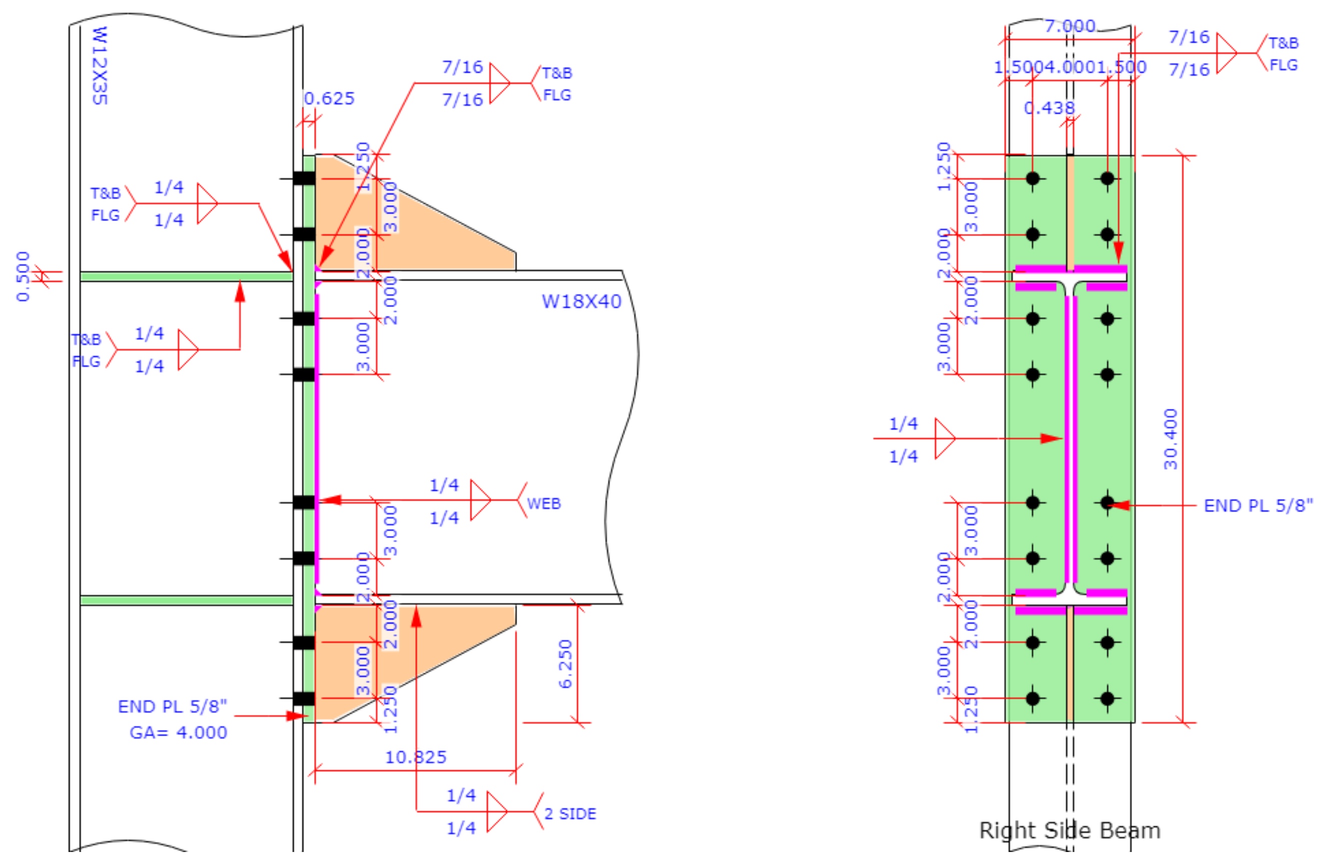
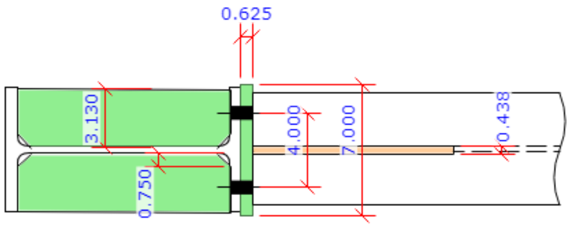
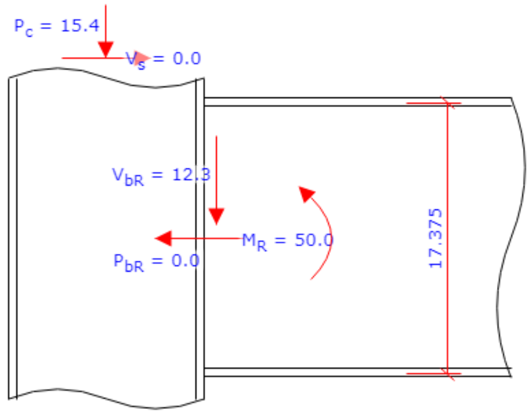
Result Summary - Overall	Moment Connection - Beam to Column	Code=AISC 360-16 LRFD

Result Summary - Overall	geometries & weld limitations = **PASS**	limit states max ratio = **1.37** **FAIL**
Right Beam to Column	geometries & weld limitations = **PASS**	limit states max ratio = **1.37** **FAIL**

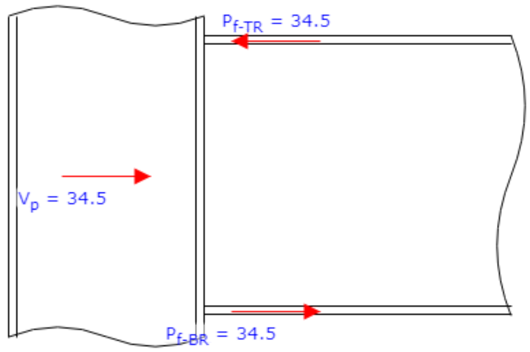
Sketch	Moment Connection - Beam to Column	Code=AISC 360-16 LRFD







Design Load



Beam Flange Force & Panel Zone Shear Vp

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	$P_{f\text{-}TR} = P_{bR} / 2 + M_R / d_m$		= **34.5**	[kips]
Beam flange force - bottom	$P_{f\text{-}BR} = P_{bR} / 2 - M_R / d_m$		= **-34.5**	[kips]

Panel Zone Shear Force Calc

Column story shear	V_s = from user input	= 0.0	[kips]
Panel zone shear force	$V_p = P_{f\text{-}TR} - P_{f\text{-}TL} - V_s$	= **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 W12X35	Z_{cx} = 51.20 [in^3]		F_{cy} = 50.0	[ksi]
	R_{cy} = 1.1			
Column expected flexural strength	$M_{ce} = 1.1\ R_{cy} F_{cy} Z_{cx}$		= 258.13	[kip-ft]

Flexural Strength by Panel Zone Shear

Depth of beam	$d_b = d_b$	= 17.900	[in]	
Column sect W12X35	d_c = 12.500 [in]	b_{cf} = 6.560	[in]	
	t_{cw} = 0.300 [in]	t_{cf} = 0.520	[in]	
	F_{cy} = 50.0 [ksi]	R_{cy} = 1.1		
Column sect W12X35	A_c = 10.300 [in^2]	F_{cy} = 50.0	[ksi]	
Column axial yield strength	$P_y = F_{cy} A_c$	= 515.0	[kips]	AISC 15th J10.6 (b)
LRFD-ASD force adjustment factor	α = for LRFD	= 1.0		AISC 15th J10.6 (b)
Column axial compression	P_r = from user input	= 15.4	[kips]	
	when $\alpha P_r \le 0.75\ P_y$, use Eq J10-11			AISC 15th Eq J10-11
Column panel zone capacity	$V_{pz} = 0.6(1.1) R_{cy} F_{cy} d_c t_{cw} \left(1 + \frac{3\ b_{cf} t_{cf}^2}{d_b d_c t_{cw}}\right)$	= 146.9	[kips]	AISC 15th Eq J10-11
Column panel zone capacity-LRFD	$V_{ue} = V_{pz} / \alpha_s = 1.0$	= 146.9	[kips]	
Beam sect W18X40	d_b = 17.900 [in]	t_{bf} = 0.525	[in]	
Flexural strength by panel zone shear	$M_{ue} = V_{ue} (d_b - t_{bf})$	= 212.72	[kip-ft]	
Min expected flexural strength	$M_{ne} = \min(M_{be} , M_{ce} , M_{ue})$	= **212.72**	[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	$V_{uc} = \dfrac{M_{ne}}{h_b}$	= 12.5	[kips]	
Flexural strength after considering story shear	$M_u = (V_{ue} + V_{uc}) (d_b - t_{bf})$	= **230.84**	[kip-ft]	

