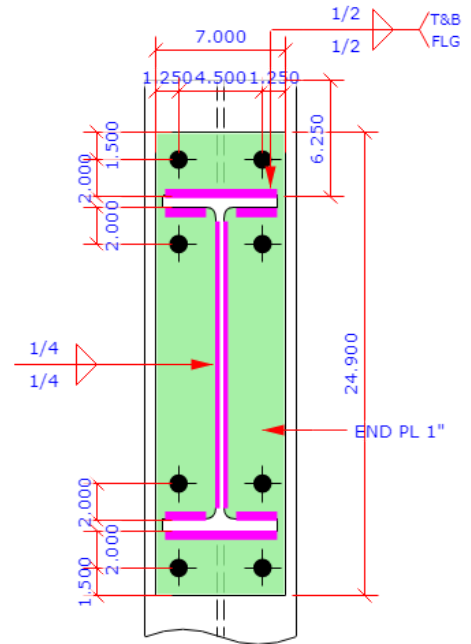
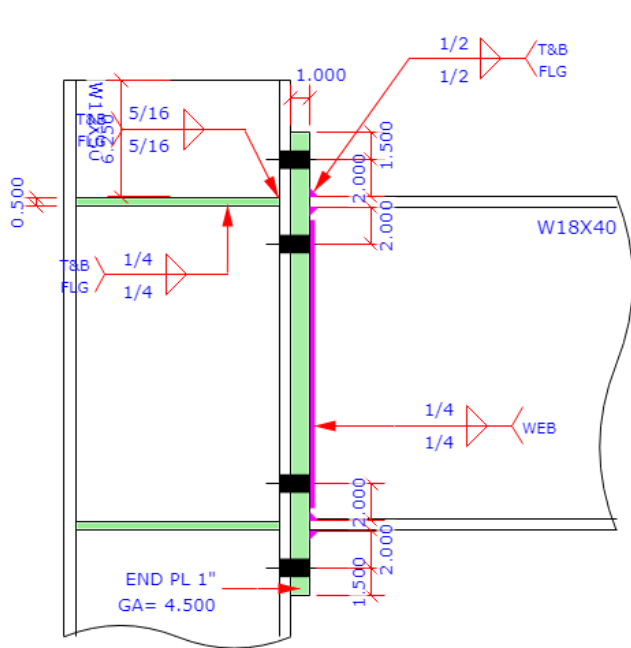


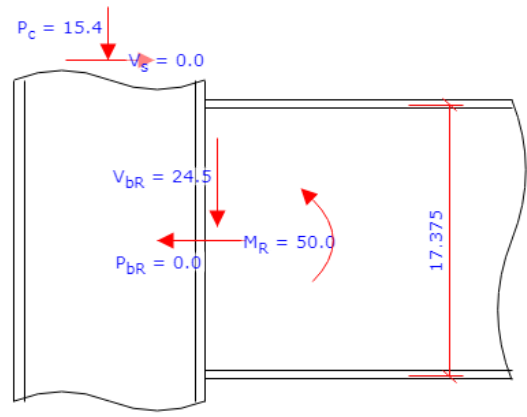
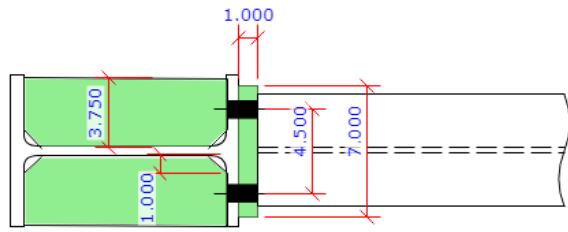
Result Summary - Overall	Moment Connection - Beam to Column	Code=AISC 360-16 LRFD
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Result Summary - Overall	geometries & weld limitations = FAIL	limit states max ratio = 1.25	FAIL
Right Beam to Column	geometries & weld limitations = FAIL	limit states max ratio = 1.25	FAIL

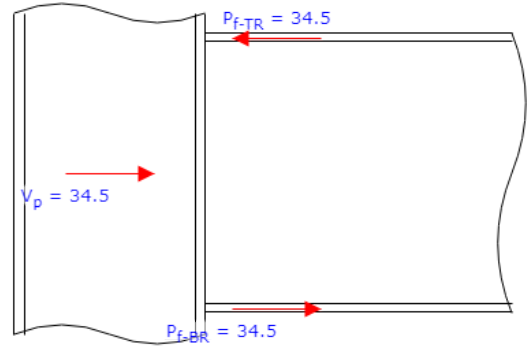
Sketch	Moment Connection - Beam to Column	Code=AISC 360-16 LRFD
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Right Side Beam



Design Load



Beam Flange Force & Panel Zone Shear V_p

Beam Flange Force Calc**Beam Flange Force - Right Side Beam**

Beam section	$d_b = 17.900$ [in]	$t_{fb} = 0.525$ [in]
Flange force moment arm	$d_m = d_b - t_{fb}$	$= 17.375$ [in]
User input load	axial $P_{bR} = 0.0$ [kips]	moment $M_R = 50.00$ [kip-ft]
Beam flange force - top	$P_{f-TR} = P_{bR} / 2 + M_R / d_m$	$= 34.5$ [kips]
Beam flange force - bottom	$P_{f-BR} = P_{bR} / 2 - M_R / d_m$	$= -34.5$ [kips]

Panel Zone Shear Force Calc

Column story shear	$V_s =$ from user input	$= 0.0$ [kips]
Panel zone shear force	$V_p = P_{f-TR} - P_{f-TL} - V_s$	$= 34.5$ [kips]

Seismic Moment and Beam Flange Force Calc**Seismic OMF Force Calc - Right Side Beam**

Refer to AISC 341-16 E1.6b (b), OMF connection design should be based on the maximum moment that can be transferred to the connection by the system, including the effects of material overstrength and strain hardening.

AISC 341-16 E1.6b (b)

The flexural strength that can be transferred is based on the smaller of the expected flexural strength of the beam or column, including a 1.1 factor for strain hardening, or the flexural strength resulting from panel zone shear.

Beam Expected Flexural Strength

Beam sect W18X40	$d_b = 17.900$ [in]	$Z_{bx} = 78.40$ [in ³]
	$F_{by} = 50.0$ [ksi]	$R_{by} = 1.1$
Beam expected flexural strength	$M_{be} = 1.1 R_{by} F_{by} Z_{bx}$	$= 395.27$ [kip-ft]

Column Expected Flexural Strength

Column sect W12X50	$Z_{cx} = 71.90$ [in ³]	$F_{cy} = 50.0$ [ksi]
	$R_{cy} = 1.1$	
Column expected flexural strength	$M_{ce} = 1.1 R_{cy} F_{cy} Z_{cx}$	$= 362.50$ [kip-ft]

Flexural Strength by Panel Zone Shear

Depth of beam	$d_b = d_b$	$= 17.900$ [in]	
Column sect W12X50	$d_c = 12.200$ [in]	$b_{cf} = 8.080$ [in]	
	$t_{cw} = 0.370$ [in]	$t_{cf} = 0.640$ [in]	
	$F_{cy} = 50.0$ [ksi]	$R_{cy} = 1.1$	
Column sect W12X50	$A_c = 14.600$ [in ²]	$F_{cy} = 50.0$ [ksi]	
Column axial yield strength	$P_y = F_{cy} A_c$	$= 730.0$ [kips]	AISC 15 th J10.6 (b)
LRFD-ASD force adjustment factor	$\alpha =$ for LRFD	$= 1.0$	AISC 15 th J10.6 (b)
Column axial compression	$P_r =$ from user input	$= 15.4$ [kips]	
	when $\alpha P_r \leq 0.75 P_y$, use Eq J10-11		AISC 15 th Eq J10-11
Column panel zone capacity	$V_{pz} = 0.6(1.1)R_{cy}F_{cy}d_c t_{cw} \left(1 + \frac{3 b_{cf} t_{cf}^2}{d_b d_c t_{cw}}\right)$	$= 184.0$ [kips]	AISC 15 th Eq J10-11
Column panel zone capacity-LRFD	$V_{ue} = V_{pz} / \alpha_s = 1.0$	$= 184.0$ [kips]	
Beam sect W18X40	$d_b = 17.900$ [in]	$t_{bf} = 0.525$ [in]	
Flexural strength by panel zone shear	$M_{ue} = V_{ue} (d_b - t_{bf})$	$= 266.41$ [kip-ft]	
Min expected flexural strength	$M_{ne} = \min(M_{be}, M_{ce}, M_{ue})$	$= 266.41$ [kip-ft]	

Calculate Story Shear

Assume column inflection point is at the mid height of story above and below beam

Column story height above/below beam	$h_t = 0.0$ [in]	$h_b = 204.0$ [in]
Story shear	$V_{uc} = \frac{M_{ne}}{h_b}$	= 15.7 [kips]
Flexural strength after considering story shear	$M_u = (V_{ue} + V_{uc})(d_b - t_{bf})$	= 289.10 [kip-ft]

Calculate Shear Load

Beam clear span	$L_{cf} =$ from user input	= 348 [in]
Shear from max expected flexural strength	$V_{ne} = 2 M_u / L_{cf}$	= 19.9 [kips]
Shear from load combination including amplified seismic load	$V =$ from user input	= 24.5 [kips]
Max shear used in design	$V_u = \max(V_{ne}, V)$	= 24.5 [kips]

Calculate Flange Force

Beam sect W18X40	$d_b = 17.900$ [in]	$t_{bf} = 0.525$ [in]
Moment arm between flanges	$d_m = d_b - t_{bf}$	= 17.375 [in]
Flange force	$F_{fu} = M_u / d_m$	= 199.7 [kips]

