REFERENCES	CALCULATIONS	RESULTS
	Shear Connection Calculations **Single Plate Connection | AISC 360-16 LRFD**	
	Design Load/s: $V_u = 30\ \mathrm{kip}$ - Vertical Shear Load	
	Beam Section Properties: $\mathrm{W16x26}$ - Beam Size $d_{bm} = 15.7\ \mathrm{in}$ - Beam Depth $t_{wb} = 0.25\ \mathrm{in}$ - Beam Web Thickness $b_{fb} = 5.5\ \mathrm{in}$ - Beam Flange Width $t_{fb} = 0.345\ \mathrm{in}$ - Beam Flange Thickness $A_{bm} = 7.68\ \mathrm{in}^2$ - Beam Area **Beam Grade Information:** $\mathrm{A992}$ - Material Grade $F_{yb} = 50\ \mathrm{ksi}$ - Beam Yield Stress $F_{ub} = 65\ \mathrm{ksi}$ - Beam Tensile Stress $E = 29000\ \mathrm{ksi}$ - Beam Modulus of Elasticity **Cope Information:** $L_{ct} = 2.875\ \mathrm{in}$ - Length of Cope at Top $D_{ct} = 3\ \mathrm{in}$ - Depth of Cope at Top	
	Girder Section Properties: $\mathrm{W18x35}$ - Girder Size $d_{sup} = 17.7\ \mathrm{in}$ - Girder Depth $t_{ws} = 0.3\ \mathrm{in}$ - Girder Web Thickness $b_{fs} = 6\ \mathrm{in}$ - Girder Flange Width $t_{fs} = 0.425\ \mathrm{in}$ - Girder Flange Thickness $A_{sup} = 10.3\ \mathrm{in}^2$ - Girder Area **Girder Grade Information:** $\mathrm{A992}$ - Material Grade $F_{ys} = 50\ \mathrm{ksi}$ - Girder Yield Stress $F_{us} = 65\ \mathrm{ksi}$ - Girder Tensile Stress $E = 29000\ \mathrm{ksi}$ - Girder Modulus of Elasticity	
	Bolt Information at Beam: $3/4\ \mathrm{in}$ - Bolt Size $\mathrm{A325N}$ - 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 Shear Planes HSSL - Bolt Hole Type at Plate STD - Bolt Hole Type at Beam	
	Weld Information at Girder: $\mathrm{E70XX}$ - Weld Classification $W = 0.3125\ \mathrm{in}$ - Fillet Weld Size $F_{EXX} = 70\ \mathrm{ksi}$ - Filler Metal Classification Strength	
	Single Plate Geometry: $b_p = 4.5\ \mathrm{in}$ - Single Plate Width $d_p = 9\ \mathrm{in}$ - Single Plate Depth $t_p = 0.5\ \mathrm{in}$ - Single Plate Thickness **Single Plate Material Grade:** $F_{yp} = 36\ \mathrm{ksi}$ - Single Plate Yield Stress $F_{up} = 58\ \mathrm{ksi}$ - Single Plate Tensile Stress **Connection Geometry:** $n_r = 3$ - Number of Bolt Rows $s_r = 3\ \mathrm{in}$ - Bolt Row Spacing $n_c = 1$ - Number of Bolt Columns $s_c = 0\ \mathrm{in}$ - Bolt Column Spacing **Distances:** $clr = 0.5\ \mathrm{in}$ - Beam Web Clearance $L_{eh\_bm} = 2\ \mathrm{in}$ - Horizontal Edge Distance on Beam Web $L_{ev\_bm} = 2.8188\ \mathrm{in}$ - Vertical Edge Distance on Top-Coped Beam Web $L_{eh\_pl} = 2\ \mathrm{in}$ - Horizontal Edge Distance on Single Plate $L_{ev\_pl} = 1.5\ \mathrm{in}$ - Vertical Edge Distance on Single Plate $e = 2.5\ \mathrm{in}$ - Bolt Group Eccentricity	

REFERENCES	CALCULATIONS	RESULTS

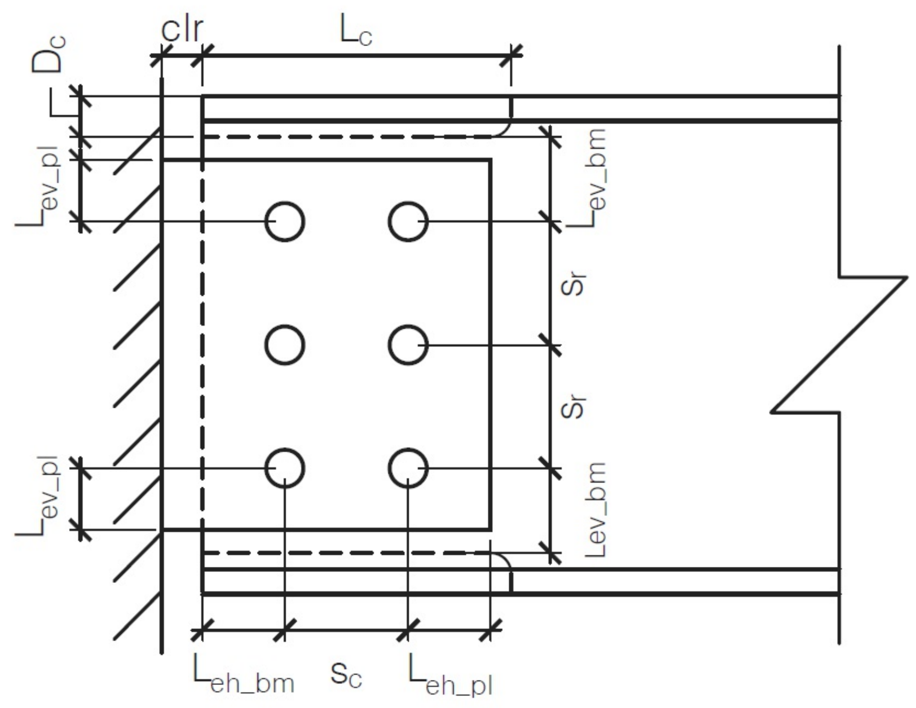

Fig. General detailing of single plate connection.

