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



REFERENCES	CALCULATIONS	RESULTS			
	Flange Plate Connection AISC 360-16 LRFD				
	Flange Plate Geometry:				
	$b_{fp}=16~{ m in}$ - Flange Plate Width				
	$L_{fp}=45~{ m in}$ - Flange Plate Length				
	$t_{fp}=1.5~{ m in}$ - Flange Plate Thickness				
	Flange Plate Material Grade:				
	$F_{yp} = 50$ ksi - Flange Plate Yield Stress $F_{i} = 65$ ksi - Flange Plate Tensile Stress				
	$\Gamma_{up} = 00$ km - hange hate lensie Stress				
	$n_r = 4$ - Number of Bolt Rows				
	$g_a = 5.5 ~{ m in}$ - Bolt Gage				
	$s_r=3~{ m in}$ - Bolt Row Spacing				
	$n_c = 12$ - Number of Bolt Columns				
	Distances:				
	$L_{ev_bf}=2~{ m in}$ - Vertical Edge Distance on Beam Flange				
	$L_{eh_bf} = 1.25~{ m in}$ - Horizontal Edge Distance on Beam Flange				
	$L_{ev_bf}=2~{ m in}$ - Vertical Edge Distance on Beam Flange				
	$L_{eh_fp}=2.25~{ m in}$ - Horizontal Edge Distance on Flange Plate				
	$L_{b_fp} = 10~{ m in}$ - 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 Spacing6.0003.0000.500PASS				
	Minimum Bolt Column Spacing2.6673.0000.889PASS				
	Maximum Bolt Column Spacing6.0003.0000.500PASS				
	Plate Minimum Vertical Edge Distance1.3752.0000.688PASS				
	Plate Minimum Horizontal Edge Distance1.3752.2500.611PASS				
	Beam Minimum Vertical Edge Distance1.2502.0000.625PASS				
	Beam Minimum Horizontal Edge Distance1.2501.000PASS				
	Result:	PASS			
	Demand over Capacity Ratio				
	$DCR = \frac{d}{c} = \frac{(1.25)}{(1.25)} = 1$				
	Check No. 2: Design Capacity of the Bolts in Shear				
	P_{uF} - Equivalent Flange Force from Strong Axis Moment				
	M D				
	$P_{uF}=rac{M_{ux}}{\left(d_{bm}-t_{fb} ight)}+rac{F_{u}}{2}$				
	$P_{\rm r} = -\frac{(500 \text{ kipft})}{(250 \text{ kip})}$				
	$\Gamma_{uF} = {((28.1 ext{ in}) - (1.34 ext{ in}))}^+ {(28.1 ext{ in})}^-$				
	$P_{uF}=349.22$ кір				
	$\phi=0.85$ - Bolt Shear Resistance Factor				
	$\mu=0.5$ - Mean Slip Coefficient				
	$D_u = 1.13$ $h_f = 0.85$ - Filler Factor for Slip Critical Bolts				
	$d_b=1~{ m in}$ - Bolt Diameter				
	$T_b=64~{ m kip}$ - Minimum Bolt Pretension				
	$N_s=1$ - Number of Slip Planes				
	$n_r=4$ - Number of Bolt Columns				
AISC 360-16 Chapter J3.8	ϕR_{r} - Design Bolt Shear Capacity				
Eq. (J3-4)					
	$\phi R_n = \phi \ \mu \ D_u \ h_f \ T_b \ N_s \ n_r \ n_c$				
	$dR = (0.85) \times (0.5) \times (1.12) \times (0.85) \times (64 \text{ km}) \times (1) \times (4) \times (12)$				
	$\varphi_{11n} = (0.00) \times (0.0) \times (0.00) \times (04 \text{ kip}) \times (1) \times (4) \times (12)$				
	$\phi R_n = 1254 ~{ m kip}$				
	Result:	PASS			
	Demand over Capacity Ratio				
	$DCR = rac{P_{uF}}{\phi R_n} = rac{(349.22 ext{ kip})}{(1254 ext{ kip})} = 0.27847$				
	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				
	$d_b=1~{ m in}$ - Bolt Diameter				
	$t_{fp}=1.5~{ m in}$ - Flange Plate Thickness				
	$r_{up} = 00$ KSI - Fidinge Plate lensile Stress $n_{up} = 12$ Number of Polt Columns				
	$n_c = 12$ - Number of Bolt Columns $n_r = 4$ - Number of Bolt Rows				
AISC 360-16 Chapter J3.10	$\phi R_{n_bearing}$ - Design Bolt Bearing Capacity of Section				
⊑ү. (ј3-6а)					
	$\phi R_{n_bearing} = \phi \ 2.4 \ d_b \ t_{fp} \ F_{up} \ n_c \ n_r$				





$\phi R_{n_bearing} = 8424 ext{ kip}$

Calculate the clear distance of outer bolts on flange plate.

 $L_{ev_fp}=2 ext{ in}$ - Vertical Edge Distance on Flange Plate

 $d_h = 1.25~{
m in}$ - Vertical Bolt Hole Dimension at Plate

 l_{c1} - Clear Distance at First Bolt Row

$$l_{c1} = L_{ev_fp} - rac{d_h}{2}$$

$$l_{c1} = (2 ext{ in}) - rac{(1.25 ext{ in})}{2}$$

$$l_{c1} = 1.375$$
 in

Calculate the clear distance of inner bolts on flange plate.

 $s_c=3~{
m in}$ - Bolt Column Spacing

 $d_h = 1.25~{
m in}$ - Vertical Bolt Hole Dimension at Plate

 $l_{c2}\,$ - Clear Distance at Rest of Bolts

 $l_{c2} = s_c - d_h$

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

 $l_{c2}=1.75~{
m in}$

Calculate the bolt tear-out capacity of the flange plate. $l_{c1}=1.375~{
m in}$ - Clear Distance at First Bolt Row $l_{c2}=1.75~{
m in}$ - Clear Distance at Rest of Bolts $t_{fp}=1.5~{
m in}$ - Flange Plate Thickness $F_{up}=65~{
m ksi}$ - 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

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

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

	$\phi R_{n_tearout} = 7239.4~{ m kip}$	
	Determine the governing bearing and tear-out capacity of the bolt group on flange plate.	
AISC 360-16 Chapter J3.10 Fg. (J3-6a)	$\phi R_{n_bearing} = 8424~{ m kip}$ - Design Bolt Bearing Capacity of Section	
AISC 360-16 Chapter J3.10	$\phi R_{n\ tearout}=7239.4~{ m kip}$ - Design Bolt Tear-out Capacity of Section	
AISC 360-16 Chapter J3.10	ϕR_n - Governing Design Capacity	
	$\phi R_n = min\left(\phi R_{n_bearing}, \phi R_{n_tearout} ight)$	
	$\phi R_n = min\left(\left(8424 \text{ kip}\right), \left(7239.4 \text{ kip}\right)\right)$	
	$\phi R_n=7239.4~{ m kip}$	
	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	
AISC 360-16 Chapter J3.10 Eq. (J3-6a)	$ \begin{array}{l} \textbf{Calculate the bolt bearing capacity of the beam flange.} \\ \phi = 0.75 \text{ - 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 \text{ - Number of Bolt Columns} \\ n_r = 4 \text{ - Number of Bolt Rows} \\ \phi R_{n_bearing} \text{ - Design Bolt Bearing Capacity of Section} \end{array} $	
	$\phi R_{n_bearing} = \phi \ 2.4 \ d_b \ t_{fb} \ F_{ub} \ n_c \ n_r$	
	$\phi R_{n_bearing} = (0.75) imes 2.4 imes (1 ext{ in}) imes (1.34 ext{ in}) imes (65 ext{ ksi}) imes (12) imes (4)$	
	$\phi R_{n_bearing} = 7525.4~{ m kip}$	
	Calculate the clear distance of outer bolts on beam flange. $L_{ev_bf} = 2$ in - Vertical Edge Distance on Beam Flange $d_h = 1.125$ in - Vertical Bolt Hole Dimension at Beam l_{c1} - Clear Distance at First Bolt Row $l_{c1} = L_{ev_bf} - \frac{d_h}{2}$ $l_{c1} = (2 \text{ in}) - \frac{(1.125 \text{ in})}{2}$	



	$l_{c1}=1.4375~{\rm in}$
	Calculate the clear distance of inner bolts on beam flange.
	$s_c = 3 \text{ in - Bolt Column Spacing}$ $d_l = 1.125 in - Vertical Bolt Hole Dimension at Beam$
	l_{c2} - Clear Distance at Rest of Bolts
	$a_{c2} = s_c - a_h$
	$l_{c2} = (3 \ {\rm in}) - (1.125 \ {\rm in})$
	$l_{c2}=1.875~{ m in}$
	Calculate the bolt tear-out capacity of the beam flange.
	$l_{c1} = 1.4375 ~{ m in}$ - Clear Distance at First Bolt Row
	$l_{c2}=1.875~{ m in}$ - Clear Distance at Rest of Bolts
	$t_{fb}=1.34~{ m in}$ - Beam Flange Thickness
	$F_{ub}=65~{ m ksi}$ - 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
	$\phi R_{n_tearout} = \phi \left[1.2 l_{c1} t_{fb} F_{ub} n_r + 1.2 l_{c2} t_{fb} F_{ub} n_r (n_c - 1) ight]$
	$\phi R_{n_tearout} = (0.75) \times [1.2 \times (1.4375 \text{ in}) \times (1.34 \text{ in}) \times (65 \text{ ksi}) \times (4) + 1.2 \times (1.875 \text{ in}) \times (1.34 \text{ in}) \times (65 \text{ ksi}) \times (4) \times ((12) - 1)]$
	$\phi R_{n_tearout} = 6917.9 ~{ m kip}$
	Determine the governing bearing and tear-out capacity of the bolt group on beam flange.
AISC 360-16 Chapter J3.10 Fg. (J3-6a)	$\phi R_{n_bearing} = 7525.4~{ m kip}$ - Design Bolt Bearing Capacity of Section
AISC 360-16 Chapter J3.10 Eq. (J3-6c)	$\phi R_{n_tearout} = 6917.9~{ m kip}$ - Design Bolt Tear-out Capacity of Section
AISC 360-16 Chapter J3.10	ϕR_n - Governing Design Capacity
	$\phi R_n = min\left(\phi R_{n_bearing}, \phi R_{n_tearout} ight)$
	$\phi R_n = min\left((7525.4~{ m kip}),(6917.9~{ m kip}) ight)$
	$\phi R_n = 6917.9 ~{ m kip}$
	Result

