REFERENCES	CALCULATIONS	RESULTS
	Beam to Column Web Moment Connection Calculations	
	Design Load/s: $V_u = 400\ \text{kip}$ - Vertical Shear Load $P_u = 250\ \text{kip}$ - Axial Load $M_{ux} = 500\ \text{kipft}$ - Strong-axis Moment Load	
	Beam Section Properties: W27x194 - Beam Size $d_{bm} = 28.1\ \text{in}$ - Beam Depth $t_{wb} = 0.75\ \text{in}$ - Beam Web Thickness $b_{fb} = 14\ \text{in}$ - Beam Flange Width $t_{fb} = 1.34\ \text{in}$ - Beam Flange Thickness $A_{bm} = 57.1\ \text{in}^2$ - Beam Area **Beam Grade Information:** A992 - Material Grade $F_{yb} = 50\ \text{ksi}$ - Beam Yield Stress $F_{ub} = 65\ \text{ksi}$ - Beam Tensile Stress $E = 29000\ \text{ksi}$ - Beam Modulus of Elasticity	
	Column Section Properties: W14x90 - Column Size $d_{sup} = 14\ \text{in}$ - Column Depth $t_{ws} = 0.44\ \text{in}$ - Column Web Thickness $b_{fs} = 14.5\ \text{in}$ - Column Flange Width $t_{fs} = 0.71\ \text{in}$ - Column Flange Thickness $A_{sup} = 26.5\ \text{in}^2$ - Column Area **Column Grade Information:** A992 - Material Grade $F_{ys} = 50\ \text{ksi}$ - Column Yield Stress $F_{us} = 65\ \text{ksi}$ - Column Tensile Stress $E = 29000\ \text{ksi}$ - Column Modulus of Elasticity	
	Bolt Information at Flange Connection (Beam Side): $1\ \text{in}$ - Bolt Size A490-SC(B) - Bolting Category $d_b = 1\ \text{in}$ - Bolt Diameter $F_{nt} = 113\ \text{ksi}$ - Bolt Nominal Tensile Strength $F_{nv} = 68\ \text{ksi}$ - Bolt Nominal Shear Strength $N_s = 1$ - Number of Slip Planes OVS - Bolt Hole Type at Plate STD - Bolt Hole Type at Beam **Bolt Information at Web Connection (Beam Side):** $1\ \text{in}$ - Bolt Size A490N - Bolting Category $d_b = 1\ \text{in}$ - Bolt Diameter $F_{nt} = 113\ \text{ksi}$ - Bolt Nominal Tensile Strength $F_{nv} = 68\ \text{ksi}$ - Bolt Nominal Shear Strength $N_s = 1$ - Number of Shear Planes HSSL - Bolt Hole Type at Plate STD - Bolt Hole Type at Beam	
	Weld Information at Flange Connection (Column Side): E70XX - Weld Classification $W = 0.5\ \text{in}$ - Fillet Weld Size $F_{EXX} = 70\ \text{ksi}$ - Filler Metal Classification Strength	
	Weld Information at Web Connection (Column Side): E70XX - Weld Classification $W = 0.5\ \text{in}$ - Fillet Weld Size $F_{EXX} = 70\ \text{ksi}$ - Filler Metal Classification Strength	

REFERENCES | **CALCULATIONS** | **RESULTS**

Flange Plate Connection | AISC 360-16 LRFD

Flange Plate Geometry:

$b_{fp} = 16 \text{ in}$ - Flange Plate Width

$L_{fp} = 45 \text{ in}$ - Flange Plate Length

$t_{fp} = 1.5 \text{ in}$ - Flange Plate Thickness

Flange Plate Material Grade:

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

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

Connection Information at Flange Plate:

$n_r = 4$ - Number of Bolt Rows

$g_a = 5.5 \text{ in}$ - Bolt Gage

$s_r = 3 \text{ in}$ - Bolt Row Spacing

$n_c = 12$ - Number of Bolt Columns

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

Distances:

$L_{ev\_bf} = 2 \text{ in}$ - Vertical Edge Distance on Beam Flange

$L_{eh\_bf} = 1.25 \text{ in}$ - Horizontal Edge Distance on Beam Flange

$L_{ev\_bf} = 2 \text{ in}$ - Vertical Edge Distance on Beam Flange

$L_{eh\_fp} = 2.25 \text{ in}$ - Horizontal Edge Distance on Flange Plate

$L_{b\_fp} = 10 \text{ in}$ - Unbraced Length at Flange Plate

Check No. 1: Connection Detailing Limitations Check at Beam Side

Detailing Limitations	Limit Value (in)	Actual Value (in)	DCR	Result
Minimum Bolt Gage	5.125	5.500	0.932	**PASS**
Minimum Bolt Row Spacing	2.667	3.000	0.889	**PASS**
Maximum Bolt Row Spacing	6.000	3.000	0.500	**PASS**
Minimum Bolt Column Spacing	2.667	3.000	0.889	**PASS**
Maximum Bolt Column Spacing	6.000	3.000	0.500	**PASS**
Plate Minimum Vertical Edge Distance	1.375	2.000	0.688	**PASS**
Plate Minimum Horizontal Edge Distance	1.375	2.250	0.611	**PASS**
Beam Minimum Vertical Edge Distance	1.250	2.000	0.625	**PASS**
Beam Minimum Horizontal Edge Distance	1.250	1.250	1.000	**PASS**

Result:

Demand over Capacity Ratio

$DCR = \frac{d}{c} = \frac{(1.25)}{(1.25)} = 1$

PASS

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

P_{uF} - Equivalent Flange Force from Strong Axis Moment

$$P_{uF} = \frac{M_{ux}}{(d_{bm} - t_{fb})} + \frac{P_u}{2}$$

$$P_{uF} = \frac{(500 \text{ kipft})}{((28.1 \text{ in}) - (1.34 \text{ in}))} + \frac{(250 \text{ kip})}{2}$$

$$P_{uF} = 349.22 \text{ kip}$$

$\phi = 0.85$ - Bolt Shear Resistance Factor

$\mu = 0.5$ - Mean Slip Coefficient

$D_u = 1.13$

$h_f = 0.85$ - Filler Factor for Slip Critical Bolts

$d_b = 1 \text{ in}$ - Bolt Diameter

$T_b = 64 \text{ kip}$ - Minimum Bolt Pretension

$N_s = 1$ - Number of Slip Planes

$n_r = 4$ - Number of Bolt Rows

$n_c = 12$ - Number of Bolt Columns

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

ϕR_n - Design Bolt Shear Capacity

$$\phi R_n = \phi \, \mu \, D_u \, h_f \, T_b \, N_s \, n_r \, n_c$$

$$\phi R_n = (0.85) \times (0.5) \times (1.13) \times (0.85) \times (64 \text{ kip}) \times (1) \times (4) \times (12)$$

$$\phi R_n = 1254 \text{ kip}$$

Result:

Demand over Capacity Ratio

$DCR = \frac{P_{uF}}{\phi R_n} = \frac{(349.22 \text{ kip})}{(1254 \text{ kip})} = 0.27847$

PASS

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

Calculate the bolt bearing capacity of the flange plate.