Check No. 1: Connection Detailing Limitations at Beam Side

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
Plate Minimum Vertical Edge Distance	1.000	1.500	0.667	PASS
Plate Minimum Horizontal Edge Distance	1.125	2.000	0.563	PASS
Beam Minimum Vertical Edge Distance	1.000	2.819	0.355	PASS
Beam Minimum Horizontal Edge Distance	1.000	2.000	0.500	PASS
Minimum Connection Depth	6.813	9.000	0.757	PASS
Maximum Connection Depth	11.563	9.000	0.778	PASS
Minimum Top Cope Depth	2.688	3.000	0.896	PASS

Result:
Demand over Capacity Ratio

$DCR = \frac{d}{c} = \frac{(2.688)}{(3)} = 0.896$

PASS

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

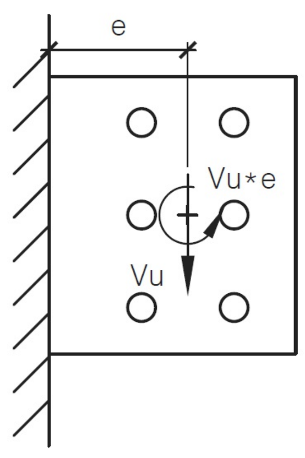

Fig. Bolt group lay-out.

Calculate the C-coefficient of the bolt group.

AISC 15th Edition Table 10-9

$e = 1.25 \text{ in}$ - Design Bolt Group Eccentricity

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

$\phi R_n = 17.892 \text{ kip}$ - Design Shear Capacity of a Bolt

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The bolt coefficient C is derived using the instantaneous center of rotation method. The coordinates of each bolt are listed in the table below.

Bolt No. 1 is located at the bottom-left corner of the bolt group.

Bolt No.	x-coordinate (in)	y-coordinate (in)
1	0.00	0.00
2	0.00	3.00
3	0.00	6.00

AISC 15th Ed. Page 7-6 to 7-8

The location of the instantaneous center with respect to Bolt No. 1 is calculated using iterations. The resulting coordinates of the I.C. are as follows.

$x_{ic} = -3.251 \text{ in}$ - Location of I.C. along the X-axis

$y_{ic} = 3 \text{ in}$ - Location of I.C. along the Y-axis

Using the AISC 15th Edition Equation (7-1), the corresponding reactions per bolt are listed below.

Bolt No.	Ru (kip)	Rux (kip)	Ruy (kip)	Mn (kip-in)
1	17.56	11.91	12.91	77.69
2	17.07	0.00	17.07	55.49
3	17.56	-11.91	12.91	77.69

$C = 2.3966$ - Calculated Bolt Group Coefficient

$C' = 5.889 \text{ in}$ - Calculated Bolt Group Coefficient for Moment-only Case

Calculate the design shear capacity of the bolt group.

$\phi = 0.75$ - Bolt Shear Resistance Factor

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

$C = 2.3966$ - Calculated Bolt Group Coefficient

$F_{nv} = 54 \text{ ksi}$ - 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

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

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

$$h_f = 1$$

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

ϕR_n - Design Bolt Shear Capacity

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

$$\phi R_n = (0.75) \times (54 \text{ ksi}) \times \frac{\pi}{4} \times ((0.75 \text{ in}))^2 \times (1) \times (2.3966) \times (1)$$

$$\phi R_n = 42.88 \text{ kip}$$

Result:
Demand over Capacity Ratio
$DCR = \frac{V_u}{\phi R_n} = \frac{(30 \text{ kip})}{(42.88 \text{ kip})} = 0.69963$

PASS

Check No. 3: Ductility Check making sure Plate Strength does not exceed Bolt Group Strength

$n_r = 3$ - Number of Bolt Rows
$n_c = 1$ - Number of Bolt Columns
STD - Bolt Hole Type at Beam
HSSL - Bolt Hole Type at Plate
$L_{eh\_pl} = 2 \text{ in}$ - Horizontal Edge Distance on Single Plate
$2d_b = 1.5 \text{ in}$ - Twice Bolt Diameter

AISC 15th Ed. page 10-90

Check not applicable since $L_{eh\_pl} \ge 2d_b$.

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

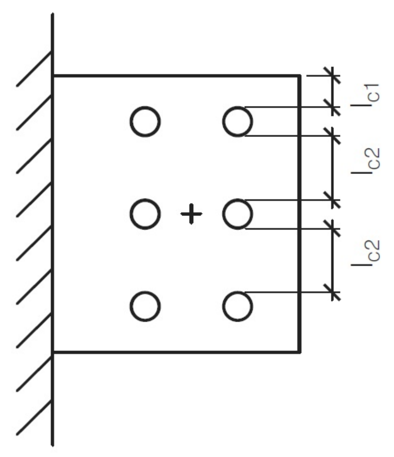

Fig. Clear distances for bolt bearing check on plate.

Calculate the bolt bearing capacity of the single plate.
$d_b = 0.75 \text{ in}$ - Bolt Diameter
$t_p = 0.5 \text{ in}$ - Single Plate Thickness
$F_{up} = 58 \text{ ksi}$ - Single Plate Tensile Stress
$C = 2.3966$ - 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

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

$$\phi R_{n\_bearing} = (0.75) \times 2.4 \times (0.75 \text{ in}) \times (0.5 \text{ in}) \times (58 \text{ ksi}) \times (2.3966)$$

$$\phi R_{n\_bearing} = 93.825 \text{ kip}$$

Calculate the clear distance of outer bolts on single plate.
$L_{ev\_pl} = 1.5 \text{ in}$ - Vertical Edge Distance on Single Plate
$d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Plate
l_{c1} - Clear Distance at First Bolt Row

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

$$l_{c1} = (1.5 \text{ in}) - \frac{(0.8125 \text{ in})}{2}$$

$$l_{c1} = 1.0938 \text{ in}$$

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

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

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

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

Calculate the bolt tear-out capacity of the single plate.
$l_{c1} = 1.0938 \text{ in}$ - Clear Distance at First Bolt Row
$l_{c2} = 2.1875 \text{ in}$ - Clear Distance at Rest of Bolts
$t_p = 0.5 \text{ in}$ - Single Plate Thickness
$F_{up} = 58 \text{ ksi}$ - Single Plate Tensile Stress