Result Demand over Capacity Ratio $DCR = rac{P_{uF}}{\phi R_n} = rac{(349.22 ext{ kip})}{(6917.9 ext{ kip})} = 0.05048$ Check No. 5: Design Block Shear Capacity of the Flange Plate Calculate the net area of the flange plate subject to tension. $t_{fp}=1.5~{
m in}$ - Flange Plate Thickness $n_r=4$ - Number of Bolt Rows $s_r=3~{
m in}$ - Bolt Row Spacing $g_a = 5.5 \ {
m in}$ - Bolt Gage $L_{eh_fp}=2.25~{
m in}$ - Horizontal Edge Distance on Flange Plate $d_h = 1.25 ~{
m in}$ - Vertical Bolt Hole Dimension at Plate $A_{nt}\,$ - Net Area Subject to Tension (L-pattern) $A_{nt} = t_{fp} \; \left[(n_r - 2) \; s_r + g_a + L_{eh_fp} - (n_r - 0.5) \; (d_h + 0.0625 \; {
m in})
ight]$ $A_{nt} = (1.5 ext{ in}) imes [((4) - 2) imes (3 ext{ in}) + (5.5 ext{ in}) + (2.25 ext{ in}) - ((4) - 0.5) imes ((1.25 ext{ in}) + (0.0625 ext{ in}))]$ $A_{nt}=13.734~{\rm in}^2$ Calculate the gross area of the flange plate subject to shear. $t_{fp}=1.5~{
m in}$ - Flange Plate Thickness $L_{ev_fp}=2~{
m in}$ - Vertical Edge Distance on Flange Plate $n_c=12$ - Number of Bolt Columns $s_c=3~{
m in}$ - Bolt Column Spacing $A_{gv}\,$ - Gross Area Subject to Shear (L-pattern) $A_{gv} = t_{fp} \; \left[L_{ev_fp} + (n_c - 1) \; s_c
ight]$

$$A_{gv} = (1.5 ext{ in}) imes [(2 ext{ in}) + ((12) - 1) imes (3 ext{ in})]$$

 $A_{gv}=52.5~{
m in}^2$

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

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

 $L_{ev_fp}=2~{
m in}$ - Vertical Edge Distance on Flange Plate

 $n_c=12$ - Number of Bolt Columns

 $s_c=3~{
m in}$ - Bolt Column Spacing

 $d_h = 1.25 ~{
m in}$ - Vertical Bolt Hole Dimension at Plate

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

$$A_{nv} = t_{fp} \, \left(L_{ev_fp} + (n_c \, - \, 1) \, \, s_c - (n_c \, - \, 0.5) \, \left(d_h + 0.0625 \, \, {
m in}
ight)
ight)$$

 $A_{nv} = (1.5 ext{ in}) imes ((2 ext{ in}) + ((12) - 1) imes (3 ext{ in}) - ((12) - 0.5) imes ((1.25 ext{ in}) + (0.0625 ext{ in})))$





	$A_{nv}=29.859~{\rm in}^2$	
	Calculate the design block shear capacity of the flange plate.	
	$\phi=0.75$ - Block Shear Resistance Factor	
	$F_{yp} = 50$ ksi - Flange Plate Yield Stress $F_{wp} = 65$ ksi - Flange Plate Tensile Stress	
	$U_{bs}=1$ - Uniformity factor for single line of bolts	
	$A_{gv}=52.5~{ m in}^2$ - Gross Area Subject to Shear (L-pattern)	
	$A_{nv}=29.859~{ m in}^2$ - Net Area Subject to Shear (L-pattern)	
AISC 360-16 Chapter I4.3	$A_{nt}=13.734~{ m in}^2$ - Net Area Subject to Tension (L-pattern)	
Éq. (J4-5)	ϕR_n - Design Block Shear Capacity of Section	
	$\phi R_n = \phi \left(0.6 F_{up} A_{nv} + U_{bs} F_{up} A_{nt} \le 0.6 F_{yp} A_{gv} + U_{bs} F_{up} A_{nt} ight)$	
	$\phi R_n = (0.75) \times \left(0.6 \times (65 \text{ ksi}) \times \left(29.859 \text{ in}^2\right) + (1) \times (65 \text{ ksi}) \times \left(13.734 \text{ in}^2\right) \le 0.6 \times (50 \text{ ksi}) \times \left(52.5 \text{ in}^2\right) + (1) \times (65 \text{ ksi}) \times \left(13.734 \text{ in}^2\right)\right)$	
	$\phi R_n = 1542.9~{ m kip}$	DASS
	Demand over Capacity Ratio	PASS
	$DCR = rac{P_{uF}}{\phi R_n} = rac{(349.22 ext{ kip})}{(1542.9 ext{ kip})} = 0.22633$	
	Check No. 6: Design Block Shear Capacity of the Beam Flange	
	Calculate the net area of the beam flange subject to tension.	
	$t_{fb}=1.34~{ m in}$ - Beam Flange Thickness	
	$b_{fb} = 14$ in - Beam Flange Width $a_{-} = 5.5$ in - Bolt Gage	
	$g_a=5.5~{ m m}$ - Bolt Gage $d_h=1.125~{ m in}$ - Vertical Bolt Hole Dimension at Beam	
	A_{nt} - Net Area Subject to Tension (2L-pattern)	
	$A_{nt} = t_{fb} \left[b_{fb} - g_a - (d_h + 0.0625 \; { m in}) ight]$	
	$A_{nt} = (1.34~{\rm in}) \times [(14~{\rm in}) - (5.5~{\rm in}) - ((1.125~{\rm in}) + (0.0625~{\rm in}))]$	
	$A_{nt}=9.7988~{\rm in}^2$	
	Calculate the gross area of the beam flange subject to shear.	
	$t_{fb}=1.34~{ m in}$ - Beam Flange Thickness	
	$n_c = 12$ - Number of Bolt Columns	
	$s_c=3~{ m in}$ - Bolt Column Spacing	
	A_{gv} - Gross Area Subject to Shear (2L-pattern)	
	$A_{gv} = 2 t_{fb} \left[L_{ev_bf} + (n_c - 1) s_c ight]$	
	$A_{gv} = 2 \times (1.34 \text{ in}) \times [(2 \text{ in}) + ((12) - 1) \times (3 \text{ in})]$	
	$A_{gv}=93.8~{\rm in}^2$	
	Calculate the net area of the beam flange subject to shear.	
	$t_{fb}=1.34~{ m in}$ - Beam Flange Thickness	
	$L_{ev_bf}=2~{ m in}$ - Vertical Edge Distance on Beam Flange $n_{+}=12$ - Number of Bolt Columns	
	$s_c=3~{ m in}$ - Bolt Column Spacing	
	$d_h = 1.125 ext{ in - Vertical Bolt Hole Dimension at Beam}$	
	A_{nv} - Net Area Subject to Shear (2L-pattern)	
	$A_{nv} = 2 t_{fb} \left(L_{ev_bf} + (n_c - 1) s_c - (n_c - 0.5) \left(d_h + 0.0625 { m in} ight) ight)$	
	$A_{nv} = 2 imes (1.34 ext{ in}) imes ((2 ext{ in}) + ((12) - 1) imes (3 ext{ in}) - ((12) - 0.5) imes ((1.125 ext{ in}) + (0.0625 ext{ in})))$	
	$A_{nv}=57.201~{\rm in}^2$	
	Calculate the design block shear capacity of the beam flange.	
	$\phi=0.75$ - Block Shear Resistance Factor	
	$F_{yb} = 50~{ m ksi}$ - Beam field Stress $F_{ub} = 65~{ m ksi}$ - Beam Tensile Stress	
	$U_{bs}=1$ - Uniformity factor for single line of bolts	
	$A_{gv}=93.8~{ m in}^2$ - Gross Area Subject to Shear (2L-pattern)	
	$A_{nv} = 57.201 \text{ in}^2$ - Net Area Subject to Shear (2L-pattern)	
AISC 360-16 Chapter J4.3	$A_{nt} = 3.1300 \text{ III}$ - Net Area Subject to Tension (2L-pattern) ϕR_{n} - Design Block Shear Capacity of Section	
Eq. (J4-5)		
	$\phi R_n = \phi \left(0.6 F_{ub} A_{nv} + U_{bs} F_{ub} A_{nt} \le 0.6 F_{yb} A_{gv} + U_{bs} F_{ub} A_{nt} ight)$	
	$\phi R_n = (0.75) \times \left(0.6 \times (65 \text{ ksi}) \times \left(57.201 \text{ in}^2\right) + (1) \times (65 \text{ ksi}) \times \left(9.7988 \text{ in}^2\right) \le 0.6 \times (50 \text{ ksi}) \times \left(93.8 \text{ in}^2\right) + (1) \times (65 \text{ ksi}) \times \left(9.7988 \text{ in}^2\right) \right)$	
	$\phi R_n = 2150.8 ~{ m kip}$	
	Result:	PASS
	Demand over Capacity Ratio	
	$DCR = \frac{1}{\phi R_n} = \frac{1}{(2150.8 \text{ kip})} = 0.16236$	