Calculate Shear Load

Beam clear span	L_{cf} = from user input	= 347	[in]	
Shear from max expected flexural strength	$V_{ne} = 2\ M_u / L_{cf}$	= 15.9	[kips]	
Shear from load combination including amplified seismic load	V = from user input	= 23.5	[kips]	
Max shear used in design	$V_u = \max(V_{ne} , V)$	= **23.5**	[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	$F_{fu} = M_u / d_m$	= **159.4**	[kips]	

Right Beam to Column MC Connection Code=AISC 360-16 LRFD

Result Summary geometries & weld limitations = **PASS** limit states max ratio = **1.37** **FAIL**

Geometry Restriction Checks **PASS**

Min Bolt Edge Distance - Column Flange

Bolt diameter	d_b =	= 0.750	[in]	
Min edge distance allowed	$L_{e\text{-}min}$ =	= **1.000**	[in]	AISC 15th Table J3.4
Min edge distance in Column Flange	L_e =	= **1.280**	[in]	
		$\geq L_{e\text{-}min}$	OK	

Min Bolt Spacing - End Plate

Bolt diameter	d_b =	= 0.750	[in]	
Min bolt spacing allowed	$L_{s\text{-}min} = 2.667\ d_b$	= **2.000**	[in]	AISC 15th J3.3
Min Bolt spacing in End Plate	L_s =	= **3.000**	[in]	
		$\geq L_{s\text{-}min}$	OK	

Min Bolt Edge Distance - End Plate

Bolt diameter	d_b =	= 0.750	[in]	
Min edge distance allowed	$L_{e\text{-}min}$ =	= **1.000**	[in]	AISC 15th Table J3.4
Min edge distance in End Plate	L_e =	= **1.250**	[in]	
		$\geq L_{e\text{-}min}$	OK	

Max Bolt Edge Distance - End Plate

Connecting plate thickness	t_p =	= 0.625	[in]	
Max edge distance allowed	$L_{e\text{-}max}$ = min (12t , 6")	= **6.000**	[in]	AISC 15th J3.5
Max edge distance in End Plate	L_e =	= **1.500**	[in]	
		$\leq L_{e\text{-}max}$	OK	

Beam Flange Fillet Weld Limitation — PASS

Min Fillet Weld Size

Thinner part joined thickness	$t =$	= 0.525	[in]	
Min fillet weld size allowed	$w_{min} =$	= **0.250**	[in]	AISC 15th Table J2.4
Fillet weld size provided	$w =$	= **0.438**	[in]	
		$\geq w_{min}$	OK	

Min Fillet Weld Length

Fillet weld size provided	$w =$	= 0.438	[in]	
Min fillet weld length allowed	$L_{min} = 4 \times w$	= **1.750**	[in]	AISC 15th J2.2b
Min fillet weld length	$L = 0.5\ b_{fb} - k_{1b}$	= **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	$w_{min} =$	= **0.188**	[in]	AISC 15th Table J2.4
Fillet weld size provided	$w =$	= **0.250**	[in]	
		$\geq w_{min}$	OK	

Min Fillet Weld Length

Fillet weld size provided	$w =$	= 0.250	[in]	
Min fillet weld length allowed	$L_{min} = 4 \times w$	= **1.000**	[in]	AISC 15th J2.2b
Min fillet weld length	$L = 0.5\ d_b - k_b$	= **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 15th G1
Fillet weld shear strength	$\phi\ R_{n\text{-}w} =$	= 1.392	[kip/in]	AISC 15th Eq 8-2a

Fillet weld strength $\phi\ R_{n\text{-}w} \times 1.5 \times 2$ 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	$D_{min} = \phi_v F_{yb} t_{wb} / (\phi\ R_{n\text{-}w} \times 1.5 \times 2)$	= **3.394**	[1/16 "]	
Fillet weld size provided	$D =$	= **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 15th Table J3.2
	bolt dia $d_b = 0.750$	[in]	bolt area $A_b = 0.442$	[in^2]	
Bolt norminal tensile strength	$P_t = F_t A_b$		= 39.8	[kips]	AISC 15th Eq J3-1

Tension bolt moment arm	$h_1 = 22.638$	[in]	$h_2 = 19.638$	[in]	
	$h_3 = 15.113$	[in]	$h_4 = 12.113$	[in]	
Bolt moment strength (no prying)	$M_{nb} = 2\ P_t (h_1 + h_2 + h_3 + h_4)$		= 460.56	[kip-ft]	AISC DG4 Table 3.3
Bolt resistance factor-LRFD	$\phi = 0.75$				AISC 15th Eq J3-1
	$\phi\ M_{nb} =$		= **345.42**	[kip-ft]	

End Plate Bending Strength

End plate width	b_{plate} = 7.000 [in]	thickness t_p = 0.625 [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} = 36.0 [ksi]			

See AISC DG4 Table 3.3 for all formulas to derive the following parameters — AISC DG4 Table 3.3

Tension bolt moment arm	h_1 = 22.638 [in]	h_2 = 19.638 [in]		
	h_3 = 15.113 [in]	h_4 = 12.113 [in]		
	g = 4.000 [in]	d_e = 1.250 [in]		
	p_{fi} = 2.000 [in]	p_{fo} = 2.000 [in]		
	p_b = 3.000 [in]			
	s = 2.646 [in]	Y_p = 231.83 [in]		
Flexure resistance factor-LRFD	ϕ_b = 0.90			AISC 15th F1 (1)
End plate bending strength	$\phi_b M_{pl} = \phi_b F_{yp} t_p^2 \ Y_p$		= **244.50** [kip-ft]	AISC DG4 Table 3.1

Max Moment in Demand

Moment by bolt strength-no prying	$\phi_b M_{nb}$ = from above calc	= 345.42 [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)	= **50.00** [kip-ft]	

Check Thick End Plate Condition

Check thick end plate condition	$\phi_b M_{pl}$ >= 1.11 X $\phi\ M_{np}$	AISC DG4 Eq 3.33
	ratio = **0.23** thick plate	

Column Flange Bending Strength

See AISC DG4 Table 3.5 for all formulas to derive the following parameters — AISC DG4 Table 3.5

Tension bolt moment arm	h_1 = 22.638 [in]	h_2 = 19.638 [in]		
	h_3 = 15.113 [in]	h_4 = 12.113 [in]		
*** Stiffened Column Flange Case ***				
Column section	b_{fc} = 6.560 [in]	t_{fc} = 0.520 [in]		
	F_{yc} = 50.0 [ksi]	bolt gage g = 4.000 [in]		
	s = 2.561 [in]	c = 4.525 [in]		
Stiffener plate thickness	t_s = 0.500 [in]			
	p_{si} = 2.013 [in]	p_{so} = 2.013 [in]		
	p_b = 3.000 [in]	d_e = 1.250 [in]		
	Y_c = 239.0 [in]			
Flexure resistance factor-LRFD	ϕ_b = 0.90			AISC 15th F1 (1)
Column flange bending strength	$\phi_b M_{cf} = \phi_b F_{yc} t_{fc}^2 \ Y_c$		= **242.33** [kip-ft]	AISC DG4 Table 3.5