Right Beam to Column

MC Connection

Code=AISC 360-16 LRFD

Result Summarygeometries & weld limitations = **FAIL**limit states max ratio = **1.25** **FAIL****Geometry Restriction Checks****PASS****Min Bolt Edge Distance - Column Flange**

Bolt diameter	$d_b =$	= 1.000 [in]	
Min edge distance allowed	$L_{e-min} =$	= 1.250 [in]	AISC 15 th Table J3.4
Min edge distance in Column Flange	$L_e =$	= 1.790 [in]	
		$\geq L_{e-min}$	OK

Min Bolt Spacing - End Plate

Bolt diameter	$d_b =$	= 1.000 [in]	
Min bolt spacing allowed	$L_{s-min} = 2.667 d_b$	= 2.667 [in]	AISC 15 th J3.3
Min Bolt spacing in End Plate	$L_s =$	= 4.500 [in]	
		$\geq L_{s-min}$	OK

Min Bolt Edge Distance - End Plate

Bolt diameter	$d_b =$	= 1.000 [in]	
Min edge distance allowed	$L_{e-min} =$	= 1.250 [in]	AISC 15 th Table J3.4
Min edge distance in End Plate	$L_e =$	= 1.250 [in]	
		$\geq L_{e-min}$	OK

Max Bolt Edge Distance - End Plate

Connecting plate thickness	$t_p =$	= 1.000 [in]	
Max edge distance allowed	$L_{e-max} = \min(12t, 6")$	= 6.000 [in]	AISC 15 th J3.5
Max edge distance in End Plate	$L_e =$	= 1.500 [in]	
		$\leq L_{e-max}$	OK

Beam Flange Fillet Weld Limitation		PASS	
Min Fillet Weld Size			
Thinner part joined thickness	$t =$	$= 0.525$ [in]	
Min fillet weld size allowed	$w_{min} =$	$= \mathbf{0.250}$ [in]	AISC 15 th Table J2.4
Fillet weld size provided	$w =$	$= \mathbf{0.500}$ [in]	
		$\geq w_{min}$	OK
Min Fillet Weld Length			
Fillet weld size provided	$w =$	$= 0.500$ [in]	
Min fillet weld length allowed	$L_{min} = 4 \times w$	$= \mathbf{2.000}$ [in]	AISC 15 th J2.2b
Min fillet weld length	$L = 0.5 b_{fb} - k_{1b}$	$= \mathbf{2.197}$ [in]	
		$\geq L_{min}$	OK

Beam Web Fillet Weld Limitation		PASS	
Min Fillet Weld Size			
Thinner part joined thickness	$t =$	$= 0.315$ [in]	
Min fillet weld size allowed	$w_{min} =$	$= \mathbf{0.188}$ [in]	AISC 15 th Table J2.4
Fillet weld size provided	$w =$	$= \mathbf{0.250}$ [in]	
		$\geq w_{min}$	OK
Min Fillet Weld Length			
Fillet weld size provided	$w =$	$= 0.250$ [in]	
Min fillet weld length allowed	$L_{min} = 4 \times w$	$= \mathbf{1.000}$ [in]	AISC 15 th J2.2b
Min fillet weld length	$L = 0.5 d_b - k_b$	$= \mathbf{7.762}$ [in]	
		$\geq L_{min}$	OK
Min Beam Web to End Plate Fillet Weld Size			
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 15 th G1
Fillet weld shear strength	$\phi R_{n-w} =$	$= 1.392$ [kip/in]	AISC 15 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	$D_{min} = \phi_v F_{yb} t_{wb} / (\phi R_{n-w} \times 1.5 \times 2)$	$= \mathbf{3.394}$ [1/16 "]	
Fillet weld size provided	$D =$	$= \mathbf{4.000}$ [1/16 "]	
		$\geq D_{min}$	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 15 th Table J3.2
	bolt dia $d_b = 1.000$ [in]	bolt area $A_b = 0.785$ [in ²]	
Bolt nominal tensile strength	$P_t = F_t A_b$	$= 70.7$ [kips]	AISC 15 th Eq J3-1
Tension bolt moment arm	$h_0 = 19.638$ [in]	$h_1 = 15.113$ [in]	
Bolt moment strength (no prying)	$M_{nb} = 2 P_t (h_0 + h_1)$	$= 409.39$ [kip-ft]	AISC DG4 Eq 3.7
Bolt resistance factor-LRFD	$\phi = 0.75$		AISC 15 th Eq J3-1
	$\phi M_{nb} =$	$= \mathbf{307.04}$ [kip-ft]	
End Plate Bending Strength			
End plate width	$L = 7.000$ [in]	Thickness $t = 1.000$ [in]	

End plate width	$D_{plate} = 7.000$ [in]	Thickness $t_p = 1.000$ [in]	
Beam flange width	$b_{fb} = 6.020$ [in]		
Effective end plate width	$b_p = \min (b_{plate}, b_{fb} + 1")$	$= 7.000$ [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	$h_0 = 19.638$ [in]	$h_1 = 15.113$ [in]
	$g = 4.500$ [in]	
	$p_{fi} = 2.000$ [in]	$p_{fo} = 2.000$ [in]
	$s = 2.806$ [in]	$Y_p = 110.19$ [in]

Flexure resistance factor-LRFD $\phi_b = 0.90$ AISC 15th F1 (1)

End plate bending strength $\phi_b M_{pl} = \phi_b F_{yp} t_p^2 Y_p = \mathbf{413.22}$ [kip-ft] AISC DG4 Table 3.1

Max Moment in Demand

Moment by bolt strength-no prying	$\phi_b M_{nb} =$ from above calc	$= 307.04$ [kip-ft]
Moment by user input	$M_r =$ from user input	$= 50.00$ [kip-ft]
Moment in demand	$\phi M_{np} = \min (\phi_b M_{nb}, M_r)$	$= \mathbf{50.00}$ [kip-ft]

Check Thick End Plate Condition

Check thick end plate condition $\phi_b M_{pl} \geq 1.11 X \phi M_{np}$ AISC DG4 Eq 3.33
 ratio = **0.13** 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	$h_0 = 19.638$ [in]	$h_1 = 15.113$ [in]
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*** Stiffened Column Flange Case ***

Column section	$b_{fc} = 8.080$ [in]	$t_{fc} = 0.640$ [in]
	$F_{yc} = 50.0$ [ksi]	bolt gage $g = 4.500$ [in]
	$s = 3.015$ [in]	$c = 4.525$ [in]
Stiffener plate thickness	$t_s = 0.500$ [in]	
	$p_{si} = 2.013$ [in]	$p_{so} = 2.013$ [in]
	$Y_c = 194.0$ [in]	

Flexure resistance factor-LRFD $\phi_b = 0.90$ AISC 15th F1 (1)

Column flange bending strength $\phi_b M_{cf} = \phi_b F_{yc} t_{fc}^2 Y_c = \mathbf{297.94}$ [kip-ft] AISC DG4 Table 3.4

Check Thick Column Flange Condition

Check thick column flange condition $\phi_b M_{cf} \geq 1.11 X \phi M_{np}$ AISC DG4 Eq 3.35
 ratio = **0.19** thick plate

The thick end plate and column flange conditions are met. AISG DG4 is used and no bolt prying is considered AISC DG4 Eq 3.33 & 3.35

Bolt Moment Strength (No Prying)		ratio = 34.5 / 212.1	= 0.16	PASS
	bolt grade = A325-N	$F_t = 90.0$	[ksi]	AISC 15 th Table J3.2
	bolt dia $d_b = 1.000$ [in]	bolt area $A_b = 0.785$	[in ²]	
Bolt nominal tensile strength	$P_t = F_t A_b$	= 70.7	[kips]	AISC 15 th Eq J3-1
Tension bolt moment arm	$h_0 = 19.638$ [in]	$h_1 = 15.113$	[in]	
Flange force moment arm	$d_m = d_b - t_{fb}$	= 17.375	[in]	
Flange force required in tension	$P_{uf,t} = P_u / 2 - M_u / d_m$	= 34.5	[kips]	
Flange force resistance by bolt	$F_n = 2 P_t (h_0 + h_1) / d_m$	= 282.7	[kips]	AISC DG4 Eq 3.7
Bolt resistance factor-LRFD	$\phi = 0.75$			AISC 15 th Eq J3-1
	$\phi F_n =$	= 212.1	[kips]	AISC DG4 Eq 3.7
	ratio = 0.16	> $P_{uf,t}$	OK	
Bolt Shear Strength		ratio = 24.5 / 127.2	= 0.19	PASS
Bolt shear stress	bolt grade = A325-N	$F_{nv} = 54.0$	[ksi]	AISC 15 th Table J3.2
	bolt dia $d_b = 1.000$ [in]	bolt area $A_b = 0.785$	[in ²]	
Number of bolt carried shear	$n_s = 4.0$	shear plane $m = 1$		
Bolt group eccentricity coefficient	$C_{ec} =$	= 1.000		
Required shear strength	$V_u =$	= 24.5	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 169.6	[kips]	AISC 15 th Eq J3-1
Bolt resistance factor-LRFD	$\phi = 0.75$			AISC 15 th Eq J3-1
	$\phi R_n =$	= 127.2	[kips]	
	ratio = 0.19	> V_u	OK	