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 $F_{ m em}=50~{ m ksi}$ - Flange Plate Yield Stress	
	$t_{fp}=1.5~{ m in}$ - Flange Plate Thickness	
	$b_{fp}=16~{ m in}$ - Flange Plate Width	
AISC 360-16 Chapter J4.1 Eq. (J4-1)	ϕR_{n_ty} - Design Tension Yielding Capacity of Section	
	$\phi R_{n_ty} = \phi \ F_{yp} \ t_{fp} \ b_{fp}$	
	$\phi R_{n_ty} = (0.9) imes (50 ext{ ksi}) imes (1.5 ext{ in}) imes (16 ext{ in})$	
	$\phi B_{\rm m}$ to $= 1080$ 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	
	$t_{fp} = 1.5 ~{ m m}$ - Flange Plate Thickness $b_{fn} = 16 ~{ m in}$ - Flange Plate Width	
	$n_r=4$ - Number of Bolt Rows	
NISC 360-16 Chapter 1/ 1	$d_h = 1.25 ~{ m in}$ - Horizontal Bolt Hole Dimension at Plate	
Eq. (J4-2)	ϕR_{n_tr} - Design Tension Rupture Capacity of Section	
	$\phi R_{n_tr} = \phi \; F_{up} t_{fp} \; [b_{fp} - n_r \; (d_h + 0.0625 \; { m in})]$	
	$\phi R_{n_tr} = (0.75) imes (65 ext{ ksi}) imes (1.5 ext{ in}) imes [(16 ext{ in}) - (4) imes ((1.25 ext{ in}) + (0.0625 ext{ in}))]$	
	$\phi R_{n_tr} = 786.09 ~{ m kip}$	
AISC 360-16 Chapter 4.1	Determine the governing tensile capacity of the flange plate. $\phi B_{\rm res} = 1080 {\rm kip}$ - Design Tension Yielding Capacity of Section	
Éq. (J́4-1) AISC 360-16 Chapter I4.1	$\phi R_{n_ty} = 1080 \text{ klp}$ - Design lension fielding Capacity of Section	
Éq. (J4-2) AISC 360-16 Chapter J4.2	$\phi R_{n_tr} = 780.09 \text{ kip}$ - Design lension Rupture Capacity of Section ϕR_n - Governing Design Capacity	
	$\phi R_n = min\left(\phi R_{n_ty}, \phi R_{n_tr} ight)$	
	$\phi R = min((1080 \text{ kin}))(786.00 \text{ kin}))$	
	$\varphi R_n = min((1000 \text{ klp}), (100.03 \text{ klp}))$	
	$\phi R_n = 780.09 ext{ klp}$	PASS
	Demand over Capacity Ratio	
	$DCR = rac{P_{uF}}{\phi R_n} = rac{(349.22 ext{ kip})}{(786.09 ext{ kip})} = 0.44424$	
	Check No. 8: Design Capacity of the Beam in Tension	
	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	
	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 $F_{ub} = 65$ ksi - Beam Tensile Stress	
	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 $F_{ub} = 65$ ksi - Beam Tensile Stress $A_{bm} = 57.1$ in ² - Beam Area $t_{re} = 1.24$ in Ream Flange Thickness	
	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 $F_{ub} = 65$ ksi - Beam Tensile Stress $A_{bm} = 57.1$ in ² - Beam Area $t_{fb} = 1.34$ in - Beam Flange Thickness $d_h = 1.125$ in - Horizontal Bolt Hole Dimension at Beam	
	Check No. 8: Design Capacity of the Beam in TensionCalculate the tensile rupture capacity of the flange plate. $\phi = 0.75$ - Tensile Rupture Resistance Factor $F_{ub} = 65$ ksi - Beam Tensile Stress $A_{bm} = 57.1$ in ² - Beam Area $t_{fb} = 1.34$ in - Beam Flange Thickness $d_h = 1.125$ in - Horizontal Bolt Hole Dimension at Beam $n_r = 4$ - Number of Bolt Rows	
	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 $F_{ub} = 65$ ksi - Beam Tensile Stress $A_{bm} = 57.1$ in ² - Beam Area $t_{fb} = 1.34$ in - Beam Flange Thickness $d_h = 1.125$ in - Horizontal Bolt Hole Dimension at Beam $n_r = 4$ - Number of Bolt Rows $n_c = 12$ - Number of Bolt Columns $c_r = 3$ in _ Bolt Columns	
	Check No. 8: Design Capacity of the Beam in TensionCalculate the tensile rupture capacity of the flange plate. $\phi = 0.75$ - Tensile Rupture Resistance Factor $F_{ub} = 65$ ksi - Beam Tensile Stress $A_{bm} = 57.1$ in ² - Beam Area $t_{fb} = 1.34$ in - Beam Flange Thickness $d_h = 1.125$ in - Horizontal Bolt Hole Dimension at Beam $n_r = 4$ - Number of Bolt Rows $n_c = 12$ - Number of Bolt Columns $s_c = 3$ in - Bolt Column Spacing $\overline{y} = 3.0789$ in - Centroid of WT section	
AISC 360-16 Chapter D3 (Table D3.1 case 2)	Check No. 8: Design Capacity of the Beam in TensionCalculate the tensile rupture capacity of the flange plate. $\phi = 0.75$ - Tensile Rupture Resistance Factor $F_{ub} = 65$ ksi - Beam Tensile Stress $A_{bm} = 57.1$ in ² - Beam Area $t_{fb} = 1.34$ in - Beam Flange Thickness $d_h = 1.125$ in - Horizontal Bolt Hole Dimension at Beam $n_r = 4$ - Number of Bolt Rows $n_c = 12$ - Number of Bolt Columns $s_c = 3$ in - Bolt Column Spacing $\bar{y} = 3.0789$ in - Centroid of WT section U - Shear Lag Factor	
AISC 360-16 Chapter D3 (Table D3.1 case 2)	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 $F_{ub} = 65$ ksi - Beam Tensile Stress $A_{bm} = 57.1$ in ² - Beam Area $t_{fb} = 1.34$ in - Beam Flange Thickness $d_h = 1.125$ in - Horizontal Bolt Hole Dimension at Beam $n_r = 4$ - Number of Bolt Rows $n_c = 12$ - Number of Bolt Columns $s_c = 3$ in - Bolt Column Spacing $\overline{y} = 3.0789$ in - Centroid of WT section U - Shear Lag Factor	
AISC 360-16 Chapter D3 (Table D3.1 case 2)	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 $F_{ub} = 65$ ksi - Beam Tensile Stress $A_{bm} = 57.1$ in ² - Beam Area $t_{fb} = 1.34$ in - Beam Flange Thickness $d_h = 1.125$ in - Horizontal Bolt Hole Dimension at Beam $n_r = 4$ - Number of Bolt Rows $n_c = 12$ - Number of Bolt Columns $s_c = 3$ in - Bolt Column Spacing $\bar{y} = 3.0789$ in - Centroid of WT section U - Shear Lag Factor $U = 1 - \frac{\bar{y}}{(n_c - 1) s_c}$	
AISC 360-16 Chapter D3 (Table D3.1 case 2)	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 $F_{ub} = 65$ ksi - Beam Tensile Stress $A_{drm} = 57.1$ in ² - Beam Area $t_{fb} = 1.34$ in - Beam Flange Thickness $d_h = 1.125$ in - Horizontal Bolt Hole Dimension at Beam $n_r = 4$ - Number of Bolt Rows $n_c = 12$ - Number of Bolt Columns $s_c = 3$ in - Bolt Column Spacing $\tilde{y} = 3.0789$ in - Centroid of WT section U - Shear Lag Factor $U = 1 - \frac{\tilde{y}}{(n_c - 1) s_c}$ $U = 1 - \frac{(3.0789 \text{ in})}{((12) - 1) \times (3 \text{ in})}$	
AISC 360-16 Chapter D3 (Table D3.1 case 2)	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 $F_{ab} = 65$ ksi - Beam Tensile Stress $A_{bm} = 57.1$ in ² - Beam Area $t_{fb} = 1.34$ in - Beam Flange Thickness $d_h = 1.125$ in - Horizontal Bolt Hole Dimension at Beam $n_r = 4$ - Number of Bolt Rows $n_c = 12$ - Number of Bolt Columns $s_c = 3$ in - Bolt Column Spacing $\tilde{y} = 3.0789$ in - Centroid of WT section U - Shear Lag Factor $U = 1 - \frac{\tilde{y}}{(n_c - 1) s_c}$ $U = 1 - \frac{(3.0789 \text{ in})}{((12) - 1) \times (3 \text{ in})}$ U = 0.9067	
AISC 360-16 Chapter D3 (Table D3.1 case 2) AISC 360-16 Chapter J4.1 Eq. (J4-2)	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 $F_{ub} = 65 \text{ ksi}$ - Beam Tensile Stress $A_{bm} = 57.1 \text{ in}^2$ - Beam Area $t_{fp} = 1.34 \text{ in}$ - Beam Flange Thickness $d_h = 1.125 \text{ in}$ - Horizontal Bolt Hole Dimension at Beam $n_r = 4$ - Number of Bolt Rows $n_c = 12$ - Number of Bolt Columns $s_c = 3 \text{ in}$ - Bolt Columns Sach y = 3.0789 in - Centroid of WT section U - Shear Lag Factor $U = 1 - \frac{\bar{y}}{(n_c - 1) s_c}$ $U = 1 - \frac{(3.0789 \text{ in})}{((12) - 1) \times (3 \text{ in})}$ U = 0.9067 $\phi R_{n_c tr}$ - Design Tension Rupture Capacity of Section	
AISC 360-16 Chapter D3 (Table D3.1 case 2) AISC 360-16 Chapter J4.1 Eq. (J4-2)	Check No. 3: Design Capacity of the Beam in Tension Calculate the tensile rupture capacity of the flange plate. $\phi = 0.75$ - Tensile Rupture Resistance Factor $F_{ub} = 65$ ksi - Beam Tensile Stress $A_{am} = 57.1 \text{ in}^2$ - Beam Area $t_{fb} = 1.34$ in - Beam Area $t_{fb} = 1.34$ in - Beam Area $t_{fb} = 1.34$ in - Beam Flange Thickness $d_h = 1.125$ in - Horizontal Bolt Hole Dimension at Beam $n_r = 4$ - Number of Bolt Rows $n_c = 12$ - Number of Bolt Columns $s_c = 3$ in - Bolt Columns $s_c = 3$ in - Bolt Columns Sacing $\tilde{y} = 3.0789$ in - Centroid of WT section U - Shear Lag Factor $U = 1 - \frac{\tilde{y}}{(n_c - 1) s_c}$ $U = 1 - \frac{(3.0789 \text{ in})}{((12) - 1) \times (3 \text{ in})}$ U = 0.9067 $\phi R_{n,tr}$ - Design Tension Rupture Capacity of Section	
AISC 360-16 Chapter D3 (Table D3.1 case 2) AISC 360-16 Chapter J4.1 Eq. (J4-2)	Check No. 8: Design Capacity of the Beam in Tension Calculate the tensile rupture capacity of the flange plate. $\phi = 0.75 \cdot \text{Tensile Rupture Resistance Factor}$ $F_{ub} = 65$ ks i - Beam Tensile Stress $A_{bm} = 57.1 \text{ in}^2 \cdot \text{Beam Area}$ $t_{fb} = 1.34 \text{ in} \cdot \text{Beam Flange Thickness}$ $d_h = 1.125 \text{ in} \cdot \text{Horizontal Bolt Hole Dimension at Beam}$ $n_r = 4 \cdot \text{Number of Bolt Rows}$ $n_e = 12 \cdot \text{Number of Bolt Rows}$ $n_e = 12 \cdot \text{Number of Bolt Columns}$ $s_e = 3 \text{ in} \cdot \text{Bolt Column Spacing}$ $\tilde{y} = 3.0789 \text{ in} \cdot \text{Centroid of WT section}$ $U = 1 - \frac{\tilde{y}}{(n_e - 1) s_e}$ $U = 1 - \frac{(3.0789 \text{ in})}{((12) - 1) \times (3 \text{ in})}$ U = 0.9067 $\phi R_{n, tr} \cdot \text{Design Tension Rupture Capacity of Section}$	
AISC 360-16 Chapter D3 (Table D3.1 case 2)	Check No. 3: Design Capacity of the Beam in Tension Calculate the tensile rupture capacity of the flange plate. $\phi = 0.75$. Tensile Rupture Resistance Factor $F_{wh} = 65$ ksi - Beam Tensile Stress $A_{born} = 57.1 \text{ in}^2$ - Beam Area $t_{fb} = 1.34$ in - Beam Flange Thickness $d_h = 1.125$ in - Horizontal Bolt Hole Dimension at Beam $n_r = 4$. Number of Bolt Columns $s_c = 12$ - Number of Bolt Columns $s_c = 3$ in - Bolt Column Spacing $\bar{y} = 3.0789$ in - Centroid of WT section $U = 1 - \frac{\bar{y}}{(n_c - 1) s_c}$ $U = 1 - \frac{(3.0789 \text{ in})}{((12) - 1) \times (3 \text{ in})}$ U = 0.9067 $\phi R_{n,tr}$ - Design Tension Rupture Capacity of Section $\phi R_{n,tr} = \phi F_{ub} U [0.5 A_{bm} - n_r (d_h + 0.0625 \text{ in}) t_B]$ $\phi R_{n,tr} = (0.75) \times (65 \text{ ksi}) \times (0.9067) \times [0.5 \times (57.1 \text{ in}^2) - (4) \times ((1.125 \text{ in}) + (0.0625 \text{ in})) \times (1.34 \text{ in})]$ $\phi R_{n,tr} = 980.61 \text{ kip}$	
AISC 360-16 Chapter D3 (Table D3.1 case 2) AISC 360-16 Chapter J4.1 Eq. (J4-2)	Check No. 8: Design Capacity of the Beam in Tension Calculate the twolife rupture capacity of the flange plate. $\phi = 0.75$. Tensile Rupture Resistance Factor $F_{ub}^{-} = 65$ ksi - Beam Tensile Stress $A_{un} = 57.1 \text{ In}^2$. Beam Area $t_{ln} = 1.34$ in - Beam Area $t_{ln} = 1.34$ in - Beam Area $t_{ln} = 1.34$ in - Beam Flange Thickness ($d_{ln} = 1.125$ in - Horizontal Bolt Hole Dimension at Beam $n_r = 4$ - Number of Bolt Columns s $a_r = 3$ in - Bolt Column Spacing $\overline{g} = 3.0789$ in - Centroid of WT section $U = 1 = \frac{\overline{y}}{(n_c - 1) s_c}$ $U = 1 = \frac{\overline{y}}{(n_c - 1) s_c}$ $U = -1 = \frac{(3.0789 \text{ in})}{((12) - 1) \times (3 \text{ in})}$ $U = 0.9067$ $\phi R_{n_s tr} \cdot \text{Design Tension Rupture Capacity of Section\phi R_{n_s tr} = \phi F_{ub} U [0.5 A_{om} - n_r (d_h + 0.0625 \text{ in}) t_{fb}]\phi R_{n_s tr} = (0.75) \times (65 \text{ ksi}) \times (0.9067) \times [0.5 \times (57.1 \text{ in}^2) - (4) \times ((1.125 \text{ in}) + (0.0625 \text{ in})) \times (1.34 \text{ in})]\phi R_{n_s tr} = 980.61 \text{ kip}Result:$	PASS
AISC 360-16 Chapter D3 (Table D3.1 case 2)	Check No. 5: Design Capacity of the Beam in Tension Calculate the tensile rupture capacity of the flange plate. $\phi = 0.75$. Tensile Rupture Resistance Factor $F_{ub} = 65$ ksi - Beam Tensile Stress $A_{un} = 57.1 \text{ in}^2$ - Beam Area $t_{fh} = 1.34 \text{ in}$ - Beam Area $t_{fh} = 1.34 \text{ in}$ - Beam Area $t_{fh} = 1.25 \text{ in}$ - Horizontal Bolt Hole Dimension at Beam $n_r = 4$ - Number of Bolt Rolum Spacing $\overline{y} = -3.0789 \text{ in}$ - Centroid of WT section $U = 1 - \frac{\overline{y}}{(n_c - 1) s_c}$ $U = 1 - \frac{(3.0789 \text{ in})}{((12) - 1) \times (3 \text{ in})}$ U = 0.9067 $\phi R_{n,tr}$ - Design Tension Rupture Capacity of Section $\phi R_{n,tr} = \phi F_{ub} U [0.5 A_{lon} - n_r (d_h + 0.0625 \text{ in}) t_{fh}]$ $\phi R_n t_r = (0.75) \times (65 \text{ ksi}) \times (0.9067) \times [0.5 \times (57.1 \text{ in}^2) - (4) \times ((1.125 \text{ in}) + (0.0625 \text{ in})) \times (1.34 \text{ in})]$ $\phi R_{n,tr} = 980.61 \text{ kip}$ Pesulti Demand over Capacity Ratio $D(R = -\frac{R_{re}}{R_{re}} - \frac{(340.22 \text{ km})}{R_{re}} = 0.35612$	PASS
AISC 360-16 Chapter D3 (Table D3.1 case 2)	Check No. 8: Design Capacity of the Beam in Tension Calculates the tensile rupture Resistance Fator $F_{ab} = 0.75$ - Tensile Rupture Resistance Fator $F_{ab} = 65$ ksi - Beam Tensile Stress $A_{beam} = 57.1 \text{ in}^3$ - Beam Area $I_{pp} = 1.34$ in - Beam Finale Thickness $d_h = 1.125$ in - Horizontal Bolt Hole Dimension at Beam $n_r = 4$ - Number of Bolt Rows $n_e = 12$ - Number of Bolt Columns $s_e = 3$ in - Bolt Columns $s_e = 3$ in - Bolt Column Sacchara $T = -\frac{y}{(n_e - 1) s_e}$ $U = 1 - \frac{y}{(n_e - 1) s_e}$ $U = 1 - \frac{(3.0789 \text{ in})}{((12) - 1) \times (3 \text{ in})}$ U = 0.9067 $\phi R_{n,tr} - Design Tension Rupture Capacity of Section \phi R_{n,tr} = \phi F_{ab} U [0.5 A_{bm} - n_r (d_h + 0.0625 \text{ in}) t_{fb}]\phi R_{n,tr} = (0.75) \times (65 \text{ ksi}) \times (0.9067) \times [0.5 \times (57.1 \text{ in}^2) - (4) \times ((1.125 \text{ in}) + (0.0625 \text{ in})) \times (1.34 \text{ in})]\phi R_{n,tr} - 980.61 \text{ kip}Persuit:Dermand over Capacity RatioDCR = \frac{F_{ab}}{\sigma R_{ab}r} = \frac{(34022 \text{ kp})}{(34042 \text{ kp})} = 0.35612Charded to Abelian Column State State$	PASS
AISC 360-16 Chapter D3 (Table D3.1 case 2)	Check No. 3: Design Capacity of the Beam in Tension Calculate the tensile rupture Resistance Factor $F_{ub} = 0.75$ - Tensile Rupture Resistance Factor $F_{ub} = 65$ ksi - Beam Tensite Stress $A_{bm} = 57.1$ in ² - Beam Area $E_{tp} = 1.34$ in - Beam Finage Thickness $d_{t} = 1.125$ in - Horizontal Bolt Hole Dimension at Beam $n_r = 4$ - Number of Bolt Columns $s_e = 3$ in - Bolt Columns g = 3.0789 in - Centroid of WT section $U = 1 - \frac{y}{(n_c - 1) s_c}$ $U = 1 - \frac{(3.0789 in)}{((12) - 1) \times (3 in)}$ U = 0.9067 $\phi R_{n,tr} - Design Tension Rupture Capacity of Section \phi R_{n,tr} = \phi F_{ub} U [0.5 A_{bm} - n_r (d_b + 0.0625 in) t_B]\phi R_{n,tr} = (0.75) \times (65 \text{ ksi}) \times (0.9067) \times [0.5 \times (57.1 in^2) - (4) \times ((1.125 in) + (0.0625 in)) \times (1.34 in)]\phi R_{n,tr} = 980.61 \text{ kip}Facult:Derma dover Capacity RalioDC'R = \frac{E_{r,r}}{\phi R_{r,r}} = \frac{(340.22 \text{ kip})}{(30.14 \text{ kip})} = 0.35612Check No. 9: Design Capacity of the Flange Plate in CompressionCalculate the compression buckling capacity of the Flange Plate.$	PASS
AISC 360-16 Chapter D3 (Table D3.1 case 2)	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 $F_{ab} = 05$ ksi - Beam Tensile Stress $A_{bm} = 57.1$ in ² - Beam Area $t_{p5} = 1.34$ in - Heam Flange Thickness $d_{h} = 1.125$ in - Horizontal Bolt Hole Dimension at Beam $n_r = 4$ - Number of Bolt Columns $s_r = 3$ in - Bolt Column Spacing $\overline{y} = 3.0789$ in - Centrold of WT section $U = 1 - \frac{\overline{y}}{(n_c - 1) s_c}$ $U = 1 - \frac{\overline{y}}{(n_c - 1) s_c}$ $U = 1 - \frac{(3.0789 in)}{((12) - 1) \times (3 in)}$ U = 0.9067 $dR_{n,tr} - Design Tension Rupture Capacity of Section \phi R_{n,tr} = \phi F_{ab} U [0.5 A_{bm} - n_r (d_b + 0.0625 in) t_{fb}]\phi R_{n,tr} = \phi S.(57.1 in2) - (4) \times ((1.125 in) + (0.0625 in)) \times (1.34 in)]\phi R_{n,tr} = 980.61 kipResult:Demand over Capacity RatioDCR = \frac{(40.27 2 in)}{(40.20 - 4)} - \frac{(40.27 2 in)}{(40.20 - 4)} - 0.35612Check No. 9: Design Capacity of the Flange Plate in CompressionCalculate the compression buckling capacity of the Flange Plate.\phi = 0.9 \cdot 0 or Compression Resistance Factor$	PASS
AISC 360-16 Chapter D3 (Table D3.1 case 2)	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 $F_{ch} = 60$ kai - Beam Flange Trickness $A_{bmi} = 57.1$ in ² - Beam Area $Y_{\mu} = 1.31$ in - Beam Flange Trickness $d_{\mu} = 1.125$ in - Horizontal Bott Hole Dimension at Beam $n_{\mu} = 4$ - Number of Bott Columns $n_{\mu} = 4$ - Number of Bott Columns $n_{\mu} = 4$ - Number of Bott Columns $n_{\mu} = 3$ in - Bott Columns Spacing $\overline{g} = -3.0789$ in - Centroid of WT section $U = 1 - \frac{\overline{y}}{(n_{\mu} - 1) \cdot s_{\mu}}$ $U = 1 - \frac{(3.0789 im)}{((12) - 1) \times (3 im)}$ U = 0.9067 dR_{n} c - Design Tension Rupture Capacity of Section ϕR_{n} $c_{\mu} = \phi F_{ab} U [0.5 A_{bm} - n_{\mu} (d_{h} + 0.0625 im) t_{fb}]$ ϕR_{n} $c_{\mu} = 980.61$ kip Fessel: Demand over Capacity Ratio $DCR = \frac{R_{\mu}}{\delta d_{h,\mu}} = \frac{(3.9789)}{(38101 \cdot 14p)} = 0.35612$ Check No. 3: Design Capacity of the Flange Plate In Compression Calculate the compression Resistance Factor $\phi = 0.9 \cdot \text{ Compression Resistance Factor F_{\mu\nu} = -30 \text{ kai} - \text{ Flange Plate Yield Stress}\delta = 2.9000 \text{ kib} - \text{ Models for Steel}$	PASS
AISC 360-16 Chapter D3 (Table D3.1 case 2)	$ \begin{array}{l} Check No. 8: Design Capacity of the Beam In Tension Calculate the tonsile rupture capacity of the flange plate. $$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$$	PASS
AISC 360-16 Chapter D3 (Table D3.1 case 2)	Check No. 3: Design Capacity of the Beam in Tension Calculate the transfer rupture capacity of the flange plate. $\phi = 0.75 \cdot \text{Tensile Rupture Resistance Factor}$ $P_{ab} = 0.6 \text{ ks}$. Beam Tensile Stress $A_{ab} = 57.1 \text{ in}^2$. Beam Area $ty_b = 1.34 \text{ in} \cdot \text{Beam Flange Thickness}$ $d_{ab} = 51.1 \text{ in}^2$. Beam Area $ty_b = 1.34 \text{ in} \cdot \text{Beam Flange Thickness}$ $d_{ab} = 51.1 \text{ in}^2$. Beam Area $ty_b = 1.34 \text{ in} \cdot \text{Beam Flange Thickness}$ $d_{ab} = 51.1 \text{ in}^2$. Beam Area $ty_b = 1.34 \text{ in} \cdot \text{Beam Flange Thickness}$ $d_{ab} = 51.1 \text{ in}^2$. Beam Area $ty_b = 1.36 \text{ in}^2$. Beam Area $t_b = 4 \cdot \text{Rumber of Bolt Columns}$ $s_c = 3 \text{ in} \text{ Bolt Columns Spacing}$ $f_{ab} = 0.1 \text{ certroid of WT section}$ $U = 1 - \frac{\bar{y}}{(n_c - 1) s_c}$ $U = 1 - \frac{(3.0789 \text{ in})}{(1(2) - 1) \times (3 \text{ in})}$ U = 0.9067 $\phi R_{a,br} \cdot \text{Design Tension Rupture Capacity of Section}$ $\phi R_{a,br} - \Phi F_{ab} U \left[0.5 \text{ A}_{bm} - n_r \left(d_b + 0.0625 \text{ in} \right) t_{jb} \right]$ $\phi R_{a,br} = (0.75) \times (65 \text{ ks}) \times (0.9067) \times \left[0.5 \times (57.1 \text{ in}^2) - (4) \times ((1.125 \text{ in}) + (0.0625 \text{ in})) \times (1.34 \text{ in}) \right]$ $\rho CR = \frac{n_{ab}}{\sigma A_{a,br}} = \frac{(20.23 \text{ tr})}{(20.04 \text{ tr})} = 0.35612$ Demand over Capacity Rulo $DCR = \frac{n_{ab}}{\sigma A_{a,br}} = \frac{(20.23 \text{ tr})}{(20.041 \text{ tp})} = 0.35612$ Check No. 8: Design Capacity of the Flange Plate in Compression Calculate the compression buckting capacity of the flange plate. $\phi = 0.9 \cdot \text{Compression Buckting Capacity of the flange plate.}$ $\phi = 0.9 \cdot \text{Compression Buckting Capacity of the flange plate.}$ $\phi = 0.9 \cdot \text{Compression Buckting Capacity of the flange plate.}$ $\phi = 0.9 \cdot \text{Compression Buckting Capacity of the flange plate.}$ $\phi = 0.9 \cdot \text{Compression Buckting Capacity of the flange plate.}$ $\phi = 0.9 \cdot \text{Compression Buckting Capacity of the flange plate.}$ $\phi = 0.9 \cdot \text{Compression Buckting Capacity of the flange plate.}$ $\phi = 0.9 \cdot \text{Compression Buckting Capacity of the flange plate.}$ $\phi = 0.9 \cdot Compression Buckting Capacity of the fla$	PASS
AISC 360-16 Chapter D3 (Table D3.1 case 2)	Check No. 3: Design Capacity of the Beam in Tension Calculate the tonsile rupture capacity of the fings plate. $\phi = 0.73$ - Tensile Rupture Resistance Factor $F_{ab} = 65$ ks. I - Beam Tensile Stress $A_{ab} = 57.11$ k ⁻¹ - Tensile Rupture Resistance Factor $F_{ab} = -56$ ks. I - Beam Tensile Stress $A_{ab} = 57.11$ k ⁻¹ - Tensile Rupture Resistance Factor $F_{ab} = 4.8$ tumber of Bolt Rows $R_{ab} = 4.8$ tumber of Bolt Rows $R_{ab} = 4.8$ tumber of Bolt Columns $R_{ab} = 4.8$ tumber of Bolt Rows $R_{ab} = 4.8$ tumber of Bolt Rows $R_{ab} = 4.8$ tumber of Bolt Rows $R_{ab} = 4.8$ tumber of Bolt Rows $U = 1 - \frac{\bar{y}}{(R_{ab} - 1)} \frac{\bar{y}}{s_{ab}}$ $U = 1 - \frac{(3.0789 \text{ in})}{((12) - 1) \times (3 \text{ in})}$ U = 0.9067 $\phi R_{ab} = (0.75) \times (65 \text{ ksi}) \times (0.9067) \times [0.5 \times (57.1 \text{ in}^2) - (4) \times ((1.125 \text{ in}) + (0.0025 \text{ in})) \times (1.34 \text{ in})]$ $\phi R_{ab} = (0.75) \times (65 \text{ ksi}) \times (0.9067) \times [0.5 \times (57.1 \text{ in}^2) - (4) \times ((1.125 \text{ in}) + (0.0025 \text{ in})) \times (1.34 \text{ in})]$ $\phi R_{ab} = (0.75) \times (0.5 \text{ ksi}) \times (0.9067) \times [0.5 \times (57.1 \text{ in}^2) - (4) \times ((1.125 \text{ in}) + (0.0025 \text{ in})) \times (1.34 \text{ in})]$ $\phi R_{ab} = (0.75) \times (0.5 \text{ ksi}) \times (0.9067) \times [0.5 \times (57.1 \text{ in}^2) - (4) \times ((1.125 \text{ in}) + (0.0025 \text{ in})) \times (1.34 \text{ in})]$ $\phi R_{ab} = (0.75) \times (0.5 \text{ ksi}) \times (0.9067) \times [0.5 \times (57.1 \text{ in}^2) - (4) \times ((1.125 \text{ in}) + (0.0025 \text{ in})) \times (1.34 \text{ in})]$ $\phi R_{ab} = 0.90 \text{ transmit} R_{ab} = Rage Plate in Compression Calculate the compression buckling capacity of the Flange Plate in Compression Calculate the compression buckling capacity of the flange plate. $	PASS