Check Thick Column Flange Condition

Check thick column flange condition	$\phi_b M_{cf}$ >= 1.11 X $\phi\ M_{np}$	AISC DG4 Eq 3.35
	ratio = **0.23** 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 / 238.6	= **0.14**	**PASS**
	bolt grade = A325-N	F_t = 90.0	[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.750 [in]	bolt area A_b = 0.442	[in^2]	
Bolt norminal tensile strength	$P_t = F_t A_b$	= 39.8	[kips]	AISC 15th Eq J3-1
Tension bolt moment arm	h_1 = 22.638 [in]	h_2 = 19.638	[in]	
	h_3 = 15.113 [in]	h_4 = 12.113	[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	$F_n = 2 P_t (h_1 + h_2 + h_3 + h_4) / d_m$	= 318.1	[kips]	AISC DG4 Eq 3.8
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J3-1
	ϕF_n =	= **238.6**	[kips]	AISC DG4 Eq 3.7
	ratio = **0.14**	> P_{uf_t}	OK	

Bolt Shear Strength		ratio = 12.3 / 143.1	= **0.09**	**PASS**
Bolt shear stress	bolt grade = A325-N	F_{nv} = 54.0	[ksi]	AISC 15th 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 = 8.0	shear plane m = 1		
Bolt group eccentricity coefficient	C_{ec} =	= 1.000		
Required shear strength	V_u =	= **12.3**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 190.9	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J3-1
	ϕR_n =	= **143.1**	[kips]	
	ratio = **0.09**	> V_u	OK	

Bolt Bearing/TearOut Strength on End Plate ratio = 12.3 / 143.1 = **0.09** **PASS**

Single Bolt Shear Strength

Bolt shear stress	bolt grade = A325-N		F_{nv} = 54.0	[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.750	[in]	bolt area A_b = 0.442	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$		= 23.9	[kips]	AISC 15th 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 15th Table J3.3
Bolt spacing & edge distance	spacing L_s = 3.000	[in]	edge distance L_e = 1.250	[in]	
Plate tensile strength	F_u = 58.0	[ksi]			
Plate thickness	t = 0.625	[in]			

Interior Bolt

Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.188	[in]	
Bolt tear out/bearing strength	$R_{n\text{-}t\&b\text{-}in} = 1.2\ L_c\ t\ F_u \le 2.4\ d_b\ t\ F_u$			AISC 15th Eq J3-6a
	= 95.2 ≤ 65.3	= 65.3	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$	= 23.9	[kips]	

Edge Bolt

Bolt hole edge clear distance	$L_c = L_e - d_h / 2$	= 0.844	[in]	
Bolt tear out/bearing strength	$R_{n\text{-}t\&b\text{-}ed} = 1.2\ L_c\ t\ F_u \le 2.4\ d_b\ t\ F_u$			AISC 15th Eq J3-6a
	= 36.7 ≤ 65.3	= 36.7	[kips]	
Bolt strength at edge	$R_{n\text{-}ed} = \min(R_{n\text{-}t\&b\text{-}ed}, R_{n\text{-}bolt})$	= 23.9	[kips]	

Number of bolt	interior n_{in} = 6	edge n_{ed} = 2		
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-}in} + n_{ed} R_{n\text{-}ed}$	= 190.9	[kips]	
Required shear strength	V_u =	= **12.3**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th J3-10
	ϕR_n =	= **143.1**	[kips]	
	ratio = **0.09**	> V_u	OK	

Bolt Bearing/TearOut Strength on Column Flange			ratio = 12.3 / 143.1	= **0.09**	**PASS**
Single Bolt Shear Strength					
Bolt shear stress	bolt grade = A325-N		F_{nv} = 54.0	[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.750	[in]	bolt area A_b = 0.442	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$		= 23.9	[kips]	AISC 15th 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 15th Table J3.3
Bolt spacing	spacing L_s = 3.000	[in]			
Plate tensile strength	F_u = 65.0	[ksi]			
Plate thickness	t = 0.520	[in]			
Interior Bolt					
Bolt hole edge clear distance	$L_c = L_s - d_h$		= 2.188	[in]	
Bolt tear out/bearing strength	$R_{n\text{-}t\&b\text{-}in} = 1.2\, L_c\, t\, F_u \le 2.4\, d_b\, t\, m\, F_u$				AISC 15th Eq J3-6a
	= 88.7 ≤ 60.8		= 60.8	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$		= 23.9	[kips]	
Number of bolt	interior n_{in} = 8				
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-}in}$		= 190.9	[kips]	
Required shear strength	V_u =		= **12.3**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 15th J3-10
	ϕR_n =		= **143.1**	[kips]	
	ratio = **0.09**		> V_u	OK	

End Plate Flexural Yielding			ratio = 34.5 / 168.9	= **0.20**	**PASS**
End Plate Bending Strength					
End plate width	b_{plate} = 7.000	[in]	thickness t_p = 0.625	[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} = 36.0	[ksi]			
See AISC DG4 Table 3.3 for all formulas to derive the following parameters					AISC DG4 Table 3.3
Tension bolt moment arm	h_1 = 22.638	[in]	h_2 = 19.638	[in]	
	h_3 = 15.113	[in]	h_4 = 12.113	[in]	
	g = 4.000	[in]	d_e = 1.250	[in]	
	p_{fi} = 2.000	[in]	p_{fo} = 2.000	[in]	
	p_b = 3.000	[in]			
	s = 2.646	[in]	Y_p = 231.83	[in]	
Flexure resistance factor-LRFD	ϕ_b = 0.90				AISC 15th F1 (1)
End plate bending strength	$\phi_b M_{pl} = \phi_b F_{yp} t_p^2\, Y_p$		= **244.50**	[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	$\phi R_{pl} = \phi M_{pl} / d_m$		= **168.9**	[kips]	AISC DG4 Eq 3.10
	ratio = **0.20**		> P_{uf_t}	OK	

End Plate Stiffener Geometry Limitations				**PASS**
Beam web thick	t_{wb} = 0.315 [in]	Stiff thick t_s = 0.438	[in]	
Beam yield strength	F_{yb} = 50.0 [ksi]	Stiff yield F_{ys} = 36.0	[ksi]	
Min Stiffener Plate Thickness				
Min stiffener plate thickness	$t_{smin} = t_{wb} F_{yb} / F_{ys}$	= **0.438**	[in]	AISC DG4 Eq 3.15
Stiffener plate thickness	t_s =	= **0.438**	[in]	
		≥ t_{smin}	OK	
Min Stiff Thick to Avoid Local Buckiling				
Stiffener plate height	h_{st} = 6.250 [in]	E = 29000	[ksi]	
Stiffener plate strength	F_{ys} = 36.0 [ksi]			
Min stiffener plate thickness	$t_{smin} = 1.79\, h_{st} \sqrt{F_{ys} / E}$	= **0.394**	[in]	AISC DG4 Eq 3.16
Stiffener plate thickness	t_s =	= **0.438**	[in]	
		≥ t_{smin}	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 = $^7/_{16}$ [in]	load angle θ = 90.0	[°]	
Electrode strength	F_{EXX} = 70.0 [ksi]	strength coeff C_1 = 1.00		AISC 15th 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 15th Page 8-9
Fillet weld shear strength	$R_{n-w} = 0.6\, (C_1 \times 70\text{ ksi})\, 0.707\, w\, n\, C_2$	= 38.973	[kip/in]	AISC 15th 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 15th Eq J4-2 is checked				AISC 15th J2.4
Base metal tensile rupture	$R_{n-b} = F_u\, t$	= 34.125	[kip/in]	AISC 15th Eq J4-2
Double fillet linear shear strength	$R_n = \min (R_{n-w}, R_{n-b})$	= **34.125**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq 8-1
	ϕR_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 = 12.3 / 71.5 = **0.17** **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} = 0.750	[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	$L_2 = d_b - 2t_{fb} - p_{fi} - 2\ d_{bolt}$		= 13.350	[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 θ = 0.0	[°]	
Electrode strength	F_{EXX} = 70.0	[ksi]	strength coeff C_1 = 1.00		AISC 15th 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 15th Page 8-9
Fillet weld shear strength	$R_{n\text{-}w} = 0.6\ (C_1 \times 70\ \text{ksi})\ 0.707\ w\ n\ C_2$		= 14.847	[kip/in]	AISC 15th 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, shear rupture as per AISC 15th Eq J4-4 is checked					AISC 15th J2.4
Base metal shear rupture	$R_{n\text{-}b} = 0.6\ F_u\ t$		= 12.285	[kip/in]	AISC 15th Eq J4-4
Double fillet linear shear strength	$R_n = \min(R_{n\text{-}w}, R_{n\text{-}b})$		= **12.285**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq 8-1
	ϕR_n =		= **9.214**	[kip/in]	
Shear resistance required	V_u =		= **12.3**	[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 = **0.17**		> V_u	OK	