Bolt Bearing/TearOut Strength on End Plate		ratio = 24.5 / 127.2	= 0.19	PASS
Single Bolt Shear Strength				
Bolt shear stress	bolt grade = A325-N	$F_{nv} = 54.0$	[ksi]	AISC 15 th Table J3.2
	bolt dia $d_b = 1.000$	[in]	bolt area $A_b = 0.785$	[in ²]
Single bolt shear strength	$R_{n-bolt} = F_{nv} A_b$	= 42.4	[kips]	AISC 15 th Eq J3-1
Bolt Bearing/TearOut Strength on Plate				
Bolt hole diameter	bolt dia $d_b = 1$	[in]	bolt hole dia $d_h = 1\frac{1}{16}$	[in] AISC 15 th Table J3.3
Bolt spacing & edge distance	spacing $L_s = 4.525$	[in]	edge distance $L_e = 1.500$	[in]
Plate tensile strength	$F_u = 65.0$	[ksi]		
Plate thickness	$t = 1.000$	[in]		
Interior Bolt				
Bolt hole edge clear distance	$L_c = L_s - d_h$	= 3.463	[in]	
Bolt tear out/bearing strength	$R_{n-t\&b-in} = 1.2 L_c t F_u \leq 2.4 d_b t F_u$	= 156.0	[kips]	AISC 15 th Eq J3-6a
	= 270.1 \leq 156.0			
Bolt strength at interior	$R_{n-in} = \min (R_{n-t\&b-in} , R_{n-bolt})$	= 42.4	[kips]	
Edge Bolt				
Bolt hole edge clear distance	$L_c = L_e - d_h / 2$	= 0.969	[in]	
Bolt tear out/bearing strength	$R_{n-t\&b-ed} = 1.2 L_c t F_u \leq 2.4 d_b t F_u$	= 75.6	[kips]	AISC 15 th Eq J3-6a
	= 75.6 \leq 156.0			
Bolt strength at edge	$R_{n-ed} = \min (R_{n-t\&b-ed} , R_{n-bolt})$	= 42.4	[kips]	
Number of bolt	interior $n_{in} = 2$	edge $n_{ed} = 2$		
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n-in} + n_{ed} R_{n-ed}$	= 169.6	[kips]	
Required shear strength	$V_u =$	= 24.5	[kips]	
Bolt resistance factor-LRFD	$\phi = 0.75$			AISC 15 th J3-10
	$\phi R_n =$	= 127.2	[kips]	
	ratio = 0.19	> V_u	OK	

Bolt Bearing/TearOut Strength on Column Flange		ratio = 24.5 / 127.2	= 0.19	PASS
Single Bolt Shear Strength				
Bolt shear stress	bolt grade = A325-N	$F_{nv} = 54.0$ [ksi]		AISC 15 th Table J3.2
	bolt dia $d_b = 1.000$ [in]	bolt area $A_b = 0.785$ [in ²]		
Single bolt shear strength	$R_{n-bolt} = F_{nv} A_b$	= 42.4 [kips]		AISC 15 th Eq J3-1
Bolt Bearing/TearOut Strength on Plate				
Bolt hole diameter	bolt dia $d_b = 1$ [in]	bolt hole dia $d_h = 1\frac{1}{16}$ [in]		AISC 15 th Table J3.3
Bolt spacing	spacing $L_s = 4.525$ [in]			
Plate tensile strength	$F_u = 65.0$ [ksi]			
Plate thickness	$t = 0.640$ [in]			
Interior Bolt				
Bolt hole edge clear distance	$L_c = L_s - d_h$	= 3.463 [in]		
Bolt tear out/bearing strength	$R_{n-t\&b-in} = 1.2 L_c t F_u \leq 2.4 d_b t m F_u$			AISC 15 th Eq J3-6a
	= 172.8 \leq 99.8	= 99.8 [kips]		
Bolt strength at interior	$R_{n-in} = \min (R_{n-t\&b-in} , R_{n-bolt})$	= 42.4 [kips]		
Number of bolt	interior $n_{in} = 4$			
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n-in}$	= 169.6 [kips]		
Required shear strength	$V_u =$	= 24.5 [kips]		
Bolt resistance factor-LRFD	$\phi = 0.75$			AISC 15 th J3-10
	$\phi R_n =$	= 127.2 [kips]		
	ratio = 0.19	> V_u	OK	

End Plate Flexural Yielding		ratio = 34.5 / 285.4	= 0.12	PASS
End Plate Bending Strength				
End plate width	$b_{plate} = 7.000$ [in]	thickness $t_p = 1.000$ [in]		
Beam flange width	$b_{fb} = 6.020$ [in]			
Effective end plate width	$b_p = \min (b_{plate} , b_{fb} + 1")$	= 7.000 [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	$h_0 = 19.638$ [in]	$h_1 = 15.113$ [in]		
	$g = 4.500$ [in]			
	$p_{fi} = 2.000$ [in]	$p_{fo} = 2.000$ [in]		
	$s = 2.806$ [in]	$Y_p = 110.19$ [in]		
Flexure resistance factor-LRFD	$\phi_b = 0.90$			AISC 15 th F1 (1)
End plate bending strength	$\phi_b M_{pl} = \phi_b F_{yp} t_p^2 Y_p$	= 413.22 [kip-ft]		AISC DG4 Table 3.1
Flange force moment arm	$d_m = d_b - t_{fb}$	= 17.375 [in]		
Flange force required in tension	$P_{uf,t} = P_u / 2 - M_u / d_m$	= 34.5 [kips]		
Flange force provided by end plate bending	$\phi R_{pl} = \phi M_{pl} / d_m$	= 285.4 [kips]		AISC DG4 Eq 3.10
	ratio = 0.12	> $P_{uf,t}$	OK	

End Plate Shear Yielding		ratio = 17.3 / 210.0	= 0.08	PASS
Flange force required in tension	$P_{uf,t} = P_u / 2 - M_u / d_m$	= 17.3	[kips]	
End plate width	$b_{plate} = 7.000$	[in]	thickness $t_p = 1.000$	[in]
Beam flange width	$b_{fb} = 6.020$	[in]		
Effective end plate width	$b_p = \min (b_{plate}, b_{fb} + 1")$	= 7.000	[in]	AISC DG4 Page 9 item 5
Plate Shear Yielding Check				
Plate size	width $b_p = 7.000$	[in]	thickness $t_p = 1.000$	[in]
Plate yield strength	$F_y = 50.0$	[ksi]		
Plate gross area in shear	$A_{gv} = b_p t_p$	= 7.000	[in ²]	
Shear force required	$0.5 P_{uf,t} =$	= 17.3	[kips]	
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$	= 210.0	[kips]	AISC 15 th Eq J4-3
Resistance factor-LRFD	$\phi = 1.00$			AISC 15 th Eq J4-3
	$\phi R_n =$	= 210.0	[kips]	
	ratio = 0.08	> 0.5 $P_{uf,t}$	OK	

End Plate Shear Rupture		ratio = 17.3 / 138.9	= 0.12	PASS
Flange force required in tension	$P_{uf,t} = P_u / 2 - M_u / d_m$	= 17.3	[kips]	
End plate width	$b_{plate} = 7.000$	[in]	thickness $t_p = 1.000$	[in]
Beam flange width	$b_{fb} = 6.020$	[in]		
Effective end plate width	$b_p = \min (b_{plate}, b_{fb} + 1")$	= 7.000	[in]	AISC DG4 Page 9 item 5
Plate Shear Rupture Check				
Bolt hole diameter	bolt dia $d_b = 1$	[in]	bolt hole dia $d_h = 1\frac{1}{8}$	[in] AISC 15 th B4.3b
Number of bolt	$n = 2$			
Plate size	width $b_p = 7.000$	[in]	thickness $t_p = 1.000$	[in]
Plate tensile strength	$F_u = 65.0$	[ksi]		
Plate net area in shear	$A_{nv} = (b_p - n d_h) t_p$	= 4.750	[in ²]	
Shear force required	$0.5 P_{uf,t} =$	= 17.3	[kips]	
Plate shear rupture strength	$R_n = 0.6 F_u A_{nv}$	= 185.3	[kips]	AISC 15 th Eq J4-4
Resistance factor-LRFD	$\phi = 0.75$			AISC 15 th Eq J4-4
	$\phi R_n =$	= 138.9	[kips]	
	ratio = 0.12	> 0.5 $P_{uf,t}$	OK	

Beam Flange Weld Strength		ratio = 34.5 / 133.3	= 0.26	PASS
Flange force required in tension	$P_{uf,t} = P_u / 2 - M_u / d_m$	= 34.5	[kips]	
Fillet weld length - double fillet	$L = [b_{fb} + (b_{fb} - 2k_{1b})] / 2$ as dbl fillet	= 5.207	[in]	
Fillet Weld Strength Check				
Fillet weld leg size	$w = 1/2$	[in]	load angle $\theta = 90.0$	[°]
Electrode strength	$F_{EXX} = 70.0$	[ksi]	strength coeff $C_1 = 1.00$	AISC 15 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 15 th Page 8-9
Fillet weld shear strength	$R_{n-w} = 0.6 (C_1 \times 70 \text{ ksi}) 0.707 w n C_2$	= 44.541	[kip/in]	AISC 15 th Eq 8-1
Base metal - beam flange	thickness $t = 0.525$	[in]	tensile $F_u = 65.0$	[ksi]
Base metal - beam flange is in tension, <u>tensile</u> rupture as per AISC 15 th Eq J4-2 is checked				AISC 15 th J2.4
Base metal tensile rupture	$R_{n-b} = F_u t$	= 34.125	[kip/in]	AISC 15 th Eq J4-2
Double fillet linear shear strength	$R_n = \min (R_{n-w}, R_{n-b})$	= 34.125	[kip/in]	AISC 15 th Eq 9-2
Resistance factor-LRFD	$\phi = 0.75$			AISC 15 th Eq 8-1
	$\phi R_n =$	= 25.594	[kip/in]	
Shear resistance required	$P_{uf,t} =$	= 34.5	[kips]	
Fillet weld length - double fillet	$L =$	= 5.207	[in]	
Shear resistance provided	$\phi F_n = \phi R_n \times L$	= 133.3	[kips]	
	ratio = 0.26	> $P_{uf,t}$		OK

