

Result Summary - Overall

Moment Connection - Beam to Column

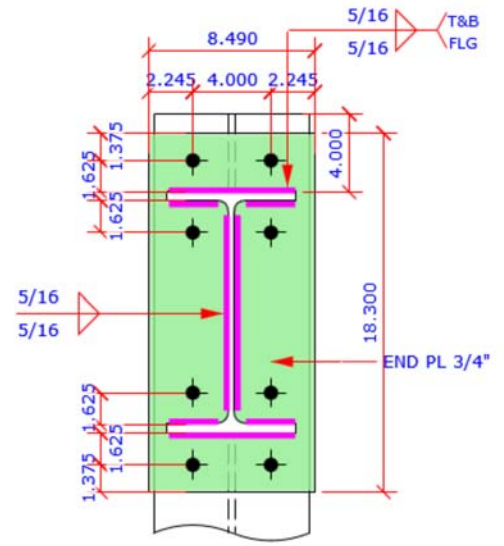
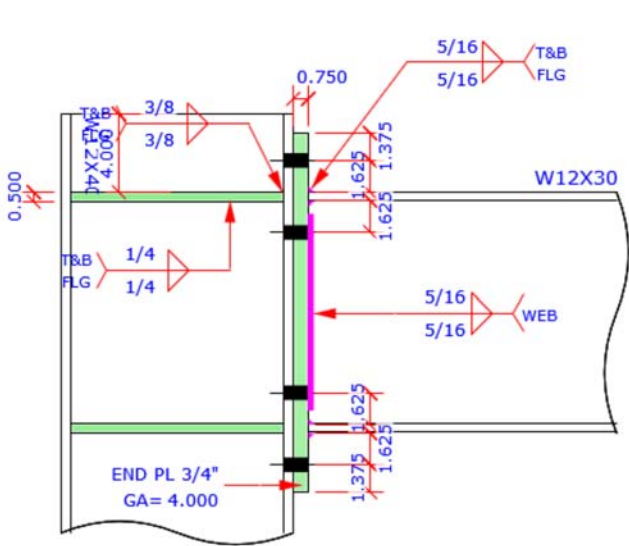
Code=AISC 360-10 LRFD

Result Summary - Overall	geometries & weld limitations = PASS	limit states max ratio = 0.95	PASS
Right Beam to Column	geometries & weld limitations = PASS	limit states max ratio = 0.95	PASS

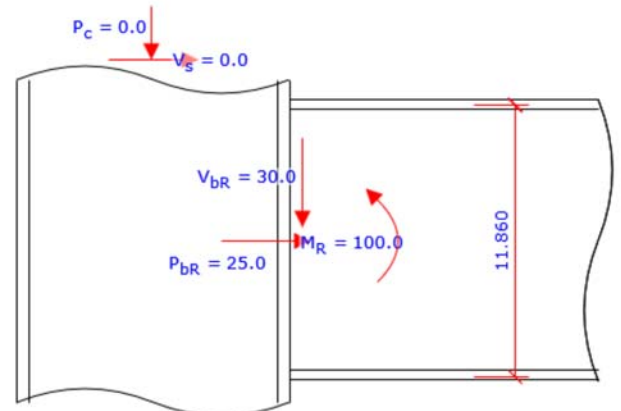
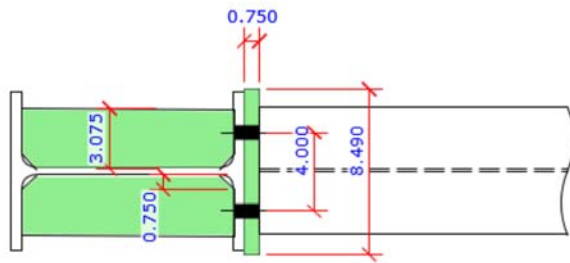
Sketch

Moment Connection - Beam to Column

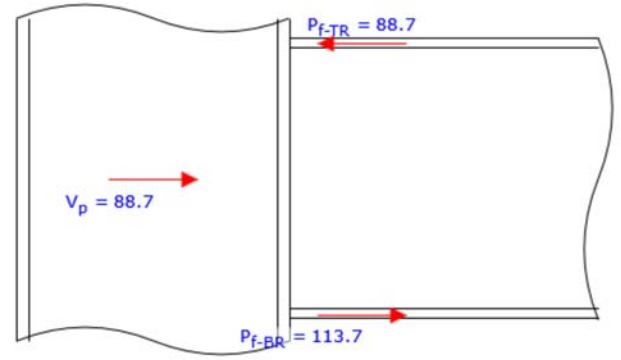
Code=AISC 360-10 LRFD



Right Side Beam



Design Load



Beam Flange Force & Panel Zone Shear V_p

Members & Components Summary

Member	Moment Connection	Code=AISC 360-10 LRFD
Column Section		
W12X40	$d = 11.900$ [in]	$b_f = 8.010$ [in]
	$t_f = 0.515$ [in]	$t_w = 0.295$ [in]
	$k_{des} = 1.020$ [in]	$k_{det} = 1.375$ [in]
	$k_1 = 0.875$ [in]	$A = 11.700$ [in ²]
	$S_x = 51.50$ [in ³]	$Z_x = 57.00$ [in ³]
Steel Grade A992	$F_y = 50.0$ [ksi]	$F_u = 65.0$ [ksi]
Right Side Beam Section		
W12X30	$d = 12.300$ [in]	$b_f = 6.520$ [in]
	$t_f = 0.440$ [in]	$t_w = 0.260$ [in]
	$k_{des} = 0.740$ [in]	$k_{det} = 1.125$ [in]
	$k_1 = 0.750$ [in]	$A = 8.790$ [in ²]
	$S_x = 38.60$ [in ³]	$Z_x = 43.10$ [in ³]
Steel Grade A992	$F_y = 50.0$ [ksi]	$F_u = 65.0$ [ksi]