$\phi = 0.75$ - Bolt Bearing Resistance Factor

$d_b = 1 \text{ in}$ - Bolt Diameter

$t_{fp} = 1.5 \text{ in}$ - Flange Plate Thickness

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

$n_c = 12$ - Number of Bolt Columns

$n_r = 4$ - Number of Bolt Rows

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

$\phi R_{n\_bearing}$ - Design Bolt Bearing Capacity of Section

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

$$\phi R_{n\_bearing} = (0.75) \times 2.4 \times (1 \text{ in}) \times (1.5 \text{ in}) \times (65 \text{ ksi}) \times (12) \times (4)$$

$$\phi R_{n\_bearing} = 8424 \text{ kip}$$

Calculate the clear distance of outer bolts on flange plate.

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

$d_h = 1.25 \text{ in}$ - Vertical Bolt Hole Dimension at Plate

l_{c1} - Clear Distance at First Bolt Row

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

$$l_{c1} = (2 \text{ in}) - \frac{(1.25 \text{ in})}{2}$$

$$l_{c1} = 1.375 \text{ in}$$

Calculate the clear distance of inner bolts on flange plate.

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

$d_h = 1.25 \text{ in}$ - Vertical Bolt Hole Dimension at Plate

l_{c2} - Clear Distance at Rest of Bolts

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

$$l_{c2} = (3 \text{ in}) - (1.25 \text{ in})$$

$$l_{c2} = 1.75 \text{ in}$$

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

$l_{c1} = 1.375 \text{ in}$ - Clear Distance at First Bolt Row

$l_{c2} = 1.75 \text{ in}$ - Clear Distance at Rest of Bolts

$t_{fp} = 1.5 \text{ in}$ - Flange Plate Thickness

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

$n_c = 12$ - Number of Bolt Columns

$n_r = 4$ - Number of Bolt Rows

$\phi = 0.75$ - Bolt Bearing Resistance Factor

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

$\phi R_{n\_tearout}$ - Design Bolt Tear-out Capacity of Section

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

$$\phi R_{n\_tearout} = (0.75) \times [1.2 \times (1.375 \text{ in}) \times (1.5 \text{ in}) \times (65 \text{ ksi}) \times (4) + 1.2 \times (1.75 \text{ in}) \times (1.5 \text{ in}) \times (65 \text{ ksi}) \times (4) \times ((12) - 1)]$$

$$\phi R_{n\_tearout} = 7239.4 \text{ kip}$$

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

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

$\phi R_{n\_bearing} = 8424 \text{ kip}$ - Design Bolt Bearing Capacity of Section

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

$\phi R_{n\_tearout} = 7239.4 \text{ kip}$ - Design Bolt Tear-out Capacity of Section

AISC 360-16 Chapter J3.10

ϕR_n - Governing Design Capacity

$$\phi R_n = min\,(\phi R_{n\_bearing}, \phi R_{n\_tearout})$$

$$\phi R_n = min\,((8424 \text{ kip})\,, (7239.4 \text{ kip}))$$

$$\phi R_n = 7239.4 \text{ kip}$$

Result:

Demand over Capacity Ratio

$DCR = \frac{P_{uF}}{\phi R_n} = \frac{(349.22 \text{ kip})}{(7239.4 \text{ kip})} = 0.048238$

PASS

Check No. 4: Design Bolt Bearing Capacity of the Beam Flange

Calculate the bolt bearing capacity of the beam flange.

$\phi = 0.75$ - Bolt Bearing Resistance Factor

$d_b = 1 \text{ in}$ - Bolt Diameter

$t_{fb} = 1.34 \text{ in}$ - Beam Flange Thickness

$F_{ub} = 65 \text{ ksi}$ - Beam Tensile Stress

$n_c = 12$ - Number of Bolt Columns

$n_r = 4$ - Number of Bolt Rows

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

$\phi R_{n\_bearing}$ - Design Bolt Bearing Capacity of Section

$$\phi R_{n\_bearing} = \phi \, 2.4 \, d_b \, t_{fb} \, F_{ub} \, n_c \, n_r$$

$$\phi R_{n\_bearing} = (0.75) \times 2.4 \times (1 \text{ in}) \times (1.34 \text{ in}) \times (65 \text{ ksi}) \times (12) \times (4)$$

$$\phi R_{n\_bearing} = 7525.4 \text{ kip}$$

Calculate the clear distance of outer bolts on beam flange.

$L_{ev\_bf} = 2 \text{ in}$ - Vertical Edge Distance on Beam Flange

$d_h = 1.125 \text{ in}$ - Vertical Bolt Hole Dimension at Beam

l_{c1} - Clear Distance at First Bolt Row

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

$$l_{c1} = (2 \text{ in}) - \frac{(1.125 \text{ in})}{2}$$

$$l_{c1} = 1.4375 \text{ in}$$

Calculate the clear distance of inner bolts on beam flange.

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

$d_h = 1.125 \text{ in}$ - Vertical Bolt Hole Dimension at Beam

l_{c2} - Clear Distance at Rest of Bolts

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

$$l_{c2} = (3 \text{ in}) - (1.125 \text{ in})$$

$$l_{c2} = 1.875 \text{ in}$$

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

$l_{c1} = 1.4375 \text{ in}$ - Clear Distance at First Bolt Row

$l_{c2} = 1.875 \text{ in}$ - Clear Distance at Rest of Bolts

$t_{fb} = 1.34 \text{ in}$ - Beam Flange Thickness

$F_{ub} = 65 \text{ ksi}$ - Beam Tensile Stress

$n_c = 12$ - Number of Bolt Columns

$n_r = 4$ - Number of Bolt Rows

$\phi = 0.75$ - Bolt Bearing Resistance Factor

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

$\phi R_{n\_tearout}$ - Design Bolt Tear-out Capacity of Section

$$\phi R_{n\_tearout} = \phi\ [1.2\ l_{c1}\ t_{fb}\ F_{ub}\ n_r + 1.2\ l_{c2}\ t_{fb}\ F_{ub}\ n_r\ \ (n_c - 1)]$$

$$\phi R_{n\_tearout} = (0.75) \times [1.2 \times (1.4375 \text{ in}) \times (1.34 \text{ in}) \times (65 \text{ ksi}) \times (4) + 1.2 \times (1.875 \text{ in}) \times (1.34 \text{ in}) \times (65 \text{ ksi}) \times (4) \times ((12) - 1)]$$

$$\phi R_{n\_tearout} = 6917.9 \text{ kip}$$

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

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

$\phi R_{n\_bearing} = 7525.4 \text{ kip}$ - Design Bolt Bearing Capacity of Section

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

$\phi R_{n\_tearout} = 6917.9 \text{ kip}$ - Design Bolt Tear-out Capacity of Section

AISC 360-16 Chapter J3.10

ϕR_n - Governing Design Capacity

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

$$\phi R_n = min\left((7525.4 \text{ kip}), (6917.9 \text{ kip})\right)$$

$$\phi R_n = 6917.9 \text{ kip}$$

Result:

Demand over Capacity Ratio

$DCR = \frac{P_{uF}}{\phi R_n} = \frac{(349.22 \text{ kip})}{(6917.9 \text{ kip})} = 0.05048$

