

REFERENCES	CALCULATIONS	RESULTS
 SkyCiv CLOUD STRUCTURAL SOFTWARE	Beam to Column Flange Vertical Brace Connection Calculations	
	Column Section Properties: W14x90 - Column Size $d_c = 14$ in - Column Depth $t_{w,c} = 0.44$ in - Column Web Thickness $b_{f,c} = 14.5$ in - Column Flange Width $t_{f,c} = 0.71$ in - Column Flange Thickness $A_c = 26.5$ in ² - Column Area Column Grade Information: A992 - Material Grade $F_{y,c} = 50$ ksi - Column Yield Stress $F_{u,c} = 65$ ksi - Column Tensile Stress	
	Beam Section Properties: W18x50 - Beam Size $d_{bm} = 18$ in - Beam Depth $t_{w,bm} = 0.355$ in - Beam Web Thickness $b_{f,bm} = 7.5$ in - Beam Flange Width $t_{f,bm} = 0.57$ in - Beam Flange Thickness $A_{bm} = 14.7$ in ² - Beam Area $clr = 1$ in - Beam Clearance Beam Grade Information: A992 - Material Grade $F_{y,bm} = 50$ ksi - Beam Yield Stress $F_{u,bm} = 65$ ksi - Beam Tensile Stress	
	Lower Brace Section Properties: 2L5x5x1/2 - Lower Brace Size $A_{l,br} = 9.58$ in ² - Lower Brace Area $L_{on,gp,l,br} = 5$ in - Lower Brace Leg on Gusset $L_{off,gp,l,br} = 5$ in - Lower Brace Leg off Gusset $t_{l,br} = 0.5$ in - Lower Brace Thickness $\theta_{l,br} = 45$ - Lower Brace Slope from Vertical in Degrees $L_{b,l,br} = 11$ in - Lower Brace Unbraced Length Lower Brace Grade Information: A36 - Material Grade $F_{y,l,br} = 36$ ksi - Lower Brace Yield Stress $F_{u,l,br} = 58$ ksi - Lower Brace Tensile Stress	
	Lower Brace Double Angle at Column Section Properties: 2L5x5x1/2 - Lower Brace Double Angle at Column Size $L_{NSL,l,br} = 5$ in - Lower Brace Double Angle Leg at Beam Side $L_{OSL,l,br} = 5$ in - Lower Brace Double Angle Leg at Column Side $t_{2L,at,col,l,br} = 0.5$ in - Lower Brace Double Angle Thickness Lower Brace Double Angle at Column Grade Information: A36 - Material Grade $F_{y,at,col,l,br} = 36$ ksi - Lower Brace Double Angle Yield Stress $F_{u,at,col,l,br} = 58$ ksi - Lower Brace Double Angle Tensile Stress	
	Upper Brace Section Properties: 2L5x5x1/2 - Upper Brace Size $A_{u,br} = 9.58$ in ² - Upper Brace Area $L_{on,gp,u,br} = 5$ in - Upper Brace Leg on Gusset $L_{off,gp,u,br} = 5$ in - Upper Brace Leg off Gusset $t_{u,br} = 0.5$ in - Upper Brace Thickness $\theta_{u,br} = 45$ - Upper Brace Slope from Vertical in Degrees $L_{b,u,br} = 11$ in - Upper Brace Unbraced Length Upper Brace Grade Information: A36 - Material Grade $F_{y,u,br} = 36$ ksi - Upper Brace Yield Stress $F_{u,u,br} = 58$ ksi - Upper Brace Tensile Stress	
	Upper Brace Double Angle at Column Section Properties: 2L5x5x1/2 - Upper Brace Double Angle at Column Size $L_{NSL,u,br} = 5$ in - Upper Brace Double Angle Leg at Beam Side $L_{OSL,u,br} = 5$ in - Upper Brace Double Angle Leg at Column Side $t_{2L,at,col,u,br} = 0.5$ in - Upper Brace Double Angle Thickness Upper Brace Double Angle at Column Grade Information: A36 - Material Grade $F_{y,at,col,u,br} = 36$ ksi - Upper Brace Double Angle Yield Stress $F_{u,at,col,u,br} = 58$ ksi - Upper Brace Double Angle Tensile Stress	
	Beam Double Angle at Column Section Properties: 2L5x5x1/2 - Beam Double Angle at Column Size $L_{NSL,bm} = 5$ in - Beam Double Angle Leg at Beam Side $L_{OSL,bm} = 5$ in - Beam Double Angle Leg at Column Side $t_{2L,at,col,bm} = 0.5$ in - Beam Double Angle Thickness Beam Double Angle at Column Grade Information: A36 - Material Grade	

	$F_{y,at,col,bm} = 36 \text{ ksi}$ - Beam Double Angle Yield Stress $F_{u,at,col,bm} = 58 \text{ ksi}$ - Beam Double Angle Tensile Stress	
	Design Loads: $V_u = 11 \text{ kip}$ - Beam Vertical Shear Load $T_u = 5 \text{ kip}$ - Beam Tension Load $C_u = 5 \text{ kip}$ - Beam Compression Load $T_{u,l,br} = 25 \text{ kip}$ - Lower Brace Tension Load $C_{u,l,br} = 25 \text{ kip}$ - Lower Brace Compression Load $T_{u,u,br} = 15 \text{ kip}$ - Upper Brace Tension Load $C_{u,u,br} = 15 \text{ kip}$ - Upper Brace Compression Load	
AISC 15th Edition page 13-3	Uniform Force Method Loads at Lower Brace: $e_c = 7 \text{ in}$ - Half Column Depth $e_b = 9 \text{ in}$ - Half Beam Depth $r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2} = \sqrt{((9.5 \text{ in}) + (7 \text{ in}))^2 + ((7.5 \text{ in}) + (9 \text{ in}))^2} = 23.335 \text{ in}$ $\alpha = e_b \tan(\theta_{l,br}) - e_c + \beta \tan(\theta_{l,br}) = (9 \text{ in}) \times \tan((45)) - (7 \text{ in}) + (7.5 \text{ in}) \times \tan((45)) = 9.5 \text{ in}$ $\bar{\alpha} = 8.5 \text{ in}$ - Distance from Face of Column to Center of Gusset to Beam Connection $\beta = 7.5 \text{ in}$ - Half of Gusset Plate Height along Column Compression Load at Gusset to Column $H_{c,C} = \frac{e_c C_{u,l,br}}{r} = \frac{(7 \text{ in}) \times (25 \text{ kip})}{(23.335 \text{ in})} = 7.4996 \text{ kip}$ Tension Load at Gusset to Column $H_{c,T} = \frac{e_c T_{u,l,br}}{r} = \frac{(7 \text{ in}) \times (25 \text{ kip})}{(23.335 \text{ in})} = 7.4996 \text{ kip}$ Vertical Shear at Gusset to Column $V_c = \frac{\beta T_{u,l,br}}{r} + \Delta V_b = \frac{(7.5 \text{ in}) \times (25 \text{ kip})}{(23.335 \text{ in})} + (9.6424 \text{ kip}) = 17.678 \text{ kip}$ Horizontal Shear at Gusset to Beam $H_b = \frac{\alpha T_{u,l,br}}{r} = \frac{(9.5 \text{ in}) \times (25 \text{ kip})}{(23.335 \text{ in})} = 10.178 \text{ kip}$ Vertical Axial Load at Gusset to Beam $V_b = \frac{e_b T_{u,l,br}}{r} - \Delta V_b = \frac{(9 \text{ in}) \times (25 \text{ kip})}{(23.335 \text{ in})} - (9.6424 \text{ kip}) = 0 \text{ kip}$ Moment Load at Gusset to Beam $M_b = V_b \alpha - \bar{\alpha} + \Delta V_b \bar{\alpha} = (0 \text{ kip}) \times (9.5 \text{ in}) - (8.5 \text{ in}) + (9.6424 \text{ kip}) \times (8.5 \text{ in}) = 6.83 \text{ kip} \cdot \text{ft}$	
AISC 15th Edition page 13-3	Uniform Force Method Loads at Upper Brace: $e_c = 7 \text{ in}$ - Half Column Depth $e_b = 9 \text{ in}$ - Half Beam Depth $r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2} = \sqrt{((8.75 \text{ in}) + (7 \text{ in}))^2 + ((6.75 \text{ in}) + (9 \text{ in}))^2} = 22.274 \text{ in}$ $\alpha = e_b \tan(\theta_{u,br}) - e_c + \beta \tan(\theta_{u,br}) = (9 \text{ in}) \times \tan((45)) - (7 \text{ in}) + (6.75 \text{ in}) \times \tan((45)) = 8.75 \text{ in}$ $\bar{\alpha} = 8.5 \text{ in}$ - Distance from Face of Column to Center of Gusset to Beam Connection $\beta = 6.75 \text{ in}$ - Half of Gusset Plate Height along Column Compression Load at Gusset to Column $H_{c,C} = \frac{e_c C_{u,u,br}}{r} = \frac{(7 \text{ in}) \times (15 \text{ kip})}{(22.274 \text{ in})} = 4.714 \text{ kip}$ Tension Load at Gusset to Column $H_{c,T} = \frac{e_c T_{u,u,br}}{r} = \frac{(7 \text{ in}) \times (15 \text{ kip})}{(22.274 \text{ in})} = 4.714 \text{ kip}$ Vertical Shear at Gusset to Column $V_c = \frac{\beta T_{u,u,br}}{r} + \Delta V_b = \frac{(6.75 \text{ in}) \times (15 \text{ kip})}{(22.274 \text{ in})} + (6.0609 \text{ kip}) = 10.607 \text{ kip}$ Horizontal Shear at Gusset to Beam $H_b = \frac{\alpha T_{u,u,br}}{r} = \frac{(8.75 \text{ in}) \times (15 \text{ kip})}{(22.274 \text{ in})} = 5.8926 \text{ kip}$ Vertical Axial Load at Gusset to Beam $V_b = \frac{e_b T_{u,u,br}}{r} - \Delta V_b = \frac{(9 \text{ in}) \times (15 \text{ kip})}{(22.274 \text{ in})} - (6.0609 \text{ kip}) = 0 \text{ kip}$ Moment Load at Gusset to Beam $M_b = V_b \alpha - \bar{\alpha} + \Delta V_b \bar{\alpha} = (0 \text{ kip}) \times (8.75 \text{ in}) - (8.5 \text{ in}) + (6.0609 \text{ kip}) \times (8.5 \text{ in}) = 4.2931 \text{ kip} \cdot \text{ft}$	
AISC 15th Edition page 13-3	Design Loads at Beam from UFM: $C_{total} = C_u + \max(H_{c,C,lower}, H_{c,C,upper}) = (5 \text{ kip}) + \max((7.4996 \text{ kip}), (4.714 \text{ kip})) = 12.5 \text{ kip}$ $T_{total} = T_u + \max(H_{c,T,lower}, H_{c,T,upper}) = (5 \text{ kip}) + \max((7.4996 \text{ kip}), (4.714 \text{ kip})) = 12.5 \text{ kip}$ $V_{total} = V_u + V_{b,lower} + V_{b,upper} = (11 \text{ kip}) + (0 \text{ kip}) + (0 \text{ kip}) = 11 \text{ kip}$	
	Connection Information at Beam Web to Column: 2L5x5x1/2 - Beam Double Angle at Column Section Size $L_{NSL,bm} = 5 \text{ in}$ - Beam Double Angle Leg at Beam Side $L_{OSL,bm} = 5 \text{ in}$ - Beam Double Angle Leg at Column Side $t_{2L,at,col,bm} = 0.5 \text{ in}$ - Beam Double Angle Thickness $L_{2L,at,col,bm} = 12 \text{ in}$ - Beam Double Angle Length Bolt Information at Double Angle to Beam Web: $d_b = 0.75 \text{ in}$ - Bolt Diameter A325-SC(A) - Bolt Type STD - Hole Type at Beam Web HSSL - Hole Type at Double Angle $n_r = 4$ - Number of Bolt Rows $s_r = 3 \text{ in}$ - Bolt Row Spacing $n_c = 1$ - Number of Bolt Columns $s_c = 0 \text{ in}$ - Bolt Column Spacing $L_{eh} = 2 \text{ in}$ - Horizontal Edge Distance at Beam Web Bolt Information at Double Angle to Column: $d_b = 0.75 \text{ in}$ - Bolt Diameter A325N - Bolt Type STD - Hole Type at Column	

	HSSL - Hole Type at Double Angle $n_r = 4$ - Number of Bolt Rows $s_r = 3$ in - Bolt Row Spacing $n_c = 1$ - Number of Bolt Columns $s_c = 0$ in - Bolt Column Spacing $g_a = 3.5$ in - Distance from beam center to first bolt column	
	Connection Information at Lower Brace to Gusset Plate: $t_{gp,l,br} = 1$ in - Lower Brace Gusset Plate Thickness $d_b = 0.75$ in - Bolt Diameter A325-SC(A) - Bolt Type STD - Hole Type at Lower Brace OVS - Hole Type at Gusset Plate $g_a = 3$ in - Bolt Gage $n_r = 3$ - Number of Bolt Rows $s_r = 3$ in - Bolt Row Spacing $n_c = 1$ - Number of Bolt Columns $L_{ev} = 2$ in - Vertical Edge Distance at Brace	
	Connection Information at Lower Gusset Plate to Beam: $W = 0.25$ in - Fillet Weld Size $L_w = 15$ in - Weld Length E70XX - Filler Metal Classification	
	Connection Information at Lower Double Angle to Column: 2L5x5x1/2 - Lower Double Angle at Column Section Size $L_{NSL,l,br} = 5$ in - Lower Brace Double Angle Leg at Beam Side $L_{OSL,l,br} = 5$ in - Lower Brace Double Angle Leg at Column Side $t_{2L,at,col,l,br} = 0.5$ in - Lower Brace Double Angle Thickness $L_{2L,at,col,l,br} = 12$ in - Lower Brace Double Angle Length Bolt Information at Lower Double Angle to Gusset: $d_b = 0.75$ in - Bolt Diameter A325-SC(A) - Bolt Type STD - Hole Type at Beam Web HSSL - Hole Type at Double Angle $n_r = 4$ - Number of Bolt Rows $s_r = 3$ in - Bolt Row Spacing $n_c = 1$ - Number of Bolt Columns $s_c = 0$ in - Bolt Column Spacing $L_{eh} = 2$ in - Horizontal Edge Distance at Gusset Plate Bolt Information at Lower Double Angle to Column: $d_b = 0.75$ in - Bolt Diameter A325N - Bolt Type STD - Hole Type at Column HSSL - Hole Type at Double Angle $n_r = 4$ - Number of Bolt Rows $s_r = 3$ in - Bolt Row Spacing $n_c = 1$ - Number of Bolt Columns $s_c = 0$ in - Bolt Column Spacing $g_a = 3.5$ in - Distance from beam center to first bolt column	
	Connection Information at Upper Brace to Gusset Plate: $t_{gp,u,br} = 1$ in - Upper Brace Gusset Plate Thickness $d_b = 0.75$ in - Bolt Diameter A325-SC(A) - Bolt Type STD - Hole Type at Upper Brace OVS - Hole Type at Gusset Plate $g_a = 3$ in - Bolt Gage $n_r = 3$ - Number of Bolt Rows $s_r = 3$ in - Bolt Row Spacing $n_c = 1$ - Number of Bolt Columns $L_{ev} = 2$ in - Vertical Edge Distance at Brace	
	Connection Information at Upper Gusset Plate to Beam: $W = 0.25$ in - Fillet Weld Size $L_w = 15$ in - Weld Length E70XX - Filler Metal Classification	
	Connection Information at Upper Double Angle to Column: 2L5x5x1/2 - Upper Double Angle at Column Section Size $L_{NSL,u,br} = 5$ in - Upper Brace Double Angle Leg at Beam Side $L_{OSL,u,br} = 5$ in - Upper Brace Double Angle Leg at Column Side $t_{2L,at,col,u,br} = 0.5$ in - Upper Brace Double Angle Thickness $L_{2L,at,col,u,br} = 12$ in - Upper Brace Double Angle Length Bolt Information at Upper Double Angle to Gusset: $d_b = 0.75$ in - Bolt Diameter A325-SC(A) - Bolt Type STD - Hole Type at Beam Web HSSL - Hole Type at Double Angle $n_r = 4$ - Number of Bolt Rows $s_r = 3$ in - Bolt Row Spacing $n_c = 1$ - Number of Bolt Columns $s_c = 0$ in - Bolt Column Spacing $L_{eh} = 2$ in - Horizontal Edge Distance at Gusset Plate Bolt Information at Upper Double Angle to Column: $d_b = 0.75$ in - Bolt Diameter A325N - Bolt Type	

STD - Hole Type at Column HSSL - Hole Type at Double Angle $n_r = 4$ - Number of Bolt Rows $s_r = 3$ in - Bolt Row Spacing $n_c = 1$ - Number of Bolt Columns $s_c = 0$ in - Bolt Column Spacing $g_a = 3.5$ in - Distance from beam center to first bolt column	
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REFERENCES	CALCULATIONS	RESULTS																																			
	Lower Brace Connection to Gusset Plate AISC 360-16 LRFD Check No. 1: Connection Detailing Limitations <table><thead><tr><th>Detailing Limitations</th><th>Limit Value (in)</th><th>Actual Value (in)</th><th>DCR</th><th>Result</th></tr></thead><tbody><tr><td>Minimum Bolt Gage</td><td>1.750</td><td>3.000</td><td>0.583</td><td>PASS</td></tr><tr><td>Minimum Bolt Row Spacing</td><td>2.000</td><td>3.000</td><td>0.667</td><td>PASS</td></tr><tr><td>Maximum Bolt Row Spacing</td><td>6.000</td><td>3.000</td><td>0.500</td><td>PASS</td></tr><tr><td>Gusset Plate Minimum Vertical Edge Distance</td><td>1.063</td><td>2.000</td><td>0.531</td><td>PASS</td></tr><tr><td>Brace Minimum Vertical Edge Distance</td><td>1.000</td><td>2.000</td><td>0.500</td><td>PASS</td></tr><tr><td>Brace Minimum Horizontal Edge Distance</td><td>1.000</td><td>2.000</td><td>0.500</td><td>PASS</td></tr></tbody></table> Result: Demand over Capacity Ratio $DCR = \frac{d}{c} = \frac{(2)}{(3)} = 0.66667$	Detailing Limitations	Limit Value (in)	Actual Value (in)	DCR	Result	Minimum Bolt Gage	1.750	3.000	0.583	PASS	Minimum Bolt Row Spacing	2.000	3.000	0.667	PASS	Maximum Bolt Row Spacing	6.000	3.000	0.500	PASS	Gusset Plate Minimum Vertical Edge Distance	1.063	2.000	0.531	PASS	Brace Minimum Vertical Edge Distance	1.000	2.000	0.500	PASS	Brace Minimum Horizontal Edge Distance	1.000	2.000	0.500	PASS	PASS
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AISC 360-16 Chapter J3.8 Eq. (J3-4)	Check No. 2: Design Capacity of the Bolts in Shear $\phi = 0.85$ - Bolt Shear Resistance Factor $\mu = 0.3$ - Mean Slip Coefficient $D_u = 1.13$ $h_f = 1$ - Filler Factor for SC Bolts $d_b = 0.75$ in - Bolt Diameter $T_b = 28$ kip - Minimum Bolt Pretension $N_s = 2$ - Number of Slip Planes $n_r = 3$ - Number of Bolt Rows $n_c = 1$ - Number of Bolt Columns ϕR_n - Design Bolt Shear Capacity $\phi R_n = \phi \mu D_u h_f T_b N_s n_r n_c$$\phi R_n = (0.85) \times (0.3) \times (1.13) \times (1) \times (28 \text{ kip}) \times (2) \times (3) \times (1)$$\phi R_n = 48.409 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{T_{u,l,br}}{\phi R_n} = \frac{(25 \text{ kip})}{(48.409 \text{ kip})} = 0.51643$	PASS																																			
AISC 360-16 Chapter J3.10 Eq. (J3-6a)	Check No. 3: Design Bolt Bearing Capacity of the Gusset Plate Calculate the bolt bearing capacity of the gusset plate. $\phi = 0.75$ - Bolt Bearing Resistance Factor $d_b = 0.75$ in - Bolt Diameter $t_{gp,l,br} = 1$ in - Lower Brace Gusset Plate Thickness $F_{u,l,br} = 65$ ksi - Lower Brace Gusset Plate Tensile Stress $n_r = 3$ - Number of Bolt Rows $n_c = 1$ - Number of Bolt Columns $\phi R_{n_bearing}$ - Design Bolt Bearing Capacity of Section $\phi R_{n_bearing} = \phi 2.4 d_b t_{gp,l,br} F_{u,l,br} n_r n_c$$\phi R_{n_bearing} = (0.75) \times 2.4 \times (0.75 \text{ in}) \times (1 \text{ in}) \times (65 \text{ ksi}) \times (3) \times (1)$$\phi R_{n_bearing} = 263.25 \text{ kip}$ Calculate the clear distance of outer bolts on gusset plate. $L_{ev} = 2$ in - Vertical Edge Distance at Brace $d_h = 0.9375$ in - Vertical Bolt Hole Dimension at Gusset Plate l_{c1} - Clear Distance at First Bolt Row $l_{c1} = L_{ev} - \frac{d_h}{2}$$l_{c1} = (2 \text{ in}) - \frac{(0.9375 \text{ in})}{2}$$l_{c1} = 1.5313 \text{ in}$ Calculate the clear distance of inner bolts on gusset plate. $s_r = 3$ in - Bolt Row Spacing $d_h = 0.9375$ in - Vertical Bolt Hole Dimension at Gusset Plate l_{c2} - Clear Distance at Rest of Bolts $l_{c2} = s_r - d_h$$l_{c2} = (3 \text{ in}) - (0.9375 \text{ in})$$l_{c2} = 2.0625 \text{ in}$ Calculate the bolt tear-out capacity of the gusset plate. $l_{c1} = 1.5313$ in - Clear Distance at First Bolt Row $l_{c2} = 2.0625$ in - Clear Distance at Rest of Bolts $t_{gp,l,br} = 1$ in - Lower Brace Gusset Plate Thickness $F_{u,l,br} = 65$ ksi - Lower Brace Gusset Plate Tensile Stress $n_r = 3$ - Number of Bolt Rows																																				

AISC 360-16 Chapter J3.10 Eq. (J3-6c) AISC 360-16 Chapter J3.10 Eq. (J3-6a) AISC 360-16 Chapter J3.10 Eq. (J3-6c) AISC 360-16 Chapter J3.10	$n_c = 1$ - Number of Bolt Columns $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_tearout}$ - Design Bolt Tear-out Capacity of Section $\phi R_{n_tearout} = \phi [1.2 l_{c1} t_{gp, l, br} F_{u, l, br} n_c + 1.2 l_{c2} t_{gp, l, br} F_{u, l, br} n_c (n_r - 1)]$ $\phi R_{n_tearout} = (0.75) \times [1.2 \times (1.5313 \text{ in}) \times (1 \text{ in}) \times (65 \text{ ksi}) \times (1) + 1.2 \times (2.0625 \text{ in}) \times (1 \text{ in}) \times (65 \text{ ksi}) \times (1) \times ((3) - 1)]$ $\phi R_{n_tearout} = 330.89 \text{ kip}$ Determine the governing bearing and tear-out capacity of the bolt group on gusset plate. $\phi R_{n_bearing} = 263.25 \text{ kip}$ - Design Bolt Bearing Capacity of Section $\phi R_{n_tearout} = 330.89 \text{ kip}$ - Design Bolt Tear-out Capacity of Section ϕR_n - Governing Design Capacity $\phi R_n = \min(\phi R_{n_bearing}, \phi R_{n_tearout})$ $\phi R_n = \min((263.25 \text{ kip}), (330.89 \text{ kip}))$ $\phi R_n = 263.25 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{T_{u, l, br}}{\phi R_n} = \frac{(25 \text{ kip})}{(263.25 \text{ kip})} = 0.094967$	PASS
AISC 360-16 Chapter J3.10 Eq. (J3-6a) AISC 360-16 Chapter J3.10 Eq. (J3-6c) AISC 360-16 Chapter J3.10	Check No. 4: Design Bolt Bearing Capacity of the Brace Calculate the bolt bearing capacity of the brace. $\phi = 0.75$ - Bolt Bearing Resistance Factor $d_b = 0.75 \text{ in}$ - Bolt Diameter $t_{l, br} = 0.5 \text{ in}$ - Lower Brace Thickness $F_{u, l, br} = 58 \text{ ksi}$ - Lower Brace Tensile Stress $n_r = 3$ - Number of Bolt Rows $n_c = 1$ - Number of Bolt Columns $\phi R_{n_bearing}$ - Design Bolt Bearing Capacity of Section $\phi R_{n_bearing} = \phi 2.4 d_b 2t_{l, br} F_{u, l, br} n_r n_c$ $\phi R_{n_bearing} = (0.75) \times 2.4 \times (0.75 \text{ in}) \times (1 \text{ in}) \times (58 \text{ ksi}) \times (3) \times (1)$ $\phi R_{n_bearing} = 234.9 \text{ kip}$ Calculate the clear distance of outer bolts on brace. $L_{ev} = 2 \text{ in}$ - Vertical Edge Distance at Brace $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Brace l_{c1} - Clear Distance at First Bolt Row $l_{c1} = L_{ev} - \frac{d_h}{2}$ $l_{c1} = (2 \text{ in}) - \frac{(0.8125 \text{ in})}{2}$ $l_{c1} = 1.5938 \text{ in}$ Calculate the clear distance of inner bolts on brace. $s_r = 3 \text{ in}$ - Bolt Row Spacing $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Brace 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 brace. $l_{c1} = 1.5938 \text{ in}$ - Clear Distance at First Bolt Row $l_{c2} = 2.1875 \text{ in}$ - Clear Distance at Rest of Bolts $t_{l, br} = 0.5 \text{ in}$ - Lower Brace Thickness $F_{u, l, br} = 58 \text{ ksi}$ - Lower Brace Tensile Stress $n_r = 3$ - Number of Bolt Rows $n_c = 1$ - Number of Bolt Columns $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_tearout}$ - Design Bolt Tear-out Capacity of Section $\phi R_{n_tearout} = \phi [1.2 l_{c1} 2t_{l, br} F_{u, l, br} n_c + 1.2 l_{c2} 2t_{l, br} F_{u, l, br} n_c (n_r - 1)]$ $\phi R_{n_tearout} = (0.75) \times [1.2 \times (1.5938 \text{ in}) \times (1 \text{ in}) \times (58 \text{ ksi}) \times (1) + 1.2 \times (2.1875 \text{ in}) \times (1 \text{ in}) \times (58 \text{ ksi}) \times (1) \times ((3) - 1)]$ $\phi R_{n_tearout} = 311.57 \text{ kip}$ Determine the governing bearing and tear-out capacity of the bolt group on brace. $\phi R_{n_bearing} = 234.9 \text{ kip}$ - Design Bolt Bearing Capacity of Section $\phi R_{n_tearout} = 311.57 \text{ kip}$ - Design Bolt Tear-out Capacity of Section	

	ϕR_n - Governing Design Capacity $\phi R_n = \min(\phi R_{n,bearing}, \phi R_{n,tearout})$ $\phi R_n = \min((234.9 \text{ kip}), (311.57 \text{ kip}))$ $\phi R_n = 234.9 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{T_{u,lb}}{\phi R_n} = \frac{(25 \text{ kip})}{(234.9 \text{ kip})} = 0.10643$	PASS
AISC 360-16 Chapter J4.3 Eq. (J4-5)	Check No. 5: Design Block Shear Capacity of the Gusset Plate Calculate the net area of the gusset plate subject to tension. $t_{gp,lb} = 1 \text{ in}$ - Lower Brace Gusset Plate Thickness $n_c = 1$ - Number of Bolt Columns $s_c = 0 \text{ in}$ - Bolt Column Spacing $L_{eh,gp} = 2 \text{ in}$ - Gusset Plate Horizontal Edge Distance $d_h = 0.9375 \text{ in}$ - Horizontal Bolt Hole Dimension at Gusset Plate A_{nt} - Net Area Subject to Tension (L-pattern) $A_{nt} = t_{gp,lb} [(n_c - 1) s_c + L_{eh,gp} - (n_c - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nt} = (1 \text{ in}) \times [(1 - 1) \times (0 \text{ in}) + (2 \text{ in}) - ((1 - 0.5) \times ((0.9375 \text{ in}) + (0.0625 \text{ in})))]$ $A_{nt} = 1.5 \text{ in}^2$ Calculate the gross area of the gusset plate subject to shear. $t_{gp,lb} = 1 \text{ in}$ - Lower Brace Gusset Plate Thickness $L_{ev} = 2 \text{ in}$ - Vertical Edge Distance at Brace $n_r = 3$ - Number of Bolt Rows $s_r = 3 \text{ in}$ - Bolt Row Spacing A_{gv} - Gross Area Subject to Shear (L-pattern) $A_{gv} = t_{gp,lb} [L_{ev} + (n_r - 1) s_r]$ $A_{gv} = (1 \text{ in}) \times [(2 \text{ in}) + ((3 - 1) \times (3 \text{ in}))]$ $A_{gv} = 8 \text{ in}^2$ Calculate the net area of the gusset plate subject to shear. $t_{gp,lb} = 1 \text{ in}$ - Lower Brace Gusset Plate Thickness $L_{br,gp} = 10 \text{ in}$ - Brace Length at Gusset Plate $L_{ev} = 2 \text{ in}$ - Vertical Edge Distance at Brace $n_r = 3$ - Number of Bolt Rows $d_h = 0.9375 \text{ in}$ - Vertical Bolt Hole Dimension at Gusset Plate A_{nv} - Net Area Subject to Shear (L-pattern) $A_{nv} = t_{gp,lb} [L_{br,gp} - L_{ev} - (n_r - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nv} = (1 \text{ in}) \times [(10 \text{ in}) - (2 \text{ in}) - ((3 - 0.5) \times ((0.9375 \text{ in}) + (0.0625 \text{ in})))]$ $A_{nv} = 5.5 \text{ in}^2$ Calculate the design block shear capacity of the gusset plate. $F_{u,lb} = 65 \text{ ksi}$ - Lower Brace Gusset Plate Tensile Stress $A_{nv} = 5.5 \text{ in}^2$ - Net Area Subject to Shear (L-pattern) $U_{bs} = 1$ - Uniformity factor $A_{nt} = 1.5 \text{ in}^2$ - Net Area Subject to Tension (L-pattern) $F_{y,lb} = 50 \text{ ksi}$ - Lower Brace Gusset Plate Yield Stress $A_{gv} = 8 \text{ in}^2$ - Gross Area Subject to Shear (L-pattern) $\phi = 0.75$ - Block Shear Resistance Factor ϕR_n - Design Block Shear Capacity of Section $\phi R_n = \phi (0.6 F_{u,lb} A_{nv} + U_{bs} F_{u,lb} A_{nt} \leq 0.6 F_{y,lb} A_{gv} + U_{bs} F_{u,lb} A_{nt})$ $\phi R_n = (0.75) \times ((0.6 \times (65 \text{ ksi}) \times (5.5 \text{ in}^2)) + (1 \times (65 \text{ ksi}) \times (1.5 \text{ in}^2)) \leq 0.6 \times (50 \text{ ksi}) \times (8 \text{ in}^2) + (1 \times (65 \text{ ksi}) \times (1.5 \text{ in}^2)))$ $\phi R_n = 234 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{T_{u,lb}}{\phi R_n} = \frac{(25 \text{ kip})}{(234 \text{ kip})} = 0.10684$	PASS
	Check No. 6: Design Block Shear Capacity of the Brace Calculate the net area of the brace subject to tension. $t_{lb} = 0.5 \text{ in}$ - Lower Brace Thickness $n_c = 1$ - Number of Bolt Columns $s_c = 0 \text{ in}$ - Bolt Column Spacing $L_{eh,br} = 2 \text{ in}$ - Brace Horizontal Edge Distance $d_h = 0.8125 \text{ in}$ - Horizontal Bolt Hole Dimension at Brace A_{nt} - Net Area Subject to Tension (L-pattern) $A_{nt} = 2t_{lb} [(n_c - 1) s_c + L_{eh,br} - (n_c - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nt} = (1 \text{ in}) \times [(1 - 1) \times (0 \text{ in}) + (2 \text{ in}) - ((1 - 0.5) \times ((0.8125 \text{ in}) + (0.0625 \text{ in})))]$	

AISC 360-16 Chapter J4.3 Eq. (J4-5)	$A_{nt} = 1.5625 \text{ in}^2$ Calculate the gross area of the brace subject to shear. $t_{l,br} = 0.5 \text{ in}$ - Lower Brace Thickness $L_{ev} = 2 \text{ in}$ - Vertical Edge Distance at Brace $n_r = 3$ - Number of Bolt Rows $s_r = 3 \text{ in}$ - Bolt Row Spacing A_{gv} - Gross Area Subject to Shear (L-pattern) $A_{gv} = 2t_{l,br} [L_{ev} + (n_r - 1) s_r]$ $A_{gv} = (1 \text{ in}) \times [(2 \text{ in}) + ((3) - 1) \times (3 \text{ in})]$ $A_{gv} = 8 \text{ in}^2$ Calculate the net area of the brace subject to shear. $t_{l,br} = 0.5 \text{ in}$ - Lower Brace Thickness $L_{br,gp} = 10 \text{ in}$ - Brace Length at Gusset Plate $L_{ev} = 2 \text{ in}$ - Vertical Edge Distance at Brace $n_r = 3$ - Number of Bolt Rows $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Brace A_{nv} - Net Area Subject to Shear (L-pattern) $A_{nv} = 2t_{l,br} [L_{br,gp} - L_{ev} - (n_r - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nv} = (1 \text{ in}) \times [(10 \text{ in}) - (2 \text{ in}) - ((3) - 0.5) \times ((0.8125 \text{ in}) + (0.0625 \text{ in}))]$ $A_{nv} = 5.8125 \text{ in}^2$ Calculate the design block shear capacity of the brace. $F_{u,l,br} = 58 \text{ ksi}$ - Lower Brace Tensile Stress $A_{nv} = 5.8125 \text{ in}^2$ - Net Area Subject to Shear (L-pattern) $U_{bs} = 1$ - Uniformity factor $A_{nt} = 1.5625 \text{ in}^2$ - Net Area Subject to Tension (L-pattern) $F_{y,l,br} = 36 \text{ ksi}$ - Lower Brace Yield Stress $A_{gv} = 8 \text{ in}^2$ - Gross Area Subject to Shear (L-pattern) $\phi = 0.75$ - Block Shear Resistance Factor ϕR_n - Design Block Shear Capacity of Section $\phi R_n = \phi (0.6 F_{u,l,br} A_{nv} + U_{bs} F_{u,l,br} A_{nt} \leq 0.6 F_{y,l,br} A_{gv} + U_{bs} F_{u,l,br} A_{nt})$ $\phi R_n = (0.75) \times (0.6 \times (58 \text{ ksi}) \times (5.8125 \text{ in}^2) + (1) \times (58 \text{ ksi}) \times (1.5625 \text{ in}^2) \leq 0.6 \times (36 \text{ ksi}) \times (8 \text{ in}^2) + (1) \times (58 \text{ ksi}) \times (1.5625 \text{ in}^2))$ $\phi R_n = 197.57 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{T_{u,l,br}}{\phi R_n} = \frac{(25 \text{ kip})}{(197.57 \text{ kip})} = 0.12654$	PASS
AISC 360-16 Chapter J4.1 Eq. (J4-1) AISC 360-16 Chapter J4.1 Eq. (J4-2) AISC 360-16 Chapter J4.1 Eq. (J4-1) AISC 360-16 Chapter J4.1 Eq. (J4-2) AISC 360-16 Chapter J4.2	Check No. 7: Design Capacity of the Gusset Plate in Tension Calculate the tensile yielding capacity of the gusset plate. $\phi = 0.9$ - Tensile Yielding Resistance Factor $F_{y,l,br} = 50 \text{ ksi}$ - Lower Brace Gusset Plate Yield Stress $t_{gp,l,br} = 1 \text{ in}$ - Lower Brace Gusset Plate Thickness $W_{br} = 6.9282 \text{ in}$ - Whitmore Section $\phi R_{n,ty}$ - Design Tension Yielding Capacity of Section $\phi R_{n,ty} = \phi F_{y,l,br} t_{gp,l,br} W_{br}$ $\phi R_{n,ty} = (0.9) \times (50 \text{ ksi}) \times (1 \text{ in}) \times (6.9282 \text{ in})$ $\phi R_{n,ty} = 311.77 \text{ kip}$ Calculate the tensile rupture capacity of the gusset plate. $\phi = 0.75$ - Tensile Rupture Resistance Factor $F_{u,l,br} = 65 \text{ ksi}$ - Lower Brace Gusset Plate Tensile Stress $t_{gp,l,br} = 1 \text{ in}$ - Lower Brace Gusset Plate Thickness $W_{br} = 6.9282 \text{ in}$ - Whitmore Section $n_c = 1$ - Number of Bolt Columns $d_h = 0.9375 \text{ in}$ - Horizontal Bolt Hole Dimension at Gusset Plate $\phi R_{n,tr}$ - Design Tension Rupture Capacity of Section $\phi R_{n,tr} = \phi F_{u,l,br} t_{gp,l,br} [W_{br} - n_c (d_h + 0.0625 \text{ in})]$ $\phi R_{n,tr} = (0.75) \times (65 \text{ ksi}) \times (1 \text{ in}) \times [(6.9282 \text{ in}) - (1) \times ((0.9375 \text{ in}) + (0.0625 \text{ in}))]$ $\phi R_{n,tr} = 289 \text{ kip}$ Determine the governing tensile capacity of the gusset plate. $\phi R_{n,ty} = 311.77 \text{ kip}$ - Design Tension Yielding Capacity of Section $\phi R_{n,tr} = 289 \text{ kip}$ - Design Tension Rupture Capacity of Section ϕR_n - Governing Design Capacity	

	$\phi R_n = \min(\phi R_{n_{ty}}, \phi R_{n_{tr}})$ $\phi R_n = \min((311.77 \text{ kip}), (289 \text{ kip}))$ $\phi R_n = 289 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{T_{u,lb}}{\phi R_n} = \frac{(25 \text{ kip})}{(289 \text{ kip})} = 0.086505$	PASS
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 Brace in Tension Rupture Calculate the tensile rupture capacity of the brace. $\phi = 0.75$ - Tensile Rupture Resistance Factor $F_{u,lb} = 58 \text{ ksi}$ - Lower Brace Tensile Stress $A_{l,br} = 9.58 \text{ in}^2$ - Lower Brace Area $t_{l,br} = 0.5 \text{ in}$ - Lower Brace Thickness $d_h = 0.8125 \text{ in}$ - Horizontal Bolt Hole Dimension at Brace $n_c = 1$ - Number of Bolt Columns $n_r = 3$ - Number of Bolt Rows $s_r = 3 \text{ in}$ - Bolt Row Spacing $\bar{y} = 1.42 \text{ in}$ - Brace Centroid U - Shear Lag Factor $U = 1 - \frac{\bar{y}}{(n_r - 1) s_r}$ $U = 1 - \frac{(1.42 \text{ in})}{((3) - 1) \times (3 \text{ in})}$ $U = 0.76343$ $\phi R_{n_{tr}}$ - Design Tension Rupture Capacity of Section $\phi R_{n_{tr}} = \phi F_{u,lb} U [A_{l,br} - n_c (d_h + 0.0625 \text{ in}) 2t_{l,br}]$ $\phi R_{n_{tr}} = (0.75) \times (58 \text{ ksi}) \times (0.76343) \times [(9.58 \text{ in}^2) - (1) \times ((0.8125 \text{ in}) + (0.0625 \text{ in})) \times (1 \text{ in})]$ $\phi R_{n_{tr}} = 289.09 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{T_{u,lb}}{\phi R_{n_{tr}}} = \frac{(25 \text{ kip})}{(289.09 \text{ kip})} = 0.08648$	PASS
AISC 360-16 Chapter E3 Eq. (E3-4) AISC 360-16 Chapter E3 Eq. (E3-2) AISC 360-16 Chapter E3 Eq. (E3-1)	Check No. 9: Design Capacity of the Gusset Plate in Compression Calculate the compression buckling capacity of the gusset plate. $\phi = 0.9$ - Compression Resistance Factor $F_{y,lb} = 50 \text{ ksi}$ - Lower Brace Gusset Plate Yield Stress $t_{gp,lb} = 1 \text{ in}$ - Lower Brace Gusset Plate Thickness $W_{br} = 6.9282 \text{ in}$ - Whitmore Section $K = 1.2$ - Effective Length Factor $L_{b,lb} = 11 \text{ in}$ - Lower Brace Unbraced Length $\frac{KL}{r} = 45.726$ - Effective Length Slenderness Ratio Since, $\frac{KL}{r} > 25$. F_e - Elastic Buckling Stress $F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$ $F_e = \frac{\pi^2 \times (29000 \text{ ksi})}{((45.726))^2}$ $F_e = 136.89 \text{ ksi}$ $4.71 \sqrt{\frac{E}{F_y}} = 113.43$ - Effective Length Slenderness Ratio Limiter Since, $\frac{KL}{r} \leq 4.71 \sqrt{\frac{E}{F_y}}$. F_{cr} - Critical Buckling Stress $F_{cr} = \left(0.658 \frac{F_{y,lb}}{F_e}\right) F_{y,lb}$ $F_{cr} = \left(0.658 \frac{(50 \text{ ksi})}{(136.89 \text{ ksi})}\right) \times (50 \text{ ksi})$ $F_{cr} = 42.912 \text{ ksi}$ ϕR_n - Design Compressive Capacity of Section $\phi R_n = \phi F_{cr} t_{gp,lb} W_{br}$ $\phi R_n = (0.9) \times (42.912 \text{ ksi}) \times (1 \text{ in}) \times (6.9282 \text{ in})$ $\phi R_n = 267.57 \text{ kip}$	

Result:
 Demand over Capacity Ratio
 $DCR = \frac{C_u Lbr}{\phi R_n} = \frac{(25 \text{ kip})}{(267.57 \text{ kip})} = 0.093433$

PASS

Summary of Checks

Design Checks	Demand	Capacity	DCR	Result
Connection Detailing Limitations	2.000	3.000	0.667	PASS
Design Capacity of the Bolts in Shear	25.000	48.409	0.516	PASS
Design Bolt Bearing Capacity of the Gusset Plate	25.000	263.250	0.095	PASS
Design Bolt Bearing Capacity of the Brace	25.000	234.900	0.106	PASS
Design Block Shear Capacity of the Gusset Plate	25.000	234.000	0.107	PASS
Design Block Shear Capacity of the Brace	25.000	197.569	0.127	PASS
Design Capacity of the Gusset Plate in Tension	25.000	289.000	0.087	PASS
Design Capacity of the Brace in Tension Rupture	25.000	289.085	0.086	PASS
Design Capacity of the Gusset Plate in Compression	25.000	267.571	0.093	PASS