	$rac{KL}{r}=27.713$ - Effective Length Slenderness Ratio	
AISC 360-16 Chapter E3	Since, $\frac{KL}{r} > 25$.	
Eq. (E3-4)	F_e - Elastic Buckling Stress	
	$F_e = rac{\pi^2 \ E}{\left(rac{KL}{r} ight)^2}$	
	$F_e = rac{\pi^2 imes (29000 ext{ ksi})}{\left((27.713) ight)^2}$	
	$F = 372.68 \ \mathrm{ksi}$	
	$T_e = 572.00$ KSI	
	4.71 $\sqrt{\frac{F_y}{F_y}} = 113.43$ - Effective Length Sienderness Ratio Limiter	
AISC 360-16 Chapter E3	Since, $rac{AB}{r} \leq 4.71 \sqrt{rac{B}{F_y}}$.	
Eq. (E3-2)	F _{cr} - Critical Buckling Stress	
	$F_{cr}=\left(0.658rac{F_{yp}}{F_e} ight)F_{yp}$	
	$F_{cr} = \left(0.658^{rac{(50 \; \mathrm{ksi})}{(372.68 \; \mathrm{ksi})}} ight) imes (50 \; \mathrm{ksi})$	
	$F_{cr}=47.27~{ m ksi}$	
AISC 360-16 Chapter E3	ϕR_n - Design Compressive Capacity of Section	
Eq. (E3-1)	$\phi R_n = \phi \; F_{cr} \; t_{fp} b_{fp}$	
	$\phi R_n = (0.9) \times (47.27 \text{ ksi}) \times (1.5 \text{ in}) \times (16 \text{ in})$	
	$\phi R_n = 1021 \; { m kip}$	
	Result: Demand over Capacity Ratio	PASS
	$DCR = rac{P_{uF}}{\phi R_n} = rac{(349.22 ext{ kip})}{(1021 ext{ kip})} = 0.34202$	
	Check No. 10: Connection Detailing Limitations Check at Support Side	
	Detailing LimitationsLimit Value (in)Actual Value (in)DCRResultMaximum Fillet Weld Size per Beam Clearance8.0000.5000.063PASS	
	Result:	PASS
	Demand over Capacity Ratio $DCR = \frac{d}{2} = \frac{(0.5)}{0.0625}$	
	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.	
	$t_{fp} = 1.5~{ m m}$ - Flange Plate Thickness $F_{up} = 65~{ m ksi}$ - Flange Plate Tensile Stress	
	$t_{fs}=0.71~{ m in}$ - Column Flange Thickness	
AISC 15th Ed. Part 9 Eq.	$F_{us} = 05$ KSI - Column lensile Stress D_{max} - Maximum Fillet Weld Size for Base Metal Strength	
(3-2)	t_{fp} , $r_{f} < t_{f}$, $r_{f} < t_{f}$	
	$D_{max} = rac{rac{1}{2} F_{up} \leq t_{fs} F_{us}}{3.09 ext{ kip/in}}$	
	$D_{max} = rac{(0.75 ext{ in}) imes (65 ext{ ksi}) \leq (0.71 ext{ in}) imes (65 ext{ ksi})}{(3.09 ext{ kip/in})}$	
	$D_{max}=14.935$	
	Calculate the total effective weld length.	
	L_w - Length of One Weld Segment	
	$L_w=rac{(b_{fs}-2k_{1_sup})}{2}$	
	$L_w = \frac{((14.5~{\rm in}) - 2 \times (1.4375~{\rm in}))}{2}$	
	$L_w=5.8125~{ m in}$	
	L_w - Total Effective Length of Weld	
	$L_w = 4 \left[L_w - 2 \left(W \le 0.3125 { m in} ight) ight]$	
	$L_w = 4 imes [(5.8125 ext{ in}) - 2 imes ((0.5 ext{ in}) \le (0.3125 ext{ in}))]$	
	$L_w=20.75~{ m in}$	
	Calculate the design capacity of weld	
	$F_{EXX}=70~{ m ksi}$ - Filler Metal Classification Strength $W=0.5~{ m in}$ - Fillet Weld Size	