Column Flexural Yielding			ratio = 34.5 / 167.4	= **0.21**	**PASS**

Column Flange Bending Strength

See AISC DG4 Table 3.5 for all formulas to derive the following parameters				AISC DG4 Table 3.5
Tension bolt moment arm	h_1 = 22.638 [in]		h_2 = 19.638 [in]	
	h_3 = 15.113 [in]		h_4 = 12.113 [in]	
*** Stiffened Column Flange Case ***				
Column section	b_{fc} = 6.560 [in]		t_{fc} = 0.520 [in]	
	F_{yc} = 50.0 [ksi]		bolt gage g = 4.000 [in]	
	s = 2.561 [in]		c = 4.525 [in]	
Stiffener plate thickness	t_s = 0.500 [in]			
	p_{si} = 2.013 [in]		p_{so} = 2.013 [in]	
	p_b = 3.000 [in]		d_e = 1.250 [in]	
	Y_c = 239.0 [in]			
Flexure resistance factor-LRFD	ϕ_b = 0.90			AISC 15th F1 (1)
Column flange bending strength	$\phi_b M_{cf} = \phi_b F_{yc} t_{fc}^2\ Y_c$		= **242.33** [kip-ft]	AISC DG4 Table 3.5
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	$\phi\ R_{cf} = \phi\ M_{cf} / d_m$		= **167.4** [kips]	AISC DG4 Eq 3.21
	ratio = **0.21**		> P_{uf_t} OK	

Column Web Yielding			ratio = 34.5 / 113.6	= **0.30**	**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 = 12.500 [in]		t_{fc} = 0.520 [in]	
	t_{wc} = 0.300 [in]		k_c = 0.820 [in]	
Column yield strength	F_{yc} = 50.0 [ksi]			
Distance from to top of column to top of beam flange	d_{end} = 13.000 [in]			
Top column reduction factor	C_t = 1.0			AISC DG4 Eq 3.24
Beam flange fillet weld size	w = 0.438 [in]		beam flange t_{fb} = 0.525 [in]	
Length of bearing	$N = t_{fb} + 2\ w$		= 1.400 [in]	AISC DG4 Eq 3.24
End plate thickness	t_p = 0.625 [in]			
Column web yielding strength	$R_n = C_t (6\ k_c + N + 2\ t_p) F_{yc} t_{wc}$		= 113.6 [kips]	AISC DG4 Eq 3.24
Resistance factor-LRFD	ϕ = 1.00			AISC 15th J10.2
	$\phi\ R_n$ =		= **113.6** [kips]	
	ratio = **0.30**		> P_{uf} OK	

Column Web Buckling		ratio = 34.5 / 64.7	= **0.53**	**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 = 12.500 [in]	t_{fc} = 0.520	[in]	
	t_{wc} = 0.300 [in]	k_c = 0.820	[in]	
	$h = d_c - 2\,k_c$	= 10.860	[in]	
Column yield strength	F_{yc} = 50.0 [ksi]	E_c = 29000	[ksi]	
Distance from top of beam flange to top of column	$d_{end\text{-}flg}$ = 13.000 [in]	beam flange t_{fb} = 0.525	[in]	
Distance from center of flange force to top of column	$d_{end\text{-}F} = d_{end\text{-}flg} + 0.5\,t_{fb}$	= 13.263	[in]	
	$d_{end\text{-}F} \geq d_c/2$, R_n has no reduction			AISC 15th J10.5
Top column reduction factor	C_t = 1.0			AISC 15th J10.5
Column web buckling strength	$R_n = \dfrac{C_t\, 24\, t_{wc}^3 \sqrt{E_c F_{yc}}}{h}$	= 71.9	[kips]	AISC 15th Eq J10-8
Resistance factor-LRFD	ϕ = 0.90			AISC 15th J10.5
	ϕR_n =	= **64.7**	[kips]	
	ratio = **0.53**	> P_{uf_c}	OK	

Column Web Crippling		ratio = 34.5 / 109.5	= **0.32**	**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 = 12.500 [in]	t_{fc} = 0.520	[in]	
	t_{wc} = 0.300 [in]	k_c = 0.820	[in]	
Column yield strength	F_{yc} = 50.0 [ksi]	E_c = 29000	[ksi]	
Beam flange fillet weld size	w = 0.438 [in]	beam flange t_{fb} = 0.525	[in]	
End plate thickness	t_p = 0.625 [in]			
Length of bearing	$l_b = t_{fb} + 2\,w + 2\,t_p$	= 2.650	[in]	
Distance from top of column to top of beam flange	$d_{end\text{-}flg}$ =	= 13.000	[in]	
Distance from top of column to center of flange force	$d_{end\text{-}F} = d_{end\text{-}flg} + 0.5\,t_{fb}$	= 13.263	[in]	
	$d_{end\text{-}F} \geq d_c/2$, use Eq J10-4			AISC 15th Eq J10-4
Column web crippling strength	$R_n = 0.8\, t_{wc}^2 \left[1+3\dfrac{l_b}{d_c}\left(\dfrac{t_{wc}}{t_{fc}}\right)^{1.5}\right] \times \left(\dfrac{E_c F_{yc} t_{fc}}{t_{wc}}\right)^{0.5}$	= 146.0	[kips]	AISC 15th Eq J10-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th J10.3
	ϕR_n =	= **109.5**	[kips]	
	ratio = **0.32**	> P_{uf_c}	OK	

Column Panel Zone Shear		ratio = 34.5 / 109.3	= **0.32**	**PASS**
Panel zone shear force	$V_p = P_{f\text{-}TR} - P_{f\text{-}TL} - V_s$	= **34.5**	[kips]	
Column W12X35	d_c = 12.500 [in]	b_{cf} = 6.560	[in]	
	t_{cf} = 0.520 [in]	t_{cw} = 0.300	[in]	
	A_c = 10.300 [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	$P_y = F_{cy} A_c$	= 515.0	[kips]	AISC 15th J10.6 (b)
LRFD-ASD force adjustment factor	α = for LRFD	= 1.0		AISC 341-16 D1.2a (b)
	when $\alpha P_r \le 0.75\, P_y$, use Eq J10-11			AISC 15th Eq J10-11
Column web panel zone capacity	$R_n = 0.6\, F_{cy}\, d_c\, t_{cw}\, (1 + \frac{3\, b_{cf}\, t_{cf}^2}{d_m\, d_c\, t_{cw}})$	= 121.4	[kips]	AISC 15th Eq J10-11
Resistance factor-LRFD	ϕ = 0.90			AISC 15th J10.6
	$\phi\, R_n$ =	= **109.3**	[kips]	
	ratio = **0.32**	> V_p	OK	

Seismic Material & Geometry Limitations — PASS

Check Max Beam Yield Stress

Condition : beam max material $F_y \le F_{y\_max}$ = 55 ksi — AISC 341-16 A3.1

Beam yield strength — F_{yb} = 50.0 [ksi]

$\le F_{y\_max}$ OK

Check Max Column Yield Stress

Condition : column max material $F_y \le F_{y\_max}$ = 55 ksi — AISC 341-16 A3.1

Column yield strength — F_{yc} = 50.0 [ksi]

$\le F_{y\_max}$ 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 = 4.000 [in]

$\le b_f$ OK

Check Min Pitch

Condition : min bolt pitch distance $p \ge d_b$ +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 = 0.750	[in]		
Min bolt pitch distance allowed	$p_{min} = d_b$ +0.50 in		= **1.250**	[in]
Min bolt pitch distance	$p = \min(p_{fi}, p_{fo})$		= **2.000**	[in]
			$\ge p_{min}$	OK

Check Min Inner Bolt Pitch P_b

Condition : min inner bolt pitch $p_b \ge$ 8/3 x bolt dia — AISC 358-16 6.9.2

Bolt dia	d_b = 0.750	[in]		
Min inner bolt pitch allowed	p_{bmin} = 8/3 x d_b		= **2.000**	[in]
Inner bolt pitch	p_b =		= **3.000**	[in]
			$\ge p_{bmin}$	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 $b_p \ge$ beam flange width b_f

Beam flange width	b_f =	= 6.020	[in]
End plate width	b_p =	= 7.000	[in]
		$\ge b_f$	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. AISC 341-16 E3.4a
It's OMF connection, so the column beam moment ratio check is not required.