REFERENCES	CALCULATIONS	RESULTS										
	Lower Gusset Plate Connection to Beam AISC 360-16 LRFD Check No. 1: Connection Detailing Limitations <table><thead><tr><th>Detailing Limitations</th><th>Limit Value (in)</th><th>Actual Value (in)</th><th>DCR</th><th>Result</th></tr></thead><tbody><tr><td>Minimum Fillet Weld Size</td><td>0.250</td><td>0.250</td><td>1.000</td><td>PASS</td></tr></tbody></table> Result: Demand over Capacity Ratio $DCR = \frac{d}{c} = \frac{(0.25)}{(0.25)} = 1$	Detailing Limitations	Limit Value (in)	Actual Value (in)	DCR	Result	Minimum Fillet Weld Size	0.250	0.250	1.000	PASS	PASS
Detailing Limitations	Limit Value (in)	Actual Value (in)	DCR	Result								
Minimum Fillet Weld Size	0.250	0.250	1.000	PASS								
AISC 15th Ed. Part 9 Eq. (9-2)	Check No. 2: Design Capacity of Weld at Gusset to Beam Connection Information: $\phi = 0.75$ - Fillet Weld Resistance Factor $W = 0.25$ in - Fillet Weld Size $F_{EXX} = 70$ ksi - Filler Metal Classification Strength $t_{f,bm} = 0.57$ in - Beam Flange Thickness $F_{u,bm} = 65$ ksi - Beam Tensile Stress $t_{gp,l,br} = 1$ in - Lower Brace Gusset Plate Thickness $F_{u,l,br} = 65$ ksi - Lower Brace Gusset Plate Tensile Stress $L_w = 15$ in - Weld Length Calculate Dmax: Maximum Fillet Weld Size for Base Metal Strength $D_{max} = \frac{t_{f,bm} F_{u,bm} \leq 0.5 t_{gp,l,br} F_{u,l,br}}{3.09 \text{ kip/in}} = \frac{(0.57 \text{ in}) \times (65 \text{ ksi}) \leq (0.5 \text{ in}) \times (65 \text{ ksi})}{(3.09 \text{ kip/in})} = 10.518$ Calculate Total Weld Length: Total Effective Length of Weld $L_w = 2 [L_w - 2 (W \leq 0.3125 \text{ in})] = 2 \times [(15 \text{ in}) - 2 \times ((0.25 \text{ in}) \leq (0.3125 \text{ in}))] = 29 \text{ in}$ Design Loads: $M_b = 6.83$ kip·ft - Moment Load at Gusset to Beam $V_{b,lower} = 0$ kip - Vertical Axial Load at Gusset to Beam $V_{b,total} = \frac{8 M_b}{L_w} + V_{b,lower} = \frac{8 \times (6.83 \text{ kip}\cdot\text{ft})}{(29 \text{ in})} + (0 \text{ kip}) = 22.61 \text{ kip}$ $H_b = 10.178$ kip - Horizontal Shear at Gusset to Beam Resultant Load $R = \sqrt{(V_{b,total})^2 + (H_b)^2} = \sqrt{((22.61 \text{ kip}))^2 + ((10.178 \text{ kip}))^2} = 24.795 \text{ kip}$ Loading Angle at Weld in Degrees $\theta = \tan^{-1} \left(\frac{V_{b,total}}{H_b} \right) = \tan^{-1} \left(\frac{(22.61 \text{ kip})}{(10.178 \text{ kip})} \right) = 65.764$ Calculate Weld Capacity: ϕR_n - Design Strength of Welds $\phi R_n = \phi 0.6 F_{EXX} \frac{\sqrt{2}}{2} \left(W \leq \frac{D_{max}}{16} \right) \left[1 + 0.5 (\sin \theta)^{1.5} \right] L_w$ $\phi R_n = (0.75) \times 0.6 \times (70 \text{ ksi}) \times \frac{\sqrt{2}}{2} \times \left((0.25 \text{ in}) \leq \frac{(10.518)}{(16)} \right) \times \left[1 + 0.5 \times (\sin(65.764))^{1.5} \right] \times (29 \text{ in})$$\phi R_n = 231.79 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{R}{\phi R_n} = \frac{(24.795 \text{ kip})}{(231.79 \text{ kip})} = 0.10697$	PASS										
AISC 360-16 Chapter J10.3 Eq. (J10-4)	Check No. 3: Design Capacity of the Beam Support Calculate the web local crippling capacity of the beam. $\phi = 0.75$ - Web Local Crippling Resistance Factor $t_{w,bm} = 0.355$ in - Beam Web Thickness $l_b = 7.5$ in - Bearing Length: Half Gusset Plate Width along Beam $d_{bm} = 18$ in - Beam Depth $t_{f,bm} = 0.57$ in - Beam Flange Thickness $F_{y,bm} = 50$ ksi - Beam Yield Stress $E = 29000$ ksi - Modulus for Steel ϕR_{n_wlc} - Design Web Local Crippling Capacity of Support $\phi R_{n_wlc} = \phi 0.4 (t_{w,bm})^2 \left[1 + \left(\frac{4 l_b}{d_{bm}} - 0.2 \right) \left(\frac{t_{w,bm}}{t_{f,bm}} \right)^{1.5} \right] \sqrt{\frac{E F_{y,bm} t_{f,bm}}{t_{w,bm}}} Q_f$ $\phi R_{n_wlc} = (0.75) \times 0.4 \times ((0.355 \text{ in}))^2 \times \left[1 + \left(\frac{4 \times (7.5 \text{ in})}{(18 \text{ in})} - 0.2 \right) \times \left(\frac{(0.355 \text{ in})}{(0.57 \text{ in})} \right)^{1.5} \right] \times \sqrt{\frac{(29000 \text{ ksi}) \times (50 \text{ ksi}) \times (0.57 \text{ in})}{(0.355 \text{ in})}} \times (1)$$\phi R_{n_wlc} = 99.274 \text{ kip}$ Calculate the web local yielding capacity of the beam. $\phi = 1$ - Web Local Yielding Resistance Factor $F_{y,bm} = 50$ ksi - Beam Yield Stress $t_{w,bm} = 0.355$ in - Beam Web Thickness $k_{des_lc} = 0.972$ in - Beam kdes $l_b = 7.5$ in - Bearing Length: Half Gusset Plate Width along Beam ϕR_{n_wly} - Design Web Local Yielding Capacity of Support $\phi R_{n_wly} = \phi F_{y,bm} t_{w,bm} (2.5 k_{des_lc} + l_b)$											
AISC 360-16 Chapter J10.2 Eq. (J10-3)												

AISC 360-16 Chapter J10	$\phi R_{n_{wly}} = (1) \times (50 \text{ ksi}) \times (0.355 \text{ in}) \times (2.5 \times (0.972 \text{ in}) + (7.5 \text{ in}))$$\phi R_{n_{wly}} = 176.26 \text{ kip}$ Calculate for the governing design capacity of the beam. ϕR_n - Governing Design Capacity of the Support $\phi R_n = \min(\phi R_{n_{wlc}}, \phi R_{n_{wly}})$$\phi R_n = \min((99.274 \text{ kip}), (176.26 \text{ kip}))$$\phi R_n = 99.274 \text{ kip}$ $V_{b,at,beam} = 11.305 \text{ kip}$ - Required Load: Half total Vb Result: Demand over Capacity Ratio $DCR = \frac{V_{b,at,beam}}{\phi R_n} = \frac{(11.305 \text{ kip})}{(99.274 \text{ kip})} = 0.11388$	PASS																				
	Summary of Checks <table><thead><tr><th>Design Checks</th><th>Demand</th><th>Capacity</th><th>DCR</th><th>Result</th></tr></thead><tbody><tr><td>Connection Detailing Limitations</td><td>0.250</td><td>0.250</td><td>1.000</td><td>PASS</td></tr><tr><td>Design Capacity of Weld at Gusset to Beam</td><td>24.795</td><td>231.793</td><td>0.107</td><td>PASS</td></tr><tr><td>Design Capacity of the Beam Support</td><td>11.305</td><td>99.274</td><td>0.114</td><td>PASS</td></tr></tbody></table>	Design Checks	Demand	Capacity	DCR	Result	Connection Detailing Limitations	0.250	0.250	1.000	PASS	Design Capacity of Weld at Gusset to Beam	24.795	231.793	0.107	PASS	Design Capacity of the Beam Support	11.305	99.274	0.114	PASS	
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REFERENCES	CALCULATIONS	RESULTS
	Lower Gusset Plate Connection to Column AISC 360-16 LRFD	

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	Check No. 1: Connection Detailing Limitations at Gusset Side <table><thead><tr><th>Detailing Limitations</th><th>Limit Value (in)</th><th>Actual Value (in)</th><th>DCR</th><th>Result</th></tr></thead><tbody><tr><td>Minimum Bolt Row Spacing</td><td>2.000</td><td>3.000</td><td>0.667</td><td>PASS</td></tr><tr><td>Maximum Bolt Row Spacing</td><td>6.000</td><td>3.000</td><td>0.500</td><td>PASS</td></tr><tr><td>Double Angle Minimum Vertical Edge Distance</td><td>1.000</td><td>1.500</td><td>0.667</td><td>PASS</td></tr><tr><td>Double Angle Minimum Horizontal Edge Distance</td><td>1.125</td><td>2.000</td><td>0.563</td><td>PASS</td></tr><tr><td>Gusset Plate Minimum Horizontal Edge Distance</td><td>1.000</td><td>2.000</td><td>0.500</td><td>PASS</td></tr><tr><td>Minimum Connection Depth</td><td>7.500</td><td>12.000</td><td>0.625</td><td>PASS</td></tr><tr><td>Maximum Connection Depth</td><td>13.500</td><td>12.000</td><td>0.889</td><td>PASS</td></tr><tr><td>Minimum Bolt Gage at Double Angle</td><td>1.750</td><td>3.000</td><td>0.583</td><td>PASS</td></tr></tbody></table> Result: Demand over Capacity Ratio $DCR = \frac{d}{c} = \frac{(12)}{(13.5)} = 0.88889$ PASS	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	Double Angle Minimum Vertical Edge Distance	1.000	1.500	0.667	PASS	Double Angle Minimum Horizontal Edge Distance	1.125	2.000	0.563	PASS	Gusset Plate Minimum Horizontal Edge Distance	1.000	2.000	0.500	PASS	Minimum Connection Depth	7.500	12.000	0.625	PASS	Maximum Connection Depth	13.500	12.000	0.889	PASS	Minimum Bolt Gage at Double Angle	1.750	3.000	0.583	PASS
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AISC 15th Edition Table 10-9 AISC 360-16 Chapter J3.8 Eq. (J3-4) AISC 15th Ed. Page 7-6 to 7-8 AISC 15th Ed. Page 7-6 to 7-8	Check No. 2: Design Capacity of the Bolt Group in Shear at Gusset Side Design Loads: Loading Angle from Vertical in Degrees $\theta = \tan^{-1}\left(\frac{H_c}{V_c}\right) = \tan^{-1}\left(\frac{(7.4996 \text{ kip})}{(17.678 \text{ kip})}\right) = 22.989$ Resultant Load $R = \sqrt{(H_c)^2 + (V_c)^2} = \sqrt{((7.4996 \text{ kip}))^2 + ((17.678 \text{ kip}))^2} = 19.203 \text{ kip}$ Calculate the C-coefficient of the bolt group. $e = 3 \text{ in}$ - Design Bolt Group Eccentricity $\phi R_n = 16.136 \text{ kip}$ - Design Bolt Shear Capacity 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. <table><thead><tr><th>Bolt No.</th><th>x-coordinate (in)</th><th>y-coordinate (in)</th></tr></thead><tbody><tr><td>1</td><td>0.00</td><td>0.00</td></tr><tr><td>2</td><td>0.00</td><td>3.00</td></tr><tr><td>3</td><td>0.00</td><td>6.00</td></tr><tr><td>4</td><td>0.00</td><td>9.00</td></tr></tbody></table>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. $x_{ic} = -2.6244 \text{ in}$ - Location of I.C. along the X-axis $y_{ic} = 6.0145 \text{ in}$ - Location of I.C. along the Y-axis Using the AISC 15th Edition Equation (7-1), the corresponding reactions per bolt are listed below. <table><thead><tr><th>Bolt No.</th><th>Ru (kip)</th><th>Rux (kip)</th><th>Ruy (kip)</th><th>Mn (kip-in)</th></tr></thead><tbody><tr><td>1</td><td>15.84</td><td>14.53</td><td>6.29</td><td>103.55</td></tr><tr><td>2</td><td>14.98</td><td>11.32</td><td>9.81</td><td>59.43</td></tr><tr><td>3</td><td>13.68</td><td>-0.00</td><td>13.68</td><td>35.56</td></tr><tr><td>4</td><td>14.98</td><td>-11.32</td><td>9.80</td><td>59.44</td></tr></tbody></table> $C = 2.6133$ - Calculated Bolt Group Coefficient $C' = 11.256 \text{ in}$ - Calculated Bolt Group Coefficient for Moment-only Case Calculate the design shear capacity of the bolt group. $\phi = 0.85$ - Bolt Shear Resistance Factor $d_b = 0.75 \text{ in}$ - Bolt Diameter $C = 2.6133$ - Calculated Bolt Group Coefficient $\mu = 0.3$ - Mean Slip Coefficient $N_s = 2$ - Number of Slip Planes $h_f = 1$ - Filler Factor for SC Bolts $T_b = 28 \text{ kip}$ - Minimum Bolt Pretension ϕR_n - Design Bolt Shear Capacity $\phi R_n = \phi \mu D_u h_f T_b N_s C$$\phi R_n = (0.85) \times (0.3) \times (1.13) \times (1) \times (28 \text{ kip}) \times (2) \times (2.6133)$$\phi R_n = 42.17 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{R}{\phi R_n} = \frac{(19.203 \text{ kip})}{(42.17 \text{ kip})} = 0.45537$ PASS	Bolt No.	x-coordinate (in)	y-coordinate (in)	1	0.00	0.00	2	0.00	3.00	3	0.00	6.00	4	0.00	9.00	Bolt No.	Ru (kip)	Rux (kip)	Ruy (kip)	Mn (kip-in)	1	15.84	14.53	6.29	103.55	2	14.98	11.32	9.81	59.43	3	13.68	-0.00	13.68	35.56	4	14.98	-11.32	9.80	59.44					
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AISC 360-16 Chapter J3.8 Eq. (J3-4) AISC 360-16 Chapter J3.10 Eq. (J3-6a)	Check No. 3: Design Capacity of the Bolt Group in Bearing and Tear-out on Double Angle at Gusset Side Calculate the bolt bearing capacity of the double angle. $d_b = 0.75 \text{ in}$ - Bolt Diameter $t_{2L,at,col,l,br} = 0.5 \text{ in}$ - Lower Brace Double Angle Thickness $F_{u,at,col,l,br} = 58 \text{ ksi}$ - Lower Brace Double Angle Tensile Stress $C = 2.6133$ - Calculated Bolt Group Coefficient $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_bearing}$ - Design Bolt Bearing Capacity of Section $\phi R_{n_bearing} = \phi 2.4 d_b [2(t_{2L,at,col,l,br})] F_{u,at,col,l,br} C$$\phi R_{n_bearing} = (0.75) \times 2.4 \times (0.75 \text{ in}) \times (1 \text{ in}) \times (58 \text{ ksi}) \times (2.6133)$																																													

AISC 360-16 Chapter J3.10 Eq. (J3-6c)	$\phi R_{n_bearing} = 204.62 \text{ kip}$ Calculate the clear distance of outer bolts on double angle. $L_{ev} = 1.5 \text{ in}$ - Vertical Edge Distance at Double Angle $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Double Angle l_{c1} - Clear Distance at First Bolt Row $l_{c1} = L_{ev} - \frac{d_h}{2}$ $l_{c1} = (1.5 \text{ in}) - \frac{(0.8125 \text{ in})}{2}$ $l_{c1} = 1.0938 \text{ in}$ Calculate the clear distance of inner bolts on double angle. $s_r = 3 \text{ in}$ - Bolt Row Spacing $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Double Angle 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 double angle. $l_{c1} = 1.0938 \text{ in}$ - Clear Distance at First Bolt Row $l_{c2} = 2.1875 \text{ in}$ - Clear Distance at Rest of Bolts $t_{2L_at_col_l_br} = 0.5 \text{ in}$ - Lower Brace Double Angle Thickness $F_{u_at_col_l_br} = 58 \text{ ksi}$ - Lower Brace Double Angle Tensile Stress $C = 2.6133$ - Calculated Bolt Group Coefficient $n_r = 4$ - Number of Bolt Rows $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_tearout}$ - Design Bolt Tear-out Capacity of Section $\phi R_{n_tearout} = \phi [1.2 l_{c1} [2(t_{2L_at_col_l_br})] F_{u_at_col_l_br} + 1.2 l_{c2} [2(t_{2L_at_col_l_br})] F_{u_at_col_l_br} (n_r - 1)] \left(\frac{C}{n_r} \right)$ $\phi R_{n_tearout} = (0.75) \times [1.2 \times (1.0938 \text{ in}) \times (1 \text{ in}) \times (58 \text{ ksi}) + 1.2 \times (2.1875 \text{ in}) \times (1 \text{ in}) \times (58 \text{ ksi}) \times ((4) - 1)] \times \left(\frac{(2.6133)}{(4)} \right)$ $\phi R_{n_tearout} = 261.11 \text{ kip}$ Determine the governing bearing and tear-out capacity of the bolt group on double angle. $\phi R_{n_bearing} = 204.62 \text{ kip}$ - Design bolt bearing capacity of double angle $\phi R_{n_tearout} = 261.11 \text{ kip}$ - Design bolt tear-out capacity of double angle ϕR_n - Governing Design Capacity $\phi R_n = \min(\phi R_{n_bearing}, \phi R_{n_tearout})$ $\phi R_n = \min((204.62 \text{ kip}), (261.11 \text{ kip}))$ $\phi R_n = 204.62 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{V_c}{\phi R_n} = \frac{(17.678 \text{ kip})}{(204.62 \text{ kip})} = 0.086392$	PASS
AISC 360-16 Chapter J3.10 Eq. (J3-6a)	Check No. 4: Design Capacity of the Bolt Group in Bearing and Tear-out on Double Angle at Gusset Side due to Axial Load Calculate the bolt bearing capacity of the double angle. $d_b = 0.75 \text{ in}$ - Bolt Diameter $t_{2L_at_col_l_br} = 0.5 \text{ in}$ - Lower Brace Double Angle Thickness $F_{u_at_col_l_br} = 58 \text{ ksi}$ - Lower Brace Double Angle Tensile Stress $C = 2.6133$ - Calculated Bolt Group Coefficient $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_bearing}$ - Design Bolt Bearing Capacity of Section $\phi R_{n_bearing} = \phi 2.4 d_b [2(t_{2L_at_col_l_br})] F_{u_at_col_l_br} C$ $\phi R_{n_bearing} = (0.75) \times 2.4 \times (0.75 \text{ in}) \times (1 \text{ in}) \times (58 \text{ ksi}) \times (2.6133)$ $\phi R_{n_bearing} = 204.62 \text{ kip}$ Calculate the clear distance of outer bolts on double angle. $L_{eh} = 2 \text{ in}$ - Horizontal Edge Distance at Double Angle $d_h = 1 \text{ in}$ - Horizontal Bolt Hole Dimension at Double Angle l_{c1} - Clear Distance at First Bolt Row $l_{c1} = L_{eh} - \frac{d_h}{2}$ $l_{c1} = (2 \text{ in}) - \frac{(1 \text{ in})}{2}$	

AISC 360-16 Chapter J3.10 Eq. (J3-6c)	$l_{c1} = 1.5 \text{ in}$ Calculate the clear distance of inner bolts on double angle. $s_c = 0 \text{ in}$ - Bolt Column Spacing $d_h = 1 \text{ in}$ - Horizontal Bolt Hole Dimension at Double Angle l_{c2} - Clear Distance at Rest of Bolts $l_{c2} = s_c - d_h$ $l_{c2} = (0 \text{ in}) - (1 \text{ in})$ $l_{c2} = -1 \text{ in}$ Calculate the bolt tear-out capacity of the double angle. $l_{c1} = 1.5 \text{ in}$ - Clear Distance at First Bolt Row $l_{c2} = -1 \text{ in}$ - Clear Distance at Rest of Bolts $t_{2L,at,col,l,br} = 0.5 \text{ in}$ - Lower Brace Double Angle Thickness $F_{u,at,col,l,br} = 58 \text{ ksi}$ - Lower Brace Double Angle Tensile Stress $C = 2.6133$ - Calculated Bolt Group Coefficient $n_c = 1$ - Number of Bolt Columns $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_tearout}$ - Design Bolt Tear-out Capacity of Section $\phi R_{n_tearout} = \phi [1.2 l_{c1} [2(t_{2L,at,col,l,br})] F_{u,at,col,l,br} + 1.2 l_{c2} [2(t_{2L,at,col,l,br})] F_{u,at,col,l,br} (n_c - 1)] \left(\frac{C}{n_c} \right)$ $\phi R_{n_tearout} = (0.75) \times [1.2 \times (1.5 \text{ in}) \times (1 \text{ in}) \times (58 \text{ ksi}) + 1.2 \times (-1 \text{ in}) \times (1 \text{ in}) \times (58 \text{ ksi}) \times ((1) - 1)] \times \left(\frac{(2.6133)}{(1)} \right)$ $\phi R_{n_tearout} = 204.62 \text{ kip}$ Determine the governing bearing and tear-out capacity of the bolt group on double angle. $\phi R_{n_bearing} = 204.62 \text{ kip}$ - Design bolt bearing capacity of double angle $\phi R_{n_tearout} = 204.62 \text{ kip}$ - Design bolt tear-out capacity of double angle ϕR_n - Governing Design Capacity $\phi R_n = \min(\phi R_{n_bearing}, \phi R_{n_tearout})$ $\phi R_n = \min((204.62 \text{ kip}), (204.62 \text{ kip}))$ $\phi R_n = 204.62 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{H_c}{\phi R_n} = \frac{(7.4996 \text{ kip})}{(204.62 \text{ kip})} = 0.036651$	PASS
	Check No. 5: Shear and Axial Interaction of Bolt Bearing/Tearout in Double Angle Calculate the shear and axial interaction of bolt bearing/tearout capacities in the double angle. $V_c = 17.678 \text{ kip}$ - Vertical Shear at Gusset to Column $H_c = 7.4996 \text{ kip}$ - Axial Load at Gusset to Column Interaction - Shear and Axial Interaction $Interaction = \sqrt{\left(\frac{V_c}{\phi R_n} \right)^2 + \left(\frac{H_c}{\phi R_n} \right)^2}$ $Interaction = \sqrt{\left(\frac{(17.678 \text{ kip})}{(204.62 \text{ kip})} \right)^2 + \left(\frac{(7.4996 \text{ kip})}{(204.62 \text{ kip})} \right)^2}$ $Interaction = 0.093845$ Result: Demand over Capacity Ratio $DCR = \frac{Interaction}{1.0} = \frac{(0.093845)}{(1)} = 0.093845$	PASS
AISC 360-16 Chapter J3.10 Eq. (J3-6a)	Check No. 6: Design Capacity of the Bolt Group in Bearing and Tear-out on Gusset Plate Calculate the bolt bearing capacity. $d_b = 0.75 \text{ in}$ - Bolt Diameter $t_{gp,l,br} = 1 \text{ in}$ - Lower Brace Gusset Plate Thickness $F_{u,l,br} = 65 \text{ ksi}$ - Lower Brace Gusset Plate Tensile Stress $C = 2.6133$ - Calculated Bolt Group Coefficient $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_bearing}$ - Design Bolt Bearing Capacity of Section $\phi R_{n_bearing} = \phi 2.4 d_b t_{gp,l,br} F_{u,l,br} C$ $\phi R_{n_bearing} = (0.75) \times 2.4 \times (0.75 \text{ in}) \times (1 \text{ in}) \times (65 \text{ ksi}) \times (2.6133)$ $\phi R_{n_bearing} = 229.32 \text{ kip}$ Calculate the clear distance of inner bolts on member. $s_r = 3 \text{ in}$ - Bolt Row Spacing $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Gusset Plate l_{c2} - Clear Distance at Rest of Bolts $l_{c2} = s_r - d_h$	

AISC 360-16 Chapter J3.10 Eq. (J3-6C)	$l_{c2} = (3 \text{ in}) - (0.8125 \text{ in})$ $l_{c2} = 2.1875 \text{ in}$ Calculate the bolt tear-out capacity. $l_{c2} = 2.1875 \text{ in}$ - Clear Distance at Rest of Bolts $t_{gp,l,br} = 1 \text{ in}$ - Lower Brace Gusset Plate Thickness $F_{u,l,br} = 65 \text{ ksi}$ - Lower Brace Gusset Plate Tensile Stress $C = 2.6133$ - Calculated Bolt Group Coefficient $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_tearout}$ - Design Bolt Tear-out Capacity of Section $\phi R_{n_tearout} = \phi 1.2 l_{c2} t_{gp,l,br} F_{u,l,br} C$ $\phi R_{n_tearout} = (0.75) \times 1.2 \times (2.1875 \text{ in}) \times (1 \text{ in}) \times (65 \text{ ksi}) \times (2.6133)$ $\phi R_{n_tearout} = 334.42 \text{ kip}$ Determine the governing bearing and tear-out capacity of the bolt group. $\phi R_{n_bearing} = 229.32 \text{ kip}$ - Design bolt bearing capacity of member $\phi R_{n_tearout} = 334.42 \text{ kip}$ - Design bolt tear-out capacity of member ϕR_n - Governing Design Capacity $\phi R_n = \min(\phi R_{n_bearing}, \phi R_{n_tearout})$ $\phi R_n = \min((229.32 \text{ kip}), (334.42 \text{ kip}))$ $\phi R_n = 229.32 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{V_c}{\phi R_n} = \frac{(17.678 \text{ kip})}{(229.32 \text{ kip})} = 0.077088$	PASS
AISC 360-16 Chapter J3.10 Eq. (J3-6a)	Check No. 7: Design Capacity of the Bolt Group in Bearing and Tear-out on Gusset Plate due to Axial Load Calculate the bolt bearing capacity. $d_b = 0.75 \text{ in}$ - Bolt Diameter $t_{gp,l,br} = 1 \text{ in}$ - Lower Brace Gusset Plate Thickness $F_{u,l,br} = 65 \text{ ksi}$ - Lower Brace Gusset Plate Tensile Stress $C = 2.6133$ - Calculated Bolt Group Coefficient $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_bearing}$ - Design Bolt Bearing Capacity of Section $\phi R_{n_bearing} = \phi 2.4 d_b t_{gp,l,br} F_{u,l,br} C$ $\phi R_{n_bearing} = (0.75) \times 2.4 \times (0.75 \text{ in}) \times (1 \text{ in}) \times (65 \text{ ksi}) \times (2.6133)$ $\phi R_{n_bearing} = 229.32 \text{ kip}$ Calculate the clear distance of outer bolts. $L_{eh} = 2 \text{ in}$ - Horizontal Edge Distance at Gusset Plate $d_h = 0.8125 \text{ in}$ - Horizontal Bolt Hole Dimension at Gusset Plate l_{c1} - Clear Distance at First Bolt Row $l_{c1} = L_{eh} - \frac{d_h}{2}$ $l_{c1} = (2 \text{ in}) - \frac{(0.8125 \text{ in})}{2}$ $l_{c1} = 1.5938 \text{ in}$ Calculate the clear distance of inner bolts. $s_c = 0 \text{ in}$ - Bolt Column Spacing $d_h = 0.8125 \text{ in}$ - Horizontal Bolt Hole Dimension at Gusset Plate l_{c2} - Clear Distance at Rest of Bolts $l_{c2} = s_c - d_h$ $l_{c2} = (0 \text{ in}) - (0.8125 \text{ in})$ $l_{c2} = -0.8125 \text{ in}$ Calculate the bolt tear-out capacity. $l_{c1} = 1.5938 \text{ in}$ - Clear Distance at First Bolt Row $l_{c2} = -0.8125 \text{ in}$ - Clear Distance at Rest of Bolts $t_{gp,l,br} = 1 \text{ in}$ - Lower Brace Gusset Plate Thickness $F_{u,l,br} = 65 \text{ ksi}$ - Lower Brace Gusset Plate Tensile Stress $C = 2.6133$ - Calculated Bolt Group Coefficient $n_c = 1$ - Number of Bolt Columns $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_tearout}$ - Design Bolt Tear-out Capacity of Section $\phi R_{n_tearout} = \phi [1.2 l_{c1} t_{gp,l,br} F_{u,l,br} + 1.2 l_{c2} t_{gp,l,br} F_{u,l,br} (n_c - 1)] \left(\frac{C}{n_c} \right)$	

AISC 360-16 Chapter J3.10 Eq. (J3-6a) AISC 360-16 Chapter J3.10 Eq. (J3-6c) AISC 360-16 Chapter J3.10	$\phi R_{n_tearout} = (0.75) \times [1.2 \times (1.5938 \text{ in}) \times (1 \text{ in}) \times (65 \text{ ksi}) + 1.2 \times (-0.8125 \text{ in}) \times (1 \text{ in}) \times (65 \text{ ksi}) \times ((1) - 1)] \times \left(\frac{(2.6133)}{(1)} \right)$ $\phi R_{n_tearout} = 243.65 \text{ kip}$ Determine the governing bearing and tear-out capacity of the bolt group. $\phi R_{n_bearing} = 229.32 \text{ kip}$ - Design bolt bearing capacity of member $\phi R_{n_tearout} = 243.65 \text{ kip}$ - Design bolt tear-out capacity of member ϕR_n - Governing Design Capacity $\phi R_n = \min(\phi R_{n_bearing}, \phi R_{n_tearout})$ $\phi R_n = \min((229.32 \text{ kip}), (243.65 \text{ kip}))$ $\phi R_n = 229.32 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{H_c}{\phi R_n} = \frac{(7.4996 \text{ kip})}{(229.32 \text{ kip})} = 0.032704$	PASS
	Check No. 8: Shear and Axial Interaction of Bolt Bearing/Tearout on Gusset Plate Calculate the shear and axial interaction of bolt bearing/tearout capacities. $V_c = 17.678 \text{ kip}$ - Vertical Shear at Gusset to Column $H_c = 7.4996 \text{ kip}$ - Axial Load at Gusset to Column <i>Interaction</i> - Shear and Axial Interaction $Interaction = \sqrt{\left(\frac{V_c}{\phi R_n} \right)^2 + \left(\frac{H_c}{\phi R_n} \right)^2}$ $Interaction = \sqrt{\left(\frac{(17.678 \text{ kip})}{(229.32 \text{ kip})} \right)^2 + \left(\frac{(7.4996 \text{ kip})}{(229.32 \text{ kip})} \right)^2}$ $Interaction = 0.083738$ Result: Demand over Capacity Ratio $DCR = \frac{Interaction}{1.0} = \frac{(0.083738)}{(1)} = 0.083738$	PASS
	Check No. 9: Design Capacity of Double Angle in Block Shear at Gusset Side Calculate the net area of the double angle subject to tension. $t_{2L,at,col,l,br} = 0.5 \text{ in}$ - Lower Brace Double Angle Thickness $n_c = 1$ - Number of Bolt Columns $s_c = 0 \text{ in}$ - Bolt Column Spacing $L_{eh} = 2 \text{ in}$ - Horizontal Edge Distance at Double Angle $d_h = 1 \text{ in}$ - Horizontal Bolt Hole Dimension at Double Angle A_{nt} - Net Area Subject to Tension (L-pattern) $A_{nt} = [2(t_{2L,at,col,l,br})] [(n_c - 1) s_c + L_{eh} - (n_c - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nt} = (1 \text{ in}) \times [(1 - 1) \times (0 \text{ in}) + (2 \text{ in}) - ((1) - 0.5) \times ((1 \text{ in}) + (0.0625 \text{ in}))]$ $A_{nt} = 1.4688 \text{ in}^2$ Calculate the gross area of the double angle subject to shear. $t_{2L,at,col,l,br} = 0.5 \text{ in}$ - Lower Brace Double Angle Thickness $L_{ev} = 1.5 \text{ in}$ - Vertical Edge Distance at Double Angle $n_r = 4$ - Number of Bolt Rows $s_r = 3 \text{ in}$ - Bolt Row Spacing A_{gv} - Gross Area Subject to Shear (L-pattern) $A_{gv} = [2(t_{2L,at,col,l,br})] [L_{ev} + (n_r - 1) s_r]$ $A_{gv} = (1 \text{ in}) \times [(1.5 \text{ in}) + ((4) - 1) \times (3 \text{ in})]$ $A_{gv} = 10.5 \text{ in}^2$ Calculate the net area of the double angle subject to shear. $t_{2L,at,col,l,br} = 0.5 \text{ in}$ - Lower Brace Double Angle Thickness $L_{2L,at,col,l,br} = 12 \text{ in}$ - Lower Brace Double Angle Length $L_{ev} = 1.5 \text{ in}$ - Vertical Edge Distance at Double Angle $n_r = 4$ - Number of Bolt Rows $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Double Angle A_{nv} - Net Area Subject to Shear (L-pattern) $A_{nv} = [2(t_{2L,at,col,l,br})] [L_{2L,at,col,l,br} - L_{ev} - (n_r - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nv} = (1 \text{ in}) \times [(12 \text{ in}) - (1.5 \text{ in}) - ((4) - 0.5) \times ((0.8125 \text{ in}) + (0.0625 \text{ in}))]$ $A_{nv} = 7.4375 \text{ in}^2$ Calculate the design block shear capacity of the double angle. $F_{u,at,col,l,br} = 58 \text{ ksi}$ - Lower Brace Double Angle Tensile Stress $A_{nv} = 7.4375 \text{ in}^2$ - Net Area Subject to Shear (L-pattern) $U_{bs} = 1$ - Uniformity factor $A_{nt} = 1.4688 \text{ in}^2$ - Net Area Subject to Tension (L-pattern)	

AISC 360-16 Chapter J4.3 Eq. (J4-5)	$F_{y,at,col,l,br} = 36 \text{ ksi}$ - Lower Brace Double Angle Yield Stress $A_{gv} = 10.5 \text{ in}^2$ - Gross Area Subject to Shear (L-pattern) $\phi = 0.75$ - Block Shear Resistance Factor ϕR_n - Design Block Shear Capacity of Section $\phi R_n = \phi \left(0.6 F_{u,at,col,l,br} A_{nv} + U_{bs} F_{u,at,col,l,br} A_{nt} \leq 0.6 F_{y,at,col,l,br} A_{gv} + U_{bs} F_{u,at,col,l,br} A_{nt} \right)$ $\phi R_n = (0.75) \times (0.6 \times (58 \text{ ksi}) \times (7.4375 \text{ in}^2) + (1) \times (58 \text{ ksi}) \times (1.4688 \text{ in}^2)) \leq 0.6 \times (36 \text{ ksi}) \times (10.5 \text{ in}^2) + (1) \times (58 \text{ ksi}) \times (1.4688 \text{ in}^2))$ $\phi R_n = 233.99 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{V_c}{\phi R_n} = \frac{(17.678 \text{ kip})}{(233.99 \text{ kip})} = 0.075549$	PASS
AISC 360-16 Chapter J4.3 Eq. (J4-5)	Check No. 10: Design Capacity of Double Angle in Block Shear at Gusset Side due to Axial Load Calculate the net area of the double angle subject to tension. $t_{2L,at,col,l,br} = 0.5 \text{ in}$ - Lower Brace Double Angle Thickness $n_r = 4$ - Number of Bolt Rows $s_r = 3 \text{ in}$ - Bolt Row Spacing $L_{ev} = 1.5 \text{ in}$ - Vertical Edge Distance at Double Angle $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Double Angle A_{nt} - Net Area Subject to Tension (L-pattern) $A_{nt} = [2(t_{2L,at,col,l,br})] [(n_r - 1) s_r + L_{ev} - (n_r - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nt} = (1 \text{ in}) \times [((4) - 1) \times (3 \text{ in}) + (1.5 \text{ in}) - ((4) - 0.5) \times ((0.8125 \text{ in}) + (0.0625 \text{ in}))]$ $A_{nt} = 7.4375 \text{ in}^2$ Calculate the gross area of the double angle subject to shear. $t_{2L,at,col,l,br} = 0.5 \text{ in}$ - Lower Brace Double Angle Thickness $L_{eh} = 2 \text{ in}$ - Horizontal Edge Distance at Double Angle $n_c = 1$ - Number of Bolt Columns $s_c = 0 \text{ in}$ - Bolt Column Spacing A_{gv} - Gross Area Subject to Shear (L-pattern) $A_{gv} = [2(t_{2L,at,col,l,br})] [L_{eh} + (n_c - 1) s_c]$ $A_{gv} = (1 \text{ in}) \times [(2 \text{ in}) + ((1) - 1) \times (0 \text{ in})]$ $A_{gv} = 2 \text{ in}^2$ Calculate the net area of the double angle subject to shear. $t_{2L,at,col,l,br} = 0.5 \text{ in}$ - Lower Brace Double Angle Thickness $L_{NSL,l,br} = 5 \text{ in}$ - Lower Brace Double Angle Leg at Beam Side $g_a = 3 \text{ in}$ - Double Angle Gage $n_c = 1$ - Number of Bolt Columns $d_h = 1 \text{ in}$ - Horizontal Bolt Hole Dimension at Double Angle A_{nv} - Net Area Subject to Shear (L-pattern) $A_{nv} = [2(t_{2L,at,col,l,br})] [L_{NSL,l,br} - g_a - (n_c - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nv} = (1 \text{ in}) \times [(5 \text{ in}) - (3 \text{ in}) - ((1) - 0.5) \times ((1 \text{ in}) + (0.0625 \text{ in}))]$ $A_{nv} = 1.4688 \text{ in}^2$ Calculate the design block shear capacity of the double angle. $F_{u,at,col,l,br} = 58 \text{ ksi}$ - Lower Brace Double Angle Tensile Stress $A_{nv} = 1.4688 \text{ in}^2$ - Net Area Subject to Shear (L-pattern) $U_{bs} = 1$ - Uniformity factor $A_{nt} = 7.4375 \text{ in}^2$ - Net Area Subject to Tension (L-pattern) $F_{y,at,col,l,br} = 36 \text{ ksi}$ - Lower Brace Double Angle Yield Stress $A_{gv} = 2 \text{ in}^2$ - Gross Area Subject to Shear (L-pattern) $\phi = 0.75$ - Block Shear Resistance Factor ϕR_n - Design Block Shear Capacity of Section $\phi R_n = \phi \left(0.6 F_{u,at,col,l,br} A_{nv} + U_{bs} F_{u,at,col,l,br} A_{nt} \leq 0.6 F_{y,at,col,l,br} A_{gv} + U_{bs} F_{u,at,col,l,br} A_{nt} \right)$ $\phi R_n = (0.75) \times (0.6 \times (58 \text{ ksi}) \times (1.4688 \text{ in}^2) + (1) \times (58 \text{ ksi}) \times (7.4375 \text{ in}^2)) \leq 0.6 \times (36 \text{ ksi}) \times (2 \text{ in}^2) + (1) \times (58 \text{ ksi}) \times (7.4375 \text{ in}^2))$ $\phi R_n = 355.93 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{H_{c,T,lower}}{\phi R_n} = \frac{(7.4996 \text{ kip})}{(355.93 \text{ kip})} = 0.02107$	PASS
	Check No. 11: Shear and Axial Interaction of Block Shear in Double Angle Calculate the shear and axial interaction of block shear capacities in the double angle. $V_c = 17.678 \text{ kip}$ - Vertical Shear at Gusset to Column $H_{c,T,lower} = 7.4996 \text{ kip}$ - Tension Load at Gusset to Column <i>Interaction</i> - Shear and Axial Interaction $Interaction = \sqrt{\left(\frac{V_c}{\phi R_n} \right)^2 + \left(\frac{H_{c,T,lower}}{\phi R_n} \right)^2}$	

	$Interaction = \sqrt{\left(\frac{(17.678 \text{ kip})}{(233.99 \text{ kip})}\right)^2 + \left(\frac{(7.4996 \text{ kip})}{(355.93 \text{ kip})}\right)^2}$ $Interaction = 0.078432$ Result: Demand over Capacity Ratio $DCR = \frac{Interaction}{1.0} = \frac{(0.078432)}{(1)} = 0.078432$	PASS
AISC 360-16 Chapter J4.2 Eq. (J4-3) AISC 360-16 Chapter J4.2 Eq. (J4-4) AISC 360-16 Chapter J4.2 Eq. (J4-3) AISC 360-16 Chapter J4.2 Eq. (J4-4) AISC 360-16 Chapter J4.2	Check No. 12: Design Capacity of Double Angle in Shear at Gusset Side Calculate the gross area of double angle subject to yielding. $L_{2L,at,col,l,br} = 12 \text{ in}$ - Lower Brace Double Angle Length $t_{2L,at,col,l,br} = 0.5 \text{ in}$ - Lower Brace Double Angle Thickness A_{gv} - Section Gross Area $A_{gv} = L_{2L,at,col,l,br} [2(t_{2L,at,col,l,br})]$ $A_{gv} = (12 \text{ in}) \times (1 \text{ in})$ $A_{gv} = 12 \text{ in}^2$ Calculate the shear yielding capacity of the double angle. $F_{y,at,col,l,br} = 36 \text{ ksi}$ - Lower Brace Double Angle Yield Stress $A_{gv} = 12 \text{ in}^2$ - Section Gross Area $\phi = 1$ - Shear Yielding Resistance Factor $\phi R_{n_{sy}}$ - Design Shear Yielding Capacity of Section $\phi R_{n_{sy}} = \phi 0.6 F_{y,at,col,l,br} A_{gv}$ $\phi R_{n_{sy}} = (1) \times 0.6 \times (36 \text{ ksi}) \times (12 \text{ in}^2)$ $\phi R_{n_{sy}} = 259.2 \text{ kip}$ Calculate the net area of double angle subject to rupture. $t_{2L,at,col,l,br} = 0.5 \text{ in}$ - Lower Brace Double Angle Thickness $L_{2L,at,col,l,br} = 12 \text{ in}$ - Lower Brace Double Angle Length $n_r = 4$ - Number of Bolt Rows $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Double Angle A_{nv} - Section Net Area $A_{nv} = [2(t_{2L,at,col,l,br})] [L_{2L,at,col,l,br} - n_r (d_h + 0.0625 \text{ in})]$ $A_{nv} = (1 \text{ in}) \times [(12 \text{ in}) - (4) \times ((0.8125 \text{ in}) + (0.0625 \text{ in}))]$ $A_{nv} = 8.5 \text{ in}^2$ Calculate the shear rupture capacity of the double angle. $F_{u,at,col,l,br} = 58 \text{ ksi}$ - Lower Brace Double Angle Tensile Stress $A_{nv} = 8.5 \text{ in}^2$ - Section Net Area $\phi = 0.75$ - Shear Rupture Resistance Factor $\phi R_{n_{sr}}$ - Design Shear Rupture Capacity of Section $\phi R_{n_{sr}} = \phi 0.6 F_{u,at,col,l,br} A_{nv}$ $\phi R_{n_{sr}} = (0.75) \times 0.6 \times (58 \text{ ksi}) \times (8.5 \text{ in}^2)$ $\phi R_{n_{sr}} = 221.85 \text{ kip}$ Determine the governing shear capacity of the double angle. $\phi R_{n_{sy}} = 259.2 \text{ kip}$ - Design shear yielding capacity of double angle $\phi R_{n_{sr}} = 221.85 \text{ kip}$ - Design shear rupture capacity of double angle ϕR_n - Governing Design Capacity $\phi R_n = \min(\phi R_{n_{sy}}, \phi R_{n_{sr}})$ $\phi R_n = \min((259.2 \text{ kip}), (221.85 \text{ kip}))$ $\phi R_n = 221.85 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{V_u}{\phi R_n} = \frac{(17.678 \text{ kip})}{(221.85 \text{ kip})} = 0.079683$	PASS
	Check No. 13: Design Capacity of Double Angle in Flexure Using Double Coped Procedure at Gusset Side Calculate and compare upper and lower limits. $E = 29000 \text{ ksi}$ - Modulus for Steel $F_{y,at,col,l,br} = 36 \text{ ksi}$ - Lower Brace Double Angle Yield Stress $L_{b,2L} = 3 \text{ in}$ - Unbraced Length at Double Angle $L_{2L,at,col,l,br} = 12 \text{ in}$ - Lower Brace Double Angle Length $t_{2L,at,col,l,br} = 0.5 \text{ in}$ - Lower Brace Double Angle Thickness Depth to Thickness Ratio	