PASS

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

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

$t_{fp} = 1.5 \text{ in}$ - Flange Plate Thickness

$n_r = 4$ - Number of Bolt Rows

$s_r = 3 \text{ in}$ - Bolt Row Spacing

$g_a = 5.5 \text{ in}$ - Bolt Gage

$L_{eh\_fp} = 2.25 \text{ in}$ - Horizontal Edge Distance on Flange Plate

$d_h = 1.25 \text{ in}$ - Vertical Bolt Hole Dimension at Plate

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

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

$$A_{nt} = (1.5 \text{ in}) \times [((4) - 2) \times (3 \text{ in}) + (5.5 \text{ in}) + (2.25 \text{ in}) - ((4) - 0.5) \times ((1.25 \text{ in}) + (0.0625 \text{ in}))]$$

$$A_{nt} = 13.734 \text{ in}^2$$

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

$t_{fp} = 1.5 \text{ in}$ - Flange Plate Thickness

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

$n_c = 12$ - Number of Bolt Columns

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

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

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

$$A_{gv} = (1.5 \text{ in}) \times [(2 \text{ in}) + ((12) - 1) \times (3 \text{ in})]$$

$$A_{gv} = 52.5 \text{ in}^2$$

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

$t_{fp} = 1.5 \text{ in}$ - Flange Plate Thickness

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

$n_c = 12$ - Number of Bolt Columns

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

$d_h = 1.25 \text{ in}$ - Vertical Bolt Hole Dimension at Plate

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

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

$$A_{nv} = (1.5 \text{ in}) \times ((2 \text{ in}) + ((12) - 1) \times (3 \text{ in}) - ((12) - 0.5) \times ((1.25 \text{ in}) + (0.0625 \text{ in})))$$

$$A_{nv} = 29.859\ \text{in}^2$$

Calculate the design block shear capacity of the flange plate.

$\phi = 0.75$ - Block Shear Resistance Factor
$F_{yp} = 50\ \text{ksi}$ - Flange Plate Yield Stress
$F_{up} = 65\ \text{ksi}$ - Flange Plate Tensile Stress
$U_{bs} = 1$ - Uniformity factor for single line of bolts
$A_{gv} = 52.5\ \text{in}^2$ - Gross Area Subject to Shear (L-pattern)
$A_{nv} = 29.859\ \text{in}^2$ - Net Area Subject to Shear (L-pattern)
$A_{nt} = 13.734\ \text{in}^2$ - Net Area Subject to Tension (L-pattern)
ϕR_n - Design Block Shear Capacity of Section

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

$$\phi R_n = \phi\ (0.6\ F_{up}\ A_{nv} + U_{bs}\ F_{up}\ A_{nt} \le 0.6\ F_{yp}\ A_{gv} + U_{bs}\ F_{up}\ A_{nt})$$

$$\phi R_n = (0.75) \times \left(0.6 \times (65\ \text{ksi}) \times \left(29.859\ \text{in}^2\right) + (1) \times (65\ \text{ksi}) \times \left(13.734\ \text{in}^2\right) \le 0.6 \times (50\ \text{ksi}) \times \left(52.5\ \text{in}^2\right) + (1) \times (65\ \text{ksi}) \times \left(13.734\ \text{in}^2\right)\right)$$

$$\phi R_n = 1542.9\ \text{kip}$$

Result:
Demand over Capacity Ratio
$DCR = \frac{P_{uF}}{\phi R_n} = \frac{(349.22\ \text{kip})}{(1542.9\ \text{kip})} = 0.22633$

PASS

Check No. 6: Design Block Shear Capacity of the Beam Flange

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

$t_{fb} = 1.34\ \text{in}$ - Beam Flange Thickness
$b_{fb} = 14\ \text{in}$ - Beam Flange Width
$g_a = 5.5\ \text{in}$ - Bolt Gage
$d_h = 1.125\ \text{in}$ - Vertical Bolt Hole Dimension at Beam
A_{nt} - Net Area Subject to Tension (2L-pattern)

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

$$A_{nt} = (1.34\ \text{in}) \times [(14\ \text{in}) - (5.5\ \text{in}) - ((1.125\ \text{in}) + (0.0625\ \text{in}))]$$

$$A_{nt} = 9.7988\ \text{in}^2$$

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

$t_{fb} = 1.34\ \text{in}$ - Beam Flange Thickness
$L_{ev\_bf} = 2\ \text{in}$ - Vertical Edge Distance on Beam Flange
$n_c = 12$ - Number of Bolt Columns
$s_c = 3\ \text{in}$ - Bolt Column Spacing
A_{gv} - Gross Area Subject to Shear (2L-pattern)

$$A_{gv} = 2\ t_{fb}\ [L_{ev\_bf} + (n_c - 1)\ s_c]$$

$$A_{gv} = 2 \times (1.34\ \text{in}) \times [(2\ \text{in}) + ((12) - 1) \times (3\ \text{in})]$$

$$A_{gv} = 93.8\ \text{in}^2$$

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

$t_{fb} = 1.34\ \text{in}$ - Beam Flange Thickness
$L_{ev\_bf} = 2\ \text{in}$ - Vertical Edge Distance on Beam Flange
$n_c = 12$ - Number of Bolt Columns
$s_c = 3\ \text{in}$ - Bolt Column Spacing
$d_h = 1.125\ \text{in}$ - Vertical Bolt Hole Dimension at Beam
A_{nv} - Net Area Subject to Shear (2L-pattern)

$$A_{nv} = 2\ t_{fb}\ (L_{ev\_bf} + (n_c - 1)\ s_c - (n_c - 0.5)\ (d_h + 0.0625\ \text{in}))$$

$$A_{nv} = 2 \times (1.34\ \text{in}) \times ((2\ \text{in}) + ((12) - 1) \times (3\ \text{in}) - ((12) - 0.5) \times ((1.125\ \text{in}) + (0.0625\ \text{in})))$$

$$A_{nv} = 57.201\ \text{in}^2$$

Calculate the design block shear capacity of the beam flange.

$\phi = 0.75$ - Block Shear Resistance Factor
$F_{yb} = 50\ \text{ksi}$ - Beam Yield Stress
$F_{ub} = 65\ \text{ksi}$ - Beam Tensile Stress
$U_{bs} = 1$ - Uniformity factor for single line of bolts
$A_{gv} = 93.8\ \text{in}^2$ - Gross Area Subject to Shear (2L-pattern)
$A_{nv} = 57.201\ \text{in}^2$ - Net Area Subject to Shear (2L-pattern)
$A_{nt} = 9.7988\ \text{in}^2$ - Net Area Subject to Tension (2L-pattern)
ϕR_n - Design Block Shear Capacity of Section

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

$$\phi R_n = \phi\ (0.6\ F_{ub}\ A_{nv} + U_{bs}\ F_{ub}\ A_{nt} \le 0.6\ F_{yb}\ A_{gv} + U_{bs}\ F_{ub}\ A_{nt})$$

$$\phi R_n = (0.75) \times \left(0.6 \times (65\ \text{ksi}) \times \left(57.201\ \text{in}^2\right) + (1) \times (65\ \text{ksi}) \times \left(9.7988\ \text{in}^2\right) \le 0.6 \times (50\ \text{ksi}) \times \left(93.8\ \text{in}^2\right) + (1) \times (65\ \text{ksi}) \times \left(9.7988\ \text{in}^2\right)\right)$$

$$\phi R_n = 2150.8\ \text{kip}$$

Result:
Demand over Capacity Ratio
$DCR = \frac{P_{uF}}{\phi R_n} = \frac{(349.22\ \text{kip})}{(2150.8\ \text{kip})} = 0.16236$

PASS

Check No. 7: Design Capacity of the Flange Plate in Tension

Calculate the tensile yielding capacity of the flange plate.

