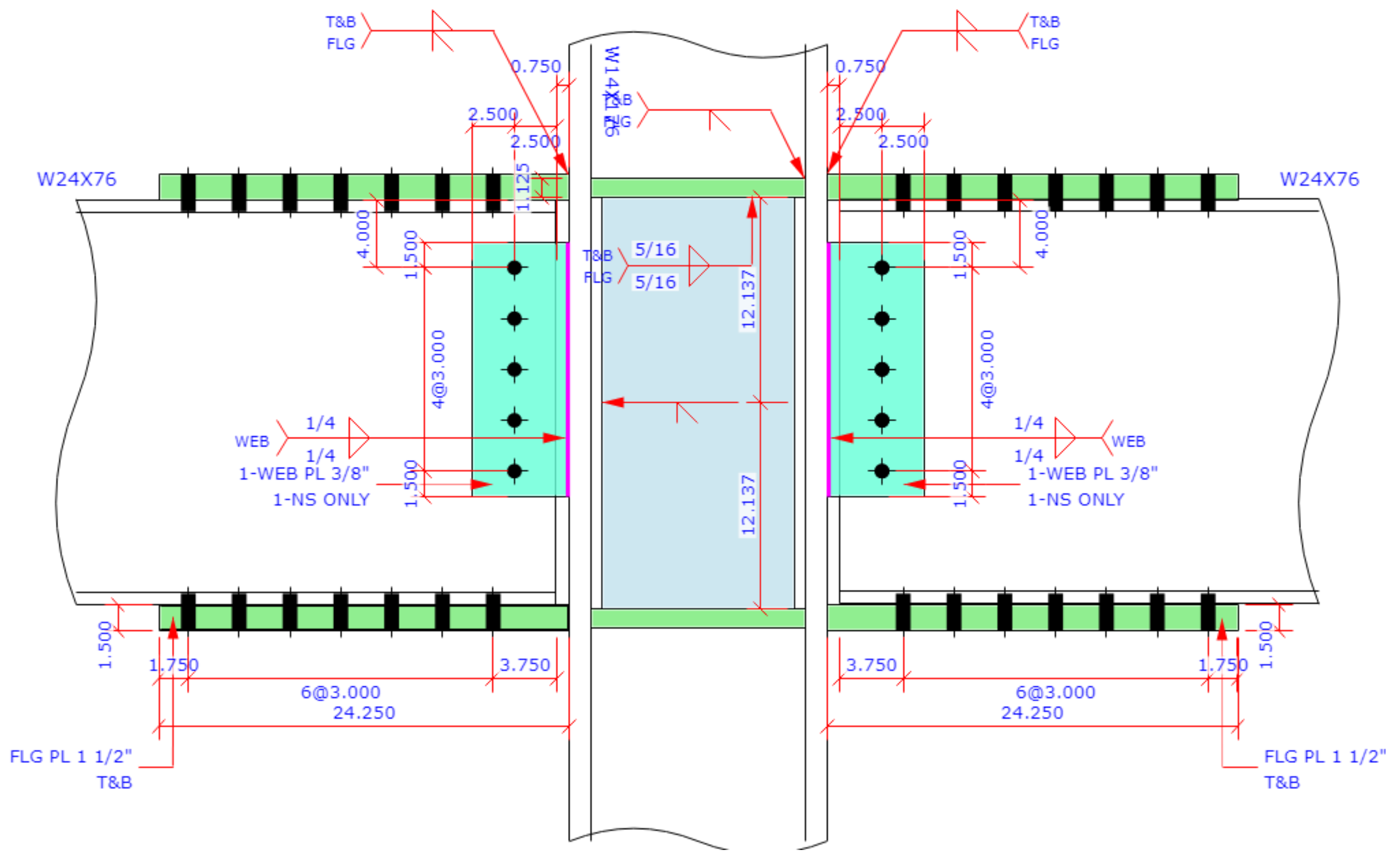
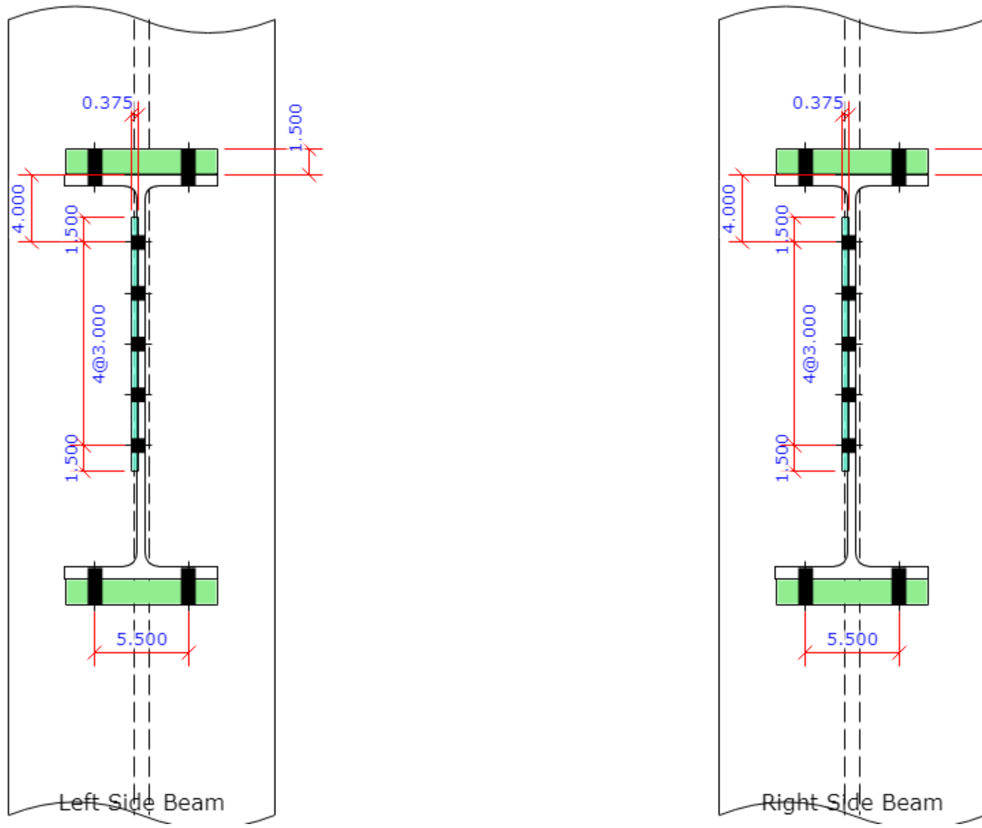
Result Summary - Overall Moment Connection - Beam to Column Code=AISC 360-16 LRFD

Result Summary - Overall	geometries & weld limitations = **FAIL**	limit states max ratio = **0.96**	**PASS**
Right Beam to Column	geometries & weld limitations = **FAIL**	limit states max ratio = **0.96**	**PASS**
Left Beam to Column	geometries & weld limitations = **FAIL**	limit states max ratio = **0.92**	**PASS**

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

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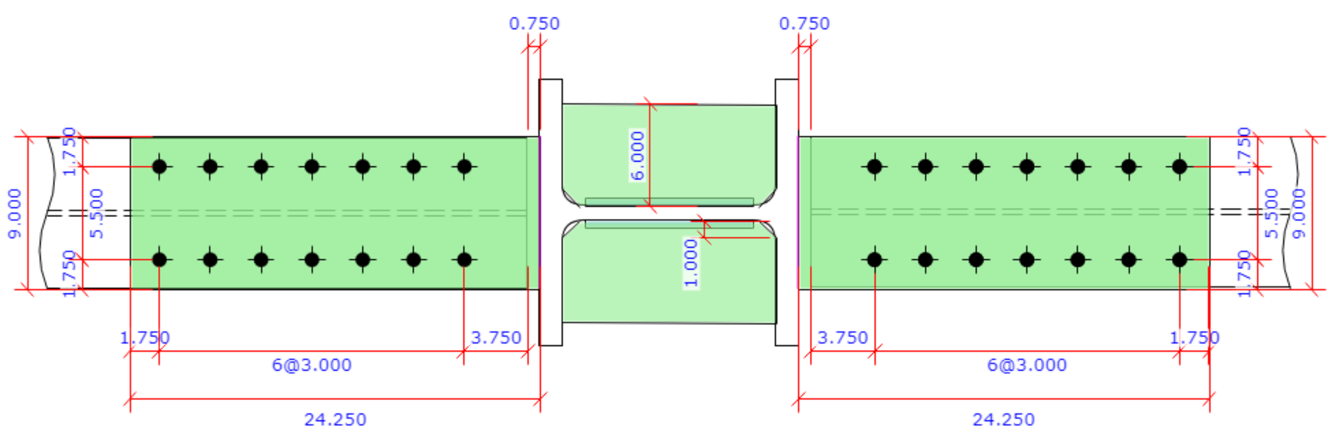
1.500
4.000
1.500
4@3.000
1.500
5.500
Left Side Beam
0.375
1.500
4.000
1.500
4@3.000
1.500
5.500
Right Side Beam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6.000
1.000
9.000
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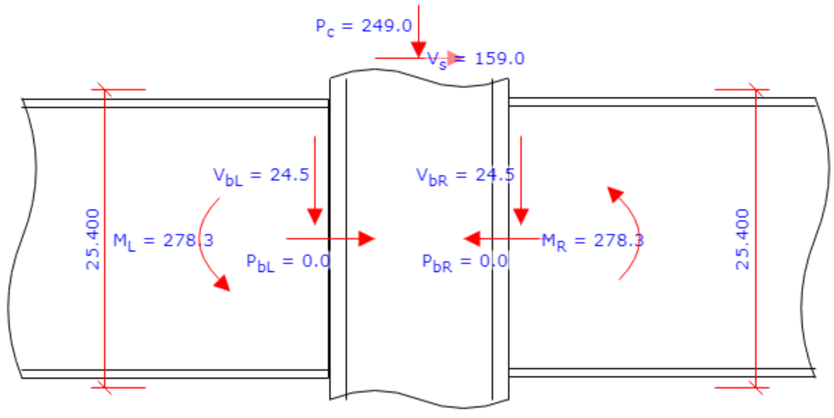
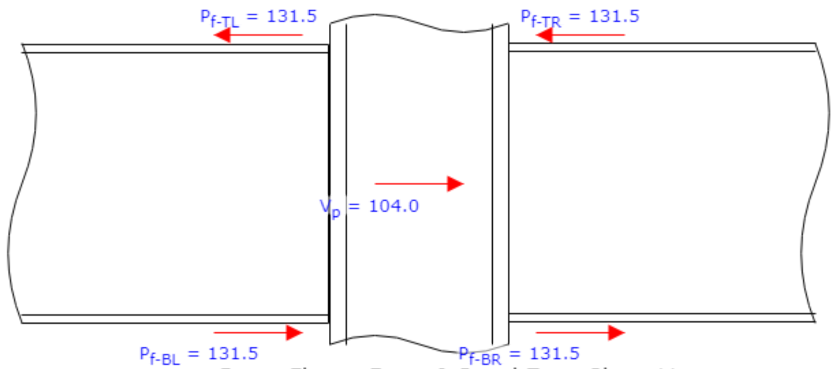
6@3.000
24.250



Design Load



Beam Flange Force & Panel Zone Shear Vp

Beam Flange Force Calc

Beam Flange Force - Right Side Beam

Beam section	d_b = 23.900 [in]	t_{fb} = 0.680	[in]	
Flange plate thickness	t_{fp} = 1.500 [in]			
Flange force moment arm	$d_m = d_b + t_{fp}$	= 25.400	[in]	
User input load	axial P_{bR} = 0.0 [kips]	moment M_R = 278.30	[kip-ft]	
Beam flange force - top	$P_{f\text{-}TR} = P_{bR} / 2 + M_R / d_m$	= **131.5**	[kips]	
Beam flange force - bottom	$P_{f\text{-}BR} = P_{bR} / 2 - M_R / d_m$	= **-131.5**	[kips]	

Beam Flange Force - Left Side Beam

Beam section	d_b = 23.900 [in]	t_{fb} = 0.680	[in]	
Flange plate thickness	t_{fp} = 1.500 [in]			
Flange force moment arm	$d_m = d_b + t_{fp}$	= 25.400	[in]	
User input load	axial P_{bL} = 0.0 [kips]	moment M_L = 278.30	[kip-ft]	
Beam flange force - top	$P_{f\text{-}TL} = P_{bL} / 2 - M_L / d_m$	= **-131.5**	[kips]	
Beam flange force - bottom	$P_{f\text{-}BL} = P_{bL} / 2 + M_L / d_m$	= **131.5**	[kips]	

Panel Zone Shear Force Calc

Column story shear	V_s = from user input	= 159.0	[kips]	
Panel zone shear force	$V_p = P_{f\text{-}TR} - P_{f\text{-}TL} - V_s$	= **104.0**	[kips]	

Seismic Moment and Beam Flange Force Calc

Seismic Force Calc - Right Side Beam

Beam steel strength	F_y = 50.0 [ksi]	F_u = 65.0	[ksi]	
Beam sect W24X76	Z_x = 200.00 [in^3]			
Beam ratio of expected/min yield stress	R_y = 1.10			AISC 341-16 Table A3.1
	$C_{pr} = \min\left(1.2, \frac{F_y + F_u}{2\,F_y}\right)$	= 1.15		AISC 358-16 Eq 2.4.3-2
Probable max M at plastic hinge	$M_{pr} = C_{pr} R_y F_y Z_x$	= 1054.2	[kip-ft]	AISC 358-16 Eq 2.4.3-1
M at plastic hinge for column panel zone shear calc	$M_h = R_y F_y Z_x$	= 916.7	[kip-ft]	
Beam sect W24X76	d = 23.900 [in]	b_f = 8.990	[in]	
	t_f = 0.680 [in]			
Flange plate first bolt to column face dist & bolt spacing	S_1 = 4.500 [in]	s = 3.000	[in]	
Flange plate no of bolt	n = 14			
Dist from column face to plastic hinge	$S_h = S_1 + s\,(n/2 - 1)$	= 22.500	[in]	AISC 358-16 Eq 7.6-5
Beam clear span between column flange	L = from user input	= 345.00	[in]	
Dist between beam 2 end plastic hinges	$L_h = L_c - 2\,S_h$	= 300.00	[in]	
Factored gravity UDL on beam	w_g = from user input	= 1.15	[kip/ft]	AISC 358-16 6.10.1
Beam shear due to gravity	$V_g = w_g L_h / 2$	= 14.4	[kips]	AISC 358-16 6.10.1
Probable max shear at plastic hinge	$V_h = 2\,M_{pr} / L_h + V_g$	= **98.7**	[kips]	AISC 358-16 Eq 6.10-2
Probable max shear at column face	$V_c = V_h + w_g S_h$	= **100.9**	[kips]	
Probable max M at column face	$M_f = M_{pr} + V_h \times S_h$	= **1239.2**	[kip-ft]	AISC 358-16 Eq 6.10-1
Flange plate thickness	t_p = from user input	= 1.500	[in]	