	$\frac{L_y d}{t^2} = \frac{(3 \text{ in}) \times (12 \text{ in})}{(0.5 \text{ in})^2} = 144$ Depth to Thickness Ratio Lower Limit $\frac{0.08 E}{F_y} = \frac{0.08 \times (29000 \text{ ksi})}{(36 \text{ ksi})} = 64.444$ Depth to Thickness Ratio Upper Limit $\frac{1.9 E}{F_y} = \frac{1.9 \times (29000 \text{ ksi})}{(36 \text{ ksi})} = 1530.6$ So, $\frac{0.08 E}{F_y} < \frac{L_y d}{t^2}$ and $\frac{L_y d}{t^2} \leq \frac{1.9 E}{F_y}$. Calculate the plastic section modulus of the double angle. $t_{2L,at,col,l,br} = 0.5 \text{ in}$ - Lower Brace Double Angle Thickness $L_{2L,at,col,l,br} = 12 \text{ in}$ - Lower Brace Double Angle Length Plastic Section Modulus $Z = \frac{t_{2L,at,col,l,br} (L_{2L,at,col,l,br})^2}{4} = \frac{(0.5 \text{ in}) \times ((12 \text{ in}))^2}{4} = 18 \text{ in}^3$ Calculate the elastic section modulus of the double angle. $t_{2L,at,col,l,br} = 0.5 \text{ in}$ - Lower Brace Double Angle Thickness $L_{2L,at,col,l,br} = 12 \text{ in}$ - Lower Brace Double Angle Length Elastic Section Modulus $S = \frac{t_{2L,at,col,l,br} (L_{2L,at,col,l,br})^3}{6} = \frac{(0.5 \text{ in}) \times ((12 \text{ in}))^3}{6} = 12 \text{ in}^3$ Calculate the lateral torsional buckling modification factor. $L_{b,2L} = 3 \text{ in}$ - Unbraced Length at Double Angle $L_{2L,at,col,l,br} = 12 \text{ in}$ - Lower Brace Double Angle Length $d_{c,2L} = 0 \text{ in}$ - Double Angle Cope Depth (Assumed 0) C_b - Lateral-Torsional Buckling Modification Factor $C_b = \left[3 + \ln \left(\frac{L_{b,2L}}{L_{2L,at,col,l,br}} \right) \right] \left(1 - \frac{d_{c,2L}}{L_{2L,at,col,l,br}} \right) \geq 1.84$ $C_b = \left[3 + \ln \left(\frac{(3 \text{ in})}{(12 \text{ in})} \right) \right] \times \left(1 - \frac{(0 \text{ in})}{(12 \text{ in})} \right) \geq 1.84$ $C_b = 1.84$ Calculate the flexural capacity of the double angle. $C_b = 1.84$ - Lateral-Torsional Buckling Modification Factor $\frac{L_y d}{t^2} = 144$ - Depth to Thickness Ratio $F_{y,at,col,l,br} = 36 \text{ ksi}$ - Lower Brace Double Angle Yield Stress $E = 29000 \text{ ksi}$ - Modulus for Steel $S = 12 \text{ in}^3$ - Elastic Section Modulus $Z = 18 \text{ in}^3$ - Plastic Section Modulus $\phi = 0.9$ - Flexure Resistance Factor ϕM_n - Design Flexural Capacity of Coped Section $\phi M_n = \phi \left[C_b \left[1.52 - 0.274 \left(\frac{L_b d}{t^2} \right) \left(\frac{F_{y,at,col,l,br}}{E} \right) \right] F_{y,at,col,l,br} S \leq F_{y,at,col,l,br} Z \right] 2$ $\phi M_n = (0.9) \times \left[(1.84) \times \left[1.52 - 0.274 \times ((144)) \times \left(\frac{(36 \text{ ksi})}{(29000 \text{ ksi})} \right) \right] \times (36 \text{ ksi}) \times (12 \text{ in}^3) \leq (36 \text{ ksi}) \times (18 \text{ in}^3) \right] \times 2$ $\phi M_n = 97.2 \text{ kip} \cdot \text{ft}$ Calculate the equivalent shear capacity of the double angle. $\phi M_n = 97.2 \text{ kip} \cdot \text{ft}$ - Design Flexural Capacity of Coped Section $L_{b,2L} = 3 \text{ in}$ - Unbraced Length at Double Angle ϕV_n - Design Capacity of Single Plate in Bending $\phi V_n = \frac{\phi M_n}{L_{b,2L}}$ $\phi V_n = \frac{(97.2 \text{ kip} \cdot \text{ft})}{(3 \text{ in})}$ $\phi V_n = 388.8 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{V_r}{\phi V_n} = \frac{(17.678 \text{ kip})}{(388.8 \text{ kip})} = 0.045467$	PASS
AISC 360-16 Chapter 14.2 Eq. (I4-3) AISC 360-16 Chapter F11.2 Eq. (F11-2) AISC 15th Edition pg. (10-90) Eq. (10-5)	Check No. 14: Shear and Moment Interaction of Double Angle at Gusset Side Calculate the shear and moment interaction of the double angle. $V_r = 17.678 \text{ kip}$ - Vertical Shear at Gusset to Column Moment at Component $M_r = V_r L_{b,2L} = (17.678 \text{ kip}) \times (3 \text{ in}) = 4.4194 \text{ kip} \cdot \text{ft}$ $\phi V_n = 259.2 \text{ kip}$ - Design shear yielding capacity of double angle $\phi M_n = 97.2 \text{ kip} \cdot \text{ft}$ - Design Flexural Capacity of Double Angle I_{nt} - Interaction for Shear Yielding/Buckling and Flexure $I_{nt} = \left(\frac{V_r}{\phi V_n} \right)^2 + \left(\frac{M_r}{\phi M_n} \right)^2$	

	$I_{nt} = \left(\frac{(17.678 \text{ kip})}{(259.2 \text{ kip})} \right)^2 + \left(\frac{(4.4194 \text{ kip}\cdot\text{ft})}{(97.2 \text{ kip}\cdot\text{ft})} \right)^2$ $I_{nt} = 0.0067186$ Result: Demand over Capacity Ratio $DCR = \frac{I_{nt}}{1.0} = \frac{(0.0067186)}{(1)} = 0.0067186$	PASS
AISC 360-16 Chapter J4.1 Eq. (J4-1) AISC 360-16 Chapter J4.1 Eq. (J4-2) AISC 360-16 Chapter J4.1 Eq. (J4-1) AISC 360-16 Chapter J4.1 Eq. (J4-2) AISC 360-16 Chapter J4.2	Check No. 15: Design Capacity of Double Angle in Tension Calculate the tensile yielding capacity of the double angle. $\phi = 0.9$ - Tensile Yielding Resistance Factor $F_{y,at,col,l,br} = 36$ ksi - Lower Brace Double Angle Yield Stress $t_{2L,at,col,l,br} = 0.5$ in - Lower Brace Double Angle Thickness $L_{2L,at,col,l,br} = 12$ in - Lower Brace Double Angle Length $\phi R_{n,ty}$ - Design Tension Yielding Capacity of Section $\phi R_{n,ty} = \phi F_{y,at,col,l,br} [2(t_{2L,at,col,l,br})] L_{2L,at,col,l,br}$ $\phi R_{n,ty} = (0.9) \times (36 \text{ ksi}) \times (1 \text{ in}) \times (12 \text{ in})$ $\phi R_{n,ty} = 388.8 \text{ kip}$ Calculate the tensile rupture capacity of the double angle. $\phi = 0.75$ - Tensile Rupture Resistance Factor $F_{u,at,col,l,br} = 58$ ksi - Lower Brace Double Angle Tensile Stress $t_{2L,at,col,l,br} = 0.5$ in - Lower Brace Double Angle Thickness $L_{2L,at,col,l,br} = 12$ in - Lower Brace Double Angle Length $n_r = 4$ - Number of Bolt Rows $d_h = 0.8125$ in - Vertical Bolt Hole Dimension at Double Angle $\phi R_{n,tr}$ - Design Tension Rupture Capacity of Section $\phi R_{n,tr} = \phi F_{u,at,col,l,br} [2(t_{2L,at,col,l,br})] [L_{2L,at,col,l,br} - n_r (d_h + 0.0625 \text{ in})]$ $\phi R_{n,tr} = (0.75) \times (58 \text{ ksi}) \times (1 \text{ in}) \times [(12 \text{ in}) - (4) \times ((0.8125 \text{ in}) + (0.0625 \text{ in}))]$ $\phi R_{n,tr} = 369.75 \text{ kip}$ Determine the governing tensile capacity of the double angle. $\phi R_{n,ty} = 388.8$ kip - Design Tension Yielding Capacity of Section $\phi R_{n,tr} = 369.75$ kip - Design Tension Rupture Capacity of Section ϕR_n - Governing Design Capacity $\phi R_n = \min(\phi R_{n,ty}, \phi R_{n,tr})$ $\phi R_n = \min((388.8 \text{ kip}), (369.75 \text{ kip}))$ $\phi R_n = 369.75 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{H_{c,T,lower}}{\phi R_n} = \frac{(7.4996 \text{ kip})}{(369.75 \text{ kip})} = 0.020283$	PASS
	Check No. 16: Tension and Flexure Interaction of the Double Angle Calculate the tension and flexure interaction of the double angle. $H_{c,T,lower} = 7.4996$ kip - Tension Load at Gusset to Column $V_r = 17.678$ kip - Vertical Shear at Gusset to Column <i>Interaction</i> - Tension and Flexure Interaction $Interaction = \frac{H_{c,T,lower}}{2 \phi R_n} + \frac{V_r}{\phi V_n}$ $Interaction = \frac{(7.4996 \text{ kip})}{2 \times (369.75 \text{ kip})} + \frac{(17.678 \text{ kip})}{(388.8 \text{ kip})}$ $Interaction = 0.055609$ Result: Demand over Capacity Ratio $DCR = \frac{Interaction}{1.0} = \frac{(0.055609)}{(1)} = 0.055609$	PASS
AISC 360-16 Chapter J4.4 Eq. (J4-6)	Check No. 17: Design Capacity of Double Angle in Compression Calculate the compression buckling capacity of the double angle. $\phi = 0.9$ - Compression Resistance Factor $F_{y,at,col,l,br} = 36$ ksi - Lower Brace Double Angle Yield Stress $t_{2L,at,col,l,br} = 0.5$ in - Lower Brace Double Angle Thickness $L_{2L,at,col,l,br} = 12$ in - Lower Brace Double Angle Length $K = 1.2$ - Effective Length Factor $g_a = 3$ in - Double Angle Gage $\frac{KL}{r} = 12.471$ - Effective Length Slenderness Ratio Since, $\frac{KL}{r} \leq 25$. ϕR_n - Design Compressive Capacity of Section $\phi R_n = \phi F_{y,at,col,l,br} [2(t_{2L,at,col,l,br})] L_{2L,at,col,l,br}$	

	$\phi R_n = (0.9) \times (36 \text{ ksi}) \times (1 \text{ in}) \times (12 \text{ in})$ $\phi R_n = 388.8 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{H_{c,C,lower}}{\phi R_n} = \frac{(7.4996 \text{ kip})}{(388.8 \text{ kip})} = 0.019289$	PASS
	Check No. 18: Compression and Flexure Interaction of the Double Angle Calculate the compression and flexure interaction of the double angle. $H_{c,C,lower} = 7.4996 \text{ kip}$ - Compression Load at Gusset to Column $V_r = 17.678 \text{ kip}$ - Vertical Shear at Gusset to Column <i>Interaction</i> - Compression and Flexure Interaction $Interaction = \frac{H_{c,C,lower}}{2 \phi R_n} + \frac{V_r}{\phi V_n}$ $Interaction = \frac{(7.4996 \text{ kip})}{2 \times (388.8 \text{ kip})} + \frac{(17.678 \text{ kip})}{(388.8 \text{ kip})}$ $Interaction = 0.055112$ Result: Demand over Capacity Ratio $DCR = \frac{Interaction}{1.0} = \frac{(0.055112)}{(1)} = 0.055112$	PASS
AISC 360-16 Chapter J4.3 Eq. (J4-5)	Check No. 19: Design Capacity of Gusset Plate in Block Shear Calculate the net area of the gusset plate subject to tension. $t_{gp,l,br} = 1 \text{ in}$ - Lower Brace Gusset Plate Thickness $n_c = 1$ - Number of Bolt Columns $s_c = 0 \text{ in}$ - Bolt Column Spacing $L_{eh} = 2 \text{ in}$ - Horizontal Edge Distance at Gusset Plate $d_h = 1 \text{ in}$ - Horizontal Bolt Hole Dimension at Double Angle A_{nt} - Net Area Subject to Tension (L-pattern) $A_{nt} = t_{gp,l,br} [(n_c - 1) s_c + L_{eh} - (n_c - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nt} = (1 \text{ in}) \times [(1 - 1) \times (0 \text{ in}) + (2 \text{ in}) - ((1) - 0.5) \times ((1 \text{ in}) + (0.0625 \text{ in}))]$ $A_{nt} = 1.4688 \text{ in}^2$ Calculate the gross area of the gusset plate subject to shear. $t_{gp,l,br} = 1 \text{ in}$ - Lower Brace Gusset Plate Thickness $L_{ev} = 3 \text{ in}$ - Vertical Edge Distance at Gusset Plate $n_r = 4$ - Number of Bolt Rows $s_r = 3 \text{ in}$ - Bolt Row Spacing A_{gv} - Gross Area Subject to Shear (L-pattern) $A_{gv} = t_{gp,l,br} [L_{ev} + (n_r - 1) s_r]$ $A_{gv} = (1 \text{ in}) \times [(3 \text{ in}) + ((4) - 1) \times (3 \text{ in})]$ $A_{gv} = 12 \text{ in}^2$ Calculate the net area of the gusset plate subject to shear. $t_{gp,l,br} = 1 \text{ in}$ - Lower Brace Gusset Plate Thickness $h_{gp,l,br} = 15 \text{ in}$ - Lower Brace Gusset Plate Height along Column $L_{ev} = 3 \text{ in}$ - Vertical Edge Distance at Gusset Plate $n_r = 4$ - Number of Bolt Rows $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Double Angle A_{nv} - Net Area Subject to Shear (L-pattern) $A_{nv} = t_{gp,l,br} [h_{gp,l,br} - L_{ev} - (n_r - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nv} = (1 \text{ in}) \times [(15 \text{ in}) - (3 \text{ in}) - ((4) - 0.5) \times ((0.8125 \text{ in}) + (0.0625 \text{ in}))]$ $A_{nv} = 8.9375 \text{ in}^2$ Calculate the design block shear capacity of the gusset plate. $F_{u,l,br} = 65 \text{ ksi}$ - Lower Brace Gusset Plate Tensile Stress $A_{nv} = 8.9375 \text{ in}^2$ - Net Area Subject to Shear (L-pattern) $U_{bs} = 1$ - Uniformity factor $A_{nt} = 1.4688 \text{ in}^2$ - Net Area Subject to Tension (L-pattern) $F_{y,l,br} = 50 \text{ ksi}$ - Lower Brace Gusset Plate Yield Stress $A_{gv} = 12 \text{ in}^2$ - Gross Area Subject to Shear (L-pattern) $\phi = 0.75$ - Block Shear Resistance Factor ϕR_n - Design Block Shear Capacity of Section $\phi R_n = \phi (0.6 F_{u,l,br} A_{nv} + U_{bs} F_{u,l,br} A_{nt} \leq 0.6 F_{y,l,br} A_{gv} + U_{bs} F_{u,l,br} A_{nt})$ $\phi R_n = (0.75) \times (0.6 \times (65 \text{ ksi}) \times (8.9375 \text{ in}^2) + (1) \times (65 \text{ ksi}) \times (1.4688 \text{ in}^2) \leq 0.6 \times (50 \text{ ksi}) \times (12 \text{ in}^2) + (1) \times (65 \text{ ksi}) \times (1.4688 \text{ in}^2))$ $\phi R_n = 333.02 \text{ kip}$ Result: Demand over Capacity Ratio	PASS

	$DCR = \frac{V_r}{\phi R_n} = \frac{(17.678 \text{ kip})}{(333.02 \text{ kip})} = 0.053082$	
	Check No. 20: Design Capacity of Gusset Plate in Block Shear due to Axial Load Calculate the net area of the gusset plate subject to tension. $t_{gp,l,br} = 1 \text{ in}$ - Lower Brace Gusset Plate Thickness $n_r = 4$ - Number of Bolt Rows $s_r = 3 \text{ in}$ - Bolt Row Spacing $L_{ev} = 3 \text{ in}$ - Vertical Edge Distance at Gusset Plate $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Double Angle A_{nt} - Net Area Subject to Tension (L-pattern) $A_{nt} = t_{gp,l,br} [(n_r - 1) s_r + L_{ev} - (n_r - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nt} = (1 \text{ in}) \times [(4 - 1) \times (3 \text{ in}) + (3 \text{ in}) - ((4) - 0.5) \times ((0.8125 \text{ in}) + (0.0625 \text{ in}))]$ $A_{nt} = 8.9375 \text{ in}^2$ Calculate the gross area of the gusset plate subject to shear. $t_{gp,l,br} = 1 \text{ in}$ - Lower Brace Gusset Plate Thickness $L_{eh} = 2 \text{ in}$ - Horizontal Edge Distance at Gusset Plate $n_c = 1$ - Number of Bolt Columns $s_c = 0 \text{ in}$ - Bolt Column Spacing A_{gv} - Gross Area Subject to Shear (L-pattern) $A_{gv} = t_{gp,l,br} [L_{eh} + (n_c - 1) s_c]$ $A_{gv} = (1 \text{ in}) \times [(2 \text{ in}) + ((1) - 1) \times (0 \text{ in})]$ $A_{gv} = 2 \text{ in}^2$ Calculate the net area of the gusset plate subject to shear. $t_{gp,l,br} = 1 \text{ in}$ - Lower Brace Gusset Plate Thickness $n_c = 1$ - Number of Bolt Columns $s_c = 0 \text{ in}$ - Bolt Column Spacing $d_h = 1 \text{ in}$ - Horizontal Bolt Hole Dimension at Double Angle $L_{eh} = 2 \text{ in}$ - Horizontal Edge Distance at Gusset Plate A_{nv} - Net Area Subject to Shear (L-pattern) $A_{nv} = t_{gp,l,br} (L_{eh} + (n_c - 1) s_c - (n_c - 0.5) (d_h + 0.0625 \text{ in}))$ $A_{nv} = (1 \text{ in}) \times ((2 \text{ in}) + ((1) - 1) \times (0 \text{ in}) - ((1) - 0.5) \times ((1 \text{ in}) + (0.0625 \text{ in})))$ $A_{nv} = 1.4688 \text{ in}^2$ Calculate the design block shear capacity of the gusset plate. $F_{u,l,br} = 65 \text{ ksi}$ - Lower Brace Gusset Plate Tensile Stress $A_{nv} = 1.4688 \text{ in}^2$ - Net Area Subject to Shear (L-pattern) $U_{bs} = 1$ - Uniformity factor $A_{nt} = 8.9375 \text{ in}^2$ - Net Area Subject to Tension (L-pattern) $F_{y,l,br} = 50 \text{ ksi}$ - Lower Brace Gusset Plate Yield Stress $A_{gv} = 2 \text{ in}^2$ - Gross Area Subject to Shear (L-pattern) $\phi = 0.75$ - Block Shear Resistance Factor ϕR_n - Design Block Shear Capacity of Section $\phi R_n = \phi (0.6 F_{u,l,br} A_{nv} + U_{bs} F_{u,l,br} A_{nt} \leq 0.6 F_{y,l,br} A_{gv} + U_{bs} F_{u,l,br} A_{nt})$ $\phi R_n = (0.75) \times (0.6 \times (65 \text{ ksi}) \times (1.4688 \text{ in}^2) + (1) \times (65 \text{ ksi}) \times (8.9375 \text{ in}^2) \leq 0.6 \times (50 \text{ ksi}) \times (2 \text{ in}^2) + (1) \times (65 \text{ ksi}) \times (8.9375 \text{ in}^2))$ $\phi R_n = 478.66 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{H_{c,T,lower}}{\phi R_n} = \frac{(7.4996 \text{ kip})}{(478.66 \text{ kip})} = 0.015668$	PASS
AISC 360-16 Chapter J4.3 Eq. (J4-5)	Check No. 21: Shear and Axial Interaction of Block Shear in Gusset Plate Calculate the shear and axial interaction of block shear capacities in the gusset plate. $V_r = 17.678 \text{ kip}$ - Vertical Shear at Gusset to Column $H_{c,T,lower} = 7.4996 \text{ kip}$ - Tension Load at Gusset to Column Interaction - Shear and Axial Interaction $Interaction = \sqrt{\left(\frac{V_r}{\phi R_n}\right)^2 + \left(\frac{H_{c,T,lower}}{\phi R_n}\right)^2}$ $Interaction = \sqrt{\left(\frac{(17.678 \text{ kip})}{(333.02 \text{ kip})}\right)^2 + \left(\frac{(7.4996 \text{ kip})}{(478.66 \text{ kip})}\right)^2}$ $Interaction = 0.055346$ Result: Demand over Capacity Ratio $DCR = \frac{Interaction}{1.0} = \frac{(0.055346)}{(1)} = 0.055346$	PASS
	Check No. 22: Connection Detailing Limitations at Support 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
Double Angle Minimum Vertical Edge Distance	1.000	1.500	0.667	PASS
Double Angle Minimum Horizontal Edge Distance	1.125	2.000	0.563	PASS
Column Minimum Horizontal Edge Distance	1.000	3.750	0.267	PASS
Minimum Bolt Gage at Column	4.375	7.000	0.625	PASS
Minimum Bolt Gage at Double Angle	4.500	7.000	0.643	PASS

Result:

Demand over Capacity Ratio

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

PASS

Check No. 23: Design Capacity of the Bolt Group in Shear at Support Side

$F_{nv} = 54$ ksi - Bolt Nominal Shear Strength

$F_{nt} = 90$ ksi - Bolt Nominal Tensile Strength

$N_T = 8$ - Total Number of Bolts in Tension

$H_c = 7.4996$ kip - Axial Load at Gusset to Column

F_{nv} - Design Modified Bolt Shear Strength

$$F_{nv} = \min \left(1.3 F_{nv} - \frac{0.84 F_{nv} H_c}{F_{nt} N_T d_b^2}, F_{nv} \right)$$

$$F_{nv} = \min \left(1.3 \times (54 \text{ ksi}) - \frac{0.84 \times (54 \text{ ksi}) \times (7.4996 \text{ kip})}{(90 \text{ ksi}) \times (8) \times (0.75 \text{ in})^2}, (54 \text{ ksi}) \right)$$

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

$d_b = 0.75$ in - Bolt Diameter

$N_s = 1$ - Number of Shear Planes

$N_V = 8$ - Total Number of Bolts in Shear

$h_f = 1$ - Filler Factor for Bearing Bolts

$\phi = 0.75$ - Bolt Shear Resistance Factor

ϕR_n - Design Bolt Shear Capacity

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

$$\phi R_n = (0.75) \times (54 \text{ ksi}) \times \frac{\pi}{4} \times ((0.75 \text{ in}))^2 \times (1) \times (8) \times (1)$$

$$\phi R_n = 143.14 \text{ kip}$$

Result:

Demand over Capacity Ratio

$$DCR = \frac{V_r}{\phi R_n} = \frac{(17.678 \text{ kip})}{(143.14 \text{ kip})} = 0.1235$$

PASS

Check No. 24: Design Capacity of the Bolt Group in Bearing and Tear-out on Double Angle at Support Side

Calculate the bolt bearing capacity of the double angle.

$d_b = 0.75$ in - Bolt Diameter

$t_{2L,at,col,l,br} = 0.5$ in - Lower Brace Double Angle Thickness

$F_{u,at,col,l,br} = 58$ ksi - Lower Brace Double Angle Tensile Stress

$N_V = 8$ - Total Number of Bolts in Shear

$\phi = 0.75$ - Bolt Bearing Resistance Factor

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

$$\phi R_{n_bearing} = \phi 2.4 d_b t_{2L,at,col,l,br} F_{u,at,col,l,br} N_V$$

$$\phi R_{n_bearing} = (0.75) \times 2.4 \times (0.75 \text{ in}) \times (0.5 \text{ in}) \times (58 \text{ ksi}) \times (8)$$

$$\phi R_{n_bearing} = 313.2 \text{ kip}$$

Calculate the clear distance of outer bolts on double angle.

$L_{ev} = 1.5$ in - Vertical Edge Distance at Double Angle

$d_h = 0.8125$ in - Vertical Bolt Hole Dimension at Double Angle

l_{c1} - Clear Distance at First Bolt Row

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

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

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

Calculate the clear distance of inner bolts on double angle.

$s_r = 3$ in - Bolt Row Spacing

$d_h = 0.8125$ in - Vertical Bolt Hole Dimension at Double Angle

l_{c2} - Clear Distance at Rest of Bolts

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

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

AISC 360-16 Chapter J3.10 Eq. (J3-6c) AISC 360-16 Chapter J3.10 Eq. (J3-6a) AISC 360-16 Chapter J3.10 Eq. (J3-6c) AISC 360-16 Chapter J3.10	$l_{c2} = 2.1875 \text{ in}$ Calculate the bolt tear-out capacity of the double angle. $l_{c1} = 1.0938 \text{ in}$ - Clear Distance at First Bolt Row $l_{c2} = 2.1875 \text{ in}$ - Clear Distance at Rest of Bolts $t_{2L,at,col,l,br} = 0.5 \text{ in}$ - Lower Brace Double Angle Thickness $F_{u,at,col,l,br} = 58 \text{ ksi}$ - Lower Brace Double Angle Tensile Stress $N_V = 8$ - Total Number of Bolts in Shear $n_r = 4$ - Number of Bolt Rows $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_tearout}$ - Design Bolt Tear-out Capacity of Section $\phi R_{n_tearout} = \phi [1.2 l_{c1} t_{2L,at,col,l,br} F_{u,at,col,l,br} + 1.2 l_{c2} t_{2L,at,col,l,br} F_{u,at,col,l,br} (n_r - 1)] \left(\frac{N_V}{n_r} \right)$ $\phi R_{n_tearout} = (0.75) \times [1.2 \times (1.0938 \text{ in}) \times (0.5 \text{ in}) \times (58 \text{ ksi}) + 1.2 \times (2.1875 \text{ in}) \times (0.5 \text{ in}) \times (58 \text{ ksi}) \times ((4) - 1)] \times \left(\frac{(8)}{(4)} \right)$ $\phi R_{n_tearout} = 399.66 \text{ kip}$ Determine the governing bearing and tear-out capacity of the bolt group on double angle. $\phi R_{n_bearing} = 313.2 \text{ kip}$ - Design bolt bearing capacity of double angle $\phi R_{n_tearout} = 399.66 \text{ kip}$ - Design bolt tear-out capacity of double angle ϕR_n - Governing Design Capacity $\phi R_n = \min(\phi R_{n_bearing}, \phi R_{n_tearout})$ $\phi R_n = \min((313.2 \text{ kip}), (399.66 \text{ kip}))$ $\phi R_n = 313.2 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{V_r}{\phi R_n} = \frac{(17.678 \text{ kip})}{(313.2 \text{ kip})} = 0.056442$	PASS
AISC 360-16 Chapter J3.10 Eq. (J3-6a) AISC 360-16 Chapter J3.10 Eq. (J3-6c) AISC 360-16 Chapter J3.10	Check No. 25: Design Capacity of the Bolt Group in Bearing and Tear-out at Support Flange Calculate the bolt bearing capacity. $d_b = 0.75 \text{ in}$ - Bolt Diameter $t_{f,c} = 0.71 \text{ in}$ - Column Flange Thickness $F_{u,c} = 65 \text{ ksi}$ - Column Tensile Stress $N_V = 8$ - Total Number of Bolts in Shear $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_bearing}$ - Design Bolt Bearing Capacity of Section $\phi R_{n_bearing} = \phi 2.4 d_b t_{f,c} F_{u,c} N_V$ $\phi R_{n_bearing} = (0.75) \times 2.4 \times (0.75 \text{ in}) \times (0.71 \text{ in}) \times (65 \text{ ksi}) \times (8)$ $\phi R_{n_bearing} = 498.42 \text{ kip}$ Calculate the clear distance of inner bolts on member. $s_r = 3 \text{ in}$ - Bolt Row Spacing $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Column 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. $l_{c2} = 2.1875 \text{ in}$ - Clear Distance at Rest of Bolts $t_{f,c} = 0.71 \text{ in}$ - Column Flange Thickness $F_{u,c} = 65 \text{ ksi}$ - Column Tensile Stress $N_V = 8$ - Total Number of Bolts in Shear $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_tearout}$ - Design Bolt Tear-out Capacity of Section $\phi R_{n_tearout} = \phi 1.2 l_{c2} t_{f,c} F_{u,c} N_V$ $\phi R_{n_tearout} = (0.75) \times 1.2 \times (2.1875 \text{ in}) \times (0.71 \text{ in}) \times (65 \text{ ksi}) \times (8)$ $\phi R_{n_tearout} = 726.86 \text{ kip}$ Determine the governing bearing and tear-out capacity of the bolt group. $\phi R_{n_bearing} = 498.42 \text{ kip}$ - Design bolt bearing capacity of member $\phi R_{n_tearout} = 726.86 \text{ kip}$ - Design bolt tear-out capacity of member ϕR_n - Governing Design Capacity $\phi R_n = \min(\phi R_{n_bearing}, \phi R_{n_tearout})$ $\phi R_n = \min((498.42 \text{ kip}), (726.86 \text{ kip}))$	

	$\phi R_n = 498.42 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{V_r}{\phi R_n} = \frac{(17.678 \text{ kip})}{(498.42 \text{ kip})} = 0.035467$	PASS
AISC 360-16 Chapter J4.3 Eq. (J4-5)	Check No. 26: Design Capacity of Double Angle in Block Shear at Support Side Calculate the net area of the double angle subject to tension. $t_{2L,at,col,l,br} = 0.5 \text{ in}$ - Lower Brace Double Angle Thickness $n_c = 1$ - Number of Bolt Columns $s_c = 0 \text{ in}$ - Bolt Column Spacing $L_{eh} = 2 \text{ in}$ - Horizontal Edge Distance at Double Angle $d_h = 1 \text{ in}$ - Horizontal Bolt Hole Dimension at Double Angle A_{nt} - Net Area Subject to Tension (L-pattern) $A_{nt} = [2(t_{2L,at,col,l,br})] [(n_c - 1) s_c + L_{eh} - (n_c - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nt} = (1 \text{ in}) \times [(1 - 1) \times (0 \text{ in}) + (2 \text{ in}) - ((1) - 0.5) \times ((1 \text{ in}) + (0.0625 \text{ in}))]$ $A_{nt} = 1.4688 \text{ in}^2$ Calculate the gross area of the double angle subject to shear. $t_{2L,at,col,l,br} = 0.5 \text{ in}$ - Lower Brace Double Angle Thickness $L_{ev} = 1.5 \text{ in}$ - Vertical Edge Distance at Double Angle $n_r = 4$ - Number of Bolt Rows $s_r = 3 \text{ in}$ - Bolt Row Spacing A_{gv} - Gross Area Subject to Shear (L-pattern) $A_{gv} = [2(t_{2L,at,col,l,br})] [L_{ev} + (n_r - 1) s_r]$ $A_{gv} = (1 \text{ in}) \times [(1.5 \text{ in}) + ((4) - 1) \times (3 \text{ in})]$ $A_{gv} = 10.5 \text{ in}^2$ Calculate the net area of the double angle subject to shear. $t_{2L,at,col,l,br} = 0.5 \text{ in}$ - Lower Brace Double Angle Thickness $L_{2L,at,col,l,br} = 12 \text{ in}$ - Lower Brace Double Angle Length $L_{ev} = 1.5 \text{ in}$ - Vertical Edge Distance at Double Angle $n_r = 4$ - Number of Bolt Rows $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Double Angle A_{nv} - Net Area Subject to Shear (L-pattern) $A_{nv} = [2(t_{2L,at,col,l,br})] [L_{2L,at,col,l,br} - L_{ev} - (n_r - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nv} = (1 \text{ in}) \times [(12 \text{ in}) - (1.5 \text{ in}) - ((4) - 0.5) \times ((0.8125 \text{ in}) + (0.0625 \text{ in}))]$ $A_{nv} = 7.4375 \text{ in}^2$ Calculate the design block shear capacity of the double angle. $F_{u,at,col,l,br} = 58 \text{ ksi}$ - Lower Brace Double Angle Tensile Stress $A_{nv} = 7.4375 \text{ in}^2$ - Net Area Subject to Shear (L-pattern) $U_{bs} = 1$ - Uniformity factor $A_{nt} = 1.4688 \text{ in}^2$ - Net Area Subject to Tension (L-pattern) $F_{y,at,col,l,br} = 36 \text{ ksi}$ - Lower Brace Double Angle Yield Stress $A_{gv} = 10.5 \text{ in}^2$ - Gross Area Subject to Shear (L-pattern) $\phi = 0.75$ - Block Shear Resistance Factor ϕR_n - Design Block Shear Capacity of Section $\phi R_n = \phi (0.6 F_{u,at,col,l,br} A_{nv} + U_{bs} F_{u,at,col,l,br} A_{nt} \leq 0.6 F_{y,at,col,l,br} A_{gv} + U_{bs} F_{u,at,col,l,br} A_{nt})$ $\phi R_n = (0.75) \times (0.6 \times (58 \text{ ksi}) \times (7.4375 \text{ in}^2) + (1) \times (58 \text{ ksi}) \times (1.4688 \text{ in}^2)) \leq 0.6 \times (36 \text{ ksi}) \times (10.5 \text{ in}^2) + (1) \times (58 \text{ ksi}) \times (1.4688 \text{ in}^2))$ $\phi R_n = 233.99 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{V_r}{\phi R_n} = \frac{(17.678 \text{ kip})}{(233.99 \text{ kip})} = 0.075549$	PASS
AISC 360-16 Chapter J4.2	Check No. 27: Design Capacity of Double Angle in Shear at Support Side Calculate the gross area of double angle subject to yielding. $L_{2L,at,col,l,br} = 12 \text{ in}$ - Lower Brace Double Angle Length $t_{2L,at,col,l,br} = 0.5 \text{ in}$ - Lower Brace Double Angle Thickness A_{gv} - Section Gross Area $A_{gv} = L_{2L,at,col,l,br} [2(t_{2L,at,col,l,br})]$ $A_{gv} = (12 \text{ in}) \times (1 \text{ in})$ $A_{gv} = 12 \text{ in}^2$ Calculate the shear yielding capacity of the double angle. $F_{y,at,col,l,br} = 36 \text{ ksi}$ - Lower Brace Double Angle Yield Stress $A_{gv} = 12 \text{ in}^2$ - Section Gross Area $\phi = 1$ - Shear Yielding Resistance Factor $\phi R_{n,vy}$ - Design Shear Yielding Capacity of Section	

Eq. (J4-3) AISC 360-16 Chapter J4.2 Eq. (J4-4) AISC 360-16 Chapter J4.2 Eq. (J4-3) AISC 360-16 Chapter J4.2 Eq. (J4-4) AISC 360-16 Chapter J4.2	$\phi R_{n_{sy}} = \phi 0.6 F_{y,at,col,l,br} A_{gv}$ $\phi R_{n_{sy}} = (1) \times 0.6 \times (36 \text{ ksi}) \times (12 \text{ in}^2)$ $\phi R_{n_{sy}} = 259.2 \text{ kip}$ Calculate the net area of double angle subject to rupture. $t_{2L,at,col,l,br} = 0.5 \text{ in}$ - Lower Brace Double Angle Thickness $L_{2L,at,col,l,br} = 12 \text{ in}$ - Lower Brace Double Angle Length $n_r = 4$ - Number of Bolt Rows $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Double Angle A_{nv} - Section Net Area $A_{nv} = [2(t_{2L,at,col,l,br})] [L_{2L,at,col,l,br} - n_r (d_h + 0.0625 \text{ in})]$ $A_{nv} = (1 \text{ in}) \times [(12 \text{ in}) - (4) \times ((0.8125 \text{ in}) + (0.0625 \text{ in}))]$ $A_{nv} = 8.5 \text{ in}^2$ Calculate the shear rupture capacity of the double angle. $F_{u,at,col,l,br} = 58 \text{ ksi}$ - Lower Brace Double Angle Tensile Stress $A_{nv} = 8.5 \text{ in}^2$ - Section Net Area $\phi = 0.75$ - Shear Rupture Resistance Factor $\phi R_{n_{sr}}$ - Design Shear Rupture Capacity of Section $\phi R_{n_{sr}} = \phi 0.6 F_{u,at,col,l,br} A_{nv}$ $\phi R_{n_{sr}} = (0.75) \times 0.6 \times (58 \text{ ksi}) \times (8.5 \text{ in}^2)$ $\phi R_{n_{sr}} = 221.85 \text{ kip}$ Determine the governing shear capacity of the double angle. $\phi R_{n_{sy}} = 259.2 \text{ kip}$ - Design shear yielding capacity of double angle $\phi R_{n_{sr}} = 221.85 \text{ kip}$ - Design shear rupture capacity of double angle ϕR_n - Governing Design Capacity $\phi R_n = \min(\phi R_{n_{sy}}, \phi R_{n_{sr}})$ $\phi R_n = \min((259.2 \text{ kip}), (221.85 \text{ kip}))$ $\phi R_n = 221.85 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{V_r}{\phi R_n} = \frac{(17.678 \text{ kip})}{(221.85 \text{ kip})} = 0.079683$	PASS
AISC 15th Edition pg. (9-9) Eq. (9-15)	Check No. 28: Design Capacity of Double Angle in Flexure Using Double Coped Procedure at Support Side Calculate and compare upper and lower limits. $E = 29000 \text{ ksi}$ - Modulus for Steel $F_{y,at,col,l,br} = 36 \text{ ksi}$ - Lower Brace Double Angle Yield Stress $L_{b,2L} = 3 \text{ in}$ - Unbraced Length at Double Angle $L_{2L,at,col,l,br} = 12 \text{ in}$ - Lower Brace Double Angle Length $t_{2L,at,col,l,br} = 0.5 \text{ in}$ - Lower Brace Double Angle Thickness Depth to Thickness Ratio $\frac{L_{bd}}{t^2} = \frac{(3 \text{ in}) \times (12 \text{ in})}{(0.5 \text{ in})^2} = 144$ 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, $\frac{0.08E}{F_y} < \frac{L_{bd}}{t^2}$ and $\frac{L_{bd}}{t^2} \leq \frac{1.9E}{F_y}$. Calculate the plastic section modulus of the double angle. $t_{2L,at,col,l,br} = 0.5 \text{ in}$ - Lower Brace Double Angle Thickness $L_{2L,at,col,l,br} = 12 \text{ in}$ - Lower Brace Double Angle Length Plastic Section Modulus $Z = \frac{t_{2L,at,col,l,br} (L_{2L,at,col,l,br})^2}{4} = \frac{(0.5 \text{ in}) \times ((12 \text{ in}))^2}{4} = 18 \text{ in}^3$ Calculate the elastic section modulus of the double angle. $t_{2L,at,col,l,br} = 0.5 \text{ in}$ - Lower Brace Double Angle Thickness $L_{2L,at,col,l,br} = 12 \text{ in}$ - Lower Brace Double Angle Length Elastic Section Modulus $S = \frac{t_{2L,at,col,l,br} (L_{2L,at,col,l,br})^2}{6} = \frac{(0.5 \text{ in}) \times ((12 \text{ in}))^2}{6} = 12 \text{ in}^3$ Calculate the lateral torsional buckling modification factor. $L_{b,2L} = 3 \text{ in}$ - Unbraced Length at Double Angle $L_{2L,at,col,l,br} = 12 \text{ in}$ - Lower Brace Double Angle Length $d_{c,2L} = 0 \text{ in}$ - Double Angle Cope Depth (Assumed 0) C_b - Lateral-Torsional Buckling Modification Factor	

AISC 15th Edition pg. (9-9) Eq. (9-15) AISC 360-16 Chapter F11.2 Eq. (F11-2) AISC 360-16 Chapter F11.2 Eq. (F11-2)	$C_b = \left[3 + \ln \left(\frac{L_{b,2L}}{L_{2L,at,col,l,br}} \right) \right] \left(1 - \frac{d_{c,2L}}{L_{2L,at,col,l,br}} \right) \geq 1.84$ $C_b = \left[3 + \ln \left(\frac{(3 \text{ in})}{(12 \text{ in})} \right) \right] \times \left(1 - \frac{(0 \text{ in})}{(12 \text{ in})} \right) \geq 1.84$ $C_b = 1.84$ Calculate the flexural capacity of the double angle. $C_b = 1.84$ - Lateral-Torsional Buckling Modification Factor $\frac{L_{bd}}{t^2} = 144$ - Depth to Thickness Ratio $F_{y,at,col,l,br} = 36 \text{ ksi}$ - Lower Brace Double Angle Yield Stress $E = 29000 \text{ ksi}$ - Modulus for Steel $S = 12 \text{ in}^3$ - Elastic Section Modulus $Z = 18 \text{ in}^3$ - Plastic Section Modulus $\phi = 0.9$ - Flexure Resistance Factor ϕM_n - Design Flexural Capacity of Coped Section $\phi M_n = \phi \left[C_b \left[1.52 - 0.274 \left(\frac{L_b d}{t^2} \right) \left(\frac{F_{y,at,col,l,br}}{E} \right) \right] F_{y,at,col,l,br} S \leq F_{y,at,col,l,br} Z \right] 2$ $\phi M_n = (0.9) \times \left[(1.84) \times \left[1.52 - 0.274 \times ((144)) \times \left(\frac{(36 \text{ ksi})}{(29000 \text{ ksi})} \right) \right] \times (36 \text{ ksi}) \times (12 \text{ in}^3) \leq (36 \text{ ksi}) \times (18 \text{ in}^3) \right] \times 2$ $\phi M_n = 97.2 \text{ kip}\cdot\text{ft}$ Calculate the equivalent shear capacity of the double angle. $\phi M_n = 97.2 \text{ kip}\cdot\text{ft}$ - Design Flexural Capacity of Coped Section $L_{b,2L} = 3 \text{ in}$ - Unbraced Length at Double Angle ϕV_n - Design Capacity of Single Plate in Bending $\phi V_n = \frac{\phi M_n}{L_{b,2L}}$ $\phi V_n = \frac{(97.2 \text{ kip}\cdot\text{ft})}{(3 \text{ in})}$ $\phi V_n = 388.8 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{V_r}{\phi V_n} = \frac{(17.678 \text{ kip})}{(388.8 \text{ kip})} = 0.045467$	PASS
AISC 360-16 Chapter J4.2 Eq. (J4-3) AISC 360-16 Chapter F11.2 Eq. (F11-2) AISC 15th Edition pg. (10-90) Eq. (10-5)	Check No. 29: Shear and Moment Interaction of Double Angle at Support Side Calculate the shear and moment interaction of the double angle. $V_r = 17.678 \text{ kip}$ - Vertical Shear at Gusset to Column Moment at Component $M_r = V_r L_{b,2L} = (17.678 \text{ kip}) \times (3 \text{ in}) = 4.4194 \text{ kip}\cdot\text{ft}$ $\phi V_n = 259.2 \text{ kip}$ - Design shear yielding capacity of double angle $\phi M_n = 97.2 \text{ kip}\cdot\text{ft}$ - Design Flexural Capacity of Double Angle I_{nt} - Interaction for Shear Yielding/Buckling and Flexure $I_{nt} = \left(\frac{V_r}{\phi V_n} \right)^2 + \left(\frac{M_r}{\phi M_n} \right)^2$ $I_{nt} = \left(\frac{(17.678 \text{ kip})}{(259.2 \text{ kip})} \right)^2 + \left(\frac{(4.4194 \text{ kip}\cdot\text{ft})}{(97.2 \text{ kip}\cdot\text{ft})} \right)^2$ $I_{nt} = 0.0067186$ Result: Demand over Capacity Ratio $DCR = \frac{I_{nt}}{1.0} = \frac{(0.0067186)}{(1)} = 0.0067186$	PASS
	Check No. 30: Design Capacity of Double Angle in Bolt Prying $d_b = 0.75 \text{ in}$ - Bolt Diameter $n_r = 4$ - Number of Bolt Rows $s_r = 3 \text{ in}$ - Bolt Row Spacing $H_{c,T,lower} = 7.4996 \text{ kip}$ - Tension Load at Gusset to Column $F_{u,at,col,l,br} = 58 \text{ ksi}$ - Lower Brace Double Angle Tensile Stress $b = 2.75 \text{ in}$ - b prying $F_{nv} = 54 \text{ ksi}$ - Bolt Nominal Shear Strength $F_{nt} = 90 \text{ ksi}$ - Bolt Nominal Tensile Strength $V_r = 17.678 \text{ kip}$ - Vertical Shear at Gusset to Column $L_{eh} = 2 \text{ in}$ - Horizontal Edge Distance at Double Angle $d_h = 1 \text{ in}$ - Horizontal Bolt Hole Dimension at Double Angle $B = 29.818 \text{ kip}$ - Available tension per bolt δ $\delta = 1 - \frac{d_h}{\min(3.5 b, s_r)}$	