$\phi = 0.9$ - Tensile Yielding Resistance Factor

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

$t_{fp} = 1.5\ \mathrm{in}$ - Flange Plate Thickness

$b_{fp} = 16\ \mathrm{in}$ - Flange Plate Width

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

ϕR_{n_ty} - Design Tension Yielding Capacity of Section

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

$$\phi R_{n\_ty} = (0.9) \times (50\ \mathrm{ksi}) \times (1.5\ \mathrm{in}) \times (16\ \mathrm{in})$$

$$\phi R_{n\_ty} = 1080\ \mathrm{kip}$$

Calculate the tensile rupture capacity of the flange plate.

$\phi = 0.75$ - Tensile Rupture Resistance Factor

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

$t_{fp} = 1.5\ \mathrm{in}$ - Flange Plate Thickness

$b_{fp} = 16\ \mathrm{in}$ - Flange Plate Width

$n_r = 4$ - Number of Bolt Rows

$d_h = 1.25\ \mathrm{in}$ - Horizontal Bolt Hole Dimension at Plate

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

ϕR_{n_tr} - Design Tension Rupture Capacity of Section

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

$$\phi R_{n\_tr} = (0.75) \times (65\ \mathrm{ksi}) \times (1.5\ \mathrm{in}) \times [(16\ \mathrm{in}) - (4) \times ((1.25\ \mathrm{in}) + (0.0625\ \mathrm{in}))]$$

$$\phi R_{n\_tr} = 786.09\ \mathrm{kip}$$

Determine the governing tensile capacity of the flange plate.

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

$\phi R_{n\_ty} = 1080\ \mathrm{kip}$ - Design Tension Yielding Capacity of Section

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

$\phi R_{n\_tr} = 786.09\ \mathrm{kip}$ - Design Tension Rupture Capacity of Section

AISC 360-16 Chapter J4.2

ϕR_n - Governing Design Capacity

$$\phi R_n = min\,(\phi R_{n\_ty}, \phi R_{n\_tr})$$

$$\phi R_n = min\,((1080\ \mathrm{kip}), (786.09\ \mathrm{kip}))$$

$$\phi R_n = 786.09\ \mathrm{kip}$$

Result:

Demand over Capacity Ratio

$DCR = \frac{P_{uF}}{\phi R_n} = \frac{(349.22\ \mathrm{kip})}{(786.09\ \mathrm{kip})} = 0.44424$

PASS

Check No. 8: Design Capacity of the Beam in Tension

Calculate the tensile rupture capacity of the flange plate.

$\phi = 0.75$ - Tensile Rupture Resistance Factor

$F_{ub} = 65\ \mathrm{ksi}$ - Beam Tensile Stress

$A_{bm} = 57.1\ \mathrm{in}^2$ - Beam Area

$t_{fb} = 1.34\ \mathrm{in}$ - Beam Flange Thickness

$d_h = 1.125\ \mathrm{in}$ - Horizontal Bolt Hole Dimension at Beam

$n_r = 4$ - Number of Bolt Rows

$n_c = 12$ - Number of Bolt Columns

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

$\bar{y} = 3.0789\ \mathrm{in}$ - Centroid of WT section

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

U - Shear Lag Factor

$$U = 1 - \frac{\bar{y}}{(n_c - 1)\ s_c}$$

$$U = 1 - \frac{(3.0789\ \mathrm{in})}{((12) - 1) \times (3\ \mathrm{in})}$$

$$U = 0.9067$$

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

ϕR_{n_tr} - Design Tension Rupture Capacity of Section

$$\phi R_{n\_tr} = \phi\, F_{ub}\, U\ [0.5\, A_{bm} - n_r\ (d_h + 0.0625\ \mathrm{in})\ t_{fb}]$$

$$\phi R_{n\_tr} = (0.75) \times (65\ \mathrm{ksi}) \times (0.9067) \times \left[0.5 \times \left(57.1\ \mathrm{in}^2\right) - (4) \times ((1.125\ \mathrm{in}) + (0.0625\ \mathrm{in})) \times (1.34\ \mathrm{in})\right]$$

$$\phi R_{n\_tr} = 980.61\ \mathrm{kip}$$

Result:

Demand over Capacity Ratio

$DCR = \frac{P_{uF}}{\phi R_{n\_tr}} = \frac{(349.22\ \mathrm{kip})}{(980.61\ \mathrm{kip})} = 0.35612$

PASS

Check No. 9: Design Capacity of the Flange Plate in Compression

Calculate the compression buckling capacity of the flange plate.

$\phi = 0.9$ - Compression Resistance Factor

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

$E = 29000\ \mathrm{ksi}$ - Modulus for Steel

$t_{fp} = 1.5\ \mathrm{in}$ - Flange Plate Thickness

$b_{fp} = 16\ \mathrm{in}$ - Flange Plate Width

$K = 1.2$ - Effective Length Factor

$L_{b\_fp} = 10\ \mathrm{in}$ - Unbraced Length at Flange Plate

$\frac{KL}{r} = 27.713$ - Effective Length Slenderness Ratio

Since, $\frac{KL}{r} > 25.$

AISC 360-16 Chapter E3 Eq. (E3-4)

F_e - Elastic Buckling Stress

$$F_e = \frac{\pi^2\,E}{\left(\frac{KL}{r}\right)^2}$$

$$F_e = \frac{\pi^2 \times (29000\ \text{ksi})}{((27.713))^2}$$

$$F_e = 372.68\ \text{ksi}$$

$4.71\sqrt{\frac{E}{F_y}} = 113.43$ - Effective Length Slenderness Ratio Limiter

Since, $\frac{KL}{r} \leq 4.71\sqrt{\frac{E}{F_y}}.$

AISC 360-16 Chapter E3 Eq. (E3-2)

F_{cr} - Critical Buckling Stress

$$F_{cr} = \left(0.658^{\frac{F_{yp}}{F_e}}\right) F_{yp}$$

$$F_{cr} = \left(0.658^{\frac{(50\ \text{ksi})}{(372.68\ \text{ksi})}}\right) \times (50\ \text{ksi})$$

$$F_{cr} = 47.27\ \text{ksi}$$

AISC 360-16 Chapter E3 Eq. (E3-1)

ϕR_n - Design Compressive Capacity of Section

$$\phi R_n = \phi\, F_{cr}\, t_{fp}\, b_{fp}$$

$$\phi R_n = (0.9) \times (47.27\ \text{ksi}) \times (1.5\ \text{in}) \times (16\ \text{in})$$

$$\phi R_n = 1021\ \text{kip}$$

Result:

Demand over Capacity Ratio

$DCR = \frac{P_{uF}}{\phi R_n} = \frac{(349.22\ \text{kip})}{(1021\ \text{kip})} = 0.34202$

PASS

Check No. 10: Connection Detailing Limitations Check at Support Side

Detailing Limitations	Limit Value (in)	Actual Value (in)	DCR	Result
Maximum Fillet Weld Size per Beam Clearance	8.000	0.500	0.063	**PASS**

Result:

Demand over Capacity Ratio

$DCR = \frac{d}{c} = \frac{(0.5)}{(8)} = 0.0625$

PASS

Check No. 11: Design Capacity of Weld to Support Flange

Calculate the maximum fillet weld size in 16th of an inch for base metal check.