$C = 2.3966$ - Calculated Bolt Group Coefficient
$n_r = 3$ - 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

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

$$\phi R_{n\_tearout} = (0.75) \times [1.2 \times (1.0938 \text{ in}) \times (0.5 \text{ in}) \times (58 \text{ ksi}) + 1.2 \times (2.1875 \text{ in}) \times (0.5 \text{ in}) \times (58 \text{ ksi}) \times ((3) - 1)] \times \left(\frac{(2.3966)}{(3)}\right)$$

$$\phi R_{n\_tearout} = 114.02 \text{ kip}$$

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

AISC 360-16 Chapter J3.10 Eq. (J3-6a) $\phi R_{n\_bearing} = 93.825 \text{ kip}$ - Design bolt bearing capacity of single plate

AISC 360-16 Chapter J3.10 Eq. (J3-6c) $\phi R_{n\_tearout} = 114.02 \text{ kip}$ - Design bolt tear-out capacity of single plate

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

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

$$\phi R_n = min\left((93.825 \text{ kip}), (114.02 \text{ kip})\right)$$

$$\phi R_n = 93.825 \text{ kip}$$

Result:
Demand over Capacity Ratio
$DCR = \frac{V_u}{\phi R_n} = \frac{(30 \text{ kip})}{(93.825 \text{ kip})} = 0.31974$

PASS

Check No. 5: Design Capacity of the Bolt Group in Bearing and Tear-out on Beam Web

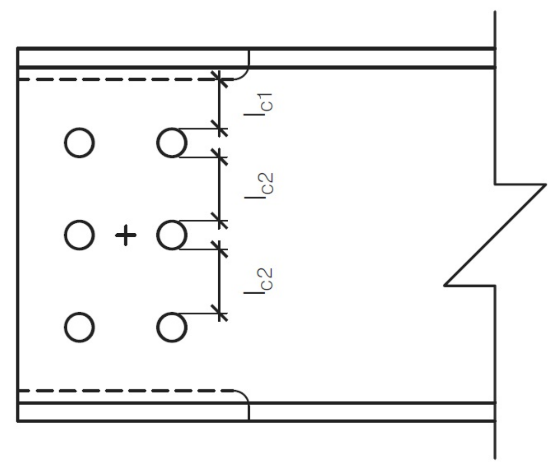

Fig. Clear distances for bolt bearing check on member.

Calculate the bolt bearing capacity of the beam web.
$d_b = 0.75 \text{ in}$ - Bolt Diameter
$t_{wb} = 0.25 \text{ in}$ - Beam Web Thickness
$F_{ub} = 65 \text{ ksi}$ - Beam Tensile Stress
$C = 2.3966$ - 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

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

$$\phi R_{n\_bearing} = (0.75) \times 2.4 \times (0.75 \text{ in}) \times (0.25 \text{ in}) \times (65 \text{ ksi}) \times (2.3966)$$

$$\phi R_{n\_bearing} = 52.574 \text{ kip}$$

Calculate the clear distance of outer bolts on beam web.
$L_{ev\_bm} = 2.8188 \text{ in}$ - Vertical Edge Distance on Top-Coped Beam Web
$L_{ev\_bm} = 2.8188 \text{ in}$ - Vertical Edge Distance on Bottom-Coped Beam Web
$d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Beam
l_{c1} - Clear Distance at First Bolt Row

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

$$l_{c1} = (2.8188 \text{ in}) - \frac{(0.8125 \text{ in})}{2}$$

$$l_{c1} = 2.4125 \text{ in}$$

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

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

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

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

Calculate the bolt tear-out capacity of the beam web.
$l_{c1} = 2.4125 \text{ in}$ - Clear Distance at First Bolt Row
$l_{c2} = 2.1875 \text{ in}$ - Clear Distance at Rest of Bolts
$t_{wb} = 0.25 \text{ in}$ - Beam Web Thickness
$F_{ub} = 65 \text{ ksi}$ - Beam Tensile Stress
$C = 2.3966$ - Calculated Bolt Group Coefficient
$n_r = 3$ - 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

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

$$\phi R_{n\_tearout} = (0.75) \times [1.2 \times (2.4125\ \text{in}) \times (0.25\ \text{in}) \times (65\ \text{ksi}) + 1.2 \times (2.1875\ \text{in}) \times (0.25\ \text{in}) \times (65\ \text{ksi}) \times ((3) - 1)] \times \left(\frac{(2.3966)}{(3)}\right)$$

$$\phi R_{n\_tearout} = 79.3\ \text{kip}$$

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

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

$\phi R_{n\_bearing} = 52.574\ \text{kip}$ - Design bolt bearing capacity of beam web

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

$\phi R_{n\_tearout} = 79.3\ \text{kip}$ - Design bolt tear-out capacity of beam web

AISC 360-16 Chapter J3.10

ϕR_n - Governing Design Capacity

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

$$\phi R_n = min\left((52.574\ \text{kip}), (79.3\ \text{kip})\right)$$

$$\phi R_n = 52.574\ \text{kip}$$

Result:

Demand over Capacity Ratio

$DCR = \frac{V_u}{\phi R_n} = \frac{(30\ \text{kip})}{(52.574\ \text{kip})} = 0.57062$

PASS

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

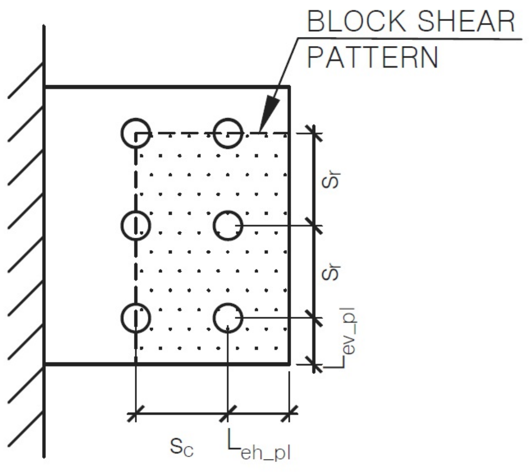

Fig. Single plate block shear pattern for shear load.