Beam Web Weld Strength		ratio = 24.5 / 71.5	= 0.34	PASS
Beam Web Effective Weld Length Calc				
Beam section	$d_b = 17.900$ [in]		$t_{fb} = 0.525$ [in]	
	$k_b = 1.188$ [in]			
Bolt diameter	$d_{bolt} = 1.000$ [in]		bolt inner pitch $p_{fi} = 2.000$ [in]	
Effective weld length case 1	$L_1 = 0.5 d_b - k_b$		= 7.762 [in]	AISC DG4 Page 38
Effective weld length case 2	$L_2 = d_b - 2t_{fb} - p_{fi} - 2 d_{bolt}$		= 12.850 [in]	AISC DG4 Page 38
Fillet weld length - double fillet	$L = \min(L_1, L_2)$		= 7.762 [in]	
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 15 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 15 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 15 th Eq 8-1
Base metal - beam web	thickness $t = 0.315$ [in]		tensile $F_u = 65.0$ [ksi]	
Base metal - beam web is in shear, <u>shear</u> rupture as per AISC 15 th Eq J4-4 is checked				
Base metal shear rupture	$R_{n-b} = 0.6 F_u t$		= 12.285 [kip/in]	AISC 15 th Eq J4-4
Double fillet linear shear strength	$R_n = \min(R_{n-w}, R_{n-b})$		= 12.285 [kip/in]	AISC 15 th Eq 9-2
Resistance factor-LRFD	$\phi = 0.75$			AISC 15 th Eq 8-1
	$\phi R_n =$		= 9.214 [kip/in]	
Shear resistance required	$V_u =$		= 24.5 [kips]	
Fillet weld length - double fillet	$L =$		= 7.762 [in]	
Shear resistance provided	$\phi F_n = \phi R_n \times L$		= 71.5 [kips]	
	ratio = 0.34		> V_u	OK

Column Flexural Yielding		ratio = 34.5 / 205.8	= 0.17	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	$h_0 = 19.638$ [in]	$h_1 = 15.113$ [in]		
*** Stiffened Column Flange Case ***				
Column section	$b_{fc} = 8.080$ [in]	$t_{fc} = 0.640$ [in]		
	$F_{yc} = 50.0$ [ksi]	bolt gage $g = 4.500$ [in]		
	$s = 3.015$ [in]	$c = 4.525$ [in]		
Stiffener plate thickness	$t_s = 0.500$ [in]			
	$p_{s1} = 2.013$ [in]	$p_{s0} = 2.013$ [in]		
	$Y_c = 194.0$ [in]			
Flexure resistance factor-LRFD	$\phi_b = 0.90$			AISC 15 th F1 (1)
Column flange bending strength	$\phi_b M_{cf} = \phi_b F_{yc} t_{fc}^2 Y_c$	= 297.94 [kip-ft]		AISC DG4 Table 3.4
Flange force moment arm	$d_m = d_b - t_{fb}$	= 17.375 [in]		
Flange force required in tension	$P_{uf,t} = P_u / 2 - M_u / d_m$	= 34.5 [kips]		
Flange force provided by column flange bending	$\phi R_{cf} = \phi M_{cf} / d_m$	= 205.8 [kips]		AISC DG4 Eq 3.21
	ratio = 0.17	> $P_{uf,t}$	OK	

Column Web Yielding		ratio = 34.5 / 95.9	= 0.36	PASS
Flange force moment arm	$d_m = d_b - t_{fb}$	= 17.375 [in]		
Flange force in demand	$P_{uf} = \max (P_{uf,t} , P_{uf,c})$	= 34.5 [kips]		AISC DG13 Eq 4.2-1
Column section	$d_c = 12.200$ [in]	$t_{fc} = 0.640$ [in]		
	$t_{wc} = 0.370$ [in]	$k_c = 1.140$ [in]		
Column yield strength	$F_{yc} = 50.0$ [ksi]			
Distance from to top of column to top of beam flange	$d_{end} = 6.250$ [in]			
Top column reduction factor	$C_t = 0.5$			AISC DG4 Eq 3.24
Beam flange fillet weld size	$w = 0.500$ [in]	beam flange $t_{fb} = 0.525$ [in]		
Length of bearing	$N = t_{fb} + 2 w$	= 1.525 [in]		AISC DG4 Eq 3.24
End plate thickness	$t_p = 1.000$ [in]			
Column web yielding strength	$R_n = C_t (6 k_c + N + 2 t_p) F_{yc} t_{wc}$	= 95.9 [kips]		AISC DG4 Eq 3.24
Resistance factor-LRFD	$\phi = 1.00$			AISC 15 th J10.2
	$\phi R_n =$	= 95.9 [kips]		
	ratio = 0.36	> P_{uf}	OK	

Column Web Buckling		ratio = 34.5 / 132.8	= 0.26	PASS
Flange force moment arm	$d_m = d_b - t_{fb}$	= 17.375	[in]	
Flange force required in compression	$P_{uf_c} = P_u / 2 - M_u / d_m$	= 34.5	[kips]	
Column section	$d_c = 12.200$ [in]	$t_{fc} = 0.640$	[in]	
	$t_{wc} = 0.370$ [in]	$k_c = 1.140$	[in]	
	$h = d_c - 2 k_c$	= 9.920	[in]	
Column yield strength	$F_{yc} = 50.0$ [ksi]	$E_c = 29000$	[ksi]	
Distance from top of beam flange to top of column	$d_{end-flg} = 6.250$ [in]	beam flange $t_{fb} = 0.525$	[in]	
Distance from center of flange force to top of column	$d_{end-F} = d_{end-flg} + 0.5 t_{fb}$	= 6.513	[in]	
	$d_{end-F} \geq d_c / 2$, R_n has no reduction			AISC 15 th J10.5
Top column reduction factor	$C_t = 1.0$			AISC 15 th J10.5
Column web buckling strength	$R_n = \frac{C_t 24 t_{wc}^3 \sqrt{E_c F_{yc}}}{h}$	= 147.6	[kips]	AISC 15 th Eq J10-8
Resistance factor-LRFD	$\phi = 0.90$			AISC 15 th J10.5
	$\phi R_n =$	= 132.8	[kips]	
	ratio = 0.26	> P_{uf_c}	OK	

Column Web Crippling		ratio = 34.5 / 179.7	= 0.19	PASS
Flange force moment arm	$d_m = d_b - t_{fb}$	= 17.375	[in]	
Flange force required in compression	$P_{uf_c} = P_u / 2 - M_u / d_m$	= 34.5	[kips]	
Column section	$d_c = 12.200$ [in]	$t_{fc} = 0.640$	[in]	
	$t_{wc} = 0.370$ [in]	$k_c = 1.140$	[in]	
Column yield strength	$F_{yc} = 50.0$ [ksi]	$E_c = 29000$	[ksi]	
Beam flange fillet weld size	$w = 0.500$ [in]	beam flange $t_{fb} = 0.525$	[in]	
End plate thickness	$t_p = 1.000$ [in]			
Length of bearing	$l_b = t_{fb} + 2 w + 2 t_p$	= 3.525	[in]	
Distance from top of column to top of beam flange	$d_{end-flg} =$	= 6.250	[in]	
Distance from top of column to center of flange force	$d_{end-F} = d_{end-flg} + 0.5 t_{fb}$	= 6.513	[in]	
	$d_{end-F} \geq d_c / 2$, use Eq J10-4			AISC 15 th Eq J10-4
Column web crippling strength	$R_n = 0.8 t_{wc}^2 \left[1 + 3 \frac{l_b}{d_c} \left(\frac{t_{wc}}{t_{fc}} \right)^{1.5} \right] \times \left(\frac{E_c F_{yc} t_{fc}}{t_{wc}} \right)^{0.5}$	= 239.5	[kips]	AISC 15 th Eq J10-4
Resistance factor-LRFD	$\phi = 0.75$			AISC 15 th J10.3
	$\phi R_n =$	= 179.7	[kips]	
	ratio = 0.19	> P_{uf_c}	OK	

Column Panel Zone Shear		ratio = 34.5 / 136.9	= 0.25	PASS
Panel zone shear force	$V_p = P_{f-TR} - P_{f-TL} - V_s$	= 34.5	[kips]	
Column W12X50	$d_c = 12.200$ [in]	$b_{cf} = 8.080$	[in]	
	$t_{cf} = 0.640$ [in]	$t_{cw} = 0.370$	[in]	
	$A_c = 14.600$ [in ²]	$F_{cy} = 50.0$	[ksi]	
Beam W18X40	$d_b = 17.900$ [in]	$t_{bf} = 0.525$	[in]	
Beam flange thickness	$t_{bf} =$	= 0.525	[in]	
Moment arm between flanges	$d_m = d - t_{bf}$	= 17.900	[in]	
Column axial compression	$P_r =$ from user input	= 15.4	[kips]	
Column axial yield strength	$P_y = F_{cy} A_c$	= 730.0	[kips]	AISC 15 th J10.6 (b)
LRFD-ASD force adjustment factor	$\alpha =$ for LRFD	= 1.0		AISC 341-16 D1.2a (b)
	when $\alpha P_r \leq 0.75 P_y$, use Eq J10-11			AISC 15 th Eq J10-11
Column web panel zone capacity	$R_n = 0.6 F_{cy} d_c t_{cw} \left(1 + \frac{3 b_{cf} t_{cf}^2}{d_m d_c t_{cw}}\right)$	= 152.1	[kips]	AISC 15 th Eq J10-11
Resistance factor-LRFD	$\phi = 0.90$			AISC 15 th J10.6
	$\phi R_n =$	= 136.9	[kips]	
	ratio = 0.25	> V_p		OK