Moment arm between flange plates	$d_m = d + t_p$	= 25.400	[in]	AISC 358-16 Eq 7.6-7
Factored beam flange force	$F_{fu} = M_f / d_m$	= **585.5**	[kips]	AISC 358-16 Eq. 6.10-6

Probable seismic moment at column face which will be used in column panel zone shear calculation

Moment at plastic hinge	$M_h = R_y F_y Z_x$	= 916.7	[kip-ft]	
Shear at plastic hinge	$V_h = 2\ M_h / L_h + V_g$	= 87.71	[kips]	
Moment at column face	$M_c = M_{pr} + V_h \times S_h$	= **1081.1**	[kip-ft]	

Seismic Force Calc - Left Side Beam

Beam steel strength	F_y = 50.0 [ksi]	F_u = 65.0	[ksi]	
Beam sect W24X76	Z_x = 200.00 [in^3]			
Beam ratio of expected/min yield stress	R_y = 1.10			AISC 341-16 Table A3.1
	$C_{pr} = \min\left(1.2, \frac{F_y + F_u}{2\ F_y}\right)$	= 1.15		AISC 358-16 Eq 2.4.3-2
Probable max M at plastic hinge	$M_{pr} = C_{pr} R_y F_y Z_x$	= 1054.2	[kip-ft]	AISC 358-16 Eq 2.4.3-1
M at plastic hinge for column panel zone shear calc	$M_h = R_y F_y Z_x$	= 916.7	[kip-ft]	
Beam sect W24X76	d = 23.900 [in]	b_f = 8.990	[in]	
	t_f = 0.680 [in]			
Flange plate first bolt to column face dist & bolt spacing	S_1 = 4.500 [in]	s = 3.000	[in]	
Flange plate no of bolt	n = 14			
Dist from column face to plastic hinge	$S_h = S_1 + s\ (\ n/2\ -1\ )$	= 22.500	[in]	AISC 358-16 Eq 7.6-5
Beam clear span between column flange	L = from user input	= 345.00	[in]	
Dist between beam 2 end plastic hinges	$L_h = L_c - 2\ S_h$	= 300.00	[in]	
Factored gravity UDL on beam	w_g = from user input	= 1.15	[kip/ft]	AISC 358-16 6.10.1
Beam shear due to gravity	$V_g = w_g L_h / 2$	= 14.4	[kips]	AISC 358-16 6.10.1
Probable max shear at plastic hinge	$V_h = 2\ M_{pr} / L_h + V_g$	= **70.0**	[kips]	AISC 358-16 Eq 6.10-2
Probable max shear at column face	$V_c = V_h + w_g S_h$	= **67.8**	[kips]	
Probable max M at column face	$M_f = M_{pr} + V_h \times S_h$	= **1185.3**	[kip-ft]	AISC 358-16 Eq 6.10-1
Flange plate thickness	t_p = from user input	= 1.500	[in]	
Moment arm between flange plates	$d_m = d + t_p$	= 25.400	[in]	AISC 358-16 Eq 7.6-7
Factored beam flange force	$F_{fu} = M_f / d_m$	= **560.0**	[kips]	AISC 358-16 Eq. 6.10-6

Probable seismic moment at column face which will be used in column panel zone shear calculation

Moment at plastic hinge	$M_h = R_y F_y Z_x$	= 916.7	[kip-ft]	
Shear at plastic hinge	$V_h = 2\ M_h / L_h - V_g$	= 58.96	[kips]	
Moment at column face	$M_c = M_h + V_h \times S_h$	= **1027.2**	[kip-ft]	

Seismic Column Panel Zone Shear Force Calc

Seismic moment at right column face	M_R = from above calc	= 1081.1	[kip-ft]
Seismic moment at left column face	M_L = from above calc	= 1027.2	[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 = 150.0 [in]	h_b = 168.0	[in]

Story shear	$V_s = \dfrac{M_R + M_L}{0.5(h_t + h_b)}$	= 159.1	[kips]	
Calculate Column Panel Zone Shear				
Moment arm between flange plates	$d_m = d + t_p$	= 25.400	[in]	AISC 358-16 Eq 7.6-7
Column panel zone shear force	$V_p = (M_R + M_L) / d_m - V_s$	= **836.9**	[kips]	

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

Result Summary geometries & weld limitations = **FAIL** limit states max ratio = **0.96** **PASS**

Geometry Restriction Checks - Flange Plate **PASS**

Min Bolt Edge Distance - Flange Plate

Bolt diameter	$d_b =$	= 0.875	[in]	
Min edge distance allowed	$L_{e\text{-}min} =$	= **1.125**	[in]	AISC 15th Table J3.4
Min edge distance in Flange Plate	$L_e =$	= **1.745**	[in]	
		$\geq L_{e\text{-}min}$	OK	

Min Bolt Spacing - Flange Plate

Bolt diameter	$d_b =$	= 0.875	[in]	
Min bolt spacing allowed	$L_{s\text{-}min} = 2.667\ d_b$	= **2.333**	[in]	AISC 15th J3.3
Min Bolt spacing in Flange Plate	$L_s =$	= **3.000**	[in]	
		$\geq L_{s\text{-}min}$	OK	

Geometry Restriction Checks - Web Plate **PASS**

Min Bolt Edge Distance - Web Plate

Bolt diameter	$d_b =$	= 0.875	[in]	
Min edge distance allowed	$L_{e\text{-}min} =$	= **1.125**	[in]	AISC 15th Table J3.4
Min edge distance in Web Plate	$L_e =$	= **1.500**	[in]	
		$\geq L_{e\text{-}min}$	OK	

Min Bolt Spacing - Web Plate

Bolt diameter	$d_b =$	= 0.875	[in]	
Min bolt spacing allowed	$L_{s\text{-}min} = 2.667\ d_b$	= **2.333**	[in]	AISC 15th J3.3
Min Bolt spacing in Web Plate	$L_s =$	= **3.000**	[in]	
		$\geq L_{s\text{-}min}$	OK	

Fillet Weld Limitation Checks - Web Plate — PASS

Min Fillet Weld Size

Thinner part joined thickness	$t =$	= 0.375	[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	$L =$	= **15.000**	[in]	
		$\geq L_{min}$	OK	

Web Plate Weld Strength — ratio = 24.5 / 164.5 = **0.15** PASS

Shear force in demand	$V_u =$	= **24.5**	[kips]	
Fillet weld length - double fillet	$L =$	= 15.000	[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 - web plate — thickness $t = 0.375$ [in] — tensile $F_u = 65.0$ [ksi]

Base metal - web 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	$R_{n\text{-}b} = 0.6 \, F_u \, t$	= 14.625	[kip/in]	AISC 15th Eq J4-4
Double fillet linear shear strength	$R_n = \min (R_{n\text{-}w} , R_{n\text{-}b})$	= **14.625**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	$\phi = 0.75$			AISC 15th Eq 8-1
	$\phi R_n =$	= **10.969**	[kip/in]	

Shear resistance required	$V_u =$	= **24.5**	[kips]	
Fillet weld length - double fillet	$L =$	= 15.000	[in]	
Shear resistance provided	$\phi F_n = \phi R_n \times L$	= **164.5**	[kips]	
	ratio = **0.15**	$> V_u$	OK	

Flange Plate - Bolt Shear

ratio = 139.7 / 530.4 = **0.26** **PASS**

Flange force moment arm	$d_m = d_b$	= 23.900	[in]	
Beam flange force as shear	$V_u = P_b / 2 + M / d_m$	= **139.7**	[kips]	
Bolt shear stress	bolt grade = A490-X	F_{nv} = 84.0	[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.875 [in]	bolt area A_b = 0.601	[in^2]	
Number of bolt carried shear	n_s = 14.0	shear plane m = 1		
Bolt group eccentricity coefficient	C_{ec} =	= 1.000		
Required shear strength	V_u =	= **139.7**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 707.2	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J3-1
	ϕR_n =	= **530.4**	[kips]	
	ratio = **0.26**	> V_u	OK	

Flange Plate - Bolt Bearing

ratio = 131.5 / 530.4 = **0.25** **PASS**

Flange force moment arm	$d_m = d_b + t_p$	= 25.400	[in]	
Beam flange force as shear	$V_u = P_b / 2 + M / d_m$	= **131.5**	[kips]	

Single Bolt Shear Strength

Bolt shear stress	bolt grade = A490-X	F_{nv} = 84.0	[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.875 [in]	bolt area A_b = 0.601	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$	= 50.5	[kips]	AISC 15th Eq J3-1

Bolt Bearing/TearOut Strength on Plate

Bolt hole diameter	bolt dia $d_b = {}^{7}/_{8}$ [in]	bolt hole dia $d_h = {}^{15}/_{16}$	[in]	AISC 15th Table J3.3
Bolt spacing & edge distance	spacing L_s = 3.000 [in]	edge distance L_e = 1.750	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate thickness	t = 1.500 [in]			

Interior Bolt

Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.063	[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
	= 241.3 ≤ 204.8	= 204.8	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$	= 50.5	[kips]	

Edge Bolt

Bolt hole edge clear distance	$L_c = L_e - d_h / 2$	= 1.281	[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
	= 149.9 ≤ 204.8	= 149.9	[kips]	
Bolt strength at edge	$R_{n\text{-}ed} = \min(R_{n\text{-}t\&b\text{-}ed}, R_{n\text{-}bolt})$	= 50.5	[kips]	

Number of bolt	interior n_{in} = 12	edge n_{ed} = 2		
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-}in} + n_{ed} R_{n\text{-}ed}$	= 707.2	[kips]	
Required shear strength	V_u =	= **131.5**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th J3-10
	ϕR_n =	= **530.4**	[kips]	
	ratio = **0.25**	> V_u	OK	

Flange Plate - Block Shear - 2 Side Strips		ratio = 131.5 / 1345.5	= **0.10**	**PASS**
Flange force moment arm	$d_m = d_b + t_p$	= 25.400	[in]	
Flange force - tension	$P_u = P_b / 2 + M / d_m$	= **131.5**	[kips]	
Plate Block Shear - 2 Side Strips				
Bolt hole diameter	bolt dia $d_b = {}^7/_8$ [in]	bolt hole dia $d_h = 1$	[in]	AISC 15th B4.3b
Plate thickness	$t_p = 1.500$ [in]			
Plate strength	$F_y = 50.0$ [ksi]	$F_u = 65.0$	[ksi]	
Bolt no in ver & hor dir	$n_v = 2$	$n_h = 7$		
Bolt spacing in ver & hor dir	$s_v = 5.500$ [in]	$s_h = 3.000$	[in]	
Bolt edge dist in ver & hor dir	$e_v = 1.750$ [in]	$e_h = 1.750$	[in]	
Gross area subject to shear	$A_{gv} = [\ (n_h - 1)\ s_h + e_h\ ]\ t_p \times 2$	= 59.250	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5]\ d_h t_p \times 2$	= 39.750	[in^2]	
Net area subject to tension				
when sheared out by 2 side strips	$A_{nt} = (\ e_v - 0.5\ d_h\ )\ t_p \times 2$	= 3.750	[in^2]	
Block shear strength required	$V_u =$	= **131.5**	[kips]	
Uniform tension stress factor	$U_{bs} = 1.00$			AISC 15th Fig C-J4.2
Bolt shear resistance provided	$R_n = \min(0.6F_u A_{nv}, 0.6F_y A_{gv}) + U_{bs} F_u A_{nt}$	= 1794.0	[kips]	AISC 15th Eq J4-5
Resistance factor-LRFD	$\phi = 0.75$			AISC 15th Eq J4-5
	$\phi R_n =$	= **1345.5**	[kips]	
	ratio = **0.10**	$> V_u$	OK	

Flange Plate - Block Shear - Center Strip		ratio = 131.5 / 1491.8	= **0.09**	**PASS**
Flange force moment arm	$d_m = d_b + t_p$	= 25.400	[in]	
Flange force - tension	$P_u = P_b / 2 + M / d_m$	= **131.5**	[kips]	
Plate Block Shear - Center Strip				
Bolt hole diameter	bolt dia $d_b = {}^7/_8$ [in]	bolt hole dia $d_h = 1$	[in]	AISC 15th B4.3b
Plate thickness	$t_p = 1.500$ [in]			
Plate strength	$F_y = 50.0$ [ksi]	$F_u = 65.0$	[ksi]	
Bolt no in ver & hor dir	$n_v = 2$	$n_h = 7$		
Bolt spacing in ver & hor dir	$s_v = 5.500$ [in]	$s_h = 3.000$	[in]	
Bolt edge dist in ver & hor dir	$e_v = 1.750$ [in]	$e_h = 1.750$	[in]	
Gross area subject to shear	$A_{gv} = [\ (n_h - 1)\ s_h + e_h\ ]\ t_p \times 2$	= 59.250	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5]\ d_h t_p \times 2$	= 39.750	[in^2]	
Net area subject to tension				
when sheared out by center strip	$A_{nt} = (\ n_v - 1)\ (\ s_v - d_h\ )\ t_p$	= 6.750	[in^2]	
Block shear strength required	$V_u =$	= **131.5**	[kips]	
Uniform tension stress factor	$U_{bs} = 1.00$			AISC 15th Fig C-J4.2
Bolt shear resistance provided	$R_n = \min(0.6F_u A_{nv}, 0.6F_y A_{gv}) + U_{bs} F_u A_{nt}$	= 1989.0	[kips]	AISC 15th Eq J4-5
Resistance factor-LRFD	$\phi = 0.75$			AISC 15th Eq J4-5
	$\phi R_n =$	= **1491.8**	[kips]	
	ratio = **0.09**	$> V_u$	OK	

Flange Plate - Tensile Yielding			ratio = 131.5 / 607.5	= **0.22**	**PASS**
Flange force moment arm	$d_m = d_b + t_p$		= 25.400	[in]	
Flange force - tension	$P_u = P_b / 2 + M / d_m$		= **131.5**	[kips]	
Plate Tensile Yielding Check					
Plate size	width b_p = 9.000	[in]	thickness t_p = 1.500	[in]	
Plate yield strength	F_y = 50.0	[ksi]			
Plate gross area in shear	$A_g = b_p t_p$		= 13.500	[in^2]	
Tensile force required	P_u =		= **131.5**	[kips]	
Plate tensile yielding strength	$R_n = F_y A_g$		= 675.0	[kips]	AISC 15th Eq J4-1
Resistance factor-LRFD	ϕ = 0.90				AISC 15th Eq J4-1
	ϕR_n =		= **607.5**	[kips]	
	ratio = **0.22**		> P_u	OK	

Flange Plate - Tensile Rupture			ratio = 131.5 / 511.9	= **0.26**	**PASS**
Flange force moment arm	$d_m = d_b + t_p$		= 25.400	[in]	
Flange force - tension	$P_u = P_b / 2 + M / d_m$		= **131.5**	[kips]	
Plate Tensile Rupture Check					
Bolt hole diameter	bolt dia d_b = 7/8	[in]	bolt hole dia d_h = 1	[in]	AISC 15th B4.3b
Number of bolt	n = 2				
Plate size	width b_p = 9.000	[in]	thickness t_p = 1.500	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate net area in tension	$A_{nt} = (b_p - n\, d_h)\, t_p$		= 10.500	[in^2]	
Tensile force required	P_u =		= **131.5**	[kips]	
Plate tensile rupture strength	$R_n = F_u A_{nt}$		= 682.5	[kips]	AISC 15th Eq J4-2
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J4-2
	ϕR_n =		= **511.9**	[kips]	AISC 15th Eq J4-2
	ratio = **0.26**		> P_u	OK	