Beam Flange Force Calc

Beam Flange Force - Right Side Beam

Beam section	$d_b = 12.300$ [in]	$t_{fb} = 0.440$ [in]
Flange force moment arm	$d_m = d_b - t_{fb}$	$= 11.860$ [in]
User input load	axial $P_{bR} = -25.00$ [kips]	moment $M_R = 100.00$ [kip-ft]
	in tension	
Beam flange force - top	$P_{f-TR} = P_{bR} / 2 + M_R / d_m$	$= 88.68$ [kips]
Beam flange force - bottom	$P_{f-BR} = P_{bR} / 2 - M_R / d_m$	$= -113.68$ [kips]

Panel Zone Shear Force Calc

Column story shear	$V_s =$ from user input	$= 0.00$ [kips]
Panel zone shear force	$V_p = P_{f-TR} - P_{f-TL} - V_s$	$= 88.68$ [kips]

Right Beam to Column MC Connection Code=AISC 360-10 LRFD

Result Summary

geometries & weld limitations = **PASS**

limit states max ratio = **0.95** **PASS**

Geometry Restriction Checks				PASS
Min Bolt Edge Distance - Column Flange				
Bolt diameter	$d_b =$	$= 0.750$ [in]		
Min edge distance allowed	$L_{e-min} =$	$= 1.000$ [in]	AISC 14 th Table J3.4	
Min edge distance in Column Flange	$L_e =$	$= 2.005$ [in]		
		$> L_{e-min}$		OK
Min Bolt Spacing - End Plate				
Bolt diameter	$d_b =$	$= 0.750$ [in]		
Min bolt spacing allowed	$L_{s-min} = 2.667 d_b$	$= 2.000$ [in]	AISC 14 th J3.3	
Min Bolt spacing in End Plate	$L_s =$	$= 3.690$ [in]		
		$> L_{s-min}$		OK
Min Bolt Edge Distance - End Plate				
Bolt diameter	$d_b =$	$= 0.750$ [in]		
Min edge distance allowed	$L_{e-min} =$	$= 1.000$ [in]	AISC 14 th Table J3.4	
Min edge distance in End Plate	$L_e =$	$= 1.375$ [in]		
		$> L_{e-min}$		OK
Max Bolt Edge Distance - End Plate				
Connecting plate thickness	$t_p =$	$= 0.750$ [in]		
Max edge distance allowed	$L_{e-max} = \min (12t , 6")$	$= 6.000$ [in]	AISC 14 th J3.5	
Max edge distance in End Plate	$L_e =$	$= 2.245$ [in]		
		$< L_{e-max}$		OK
Beam Flange Fillet Weld Limitation				
Min Fillet Weld Size				
Thinner part joined thickness	$t =$	$= 0.440$ [in]		
Min fillet weld size allowed	$w_{min} =$	$= 0.188$ [in]	AISC 14 th Table J2.4	
Fillet weld size provided	$w =$	$= 0.313$ [in]		
		$> w_{min}$		OK
Min Fillet Weld Length				
Fillet weld size provided	$w =$	$= 0.313$ [in]		
Min fillet weld length allowed	$L_{min} = 4 \times w$	$= 1.250$ [in]	AISC 14 th J2.2b	
Min fillet weld length	$L = 0.5 b_{fb} - k_{1b}$	$= 2.510$ [in]		
		$> L_{min}$		OK

Beam Web Fillet Weld Limitation			PASS
Min Fillet Weld Size			
<hr/>			
Thinner part joined thickness	$t =$	$= 0.260$ [in]	
Min fillet weld size allowed	$w_{min} =$	$= 0.188$ [in]	AISC 14 th Table J2.4
Fillet weld size provided	$w =$	$= 0.313$ [in]	
		$> w_{min}$	OK
Min Fillet Weld Length			
<hr/>			
Fillet weld size provided	$w =$	$= 0.313$ [in]	
Min fillet weld length allowed	$L_{min} = 4 \times w$	$= 1.250$ [in]	AISC 14 th J2.2b
Min fillet weld length	$L = 0.5 d_b - k_b$	$= 5.025$ [in]	
		$> L_{min}$	OK
Min Beam Web to End Plate Fillet Weld Size			
<hr/>			
Beam web to end-plate fillet weld in the tension-bolt region to develop the yield strength of the beam web			AISC DG4 Page 9 Item 7
Shear resistance factor-LRFD	$\phi_v = 0.90$		AISC 14 th G1
Fillet weld shear strength	$\phi R_{n-w} =$	$= 1.392$ [kip/in]	AISC 14 th Eq 8-2a
Fillet weld strength $\phi R_{n-w} \times 1.5 \times 2$ to account for 90° load angle when it's in tension and double fillet			
Min double fillet weld size to match beam web yield strength	$D_{min} = \phi_v F_{yb} t_{wb} / (\phi R_{n-w} \times 1.5 \times 2)$	$= 2.802$ [1/16 "]	
Fillet weld size provided	$D =$	$= 5.000$ [1/16 "]	
		$> D_{min}$	OK