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 358-16 6.10
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 15th Table J3.2
	bolt dia d_b = 0.750 [in]	bolt area A_b = 0.442	[in^2]	
Bolt norminal tensile strength	$P_t = F_t A_b$	= 39.8	[kips]	AISC 15th Eq J3-1
Tension bolt moment arm	h_1 = 22.638 [in]	h_2 = 19.638	[in]	
	h_3 = 15.113 [in]	h_4 = 12.113	[in]	
Bolt moment strength (no prying)	$M_{nb} = 2\ P_t (\ h_1 + h_2 + h_3 + h_4\ )$	= 460.56	[kip-ft]	AISC DG4 Table 3.3
Bolt resistance factor-LRFD	ϕ_n = 0.90			AISC 358-16 2.4.1
	$\phi_n M_{nb}$ =	= **414.51**	[kip-ft]	

End Plate Bending Strength

End plate width	b_{plate} = 7.000 [in]	thickness t_p = 0.625	[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} = 36.0 [ksi]			

See AISC DG4 Table 3.3 for all formulas to derive the following parameters AISC DG4 Table 3.3

Tension bolt moment arm	h_1 = 22.638 [in]	h_2 = 19.638	[in]	
	h_3 = 15.113 [in]	h_4 = 12.113	[in]	
	g = 4.000 [in]	d_e = 1.250	[in]	
	p_{fi} = 2.000 [in]	p_{fo} = 2.000	[in]	
	p_b = 3.000 [in]			
	s = 2.646 [in]	Y_p = 231.83	[in]	
Flexure resistance factor-LRFD	ϕ_d = 1.00			AISC 358-16 2.4.1
End plate bending strength	$\phi_d M_{pl} = \phi_d F_{yp} t_p^2\ Y_p$	= **271.67**	[kip-ft]	AISC DG4 Table 3.1

Max Moment in Demand

Moment by bolt strength-no prying	$\phi_d M_{nb}$ = from above calc	= 414.51	[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 = **0.20** thick plate

Column Flange Bending Strength

See AISC DG4 Table 3.5 for all formulas to derive the following parameters — AISC DG4 Table 3.5

Tension bolt moment arm	h_1 = 22.638	[in]	h_2 = 19.638	[in]	
	h_3 = 15.113	[in]	h_4 = 12.113	[in]	

*** Stiffened Column Flange Case ***

Column section	b_{fc} = 6.560	[in]	t_{fc} = 0.520	[in]	
	F_{yc} = 50.0	[ksi]	bolt gage g = 4.000	[in]	
	s = 2.561	[in]	c = 4.525	[in]	
Stiffener plate thickness	t_s = 0.500	[in]			
	p_{si} = 2.013	[in]	p_{so} = 2.013	[in]	
	p_b = 3.000	[in]	d_e = 1.250	[in]	
	Y_c = 239.0	[in]			
Flexure resistance factor-LRFD	ϕ_d = 1.00				AISC 358-16 2.4.1
Column flange bending strength	$\phi_d M_{cf} = \phi_d F_{yc} t_{fc}^2 Y_c$		= **269.25**	[kip-ft]	AISC DG4 Table 3.5

Check Thick Column Flange Condition

Check thick column flange condition	$\phi_d M_{cf}$ >= 1.11 X ϕ M_{np}		AISC DG4 Eq 3.35
	ratio = **0.21**	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

Seismic Bolt Moment Strength (No Prying)		ratio = 159.4 / 286.3	= **0.56**	**PASS**	
	bolt grade = A325-N		F_t = 90.0	[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.750	[in]	bolt area A_b = 0.442	[in^2]	
Bolt norminal tensile strength	$P_t = F_t A_b$		= 39.8	[kips]	AISC 15th Eq J3-1
Tension bolt moment arm	h_1 = 22.638	[in]	h_2 = 19.638	[in]	
	h_3 = 15.113	[in]	h_4 = 12.113	[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		= **159.4**	[kips]	
Flange force resistance by bolt	$F_n = 2 P_t (h_1 + h_2 + h_3 + h_4) / d_m$		= 318.1	[kips]	AISC DG4 Eq 3.8
Bolt resistance factor-LRFD	ϕ_n = 0.90				AISC 358-16 2.4.1
	$\phi_n F_n$ =		= **286.3**	[kips]	AISC DG4 Eq 3.7
	ratio = **0.56**		> P_{uf_t}	OK	

Seismic Bolt Shear Strength		ratio = 23.5 / 171.8	= **0.14**	**PASS**
Bolt shear stress	bolt grade = A325-N	F_{nv} = 54.0	[ksi]	AISC 15th 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 = 8.0	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	= **23.5**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 190.9	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	ϕ_n = 0.90			AISC 358-16 2.4.1
	$\phi_n R_n$ =	= **171.8**	[kips]	
	ratio = **0.14**	> V_u	OK	

Seismic Bolt Bearing/TearOut Strength on End Plate		ratio = 23.5 / 171.8	= **0.14**	**PASS**
Single Bolt Shear Strength				
Bolt shear stress	bolt grade = A325-N	F_{nv} = 54.0	[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.750 [in]	bolt area A_b = 0.442	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$	= 23.9	[kips]	AISC 15th 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 15th Table J3.3
Bolt spacing & edge distance	spacing L_s = 3.000 [in]	edge distance L_e = 1.250	[in]	
Plate tensile strength	F_u = 58.0 [ksi]			
Plate thickness	t = 0.625 [in]			
Interior Bolt				
Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.188	[in]	
Bolt tear out/bearing strength	$R_{n\text{-}t\&b\text{-}in} = 1.2 L_c t F_u \le 2.4 d_b t F_u$			AISC 15th Eq J3-6a
	= 95.2 ≤ 65.3	= 65.3	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$	= 23.9	[kips]	
Edge Bolt				
Bolt hole edge clear distance	$L_c = L_e - d_h / 2$	= 0.844	[in]	
Bolt tear out/bearing strength	$R_{n\text{-}t\&b\text{-}ed} = 1.2 L_c t F_u \le 2.4 d_b t F_u$			AISC 15th Eq J3-6a
	= 36.7 ≤ 65.3	= 36.7	[kips]	
Bolt strength at edge	$R_{n\text{-}ed} = \min(R_{n\text{-}t\&b\text{-}ed}, R_{n\text{-}bolt})$	= 23.9	[kips]	
Number of bolt	interior n_{in} = 6	edge n_{ed} = 2		
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-}in} + n_{ed} R_{n\text{-}ed}$	= 190.9	[kips]	
Required shear strength	V_u = see Seismic Moment and Beam Flange Force Calc	= **23.5**	[kips]	
Bolt resistance factor-LRFD	ϕ_n = 0.90			AISC 358-16 2.4.1
	$\phi_n R_n$ =	= **171.8**	[kips]	
	ratio = **0.14**	> V_u	OK	

Seismic Bolt Bearing/TearOut Strength on Column Flange — ratio = 23.5 / 171.8 = **0.14** PASS

Single Bolt Shear Strength

Bolt shear stress	bolt grade = A325-N		F_{nv} = 54.0	[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.750	[in]	bolt area A_b = 0.442	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$		= 23.9	[kips]	AISC 15th 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 15th Table J3.3
Bolt spacing	spacing L_s = 3.000	[in]			
Plate tensile strength	F_u = 65.0	[ksi]			
Plate thickness	t = 0.520	[in]			