AISC 15th Ed. Part 9 Eq. (9-2)	$D_{max} = 14.935$ - Maximum Fillet Weld Size for Base Metal Strength	
(3.2)	$L_w=20.75~{ m in}$ - 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	
	$\sqrt{2} \left(\sum_{max} D_{max} \right)$	
	$\phi R_n = \phi \ 0.6 \ F_{EXX} \ rac{1}{2} \left(W \leq rac{-max}{16} ight) \ L_w$	
	$\phi R_n = (0.75) imes 0.6 imes (70 ext{ ksi}) imes rac{\sqrt{2}}{2} imes \left((0.5 ext{ in}) \leq rac{(14.935)}{(10)} ight) imes (20.75 ext{ in})$	
	2 (16) /	
	$\phi R_n = 231.09 { m ~kip}$	
	Result:	PASS
	Demand over Capacity Ratio	1455
	$DCR = rac{P_f}{\phi R_r} = rac{(231.09 ext{ kip})}{(231.09 ext{ kip})} = 1$	
	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.	
	$t_{fp}=1.5~{ m in}$ - Flange Plate Thickness	
	$F_{up}=65~{ m ksi}$ - Flange Plate Tensile Stress	
	$t_{ws} = 0.44 ext{ in - Column Web Thickness}$	
AISC 15th Ed. Part 9 Eq.	$P_{us} = 03$ ksi - Column lensile Stress D_{us} Maximum Fillet Weld Size for Pase Metal Strength	
(9-2)	D _{max} - Maximum milet weld Size for base Metal Strength	
	$rac{t_{fp}}{2}\;F_{up}\leq t_{ws}\;F_{us}$	
	$D_{max} = rac{-2}{3.09 ext{ kip/in}}$	
	$D_{max} = rac{(0.75 ext{ in}) imes (65 ext{ ksi}) \leq (0.44 ext{ in}) imes (65 ext{ ksi})}{(2.00 ext{ ksi} + (1.0) ext{ ksi})}$	
	(3.09 kip/in)	
	$D_{max}=9.2557$	
	Calculate the total effective weld length. L_w - Total Effective Length of Weld	
	U = 2 [T = 2 (W < 0.2125 ; m)]	
	$L_w = 2 \left[I_{sup} - 2 \left(W \ge 0.3123 \text{ III} \right) \right]$	
	$L_w = 2 imes [(10 ext{ in}) - 2 imes ((0.5 ext{ in}) \le (0.3125 ext{ in}))]$	
	$L_w=18.75~{ m in}$	
	Calculate the design capacity of weld.	
	$P_f = 118.12 ~{ m kip}$ - Excess Flange force to be carried by the web welds	
	$F_{EXX} = 70 \text{ ksi}$ - Filler Metal Classification Strength	
AISC 15th Ed. Part 9 Eg.	$W = 0.5 ext{ in - Fillet Weld Size}$	
(9-2)	$D_{max} = 9.2557$ - Maximum Fillet weld Size for Base Metal Strength $L_{max} = 18.75$ in Total Effective Length of Wold	
	$\phi = 0.75$ - Fillet Weld Resistance Factor	
AISC 360-16 Chapter J2.4 Fa (12-4)	ϕR_n - Design Strength of Welds	
29.02 4		
	$\phi R_n = \phi \ 0.6 \ F_{EXX} \ rac{\sqrt{2}}{2} \left(W \leq rac{D_{max}}{16} ight) \ L_w$	
	$\phi P = (0.75) \times 0.6 \times (70 \text{ kgi}) \times \sqrt{2} \times ((0.5 \text{ in}) < (9.2557)) \times (18.75 \text{ in})$	
	$\varphi R_n = (0.75) \times 0.0 \times (70 \text{ ksr}) \times \frac{1}{2} \times ((0.5 \text{ m}) \le \frac{1}{(16)}) \times (18.75 \text{ m})$	
	$\phi R_n = 208.82~{ m kip}$	
	Result: Demand over Capacity Batio	PASS
	$DCR = \frac{P_f}{P_f} = \frac{(118.12 \text{ kip})}{0.56568} = 0.56568$	
	$DOR = \frac{1}{\phi R_n} = \frac{1}{(208.82 \text{ kip})} = 0.50500$	
	Check No. 13: Design Capacity of Support Web in Punching	
	Calculate the web capacity in punching of the supporting member. $P_f = 118.12 \text{ kip}$ - Excess Flange force to be carried by the web welds	
	$\phi=0.95$ - Web Punching Safety Factor	
	$F_{ys}=50~{ m ksi}$ - Column Yield Stress	
	$t_{ws}=0.44~{ m in}$ - Column Web Thickness	
	$r_{sup} = 10 \text{ m}$ - Column 1-Dimension ϕR_n - Design Web Shear Yielding (Punching)	
	$\phi \kappa_n = \phi \ 0.0 \ F_{ys} \ t_{ws} \ 2 \ T_{sup}$	
	$\phi R_n = (0.95) imes 0.6 imes (50 ext{ ksi}) imes (0.44 ext{ in}) imes 2 imes (10 ext{ in})$	
	$\phi R_n = 250.8 ~{ m kip}$	
	Result:	PASS
	Demand over Capacity Ratio	
	$DCR = rac{1}{\phi R_n} = rac{(10012 ext{ MP})}{(250.8 ext{ kip})} = 0.47099$	



