $$l_{c2} = s_c - d_h$$ $$l_{c2} = (3\text{ in}) - (0.8125\text{ in})$$ $$l_{c2} = 2.1875\text{ in}$$ **Calculate the bolt tear-out capacity of the column web.** $l_{c1} = 1.5938$ in - Clear Distance at First Bolt Row $l_{c2} = 2.1875$ in - Clear Distance at Rest of Bolts $t_{w,lc} = 0.44$ in - Lower Column Web Thickness $F_{u,lc} = 65$ ksi - Lower Column Tensile Stress $C = 4$ - Bolt Group Coefficient $n_c = 2$ - Number of Bolt Columns $\Omega = 2$ - Bolt Bearing Safety Factor	
AISC 360-16 Chapter J3.10 Eq. (J3-6c)	$\frac{R_{n\_tearout}}{\Omega}$ - Allowable Bolt Tear-out Capacity of Section $$\frac{R_{n\_tearout}}{\Omega} = \frac{[1.2\, l_{c1}\, t_{w,lc}\, F_{u,lc} + 1.2\, l_{c2}\, t_{w,lc}\, F_{u,lc}\, (n_c - 1)] \left(\frac{C}{n_c}\right)}{\Omega}$$ $$\frac{R_{n\_tearout}}{\Omega} = \frac{[1.2 \times (1.5938\text{ in}) \times (0.44\text{ in}) \times (65\text{ ksi}) + 1.2 \times (2.1875\text{ in}) \times (0.44\text{ in}) \times (65\text{ ksi}) \times ((2) - 1)] \times \left(\frac{(4)}{(2)}\right)}{(2)}$$ $$\frac{R_{n\_tearout}}{\Omega} = 129.77\text{ kip}$$ **Determine the governing bearing and tear-out capacity of the bolt group on column web.**	
AISC 360-16 Chapter J3.10 Eq. (J3-6a)	$\frac{R_{n\_bearing}}{\Omega} = 102.96$ kip - Allowable bolt bearing capacity of column web	
AISC 360-16 Chapter J3.10 Eq. (J3-6c)	$\frac{R_{n\_tearout}}{\Omega} = 129.77$ kip - Allowable bolt tear-out capacity of column web	
AISC 360-16 Chapter J3.10	$\frac{R_n}{\Omega}$ - Governing Allowable Capacity $$\frac{R_n}{\Omega} = min\left(\frac{R_{n\_bearing}}{\Omega}, \frac{R_{n\_tearout}}{\Omega}\right)$$ $$\frac{R_n}{\Omega} = min\left((102.96\text{ kip}), (129.77\text{ kip})\right)$$ $$\frac{R_n}{\Omega} = 102.96\text{ kip}$$ **Result:** Demand over Capacity Ratio $DCR = \frac{P_{web}}{\frac{R_n}{\Omega}} = \frac{(208.88)}{(102.96\text{ kip})} = 0.000063054$	FAIL
	Check No. 8: Shear and Axial Interaction of Bolt Bearing/Tearout at Column Web **Calculate the shear and axial interaction of bolt bearing/tearout capacities in the column web.** $V_a = 150$ kip - Shear Load $P_{web} = 208.88$ - Axial Load at Web $Interaction$ - Shear and Axial Interaction $$Interaction = \sqrt{\left(\frac{V_a}{\frac{R_n}{\Omega}}\right)^2 + \left(\frac{P_{web}}{\frac{R_n}{\Omega}}\right)^2}$$ $$Interaction = \sqrt{\left(\frac{(150\text{ kip})}{(102.96\text{ kip})}\right)^2 + \left(\frac{(208.88)}{(102.96\text{ kip})}\right)^2}$$ $$Interaction = 1.4569$$ **Result:** Demand over Capacity Ratio $DCR = \frac{Interaction}{1.0} = \frac{(1.4569)}{(1)} = 1.4569$	FAIL
	Check No. 9: Allowable Capacity of Web Plate in Block Shear due to Shear Load **Calculate the net area of the web plate subject to tension.** $t_{wp} = 1$ in - Web Plate Thickness $n_c = 2$ - Number of Bolt Columns $s_c = 3$ in - Bolt Column Spacing	

$L_{eh,wp} = 2 \text{ in}$ - Web Plate Horizontal Edge Distance
$d_h = 0.9375 \text{ in}$ - Horizontal Bolt Hole Dimension at Web Plate
A_{nt} - Net Area Subject to Tension (L-pattern)

$$A_{nt} = t_{wp} \left[(n_c - 1)\ s_c + L_{eh,wp} - (n_c - 0.5)\ (d_h + 0.0625 \text{ in})\right]$$

$$A_{nt} = (1 \text{ in}) \times [((2) - 1) \times (3 \text{ in}) + (2 \text{ in}) - ((2) - 0.5) \times ((0.9375 \text{ in}) + (0.0625 \text{ in}))]$$

$$A_{nt} = 3.5 \text{ in}^2$$

Calculate the gross area of the web plate subject to shear.
$t_{wp} = 1 \text{ in}$ - Web Plate Thickness
$L_{ev,wp} = 2 \text{ in}$ - Web Plate Vertical Edge Distance
$n_r = 2$ - Number of Bolt Rows
$s_r = 3 \text{ in}$ - Bolt Row Spacing
A_{gv} - Gross Area Subject to Shear (L-pattern)

$$A_{gv} = t_{wp}\ [L_{ev,wp} + (n_r - 1)\ s_r]$$

$$A_{gv} = (1 \text{ in}) \times [(2 \text{ in}) + ((2) - 1) \times (3 \text{ in})]$$

$$A_{gv} = 5 \text{ in}^2$$

Calculate the net area of the web plate subject to shear.
$t_{wp} = 1 \text{ in}$ - Web Plate Thickness
$b_{wp} = 7 \text{ in}$ - Web Plate Width
$L_{ev,wp} = 2 \text{ in}$ - Web Plate Vertical Edge Distance
$n_r = 2$ - Number of Bolt Rows
$d_h = 0.9375 \text{ in}$ - Vertical Bolt Hole Dimension at Web Plate
A_{nv} - Net Area Subject to Shear (L-pattern)

$$A_{nv} = t_{wp}\ [b_{wp} - L_{ev,wp} - (n_r - 0.5)\ (d_h + 0.0625 \text{ in})]$$

$$A_{nv} = (1 \text{ in}) \times [(7 \text{ in}) - (2 \text{ in}) - ((2) - 0.5) \times ((0.9375 \text{ in}) + (0.0625 \text{ in}))]$$

$$A_{nv} = 3.5 \text{ in}^2$$

Calculate the allowable block shear capacity of the web plate.
$F_{up} = 65 \text{ ksi}$ - Web Plate Tensile Stress
$A_{nv} = 3.5 \text{ in}^2$ - Net Area Subject to Shear (L-pattern)
$U_{bs} = 0.5$ - Uniformity factor
$A_{nt} = 3.5 \text{ in}^2$ - Net Area Subject to Tension (L-pattern)
$F_{yp} = 50 \text{ ksi}$ - Web Plate Yield Stress
$A_{gv} = 5 \text{ in}^2$ - Gross Area Subject to Shear (L-pattern)
$\Omega = 2$ - Block Shear Safety Factor

AISC 360-16 Chapter J4.3 Eq. (J4-5)

$\frac{R_n}{\Omega}$ - Allowable Block Shear Capacity of Section

$$\frac{R_n}{\Omega} = \frac{0.6\ F_{up}\ A_{nv} + U_{bs}\ F_{up}\ A_{nt} \le 0.6\ F_{yp}\ A_{gv} + U_{bs}\ F_{up}\ A_{nt}}{\Omega}$$

$$\frac{R_n}{\Omega} = \frac{0.6 \times (65 \text{ ksi}) \times \left(3.5 \text{ in}^2\right) + (0.5) \times (65 \text{ ksi}) \times \left(3.5 \text{ in}^2\right) \le 0.6 \times (50 \text{ ksi}) \times \left(5 \text{ in}^2\right) + (0.5) \times (65 \text{ ksi}) \times \left(3.5 \text{ in}^2\right)}{(2)}$$

$$\frac{R_n}{\Omega} = 125.13 \text{ kip}$$

Result:
Demand over Capacity Ratio
$DCR = \frac{V_a}{\frac{R_n}{\Omega}} = \frac{(150 \text{ kip})}{(125.13 \text{ kip})} = 1.1988$

FAIL

Check No. 10: Allowable Capacity of Web Plate in Block Shear due to Axial Load

Calculate the net area of the web plate subject to tension.
$t_{wp} = 1 \text{ in}$ - Web Plate Thickness
$n_r = 2$ - Number of Bolt Rows
$s_r = 3 \text{ in}$ - Bolt Row Spacing
$L_{ev,wp} = 2 \text{ in}$ - Web Plate Vertical Edge Distance
$d_h = 0.9375 \text{ in}$ - Vertical Bolt Hole Dimension at Web Plate
A_{nt} - Net Area Subject to Tension (L-pattern)

$$A_{nt} = t_{wp} \left[(n_r - 1)\ s_r + L_{ev,wp} - (n_r - 0.5)\ (d_h + 0.0625 \text{ in})\right]$$

$$A_{nt} = (1 \text{ in}) \times [((2) - 1) \times (3 \text{ in}) + (2 \text{ in}) - ((2) - 0.5) \times ((0.9375 \text{ in}) + (0.0625 \text{ in}))]$$

$$A_{nt} = 3.5 \text{ in}^2$$

Calculate the gross area of the web plate subject to shear.
$t_{wp} = 1 \text{ in}$ - Web Plate Thickness
$L_{eh,wp} = 2 \text{ in}$ - Web Plate Horizontal Edge Distance
$n_c = 2$ - Number of Bolt Columns
$s_c = 3 \text{ in}$ - Bolt Column Spacing
A_{gv} - Gross Area Subject to Shear (L-pattern)

$$A_{gv} = t_{wp}\ [L_{eh,wp} + (n_c - 1)\ s_c]$$

$$A_{gv} = (1 \text{ in}) \times [(2 \text{ in}) + ((2) - 1) \times (3 \text{ in})]$$

$$A_{gv} = 5 \text{ in}^2$$

Calculate the net area of the web plate subject to shear.