Flange Plate - Compression		ratio = 131.5 / 607.5	= **0.22**	**PASS**
Flange force moment arm	$d_m = d_b + t_p$	= 25.400	[in]	
Flange force - compression	$P_u = P_b / 2 + M / d_m$	= **131.5**	[kips]	
Plate Compression Check				
Plate size	width b_p = 9.000 [in]	thickness t_p = 1.500	[in]	
	F_y = 50.0 [ksi]	E = 29000	[ksi]	
Plate gross area in compression	$A_g = b_p t_p$	= 13.500	[in^2]	
Plate radius of gyration	$r = t_p / \sqrt{12}$	= 0.433	[in]	
Plate effective length factor	K =	= 0.65		
Plate unbraced length	L_u =	= 4.500	[in]	
Plate slenderness	$KL/r = 0.65 \times L_u / r$	= 6.75		
Plate compression required	P_u =	= **131.5**	[kips]	
	when $\frac{KL}{r} \leq 25$			AISC 15th J4.4 (a)
Plate compression provided	$R_n = F_y \times A_g$	= 675.0	[kips]	AISC 15th Eq J4-6
Resistance factor-LRFD	ϕ = 0.90			AISC 15th J4.4 (a)
	ϕR_n =	= **607.5**	[kips]	
	ratio = **0.22**	> P_u	OK	

Flange Plate - Slip Critical		ratio = 139.7 / 232.6	= **0.60**	**PASS**
Flange force moment arm	$d_m = d_b$	= 23.900	[in]	
Beam flange force as shear	$V_u = P_b / 2 + M / d_m$	= **139.7**	[kips]	
Bolt dia & bolt pretension	dia d_b = $^7/_8$ [in]	Pretension T_b = 49.0	[kips]	AISC 15th Table J3.1
Surface class	= Class A	Slip coeff. μ = 0.30		AISC 15th J3.8
Min. bolt pretension	D_u = 1.13	Filler factor h_f = 1.00		AISC 15th J3.8
No of bolt row & column	n_r = 2	n_c = 7		
No of slip plane	n_s = 1			
Bolt group eccentricity coefficient	C_{ec} =	= 1.000		
Required shear strength	V_u =	= **139.7**	[kips]	
Slip resistance	$R_n = \mu D_u h_f T_b n_s n_r n_c C_{ec}$	= 232.6	[kips]	AISC 15th Eq J3-4
Resistance factor-LRFD	ϕ = 1.00 for standard size or SSLT hole			AISC 15th J3.8
	ϕR_n =	= **232.6**	[kips]	
	ratio = **0.60**	> V_u	OK	

Web Plate - Bolt Bearing				ratio = 24.5 / 120.0	= **0.20**	**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 = 0.875	[in]	bolt area A_b = 0.601	[in^2]		
Single bolt shear strength	$R_{n\text{-bolt}} = F_{nv} A_b$		= 32.5	[kips]	AISC 15th Eq J3-1	
Bolt Bearing/TearOut Strength on Plate						
Bolt hole diameter	bolt dia d_b = $^7/_8$	[in]	bolt hole dia d_h = $^{15}/_{16}$	[in]	AISC 15th Table J3.3	
Bolt spacing & edge distance	spacing L_s = 3.000	[in]	edge distance L_e = 1.500	[in]		
Plate tensile strength	F_u = 65.0	[ksi]				
Plate thickness	t = 0.375	[in]				
Interior Bolt						
Bolt hole edge clear distance	$L_c = L_s - d_h$		= 2.063	[in]		
Bolt tear out/bearing strength	$R_{n\text{-t\&b-in}} = 1.2\ L_c\ t\ F_u \le 2.4\ d_b\ t\ F_u$				AISC 15th Eq J3-6a	
	= 60.3 ≤ 51.2		= 51.2	[kips]		
Bolt strength at interior	$R_{n\text{-in}} = \min(R_{n\text{-t\&b-in}}, R_{n\text{-bolt}})$		= 32.5	[kips]		
Edge Bolt						
Bolt hole edge clear distance	$L_c = L_e - d_h / 2$		= 1.031	[in]		
Bolt tear out/bearing strength	$R_{n\text{-t\&b-ed}} = 1.2\ L_c\ t\ F_u \le 2.4\ d_b\ t\ F_u$				AISC 15th Eq J3-6a	
	= 30.2 ≤ 51.2		= 30.2	[kips]		
Bolt strength at edge	$R_{n\text{-ed}} = \min(R_{n\text{-t\&b-ed}}, R_{n\text{-bolt}})$		= 30.2	[kips]		
Number of bolt	interior n_{in} = 4		edge n_{ed} = 1			
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-in}} + n_{ed} R_{n\text{-ed}}$		= 160.0	[kips]		
Required shear strength	V_u =		= **24.5**	[kips]		
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 15th J3-10	
	$\phi\ R_n$ =		= **120.0**	[kips]		
	ratio = **0.20**		> V_u	OK		

Web Plate - Shear Yielding				ratio = 24.5 / 168.8	= **0.15**	**PASS**
Plate Shear Yielding Check						
Plate size	width b_p = 15.000	[in]	thickness t_p = 0.375	[in]		
Plate yield strength	F_y = 50.0	[ksi]				
Plate gross area in shear	$A_{gv} = b_p t_p$		= 5.625	[in^2]		
Shear force required	V_u =		= **24.5**	[kips]		
Plate shear yielding strength	$R_n = 0.6\ F_y A_{gv}$		= 168.8	[kips]	AISC 15th Eq J4-3	
Resistance factor-LRFD	ϕ = 1.00				AISC 15th Eq J4-3	
	$\phi\ R_n$ =		= **168.8**	[kips]		
	ratio = **0.15**		> V_u	OK		

Web Plate - Shear Rupture			ratio = 24.5 / 109.7	= **0.22**	**PASS**
Plate Shear Rupture Check					
Bolt hole diameter	bolt dia $d_b = {}^7/_8$	[in]	bolt hole dia d_h = 1	[in]	AISC 15th B4.3b
Number of bolt	n = 5				
Plate size	width b_p = 15.000	[in]	thickness t_p = 0.375	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate net area in shear	$A_{nv} = (b_p - n\, d_h)\, t_p$		= 3.750	[in^2]	
Shear force required	V_u =		= **24.5**	[kips]	
Plate shear rupture strength	$R_n = 0.6\, F_u A_{nv}$		= 146.3	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J4-4
	$\phi\, R_n$ =		= **109.7**	[kips]	
	ratio = **0.22**		> V_u	OK	

Web Plate - Slip Critical			ratio = 24.5 / 66.1	= **0.37**	**PASS**
Bolt dia & bolt pretension	dia $d_b = {}^7/_8$	[in]	Pretension T_b = 39.0	[kips]	AISC 15th Table J3.1
Surface class	= Class A		Slip coeff. μ = 0.30		AISC 15th J3.8
Min. bolt pretension	D_u = 1.13		Filler factor h_f = 1.00		AISC 15th J3.8
No of bolt row & column	n_r = 5		n_c = 1		
No of slip plane	n_s = 1				
Bolt group eccentricity coefficient	C_{ec} =		= 1.000		
Required shear strength	V_u =		= **24.5**	[kips]	
Slip resistance	$R_n = \mu\, D_u h_f T_b n_s n_r n_c C_{ec}$		= 66.1	[kips]	AISC 15th Eq J3-4
Resistance factor-LRFD	ϕ = 1.00	for standard size or SSLT hole			AISC 15th J3.8
	$\phi\, R_n$ =		= **66.1**	[kips]	
	ratio = **0.37**		> V_u	OK	

Web Plate - Block Shear - 1-Side Strip			ratio = 24.5 / 135.3	= **0.18**	**PASS**
Plate Block Shear - Side Strip					
Bolt hole diameter	bolt dia $d_b = {}^7/_8$	[in]	bolt hole dia d_h = 1	[in]	AISC 15th B4.3b
Plate thickness	t_p = 0.375	[in]			
Plate strength	F_y = 50.0	[ksi]	F_u = 65.0	[ksi]	
Bolt no in ver & hor dir	n_v = 1		n_h = 5		
Bolt spacing in hor dir	s_h = 3.000	[in]			
Bolt edge dist in ver & hor dir	e_v = 2.500	[in]	e_h = 1.500	[in]	
Gross area subject to shear	$A_{gv} = [\,(n_h - 1)\, s_h + e_h\,]\, t_p$		= 5.063	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [\,(n_h - 1) + 0.5\,]\, d_h\, t_p$		= 3.375	[in^2]	
Net area subject to tension	$A_{nt} = (e_v - 0.5\, d_h)\, t_p$		= 0.750	[in^2]	
Block shear strength required	V_u =		= **24.5**	[kips]	
Uniform tension stress factor	U_{bs} = 1.00				AISC 15th Fig C-J4.2
Bolt shear resistance provided	$R_n = \min(0.6F_u A_{nv},\, 0.6F_y A_{gv}) + U_{bs} F_u A_{nt}$		= 180.4	[kips]	AISC 15th Eq J4-5
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J4-5
	$\phi\, R_n$ =		= **135.3**	[kips]	
	ratio = **0.18**		> V_u	OK	

Beam Flange - Bolt Shear		ratio = 139.7 / 530.4	= **0.26**	**PASS**
Flange force moment arm	$d_m = d_b$	= 23.900	[in]	
Beam flange force as shear	$V_u = P_b / 2 + M / d_m$	= **139.7**	[kips]	
Bolt shear stress	bolt grade = A490-X	F_{nv} = 84.0	[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.875 [in]	bolt area A_b = 0.601	[in^2]	
Number of bolt carried shear	n_s = 14.0	shear plane m = 1		
Bolt group eccentricity coefficient	C_{ec} =	= 1.000		
Required shear strength	V_u =	= **139.7**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 707.2	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J3-1
	ϕR_n =	= **530.4**	[kips]	
	ratio = **0.26**	> V_u	OK	

Beam Flange - Bolt Bearing		ratio = 143.8 / 530.4	= **0.27**	**PASS**
Flange force moment arm	$d_m = d_b - t_{fb}$	= 23.220	[in]	
Beam flange force as shear	$V_u = P_b / 2 + M / d_m$	= **143.8**	[kips]	
Single Bolt Shear Strength				
Bolt shear stress	bolt grade = A490-X	F_{nv} = 84.0	[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.875 [in]	bolt area A_b = 0.601	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$	= 50.5	[kips]	AISC 15th Eq J3-1
Bolt Bearing/TearOut Strength on Plate				
Bolt hole diameter	bolt dia $d_b = 7/8$ [in]	bolt hole dia $d_h = 15/16$	[in]	AISC 15th Table J3.3
Bolt spacing & edge distance	spacing L_s = 3.000 [in]	edge distance L_e = 3.750	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate thickness	t = 0.680 [in]			
Interior Bolt				
Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.063	[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
	= 109.4 ≤ 92.8	= 92.8	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$	= 50.5	[kips]	
Edge Bolt				
Bolt hole edge clear distance	$L_c = L_e - d_h / 2$	= 3.281	[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
	= 174.0 ≤ 92.8	= 92.8	[kips]	
Bolt strength at edge	$R_{n\text{-}ed} = \min(R_{n\text{-}t\&b\text{-}ed}, R_{n\text{-}bolt})$	= 50.5	[kips]	
Number of bolt	interior n_{in} = 12	edge n_{ed} = 2		
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-}in} + n_{ed} R_{n\text{-}ed}$	= 707.2	[kips]	
Required shear strength	V_u =	= **143.8**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th J3-10
	ϕR_n =	= **530.4**	[kips]	
	ratio = **0.27**	> V_u	OK	

Beam Flange - Block Shear - 2 Side Strips			ratio = 143.8 / 689.2	= **0.21**	**PASS**
Flange force moment arm	$d_m = d_b - t_{fb}$		= 23.220	[in]	
Beam flange force - tension	$V_u = P_b / 2 + M / d_m$		= **143.8**	[kips]	

Plate Block Shear - 2 Side Strips

Bolt hole diameter	bolt dia $d_b = {}^7/_8$	[in]	bolt hole dia $d_h = 1$	[in]	AISC 15th B4.3b
Plate thickness	$t_p = 0.680$	[in]			
Plate strength	$F_y = 50.0$	[ksi]	$F_u = 65.0$	[ksi]	
Bolt no in ver & hor dir	$n_v = 2$		$n_h = 7$		
Bolt spacing in ver & hor dir	$s_v = 5.500$	[in]	$s_h = 3.000$	[in]	
Bolt edge dist in ver & hor dir	$e_v = 1.745$	[in]	$e_h = 3.750$	[in]	

Gross area subject to shear	$A_{gv} = [(n_h - 1) s_h + e_h] t_p \times 2$	= 29.580	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5] d_h t_p \times 2$	= 20.740	[in^2]	
Net area subject to tension when sheared out by 2 side strips	$A_{nt} = (e_v - 0.5 d_h) t_p \times 2$	= 1.693	[in^2]	

Block shear strength required	$V_u =$	= **143.8**	[kips]	
Uniform tension stress factor	$U_{bs} = 1.00$			AISC 15th Fig C-J4.2
Bolt shear resistance provided	$R_n = \min (0.6F_u A_{nv}, 0.6F_y A_{gv}) + U_{bs} F_u A_{nt}$	= 918.9	[kips]	AISC 15th Eq J4-5
Resistance factor-LRFD	$\phi = 0.75$			AISC 15th Eq J4-5
	$\phi R_n =$	= **689.2**	[kips]	
	ratio = **0.21**	$> V_u$	OK	

Beam Web - Bolt Shear			ratio = 24.5 / 121.8	= **0.20**	**PASS**
Bolt shear stress	bolt grade = A325-N		$F_{nv} = 54.0$	[ksi]	AISC 15th Table J3.2
	bolt dia $d_b = 0.875$	[in]	bolt area $A_b = 0.601$	[in^2]	
Number of bolt carried shear	$n_s = 5.0$		shear plane m = 1		
Bolt group eccentricity coefficient	$C_{ec} =$		= 1.000		
Required shear strength	$V_u =$		= **24.5**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$		= 162.4	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	$\phi = 0.75$				AISC 15th Eq J3-1
	$\phi R_n =$		= **121.8**	[kips]	
	ratio = **0.20**		$> V_u$	OK	

Beam Web - Bolt Bearing			ratio = 24.5 / 121.8	= **0.20**	**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 = 0.875	[in]	bolt area A_b = 0.601	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$		= 32.5	[kips]	AISC 15th Eq J3-1

Bolt Bearing/TearOut Strength on Plate

Bolt hole diameter	bolt dia $d_b = {}^7/_8$	[in]	bolt hole dia $d_h = {}^{15}/_{16}$	[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 = 0.440	[in]			