$t_{fp} = 1.5\ \text{in}$ - Flange Plate Thickness

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

$t_{fs} = 0.71\ \text{in}$ - Column Flange Thickness

$F_{us} = 65\ \text{ksi}$ - Column Tensile Stress

AISC 15th Ed. Part 9 Eq. (9-2)

D_{max} - Maximum Fillet Weld Size for Base Metal Strength

$$D_{max} = \frac{\frac{t_{fp}}{2}\,F_{up} \leq t_{fs}\,F_{us}}{3.09\ \text{kip/in}}$$

$$D_{max} = \frac{(0.75\ \text{in}) \times (65\ \text{ksi}) \leq (0.71\ \text{in}) \times (65\ \text{ksi})}{(3.09\ \text{kip/in})}$$

$$D_{max} = 14.935$$

Calculate the total effective weld length.

L_w - Length of One Weld Segment

$$L_w = \frac{(b_{fs} - 2\,k_{1\_sup})}{2}$$

$$L_w = \frac{((14.5\ \text{in}) - 2 \times (1.4375\ \text{in}))}{2}$$

$$L_w = 5.8125\ \text{in}$$

L_w - Total Effective Length of Weld

$$L_w = 4\ [L_w - 2\ (W \leq 0.3125\ \text{in})]$$

$$L_w = 4 \times [(5.8125\ \text{in}) - 2 \times ((0.5\ \text{in}) \leq (0.3125\ \text{in}))]$$

$$L_w = 20.75\ \text{in}$$

Calculate the design capacity of weld.

$F_{EXX} = 70\ \text{ksi}$ - Filler Metal Classification Strength

$W = 0.5\ \text{in}$ - Fillet Weld Size

AISC 15th Ed. Part 9 Eq. (9-2)

$D_{max} = 14.935$ - Maximum Fillet Weld Size for Base Metal Strength

$L_w = 20.75$ in - Total Effective Length of Weld

$\phi = 0.75$ - Fillet Weld Resistance Factor

AISC 360-16 Chapter J2.4 Eq. (J2-4)

ϕR_n - Design Strength of Welds

$$\phi R_n = \phi\, 0.6\, F_{EXX}\, \frac{\sqrt{2}}{2} \left(W \le \frac{D_{max}}{16} \right) L_w$$

$$\phi R_n = (0.75) \times 0.6 \times (70\ \text{ksi}) \times \frac{\sqrt{2}}{2} \times \left((0.5\ \text{in}) \le \frac{(14.935)}{(16)} \right) \times (20.75\ \text{in})$$

$$\phi R_n = 231.09\ \text{kip}$$

Result:

Demand over Capacity Ratio

$DCR = \frac{P_f}{\phi R_n} = \frac{(231.09\ \text{kip})}{(231.09\ \text{kip})} = 1$

PASS

Check No. 12: Design Capacity of Weld to Support Web

Calculate the maximum fillet weld size in 16th of an inch for base metal check.

$t_{fp} = 1.5$ in - Flange Plate Thickness

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

$t_{ws} = 0.44$ in - Column Web Thickness

$F_{us} = 65$ ksi - Column Tensile Stress

AISC 15th Ed. Part 9 Eq. (9-2)

D_{max} - Maximum Fillet Weld Size for Base Metal Strength

$$D_{max} = \frac{\frac{t_{fp}}{2}\, F_{up} \le t_{ws}\, F_{us}}{3.09\ \text{kip/in}}$$

$$D_{max} = \frac{(0.75\ \text{in}) \times (65\ \text{ksi}) \le (0.44\ \text{in}) \times (65\ \text{ksi})}{(3.09\ \text{kip/in})}$$

$$D_{max} = 9.2557$$

Calculate the total effective weld length.

L_w - Total Effective Length of Weld

$$L_w = 2\ [T_{sup} - 2\ (W \le 0.3125\ \text{in})]$$

$$L_w = 2 \times [(10\ \text{in}) - 2 \times ((0.5\ \text{in}) \le (0.3125\ \text{in}))]$$

$$L_w = 18.75\ \text{in}$$

Calculate the design capacity of weld.

$P_f = 118.12$ kip - Excess Flange force to be carried by the web welds

$F_{EXX} = 70$ ksi - Filler Metal Classification Strength

$W = 0.5$ in - Fillet Weld Size

AISC 15th Ed. Part 9 Eq. (9-2)

$D_{max} = 9.2557$ - Maximum Fillet Weld Size for Base Metal Strength

$L_w = 18.75$ in - Total Effective Length of Weld

$\phi = 0.75$ - Fillet Weld Resistance Factor

AISC 360-16 Chapter J2.4 Eq. (J2-4)

ϕR_n - Design Strength of Welds

$$\phi R_n = \phi\, 0.6\, F_{EXX}\, \frac{\sqrt{2}}{2} \left(W \le \frac{D_{max}}{16} \right) L_w$$

$$\phi R_n = (0.75) \times 0.6 \times (70\ \text{ksi}) \times \frac{\sqrt{2}}{2} \times \left((0.5\ \text{in}) \le \frac{(9.2557)}{(16)} \right) \times (18.75\ \text{in})$$

$$\phi R_n = 208.82\ \text{kip}$$

Result:

Demand over Capacity Ratio

$DCR = \frac{P_f}{\phi R_n} = \frac{(118.12\ \text{kip})}{(208.82\ \text{kip})} = 0.56568$

PASS

Check No. 13: Design Capacity of Support Web in Punching

Calculate the web capacity in punching of the supporting member.

$P_f = 118.12$ kip - Excess Flange force to be carried by the web welds

$\phi = 0.95$ - Web Punching Safety Factor

$F_{ys} = 50$ ksi - Column Yield Stress

$t_{ws} = 0.44$ in - Column Web Thickness

$T_{sup} = 10$ in - Column T-Dimension

ϕR_n - Design Web Shear Yielding (Punching)

$$\phi R_n = \phi\, 0.6\, F_{ys}\, t_{ws}\, 2\, T_{sup}$$

$$\phi R_n = (0.95) \times 0.6 \times (50\ \text{ksi}) \times (0.44\ \text{in}) \times 2 \times (10\ \text{in})$$

$$\phi R_n = 250.8\ \text{kip}$$

Result:

Demand over Capacity Ratio

$DCR = \frac{P_f}{\phi R_n} = \frac{(118.12\ \text{kip})}{(250.8\ \text{kip})} = 0.47099$