Interior Bolt

Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.188	[in]	
Bolt tear out/bearing strength	$R_{n\text{-}t\&b\text{-}in} = 1.2\, L_c\, t\, F_u \le 2.4\, d_b\, t\, m\, F_u$			AISC 15th Eq J3-6a
	= 88.7 ≤ 60.8	= 60.8	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$	= 23.9	[kips]	

Number of bolt	interior n_{in} = 8			
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-}in}$	= 190.9	[kips]	
Required shear strength	V_u = see Seismic Moment and Beam Flange Force Calc	= **23.5**	[kips]	
Bolt resistance factor-LRFD	ϕ_n = 0.90			AISC 358-16 2.4.1
	$\phi_n R_n$ =	= **171.8**	[kips]	
	ratio = **0.14**	> V_u	OK	

Seismic End Plate Shear — ratio = 79.7 / 94.5 = **0.84** PASS

Seismic flange force	$0.5F_{fu}$ = see Seismic Moment and Beam Flange Force Calc		= **79.7**	[kips]	
End plate	width b_p = 7.000	[in]	thickness t_p = 0.625	[in]	
	F_{yp} = 36.0	[ksi]	F_{up} = 58.0	[ksi]	

Check End Plate Shear Yielding — AISC 358-16 Eq 6.10-7

Plate shear yield strength	$R_{ny} = 0.6\, F_{yp}\, b_p\, t_p$	= 94.5	[kips]	AISC 358-16 Eq 6.10-7
Resistance factor-LRFD	ϕ_d = 1.00			AISC 358-16 2.4.1
	$\phi_d R_{ny}$ =	= **94.5**	[kips]	
	ratio = **0.84**	> $0.5F_{fu}$	OK	

Check End Plate Shear Rupture — AISC 358-16 Eq 6.10-8

Bolt hole diameter	bolt dia d_b = $^3/_4$	[in]	bolt hole dia d_h = $^7/_8$	[in]	AISC 15th B4.3b
Number of bolt	n = 2				
Plate net area in shear	$A_{nv} = (b_p - n\, d_h)\, t_p$		= 3.281	[in^2]	
Plate shear rupture strength	$R_{nr} = 0.6\, F_{up} A_{nv}$		= 114.2	[kips]	AISC 358-16 Eq 6.10-8
Resistance factor-LRFD	ϕ_n = 0.90				AISC 358-16 2.4.1
	$\phi_n R_{nr}$ =		= **102.8**	[kips]	
	ratio = **0.78**		> $0.5F_{fu}$	OK	

Seismic End Plate Flexural Yielding		ratio = 159.4 / 187.6	= **0.85**	**PASS**

End Plate Bending Strength

End plate width	b_{plate} = 7.000 [in]	thickness t_p = 0.625	[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} = 36.0 [ksi]			

See AISC DG4 Table 3.3 for all formulas to derive the following parameters — AISC DG4 Table 3.3

Tension bolt moment arm	h_1 = 22.638 [in]	h_2 = 19.638	[in]	
	h_3 = 15.113 [in]	h_4 = 12.113	[in]	
	g = 4.000 [in]	d_e = 1.250	[in]	
	p_{fi} = 2.000 [in]	p_{fo} = 2.000	[in]	
	p_b = 3.000 [in]			
	s = 2.646 [in]	Y_p = 231.83	[in]	
Flexure resistance factor-LRFD	ϕ_d = 1.00			AISC 358-16 2.4.1
End plate bending strength	$\phi_d M_{pl} = \phi_d F_{yp} t_p^2 Y_p$	= **271.67**	[kip-ft]	AISC DG4 Table 3.1
Seismic flange force	P_{uf_t} = see Seismic Moment and Beam Flange Force Calc	= **159.4**	[kips]	
Flange force provided by end plate bending	$\phi R_{pl} = \phi M_{pl} / d_m$	= **187.6**	[kips]	AISC DG4 Eq 3.10
	ratio = **0.85**	> P_{uf_t}	OK	

Seismic End Plate Stiffener Geometry Limitations	**PASS**

Beam web thick	t_{wb} = 0.315 [in]	Stiff thick t_s = 0.438	[in]	
Beam yield strength	F_{yb} = 50.0 [ksi]	Stiff yield F_{ys} = 36.0	[ksi]	

Min Stiffener Plate Thickness

Min stiffener plate thickness	$t_{smin} = t_{wb} F_{yb} / F_{ys}$	= **0.438**	[in]	AISC DG4 Eq 3.15
Stiffener plate thickness	t_s =	= **0.438**	[in]	
		≥ t_{smin}	OK	

Min Stiff Thick to Avoid Local Buckiling

Stiffener plate height	h_{st} = 6.250 [in]	E = 29000	[ksi]	
Stiffener plate strength	F_{ys} = 36.0 [ksi]			
Min stiffener plate thickness	$t_{smin} = 1.79\, h_{st} \sqrt{F_{ys} / E}$	= **0.394**	[in]	AISC DG4 Eq 3.16
Stiffener plate thickness	t_s =	= **0.438**	[in]	
		≥ t_{smin}	OK	

Seismic Beam Flange Weld Strength		ratio = 159.4 / 152.2	= **1.05**	**FAIL**	
Flange force required in tension	$P_{uf\_t} = P_u\ /\ 2 - M_u\ /\ d_m$		= **159.4**	[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	$w = {}^{7}/_{16}$ [in]	load angle θ = 90.0	[°]		
Electrode strength	F_{EXX} = 70.0 [ksi]	strength coeff C_1 = 1.00			AISC 15th 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 15th Page 8-9
Fillet weld shear strength	$R_{n-w} = 0.6\ (C_1 \times 70\ \text{ksi})\ 0.707\ w\ n\ C_2$		= 38.973	[kip/in]	AISC 15th 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, tensile rupture as per AISC 15th Eq J4-2 is checked					AISC 15th J2.4
Base metal tensile rupture	$R_{n-b} = F_u\ t$		= 34.125	[kip/in]	AISC 15th Eq J4-2

For seismic nonductile limit state, weld strength use ϕ = 0.75 , base metal rupture use ϕ_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	$R_n = \min\left(R_{n-w},\ R_{n-b} \times \frac{0.90}{0.75}\right)$	= **38.973**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq 8-1
	$\phi\ R_n$ =	= **29.230**	[kip/in]	

Shear resistance required	P_{uf_t} =	= **159.4**	[kips]
Fillet weld length - double fillet	L =	= 5.207	[in]
Shear resistance provided	$\phi\ F_n = \phi\ R_n \times L$	= **152.2**	[kips]
	ratio = **1.05**	< P_{uf_t}	**NG**

Seismic Beam Web Weld Strength ratio = 23.5 / 85.8 = **0.27** 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} = 0.750 [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	$L_2 = d_b - 2t_{fb} - p_{fi} - 2\ d_{bolt}$	= 13.350	[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 θ = 0.0	[°]	
Electrode strength	F_{EXX} = 70.0 [ksi]	strength coeff C_1 = 1.00		AISC 15th 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 15th 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 15th 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 15th Eq J4-4 is checked				AISC 15th J2.4
Base metal shear rupture	$R_{n-b} = 0.6\ F_u\ t$	= 12.285	[kip/in]	AISC 15th Eq J4-4

For seismic nonductile limit state, weld strength use ϕ = 0.75 , base metal rupture use ϕ_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	$R_n = \min\left(R_{n-w}, R_{n-b} \times \frac{0.90}{0.75}\right)$	= **14.742**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq 8-1
	ϕR_n =	= **11.057**	[kip/in]	

Shear resistance required	V_u =	= **23.5**	[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 = **0.27**	> V_u	OK	

Seismic Column Flexural Yielding		ratio = 159.4 / 186.0	= **0.86**	**PASS**

Column Flange Bending Strength

See AISC DG4 Table 3.5 for all formulas to derive the following parameters — AISC DG4 Table 3.5