Interior Bolt

Bolt hole edge clear distance	$L_c = L_s - d_h$		= 2.063	[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
	= 70.8 ≤ 60.1		= 60.1	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$		= 32.5	[kips]	

Number of bolt	interior n_{in} = 5				
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-}in}$		= 162.4	[kips]	
Required shear strength	V_u =		= **24.5**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 15th J3-10
	ϕR_n =		= **121.8**	[kips]	
	ratio = **0.20**		> V_u	OK	

Beam Web - Shear Yielding			ratio = 24.5 / 315.5	= **0.08**	**PASS**

Plate Shear Yielding Check

Plate size	width b_p = 23.900	[in]	thickness t_p = 0.440	[in]	
Plate yield strength	F_y = 50.0	[ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$		= 10.516	[in^2]	
Shear force required	V_u =		= **24.5**	[kips]	
Plate shear yielding strength	$R_n = 0.6\ F_y A_{gv}$		= 315.5	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00				AISC 15th Eq J4-3
	ϕR_n =		= **315.5**	[kips]	
	ratio = **0.08**		> V_u	OK	

Beam Web - Shear Rupture			ratio = 24.5 / 243.2	= **0.10**	**PASS**

Plate Shear Rupture Check

Bolt hole diameter	bolt dia $d_b = {}^7/_8$	[in]	bolt hole dia d_h = 1	[in]	AISC 15th B4.3b
Number of bolt	n = 5				
Plate size	width b_p = 23.900	[in]	thickness t_p = 0.440	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate net area in shear	$A_{nv} = (b_p - n\ d_h)\ t_p$		= 8.316	[in^2]	
Shear force required	V_u =		= **24.5**	[kips]	
Plate shear rupture strength	$R_n = 0.6\ F_u A_{nv}$		= 324.3	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J4-4
	ϕR_n =		= **243.2**	[kips]	
	ratio = **0.10**		> V_u	OK	

Beam Flange With Holes - Beam Flexural Rupture					**Not Applicable**
Beam sect W24X76	$b_f = 8.990$	[in]	$t_f = 0.680$	[in]	
	$S_x = 176.00$	[in^3]			
	$F_y = 50.0$	[ksi]	$F_u = 65.0$	[ksi]	
Gross area of tension flange	$A_{fg} = b_f t_f$		$= 6.113$	[in^2]	
Bolt hole diameter	bolt dia $d_b = {}^7/_8$	[in]	bolt hole dia $d_h = 1$	[in]	AISC 15th B4.3b
Number of bolt	$n = 2$				
Net area of tension flange	$A_{fn} = (b_f - n\, d_h)\, t_f$		$= 4.753$	[in^2]	
	$Y_t = 1.0$				
	When $F_u A_{fn} \geq Y_t F_y A_{fg}$ this limit state doesn't apply				AISC 15th Eq F13.1 (a)

Column Flange Bending			ratio = 131.5 / 482.7	= **0.27**	**PASS**
Flange force moment arm	$d_m = d_b + t_p$		$= 25.400$	[in]	
Flange force required - tension	$P_{uf\_t} = P_u / 2 - M_u / d_m$		= **131.5**	[kips]	
Column flange thickness	$t_{fc} = 1.310$	[in]	yield $F_{yc} = 50.0$	[ksi]	
Top column condition	it's not a top column case		$C_t = 1.0$		AISC 15th J10.1
Column flange tensile resistance	$R_n = C_t\, 6.25\, F_{yc}\, t_{fc}^2$		$= 536.3$	[kips]	AISC 15th Eq J10-1
Resistance factor-LRFD	$\phi = 0.90$				AISC 15th J10.1
	$\phi R_n =$		= **482.7**	[kips]	
	ratio = **0.27**		$> P_{uf\_t}$	OK	

Column Web Yielding			ratio = 131.5 / 1011.1	= **0.13**	**PASS**
Column web thickness	$t_{wc} = 0.830$	[in]	yield $F_y = 50.0$	[ksi]	
Doubler plate thickness	$t_{dp} = 0.500$	[in]	yield $F_{ydp} = 50.0$	[ksi]	
Equivalent web thickness when considering doubler plate	$t_{w\text{-}eq} = t_{wc} + t_{dp} \times F_{ydp} / F_y \times 2$ sides		$= 1.830$	[in]	
Flange force moment arm	$d_m = d_b + t_p$		$= 25.400$	[in]	
Flange force in demand	$P_{uf} = \max(P_{uf\_t} , P_{uf\_c})$		= **131.5**	[kips]	AISC DG13 Eq 4.2-1
Column section	$d_c = 15.200$	[in]	$t_{fc} = 1.310$	[in]	
	$t_{wc} = 0.830$	[in]	$k_c = 1.910$	[in]	
Column yield strength	$F_{yc} = 50.0$	[ksi]			
Top column condition	it's not a top column case				AISC 15th J10.2 (a)
Flange plate fillet weld size	$w = 0.000$	[in]	flange plate $t_p = 1.500$	[in]	
Length of bearing	$N = t_p + 2\, w$		$= 1.500$	[in]	AISC DG4 Eq 3.24
Column web yielding strength	$R_n = (5\, k_c + N)\, F_{yc}\, t_{w\text{-}eq}$		$= 1011.1$	[kips]	AISC 15th Eq J10-2
Resistance factor-LRFD	$\phi = 1.00$				AISC 15th J10.2
	$\phi R_n =$		= **1011.1**	[kips]	
	ratio = **0.13**		$> P_{uf}$	OK	

Column Web Buckling		ratio = 131.5 / 1878.3	= **0.07**	**PASS**
Flange force moment arm	$d_m = d_b + t_p$	= 25.400 [in]		
Flange force required in compression	$P_{uf\_c} = P_u / 2 - M_u / d_m$	= **131.5** [kips]		
Column section	d_c = 15.200 [in]	t_{fc} = 1.310 [in]		
	t_{wc} = 0.830 [in]	k_c = 1.910 [in]		
	$h = d_c - 2\ k_c$	= 11.380 [in]		
Column yield strength	F_{yc} = 50.0 [ksi]	E_c = 29000 [ksi]		
Top column condition	it's not a top column case	C_t = 1.0	AISC 15th J10.5	
Column web buckling strength	$R_{n1} = \dfrac{C_t\, 24\, t_{wc}^3 \sqrt{E_c F_{yc}}}{h}$	= 1452.1 [kips]	AISC 15th Eq J10-8	
Doubler plate thickness	t_{dp} = 0.500 [in]	yield F_{ydp} = 50.0 [ksi]		
Doubler plate buckling strength	$R_{n2} = \dfrac{2\ C_t\, 24\, t_{dp}^3 \sqrt{E_c F_{ydp}}}{h}$	= 634.9 [kips]	AISC 15th Eq J10-8	
Total buckling strength	$R_n = R_{n1} + R_{n2}$	= 2087.0 [kips]		
Resistance factor-LRFD	ϕ = 0.90		AISC 15th J10.5	
	$\phi\ R_n$ =	= **1878.3** [kips]		
	ratio = **0.07**	> P_{uf_c}	OK	

Column Web Crippling		ratio = 131.5 / 1344.2	= **0.10**	**PASS**
Flange force moment arm	$d_m = d_b + t_p$	= 25.400 [in]		
Flange force required in compression	$P_{uf\_c} = P_u / 2 - M_u / d_m$	= **131.5** [kips]		
Column section	d_c = 15.200 [in]	t_{fc} = 1.310 [in]		
	t_{wc} = 0.830 [in]	k_c = 1.910 [in]		
Column yield strength	F_{yc} = 50.0 [ksi]	E_c = 29000 [ksi]		
Flange plate fillet weld size	w = 0.000 [in]	flange plate t_p = 1.500 [in]		
Length of bearing	$l_b = t_p + 2\ w$	= 1.500 [in]		
Distance from top of column to top of beam flange	$d_{end\text{-}flg}$ =	= 4.000 [in]		
Top column condition	it's not a top column case, use Eq J10-4		AISC 15th J10.3 (a)	
Column web crippling strength	$R_{n1} = 0.8\ t_{wc}^2\ [1+3\dfrac{l_b}{d_c}(\dfrac{t_{wc}}{t_{fc}})^{1.5}]\text{x}\ (\dfrac{E_c F_{yc} t_{fc}}{t_{wc}})^{0.5}$	= 958.2 [kips]	AISC 15th Eq J10-4	
Doubler plate thickness	t_{dp} = 0.500 [in]	yield F_{ydp} = 50.0 [ksi]		
Doubler plate crippling strength	$R_{n2} = 0.8\ t_{dp}^2\ [1+3\dfrac{l_b}{d_c}(\dfrac{t_{dp}}{t_{fc}})^{1.5}]\text{x}\ (\dfrac{E_c F_{ydp} t_{fc}}{t_{dp}})^{0.5}$	= 834.1 [kips]	AISC 15th Eq J10-4	
Total crippling strength	$R_n = R_{n1} + R_{n2}$	= 1792.3 [kips]		
Resistance factor-LRFD	ϕ = 0.75		AISC 15th J10.3	
	$\phi\ R_n$ =	= **1344.2** [kips]		
	ratio = **0.10**	> P_{uf_c}	OK	

Column Panel Zone Shear		ratio = 104.0 / 421.8	= **0.25**	**PASS**
Panel zone shear force	$V_p = P_{f\text{-}TR} - P_{f\text{-}TL} - V_s$	= **104.0**	[kips]	
Column W14X176	d_c = 15.200 [in]	b_{cf} = 15.700	[in]	
	t_{cf} = 1.310 [in]	t_{cw} = 0.830	[in]	
	A_c = 51.800 [in^2]	F_{cy} = 50.0	[ksi]	
Beam W24X76	d_b = 23.900 [in]	t_{bf} = 0.680	[in]	
Flange plate thickness	t_p = from user input	= 1.500	[in]	
Moment arm between flange plates	$d_m = d + t_p$	= 26.900	[in]	AISC 358-16 Eq 7.6-7
Column axial compression	P_r = from user input	= 249.0	[kips]	
Column axial yield strength	$P_y = F_{cy} A_c$	= 2590.0	[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	$R_n = 0.6 F_{cy} d_c t_{cw} \left(1 + \frac{3 b_{cf} t_{cf}^2}{d_m d_c t_{cw}}\right)$	= 468.6	[kips]	AISC 15th Eq J10-11
Resistance factor-LRFD	ϕ = 0.90			AISC 15th J10.6
	ϕR_n =	= **421.8**	[kips]	
	ratio = **0.25**	> V_p	OK	

Seismic Material & Geometry Limitations — PASS

Check Max Beam Yield Stress

Condition : beam max material $F_y \le F_{y\_max}$ = 50 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 $F_y \le F_{y\_max}$ = 65 ksi — AISC 341-16 A3.1

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

$\le F_{y\_max}$ OK

Check Max Beam Depth

Condition : beam depth shall be limited to W36 maximum — AISC 358-16 7.3.1 (2)

Beam W24X76 depth d = = **23.900** [in]

≤ W36 OK

Check Max Beam Weight

Condition : beam weight shall be limited to 150 lb/ft maximum — AISC 358-16 7.3.1 (3)

Beam W24X76 weight wt = = **75** [lb/ft]

≤ 150 OK

Check Max Beam Flange Thickness

Condition : beam flange thickness shall be limited to 1 in maximum — AISC 358-16 7.3.1 (4)

Beam W24X76 flange thickness t_f = = **0.680** [in]

$\le t_f$ OK

Check Min Beam Clear Span to Depth Ratio

Condition : beam clear span-to-depth ratio ≥ 9 — AISC 358-16 7.3.1 (5)

Beam W24X76 clear span L = 345.0 [in] depth d = 23.900 [in]

Beam clear span to depth ratio L/d = L / d = **14.44**

≥ 9 OK

Check Max Column Depth

Condition : column depth shall be limited to W36 maximum — AISC 358-16 7.3.2 (3)

Column W14X176 depth d = = **15.200** [in]

≤ W36 OK

Check Flange Plate Bolt Hole Type

Condition : Flange plate bolt hole type shall be STD

Flange plate bolt hole type = STD + STD

OK AISC 358-16 7.5.4

Check Web Plate Bolt Hole Type

Condition : Web plate bolt hole type shall be STD or SSLT

Web plate bolt hole type = STD + STD

OK AISC 341-16 D2.2 (3)

Seismic Geometry SMF Column Flange Thickness — PASS

Check SMF Column Condition Without Continuity Plate

SMF column flange thcikness shall satisfy AISC 341-16 Eq E3-8 , otherwise continuity plate is required — AISC 341-16 E3.6f.1

Siffener palte is provided as continuity plate, so this check is not required.

OK — AISC 341-16 E3.6f.1

Seismic Geometry SMF Column Stiffener Plate Thickness — PASS

Check SMF Column Min Stiffener Plate Thickness

SMF column minimum stiffener plate thcikness shall satisfy requirements in AISC 341-16 E3.6f.2(b) — AISC 341-16 E3.6f.2(b)

Right beam W24X76 flange thickness	t_{fb1} =	= 0.680	[in]		
Left beam W24X76 flange thickness	t_{fb2} =	= 0.680	[in]		
Thicker beam flange thickness	t_{fb} = max (t_{fb1} , t_{fb2})	= 0.680	[in]		
Min stiffener plate thickness	t_{min} = t_{fb} x 0.75	= **0.510**	[in]		
Stiffener plate thickness	t_s = from user input	= **1.125**	[in]		
		≥ t_{min}		OK	AISC 341-16 E3.6f.2(b)

Seismic Width to Thickness Ratio — PASS

Check Beam Flange

Beam half flange width	$b = 4.495$ [in]	$t_f = 0.680$	[in]	
Beam yield strength	$F_y = 50.0$ [ksi]	$E = 29000$	[ksi]	
Width-to-thickness ratio-allow	$\lambda_{hd} = 0.30 \sqrt{E / F_y}$	$=$ **7.22**		
Width-to-thickness ratio-actual	$b/t_f = b / t_f$	$=$ **6.61**		
		$\leq \lambda_{hd}$	OK	AISC 341-16 Table D1.1

Check Beam Web

Clear dist between beam flange	$h_b = 22.540$ [in]	$t_w = 0.440$	[in]	
Beam yield strength	$F_{yb} = 50.0$ [ksi]	$E = 29000$	[ksi]	
Resistance factor for compression	$\phi_c = 0.90$	$A_b = 22.400$	[in^2]	
Nominal axial yield strength	$P_{yb} = F_{yb} A_b$	$= 1120.0$	[kips]	
Beam axial compression	$P_{ub} =$ from user input	$= 0.0$	[kips]	
Ratio of req'd to available strength	$C_{ab} = P_{ub} / (\phi_c \times P_{yb})$	$= 0.00$		
Width-to-thickness ratio-allow	$\lambda_{hd} =$	$=$ **59.00**		
Width-to-thickness ratio-actual	$h/t_w = h_b / t_w$	$=$ **51.23**		
		$\leq \lambda_{hd}$	OK	AISC 341-16 Table D1.1