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

$t_p = 0.5\ \text{in}$ - Single Plate Thickness

$n_c = 1$ - Number of Bolt Columns

$s_c = 0\ \text{in}$ - Bolt Column Spacing

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

$d_h = 1\ \text{in}$ - Horizontal Bolt Hole Dimension at Plate

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

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

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

$$A_{nt} = 0.73438\ \text{in}^2$$

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

$t_p = 0.5\ \text{in}$ - Single Plate Thickness

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

$n_r = 3$ - Number of Bolt Rows

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

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

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

$$A_{gv} = (0.5\ \text{in}) \times [(1.5\ \text{in}) + ((3) - 1) \times (3\ \text{in})]$$

$$A_{gv} = 3.75\ \text{in}^2$$

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

$t_p = 0.5\ \text{in}$ - Single Plate Thickness

$d_p = 9\ \text{in}$ - Single Plate Depth

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

$n_r = 3$ - Number of Bolt Rows

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

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

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

$$A_{nv} = (0.5\ \text{in}) \times [(9\ \text{in}) - (1.5\ \text{in}) - ((3) - 0.5) \times ((0.8125\ \text{in}) + (0.0625\ \text{in}))]$$

$$A_{nv} = 2.6563\ \text{in}^2$$

Calculate the design block shear capacity of the single plate.

$F_{up} = 58\ \text{ksi}$ - Single Plate Tensile Stress

$A_{nv} = 2.6563\ \text{in}^2$ - Net Area Subject to Shear (L-pattern)

$U_{bs} = 1$ - Uniformity factor for single line of bolts

$A_{nt} = 0.73438 \text{ in}^2$ - Net Area Subject to Tension (L-pattern)
$F_{yp} = 36 \text{ ksi}$ - Single Plate Yield Stress
$A_{gv} = 3.75 \text{ in}^2$ - 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

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

$$\phi R_n = (0.75) \times \left(0.6 \times (58 \text{ ksi}) \times \left(2.6563 \text{ in}^2\right) + (1) \times (58 \text{ ksi}) \times \left(0.73438 \text{ in}^2\right) \leq 0.6 \times (36 \text{ ksi}) \times \left(3.75 \text{ in}^2\right) + (1) \times (58 \text{ ksi}) \times \left(0.73438 \text{ in}^2\right)\right)$$

$$\phi R_n = 92.695 \text{ kip}$$

Result:
Demand over Capacity Ratio
$DCR = \frac{V_u}{\phi R_n} = \frac{(30 \text{ kip})}{(92.695 \text{ kip})} = 0.32364$

PASS

Check No. 7: Design Capacity of Single Plate in Shear

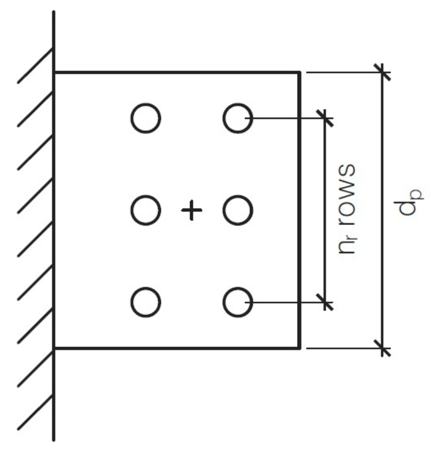

Fig. Gross and net section areas for single plate.

Calculate the gross area of single plate subject to yielding.
$d_p = 9 \text{ in}$ - Single Plate Depth
$t_p = 0.5 \text{ in}$ - Single Plate Thickness
A_{gv} - Section Gross Area

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

$$A_{gv} = (9 \text{ in}) \times (0.5 \text{ in})$$

$$A_{gv} = 4.5 \text{ in}^2$$

Calculate the shear yielding capacity of the single plate.
$F_{yp} = 36 \text{ ksi}$ - Single Plate Yield Stress
$A_{gv} = 4.5 \text{ in}^2$ - 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

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

$$\phi R_{n\_sy} = (1) \times 0.6 \times (36 \text{ ksi}) \times \left(4.5 \text{ in}^2\right)$$

$$\phi R_{n\_sy} = 97.2 \text{ kip}$$

Calculate the net area of single plate subject to rupture.
$t_p = 0.5 \text{ in}$ - Single Plate Thickness
$d_p = 9 \text{ in}$ - Single Plate Depth
$n_r = 3$ - Number of Bolt Rows
$d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Plate
A_{nv} - Section Net Area

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

$$A_{nv} = (0.5 \text{ in}) \times [(9 \text{ in}) - (3) \times ((0.8125 \text{ in}) + (0.0625 \text{ in}))]$$

$$A_{nv} = 3.1875 \text{ in}^2$$

Calculate the shear rupture capacity of the single plate.
$F_{up} = 58 \text{ ksi}$ - Single Plate Tensile Stress
$A_{nv} = 3.1875 \text{ in}^2$ - 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

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

$$\phi R_{n\_sr} = (1) \times 0.6 \times (58 \text{ ksi}) \times \left(3.1875 \text{ in}^2\right)$$

$$\phi R_{n\_sr} = 110.92 \text{ kip}$$

Determine the governing shear capacity of the single plate.

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

$\phi R_{n\_sy} = 97.2 \text{ kip}$ - Design shear yielding capacity of single plate

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

$\phi R_{n\_sr} = 110.92 \text{ kip}$ - Design shear rupture capacity of single plate

AISC 360-16 Chapter J4.2

ϕR_n - Governing Design Capacity

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

$$\phi R_n = min\left((97.2\ \text{kip}), (110.92\ \text{kip})\right)$$

$$\phi R_n = 97.2\ \text{kip}$$

Result:
Demand over Capacity Ratio
$DCR = \frac{V_u}{\phi R_n} = \frac{(30\ \text{kip})}{(97.2\ \text{kip})} = 0.30864$

PASS

Check No. 8: Design Capacity of Single Plate in Flexure Using Double Coped Procedure

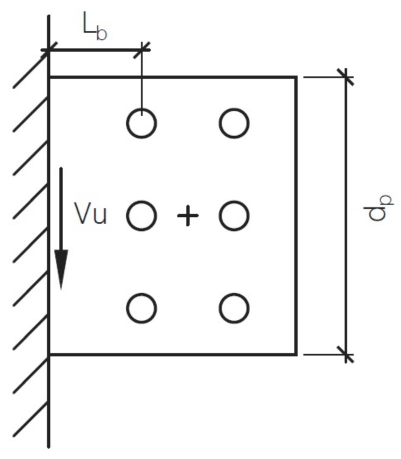

Fig. Single plate subject to flexure.