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 Con	nection A	ISC 360-16 LI	RFD			
	Single Plate Geometry:	·					
	$b_p=21~{ m in}$ - Single Plate Width						
	$d_p=21~{ m in}$ - Single Plate Depth						
	$t_p=1~{ m in}$ - Single Plate Thickness						
	Single Plate Material Grade: $F_{em} = 50 \text{ ksi}$ - Single Plate Yield Stress						
	$F_{up}=65~{ m ksi}$ - Single Plate Tensile Stress						
	Connection Geometry:						
	$n_r=7$ - Number of Bolt Rows						
	$s_r = 3$ in - Bolt Row Spacing $n_r = 4$ - Number of Bolt Columns						
	$s_c = 3 ext{ in - Bolt Column Spacing}$						
	Distances:						
	$clr=8~{ m in}$ - Beam Web Clearance						
	$L_{eh_bm} = 2$ in - Horizontal Edge Distance on Beam Web						
	$L_{ev_pl} = 2.5 ext{ in}$ - Vertical Edge Distance on Single Plate						
	$e=14.5~{ m in}$ - Bolt Group Eccentricity						
	Check No. 1: Connection Detailing Limitations at Beam Side	1					
						-	
	Detailing Limitations	Limit Value (in)	Actual Value (in)	DCR	Result		
	Minimum Bolt Row Spacing	2.667	3.000	0.889	PASS	-	
	Minimum Bolt Column Spacing	6.000	3.000	0.500	PASS	4	
	Maximum Bolt Column Spacing	6.000	3.000	0.500	PASS	1	
	Plate Minimum Vertical Edge Distance	1.250	1.500	0.833	PASS	1	
	Plate Minimum Horizontal Edge Distance	1.375	2.000	0.688	PASS	1	
	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:						PASS
	Demand over Capacity Ratio						
	$DCR = \frac{1}{c} = \frac{1}{(3)} = 0.889$						
	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						
	ϕ^{-} . Bolt Diameter						
	C=28 - Calculated Bolt Group Coefficient						
	$F_{nv}=68~{ m ksi}$ - Bolt Nominal Shear Strength						
AISC 360-16 Chapter	$N_s = 1$ - Number of Shear Planes						
J5.2(a)	Ny Thiel Tactor for Dearing Doits						
	$h_f=0.85$	5 < 1 - 0.4 (t - 0.5)	$25 ext{ in}) \leq 1$				
	$b_{1} = 0.85 < 1$	$(0,4) \times ((0,in))$	(0.25 in)) < 1				
	$n_f = 0.85 < 1$	$-(0.4) \times ((0 \text{ m}) -$	$(0.25 \text{ III})) \leq 1$				
		$h_f=1$					
AISC 360-16 Chapter	A Design Balt Cheer Canacity						
J3.6 Eq. (J́3-1)	$\phi \kappa_n$ - Design Bolt Shear Capacity						
	ϕR_n =	$=\phiF_{nv}\;rac{\pi}{4}\left(d_b ight)^2N$	$f_s \ C \ h_f$				
		Ŧ					
	$\phi R_n = (0.75) imes (68$	ksi) $\times \frac{\pi}{4} \times ((1 \text{ in}))$	$\left(1 ight)^2 imes (1) imes (28) imes (1)$)			
		4					
		$\phi R_n = 1121.5 \; { m kip}$	•				
	Result:						PASS
	Demand over Capacity Ratio						
	$DCR = rac{v_u}{\phi R_n} = rac{(400 ext{ kip})}{(1121.5 ext{ kip})} = 0.35665$						
	Check No. 3: Design Capacity of the Bolt Group in Bearing a	nd Tear-out on Si	ngle Plate				
	Calculate the bolt bearing capacity of the single plate.						
	$d_b = 1$ in - Bolt Diameter $t_n = 1$ in - Single Plate Thickness						
	$F_{up}=65~{ m ksi}$ - Single Plate Tensile Stress						
	C=28 - Calculated Bolt Group Coefficient						
AISC 260 16 Chapter	$\phi=0.75$ - Bolt Bearing Resistance Factor						
J3.10 Eq. (J3-6a)	$\phi R_{n_bearing}$ - Design Bolt Bearing Capacity of Section						
	ϕR_{n_l}	$_{bearing}=\phi~2.4d_bt_p$	$F_{up} C$				
				`			
	$\phi R_{n_bearing} = (0.75)$:	imes 2.4 $ imes$ (1 in) $ imes$ (1	(1 in) imes (65 ksi) imes (28)	5)			
	φ	$R_{n\ bearing}=3276$ l	cip				
	φ	ocarring 0=101	-				
	Calculate the clear distance of outer bolts on single plate.						