Verify AISC DG4 Bolt No Prying Assumption			AISC DG4 Is Used
Bolt Moment Strength (No Prying)			
	bolt grade = A325-N	$F_t = 90.0$ [ksi]	AISC 14 th Table J3.2
	bolt dia $d_b = 0.750$ [in]	bolt area $A_b = 0.442$ [in ²]	
Bolt nominal tensile strength	$P_t = F_t A_b$	$= 39.76$ [kips]	AISC 14 th Eq J3-1
Tension bolt moment arm	$h_0 = 13.705$ [in]	$h_1 = 10.015$ [in]	
Bolt moment strength (no prying)	$M_{np} = 2 P_t (h_0 + h_1)$	$= 157.19$ [kip-ft]	AISC DG4 Eq 3.7
Bolt resistance factor-LRFD	$\phi = 0.75$		AISC 14 th Eq J3-1
	$\phi M_{np} =$	$= 117.89$ [kip-ft]	
End Plate Bending Strength			
End plate width	$b_{plate} = 8.490$ [in]	thickness $t_p = 0.750$ [in]	
Beam flange width	$b_{fb} = 6.520$ [in]		
Effective end plate width	$b_p = \min (b_{plate}, b_{fb} + 1")$	$= 7.520$ [in]	AISC DG4 Page 9 item 5
End plate yield strength	$F_{yp} = 50.0$ [ksi]		
See AISC DG4 Table 3.1 for all formulas to derive the following parameters			AISC DG4 Table 3.1
Tension bolt moment arm	$h_0 = 13.705$ [in]	$h_1 = 10.015$ [in]	
	$g = 4.000$ [in]		
	$p_{fi} = 1.625$ [in]	$p_{fo} = 1.625$ [in]	
	$s = 2.742$ [in]	$Y_p = 88.61$ [in]	
Flexure resistance factor-LRFD	$\phi_b = 0.90$		AISC 14 th F1 (1)
End plate bending strength	$\phi_b M_{pl} = \phi_b F_{yp} t_p^2 Y_p$	$= 186.90$ [kip-ft]	AISC DG4 Table 3.1
Check thick end plate condition	$\phi_b M_{pl} \geq 1.11 \times \phi M_{np}$		AISC DG4 Eq 3.33
	ratio = 0.70 thick plate		
Column Flange Bending Strength			
See AISC DG4 Table 3.4 for all formulas to derive the following parameters			AISC DG4 Table 3.4
Tension bolt moment arm	$h_0 = 13.705$ [in]	$h_1 = 10.015$ [in]	
*** Stiffened Column Flange Case ***			
Column section	$b_{fc} = 8.010$ [in]	$t_{fc} = 0.515$ [in]	
	$F_{yc} = 50.0$ [ksi]	bolt gage $g = 4.000$ [in]	
	$s = 2.830$ [in]	$c = 3.690$ [in]	
Stiffener plate thickness	$t_s = 0.500$ [in]		
	$p_{si} = 1.595$ [in]	$p_{so} = 1.595$ [in]	
	$Y_c = 145.6$ [in]		
Flexure resistance factor-LRFD	$\phi_b = 0.90$		AISC 14 th F1 (1)
Column flange bending strength	$\phi_b M_{cf} = \phi_b F_{yc} t_{fc}^2 Y_c$	$= 144.82$ [kip-ft]	AISC DG4 Table 3.4
Check thick column flange condition	$\phi_b M_{cf} \geq 1.11 \times \phi M_{np}$		AISC DG4 Eq 3.35
	ratio = 0.90 thick plate		
The thick end plate and column flange conditions are met. AISC DG4 is used and no bolt prying is considered			AISC DG4 Eq 3.33 & 3.35

Bolt Moment Strength (No Prying)		ratio = 113.68 / 119.28	= 0.95	PASS
	bolt grade = A325-N	$F_t = 90.0$	[ksi]	AISC 14 th Table J3.2
	bolt dia $d_b = 0.750$ [in]	bolt area $A_b = 0.442$	[in ²]	
Bolt nominal tensile strength	$P_t = F_t A_b$	= 39.76	[kips]	AISC 14 th Eq J3-1
Tension bolt moment arm	$h_0 = 13.705$ [in]	$h_1 = 10.015$	[in]	
Flange force moment arm	$d_m = d_b - t_{fb}$	= 11.860	[in]	
Flange force required in tension	$P_{uf,t} = P_u / 2 - M_u / d_m$	= 113.68	[kips]	
Flange force resistance by bolt	$F_n = 2 P_t (h_0 + h_1) / d_m$	= 159.04	[kips]	AISC DG4 Eq 3.7
Bolt resistance factor-LRFD	$\phi = 0.75$			AISC 14 th Eq J3-1
	$\phi F_n =$	= 119.28	[kips]	AISC DG4 Eq 3.7
	ratio = 0.95	> $P_{uf,t}$	OK	
Bolt Shear Strength		ratio = 30.00 / 71.57	= 0.42	PASS
Bolt shear stress	bolt grade = A325-N	$F_{nv} = 54.0$	[ksi]	AISC 14 th Table J3.2
	bolt dia $d_b = 0.750$ [in]	bolt area $A_b = 0.442$	[in ²]	
Number of bolt carried shear	$n_s = 4.0$	shear plane $m = 1$		
Required shear strength	$V_u =$	= 30.00	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 95.43	[kips]	AISC 14 th Eq J3-1
Bolt resistance factor-LRFD	$\phi = 0.75$			AISC 14 th Eq J3-1
	$\phi R_n =$	= 71.57	[kips]	
	ratio = 0.42	> V_u	OK	

Bolt Bearing/TearOut Strength on End Plate		ratio = 30.00 / 71.57	= 0.42	PASS
Single Bolt Shear Strength				
Bolt shear stress	bolt grade = A325-N	$F_{nv} = 54.0$	[ksi]	AISC 14 th Table J3.2
	bolt dia $d_b = 0.750$	$A_b = 0.442$	[in] [in ²]	
Single bolt shear strength	$R_{n-bolt} = F_{nv} A_b$	= 23.86	[kips]	AISC 14 th Eq J3-1
Bolt Bearing/TearOut Strength on Plate				
Bolt hole diameter	bolt dia $d_b = 3/4$	$d_h = 13/16$	[in]	AISC 14 th Table J3.3
Bolt spacing & edge distance	spacing $L_s = 3.690$	edge distance $L_e = 1.375$	[in]	
Plate tensile strength	$F_u = 65.0$		[ksi]	
Plate thickness	$t = 0.750$		[in]	
Interior Bolt				
Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.878	[in]	
Bolt tear out/bearing strength	$R_{n-t\&b-in} = 1.5 L_c t F_u \leq 3.0 d_b t F_u$ = 210.42 ≤ 109.69	= 109.69	[kips]	AISC 14 th Eq J3-6b
Bolt strength at interior	$R_{n-in} = \min (R_{n-t\&b-in}, R_{n-bolt})$	= 23.86	[kips]	
Edge Bolt				
Bolt hole edge clear distance	$L_c = L_e - d_h / 2$	= 0.969	[in]	
Bolt tear out/bearing strength	$R_{n-t\&b-ed} = 1.5 L_c t F_u \leq 3.0 d_b t F_u$ = 70.84 ≤ 109.69	= 70.84	[kips]	AISC 14 th Eq J3-6b
Bolt strength at edge	$R_{n-ed} = \min (R_{n-t\&b-ed}, R_{n-bolt})$	= 23.86	[kips]	
Number of bolt	interior $n_{in} = 2$	edge $n_{ed} = 2$		
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n-in} + n_{ed} R_{n-ed}$	= 95.43	[kips]	
Required shear strength	$V_u =$	= 30.00	[kips]	
Bolt resistance factor-LRFD	$\phi = 0.75$			AISC 14 th J3-10
	$\phi R_n =$	= 71.57	[kips]	
	ratio = 0.42	> V_u	OK	