Check Column Flange

Column half flange width	$b = 7.850$ [in]	$t_f = 1.310$	[in]	
Column yield strength	$F_y = 50.0$ [ksi]	$E = 29000$	[ksi]	
Width-to-thickness ratio-allow	$\lambda_{hd} = 0.30 \sqrt{E / F_y}$	$=$ **7.22**		
Width-to-thickness ratio-actual	$b/t_f = b / t_f$	$=$ **5.99**		
		$\leq \lambda_{hd}$	OK	AISC 341-16 Table D1.1

Check Column Web

Clear dist between column flange	$h_c = 12.580$ [in]	$t_w = 0.830$	[in]	
Column yield strength	$F_{yc} = 50.0$ [ksi]	$E = 29000$	[ksi]	
Resistance factor for compression	$\phi_c = 0.90$	$A_c = 51.800$	[in^2]	
Nominal axial yield strength	$P_{yc} = F_{yc} A_c$	$= 2590.0$	[kips]	
Column axial compression	$P_{uc} =$ from user input	$= 249.0$	[kips]	
Ratio of req'd to available strength	$C_{ac} = P_{uc} / (\phi_c \times P_{yc})$	$= 0.11$		
Width-to-thickness ratio-allow	$\lambda_{hd} =$	$=$ **53.14**		
Width-to-thickness ratio-actual	$h/t_w = h_c / t_w$	$=$ **15.16**		
		$\leq \lambda_{hd}$	OK	AISC 341-16 Table D1.1

Seismic Weld Limitation				**PASS**
Check Beam Web Weld Material				
Condition : Web weld electrode must be E70 or E80				
Beam web weld electrode material	=	E70XX		
			OK	AISC 341-16 A3.4b
Check Beam Flange Weld Material				
Condition : Flange weld electrode must be E70 or E80				
Beam flange weld electrode material	=	E70XX		
			OK	AISC 341-16 A3.4b
Check Beam Flange Weld Type				
Condition : Flange weld type shall be CJP weld				
Beam flange to end plate weld	=	CJP		
			OK	AISC 358-16 7.5.2
Check Column Stiff PL to Column Flange Weld Type				
Condition : Continuity plate (column stiff) shall be welded to column flanges using CJP weld				
Column stiff to column flange weld	=	CJP		
			OK	AISC 341-16 E3.6f (3)

Seismic Flange Plate Bolt Limitation — FAIL

Check Length of Bolt Group

Condition : Length of bolt group L ≤ beam depth — AISC 358-16 7.5.4

Beam depth	$d = 23.900$ [in]				
Bolt row & spacing	$n = 7$	$s = 3.000$	[in]		
Length of bolt group	$L = (n-1)\,s$	$= 18.000$	[in]		
		$\le d$		OK	AISC 358-16 7.5.4

Check Check Max Bolt Diameter

Condition : Max bolt diameter $d_b \le 1.125$ inch

Bolt diameter	=	0.875		
			OK	AISC 358-16 7.5.4

Check Max Bolt Dia to Prevent Beam Flange Tensile Rupture

Ratio of expected/min yield & tensile strength	$R_y = 1.1$	$R_t = 1.2$			
Beam sect W24X76	$b_f = 8.990$ [in]				
Beam sect strength	$F_y = 50.0$ [ksi]	$F_u = 65.0$	[ksi]		
Flange plate nax bolt diameter	$d_{max} = \frac{b_f}{2}\left(1 - \frac{R_y F_y}{R_t F_u}\right) - \frac{1}{8}$	$= 1.200$	[in]		AISC 358-16 Eq 7.6-2
Flange plate bolt diameter	$d_b =$	$= 0.875$	[in]		
		$\le d_{max}$		OK	

Check Bolt Grade Material

Condition : Bolt grade must be A490-X or A490M-X

Bolt grade	=	A490-X		
			OK	AISC 358-16 7.5.4

Check Bolt Clear Edge Dist at Beam Flange

Bolt and bolt hole dia	$d_b = 0.875$ [in]	$d_h = 0.938$	[in]		
Bolt edge dist at beam flange	$L_e = 3.750$ [in]				
Bolt clear edge dist	$L_{ce} = L_e - 0.5\,d_h$	$= 3.281$	[in]		
		$\ge 2d_b$		OK	AISC 358-16 7.6 Step 5

Check Bolt Clear Edge Dist at Flange Plate

Bolt and bolt hole dia	$d_b = 0.875$ [in]	$d_h = 0.938$	[in]		
Bolt edge dist at flange plate	$L_e = 1.750$ [in]				
Bolt clear edge dist	$L_{ce} = L_e - 0.5\,d_h$	$= 1.281$	[in]		
		$< 2d_b$		NG	AISC 358-16 7.6 Step 5

Check Bolt Clear Dist at Bolt Spacing

Bolt and bolt hole dia	$d_b = 0.875$ [in]	$d_h = 0.938$	[in]		
Bolt spacing at flange plate	$L_s = 3.000$ [in]				
Bolt clear dist spacing	$L_{cs} = L_s - d_h$	$= 2.063$	[in]		
		$\ge 2d_b$		OK	AISC 358-16 7.6 Step 5

Seismic Column Beam Moment Ratio — PASS

Seismic column beam moment ratio check as per AISC 341-16 E3.4a — AISC 341-16 E3.4a

Column W14X176	A_c = 51.800	[in^2]	Z_c = 320.00	[in^3]	
	d_c = 15.200	[in]	F_{yc} = 50.0	[ksi]	
Column axial force	P_{uc} = from user input		= 249.0	[kips]	
LRFD-ASD force adjustment factor	α = for LRFD		= 1.0		AISC 341-16 D1.2a (b)
Half of column height above/below beam	h_t = 75.0	[in]	h_b = 84.0	[in]	
Beam W24X76 depth	d_b = 23.900	[in]			
Top column flexural strength	$M_t = Z_c (F_{yc} - \alpha P_{uc} / A_c) \dfrac{h_t}{h_t - 0.5 d_b}$		= 1433.6	[kip-ft]	AISC 341-16 Eq E3-2
Bot column flexural strength	$M_b = Z_c (F_{yc} - \alpha P_{uc} / A_c) \dfrac{h_b}{h_b - 0.5 d_b}$		= 1405.0	[kip-ft]	
Total column flexural strength	$M_{pc} = M_t + M_b$		= **2838.6**	[kip-ft]	AISC 341-16 Eq E3-2
Beam W24X76	Z_b = 200.00	[in^3]	F_{yb} = 50.0	[ksi]	
Beam ratio of expected/min yield stress	R_{yb} = 1.10				AISC 341-16 Table A3.1

see Seismic Moment and Beam Flange Force Calc section on how below seismic moments are derived

Seismic M at right column face	M_R =	= 1239.2	[kip-ft]	
Seismic M at left column face	M_L =	= 1185.3	[kip-ft]	
Seismic V at right column face	V_R =	= 100.9	[kips]	
Seismic V at left column face	V_L =	= 67.8	[kips]	
Expected flexural strength of beam	$M_{pb} = M_R + M_L + (V_R + V_L) \times 0.5\, d_c$	= **2531.4**	[kip-ft]	AISC 341-16 Eq E3-3
Seismic column beam moment ratio	$= M_{pc} / M_{pb}$	= 1.12		AISC 341-16 Eq E3-1
		≥ 1.0	OK	

Seismic Flange PL Thickness AISC 358-10 7.6 Step 10 — PASS

Check Flange Plate Material

Condition : Flange plate material must be A36 or A572

Flange plate material = A572 Gr.50

OK — AISC 358-16 7.5.1

Check Min Flange Plate Thickness

Flange plate force	F_{pr} = See calc in Seismic Moment and Beam Flange Force Calc		= 585.5	[kips]	
Flange plate	F_y = 50.0	[ksi]	b_{fp} = 9.000	[in]	
Resistance factor-ductile limit state	ϕ_d = 1.00				AISC 358-16 2.4.1
Min flange plate thickness	$t_{min} = \dfrac{F_{pr}}{\phi_d F_y b_{fp}}$		= 1.301	[in]	AISC 358-16 Eq 7.6-9
Flange plate thickness	t =		= 1.500	[in]	
			≥ t_{min}	OK	

Seismic Flange PL Tensile Yielding			ratio = 585.5 / 675.0	= **0.87**	**PASS**
Flange force moment arm	$d_m = d_b + t_p$		= 25.400	[in]	
Flange force - tension	$P_u = P_b / 2 + M / d_m$		= **585.5**	[kips]	
Plate Tensile Yielding Check					
Plate size	width b_p = 9.000	[in]	thickness t_p = 1.500	[in]	
Plate yield strength	F_y = 50.0	[ksi]			
Plate gross area in shear	$A_g = b_p t_p$		= 13.500	[in^2]	
Tensile force required	P_u =		= **585.5**	[kips]	
Plate tensile yielding strength	$R_n = F_y A_g$		= 675.0	[kips]	AISC 15th Eq J4-1
Resistance factor-LRFD	ϕ_d = 1.00				AISC 358-16 2.4.1
	ϕR_n =		= **675.0**	[kips]	
	ratio = **0.87**		> P_u	OK	

Seismic Flange PL Tensile Rupture AISC 358-10 7.6 - Step 11			ratio = 585.5 / 614.3	= **0.95**	**PASS**
Flange force moment arm	$d_m = d_b + t_p$		= 25.400	[in]	
Flange force - tension	$P_u = P_b / 2 + M / d_m$		= **585.5**	[kips]	
Plate Tensile Rupture Check					
Bolt hole diameter	bolt dia d_b = $^7/_8$	[in]	bolt hole dia d_h = 1	[in]	AISC 15th B4.3b
Number of bolt	n = 2				
Plate size	width b_p = 9.000	[in]	thickness t_p = 1.500	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate net area in tension	$A_{nt} = (b_p - n\, d_h)\, t_p$		= 10.500	[in^2]	
Tensile force required	P_u =		= **585.5**	[kips]	
Plate tensile rupture strength	$R_n = F_u A_{nt}$		= 682.5	[kips]	AISC 15th Eq J4-2
Resistance factor-LRFD	ϕ_n = 0.90				AISC 358-16 2.4.1
	ϕR_n =		= **614.3**	[kips]	AISC 15th Eq J4-2
	ratio = **0.95**		> P_u	OK	

Seismic Flange PL Block Shear-2 Side Strips				ratio = 585.5 / 1937.5	= **0.30**	**PASS**
Flange force moment arm	$d_m = d_b + t_p$			= 25.400	[in]	
Flange force - tension	$P_u = P_b / 2 + M / d_m$			= **585.5**	[kips]	
Plate Block Shear - 2 Side Strips						
Bolt hole diameter	bolt dia $d_b = {}^7/_8$	[in]	bolt hole dia $d_h = 1$		[in]	AISC 15th B4.3b
Plate thickness	$t_p = 1.500$	[in]				
Plate strength	$F_y = 50.0$	[ksi]	$F_u = 65.0$		[ksi]	
Bolt no in ver & hor dir	$n_v = 2$		$n_h = 7$			
Bolt spacing in ver & hor dir	$s_v = 5.500$	[in]	$s_h = 3.000$		[in]	
Bolt edge dist in ver & hor dir	$e_v = 1.750$	[in]	$e_h = 1.750$		[in]	
Gross area subject to shear	$A_{gv} = [\ (n_h - 1)\ s_h + e_h\ ]\ t_p \times 2$			= 59.250	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5]\ d_h\ t_p \times 2$			= 39.750	[in^2]	
Net area subject to tension						
when sheared out by 2 side strips	$A_{nt} = (\ e_v - 0.5\ d_h\ )\ t_p \times 2$			= 3.750	[in^2]	
Block shear strength required	$V_u =$			= **585.5**	[kips]	
Uniform tension stress factor	$U_{bs} = 1.00$					AISC 15th Fig C-J4.2
Ratio of expected Fy/Fu to specified min Fy/Fu	$R_y = 1.10$		$R_t = 1.20$			AISC 341-16 Table A3.1
Bolt shear resistance provided	$R_n = \min\ (0.6\ R_t F_u A_{nv}\ ,\ 0.6\ R_y F_y A_{gv}\ ) + U_{bs} R_t F_u A_{nt}$			= 2152.8	[kips]	AISC 15th Eq J4-5
Resistance factor-LRFD	$\phi_n = 0.90$					AISC 358-16 2.4.1
	$\phi\ R_n =$			= **1937.5**	[kips]	
	ratio = **0.30**			> V_u	OK	

Seismic Flange PL Block Shear-Center Strip ratio = 585.5 / 2148.1 = **0.27** **PASS**

Flange force moment arm	$d_m = d_b + t_p$	= 25.400	[in]	
Flange force - tension	$P_u = P_b / 2 + M / d_m$	= **585.5**	[kips]	

Plate Block Shear - Center Strip

Bolt hole diameter	bolt dia $d_b = 7/8$	[in]	bolt hole dia $d_h = 1$	[in]	AISC 15th B4.3b
Plate thickness	$t_p = 1.500$	[in]			
Plate strength	$F_y = 50.0$	[ksi]	$F_u = 65.0$	[ksi]	
Bolt no in ver & hor dir	$n_v = 2$		$n_h = 7$		
Bolt spacing in ver & hor dir	$s_v = 5.500$	[in]	$s_h = 3.000$	[in]	
Bolt edge dist in ver & hor dir	$e_v = 1.750$	[in]	$e_h = 1.750$	[in]	

Gross area subject to shear	$A_{gv} = [(n_h - 1) s_h + e_h] t_p x 2$	= 59.250	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1)+ 0.5] d_h t_p x2$	= 39.750	[in^2]	
Net area subject to tension when sheared out by center strip	$A_{nt} = (n_v - 1) (s_v - d_h) t_p$	= 6.750	[in^2]	
Block shear strength required	$V_u =$	= **585.5**	[kips]	
Uniform tension stress factor	$U_{bs} = 1.00$			AISC 15th Fig C-J4.2
Ratio of expected Fy/Fu to specified min Fy/Fu	$R_y = 1.10$	$R_t = 1.20$		AISC 341-16 Table A3.1
Bolt shear resistance provided	$R_n = \min (0.6 R_t F_u A_{nv} , 0.6 R_y F_y A_{gv}) + U_{bs} R_t F_u A_{nt}$	= 2386.8	[kips]	AISC 15th Eq J4-5
Resistance factor-LRFD	$\phi_n = 0.90$			AISC 358-16 2.4.1
	$\phi R_n =$	= **2148.1**	[kips]	
	ratio = **0.27**	$> V_u$	OK	

Seismic Flange Plate - Bolt Shear ratio = 585.5 / 636.4 = **0.92** **PASS**

Beam flange force as shear	$V_u =$ See calc in Seismic Moment and Beam Flange Force Calc		= **585.5**	[kips]	
Bolt shear stress	bolt grade = A490-X		$F_{nv} = 84.0$	[ksi]	AISC 15th Table J3.2
	bolt dia $d_b = 0.875$	[in]	bolt area $A_b = 0.601$	[in^2]	
Number of bolt carried shear	$n_s = 14.0$		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		= **585.5**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$		= 707.2	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	$\phi_n = 0.90$				AISC 358-16 2.4.1
	$\phi_n R_n =$		= **636.4**	[kips]	
	ratio = **0.92**		$> V_u$	OK	