Seismic Material & Geometry Limitations		PASS		
Check Max Beam Yield Stress				
Condition : beam max material $F_y \leq F_{y_max} = 55$ ksi AISC 341-16 A3.1				
Beam yield strength		$F_{yb} = 50.0$	[ksi]	
		$\leq F_{y_max}$		OK
Check Max Column Yield Stress				
Condition : column max material $F_y \leq F_{y_max} = 55$ ksi AISC 341-16 A3.1				
Column yield strength		$F_{yc} = 50.0$	[ksi]	
		$\leq F_{y_max}$		OK
Check Max Gage				
Condition : max bolt gage is limited to beam b_f AISC 358-16 6.9.1				
Beam flange width	$b_f = 6.020$ [in]	gage = 4.500	[in]	
		$\leq b_f$		OK
Check Min Pitch				
Condition : min bolt pitch distance $p \geq d_b + 0.50$ in AISC 358-16 6.9.2				
Bolt pitch	$p_{fi} = 2.000$ [in]	$p_{fo} = 2.000$	[in]	
Bolt dia	$d_b = 1.000$ [in]			
Min bolt pitch distance allowed	$p_{min} = d_b + 0.50$ in	= 1.500	[in]	
Min bolt pitch distance	$p = \min(p_{fi}, p_{fo})$	= 2.000	[in]	
		$\geq p_{min}$		OK

Seismic Bolt Limitation	PASS
Check Bolt Grade Material	
Condition : Bolt grade must be A325 or A490	
Bolt grade = A325-N	OK AISC 358-16 4.1
Check Bolt Hole Type	
Condition : Bolt hole type shall be STD or SSLT	
Bolt hole type = PL1=STD PL2=STD	OK AISC 341-16 D2.2 (3)

Seismic End Plate Width Limitation	PASS
Check End Plate Width	
Condition : end palte width $b_p \geq$ beam flange width b_f	
Beam flange width $b_f = 6.020$ [in]	
End plate width $b_p = 7.000$ [in]	
	$\geq b_f$ OK AISC 358-16 6.9.3

Seismic Column Beam Moment Ratio	N/A
The column beam moment ratio requirement applys to SMF connection only. It's OMF connection, so the column beam moment ratio check is not required.	AISC 341-16 E3.4a

Seismic Check Thick Plate to Meet AISC 358-10 6.10 Assumption	PASS
The seismic design of extended end plate moment connection is based on the Design Procedure stated in AISC 358-10 6.10	
Refer to AISC 358-10 6.10 commentary on page 9.2-139. The Design Procedure is very similar to that in AISC Design Guide 4 (Murray and Sumner, 2003) except that different resistance factors are used.	
AISC Design Guide 4 is based on the thick plate assumption and the bolt has no prying action.	
For this reason the thick plate assumption is checked here and it will flag FAIL if the thick plate condition is not met. If FAIL user can increase the end plate thickness to get this check pass.	
Bolt Moment Strength (No Prying)	
bolt grade = A325-N	$F_t = 90.0$ [ksi] AISC 15 th Table J3.2
bolt dia $d_b = 1.000$ [in]	bolt area $A_b = 0.785$ [in ²]
Bolt nominal tensile strength $P_t = F_t A_b$	$= 70.7$ [kips] AISC 15 th Eq J3-1
Tension bolt moment arm $h_0 = 19.638$ [in]	$h_1 = 15.113$ [in]
Bolt moment strength (no prying) $M_{nb} = 2 P_t (h_0 + h_1)$	$= 409.39$ [kip-ft] AISC DG4 Eq 3.7
Bolt resistance factor-LRFD $\phi_n = 0.90$	AISC 358-16 2.4.1
	$\phi_n M_{nb} = 368.45$ [kip-ft]
End Plate Bending Strength	
End plate width $b_{plate} = 7.000$ [in]	thickness $t_p = 1.000$ [in]
Beam flange width $b_{fb} = 6.020$ [in]	
Effective end plate width $b_p = \min (b_{plate}, b_{fb} + 1")$	$= 7.000$ [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	
Tension bolt moment arm $h_0 = 19.638$ [in]	$h_1 = 15.113$ [in]
	$g = 4.500$ [in]

	$p_{fi} = 2.000$ [in]	$p_{fo} = 2.000$ [in]	
	$s = 2.806$ [in]	$Y_p = 110.19$ [in]	
Flexure resistance factor-LRFD	$\phi_d = 1.00$		AISC 358-16 2.4.1
End plate bending strength	$\phi_d M_{pl} = \phi_d F_{yp} t_p^2 Y_p$	= 459.14 [kip-ft]	AISC DG4 Table 3.1

Max Moment in Demand

Moment by bolt strength-no prying	$\phi_d M_{nb} =$ from above calc	= 368.45 [kip-ft]
Moment by user input	$M_r =$ from user input	= 50.00 [kip-ft]
Moment in demand	$\phi M_{np} = \min(\phi_d M_{nb}, M_r)$	= 50.00 [kip-ft]

Check Thick End Plate Condition

Check thick end plate condition	$\phi_d M_{pl} >= 1.11 \times \phi M_{np}$	AISC DG4 Eq 3.33
	ratio = 0.12 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	$h_0 = 19.638$ [in]	$h_1 = 15.113$ [in]
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*** Stiffened Column Flange Case ***

Column section	$b_{fc} = 8.080$ [in]	$t_{fc} = 0.640$ [in]	
	$F_{yc} = 50.0$ [ksi]	bolt gage $g = 4.500$ [in]	
	$s = 3.015$ [in]	$c = 4.525$ [in]	
Stiffener plate thickness	$t_s = 0.500$ [in]		
	$p_{si} = 2.013$ [in]	$p_{so} = 2.013$ [in]	
	$Y_c = 194.0$ [in]		
Flexure resistance factor-LRFD	$\phi_d = 1.00$	AISC 358-16 2.4.1	
Column flange bending strength	$\phi_d M_{cf} = \phi_d F_{yc} t_{fc}^2 Y_c$	= 331.04 [kip-ft]	AISC DG4 Table 3.4

Check Thick Column Flange Condition

Check thick column flange condition	$\phi_d M_{cf} >= 1.11 \times \phi M_{np}$	AISC DG4 Eq 3.35
	ratio = 0.17 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

Seismic Bolt Moment Strength (No Prying)		ratio = 199.7 / 254.5	= 0.78	PASS
	bolt grade = A325-N	$F_t = 90.0$	[ksi]	AISC 15 th Table J3.2
	bolt dia $d_b = 1.000$ [in]	bolt area $A_b = 0.785$	[in ²]	
Bolt nominal tensile strength	$P_t = F_t A_b$	= 70.7	[kips]	AISC 15 th Eq J3-1
Tension bolt moment arm	$h_0 = 19.638$ [in]	$h_1 = 15.113$	[in]	
Flange force moment arm	$d_m = d_b - t_{fb}$	= 17.375	[in]	
Seismic flange force	$P_{uf,t}$ = see Seismic Moment and Beam Flange Force Calc	= 199.7	[kips]	
Flange force resistance by bolt	$F_n = 2 P_t (h_0 + h_1) / d_m$	= 282.7	[kips]	AISC DG4 Eq 3.7
Bolt resistance factor-LRFD	$\phi_n = 0.90$			AISC 358-16 2.4.1
	$\phi_n F_n =$	= 254.5	[kips]	AISC DG4 Eq 3.7
	ratio = 0.78	> $P_{uf,t}$	OK	

Seismic Bolt Shear Strength		ratio = 24.5 / 152.7	= 0.16	PASS
Bolt shear stress	bolt grade = A325-N	$F_{nv} = 54.0$	[ksi]	AISC 15 th Table J3.2
	bolt dia $d_b = 1.000$ [in]	bolt area $A_b = 0.785$	[in ²]	
Number of bolt carried shear	$n_s = 4.0$	shear plane $m = 1$		
Bolt group eccentricity coefficient	$C_{ec} =$	= 1.000		
Required shear strength	V_u = see Seismic Moment and Beam Flange Force Calc	= 24.5	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 169.6	[kips]	AISC 15 th Eq J3-1
Bolt resistance factor-LRFD	$\phi_n = 0.90$			AISC 358-16 2.4.1
	$\phi_n R_n =$	= 152.7	[kips]	
	ratio = 0.16	> V_u	OK	

Seismic Bolt Bearing/TearOut Strength on End Plate		ratio = 24.5 / 152.7	= 0.16	PASS
Single Bolt Shear Strength				
Bolt shear stress	bolt grade = A325-N	$F_{nv} = 54.0$	[ksi]	AISC 15 th Table J3.2
	bolt dia $d_b = 1.000$	[in]	bolt area $A_b = 0.785$	[in ²]
Single bolt shear strength	$R_{n-bolt} = F_{nv} A_b$	= 42.4	[kips]	AISC 15 th Eq J3-1
Bolt Bearing/TearOut Strength on Plate				
Bolt hole diameter	bolt dia $d_b = 1$	[in]	bolt hole dia $d_h = 1\frac{1}{16}$	[in] AISC 15 th Table J3.3
Bolt spacing & edge distance	spacing $L_s = 4.525$	[in]	edge distance $L_e = 1.500$	[in]
Plate tensile strength	$F_u = 65.0$	[ksi]		
Plate thickness	$t = 1.000$	[in]		
Interior Bolt				
Bolt hole edge clear distance	$L_c = L_s - d_h$	= 3.463	[in]	
Bolt tear out/bearing strength	$R_{n-t\&b-in} = 1.2 L_c t F_u \leq 2.4 d_b t F_u$	= 156.0	[kips]	AISC 15 th Eq J3-6a
	= 270.1 \leq 156.0			
Bolt strength at interior	$R_{n-in} = \min (R_{n-t\&b-in} , R_{n-bolt})$	= 42.4	[kips]	
Edge Bolt				
Bolt hole edge clear distance	$L_c = L_e - d_h / 2$	= 0.969	[in]	
Bolt tear out/bearing strength	$R_{n-t\&b-ed} = 1.2 L_c t F_u \leq 2.4 d_b t F_u$	= 75.6	[kips]	AISC 15 th Eq J3-6a
	= 75.6 \leq 156.0			
Bolt strength at edge	$R_{n-ed} = \min (R_{n-t\&b-ed} , R_{n-bolt})$	= 42.4	[kips]	
Number of bolt	interior $n_{in} = 2$	edge $n_{ed} = 2$		
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n-in} + n_{ed} R_{n-ed}$	= 169.6	[kips]	
Required shear strength	$V_u =$ see Seismic Moment and Beam Flange Force Calc	= 24.5	[kips]	
Bolt resistance factor-LRFD	$\phi_n = 0.90$			AISC 358-16 2.4.1
	$\phi_n R_n =$	= 152.7	[kips]	
	ratio = 0.16	$> V_u$	OK	