Bolt Bearing/TearOut Strength on Column Flange		ratio = 30.00 / 71.57	= 0.42	PASS
Single Bolt Shear Strength				
Bolt shear stress	bolt grade = A325-N	$F_{nv} = 54.0$	[ksi]	AISC 14 th Table J3.2
	bolt dia $d_b = 0.750$	[in]	bolt area $A_b = 0.442$	[in ²]
Single bolt shear strength	$R_{n-bolt} = F_{nv} A_b$	= 23.86	[kips]	AISC 14 th Eq J3-1
Bolt Bearing/TearOut Strength on Plate				
Bolt hole diameter	bolt dia $d_b = 3/4$	[in]	bolt hole dia $d_h = 13/16$	[in] AISC 14 th Table J3.3
Bolt spacing	spacing $L_s = 3.690$	[in]		
Plate tensile strength	$F_u = 65.0$	[ksi]		
Plate thickness	$t = 0.515$	[in]		
Interior Bolt				
Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.878	[in]	
Bolt tear out/bearing strength	$R_{n-t\&b-in} = 1.5 L_c t F_u \leq 3.0 d_b t m F_u$	= 75.32	[kips]	AISC 14 th Eq J3-6b
	= 144.49 \leq 75.32			
Bolt strength at interior	$R_{n-in} = \min (R_{n-t\&b-in}, R_{n-bolt})$	= 23.86	[kips]	
Number of bolt	interior $n_{in} = 4$			
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n-in}$	= 95.43	[kips]	
Required shear strength	$V_u =$	= 30.00	[kips]	
Bolt resistance factor-LRFD	$\phi = 0.75$			AISC 14 th J3-10
	$\phi R_n =$	= 71.57	[kips]	
	ratio = 0.42	> V_u	OK	

End Plate Flexural Yielding		ratio = 113.68 / 189.11 = 0.60		PASS
End Plate Bending Strength				
End plate width	$b_{plate} = 8.490$ [in]	thickness $t_p = 0.750$ [in]		
Beam flange width	$b_{fb} = 6.520$ [in]			
Effective end plate width	$b_p = \min (b_{plate}, b_{fb} + 1")$	= 7.520 [in]		AISC DG4 Page 9 item 5
End plate yield strength	$F_{yp} = 50.0$ [ksi]			
See AISC DG4 Table 3.1 for all formulas to derive the following parameters				AISC DG4 Table 3.1
Tension bolt moment arm	$h_0 = 13.705$ [in]	$h_1 = 10.015$ [in]		
	$g = 4.000$ [in]			
	$p_{fi} = 1.625$ [in]	$p_{fo} = 1.625$ [in]		
	$s = 2.742$ [in]	$Y_p = 88.61$ [in]		
Flexure resistance factor-LRFD	$\phi_b = 0.90$			AISC 14 th F1 (1)
End plate bending strength	$\phi_b M_{pl} = \phi_b F_{yp} t_p^2 Y_p$	= 186.90 [kip-ft]		AISC DG4 Table 3.1
Flange force moment arm	$d_m = d_b - t_{fb}$	= 11.860 [in]		
Flange force required in tension	$P_{uf,t} = P_u / 2 - M_u / d_m$	= 113.68 [kips]		
Flange force provided by end plate bending	$\phi R_{pl} = \phi M_{pl} / d_m$	= 189.11 [kips]		AISC DG4 Eq 3.10
	ratio = 0.60	> $P_{uf,t}$	OK	
End Plate Shear Yielding		ratio = 56.84 / 169.20 = 0.34		PASS
Flange force required in tension	$P_{uf,t} = P_u / 2 - M_u / d_m$	= 56.84 [kips]		
End plate width	$b_{plate} = 8.490$ [in]	thickness $t_p = 0.750$ [in]		
Beam flange width	$b_{fb} = 6.520$ [in]			
Effective end plate width	$b_p = \min (b_{plate}, b_{fb} + 1")$	= 7.520 [in]		AISC DG4 Page 9 item 5
Plate Shear Yielding Check				
Plate size	width $b_p = 7.520$ [in]	thickness $t_p = 0.750$ [in]		
Plate yield strength	$F_y = 50.0$ [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$	= 5.640 [in ²]		
Shear force required	$0.5 P_{uf,t} =$	= 56.84 [kips]		
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$	= 169.20 [kips]		AISC 14 th Eq J4-3
Resistance factor-LRFD	$\phi = 1.00$			AISC 14 th Eq J4-3
	$\phi R_n =$	= 169.20 [kips]		
	ratio = 0.34	> $0.5 P_{uf,t}$	OK	

End Plate Shear Rupture		ratio = 56.84 / 126.58	= 0.45	PASS
Flange force required in tension	$P_{uf_t} = P_u / 2 - M_u / d_m$	= 56.84	[kips]	
End plate width	$b_{plate} = 8.490$ [in]	thickness $t_p = 0.750$	[in]	
Beam flange width	$b_{fb} = 6.520$ [in]			
Effective end plate width	$b_p = \min (b_{plate}, b_{fb} + 1")$	= 7.520	[in]	AISC DG4 Page 9 item 5
Plate Shear Rupture Check				
Bolt hole diameter	bolt dia $d_b = 3/4$ [in]	bolt hole dia $d_h = 7/8$	[in]	AISC 14 th B4.3b
Number of bolt	$n = 2$			
Plate size	width $b_p = 7.520$ [in]	thickness $t_p = 0.750$	[in]	
Plate tensile strength	$F_u = 65.0$ [ksi]			
Plate net area in shear	$A_{nv} = (b_p - n d_h) t_p$	= 4.328	[in ²]	
Shear force required	$0.5 P_{uf_t} =$	= 56.84	[kips]	
Plate shear rupture strength	$R_n = 0.6 F_u A_{nv}$	= 168.77	[kips]	AISC 14 th Eq J4-4
Resistance factor-LRFD	$\phi = 0.75$			AISC 14 th Eq J4-4
	$\phi R_n =$	= 126.58	[kips]	
	ratio = 0.45	> 0.5 P_{uf_t}	OK	