AISC 15th Edition Equation (9-19a)	$\delta = 1 - \frac{(1 \text{ in})}{\min(3.5 \times (2.75 \text{ in}), (3 \text{ in}))}$ $\delta = 0.66667$ β $\beta = \frac{\min(L_{eh} + 0.5 d_b, 1.25 b + 0.5 d_b) (2 H_{c,T,lower} - 4 B n_r)}{H_{c,T,lower} (d_b - 2 b)}$ $\beta = \frac{\min((2 \text{ in}) + 0.5 \times (0.75 \text{ in}), 1.25 \times (2.75 \text{ in}) + 0.5 \times (0.75 \text{ in})) \times (2 \times (7.4996 \text{ kip}) - 4 \times (29.818 \text{ kip}) \times (4))}{(7.4996 \text{ kip}) \times ((0.75 \text{ in}) - 2 \times (2.75 \text{ in}))}$ $\beta = 30.808$ α' $\alpha' = \min \left[1, \left \frac{1}{\delta} \left(\frac{\beta}{1 - \beta} \right) \right \right]$ $\alpha' = \min \left[1, \left \frac{1}{(0.66667)} \times \left(\frac{(30.808)}{1 - (30.808)} \right) \right \right]$ $\alpha' = 1$ t_{min} - Minimum thickness required to ensure an acceptable combination of fitting strength and stiffness and bolt strength $t_{min} = \sqrt{\frac{2.22 H_{c,T,lower} (b - 0.5 d_b)}{\min(3.5 b, s_r) F_{u,at,col,l,br} n_r (1 + \delta \alpha')}}}$ $t_{min} = \sqrt{\frac{2.22 \times (7.4996 \text{ kip}) \times ((2.75 \text{ in}) - 0.5 \times (0.75 \text{ in}))}{\min(3.5 \times (2.75 \text{ in}), (3 \text{ in})) \times (58 \text{ ksi}) \times (4) \times (1 + (0.66667) \times (1))}}$ $t_{min} = 0.18463 \text{ in}$ Result: Demand over Capacity Ratio $DCR = \frac{t_{min}}{t_{2L,at,col,l,br}} = \frac{(0.18463 \text{ in})}{(0.5 \text{ in})} = 0.36926$	PASS
AISC 15th Edition Equation (9-19a)	Check No. 31: Design Capacity of Column Flange in Bolt Prying $d_b = 0.75 \text{ in}$ - Bolt Diameter $n_r = 4$ - Number of Bolt Rows $s_r = 3 \text{ in}$ - Bolt Row Spacing $V_r = 17.678 \text{ kip}$ - Vertical Shear at Gusset to Column $H_{c,T,lower} = 7.4996 \text{ kip}$ - Tension Load at Gusset to Column $F_{u,c} = 65 \text{ ksi}$ - Column Tensile Stress $b = 3.28 \text{ in}$ - b prying $F_{nv} = 54 \text{ ksi}$ - Bolt Nominal Shear Strength $F_{nt} = 90 \text{ ksi}$ - Bolt Nominal Tensile Strength $L_{eh} = 3.75 \text{ in}$ - Horizontal Edge Distance at Column $d_h = 0.8125 \text{ in}$ - Horizontal Bolt Hole Dimension at Column $B = 29.818 \text{ kip}$ - Available tension per bolt δ $\delta = 1 - \frac{d_h}{\min(3.5 b, s_r)}$ $\delta = 1 - \frac{(0.8125 \text{ in})}{\min(3.5 \times (3.28 \text{ in}), (3 \text{ in}))}$ $\delta = 0.72917$ β $\beta = \frac{\min(L_{eh} + 0.5 d_b, 1.25 b + 0.5 d_b) (2 H_{c,T,lower} - 4 B n_r)}{H_{c,T,lower} (d_b - 2 b)}$ $\beta = \frac{\min((3.75 \text{ in}) + 0.5 \times (0.75 \text{ in}), 1.25 \times (3.28 \text{ in}) + 0.5 \times (0.75 \text{ in})) \times (2 \times (7.4996 \text{ kip}) - 4 \times (29.818 \text{ kip}) \times (4))}{(7.4996 \text{ kip}) \times ((0.75 \text{ in}) - 2 \times (3.28 \text{ in}))}$ $\beta = 43.746$ α' $\alpha' = \min \left[1, \left \frac{1}{\delta} \left(\frac{\beta}{1 - \beta} \right) \right \right]$ $\alpha' = \min \left[1, \left \frac{1}{(0.72917)} \times \left(\frac{(43.746)}{1 - (43.746)} \right) \right \right]$ $\alpha' = 1$ t_{min} - Minimum thickness required to ensure an acceptable combination of fitting strength and stiffness and bolt strength $t_{min} = \sqrt{\frac{2.22 H_{c,T,lower} (b - 0.5 d_b)}{\min(3.5 b, s_r) F_{u,c} n_r (1 + \delta \alpha')}}}$	

	$t_{min} = \sqrt{\frac{2.22 \times (7.4996 \text{ kip}) \times ((3.28 \text{ in}) - 0.5 \times (0.75 \text{ in}))}{\min(3.5 \times (3.28 \text{ in}), (3 \text{ in})) \times (65 \text{ ksi}) \times (4) \times (1 + (0.72917) \times (1))}}$ $t_{min} = 0.18937 \text{ in}$ Result: Demand over Capacity Ratio $DCR = \frac{t_{min}}{t_{fc}} = \frac{(0.18937 \text{ in})}{(0.71 \text{ in})} = 0.26671$	PASS
AISC 360-16 Chapter J10.3 Eq. (J10-4) AISC 360-16 Chapter J10.2 Eq. (J10-2) AISC 360-16 Chapter J10	Check No. 32: Design Capacity of Support due to Compressive Load Calculate the web local crippling capacity of the column. $\phi = 0.75$ - Web Local Crippling Resistance Factor $t_{w,c} = 0.44 \text{ in}$ - Column Web Thickness $L_{2L,at,col,l,br} = 12 \text{ in}$ - Lower Brace Double Angle Length $d_c = 14 \text{ in}$ - Column Depth $t_{f,c} = 0.71 \text{ in}$ - Column Flange Thickness $F_{y,c} = 50 \text{ ksi}$ - Column Yield Stress $E = 29000 \text{ ksi}$ - Modulus for Steel ϕR_{n_wlc} - Design Web Local Crippling Capacity of Support $\phi R_{n_wlc} = \phi 0.8 (t_{w,c})^2 \left[1 + 3 \left(\frac{L_{2L,at,col,l,br}}{d_c} \right) \left(\frac{t_{w,c}}{t_{f,c}} \right)^{1.5} \right] \sqrt{\frac{E F_{y,c} t_{f,c}}{t_{w,c}}} Q_f$ $\phi R_{n_wlc} = (0.75) \times 0.8 \times ((0.44 \text{ in}))^2 \times \left[1 + 3 \times \left(\frac{(12 \text{ in})}{(14 \text{ in})} \right) \times \left(\frac{(0.44 \text{ in})}{(0.71 \text{ in})} \right)^{1.5} \right] \times \sqrt{\frac{(29000 \text{ ksi}) \times (50 \text{ ksi}) \times (0.71 \text{ in})}{(0.44 \text{ in})}} \times (1)$ $\phi R_{n_wlc} = 400.58 \text{ kip}$ Calculate the web local yielding capacity of the column. $\phi = 1$ - Web Local Yielding Resistance Factor $F_{y,c} = 50 \text{ ksi}$ - Column Yield Stress $t_{w,c} = 0.44 \text{ in}$ - Column Web Thickness $k_{des,c} = 1.31 \text{ in}$ - Column kdes $L_{2L,at,col,l,br} = 12 \text{ in}$ - Lower Brace Double Angle Length ϕR_{n_wly} - Design Web Local Yielding Capacity of Support $\phi R_{n_wly} = \phi F_{y,c} t_{w,c} (5 k_{des,c} + L_{2L,at,col,l,br})$ $\phi R_{n_wly} = (1) \times (50 \text{ ksi}) \times (0.44 \text{ in}) \times (5 \times (1.31 \text{ in}) + (12 \text{ in}))$ $\phi R_{n_wly} = 408.1 \text{ kip}$ Calculate for the governing design capacity of the column. ϕR_n - Governing Design Capacity of the Support $\phi R_n = \min(\phi R_{n_wlc}, \phi R_{n_wly})$ $\phi R_n = \min((400.58 \text{ kip}), (408.1 \text{ kip}))$ $\phi R_n = 400.58 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{H_{n,C,lower}}{\phi R_n} = \frac{(7.4996 \text{ kip})}{(400.58 \text{ kip})} = 0.018722$	PASS
	Summary of Checks	

Design Checks	Demand	Capacity	DCR	Result
Connection Detailing Limitations at Gusset Side	12.000	13.500	0.889	PASS
Design Capacity of the Bolt Group in Shear at Gusset Side	19.203	42.170	0.455	PASS
Design Capacity of the Bolt Group in Bearing and Tear-out on Double Angle at Gusset Side	17.678	204.623	0.086	PASS
Design Capacity of the Bolt Group in Bearing and Tear-out on Double Angle at Gusset Side due to Axial Load	7.500	204.623	0.037	PASS
Shear and Axial Interaction of Bolt Bearing/Tearout in Double Angle	0.094	1.000	0.094	PASS
Design Capacity of the Bolt Group in Bearing and Tear-out on Gusset Plate	17.678	229.319	0.077	PASS
Design Capacity of the Bolt Group in Bearing and Tear-out on Gusset Plate due to Axial Load	7.500	229.319	0.033	PASS
Shear and Axial Interaction of Bolt Bearing/Tearout on Gusset Plate	0.084	1.000	0.084	PASS
Design Capacity of Double Angle in Block Shear at Gusset Side	17.678	233.991	0.076	PASS
Design Capacity of Double Angle in Block Shear at Gusset Side due to Axial Load	7.500	355.931	0.021	PASS
Shear and Axial Interaction of Block Shear in Double Angle	0.078	1.000	0.078	PASS
Design Capacity of Double Angle in Shear at Gusset Side	17.678	221.850	0.080	PASS
Design Capacity of Double Angle in Flexure Using Double Coped Procedure at Gusset Side	17.678	388.800	0.045	PASS
Shear and Moment Interaction of Double Angle at Gusset Side	0.007	1.000	0.007	PASS
Design Capacity of Double Angle in Tension	7.500	369.750	0.020	PASS
Tension and Flexure Interaction of the Double Angle	0.056	1.000	0.056	PASS
Design Capacity of Double Angle in Compression	7.500	388.800	0.019	PASS
Compression and Flexure Interaction of the Double Angle	0.055	1.000	0.055	PASS
Design Capacity of Gusset Plate in Block Shear	17.678	333.023	0.053	PASS
Design Capacity of Gusset Plate in Block Shear due to Axial Load	7.500	478.664	0.016	PASS
Shear and Axial Interaction of Block Shear in Gusset Plate	0.055	1.000	0.055	PASS
Connection Detailing Limitations at Support Side	2.000	3.000	0.667	PASS
Design Capacity of the Bolt Group in Shear at Support Side	17.678	143.139	0.124	PASS
Design Capacity of the Bolt Group in Bearing and Tear-out on Double Angle at Support Side	17.678	313.200	0.056	PASS
Design Capacity of the Bolt Group in Bearing and Tear-out at Support Flange	17.678	498.420	0.035	PASS
Design Capacity of Double Angle in Block Shear at Support Side	17.678	233.991	0.076	PASS
Design Capacity of Double Angle in Shear at Support Side	17.678	221.850	0.080	PASS
Design Capacity of Double Angle in Flexure Using Double Coped Procedure at Support Side	17.678	388.800	0.045	PASS
Shear and Moment Interaction of Double Angle at Support Side	0.007	1.000	0.007	PASS
Design Capacity of Double Angle in Bolt Prying	0.185	0.500	0.369	PASS
Design Capacity of Column Flange in Bolt Prying	0.189	0.710	0.267	PASS
Design Capacity of Support due to Compressive Load	7.500	400.582	0.019	PASS

REFERENCES	CALCULATIONS	RESULTS																																			
	Upper Brace Connection to Gusset Plate AISC 360-16 LRFD Check No. 1: Connection Detailing Limitations <table><thead><tr><th>Detailing Limitations</th><th>Limit Value (in)</th><th>Actual Value (in)</th><th>DCR</th><th>Result</th></tr></thead><tbody><tr><td>Minimum Bolt Gage</td><td>1.750</td><td>3.000</td><td>0.583</td><td>PASS</td></tr><tr><td>Minimum Bolt Row Spacing</td><td>2.000</td><td>3.000</td><td>0.667</td><td>PASS</td></tr><tr><td>Maximum Bolt Row Spacing</td><td>6.000</td><td>3.000</td><td>0.500</td><td>PASS</td></tr><tr><td>Gusset Plate Minimum Vertical Edge Distance</td><td>1.063</td><td>2.000</td><td>0.531</td><td>PASS</td></tr><tr><td>Brace Minimum Vertical Edge Distance</td><td>1.000</td><td>2.000</td><td>0.500</td><td>PASS</td></tr><tr><td>Brace Minimum Horizontal Edge Distance</td><td>1.000</td><td>2.000</td><td>0.500</td><td>PASS</td></tr></tbody></table> Result: Demand over Capacity Ratio $DCR = \frac{d}{c} = \frac{(2)}{(3)} = 0.66667$	Detailing Limitations	Limit Value (in)	Actual Value (in)	DCR	Result	Minimum Bolt Gage	1.750	3.000	0.583	PASS	Minimum Bolt Row Spacing	2.000	3.000	0.667	PASS	Maximum Bolt Row Spacing	6.000	3.000	0.500	PASS	Gusset Plate Minimum Vertical Edge Distance	1.063	2.000	0.531	PASS	Brace Minimum Vertical Edge Distance	1.000	2.000	0.500	PASS	Brace Minimum Horizontal Edge Distance	1.000	2.000	0.500	PASS	PASS
Detailing Limitations	Limit Value (in)	Actual Value (in)	DCR	Result																																	
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AISC 360-16 Chapter J3.8 Eq. (J3-4)	Check No. 2: Design Capacity of the Bolts in Shear $\phi = 0.85$ - Bolt Shear Resistance Factor $\mu = 0.3$ - Mean Slip Coefficient $D_u = 1.13$ $h_f = 1$ - Filler Factor for SC Bolts $d_b = 0.75$ in - Bolt Diameter $T_b = 28$ kip - Minimum Bolt Pretension $N_s = 2$ - Number of Slip Planes $n_r = 3$ - Number of Bolt Rows $n_c = 1$ - Number of Bolt Columns ϕR_n - Design Bolt Shear Capacity $\phi R_n = \phi \mu D_u h_f T_b N_s n_r n_c$$\phi R_n = (0.85) \times (0.3) \times (1.13) \times (1) \times (28 \text{ kip}) \times (2) \times (3) \times (1)$$\phi R_n = 48.409 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{T_{u,u,br}}{\phi R_n} = \frac{(15 \text{ kip})}{(48.409 \text{ kip})} = 0.30986$	PASS																																			
AISC 360-16 Chapter J3.10 Eq. (J3-6a)	Check No. 3: Design Bolt Bearing Capacity of the Gusset Plate Calculate the bolt bearing capacity of the gusset plate. $\phi = 0.75$ - Bolt Bearing Resistance Factor $d_b = 0.75$ in - Bolt Diameter $t_{gp,u,br} = 1$ in - Upper Brace Gusset Plate Thickness $F_{u,u,br} = 65$ ksi - Upper Brace Gusset Plate Tensile Stress $n_r = 3$ - Number of Bolt Rows $n_c = 1$ - Number of Bolt Columns $\phi R_{n_bearing}$ - Design Bolt Bearing Capacity of Section $\phi R_{n_bearing} = \phi 2.4 d_b t_{gp,u,br} F_{u,u,br} n_r n_c$$\phi R_{n_bearing} = (0.75) \times 2.4 \times (0.75 \text{ in}) \times (1 \text{ in}) \times (65 \text{ ksi}) \times (3) \times (1)$$\phi R_{n_bearing} = 263.25 \text{ kip}$ Calculate the clear distance of outer bolts on gusset plate. $L_{ev} = 2$ in - Vertical Edge Distance at Brace $d_h = 0.9375$ in - Vertical Bolt Hole Dimension at Gusset Plate l_{c1} - Clear Distance at First Bolt Row $l_{c1} = L_{ev} - \frac{d_h}{2}$$l_{c1} = (2 \text{ in}) - \frac{(0.9375 \text{ in})}{2}$$l_{c1} = 1.5313 \text{ in}$ Calculate the clear distance of inner bolts on gusset plate. $s_r = 3$ in - Bolt Row Spacing $d_h = 0.9375$ in - Vertical Bolt Hole Dimension at Gusset Plate l_{c2} - Clear Distance at Rest of Bolts $l_{c2} = s_r - d_h$$l_{c2} = (3 \text{ in}) - (0.9375 \text{ in})$$l_{c2} = 2.0625 \text{ in}$ Calculate the bolt tear-out capacity of the gusset plate. $l_{c1} = 1.5313$ in - Clear Distance at First Bolt Row $l_{c2} = 2.0625$ in - Clear Distance at Rest of Bolts $t_{gp,u,br} = 1$ in - Upper Brace Gusset Plate Thickness $F_{u,u,br} = 65$ ksi - Upper Brace Gusset Plate Tensile Stress $n_r = 3$ - Number of Bolt Rows																																				

AISC 360-16 Chapter J3.10 Eq. (J3-6c) AISC 360-16 Chapter J3.10 Eq. (J3-6a) AISC 360-16 Chapter J3.10 Eq. (J3-6c) AISC 360-16 Chapter J3.10	$n_c = 1$ - Number of Bolt Columns $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_tearout}$ - Design Bolt Tear-out Capacity of Section $\phi R_{n_tearout} = \phi [1.2 l_{c1} t_{gp,u,br} F_{u,u,br} n_c + 1.2 l_{c2} t_{gp,u,br} F_{u,u,br} n_c (n_r - 1)]$ $\phi R_{n_tearout} = (0.75) \times [1.2 \times (1.5313 \text{ in}) \times (1 \text{ in}) \times (65 \text{ ksi}) \times (1) + 1.2 \times (2.0625 \text{ in}) \times (1 \text{ in}) \times (65 \text{ ksi}) \times (1) \times ((3) - 1)]$ $\phi R_{n_tearout} = 330.89 \text{ kip}$ Determine the governing bearing and tear-out capacity of the bolt group on gusset plate. $\phi R_{n_bearing} = 263.25 \text{ kip}$ - Design Bolt Bearing Capacity of Section $\phi R_{n_tearout} = 330.89 \text{ kip}$ - Design Bolt Tear-out Capacity of Section ϕR_n - Governing Design Capacity $\phi R_n = \min(\phi R_{n_bearing}, \phi R_{n_tearout})$ $\phi R_n = \min((263.25 \text{ kip}), (330.89 \text{ kip}))$ $\phi R_n = 263.25 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{T_{u,u,br}}{\phi R_n} = \frac{(15 \text{ kip})}{(263.25 \text{ kip})} = 0.05698$	PASS
AISC 360-16 Chapter J3.10 Eq. (J3-6a) AISC 360-16 Chapter J3.10 Eq. (J3-6c) AISC 360-16 Chapter J3.10	Check No. 4: Design Bolt Bearing Capacity of the Brace Calculate the bolt bearing capacity of the brace. $\phi = 0.75$ - Bolt Bearing Resistance Factor $d_b = 0.75 \text{ in}$ - Bolt Diameter $t_{u,br} = 0.5 \text{ in}$ - Upper Brace Thickness $F_{u,u,br} = 58 \text{ ksi}$ - Upper Brace Tensile Stress $n_r = 3$ - Number of Bolt Rows $n_c = 1$ - Number of Bolt Columns $\phi R_{n_bearing}$ - Design Bolt Bearing Capacity of Section $\phi R_{n_bearing} = \phi 2.4 d_b 2 t_{u,br} F_{u,u,br} n_r n_c$ $\phi R_{n_bearing} = (0.75) \times 2.4 \times (0.75 \text{ in}) \times (1 \text{ in}) \times (58 \text{ ksi}) \times (3) \times (1)$ $\phi R_{n_bearing} = 234.9 \text{ kip}$ Calculate the clear distance of outer bolts on brace. $L_{ev} = 2 \text{ in}$ - Vertical Edge Distance at Brace $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Brace l_{c1} - Clear Distance at First Bolt Row $l_{c1} = L_{ev} - \frac{d_h}{2}$ $l_{c1} = (2 \text{ in}) - \frac{(0.8125 \text{ in})}{2}$ $l_{c1} = 1.5938 \text{ in}$ Calculate the clear distance of inner bolts on brace. $s_r = 3 \text{ in}$ - Bolt Row Spacing $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Brace 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 brace. $l_{c1} = 1.5938 \text{ in}$ - Clear Distance at First Bolt Row $l_{c2} = 2.1875 \text{ in}$ - Clear Distance at Rest of Bolts $t_{u,br} = 0.5 \text{ in}$ - Upper Brace Thickness $F_{u,u,br} = 58 \text{ ksi}$ - Upper Brace Tensile Stress $n_r = 3$ - Number of Bolt Rows $n_c = 1$ - Number of Bolt Columns $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_tearout}$ - Design Bolt Tear-out Capacity of Section $\phi R_{n_tearout} = \phi [1.2 l_{c1} 2 t_{u,br} F_{u,u,br} n_c + 1.2 l_{c2} 2 t_{u,br} F_{u,u,br} n_c (n_r - 1)]$ $\phi R_{n_tearout} = (0.75) \times [1.2 \times (1.5938 \text{ in}) \times (1 \text{ in}) \times (58 \text{ ksi}) \times (1) + 1.2 \times (2.1875 \text{ in}) \times (1 \text{ in}) \times (58 \text{ ksi}) \times (1) \times ((3) - 1)]$ $\phi R_{n_tearout} = 311.57 \text{ kip}$ Determine the governing bearing and tear-out capacity of the bolt group on brace. $\phi R_{n_bearing} = 234.9 \text{ kip}$ - Design Bolt Bearing Capacity of Section $\phi R_{n_tearout} = 311.57 \text{ kip}$ - Design Bolt Tear-out Capacity of Section	

	ϕR_n - Governing Design Capacity $\phi R_n = \min(\phi R_{n,bearing}, \phi R_{n,tearout})$ $\phi R_n = \min((234.9 \text{ kip}), (311.57 \text{ kip}))$ $\phi R_n = 234.9 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{T_{u,u,br}}{\phi R_n} = \frac{(15 \text{ kip})}{(234.9 \text{ kip})} = 0.063857$	PASS
AISC 360-16 Chapter J4.3 Eq. (J4-5)	Check No. 5: Design Block Shear Capacity of the Gusset Plate Calculate the net area of the gusset plate subject to tension. $t_{gp,u,br} = 1 \text{ in}$ - Upper Brace Gusset Plate Thickness $n_c = 1$ - Number of Bolt Columns $s_c = 0 \text{ in}$ - Bolt Column Spacing $L_{eh,gp} = 2 \text{ in}$ - Gusset Plate Horizontal Edge Distance $d_h = 0.9375 \text{ in}$ - Horizontal Bolt Hole Dimension at Gusset Plate A_{nt} - Net Area Subject to Tension (L-pattern) $A_{nt} = t_{gp,u,br} [(n_c - 1) s_c + L_{eh,gp} - (n_c - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nt} = (1 \text{ in}) \times [(1 - 1) \times (0 \text{ in}) + (2 \text{ in}) - ((1 - 0.5) \times ((0.9375 \text{ in}) + (0.0625 \text{ in})))]$ $A_{nt} = 1.5 \text{ in}^2$ Calculate the gross area of the gusset plate subject to shear. $t_{gp,u,br} = 1 \text{ in}$ - Upper Brace Gusset Plate Thickness $L_{ev} = 2 \text{ in}$ - Vertical Edge Distance at Brace $n_r = 3$ - Number of Bolt Rows $s_r = 3 \text{ in}$ - Bolt Row Spacing A_{gv} - Gross Area Subject to Shear (L-pattern) $A_{gv} = t_{gp,u,br} [L_{ev} + (n_r - 1) s_r]$ $A_{gv} = (1 \text{ in}) \times [(2 \text{ in}) + ((3 - 1) \times (3 \text{ in}))]$ $A_{gv} = 8 \text{ in}^2$ Calculate the net area of the gusset plate subject to shear. $t_{gp,u,br} = 1 \text{ in}$ - Upper Brace Gusset Plate Thickness $L_{br,gp} = 10 \text{ in}$ - Brace Length at Gusset Plate $L_{ev} = 2 \text{ in}$ - Vertical Edge Distance at Brace $n_r = 3$ - Number of Bolt Rows $d_h = 0.9375 \text{ in}$ - Vertical Bolt Hole Dimension at Gusset Plate A_{nv} - Net Area Subject to Shear (L-pattern) $A_{nv} = t_{gp,u,br} [L_{br,gp} - L_{ev} - (n_r - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nv} = (1 \text{ in}) \times [(10 \text{ in}) - (2 \text{ in}) - ((3 - 0.5) \times ((0.9375 \text{ in}) + (0.0625 \text{ in})))]$ $A_{nv} = 5.5 \text{ in}^2$ Calculate the design block shear capacity of the gusset plate. $F_{u,u,br} = 65 \text{ ksi}$ - Upper Brace Gusset Plate Tensile Stress $A_{nv} = 5.5 \text{ in}^2$ - Net Area Subject to Shear (L-pattern) $U_{bs} = 1$ - Uniformity factor $A_{nt} = 1.5 \text{ in}^2$ - Net Area Subject to Tension (L-pattern) $F_{y,u,br} = 50 \text{ ksi}$ - Upper Brace Gusset Plate Yield Stress $A_{gv} = 8 \text{ in}^2$ - Gross Area Subject to Shear (L-pattern) $\phi = 0.75$ - Block Shear Resistance Factor ϕR_n - Design Block Shear Capacity of Section $\phi R_n = \phi (0.6 F_{u,u,br} A_{nv} + U_{bs} F_{u,u,br} A_{nt} \leq 0.6 F_{y,u,br} A_{gv} + U_{bs} F_{u,u,br} A_{nt})$ $\phi R_n = (0.75) \times (0.6 \times (65 \text{ ksi}) \times (5.5 \text{ in}^2) + (1) \times (65 \text{ ksi}) \times (1.5 \text{ in}^2) \leq 0.6 \times (50 \text{ ksi}) \times (8 \text{ in}^2) + (1) \times (65 \text{ ksi}) \times (1.5 \text{ in}^2))$ $\phi R_n = 234 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{T_{u,u,br}}{\phi R_n} = \frac{(15 \text{ kip})}{(234 \text{ kip})} = 0.064103$	PASS
	Check No. 6: Design Block Shear Capacity of the Brace Calculate the net area of the brace subject to tension. $t_{u,br} = 0.5 \text{ in}$ - Upper Brace Thickness $n_c = 1$ - Number of Bolt Columns $s_c = 0 \text{ in}$ - Bolt Column Spacing $L_{eh,br} = 2 \text{ in}$ - Brace Horizontal Edge Distance $d_h = 0.8125 \text{ in}$ - Horizontal Bolt Hole Dimension at Brace A_{nt} - Net Area Subject to Tension (L-pattern) $A_{nt} = 2t_{u,br} [(n_c - 1) s_c + L_{eh,br} - (n_c - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nt} = (1 \text{ in}) \times [(1 - 1) \times (0 \text{ in}) + (2 \text{ in}) - ((1 - 0.5) \times ((0.8125 \text{ in}) + (0.0625 \text{ in})))]$	

	$A_{nt} = 1.5625 \text{ in}^2$ Calculate the gross area of the brace subject to shear. $t_{u,br} = 0.5 \text{ in}$ - Upper Brace Thickness $L_{ev} = 2 \text{ in}$ - Vertical Edge Distance at Brace $n_r = 3$ - Number of Bolt Rows $s_r = 3 \text{ in}$ - Bolt Row Spacing A_{gv} - Gross Area Subject to Shear (L-pattern) $A_{gv} = 2t_{u,br} [L_{ev} + (n_r - 1) s_r]$ $A_{gv} = (1 \text{ in}) \times [(2 \text{ in}) + ((3) - 1) \times (3 \text{ in})]$ $A_{gv} = 8 \text{ in}^2$ Calculate the net area of the brace subject to shear. $t_{u,br} = 0.5 \text{ in}$ - Upper Brace Thickness $L_{br,gp} = 10 \text{ in}$ - Brace Length at Gusset Plate $L_{ev} = 2 \text{ in}$ - Vertical Edge Distance at Brace $n_r = 3$ - Number of Bolt Rows $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Brace A_{nv} - Net Area Subject to Shear (L-pattern) $A_{nv} = 2t_{u,br} [L_{br,gp} - L_{ev} - (n_r - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nv} = (1 \text{ in}) \times [(10 \text{ in}) - (2 \text{ in}) - ((3) - 0.5) \times ((0.8125 \text{ in}) + (0.0625 \text{ in}))]$ $A_{nv} = 5.8125 \text{ in}^2$ Calculate the design block shear capacity of the brace. $F_{u,u,br} = 58 \text{ ksi}$ - Upper Brace Tensile Stress $A_{nv} = 5.8125 \text{ in}^2$ - Net Area Subject to Shear (L-pattern) $U_{bs} = 1$ - Uniformity factor $A_{nt} = 1.5625 \text{ in}^2$ - Net Area Subject to Tension (L-pattern) $F_{y,u,br} = 36 \text{ ksi}$ - Upper Brace Yield Stress $A_{gv} = 8 \text{ in}^2$ - Gross Area Subject to Shear (L-pattern) $\phi = 0.75$ - Block Shear Resistance Factor ϕR_n - Design Block Shear Capacity of Section $\phi R_n = \phi (0.6 F_{u,u,br} A_{nv} + U_{bs} F_{u,u,br} A_{nt} \leq 0.6 F_{y,u,br} A_{gv} + U_{bs} F_{u,u,br} A_{nt})$ $\phi R_n = (0.75) \times (0.6 \times (58 \text{ ksi}) \times (5.8125 \text{ in}^2) + (1) \times (58 \text{ ksi}) \times (1.5625 \text{ in}^2) \leq 0.6 \times (36 \text{ ksi}) \times (8 \text{ in}^2) + (1) \times (58 \text{ ksi}) \times (1.5625 \text{ in}^2))$ $\phi R_n = 197.57 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{T_{u,u,br}}{\phi R_n} = \frac{(15 \text{ kip})}{(197.57 \text{ kip})} = 0.075923$	PASS
AISC 360-16 Chapter J4.3 Eq. (J4-5) AISC 360-16 Chapter J4.1 Eq. (J4-1) AISC 360-16 Chapter J4.1 Eq. (J4-2) AISC 360-16 Chapter J4.1 Eq. (J4-1) AISC 360-16 Chapter J4.1 Eq. (J4-2) AISC 360-16 Chapter J4.2	Check No. 7: Design Capacity of the Gusset Plate in Tension Calculate the tensile yielding capacity of the gusset plate. $\phi = 0.9$ - Tensile Yielding Resistance Factor $F_{y,u,br} = 50 \text{ ksi}$ - Upper Brace Gusset Plate Yield Stress $t_{gp,u,br} = 1 \text{ in}$ - Upper Brace Gusset Plate Thickness $W_{br} = 6.9282 \text{ in}$ - Whitmore Section $\phi R_{n,ty}$ - Design Tension Yielding Capacity of Section $\phi R_{n,ty} = \phi F_{y,u,br} t_{gp,u,br} W_{br}$ $\phi R_{n,ty} = (0.9) \times (50 \text{ ksi}) \times (1 \text{ in}) \times (6.9282 \text{ in})$ $\phi R_{n,ty} = 311.77 \text{ kip}$ Calculate the tensile rupture capacity of the gusset plate. $\phi = 0.75$ - Tensile Rupture Resistance Factor $F_{u,u,br} = 65 \text{ ksi}$ - Upper Brace Gusset Plate Tensile Stress $t_{gp,u,br} = 1 \text{ in}$ - Upper Brace Gusset Plate Thickness $W_{br} = 6.9282 \text{ in}$ - Whitmore Section $n_c = 1$ - Number of Bolt Columns $d_h = 0.9375 \text{ in}$ - Horizontal Bolt Hole Dimension at Gusset Plate $\phi R_{n,tr}$ - Design Tension Rupture Capacity of Section $\phi R_{n,tr} = \phi F_{u,u,br} t_{gp,u,br} [W_{br} - n_c (d_h + 0.0625 \text{ in})]$ $\phi R_{n,tr} = (0.75) \times (65 \text{ ksi}) \times (1 \text{ in}) \times [(6.9282 \text{ in}) - (1) \times ((0.9375 \text{ in}) + (0.0625 \text{ in}))]$ $\phi R_{n,tr} = 289 \text{ kip}$ Determine the governing tensile capacity of the gusset plate. $\phi R_{n,ty} = 311.77 \text{ kip}$ - Design Tension Yielding Capacity of Section $\phi R_{n,tr} = 289 \text{ kip}$ - Design Tension Rupture Capacity of Section ϕR_n - Governing Design Capacity	

	$\phi R_n = \min(\phi R_{n_{ty}}, \phi R_{n_{tr}})$ $\phi R_n = \min((311.77 \text{ kip}), (289 \text{ kip}))$ $\phi R_n = 289 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{T_{u,u,br}}{\phi R_n} = \frac{(15 \text{ kip})}{(289 \text{ kip})} = 0.051903$	PASS
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 Brace in Tension Rupture Calculate the tensile rupture capacity of the brace. $\phi = 0.75$ - Tensile Rupture Resistance Factor $F_{u,u,br} = 58 \text{ ksi}$ - Upper Brace Tensile Stress $A_{u,br} = 9.58 \text{ in}^2$ - Upper Brace Area $t_{u,br} = 0.5 \text{ in}$ - Upper Brace Thickness $d_h = 0.8125 \text{ in}$ - Horizontal Bolt Hole Dimension at Brace $n_c = 1$ - Number of Bolt Columns $n_r = 3$ - Number of Bolt Rows $s_r = 3 \text{ in}$ - Bolt Row Spacing $\bar{y} = 1.42 \text{ in}$ - Brace Centroid U - Shear Lag Factor $U = 1 - \frac{\bar{y}}{(n_r - 1) s_r}$ $U = 1 - \frac{(1.42 \text{ in})}{((3) - 1) \times (3 \text{ in})}$ $U = 0.76343$ $\phi R_{n_{tr}}$ - Design Tension Rupture Capacity of Section $\phi R_{n_{tr}} = \phi F_{u,u,br} U [A_{u,br} - n_c (d_h + 0.0625 \text{ in}) 2t_{u,br}]$ $\phi R_{n_{tr}} = (0.75) \times (58 \text{ ksi}) \times (0.76343) \times [(9.58 \text{ in}^2) - (1) \times ((0.8125 \text{ in}) + (0.0625 \text{ in})) \times (1 \text{ in})]$ $\phi R_{n_{tr}} = 289.09 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{T_{u,u,br}}{\phi R_{n_{tr}}} = \frac{(15 \text{ kip})}{(289.09 \text{ kip})} = 0.051888$	PASS
AISC 360-16 Chapter E3 Eq. (E3-4) AISC 360-16 Chapter E3 Eq. (E3-2) AISC 360-16 Chapter E3 Eq. (E3-1)	Check No. 9: Design Capacity of the Gusset Plate in Compression Calculate the compression buckling capacity of the gusset plate. $\phi = 0.9$ - Compression Resistance Factor $F_{y,u,br} = 50 \text{ ksi}$ - Upper Brace Gusset Plate Yield Stress $t_{gp,u,br} = 1 \text{ in}$ - Upper Brace Gusset Plate Thickness $W_{br} = 6.9282 \text{ in}$ - Whitmore Section $K = 1.2$ - Effective Length Factor $L_{b,u,br} = 11 \text{ in}$ - Upper Brace Unbraced Length $\frac{KL}{r} = 45.726$ - Effective Length Slenderness Ratio Since, $\frac{KL}{r} > 25$. F_e - Elastic Buckling Stress $F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$ $F_e = \frac{\pi^2 \times (29000 \text{ ksi})}{((45.726))^2}$ $F_e = 136.89 \text{ ksi}$ $4.71 \sqrt{\frac{E}{F_y}} = 113.43$ - Effective Length Slenderness Ratio Limiter Since, $\frac{KL}{r} \leq 4.71 \sqrt{\frac{E}{F_y}}$. F_{cr} - Critical Buckling Stress $F_{cr} = \left(0.658 \frac{F_{y,u,br}}{F_e}\right) F_{y,u,br}$ $F_{cr} = \left(0.658 \frac{(50 \text{ ksi})}{(136.89 \text{ ksi})}\right) \times (50 \text{ ksi})$ $F_{cr} = 42.912 \text{ ksi}$ ϕR_n - Design Compressive Capacity of Section $\phi R_n = \phi F_{cr} t_{gp,u,br} W_{br}$ $\phi R_n = (0.9) \times (42.912 \text{ ksi}) \times (1 \text{ in}) \times (6.9282 \text{ in})$ $\phi R_n = 267.57 \text{ kip}$	

	Result: Demand over Capacity Ratio $DCR = \frac{C_{u,ubr}}{\phi R_n} = \frac{(15 \text{ kip})}{(267.57 \text{ kip})} = 0.05606$	PASS																																																		
	Summary of Checks <table><thead><tr><th>Design Checks</th><th>Demand</th><th>Capacity</th><th>DCR</th><th>Result</th></tr></thead><tbody><tr><td>Connection Detailing Limitations</td><td>2.000</td><td>3.000</td><td>0.667</td><td>PASS</td></tr><tr><td>Design Capacity of the Bolts in Shear</td><td>15.000</td><td>48.409</td><td>0.310</td><td>PASS</td></tr><tr><td>Design Bolt Bearing Capacity of the Gusset Plate</td><td>15.000</td><td>263.250</td><td>0.057</td><td>PASS</td></tr><tr><td>Design Bolt Bearing Capacity of the Brace</td><td>15.000</td><td>234.900</td><td>0.064</td><td>PASS</td></tr><tr><td>Design Block Shear Capacity of the Gusset Plate</td><td>15.000</td><td>234.000</td><td>0.064</td><td>PASS</td></tr><tr><td>Design Block Shear Capacity of the Brace</td><td>15.000</td><td>197.569</td><td>0.076</td><td>PASS</td></tr><tr><td>Design Capacity of the Gusset Plate in Tension</td><td>15.000</td><td>289.000</td><td>0.052</td><td>PASS</td></tr><tr><td>Design Capacity of the Brace in Tension Rupture</td><td>15.000</td><td>289.085</td><td>0.052</td><td>PASS</td></tr><tr><td>Design Capacity of the Gusset Plate in Compression</td><td>15.000</td><td>267.571</td><td>0.056</td><td>PASS</td></tr></tbody></table>	Design Checks	Demand	Capacity	DCR	Result	Connection Detailing Limitations	2.000	3.000	0.667	PASS	Design Capacity of the Bolts in Shear	15.000	48.409	0.310	PASS	Design Bolt Bearing Capacity of the Gusset Plate	15.000	263.250	0.057	PASS	Design Bolt Bearing Capacity of the Brace	15.000	234.900	0.064	PASS	Design Block Shear Capacity of the Gusset Plate	15.000	234.000	0.064	PASS	Design Block Shear Capacity of the Brace	15.000	197.569	0.076	PASS	Design Capacity of the Gusset Plate in Tension	15.000	289.000	0.052	PASS	Design Capacity of the Brace in Tension Rupture	15.000	289.085	0.052	PASS	Design Capacity of the Gusset Plate in Compression	15.000	267.571	0.056	PASS	
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	Upper Gusset Plate Connection to Beam AISC 360-16 LRFD Check No. 1: Connection Detailing Limitations <table><thead><tr><th>Detailing Limitations</th><th>Limit Value (in)</th><th>Actual Value (in)</th><th>DCR</th><th>Result</th></tr></thead><tbody><tr><td>Minimum Fillet Weld Size</td><td>0.250</td><td>0.250</td><td>1.000</td><td>PASS</td></tr></tbody></table> Result: Demand over Capacity Ratio $DCR = \frac{d}{c} = \frac{(0.25)}{(0.25)} = 1$	Detailing Limitations	Limit Value (in)	Actual Value (in)	DCR	Result	Minimum Fillet Weld Size	0.250	0.250	1.000	PASS	PASS
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Minimum Fillet Weld Size	0.250	0.250	1.000	PASS								
AISC 15th Ed. Part 9 Eq. (9-2) AISC 360-16 Chapter J2.4 Eq. (J2-4)	Check No. 2: Design Capacity of Weld at Gusset to Beam Connection Information: $\phi = 0.75$ - Fillet Weld Resistance Factor $W = 0.25$ in - Fillet Weld Size $F_{EXX} = 70$ ksi - Filler Metal Classification Strength $t_{f,bm} = 0.57$ in - Beam Flange Thickness $F_{u,bm} = 65$ ksi - Beam Tensile Stress $t_{gp,u,br} = 1$ in - Upper Brace Gusset Plate Thickness $F_{u,u,br} = 65$ ksi - Upper Brace Gusset Plate Tensile Stress $L_w = 15$ in - Weld Length Calculate Dmax: Maximum Fillet Weld Size for Base Metal Strength $D_{max} = \frac{t_{f,bm} F_{u,bm} \leq 0.5 t_{gp,u,br} F_{u,u,br}}{3.09 \text{ kip/in}} = \frac{(0.57 \text{ in}) \times (65 \text{ ksi}) \leq (0.5 \text{ in}) \times (65 \text{ ksi})}{(3.09 \text{ kip/in})} = 10.518$ Calculate Total Weld Length: Total Effective Length of Weld $L_w = 2 [L_w - 2 (W \leq 0.3125 \text{ in})] = 2 \times [(15 \text{ in}) - 2 \times ((0.25 \text{ in}) \leq (0.3125 \text{ in}))] = 29 \text{ in}$ Design Loads: $M_b = 4.2931$ kip·ft - Moment Load at Gusset to Beam $V_{b,upper} = 0$ kip - Vertical Axial Load at Gusset to Beam $V_{b,total} = \frac{8 M_b}{L_w} + V_{b,upper} = \frac{8 \times (4.2931 \text{ kipft})}{(29 \text{ in})} + (0 \text{ kip}) = 14.212$ kip $H_b = 5.8926$ kip - Horizontal Shear at Gusset to Beam Resultant Load $R = \sqrt{(V_{b,total})^2 + (H_b)^2} = \sqrt{((14.212 \text{ kip}))^2 + ((5.8926 \text{ kip}))^2} = 15.385$ kip Loading Angle at Weld in Degrees $\theta = \tan^{-1} \left(\frac{V_{b,total}}{H_b} \right) = \tan^{-1} \left(\frac{(14.212 \text{ kip})}{(5.8926 \text{ kip})} \right) = 67.48$ Calculate Weld Capacity: 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) \left[1 + 0.5 (\sin \theta)^{1.5} \right] L_w$$\phi R_n = (0.75) \times 0.6 \times (70 \text{ ksi}) \times \frac{\sqrt{2}}{2} \times \left((0.25 \text{ in}) \leq \frac{(10.518)}{(16)} \right) \times \left[1 + 0.5 \times (\sin(67.48))^{1.5} \right] \times (29 \text{ in})$$\phi R_n = 233.17 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{R}{\phi R_n} = \frac{(15.385 \text{ kip})}{(233.17 \text{ kip})} = 0.065981$	PASS										
AISC 360-16 Chapter J10.3 Eq. (J10-4) AISC 360-16 Chapter J10.2 Eq. (J10-3)	Check No. 3: Design Capacity of the Beam Support Calculate the web local crippling capacity of the beam. $\phi = 0.75$ - Web Local Crippling Resistance Factor $t_{w,bm} = 0.355$ in - Beam Web Thickness $l_b = 7.5$ in - Bearing Length: Half Gusset Plate Width along Beam $d_{bm} = 18$ in - Beam Depth $t_{f,bm} = 0.57$ in - Beam Flange Thickness $F_{y,bm} = 50$ ksi - Beam Yield Stress $E = 29000$ ksi - Modulus for Steel ϕR_{n_wlc} - Design Web Local Crippling Capacity of Support $\phi R_{n_wlc} = \phi 0.4 (t_{w,bm})^2 \left[1 + \left(\frac{4 l_b}{d_{bm}} - 0.2 \right) \left(\frac{t_{w,bm}}{t_{f,bm}} \right)^{1.5} \right] \sqrt{\frac{E F_{y,bm} t_{f,bm}}{t_{w,bm}}} Q_f$$\phi R_{n_wlc} = (0.75) \times 0.4 \times ((0.355 \text{ in}))^2 \times \left[1 + \left(\frac{4 \times (7.5 \text{ in})}{(18 \text{ in})} - 0.2 \right) \times \left(\frac{(0.355 \text{ in})}{(0.57 \text{ in})} \right)^{1.5} \right] \times \sqrt{\frac{(29000 \text{ ksi}) \times (50 \text{ ksi}) \times (0.57 \text{ in})}{(0.355 \text{ in})}} \times (1)$$\phi R_{n_wlc} = 99.274 \text{ kip}$ Calculate the web local yielding capacity of the beam. $\phi = 1$ - Web Local Yielding Resistance Factor $F_{y,bm} = 50$ ksi - Beam Yield Stress $t_{w,bm} = 0.355$ in - Beam Web Thickness $k_{des_lc} = 0.972$ in - Beam kdes $l_b = 7.5$ in - Bearing Length: Half Gusset Plate Width along Beam ϕR_{n_wly} - Design Web Local Yielding Capacity of Support $\phi R_{n_wly} = \phi F_{y,bm} t_{w,bm} (2.5 k_{des_lc} + l_b)$											