Seismic Bolt Bearing/TearOut Strength on Column Flange		ratio = 24.5 / 152.7	= 0.16	PASS
Single Bolt Shear Strength				
Bolt shear stress	bolt grade = A325-N	$F_{nv} = 54.0$ [ksi]		AISC 15 th Table J3.2
	bolt dia $d_b = 1.000$ [in]	bolt area $A_b = 0.785$ [in ²]		
Single bolt shear strength	$R_{n-bolt} = F_{nv} A_b$	= 42.4 [kips]		AISC 15 th Eq J3-1
Bolt Bearing/TearOut Strength on Plate				
Bolt hole diameter	bolt dia $d_b = 1$ [in]	bolt hole dia $d_h = 1\frac{1}{16}$ [in]		AISC 15 th Table J3.3
Bolt spacing	spacing $L_s = 4.525$ [in]			
Plate tensile strength	$F_u = 65.0$ [ksi]			
Plate thickness	$t = 0.640$ [in]			
Interior Bolt				
Bolt hole edge clear distance	$L_c = L_s - d_h$	= 3.463 [in]		
Bolt tear out/bearing strength	$R_{n-t\&b-in} = 1.2 L_c t F_u \leq 2.4 d_b t m F_u$	= 99.8 [kips]		AISC 15 th Eq J3-6a
	= 172.8 \leq 99.8			
Bolt strength at interior	$R_{n-in} = \min (R_{n-t\&b-in}, R_{n-bolt})$	= 42.4 [kips]		
Number of bolt	interior $n_{in} = 4$			
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n-in}$	= 169.6 [kips]		
Required shear strength	$V_u =$ see Seismic Moment and Beam Flange Force Calc	= 24.5 [kips]		
Bolt resistance factor-LRFD	$\phi_n = 0.90$			AISC 358-16 2.4.1
	$\phi_n R_n =$	= 152.7 [kips]		
	ratio = 0.16	> V_u	OK	

Seismic End Plate Shear		ratio = 99.8 / 166.7	= 0.60	PASS
Seismic flange force	$0.5F_{fu} =$ see Seismic Moment and Beam Flange Force Calc	= 99.8 [kips]		
End plate	width $b_p = 7.000$ [in]	thickness $t_p = 1.000$ [in]		
	$F_{yp} = 50.0$ [ksi]	$F_{up} = 65.0$ [ksi]		
Check End Plate Shear Yielding				
Plate shear yield strength	$R_{ny} = 0.6 F_{yp} b_p t_p$	= 210.0 [kips]		AISC 358-16 Eq 6.10-7
Resistance factor-LRFD	$\phi_d = 1.00$			AISC 358-16 2.4.1
	$\phi_d R_{ny} =$	= 210.0 [kips]		
	ratio = 0.48	> $0.5F_{fu}$	OK	
Check End Plate Shear Rupture				
Bolt hole diameter	bolt dia $d_b = 1$ [in]	bolt hole dia $d_h = 1\frac{1}{8}$ [in]		AISC 15 th B4.3b
Number of bolt	$n = 2$			
Plate net area in shear	$A_{nv} = (b_p - n d_h) t_p$	= 4.750 [in ²]		
Plate shear rupture strength	$R_{nr} = 0.6 F_{up} A_{nv}$	= 185.3 [kips]		AISC 358-16 Eq 6.10-8
Resistance factor-LRFD	$\phi_n = 0.90$			AISC 358-16 2.4.1
	$\phi_n R_{nr} =$	= 166.7 [kips]		
	ratio = 0.60	> $0.5F_{fu}$	OK	

Seismic End Plate Flexural Yielding		ratio = 199.7 / 317.1	= 0.63	PASS
End Plate Bending Strength				
End plate width	$b_{plate} = 7.000$ [in]	thickness $t_p = 1.000$ [in]		
Beam flange width	$b_{fb} = 6.020$ [in]			
Effective end plate width	$b_p = \min (b_{plate}, b_{fb} + 1")$	= 7.000 [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	$h_0 = 19.638$ [in]	$h_1 = 15.113$ [in]		
	$g = 4.500$ [in]			
	$p_{fi} = 2.000$ [in]	$p_{fo} = 2.000$ [in]		
	$s = 2.806$ [in]	$Y_p = 110.19$ [in]		
Flexure resistance factor-LRFD	$\phi_d = 1.00$			AISC 358-16 2.4.1
End plate bending strength	$\phi_d M_{pl} = \phi_d F_{yp} t_p^2 Y_p$	= 459.14 [kip-ft]		AISC DG4 Table 3.1
Seismic flange force	$P_{uf,t} =$ see Seismic Moment and Beam Flange Force Calc	= 199.7 [kips]		
Flange force provided by end plate bending	$\phi R_{pl} = \phi M_{pl} / d_m$	= 317.1 [kips]		AISC DG4 Eq 3.10
	ratio = 0.63	> $P_{uf,t}$	OK	

Seismic End Plate Shear Yielding		ratio = 99.8 / 210.0	= 0.48	PASS
Flange force required in tension	$P_{uf,t} = P_u / 2 - M_u / d_m$	= 99.8 [kips]		
End plate width	$b_{plate} = 7.000$ [in]	thickness $t_p = 1.000$ [in]		
Beam flange width	$b_{fb} = 6.020$ [in]			
Effective end plate width	$b_p = \min (b_{plate}, b_{fb} + 1")$	= 7.000 [in]		AISC DG4 Page 9 item 5
Plate Shear Yielding Check				
Plate size	width $b_p = 7.000$ [in]	thickness $t_p = 1.000$ [in]		
Plate yield strength	$F_y = 50.0$ [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$	= 7.000 [in ²]		
Shear force required	$0.5 P_{uf,t} =$	= 99.8 [kips]		
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$	= 210.0 [kips]		AISC 15 th Eq J4-3
Resistance factor-LRFD	$\phi = 1.00$			AISC 15 th Eq J4-3
	$\phi R_n =$	= 210.0 [kips]		
	ratio = 0.48	> $0.5 P_{uf,t}$	OK	

Seismic End Plate Shear Rupture		ratio = 99.8 / 138.9	= 0.72	PASS
Flange force required in tension	$P_{uf,t} = P_u / 2 - M_u / d_m$	= 99.8	[kips]	
End plate width	$b_{plate} = 7.000$ [in]	thickness $t_p = 1.000$	[in]	
Beam flange width	$b_{fb} = 6.020$ [in]			
Effective end plate width	$b_p = \min (b_{plate}, b_{fb} + 1")$	= 7.000	[in]	AISC DG4 Page 9 item 5
Plate Shear Rupture Check				
Bolt hole diameter	bolt dia $d_b = 1$ [in]	bolt hole dia $d_h = 1\frac{1}{8}$	[in]	AISC 15 th B4.3b
Number of bolt	$n = 2$			
Plate size	width $b_p = 7.000$ [in]	thickness $t_p = 1.000$	[in]	
Plate tensile strength	$F_u = 65.0$ [ksi]			
Plate net area in shear	$A_{nv} = (b_p - n d_h) t_p$	= 4.750	[in ²]	
Shear force required	$0.5 P_{uf,t} =$	= 99.8	[kips]	
Plate shear rupture strength	$R_n = 0.6 F_u A_{nv}$	= 185.3	[kips]	AISC 15 th Eq J4-4
Resistance factor-LRFD	$\phi = 0.75$			AISC 15 th Eq J4-4
	$\phi R_n =$	= 138.9	[kips]	
	ratio = 0.72	> 0.5 $P_{uf,t}$		OK

Seismic Beam Flange Weld Strength		ratio = 199.7 / 159.9	= 1.25	FAIL
Flange force required in tension	$P_{uf,t} = P_u / 2 - M_u / d_m$		= 199.7	[kips]
Fillet weld length - double fillet	$L = [b_{fb} + (b_{fb} - 2k_{1b})] / 2$ as dbl fillet		= 5.207	[in]
Fillet Weld Strength Check				
Fillet weld leg size	$w = 1/2$	[in]	load angle $\theta = 90.0$	[°]
Electrode strength	$F_{EXX} = 70.0$	[ksi]	strength coeff $C_1 = 1.00$	AISC 15 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 15 th Page 8-9
Fillet weld shear strength	$R_{n-w} = 0.6 (C_1 \times 70 \text{ ksi}) 0.707 w n C_2$		= 44.541	[kip/in] AISC 15 th Eq 8-1
Base metal - beam flange	thickness $t = 0.525$	[in]	tensile $F_u = 65.0$	[ksi]
Base metal - beam flange is in tension, <u>tensile</u> rupture as per AISC 15 th Eq J4-2 is checked				AISC 15 th J2.4
Base metal tensile rupture	$R_{n-b} = F_u t$		= 34.125	[kip/in] AISC 15 th Eq J4-2
For seismic nonductile limit state, weld strength use $\phi = 0.75$, base metal rupture use $\phi_n = 0.9$				AISC 358-16 2.4.1
Increase base metal rupture strength due to higher ϕ value when compare to weld strength				
Double fillet linear shear strength	$R_n = \min (R_{n-w}, R_{n-b} \times \frac{0.90}{0.75})$		= 40.950	[kip/in] AISC 15 th Eq 9-2
Resistance factor-LRFD	$\phi = 0.75$			AISC 15 th Eq 8-1
	$\phi R_n =$		= 30.713	[kip/in]
Shear resistance required	$P_{uf,t} =$		= 199.7	[kips]
Fillet weld length - double fillet	$L =$		= 5.207	[in]
Shear resistance provided	$\phi F_n = \phi R_n \times L$		= 159.9	[kips]
	ratio = 1.25		< $P_{uf,t}$	NG
The fail is caused by base metal rupture not by weld metal rupture as such increasing weld size won't help.				
The user has the following options to get this check passed				
1) Increase the beam flange thickness or strength				
2) Increase the weld length				
3) Reduce the force in demand				