Beam Flange Weld Strength		ratio = 113.68 / 120.47	= 0.94	PASS
Flange force required in tension	$P_{uf_t} = P_u / 2 - M_u / d_m$	= 113.68	[kips]	
Fillet weld length - double fillet	$L = [b_{fb} + (b_{fb} - 2k_{1b})] / 2$ as dbl fillet	= 5.770	[in]	
Fillet Weld Strength Check				
Fillet weld leg size	$w = 5/16$ [in]	load angle $\theta = 90.0$	[°]	
Electrode strength	$F_{EXX} = 70.0$ [ksi]	strength coeff $C_1 = 1.00$		AISC 14 th Table 8-3
Number of weld line	$n = 2$ for double fillet			
Load angle coefficient	$C_2 = (1 + 0.5 \sin^{1.5} \theta)$	= 1.50		AISC 14 th Page 8-9
Fillet weld shear strength	$R_{n-w} = 0.6 (C_1 \times 70 \text{ ksi}) 0.707 w n C_2$	= 27.838	[kip/in]	AISC 14 th Eq 8-1
Base metal - beam flange	thickness $t = 0.440$ [in]	tensile $F_u = 65.0$	[ksi]	
Base metal - beam flange is in tension, <u>tensile</u> rupture as per AISC 14 th Eq J4-2 is checked				
Base metal tensile rupture	$R_{n-b} = F_u t$	= 28.600	[kip/in]	AISC 14 th Eq J4-2
Double fillet linear shear strength	$R_n = \min (R_{n-w}, R_{n-b})$	= 27.838	[kip/in]	AISC 14 th Eq 9-2
Resistance factor-LRFD	$\phi = 0.75$			AISC 14 th Eq 8-1
	$\phi R_n =$	= 20.879	[kip/in]	
Shear resistance required	$P_{uf_t} =$	= 113.68	[kips]	
Fillet weld length - double fillet	$L =$	= 5.770	[in]	
Shear resistance provided	$\phi F_n = \phi R_n \times L$	= 120.47	[kips]	
	ratio = 0.94	> P_{uf_t}	OK	

Beam Web Weld Strength		ratio = 30.00 / 38.22	= 0.79	PASS
Beam Web Effective Weld Length Calc				
Beam section	$d_b = 12.300$ [in]	$t_{fb} = 0.440$ [in]		
	$k_b = 1.125$ [in]			
Bolt diameter	$d_{bolt} = 0.750$ [in]	bolt inner pitch $p_{fi} = 1.625$ [in]		
Effective weld length case 1	$L_1 = 0.5 d_b - k_b$	$= 5.025$ [in]		AISC DG4 Page 38
Effective weld length case 2	$L_2 = d_b - 2t_{fb} - p_{fi} - 2 d_{bolt}$	$= 8.295$ [in]		AISC DG4 Page 38
Fillet weld length - double fillet	$L = \min(L_1, L_2)$	$= 5.025$ [in]		
Fillet Weld Strength Check				
Fillet weld leg size	$w = 5/16$ [in]	load angle $\theta = 0.0$ [°]		
Electrode strength	$F_{EXX} = 70.0$ [ksi]	strength coeff $C_1 = 1.00$		AISC 14 th Table 8-3
Number of weld line	$n = 2$ for double fillet			
Load angle coefficient	$C_2 = (1 + 0.5 \sin^{1.5} \theta)$	$= 1.00$		AISC 14 th Page 8-9
Fillet weld shear strength	$R_{n-w} = 0.6 (C_1 \times 70 \text{ ksi}) 0.707 w n C_2$	$= 18.559$ [kip/in]		AISC 14 th Eq 8-1
Base metal - beam web	thickness $t = 0.260$ [in]	tensile $F_u = 65.0$ [ksi]		
Base metal - beam web is in shear, <u>shear</u> rupture as per AISC 14 th Eq J4-4 is checked				AISC 14 th J2.4
Base metal shear rupture	$R_{n-b} = 0.6 F_u t$	$= 10.140$ [kip/in]		AISC 14 th Eq J4-4
Double fillet linear shear strength	$R_n = \min(R_{n-w}, R_{n-b})$	$= 10.140$ [kip/in]		AISC 14 th Eq 9-2
Resistance factor-LRFD	$\phi = 0.75$			AISC 14 th Eq 8-1
	$\phi R_n =$	$= 7.605$ [kip/in]		
Shear resistance required	$V_u =$	$= 30.00$ [kips]		
Fillet weld length - double fillet	$L =$	$= 5.025$ [in]		
Shear resistance provided	$\phi F_n = \phi R_n \times L$	$= 38.22$ [kips]		
	ratio = 0.79	$> V_u$	OK	