 $L_{ev_pl} = 1.5 ~{
m in}$ - Vertical Edge Distance on Single Plate

 $d_h = 1.125 ~{
m in}$ - Vertical Bolt Hole Dimension at Plate

 l_{c1} - Clear Distance at First Bolt Row

$$l_{c1} = L_{ev_pl} - rac{d_h}{2}$$

$$l_{c1} = (1.5 ext{ in}) - rac{(1.125 ext{ in})}{2}$$

$$l_{c1} = 0.9375$$
 in

Calculate the clear distance of inner bolts on single plate.

 $s_r=3~{
m in}$ - Bolt Row Spacing

 $d_h = 1.125~{
m in}$ - Vertical Bolt Hole Dimension at Plate

 $l_{c2}\,$ - Clear Distance at Rest of Bolts

 $l_{c2} = s_r - d_h$

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

$$l_{c2} = 1.875$$
 in

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

 $l_{c1}=0.9375~{
m in}$ - Clear Distance at First Bolt Row

 $l_{c2}=1.875~{
m in}$ - Clear Distance at Rest of Bolts

 $t_p=1~{
m in}$ - Single Plate Thickness

 $F_{up}=65~{
m ksi}$ - 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

$$\phi R_{n_tearout} = \phi \, \left[1.2 \, l_{c1} \, t_p \, F_{up} + 1.2 \, l_{c2} \, t_p \, F_{up} \, (n_r - 1)
ight] \, \left(rac{C}{n_r}
ight)$$

$$\phi R_{n_tearout} = (0.75) imes [1.2 imes (0.9375 ext{ in}) imes (1 ext{ in}) imes (65 ext{ ksi}) + 1.2 imes (1.875 ext{ in}) imes (1 ext{ in}) imes (65 ext{ ksi}) imes ((7) - 1)] imes \left(rac{(28)}{(7)}
ight)$$

 $\phi R_{n_tearout} = 2851.9 ~{
m kip}$

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

AISC 360-16 Chapter $\phi R_{n, hearing} = 3276 ext{ kip}$ - Design bolt bearing capacity of single plate

JS.10 Eq. (JS-0a)		
AISC 360-16 Chapter J3.10 Eq. (J3-6c)	$\phi R_{n_tearout} = 2851.9~{ m kip}$ - Design bolt tear-out capacity of single plate	
AISC 360-16 Chapter J3.10	ϕR_n - Governing Design Capacity	
	$\phi R_n = min\left(\phi R_{n_bearing}, \phi R_{n_tearout} ight)$	
	$\phi R_n = min\left(\left(3276 \text{ kip}\right), \left(2851.9 \text{ kip}\right)\right)$	
	$\phi R_n = 2851.9~{ m kip}$	
	Result:	PASS
	$DCR = \frac{V_u}{V_u} = \frac{(400 \text{ kip})}{-0.14026} = 0.14026$	
	$DCR = \frac{1}{\phi R_n} = \frac{1}{(2851.9 \text{ kip})} = 0.14020$	
	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.	
	$d_b=1~{ m in}$ - Bolt Diameter	
	$t_{wb}=0.75~{ m in}$ - Beam Web Thickness	
	$F_{ub} = 65$ ksi - Beam Tensile Stress	
	C = 28 - Calculated Bolt Group Coefficient	
NISC 360-16 Chapter	$\phi = 0.75$ - Bolt Bearing Resistance Factor	
J3.10 Eq. (J3-6a)	$\phi R_{n_bearing}$ - Design Bolt Bearing Capacity of Section	
	$\phi R_{n_bearing} = \phi \ 2.4 \ d_b \ t_{wb} \ F_{ub} \ C$	
	$\phi R_{n_bearing} = (0.75) imes 2.4 imes (1 ext{ in}) imes (0.75 ext{ in}) imes (65 ext{ ksi}) imes (28)$	
	$\phi R_{n_bearing} = 2457 \; { m kip}$	
	Calculate the clear distance of inner bolts on beam web.	
	$s_r=3~{ m in}$ - Bolt Row Spacing	
	$d_h = 1.125 ~{ m in}$ - Vertical Bolt Hole Dimension at Beam	
	l_{c2} - Clear Distance at Rest of Bolts	
	$l_{c2}=s_r-d_h$	
	$l_{c2} = (3 \text{ in}) - (1.125 \text{ in})$	
	$l_{c2}=1.875~{\rm in}$	
	Calculate the bolt tear-out capacity of the beam web.	
	$l_{c2}=1.875~{ m in}$ - Clear Distance at Rest of Bolts	
	$t_{wb}=0.75~{ m in}$ - Beam Web Thickness	
	$F_{ub}=65~{ m ksi}$ - Beam Tensile Stress	



AISC 360-16 Chapter	C=28 - Calculated Bolt Group Coefficient $\phi=0.75$ - Bolt Bearing Resistance Factor ϕB_{rr} togget - Design Bolt Tear-out Capacity of Section	
J3.10 Eq. (J3-6C)		
	$\phi R_{n_tearout} = \phi \ 1.2 \ l_{c2} \ t_{wb} \ F_{ub} \ C$	
	$\phi R_{n_tearout} = (0.75) imes 1.2 imes (1.875 ext{ in}) imes (0.75 ext{ in}) imes (65 ext{ ksi}) imes (28)$	
	$\phi R_{n_tearout} = 2303.4 ~{ m kip}$	
	Determine the governing bearing and tear-out capacity of the bolt group on beam web.	
AISC 360-16 Chapter J3.10 Eq. (J3-6a)	$\phi R_{n_bearing} = 2457~{ m kip}$ - Design bolt bearing capacity of beam web	
AISC 360-16 Chapter J3.10 Eq. (J3-6c)	$\phi R_{n_tearout} = 2303.4~{ m kip}$ - Design bolt tear-out capacity of beam web	
AISC 360-16 Chapter J3.10	ϕR_n - Governing Design Capacity	
	$\phi R_n = min\left(\phi R_{n_bearing}, \phi R_{n_tearout} ight)$	
	$\phi R_n = min\left(\left(2457~\mathrm{kip}\right), \left(2303.4~\mathrm{kip}\right)\right)$	
	$\phi R_n = 2303.4~{ m kip}$	
	Result:	PASS
	Demand over Capacity Ratio	
	$DCR = rac{v_u}{\phi R_n} = rac{(400 ext{ Mp})}{(2303.4 ext{ kip})} = 0.17365$	
	Check No. 5: Design Capacity of Single Plate in Block Shear	
	Calculate the net area of the single plate subject to tension.	
	$v_p = 1 \text{ III}$ - Single Plate Thickness $n_p = A$. Number of Bolt Columns	
	$n_c = 4$ - Number of Bolt Columns $s_c = 3$ in - Bolt Column Spacing	
	$n_c = 4$ - Number of Bolt Columns $s_c = 3$ in - Bolt Column Spacing $L_{eh_pl} = 2$ in - Horizontal Edge Distance on Single Plate	
	$n_c = 4$ - Number of Bolt Columns $s_c = 3$ in - Bolt Column Spacing $L_{eh_pl} = 2$ in - Horizontal Edge Distance on Single Plate $d_h = 1.3125$ in - Horizontal Bolt Hole Dimension at Plate	
	$r_p = 1 \text{ m}$ - Single Plate Trickness $n_c = 4$ - 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)	
	$t_p = 1$ m - Single Plate Mitchless $n_c = 4$ - Number of Bolt Columns $s_c = 3$ in - Bolt Column Spacing $L_{ch_pl} = 2$ in - Horizontal Edge Distance on Single Plate $d_h = 1.3125$ in - Horizontal Bolt Hole Dimension at Plate A_{nt} - Net Area Subject to Tension (L-pattern) $A_{nt} = t_p [(n_c - 1) \ s_c + L_{eh_pl} - (n_c - 0.5) \ (d_h + 0.0625 \ in)]$	
	$\begin{aligned} n_c &= 4 \text{ Number of Bolt Columns} \\ s_c &= 3 \text{ in - Bolt Column Spacing} \\ L_{eh_pl} &= 2 \text{ in - Horizontal Edge Distance on Single Plate} \\ d_h &= 1.3125 \text{ in - Horizontal Bolt Hole Dimension at Plate} \\ A_{nt} &= \text{Net Area Subject to Tension (L-pattern)} \end{aligned}$ $\begin{aligned} A_{nt} &= t_p \left[(n_c - 1) \ s_c + L_{eh_pl} - (n_c - 0.5) \ (d_h + 0.0625 \text{ in}) \right] \\ A_{nt} &= (1 \text{ in}) \times \left[((4) - 1) \times (3 \text{ in}) + (2 \text{ in}) - ((4) - 0.5) \times ((1.3125 \text{ in}) + (0.0625 \text{ in})) \right] \end{aligned}$	
	$\begin{aligned} & t_p = 1 \text{ in - Single Flate Hitchless} \\ & n_c = 4 \text{ - Number of Bolt Columns} \\ & s_c = 3 \text{ in - Bolt Column Spacing} \\ & L_{eh_pl} = 2 \text{ in - Horizontal Edge Distance on Single Plate} \\ & d_h = 1.3125 \text{ in - Horizontal Bolt Hole Dimension at Plate} \\ & A_{nt} \text{ - Net Area Subject to Tension (L-pattern)} \end{aligned}$ $\begin{aligned} & A_{nt} = t_p \left[(n_c - 1) \ s_c + L_{eh_pl} - (n_c - 0.5) \ (d_h + 0.0625 \text{ in}) \right] \\ & A_{nt} = (1 \text{ in}) \times \left[((4) - 1) \times (3 \text{ in}) + (2 \text{ in}) - ((4) - 0.5) \times ((1.3125 \text{ in}) + (0.0625 \text{ in})) \right] \end{aligned}$ $\begin{aligned} & A_{nt} = 6.1875 \text{ in}^2 \end{aligned}$	