PASS

REFERENCES	CALCULATIONS	RESULTS

Summary of Checks

Design Checks	Demand	Capacity	DCR	Result
Connection Detailing Limitations Check at Beam Side	1.250	1.250	1.000	PASS
Design Capacity of the Bolts in Shear	349.215	1254.029	0.278	PASS
Design Bolt Bearing Capacity of the Flange Plate	349.215	7239.375	0.048	PASS
Design Bolt Bearing Capacity of the Beam Flange	349.215	6917.918	0.050	PASS
Design Block Shear Capacity of the Flange Plate	349.215	1542.938	0.226	PASS
Design Block Shear Capacity of the Beam Flange	349.215	2150.826	0.162	PASS
Design Capacity of the Flange Plate in Tension	349.215	786.094	0.444	PASS
Design Capacity of the Beam in Tension	349.215	980.612	0.356	PASS
Design Capacity of the Flange Plate in Compression	349.215	1021.025	0.342	PASS
Connection Detailing Limitations Check at Support Side	0.500	8.000	0.063	PASS
Design Capacity of Weld to Support Flange	231.091	231.091	1.000	PASS
Design Capacity of Weld to Support Web	118.124	208.817	0.566	PASS
Design Capacity of Support Web in Punching	118.124	250.800	0.471	PASS

REFERENCES | **CALCULATIONS** | **RESULTS**

Shear Plate Connection | AISC 360-16 LRFD

Single Plate Geometry:

$b_p = 21 \text{ in}$ - Single Plate Width

$d_p = 21 \text{ in}$ - Single Plate Depth

$t_p = 1 \text{ in}$ - Single Plate Thickness

Single Plate Material Grade:

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

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

Connection Geometry:

$n_r = 7$ - Number of Bolt Rows

$s_r = 3 \text{ in}$ - Bolt Row Spacing

$n_c = 4$ - Number of Bolt Columns

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

Distances:

$clr = 8 \text{ in}$ - Beam Web Clearance

$L_{eh\_bm} = 2 \text{ in}$ - Horizontal Edge Distance on Beam Web

$L_{eh\_pl} = 2 \text{ in}$ - Horizontal Edge Distance on Single Plate

$L_{ev\_pl} = 1.5 \text{ in}$ - Vertical Edge Distance on Single Plate

$e = 14.5 \text{ in}$ - Bolt Group Eccentricity

Check No. 1: Connection Detailing Limitations at Beam Side

Detailing Limitations	Limit Value (in)	Actual Value (in)	DCR	Result
Minimum Bolt Row Spacing	2.667	3.000	0.889	**PASS**
Maximum Bolt Row Spacing	6.000	3.000	0.500	**PASS**
Minimum Bolt Column Spacing	2.667	3.000	0.889	**PASS**
Maximum Bolt Column Spacing	6.000	3.000	0.500	**PASS**
Plate Minimum Vertical Edge Distance	1.250	1.500	0.833	**PASS**
Plate Minimum Horizontal Edge Distance	1.375	2.000	0.688	**PASS**
Beam Minimum Horizontal Edge Distance	1.250	2.000	0.625	**PASS**
Minimum Connection Depth	11.500	21.000	0.548	**PASS**
Maximum Connection Depth	23.625	21.000	0.889	**PASS**

Result:

Demand over Capacity Ratio

$DCR = \frac{d}{c} = \frac{(2.667)}{(3)} = 0.889$

PASS

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

Calculate the design shear capacity of the bolt group.

$\phi = 0.75$ - Bolt Shear Resistance Factor

$d_b = 1 \text{ in}$ - Bolt Diameter

$C = 28$ - Calculated Bolt Group Coefficient

$F_{nv} = 68 \text{ ksi}$ - Bolt Nominal Shear Strength

$N_s = 1$ - Number of Shear Planes

AISC 360-16 Chapter J5.2(a)

h_f - Filler Factor for Bearing Bolts

$$h_f = 0.85 < 1 - 0.4\ (t - 0.25 \text{ in}) \leq 1$$

$$h_f = 0.85 < 1 - (0.4) \times ((0 \text{ in}) - (0.25 \text{ in})) \leq 1$$

$$h_f = 1$$

AISC 360-16 Chapter J3.6 Eq. (J3-1)

ϕR_n - Design Bolt Shear Capacity

$$\phi R_n = \phi\, F_{nv}\, \frac{\pi}{4}\, (d_b)^2\, N_s\, C\, h_f$$

$$\phi R_n = (0.75) \times (68 \text{ ksi}) \times \frac{\pi}{4} \times ((1 \text{ in}))^2 \times (1) \times (28) \times (1)$$

$$\phi R_n = 1121.5 \text{ kip}$$

Result:

Demand over Capacity Ratio

$DCR = \frac{V_u}{\phi R_n} = \frac{(400 \text{ kip})}{(1121.5 \text{ kip})} = 0.35665$

PASS

Check No. 3: Design Capacity of the Bolt Group in Bearing and Tear-out on Single Plate

Calculate the bolt bearing capacity of the single plate.

$d_b = 1 \text{ in}$ - Bolt Diameter

$t_p = 1 \text{ in}$ - Single Plate Thickness

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

$C = 28$ - Calculated Bolt Group Coefficient

$\phi = 0.75$ - Bolt Bearing Resistance Factor

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

$\phi R_{n\_bearing}$ - Design Bolt Bearing Capacity of Section

$$\phi R_{n\_bearing} = \phi\, 2.4\, d_b\, t_p\, F_{up}\, C$$

$$\phi R_{n\_bearing} = (0.75) \times 2.4 \times (1 \text{ in}) \times (1 \text{ in}) \times (65 \text{ ksi}) \times (28)$$

$$\phi R_{n\_bearing} = 3276 \text{ kip}$$

Calculate the clear distance of outer bolts on single plate.

$L_{ev\_pl} = 1.5 \text{ in}$ - Vertical Edge Distance on Single Plate
$d_h = 1.125 \text{ in}$ - Vertical Bolt Hole Dimension at Plate
l_{c1} - Clear Distance at First Bolt Row

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

$$l_{c1} = (1.5 \text{ in}) - \frac{(1.125 \text{ in})}{2}$$

$$l_{c1} = 0.9375 \text{ in}$$

Calculate the clear distance of inner bolts on single plate.
$s_r = 3 \text{ in}$ - Bolt Row Spacing
$d_h = 1.125 \text{ in}$ - Vertical Bolt Hole Dimension at Plate
l_{c2} - Clear Distance at Rest of Bolts

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

$$l_{c2} = (3 \text{ in}) - (1.125 \text{ in})$$

$$l_{c2} = 1.875 \text{ in}$$

Calculate the bolt tear-out capacity of the single plate.
$l_{c1} = 0.9375 \text{ in}$ - Clear Distance at First Bolt Row
$l_{c2} = 1.875 \text{ in}$ - Clear Distance at Rest of Bolts
$t_p = 1 \text{ in}$ - Single Plate Thickness
$F_{up} = 65 \text{ ksi}$ - Single Plate Tensile Stress
$C = 28$ - Calculated Bolt Group Coefficient
$n_r = 7$ - Number of Bolt Rows
$\phi = 0.75$ - Bolt Bearing Resistance Factor

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

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

$$\phi R_{n\_tearout} = (0.75) \times [1.2 \times (0.9375 \text{ in}) \times (1 \text{ in}) \times (65 \text{ ksi}) + 1.2 \times (1.875 \text{ in}) \times (1 \text{ in}) \times (65 \text{ ksi}) \times ((7) - 1)] \times \left(\frac{(28)}{(7)}\right)$$

$$\phi R_{n\_tearout} = 2851.9 \text{ kip}$$

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

AISC 360-16 Chapter J3.10 Eq. (J3-6a)
$\phi R_{n\_bearing} = 3276 \text{ kip}$ - Design bolt bearing capacity of single plate