Calculate and compare upper and lower limits.
$E = 29000\ \text{ksi}$ - Modulus for Steel
$F_{yp} = 36\ \text{ksi}$ - Single Plate Yield Stress
$L_b = 2.5\ \text{in}$ - Single Plate Unbraced Length
$d_p = 9\ \text{in}$ - Single Plate Depth
$t_p = 0.5\ \text{in}$ - Single Plate Thickness

Depth to Thickness Ratio
$\frac{L_b d}{t^2} = \frac{(2.5\ \text{in})\times(9\ \text{in})}{(0.5\ \text{in})^2} = 90$

Depth to Thickness Ratio Lower Limit
$\frac{0.08E}{F_y} = \frac{0.08\times(29000\ \text{ksi})}{(36\ \text{ksi})} = 64.444$

Depth to Thickness Ratio Upper Limit
$\frac{1.9E}{F_y} = \frac{1.9\times(29000\ \text{ksi})}{(36\ \text{ksi})} = 1530.6$

So, $\frac{0.08E}{F_y} < \frac{L_b d}{t^2}$ and $\frac{L_b d}{t^2} \le \frac{1.9E}{F_y}$.

Calculate the plastic section modulus of the single plate.
$t_p = 0.5\ \text{in}$ - Single Plate Thickness
$d_p = 9\ \text{in}$ - Single Plate Depth
Z - Plastic Section Modulus

$$Z = \frac{t_p\left(d_p\right)^2}{4}$$

$$Z = \frac{(0.5\ \text{in}) \times \left((9\ \text{in})\right)^2}{4}$$

$$Z = 10.125\ \text{in}^3$$

Calculate the elastic section modulus of the single plate.
$t_p = 0.5\ \text{in}$ - Single Plate Thickness
$d_p = 9\ \text{in}$ - Single Plate Depth
S - Elastic Section Modulus

$$S = \frac{t_p\left(d_p\right)^2}{6}$$

$$S = \frac{(0.5\ \text{in}) \times \left((9\ \text{in})\right)^2}{6}$$

$$S = 6.75\ \text{in}^3$$

Calculate the lateral torsional buckling modification factor.
$L_b = 2.5\ \text{in}$ - Single Plate Unbraced Length
$d_p = 9\ \text{in}$ - Single Plate Depth
$d_{ct} = 0\ \text{in}$ - Cope Depth at Single Plate (assume 0)

AISC 15th Edition pg. (9-9) Eq. (9-15)

C_b - Lateral-Torsional Buckling Modification Factor

$$C_b = \left[3 + \ln\left(\frac{L_b}{d_p}\right)\right]\left(1 - \frac{D_{ct}}{d_p}\right) \geq 1.84$$

$$C_b = \left[3 + \ln\left(\frac{(2.5\ \text{in})}{(9\ \text{in})}\right)\right] \times \left(1 - \frac{(3\ \text{in})}{(9\ \text{in})}\right) \geq 1.84$$

$$C_b = 1.84$$

Calculate the flexural capacity of the single plate.

AISC 15th Edition pg. (9-9) Eq. (9-15)

$C_b = 1.84$ - Lateral-Torsional Buckling Modification Factor
$\frac{L_b d}{t^2} = 90$ - Depth to Thickness Ratio

$F_{yp} = 36 \text{ ksi}$ - Single Plate Yield Stress

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

$S = 6.75 \text{ in}^3$ - Elastic Section Modulus

$Z = 10.125 \text{ in}^3$ - Plastic Section Modulus

$\phi = 0.9$ - Flexure Resistance Factor

AISC 360-16 Chapter F11.2 Eq. (F11-2)

ϕM_n - Design Flexural Capacity of Coped Section

$$\phi M_n = \phi \left[C_b \left[1.52 - 0.274 \left(\frac{L_b d}{t^2}\right) \left(\frac{F_{yp}}{E}\right)\right] F_{yp}\, S \le F_{yp}\, Z\right]$$

$$\phi M_n = (0.9) \times \left[(1.84) \times \left[1.52 - 0.274 \times ((90)) \times \left(\frac{(36 \text{ ksi})}{(29000 \text{ ksi})}\right)\right] \times (36 \text{ ksi}) \times \left(6.75 \text{ in}^3\right) \le (36 \text{ ksi}) \times \left(10.125 \text{ in}^3\right)\right]$$

$$\phi M_n = 27.337 \text{ kipft}$$

Calculate the equivalent shear capacity of the single plate.

AISC 360-16 Chapter F11.2 Eq. (F11-2)

$\phi M_n = 27.337 \text{ kipft}$ - Design Flexural Capacity of Coped Section

$L_b = 2.5 \text{ in}$ - Single Plate Unbraced Length

ϕV_n - Design Capacity of Single Plate in Bending

$$\phi V_n = \frac{\phi M_n}{L_b}$$

$$\phi V_n = \frac{(27.337 \text{ kipft})}{(2.5 \text{ in})}$$

$$\phi V_n = 131.22 \text{ kip}$$

Result: **PASS**

Demand over Capacity Ratio

$DCR = \frac{V_u}{\phi V_n} = \frac{(30 \text{ kip})}{(131.22 \text{ kip})} = 0.22862$

Check No. 9: Shear and Moment Interaction of Single Plate

Calculate the shear and moment interaction of the single plate.