$t_{wp} = 1 \text{ in}$ - Web Plate Thickness
$L_{wp} = 7 \text{ in}$ - Web Plate Length
$L_{eh,c} = 2 \text{ in}$ - Column Web Horizontal Edge Distance
$n_c = 2$ - Number of Bolt Columns
$d_h = 0.9375 \text{ in}$ - Horizontal Bolt Hole Dimension at Web Plate
A_{nv} - Net Area Subject to Shear (L-pattern)

$$A_{nv} = t_{wp} \left[L_{wp} - L_{eh,c} - (n_c - 0.5)\ (d_h + 0.0625 \text{ in})\right]$$

$$A_{nv} = (1 \text{ in}) \times [(7 \text{ in}) - (2 \text{ in}) - ((2) - 0.5) \times ((0.9375 \text{ in}) + (0.0625 \text{ in}))]$$

$$A_{nv} = 3.5 \text{ in}^2$$

Calculate the allowable block shear capacity of the web plate.

$F_{up} = 65 \text{ ksi}$ - Web Plate Tensile Stress
$A_{nv} = 3.5 \text{ in}^2$ - Net Area Subject to Shear (L-pattern)
$U_{bs} = 1$ - Uniformity factor
$A_{nt} = 3.5 \text{ in}^2$ - Net Area Subject to Tension (L-pattern)
$F_{yp} = 50 \text{ ksi}$ - Web Plate Yield Stress
$A_{gv} = 5 \text{ in}^2$ - Gross Area Subject to Shear (L-pattern)
$\Omega = 2$ - Block Shear Safety Factor

AISC 360-16 Chapter J4.3 Eq. (J4-5)

$\frac{R_n}{\Omega}$ - Allowable Block Shear Capacity of Section

$$\frac{R_n}{\Omega} = \frac{0.6\ F_{up}\ A_{nv} + U_{bs}\ F_{up}\ A_{nt} \leq 0.6\ F_{yp}\ A_{gv} + U_{bs}\ F_{up}\ A_{nt}}{\Omega}$$

$$\frac{R_n}{\Omega} = \frac{0.6 \times (65 \text{ ksi}) \times \left(3.5 \text{ in}^2\right) + (1) \times (65 \text{ ksi}) \times \left(3.5 \text{ in}^2\right) \leq 0.6 \times (50 \text{ ksi}) \times \left(5 \text{ in}^2\right) + (1) \times (65 \text{ ksi}) \times \left(3.5 \text{ in}^2\right)}{(2)}$$

$$\frac{R_n}{\Omega} = 182 \text{ kip}$$

Result:
Demand over Capacity Ratio

$DCR = \frac{T_{web}}{\frac{R_n}{\Omega}} = \frac{(208.88 \text{ kip})}{(182 \text{ kip})} = 1.1477$

FAIL

Check No. 11: Shear and Axial Interaction of Block Shear in Web Plate

Calculate the shear and axial interaction of block shear capacities in the web plate.

$V_a = 150 \text{ kip}$ - Shear Load
$T_{web} = 208.88 \text{ kip}$ - Tension load at web connection. This is proportioned based on web area over the total gross area.
$Interaction$ - Shear and Axial Interaction

$$Interaction = \sqrt{\left(\frac{V_a}{\frac{R_n}{\Omega}}\right)^2 + \left(\frac{T_{web}}{\frac{R_n}{\Omega}}\right)^2}$$

$$Interaction = \sqrt{\left(\frac{(150 \text{ kip})}{(125.13 \text{ kip})}\right)^2 + \left(\frac{(208.88 \text{ kip})}{(182 \text{ kip})}\right)^2}$$

$$Interaction = 1.6596$$

Result:
Demand over Capacity Ratio

$DCR = \frac{Interaction}{1.0} = \frac{(1.6596)}{(1)} = 1.6596$

FAIL

Check No. 12: Allowable Capacity of Column Web in Block Shear due to Axial Load

Calculate the net area of the column web subject to tension.

$t_{w,lc} = 0.44 \text{ in}$ - Lower Column Web Thickness
$n_r = 2$ - Number of Bolt Rows
$s_r = 3 \text{ in}$ - Bolt Row Spacing
$d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Column Web
A_{nt} - Net Area Subject to Tension (C-pattern)

$$A_{nt} = t_{w,lc}\ [(n_r - 1)\ s_r - (n_r - 1)\ (d_h + 0.0625 \text{ in})]$$

$$A_{nt} = (0.44 \text{ in}) \times [((2) - 1) \times (3 \text{ in}) - ((2) - 1) \times ((0.8125 \text{ in}) + (0.0625 \text{ in}))]$$

$$A_{nt} = 0.935 \text{ in}^2$$

Calculate the gross area of the column web subject to shear.

$t_{w,lc} = 0.44 \text{ in}$ - Lower Column Web Thickness
$L_{eh,c} = 2 \text{ in}$ - Column Web Horizontal Edge Distance
$n_c = 2$ - Number of Bolt Columns
$s_c = 3 \text{ in}$ - Bolt Column Spacing
A_{gv} - Gross Area Subject to Shear (C-pattern)

$$A_{gv} = 2\ t_{w,lc}\ [L_{eh,c} + (n_c - 1)\ s_c]$$

$$A_{gv} = 2 \times (0.44 \text{ in}) \times [(2 \text{ in}) + ((2) - 1) \times (3 \text{ in})]$$

$$A_{gv} = 4.4 \text{ in}^2$$

	Calculate the net area of the column web subject to shear. $t_{w,lc} = 0.44$ in - Lower Column Web Thickness $L_{eh,c} = 2$ in - Column Web Horizontal Edge Distance $n_c = 2$ - Number of Bolt Columns $s_c = 3$ in - Bolt Column Spacing $d_h = 0.8125$ in - Horizontal Bolt Hole Dimension at Column Web A_{nv} - Net Area Subject to Shear (C-pattern) $$A_{nv} = 2\, t_{w,lc}\, [(n_c - 1)\, s_c + L_{eh,c} - (n_c - 0.5)\, (d_h + 0.0625 \text{ in})]$$ $$A_{nv} = 2 \times (0.44 \text{ in}) \times [((2) - 1) \times (3 \text{ in}) + (2 \text{ in}) - ((2) - 0.5) \times ((0.8125 \text{ in}) + (0.0625 \text{ in}))]$$ $$A_{nv} = 3.245 \text{ in}^2$$	
AISC 360-16 Chapter J4.3 Eq. (J4-5)	**Calculate the allowable block shear capacity of the column web.** $F_{u,lc} = 65$ ksi - Lower Column Tensile Stress $A_{nt} = 0.935 \text{ in}^2$ - Net Area Subject to Tension (C-pattern) $U_{bs} = 1$ - Uniformity factor $A_{gv} = 4.4 \text{ in}^2$ - Gross Area Subject to Shear (C-pattern) $F_{y,lc} = 50$ ksi - Lower Column Yield Stress $A_{nv} = 3.245 \text{ in}^2$ - Net Area Subject to Shear (C-pattern) $\Omega = 2$ - Block Shear Safety Factor $\frac{R_n}{\Omega}$ - Allowable Block Shear Capacity of Section $$\frac{R_n}{\Omega} = \frac{0.6\, F_{u,lc}\, A_{nv} + U_{bs}\, F_{u,lc}\, A_{nt} \leq 0.6\, F_{y,lc}\, A_{gv} + U_{bs}\, F_{u,lc}\, A_{nt}}{\Omega}$$ $$\frac{R_n}{\Omega} = \frac{0.6 \times (65 \text{ ksi}) \times \left(3.245 \text{ in}^2\right) + (1) \times (65 \text{ ksi}) \times \left(0.935 \text{ in}^2\right) \leq 0.6 \times (50 \text{ ksi}) \times \left(4.4 \text{ in}^2\right) + (1) \times (65 \text{ ksi}) \times \left(0.935 \text{ in}^2\right)}{(2)}$$ $$\frac{R_n}{\Omega} = 93.665 \text{ kip}$$ **Result:** Demand over Capacity Ratio $DCR = \frac{T_{web}}{\frac{R_n}{\Omega}} = \frac{(208.88 \text{ kip})}{(93.665 \text{ kip})} = 2.23$	**FAIL**
	Check No. 13: Allowable Capacity of Web Plate in Shear **Calculate the gross area of web plate subject to yielding.** $b_{wp} = 7$ in - Web Plate Width $t_{wp} = 1$ in - Web Plate Thickness A_{gv} - Section Gross Area $$A_{gv} = b_{wp}\, t_{wp}$$ $$A_{gv} = (7 \text{ in}) \times (1 \text{ in})$$ $$A_{gv} = 7 \text{ in}^2$$	
AISC 360-16 Chapter J4.2 Eq. (J4-3)	**Calculate the shear yielding capacity of the web plate.** $F_{yp} = 50$ ksi - Web Plate Yield Stress $A_{gv} = 7 \text{ in}^2$ - Section Gross Area $\Omega = 1.5$ - Shear Yielding Safety Factor $\frac{R_{n\_sy}}{\Omega}$ - Allowable Shear Yielding Capacity of Section $$\frac{R_{n\_sy}}{\Omega} = \frac{0.6\, F_{yp}\, A_{gv}}{\Omega}$$ $$\frac{R_{n\_sy}}{\Omega} = \frac{0.6 \times (50 \text{ ksi}) \times \left(7 \text{ in}^2\right)}{(1.5)}$$ $$\frac{R_{n\_sy}}{\Omega} = 140 \text{ kip}$$	
	Calculate the net area of web plate subject to rupture. $t_{wp} = 1$ in - Web Plate Thickness $b_{wp} = 7$ in - Web Plate Width $n_r = 2$ - Number of Bolt Rows $d_h = 0.9375$ in - Vertical Bolt Hole Dimension at Web Plate A_{nv} - Section Net Area $$A_{nv} = t_{wp}\, [b_{wp} - n_r\, (d_h + 0.0625 \text{ in})]$$ $$A_{nv} = (1 \text{ in}) \times [(7 \text{ in}) - (2) \times ((0.9375 \text{ in}) + (0.0625 \text{ in}))]$$ $$A_{nv} = 5 \text{ in}^2$$	
AISC 360-16 Chapter J4.2 Eq. (J4-4)	**Calculate the shear rupture capacity of the web plate.** $F_{up} = 65$ ksi - Web Plate Tensile Stress $A_{nv} = 5 \text{ in}^2$ - Section Net Area $\Omega = 1.5$ - Shear Yielding Safety Factor $\frac{R_{n\_sr}}{\Omega}$ - Allowable Shear Rupture Capacity of Section	

$$\frac{R_{n\_sr}}{\Omega} = \frac{0.6\,F_{up}\,A_{nv}}{\Omega}$$

$$\frac{R_{n\_sr}}{\Omega} = \frac{0.6 \times (65\text{ ksi}) \times (5\text{ in}^2)}{(1.5)}$$

$$\frac{R_{n\_sr}}{\Omega} = 130\text{ kip}$$

Determine the governing shear capacity of the web plate.