Column Flexural Yielding		ratio = 113.68 / 146.53 = 0.78	PASS
Column Flange Bending Strength			
See AISC DG4 Table 3.4 for all formulas to derive the following parameters		AISC DG4 Table 3.4	
Tension bolt moment arm	$h_0 = 13.705$ [in]	$h_1 = 10.015$ [in]	
*** Stiffened Column Flange Case ***			
Column section	$b_{fc} = 8.010$ [in]	$t_{fc} = 0.515$ [in]	
	$F_{yc} = 50.0$ [ksi]	bolt gage $g = 4.000$ [in]	
	$s = 2.830$ [in]	$c = 3.690$ [in]	
Stiffener plate thickness	$t_s = 0.500$ [in]		
	$p_{si} = 1.595$ [in]	$p_{so} = 1.595$ [in]	
	$Y_c = 145.6$ [in]		
Flexure resistance factor-LRFD	$\phi_b = 0.90$		AISC 14 th F1 (1)
Column flange bending strength	$\phi_b M_{cf} = \phi_b F_{yc} t_{fc}^2 Y_c$	= 144.82 [kip-ft]	AISC DG4 Table 3.4
Flange force moment arm	$d_m = d_b - t_{fb}$	= 11.860 [in]	
Flange force required in tension	$P_{uf,t} = P_u / 2 - M_u / d_m$	= 113.68 [kips]	
Flange force provided by column flange bending	$\phi R_{cf} = \phi M_{cf} / d_m$	= 146.53 [kips]	AISC DG4 Eq 3.21
	ratio = 0.78	> $P_{uf,t}$	OK
Column Web Yielding		113.68 / 64.05	N/A
Flange force moment arm	$d_m = d_b - t_{fb}$	= 11.860 [in]	
Flange force in demand	$P_{uf} = \max (P_{uf,t} , P_{uf,c})$	= 113.68 [kips]	AISC DG13 Eq 4.2-1
Column section	$d_c = 11.900$ [in]	$t_{fc} = 0.515$ [in]	
	$t_{wc} = 0.295$ [in]	$k_c = 1.020$ [in]	
Column yield strength	$F_{yc} = 50.0$ [ksi]		
Distance from to top of column to top of beam flange	$d_{end} = 4.000$ [in]		
Top column reduction factor	$C_t = 0.5$		AISC DG4 Eq 3.24
Beam flange fillet weld size	$w = 0.313$ [in]	beam flange $t_{fb} = 0.440$ [in]	
Length of bearing	$N = t_{fb} + 2 w$	= 1.065 [in]	AISC DG4 Eq 3.24
End plate thickness	$t_p = 0.750$ [in]		
Column web yielding strength	$R_n = C_t (6 k_c + N + 2 t_p) F_{yc} t_{wc}$	= 64.05 [kips]	AISC DG4 Eq 3.24
Resistance factor-LRFD	$\phi = 1.00$		AISC 14 th J10.2
	$\phi R_n =$	= 64.05 [kips]	
Unbalanced force to be resisted by transverse stiffeners	$R_s = P_{uf} - \phi R_n$	= 49.63 [kips]	

Column Web Buckling		88.68 / 33.86	N/A
Flange force moment arm	$d_m = d_b - t_{fb}$	= 11.860 [in]	
Flange force required in compression	$P_{uf_c} = P_u / 2 - M_u / d_m$	= 88.68 [kips]	
Column section	$d_c = 11.900$ [in]	$t_{fc} = 0.515$ [in]	
	$t_{wc} = 0.295$ [in]	$k_c = 1.020$ [in]	
	$h = d_c - 2 k_c$	= 9.860 [in]	
Column yield strength	$F_{yc} = 50.0$ [ksi]	$E_c = 29000$ [ksi]	
Distance from top of beam flange to top of column	$d_{end-flg} = 4.000$ [in]	beam flange $t_{fb} = 0.440$ [in]	
Distance from center of flange force to top of column	$d_{end-F} = d_{end-flg} + 0.5 t_{fb}$	= 4.220 [in]	
	$d_{end-F} < d_c/2$, R_n reduced by 50%		AISC 14 th J10.5
Top column reduction factor	$C_t = 0.5$		AISC 14 th J10.5
Column web buckling strength	$R_n = \frac{C_t 24 t_{wc}^3 \sqrt{E_c F_{yc}}}{h}$	= 37.62 [kips]	AISC 14 th Eq J10-8
Resistance factor-LRFD	$\phi = 0.90$		AISC 14 th J10.5
	$\phi R_n =$	= 33.86 [kips]	
Unbalanced force to be resisted by transverse stiffeners	$R_s = P_{uf_c} - \phi R_n$	= 54.82 [kips]	
Column Web Crippling		88.68 / 53.46	N/A
Flange force moment arm	$d_m = d_b - t_{fb}$	= 11.860 [in]	
Flange force required in compression	$P_{uf_c} = P_u / 2 - M_u / d_m$	= 88.68 [kips]	
Column section	$d_c = 11.900$ [in]	$t_{fc} = 0.515$ [in]	
	$t_{wc} = 0.295$ [in]	$k_c = 1.020$ [in]	
Column yield strength	$F_{yc} = 50.0$ [ksi]	$E_c = 29000$ [ksi]	
Beam flange fillet weld size	$w = 0.313$ [in]	beam flange $t_{fb} = 0.440$ [in]	
End plate thickness	$t_p = 0.750$ [in]		
Length of bearing	$l_b = t_{fb} + 2 w + 2 t_p$	= 2.565 [in]	
Distance from top of column to top of beam flange	$d_{end-flg} =$	= 4.000 [in]	
Distance from top of column to center of flange force	$d_{end-F} = d_{end-flg} + 0.5 t_{fb}$	= 4.220 [in]	
	$d_{end-F} < d_c/2$ and $l_b / d_c = 0.22 > 0.2$, use Eq J10-5b		AISC 14 th Eq J10-5b
Column web crippling strength	$R_n = 0.4 t_{wc}^2 \left[1 + \left(\frac{4 l_b}{d_c} - 0.2 \right) \left(\frac{t_{wc}}{t_{fc}} \right)^{1.5} \right] \times \left(\frac{E_c F_{yc} t_{fc}}{t_{wc}} \right)^{0.5}$	= 71.28 [kips]	AISC 14 th Eq J10-5b
Resistance factor-LRFD	$\phi = 0.75$		AISC 14 th J10.3
	$\phi R_n =$	= 53.46 [kips]	
Unbalanced force to be resisted by transverse stiffeners	$R_s = P_{uf_c} - \phi R_n$	= 35.22 [kips]	

Column Panel Zone Shear		ratio = 88.68 / 94.78	= 0.94	PASS
Panel zone shear force	$V_p = P_{f-TR} - P_{f-TL} - V_s$		= 88.68 [kips]	
Column section	$d_c = 11.900$ [in]		$t_{wc} = 0.295$ [in]	
	$A_c = 11.700$ [in ²]		$F_{yc} = 50.0$ [ksi]	
Column axial compression - user input	$P_r =$		= 0.00 [kips]	
	$P_c = P_y = F_{yc} A_c$		= 585.00 [kips]	AISC 14 th J10.6
	$P_r \leq 0.4 P_c$, use Eq J10-9			AISC 14 th Eq J10-9
Web panel zone capacity	$R_n = 0.6 F_{yc} d_c t_{wc}$		= 105.32 [kips]	AISC 14 th Eq J10-9
Resistance factor-LRFD	$\phi = 0.90$			AISC 14 th J10.6
	$\phi R_n =$		= 94.78 [kips]	
	ratio = 0.94		> V_p	OK