Seismic Flange Plate - Bolt Bearing		ratio = 585.5 / 636.4	= **0.92**	**PASS**	
Beam flange force as shear	V_u = See calc in Seismic Moment and Beam Flange Force Calc		= **585.5**	[kips]	

Single Bolt Shear Strength

Bolt shear stress	bolt grade = A490-X		F_{nv} = 84.0	[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.875	[in]	bolt area A_b = 0.601	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$		= 50.5	[kips]	AISC 15th Eq J3-1

Bolt Bearing/TearOut Strength on Plate

Bolt hole diameter	bolt dia d_b = $^7/_8$	[in]	bolt hole dia d_h = $^{15}/_{16}$	[in]	AISC 15th Table J3.3
Bolt spacing & edge distance	spacing L_s = 3.000	[in]	edge distance L_e = 1.750	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate thickness	t = 1.500	[in]			

Interior Bolt

Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.063	[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
	= 241.3 ≤ 204.8	= 204.8	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$	= 50.5	[kips]	

Edge Bolt

Bolt hole edge clear distance	$L_c = L_e - d_h / 2$	= 1.281	[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
	= 149.9 ≤ 204.8	= 149.9	[kips]	
Bolt strength at edge	$R_{n\text{-}ed} = \min(R_{n\text{-}t\&b\text{-}ed}, R_{n\text{-}bolt})$	= 50.5	[kips]	

Number of bolt	interior n_{in} = 12	edge n_{ed} = 2		
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-}in} + n_{ed} R_{n\text{-}ed}$	= 707.2	[kips]	
Required shear strength	V_u = see Seismic Moment and Beam Flange Force Calc	= **585.5**	[kips]	
Bolt resistance factor-LRFD	ϕ_n = 0.90			AISC 358-16 2.4.1
	$\phi_n R_n$ =	= **636.4**	[kips]	
	ratio = **0.92**	> V_u	OK	

Seismic Beam Flange Block Shear-2 Side Strip - Step 12 ratio = 585.5 / 909.7 = **0.64** PASS

Flange force moment arm	$d_m = d_b - t_{fb}$		= 23.220	[in]	
Beam flange force - tension	$V_u = P_b / 2 + M / d_m$		= **585.5**	[kips]	

Plate Block Shear - 2 Side Strips

Bolt hole diameter	bolt dia $d_b = 7/8$ [in]	bolt hole dia $d_h = 1$		[in]	AISC 15th B4.3b
Plate thickness	$t_p = 0.680$ [in]				
Plate strength	$F_y = 50.0$ [ksi]	$F_u = 65.0$		[ksi]	
Bolt no in ver & hor dir	$n_v = 2$	$n_h = 7$			
Bolt spacing in ver & hor dir	$s_v = 5.500$ [in]	$s_h = 3.000$		[in]	
Bolt edge dist in ver & hor dir	$e_v = 1.745$ [in]	$e_h = 3.750$		[in]	

Gross area subject to shear	$A_{gv} = [(n_h - 1) s_h + e_h] t_p \times 2$	= 29.580	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1)+ 0.5] d_h t_p \times 2$	= 20.740	[in^2]	
Net area subject to tension when sheared out by 2 side strips	$A_{nt} = (e_v - 0.5 d_h) t_p \times 2$	= 1.693	[in^2]	

Block shear strength required	$V_u =$	= **585.5**	[kips]	
Uniform tension stress factor	$U_{bs} = 1.00$			AISC 15th Fig C-J4.2
Ratio of expected Fy/Fu to specified min Fy/Fu	$R_y = 1.10$; $R_t = 1.10$			AISC 341-16 Table A3.1
Bolt shear resistance provided	$R_n = \min (0.6 R_t F_u A_{nv} , 0.6 R_y F_y A_{gv}) + U_{bs} R_t F_u A_{nt}$	= 1010.8	[kips]	AISC 15th Eq J4-5
Resistance factor-LRFD	$\phi_n = 0.90$			AISC 358-16 2.4.1
	$\phi R_n =$	= **909.7**	[kips]	
	ratio = **0.64**	$> V_u$	OK	

Seismic Flange Plate - Compression - Step 13 ratio = 585.5 / 607.5 = **0.96** PASS

Flange force moment arm	$d_m = d_b + t_p$	= 25.400	[in]	
Flange force - compression	$P_u = P_b / 2 + M / d_m$	= **585.5**	[kips]	

Plate Compression Check

Plate size	width $b_p = 9.000$ [in]	thickness $t_p = 1.500$	[in]	
	$F_y = 50.0$ [ksi]	E = 29000	[ksi]	
Plate gross area in compression	$A_g = b_p t_p$	= 13.500	[in^2]	

Plate radius of gyration	$r = t_p / \sqrt{12}$	= 0.433	[in]	
Plate effective length factor	K =	= 0.65		
Plate unbraced length	$L_u =$	= 4.500	[in]	
Plate slenderness	$KL/r = 0.65 \times L_u / r$	= 6.75		
Plate compression required	$P_u =$	= **585.5**	[kips]	

	when $\frac{KL}{r} \leq 25$			AISC 15th J4.4 (a)
Plate compression provided	$R_n = F_y \times A_g$	= 675.0	[kips]	AISC 15th Eq J4-6
Resistance factor-LRFD	$\phi_n = 0.90$			AISC 358-16 2.4.1
	$\phi R_n =$	= **607.5**	[kips]	
	ratio = **0.96**	$> P_u$	OK	

Seismic Beam Section Shear Strength - Step 14		ratio = 100.9 / 315.5	= **0.32**		**PASS**
W Shape Beam Shear Yielding Check					
W sect W24X76	d = 23.900 [in]	t_w = 0.440		[in]	
	F_y = 50.0 [ksi]				
Beam to column shear	V_u = from calc above Seismic Moment and Beam Flange Force Calc		= **100.9**	[kips]	
Beam shear strength	$R_n = 0.6\ F_y\ d\ t_w$		= 315.5	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00				AISC 15th Eq J4-3
	$\phi\ R_n$ =		= **315.5**	[kips]	
	ratio = **0.32**		> V_u	OK	

Seismic Beam Flange - Bolt Shear		ratio = 585.5 / 636.4	= **0.92**		**PASS**
Flange force moment arm	$d_m = d_b$		= 23.900	[in]	
Beam flange force as shear	$V_u = P_b / 2 + M / d_m$		= **585.5**	[kips]	
Bolt shear stress	bolt grade = A490-X	F_{nv} = 84.0		[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.875 [in]	bolt area A_b = 0.601		[in^2]	
Number of bolt carried shear	n_s = 14.0	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		= **585.5**	[kips]	
Bolt shear strength	$R_n = F_{nv}\ A_b\ n_s\ m\ C_{ec}$		= 707.2	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	ϕ_n = 0.90				AISC 358-16 2.4.1
	$\phi_n\ R_n$ =		= **636.4**	[kips]	
	ratio = **0.92**		> V_u	OK	

Seismic Beam Flange - Bolt Bearing

ratio = 585.5 / 636.4 = **0.92** **PASS**

Flange force moment arm	$d_m = d_b - t_{fb}$	= 23.220	[in]	
Beam flange force as shear	$V_u = P_b / 2 + M / d_m$	= **585.5**	[kips]	

Single Bolt Shear Strength

Bolt shear stress	bolt grade = A490-X		F_{nv} = 84.0	[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.875	[in]	bolt area A_b = 0.601	[in^2]	
Single bolt shear strength	$R_{n\text{-bolt}} = F_{nv} A_b$		= 50.5	[kips]	AISC 15th Eq J3-1

Bolt Bearing/TearOut Strength on Plate

Bolt hole diameter	bolt dia $d_b = {}^{7}/_{8}$	[in]	bolt hole dia $d_h = {}^{15}/_{16}$	[in]	AISC 15th Table J3.3
Bolt spacing & edge distance	spacing L_s = 3.000	[in]	edge distance L_e = 3.750	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate thickness	t = 0.680	[in]			

Interior Bolt

Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.063	[in]	
Bolt tear out/bearing strength	$R_{n\text{-t\&b-in}} = 1.2\ L_c\, t\, F_u \le 2.4\ d_b\, t\, F_u$			AISC 15th Eq J3-6a
	= 109.4 ≤ 92.8	= 92.8	[kips]	
Bolt strength at interior	$R_{n\text{-in}} = \min(R_{n\text{-t\&b-in}}, R_{n\text{-bolt}})$	= 50.5	[kips]	

Edge Bolt

Bolt hole edge clear distance	$L_c = L_e - d_h / 2$	= 3.281	[in]	
Bolt tear out/bearing strength	$R_{n\text{-t\&b-ed}} = 1.2\ L_c\, t\, F_u \le 2.4\ d_b\, t\, F_u$			AISC 15th Eq J3-6a
	= 174.0 ≤ 92.8	= 92.8	[kips]	
Bolt strength at edge	$R_{n\text{-ed}} = \min(R_{n\text{-t\&b-ed}}, R_{n\text{-bolt}})$	= 50.5	[kips]	

Number of bolt	interior n_{in} = 12	edge n_{ed} = 2		
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-in}} + n_{ed} R_{n\text{-ed}}$	= 707.2	[kips]	
Required shear strength	V_u = see Seismic Moment and Beam Flange Force Calc	= **585.5**	[kips]	
Bolt resistance factor-LRFD	ϕ_n = 0.90			AISC 358-16 2.4.1
	$\phi_n R_n$ =	= **636.4**	[kips]	
	ratio = **0.92**	> V_u	OK	

Seismic Beam Web - Bolt Shear

ratio = 100.9 / 146.1 = **0.69** **PASS**

Bolt shear stress	bolt grade = A325-N		F_{nv} = 54.0	[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.875	[in]	bolt area A_b = 0.601	[in^2]	
Number of bolt carried shear	n_s = 5.0		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		= **100.9**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m\, C_{ec}$		= 162.4	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	ϕ_n = 0.90				AISC 358-16 2.4.1
	$\phi_n R_n$ =		= **146.1**	[kips]	
	ratio = **0.69**		> V_u	OK	

Seismic Beam Web - Bolt Bearing

ratio = 100.9 / 146.1 = **0.69** **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 = 0.875 [in]	bolt area A_b = 0.601 [in^2]		
Single bolt shear strength	$R_{n\text{-bolt}} = F_{nv} A_b$	= 32.5	[kips]	AISC 15th Eq J3-1

Bolt Bearing/TearOut Strength on Plate

Bolt hole diameter	bolt dia d_b = $^7/_8$ [in]	bolt hole dia d_h = $^{15}/_{16}$	[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 = 0.440 [in]			

Interior Bolt

Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.063	[in]	
Bolt tear out/bearing strength	$R_{n\text{-t\&b-in}} = 1.2\ L_c\ t\ F_u \leq 2.4\ d_b\ t\ m\ F_u$			AISC 15th Eq J3-6a
	= 70.8 ≤ 60.1	= 60.1	[kips]	
Bolt strength at interior	$R_{n\text{-in}} = \min(R_{n\text{-t\&b-in}}, R_{n\text{-bolt}})$	= 32.5	[kips]	

Number of bolt	interior n_{in} = 5			
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-in}}$	= 162.4	[kips]	
Required shear strength	V_u = see Seismic Moment and Beam Flange Force Calc	= **100.9**	[kips]	
Bolt resistance factor-LRFD	ϕ_n = 0.90			AISC 358-16 2.4.1
	$\phi_n R_n$ =	= **146.1**	[kips]	
	ratio = **0.69**	> V_u	OK	

Seismic Beam Web - Shear Yielding

ratio = 100.9 / 315.5 = **0.32** **PASS**

Plate Shear Yielding Check

Plate size	width b_p = 23.900 [in]	thickness t_p = 0.440	[in]	
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$	= 10.516	[in^2]	
Shear force required	V_u =	= **100.9**	[kips]	
Plate shear yielding strength	$R_n = 0.6\ F_y A_{gv}$	= 315.5	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	ϕR_n =	= **315.5**	[kips]	
	ratio = **0.32**	> V_u	OK	

Seismic Beam Web - Shear Rupture		ratio = 100.9 / 243.2	= **0.41**	**PASS**

Plate Shear Rupture Check

Bolt hole diameter	bolt dia d_b = $^7/_8$	[in]	bolt hole dia d_h = 1	[in]	AISC 15th B4.3b
Number of bolt	n = 5				
Plate size	width b_p = 23.900	[in]	thickness t_p = 0.440	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate net area in shear	$A_{nv} = (b_p - n\, d_h)\, t_p$		= 8.316	[in^2]	
Shear force required	V_u =		= **100.9**	[kips]	
Plate shear rupture strength	$R_n = 0.6\, F_u\, A_{nv}$		= 324.3	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J4-4
	$\phi\, R_n$ =		= **243.2**	[kips]	
	ratio = **0.41**		> V_u	OK	

Seismic Web Plate - Bolt Bearing		ratio = 100.9 / 144.0	= **0.70**	**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 = 0.875	[in]	bolt area A_b = 0.601	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv}\, A_b$		= 32.5	[kips]	AISC 15th Eq J3-1

Bolt Bearing/TearOut Strength on Plate

Bolt hole diameter	bolt dia d_b = $^7/_8$	[in]	bolt hole dia d_h = $^{15}/_{16}$	[in]	AISC 15th Table J3.3
Bolt spacing & edge distance	spacing L_s = 3.000	[in]	edge distance L_e = 1.500	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate thickness	t = 0.375	[in]			

Interior Bolt

Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.063	[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
	$= 60.3 \le 51.2$	= 51.2	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$	= 32.5	[kips]	

Edge Bolt

Bolt hole edge clear distance	$L_c = L_e - d_h / 2$	= 1.031	[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
	$= 30.2 \le 51.2$	= 30.2	[kips]	
Bolt strength at edge	$R_{n\text{-}ed} = \min(R_{n\text{-}t\&b\text{-}ed}, R_{n\text{-}bolt})$	= 30.2	[kips]	

Number of bolt	interior n_{in} = 4	edge n_{ed} = 1			
Bolt bearing strength for all bolts	$R_n = n_{in}\, R_{n\text{-}in} + n_{ed}\, R_{n\text{-}ed}$		= 160.0	[kips]	
Required shear strength	V_u = see Seismic Moment and Beam Flange Force Calc		= **100.9**	[kips]	
Bolt resistance factor-LRFD	ϕ_n = 0.90				AISC 358-16 2.4.1
	$\phi_n\, R_n$ =		= **144.0**	[kips]	
	ratio = **0.70**		> V_u	OK	

Seismic Web Plate - Shear Yielding — ratio = 100.9 / 168.8 = **0.60** **PASS**

Plate Shear Yielding Check

Plate size	width b_p = 15.000	[in]	thickness t_p = 0.375	[in]	
Plate yield strength	F_y = 50.0	[ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$		= 5.625	[in^2]	
Shear force required	V_u =		= **100.9**	[kips]	
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$		= 168.8	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00				AISC 15th Eq J4-3
	ϕR_n =		= **168.8**	[kips]	
	ratio = **0.60**		> V_u	OK	