AISC 360-16 Chapter J3.10 Eq. (J3-6c)
$\phi R_{n\_tearout} = 2851.9 \text{ kip}$ - Design bolt tear-out capacity of single plate

AISC 360-16 Chapter J3.10
ϕR_n - Governing Design Capacity

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

$$\phi R_n = min\left((3276 \text{ kip}), (2851.9 \text{ kip})\right)$$

$$\phi R_n = 2851.9 \text{ kip}$$

Result:
Demand over Capacity Ratio
$DCR = \frac{V_u}{\phi R_n} = \frac{(400 \text{ kip})}{(2851.9 \text{ kip})} = 0.14026$

PASS

Check No. 4: Design Capacity of the Bolt Group in Bearing and Tear-out on Beam Web

Calculate the bolt bearing capacity of the beam web.
$d_b = 1 \text{ in}$ - Bolt Diameter
$t_{wb} = 0.75 \text{ in}$ - Beam Web Thickness
$F_{ub} = 65 \text{ ksi}$ - Beam Tensile Stress
$C = 28$ - Calculated Bolt Group Coefficient
$\phi = 0.75$ - Bolt Bearing Resistance Factor

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

$$\phi R_{n\_bearing} = \phi\, 2.4\, d_b\, t_{wb}\, F_{ub}\, C$$

$$\phi R_{n\_bearing} = (0.75) \times 2.4 \times (1 \text{ in}) \times (0.75 \text{ in}) \times (65 \text{ ksi}) \times (28)$$

$$\phi R_{n\_bearing} = 2457 \text{ kip}$$

Calculate the clear distance of inner bolts on beam web.
$s_r = 3 \text{ in}$ - Bolt Row Spacing
$d_h = 1.125 \text{ in}$ - Vertical Bolt Hole Dimension at Beam
l_{c2} - Clear Distance at Rest of Bolts

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

$$l_{c2} = (3 \text{ in}) - (1.125 \text{ in})$$

$$l_{c2} = 1.875 \text{ in}$$

Calculate the bolt tear-out capacity of the beam web.
$l_{c2} = 1.875 \text{ in}$ - Clear Distance at Rest of Bolts
$t_{wb} = 0.75 \text{ in}$ - Beam Web Thickness
$F_{ub} = 65 \text{ ksi}$ - Beam Tensile Stress

$C = 28$ - Calculated Bolt Group Coefficient

$\phi = 0.75$ - Bolt Bearing Resistance Factor

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

$\phi R_{n\_tearout}$ - Design Bolt Tear-out Capacity of Section

$$\phi R_{n\_tearout} = \phi\, 1.2\, l_{c2}\, t_{wb}\, F_{ub}\, C$$

$$\phi R_{n\_tearout} = (0.75) \times 1.2 \times (1.875 \text{ in}) \times (0.75 \text{ in}) \times (65 \text{ ksi}) \times (28)$$

$$\phi R_{n\_tearout} = 2303.4 \text{ kip}$$

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

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

$\phi R_{n\_bearing} = 2457 \text{ kip}$ - Design bolt bearing capacity of beam web

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

$\phi R_{n\_tearout} = 2303.4 \text{ kip}$ - Design bolt tear-out capacity of beam web

AISC 360-16 Chapter J3.10

ϕR_n - Governing Design Capacity

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

$$\phi R_n = min\left((2457 \text{ kip}), (2303.4 \text{ kip})\right)$$

$$\phi R_n = 2303.4 \text{ kip}$$

Result:

PASS

Demand over Capacity Ratio

$DCR = \frac{V_u}{\phi R_n} = \frac{(400 \text{ kip})}{(2303.4 \text{ kip})} = 0.17365$

Check No. 5: Design Capacity of Single Plate in Block Shear

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

$t_p = 1 \text{ in}$ - Single Plate Thickness

$n_c = 4$ - Number of Bolt Columns

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

$L_{eh\_pl} = 2 \text{ in}$ - Horizontal Edge Distance on Single Plate

$d_h = 1.3125 \text{ in}$ - Horizontal Bolt Hole Dimension at Plate

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

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

$$A_{nt} = (1 \text{ in}) \times \left[((4) - 1) \times (3 \text{ in}) + (2 \text{ in}) - ((4) - 0.5) \times ((1.3125 \text{ in}) + (0.0625 \text{ in}))\right]$$

$$A_{nt} = 6.1875 \text{ in}^2$$

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

$t_p = 1 \text{ in}$ - Single Plate Thickness

$L_{ev\_pl} = 1.5 \text{ in}$ - Vertical Edge Distance on Single Plate

$n_r = 7$ - Number of Bolt Rows

$s_r = 3 \text{ in}$ - Bolt Row Spacing

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

$$A_{gv} = t_p\ \left[L_{ev\_pl} + (n_r - 1)\ s_r\right]$$

$$A_{gv} = (1 \text{ in}) \times \left[(1.5 \text{ in}) + ((7) - 1) \times (3 \text{ in})\right]$$

$$A_{gv} = 19.5 \text{ in}^2$$

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

$t_p = 1 \text{ in}$ - Single Plate Thickness

$d_p = 21 \text{ in}$ - Single Plate Depth

$L_{ev\_pl} = 1.5 \text{ in}$ - Vertical Edge Distance on Single Plate

$n_r = 7$ - Number of Bolt Rows

$d_h = 1.125 \text{ in}$ - Vertical Bolt Hole Dimension at Plate

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

$$A_{nv} = t_p\ \left[d_p - L_{ev\_pl} - (n_r - 0.5)\ (d_h + 0.0625 \text{ in})\right]$$

$$A_{nv} = (1 \text{ in}) \times \left[(21 \text{ in}) - (1.5 \text{ in}) - ((7) - 0.5) \times ((1.125 \text{ in}) + (0.0625 \text{ in}))\right]$$

$$A_{nv} = 11.781 \text{ in}^2$$

Calculate the design block shear capacity of the single plate.

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

$A_{nv} = 11.781 \text{ in}^2$ - Net Area Subject to Shear (L-pattern)

$U_{bs} = 0.5$ - Uniformity factor for multiple line of bolts

$A_{nt} = 6.1875 \text{ in}^2$ - Net Area Subject to Tension (L-pattern)

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

$A_{gv} = 19.5 \text{ in}^2$ - Gross Area Subject to Shear (L-pattern)

$\phi = 0.75$ - Block Shear Resistance Factor

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

ϕR_n - Design Block Shear Capacity of Section

$$\phi R_n = \phi\ (0.6\, F_{up}\, A_{nv} + U_{bs}\, F_{up}\, A_{nt} \leq 0.6\, F_{yp}\, A_{gv} + U_{bs}\, F_{up}\, A_{nt})$$

$$\phi R_n = (0.75) \times \left(0.6 \times (65 \text{ ksi}) \times \left(11.781 \text{ in}^2\right) + (0.5) \times (65 \text{ ksi}) \times \left(6.1875 \text{ in}^2\right) \leq 0.6 \times (50 \text{ ksi}) \times \left(19.5 \text{ in}^2\right) + (0.5) \times (65 \text{ ksi}) \times \left(6.1875 \text{ in}^2\right)\right)$$

$$\phi R_n = 495.42 \text{ kip}$$

Result:

PASS

Demand over Capacity Ratio

$DCR = \frac{V_u}{\phi R_n} = \frac{(400\ \text{kip})}{(495.42\ \text{kip})} = 0.80739$

Check No. 6: Design Capacity of Single Plate in Shear

Calculate the gross area of single plate subject to yielding.