AISC 360-16 Chapter J4.2 Eq. (J4-3) — $\frac{R_{n\_sy}}{\Omega} = 140\text{ kip}$ - Allowable shear yielding capacity of web plate

AISC 360-16 Chapter J4.2 Eq. (J4-4) — $\frac{R_{n\_sr}}{\Omega} = 130\text{ kip}$ - Allowable shear rupture capacity of web plate

AISC 360-16 Chapter J4.2 — $\frac{R_n}{\Omega}$ - Governing Allowable Capacity

$$\frac{R_n}{\Omega} = min\left(\frac{R_{n\_sy}}{\Omega}, \frac{R_{n\_sr}}{\Omega}\right)$$

$$\frac{R_n}{\Omega} = min\left((140\text{ kip}), (130\text{ kip})\right)$$

$$\frac{R_n}{\Omega} = 130\text{ kip}$$

Result: — **FAIL**

Demand over Capacity Ratio

$DCR = \frac{V_a}{\frac{R_n}{\Omega}} = \frac{(150\text{ kip})}{(130\text{ kip})} = 1.1538$

Check No. 14: Allowable Capacity of Web Plate in Tension

Calculate the tensile yielding capacity of the web plate.

$\Omega = 1.67$ - Tensile Yielding Safety Factor

$F_{yp} = 50\text{ ksi}$ - Web Plate Yield Stress

$t_{wp} = 1\text{ in}$ - Web Plate Thickness

$b_{wp} = 7\text{ in}$ - Web Plate Width

AISC 360-16 Chapter J4.1 Eq. (J4-1) — $\frac{R_{n\_ty}}{\Omega}$ - Allowable Tension Yielding Capacity of Section

$$\frac{R_{n\_ty}}{\Omega} = \frac{F_{yp}\,t_{wp}\,b_{wp}}{\Omega}$$

$$\frac{R_{n\_ty}}{\Omega} = \frac{(50\text{ ksi}) \times (1\text{ in}) \times (7\text{ in})}{(1.67)}$$

$$\frac{R_{n\_ty}}{\Omega} = 209.58\text{ kip}$$

Calculate the tensile rupture capacity of the web plate.

$\Omega = 2$ - Tensile Rupture Safety Factor

$F_{up} = 65\text{ ksi}$ - Web Plate Tensile Stress

$t_{wp} = 1\text{ in}$ - Web Plate Thickness

$b_{wp} = 7\text{ in}$ - Web Plate Width

$n_r = 2$ - Number of Bolt Rows

$d_h = 0.9375\text{ in}$ - Vertical Bolt Hole Dimension at Web Plate

AISC 360-16 Chapter J4.1 Eq. (J4-2) — $\frac{R_{n\_tr}}{\Omega}$ - Allowable Tension Rupture Capacity of Section

$$\frac{R_{n\_tr}}{\Omega} = \frac{F_{up}\,t_{wp}\,[b_{wp} - n_r\,(d_h + 0.0625\text{ in})]}{\Omega}$$

$$\frac{R_{n\_tr}}{\Omega} = \frac{(65\text{ ksi}) \times (1\text{ in}) \times [(7\text{ in}) - (2) \times ((0.9375\text{ in}) + (0.0625\text{ in}))]}{(2)}$$

$$\frac{R_{n\_tr}}{\Omega} = 162.5\text{ kip}$$

Determine the governing tensile capacity of the web plate.

AISC 360-16 Chapter J4.1 Eq. (J4-1) — $\frac{R_{n\_ty}}{\Omega} = 209.58\text{ kip}$ - Allowable Tension Yielding Capacity of Section

AISC 360-16 Chapter J4.1 Eq. (J4-2) — $\frac{R_{n\_tr}}{\Omega} = 162.5\text{ kip}$ - Allowable Tension Rupture Capacity of Section

AISC 360-16 Chapter J4.1 — $\frac{R_n}{\Omega}$ - Governing Allowable Capacity

$$\frac{R_n}{\Omega} = min\left(\frac{R_{n\_ty}}{\Omega}, \frac{R_{n\_tr}}{\Omega}\right)$$

$$\frac{R_n}{\Omega} = min\left((209.58\text{ kip}), (162.5\text{ kip})\right)$$

$$\frac{R_n}{\Omega} = 162.5\text{ kip}$$

Result: — **FAIL**

Demand over Capacity Ratio

$DCR = \frac{T_{web}}{\frac{R_n}{\Omega}} = \frac{(208.88\text{ kip})}{(162.5\text{ kip})} = 1.2854$

Check No. 15: Allowable Capacity of Beam in Tension

Calculate the tensile rupture capacity of the beam.

$\Omega = 2$ - Tensile Rupture Safety Factor

$F_{u,lc} = 65\text{ ksi}$ - Lower Column Tensile Stress

	$A_{lc} = 26.5\ \text{in}^2$ - Lower Column Area $t_{w,lc} = 0.44\ \text{in}$ - Lower Column Web Thickness $d_h = 0.8125\ \text{in}$ - Vertical Bolt Hole Dimension at Column Web $n_r = 2$ - Number of Bolt Rows $n_c = 2$ - Number of Bolt Columns $s_c = 3\ \text{in}$ - Bolt Column Spacing $\bar{x} = 3.1003\ \text{in}$ - Centroid of WF section	
AISC 360-16 Chapter D3 (Table D3.1 case 2)	$U = 0.175$ - Shear Lag Factor	
AISC 360-16 Chapter J4.1 Eq. (J4-2)	$\frac{R_{n\_tr}}{\Omega}$ - Allowable Tension Rupture Capacity of Section $$\frac{R_{n\_tr}}{\Omega} = \frac{F_{u,lc}\, U\, \left[A_{lc} - n_r\ \left(d_h + 0.0625\ \text{in}\right)\ t_{w,lc}\right]}{\Omega}$$ $$\frac{R_{n\_tr}}{\Omega} = \frac{(65\ \text{ksi}) \times (0.175) \times \left[\left(26.5\ \text{in}^2\right) - (2) \times \left((0.8125\ \text{in}) + (0.0625\ \text{in})\right) \times (0.44\ \text{in})\right]}{(2)}$$ $$\frac{R_{n\_tr}}{\Omega} = 146.34\ \text{kip}$$ **Result:** Demand over Capacity Ratio $DCR = \frac{T_{web}}{\frac{R_{n\_tr}}{\Omega}} = \frac{(208.88\ \text{kip})}{(146.34\ \text{kip})} = 1.4273$	FAIL
	Check No. 16: Allowable Capacity of Web Plate in Compression **Calculate the compression buckling capacity of the web plate.** $\Omega = 1.67$ - Compression Safety Factor $F_{yp} = 50\ \text{ksi}$ - Web Plate Yield Stress $t_{wp} = 1\ \text{in}$ - Web Plate Thickness $b_{wp} = 7\ \text{in}$ - Web Plate Width $K = 1.2$ - Effective Length Factor $L_b = 2\ \text{in}$ - Web Plate Unbraced Length $\frac{KL}{r} = 8.3138$ - Effective Length Slenderness Ratio Since, $\frac{KL}{r} \leq 25$.	
AISC 360-16 Chapter J4.4 Eq. (J4-6)	$\frac{R_n}{\Omega}$ - Allowable Compressive Capacity of Section $$\frac{R_n}{\Omega} = \frac{F_{yp}\, t_{wp}\, b_{wp}}{\Omega}$$ $$\frac{R_n}{\Omega} = \frac{(50\ \text{ksi}) \times (1\ \text{in}) \times (7\ \text{in})}{(1.67)}$$ $$\frac{R_n}{\Omega} = 209.58\ \text{kip}$$ **Result:** Demand over Capacity Ratio $DCR = \frac{C_{web}}{\frac{R_n}{\Omega}} = \frac{(67.752\ \text{kip})}{(209.58\ \text{kip})} = 0.32327$	PASS

REFERENCES	CALCULATIONS	RESULTS

Summary of Checks

Design Checks	Demand	Capacity	DCR	Result
Connection Detailing Limitations	5.000	7.000	0.714	PASS
Allowable Capacity of the Bolt Group in Shear	257.156	25.312	10.159	FAIL
Allowable Capacity of the Bolt Group in Bearing and Tear-out at Web Plate due to Shear Load	150.000	234.000	0.641	PASS
Allowable Capacity of the Bolt Group in Bearing and Tear-out at Web Plate due to Axial Load	208.875	234.000	0.893	PASS
Shear and Axial Interaction of Bolt Bearing/Tearout at Web Plate	0.641	1.000	0.641	PASS
Allowable Capacity of the Bolt Group in Bearing and Tear-out at Column Web due to Shear Load	150.000	102.960	1.457	FAIL
Allowable Capacity of the Bolt Group in Bearing and Tear-out at Column Web due to Axial Load	208.875	102.960	2.029	FAIL
Shear and Axial Interaction of Bolt Bearing/Tearout at Column Web	1.457	1.000	1.457	FAIL
Allowable Capacity of Web Plate in Block Shear due to Shear Load	150.000	125.125	1.199	FAIL
Allowable Capacity of Web Plate in Block Shear due to Axial Load	208.875	182.000	1.148	FAIL
Shear and Axial Interaction of Block Shear in Web Plate	1.660	1.000	1.660	FAIL
Allowable Capacity of Column Web in Block Shear due to Axial Load	208.875	93.665	2.230	FAIL
Allowable Capacity of Web Plate in Shear	150.000	130.000	1.154	FAIL
Allowable Capacity of Web Plate in Tension	208.875	162.500	1.285	FAIL
Allowable Capacity of Beam in Tension	208.875	146.339	1.427	FAIL
Allowable Capacity of Web Plate in Compression	67.752	209.581	0.323	PASS