$V_r = 30 \text{ kip}$ - Vertical Shear Load

Moment at Component

$M_r = V_r\, L_b = (30 \text{ kip}) \times (2.5 \text{ in}) = 6.25 \text{ kipft}$

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

$\phi V_n = 97.2 \text{ kip}$ - Design shear yielding capacity of single plate

AISC 360-16 Chapter F11.2 Eq. (F11-2)

$\phi M_n = 27.337 \text{ kipft}$ - Design Flexural Capacity of Single Plate

AISC 15th Edition pg. (10-90) Eq. (10-5)

I_{nt} - Interaction for Shear Yielding/Buckling and Flexure

$$I_{nt} = \left(\frac{V_r}{\phi V_n}\right)^2 + \left(\frac{M_r}{\phi M_n}\right)^2$$

$$I_{nt} = \left(\frac{(30 \text{ kip})}{(97.2 \text{ kip})}\right)^2 + \left(\frac{(6.25 \text{ kipft})}{(27.337 \text{ kipft})}\right)^2$$

$$I_{nt} = 0.14753$$

Result: **PASS**

Demand over Capacity Ratio

$DCR = \frac{I_{nt}}{1.0} = \frac{(0.14753)}{(1)} = 0.14753$

Check No. 10: Design Capacity of Coped Beam in Block Shear

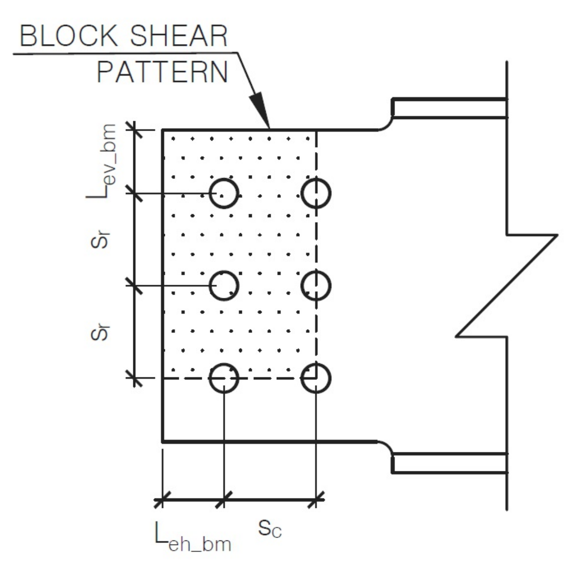

Fig. Beam web block shear pattern for shear load.

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

$t_{wb} = 0.25 \text{ in}$ - Beam Web Thickness

$n_c = 1$ - Number of Bolt Columns

$s_c = 0 \text{ in}$ - Bolt Column Spacing

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

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

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

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

$$A_{nt} = (0.25 \text{ in}) \times \left[((1) - 1) \times (0 \text{ in}) + (2 \text{ in}) - ((1) - 0.5) \times ((0.8125 \text{ in}) + (0.0625 \text{ in}))\right]$$

$$A_{nt} = 0.39063 \text{ in}^2$$

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

$t_{wb} = 0.25 \text{ in}$ - Beam Web Thickness

$L_{ev\_bm} = 2.8188 \text{ in}$ - Vertical Edge Distance on Top-Coped Beam Web

$n_r = 3$ - Number of Bolt Rows
$s_r = 3$ in - Bolt Row Spacing
A_{gv} - Gross Area Subject to Shear (L-pattern)

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

$$A_{gv} = (0.25\ \text{in}) \times [(2.8188\ \text{in}) + ((3) - 1) \times (3\ \text{in})]$$

$$A_{gv} = 2.2047\ \text{in}^2$$

Calculate the net area of the beam web subject to shear.
$t_{wb} = 0.25$ in - Beam Web Thickness
$d_{coped} = 11.637$ in - Depth of Web Remaining at Cope
$L_{ev\_bm} = 2.8188$ in - Vertical Edge Distance on Top-Coped Beam Web
$L_{ev\_bm} = 2.8188$ in - Vertical Edge Distance on Bottom-Coped Beam Web
$n_r = 3$ - Number of Bolt Rows
$d_h = 0.8125$ in - Horizontal Bolt Hole Dimension at Beam
A_{nv} - Net Area Subject to Shear (L-pattern)

$$A_{nv} = t_{wb}\ [d_{coped} - L_{ev\_bm} - (n_r - 0.5)\ (d_h + 0.0625\ \text{in})]$$

$$A_{nv} = (0.25\ \text{in}) \times [(11.637\ \text{in}) - (2.8188\ \text{in}) - ((3) - 0.5) \times ((0.8125\ \text{in}) + (0.0625\ \text{in}))]$$

$$A_{nv} = 1.6578\ \text{in}^2$$

Calculate the design block shear capacity of the coped beam web.
$F_{ub} = 65$ ksi - Beam Tensile Stress
$A_{nv} = 1.6578\ \text{in}^2$ - Net Area Subject to Shear (L-pattern)
$U_{bs} = 1$ - Uniformity factor for single line of bolts
$A_{nt} = 0.39063\ \text{in}^2$ - Net Area Subject to Tension (L-pattern)
$F_{yb} = 50$ ksi - Beam Yield Stress
$A_{gv} = 2.2047\ \text{in}^2$ - 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

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

$$\phi R_n = (0.75) \times \left(0.6 \times (65\ \text{ksi}) \times \left(1.6578\ \text{in}^2\right) + (1) \times (65\ \text{ksi}) \times \left(0.39063\ \text{in}^2\right) \leq 0.6 \times (50\ \text{ksi}) \times \left(2.2047\ \text{in}^2\right) + (1) \times (65\ \text{ksi}) \times \left(0.39063\ \text{in}^2\right)\right)$$

$$\phi R_n = 67.534\ \text{kip}$$

Result:
Demand over Capacity Ratio
$DCR = \frac{V_r}{\phi R_n} = \frac{(30\ \text{kip})}{(67.534\ \text{kip})} = 0.44422$

PASS

Check No. 11: Design Capacity of Coped Beam in Shear

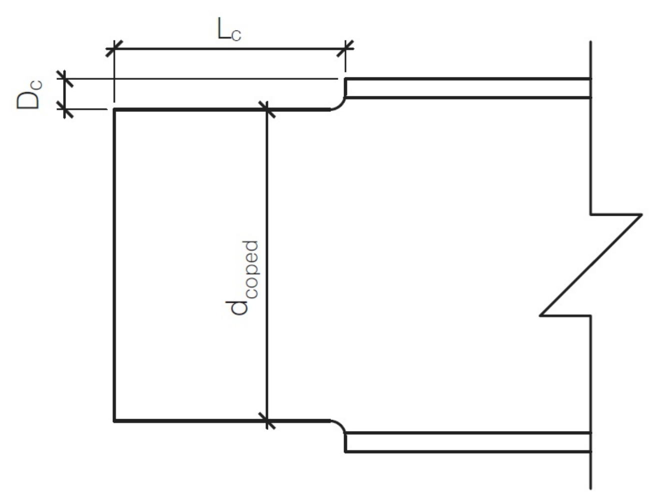

Fig. Gross section area for coped beam web.