Seismic Beam Web Weld Strength		ratio = 24.5 / 85.8	= 0.29	PASS
Beam Web Effective Weld Length Calc				
Beam section	$d_b = 17.900$ [in]		$t_{fb} = 0.525$ [in]	
	$k_b = 1.188$ [in]			
Bolt diameter	$d_{bolt} = 1.000$ [in]		bolt inner pitch $p_{fi} = 2.000$ [in]	
Effective weld length case 1	$L_1 = 0.5 d_b - k_b$		= 7.762 [in]	AISC DG4 Page 38
Effective weld length case 2	$L_2 = d_b - 2t_{fb} - p_{fi} - 2 d_{bolt}$		= 12.850 [in]	AISC DG4 Page 38
Fillet weld length - double fillet	$L = \min(L_1, L_2)$		= 7.762 [in]	
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 15 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 15 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 15 th Eq 8-1
Base metal - beam web	thickness $t = 0.315$ [in]		tensile $F_u = 65.0$ [ksi]	
Base metal - beam web is in shear, <u>shear</u> rupture as per AISC 15 th Eq J4-4 is checked				AISC 15 th J2.4
Base metal shear rupture	$R_{n-b} = 0.6 F_u t$		= 12.285 [kip/in]	AISC 15 th Eq J4-4
For seismic nonductile limit state, weld strength use $\phi = 0.75$, base metal rupture use $\phi_n = 0.9$ Increase base metal rupture strength due to higher ϕ value when compare to weld strength				AISC 358-16 2.4.1
Double fillet linear shear strength	$R_n = \min (R_{n-w}, R_{n-b} \times \frac{0.90}{0.75})$		= 14.742 [kip/in]	AISC 15 th Eq 9-2
Resistance factor-LRFD	$\phi = 0.75$			AISC 15 th Eq 8-1
	$\phi R_n =$		= 11.057 [kip/in]	
Shear resistance required	$V_u =$		= 24.5 [kips]	
Fillet weld length - double fillet	$L =$		= 7.762 [in]	
Shear resistance provided	$\phi F_n = \phi R_n \times L$		= 85.8 [kips]	
	ratio = 0.29		> V_u	OK

Seismic Column Flexural Yielding		ratio = 199.7 / 228.6	= 0.87	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	$h_0 = 19.638$ [in]	$h_1 = 15.113$ [in]		
*** Stiffened Column Flange Case ***				
Column section	$b_{fc} = 8.080$ [in]	$t_{fc} = 0.640$ [in]		
	$F_{yc} = 50.0$ [ksi]	bolt gage $g = 4.500$ [in]		
	$s = 3.015$ [in]	$c = 4.525$ [in]		
Stiffener plate thickness	$t_s = 0.500$ [in]			
	$p_{si} = 2.013$ [in]	$p_{so} = 2.013$ [in]		
	$Y_c = 194.0$ [in]			
Flexure resistance factor-LRFD	$\phi_d = 1.00$		AISC 358-16 2.4.1	
Column flange bending strength	$\phi_d M_{cf} = \phi_d F_{yc} t_{fc}^2 Y_c$	= 331.04 [kip-ft]	AISC DG4 Table 3.4	
Seismic flange force	$P_{uf,t} =$ see Seismic Moment and Beam Flange Force Calc	= 199.7 [kips]		
Flange force provided by column flange bending	$\phi R_{cf} = \phi M_{cf} / d_m$	= 228.6 [kips]	AISC DG4 Eq 3.21	
	ratio = 0.87	> $P_{uf,t}$	OK	
Seismic Column Web Yielding		199.7 / 95.9	N/A	
Flange force moment arm	$d_m = d_b - t_{fb}$	= 17.375 [in]		
Seismic flange force	$P_{uf,c} =$ see Seismic Moment and Beam Flange Force Calc	= 199.7 [kips]		
Column section	$d_c = 12.200$ [in]	$t_{fc} = 0.640$ [in]		
	$t_{wc} = 0.370$ [in]	$k_c = 1.140$ [in]		
Column yield strength	$F_{yc} = 50.0$ [ksi]			
Distance from to top of column to top of beam flange	$d_{end} = 6.250$ [in]			
Top column reduction factor	$C_t = 0.5$		AISC DG4 Eq 3.24	
Beam flange fillet weld size	$w = 0.500$ [in]	beam flange $t_{fb} = 0.525$ [in]		
Length of bearing	$N = t_{fb} + 2 w$	= 1.525 [in]	AISC DG4 Eq 3.24	
End plate thickness	$t_p = 1.000$ [in]			
Column web yielding strength	$R_n = C_t (6 k_c + N + 2 t_p) F_{yc} t_{wc}$	= 95.9 [kips]	AISC DG4 Eq 3.24	
Resistance factor-LRFD	$\phi = 1.00$		AISC 341-16 E3.6e (1)	
	$\phi R_n =$	= 95.9 [kips]		
Unbalanced force to be resisted by transverse stiffeners	$R_s = P_{uf} - \phi R_n$	= 103.8 [kips]		

Seismic Column Web Buckling		199.7 / 147.6	N/A
Flange force moment arm	$d_m = d_b - t_{fb}$	= 17.375 [in]	
Seismic flange force	P_{uf_c} = see Seismic Moment and Beam Flange Force Calc	= 199.7 [kips]	
Column section	$d_c = 12.200$ [in] $t_{wc} = 0.370$ [in] $h = d_c - 2 k_c$	$t_{fc} = 0.640$ [in] $k_c = 1.140$ [in] = 9.920 [in]	
Column yield strength	$F_{yc} = 50.0$ [ksi]	$E_c = 29000$ [ksi]	
Distance from top of beam flange to top of column	$d_{end-flg} = 6.250$ [in]	beam flange $t_{fb} = 0.525$ [in]	
Distance from center of flange force to top of column	$d_{end-F} = d_{end-flg} + 0.5 t_{fb}$ $d_{end-F} \geq d_c/2$, R_n has no reduction	= 6.513 [in]	AISC 15 th J10.5
Top column reduction factor	$C_t = 1.0$		AISC 15 th J10.5
Column web buckling strength	$R_n = \frac{C_t 24 t_{wc}^3 \sqrt{E_c F_{yc}}}{h}$	= 147.6 [kips]	AISC 15 th Eq J10-8
Resistance factor-LRFD	$\phi = 1.00$ $\phi R_n =$	= 147.6 [kips]	AISC 341-16 E3.6e (1)
Unbalanced force to be resisted by transverse stiffeners	$R_s = P_{uf_c} - \phi R_n$	= 52.1 [kips]	

Seismic Column Web Crippling		ratio = 199.7 / 239.5	= 0.83 PASS
Flange force moment arm	$d_m = d_b - t_{fb}$	= 17.375 [in]	
Seismic flange force	P_{uf_c} = see Seismic Moment and Beam Flange Force Calc	= 199.7 [kips]	
Column section	$d_c = 12.200$ [in] $t_{wc} = 0.370$ [in]	$t_{fc} = 0.640$ [in] $k_c = 1.140$ [in]	
Column yield strength	$F_{yc} = 50.0$ [ksi]	$E_c = 29000$ [ksi]	
Beam flange fillet weld size	$w = 0.500$ [in]	beam flange $t_{fb} = 0.525$ [in]	
End plate thickness	$t_p = 1.000$ [in]		
Length of bearing	$l_b = t_{fb} + 2 w + 2 t_p$	= 3.525 [in]	
Distance from top of column to top of beam flange	$d_{end-flg} =$	= 6.250 [in]	
Distance from top of column to center of flange force	$d_{end-F} = d_{end-flg} + 0.5 t_{fb}$ $d_{end-F} \geq d_c/2$, use Eq J10-4	= 6.513 [in]	AISC 15 th Eq J10-4
Column web crippling strength	$R_n = 0.8 t_{wc}^2 \left[1 + 3 \frac{l_b}{d_c} \left(\frac{t_{wc}}{t_{fc}} \right)^{1.5} \right] \times \left(\frac{E_c F_{yc} t_{fc}}{t_{wc}} \right)^{0.5}$	= 239.5 [kips]	AISC 15 th Eq J10-4
Resistance factor-LRFD	$\phi = 1.00$ $\phi R_n =$ ratio = 0.83	= 239.5 [kips] > P_{uf_c}	AISC 341-16 E3.6e (1) OK

Seismic Column Panel Zone Shear	N/A
For OMF connection, there is no additional panel zone check requirements for amplified seismic load.	AISC 341-16 E1.6b
Panel zone shear strength should be checked in accordance with Section J10.6 of the Specification.	
The required shear strength of the panel zone should be based on the beam end moments computed from the load combinations stipulated by the applicable building code, not including the amplified seismic load.	