Web Plate Connection at Upper Column | AISC 360-16 ASD

Web Plate Geometry:

$b_{wp} = 7$ in - Web Plate Width

$L_{wp} = 14$ in - Web Plate Length

$t_{wp} = 1$ in - Web Plate Thickness

Web Plate Material Grade:

$F_{yp} = 50$ ksi - Web Plate Yield Stress

$F_{up} = 65$ ksi - Web Plate Tensile Stress

Connection Information at Web Plate:

$n_r = 2$ - Number of Bolt Rows

$s_r = 3$ in - Bolt Row Spacing

$n_c = 2$ - Number of Bolt Columns

$s_c = 3$ in - Bolt Column Spacing

Distances:

$L_{ev,wp} = 3.5$ in - Web Plate Vertical Edge Distance

$L_{eh,c} = 2$ in - Column Web Horizontal Edge Distance

$L_{eh,wp} = 2$ in - Web Plate Horizontal Edge Distance

Check No. 1: Connection Detailing Limitations

Detailing Limitations	Limit Value (in)	Actual Value (in)	DCR	Result
Minimum Bolt Row Spacing	2.000	3.000	0.667	PASS
Maximum Bolt Row Spacing	6.000	3.000	0.500	PASS
Minimum Bolt Column Spacing	2.000	3.000	0.667	PASS
Maximum Bolt Column Spacing	6.000	3.000	0.500	PASS
Web Plate Minimum Vertical Edge Distance	1.063	3.500	0.304	PASS
Web Plate Minimum Horizontal Edge Distance	1.063	2.000	0.531	PASS
Minimum Connection Depth	5.000	7.000	0.714	PASS
Maximum Connection Depth	10.000	7.000	0.700	PASS

Result:

Demand over Capacity Ratio

$DCR = \frac{d}{c} = \frac{(5)}{(7)} = 0.71429$

PASS

Check No. 2: Allowable Capacity of the Bolt Group in Shear

$\Omega = 1.5$ - Bolt Shear Safety Factor

$\mu = 0.3$ - Mean Slip Coefficient

$D_u = 1.13$

$h_f = 1$ - Filler Factor for SC Bolts

$d_b = 0.75$ in - Bolt Diameter

$T_b = 28$ kip - Minimum Bolt Pretension

$N_s = 1$ - Number of Slip Planes

$n_r = 2$ - Number of Bolt Rows

$n_c = 2$ - Number of Bolt Columns

AISC 360-16 Chapter J3.8 Eq. (J3-4)

$\frac{R_n}{\Omega}$ - Allowable Bolt Shear Capacity

$$\frac{R_n}{\Omega} = \frac{\mu\, D_u\, h_f\, T_b\, N_s\, n_r\, n_c}{\Omega}$$

$$\frac{R_n}{\Omega} = \frac{(0.3) \times (1.13) \times (1) \times (28 \text{ kip}) \times (1) \times (2) \times (2)}{(1.5)}$$

$$\frac{R_n}{\Omega} = 25.312 \text{ kip}$$

Result:

Demand over Capacity Ratio

$DCR = \frac{R_a}{\frac{R_n}{\Omega}} = \frac{(257.16 \text{ kip})}{(25.312 \text{ kip})} = 10.159$

FAIL

Check No. 3: Allowable Capacity of the Bolt Group in Bearing and Tear-out at Web Plate due to Shear Load

Calculate the bolt bearing capacity of the web plate.

$d_b = 0.75\ \text{in}$ - Bolt Diameter
$t_{wp} = 1\ \text{in}$ - Web Plate Thickness
$F_{up} = 65\ \text{ksi}$ - Web Plate Tensile Stress
$C = 4$ - Bolt Group Coefficient
$\Omega = 2$ - Bolt Bearing Safety Factor

AISC 360-16 Chapter J3.10 Eq. (J3-6a) $\frac{R_{n\_bearing}}{\Omega}$ - Allowable Bolt Bearing Capacity of Section

$$\frac{R_{n\_bearing}}{\Omega} = \frac{2.4\, d_b\, t_{wp}\, F_{up}\, C}{\Omega}$$

$$\frac{R_{n\_bearing}}{\Omega} = \frac{2.4 \times (0.75\ \text{in}) \times (1\ \text{in}) \times (65\ \text{ksi}) \times (4)}{(2)}$$

$$\frac{R_{n\_bearing}}{\Omega} = 234\ \text{kip}$$

Calculate the clear distance of outer bolts on web plate.

$L_{ev,wp} = 3.5\ \text{in}$ - Web Plate Vertical Edge Distance
$d_h = 0.9375\ \text{in}$ - Vertical Bolt Hole Dimension at Web Plate
l_{c1} - Clear Distance at First Bolt Row

$$l_{c1} = L_{ev,wp} - \frac{d_h}{2}$$

$$l_{c1} = (3.5\ \text{in}) - \frac{(0.9375\ \text{in})}{2}$$

$$l_{c1} = 3.0313\ \text{in}$$

Calculate the clear distance of inner bolts on web plate.

$s_r = 3\ \text{in}$ - Bolt Row Spacing
$d_h = 0.9375\ \text{in}$ - Vertical Bolt Hole Dimension at Web Plate
l_{c2} - Clear Distance at Rest of Bolts

$$l_{c2} = s_r - d_h$$

$$l_{c2} = (3\ \text{in}) - (0.9375\ \text{in})$$

$$l_{c2} = 2.0625\ \text{in}$$

Calculate the bolt tear-out capacity of the web plate.

$l_{c1} = 3.0313\ \text{in}$ - Clear Distance at First Bolt Row
$l_{c2} = 2.0625\ \text{in}$ - Clear Distance at Rest of Bolts
$t_{wp} = 1\ \text{in}$ - Web Plate Thickness
$F_{up} = 65\ \text{ksi}$ - Web Plate Tensile Stress
$C = 4$ - Bolt Group Coefficient
$n_r = 2$ - Number of Bolt Rows
$\Omega = 2$ - Bolt Bearing Safety Factor

AISC 360-16 Chapter J3.10 Eq. (J3-6c) $\frac{R_{n\_tearout}}{\Omega}$ - Allowable Bolt Tear-out Capacity of Section

$$\frac{R_{n\_tearout}}{\Omega} = \frac{[1.2\, l_{c1}\, t_{wp}\, F_{up} + 1.2\, l_{c2}\, t_{wp}\, F_{up}\, (n_r - 1)]\left(\frac{C}{n_r}\right)}{\Omega}$$

$$\frac{R_{n\_tearout}}{\Omega} = \frac{[1.2 \times (3.0313\ \text{in}) \times (1\ \text{in}) \times (65\ \text{ksi}) + 1.2 \times (2.0625\ \text{in}) \times (1\ \text{in}) \times (65\ \text{ksi}) \times ((2) - 1)] \times \left(\frac{(4)}{(2)}\right)}{(2)}$$

$$\frac{R_{n\_tearout}}{\Omega} = 397.31\ \text{kip}$$

Determine the governing bearing and tear-out capacity of the bolt group on web plate.

AISC 360-16 Chapter J3.10 Eq. (J3-6a) $\frac{R_{n\_bearing}}{\Omega} = 234\ \text{kip}$ - Allowable bolt bearing capacity of web plate
AISC 360-16 Chapter J3.10 Eq. (J3-6c) $\frac{R_{n\_tearout}}{\Omega} = 397.31\ \text{kip}$ - Allowable bolt tear-out capacity of web plate
AISC 360-16 Chapter J3.10 $\frac{R_n}{\Omega}$ - Governing Allowable Capacity

$$\frac{R_n}{\Omega} = min\left(\frac{R_{n\_bearing}}{\Omega}, \frac{R_{n\_tearout}}{\Omega}\right)$$

$$\frac{R_n}{\Omega} = min\left((234\ \text{kip}), (397.31\ \text{kip})\right)$$

$$\frac{R_n}{\Omega} = 234\ \text{kip}$$

Result:
Demand over Capacity Ratio

$DCR = \frac{V_a}{\frac{R_n}{\Omega}} = \frac{(150\ \text{kip})}{(234\ \text{kip})} = 0.64103$

PASS

Check No. 4: Allowable Capacity of the Bolt Group in Bearing and Tear-out at Web Plate due to Axial Load

Calculate the bolt bearing capacity of the web plate.

$d_b = 0.75\ \text{in}$ - Bolt Diameter
$t_{wp} = 1\ \text{in}$ - Web Plate Thickness
$F_{up} = 65\ \text{ksi}$ - Web Plate Tensile Stress