 $t_p = 1$ in - Single Plate Thickness $L_{ev_pl} = 1.5 ~{
m in}$ - Vertical Edge Distance on Single Plate $n_r=7$ - Number of Bolt Rows $s_r=3~{
m in}$ - Bolt Row Spacing $A_{gv}\,$ - Gross Area Subject to Shear (L-pattern) $A_{gv} = t_p \, \left[L_{ev_pl} + (n_r - 1) \, \, s_r
ight]$ $A_{qv} = (1 ext{ in}) imes [(1.5 ext{ in}) + ((7) - 1) imes (3 ext{ in})]$ $A_{gv}=19.5~{
m in}^2$ Calculate the net area of the single plate subject to shear. $t_p=1~{
m in}$ - Single Plate Thickness $d_p=21~{
m in}$ - Single Plate Depth $L_{ev_pl} = 1.5 ~{
m in}$ - Vertical Edge Distance on Single Plate $n_r=7$ - Number of Bolt Rows $d_h = 1.125 \; {
m in}$ - Vertical Bolt Hole Dimension at Plate $A_{nv}\,$ - Net Area Subject to Shear (L-pattern) $A_{nv} = t_p \, \left[d_p - L_{ev_pl} - (n_r - 0.5) \, \left(d_h + 0.0625 \ {
m in}
ight)
ight]$ $A_{nv} = (1 ext{ in}) imes [(21 ext{ in}) - (1.5 ext{ in}) - ((7) - 0.5) imes ((1.125 ext{ in}) + (0.0625 ext{ in}))]$ $A_{nv}=11.781~{\rm in}^2$ Calculate the design block shear capacity of the single plate. $F_{up}=65~{
m ksi}$ - Single Plate Tensile Stress $A_{nv}=11.781~{
m in}^2$ - Net Area Subject to Shear (L-pattern) $U_{bs}=0.5$ - Uniformity factor for multiple line of bolts $A_{nt}=6.1875~{
m in}^2$ - Net Area Subject to Tension (L-pattern) $F_{yp}=50~{
m ksi}$ - Single Plate Yield Stress $A_{gv}=19.5~{
m in}^2$ - 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 $\phi R_n = \phi \; (0.6 \; F_{up} \, A_{nv} + U_{bs} \; F_{up} \, A_{nt} \leq 0.6 \; F_{yp} \; A_{gv} + U_{bs} \; F_{up} \; A_{nt})$ $\phi R_n = (0.75) \times \left(0.6 \times (65 \text{ ksi}) \times \left(11.781 \text{ in}^2\right) + (0.5) \times (65 \text{ ksi}) \times \left(6.1875 \text{ in}^2\right) \le 0.6 \times (50 \text{ ksi}) \times \left(19.5 \text{ in}^2\right) + (0.5) \times (65 \text{ ksi}) \times \left(6.1875 \text{ in}^2\right)\right)$

Result:



 $\phi R_n = 495.42 ext{ kip}$

PASS

	Demand over Capacity Ratio $D C B = \frac{V_u}{V_u} = \frac{(400 \text{ kip})}{0.00730} = 0.00730$	
	$DCR = \frac{1}{\phi R_n} = \frac{1}{(495.42 \text{ kip})} = 0.80739$ Check No. 6: Design Capacity of Single Plate in Shear	
	Calculate the gross area of single plate subject to yielding.	
	$d_p=21~{ m in}$ - Single Plate Depth $t_n=1~{ m in}$ - Single Plate Thickness	
	A_{gv} - Section Gross Area	
	$A_{gv}=d_p \ t_p$	
	$A_{gv} = (21 \text{ in}) \times (1 \text{ in})$	
	$A_{gv}=21~{\rm in}^2$	
	Calculate the shear yielding capacity of the single plate.	
	$F_{yp}=50~{ m ksi}$ - Single Plate Yield Stress $A_{av}=21~{ m in}^2$ - Section Gross Area	
AISC 360-16 Chapter J4.2 Eq. (J4-3)	$\phi=1$ - Shear Yielding Resistance Factor	
	ϕR_{n_sy} - Design Shear Yielding Capacity of Section	
	$\phi R_{n_sy} = \phi \ 0.6 \ F_{yp} \ A_{gv}$	
	$\phi R_{n_sy} = (1) imes 0.6 imes (50 ext{ ksi}) imes (21 ext{ in}^2)$	
	$\phi R_{n_sy} = 630 \; \rm kip$	
	Calculate the net area of single plate subject to rupture. $t_p=1 { m in}$ - Single Plate Thickness	
	$d_p=21~{ m in}$ - Single Plate Depth	
	$n_r=7$ - Number of Bolt Rows $d_h=1.125~{ m in}$ - Vertical Bolt Hole Dimension at Plate	
	A_{nv} - Section Net Area	
	$A_{nv} = t_p \left[a_p - n_r \left(a_h + 0.0025 \mathrm{m} ight) ight]$	
	$A_{nv} = (1 \; \mathrm{in}) \times [(21 \; \mathrm{in}) - (7) \times ((1.125 \; \mathrm{in}) + (0.0625 \; \mathrm{in}))]$	
	$A_{nv}=12.688~{\rm in}^2$	
	Calculate the shear rupture capacity of the single plate. $F_{up}=65~{ m ksi}$ - Single Plate Tensile Stress	
	$A_{nv}=12.688~{ m in}^2$ - Section Net Area $\phi=1$ - Shear Yielding Resistance Factor	
AISC 360-16 Chapter 14.2 Eg. (14-4)	$\phi = 1$ - Shear Heiding Resistance Factor ϕR_{n_sr} - Design Shear Rupture Capacity of Section	
	$\phi R_{n_sr} = \phi \ 0.6 \ F_{up} \ A_{nv}$	
	$\phi R_{n_sr} = (1) imes 0.6 imes (65 ext{ ksi}) imes ig(12.688 ext{ in}^2 ig)$	
	$\phi R_{n_sr} = 494.81 { m ~kip}$	
	Determine the governing shear capacity of the single plate.	
AISC 360-16 Chapter J4.2 Eq. (J4-3)	$\phi R_{n_sy} = 630 ~{ m kip}$ - Design shear yielding capacity of single plate	
AISC 300-10 Chapter J4.2 Eq. (J4-4) AISC 360-16 Chapter	$\phi R_{n_sr} = 494.81 ext{ kip}$ - Design shear rupture capacity of single plate ϕR_n - Governing Design Capacity	
J4.2	$\phi R_n = min \left(\phi R_n _{su}, \phi R_n _{sr} \right)$	
	dP = min((620 kin) (404.81 kin))	
	$\varphi_{1} u_n = min((0.50 \text{ klp}), (4.94.01 \text{ klp}))$	
	$\phi R_n = 494.81 ext{ kip}$ Result:	PASS
	Demand over Capacity Ratio $V_{\mu} = \frac{V_{\mu}}{V_{\mu}} = 0.00000$	
	$DCR = \frac{1}{\phi R_n} = \frac{1}{(494.81 \text{ kip})} = 0.80839$ 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 Clearance8.0000.5000.063PASS	
	Result:	PASS
	$DCR = \frac{d}{c} = \frac{(0.5)}{(8)} = 0.0625$	
	Check No. 8: Design Capacity of Weld to Support Web	
	$t_p = 1 ext{ in - Single Plate Thickness}$	
	$F_{up}=65~{ m ksi}$ - Single Plate Tensile Stress	
	$ au_{ws}=0.44~{ m m}$ - Column Web Thickness $F_{us}=65~{ m ksi}$ - Column Tensile Stress	
AISC 15th Ed. Part 9 Eq. (9-2)	D_{max} - Maximum Fillet Weld Size for Base Metal Strength	





$$D_{max} = \frac{\frac{1}{2} F_{max}}{3.00 \text{ kip}/\text{im}}$$

$$D_{max} = \frac{(1.5 \text{ m}) \times (65 \text{ ks}) \leq (0.41 \text{ m}) \times (65 \text{ ks})}{(3.00 \text{ kip}/\text{m})}$$

$$D_{max} = 9.2567$$
Calculate the total effective weld length for NS/FS fillet welds.

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

$$d_{\mu} = 21 \text{ in : Single Fillet Oright}$$

$$L_{waster} = 0.3255 \text{ in : Maximum weld kength reduction}$$

$$L_{\mu} = 0.215 \text{ in : Here the Deglth}$$

$$L_{w} = 2 \cdot (d_{\mu} - 2 \cdot (W \leq 0.3125 \text{ in}))$$

$$L_{w} = 2 \cdot (0.5 \text{ in}) \leq (0.3125 \text{ in}))$$

$$L_{w} = 2 \cdot (0.5 \text{ in}) \leq (0.3125 \text{ in}))$$

$$L_{w} = 40.75 \text{ in}$$
Calculate the design capacity of weld in shear.

$$P_{2XX} = 70 \text{ ks} \cdot \text{Filler Weld Size}$$

$$W = 0.5 \text{ in . Filler Weld Size for Base Metal Strength}$$

$$L_{w} = 40.75 \text{ in}$$
Calculate the design capacity of weld in shear.

$$P_{2XX} = 70 \text{ ks} \cdot \text{Filler Weld Size}$$

$$d_{R_{w}} = 0.055 \text{ in . Here Weld Size}$$

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$$d_{R_{w}} = 0.055 \text{ in . Here Weld Size}$$

$$d_{R_{w}} = 0.055 \text{ in . Steppendow Weld}$$

$$d_{R_{w}} = 0.055 \text{ in . Steppendow Weld}$$

$$d_{R_{w}} = 0.055 \text{ in . Steppendow Weld}$$

$$d_{R_{w}} = 0.055 \times 0.6 \times (70 \text{ ks}) \times \frac{\sqrt{2}}{2} \times \left((0.5 \text{ in}) \le \frac{(0.2557)}{(28)} \right) \times (40.75 \text{ in})$$

$$d_{R_{w}} = 453.83 \text{ kip}$$

$$Examt:$$

$$Dematin dower Capacity fieldo
$$DCR = \frac{V_{K_{w}}}{V_{K_{w}}}} = \frac{(0.8320 \text{ Here}}{V_{K_{w}}} = 0.83130 \text{ Here}$$$$

PASS



REFERENCES	CALCULATIONS						
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		