AISC 360-16 Chapter J10	$\phi R_{n_{wly}} = (1) \times (50 \text{ ksi}) \times (0.355 \text{ in}) \times (2.5 \times (0.972 \text{ in}) + (7.5 \text{ in}))$$\phi R_{n_{wly}} = 176.26 \text{ kip}$ Calculate for the governing design capacity of the beam. ϕR_n - Governing Design Capacity of the Support $\phi R_n = \min(\phi R_{n_{wlc}}, \phi R_{n_{wly}})$$\phi R_n = \min((99.274 \text{ kip}), (176.26 \text{ kip}))$$\phi R_n = 99.274 \text{ kip}$ $V_{b,at,beam} = 7.1059 \text{ kip}$ - Required Load: Half total Vb Result: Demand over Capacity Ratio $DCR = \frac{V_{b,at,beam}}{\phi R_n} = \frac{(7.1059 \text{ kip})}{(99.274 \text{ kip})} = 0.071579$	PASS																				
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	Upper Gusset Plate Connection to Column AISC 360-16 LRFD	

REFERENCES	CALCULATIONS	RESULTS																																													
	Check No. 1: Connection Detailing Limitations at Gusset Side <table><thead><tr><th>Detailing Limitations</th><th>Limit Value (in)</th><th>Actual Value (in)</th><th>DCR</th><th>Result</th></tr></thead><tbody><tr><td>Minimum Bolt Row Spacing</td><td>2.000</td><td>3.000</td><td>0.667</td><td>PASS</td></tr><tr><td>Maximum Bolt Row Spacing</td><td>6.000</td><td>3.000</td><td>0.500</td><td>PASS</td></tr><tr><td>Double Angle Minimum Vertical Edge Distance</td><td>1.000</td><td>1.500</td><td>0.667</td><td>PASS</td></tr><tr><td>Double Angle Minimum Horizontal Edge Distance</td><td>1.125</td><td>2.000</td><td>0.563</td><td>PASS</td></tr><tr><td>Gusset Plate Minimum Horizontal Edge Distance</td><td>1.000</td><td>2.000</td><td>0.500</td><td>PASS</td></tr><tr><td>Minimum Connection Depth</td><td>6.750</td><td>12.000</td><td>0.563</td><td>PASS</td></tr><tr><td>Maximum Connection Depth</td><td>12.000</td><td>12.000</td><td>1.000</td><td>PASS</td></tr><tr><td>Minimum Bolt Gage at Double Angle</td><td>1.750</td><td>3.000</td><td>0.583</td><td>PASS</td></tr></tbody></table> Result: Demand over Capacity Ratio $DCR = \frac{d}{c} = \frac{(12)}{(12)} = 1$	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	Double Angle Minimum Vertical Edge Distance	1.000	1.500	0.667	PASS	Double Angle Minimum Horizontal Edge Distance	1.125	2.000	0.563	PASS	Gusset Plate Minimum Horizontal Edge Distance	1.000	2.000	0.500	PASS	Minimum Connection Depth	6.750	12.000	0.563	PASS	Maximum Connection Depth	12.000	12.000	1.000	PASS	Minimum Bolt Gage at Double Angle	1.750	3.000	0.583	PASS	PASS
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AISC 15th Edition Table 10-9 AISC 360-16 Chapter J3.8 Eq. (J3-4) AISC 15th Ed. Page 7-6 to 7-8	Check No. 2: Design Capacity of the Bolt Group in Shear at Gusset Side Design Loads: Loading Angle from Vertical in Degrees $\theta = \tan^{-1}\left(\frac{H_c}{V_c}\right) = \tan^{-1}\left(\frac{(4.714 \text{ kip})}{(10.607 \text{ kip})}\right) = 23.962$ Resultant Load $R = \sqrt{(H_c)^2 + (V_c)^2} = \sqrt{((4.714 \text{ kip}))^2 + ((10.607 \text{ kip}))^2} = 11.607 \text{ kip}$ Calculate the C-coefficient of the bolt group. $e = 3 \text{ in}$ - Design Bolt Group Eccentricity $\phi R_n = 16.136 \text{ kip}$ - Design Bolt Shear Capacity 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. <table><thead><tr><th>Bolt No.</th><th>x-coordinate (in)</th><th>y-coordinate (in)</th></tr></thead><tbody><tr><td>1</td><td>0.00</td><td>0.00</td></tr><tr><td>2</td><td>0.00</td><td>3.00</td></tr><tr><td>3</td><td>0.00</td><td>6.00</td></tr><tr><td>4</td><td>0.00</td><td>9.00</td></tr></tbody></table> 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. $x_{ic} = -2.624 \text{ in}$ - Location of I.C. along the X-axis $y_{ic} = 6.0743 \text{ in}$ - Location of I.C. along the Y-axis Using the AISC 15th Edition Equation (7-1), the corresponding reactions per bolt are listed below. <table><thead><tr><th>Bolt No.</th><th>Ru (kip)</th><th>Rux (kip)</th><th>Ruy (kip)</th><th>Mn (kip-in)</th></tr></thead><tbody><tr><td>1</td><td>15.84</td><td>14.56</td><td>6.24</td><td>104.41</td></tr><tr><td>2</td><td>14.98</td><td>11.42</td><td>9.70</td><td>60.13</td></tr><tr><td>3</td><td>13.66</td><td>0.31</td><td>13.65</td><td>35.49</td></tr><tr><td>4</td><td>14.93</td><td>-11.19</td><td>9.88</td><td>58.58</td></tr></tbody></table> $C = 2.6192$ - Calculated Bolt Group Coefficient $C' = 11.256 \text{ in}$ - Calculated Bolt Group Coefficient for Moment-only Case Calculate the design shear capacity of the bolt group. $\phi = 0.85$ - Bolt Shear Resistance Factor $d_b = 0.75 \text{ in}$ - Bolt Diameter $C = 2.6192$ - Calculated Bolt Group Coefficient $\mu = 0.3$ - Mean Slip Coefficient $N_s = 2$ - Number of Slip Planes $h_f = 1$ - Filler Factor for SC Bolts $T_b = 28 \text{ kip}$ - Minimum Bolt Pretension ϕR_n - Design Bolt Shear Capacity $\phi R_n = \phi \mu D_u h_f T_b N_s C$$\phi R_n = (0.85) \times (0.3) \times (1.13) \times (1) \times (28 \text{ kip}) \times (2) \times (2.6192)$$\phi R_n = 42.265 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{R}{\phi R_n} = \frac{(11.607 \text{ kip})}{(42.265 \text{ kip})} = 0.27463$	Bolt No.	x-coordinate (in)	y-coordinate (in)	1	0.00	0.00	2	0.00	3.00	3	0.00	6.00	4	0.00	9.00	Bolt No.	Ru (kip)	Rux (kip)	Ruy (kip)	Mn (kip-in)	1	15.84	14.56	6.24	104.41	2	14.98	11.42	9.70	60.13	3	13.66	0.31	13.65	35.49	4	14.93	-11.19	9.88	58.58	PASS					
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AISC 360-16 Chapter J3.10 Eq. (J3-6a)	Check No. 3: Design Capacity of the Bolt Group in Bearing and Tear-out on Double Angle at Gusset Side Calculate the bolt bearing capacity of the double angle. $d_b = 0.75 \text{ in}$ - Bolt Diameter $t_{2L,at,col,u,br} = 0.5 \text{ in}$ - Upper Brace Double Angle Thickness $F_{u,at,col,u,br} = 58 \text{ ksi}$ - Upper Brace Double Angle Tensile Stress $C = 2.6192$ - Calculated Bolt Group Coefficient $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n,bearing}$ - Design Bolt Bearing Capacity of Section $\phi R_{n,bearing} = \phi 2.4 d_b [2(t_{2L,at,col,u,br})] F_{u,at,col,u,br} C$$\phi R_{n,bearing} = (0.75) \times 2.4 \times (0.75 \text{ in}) \times (1 \text{ in}) \times (58 \text{ ksi}) \times (2.6192)$																																														

AISC 360-16 Chapter J3.10 Eq. (J3-6c) AISC 360-16 Chapter J3.10 Eq. (J3-6a) AISC 360-16 Chapter J3.10 Eq. (J3-6c) AISC 360-16 Chapter J3.10	$\phi R_{n_bearing} = 205.09 \text{ kip}$ Calculate the clear distance of outer bolts on double angle. $L_{ev} = 1.5 \text{ in}$ - Vertical Edge Distance at Double Angle $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Double Angle l_{c1} - Clear Distance at First Bolt Row $l_{c1} = L_{ev} - \frac{d_h}{2}$ $l_{c1} = (1.5 \text{ in}) - \frac{(0.8125 \text{ in})}{2}$ $l_{c1} = 1.0938 \text{ in}$ Calculate the clear distance of inner bolts on double angle. $s_r = 3 \text{ in}$ - Bolt Row Spacing $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Double Angle 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 double angle. $l_{c1} = 1.0938 \text{ in}$ - Clear Distance at First Bolt Row $l_{c2} = 2.1875 \text{ in}$ - Clear Distance at Rest of Bolts $t_{2L_at_col_u_br} = 0.5 \text{ in}$ - Upper Brace Double Angle Thickness $F_{u_at_col_u_br} = 58 \text{ ksi}$ - Upper Brace Double Angle Tensile Stress $C = 2.6192$ - Calculated Bolt Group Coefficient $n_r = 4$ - Number of Bolt Rows $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_tearout}$ - Design Bolt Tear-out Capacity of Section $\phi R_{n_tearout} = \phi [1.2 l_{c1} [2(t_{2L_at_col_u_br})] F_{u_at_col_u_br} + 1.2 l_{c2} [2(t_{2L_at_col_u_br})] F_{u_at_col_u_br} (n_r - 1)] \left(\frac{C}{n_r} \right)$ $\phi R_{n_tearout} = (0.75) \times [1.2 \times (1.0938 \text{ in}) \times (1 \text{ in}) \times (58 \text{ ksi}) + 1.2 \times (2.1875 \text{ in}) \times (1 \text{ in}) \times (58 \text{ ksi}) \times ((4) - 1)] \times \left(\frac{(2.6192)}{(4)} \right)$ $\phi R_{n_tearout} = 261.7 \text{ kip}$ Determine the governing bearing and tear-out capacity of the bolt group on double angle. $\phi R_{n_bearing} = 205.09 \text{ kip}$ - Design bolt bearing capacity of double angle $\phi R_{n_tearout} = 261.7 \text{ kip}$ - Design bolt tear-out capacity of double angle ϕR_n - Governing Design Capacity $\phi R_n = \min(\phi R_{n_bearing}, \phi R_{n_tearout})$ $\phi R_n = \min((205.09 \text{ kip}), (261.7 \text{ kip}))$ $\phi R_n = 205.09 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{V_c}{\phi R_n} = \frac{(10.607 \text{ kip})}{(205.09 \text{ kip})} = 0.051718$	PASS
AISC 360-16 Chapter J3.10 Eq. (J3-6a)	Check No. 4: Design Capacity of the Bolt Group in Bearing and Tear-out on Double Angle at Gusset Side due to Axial Load Calculate the bolt bearing capacity of the double angle. $d_b = 0.75 \text{ in}$ - Bolt Diameter $t_{2L_at_col_u_br} = 0.5 \text{ in}$ - Upper Brace Double Angle Thickness $F_{u_at_col_u_br} = 58 \text{ ksi}$ - Upper Brace Double Angle Tensile Stress $C = 2.6192$ - Calculated Bolt Group Coefficient $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_bearing}$ - Design Bolt Bearing Capacity of Section $\phi R_{n_bearing} = \phi 2.4 d_b [2(t_{2L_at_col_u_br})] F_{u_at_col_u_br} C$ $\phi R_{n_bearing} = (0.75) \times 2.4 \times (0.75 \text{ in}) \times (1 \text{ in}) \times (58 \text{ ksi}) \times (2.6192)$ $\phi R_{n_bearing} = 205.09 \text{ kip}$ Calculate the clear distance of outer bolts on double angle. $L_{eh} = 2 \text{ in}$ - Horizontal Edge Distance at Double Angle $d_h = 1 \text{ in}$ - Horizontal Bolt Hole Dimension at Double Angle l_{c1} - Clear Distance at First Bolt Row $l_{c1} = L_{eh} - \frac{d_h}{2}$ $l_{c1} = (2 \text{ in}) - \frac{(1 \text{ in})}{2}$	

AISC 360-16 Chapter J3.10 Eq. (J3-6c) AISC 360-16 Chapter J3.10 Eq. (J3-6a) AISC 360-16 Chapter J3.10 Eq. (J3-6c) AISC 360-16 Chapter J3.10	$l_{c1} = 1.5 \text{ in}$ Calculate the clear distance of inner bolts on double angle. $s_c = 0 \text{ in}$ - Bolt Column Spacing $d_h = 1 \text{ in}$ - Horizontal Bolt Hole Dimension at Double Angle l_{c2} - Clear Distance at Rest of Bolts $l_{c2} = s_c - d_h$ $l_{c2} = (0 \text{ in}) - (1 \text{ in})$ $l_{c2} = -1 \text{ in}$ Calculate the bolt tear-out capacity of the double angle. $l_{c1} = 1.5 \text{ in}$ - Clear Distance at First Bolt Row $l_{c2} = -1 \text{ in}$ - Clear Distance at Rest of Bolts $t_{2L,at,col,u,br} = 0.5 \text{ in}$ - Upper Brace Double Angle Thickness $F_{u,at,col,u,br} = 58 \text{ ksi}$ - Upper Brace Double Angle Tensile Stress $C = 2.6192$ - Calculated Bolt Group Coefficient $n_c = 1$ - Number of Bolt Columns $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_tearout}$ - Design Bolt Tear-out Capacity of Section $\phi R_{n_tearout} = \phi [1.2 l_{c1} [2(t_{2L,at,col,u,br})] F_{u,at,col,u,br} + 1.2 l_{c2} [2(t_{2L,at,col,u,br})] F_{u,at,col,u,br} (n_c - 1)] \left(\frac{C}{n_c} \right)$ $\phi R_{n_tearout} = (0.75) \times [1.2 \times (1.5 \text{ in}) \times (1 \text{ in}) \times (58 \text{ ksi}) + 1.2 \times (-1 \text{ in}) \times (1 \text{ in}) \times (58 \text{ ksi}) \times ((1) - 1)] \times \left(\frac{(2.6192)}{(1)} \right)$ $\phi R_{n_tearout} = 205.09 \text{ kip}$ Determine the governing bearing and tear-out capacity of the bolt group on double angle. $\phi R_{n_bearing} = 205.09 \text{ kip}$ - Design bolt bearing capacity of double angle $\phi R_{n_tearout} = 205.09 \text{ kip}$ - Design bolt tear-out capacity of double angle ϕR_n - Governing Design Capacity $\phi R_n = \min(\phi R_{n_bearing}, \phi R_{n_tearout})$ $\phi R_n = \min((205.09 \text{ kip}), (205.09 \text{ kip}))$ $\phi R_n = 205.09 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{H_c}{\phi R_n} = \frac{(4.714 \text{ kip})}{(205.09 \text{ kip})} = 0.022986$	PASS
	Check No. 5: Shear and Axial Interaction of Bolt Bearing/Tearout in Double Angle Calculate the shear and axial interaction of bolt bearing/tearout capacities in the double angle. $V_c = 10.607 \text{ kip}$ - Vertical Shear at Gusset to Column $H_c = 4.714 \text{ kip}$ - Axial Load at Gusset to Column Interaction - Shear and Axial Interaction $Interaction = \sqrt{\left(\frac{V_c}{\phi R_n} \right)^2 + \left(\frac{H_c}{\phi R_n} \right)^2}$ $Interaction = \sqrt{\left(\frac{(10.607 \text{ kip})}{(205.09 \text{ kip})} \right)^2 + \left(\frac{(4.714 \text{ kip})}{(205.09 \text{ kip})} \right)^2}$ $Interaction = 0.056596$ Result: Demand over Capacity Ratio $DCR = \frac{Interaction}{1.0} = \frac{(0.056596)}{(1)} = 0.056596$	PASS
AISC 360-16 Chapter J3.10 Eq. (J3-6a)	Check No. 6: Design Capacity of the Bolt Group in Bearing and Tear-out on Gusset Plate Calculate the bolt bearing capacity. $d_b = 0.75 \text{ in}$ - Bolt Diameter $t_{gp,u,br} = 1 \text{ in}$ - Upper Brace Gusset Plate Thickness $F_{u,u,br} = 65 \text{ ksi}$ - Upper Brace Gusset Plate Tensile Stress $C = 2.6192$ - Calculated Bolt Group Coefficient $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_bearing}$ - Design Bolt Bearing Capacity of Section $\phi R_{n_bearing} = \phi 2.4 d_b t_{gp,u,br} F_{u,u,br} C$ $\phi R_{n_bearing} = (0.75) \times 2.4 \times (0.75 \text{ in}) \times (1 \text{ in}) \times (65 \text{ ksi}) \times (2.6192)$ $\phi R_{n_bearing} = 229.84 \text{ kip}$ Calculate the clear distance of inner bolts on member. $s_r = 3 \text{ in}$ - Bolt Row Spacing $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Gusset Plate l_{c2} - Clear Distance at Rest of Bolts $l_{c2} = s_r - d_h$	

AISC 360-16 Chapter J3.10 Eq. (J3-6C) AISC 360-16 Chapter J3.10 Eq. (J3-6a) AISC 360-16 Chapter J3.10 Eq. (J3-6c) AISC 360-16 Chapter J3.10	$l_{c2} = (3 \text{ in}) - (0.8125 \text{ in})$ $l_{c2} = 2.1875 \text{ in}$ Calculate the bolt tear-out capacity. $l_{c2} = 2.1875 \text{ in}$ - Clear Distance at Rest of Bolts $t_{gp,u,br} = 1 \text{ in}$ - Upper Brace Gusset Plate Thickness $F_{u,u,br} = 65 \text{ ksi}$ - Upper Brace Gusset Plate Tensile Stress $C = 2.6192$ - Calculated Bolt Group Coefficient $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_tearout}$ - Design Bolt Tear-out Capacity of Section $\phi R_{n_tearout} = \phi 1.2 l_{c2} t_{gp,u,br} F_{u,u,br} C$ $\phi R_{n_tearout} = (0.75) \times 1.2 \times (2.1875 \text{ in}) \times (1 \text{ in}) \times (65 \text{ ksi}) \times (2.6192)$ $\phi R_{n_tearout} = 335.18 \text{ kip}$ Determine the governing bearing and tear-out capacity of the bolt group. $\phi R_{n_bearing} = 229.84 \text{ kip}$ - Design bolt bearing capacity of member $\phi R_{n_tearout} = 335.18 \text{ kip}$ - Design bolt tear-out capacity of member ϕR_n - Governing Design Capacity $\phi R_n = \min(\phi R_{n_bearing}, \phi R_{n_tearout})$ $\phi R_n = \min((229.84 \text{ kip}), (335.18 \text{ kip}))$ $\phi R_n = 229.84 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{V_c}{\phi R_n} = \frac{(10.607 \text{ kip})}{(229.84 \text{ kip})} = 0.046148$	PASS
AISC 360-16 Chapter J3.10 Eq. (J3-6a) AISC 360-16 Chapter J3.10 Eq. (J3-6c)	Check No. 7: Design Capacity of the Bolt Group in Bearing and Tear-out on Gusset Plate due to Axial Load Calculate the bolt bearing capacity. $d_b = 0.75 \text{ in}$ - Bolt Diameter $t_{gp,u,br} = 1 \text{ in}$ - Upper Brace Gusset Plate Thickness $F_{u,u,br} = 65 \text{ ksi}$ - Upper Brace Gusset Plate Tensile Stress $C = 2.6192$ - Calculated Bolt Group Coefficient $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_bearing}$ - Design Bolt Bearing Capacity of Section $\phi R_{n_bearing} = \phi 2.4 d_b t_{gp,u,br} F_{u,u,br} C$ $\phi R_{n_bearing} = (0.75) \times 2.4 \times (0.75 \text{ in}) \times (1 \text{ in}) \times (65 \text{ ksi}) \times (2.6192)$ $\phi R_{n_bearing} = 229.84 \text{ kip}$ Calculate the clear distance of outer bolts. $L_{eh} = 2 \text{ in}$ - Horizontal Edge Distance at Gusset Plate $d_h = 0.8125 \text{ in}$ - Horizontal Bolt Hole Dimension at Gusset Plate l_{c1} - Clear Distance at First Bolt Row $l_{c1} = L_{eh} - \frac{d_h}{2}$ $l_{c1} = (2 \text{ in}) - \frac{(0.8125 \text{ in})}{2}$ $l_{c1} = 1.5938 \text{ in}$ Calculate the clear distance of inner bolts. $s_c = 0 \text{ in}$ - Bolt Column Spacing $d_h = 0.8125 \text{ in}$ - Horizontal Bolt Hole Dimension at Gusset Plate l_{c2} - Clear Distance at Rest of Bolts $l_{c2} = s_c - d_h$ $l_{c2} = (0 \text{ in}) - (0.8125 \text{ in})$ $l_{c2} = -0.8125 \text{ in}$ Calculate the bolt tear-out capacity. $l_{c1} = 1.5938 \text{ in}$ - Clear Distance at First Bolt Row $l_{c2} = -0.8125 \text{ in}$ - Clear Distance at Rest of Bolts $t_{gp,u,br} = 1 \text{ in}$ - Upper Brace Gusset Plate Thickness $F_{u,u,br} = 65 \text{ ksi}$ - Upper Brace Gusset Plate Tensile Stress $C = 2.6192$ - Calculated Bolt Group Coefficient $n_c = 1$ - Number of Bolt Columns $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_tearout}$ - Design Bolt Tear-out Capacity of Section $\phi R_{n_tearout} = \phi [1.2 l_{c1} t_{gp,u,br} F_{u,u,br} + 1.2 l_{c2} t_{gp,u,br} F_{u,u,br} (n_c - 1)] \left(\frac{C}{n_c} \right)$	

AISC 360-16 Chapter J3.10 Eq. (J3-6a) AISC 360-16 Chapter J3.10 Eq. (J3-6c) AISC 360-16 Chapter J3.10	$\phi R_{n_tearout} = (0.75) \times [1.2 \times (1.5938 \text{ in}) \times (1 \text{ in}) \times (65 \text{ ksi}) + 1.2 \times (-0.8125 \text{ in}) \times (1 \text{ in}) \times (65 \text{ ksi}) \times ((1) - 1)] \times \left(\frac{(2.6192)}{(1)} \right)$ $\phi R_{n_tearout} = 244.2 \text{ kip}$ Determine the governing bearing and tear-out capacity of the bolt group. $\phi R_{n_bearing} = 229.84 \text{ kip}$ - Design bolt bearing capacity of member $\phi R_{n_tearout} = 244.2 \text{ kip}$ - Design bolt tear-out capacity of member ϕR_n - Governing Design Capacity $\phi R_n = \min(\phi R_{n_bearing}, \phi R_{n_tearout})$ $\phi R_n = \min((229.84 \text{ kip}), (244.2 \text{ kip}))$ $\phi R_n = 229.84 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{H_c}{\phi R_n} = \frac{(4.714 \text{ kip})}{(229.84 \text{ kip})} = 0.02051$	PASS
	Check No. 8: Shear and Axial Interaction of Bolt Bearing/Tearout on Gusset Plate Calculate the shear and axial interaction of bolt bearing/tearout capacities. $V_c = 10.607 \text{ kip}$ - Vertical Shear at Gusset to Column $H_c = 4.714 \text{ kip}$ - Axial Load at Gusset to Column Interaction - Shear and Axial Interaction $Interaction = \sqrt{\left(\frac{V_c}{\phi R_n} \right)^2 + \left(\frac{H_c}{\phi R_n} \right)^2}$ $Interaction = \sqrt{\left(\frac{(10.607 \text{ kip})}{(229.84 \text{ kip})} \right)^2 + \left(\frac{(4.714 \text{ kip})}{(229.84 \text{ kip})} \right)^2}$ $Interaction = 0.050501$ Result: Demand over Capacity Ratio $DCR = \frac{Interaction}{1.0} = \frac{(0.050501)}{(1)} = 0.050501$	PASS
	Check No. 9: Design Capacity of Double Angle in Block Shear at Gusset Side Calculate the net area of the double angle subject to tension. $t_{2L,at,col,u,br} = 0.5 \text{ in}$ - Upper Brace Double Angle Thickness $n_c = 1$ - Number of Bolt Columns $s_c = 0 \text{ in}$ - Bolt Column Spacing $L_{eh} = 2 \text{ in}$ - Horizontal Edge Distance at Double Angle $d_h = 1 \text{ in}$ - Horizontal Bolt Hole Dimension at Double Angle A_{nt} - Net Area Subject to Tension (L-pattern) $A_{nt} = [2(t_{2L,at,col,u,br})] [(n_c - 1) s_c + L_{eh} - (n_c - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nt} = (1 \text{ in}) \times [((1) - 1) \times (0 \text{ in}) + (2 \text{ in}) - ((1) - 0.5) \times ((1 \text{ in}) + (0.0625 \text{ in}))]$ $A_{nt} = 1.4688 \text{ in}^2$ Calculate the gross area of the double angle subject to shear. $t_{2L,at,col,u,br} = 0.5 \text{ in}$ - Upper Brace Double Angle Thickness $L_{ev} = 1.5 \text{ in}$ - Vertical Edge Distance at Double Angle $n_r = 4$ - Number of Bolt Rows $s_r = 3 \text{ in}$ - Bolt Row Spacing A_{gv} - Gross Area Subject to Shear (L-pattern) $A_{gv} = [2(t_{2L,at,col,u,br})] [L_{ev} + (n_r - 1) s_r]$ $A_{gv} = (1 \text{ in}) \times [(1.5 \text{ in}) + ((4) - 1) \times (3 \text{ in})]$ $A_{gv} = 10.5 \text{ in}^2$ Calculate the net area of the double angle subject to shear. $t_{2L,at,col,u,br} = 0.5 \text{ in}$ - Upper Brace Double Angle Thickness $L_{2L,at,col,u,br} = 12 \text{ in}$ - Upper Brace Double Angle Length $L_{ev} = 1.5 \text{ in}$ - Vertical Edge Distance at Double Angle $n_r = 4$ - Number of Bolt Rows $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Double Angle A_{nv} - Net Area Subject to Shear (L-pattern) $A_{nv} = [2(t_{2L,at,col,u,br})] [L_{2L,at,col,u,br} - L_{ev} - (n_r - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nv} = (1 \text{ in}) \times [(12 \text{ in}) - (1.5 \text{ in}) - ((4) - 0.5) \times ((0.8125 \text{ in}) + (0.0625 \text{ in}))]$ $A_{nv} = 7.4375 \text{ in}^2$ Calculate the design block shear capacity of the double angle. $F_{u,at,col,u,br} = 58 \text{ ksi}$ - Upper Brace Double Angle Tensile Stress $A_{nv} = 7.4375 \text{ in}^2$ - Net Area Subject to Shear (L-pattern) $U_{bs} = 1$ - Uniformity factor $A_{nt} = 1.4688 \text{ in}^2$ - Net Area Subject to Tension (L-pattern)	

AISC 360-16 Chapter J4.3 Eq. (J4-5)	$F_{y,at,col,u,br} = 36 \text{ ksi}$ - Upper Brace Double Angle Yield Stress $A_{gv} = 10.5 \text{ in}^2$ - Gross Area Subject to Shear (L-pattern) $\phi = 0.75$ - Block Shear Resistance Factor ϕR_n - Design Block Shear Capacity of Section $\phi R_n = \phi (0.6 F_{u,at,col,u,br} A_{nv} + U_{bs} F_{u,at,col,u,br} A_{nt} \leq 0.6 F_{y,at,col,u,br} A_{gv} + U_{bs} F_{u,at,col,u,br} A_{nt})$ $\phi R_n = (0.75) \times (0.6 \times (58 \text{ ksi}) \times (7.4375 \text{ in}^2) + (1) \times (58 \text{ ksi}) \times (1.4688 \text{ in}^2)) \leq 0.6 \times (36 \text{ ksi}) \times (10.5 \text{ in}^2) + (1) \times (58 \text{ ksi}) \times (1.4688 \text{ in}^2)$ $\phi R_n = 233.99 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{V_c}{\phi R_n} = \frac{(10.607 \text{ kip})}{(233.99 \text{ kip})} = 0.045329$	PASS
AISC 360-16 Chapter J4.3 Eq. (J4-5)	Check No. 10: Design Capacity of Double Angle in Block Shear at Gusset Side due to Axial Load Calculate the net area of the double angle subject to tension. $t_{2L,at,col,u,br} = 0.5 \text{ in}$ - Upper Brace Double Angle Thickness $n_r = 4$ - Number of Bolt Rows $s_r = 3 \text{ in}$ - Bolt Row Spacing $L_{ev} = 1.5 \text{ in}$ - Vertical Edge Distance at Double Angle $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Double Angle A_{nt} - Net Area Subject to Tension (L-pattern) $A_{nt} = [2(t_{2L,at,col,u,br})] [(n_r - 1) s_r + L_{ev} - (n_r - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nt} = (1 \text{ in}) \times [((4) - 1) \times (3 \text{ in}) + (1.5 \text{ in}) - ((4) - 0.5) \times ((0.8125 \text{ in}) + (0.0625 \text{ in}))]$ $A_{nt} = 7.4375 \text{ in}^2$ Calculate the gross area of the double angle subject to shear. $t_{2L,at,col,u,br} = 0.5 \text{ in}$ - Upper Brace Double Angle Thickness $L_{eh} = 2 \text{ in}$ - Horizontal Edge Distance at Double Angle $n_c = 1$ - Number of Bolt Columns $s_c = 0 \text{ in}$ - Bolt Column Spacing A_{gv} - Gross Area Subject to Shear (L-pattern) $A_{gv} = [2(t_{2L,at,col,u,br})] [L_{eh} + (n_c - 1) s_c]$ $A_{gv} = (1 \text{ in}) \times [(2 \text{ in}) + ((1) - 1) \times (0 \text{ in})]$ $A_{gv} = 2 \text{ in}^2$ Calculate the net area of the double angle subject to shear. $t_{2L,at,col,u,br} = 0.5 \text{ in}$ - Upper Brace Double Angle Thickness $L_{NSL,u,br} = 5 \text{ in}$ - Upper Brace Double Angle Leg at Beam Side $g_a = 3 \text{ in}$ - Double Angle Gage $n_c = 1$ - Number of Bolt Columns $d_h = 1 \text{ in}$ - Horizontal Bolt Hole Dimension at Double Angle A_{nv} - Net Area Subject to Shear (L-pattern) $A_{nv} = [2(t_{2L,at,col,u,br})] [L_{NSL,u,br} - g_a - (n_c - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nv} = (1 \text{ in}) \times [(5 \text{ in}) - (3 \text{ in}) - ((1) - 0.5) \times ((1 \text{ in}) + (0.0625 \text{ in}))]$ $A_{nv} = 1.4688 \text{ in}^2$ Calculate the design block shear capacity of the double angle. $F_{u,at,col,u,br} = 58 \text{ ksi}$ - Upper Brace Double Angle Tensile Stress $A_{nv} = 1.4688 \text{ in}^2$ - Net Area Subject to Shear (L-pattern) $U_{bs} = 1$ - Uniformity factor $A_{nt} = 7.4375 \text{ in}^2$ - Net Area Subject to Tension (L-pattern) $F_{y,at,col,u,br} = 36 \text{ ksi}$ - Upper Brace Double Angle Yield Stress $A_{gv} = 2 \text{ in}^2$ - Gross Area Subject to Shear (L-pattern) $\phi = 0.75$ - Block Shear Resistance Factor ϕR_n - Design Block Shear Capacity of Section $\phi R_n = \phi (0.6 F_{u,at,col,u,br} A_{nv} + U_{bs} F_{u,at,col,u,br} A_{nt} \leq 0.6 F_{y,at,col,u,br} A_{gv} + U_{bs} F_{u,at,col,u,br} A_{nt})$ $\phi R_n = (0.75) \times (0.6 \times (58 \text{ ksi}) \times (1.4688 \text{ in}^2) + (1) \times (58 \text{ ksi}) \times (7.4375 \text{ in}^2)) \leq 0.6 \times (36 \text{ ksi}) \times (2 \text{ in}^2) + (1) \times (58 \text{ ksi}) \times (7.4375 \text{ in}^2)$ $\phi R_n = 355.93 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{H_{c,T,upper}}{\phi R_n} = \frac{(4.714 \text{ kip})}{(355.93 \text{ kip})} = 0.013244$	PASS
	Check No. 11: Shear and Axial Interaction of Block Shear in Double Angle Calculate the shear and axial interaction of block shear capacities in the double angle. $V_c = 10.607 \text{ kip}$ - Vertical Shear at Gusset to Column $H_{c,T,upper} = 4.714 \text{ kip}$ - Tension Load at Gusset to Column $Interaction$ - Shear and Axial Interaction $Interaction = \sqrt{\left(\frac{V_c}{\phi R_n}\right)^2 + \left(\frac{H_{c,T,upper}}{\phi R_n}\right)^2}$	

	$Interaction = \sqrt{\left(\frac{(10.607 \text{ kip})}{(233.99 \text{ kip})}\right)^2 + \left(\frac{(4.714 \text{ kip})}{(355.93 \text{ kip})}\right)^2}$ $Interaction = 0.047224$ Result: Demand over Capacity Ratio $DCR = \frac{Interaction}{1.0} = \frac{(0.047224)}{(1)} = 0.047224$	PASS
AISC 360-16 Chapter J4.2 Eq. (J4-3) AISC 360-16 Chapter J4.2 Eq. (J4-4) AISC 360-16 Chapter J4.2 Eq. (J4-3) AISC 360-16 Chapter J4.2 Eq. (J4-4) AISC 360-16 Chapter J4.2	Check No. 12: Design Capacity of Double Angle in Shear at Gusset Side Calculate the gross area of double angle subject to yielding. $L_{2L,at,col,u,br} = 12 \text{ in}$ - Upper Brace Double Angle Length $t_{2L,at,col,u,br} = 0.5 \text{ in}$ - Upper Brace Double Angle Thickness A_{gv} - Section Gross Area $A_{gv} = L_{2L,at,col,u,br} [2(t_{2L,at,col,u,br})]$ $A_{gv} = (12 \text{ in}) \times (1 \text{ in})$ $A_{gv} = 12 \text{ in}^2$ Calculate the shear yielding capacity of the double angle. $F_{y,at,col,u,br} = 36 \text{ ksi}$ - Upper Brace Double Angle Yield Stress $A_{gv} = 12 \text{ in}^2$ - Section Gross Area $\phi = 1$ - Shear Yielding Resistance Factor $\phi R_{n,sy}$ - Design Shear Yielding Capacity of Section $\phi R_{n,sy} = \phi 0.6 F_{y,at,col,u,br} A_{gv}$ $\phi R_{n,sy} = (1) \times 0.6 \times (36 \text{ ksi}) \times (12 \text{ in}^2)$ $\phi R_{n,sy} = 259.2 \text{ kip}$ Calculate the net area of double angle subject to rupture. $t_{2L,at,col,u,br} = 0.5 \text{ in}$ - Upper Brace Double Angle Thickness $L_{2L,at,col,u,br} = 12 \text{ in}$ - Upper Brace Double Angle Length $n_r = 4$ - Number of Bolt Rows $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Double Angle A_{nv} - Section Net Area $A_{nv} = [2(t_{2L,at,col,u,br})] [L_{2L,at,col,u,br} - n_r (d_h + 0.0625 \text{ in})]$ $A_{nv} = (1 \text{ in}) \times [(12 \text{ in}) - (4) \times ((0.8125 \text{ in}) + (0.0625 \text{ in}))]$ $A_{nv} = 8.5 \text{ in}^2$ Calculate the shear rupture capacity of the double angle. $F_{u,at,col,u,br} = 58 \text{ ksi}$ - Upper Brace Double Angle Tensile Stress $A_{nv} = 8.5 \text{ in}^2$ - Section Net Area $\phi = 0.75$ - Shear Rupture Resistance Factor $\phi R_{n,sr}$ - Design Shear Rupture Capacity of Section $\phi R_{n,sr} = \phi 0.6 F_{u,at,col,u,br} A_{nv}$ $\phi R_{n,sr} = (0.75) \times 0.6 \times (58 \text{ ksi}) \times (8.5 \text{ in}^2)$ $\phi R_{n,sr} = 221.85 \text{ kip}$ Determine the governing shear capacity of the double angle. $\phi R_{n,sy} = 259.2 \text{ kip}$ - Design shear yielding capacity of double angle $\phi R_{n,sr} = 221.85 \text{ kip}$ - Design shear rupture capacity of double angle ϕR_n - Governing Design Capacity $\phi R_n = \min(\phi R_{n,sy}, \phi R_{n,sr})$ $\phi R_n = \min((259.2 \text{ kip}), (221.85 \text{ kip}))$ $\phi R_n = 221.85 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{V_c}{\phi R_n} = \frac{(10.607 \text{ kip})}{(221.85 \text{ kip})} = 0.04781$	PASS
	Check No. 13: Design Capacity of Double Angle in Flexure Using Double Coped Procedure at Gusset Side Calculate and compare upper and lower limits. $E = 29000 \text{ ksi}$ - Modulus for Steel $F_{y,at,col,u,br} = 36 \text{ ksi}$ - Upper Brace Double Angle Yield Stress $L_{b,2L} = 3 \text{ in}$ - Unbraced Length at Double Angle $L_{2L,at,col,u,br} = 12 \text{ in}$ - Upper Brace Double Angle Length $t_{2L,at,col,u,br} = 0.5 \text{ in}$ - Upper Brace Double Angle Thickness Depth to Thickness Ratio	

AISC 15th Edition pg. (9-9) Eq. (9-15)	$\frac{L_b d}{t^2} = \frac{(3 \text{ in}) \times (12 \text{ in})}{(0.5 \text{ in})^2} = 144$ 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 $\frac{L_b d}{t^2} \leq \frac{1.9E}{F_y}$. Calculate the plastic section modulus of the double angle. $t_{2L,at,col,u,br} = 0.5 \text{ in}$ - Upper Brace Double Angle Thickness $L_{2L,at,col,u,br} = 12 \text{ in}$ - Upper Brace Double Angle Length Plastic Section Modulus $Z = \frac{t_{2L,at,col,u,br} (L_{2L,at,col,u,br})^2}{4} = \frac{(0.5 \text{ in}) \times ((12 \text{ in}))^2}{4} = 18 \text{ in}^3$ Calculate the elastic section modulus of the double angle. $t_{2L,at,col,u,br} = 0.5 \text{ in}$ - Upper Brace Double Angle Thickness $L_{2L,at,col,u,br} = 12 \text{ in}$ - Upper Brace Double Angle Length Elastic Section Modulus $S = \frac{t_{2L,at,col,u,br} (L_{2L,at,col,u,br})^3}{6} = \frac{(0.5 \text{ in}) \times ((12 \text{ in}))^3}{6} = 12 \text{ in}^3$ Calculate the lateral torsional buckling modification factor. $L_{b,2L} = 3 \text{ in}$ - Unbraced Length at Double Angle $L_{2L,at,col,u,br} = 12 \text{ in}$ - Upper Brace Double Angle Length $d_{c,2L} = 0 \text{ in}$ - Double Angle Cope Depth (Assumed 0) C_b - Lateral-Torsional Buckling Modification Factor $C_b = \left[3 + \ln \left(\frac{L_{b,2L}}{L_{2L,at,col,u,br}} \right) \right] \left(1 - \frac{d_{c,2L}}{L_{2L,at,col,u,br}} \right) \geq 1.84$ $C_b = \left[3 + \ln \left(\frac{(3 \text{ in})}{(12 \text{ in})} \right) \right] \times \left(1 - \frac{(0 \text{ in})}{(12 \text{ in})} \right) \geq 1.84$ $C_b = 1.84$	
AISC 15th Edition pg. (9-9) Eq. (9-15) AISC 360-16 Chapter F11.2 Eq. (F11-2)	Calculate the flexural capacity of the double angle. $C_b = 1.84$ - Lateral-Torsional Buckling Modification Factor $\frac{L_b d}{t^2} = 144$ - Depth to Thickness Ratio $F_{y,at,col,u,br} = 36 \text{ ksi}$ - Upper Brace Double Angle Yield Stress $E = 29000 \text{ ksi}$ - Modulus for Steel $S = 12 \text{ in}^3$ - Elastic Section Modulus $Z = 18 \text{ in}^3$ - Plastic Section Modulus $\phi = 0.9$ - Flexure Resistance Factor ϕM_n - Design Flexural Capacity of Coped Section $\phi M_n = \phi \left[C_b \left[1.52 - 0.274 \left(\frac{L_b d}{t^2} \right) \left(\frac{F_{y,at,col,u,br}}{E} \right) \right] F_{y,at,col,u,br} S \leq F_{y,at,col,u,br} Z \right] 2$ $\phi M_n = (0.9) \times \left[(1.84) \times \left[1.52 - 0.274 \times ((144)) \times \left(\frac{(36 \text{ ksi})}{(29000 \text{ ksi})} \right) \right] \times (36 \text{ ksi}) \times (12 \text{ in}^3) \leq (36 \text{ ksi}) \times (18 \text{ in}^3) \right] \times 2$ $\phi M_n = 97.2 \text{ kip} \cdot \text{ft}$	
AISC 360-16 Chapter F11.2 Eq. (F11-2)	Calculate the equivalent shear capacity of the double angle. $\phi M_n = 97.2 \text{ kip} \cdot \text{ft}$ - Design Flexural Capacity of Coped Section $L_{b,2L} = 3 \text{ in}$ - Unbraced Length at Double Angle ϕV_n - Design Capacity of Single Plate in Bending $\phi V_n = \frac{\phi M_n}{L_{b,2L}}$ $\phi V_n = \frac{(97.2 \text{ kip} \cdot \text{ft})}{(3 \text{ in})}$ $\phi V_n = 388.8 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{V_r}{\phi V_n} = \frac{(10.607 \text{ kip})}{(388.8 \text{ kip})} = 0.02728$	PASS
AISC 360-16 Chapter J4.2 Eq. (J4-3) AISC 360-16 Chapter F11.2 Eq. (F11-2) AISC 15th Edition pg. (10-90) Eq. (10-5)	Check No. 14: Shear and Moment Interaction of Double Angle at Gusset Side Calculate the shear and moment interaction of the double angle. $V_r = 10.607 \text{ kip}$ - Vertical Shear at Gusset to Column Moment at Component $M_r = V_r L_{b,2L} = (10.607 \text{ kip}) \times (3 \text{ in}) = 2.6517 \text{ kip} \cdot \text{ft}$ $\phi V_n = 259.2 \text{ kip}$ - Design shear yielding capacity of double angle $\phi M_n = 97.2 \text{ kip} \cdot \text{ft}$ - Design Flexural Capacity of Double Angle I_{nt} - Interaction for Shear Yielding/Buckling and Flexure $I_{nt} = \left(\frac{V_r}{\phi V_n} \right)^2 + \left(\frac{M_r}{\phi M_n} \right)^2$	