AISC 360-16 Chapter J3.10 Eq. (J3-6a)	$C = 4$ - Bolt Group Coefficient $\Omega = 2$ - Bolt Bearing Safety Factor $\frac{R_{n\_bearing}}{\Omega}$ - Allowable Bolt Bearing Capacity of Section $$\frac{R_{n\_bearing}}{\Omega} = \frac{2.4\, d_b\, t_{wp}\, F_{up}\, C}{\Omega}$$ $$\frac{R_{n\_bearing}}{\Omega} = \frac{2.4 \times (0.75\ \text{in}) \times (1\ \text{in}) \times (65\ \text{ksi}) \times (4)}{(2)}$$ $$\frac{R_{n\_bearing}}{\Omega} = 234\ \text{kip}$$	
	Calculate the clear distance of outer bolts on web plate. $L_{eh,wp} = 2\ \text{in}$ - Web Plate Horizontal Edge Distance $d_h = 0.9375\ \text{in}$ - Horizontal Bolt Hole Dimension at Web Plate l_{c1} - Clear Distance at First Bolt Row $$l_{c1} = L_{eh,wp} - \frac{d_h}{2}$$ $$l_{c1} = (2\ \text{in}) - \frac{(0.9375\ \text{in})}{2}$$ $$l_{c1} = 1.5313\ \text{in}$$	
	Calculate the clear distance of inner bolts on web plate. $s_c = 3\ \text{in}$ - Bolt Column Spacing $d_h = 0.9375\ \text{in}$ - Horizontal Bolt Hole Dimension at Web Plate l_{c2} - Clear Distance at Rest of Bolts $$l_{c2} = s_c - d_h$$ $$l_{c2} = (3\ \text{in}) - (0.9375\ \text{in})$$ $$l_{c2} = 2.0625\ \text{in}$$	
AISC 360-16 Chapter J3.10 Eq. (J3-6c)	**Calculate the bolt tear-out capacity of the web plate.** $l_{c1} = 1.5313\ \text{in}$ - Clear Distance at First Bolt Row $l_{c2} = 2.0625\ \text{in}$ - Clear Distance at Rest of Bolts $t_{wp} = 1\ \text{in}$ - Web Plate Thickness $F_{up} = 65\ \text{ksi}$ - Web Plate Tensile Stress $C = 4$ - Bolt Group Coefficient $n_c = 2$ - Number of Bolt Columns $\Omega = 2$ - Bolt Bearing Safety Factor $\frac{R_{n\_tearout}}{\Omega}$ - Allowable Bolt Tear-out Capacity of Section $$\frac{R_{n\_tearout}}{\Omega} = \frac{[1.2\, l_{c1}\, t_{wp}\, F_{up} + 1.2\, l_{c2}\, t_{wp}\, F_{up}\, (n_c - 1)] \left(\frac{C}{n_c}\right)}{\Omega}$$ $$\frac{R_{n\_tearout}}{\Omega} = \frac{[1.2 \times (1.5313\ \text{in}) \times (1\ \text{in}) \times (65\ \text{ksi}) + 1.2 \times (2.0625\ \text{in}) \times (1\ \text{in}) \times (65\ \text{ksi}) \times ((2) - 1)] \times \left(\frac{(4)}{(2)}\right)}{(2)}$$ $$\frac{R_{n\_tearout}}{\Omega} = 280.31\ \text{kip}$$	
AISC 360-16 Chapter J3.10 Eq. (J3-6a) *AISC 360-16 Chapter J3.10 Eq. (J3-6c)* *AISC 360-16 Chapter J3.10*	**Determine the governing bearing and tear-out capacity of the bolt group on web plate.** $\frac{R_{n\_bearing}}{\Omega} = 234\ \text{kip}$ - Allowable bolt bearing capacity of web plate $\frac{R_{n\_tearout}}{\Omega} = 280.31\ \text{kip}$ - Allowable bolt tear-out capacity of web plate $\frac{R_n}{\Omega}$ - Governing Allowable Capacity $$\frac{R_n}{\Omega} = min\left(\frac{R_{n\_bearing}}{\Omega}, \frac{R_{n\_tearout}}{\Omega}\right)$$ $$\frac{R_n}{\Omega} = min\left((234\ \text{kip}), (280.31\ \text{kip})\right)$$ $$\frac{R_n}{\Omega} = 234\ \text{kip}$$ **Result:** Demand over Capacity Ratio $DCR = \frac{P_{web}}{\frac{R_n}{\Omega}} = \frac{(208.88)}{(234\ \text{kip})} = 0.000027744$	**PASS**
	Check No. 5: Shear and Axial Interaction of Bolt Bearing/Tearout at Web Plate **Calculate the shear and axial interaction of bolt bearing/tearout capacities in the web plate.** $V_a = 150\ \text{kip}$ - Shear Load $P_{web} = 208.88$ - Axial Load at Web $Interaction$ - Shear and Axial Interaction $$Interaction = \sqrt{\left(\frac{V_a}{\frac{R_n}{\Omega}}\right)^2 + \left(\frac{P_{web}}{\frac{R_n}{\Omega}}\right)^2}$$	

$$Interaction = \sqrt{\left(\frac{(150\ \text{kip})}{(234\ \text{kip})}\right)^2 + \left(\frac{(208.88)}{(234\ \text{kip})}\right)^2}$$

$$Interaction = 0.64103$$

Result:
Demand over Capacity Ratio
$DCR = \frac{Interaction}{1.0} = \frac{(0.64103)}{(1)} = 0.64103$

PASS

Check No. 6: Allowable Capacity of the Bolt Group in Bearing and Tear-out at Column Web due to Shear Load

Calculate the bolt bearing capacity of the column web.
$d_b = 0.75\ \text{in}$ - Bolt Diameter
$t_{w,uc} = 0.44\ \text{in}$ - Upper Column Web Thickness
$F_{u,uc} = 65\ \text{ksi}$ - Upper Column Tensile Stress
$C = 4$ - Bolt Group Coefficient
$\Omega = 2$ - Bolt Bearing Safety Factor

AISC 360-16 Chapter J3.10 Eq. (J3-6a)

$\frac{R_{n\_bearing}}{\Omega}$ - Allowable Bolt Bearing Capacity of Section

$$\frac{R_{n\_bearing}}{\Omega} = \frac{2.4\, d_b\, t_{w,uc}\, F_{u,uc}\, C}{\Omega}$$

$$\frac{R_{n\_bearing}}{\Omega} = \frac{2.4 \times (0.75\ \text{in}) \times (0.44\ \text{in}) \times (65\ \text{ksi}) \times (4)}{(2)}$$

$$\frac{R_{n\_bearing}}{\Omega} = 102.96\ \text{kip}$$

Calculate the clear distance of inner bolts on column web.
$s_r = 3\ \text{in}$ - Bolt Row Spacing
$d_h = 0.8125\ \text{in}$ - Vertical Bolt Hole Dimension at Column Web
l_{c2} - Clear Distance at Rest of Bolts

$$l_{c2} = s_r - d_h$$

$$l_{c2} = (3\ \text{in}) - (0.8125\ \text{in})$$

$$l_{c2} = 2.1875\ \text{in}$$

Calculate the bolt tear-out capacity of the column web.
$l_{c2} = 2.1875\ \text{in}$ - Clear Distance at Rest of Bolts
$t_{w,uc} = 0.44\ \text{in}$ - Upper Column Web Thickness
$F_{u,uc} = 65\ \text{ksi}$ - Upper Column Tensile Stress
$C = 4$ - Bolt Group Coefficient
$\Omega = 2$ - Bolt Bearing Safety Factor

AISC 360-16 Chapter J3.10 Eq. (J3-6c)

$\frac{R_{n\_tearout}}{\Omega}$ - Allowable Bolt Tear-out Capacity of Section

$$\frac{R_{n\_tearout}}{\Omega} = \frac{1.2\, l_{c2}\, t_{w,uc}\, F_{u,uc}\, C}{\Omega}$$

$$\frac{R_{n\_tearout}}{\Omega} = \frac{1.2 \times (2.1875\ \text{in}) \times (0.44\ \text{in}) \times (65\ \text{ksi}) \times (4)}{(2)}$$

$$\frac{R_{n\_tearout}}{\Omega} = 150.15\ \text{kip}$$

Determine the governing bearing and tear-out capacity of the bolt group on column web.

AISC 360-16 Chapter J3.10 Eq. (J3-6a)
$\frac{R_{n\_bearing}}{\Omega} = 102.96\ \text{kip}$ - Allowable bolt bearing capacity of column web

AISC 360-16 Chapter J3.10 Eq. (J3-6c)
$\frac{R_{n\_tearout}}{\Omega} = 150.15\ \text{kip}$ - Allowable bolt tear-out capacity of column web

AISC 360-16 Chapter J3.10
$\frac{R_n}{\Omega}$ - Governing Allowable Capacity

$$\frac{R_n}{\Omega} = min\left(\frac{R_{n\_bearing}}{\Omega}, \frac{R_{n\_tearout}}{\Omega}\right)$$

$$\frac{R_n}{\Omega} = min\left((102.96\ \text{kip}), (150.15\ \text{kip})\right)$$

$$\frac{R_n}{\Omega} = 102.96\ \text{kip}$$

Result:
Demand over Capacity Ratio
$DCR = \frac{V_a}{\frac{R_n}{\Omega}} = \frac{(150\ \text{kip})}{(102.96\ \text{kip})} = 1.4569$

FAIL

Check No. 7: Allowable Capacity of the Bolt Group in Bearing and Tear-out at Column Web due to Axial Load

Calculate the bolt bearing capacity of the column web.
$d_b = 0.75\ \text{in}$ - Bolt Diameter
$t_{w,uc} = 0.44\ \text{in}$ - Upper Column Web Thickness
$F_{u,uc} = 65\ \text{ksi}$ - Upper Column Tensile Stress
$C = 4$ - Bolt Group Coefficient
$\Omega = 2$ - Bolt Bearing Safety Factor

AISC 360-16 Chapter J3.10 Eq. (J3-6a)

$\frac{R_{n\_bearing}}{\Omega}$ - Allowable Bolt Bearing Capacity of Section

$$\frac{R_{n\_bearing}}{\Omega} = \frac{2.4\, d_b\, t_{w,uc}\, F_{u,uc}\, C}{\Omega}$$

$$\frac{R_{n\_bearing}}{\Omega} = \frac{2.4 \times (0.75 \text{ in}) \times (0.44 \text{ in}) \times (65 \text{ ksi}) \times (4)}{(2)}$$

$$\frac{R_{n\_bearing}}{\Omega} = 102.96 \text{ kip}$$

Calculate the clear distance of outer bolts on column web.

$L_{eh,c} = 2 \text{ in}$ - Column Web Horizontal Edge Distance

$d_h = 0.8125 \text{ in}$ - Horizontal Bolt Hole Dimension at Column Web

l_{c1} - Clear Distance at First Bolt Row

$$l_{c1} = L_{eh,c} - \frac{d_h}{2}$$

$$l_{c1} = (2 \text{ in}) - \frac{(0.8125 \text{ in})}{2}$$

$$l_{c1} = 1.5938 \text{ in}$$

Calculate the clear distance of inner bolts on column web.

$s_c = 3 \text{ in}$ - Bolt Column Spacing

$d_h = 0.8125 \text{ in}$ - Horizontal Bolt Hole Dimension at Column Web

l_{c2} - Clear Distance at Rest of Bolts

$$l_{c2} = s_c - d_h$$

$$l_{c2} = (3 \text{ in}) - (0.8125 \text{ in})$$

$$l_{c2} = 2.1875 \text{ in}$$

Calculate the bolt tear-out capacity of the column web.