Calculate the gross area of beam web subject to yielding.
$d_{coped} = 12.7$ in - Depth of Web Remaining at Cope
$t_{wb} = 0.25$ in - Beam Web Thickness
A_{gv} - Section Gross Area

$$A_{gv} = d_{coped}\ t_{wb}$$

$$A_{gv} = (12.7\ \text{in}) \times (0.25\ \text{in})$$

$$A_{gv} = 3.175\ \text{in}^2$$

Calculate the shear yielding capacity of the beam web.
$F_{yb} = 50$ ksi - Beam Yield Stress
$A_{gv} = 3.175\ \text{in}^2$ - 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

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

$$\phi R_{n\_sy} = (1) \times 0.6 \times (50\ \text{ksi}) \times \left(3.175\ \text{in}^2\right)$$

$$\phi R_{n\_sy} = 95.25\ \text{kip}$$

Result:
Demand over Capacity Ratio
$DCR = \frac{V_r}{\phi R_{n\_sy}} = \frac{(30\ \text{kip})}{(95.25\ \text{kip})} = 0.31496$

PASS

Check No. 12: Design Capacity of Coped Beam in Flexure

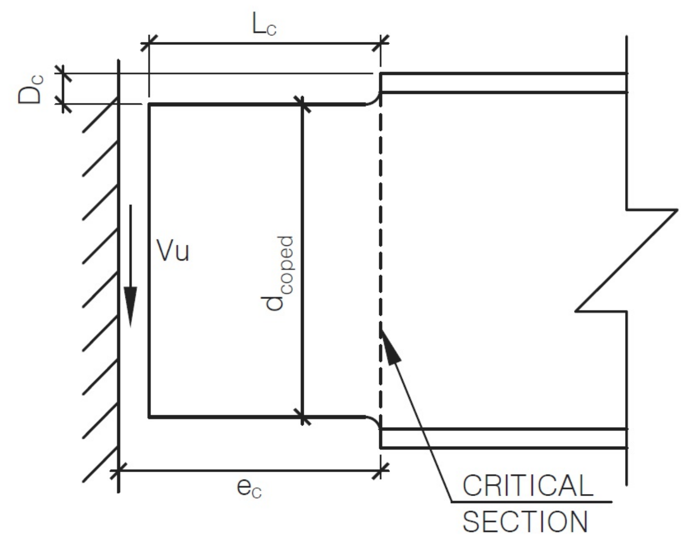

Fig. Coped beam web subject to flexure.

$E = 29000\ \mathrm{ksi}$ - Modulus for Steel
$F_{yb} = 50\ \mathrm{ksi}$ - Beam Yield Stress
$D_{ct} = 3\ \mathrm{in}$ - Depth of Cope at Top
$L_{ct} = 2.875\ \mathrm{in}$ - Length of Cope at Top
$D_{cb} = 0\ \mathrm{in}$ - Depth of Cope at Bottom
$L_{cb} = 0\ \mathrm{in}$ - Length of Cope at Bottom
$\phi = 0.9$ - Flexure Resistance Factor

$S_x = 10.089\ \mathrm{in}^3$ - Coped Beam Elastic Section Modulus
$Z_x = 18.464\ \mathrm{in}^3$ - Coped Beam Plastic Section Modulus

AISC 15th Edition pg. (9-7) Eq. (9-11)

Web Slenderness

$\lambda = \frac{h_o}{t_{wb}} = \frac{(12.7\ \mathrm{in})}{(0.25\ \mathrm{in})} = 50.8$

Since, $c \leq d_{bm}$.

AISC 15th Edition pg. (9-8) Eq. (9-14a)

Buckling Adjustment factor

$f = 2\left(\frac{c}{d_{bm}}\right) = 2 \times \left(\frac{(2.875\ \mathrm{in})}{(15.7\ \mathrm{in})}\right) = 0.36624$

Since, $c \leq h_o$.

AISC 15th Edition pg. (9-8) Eq. (9-13a)

Plate Buckling coefficient

$k = 2.2\left(\frac{h_o}{c}\right)^{1.65} = 2.2 \times \left(\frac{(12.7\ \mathrm{in})}{(2.875\ \mathrm{in})}\right)^{1.65} = 25.524$

AISC 15th Edition pg. (9-8) Eq. (9-12)

λ_p - Limiting Slenderness for a compact web

$$\lambda_p = 0.475\sqrt{\frac{(f\,k \geq 1.61)\,E}{F_{yb}}}$$

$$\lambda_p = 0.475 \times \sqrt{\frac{((0.36624) \times (25.524) \geq 1.61) \times (29000\ \mathrm{ksi})}{(50\ \mathrm{ksi})}}$$

$$\lambda_p = 34.976$$

Since, $\lambda_p < \lambda$ and $\lambda \leq 2\lambda_p$.

AISC 15th Edition pg. (9-7) Eq. (9-7)

ϕM_n - Design Flexural Capacity of Coped Section

$$\phi M_n = \phi\left[F_{yb}\,Z_x - (F_{yb}\,Z_x - F_{yb}\,S_x)\left(\frac{\lambda}{\lambda_p} - 1\right)\right]$$

$$\phi M_n = (0.9) \times \left[(50\ \mathrm{ksi}) \times \left(18.464\ \mathrm{in}^3\right) - \left((50\ \mathrm{ksi}) \times \left(18.464\ \mathrm{in}^3\right) - (50\ \mathrm{ksi}) \times \left(10.089\ \mathrm{in}^3\right)\right) \times \left(\frac{(50.8)}{(34.976)} - 1\right)\right]$$

$$\phi M_n = 55.031\ \mathrm{kipft}$$

Calculate the equivalent shear capacity of the coped beam.