Tension bolt moment arm	h_1 = 22.638	[in]	h_2 = 19.638	[in]		
	h_3 = 15.113	[in]	h_4 = 12.113	[in]		
*** Stiffened Column Flange Case ***						
Column section	b_{fc} = 6.560	[in]	t_{fc} = 0.520	[in]		
	F_{yc} = 50.0	[ksi]	bolt gage g = 4.000	[in]		
	s = 2.561	[in]	c = 4.525	[in]		
Stiffener plate thickness	t_s = 0.500	[in]				
	p_{si} = 2.013	[in]	p_{so} = 2.013	[in]		
	p_b = 3.000	[in]	d_e = 1.250	[in]		
	Y_c = 239.0	[in]				
Flexure resistance factor-LRFD	ϕ_d = 1.00					AISC 358-16 2.4.1
Column flange bending strength	$\phi_d M_{cf} = \phi_d F_{yc} t_{fc}^2 \ Y_c$		= **269.25**	[kip-ft]		AISC DG4 Table 3.5
Seismic flange force	P_{uf_t} = see Seismic Moment and Beam Flange Force Calc		= **159.4**	[kips]		
Flange force provided by column flange bending	$\phi R_{cf} = \phi M_{cf} / d_m$		= **186.0**	[kips]		AISC DG4 Eq 3.21
	ratio = **0.86**		> P_{uf_t}	OK		

Seismic Column Web Yielding	159.4 / 113.6	**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		= **159.4**	[kips]	
Column section	d_c = 12.500 [in]		t_{fc} = 0.520	[in]	
	t_{wc} = 0.300 [in]		k_c = 0.820	[in]	
Column yield strength	F_{yc} = 50.0 [ksi]				
Distance from to top of column to top of beam flange	d_{end} = 13.000 [in]				
Top column reduction factor	C_t = 1.0				AISC DG4 Eq 3.24
Beam flange fillet weld size	w = 0.438 [in]	beam flange	t_{fb} = 0.525	[in]	
Length of bearing	$N = t_{fb} + 2w$		= 1.400	[in]	AISC DG4 Eq 3.24
End plate thickness	t_p = 0.625 [in]				
Column web yielding strength	$R_n = C_t (6 k_c + N + 2 t_p) F_{yc} t_{wc}$		= 113.6	[kips]	AISC DG4 Eq 3.24
Resistance factor-LRFD	ϕ = 1.00				AISC 341-16 E3.6e (1)
	ϕR_n =		= **113.6**	[kips]	
Unbalanced force to be resisted by transverse stiffeners	$R_s = P_{uf} - \phi R_n$		= **45.9**	[kips]	

Seismic Column Web Buckling		159.4 / 71.9		**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	= **159.4**	[kips]	
Column section	d_c = 12.500 [in]	t_{fc} = 0.520	[in]	
	t_{wc} = 0.300 [in]	k_c = 0.820	[in]	
	$h = d_c - 2\,k_c$	= 10.860	[in]	
Column yield strength	F_{yc} = 50.0 [ksi]	E_c = 29000	[ksi]	
Distance from top of beam flange to top of column	$d_{end\text{-}flg}$ = 13.000 [in]	beam flange t_{fb} = 0.525	[in]	
Distance from center of flange force to top of column	$d_{end\text{-}F} = d_{end\text{-}flg} + 0.5\,t_{fb}$	= 13.263	[in]	
	$d_{end\text{-}F} \ge d_c/2$, R_n has no reduction			AISC 15th J10.5
Top column reduction factor	C_t = 1.0			AISC 15th J10.5
Column web buckling strength	$R_n = \dfrac{C_t\,24\,t_{wc}^3\,\sqrt{E_c F_{yc}}}{h}$	= 71.9	[kips]	AISC 15th Eq J10-8
Resistance factor-LRFD	ϕ = 1.00			AISC 341-16 E3.6e (1)
	ϕR_n =	= **71.9**	[kips]	
Unbalanced force to be resisted by transverse stiffeners	$R_s = P_{uf\_c} - \phi R_n$	= **87.6**	[kips]	

Seismic Column Web Crippling		159.4 / 146.0		**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	= **159.4**	[kips]	
Column section	d_c = 12.500 [in]	t_{fc} = 0.520	[in]	
	t_{wc} = 0.300 [in]	k_c = 0.820	[in]	
Column yield strength	F_{yc} = 50.0 [ksi]	E_c = 29000	[ksi]	
Beam flange fillet weld size	w = 0.438 [in]	beam flange t_{fb} = 0.525	[in]	
End plate thickness	t_p = 0.625 [in]			
Length of bearing	$l_b = t_{fb} + 2\,w + 2\,t_p$	= 2.650	[in]	
Distance from top of column to top of beam flange	$d_{end\text{-}flg}$ =	= 13.000	[in]	
Distance from top of column to center of flange force	$d_{end\text{-}F} = d_{end\text{-}flg} + 0.5\,t_{fb}$	= 13.263	[in]	
	$d_{end\text{-}F} \ge d_c/2$, use Eq J10-4			AISC 15th Eq J10-4
Column web crippling strength	$R_n = 0.8\,t_{wc}^2\,\left[1+3\dfrac{l_b}{d_c}\left(\dfrac{t_{wc}}{t_{fc}}\right)^{1.5}\right] \times \left(\dfrac{E_c F_{yc} t_{fc}}{t_{wc}}\right)^{0.5}$	= 146.0	[kips]	AISC 15th Eq J10-4
Resistance factor-LRFD	ϕ = 1.00			AISC 341-16 E3.6e (1)
	ϕR_n =	= **146.0**	[kips]	
Unbalanced force to be resisted by transverse stiffeners	$R_s = P_{uf\_c} - \phi R_n$	= **13.5**	[kips]	

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 = 3.130	[in]	depth d_s = 11.460	[in]		
Stiffener plate thickness	t_s = 0.500	[in]				
Column flange thickness	t_{fc} = 0.520	[in]	column depth d_c = 12.500	[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 15th J10.8 (2)
Stiffener plate thickness	t_s =		= **0.500**	[in]		
			≥ t_{smin}	OK		
Min Stiffener Plate Depth						
Min stiffener plate depth	$d_{smin} = (d_c - 2\,t_{fc}) / 2$		= **5.730**	[in]		AISC 15th J10.8 (3)
Stiffener plate depth	d_s =		= **11.460**	[in]		
			≥ d_{smin}	OK		

Seismic Stiffener Yield at Column Flange			ratio = 87.6 / 77.1	= **1.14**	**FAIL**
Stiffener plate width	b_s = 3.130	[in]	thickness t_s = 0.500	[in]	
Stiffener plate corner clip	clip =		= 0.750	[in]	AISC 15th Page 8-18
Stiffener plate yield strength	F_y =		= 36.0	[ksi]	
Stiffener plate cross sect area	$A_{st} = (b_s - \text{clip})\, t_s$		= 1.190	[in^2]	
Trans stiffener strength required	R_s =		= **87.6**	[kips]	
Trans stiffener strength provided	$R_n = F_y \times 2 \times A_{st}$		= 85.7	[kips]	AISC 15th Eq J4-1
Bolt resistance factor-LRFD	ϕ = 0.90				AISC 15th Eq J4-1
	$\phi\, R_n$ =		= **77.1**	[kips]	
	ratio = **1.14**		< R_s	NG	

Seismic Stiffener Shear at Column Web			ratio = 87.6 / 215.1	= **0.41**	**PASS**
Stiffener plate depth	d_s = 11.460	[in]	thickness t_s = 0.500	[in]	
Stiffener plate corner clip	clip =		= 0.750	[in]	AISC 15th Page 8-18
Stiffener plate yield strength	F_y =		= 36.0	[ksi]	
Stiffener plate cross sect area	$A_{gv} = (d_s - 2 \times \text{clip})\, t_s$		= 4.980	[in^2]	
Trans stiffener strength required	R_s =		= **87.6**	[kips]	
Trans stiffener strength provided	$R_n = 2 \times 0.6 \times F_y \times A_{gv}$		= 215.1	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00				AISC 15th Eq J4-3
	$\phi\, R_n$ =		= **215.1**	[kips]	
	ratio = **0.41**		> R_s	OK	