$l_{c1} = 1.5938 \text{ in}$ - Clear Distance at First Bolt Row

$l_{c2} = 2.1875 \text{ in}$ - Clear Distance at Rest of Bolts

$t_{w,uc} = 0.44 \text{ in}$ - Upper Column Web Thickness

$F_{u,uc} = 65 \text{ ksi}$ - Upper Column Tensile Stress

$C = 4$ - Bolt Group Coefficient

$n_c = 2$ - Number of Bolt Columns

$\Omega = 2$ - Bolt Bearing Safety Factor

AISC 360-16 Chapter J3.10 Eq. (J3-6c) $\frac{R_{n\_tearout}}{\Omega}$ - Allowable Bolt Tear-out Capacity of Section

$$\frac{R_{n\_tearout}}{\Omega} = \frac{\left[1.2\, l_{c1}\, t_{w,uc}\, F_{u,uc} + 1.2\, l_{c2}\, t_{w,uc}\, F_{u,uc}\, (n_c - 1)\right] \left(\frac{C}{n_c}\right)}{\Omega}$$

$$\frac{R_{n\_tearout}}{\Omega} = \frac{[1.2 \times (1.5938 \text{ in}) \times (0.44 \text{ in}) \times (65 \text{ ksi}) + 1.2 \times (2.1875 \text{ in}) \times (0.44 \text{ in}) \times (65 \text{ ksi}) \times ((2) - 1)] \times \left(\frac{(4)}{(2)}\right)}{(2)}$$

$$\frac{R_{n\_tearout}}{\Omega} = 129.77 \text{ kip}$$

Determine the governing bearing and tear-out capacity of the bolt group on column web.

AISC 360-16 Chapter J3.10 Eq. (J3-6a) $\frac{R_{n\_bearing}}{\Omega} = 102.96 \text{ kip}$ - Allowable bolt bearing capacity of column web

AISC 360-16 Chapter J3.10 Eq. (J3-6c) $\frac{R_{n\_tearout}}{\Omega} = 129.77 \text{ kip}$ - Allowable bolt tear-out capacity of column web

AISC 360-16 Chapter J3.10 $\frac{R_n}{\Omega}$ - Governing Allowable Capacity

$$\frac{R_n}{\Omega} = min\left(\frac{R_{n\_bearing}}{\Omega}, \frac{R_{n\_tearout}}{\Omega}\right)$$

$$\frac{R_n}{\Omega} = min\left((102.96 \text{ kip}), (129.77 \text{ kip})\right)$$

$$\frac{R_n}{\Omega} = 102.96 \text{ kip}$$

Result: **FAIL**

Demand over Capacity Ratio

$DCR = \frac{P_{web}}{\frac{R_n}{\Omega}} = \frac{(208.88)}{(102.96 \text{ kip})} = 0.000063054$

Check No. 8: Shear and Axial Interaction of Bolt Bearing/Tearout at Column Web

Calculate the shear and axial interaction of bolt bearing/tearout capacities in the column web.

$V_a = 150 \text{ kip}$ - Shear Load

$P_{web} = 208.88$ - Axial Load at Web

$Interaction$ - Shear and Axial Interaction

$$Interaction = \sqrt{\left(\frac{V_a}{\frac{R_n}{\Omega}}\right)^2 + \left(\frac{P_{web}}{\frac{R_n}{\Omega}}\right)^2}$$

$$Interaction = \sqrt{\left(\frac{(150 \text{ kip})}{(102.96 \text{ kip})}\right)^2 + \left(\frac{(208.88)}{(102.96 \text{ kip})}\right)^2}$$

$$Interaction = 1.4569$$

Result: **FAIL**

Demand over Capacity Ratio

$DCR = \frac{Interaction}{1.0} = \frac{(1.4569)}{(1)} = 1.4569$

Check No. 9: Allowable Capacity of Web Plate in Block Shear due to Shear Load

Calculate the net area of the web plate subject to tension.

$t_{wp} = 1$ in - Web Plate Thickness

$n_c = 2$ - Number of Bolt Columns

$s_c = 3$ in - Bolt Column Spacing

$L_{eh,wp} = 2$ in - Web Plate Horizontal Edge Distance

$d_h = 0.9375$ in - Horizontal Bolt Hole Dimension at Web Plate

A_{nt} - Net Area Subject to Tension (L-pattern)

$$A_{nt} = t_{wp}\ [(n_c - 1)\ s_c + L_{eh,wp} - (n_c - 0.5)\ (d_h + 0.0625\ \text{in})]$$

$$A_{nt} = (1\ \text{in}) \times [((2) - 1) \times (3\ \text{in}) + (2\ \text{in}) - ((2) - 0.5) \times ((0.9375\ \text{in}) + (0.0625\ \text{in}))]$$

$$A_{nt} = 3.5\ \text{in}^2$$

Calculate the gross area of the web plate subject to shear.

$t_{wp} = 1$ in - Web Plate Thickness

$L_{ev,wp} = 3.5$ in - Web Plate Vertical Edge Distance

$n_r = 2$ - Number of Bolt Rows

$s_r = 3$ in - Bolt Row Spacing

A_{gv} - Gross Area Subject to Shear (L-pattern)

$$A_{gv} = t_{wp}\ [L_{ev,wp} + (n_r - 1)\ s_r]$$

$$A_{gv} = (1\ \text{in}) \times [(3.5\ \text{in}) + ((2) - 1) \times (3\ \text{in})]$$

$$A_{gv} = 6.5\ \text{in}^2$$

Calculate the net area of the web plate subject to shear.

$t_{wp} = 1$ in - Web Plate Thickness

$b_{wp} = 7$ in - Web Plate Width

$L_{ev,wp} = 3.5$ in - Web Plate Vertical Edge Distance

$n_r = 2$ - Number of Bolt Rows

$d_h = 0.9375$ in - Vertical Bolt Hole Dimension at Web Plate

A_{nv} - Net Area Subject to Shear (L-pattern)

$$A_{nv} = t_{wp}\ [b_{wp} - L_{ev,wp} - (n_r - 0.5)\ (d_h + 0.0625\ \text{in})]$$

$$A_{nv} = (1\ \text{in}) \times [(7\ \text{in}) - (3.5\ \text{in}) - ((2) - 0.5) \times ((0.9375\ \text{in}) + (0.0625\ \text{in}))]$$

$$A_{nv} = 2\ \text{in}^2$$

Calculate the allowable block shear capacity of the web plate.

$F_{up} = 65$ ksi - Web Plate Tensile Stress

$A_{nv} = 2\ \text{in}^2$ - Net Area Subject to Shear (L-pattern)

$U_{bs} = 0.5$ - Uniformity factor

$A_{nt} = 3.5\ \text{in}^2$ - Net Area Subject to Tension (L-pattern)

$F_{yp} = 50$ ksi - Web Plate Yield Stress

$A_{gv} = 6.5\ \text{in}^2$ - Gross Area Subject to Shear (L-pattern)

$\Omega = 2$ - Block Shear Safety Factor

AISC 360-16 Chapter J4.3 Eq. (J4-5)

$\frac{R_n}{\Omega}$ - Allowable Block Shear Capacity of Section

$$\frac{R_n}{\Omega} = \frac{0.6\ F_{up}\ A_{nv} + U_{bs}\ F_{up}\ A_{nt} \le 0.6\ F_{yp}\ A_{gv} + U_{bs}\ F_{up}\ A_{nt}}{\Omega}$$

$$\frac{R_n}{\Omega} = \frac{0.6 \times (65\ \text{ksi}) \times \left(2\ \text{in}^2\right) + (0.5) \times (65\ \text{ksi}) \times \left(3.5\ \text{in}^2\right) \le 0.6 \times (50\ \text{ksi}) \times \left(6.5\ \text{in}^2\right) + (0.5) \times (65\ \text{ksi}) \times \left(3.5\ \text{in}^2\right)}{(2)}$$

$$\frac{R_n}{\Omega} = 95.875\ \text{kip}$$

Result:

Demand over Capacity Ratio

$DCR = \frac{V_a}{\frac{R_n}{\Omega}} = \frac{(150\ \text{kip})}{(95.875\ \text{kip})} = 1.5645$

FAIL

Check No. 10: Allowable Capacity of Web Plate in Block Shear due to Axial Load

Calculate the net area of the web plate subject to tension.

$t_{wp} = 1$ in - Web Plate Thickness

$n_r = 2$ - Number of Bolt Rows

$s_r = 3$ in - Bolt Row Spacing

$L_{ev,wp} = 3.5$ in - Web Plate Vertical Edge Distance

$d_h = 0.9375$ in - Vertical Bolt Hole Dimension at Web Plate

A_{nt} - Net Area Subject to Tension (L-pattern)

$$A_{nt} = t_{wp}\ [(n_r - 1)\ s_r + L_{ev,wp} - (n_r - 0.5)\ (d_h + 0.0625\ \text{in})]$$

$$A_{nt} = (1\ \text{in}) \times [((2) - 1) \times (3\ \text{in}) + (3.5\ \text{in}) - ((2) - 0.5) \times ((0.9375\ \text{in}) + (0.0625\ \text{in}))]$$

$$A_{nt} = 5\ \text{in}^2$$

Calculate the gross area of the web plate subject to shear.