AISC 15th Edition pg. (9-7) Eq. (9-7)

$\phi M_n = 55.031\ \mathrm{kipft}$ - Design Flexural Capacity of Coped Section
$e_c = 3.375\ \mathrm{in}$ - Cope Eccentricity
ϕV_n - Design Capacity of Coped Beam in Bending

$$\phi V_n = \frac{\phi M_n}{e_c}$$

$$\phi V_n = \frac{(55.031\ \mathrm{kipft})}{(3.375\ \mathrm{in})}$$

$$\phi V_n = 195.66\ \mathrm{kip}$$

Result: **PASS**

Demand over Capacity Ratio

$DCR = \frac{V_r}{\phi V_n} = \frac{(30\ \mathrm{kip})}{(195.66\ \mathrm{kip})} = 0.15332$

Check No. 13: Connection Detailing Limitations at Support Side

Detailing Limitations	Limit Value (in)	Actual Value (in)	DCR	Result
Maximum Fillet Weld Size per Beam Clearance	0.500	0.313	0.625	PASS

Result: **PASS**

Demand over Capacity Ratio

$DCR = \frac{d}{c} = \frac{(0.3125)}{(0.5)} = 0.625$

Check No. 14: Required Weld to Develop Single Plate at Support Web

Check the weld size at support if it can develop the single plate.

$W = 0.3125\ \mathrm{in}$ - Fillet Weld Size

AISC 15th Edition page 10-87

W_{duc} - Weld required to develop Plate

$$W_{duc} = \frac{5}{8}\, t_p$$

$$W_{duc} = \frac{5}{8} \times (0.5 \text{ in})$$

$$W_{duc} = 0.3125 \text{ in}$$

Result:

Demand over Capacity Ratio

$DCR = \frac{W_{duc}}{W} = \frac{(0.3125 \text{ in})}{(0.3125 \text{ in})} = 1$

PASS

Check No. 15: Design Capacity of Weld to Support Web

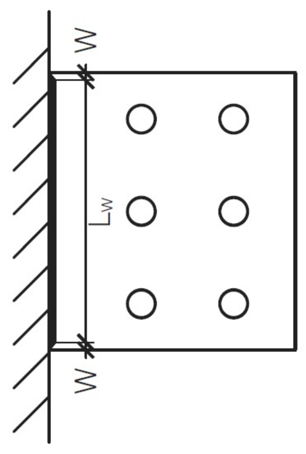

Fig. Fillet weld lines at the support.

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

$t_p = 0.5 \text{ in}$ - Single Plate Thickness

$F_{up} = 58 \text{ ksi}$ - Single Plate Tensile Stress

$t_{ws} = 0.3 \text{ in}$ - Girder Web Thickness

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

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

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

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

$$D_{max} = \frac{(0.25 \text{ in}) \times (58 \text{ ksi}) \leq (0.3 \text{ in}) \times (65 \text{ ksi})}{(3.09 \text{ kip/in})}$$

$$D_{max} = 4.6926$$

Calculate the total effective weld length for NS/FS fillet welds.

$W = 0.3125 \text{ in}$ - Fillet Weld Size

$d_p = 9 \text{ in}$ - Single Plate Depth

$l_{w\_min} = 0.3125 \text{ in}$ - Maximum weld length reduction

L_w - Total Effective Length of Weld

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

$$L_w = 2 \times [(9 \text{ in}) - 2 \times ((0.3125 \text{ in}) \leq (0.3125 \text{ in}))]$$

$$L_w = 16.75 \text{ in}$$

Calculate the design capacity of weld in shear.

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

$W = 0.3125 \text{ in}$ - Fillet Weld Size

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

$D_{max} = 4.6926$ - Maximum Fillet Weld Size for Base Metal Strength

$L_w = 16.75 \text{ in}$ - 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

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

$$\phi R_n = (0.75) \times 0.6 \times (70 \text{ ksi}) \times \frac{\sqrt{2}}{2} \times \left((0.3125 \text{ in}) \leq \frac{(4.6926)}{(16)}\right) \times (16.75 \text{ in})$$

$$\phi R_n = 109.42 \text{ kip}$$

Result:

Demand over Capacity Ratio

$DCR = \frac{V_r}{\phi R_n} = \frac{(30 \text{ kip})}{(109.42 \text{ kip})} = 0.27417$

PASS

REFERENCES	CALCULATIONS	RESULTS

Summary of Checks

Design Checks	Demand	Capacity	DCR	Result
Connection Detailing Limitations at Beam Side	2.688	3.000	0.896	PASS
Design Capacity of the Bolt Group in Shear	30.000	42.880	0.700	PASS
Design Capacity of the Bolt Group in Bearing and Tear-out on Single Plate	30.000	93.825	0.320	PASS
Design Capacity of the Bolt Group in Bearing and Tear-out on Beam Web	30.000	52.574	0.571	PASS
Design Capacity of Single Plate in Block Shear	30.000	92.695	0.324	PASS
Design Capacity of Single Plate in Shear	30.000	97.200	0.309	PASS
Design Capacity of Single Plate in Flexure Using Double Coped Procedure	30.000	131.220	0.229	PASS
Shear and Moment Interaction of Single Plate	0.148	1.000	0.148	PASS
Design Capacity of Coped Beam in Block Shear	30.000	67.534	0.444	PASS
Design Capacity of Coped Beam in Shear	30.000	95.250	0.315	PASS
Design Capacity of Coped Beam in Flexure	30.000	195.665	0.153	PASS
Connection Detailing Limitations at Support Side	0.313	0.500	0.625	PASS
Required Weld to Develop Single Plate at Support Web	0.313	0.313	1.000	PASS
Design Capacity of Weld to Support Web	30.000	109.421	0.274	PASS