Stiffener Geometry Restriction				PASS
Stiffener plate width	$b_s = 3.075$ [in]		depth $d_s = 10.870$ [in]	
Stiffener plate thickness	$t_s = 0.500$ [in]			
Column flange thickness	$t_{fc} = 0.515$ [in]		column depth $d_c = 11.900$ [in]	
Beam flange thickness	$t_{fb} = 0.440$ [in]			
Min Stiffener Plate Thickness				
Min stiffener plate thickness	$t_{smin} = \max (t_{fb} / 2 , b_s / 16)$		= 0.220 [in]	AISC 14 th J10.8 (2)
Stiffener plate thickness	$t_s =$		= 0.500 [in]	
			> t_{smin}	OK
Min Stiffener Plate Depth				
Min stiffener plate depth	$d_{smin} = (d_c - 2 t_{fc}) / 2$		= 5.435 [in]	AISC 14 th J10.8 (3)
Stiffener plate depth	$d_s =$		= 10.870 [in]	
			> d_{smin}	OK

Stiffener Yield at Column Flange		ratio = 54.82 / 104.63	= 0.52	PASS
Stiffener plate width	$b_s = 3.075$ [in]		thickness $t_s = 0.500$ [in]	
Stiffener plate corner clip	clip =		= 0.750 [in]	AISC 14 th Page 8-18
Stiffener plate yield strength	$F_y =$		= 50.0 [ksi]	
Stiffener plate cross sect area	$A_{st} = (b_s - clip) t_s$		= 1.163 [in ²]	
Trans stiffener strength required	$R_s =$		= 54.82 [kips]	
Trans stiffener strength provided	$R_n = F_y \times 2 \times A_{st}$		= 116.25 [kips]	AISC 14 th Eq J4-1
Bolt resistance factor-LRFD	$\phi = 0.90$			AISC 14 th Eq J4-1
	$\phi R_n =$		= 104.63 [kips]	
	ratio = 0.52		> R_s	OK

Stiffener Shear at Column Web		ratio = 54.82 / 281.10	= 0.20	PASS
Stiffener plate depth	$d_s = 10.870$ [in]	thickness $t_s = 0.500$	[in]	
Stiffener plate corner clip	clip =	= 0.750	[in]	AISC 14 th Page 8-18
Stiffener plate yield strength	$F_y =$	= 50.0	[ksi]	
Stiffener plate cross sect area	$A_{gv} = (d_s - 2 \times \text{clip}) t_s$	= 4.685	[in ²]	
Trans stiffener strength required	$R_s =$	= 54.82	[kips]	
Trans stiffener strength provided	$R_n = 2 \times 0.6 \times F_y \times A_{gv}$	= 281.10	[kips]	AISC 14 th Eq J4-3
Bolt resistance factor-LRFD	$\phi = 1.00$			AISC 14 th Eq J4-3
	$\phi R_n =$	= 281.10	[kips]	
	ratio = 0.20	> R_s	OK	

Stiffener Compression		ratio = 54.82 / 82.86	= 0.66	PASS
Stiffener plate width	$b_s = 3.075$ [in]	thickness $t_s = 0.500$	[in]	
Stiffener plate depth	$d_s = d - 2 t_f$	= 10.870	[in]	
Stiffener plate corner clip	clip =	= 0.750	[in]	AISC 14 th Page 8-18
Stiffener plate yield strength	$F_y = 50.0$ [ksi]	$E = 29000$	[ksi]	
Stiffener plate cross sect area	$A_{st} = (b_s - \text{clip}) t_s$	= 1.163	[in ²]	
Stiffener plate unbraced length	$L = d_s = d - 2 t_f$	= 10.870	[in]	
Plate radius of gyration	$r = t_s / \sqrt{12}$	= 0.144	[in]	
Stiffener plate slenderness	$KL/r = 0.75 \times L / r$	= 56.48		
Elastic buckling stress	$F_e = \frac{\pi^2 E}{(KL/r)^2}$	= 89.72	[ksi]	AISC 14 th Eq E3-4
	when $\frac{KL}{r} \leq 4.71 \left(\frac{E}{F_y} \right)^{0.5} = 113.43$			AISC 14 th E3
Critical stress	$F_{cr} = 0.658 (F_y/F_e) F_y$	= 39.60	[ksi]	AISC 14 th Eq E3-2
Trans stiffener strength required	$R_s =$	= 54.82	[kips]	
Trans stiffener strength provided	$R_n = 2 \times F_{cr} \times A_{st}$	= 92.06	[kips]	AISC 14 th Eq E3-1
Bolt resistance factor-LRFD	$\phi = 0.90$			AISC 14 th E1
	$\phi R_n =$	= 82.86	[kips]	
	ratio = 0.66	> R_s	OK	

Stiffener to Column Flange Fillet Weld Limitation			PASS
Min Fillet Weld Size			
<hr/>			
Thinner part joined thickness	$t =$	$= 0.500$ [in]	
Min fillet weld size allowed	$w_{min} =$	$= 0.188$ [in]	AISC 14 th Table J2.4
Fillet weld size provided	$w =$	$= 0.375$ [in]	
		$> w_{min}$	OK
Min Fillet Weld Length			
<hr/>			
Fillet weld size provided	$w =$	$= 0.375$ [in]	
Min fillet weld length allowed	$L_{min} = 4 \times w$	$= 1.500$ [in]	AISC 14 th J2.2b
Min fillet weld length	$L = 0.5 b_{fc} - k_{1c}$	$= 3.130$ [in]	
		$> L_{min}$	OK
Min Stiffener to Column Flange Fillet Weld Size			
<hr/>			
Stiffener plate to column flange fillet weld to develop the yield strength of the stiffener plate			AISC DG13 Eq 4.3-6
Shear resistance factor-LRFD	$\phi_v = 0.90$		AISC 14 th G1
Fillet weld shear strength	$\phi R_{n-w} =$	$= 1.392$ [kip/in]	AISC 14 th Eq 8-2a
Fillet weld strength $\phi R_{n-w} \times 1.5 \times 2$ to account for 90° load angle when it's in tension and double fillet			
Stiffener plate thickness	$t_s = 0.500$ [in]	$F_y = 50.0$ [ksi]	
Min double fillet weld size to match stiffener plate yield strength	$D_{min} = \phi_v F_y t_s / (\phi R_{n-w} \times 1.5 \times 2)$	$= 5.389$ [1/16 "]	
Fillet weld size provided	$D =$	$= 6.000$ [1/16 "]	
		$> D_{min}$	OK