$t_{wp} = 1$ in - Web Plate Thickness

$L_{eh,wp} = 2$ in - Web Plate Horizontal Edge Distance

$n_c = 2$ - Number of Bolt Columns
$s_c = 3$ in - Bolt Column Spacing
A_{gv} - Gross Area Subject to Shear (L-pattern)

$$A_{gv} = t_{wp}\ [L_{eh,wp} + (n_c - 1)\ s_c]$$

$$A_{gv} = (1 \text{ in}) \times [(2 \text{ in}) + ((2) - 1) \times (3 \text{ in})]$$

$$A_{gv} = 5 \text{ in}^2$$

Calculate the net area of the web plate subject to shear.
$t_{wp} = 1$ in - Web Plate Thickness
$L_{wp} = 7$ in - Web Plate Length
$L_{eh,c} = 2$ in - Column Web Horizontal Edge Distance
$n_c = 2$ - Number of Bolt Columns
$d_h = 0.9375$ in - Horizontal Bolt Hole Dimension at Web Plate
A_{nv} - Net Area Subject to Shear (L-pattern)

$$A_{nv} = t_{wp}\ [L_{wp} - L_{eh,c} - (n_c - 0.5)\ (d_h + 0.0625 \text{ in})]$$

$$A_{nv} = (1 \text{ in}) \times [(7 \text{ in}) - (2 \text{ in}) - ((2) - 0.5) \times ((0.9375 \text{ in}) + (0.0625 \text{ in}))]$$

$$A_{nv} = 3.5 \text{ in}^2$$

Calculate the allowable block shear capacity of the web plate.
$F_{up} = 65$ ksi - Web Plate Tensile Stress
$A_{nv} = 3.5 \text{ in}^2$ - Net Area Subject to Shear (L-pattern)
$U_{bs} = 1$ - Uniformity factor
$A_{nt} = 5 \text{ in}^2$ - Net Area Subject to Tension (L-pattern)
$F_{yp} = 50$ ksi - Web Plate Yield Stress
$A_{gv} = 5 \text{ in}^2$ - Gross Area Subject to Shear (L-pattern)
$\Omega = 2$ - Block Shear Safety Factor

AISC 360-16 Chapter J4.3 Eq. (J4-5)

$\frac{R_n}{\Omega}$ - Allowable Block Shear Capacity of Section

$$\frac{R_n}{\Omega} = \frac{0.6\ F_{up}\ A_{nv} + U_{bs}\ F_{up}\ A_{nt} \leq 0.6\ F_{yp}\ A_{gv} + U_{bs}\ F_{up}\ A_{nt}}{\Omega}$$

$$\frac{R_n}{\Omega} = \frac{0.6 \times (65 \text{ ksi}) \times \left(3.5 \text{ in}^2\right) + (1) \times (65 \text{ ksi}) \times \left(5 \text{ in}^2\right) \leq 0.6 \times (50 \text{ ksi}) \times \left(5 \text{ in}^2\right) + (1) \times (65 \text{ ksi}) \times \left(5 \text{ in}^2\right)}{(2)}$$

$$\frac{R_n}{\Omega} = 230.75 \text{ kip}$$

Result:
Demand over Capacity Ratio

$DCR = \frac{T_{web}}{\frac{R_n}{\Omega}} = \frac{(208.88 \text{ kip})}{(230.75 \text{ kip})} = 0.9052$

PASS

Check No. 11: Shear and Axial Interaction of Block Shear in Web Plate

Calculate the shear and axial interaction of block shear capacities in the web plate.
$V_a = 150$ kip - Shear Load
$T_{web} = 208.88$ kip - Tension load at web connection. This is proportioned based on web area over the total gross area.
$Interaction$ - Shear and Axial Interaction

$$Interaction = \sqrt{\left(\frac{V_a}{\frac{R_n}{\Omega}}\right)^2 + \left(\frac{T_{web}}{\frac{R_n}{\Omega}}\right)^2}$$

$$Interaction = \sqrt{\left(\frac{(150 \text{ kip})}{(95.875 \text{ kip})}\right)^2 + \left(\frac{(208.88 \text{ kip})}{(230.75 \text{ kip})}\right)^2}$$

$$Interaction = 1.8075$$

Result:
Demand over Capacity Ratio

$DCR = \frac{Interaction}{1.0} = \frac{(1.8075)}{(1)} = 1.8075$

FAIL

Check No. 12: Allowable Capacity of Column Web in Block Shear due to Axial Load

Calculate the net area of the column web subject to tension.
$t_{w,uc} = 0.44$ in - Upper Column Web Thickness
$n_r = 2$ - Number of Bolt Rows
$s_r = 3$ in - Bolt Row Spacing
$d_h = 0.8125$ in - Vertical Bolt Hole Dimension at Column Web
A_{nt} - Net Area Subject to Tension (C-pattern)

$$A_{nt} = t_{w,uc}\ [(n_r - 1)\ s_r - (n_r - 1)\ (d_h + 0.0625 \text{ in})]$$

$$A_{nt} = (0.44 \text{ in}) \times [((2) - 1) \times (3 \text{ in}) - ((2) - 1) \times ((0.8125 \text{ in}) + (0.0625 \text{ in}))]$$

$$A_{nt} = 0.935 \text{ in}^2$$

Calculate the gross area of the column web subject to shear.
$t_{w,uc} = 0.44$ in - Upper Column Web Thickness
$L_{eh,c} = 2$ in - Column Web Horizontal Edge Distance

	$n_c = 2$ - Number of Bolt Columns $s_c = 3\ \text{in}$ - Bolt Column Spacing A_{gv} - Gross Area Subject to Shear (C-pattern) $$A_{gv} = 2\ t_{w,uc}\ \left[L_{eh,c} + (n_c - 1)\ s_c\right]$$ $$A_{gv} = 2 \times (0.44\ \text{in}) \times \left[(2\ \text{in}) + ((2) - 1) \times (3\ \text{in})\right]$$ $$A_{gv} = 4.4\ \text{in}^2$$ **Calculate the net area of the column web subject to shear.** $t_{w,uc} = 0.44\ \text{in}$ - Upper Column Web Thickness $L_{eh,c} = 2\ \text{in}$ - Column Web Horizontal Edge Distance $n_c = 2$ - Number of Bolt Columns $s_c = 3\ \text{in}$ - Bolt Column Spacing $d_h = 0.8125\ \text{in}$ - Horizontal Bolt Hole Dimension at Column Web A_{nv} - Net Area Subject to Shear (C-pattern) $$A_{nv} = 2\ t_{w,uc}\ \left[(n_c - 1)\ s_c + L_{eh,c} - (n_c - 0.5)\ (d_h + 0.0625\ \text{in})\right]$$ $$A_{nv} = 2 \times (0.44\ \text{in}) \times \left[((2) - 1) \times (3\ \text{in}) + (2\ \text{in}) - ((2) - 0.5) \times ((0.8125\ \text{in}) + (0.0625\ \text{in}))\right]$$ $$A_{nv} = 3.245\ \text{in}^2$$ **Calculate the allowable block shear capacity of the column web.** $F_{u,uc} = 65\ \text{ksi}$ - Upper Column Tensile Stress $A_{nt} = 0.935\ \text{in}^2$ - Net Area Subject to Tension (C-pattern) $U_{bs} = 1$ - Uniformity factor $A_{gv} = 4.4\ \text{in}^2$ - Gross Area Subject to Shear (C-pattern) $F_{y,uc} = 50\ \text{ksi}$ - Upper Column Yield Stress $A_{nv} = 3.245\ \text{in}^2$ - Net Area Subject to Shear (C-pattern) $\Omega = 2$ - Block Shear Safety Factor	
AISC 360-16 Chapter J4.3 Eq. (J4-5)	$\frac{R_n}{\Omega}$ - Allowable Block Shear Capacity of Section $$\frac{R_n}{\Omega} = \frac{0.6\ F_{u,uc}\ A_{nv} + U_{bs}\ F_{u,uc}\ A_{nt} \leq 0.6\ F_{y,uc}\ A_{gv} + U_{bs}\ F_{u,uc}\ A_{nt}}{\Omega}$$ $$\frac{R_n}{\Omega} = \frac{0.6 \times (65\ \text{ksi}) \times \left(3.245\ \text{in}^2\right) + (1) \times (65\ \text{ksi}) \times \left(0.935\ \text{in}^2\right) \leq 0.6 \times (50\ \text{ksi}) \times \left(4.4\ \text{in}^2\right) + (1) \times (65\ \text{ksi}) \times \left(0.935\ \text{in}^2\right)}{(2)}$$ $$\frac{R_n}{\Omega} = 93.665\ \text{kip}$$ **Result:** Demand over Capacity Ratio $DCR = \frac{T_{web}}{\frac{R_n}{\Omega}} = \frac{(208.88\ \text{kip})}{(93.665\ \text{kip})} = 2.23$	**FAIL**
	Check No. 13: Allowable Capacity of Web Plate in Shear **Calculate the gross area of web plate subject to yielding.** $b_{wp} = 7\ \text{in}$ - Web Plate Width $t_{wp} = 1\ \text{in}$ - Web Plate Thickness A_{gv} - Section Gross Area $$A_{gv} = b_{wp}\ t_{wp}$$ $$A_{gv} = (7\ \text{in}) \times (1\ \text{in})$$ $$A_{gv} = 7\ \text{in}^2$$ **Calculate the shear yielding capacity of the web plate.** $F_{yp} = 50\ \text{ksi}$ - Web Plate Yield Stress $A_{gv} = 7\ \text{in}^2$ - Section Gross Area $\Omega = 1.5$ - Shear Yielding Safety Factor	
AISC 360-16 Chapter J4.2 Eq. (J4-3)	$\frac{R_{n\_sy}}{\Omega}$ - Allowable Shear Yielding Capacity of Section $$\frac{R_{n\_sy}}{\Omega} = \frac{0.6\ F_{yp}\ A_{gv}}{\Omega}$$ $$\frac{R_{n\_sy}}{\Omega} = \frac{0.6 \times (50\ \text{ksi}) \times \left(7\ \text{in}^2\right)}{(1.5)}$$ $$\frac{R_{n\_sy}}{\Omega} = 140\ \text{kip}$$ **Calculate the net area of web plate subject to rupture.** $t_{wp} = 1\ \text{in}$ - Web Plate Thickness $b_{wp} = 7\ \text{in}$ - Web Plate Width $n_r = 2$ - Number of Bolt Rows $d_h = 0.9375\ \text{in}$ - Vertical Bolt Hole Dimension at Web Plate A_{nv} - Section Net Area $$A_{nv} = t_{wp}\ \left[b_{wp} - n_r\ (d_h + 0.0625\ \text{in})\right]$$ $$A_{nv} = (1\ \text{in}) \times \left[(7\ \text{in}) - (2) \times ((0.9375\ \text{in}) + (0.0625\ \text{in}))\right]$$	

$$A_{nv} = 5 \text{ in}^2$$

Calculate the shear rupture capacity of the web plate.