Seismic Web Plate - Shear Rupture — ratio = 100.9 / 109.7 = **0.92** **PASS**

Plate Shear Rupture Check

Bolt hole diameter	bolt dia $d_b = {}^7/_8$	[in]	bolt hole dia d_h = 1	[in]	AISC 15th B4.3b
Number of bolt	n = 5				
Plate size	width b_p = 15.000	[in]	thickness t_p = 0.375	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate net area in shear	$A_{nv} = (b_p - n\, d_h) t_p$		= 3.750	[in^2]	
Shear force required	V_u =		= **100.9**	[kips]	
Plate shear rupture strength	$R_n = 0.6 F_u A_{nv}$		= 146.3	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J4-4
	ϕR_n =		= **109.7**	[kips]	
	ratio = **0.92**		> V_u	OK	

Seismic Web Plate - Block Shear - 1-Side Strip — ratio = 100.9 / 194.8 = **0.52** **PASS**

Plate Block Shear - Side Strip

Bolt hole diameter	bolt dia $d_b = {}^7/_8$	[in]	bolt hole dia d_h = 1	[in]	AISC 15th B4.3b
Plate thickness	t_p = 0.375	[in]			
Plate strength	F_y = 50.0	[ksi]	F_u = 65.0	[ksi]	
Bolt no in ver & hor dir	n_v = 1		n_h = 5		
Bolt spacing in hor dir	s_h = 3.000	[in]			
Bolt edge dist in ver & hor dir	e_v = 2.500	[in]	e_h = 1.500	[in]	
Gross area subject to shear	$A_{gv} = [(n_h - 1) s_h + e_h] t_p$		= 5.063	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5] d_h t_p$		= 3.375	[in^2]	
Net area subject to tension	$A_{nt} = (e_v - 0.5 d_h) t_p$		= 0.750	[in^2]	
Block shear strength required	V_u =		= **100.9**	[kips]	
Uniform tension stress factor	U_{bs} = 1.00				AISC 15th Fig C-J4.2
Ratio of expected Fy/Fu to specified min Fy/Fu	R_y = 1.10		R_t = 1.20		AISC 341-16 Table A3.1
Bolt shear resistance provided	$R_n = \min (0.6 R_t F_u A_{nv} , 0.6 R_y F_y A_{gv}) + U_{bs} R_t F_u A_{nt}$		= 216.5	[kips]	AISC 15th Eq J4-5
Resistance factor-LRFD	ϕ_n = 0.90				AISC 358-16 2.4.1
	ϕR_n =		= **194.8**	[kips]	
	ratio = **0.52**		> V_u	OK	

Seismic Web Plate Weld Strength ratio = 100.9 / 167.0 = **0.60** **PASS**

Shear force in demand	V_u =	= **100.9**	[kips]
Fillet weld length - double fillet	L =	= 15.000	[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 - web plate	thickness t = 0.375 [in]	tensile F_u = 65.0	[ksi]	
Base metal - web plate is in shear, shear rupture as per AISC 15th Eq J4-4 is checked				AISC 15th J2.4
Base metal shear rupture	$R_{n\text{-}b} = 0.6\ F_u\ t$	= 14.625	[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	$R_n = \min \left(R_{n\text{-}w} , R_{n\text{-}b} \times \frac{0.90}{0.75} \right)$	= **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	V_u =	= **100.9**	[kips]
Fillet weld length - double fillet	L =	= 15.000	[in]
Shear resistance provided	$\phi F_n = \phi R_n \times L$	= **167.0**	[kips]
	ratio = **0.60**	> V_u	OK

Seismic Column Panel Zone Thickness **PASS**

Check Column Web Thickness

Condition : column web thickness $t_{wc} \geq (d_z + w_z) /90$ — AISC 341-16 Eq E3-7

Beam sect W24X76	d_b = 23.900 [in]	t_{fb} = 0.680	[in]	
Beam clear dist between flanges	$d_z = d_b - 2\ t_{fb}$	= 22.540	[in]	
Column sect W24X76	d_c = 15.200 [in]	t_{fc} = 1.310	[in]	
Column width of panel zone between flanges	$w_z = d_c - 2\ t_{fc}$	= 12.580	[in]	
Column web thickness required	$t_{min} = (d_z + w_z) /90$	= **0.390**	[in]	AISC 341-16 Eq E3-7
Doubler plate No & thickness	No = 2	t_{db} = 0.500	[in]	
Column web thickness	t_{wc} = 0.830 [in]			

When the doubler plates are plug welded to the column web, the total panel zone thickness is the sum of doubler plates and column web — AISC 341-16 E3.6e (2)

Total panel zone thickness	$t = t_{wc} + 2 \times t_{db}$	= **1.830**	[in]	
		≥ t_{min}	OK	AISC 341-16 E3.6e (2)

Seismic Column Flange Bending		585.5 / 482.7		**N/A**
Flange force moment arm	$d_m = d_b + t_p$	= 25.400	[in]	
Flange force required - tension	$P_{uf\_t} = P_u \ / \ 2 - M_u \ / \ d_m$	= **585.5**	[kips]	
Column flange thickness	t_{fc} = 1.310 [in]	yield F_{yc} = 50.0	[ksi]	
Top column condition	it's not a top column case	C_t = 1.0		AISC 15th J10.1
Column flange tensile resistance	$R_n = C_t \, 6.25 \, F_{yc} \, t_{fc}^2$	= 536.3	[kips]	AISC 15th Eq J10-1
Resistance factor-LRFD	ϕ = 0.90			AISC 15th J10.1
	$\phi \, R_n$ =	= **482.7**	[kips]	
Unbalanced force to be resisted by transverse stiffeners	$R_s = P_{uf\_t} - \phi \, R_n$	= **102.8**	[kips]	

Seismic Column Web Yielding		ratio = 585.5 / 1011.1	= **0.58**	**PASS**
Column web thickness	t_{wc} = 0.830 [in]	yield F_y = 50.0	[ksi]	
Doubler plate thickness	t_{dp} = 0.500 [in]	yield F_{ydp} = 50.0	[ksi]	
Equivalent web thickness when considering doubler plate	$t_{w\text{-}eq} = t_{wc} + t_{dp} \times F_{ydp} / F_y \times 2$ sides	= 1.830	[in]	
Flange force moment arm	$d_m = d_b + t_p$	= 25.400	[in]	
Seismic flange force	P_{uf_c} = see Seismic Moment and Beam Flange Force Calc	= **585.5**	[kips]	
Column section	d_c = 15.200 [in]	t_{fc} = 1.310	[in]	
	t_{wc} = 0.830 [in]	k_c = 1.910	[in]	
Column yield strength	F_{yc} = 50.0 [ksi]			
Top column condition	it's not a top column case			AISC 15th J10.2 (a)
Flange plate fillet weld size	w = 0.000 [in]	flange plate t_p = 1.500	[in]	
Length of bearing	$N = t_p + 2\,w$	= 1.500	[in]	AISC DG4 Eq 3.24
Column web yielding strength	$R_n = (5\,k_c + N)\,F_{yc}\,t_{w\text{-}eq}$	= 1011.1	[kips]	AISC 15th Eq J10-2
Resistance factor-LRFD	ϕ = 1.00			AISC 341-16 E3.6e (1)
	$\phi \, R_n$ =	= **1011.1**	[kips]	
	ratio = **0.58**	> P_{uf}	OK	

Seismic Column Web Buckling		ratio = 585.5 / 2087.0	= **0.28**	**PASS**
Flange force moment arm	$d_m = d_b + t_p$	= 25.400	[in]	
Seismic flange force	P_{uf_c} = see Seismic Moment and Beam Flange Force Calc	= **585.5**	[kips]	
Column section	d_c = 15.200 [in]	t_{fc} = 1.310	[in]	
	t_{wc} = 0.830 [in]	k_c = 1.910	[in]	
	$h = d_c - 2\,k_c$	= 11.380	[in]	
Column yield strength	F_{yc} = 50.0 [ksi]	E_c = 29000	[ksi]	
Top column condition	it's not a top column case	C_t = 1.0		AISC 15th J10.5
Column web buckling strength	$R_{n1} = \dfrac{C_t\,24\,t_{wc}^3\sqrt{E_c F_{yc}}}{h}$	= 1452.1	[kips]	AISC 15th Eq J10-8
Doubler plate thickness	t_{dp} = 0.500 [in]	yield F_{ydp} = 50.0	[ksi]	
Doubler plate buckling strength	$R_{n2} = \dfrac{2\,C_t\,24\,t_{dp}^3\sqrt{E_c F_{ydp}}}{h}$	= 634.9	[kips]	AISC 15th Eq J10-8
Total buckling strength	$R_n = R_{n1} + R_{n2}$	= 2087.0	[kips]	
Resistance factor-LRFD	ϕ = 1.00			AISC 341-16 E3.6e (1)
	$\phi\,R_n$ =	= **2087.0**	[kips]	
	ratio = **0.28**	> P_{uf_c}	OK	

Seismic Column Web Crippling		ratio = 585.5 / 1792.3	= **0.33**	**PASS**
Flange force moment arm	$d_m = d_b + t_p$	= 25.400	[in]	
Seismic flange force	P_{uf_c} = see Seismic Moment and Beam Flange Force Calc	= **585.5**	[kips]	
Column section	d_c = 15.200 [in]	t_{fc} = 1.310	[in]	
	t_{wc} = 0.830 [in]	k_c = 1.910	[in]	
Column yield strength	F_{yc} = 50.0 [ksi]	E_c = 29000	[ksi]	
Flange plate fillet weld size	w = 0.000 [in]	flange plate t_p = 1.500	[in]	
Length of bearing	$l_b = t_p + 2\,w$	= 1.500	[in]	
Distance from top of column to top of beam flange	$d_{end\text{-}flg}$ =	= 4.000	[in]	
Top column condition	it's not a top column case, use Eq J10-4			AISC 15th J10.3 (a)
Column web crippling strength	$R_{n1} = 0.8\, t_{wc}^2 \left[1+3\frac{l_b}{d_c}\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right] \times \left(\frac{E_c F_{yc} t_{fc}}{t_{wc}}\right)^{0.5}$	= 958.2	[kips]	AISC 15th Eq J10-4
Doubler plate thickness	t_{dp} = 0.500 [in]	yield F_{ydp} = 50.0	[ksi]	
Doubler plate crippling strength	$R_{n2} = 0.8\, t_{dp}^2 \left[1+3\frac{l_b}{d_c}\left(\frac{t_{dp}}{t_{fc}}\right)^{1.5}\right] \times \left(\frac{E_c F_{ydp} t_{fc}}{t_{dp}}\right)^{0.5}$	= 834.1	[kips]	AISC 15th Eq J10-4
Total crippling strength	$R_n = R_{n1} + R_{n2}$	= 1792.3	[kips]	
Resistance factor-LRFD	ϕ = 1.00			AISC 341-16 E3.6e (1)
	$\phi\, R_n$ =	= **1792.3**	[kips]	
	ratio = **0.33**	> P_{uf_c}	OK	

Seismic Column Panel Zone Shear		836.9 / 468.6		**N/A**
Seismic panel zone shear in demand	V_p = see Seismic Moment and Beam Flange Force Calc	= **836.9**	[kips]	
Column W14X176	d_c = 15.200 [in]	b_{cf} = 15.700	[in]	
	t_{cf} = 1.310 [in]	t_{cw} = 0.830	[in]	
	A_c = 51.800 [in^2]	F_{cy} = 50.0	[ksi]	
Beam W24X76	d_b = 23.900 [in]	t_{bf} = 0.680	[in]	
Flange plate thickness	t_p = from user input	= 1.500	[in]	
Moment arm between flange plates	$d_m = d + t_p$	= 26.900	[in]	AISC 358-16 Eq 7.6-7
Column axial compression	P_r = from user input	= 249.0	[kips]	
Column axial yield strength	$P_y = F_{cy} A_c$	= 2590.0	[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	$R_n = 0.6\ F_{cy} d_c t_{cw} (1 + \frac{3\ b_{cf} t_{cf}^2}{d_m d_c t_{cw}})$	= 468.6	[kips]	AISC 15th Eq J10-11
Resistance factor-LRFD	ϕ_v = 1.00			AISC 341-16 E3.6e (1)
	$\phi_v R_n$ =	= **468.6**	[kips]	
Unbalanced force to be resisted by doubler plate	$V_{dp} = R_p - \phi R_n$	= **368.3**	[kips]	

Seismic Stiffener Geometry Restriction				**PASS**
Stiffener plate width	b_s = 6.000 [in]	depth d_s = 12.580	[in]	
Stiffener plate thickness	t_s = 1.125 [in]			
Column flange thickness	t_{fc} = 1.310 [in]	column depth d_c = 15.200	[in]	
Beam flange thickness	t_{fb} = 0.680 [in]			
Min Stiffener Plate Thickness				
Min stiffener plate thickness	t_{smin} = max (t_{fb} / 2 , b_s / 16)	= **0.375**	[in]	AISC 15th J10.8 (2)
Stiffener plate thickness	t_s =	= **1.125**	[in]	
		≥ t_{smin}	OK	
Min Stiffener Plate Depth				
Min stiffener plate depth	d_{smin} = (d_c - 2 t_{fc}) / 2	= **6.290**	[in]	AISC 15th J10.8 (3)
Stiffener plate depth	d_s =	= **12.580**	[in]	
		≥ d_{smin}	OK	

Seismic Stiffener Yield at Column Flange		ratio = 102.8 / 455.6	= **0.23**	**PASS**
Stiffener plate width	b_s = 5.500 [in]	thickness t_s = 1.125	[in]	
Stiffener plate corner clip	clip =	= 1.000	[in]	AISC 15th Page 8-18
Stiffener plate yield strength	F_y =	= 50.0	[ksi]	
Stiffener plate cross sect area	A_{st} = (b_s - clip) t_s	= 5.063	[in^2]	
Trans stiffener strength required	R_s =	= **102.8**	[kips]	
Trans stiffener strength provided	$R_n = F_y \times 2 \times A_{st}$	= 506.3	[kips]	AISC 15th Eq J4-1
Bolt resistance factor-LRFD	ϕ = 0.90			AISC 15th Eq J4-1
	ϕ R_n =	= **455.6**	[kips]	
	ratio = **0.23**	> R_s	OK	

Seismic Stiffener Shear at Column Web		ratio = 102.8 / 714.2	= **0.14**		**PASS**
Stiffener plate depth	d_s = 12.580 [in]	thickness t_s	= 1.125	[in]	
Stiffener plate corner clip	clip =		= 1.000	[in]	AISC 15th Page 8-18
Stiffener plate yield strength	F_y =		= 50.0	[ksi]	
Stiffener plate cross sect area	$A_{gv} = (d_s - 2 \times clip) t_s$		= 11.903	[in^2]	
Trans stiffener strength required	R_s =		= **102.8**	[kips]	
Trans stiffener strength provided	$R_n = 2 \times 0.6 \times F_y \times A_{gv}$		= 714.2	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00				AISC 15th Eq J4-3
	ϕR_n =		= **714.2**	[kips]	
	ratio = **0.14**		$> R_s$	OK	