Seismic Stiffener Geometry Restriction	PASS		
Stiffener plate width	$b_s = 3.750$ [in]	depth $d_s = 10.920$ [in]	
Stiffener plate thickness	$t_s = 0.500$ [in]		
Column flange thickness	$t_{fc} = 0.640$ [in]	column depth $d_c = 12.200$ [in]	
Beam flange thickness	$t_{fb} = 0.525$ [in]		
Min Stiffener Plate Thickness			
Min stiffener plate thickness	$t_{smin} = \max (t_{fb} / 2 , b_s / 16)$	= 0.263 [in]	AISC 15 th J10.8 (2)
Stiffener plate thickness	$t_s =$	= 0.500 [in]	
		$\geq t_{smin}$	OK
Min Stiffener Plate Depth			
Min stiffener plate depth	$d_{smin} = (d_c - 2 t_{fc}) / 2$	= 5.460 [in]	AISC 15 th J10.8 (3)
Stiffener plate depth	$d_s =$	= 10.920 [in]	
		$\geq d_{smin}$	OK

Seismic Stiffener Yield at Column Flange	ratio = 103.8 / 123.8	= 0.84	PASS
Stiffener plate width	$b_s = 3.750$ [in]	thickness $t_s = 0.500$ [in]	
Stiffener plate corner clip	clip =	= 1.000 [in]	AISC 15 th Page 8-18
Stiffener plate yield strength	$F_y =$	= 50.0 [ksi]	
Stiffener plate cross sect area	$A_{st} = (b_s - clip) t_s$	= 1.375 [in ²]	
Trans stiffener strength required	$R_s =$	= 103.8 [kips]	
Trans stiffener strength provided	$R_n = F_y \times 2 \times A_{st}$	= 137.5 [kips]	AISC 15 th Eq J4-1
Bolt resistance factor-LRFD	$\phi = 0.90$		AISC 15 th Eq J4-1
	$\phi R_n =$	= 123.8 [kips]	
	ratio = 0.84	$> R_s$	OK

Seismic Stiffener Shear at Column Web	ratio = 103.8 / 267.6	= 0.39	PASS
Stiffener plate depth	$d_s = 10.920$ [in]	thickness $t_s = 0.500$ [in]	
Stiffener plate corner clip	clip =	= 1.000 [in]	AISC 15 th Page 8-18
Stiffener plate yield strength	$F_y =$	= 50.0 [ksi]	
Stiffener plate cross sect area	$A_{gv} = (d_s - 2 \times clip) t_s$	= 4.460 [in ²]	
Trans stiffener strength required	$R_s =$	= 103.8 [kips]	
Trans stiffener strength provided	$R_n = 2 \times 0.6 \times F_y \times A_{gv}$	= 267.6 [kips]	AISC 15 th Eq J4-3
Resistance factor-LRFD	$\phi = 1.00$		AISC 15 th Eq J4-3
	$\phi R_n =$	= 267.6 [kips]	
	ratio = 0.39	$> R_s$	OK

Seismic Stiffener Compression		ratio = 103.8 / 97.8	= 1.06	FAIL
Column section	$d = 12.200$ [in]	$t_f = 0.640$ [in]		
Stiffener plate depth	$d_s = d - 2 t_f$	$= 10.920$ [in]		
Stiffener plate width	$b_s = 3.750$ [in]	thickness $t_s = 0.500$ [in]		
Stiffener plate corner clip	clip =	$= 1.000$ [in]		AISC 15 th Page 8-18
Stiffener plate yield strength	$F_y = 50.0$ [ksi]	$E = 29000$ [ksi]		
Stiffener plate cross sect area	$A_{st} = (b_s - clip) t_s$	$= 1.375$ [in ²]		
Stiffener plate unbraced length	$L = d_s = d - 2 t_f$	$= 10.920$ [in]		
Plate radius of gyration	$r = t_s / \sqrt{12}$	$= 0.144$ [in]		
Stiffener plate slenderness	$KL/r = 0.75 \times L / r$	$= 56.74$		
Elastic buckling stress	$F_e = \frac{\pi^2 E}{(KL/r)^2}$	$= 88.90$ [ksi]		AISC 15 th Eq E3-4
	when $\frac{KL}{r} \leq 4.71 \left(\frac{E}{F_y} \right)^{0.5} = 113.43$			AISC 15 th E3
Critical stress	$F_{cr} = 0.658 \left(F_y / F_e \right) F_y$	$= 39.51$ [ksi]		AISC 15 th Eq E3-2
Trans stiffener strength required	$R_s =$	$= 103.8$ [kips]		
Trans stiffener strength provided	$R_n = 2 \times F_{cr} \times A_{st}$	$= 108.7$ [kips]		AISC 15 th Eq E3-1
Bolt resistance factor-LRFD	$\phi = 0.90$			AISC 15 th E1
	$\phi R_n =$	$= 97.8$ [kips]		
	ratio = 1.06	$< R_s$	NG	

Seismic Stiffener to Column Flange Fillet Weld Limitation		FAIL		
Min Fillet Weld Size				
Thinner part joined thickness	$t =$	$= 0.500$ [in]		
Min fillet weld size allowed	$w_{min} =$	$= 0.188$ [in]		AISC 15 th Table J2.4
Fillet weld size provided	$w =$	$= 0.313$ [in]		
		$\geq w_{min}$	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 15 th J2.2b
Min fillet weld length	$L = b_s - clip$	$= 2.750$ [in]		
		$\geq L_{min}$	OK	
Min Stiffener to Column Flange Fillet Weld Size				
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 15 th G1
Fillet weld shear strength	$\phi R_{n-w} =$	$= 1.392$ [kip/in]		AISC 15 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	$D_{min} = \phi_v F_y t_s / (\phi R_{n-w} \times 1.5 \times 2)$	$= 5.388$ [1/16 "]		
Fillet weld size provided	$D =$	$= 5.000$ [1/16 "]		
		$< D_{min}$	NG	

Seismic Stiffener to Column Web Fillet Weld Limitation		PASS	
Min Fillet Weld Size			
Thinner part joined thickness	$t =$	$= 0.370$ [in]	
Min fillet weld size allowed	$w_{min} =$	$= \mathbf{0.188}$ [in]	AISC 15 th Table J2.4
Fillet weld size provided	$w =$	$= \mathbf{0.250}$ [in]	
		$\geq w_{min}$	OK
Min Fillet Weld Length			
Fillet weld size provided	$w =$	$= 0.250$ [in]	
Min fillet weld length allowed	$L_{min} = 4 \times w$	$= \mathbf{1.000}$ [in]	AISC 15 th J2.2b
Min fillet weld length	$L = d_c - 2 \times t_{fc} - 2 \times \text{clip}$	$= \mathbf{8.920}$ [in]	
		$\geq L_{min}$	OK
Seismic Stiffener Weld Strength at Column Flange		ratio = 103.8 / 114.8	= 0.90 PASS
Stiffener to Column Flange Weld Length Calc			
Stiffener plate width & clip	$b_s = 3.750$ [in]	clip = 1.000 [in]	
Stiffener to column flange weld length - double fillet	$L = (b_s - \text{clip}) \times 2$ stiffener	$= \mathbf{5.500}$ [in]	
Trans stiffener strength required	$R_s =$	$= \mathbf{103.8}$ [kips]	
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 15 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 15 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 15 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 15 th Eq J4-2 is checked			AISC 15 th J2.4
Base metal tensile rupture	$R_{n-b} = F_u t$	$= 32.500$ [kip/in]	AISC 15 th Eq J4-2
For seismic nonductile limit state, weld strength use $\phi = 0.75$, base metal rupture use $\phi_n = 0.9$			AISC 358-16 2.4.1
Increase base metal rupture strength due to higher ϕ value when compare to weld strength			
Double fillet linear shear strength	$R_n = \min (R_{n-w}, R_{n-b} \times \frac{0.90}{0.75})$	$= \mathbf{27.838}$ [kip/in]	AISC 15 th Eq 9-2
Resistance factor-LRFD	$\phi = 0.75$		AISC 15 th Eq 8-1
	$\phi R_n =$	$= \mathbf{20.879}$ [kip/in]	
Shear resistance required	$R_s =$	$= \mathbf{103.8}$ [kips]	
Fillet weld length - double fillet	$L =$	$= 5.500$ [in]	
Shear resistance provided	$\phi F_n = \phi R_n \times L$	$= \mathbf{114.8}$ [kips]	
	ratio = 0.90	$> R_s$	OK

Seismic Stiffener Weld Strength at Column Web		ratio = 103.8 / 198.7	= 0.52	PASS
Stiffener to Column Web Weld Length Calc				
Column section	$d_c = 12.200$ [in]		$t_{fc} = 0.640$ [in]	
Stiffener plate corner clip	$clip = 1.000$ [in]			
Stiffener to column web weld length - double fillet	$L = (d_c - 2 \times t_{fc} - 2 \times clip) \times 2$ stiffener		= 17.840 [in]	
Trans stiffener strength required	$R_s =$		= 103.8 [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 15 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 15 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 15 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 15 th Eq J4-4 is checked				AISC 15 th J2.4
Base metal shear rupture	$R_{n-b} = 0.6 F_u t$		= 19.500 [kip/in]	AISC 15 th Eq J4-4
For seismic nonductile limit state, weld strength use $\phi = 0.75$, base metal rupture use $\phi_n = 0.9$ Increase base metal rupture strength due to higher ϕ value when compare to weld strength				
Double fillet linear shear strength	$R_n = \min (R_{n-w}, R_{n-b} \times \frac{0.90}{0.75})$		= 14.847 [kip/in]	AISC 15 th Eq 9-2
Resistance factor-LRFD	$\phi = 0.75$			AISC 15 th Eq 8-1
	$\phi R_n =$		= 11.135 [kip/in]	
Shear resistance required	$R_s =$		= 103.8 [kips]	
Fillet weld length - double fillet	$L =$		= 17.840 [in]	
Shear resistance provided	$\phi F_n = \phi R_n \times L$		= 198.7 [kips]	
	ratio = 0.52		> R_s	OK