$F_{up} = 65 \text{ ksi}$ - Web Plate Tensile Stress

$A_{nv} = 5 \text{ in}^2$ - Section Net Area

$\Omega = 1.5$ - Shear Yielding Safety Factor

AISC 360-16 Chapter J4.2 Eq. (J4-4)

$\frac{R_{n\_sr}}{\Omega}$ - Allowable Shear Rupture Capacity of Section

$$\frac{R_{n\_sr}}{\Omega} = \frac{0.6\, F_{up}\, A_{nv}}{\Omega}$$

$$\frac{R_{n\_sr}}{\Omega} = \frac{0.6 \times (65 \text{ ksi}) \times \left(5 \text{ in}^2\right)}{(1.5)}$$

$$\frac{R_{n\_sr}}{\Omega} = 130 \text{ kip}$$

Determine the governing shear capacity of the web plate.

AISC 360-16 Chapter J4.2 Eq. (J4-3)

$\frac{R_{n\_sy}}{\Omega} = 140 \text{ kip}$ - Allowable shear yielding capacity of web plate

AISC 360-16 Chapter J4.2 Eq. (J4-4)

$\frac{R_{n\_sr}}{\Omega} = 130 \text{ kip}$ - Allowable shear rupture capacity of web plate

AISC 360-16 Chapter J4.2

$\frac{R_n}{\Omega}$ - Governing Allowable Capacity

$$\frac{R_n}{\Omega} = min\left(\frac{R_{n\_sy}}{\Omega}, \frac{R_{n\_sr}}{\Omega}\right)$$

$$\frac{R_n}{\Omega} = min\left((140 \text{ kip}), (130 \text{ kip})\right)$$

$$\frac{R_n}{\Omega} = 130 \text{ kip}$$

Result:

Demand over Capacity Ratio

$DCR = \frac{V_a}{\frac{R_n}{\Omega}} = \frac{(150 \text{ kip})}{(130 \text{ kip})} = 1.1538$

FAIL

Check No. 14: Allowable Capacity of Web Plate in Tension

Calculate the tensile yielding capacity of the web plate.

$\Omega = 1.67$ - Tensile Yielding Safety Factor

$F_{yp} = 50 \text{ ksi}$ - Web Plate Yield Stress

$t_{wp} = 1 \text{ in}$ - Web Plate Thickness

$b_{wp} = 7 \text{ in}$ - Web Plate Width

AISC 360-16 Chapter J4.1 Eq. (J4-1)

$\frac{R_{n\_ty}}{\Omega}$ - Allowable Tension Yielding Capacity of Section

$$\frac{R_{n\_ty}}{\Omega} = \frac{F_{yp}\, t_{wp}\, b_{wp}}{\Omega}$$

$$\frac{R_{n\_ty}}{\Omega} = \frac{(50 \text{ ksi}) \times (1 \text{ in}) \times (7 \text{ in})}{(1.67)}$$

$$\frac{R_{n\_ty}}{\Omega} = 209.58 \text{ kip}$$

Calculate the tensile rupture capacity of the web plate.

$\Omega = 2$ - Tensile Rupture Safety Factor

$F_{up} = 65 \text{ ksi}$ - Web Plate Tensile Stress

$t_{wp} = 1 \text{ in}$ - Web Plate Thickness

$b_{wp} = 7 \text{ in}$ - Web Plate Width

$n_r = 2$ - Number of Bolt Rows

$d_h = 0.9375 \text{ in}$ - Vertical Bolt Hole Dimension at Web Plate

AISC 360-16 Chapter J4.1 Eq. (J4-2)

$\frac{R_{n\_tr}}{\Omega}$ - Allowable Tension Rupture Capacity of Section

$$\frac{R_{n\_tr}}{\Omega} = \frac{F_{up}\, t_{wp}\, \left[b_{wp} - n_r\, \left(d_h + 0.0625 \text{ in}\right)\right]}{\Omega}$$

$$\frac{R_{n\_tr}}{\Omega} = \frac{(65 \text{ ksi}) \times (1 \text{ in}) \times [(7 \text{ in}) - (2) \times ((0.9375 \text{ in}) + (0.0625 \text{ in}))]}{(2)}$$

$$\frac{R_{n\_tr}}{\Omega} = 162.5 \text{ kip}$$

Determine the governing tensile capacity of the web plate.

AISC 360-16 Chapter J4.1 Eq. (J4-1)

$\frac{R_{n\_ty}}{\Omega} = 209.58 \text{ kip}$ - Allowable Tension Yielding Capacity of Section

AISC 360-16 Chapter J4.1 Eq. (J4-2)

$\frac{R_{n\_tr}}{\Omega} = 162.5 \text{ kip}$ - Allowable Tension Rupture Capacity of Section

AISC 360-16 Chapter J4.1

$\frac{R_n}{\Omega}$ - Governing Allowable Capacity

$$\frac{R_n}{\Omega} = min\left(\frac{R_{n\_ty}}{\Omega}, \frac{R_{n\_tr}}{\Omega}\right)$$

$$\frac{R_n}{\Omega} = min\left((209.58 \text{ kip}), (162.5 \text{ kip})\right)$$

$$\frac{R_n}{\Omega} = 162.5 \text{ kip}$$

	Result: Demand over Capacity Ratio $DCR = \frac{T_{web}}{\frac{R_n}{\Omega}} = \frac{(208.88\text{ kip})}{(162.5\text{ kip})} = 1.2854$	**FAIL**
AISC 360-16 Chapter D3 (Table D3.1 case 2) *AISC 360-16 Chapter J4.1 Eq. (J4-2)*	**Check No. 15: Allowable Capacity of Beam in Tension** **Calculate the tensile rupture capacity of the beam.** $\Omega = 2$ - Tensile Rupture Safety Factor $F_{u,uc} = 65\text{ ksi}$ - Upper Column Tensile Stress $A_{uc} = 26.5\text{ in}^2$ - Upper Column Area $t_{w,uc} = 0.44\text{ in}$ - Upper Column Web Thickness $d_h = 0.8125\text{ in}$ - Vertical Bolt Hole Dimension at Column Web $n_r = 2$ - Number of Bolt Rows $n_c = 2$ - Number of Bolt Columns $s_c = 3\text{ in}$ - Bolt Column Spacing $\bar{x} = 3.1003\text{ in}$ - Centroid of WF section $U = 0.175$ - Shear Lag Factor $\frac{R_{n\_tr}}{\Omega}$ - Allowable Tension Rupture Capacity of Section $$\frac{R_{n\_tr}}{\Omega} = \frac{F_{u,uc}\, U\, [A_{uc} - n_r\, (d_h + 0.0625\text{ in})\, t_{w,uc}]}{\Omega}$$ $$\frac{R_{n\_tr}}{\Omega} = \frac{(65\text{ ksi}) \times (0.175) \times \left[(26.5\text{ in}^2) - (2) \times ((0.8125\text{ in}) + (0.0625\text{ in})) \times (0.44\text{ in})\right]}{(2)}$$ $$\frac{R_{n\_tr}}{\Omega} = 146.34\text{ kip}$$ **Result:** Demand over Capacity Ratio $DCR = \frac{T_{web}}{\frac{R_{n\_tr}}{\Omega}} = \frac{(208.88\text{ kip})}{(146.34\text{ kip})} = 1.4273$	**FAIL**
AISC 360-16 Chapter J4.4 Eq. (J4-6)	**Check No. 16: Allowable Capacity of Web Plate in Compression** **Calculate the compression buckling capacity of the web plate.** $\Omega = 1.67$ - Compression Safety Factor $F_{yp} = 50\text{ ksi}$ - Web Plate Yield Stress $t_{wp} = 1\text{ in}$ - Web Plate Thickness $b_{wp} = 7\text{ in}$ - Web Plate Width $K = 1.2$ - Effective Length Factor $L_b = 2\text{ in}$ - Web Plate Unbraced Length $\frac{KL}{r} = 8.3138$ - Effective Length Slenderness Ratio Since, $\frac{KL}{r} \le 25.$ $\frac{R_n}{\Omega}$ - Allowable Compressive Capacity of Section $$\frac{R_n}{\Omega} = \frac{F_{yp}\, t_{wp}\, b_{wp}}{\Omega}$$ $$\frac{R_n}{\Omega} = \frac{(50\text{ ksi}) \times (1\text{ in}) \times (7\text{ in})}{(1.67)}$$ $$\frac{R_n}{\Omega} = 209.58\text{ kip}$$ **Result:** Demand over Capacity Ratio $DCR = \frac{C_{web}}{\frac{R_n}{\Omega}} = \frac{(67.752\text{ kip})}{(209.58\text{ kip})} = 0.32327$	**PASS**

REFERENCES	CALCULATIONS	RESULTS

Summary of Checks

Design Checks	Demand	Capacity	DCR	Result
Connection Detailing Limitations	5.000	7.000	0.714	PASS
Allowable Capacity of the Bolt Group in Shear	257.156	25.312	10.159	FAIL
Allowable Capacity of the Bolt Group in Bearing and Tear-out at Web Plate due to Shear Load	150.000	234.000	0.641	PASS
Allowable Capacity of the Bolt Group in Bearing and Tear-out at Web Plate due to Axial Load	208.875	234.000	0.893	PASS
Shear and Axial Interaction of Bolt Bearing/Tearout at Web Plate	0.641	1.000	0.641	PASS
Allowable Capacity of the Bolt Group in Bearing and Tear-out at Column Web due to Shear Load	150.000	102.960	1.457	FAIL
Allowable Capacity of the Bolt Group in Bearing and Tear-out at Column Web due to Axial Load	208.875	102.960	2.029	FAIL
Shear and Axial Interaction of Bolt Bearing/Tearout at Column Web	1.457	1.000	1.457	FAIL
Allowable Capacity of Web Plate in Block Shear due to Shear Load	150.000	95.875	1.565	FAIL
Allowable Capacity of Web Plate in Block Shear due to Axial Load	208.875	230.750	0.905	PASS
Shear and Axial Interaction of Block Shear in Web Plate	1.808	1.000	1.808	FAIL
Allowable Capacity of Column Web in Block Shear due to Axial Load	208.875	93.665	2.230	FAIL
Allowable Capacity of Web Plate in Shear	150.000	130.000	1.154	FAIL
Allowable Capacity of Web Plate in Tension	208.875	162.500	1.285	FAIL
Allowable Capacity of Beam in Tension	208.875	146.339	1.427	FAIL
Allowable Capacity of Web Plate in Compression	67.752	209.581	0.323	PASS