Stiffener to Column Web Fillet Weld Limitation			PASS
Min Fillet Weld Size			
<hr/>			
Thinner part joined thickness	$t =$	$= 0.295$ [in]	
Min fillet weld size allowed	$w_{min} =$	$= 0.188$ [in]	AISC 14 th Table J2.4
Fillet weld size provided	$w =$	$= 0.250$ [in]	
		$> w_{min}$	OK
Min Fillet Weld Length			
<hr/>			
Fillet weld size provided	$w =$	$= 0.250$ [in]	
Min fillet weld length allowed	$L_{min} = 4 \times w$	$= 1.000$ [in]	AISC 14 th J2.2b
Min fillet weld length	$L = d_c - 2 \times t_{fc} - 2 \times clip$	$= 9.370$ [in]	
		$> L_{min}$	OK

Stiffener Weld Strength at Column Flange		ratio = 54.82 / 152.59	= 0.36	PASS
Stiffener to Column Flange Weld Length Calc				
Column section	$b_{fc} = 8.010$ [in]	$k_{1c} = 0.875$ [in]		
Stiffener plate corner clip	clip = 0.750 [in]			
Stiffener to column flange weld length - double fillet	$L = (0.5 b_{fc} - k_{1c}) \times 2$ stiffener		= 6.260 [in]	
Trans stiffener strength required	$R_s =$		= 54.82 [kips]	
Fillet Weld Strength Check				
Fillet weld leg size	$w = \frac{3}{8}$ [in]	load angle $\theta = 90.0$ [°]		
Electrode strength	$F_{EXX} = 70.0$ [ksi]	strength coeff $C_1 = 1.00$		AISC 14 th Table 8-3
Number of weld line	$n = 2$ for double fillet			
Load angle coefficient	$C_2 = (1 + 0.5 \sin^{1.5} \theta)$		= 1.50	AISC 14 th Page 8-9
Fillet weld shear strength	$R_{n-w} = 0.6 (C_1 \times 70 \text{ ksi}) 0.707 w n C_2$		= 33.406 [kip/in]	AISC 14 th Eq 8-1
Base metal - stiffener plate	thickness $t = 0.500$ [in]	tensile $F_u = 65.0$ [ksi]		
Base metal - stiffener plate is in tension, <u>tensile</u> rupture as per AISC 14 th Eq J4-2 is checked				AISC 14 th J2.4
Base metal tensile rupture	$R_{n-b} = F_u t$		= 32.500 [kip/in]	AISC 14 th Eq J4-2
Double fillet linear shear strength	$R_n = \min (R_{n-w}, R_{n-b})$		= 32.500 [kip/in]	AISC 14 th Eq 9-2
Resistance factor-LRFD	$\phi = 0.75$			AISC 14 th Eq 8-1
	$\phi R_n =$		= 24.375 [kip/in]	
Shear resistance required	$R_s =$		= 54.82 [kips]	
Fillet weld length - double fillet	$L =$		= 6.260 [in]	
Shear resistance provided	$\phi F_n = \phi R_n \times L$		= 152.59 [kips]	
	ratio = 0.36		> R_s	OK

Stiffener Weld Strength at Column Web		ratio = 54.82 / 208.67	= 0.26	PASS
Stiffener to Column Web Weld Length Calc				
Column section	$d_c = 11.900$ [in]		$t_{fc} = 0.515$ [in]	
Stiffener plate corner clip	clip = 0.750 [in]			
Stiffener to column web weld length - double fillet	$L = (d_c - 2 \times t_{fc} - 2 \times \text{clip}) \times 2$ stiffener		= 18.740 [in]	
Trans stiffener strength required	$R_s =$		= 54.82 [kips]	
Fillet Weld Strength Check				
Fillet weld leg size	$w = 1/4$ [in]		load angle $\theta = 0.0$ [°]	
Electrode strength	$F_{EXX} = 70.0$ [ksi]		strength coeff $C_1 = 1.00$	AISC 14 th Table 8-3
Number of weld line	$n = 2$ for double fillet			
Load angle coefficient	$C_2 = (1 + 0.5 \sin^{1.5} \theta)$		= 1.00	AISC 14 th Page 8-9
Fillet weld shear strength	$R_{n-w} = 0.6 (C_1 \times 70 \text{ ksi}) 0.707 w n C_2$		= 14.847 [kip/in]	AISC 14 th Eq 8-1
Base metal - stiffener plate	thickness $t = 0.500$ [in]		tensile $F_u = 65.0$ [ksi]	
Base metal - stiffener plate is in shear, <u>shear</u> rupture as per AISC 14 th Eq J4-4 is checked				AISC 14 th J2.4
Base metal shear rupture	$R_{n-b} = 0.6 F_u t$		= 19.500 [kip/in]	AISC 14 th Eq J4-4
Double fillet linear shear strength	$R_n = \min (R_{n-w}, R_{n-b})$		= 14.847 [kip/in]	AISC 14 th Eq 9-2
Resistance factor-LRFD	$\phi = 0.75$			AISC 14 th Eq 8-1
	$\phi R_n =$		= 11.135 [kip/in]	
Shear resistance required	$R_s =$		= 54.82 [kips]	
Fillet weld length - double fillet	$L =$		= 18.740 [in]	
Shear resistance provided	$\phi F_n = \phi R_n \times L$		= 208.67 [kips]	
	ratio = 0.26		> R_s	OK