	$I_{nt} = \left(\frac{(10.607 \text{ kip})}{(259.2 \text{ kip})} \right)^2 + \left(\frac{(2.6517 \text{ kip}\cdot\text{ft})}{(97.2 \text{ kip}\cdot\text{ft})} \right)^2$ $I_{nt} = 0.0024187$ Result: Demand over Capacity Ratio $DCR = \frac{I_{nt}}{1.0} = \frac{(0.0024187)}{(1)} = 0.0024187$	PASS
AISC 360-16 Chapter J4.1 Eq. (J4-1) AISC 360-16 Chapter J4.1 Eq. (J4-2) AISC 360-16 Chapter J4.1 Eq. (J4-1) AISC 360-16 Chapter J4.1 Eq. (J4-2) AISC 360-16 Chapter J4.2	Check No. 15: Design Capacity of Double Angle in Tension Calculate the tensile yielding capacity of the double angle. $\phi = 0.9$ - Tensile Yielding Resistance Factor $F_{y,at,col,u,br} = 36 \text{ ksi}$ - Upper Brace Double Angle Yield Stress $t_{2L,at,col,u,br} = 0.5 \text{ in}$ - Upper Brace Double Angle Thickness $L_{2L,at,col,u,br} = 12 \text{ in}$ - Upper Brace Double Angle Length $\phi R_{n,ty}$ - Design Tension Yielding Capacity of Section $\phi R_{n,ty} = \phi F_{y,at,col,u,br} [2(t_{2L,at,col,u,br})] L_{2L,at,col,u,br}$ $\phi R_{n,ty} = (0.9) \times (36 \text{ ksi}) \times (1 \text{ in}) \times (12 \text{ in})$ $\phi R_{n,ty} = 388.8 \text{ kip}$ Calculate the tensile rupture capacity of the double angle. $\phi = 0.75$ - Tensile Rupture Resistance Factor $F_{u,at,col,u,br} = 58 \text{ ksi}$ - Upper Brace Double Angle Tensile Stress $t_{2L,at,col,u,br} = 0.5 \text{ in}$ - Upper Brace Double Angle Thickness $L_{2L,at,col,u,br} = 12 \text{ in}$ - Upper Brace Double Angle Length $n_r = 4$ - Number of Bolt Rows $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Double Angle $\phi R_{n,tr}$ - Design Tension Rupture Capacity of Section $\phi R_{n,tr} = \phi F_{u,at,col,u,br} [2(t_{2L,at,col,u,br})] [L_{2L,at,col,u,br} - n_r (d_h + 0.0625 \text{ in})]$ $\phi R_{n,tr} = (0.75) \times (58 \text{ ksi}) \times (1 \text{ in}) \times [(12 \text{ in}) - (4) \times ((0.8125 \text{ in}) + (0.0625 \text{ in}))]$ $\phi R_{n,tr} = 369.75 \text{ kip}$ Determine the governing tensile capacity of the double angle. $\phi R_{n,ty} = 388.8 \text{ kip}$ - Design Tension Yielding Capacity of Section $\phi R_{n,tr} = 369.75 \text{ kip}$ - Design Tension Rupture Capacity of Section ϕR_n - Governing Design Capacity $\phi R_n = \min(\phi R_{n,ty}, \phi R_{n,tr})$ $\phi R_n = \min((388.8 \text{ kip}), (369.75 \text{ kip}))$ $\phi R_n = 369.75 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{H_{c,T,upper}}{\phi R_n} = \frac{(4.714 \text{ kip})}{(369.75 \text{ kip})} = 0.012749$	PASS
	Check No. 16: Tension and Flexure Interaction of the Double Angle Calculate the tension and flexure interaction of the double angle. $H_{c,T,upper} = 4.714 \text{ kip}$ - Tension Load at Gusset to Column $V_r = 10.607 \text{ kip}$ - Vertical Shear at Gusset to Column Interaction - Tension and Flexure Interaction $Interaction = \frac{H_{c,T,upper}}{2 \phi R_n} + \frac{V_r}{\phi V_n}$ $Interaction = \frac{(4.714 \text{ kip})}{2 \times (369.75 \text{ kip})} + \frac{(10.607 \text{ kip})}{(388.8 \text{ kip})}$ $Interaction = 0.033655$ Result: Demand over Capacity Ratio $DCR = \frac{Interaction}{1.0} = \frac{(0.033655)}{(1)} = 0.033655$	PASS
AISC 360-16 Chapter J4.4 Eq. (J4-6)	Check No. 17: Design Capacity of Double Angle in Compression Calculate the compression buckling capacity of the double angle. $\phi = 0.9$ - Compression Resistance Factor $F_{y,at,col,u,br} = 36 \text{ ksi}$ - Upper Brace Double Angle Yield Stress $t_{2L,at,col,u,br} = 0.5 \text{ in}$ - Upper Brace Double Angle Thickness $L_{2L,at,col,u,br} = 12 \text{ in}$ - Upper Brace Double Angle Length $K = 1.2$ - Effective Length Factor $g_a = 3 \text{ in}$ - Double Angle Gage $\frac{KL}{r} = 12.471$ - Effective Length Slenderness Ratio Since, $\frac{KL}{r} \leq 25$. ϕR_n - Design Compressive Capacity of Section $\phi R_n = \phi F_{y,at,col,u,br} [2(t_{2L,at,col,u,br})] L_{2L,at,col,u,br}$	

	$\phi R_n = (0.9) \times (36 \text{ ksi}) \times (1 \text{ in}) \times (12 \text{ in})$ $\phi R_n = 388.8 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{H_{c,C,upper}}{\phi R_n} = \frac{(4.714 \text{ kip})}{(388.8 \text{ kip})} = 0.012125$	PASS
	Check No. 18: Compression and Flexure Interaction of the Double Angle Calculate the compression and flexure interaction of the double angle. $H_{c,C,upper} = 4.714 \text{ kip}$ - Compression Load at Gusset to Column $V_r = 10.607 \text{ kip}$ - Vertical Shear at Gusset to Column <i>Interaction</i> - Compression and Flexure Interaction $Interaction = \frac{H_{c,C,upper}}{2 \phi R_n} + \frac{V_r}{\phi V_n}$ $Interaction = \frac{(4.714 \text{ kip})}{2 \times (388.8 \text{ kip})} + \frac{(10.607 \text{ kip})}{(388.8 \text{ kip})}$ $Interaction = 0.033343$ Result: Demand over Capacity Ratio $DCR = \frac{Interaction}{1.0} = \frac{(0.033343)}{(1)} = 0.033343$	PASS
AISC 360-16 Chapter J4.3 Eq. (J4-5)	Check No. 19: Design Capacity of Gusset Plate in Block Shear Calculate the net area of the gusset plate subject to tension. $t_{gp,u,br} = 1 \text{ in}$ - Upper Brace Gusset Plate Thickness $n_c = 1$ - Number of Bolt Columns $s_c = 0 \text{ in}$ - Bolt Column Spacing $L_{eh} = 2 \text{ in}$ - Horizontal Edge Distance at Gusset Plate $d_h = 1 \text{ in}$ - Horizontal Bolt Hole Dimension at Double Angle A_{nt} - Net Area Subject to Tension (L-pattern) $A_{nt} = t_{gp,u,br} [(n_c - 1) s_c + L_{eh} - (n_c - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nt} = (1 \text{ in}) \times [(1 - 1) \times (0 \text{ in}) + (2 \text{ in}) - ((1 - 0.5) \times ((1 \text{ in}) + (0.0625 \text{ in})))]$ $A_{nt} = 1.4688 \text{ in}^2$ Calculate the gross area of the gusset plate subject to shear. $t_{gp,u,br} = 1 \text{ in}$ - Upper Brace Gusset Plate Thickness $L_{ev} = 2.25 \text{ in}$ - Vertical Edge Distance at Gusset Plate $n_r = 4$ - Number of Bolt Rows $s_r = 3 \text{ in}$ - Bolt Row Spacing A_{gv} - Gross Area Subject to Shear (L-pattern) $A_{gv} = t_{gp,u,br} [L_{ev} + (n_r - 1) s_r]$ $A_{gv} = (1 \text{ in}) \times [(2.25 \text{ in}) + ((4 - 1) \times (3 \text{ in}))]$ $A_{gv} = 11.25 \text{ in}^2$ Calculate the net area of the gusset plate subject to shear. $t_{gp,u,br} = 1 \text{ in}$ - Upper Brace Gusset Plate Thickness $h_{gp,u,br} = 13.5 \text{ in}$ - Upper Brace Gusset Plate Height along Column $L_{ev} = 2.25 \text{ in}$ - Vertical Edge Distance at Gusset Plate $n_r = 4$ - Number of Bolt Rows $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Double Angle A_{nv} - Net Area Subject to Shear (L-pattern) $A_{nv} = t_{gp,u,br} [h_{gp,u,br} - L_{ev} - (n_r - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nv} = (1 \text{ in}) \times [(13.5 \text{ in}) - (2.25 \text{ in}) - ((4 - 0.5) \times ((0.8125 \text{ in}) + (0.0625 \text{ in})))]$ $A_{nv} = 8.1875 \text{ in}^2$ Calculate the design block shear capacity of the gusset plate. $F_{u,u,br} = 65 \text{ ksi}$ - Upper Brace Gusset Plate Tensile Stress $A_{nv} = 8.1875 \text{ in}^2$ - Net Area Subject to Shear (L-pattern) $U_{bs} = 1$ - Uniformity factor $A_{nt} = 1.4688 \text{ in}^2$ - Net Area Subject to Tension (L-pattern) $F_{y,u,br} = 50 \text{ ksi}$ - Upper Brace Gusset Plate Yield Stress $A_{gv} = 11.25 \text{ in}^2$ - Gross Area Subject to Shear (L-pattern) $\phi = 0.75$ - Block Shear Resistance Factor ϕR_n - Design Block Shear Capacity of Section $\phi R_n = \phi (0.6 F_{u,u,br} A_{nv} + U_{bs} F_{u,u,br} A_{nt} \leq 0.6 F_{y,u,br} A_{gv} + U_{bs} F_{u,u,br} A_{nt})$ $\phi R_n = (0.75) \times (0.6 \times (65 \text{ ksi}) \times (8.1875 \text{ in}^2) + (1) \times (65 \text{ ksi}) \times (1.4688 \text{ in}^2) \leq 0.6 \times (50 \text{ ksi}) \times (11.25 \text{ in}^2) + (1) \times (65 \text{ ksi}) \times (1.4688 \text{ in}^2))$ $\phi R_n = 311.09 \text{ kip}$ Result: Demand over Capacity Ratio	PASS

	$DCR = \frac{V_r}{\phi R_n} = \frac{(10.607 \text{ kip})}{(311.09 \text{ kip})} = 0.034095$	
	Check No. 20: Design Capacity of Gusset Plate in Block Shear due to Axial Load Calculate the net area of the gusset plate subject to tension. $t_{gp,u,br} = 1 \text{ in}$ - Upper Brace Gusset Plate Thickness $n_r = 4$ - Number of Bolt Rows $s_r = 3 \text{ in}$ - Bolt Row Spacing $L_{ev} = 2.25 \text{ in}$ - Vertical Edge Distance at Gusset Plate $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Double Angle A_{nt} - Net Area Subject to Tension (L-pattern) $A_{nt} = t_{gp,u,br} [(n_r - 1) s_r + L_{ev} - (n_r - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nt} = (1 \text{ in}) \times [(4 - 1) \times (3 \text{ in}) + (2.25 \text{ in}) - ((4 - 0.5) \times ((0.8125 \text{ in}) + (0.0625 \text{ in})))]$ $A_{nt} = 8.1875 \text{ in}^2$ Calculate the gross area of the gusset plate subject to shear. $t_{gp,u,br} = 1 \text{ in}$ - Upper Brace Gusset Plate Thickness $L_{eh} = 2 \text{ in}$ - Horizontal Edge Distance at Gusset Plate $n_c = 1$ - Number of Bolt Columns $s_c = 0 \text{ in}$ - Bolt Column Spacing A_{gv} - Gross Area Subject to Shear (L-pattern) $A_{gv} = t_{gp,u,br} [L_{eh} + (n_c - 1) s_c]$ $A_{gv} = (1 \text{ in}) \times [(2 \text{ in}) + ((1 - 1) \times (0 \text{ in}))]$ $A_{gv} = 2 \text{ in}^2$ Calculate the net area of the gusset plate subject to shear. $t_{gp,u,br} = 1 \text{ in}$ - Upper Brace Gusset Plate Thickness $n_c = 1$ - Number of Bolt Columns $s_c = 0 \text{ in}$ - Bolt Column Spacing $d_h = 1 \text{ in}$ - Horizontal Bolt Hole Dimension at Double Angle $L_{eh} = 2 \text{ in}$ - Horizontal Edge Distance at Gusset Plate A_{nv} - Net Area Subject to Shear (L-pattern) $A_{nv} = t_{gp,u,br} (L_{eh} + (n_c - 1) s_c - (n_c - 0.5) (d_h + 0.0625 \text{ in}))$ $A_{nv} = (1 \text{ in}) \times ((2 \text{ in}) + ((1 - 1) \times (0 \text{ in})) - ((1 - 0.5) \times ((1 \text{ in}) + (0.0625 \text{ in}))))$ $A_{nv} = 1.4688 \text{ in}^2$ Calculate the design block shear capacity of the gusset plate. $F_{u,u,br} = 65 \text{ ksi}$ - Upper Brace Gusset Plate Tensile Stress $A_{nv} = 1.4688 \text{ in}^2$ - Net Area Subject to Shear (L-pattern) $U_{bs} = 1$ - Uniformity factor $A_{nt} = 8.1875 \text{ in}^2$ - Net Area Subject to Tension (L-pattern) $F_{y,u,br} = 50 \text{ ksi}$ - Upper Brace Gusset Plate Yield Stress $A_{gv} = 2 \text{ in}^2$ - Gross Area Subject to Shear (L-pattern) $\phi = 0.75$ - Block Shear Resistance Factor ϕR_n - Design Block Shear Capacity of Section $\phi R_n = \phi (0.6 F_{u,u,br} A_{nv} + U_{bs} F_{u,u,br} A_{nt} \leq 0.6 F_{y,u,br} A_{gv} + U_{bs} F_{u,u,br} A_{nt})$ $\phi R_n = (0.75) \times (0.6 \times (65 \text{ ksi}) \times (1.4688 \text{ in}^2) + (1) \times (65 \text{ ksi}) \times (8.1875 \text{ in}^2)) \leq 0.6 \times (50 \text{ ksi}) \times (2 \text{ in}^2) + (1) \times (65 \text{ ksi}) \times (8.1875 \text{ in}^2)$ $\phi R_n = 442.1 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{H_{c,T,upper}}{\phi R_n} = \frac{(4.714 \text{ kip})}{(442.1 \text{ kip})} = 0.010663$	PASS
AISC 360-16 Chapter J4.3 Eq. (J4-5)	Check No. 21: Shear and Axial Interaction of Block Shear in Gusset Plate Calculate the shear and axial interaction of block shear capacities in the gusset plate. $V_r = 10.607 \text{ kip}$ - Vertical Shear at Gusset to Column $H_{c,T,upper} = 4.714 \text{ kip}$ - Tension Load at Gusset to Column $Interaction$ - Shear and Axial Interaction $Interaction = \sqrt{\left(\frac{V_r}{\phi R_n}\right)^2 + \left(\frac{H_{c,T,upper}}{\phi R_n}\right)^2}$ $Interaction = \sqrt{\left(\frac{(10.607 \text{ kip})}{(311.09 \text{ kip})}\right)^2 + \left(\frac{(4.714 \text{ kip})}{(442.1 \text{ kip})}\right)^2}$ $Interaction = 0.035724$ Result: Demand over Capacity Ratio $DCR = \frac{Interaction}{1.0} = \frac{(0.035724)}{(1)} = 0.035724$	PASS
	Check No. 22: Connection Detailing Limitations at Support 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
Double Angle Minimum Vertical Edge Distance	1.000	1.500	0.667	PASS
Double Angle Minimum Horizontal Edge Distance	1.125	2.000	0.563	PASS
Column Minimum Horizontal Edge Distance	1.000	3.750	0.267	PASS
Minimum Bolt Gage at Column	4.375	7.000	0.625	PASS
Minimum Bolt Gage at Double Angle	4.500	7.000	0.643	PASS

Result:

Demand over Capacity Ratio

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

PASS

Check No. 23: Design Capacity of the Bolt Group in Shear at Support Side

$F_{nv} = 54$ ksi - Bolt Nominal Shear Strength

$F_{nt} = 90$ ksi - Bolt Nominal Tensile Strength

$N_T = 8$ - Total Number of Bolts in Tension

$H_c = 4.714$ kip - Axial Load at Gusset to Column

F'_{nv} - Design Modified Bolt Shear Strength

$$F_{nv} = \min \left(1.3 F_{nv} - \frac{0.84 F_{nv} H_c}{F_{nt} N_T d_b^2}, F_{nv} \right)$$

$$F'_{nv} = \min \left(1.3 \times (54 \text{ ksi}) - \frac{0.84 \times (54 \text{ ksi}) \times (4.714 \text{ kip})}{(90 \text{ ksi}) \times (8) \times (0.75 \text{ in})^2}, (54 \text{ ksi}) \right)$$

$$F'_{nv} = 54 \text{ ksi}$$

$d_b = 0.75$ in - Bolt Diameter

$N_s = 1$ - Number of Shear Planes

$N_V = 8$ - Total Number of Bolts in Shear

$h_f = 1$ - Filler Factor for Bearing Bolts

$\phi = 0.75$ - Bolt Shear Resistance Factor

ϕR_n - Design Bolt Shear Capacity

$$\phi R_n = \phi F'_{nv} \frac{\pi}{4} (d_b)^2 N_s N_V h_f$$

$$\phi R_n = (0.75) \times (54 \text{ ksi}) \times \frac{\pi}{4} \times ((0.75 \text{ in}))^2 \times (1) \times (8) \times (1)$$

$$\phi R_n = 143.14 \text{ kip}$$

Result:

Demand over Capacity Ratio

$$DCR = \frac{V_r}{\phi R_n} = \frac{(10.607 \text{ kip})}{(143.14 \text{ kip})} = 0.0741$$

PASS

Check No. 24: Design Capacity of the Bolt Group in Bearing and Tear-out on Double Angle at Support Side

Calculate the bolt bearing capacity of the double angle.

$d_b = 0.75$ in - Bolt Diameter

$t_{2L,at,col,u,br} = 0.5$ in - Upper Brace Double Angle Thickness

$F_{u,at,col,u,br} = 58$ ksi - Upper Brace Double Angle Tensile Stress

$N_V = 8$ - Total Number of Bolts in Shear

$\phi = 0.75$ - Bolt Bearing Resistance Factor

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

$$\phi R_{n_bearing} = \phi 2.4 d_b t_{2L,at,col,u,br} F_{u,at,col,u,br} N_V$$

$$\phi R_{n_bearing} = (0.75) \times 2.4 \times (0.75 \text{ in}) \times (0.5 \text{ in}) \times (58 \text{ ksi}) \times (8)$$

$$\phi R_{n_bearing} = 313.2 \text{ kip}$$

Calculate the clear distance of outer bolts on double angle.

$L_{ev} = 1.5$ in - Vertical Edge Distance at Double Angle

$d_h = 0.8125$ in - Vertical Bolt Hole Dimension at Double Angle

l_{c1} - Clear Distance at First Bolt Row

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

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

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

Calculate the clear distance of inner bolts on double angle.

$s_r = 3$ in - Bolt Row Spacing

$d_h = 0.8125$ in - Vertical Bolt Hole Dimension at Double Angle

l_{c2} - Clear Distance at Rest of Bolts

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

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

AISC 360-16 Chapter J3.10 Eq. (J3-6C) AISC 360-16 Chapter J3.10 Eq. (J3-6a) AISC 360-16 Chapter J3.10 Eq. (J3-6C) AISC 360-16 Chapter J3.10	$l_{c2} = 2.1875$ in Calculate the bolt tear-out capacity of the double angle. $l_{c1} = 1.0938$ in - Clear Distance at First Bolt Row $l_{c2} = 2.1875$ in - Clear Distance at Rest of Bolts $t_{2L,at,col,u,br} = 0.5$ in - Upper Brace Double Angle Thickness $F_{u,at,col,u,br} = 58$ ksi - Upper Brace Double Angle Tensile Stress $N_V = 8$ - Total Number of Bolts in Shear $n_r = 4$ - Number of Bolt Rows $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_tearout}$ - Design Bolt Tear-out Capacity of Section $\phi R_{n_tearout} = \phi [1.2 l_{c1} t_{2L,at,col,u,br} F_{u,at,col,u,br} + 1.2 l_{c2} t_{2L,at,col,u,br} F_{u,at,col,u,br} (n_r - 1)] \left(\frac{N_V}{n_r} \right)$ $\phi R_{n_tearout} = (0.75) \times [1.2 \times (1.0938 \text{ in}) \times (0.5 \text{ in}) \times (58 \text{ ksi}) + 1.2 \times (2.1875 \text{ in}) \times (0.5 \text{ in}) \times (58 \text{ ksi}) \times ((4) - 1)] \times \left(\frac{(8)}{(4)} \right)$ $\phi R_{n_tearout} = 399.66 \text{ kip}$ Determine the governing bearing and tear-out capacity of the bolt group on double angle. $\phi R_{n_bearing} = 313.2$ kip - Design bolt bearing capacity of double angle $\phi R_{n_tearout} = 399.66$ kip - Design bolt tear-out capacity of double angle ϕR_n - Governing Design Capacity $\phi R_n = \min(\phi R_{n_bearing}, \phi R_{n_tearout})$ $\phi R_n = \min((313.2 \text{ kip}), (399.66 \text{ kip}))$ $\phi R_n = 313.2 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{V_r}{\phi R_n} = \frac{(10.607 \text{ kip})}{(313.2 \text{ kip})} = 0.033865$	PASS
AISC 360-16 Chapter J3.10 Eq. (J3-6a) AISC 360-16 Chapter J3.10 Eq. (J3-6C) AISC 360-16 Chapter J3.10	Check No. 25: Design Capacity of the Bolt Group in Bearing and Tear-out at Support Flange Calculate the bolt bearing capacity. $d_b = 0.75$ in - Bolt Diameter $t_{f,c} = 0.71$ in - Column Flange Thickness $F_{u,c} = 65$ ksi - Column Tensile Stress $N_V = 8$ - Total Number of Bolts in Shear $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_bearing}$ - Design Bolt Bearing Capacity of Section $\phi R_{n_bearing} = \phi 2.4 d_b t_{f,c} F_{u,c} N_V$ $\phi R_{n_bearing} = (0.75) \times 2.4 \times (0.75 \text{ in}) \times (0.71 \text{ in}) \times (65 \text{ ksi}) \times (8)$ $\phi R_{n_bearing} = 498.42 \text{ kip}$ Calculate the clear distance of inner bolts on member. $s_r = 3$ in - Bolt Row Spacing $d_h = 0.8125$ in - Vertical Bolt Hole Dimension at Column 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. $l_{c2} = 2.1875$ in - Clear Distance at Rest of Bolts $t_{f,c} = 0.71$ in - Column Flange Thickness $F_{u,c} = 65$ ksi - Column Tensile Stress $N_V = 8$ - Total Number of Bolts in Shear $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_tearout}$ - Design Bolt Tear-out Capacity of Section $\phi R_{n_tearout} = \phi 1.2 l_{c2} t_{f,c} F_{u,c} N_V$ $\phi R_{n_tearout} = (0.75) \times 1.2 \times (2.1875 \text{ in}) \times (0.71 \text{ in}) \times (65 \text{ ksi}) \times (8)$ $\phi R_{n_tearout} = 726.86 \text{ kip}$ Determine the governing bearing and tear-out capacity of the bolt group. $\phi R_{n_bearing} = 498.42$ kip - Design bolt bearing capacity of member $\phi R_{n_tearout} = 726.86$ kip - Design bolt tear-out capacity of member ϕR_n - Governing Design Capacity $\phi R_n = \min(\phi R_{n_bearing}, \phi R_{n_tearout})$ $\phi R_n = \min((498.42 \text{ kip}), (726.86 \text{ kip}))$	

	$\phi R_n = 498.42 \text{ kip}$	
	Result: Demand over Capacity Ratio $DCR = \frac{V_r}{\phi R_n} = \frac{(10.607 \text{ kip})}{(498.42 \text{ kip})} = 0.02128$	PASS
	Check No. 26: Design Capacity of Double Angle in Block Shear at Support Side Calculate the net area of the double angle subject to tension. $t_{2L,at,col,u,br} = 0.5 \text{ in}$ - Upper Brace Double Angle Thickness $n_c = 1$ - Number of Bolt Columns $s_c = 0 \text{ in}$ - Bolt Column Spacing $L_{eh} = 2 \text{ in}$ - Horizontal Edge Distance at Double Angle $d_h = 1 \text{ in}$ - Horizontal Bolt Hole Dimension at Double Angle A_{nt} - Net Area Subject to Tension (L-pattern) $A_{nt} = [2(t_{2L,at,col,u,br})] [(n_c - 1) s_c + L_{eh} - (n_c - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nt} = (1 \text{ in}) \times [(1 - 1) \times (0 \text{ in}) + (2 \text{ in}) - ((1) - 0.5) \times ((1 \text{ in}) + (0.0625 \text{ in}))]$ $A_{nt} = 1.4688 \text{ in}^2$ Calculate the gross area of the double angle subject to shear. $t_{2L,at,col,u,br} = 0.5 \text{ in}$ - Upper Brace Double Angle Thickness $L_{ev} = 1.5 \text{ in}$ - Vertical Edge Distance at Double Angle $n_r = 4$ - Number of Bolt Rows $s_r = 3 \text{ in}$ - Bolt Row Spacing A_{gv} - Gross Area Subject to Shear (L-pattern) $A_{gv} = [2(t_{2L,at,col,u,br})] [L_{ev} + (n_r - 1) s_r]$ $A_{gv} = (1 \text{ in}) \times [(1.5 \text{ in}) + ((4) - 1) \times (3 \text{ in})]$ $A_{gv} = 10.5 \text{ in}^2$ Calculate the net area of the double angle subject to shear. $t_{2L,at,col,u,br} = 0.5 \text{ in}$ - Upper Brace Double Angle Thickness $L_{2L,at,col,u,br} = 12 \text{ in}$ - Upper Brace Double Angle Length $L_{ev} = 1.5 \text{ in}$ - Vertical Edge Distance at Double Angle $n_r = 4$ - Number of Bolt Rows $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Double Angle A_{nv} - Net Area Subject to Shear (L-pattern) $A_{nv} = [2(t_{2L,at,col,u,br})] [L_{2L,at,col,u,br} - L_{ev} - (n_r - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nv} = (1 \text{ in}) \times [(12 \text{ in}) - (1.5 \text{ in}) - ((4) - 0.5) \times ((0.8125 \text{ in}) + (0.0625 \text{ in}))]$ $A_{nv} = 7.4375 \text{ in}^2$ Calculate the design block shear capacity of the double angle. $F_{u,at,col,u,br} = 58 \text{ ksi}$ - Upper Brace Double Angle Tensile Stress $A_{nv} = 7.4375 \text{ in}^2$ - Net Area Subject to Shear (L-pattern) $U_{bs} = 1$ - Uniformity factor $A_{nt} = 1.4688 \text{ in}^2$ - Net Area Subject to Tension (L-pattern) $F_{y,at,col,u,br} = 36 \text{ ksi}$ - Upper Brace Double Angle Yield Stress $A_{gv} = 10.5 \text{ in}^2$ - Gross Area Subject to Shear (L-pattern) $\phi = 0.75$ - Block Shear Resistance Factor ϕR_n - Design Block Shear Capacity of Section $\phi R_n = \phi (0.6 F_{u,at,col,u,br} A_{nv} + U_{bs} F_{u,at,col,u,br} A_{nt} \leq 0.6 F_{y,at,col,u,br} A_{gv} + U_{bs} F_{u,at,col,u,br} A_{nt})$ $\phi R_n = (0.75) \times (0.6 \times (58 \text{ ksi}) \times (7.4375 \text{ in}^2) + (1) \times (58 \text{ ksi}) \times (1.4688 \text{ in}^2)) \leq 0.6 \times (36 \text{ ksi}) \times (10.5 \text{ in}^2) + (1) \times (58 \text{ ksi}) \times (1.4688 \text{ in}^2))$ $\phi R_n = 233.99 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{V_r}{\phi R_n} = \frac{(10.607 \text{ kip})}{(233.99 \text{ kip})} = 0.045329$	PASS
	Check No. 27: Design Capacity of Double Angle in Shear at Support Side Calculate the gross area of double angle subject to yielding. $L_{2L,at,col,u,br} = 12 \text{ in}$ - Upper Brace Double Angle Length $t_{2L,at,col,u,br} = 0.5 \text{ in}$ - Upper Brace Double Angle Thickness A_{gv} - Section Gross Area $A_{gv} = L_{2L,at,col,u,br} [2(t_{2L,at,col,u,br})]$ $A_{gv} = (12 \text{ in}) \times (1 \text{ in})$ $A_{gv} = 12 \text{ in}^2$ Calculate the shear yielding capacity of the double angle. $F_{y,at,col,u,br} = 36 \text{ ksi}$ - Upper Brace Double Angle Yield Stress $A_{gv} = 12 \text{ in}^2$ - Section Gross Area $\phi = 1$ - Shear Yielding Resistance Factor $\phi R_{n,sv}$ - Design Shear Yielding Capacity of Section	

AISC 360-16 Chapter J4.2 Eq. (J4-4) AISC 360-16 Chapter J4.2 Eq. (J4-3) AISC 360-16 Chapter J4.2 Eq. (J4-4) AISC 360-16 Chapter J4.2	$\phi R_{n_{sy}} = \phi 0.6 F_{y,at,col,u,br} A_{gv}$ $\phi R_{n_{sy}} = (1) \times 0.6 \times (36 \text{ ksi}) \times (12 \text{ in}^2)$ $\phi R_{n_{sy}} = 259.2 \text{ kip}$ Calculate the net area of double angle subject to rupture. $t_{2L,at,col,u,br} = 0.5 \text{ in}$ - Upper Brace Double Angle Thickness $L_{2L,at,col,u,br} = 12 \text{ in}$ - Upper Brace Double Angle Length $n_r = 4$ - Number of Bolt Rows $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Double Angle A_{nv} - Section Net Area $A_{nv} = [2(t_{2L,at,col,u,br})] [L_{2L,at,col,u,br} - n_r (d_h + 0.0625 \text{ in})]$ $A_{nv} = (1 \text{ in}) \times [(12 \text{ in}) - (4) \times ((0.8125 \text{ in}) + (0.0625 \text{ in}))]$ $A_{nv} = 8.5 \text{ in}^2$ Calculate the shear rupture capacity of the double angle. $F_{u,at,col,u,br} = 58 \text{ ksi}$ - Upper Brace Double Angle Tensile Stress $A_{nv} = 8.5 \text{ in}^2$ - Section Net Area $\phi = 0.75$ - Shear Rupture Resistance Factor $\phi R_{n_{sr}}$ - Design Shear Rupture Capacity of Section $\phi R_{n_{sr}} = \phi 0.6 F_{u,at,col,u,br} A_{nv}$ $\phi R_{n_{sr}} = (0.75) \times 0.6 \times (58 \text{ ksi}) \times (8.5 \text{ in}^2)$ $\phi R_{n_{sr}} = 221.85 \text{ kip}$ Determine the governing shear capacity of the double angle. $\phi R_{n_{sy}} = 259.2 \text{ kip}$ - Design shear yielding capacity of double angle $\phi R_{n_{sr}} = 221.85 \text{ kip}$ - Design shear rupture capacity of double angle ϕR_n - Governing Design Capacity $\phi R_n = \min(\phi R_{n_{sy}}, \phi R_{n_{sr}})$ $\phi R_n = \min((259.2 \text{ kip}), (221.85 \text{ kip}))$ $\phi R_n = 221.85 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{V_r}{\phi R_n} = \frac{(10.607 \text{ kip})}{(221.85 \text{ kip})} = 0.04781$	PASS
AISC 15th Edition pg. (9-9) Eq. (9-15)	Check No. 28: Design Capacity of Double Angle in Flexure Using Double Coped Procedure at Support Side Calculate and compare upper and lower limits. $E = 29000 \text{ ksi}$ - Modulus for Steel $F_{y,at,col,u,br} = 36 \text{ ksi}$ - Upper Brace Double Angle Yield Stress $L_{b,2L} = 3 \text{ in}$ - Unbraced Length at Double Angle $L_{2L,at,col,u,br} = 12 \text{ in}$ - Upper Brace Double Angle Length $t_{2L,at,col,u,br} = 0.5 \text{ in}$ - Upper Brace Double Angle Thickness Depth to Thickness Ratio $\frac{L_b d}{t^2} = \frac{(3 \text{ in}) \times (12 \text{ in})}{(0.5 \text{ in})^2} = 144$ 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 $\frac{L_b d}{t^2} \leq \frac{1.9E}{F_y}$. Calculate the plastic section modulus of the double angle. $t_{2L,at,col,u,br} = 0.5 \text{ in}$ - Upper Brace Double Angle Thickness $L_{2L,at,col,u,br} = 12 \text{ in}$ - Upper Brace Double Angle Length Plastic Section Modulus $Z = \frac{t_{2L,at,col,u,br} (L_{2L,at,col,u,br})^2}{4} = \frac{(0.5 \text{ in}) \times ((12 \text{ in}))^2}{4} = 18 \text{ in}^3$ Calculate the elastic section modulus of the double angle. $t_{2L,at,col,u,br} = 0.5 \text{ in}$ - Upper Brace Double Angle Thickness $L_{2L,at,col,u,br} = 12 \text{ in}$ - Upper Brace Double Angle Length Elastic Section Modulus $S = \frac{t_{2L,at,col,u,br} (L_{2L,at,col,u,br})^2}{6} = \frac{(0.5 \text{ in}) \times ((12 \text{ in}))^2}{6} = 12 \text{ in}^3$ Calculate the lateral torsional buckling modification factor. $L_{b,2L} = 3 \text{ in}$ - Unbraced Length at Double Angle $L_{2L,at,col,u,br} = 12 \text{ in}$ - Upper Brace Double Angle Length $d_{c,2L} = 0 \text{ in}$ - Double Angle Cope Depth (Assumed 0) C_b - Lateral-Torsional Buckling Modification Factor $C_b = \left[3 + \ln \left(\frac{L_{b,2L}}{L_{2L,at,col,u,br}} \right) \right] \left(1 - \frac{d_{c,2L}}{L_{2L,at,col,u,br}} \right) \geq 1.84$	

AISC 15th Edition pg. (9-9) Eq. (9-15) AISC 360-16 Chapter F11.2 Eq. (F11-2) AISC 360-16 Chapter F11.2 Eq. (F11-2)	$C_b = \left[3 + \ln \left(\frac{(3 \text{ in})}{(12 \text{ in})} \right) \right] \times \left(1 - \frac{(0 \text{ in})}{(12 \text{ in})} \right) \geq 1.84$ $C_b = 1.84$ Calculate the flexural capacity of the double angle. $C_b = 1.84$ - Lateral-Torsional Buckling Modification Factor $\frac{L_{bd}}{t^2} = 144$ - Depth to Thickness Ratio $F_{y,at,col,u,br} = 36 \text{ ksi}$ - Upper Brace Double Angle Yield Stress $E = 29000 \text{ ksi}$ - Modulus for Steel $S = 12 \text{ in}^3$ - Elastic Section Modulus $Z = 18 \text{ in}^3$ - Plastic Section Modulus $\phi = 0.9$ - Flexure Resistance Factor ϕM_n - Design Flexural Capacity of Coped Section $\phi M_n = \phi \left[C_b \left[1.52 - 0.274 \left(\frac{L_{bd}}{t^2} \right) \left(\frac{F_{y,at,col,u,br}}{E} \right) \right] F_{y,at,col,u,br} S \leq F_{y,at,col,u,br} Z \right] 2$ $\phi M_n = (0.9) \times \left[(1.84) \times \left[1.52 - 0.274 \times ((144)) \times \left(\frac{(36 \text{ ksi})}{(29000 \text{ ksi})} \right) \right] \times (36 \text{ ksi}) \times (12 \text{ in}^3) \leq (36 \text{ ksi}) \times (18 \text{ in}^3) \right] \times 2$ $\phi M_n = 97.2 \text{ kip}\cdot\text{ft}$ Calculate the equivalent shear capacity of the double angle. $\phi M_n = 97.2 \text{ kip}\cdot\text{ft}$ - Design Flexural Capacity of Coped Section $L_{b,2L} = 3 \text{ in}$ - Unbraced Length at Double Angle ϕV_n - Design Capacity of Single Plate in Bending $\phi V_n = \frac{\phi M_n}{L_{b,2L}}$ $\phi V_n = \frac{(97.2 \text{ kip}\cdot\text{ft})}{(3 \text{ in})}$ $\phi V_n = 388.8 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{V_r}{\phi V_n} = \frac{(10.607 \text{ kip})}{(388.8 \text{ kip})} = 0.02728$	PASS
AISC 360-16 Chapter J4.2 Eq. (J4-3) AISC 360-16 Chapter F11.2 Eq. (F11-2) AISC 15th Edition pg. (10-90) Eq. (10-5)	Check No. 29: Shear and Moment Interaction of Double Angle at Support Side Calculate the shear and moment interaction of the double angle. $V_r = 10.607 \text{ kip}$ - Vertical Shear at Gusset to Column Moment at Component $M_r = V_r L_{b,2L} = (10.607 \text{ kip}) \times (3 \text{ in}) = 2.6517 \text{ kip}\cdot\text{ft}$ $\phi V_n = 259.2 \text{ kip}$ - Design shear yielding capacity of double angle $\phi M_n = 97.2 \text{ kip}\cdot\text{ft}$ - Design Flexural Capacity of Double Angle I_{nt} - Interaction for Shear Yielding/Buckling and Flexure $I_{nt} = \left(\frac{V_r}{\phi V_n} \right)^2 + \left(\frac{M_r}{\phi M_n} \right)^2$ $I_{nt} = \left(\frac{(10.607 \text{ kip})}{(259.2 \text{ kip})} \right)^2 + \left(\frac{(2.6517 \text{ kip}\cdot\text{ft})}{(97.2 \text{ kip}\cdot\text{ft})} \right)^2$ $I_{nt} = 0.0024187$ Result: Demand over Capacity Ratio $DCR = \frac{I_{nt}}{1.0} = \frac{(0.0024187)}{(1)} = 0.0024187$	PASS
	Check No. 30: Design Capacity of Double Angle in Bolt Prying $d_b = 0.75 \text{ in}$ - Bolt Diameter $n_r = 4$ - Number of Bolt Rows $s_r = 3 \text{ in}$ - Bolt Row Spacing $H_{c,T,upper} = 4.714 \text{ kip}$ - Tension Load at Gusset to Column $F_{u,at,col,u,br} = 58 \text{ ksi}$ - Upper Brace Double Angle Tensile Stress $b = 2.75 \text{ in}$ - b prying $F_{nv} = 54 \text{ ksi}$ - Bolt Nominal Shear Strength $F_{nt} = 90 \text{ ksi}$ - Bolt Nominal Tensile Strength $V_r = 10.607 \text{ kip}$ - Vertical Shear at Gusset to Column $L_{eh} = 2 \text{ in}$ - Horizontal Edge Distance at Double Angle $d_h = 1 \text{ in}$ - Horizontal Bolt Hole Dimension at Double Angle $B = 29.818 \text{ kip}$ - Available tension per bolt δ $\delta = 1 - \frac{d_h}{\min(3.5 b, s_r)}$ $\delta = 1 - \frac{(1 \text{ in})}{\min(3.5 \times (2.75 \text{ in}), (3 \text{ in}))}$	

	$\delta = 0.66667$	
	$\beta = \frac{\min(L_{eh} + 0.5 d_b, 1.25 b + 0.5 d_b) (2 H_{c,T,upper} - 4 B n_r)}{H_{c,T,upper} (d_b - 2 b)}$ $\beta = \frac{\min((2 \text{ in}) + 0.5 \times (0.75 \text{ in}), 1.25 \times (2.75 \text{ in}) + 0.5 \times (0.75 \text{ in})) \times (2 \times (4.714 \text{ kip}) - 4 \times (29.818 \text{ kip}) \times (4))}{(4.714 \text{ kip}) \times ((0.75 \text{ in}) - 2 \times (2.75 \text{ in}))}$ $\beta = 49.603$	
	$\alpha' = \min \left[1, \left \frac{1}{\delta} \left(\frac{\beta}{1 - \beta} \right) \right \right]$ $\alpha' = \min \left[1, \left \frac{1}{(0.66667)} \times \left(\frac{(49.603)}{1 - (49.603)} \right) \right \right]$ $\alpha' = 1$	
AISC 15th Edition Equation (9-19a)	$t_{min} \text{ - Minimum thickness required to ensure an acceptable combination of fitting strength and stiffness and bolt strength}$ $t_{min} = \sqrt{\frac{2.22 H_{c,T,upper} (b - 0.5 d_b)}{\min(3.5 b, s_r) F_{u,at,col,u,br} n_r (1 + \delta \alpha')}} = \sqrt{\frac{2.22 \times (4.714 \text{ kip}) \times ((2.75 \text{ in}) - 0.5 \times (0.75 \text{ in}))}{\min(3.5 \times (2.75 \text{ in}), (3 \text{ in})) \times (58 \text{ ksi}) \times (4) \times (1 + (0.66667) \times (1))}}$ $t_{min} = 0.14638 \text{ in}$ Result: Demand over Capacity Ratio $DCR = \frac{t_{min}}{t_{2L,at,col,u,br}} = \frac{(0.14638 \text{ in})}{(0.5 \text{ in})} = 0.29276$	PASS
	Check No. 31: Design Capacity of Column Flange in Bolt Prying $d_b = 0.75 \text{ in}$ - Bolt Diameter $n_r = 4$ - Number of Bolt Rows $s_r = 3 \text{ in}$ - Bolt Row Spacing $V_r = 10.607 \text{ kip}$ - Vertical Shear at Gusset to Column $H_{c,T,upper} = 4.714 \text{ kip}$ - Tension Load at Gusset to Column $F_{u,c} = 65 \text{ ksi}$ - Column Tensile Stress $b = 3.28 \text{ in}$ - b prying $F_{nv} = 54 \text{ ksi}$ - Bolt Nominal Shear Strength $F_{nt} = 90 \text{ ksi}$ - Bolt Nominal Tensile Strength $L_{eh} = 3.75 \text{ in}$ - Horizontal Edge Distance at Column $d_h = 0.8125 \text{ in}$ - Horizontal Bolt Hole Dimension at Column $B = 29.818 \text{ kip}$ - Available tension per bolt $\delta = 1 - \frac{d_h}{\min(3.5 b, s_r)}$ $\delta = 1 - \frac{(0.8125 \text{ in})}{\min(3.5 \times (3.28 \text{ in}), (3 \text{ in}))}$ $\delta = 0.72917$	
	$\beta = \frac{\min(L_{eh} + 0.5 d_b, 1.25 b + 0.5 d_b) (2 H_{c,T,upper} - 4 B n_r)}{H_{c,T,upper} (d_b - 2 b)}$ $\beta = \frac{\min((3.75 \text{ in}) + 0.5 \times (0.75 \text{ in}), 1.25 \times (3.28 \text{ in}) + 0.5 \times (0.75 \text{ in})) \times (2 \times (4.714 \text{ kip}) - 4 \times (29.818 \text{ kip}) \times (4))}{(4.714 \text{ kip}) \times ((0.75 \text{ in}) - 2 \times (3.28 \text{ in}))}$ $\beta = 70.435$	
	$\alpha' = \min \left[1, \left \frac{1}{\delta} \left(\frac{\beta}{1 - \beta} \right) \right \right]$ $\alpha' = \min \left[1, \left \frac{1}{(0.72917)} \times \left(\frac{(70.435)}{1 - (70.435)} \right) \right \right]$ $\alpha' = 1$	
AISC 15th Edition Equation (9-19a)	$t_{min} \text{ - Minimum thickness required to ensure an acceptable combination of fitting strength and stiffness and bolt strength}$ $t_{min} = \sqrt{\frac{2.22 H_{c,T,upper} (b - 0.5 d_b)}{\min(3.5 b, s_r) F_{u,c} n_r (1 + \delta \alpha')}} = \sqrt{\frac{2.22 \times (4.714 \text{ kip}) \times ((3.28 \text{ in}) - 0.5 \times (0.75 \text{ in}))}{\min(3.5 \times (3.28 \text{ in}), (3 \text{ in})) \times (65 \text{ ksi}) \times (4) \times (1 + (0.72917) \times (1))}}$	