Seismic Stiffener Compression		ratio = 87.6 / 64.0	= **1.37**	**FAIL**
Column section	d = 12.500 [in]	t_f = 0.520	[in]	
Stiffener plate depth	$d_s = d - 2\,t_f$	= 11.460	[in]	
Stiffener plate width	b_s = 3.130 [in]	thickness t_s = 0.500	[in]	
Stiffener plate corner clip	clip =	= 0.750	[in]	AISC 15th Page 8-18
Stiffener plate yield strength	F_y = 36.0 [ksi]	E = 29000	[ksi]	
Stiffener plate cross sect area	$A_{st} = (b_s - clip)\,t_s$	= 1.190	[in^2]	
Stiffener plate unbraced length	$L = d_s = d - 2\,t_f$	= 11.460	[in]	
Plate radius of gyration	$r = t_s / \sqrt{12}$	= 0.144	[in]	
Stiffener plate slenderness	$KL/r = 0.75 \times L / r$	= 59.55		
Elastic buckling stress	$F_e = \dfrac{\pi^2 E}{(KL/r)^2}$	= 80.72	[ksi]	AISC 15th Eq E3-4
	when $\dfrac{KL}{r} \le 4.71 \left(\dfrac{E}{F_y}\right)^{0.5} = 133.68$			AISC 15th E3
Critical stress	$F_{cr} = 0.658^{(F_y/F_e)}\,F_y$	= 29.87	[ksi]	AISC 15th Eq E3-2
Trans stiffener strength required	R_s =	= **87.6**	[kips]	
Trans stiffener strength provided	$R_n = 2 \times F_{cr} \times A_{st}$	= 71.1	[kips]	AISC 15th Eq E3-1
Bolt resistance factor-LRFD	ϕ = 0.90			AISC 15th E1
	$\phi\,R_n$ =	= **64.0**	[kips]	
	ratio = **1.37**	$< R_s$	**NG**	

Seismic Stiffener to Column Flange Fillet Weld Limitation				**PASS**
Min Fillet Weld Size				
Thinner part joined thickness	t =	= 0.500	[in]	
Min fillet weld size allowed	w_{min} =	= **0.188**	[in]	AISC 15th Table J2.4
Fillet weld size provided	w =	= **0.250**	[in]	
		$\ge w_{min}$	OK	
Min Fillet Weld Length				
Fillet weld size provided	w =	= 0.250	[in]	
Min fillet weld length allowed	$L_{min} = 4 \times w$	= **1.000**	[in]	AISC 15th J2.2b
Min fillet weld length	$L = b_s - clip$	= **2.380**	[in]	
		$\ge L_{min}$	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	ϕ_v = 0.90			AISC 15th G1
Fillet weld shear strength	$\phi\,R_{n\text{-}w}$ =	= 1.392	[kip/in]	AISC 15th Eq 8-2a
Fillet weld strength $\phi\,R_{n\text{-}w} \times 1.5 \times 2$ to account for 90° load angle when it's in tension and double fillet				
Stiffener plate thickness	t_s = 0.500 [in]	F_y = 36.0	[ksi]	
Min double fillet weld size to match stiffener plate yield strength	$D_{min} = \phi_v F_y t_s / (\phi\,R_{n\text{-}w} \times 1.5 \times 2)$	= **3.879**	[1/16 "]	
Fillet weld size provided	D =	= **4.000**	[1/16 "]	
		$\ge D_{min}$	OK	

Seismic Stiffener to Column Web Fillet Weld Limitation — PASS

Min Fillet Weld Size

Thinner part joined thickness	$t =$	= 0.300	[in]	
Min fillet weld size allowed	$w_{min} =$	= **0.188**	[in]	AISC 15th Table J2.4
Fillet weld size provided	$w =$	= **0.250**	[in]	
		≥ w_{min}	OK	

Min Fillet Weld Length

Fillet weld size provided	$w =$	= 0.250	[in]	
Min fillet weld length allowed	$L_{min} = 4 \times w$	= **1.000**	[in]	AISC 15th J2.2b
Min fillet weld length	$L = d_c - 2 \times t_{fc} - 2 \times clip$	= **9.960**	[in]	
		≥ L_{min}	OK	

Seismic Stiffener Weld Strength at Column Flange — ratio = 87.6 / 79.5 = **1.10** FAIL

Stiffener to Column Flange Weld Length Calc

Stiffener plate width & clip	$b_s = 3.130$ [in]	clip = 0.750	[in]	
Stiffener to column flange weld length - double fillet	$L = (b_s - clip) \times 2$ stiffener	= **4.760**	[in]	
Trans stiffener strength required	$R_s =$	= **87.6**	[kips]	

Fillet Weld Strength Check

Fillet weld leg size	$w = {}^1/_4$ [in]	load angle θ = 90.0	[°]	
Electrode strength	$F_{EXX} = 70.0$ [ksi]	strength coeff C_1 = 1.00		AISC 15th 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 15th Page 8-9
Fillet weld shear strength	$R_{n-w} = 0.6 (C_1 \times 70 \text{ ksi}) 0.707\, w\, n\, C_2$	= 22.271	[kip/in]	AISC 15th Eq 8-1

Base metal - stiffener plate	thickness t = 0.500 [in]	tensile F_u = 58.0	[ksi]	

Base metal - stiffener plate is in tension, tensile rupture as per AISC 15th Eq J4-2 is checked — AISC 15th J2.4

Base metal tensile rupture	$R_{n-b} = F_u\, t$	= 29.000	[kip/in]	AISC 15th Eq J4-2

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	$R_n = \min \left(R_{n-w} ,\ R_{n-b} \times \frac{0.90}{0.75} \right)$	= **22.271**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	$\phi = 0.75$			AISC 15th Eq 8-1
	$\phi R_n =$	= **16.703**	[kip/in]	

Shear resistance required	$R_s =$	= **87.6**	[kips]	
Fillet weld length - double fillet	$L =$	= 4.760	[in]	
Shear resistance provided	$\phi F_n = \phi R_n \times L$	= **79.5**	[kips]	
	ratio = **1.10**	< R_s	NG	

Seismic Stiffener Weld Strength at Column Web ratio = 87.6 / 221.8 = **0.39** **PASS**

Stiffener to Column Web Weld Length Calc

Column section	d_c = 12.500	[in]	t_{fc} = 0.520	[in]	
Stiffener plate corner clip	clip = 0.750	[in]			
Stiffener to column web weld length - double fillet	L = (d_c - 2 x t_{fc} - 2 x clip) x 2 stiffener		= **19.920**	[in]	
Trans stiffener strength required	R_s =		= **87.6**	[kips]	

Fillet Weld Strength Check

Fillet weld leg size	w = $^1/_4$	[in]	load angle θ = 0.0	[°]	
Electrode strength	F_{EXX} = 70.0	[ksi]	strength coeff C_1 = 1.00		AISC 15th 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 15th Page 8-9
Fillet weld shear strength	R_{n-w} = 0.6 (C_1 x 70 ksi) 0.707 w n C_2		= 14.847	[kip/in]	AISC 15th Eq 8-1

Base metal - stiffener plate	thickness t = 0.500	[in]	tensile F_u = 58.0	[ksi]	
Base metal - stiffener plate is in shear, <u>shear</u> rupture as per AISC 15th Eq J4-4 is checked					AISC 15th J2.4
Base metal shear rupture	R_{n-b} = 0.6 F_u t		= 17.400	[kip/in]	AISC 15th Eq J4-4

For seismic nonductile limit state, weld strength use ϕ = 0.75 , base metal rupture use ϕ_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	$R_n = \min\left(R_{n-w} , R_{n-b} \times \frac{0.90}{0.75} \right)$	= **14.847**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq 8-1
	ϕR_n =	= **11.135**	[kip/in]	

Shear resistance required	R_s =	= **87.6**	[kips]
Fillet weld length - double fillet	L =	= 19.920	[in]
Shear resistance provided	$\phi F_n = \phi R_n$ x L	= **221.8**	[kips]
	ratio = **0.39**	> R_s	OK