$d_p = 21\ \text{in}$ - Single Plate Depth

$t_p = 1\ \text{in}$ - Single Plate Thickness

A_{gv} - Section Gross Area

$$A_{gv} = d_p\, t_p$$

$$A_{gv} = (21\ \text{in}) \times (1\ \text{in})$$

$$A_{gv} = 21\ \text{in}^2$$

Calculate the shear yielding capacity of the single plate.

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

$A_{gv} = 21\ \text{in}^2$ - Section Gross Area

$\phi = 1$ - Shear Yielding Resistance Factor

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

ϕR_{n_sy} - Design Shear Yielding Capacity of Section

$$\phi R_{n\_sy} = \phi\, 0.6\, F_{yp}\, A_{gv}$$

$$\phi R_{n\_sy} = (1) \times 0.6 \times (50\ \text{ksi}) \times \left(21\ \text{in}^2\right)$$

$$\phi R_{n\_sy} = 630\ \text{kip}$$

Calculate the net area of single plate subject to rupture.

$t_p = 1\ \text{in}$ - Single Plate Thickness

$d_p = 21\ \text{in}$ - Single Plate Depth

$n_r = 7$ - Number of Bolt Rows

$d_h = 1.125\ \text{in}$ - Vertical Bolt Hole Dimension at Plate

A_{nv} - Section Net Area

$$A_{nv} = t_p\ \left[d_p - n_r\ \left(d_h + 0.0625\ \text{in}\right)\right]$$

$$A_{nv} = (1\ \text{in}) \times \left[(21\ \text{in}) - (7) \times \left((1.125\ \text{in}) + (0.0625\ \text{in})\right)\right]$$

$$A_{nv} = 12.688\ \text{in}^2$$

Calculate the shear rupture capacity of the single plate.

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

$A_{nv} = 12.688\ \text{in}^2$ - Section Net Area

$\phi = 1$ - Shear Yielding Resistance Factor

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

ϕR_{n_sr} - Design Shear Rupture Capacity of Section

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

$$\phi R_{n\_sr} = (1) \times 0.6 \times (65\ \text{ksi}) \times \left(12.688\ \text{in}^2\right)$$

$$\phi R_{n\_sr} = 494.81\ \text{kip}$$

Determine the governing shear capacity of the single plate.

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

$\phi R_{n\_sy} = 630\ \text{kip}$ - Design shear yielding capacity of single plate

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

$\phi R_{n\_sr} = 494.81\ \text{kip}$ - Design shear rupture capacity of single plate

AISC 360-16 Chapter J4.2

ϕR_n - Governing Design Capacity

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

$$\phi R_n = min\left((630\ \text{kip}), (494.81\ \text{kip})\right)$$

$$\phi R_n = 494.81\ \text{kip}$$

Result:

Demand over Capacity Ratio

$DCR = \frac{V_u}{\phi R_n} = \frac{(400\ \text{kip})}{(494.81\ \text{kip})} = 0.80839$

PASS

Check No. 7: Connection Detailing Limitations at Support Side

Detailing Limitations	Limit Value (in)	Actual Value (in)	DCR	Result
Maximum Fillet Weld Size per Beam Clearance	8.000	0.500	0.063	PASS

Result:

Demand over Capacity Ratio

$DCR = \frac{d}{c} = \frac{(0.5)}{(8)} = 0.0625$

PASS

Check No. 8: Design Capacity of Weld to Support Web

Calculate the maximum fillet weld size in 16th of an inch for base metal check.

$t_p = 1\ \text{in}$ - Single Plate Thickness

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

$t_{ws} = 0.44\ \text{in}$ - Column Web Thickness

$F_{us} = 65\ \text{ksi}$ - Column Tensile Stress

AISC 15th Ed. Part 9 Eq. (9-2)

D_{max} - Maximum Fillet Weld Size for Base Metal Strength

	$D_{max} = \frac{\frac{t_p}{2}\, F_{up} \le t_{ws}\, F_{us}}{3.09 \text{ kip/in}}$	
	$D_{max} = \frac{(0.5 \text{ in}) \times (65 \text{ ksi}) \le (0.44 \text{ in}) \times (65 \text{ ksi})}{(3.09 \text{ kip/in})}$	
	$D_{max} = 9.2557$	
	Calculate the total effective weld length for NS/FS fillet welds.	
	$W = 0.5 \text{ in}$ - Fillet Weld Size	
	$d_p = 21 \text{ in}$ - Single Plate Depth	
	$l_{w\_min} = 0.3125 \text{ in}$ - Maximum weld length reduction	
	L_w - Total Effective Length of Weld	
	$L_w = 2\ [d_p - 2\ (W \le 0.3125 \text{ in})]$	
	$L_w = 2 \times [(21 \text{ in}) - 2 \times ((0.5 \text{ in}) \le (0.3125 \text{ in}))]$	
	$L_w = 40.75 \text{ in}$	
	Calculate the design capacity of weld in shear.	
	$F_{EXX} = 70 \text{ ksi}$ - Filler Metal Classification Strength	
	$W = 0.5 \text{ in}$ - Fillet Weld Size	
AISC 15th Ed. Part 9 Eq. (9-2)	$D_{max} = 9.2557$ - Maximum Fillet Weld Size for Base Metal Strength	
	$L_w = 40.75 \text{ in}$ - Total Effective Length of Weld	
	$\phi = 0.75$ - Fillet Weld Resistance Factor	
AISC 360-16 Chapter J2.4 Eq. (J2-4)	ϕR_n - Design Strength of Welds	
	$\phi R_n = \phi\, 0.6\, F_{EXX}\, \frac{\sqrt{2}}{2} \left(W \le \frac{D_{max}}{16}\right) L_w$	
	$\phi R_n = (0.75) \times 0.6 \times (70 \text{ ksi}) \times \frac{\sqrt{2}}{2} \times \left((0.5 \text{ in}) \le \frac{(9.2557)}{(16)}\right) \times (40.75 \text{ in})$	
	$\phi R_n = 453.83 \text{ kip}$	
	Result:	**PASS**
	Demand over Capacity Ratio	
	$DCR = \frac{V_u}{\phi R_n} = \frac{(400 \text{ kip})}{(453.83 \text{ kip})} = 0.88139$	

Summary of Checks

Design Checks	Demand	Capacity	DCR	Result
Connection Detailing Limitations at Beam Side	2.667	3.000	0.889	**PASS**
Design Capacity of the Bolt Group in Shear	400.000	1121.549	0.357	**PASS**
Design Capacity of the Bolt Group in Bearing and Tear-out on Single Plate	400.000	2851.875	0.140	**PASS**
Design Capacity of the Bolt Group in Bearing and Tear-out on Beam Web	400.000	2303.438	0.174	**PASS**
Design Capacity of Single Plate in Block Shear	400.000	495.422	0.807	**PASS**
Design Capacity of Single Plate in Shear	400.000	494.813	0.808	**PASS**
Connection Detailing Limitations at Support Side	0.500	8.000	0.063	**PASS**
Design Capacity of Weld to Support Web	400.000	453.830	0.881	**PASS**