	$t_{min} = 0.15013 \text{ in}$ Result: Demand over Capacity Ratio $DCR = \frac{t_{min}}{t_{fc}} = \frac{(0.15013 \text{ in})}{(0.71 \text{ in})} = 0.21146$	PASS
AISC 360-16 Chapter J10.3 Eq. (J10-4)	Check No. 32: Design Capacity of Support due to Compressive Load Calculate the web local crippling capacity of the column. $\phi = 0.75$ - Web Local Crippling Resistance Factor $t_{w,c} = 0.44 \text{ in}$ - Column Web Thickness $L_{2L,at,col,u,br} = 12 \text{ in}$ - Upper Brace Double Angle Length $d_c = 14 \text{ in}$ - Column Depth $t_{f,c} = 0.71 \text{ in}$ - Column Flange Thickness $F_{y,c} = 50 \text{ ksi}$ - Column Yield Stress $E = 29000 \text{ ksi}$ - Modulus for Steel $\phi R_{n,wlc}$ - Design Web Local Crippling Capacity of Support $\phi R_{n,wlc} = \phi 0.8 (t_{w,c})^2 \left[1 + 3 \left(\frac{L_{2L,at,col,u,br}}{d_c} \right) \left(\frac{t_{w,c}}{t_{f,c}} \right)^{1.5} \right] \sqrt{\frac{E F_{y,c} t_{f,c}}{t_{w,c}}} Q_f$ $\phi R_{n,wlc} = (0.75) \times 0.8 \times ((0.44 \text{ in}))^2 \times \left[1 + 3 \times \left(\frac{(12 \text{ in})}{(14 \text{ in})} \right) \times \left(\frac{(0.44 \text{ in})}{(0.71 \text{ in})} \right)^{1.5} \right] \times \sqrt{\frac{(29000 \text{ ksi}) \times (50 \text{ ksi}) \times (0.71 \text{ in})}{(0.44 \text{ in})}} \times (1)$ $\phi R_{n,wlc} = 400.58 \text{ kip}$ Calculate the web local yielding capacity of the column. $\phi = 1$ - Web Local Yielding Resistance Factor $F_{y,c} = 50 \text{ ksi}$ - Column Yield Stress $t_{w,c} = 0.44 \text{ in}$ - Column Web Thickness $k_{des,c} = 1.31 \text{ in}$ - Column kdes $L_{2L,at,col,u,br} = 12 \text{ in}$ - Upper Brace Double Angle Length $\phi R_{n,wly}$ - Design Web Local Yielding Capacity of Support $\phi R_{n,wly} = \phi F_{y,c} t_{w,c} (5 k_{des,c} + L_{2L,at,col,u,br})$ $\phi R_{n,wly} = (1) \times (50 \text{ ksi}) \times (0.44 \text{ in}) \times (5 \times (1.31 \text{ in}) + (12 \text{ in}))$ $\phi R_{n,wly} = 408.1 \text{ kip}$ Calculate for the governing design capacity of the column. ϕR_n - Governing Design Capacity of the Support $\phi R_n = \min(\phi R_{n,wlc}, \phi R_{n,wly})$ $\phi R_n = \min((400.58 \text{ kip}), (408.1 \text{ kip}))$ $\phi R_n = 400.58 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{H_c, C_{upper}}{\phi R_n} = \frac{(4.714 \text{ kip})}{(400.58 \text{ kip})} = 0.011768$	PASS
	Summary of Checks	

Design Checks	Demand	Capacity	DCR	Result
Connection Detailing Limitations at Gusset Side	12.000	12.000	1.000	PASS
Design Capacity of the Bolt Group in Shear at Gusset Side	11.607	42.265	0.275	PASS
Design Capacity of the Bolt Group in Bearing and Tear-out on Double Angle at Gusset Side	10.607	205.085	0.052	PASS
Design Capacity of the Bolt Group in Bearing and Tear-out on Double Angle at Gusset Side due to Axial Load	4.714	205.085	0.023	PASS
Shear and Axial Interaction of Bolt Bearing/Tearout in Double Angle	0.057	1.000	0.057	PASS
Design Capacity of the Bolt Group in Bearing and Tear-out on Gusset Plate	10.607	229.837	0.046	PASS
Design Capacity of the Bolt Group in Bearing and Tear-out on Gusset Plate due to Axial Load	4.714	229.837	0.021	PASS
Shear and Axial Interaction of Bolt Bearing/Tearout on Gusset Plate	0.051	1.000	0.051	PASS
Design Capacity of Double Angle in Block Shear at Gusset Side	10.607	233.991	0.045	PASS
Design Capacity of Double Angle in Block Shear at Gusset Side due to Axial Load	4.714	355.931	0.013	PASS
Shear and Axial Interaction of Block Shear in Double Angle	0.047	1.000	0.047	PASS
Design Capacity of Double Angle in Shear at Gusset Side	10.607	221.850	0.048	PASS
Design Capacity of Double Angle in Flexure Using Double Coped Procedure at Gusset Side	10.607	388.800	0.027	PASS
Shear and Moment Interaction of Double Angle at Gusset Side	0.002	1.000	0.002	PASS
Design Capacity of Double Angle in Tension	4.714	369.750	0.013	PASS
Tension and Flexure Interaction of the Double Angle	0.034	1.000	0.034	PASS
Design Capacity of Double Angle in Compression	4.714	388.800	0.012	PASS
Compression and Flexure Interaction of the Double Angle	0.033	1.000	0.033	PASS
Design Capacity of Gusset Plate in Block Shear	10.607	311.086	0.034	PASS
Design Capacity of Gusset Plate in Block Shear due to Axial Load	4.714	442.102	0.011	PASS
Shear and Axial Interaction of Block Shear in Gusset Plate	0.036	1.000	0.036	PASS
Connection Detailing Limitations at Support Side	2.000	3.000	0.667	PASS
Design Capacity of the Bolt Group in Shear at Support Side	10.607	143.139	0.074	PASS
Design Capacity of the Bolt Group in Bearing and Tear-out on Double Angle at Support Side	10.607	313.200	0.034	PASS
Design Capacity of the Bolt Group in Bearing and Tear-out at Support Flange	10.607	498.420	0.021	PASS
Design Capacity of Double Angle in Block Shear at Support Side	10.607	233.991	0.045	PASS
Design Capacity of Double Angle in Shear at Support Side	10.607	221.850	0.048	PASS
Design Capacity of Double Angle in Flexure Using Double Coped Procedure at Support Side	10.607	388.800	0.027	PASS
Shear and Moment Interaction of Double Angle at Support Side	0.002	1.000	0.002	PASS
Design Capacity of Double Angle in Bolt Prying	0.146	0.500	0.293	PASS
Design Capacity of Column Flange in Bolt Prying	0.150	0.710	0.211	PASS
Design Capacity of Support due to Compressive Load	4.714	400.582	0.012	PASS

REFERENCES	CALCULATIONS	RESULTS
	Beam Connection to Column AISC 360-16 LRFD	

REFERENCES	CALCULATIONS	RESULTS																																													
	Check No. 1: Connection Detailing Limitations at Beam Side <table><thead><tr><th>Detailing Limitations</th><th>Limit Value (in)</th><th>Actual Value (in)</th><th>DCR</th><th>Result</th></tr></thead><tbody><tr><td>Minimum Bolt Row Spacing</td><td>2.000</td><td>3.000</td><td>0.667</td><td>PASS</td></tr><tr><td>Maximum Bolt Row Spacing</td><td>6.000</td><td>3.000</td><td>0.500</td><td>PASS</td></tr><tr><td>Double Angle Minimum Vertical Edge Distance</td><td>1.000</td><td>1.500</td><td>0.667</td><td>PASS</td></tr><tr><td>Double Angle Minimum Horizontal Edge Distance</td><td>1.125</td><td>2.000</td><td>0.563</td><td>PASS</td></tr><tr><td>Beam Web Minimum Horizontal Edge Distance</td><td>1.000</td><td>2.000</td><td>0.500</td><td>PASS</td></tr><tr><td>Minimum Connection Depth</td><td>7.750</td><td>12.000</td><td>0.646</td><td>PASS</td></tr><tr><td>Maximum Connection Depth</td><td>16.000</td><td>12.000</td><td>0.750</td><td>PASS</td></tr><tr><td>Minimum Bolt Gage at Double Angle</td><td>1.750</td><td>3.000</td><td>0.583</td><td>PASS</td></tr></tbody></table> Result: Demand over Capacity Ratio $DCR = \frac{d}{c} = \frac{(12)}{(16)} = 0.75$	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	Double Angle Minimum Vertical Edge Distance	1.000	1.500	0.667	PASS	Double Angle Minimum Horizontal Edge Distance	1.125	2.000	0.563	PASS	Beam Web Minimum Horizontal Edge Distance	1.000	2.000	0.500	PASS	Minimum Connection Depth	7.750	12.000	0.646	PASS	Maximum Connection Depth	16.000	12.000	0.750	PASS	Minimum Bolt Gage at Double Angle	1.750	3.000	0.583	PASS	PASS
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AISC 15th Edition Table 10-9 AISC 360-16 Chapter J3.8 Eq. (J3-4) AISC 15th Ed. Page 7-6 to 7-8 AISC 15th Ed. Page 7-6 to 7-8	Check No. 2: Design Capacity of the Bolt Group in Shear at Beam Side Calculate the C-coefficient of the bolt group. $e = 3$ in - Design Bolt Group Eccentricity $\phi R_n = 16.136$ kip - Design Bolt Shear Capacity 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. <table><thead><tr><th>Bolt No.</th><th>x-coordinate (in)</th><th>y-coordinate (in)</th></tr></thead><tbody><tr><td>1</td><td>0.00</td><td>0.00</td></tr><tr><td>2</td><td>0.00</td><td>3.00</td></tr><tr><td>3</td><td>0.00</td><td>6.00</td></tr><tr><td>4</td><td>0.00</td><td>9.00</td></tr></tbody></table> 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. $x_{ic} = -2.2174$ in - Location of I.C. along the X-axis $y_{ic} = 7.7447$ 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. <table><thead><tr><th>Bolt No.</th><th>Ru (kip)</th><th>Rux (kip)</th><th>Ruy (kip)</th><th>Mn (kip-in)</th></tr></thead><tbody><tr><td>1</td><td>15.84</td><td>15.23</td><td>4.34</td><td>127.14</td></tr><tr><td>2</td><td>15.13</td><td>13.72</td><td>6.39</td><td>78.81</td></tr><tr><td>3</td><td>13.20</td><td>8.13</td><td>10.40</td><td>36.85</td></tr><tr><td>4</td><td>12.84</td><td>-6.46</td><td>11.10</td><td>32.68</td></tr></tbody></table> $C = 2.7549$ - Calculated Bolt Group Coefficient $C' = 11.256$ in - Calculated Bolt Group Coefficient for Moment-only Case Calculate the design shear capacity of the bolt group. $\phi = 0.85$ - Bolt Shear Resistance Factor $d_b = 0.75$ in - Bolt Diameter $C = 2.7549$ - Calculated Bolt Group Coefficient $\mu = 0.3$ - Mean Slip Coefficient $N_s = 2$ - Number of Slip Planes $h_f = 1$ - Filler Factor for SC Bolts $T_b = 28$ kip - Minimum Bolt Pretension ϕR_n - Design Bolt Shear Capacity $\phi R_n = \phi \mu D_u h_f T_b N_s C$ $\phi R_n = (0.85) \times (0.3) \times (1.13) \times (1) \times (28 \text{ kip}) \times (2) \times (2.7549)$ $\phi R_n = 44.454 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{R}{\phi R_n} = \frac{(16.651 \text{ kip})}{(44.454 \text{ kip})} = 0.37456$	Bolt No.	x-coordinate (in)	y-coordinate (in)	1	0.00	0.00	2	0.00	3.00	3	0.00	6.00	4	0.00	9.00	Bolt No.	Ru (kip)	Rux (kip)	Ruy (kip)	Mn (kip-in)	1	15.84	15.23	4.34	127.14	2	15.13	13.72	6.39	78.81	3	13.20	8.13	10.40	36.85	4	12.84	-6.46	11.10	32.68	PASS					
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AISC 360-16 Chapter J3.10 Eq. (J3-6a)	Check No. 3: Design Capacity of the Bolt Group in Bearing and Tear-out on Double Angle at Beam Side Calculate the bolt bearing capacity of the double angle. $d_b = 0.75$ in - Bolt Diameter $t_{2L,at,col,bm} = 0.5$ in - Beam Double Angle Thickness $F_{u,at,col,bm} = 58$ ksi - Beam Double Angle Tensile Stress $C = 2.7549$ - Calculated Bolt Group Coefficient $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_bearing}$ - Design Bolt Bearing Capacity of Section $\phi R_{n_bearing} = \phi 2.4 d_b [2(t_{2L,at,col,bm})] F_{u,at,col,bm} C$ $\phi R_{n_bearing} = (0.75) \times 2.4 \times (0.75 \text{ in}) \times (1 \text{ in}) \times (58 \text{ ksi}) \times (2.7549)$ $\phi R_{n_bearing} = 215.71 \text{ kip}$ Calculate the clear distance of outer bolts on double angle. $L_{ev} = 1.5$ in - Vertical Edge Distance at Double Angle $d_h = 0.8125$ in - Vertical Bolt Hole Dimension at Double Angle																																														

AISC 360-16 Chapter J3.10 Eq. (J3-6c) AISC 360-16 Chapter J3.10 Eq. (J3-6a) AISC 360-16 Chapter J3.10 Eq. (J3-6c) AISC 360-16 Chapter J3.10	l_{c1} - Clear Distance at First Bolt Row $l_{c1} = L_{ev} - \frac{d_h}{2}$ $l_{c1} = (1.5 \text{ in}) - \frac{(0.8125 \text{ in})}{2}$ $l_{c1} = 1.0938 \text{ in}$ Calculate the clear distance of inner bolts on double angle. $s_r = 3 \text{ in}$ - Bolt Row Spacing $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Double Angle 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 double angle. $l_{c1} = 1.0938 \text{ in}$ - Clear Distance at First Bolt Row $l_{c2} = 2.1875 \text{ in}$ - Clear Distance at Rest of Bolts $t_{2L,at,col,bm} = 0.5 \text{ in}$ - Beam Double Angle Thickness $F_{u,at,col,bm} = 58 \text{ ksi}$ - Beam Double Angle Tensile Stress $C = 2.7549$ - Calculated Bolt Group Coefficient $n_r = 4$ - Number of Bolt Rows $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_tearout}$ - Design Bolt Tear-out Capacity of Section $\phi R_{n_tearout} = \phi [1.2 l_{c1} [2(t_{2L,at,col,bm})] F_{u,at,col,bm} + 1.2 l_{c2} [2(t_{2L,at,col,bm})] F_{u,at,col,bm} (n_r - 1)] \left(\frac{C}{n_r} \right)$ $\phi R_{n_tearout} = (0.75) \times [1.2 \times (1.0938 \text{ in}) \times (1 \text{ in}) \times (58 \text{ ksi}) + 1.2 \times (2.1875 \text{ in}) \times (1 \text{ in}) \times (58 \text{ ksi}) \times ((4) - 1)] \times \left(\frac{(2.7549)}{(4)} \right)$ $\phi R_{n_tearout} = 275.25 \text{ kip}$ Determine the governing bearing and tear-out capacity of the bolt group on double angle. $\phi R_{n_bearing} = 215.71 \text{ kip}$ - Design bolt bearing capacity of double angle $\phi R_{n_tearout} = 275.25 \text{ kip}$ - Design bolt tear-out capacity of double angle ϕR_n - Governing Design Capacity $\phi R_n = \min(\phi R_{n_bearing}, \phi R_{n_tearout})$ $\phi R_n = \min((215.71 \text{ kip}), (275.25 \text{ kip}))$ $\phi R_n = 215.71 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{V_{fact}}{\phi R_n} = \frac{(11 \text{ kip})}{(215.71 \text{ kip})} = 0.050995$	PASS
AISC 360-16 Chapter J3.10 Eq. (J3-6a)	Check No. 4: Design Capacity of the Bolt Group in Bearing and Tear-out on Double Angle at Beam Side due to Axial Load Calculate the bolt bearing capacity of the double angle. $d_b = 0.75 \text{ in}$ - Bolt Diameter $t_{2L,at,col,bm} = 0.5 \text{ in}$ - Beam Double Angle Thickness $F_{u,at,col,bm} = 58 \text{ ksi}$ - Beam Double Angle Tensile Stress $C = 2.7549$ - Calculated Bolt Group Coefficient $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_bearing}$ - Design Bolt Bearing Capacity of Section $\phi R_{n_bearing} = \phi 2.4 d_b [2(t_{2L,at,col,bm})] F_{u,at,col,bm} C$ $\phi R_{n_bearing} = (0.75) \times 2.4 \times (0.75 \text{ in}) \times (1 \text{ in}) \times (58 \text{ ksi}) \times (2.7549)$ $\phi R_{n_bearing} = 215.71 \text{ kip}$ Calculate the clear distance of outer bolts on double angle. $L_{eh} = 2 \text{ in}$ - Horizontal Edge Distance at Double Angle $d_h = 1 \text{ in}$ - Horizontal Bolt Hole Dimension at Double Angle l_{c1} - Clear Distance at First Bolt Row $l_{c1} = L_{eh} - \frac{d_h}{2}$ $l_{c1} = (2 \text{ in}) - \frac{(1 \text{ in})}{2}$ $l_{c1} = 1.5 \text{ in}$ Calculate the clear distance of inner bolts on double angle. $s_c = 0 \text{ in}$ - Bolt Column Spacing $d_h = 1 \text{ in}$ - Horizontal Bolt Hole Dimension at Double Angle l_{c2} - Clear Distance at Rest of Bolts	

AISC 360-16 Chapter J3.10 Eq. (J3-6c) AISC 360-16 Chapter J3.10 Eq. (J3-6a) AISC 360-16 Chapter J3.10 Eq. (J3-6c) AISC 360-16 Chapter J3.10	$l_{c2} = s_c - d_h$ $l_{c2} = (0 \text{ in}) - (1 \text{ in})$ $l_{c2} = -1 \text{ in}$ Calculate the bolt tear-out capacity of the double angle. $l_{c1} = 1.5 \text{ in}$ - Clear Distance at First Bolt Row $l_{c2} = -1 \text{ in}$ - Clear Distance at Rest of Bolts $t_{2L,at,col,bm} = 0.5 \text{ in}$ - Beam Double Angle Thickness $F_{u,at,col,bm} = 58 \text{ ksi}$ - Beam Double Angle Tensile Stress $C = 2.7549$ - Calculated Bolt Group Coefficient $n_c = 1$ - Number of Bolt Columns $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_tearout}$ - Design Bolt Tear-out Capacity of Section $\phi R_{n_tearout} = \phi [1.2 l_{c1} [2(t_{2L,at,col,bm})] F_{u,at,col,bm} + 1.2 l_{c2} [2(t_{2L,at,col,bm})] F_{u,at,col,bm} (n_c - 1)] \left(\frac{C}{n_c} \right)$ $\phi R_{n_tearout} = (0.75) \times [1.2 \times (1.5 \text{ in}) \times (1 \text{ in}) \times (58 \text{ ksi}) + 1.2 \times (-1 \text{ in}) \times (1 \text{ in}) \times (58 \text{ ksi}) \times ((1) - 1)] \times \left(\frac{(2.7549)}{(1)} \right)$ $\phi R_{n_tearout} = 215.71 \text{ kip}$ Determine the governing bearing and tear-out capacity of the bolt group on double angle. $\phi R_{n_bearing} = 215.71 \text{ kip}$ - Design bolt bearing capacity of double angle $\phi R_{n_tearout} = 215.71 \text{ kip}$ - Design bolt tear-out capacity of double angle ϕR_n - Governing Design Capacity $\phi R_n = \min(\phi R_{n_bearing}, \phi R_{n_tearout})$ $\phi R_n = \min((215.71 \text{ kip}), (215.71 \text{ kip}))$ $\phi R_n = 215.71 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{P_{bm}}{\phi R_n} = \frac{(12.5 \text{ kip})}{(215.71 \text{ kip})} = 0.057947$	PASS
	Check No. 5: Shear and Axial Interaction of Bolt Bearing/Tearout in Double Angle Calculate the shear and axial interaction of bolt bearing/tearout capacities in the double angle. $V_{total} = 11 \text{ kip}$ $P_{bm} = 12.5 \text{ kip}$ $Interaction$ - Shear and Axial Interaction $Interaction = \sqrt{\left(\frac{V_{total}}{\phi R_n} \right)^2 + \left(\frac{P_{bm}}{\phi R_n} \right)^2}$ $Interaction = \sqrt{\left(\frac{(11 \text{ kip})}{(215.71 \text{ kip})} \right)^2 + \left(\frac{(12.5 \text{ kip})}{(215.71 \text{ kip})} \right)^2}$ $Interaction = 0.07719$ Result: Demand over Capacity Ratio $DCR = \frac{Interaction}{1.0} = \frac{(0.07719)}{(1)} = 0.07719$	PASS
AISC 360-16 Chapter J3.10 Eq. (J3-6a)	Check No. 6: Design Capacity of the Bolt Group in Bearing and Tear-out on Beam Web Calculate the bolt bearing capacity of the beam web. $d_b = 0.75 \text{ in}$ - Bolt Diameter $t_{w,bm} = 0.355 \text{ in}$ - Beam Web Thickness $F_{u,bm} = 65 \text{ ksi}$ - Beam Tensile Stress $C = 2.7549$ - Calculated Bolt Group Coefficient $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_bearing}$ - Design Bolt Bearing Capacity of Section $\phi R_{n_bearing} = \phi 2.4 d_b t_{w,bm} F_{u,bm} C$ $\phi R_{n_bearing} = (0.75) \times 2.4 \times (0.75 \text{ in}) \times (0.355 \text{ in}) \times (65 \text{ ksi}) \times (2.7549)$ $\phi R_{n_bearing} = 85.818 \text{ kip}$ 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 Web l_{c2} - Clear Distance at Rest of Bolts $l_{c2} = s_r - d_h$ $l_{c2} = (3 \text{ in}) - (0.8125 \text{ in})$ $l_{c2} = 2.1875 \text{ in}$ Calculate the bolt tear-out capacity of the beam web. $l_{c2} = 2.1875 \text{ in}$ - Clear Distance at Rest of Bolts	

AISC 360-16 Chapter J3.10 Eq. (J3-6c)	$t_{w,bm} = 0.355$ in - Beam Web Thickness $F_{u,bm} = 65$ ksi - Beam Tensile Stress $C = 2.7549$ - Calculated Bolt Group Coefficient $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_tearout}$ - Design Bolt Tear-out Capacity of Section $\phi R_{n_tearout} = \phi 1.2 l_{c2} t_{w,bm} F_{u,bm} C$ $\phi R_{n_tearout} = (0.75) \times 1.2 \times (2.1875 \text{ in}) \times (0.355 \text{ in}) \times (65 \text{ ksi}) \times (2.7549)$ $\phi R_{n_tearout} = 125.15 \text{ kip}$ Determine the governing bearing and tear-out capacity of the bolt group on beam web. $\phi R_{n_bearing} = 85.818$ kip - Design bolt bearing capacity of beam web $\phi R_{n_tearout} = 125.15$ kip - Design bolt tear-out capacity of beam web ϕR_n - Governing Design Capacity $\phi R_n = \min(\phi R_{n_bearing}, \phi R_{n_tearout})$ $\phi R_n = \min((85.818 \text{ kip}), (125.15 \text{ kip}))$ $\phi R_n = 85.818 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{V_{total}}{\phi R_n} = \frac{(11 \text{ kip})}{(85.818 \text{ kip})} = 0.12818$	PASS
AISC 360-16 Chapter J3.10 Eq. (J3-6a) AISC 360-16 Chapter J3.10 Eq. (J3-6c)	Check No. 7: Design Capacity of the Bolt Group in Bearing and Tear-out on Beam Web due to Axial Load Calculate the bolt bearing capacity of the beam web. $d_b = 0.75$ in - Bolt Diameter $t_{w,bm} = 0.355$ in - Beam Web Thickness $F_{u,bm} = 65$ ksi - Beam Tensile Stress $C = 2.7549$ - Calculated Bolt Group Coefficient $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_bearing}$ - Design Bolt Bearing Capacity of Section $\phi R_{n_bearing} = \phi 2.4 d_b t_{w,bm} F_{u,bm} C$ $\phi R_{n_bearing} = (0.75) \times 2.4 \times (0.75 \text{ in}) \times (0.355 \text{ in}) \times (65 \text{ ksi}) \times (2.7549)$ $\phi R_{n_bearing} = 85.818 \text{ kip}$ Calculate the clear distance of outer bolts on beam web. $L_{eh} = 2$ in - Horizontal Edge Distance at Beam Web $d_h = 0.8125$ in - Horizontal Bolt Hole Dimension at Beam Web l_{c1} - Clear Distance at First Bolt Row $l_{c1} = L_{eh} - \frac{d_h}{2}$ $l_{c1} = (2 \text{ in}) - \frac{(0.8125 \text{ in})}{2}$ $l_{c1} = 1.5938 \text{ in}$ Calculate the clear distance of inner bolts on beam web. $s_c = 0$ in - Bolt Column Spacing $d_h = 0.8125$ in - Horizontal Bolt Hole Dimension at Beam Web l_{c2} - Clear Distance at Rest of Bolts $l_{c2} = s_c - d_h$ $l_{c2} = (0 \text{ in}) - (0.8125 \text{ in})$ $l_{c2} = -0.8125 \text{ in}$ Calculate the bolt tear-out capacity of the beam web. $l_{c1} = 1.5938$ in - Clear Distance at First Bolt Row $l_{c2} = -0.8125$ in - Clear Distance at Rest of Bolts $t_{w,bm} = 0.355$ in - Beam Web Thickness $F_{u,bm} = 65$ ksi - Beam Tensile Stress $C = 2.7549$ - Calculated Bolt Group Coefficient $n_c = 1$ - Number of Bolt Columns $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_tearout}$ - Design Bolt Tear-out Capacity of Section $\phi R_{n_tearout} = \phi [1.2 l_{c1} t_{w,bm} F_{u,bm} + 1.2 l_{c2} t_{w,bm} F_{u,bm} (n_c - 1)] \left(\frac{C}{n_c} \right)$ $\phi R_{n_tearout} = (0.75) \times [1.2 \times (1.5938 \text{ in}) \times (0.355 \text{ in}) \times (65 \text{ ksi}) + 1.2 \times (-0.8125 \text{ in}) \times (0.355 \text{ in}) \times (65 \text{ ksi}) \times ((1) - 1)] \times \left(\frac{(2.7549)}{(1)} \right)$ $\phi R_{n_tearout} = 91.182 \text{ kip}$ Determine the governing bearing and tear-out capacity of the bolt group on beam web.	

AISC 360-16 Chapter J3.10 Eq. (J3-6a) AISC 360-16 Chapter J3.10 Eq. (J3-6c) AISC 360-16 Chapter J3.10	$\phi R_{n_bearing} = 85.818$ kip - Design bolt bearing capacity of beam web $\phi R_{n_tearout} = 91.182$ kip - Design bolt tear-out capacity of beam web ϕR_n - Governing Design Capacity $\phi R_n = \min(\phi R_{n_bearing}, \phi R_{n_tearout})$ $\phi R_n = \min((85.818 \text{ kip}), (91.182 \text{ kip}))$ $\phi R_n = 85.818 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{P_{bm}}{\phi R_n} = \frac{(12.5 \text{ kip})}{(85.818 \text{ kip})} = 0.14565$	PASS
	Check No. 8: Shear and Axial Interaction of Bolt Bearing/Tearout on Beam Web Calculate the shear and axial interaction of bolt bearing/tearout capacities in the beam web. $V_{total} = 11$ kip $P_{bm} = 12.5$ kip <i>Interaction</i> - Shear and Axial Interaction $Interaction = \sqrt{\left(\frac{V_{total}}{\phi R_n}\right)^2 + \left(\frac{P_{bm}}{\phi R_n}\right)^2}$ $Interaction = \sqrt{\left(\frac{(11 \text{ kip})}{(85.818 \text{ kip})}\right)^2 + \left(\frac{(12.5 \text{ kip})}{(85.818 \text{ kip})}\right)^2}$ $Interaction = 0.19402$ Result: Demand over Capacity Ratio $DCR = \frac{Interaction}{1.0} = \frac{(0.19402)}{(1)} = 0.19402$	PASS
AISC 360-16 Chapter J4.3 Eq. (J4-5)	Check No. 9: Design Capacity of Double Angle in Block Shear at Beam Side Calculate the net area of the double angle subject to tension. $t_{2L,at,col,bm} = 0.5$ in - Beam Double Angle Thickness $n_c = 1$ - Number of Bolt Columns $s_c = 0$ in - Bolt Column Spacing $L_{eh} = 2$ in - Horizontal Edge Distance at Double Angle $d_h = 1$ in - Horizontal Bolt Hole Dimension at Double Angle A_{nt} - Net Area Subject to Tension (L-pattern) $A_{nt} = [2(t_{2L,at,col,bm})] [(n_c - 1) s_c + L_{eh} - (n_c - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nt} = (1 \text{ in}) \times [(1 - 1) \times (0 \text{ in}) + (2 \text{ in}) - ((1) - 0.5) \times ((1 \text{ in}) + (0.0625 \text{ in}))]$ $A_{nt} = 1.4688 \text{ in}^2$ Calculate the gross area of the double angle subject to shear. $t_{2L,at,col,bm} = 0.5$ in - Beam Double Angle Thickness $L_{ev} = 1.5$ in - Vertical Edge Distance at Double Angle $n_r = 4$ - Number of Bolt Rows $s_r = 3$ in - Bolt Row Spacing A_{gv} - Gross Area Subject to Shear (L-pattern) $A_{gv} = [2(t_{2L,at,col,bm})] [L_{ev} + (n_r - 1) s_r]$ $A_{gv} = (1 \text{ in}) \times [(1.5 \text{ in}) + ((4) - 1) \times (3 \text{ in})]$ $A_{gv} = 10.5 \text{ in}^2$ Calculate the net area of the double angle subject to shear. $t_{2L,at,col,bm} = 0.5$ in - Beam Double Angle Thickness $L_{2L,at,col,bm} = 12$ in - Beam Double Angle Length $L_{ev} = 1.5$ in - Vertical Edge Distance at Double Angle $n_r = 4$ - Number of Bolt Rows $d_h = 0.8125$ in - Vertical Bolt Hole Dimension at Double Angle A_{nv} - Net Area Subject to Shear (L-pattern) $A_{nv} = [2(t_{2L,at,col,bm})] [L_{2L,at,col,bm} - L_{ev} - (n_r - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nv} = (1 \text{ in}) \times [(12 \text{ in}) - (1.5 \text{ in}) - ((4) - 0.5) \times ((0.8125 \text{ in}) + (0.0625 \text{ in}))]$ $A_{nv} = 7.4375 \text{ in}^2$ Calculate the design block shear capacity of the double angle. $F_{u,at,col,bm} = 58$ ksi - Beam Double Angle Tensile Stress $A_{nv} = 7.4375 \text{ in}^2$ - Net Area Subject to Shear (L-pattern) $U_{bs} = 1$ - Uniformity factor $A_{nt} = 1.4688 \text{ in}^2$ - Net Area Subject to Tension (L-pattern) $F_{y,at,col,bm} = 36$ ksi - Beam Double Angle Yield Stress $A_{gv} = 10.5 \text{ in}^2$ - Gross Area Subject to Shear (L-pattern) $\phi = 0.75$ - Block Shear Resistance Factor ϕR_n - Design Block Shear Capacity of Section $\phi R_n = \phi (0.6 F_{u,at,col,bm} A_{nv} + U_{bs} F_{u,at,col,bm} A_{nt} \leq 0.6 F_{y,at,col,bm} A_{gv} + U_{bs} F_{u,at,col,bm} A_{nt})$	

	$\phi R_n = (0.75) \times (0.6 \times (58 \text{ ksi}) \times (7.4375 \text{ in}^2) + (1) \times (58 \text{ ksi}) \times (1.4688 \text{ in}^2)) \leq 0.6 \times (36 \text{ ksi}) \times (10.5 \text{ in}^2) + (1) \times (58 \text{ ksi}) \times (1.4688 \text{ in}^2)$ $\phi R_n = 233.99 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{V_{total}}{\phi R_n} = \frac{(11 \text{ kip})}{(233.99 \text{ kip})} = 0.04701$	PASS
AISC 360-16 Chapter J4.3 Eq. (J4-5)	Check No. 10: Design Capacity of Double Angle in Block Shear at Beam Side due to Axial Load Calculate the net area of the double angle subject to tension. $t_{2L,at,col,bm} = 0.5 \text{ in}$ - Beam Double Angle Thickness $n_r = 4$ - Number of Bolt Rows $s_r = 3 \text{ in}$ - Bolt Row Spacing $L_{ev} = 1.5 \text{ in}$ - Vertical Edge Distance at Double Angle $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Double Angle A_{nt} - Net Area Subject to Tension (L-pattern) $A_{nt} = [2(t_{2L,at,col,bm})] [(n_r - 1) s_r + L_{ev} - (n_r - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nt} = (1 \text{ in}) \times [(4 - 1) \times (3 \text{ in}) + (1.5 \text{ in}) - ((4 - 0.5) \times ((0.8125 \text{ in}) + (0.0625 \text{ in})))]$ $A_{nt} = 7.4375 \text{ in}^2$ Calculate the gross area of the double angle subject to shear. $t_{2L,at,col,bm} = 0.5 \text{ in}$ - Beam Double Angle Thickness $L_{eh} = 2 \text{ in}$ - Horizontal Edge Distance at Double Angle $n_c = 1$ - Number of Bolt Columns $s_c = 0 \text{ in}$ - Bolt Column Spacing A_{gv} - Gross Area Subject to Shear (L-pattern) $A_{gv} = [2(t_{2L,at,col,bm})] [L_{eh} + (n_c - 1) s_c]$ $A_{gv} = (1 \text{ in}) \times [(2 \text{ in}) + ((1 - 1) \times (0 \text{ in}))]$ $A_{gv} = 2 \text{ in}^2$ Calculate the net area of the double angle subject to shear. $t_{2L,at,col,bm} = 0.5 \text{ in}$ - Beam Double Angle Thickness $L_{NSL,bm} = 5 \text{ in}$ - Beam Double Angle Leg at Beam Side $g_a = 3 \text{ in}$ - Double Angle Gage $n_c = 1$ - Number of Bolt Columns $d_h = 1 \text{ in}$ - Horizontal Bolt Hole Dimension at Double Angle A_{nv} - Net Area Subject to Shear (L-pattern) $A_{nv} = [2(t_{2L,at,col,bm})] [L_{NSL,bm} - g_a - (n_c - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nv} = (1 \text{ in}) \times [(5 \text{ in}) - (3 \text{ in}) - ((1 - 0.5) \times ((1 \text{ in}) + (0.0625 \text{ in})))]$ $A_{nv} = 1.4688 \text{ in}^2$ Calculate the design block shear capacity of the double angle. $F_{u,at,col,bm} = 58 \text{ ksi}$ - Beam Double Angle Tensile Stress $A_{nv} = 1.4688 \text{ in}^2$ - Net Area Subject to Shear (L-pattern) $U_{bs} = 1$ - Uniformity factor $A_{nt} = 7.4375 \text{ in}^2$ - Net Area Subject to Tension (L-pattern) $F_{y,at,col,bm} = 36 \text{ ksi}$ - Beam Double Angle Yield Stress $A_{gv} = 2 \text{ in}^2$ - Gross Area Subject to Shear (L-pattern) $\phi = 0.75$ - Block Shear Resistance Factor ϕR_n - Design Block Shear Capacity of Section $\phi R_n = \phi (0.6 F_{u,at,col,bm} A_{nv} + U_{bs} F_{u,at,col,bm} A_{nt}) \leq 0.6 F_{y,at,col,bm} A_{gv} + U_{bs} F_{u,at,col,bm} A_{nt}$ $\phi R_n = (0.75) \times (0.6 \times (58 \text{ ksi}) \times (1.4688 \text{ in}^2) + (1) \times (58 \text{ ksi}) \times (7.4375 \text{ in}^2)) \leq 0.6 \times (36 \text{ ksi}) \times (2 \text{ in}^2) + (1) \times (58 \text{ ksi}) \times (7.4375 \text{ in}^2)$ $\phi R_n = 355.93 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{T_{total}}{\phi R_n} = \frac{(12.5 \text{ kip})}{(355.93 \text{ kip})} = 0.035118$	PASS
	Check No. 11: Shear and Axial Interaction of Block Shear in Double Angle Calculate the shear and axial interaction of block shear capacities in the double angle. $V_{total} = 11 \text{ kip}$ $T_{total} = 12.5 \text{ kip}$ $Interaction$ - Shear and Axial Interaction $Interaction = \sqrt{\left(\frac{V_{total}}{\phi R_n}\right)^2 + \left(\frac{T_{total}}{\phi R_n}\right)^2}$ $Interaction = \sqrt{\left(\frac{(11 \text{ kip})}{(233.99 \text{ kip})}\right)^2 + \left(\frac{(12.5 \text{ kip})}{(355.93 \text{ kip})}\right)^2}$ $Interaction = 0.058679$	

	Result: Demand over Capacity Ratio $DCR = \frac{Interaction}{1.0} = \frac{(0.058679)}{(1)} = 0.058679$	PASS
AISC 360-16 Chapter J4.2 Eq. (J4-3)	Check No. 12: Design Capacity of Double Angle in Shear at Beam Side Calculate the gross area of double angle subject to yielding. $L_{2L,at,col,bm} = 12$ in - Beam Double Angle Length $t_{2L,at,col,bm} = 0.5$ in - Beam Double Angle Thickness A_{gv} - Section Gross Area $A_{gv} = L_{2L,at,col,bm} [2(t_{2L,at,col,bm})]$ $A_{gv} = (12 \text{ in}) \times (1 \text{ in})$ $A_{gv} = 12 \text{ in}^2$ Calculate the shear yielding capacity of the double angle. $F_{y,at,col,bm} = 36$ ksi - Beam Double Angle Yield Stress $A_{gv} = 12 \text{ in}^2$ - Section Gross Area $\phi = 1$ - Shear Yielding Resistance Factor $\phi R_{n_{sy}}$ - Design Shear Yielding Capacity of Section $\phi R_{n_{sy}} = \phi 0.6 F_{y,at,col,bm} A_{gv}$ $\phi R_{n_{sy}} = (1) \times 0.6 \times (36 \text{ ksi}) \times (12 \text{ in}^2)$ $\phi R_{n_{sy}} = 259.2 \text{ kip}$ Calculate the net area of double angle subject to rupture. $t_{2L,at,col,bm} = 0.5$ in - Beam Double Angle Thickness $L_{2L,at,col,bm} = 12$ in - Beam Double Angle Length $n_r = 4$ - Number of Bolt Rows $d_h = 0.8125$ in - Vertical Bolt Hole Dimension at Double Angle A_{nv} - Section Net Area $A_{nv} = [2(t_{2L,at,col,bm})] [L_{2L,at,col,bm} - n_r (d_h + 0.0625 \text{ in})]$ $A_{nv} = (1 \text{ in}) \times [(12 \text{ in}) - (4) \times ((0.8125 \text{ in}) + (0.0625 \text{ in}))]$ $A_{nv} = 8.5 \text{ in}^2$ Calculate the shear rupture capacity of the double angle. $F_{u,at,col,bm} = 58$ ksi - Beam Double Angle Tensile Stress $A_{nv} = 8.5 \text{ in}^2$ - Section Net Area $\phi = 0.75$ - Shear Rupture Resistance Factor $\phi R_{n_{sr}}$ - Design Shear Rupture Capacity of Section $\phi R_{n_{sr}} = \phi 0.6 F_{u,at,col,bm} A_{nv}$ $\phi R_{n_{sr}} = (0.75) \times 0.6 \times (58 \text{ ksi}) \times (8.5 \text{ in}^2)$ $\phi R_{n_{sr}} = 221.85 \text{ kip}$ Determine the governing shear capacity of the double angle. $\phi R_{n_{sy}} = 259.2$ kip - Design shear yielding capacity of double angle $\phi R_{n_{sr}} = 221.85$ kip - Design shear rupture capacity of double angle ϕR_n - Governing Design Capacity $\phi R_n = \min(\phi R_{n_{sy}}, \phi R_{n_{sr}})$ $\phi R_n = \min((259.2 \text{ kip}), (221.85 \text{ kip}))$ $\phi R_n = 221.85 \text{ kip}$	
AISC 360-16 Chapter J4.2 Eq. (J4-4)	Result: Demand over Capacity Ratio $DCR = \frac{V_{fact}}{\phi R_n} = \frac{(11 \text{ kip})}{(221.85 \text{ kip})} = 0.049583$	PASS
AISC 360-16 Chapter J4.2 Eq. (J4-3) AISC 360-16 Chapter J4.2 Eq. (J4-4) AISC 360-16 Chapter J4.2	Check No. 13: Design Capacity of Double Angle in Flexure Using Double Coped Procedure at Beam Side Calculate and compare upper and lower limits. $E = 29000$ ksi - Modulus for Steel $F_{y,at,col,bm} = 36$ ksi - Beam Double Angle Yield Stress $L_{b,2L} = 3$ in - Unbraced Length at Double Angle $L_{2L,at,col,bm} = 12$ in - Beam Double Angle Length $t_{2L,at,col,bm} = 0.5$ in - Beam Double Angle Thickness Depth to Thickness Ratio $\frac{L_{bd}}{t^2} = \frac{(3 \text{ in}) \times (12 \text{ in})}{(0.5 \text{ in})^2} = 144$ 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 $\frac{1.9E}{F_y} = \frac{1.9 \times (29000 \text{ ksi})}{(36 \text{ ksi})} = 1530.6$	

AISC 15th Edition pg. (9-9) Eq. (9-15) AISC 15th Edition pg. (9-9) Eq. (9-15) AISC 360-16 Chapter F11.2 Eq. (F11-2) AISC 360-16 Chapter F11.2 Eq. (F11-2)	So, $\frac{0.08E}{F_y} < \frac{L_b d}{t^2}$ and $\frac{L_b d}{t^2} \leq \frac{1.9E}{F_y}$. Calculate the plastic section modulus of the double angle. $t_{2L,at,col,bm} = 0.5$ in - Beam Double Angle Thickness $L_{2L,at,col,bm} = 12$ in - Beam Double Angle Length Plastic Section Modulus $Z = \frac{t_{2L,at,col,bm} (L_{2L,at,col,bm})^2}{4} = \frac{(0.5 \text{ in}) \times ((12 \text{ in})^2)}{4} = 18 \text{ in}^3$ Calculate the elastic section modulus of the double angle. $t_{2L,at,col,bm} = 0.5$ in - Beam Double Angle Thickness $L_{2L,at,col,bm} = 12$ in - Beam Double Angle Length Elastic Section Modulus $S = \frac{t_{2L,at,col,bm} (L_{2L,at,col,bm})^2}{6} = \frac{(0.5 \text{ in}) \times ((12 \text{ in})^2)}{6} = 12 \text{ in}^3$ Calculate the lateral torsional buckling modification factor. $L_{b,2L} = 3$ in - Unbraced Length at Double Angle $L_{2L,at,col,bm} = 12$ in - Beam Double Angle Length $d_{c,2L} = 0$ in - Double Angle Cope Depth (Assumed 0) C_b - Lateral-Torsional Buckling Modification Factor $C_b = \left[3 + \ln \left(\frac{L_{b,2L}}{L_{2L,at,col,bm}} \right) \right] \left(1 - \frac{d_{c,2L}}{L_{2L,at,col,bm}} \right) \geq 1.84$ $C_b = \left[3 + \ln \left(\frac{(3 \text{ in})}{(12 \text{ in})} \right) \right] \times \left(1 - \frac{(0 \text{ in})}{(12 \text{ in})} \right) \geq 1.84$ $C_b = 1.84$ Calculate the flexural capacity of the double angle. $C_b = 1.84$ - Lateral-Torsional Buckling Modification Factor $\frac{L_b d}{t^2} = 144$ - Depth to Thickness Ratio $F_{y,at,col,bm} = 36$ ksi - Beam Double Angle Yield Stress $E = 29000$ ksi - Modulus for Steel $S = 12 \text{ in}^3$ - Elastic Section Modulus $Z = 18 \text{ in}^3$ - Plastic Section Modulus $\phi = 0.9$ - Flexure Resistance Factor ϕM_n - Design Flexural Capacity of Coped Section $\phi M_n = \phi \left[C_b \left[1.52 - 0.274 \left(\frac{L_b d}{t^2} \right) \left(\frac{F_{y,at,col,bm}}{E} \right) \right] F_{y,at,col,bm} S \leq F_{y,at,col,bm} Z \right] 2$ $\phi M_n = (0.9) \times \left[(1.84) \times \left[1.52 - 0.274 \times ((144)) \times \left(\frac{(36 \text{ ksi})}{(29000 \text{ ksi})} \right) \right] \times (36 \text{ ksi}) \times (12 \text{ in}^3) \leq (36 \text{ ksi}) \times (18 \text{ in}^3) \right] \times 2$ $\phi M_n = 97.2 \text{ kip} \cdot \text{ft}$ Calculate the equivalent shear capacity of the double angle. $\phi M_n = 97.2 \text{ kip} \cdot \text{ft}$ - Design Flexural Capacity of Coped Section $L_{b,2L} = 3$ in - Unbraced Length at Double Angle ϕV_n - Design Capacity of Single Plate in Bending $\phi V_n = \frac{\phi M_n}{L_{b,2L}}$ $\phi V_n = \frac{(97.2 \text{ kip} \cdot \text{ft})}{(3 \text{ in})}$ $\phi V_n = 388.8 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{V_{actual}}{\phi V_n} = \frac{(11 \text{ kip})}{(388.8 \text{ kip})} = 0.028292$	PASS
AISC 360-16 Chapter J4.2 Eq. (J4-3) AISC 360-16 Chapter F11.2 Eq. (F11-2) AISC 15th Edition pg. (10-90) Eq. (10-5)	Check No. 14: Shear and Moment Interaction of Double Angle at Beam Side Calculate the shear and moment interaction of the double angle. $V_r = 11$ kip Moment at Component $M_r = V_r L_{b,2L} = (11 \text{ kip}) \times (3 \text{ in}) = 2.75 \text{ kip} \cdot \text{ft}$ $\phi V_n = 259.2 \text{ kip}$ - Design shear yielding capacity of double angle $\phi M_n = 97.2 \text{ kip} \cdot \text{ft}$ - Design Flexural Capacity of Double Angle I_{nt} - Interaction for Shear Yielding/Buckling and Flexure $I_{nt} = \left(\frac{V_r}{\phi V_n} \right)^2 + \left(\frac{M_r}{\phi M_n} \right)^2$ $I_{nt} = \left(\frac{(11 \text{ kip})}{(259.2 \text{ kip})} \right)^2 + \left(\frac{(2.75 \text{ kip} \cdot \text{ft})}{(97.2 \text{ kip} \cdot \text{ft})} \right)^2$ $I_{nt} = 0.0026015$ Result: Demand over Capacity Ratio $DCR = \frac{I_{nt}}{1.0} = \frac{(0.0026015)}{(1)} = 0.0026015$	PASS

AISC 360-16 Chapter J4.1 Eq. (J4-1) AISC 360-16 Chapter J4.1 Eq. (J4-2) AISC 360-16 Chapter J4.1 Eq. (J4-1) AISC 360-16 Chapter J4.1 Eq. (J4-2) AISC 360-16 Chapter J4.2	Check No. 15: Design Capacity of Double Angle in Tension Calculate the tensile yielding capacity of the double angle. $\phi = 0.9$ - Tensile Yielding Resistance Factor $F_{y,at,col,bm} = 36$ ksi - Beam Double Angle Yield Stress $t_{2L,at,col,bm} = 0.5$ in - Beam Double Angle Thickness $L_{2L,at,col,bm} = 12$ in - Beam Double Angle Length $\phi R_{n_{ty}}$ - Design Tension Yielding Capacity of Section $\phi R_{n_{ty}} = \phi F_{y,at,col,bm} [2(t_{2L,at,col,bm})] L_{2L,at,col,bm}$ $\phi R_{n_{ty}} = (0.9) \times (36 \text{ ksi}) \times (1 \text{ in}) \times (12 \text{ in})$ $\phi R_{n_{ty}} = 388.8 \text{ kip}$ Calculate the tensile rupture capacity of the double angle. $\phi = 0.75$ - Tensile Rupture Resistance Factor $F_{u,at,col,bm} = 58$ ksi - Beam Double Angle Tensile Stress $t_{2L,at,col,bm} = 0.5$ in - Beam Double Angle Thickness $L_{2L,at,col,bm} = 12$ in - Beam Double Angle Length $n_r = 4$ - Number of Bolt Rows $d_h = 0.8125$ in - Vertical Bolt Hole Dimension at Double Angle $\phi R_{n_{tr}}$ - Design Tension Rupture Capacity of Section $\phi R_{n_{tr}} = \phi F_{u,at,col,bm} [2(t_{2L,at,col,bm})] [L_{2L,at,col,bm} - n_r (d_h + 0.0625 \text{ in})]$ $\phi R_{n_{tr}} = (0.75) \times (58 \text{ ksi}) \times (1 \text{ in}) \times [(12 \text{ in}) - (4) \times ((0.8125 \text{ in}) + (0.0625 \text{ in}))]$ $\phi R_{n_{tr}} = 369.75 \text{ kip}$ Determine the governing tensile capacity of the double angle. $\phi R_{n_{ty}} = 388.8 \text{ kip}$ - Design Tension Yielding Capacity of Section $\phi R_{n_{tr}} = 369.75 \text{ kip}$ - Design Tension Rupture Capacity of Section ϕR_n - Governing Design Capacity $\phi R_n = \min(\phi R_{n_{ty}}, \phi R_{n_{tr}})$ $\phi R_n = \min((388.8 \text{ kip}), (369.75 \text{ kip}))$ $\phi R_n = 369.75 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{T_{total}}{\phi R_n} = \frac{(12.5 \text{ kip})}{(369.75 \text{ kip})} = 0.033806$	PASS
	Check No. 16: Tension and Flexure Interaction of the Double Angle Calculate the tension and flexure interaction of the double angle. $T_{total} = 12.5$ kip $V_r = 11$ kip <i>Interaction</i> - Tension and Flexure Interaction $Interaction = \frac{T_{total}}{2 \phi R_n} + \frac{V_r}{\phi V_n}$ $Interaction = \frac{(12.5 \text{ kip})}{2 \times (369.75 \text{ kip})} + \frac{(11 \text{ kip})}{(388.8 \text{ kip})}$ $Interaction = 0.045195$ Result: Demand over Capacity Ratio $DCR = \frac{Interaction}{1.0} = \frac{(0.045195)}{(1)} = 0.045195$	PASS
AISC 360-16 Chapter J4.4 Eq. (J4-6)	Check No. 17: Design Capacity of Double Angle in Compression Calculate the compression buckling capacity of the double angle. $\phi = 0.9$ - Compression Resistance Factor $F_{y,at,col,bm} = 36$ ksi - Beam Double Angle Yield Stress $t_{2L,at,col,bm} = 0.5$ in - Beam Double Angle Thickness $L_{2L,at,col,bm} = 12$ in - Beam Double Angle Length $K = 1.2$ - Effective Length Factor $g_a = 3$ in - Double Angle Gage $\frac{KL}{r} = 12.471$ - Effective Length Slenderness Ratio Since, $\frac{KL}{r} \leq 25$, ϕR_n - Design Compressive Capacity of Section $\phi R_n = \phi F_{y,at,col,bm} [2(t_{2L,at,col,bm})] L_{2L,at,col,bm}$ $\phi R_n = (0.9) \times (36 \text{ ksi}) \times (1 \text{ in}) \times (12 \text{ in})$ $\phi R_n = 388.8 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{C_{total}}{\phi R_n} = \frac{(12.5 \text{ kip})}{(388.8 \text{ kip})} = 0.032149$	PASS
	Check No. 18: Compression and Flexure Interaction of the Double Angle Calculate the compression and flexure interaction of the double angle.	

	$C_{total} = 12.5 \text{ kip}$ $V_r = 11 \text{ kip}$ <i>Interaction</i> - Compression and Flexure Interaction $Interaction = \frac{C_{total}}{2 \phi R_n} + \frac{V_r}{\phi V_n}$ $Interaction = \frac{(12.5 \text{ kip})}{2 \times (388.8 \text{ kip})} + \frac{(11 \text{ kip})}{(388.8 \text{ kip})}$ $Interaction = 0.044367$ Result: Demand over Capacity Ratio $DCR = \frac{Interaction}{1.0} = \frac{(0.044367)}{(1)} = 0.044367$	PASS
AISC 360-16 Chapter J4.3 Eq. (J4-5)	Check No. 19: Design Capacity of Beam Web in Block Shear due to Axial Load Calculate the net area of the beam web subject to tension. $t_{w,bm} = 0.355 \text{ in}$ - Beam Web Thickness $n_r = 4$ - Number of Bolt Rows $s_r = 3 \text{ in}$ - Bolt Row Spacing $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Beam Web A_{nt} - Net Area Subject to Tension (C-pattern) $A_{nt} = t_{w,bm} [(n_r - 1) s_r - (n_r - 1) (d_h + 0.0625 \text{ in})]$ $A_{nt} = (0.355 \text{ in}) \times [(4 - 1) \times (3 \text{ in}) - ((4 - 1) \times ((0.8125 \text{ in}) + (0.0625 \text{ in})))]$ $A_{nt} = 2.2631 \text{ in}^2$ Calculate the gross area of the beam web subject to shear. $t_{w,bm} = 0.355 \text{ in}$ - Beam Web Thickness $L_{eh} = 2 \text{ in}$ - Horizontal Edge Distance at Beam Web $n_c = 1$ - Number of Bolt Columns $s_c = 0 \text{ in}$ - Bolt Column Spacing A_{gv} - Gross Area Subject to Shear (C-pattern) $A_{gv} = 2 t_{w,bm} [L_{eh} + (n_c - 1) s_c]$ $A_{gv} = 2 \times (0.355 \text{ in}) \times [(2 \text{ in}) + ((1 - 1) \times (0 \text{ in}))]$ $A_{gv} = 1.42 \text{ in}^2$ Calculate the net area of the beam web subject to shear. $t_{w,bm} = 0.355 \text{ in}$ - Beam Web Thickness $L_{eh} = 2 \text{ in}$ - Horizontal Edge Distance at Beam Web $n_c = 1$ - Number of Bolt Columns $s_c = 0 \text{ in}$ - Bolt Column Spacing $d_h = 0.8125 \text{ in}$ - Horizontal Bolt Hole Dimension at Beam Web A_{nv} - Net Area Subject to Shear (C-pattern) $A_{nv} = 2 t_{w,bm} [(n_c - 1) s_c + L_{eh} - (n_c - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nv} = 2 \times (0.355 \text{ in}) \times [((1 - 1) \times (0 \text{ in}) + (2 \text{ in}) - ((1 - 0.5) \times ((0.8125 \text{ in}) + (0.0625 \text{ in})))]$ $A_{nv} = 1.1094 \text{ in}^2$ Calculate the design block shear capacity of the beam web. $F_{u,bm} = 65 \text{ ksi}$ - Beam Tensile Stress $A_{nt} = 2.2631 \text{ in}^2$ - Net Area Subject to Tension (C-pattern) $U_{bs} = 1$ - Uniformity factor $A_{gv} = 1.42 \text{ in}^2$ - Gross Area Subject to Shear (C-pattern) $F_{y,bm} = 50 \text{ ksi}$ - Beam Yield Stress $A_{nv} = 1.1094 \text{ in}^2$ - Net Area Subject to Shear (C-pattern) $\phi = 0.75$ - Block Shear Resistance Factor ϕR_n - Design Block Shear Capacity of Section $\phi R_n = \phi (0.6 F_{u,bm} A_{nv} + U_{bs} F_{u,bm} A_{nt} \leq 0.6 F_{y,bm} A_{gv} + U_{bs} F_{u,bm} A_{nt})$ $\phi R_n = (0.75) \times (0.6 \times (65 \text{ ksi}) \times (1.1094 \text{ in}^2) + (1) \times (65 \text{ ksi}) \times (2.2631 \text{ in}^2) \leq 0.6 \times (50 \text{ ksi}) \times (1.42 \text{ in}^2) + (1) \times (65 \text{ ksi}) \times (2.2631 \text{ in}^2))$ $\phi R_n = 142.28 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{T_{total}}{\phi R_n} = \frac{(12.5 \text{ kip})}{(142.28 \text{ kip})} = 0.087854$	PASS
	Check No. 20: Design Capacity of Beam in Tension Calculate the tensile rupture capacity of the beam. $\phi = 0.75$ - Tensile Rupture Resistance Factor $F_{u,bm} = 65 \text{ ksi}$ - Beam Tensile Stress $A_{bm} = 14.7 \text{ in}^2$ - Beam Area $t_{w,bm} = 0.355 \text{ in}$ - Beam Web Thickness $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Beam Web $n_r = 4$ - Number of Bolt Rows $n_c = 1$ - Number of Bolt Columns	

[illegible]

AISC 360-16 Chapter J3.10 Eq. (J3-6c) AISC 360-16 Chapter J3.10 Eq. (J3-6a) AISC 360-16 Chapter J3.10 Eq. (J3-6c) AISC 360-16 Chapter J3.10	$l_{c1} = (1.5 \text{ in}) - \frac{(0.8125 \text{ in})}{2}$ $l_{c1} = 1.0938 \text{ in}$ Calculate the clear distance of inner bolts on double angle. $s_r = 3 \text{ in}$ - Bolt Row Spacing $d_h = 0.8125 \text{ in}$ - Vertical Bolt Hole Dimension at Beam Double Angle 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 double angle. $l_{c1} = 1.0938 \text{ in}$ - Clear Distance at First Bolt Row $l_{c2} = 2.1875 \text{ in}$ - Clear Distance at Rest of Bolts $t_{2L,at,col,bm} = 0.5 \text{ in}$ - Beam Double Angle Thickness $F_{u,at,col,bm} = 58 \text{ ksi}$ - Beam Double Angle Tensile Stress $N_V = 8$ - Total Number of Bolts in Shear $n_r = 4$ - Number of Bolt Rows $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_tearout}$ - Design Bolt Tear-out Capacity of Section $\phi R_{n_tearout} = \phi [1.2 l_{c1} t_{2L,at,col,bm} F_{u,at,col,bm} + 1.2 l_{c2} t_{2L,at,col,bm} F_{u,at,col,bm} (n_r - 1)] \left(\frac{N_V}{n_r} \right)$ $\phi R_{n_tearout} = (0.75) \times [1.2 \times (1.0938 \text{ in}) \times (0.5 \text{ in}) \times (58 \text{ ksi}) + 1.2 \times (2.1875 \text{ in}) \times (0.5 \text{ in}) \times (58 \text{ ksi}) \times ((4) - 1)] \times \left(\frac{(8)}{(4)} \right)$ $\phi R_{n_tearout} = 399.66 \text{ kip}$ Determine the governing bearing and tear-out capacity of the bolt group on double angle. $\phi R_{n_bearing} = 313.2 \text{ kip}$ - Design bolt bearing capacity of double angle $\phi R_{n_tearout} = 399.66 \text{ kip}$ - Design bolt tear-out capacity of double angle ϕR_n - Governing Design Capacity $\phi R_n = \min(\phi R_{n_bearing}, \phi R_{n_tearout})$ $\phi R_n = \min((313.2 \text{ kip}), (399.66 \text{ kip}))$ $\phi R_n = 313.2 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{V_r}{\phi R_n} = \frac{(11 \text{ kip})}{(313.2 \text{ kip})} = 0.035121$	PASS
AISC 360-16 Chapter J3.10 Eq. (J3-6a) AISC 360-16 Chapter J3.10 Eq. (J3-6c)	Check No. 24: Design Capacity of the Bolt Group in Bearing and Tear-out at Support Flange Calculate the bolt bearing capacity of the beam web. $d_b = 0.75 \text{ in}$ - Bolt Diameter $t_{f,c} = 0.71 \text{ in}$ - Column Flange Thickness $F_{u,c} = 65 \text{ ksi}$ - Column Tensile Stress $N_V = 8$ - Total Number of Bolts in Shear $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_bearing}$ - Design Bolt Bearing Capacity of Section $\phi R_{n_bearing} = \phi 2.4 d_b t_{f,c} F_{u,c} N_V$ $\phi R_{n_bearing} = (0.75) \times 2.4 \times (0.75 \text{ in}) \times (0.71 \text{ in}) \times (65 \text{ ksi}) \times (8)$ $\phi R_{n_bearing} = 498.42 \text{ kip}$ 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 Column 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. $l_{c2} = 2.1875 \text{ in}$ - Clear Distance at Rest of Bolts $t_{f,c} = 0.71 \text{ in}$ - Column Flange Thickness $F_{u,c} = 65 \text{ ksi}$ - Column Tensile Stress $N_V = 8$ - Total Number of Bolts in Shear $\phi = 0.75$ - Bolt Bearing Resistance Factor $\phi R_{n_tearout}$ - Design Bolt Tear-out Capacity of Section $\phi R_{n_tearout} = \phi 1.2 l_{c2} t_{f,c} F_{u,c} N_V$	

AISC 360-16 Chapter J3.10 Eq. (J3-6a) AISC 360-16 Chapter J3.10 Eq. (J3-6c) AISC 360-16 Chapter J3.10	$\phi R_{n_tearout} = (0.75) \times 1.2 \times (2.1875 \text{ in}) \times (0.71 \text{ in}) \times (65 \text{ ksi}) \times (8)$ $\phi R_{n_tearout} = 726.86 \text{ kip}$ Determine the governing bearing and tear-out capacity of the bolt group on beam web. $\phi R_{n_bearing} = 498.42 \text{ kip} - \text{Design bolt bearing capacity of beam web}$ $\phi R_{n_tearout} = 726.86 \text{ kip} - \text{Design bolt tear-out capacity of beam web}$ $\phi R_n - \text{Governing Design Capacity}$ $\phi R_n = \min(\phi R_{n_bearing}, \phi R_{n_tearout})$ $\phi R_n = \min((498.42 \text{ kip}), (726.86 \text{ kip}))$ $\phi R_n = 498.42 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{V_r}{\phi R_n} = \frac{(11 \text{ kip})}{(498.42 \text{ kip})} = 0.02207$	PASS
AISC 360-16 Chapter J4.3 Eq. (J4-5)	Check No. 25: Design Capacity of Double Angle in Block Shear at Support Side Calculate the net area of the double angle subject to tension. $t_{2L,at,col,bm} = 0.5 \text{ in} - \text{Beam Double Angle Thickness}$ $n_c = 1 - \text{Number of Bolt Columns}$ $s_c = 0 \text{ in} - \text{Bolt Column Spacing}$ $L_{eh} = 1.6775 \text{ in} - \text{Horizontal Edge Distance at Double Angle}$ $d_h = 1 \text{ in} - \text{Horizontal Bolt Hole Dimension at Beam Double Angle}$ $A_{nt} - \text{Net Area Subject to Tension (L-pattern)}$ $A_{nt} = [2(t_{2L,at,col,bm})] [(n_c - 1) s_c + L_{eh} - (n_c - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nt} = (1 \text{ in}) \times [(1 - 1) \times (0 \text{ in}) + (1.6775 \text{ in}) - ((1) - 0.5) \times ((1 \text{ in}) + (0.0625 \text{ in}))]$ $A_{nt} = 1.1462 \text{ in}^2$ Calculate the gross area of the double angle subject to shear. $t_{2L,at,col,bm} = 0.5 \text{ in} - \text{Beam Double Angle Thickness}$ $L_{ev} = 1.5 \text{ in} - \text{Vertical Edge Distance at Double Angle}$ $n_r = 4 - \text{Number of Bolt Rows}$ $s_r = 3 \text{ in} - \text{Bolt Row Spacing}$ $A_{gv} - \text{Gross Area Subject to Shear (L-pattern)}$ $A_{gv} = [2(t_{2L,at,col,bm})] [L_{ev} + (n_r - 1) s_r]$ $A_{gv} = (1 \text{ in}) \times [(1.5 \text{ in}) + ((4) - 1) \times (3 \text{ in})]$ $A_{gv} = 10.5 \text{ in}^2$ Calculate the net area of the double angle subject to shear. $t_{2L,at,col,bm} = 0.5 \text{ in} - \text{Beam Double Angle Thickness}$ $L_{2L,at,col,bm} = 12 \text{ in} - \text{Beam Double Angle Length}$ $L_{ev} = 1.5 \text{ in} - \text{Vertical Edge Distance at Double Angle}$ $n_r = 4 - \text{Number of Bolt Rows}$ $d_h = 0.8125 \text{ in} - \text{Vertical Bolt Hole Dimension at Beam Double Angle}$ $A_{nv} - \text{Net Area Subject to Shear (L-pattern)}$ $A_{nv} = [2(t_{2L,at,col,bm})] [L_{2L,at,col,bm} - L_{ev} - (n_r - 0.5) (d_h + 0.0625 \text{ in})]$ $A_{nv} = (1 \text{ in}) \times [(12 \text{ in}) - (1.5 \text{ in}) - ((4) - 0.5) \times ((0.8125 \text{ in}) + (0.0625 \text{ in}))]$ $A_{nv} = 7.4375 \text{ in}^2$ Calculate the design block shear capacity of the double angle. $F_{u,at,col,bm} = 58 \text{ ksi} - \text{Beam Double Angle Tensile Stress}$ $A_{nv} = 7.4375 \text{ in}^2 - \text{Net Area Subject to Shear (L-pattern)}$ $U_{bs} = 1 - \text{Uniformity factor}$ $A_{nt} = 1.1462 \text{ in}^2 - \text{Net Area Subject to Tension (L-pattern)}$ $F_{y,at,col,bm} = 36 \text{ ksi} - \text{Beam Double Angle Yield Stress}$ $A_{gv} = 10.5 \text{ in}^2 - \text{Gross Area Subject to Shear (L-pattern)}$ $\phi = 0.75 - \text{Block Shear Resistance Factor}$ $\phi R_n - \text{Design Block Shear Capacity of Section}$ $\phi R_n = \phi (0.6 F_{u,at,col,bm} A_{nv} + U_{bs} F_{u,at,col,bm} A_{nt} \leq 0.6 F_{y,at,col,bm} A_{gv} + U_{bs} F_{u,at,col,bm} A_{nt})$ $\phi R_n = (0.75) \times (0.6 \times (58 \text{ ksi}) \times (7.4375 \text{ in}^2) + (1) \times (58 \text{ ksi}) \times (1.1462 \text{ in}^2) \leq 0.6 \times (36 \text{ ksi}) \times (10.5 \text{ in}^2) + (1) \times (58 \text{ ksi}) \times (1.1462 \text{ in}^2))$ $\phi R_n = 219.96 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{V_r}{\phi R_n} = \frac{(11 \text{ kip})}{(219.96 \text{ kip})} = 0.050009$	PASS
	Check No. 26: Design Capacity of Double Angle in Shear at Support Side Calculate the gross area of double angle subject to yielding. $L_{2L,at,col,bm} = 12 \text{ in} - \text{Beam Double Angle Length}$	

AISC 360-16 Chapter J4.2 Eq. (J4-3)	$t_{2L,at,col,bm} = 0.5$ in - Beam Double Angle Thickness A_{gv} - Section Gross Area $A_{gv} = L_{2L,at,col,bm} [2(t_{2L,at,col,bm})]$ $A_{gv} = (12 \text{ in}) \times (1 \text{ in})$ $A_{gv} = 12 \text{ in}^2$ Calculate the shear yielding capacity of the double angle. $F_{y,at,col,bm} = 36$ ksi - Beam Double Angle Yield Stress $A_{gv} = 12 \text{ in}^2$ - Section Gross Area $\phi = 1$ - Shear Yielding Resistance Factor $\phi R_{n_{sy}}$ - Design Shear Yielding Capacity of Section $\phi R_{n_{sy}} = \phi 0.6 F_{y,at,col,bm} A_{gv}$ $\phi R_{n_{sy}} = (1) \times 0.6 \times (36 \text{ ksi}) \times (12 \text{ in}^2)$ $\phi R_{n_{sy}} = 259.2 \text{ kip}$ Calculate the net area of double angle subject to rupture. $t_{2L,at,col,bm} = 0.5$ in - Beam Double Angle Thickness $L_{2L,at,col,bm} = 12$ in - Beam Double Angle Length $n_r = 4$ - Number of Bolt Rows $d_h = 0.8125$ in - Vertical Bolt Hole Dimension at Beam Double Angle A_{nv} - Section Net Area $A_{nv} = [2(t_{2L,at,col,bm})] [L_{2L,at,col,bm} - n_r (d_h + 0.0625 \text{ in})]$ $A_{nv} = (1 \text{ in}) \times [(12 \text{ in}) - (4) \times ((0.8125 \text{ in}) + (0.0625 \text{ in}))]$ $A_{nv} = 8.5 \text{ in}^2$ Calculate the shear rupture capacity of the double angle. $F_{u,at,col,bm} = 58$ ksi - Beam Double Angle Tensile Stress $A_{nv} = 8.5 \text{ in}^2$ - Section Net Area $\phi = 0.75$ - Shear Rupture Resistance Factor $\phi R_{n_{sr}}$ - Design Shear Rupture Capacity of Section $\phi R_{n_{sr}} = \phi 0.6 F_{u,at,col,bm} A_{nv}$ $\phi R_{n_{sr}} = (0.75) \times 0.6 \times (58 \text{ ksi}) \times (8.5 \text{ in}^2)$ $\phi R_{n_{sr}} = 221.85 \text{ kip}$ Determine the governing shear capacity of the double angle. $\phi R_{n_{sy}} = 259.2$ kip - Design shear yielding capacity of double angle $\phi R_{n_{sr}} = 221.85$ kip - Design shear rupture capacity of double angle ϕR_n - Governing Design Capacity $\phi R_n = \min(\phi R_{n_{sy}}, \phi R_{n_{sr}})$ $\phi R_n = \min((259.2 \text{ kip}), (221.85 \text{ kip}))$ $\phi R_n = 221.85 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{V_r}{\phi R_n} = \frac{(11 \text{ kip})}{(221.85 \text{ kip})} = 0.049583$	PASS
	Check No. 27: Design Capacity of Double Angle in Flexure Using Double Coped Procedure at Support Side Calculate and compare upper and lower limits. $E = 29000$ ksi - Modulus for Steel $F_{y,at,col,bm} = 36$ ksi - Beam Double Angle Yield Stress $L_{b,2L} = 3.3225$ in - Unbraced Length at Double Angle $L_{2L,at,col,bm} = 12$ in - Beam Double Angle Length $t_{2L,at,col,bm} = 0.5$ in - Beam Double Angle Thickness Depth to Thickness Ratio $\frac{L_{bd}}{t^2} = \frac{(3.3225 \text{ in}) \times (12 \text{ in})}{(0.5 \text{ in})^2} = 159.48$ 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, $\frac{0.08E}{F_y} < \frac{L_{bd}}{t^2}$ and $\frac{L_{bd}}{t^2} \leq \frac{1.9E}{F_y}$. Calculate the plastic section modulus of the double angle. $t_{2L,at,col,bm} = 0.5$ in - Beam Double Angle Thickness $L_{2L,at,col,bm} = 12$ in - Beam Double Angle Length Plastic Section Modulus $Z = \frac{t_{2L,at,col,bm} (L_{2L,at,col,bm})^2}{4} = \frac{(0.5 \text{ in}) \times ((12 \text{ in}))^2}{4} = 18 \text{ in}^3$	

AISC 15th Edition pg. (9-9) Eq. (9-15) AISC 15th Edition pg. (9-9) Eq. (9-15) AISC 360-16 Chapter F11.2 Eq. (F11-2) AISC 360-16 Chapter F11.2 Eq. (F11-2)	Calculate the elastic section modulus of the double angle. $t_{2L,at,col,bm} = 0.5$ in - Beam Double Angle Thickness $L_{2L,at,col,bm} = 12$ in - Beam Double Angle Length Elastic Section Modulus $S = \frac{t_{2L,at,col,bm} (L_{2L,at,col,bm})^2}{6} = \frac{(0.5 \text{ in}) \times ((12 \text{ in})^2)}{6} = 12 \text{ in}^3$ Calculate the lateral torsional buckling modification factor. $L_{b,2L} = 3.3225$ in - Unbraced Length at Double Angle $L_{2L,at,col,bm} = 12$ in - Beam Double Angle Length $d_{c,2L} = 0$ in - Double Angle Cope Depth (Assumed 0) C_b - Lateral-Torsional Buckling Modification Factor $C_b = \left[3 + \ln \left(\frac{L_{b,2L}}{L_{2L,at,col,bm}} \right) \right] \left(1 - \frac{d_{c,2L}}{L_{2L,at,col,bm}} \right) \geq 1.84$ $C_b = \left[3 + \ln \left(\frac{(3.3225 \text{ in})}{(12 \text{ in})} \right) \right] \times \left(1 - \frac{(0 \text{ in})}{(12 \text{ in})} \right) \geq 1.84$ $C_b = 1.84$ Calculate the flexural capacity of the double angle. $C_b = 1.84$ - Lateral-Torsional Buckling Modification Factor $\frac{L_{yd}}{t^2} = 159.48$ - Depth to Thickness Ratio $F_{y,at,col,bm} = 36$ ksi - Beam Double Angle Yield Stress $E = 29000$ ksi - Modulus for Steel $S = 12 \text{ in}^3$ - Elastic Section Modulus $Z = 18 \text{ in}^3$ - Plastic Section Modulus $\phi = 0.9$ - Flexure Resistance Factor ϕM_n - Design Flexural Capacity of Coped Section $\phi M_n = \phi \left[C_b \left[1.52 - 0.274 \left(\frac{L_b d}{t^2} \right) \left(\frac{F_{y,at,col,bm}}{E} \right) \right] F_{y,at,col,bm} S \leq F_{y,at,col,bm} Z \right] 2$ $\phi M_n = (0.9) \times \left[(1.84) \times \left[1.52 - 0.274 \times ((159.48)) \times \left(\frac{(36 \text{ ksi})}{(29000 \text{ ksi})} \right) \right] \times (36 \text{ ksi}) \times (12 \text{ in}^3) \leq (36 \text{ ksi}) \times (18 \text{ in}^3) \right] \times 2$ $\phi M_n = 97.2 \text{ kip} \cdot \text{ft}$ Calculate the equivalent shear capacity of the double angle. $\phi M_n = 97.2 \text{ kip} \cdot \text{ft}$ - Design Flexural Capacity of Coped Section $L_{b,2L} = 3.3225$ in - Unbraced Length at Double Angle ϕV_n - Design Capacity of Single Plate in Bending $\phi V_n = \frac{\phi M_n}{L_{b,2L}}$ $\phi V_n = \frac{(97.2 \text{ kip} \cdot \text{ft})}{(3.3225 \text{ in})}$ $\phi V_n = 351.06 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{V_r}{\phi V_n} = \frac{(11 \text{ kip})}{(351.06 \text{ kip})} = 0.031334$	PASS
AISC 360-16 Chapter F4.2 Eq. (F4-3) AISC 360-16 Chapter F11.2 Eq. (F11-2) AISC 15th Edition pg. (10-90) Eq. (10-5)	Check No. 28: Shear and Moment Interaction of Double Angle at Support Side Calculate the shear and moment interaction of the double angle. $V_r = 11$ kip Moment at Component $M_r = V_r L_{b,2L} = (11 \text{ kip}) \times (3.3225 \text{ in}) = 3.0456 \text{ kip} \cdot \text{ft}$ $\phi V_n = 259.2$ kip - Design shear yielding capacity of double angle $\phi M_n = 97.2 \text{ kip} \cdot \text{ft}$ - Design Flexural Capacity of Double Angle I_{nt} - Interaction for Shear Yielding/Buckling and Flexure $I_{nt} = \left(\frac{V_r}{\phi V_n} \right)^2 + \left(\frac{M_r}{\phi M_n} \right)^2$ $I_{nt} = \left(\frac{(11 \text{ kip})}{(259.2 \text{ kip})} \right)^2 + \left(\frac{(3.0456 \text{ kip} \cdot \text{ft})}{(97.2 \text{ kip} \cdot \text{ft})} \right)^2$ $I_{nt} = 0.0027828$ Result: Demand over Capacity Ratio $DCR = \frac{I_{nt}}{1.0} = \frac{(0.0027828)}{(1)} = 0.0027828$	PASS
	Check No. 29: Design Capacity of Double Angle in Bolt Prying $d_b = 0.75$ in - Bolt Diameter $n_r = 4$ - Number of Bolt Rows $s_r = 3$ in - Bolt Row Spacing $T_{total} = 12.5$ kip $F_{u,at,col,bm} = 58$ ksi - Beam Double Angle Tensile Stress $b = 3.0725$ in - b prying	

AISC 15th Edition Equation (9-19a)	$F_{nv} = 54 \text{ ksi}$ - Bolt Nominal Shear Strength $F_{nt} = 90 \text{ ksi}$ - Bolt Nominal Tensile Strength $V_r = 11 \text{ kip}$ $L_{eh} = 1.6775 \text{ in}$ - Horizontal Edge Distance at Double Angle $d_h = 1 \text{ in}$ - Horizontal Bolt Hole Dimension at Beam Double Angle $B = 29.818 \text{ kip}$ - Available tension per bolt δ $\delta = 1 - \frac{d_h}{\min(3.5 b, s_r)}$ $\delta = 1 - \frac{(1 \text{ in})}{\min(3.5 \times (3.0725 \text{ in}), (3 \text{ in}))}$ $\delta = 0.66667$ β $\beta = \frac{\min(L_{eh} + 0.5 d_b, 1.25 b + 0.5 d_b) (2 T_{total} - 4 B n_r)}{T_{total} (d_b - 2 b)}$ $\beta = \frac{\min((1.6775 \text{ in}) + 0.5 \times (0.75 \text{ in}), 1.25 \times (3.0725 \text{ in}) + 0.5 \times (0.75 \text{ in})) \times (2 \times (12.5 \text{ kip}) - 4 \times (29.818 \text{ kip}) \times (4))}{(12.5 \text{ kip}) \times ((0.75 \text{ in}) - 2 \times (3.0725 \text{ in}))}$ $\beta = 13.76$ α' $\alpha' = \min \left[1, \left \frac{1}{\delta} \left(\frac{\beta}{1 - \beta} \right) \right \right]$ $\alpha' = \min \left[1, \left \frac{1}{(0.66667)} \times \left(\frac{(13.76)}{1 - (13.76)} \right) \right \right]$ $\alpha' = 1$ t_{min} - Minimum thickness required to ensure an acceptable combination of fitting strength and stiffness and bolt strength $t_{min} = \sqrt{\frac{2.22 T_{total} (b - 0.5 d_b)}{\min(3.5 b, s_r) F_{u,at,col,bm} n_r (1 + \delta \alpha')}}}$ $t_{min} = \sqrt{\frac{2.22 \times (12.5 \text{ kip}) \times ((3.0725 \text{ in}) - 0.5 \times (0.75 \text{ in}))}{\min(3.5 \times (3.0725 \text{ in}), (3 \text{ in})) \times (58 \text{ ksi}) \times (4) \times (1 + (0.66667) \times (1))}}$ $t_{min} = 0.25403 \text{ in}$ Result: Demand over Capacity Ratio $DCR = \frac{t_{min}}{t_{2L,at,col,bm}} = \frac{(0.25403 \text{ in})}{(0.5 \text{ in})} = 0.50805$	PASS
	Check No. 30: Design Capacity of Column Flange in Bolt Prying $d_b = 0.75 \text{ in}$ - Bolt Diameter $n_r = 4$ - Number of Bolt Rows $s_r = 3 \text{ in}$ - Bolt Row Spacing $V_r = 11 \text{ kip}$ $T_{total} = 12.5 \text{ kip}$ $F_{u,c} = 65 \text{ ksi}$ - Column Tensile Stress $b = 3.28 \text{ in}$ - b prying $F_{nv} = 54 \text{ ksi}$ - Bolt Nominal Shear Strength $F_{nt} = 90 \text{ ksi}$ - Bolt Nominal Tensile Strength $L_{eh} = 3.75 \text{ in}$ - Horizontal Edge Distance at Column $d_h = 0.8125 \text{ in}$ - Horizontal Bolt Hole Dimension at Column $B = 29.818 \text{ kip}$ - Available tension per bolt δ $\delta = 1 - \frac{d_h}{\min(3.5 b, s_r)}$ $\delta = 1 - \frac{(0.8125 \text{ in})}{\min(3.5 \times (3.28 \text{ in}), (3 \text{ in}))}$ $\delta = 0.72917$ β $\beta = \frac{\min(L_{eh} + 0.5 d_b, 1.25 b + 0.5 d_b) (2 T_{total} - 4 B n_r)}{T_{total} (d_b - 2 b)}$ $\beta = \frac{\min((3.75 \text{ in}) + 0.5 \times (0.75 \text{ in}), 1.25 \times (3.28 \text{ in}) + 0.5 \times (0.75 \text{ in})) \times (2 \times (12.5 \text{ kip}) - 4 \times (29.818 \text{ kip}) \times (4))}{(12.5 \text{ kip}) \times ((0.75 \text{ in}) - 2 \times (3.28 \text{ in}))}$ $\beta = 25.679$ α' $\alpha' = \min \left[1, \left \frac{1}{\delta} \left(\frac{\beta}{1 - \beta} \right) \right \right]$	

AISC 15th Edition Equation (9-19a)	$\alpha' = \min \left[1, \left \frac{1}{(0.72917)} \times \left(\frac{(25.679)}{1 - (25.679)} \right) \right \right]$ $\alpha' = 1$ t_{min} - Minimum thickness required to ensure an acceptable combination of fitting strength and stiffness and bolt strength $t_{min} = \sqrt{\frac{2.22 T_{total} (b - 0.5 d_b)}{\min(3.5 b, s_r) F_{u,c} n_r (1 + \delta \alpha')}}}$ $t_{min} = \sqrt{\frac{2.22 \times (12.5 \text{ kip}) \times ((3.28 \text{ in}) - 0.5 \times (0.75 \text{ in}))}{\min(3.5 \times (3.28 \text{ in}), (3 \text{ in})) \times (65 \text{ ksi}) \times (4) \times (1 + (0.72917) \times (1))}}$ $t_{min} = 0.24447 \text{ in}$ Result: Demand over Capacity Ratio $DCR = \frac{t_{min}}{t_{f,c}} = \frac{(0.24447 \text{ in})}{(0.71 \text{ in})} = 0.34433$	PASS
AISC 360-16 Chapter J10.3 Eq. (J10-4) AISC 360-16 Chapter J10.2 Eq. (J10-2) AISC 360-16 Chapter J10	Check No. 31: Design Capacity of Support due to Compressive Load Calculate the web local crippling capacity of the column. $\phi = 0.75$ - Web Local Crippling Resistance Factor $t_{w,c} = 0.44 \text{ in}$ - Column Web Thickness $L_{2L,at,col,bm} = 12 \text{ in}$ - Beam Double Angle Length $d_c = 14 \text{ in}$ - Column Depth $t_{f,c} = 0.71 \text{ in}$ - Column Flange Thickness $F_{y,c} = 50 \text{ ksi}$ - Column Yield Stress $E = 29000 \text{ ksi}$ - Modulus for Steel $\phi R_{n,wlc}$ - Design Web Local Crippling Capacity of Support $\phi R_{n,wlc} = \phi 0.8 (t_{w,c})^2 \left[1 + 3 \left(\frac{L_{2L,at,col,bm}}{d_c} \right) \left(\frac{t_{w,c}}{t_{f,c}} \right)^{1.5} \right] \sqrt{\frac{E F_{y,c} t_{f,c}}{t_{w,c}}} Q_f$ $\phi R_{n,wlc} = (0.75) \times 0.8 \times ((0.44 \text{ in}))^2 \times \left[1 + 3 \times \left(\frac{(12 \text{ in})}{(14 \text{ in})} \right) \times \left(\frac{(0.44 \text{ in})}{(0.71 \text{ in})} \right)^{1.5} \right] \times \sqrt{\frac{(29000 \text{ ksi}) \times (50 \text{ ksi}) \times (0.71 \text{ in})}{(0.44 \text{ in})}} \times (1)$ $\phi R_{n,wlc} = 400.58 \text{ kip}$ Calculate the web local yielding capacity of the column. $\phi = 1$ - Web Local Yielding Resistance Factor $F_{y,c} = 50 \text{ ksi}$ - Column Yield Stress $t_{w,c} = 0.44 \text{ in}$ - Column Web Thickness $k_{des,c} = 1.31 \text{ in}$ - Column kdes $L_{2L,at,col,bm} = 12 \text{ in}$ - Beam Double Angle Length $\phi R_{n,wly}$ - Design Web Local Yielding Capacity of Support $\phi R_{n,wly} = \phi F_{y,c} t_{w,c} (5 k_{des,c} + L_{2L,at,col,bm})$ $\phi R_{n,wly} = (1) \times (50 \text{ ksi}) \times (0.44 \text{ in}) \times (5 \times (1.31 \text{ in}) + (12 \text{ in}))$ $\phi R_{n,wly} = 408.1 \text{ kip}$ Calculate for the governing design capacity of the column. ϕR_n - Governing Design Capacity of the Support $\phi R_n = \min(\phi R_{n,wlc}, \phi R_{n,wly})$ $\phi R_n = \min((400.58 \text{ kip}), (408.1 \text{ kip}))$ $\phi R_n = 400.58 \text{ kip}$ Result: Demand over Capacity Ratio $DCR = \frac{C_{total}}{\phi R_n} = \frac{(12.5 \text{ kip})}{(400.58 \text{ kip})} = 0.031204$	PASS
	Summary of Checks	

Design Checks	Demand	Capacity	DCR	Result
Connection Detailing Limitations at Beam Side	12.000	16.000	0.750	PASS
Design Capacity of the Bolt Group in Shear at Beam Side	16.651	44.454	0.375	PASS
Design Capacity of the Bolt Group in Bearing and Tear-out on Double Angle at Beam Side	11.000	215.708	0.051	PASS
Design Capacity of the Bolt Group in Bearing and Tear-out on Double Angle at Beam Side due to Axial Load	12.500	215.708	0.058	PASS
Shear and Axial Interaction of Bolt Bearing/Tearout in Double Angle	0.077	1.000	0.077	PASS
Design Capacity of the Bolt Group in Bearing and Tear-out on Beam Web	11.000	85.818	0.128	PASS
Design Capacity of the Bolt Group in Bearing and Tear-out on Beam Web due to Axial Load	12.500	85.818	0.146	PASS
Shear and Axial Interaction of Bolt Bearing/Tearout on Beam Web	0.194	1.000	0.194	PASS
Design Capacity of Double Angle in Block Shear at Beam Side	11.000	233.991	0.047	PASS
Design Capacity of Double Angle in Block Shear at Beam Side due to Axial Load	12.500	355.931	0.035	PASS
Shear and Axial Interaction of Block Shear in Double Angle	0.059	1.000	0.059	PASS
Design Capacity of Double Angle in Shear at Beam Side	11.000	221.850	0.050	PASS
Design Capacity of Double Angle in Flexure Using Double Coped Procedure at Beam Side	11.000	388.800	0.028	PASS
Shear and Moment Interaction of Double Angle at Beam Side	0.003	1.000	0.003	PASS
Design Capacity of Double Angle in Tension	12.500	369.750	0.034	PASS
Tension and Flexure Interaction of the Double Angle	0.045	1.000	0.045	PASS
Design Capacity of Double Angle in Compression	12.500	388.800	0.032	PASS
Compression and Flexure Interaction of the Double Angle	0.044	1.000	0.044	PASS
Design Capacity of Beam Web in Block Shear due to Axial Load	12.500	142.277	0.088	PASS
Design Capacity of Beam in Tension	12.500	114.809	0.109	PASS
Connection Detailing Limitations at Support Side	1.125	1.677	0.671	PASS
Design Capacity of the Bolt Group in Shear at Support Side	11.000	143.139	0.077	PASS
Design Capacity of the Bolt Group in Bearing and Tear-out on Double Angle at Support Side	11.000	313.200	0.035	PASS
Design Capacity of the Bolt Group in Bearing and Tear-out at Support Flange	11.000	498.420	0.022	PASS
Design Capacity of Double Angle in Block Shear at Support Side	11.000	219.962	0.050	PASS
Design Capacity of Double Angle in Shear at Support Side	11.000	221.850	0.050	PASS
Design Capacity of Double Angle in Flexure Using Double Coped Procedure at Support Side	11.000	351.061	0.031	PASS
Shear and Moment Interaction of Double Angle at Support Side	0.003	1.000	0.003	PASS
Design Capacity of Double Angle in Bolt Prying	0.254	0.500	0.508	PASS
Design Capacity of Column Flange in Bolt Prying	0.244	0.710	0.344	PASS
Design Capacity of Support due to Compressive Load	12.500	400.582	0.031	PASS