Seismic Stiffener to Column Web Fillet Weld Limitation				**PASS**
Min Fillet Weld Size				
Thinner part joined thickness	t =	= 0.830	[in]	
Min fillet weld size allowed	w_{min} =	= **0.313**	[in]	AISC 15th 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 15th J2.2b
Min fillet weld length	$L = d_c - 2 \times t_{fc} - 2 \times clip$	= **10.580**	[in]	
		$\geq L_{min}$	OK	

Seismic Stiffener Weld Strength at Column Web — ratio = 102.8 / 294.5 = **0.35** PASS

Stiffener to Column Web Weld Length Calc

Column section	d_c = 15.200 [in]	t_{fc} = 1.310 [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	= **21.160**	[in]	
Trans stiffener strength required	R_s =	= **102.8**	[kips]	

Fillet Weld Strength Check

Fillet weld leg size	w = 5/16 [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	$R_{n\text{-}w} = 0.6 (C_1 \times 70 \text{ ksi}) 0.707\ w\ n\ C_2$	= 18.559	[kip/in]	AISC 15th Eq 8-1
Base metal - stiffener plate	thickness t = 1.125 [in]	tensile F_u = 65.0 [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	$R_{n\text{-}b} = 0.6\ F_u\ t$	= 43.875	[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	$R_n = \min\left(R_{n\text{-}w} , R_{n\text{-}b} \times \frac{0.90}{0.75} \right)$	= **18.559**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq 8-1
	ϕR_n =	= **13.919**	[kip/in]	
Shear resistance required	R_s =	= **102.8**	[kips]	
Fillet weld length - double fillet	L =	= 21.160	[in]	
Shear resistance provided	$\phi F_n = \phi R_n \times L$	= **294.5**	[kips]	
	ratio = **0.35**	> R_s	OK	

Seismic Doubler Plate Thickness — PASS

Check Doubler Plate Thickness

Condition : Doubler plate thickness $t_{db} \geq (d_z + w_z) /90$ — AISC 341-16 Eq E3-7

Beam sect W24X76	d_b = 23.900 [in]	t_{fb} = 0.680 [in]		
Beam clear dist between flanges	$d_z = d_b - 2\ t_{fb}$	= 22.540	[in]	
Column sect W24X76	d_c = 15.200 [in]	t_{fc} = 1.310 [in]		
Column width of panel zone between flanges	$w_z = d_c - 2\ t_{fc}$	= 12.580	[in]	
Doubler plate thickness required	$t = (d_z + w_z) /90$	= **0.390**	[in]	AISC 341-16 Eq E3-7
Doubler plate thickness	t_{db} =	= **0.500**	[in]	
		≥ t	OK	AISC 341-16 E3.6e (2)

Seismic Doubler Plate Shear Yield		ratio = 368.3 / 456.0	= **0.81**	**PASS**
Doubler plate thickness	t_{dp} = 0.500 [in]	yield strength F_y = 50.0	[ksi]	
Number of doubler	N_{dbl} = 2 for two side			
Column depth	d_c = 15.200 [in]			
Area of web	$A_w = t_{dp} \times d_c$	= 7.600	[in^2]	AISC DG13 Eq 4.4-1
Shear coefficient	C_v = 1.0			AISC 15th Eq G2-2
Doubler plate shear in demand	R_{dp} = unbalanced force from column panel zone shear calc	= **368.3**	[kips]	
Doubler plate shear strength	$R_n = N_{dbl}\, 0.6\, F_y A_w C_v$	= 456.0	[kips]	AISC 15th Eq G2-1
Resistance factor-LRFD	ϕ = 1.00			AISC 15th G2.1 (a)
	ϕR_n =	= **456.0**	[kips]	
	ratio = **0.81**	> R_{dp}	OK	

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

Result Summary	geometries & weld limitations = **FAIL**	limit states max ratio = **0.92**	**PASS**

Geometry Restriction Checks - Flange Plate — PASS

Min Bolt Edge Distance - Flange Plate

Bolt diameter	d_b =	= 0.875	[in]	
Min edge distance allowed	$L_{e\text{-}min}$ =	= **1.125**	[in]	AISC 15th Table J3.4
Min edge distance in Flange Plate	L_e =	= **1.745**	[in]	
		$\geq L_{e\text{-}min}$	OK	

Min Bolt Spacing - Flange Plate

Bolt diameter	d_b =	= 0.875	[in]	
Min bolt spacing allowed	$L_{s\text{-}min} = 2.667\ d_b$	= **2.333**	[in]	AISC 15th J3.3
Min Bolt spacing in Flange Plate	L_s =	= **3.000**	[in]	
		$\geq L_{s\text{-}min}$	OK	

Geometry Restriction Checks - Web Plate — PASS

Min Bolt Edge Distance - Web Plate

Bolt diameter	d_b =	= 0.875	[in]	
Min edge distance allowed	$L_{e\text{-}min}$ =	= **1.125**	[in]	AISC 15th Table J3.4
Min edge distance in Web Plate	L_e =	= **1.500**	[in]	
		$\geq L_{e\text{-}min}$	OK	

Min Bolt Spacing - Web Plate

Bolt diameter	d_b =	= 0.875	[in]	
Min bolt spacing allowed	$L_{s\text{-}min} = 2.667\ d_b$	= **2.333**	[in]	AISC 15th J3.3
Min Bolt spacing in Web Plate	L_s =	= **3.000**	[in]	
		$\geq L_{s\text{-}min}$	OK	

Fillet Weld Limitation Checks - Web Plate — PASS

Min Fillet Weld Size

Thinner part joined thickness	t =	= 0.375	[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 =	= **15.000**	[in]	
		$\geq L_{min}$	OK	

Web Plate Weld Strength		ratio = 24.5 / 164.5	= **0.15**	**PASS**	
Shear force in demand	V_u =		= **24.5**	[kips]	
Fillet weld length - double fillet	L =		= 15.000	[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 - web plate	thickness t = 0.375 [in]	tensile F_u = 65.0		[ksi]	
Base metal - web plate is in shear, shear rupture as per AISC 15th Eq J4-4 is checked					AISC 15th J2.4
Base metal shear rupture	$R_{n\text{-}b} = 0.6\ F_u\ t$		= 14.625	[kip/in]	AISC 15th Eq J4-4
Double fillet linear shear strength	$R_n = \min (R_{n\text{-}w} , R_{n\text{-}b})$		= **14.625**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq 8-1
	$\phi\ R_n$ =		= **10.969**	[kip/in]	
Shear resistance required	V_u =		= **24.5**	[kips]	
Fillet weld length - double fillet	L =		= 15.000	[in]	
Shear resistance provided	$\phi\ F_n = \phi\ R_n \times L$		= **164.5**	[kips]	
	ratio = **0.15**		> V_u	OK	

Flange Plate - Bolt Shear		ratio = 139.7 / 530.4	= **0.26**	**PASS**	
Flange force moment arm	$d_m = d_b$		= 23.900	[in]	
Beam flange force as shear	$V_u = P_b / 2 + M / d_m$		= **139.7**	[kips]	
Bolt shear stress	bolt grade = A490-X	F_{nv} = 84.0		[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.875 [in]	bolt area A_b = 0.601		[in^2]	
Number of bolt carried shear	n_s = 14.0	shear plane m = 1			
Bolt group eccentricity coefficient	C_{ec} =		= 1.000		
Required shear strength	V_u =		= **139.7**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$		= 707.2	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J3-1
	$\phi\ R_n$ =		= **530.4**	[kips]	
	ratio = **0.26**		> V_u	OK	

Flange Plate - Bolt Bearing		ratio = 131.5 / 530.4 = **0.25**	**PASS**	
Flange force moment arm	$d_m = d_b + t_p$	= 25.400	[in]	
Beam flange force as shear	$V_u = P_b / 2 + M / d_m$	= **131.5**	[kips]	

Single Bolt Shear Strength

Bolt shear stress	bolt grade = A490-X	F_{nv} = 84.0	[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.875 [in]	bolt area A_b = 0.601	[in^2]	
Single bolt shear strength	$R_{n\text{-bolt}} = F_{nv} A_b$	= 50.5	[kips]	AISC 15th Eq J3-1

Bolt Bearing/TearOut Strength on Plate

Bolt hole diameter	bolt dia $d_b = ^7/_8$ [in]	bolt hole dia $d_h = ^{15}/_{16}$	[in]	AISC 15th Table J3.3
Bolt spacing & edge distance	spacing L_s = 3.000 [in]	edge distance L_e = 1.750	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate thickness	t = 1.500 [in]			

Interior Bolt

Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.063	[in]	
Bolt tear out/bearing strength	$R_{n\text{-t\&b-in}} = 1.2\ L_c\ t\ F_u \le 2.4\ d_b\ t\ F_u$			AISC 15th Eq J3-6a
	= 241.3 ≤ 204.8	= 204.8	[kips]	
Bolt strength at interior	$R_{n\text{-in}} = \min(R_{n\text{-t\&b-in}}, R_{n\text{-bolt}})$	= 50.5	[kips]	

Edge Bolt

Bolt hole edge clear distance	$L_c = L_e - d_h / 2$	= 1.281	[in]	
Bolt tear out/bearing strength	$R_{n\text{-t\&b-ed}} = 1.2\ L_c\ t\ F_u \le 2.4\ d_b\ t\ F_u$			AISC 15th Eq J3-6a
	= 149.9 ≤ 204.8	= 149.9	[kips]	
Bolt strength at edge	$R_{n\text{-ed}} = \min(R_{n\text{-t\&b-ed}}, R_{n\text{-bolt}})$	= 50.5	[kips]	

Number of bolt	interior n_{in} = 12	edge n_{ed} = 2		
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-in}} + n_{ed} R_{n\text{-ed}}$	= 707.2	[kips]	
Required shear strength	V_u =	= **131.5**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th J3-10
	ϕR_n =	= **530.4**	[kips]	
	ratio = **0.25**	> V_u	OK	

Flange Plate - Block Shear - 2 Side Strips ratio = 131.5 / 1345.5 = **0.10** **PASS**

Flange force moment arm	$d_m = d_b + t_p$		= 25.400	[in]	
Flange force - tension	$P_u = P_b / 2 + M / d_m$		= **131.5**	[kips]	
Plate Block Shear - 2 Side Strips					
Bolt hole diameter	bolt dia $d_b = 7/8$ [in]		bolt hole dia $d_h = 1$	[in]	AISC 15^{th} B4.3b
Plate thickness	$t_p = 1.500$	[in]			
Plate strength	$F_y = 50.0$	[ksi]	$F_u = 65.0$	[ksi]	
Bolt no in ver & hor dir	$n_v = 2$		$n_h = 7$		
Bolt spacing in ver & hor dir	$s_v = 5.500$	[in]	$s_h = 3.000$	[in]	
Bolt edge dist in ver & hor dir	$e_v = 1.750$	[in]	$e_h = 1.750$	[in]	
Gross area subject to shear	$A_{gv} = [\ (n_h - 1)\ s_h + e_h\ ]\ t_p \times 2$		= 59.250	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5]\ d_h t_p \times 2$		= 39.750	[in^2]	
Net area subject to tension					
when sheared out by 2 side strips	$A_{nt} = (\ e_v - 0.5\ d_h\ )\ t_p \times 2$		= 3.750	[in^2]	
Block shear strength required	$V_u =$		= **131.5**	[kips]	
Uniform tension stress factor	$U_{bs} = 1.00$				AISC 15^{th} Fig C-J4.2
Bolt shear resistance provided	$R_n = \min\ (0.6F_u A_{nv}\ ,\ 0.6F_y A_{gv}\ ) + U_{bs} F_u A_{nt}$		= 1794.0	[kips]	AISC 15^{th} Eq J4-5
Resistance factor-LRFD	$\phi = 0.75$				AISC 15^{th} Eq J4-5
	$\phi\ R_n =$		= **1345.5**	[kips]	
	ratio = **0.10**		$> V_u$	OK	

Flange Plate - Block Shear - Center Strip ratio = 131.5 / 1491.8 = **0.09** **PASS**

Flange force moment arm	$d_m = d_b + t_p$		= 25.400	[in]	
Flange force - tension	$P_u = P_b / 2 + M / d_m$		= **131.5**	[kips]	
Plate Block Shear - Center Strip					
Bolt hole diameter	bolt dia $d_b = 7/8$ [in]		bolt hole dia $d_h = 1$	[in]	AISC 15^{th} B4.3b
Plate thickness	$t_p = 1.500$	[in]			
Plate strength	$F_y = 50.0$	[ksi]	$F_u = 65.0$	[ksi]	
Bolt no in ver & hor dir	$n_v = 2$		$n_h = 7$		
Bolt spacing in ver & hor dir	$s_v = 5.500$	[in]	$s_h = 3.000$	[in]	
Bolt edge dist in ver & hor dir	$e_v = 1.750$	[in]	$e_h = 1.750$	[in]	
Gross area subject to shear	$A_{gv} = [\ (n_h - 1)\ s_h + e_h\ ]\ t_p \times 2$		= 59.250	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5]\ d_h t_p \times 2$		= 39.750	[in^2]	
Net area subject to tension					
when sheared out by center strip	$A_{nt} = (\ n_v - 1)\ (\ s_v - d_h\ )\ t_p$		= 6.750	[in^2]	
Block shear strength required	$V_u =$		= **131.5**	[kips]	
Uniform tension stress factor	$U_{bs} = 1.00$				AISC 15^{th} Fig C-J4.2
Bolt shear resistance provided	$R_n = \min\ (0.6F_u A_{nv}\ ,\ 0.6F_y A_{gv}\ ) + U_{bs} F_u A_{nt}$		= 1989.0	[kips]	AISC 15^{th} Eq J4-5
Resistance factor-LRFD	$\phi = 0.75$				AISC 15^{th} Eq J4-5
	$\phi\ R_n =$		= **1491.8**	[kips]	
	ratio = **0.09**		$> V_u$	OK	

Flange Plate - Tensile Yielding			ratio = 131.5 / 607.5	= **0.22**	**PASS**
Flange force moment arm	$d_m = d_b + t_p$		= 25.400	[in]	
Flange force - tension	$P_u = P_b / 2 + M / d_m$		= **131.5**	[kips]	
Plate Tensile Yielding Check					
Plate size	width b_p = 9.000	[in]	thickness t_p = 1.500	[in]	
Plate yield strength	F_y = 50.0	[ksi]			
Plate gross area in shear	$A_g = b_p t_p$		= 13.500	[in^2]	
Tensile force required	P_u =		= **131.5**	[kips]	
Plate tensile yielding strength	$R_n = F_y A_g$		= 675.0	[kips]	AISC 15th Eq J4-1
Resistance factor-LRFD	ϕ = 0.90				AISC 15th Eq J4-1
	ϕR_n =		= **607.5**	[kips]	
	ratio = **0.22**		> P_u	OK	

Flange Plate - Tensile Rupture			ratio = 131.5 / 511.9	= **0.26**	**PASS**
Flange force moment arm	$d_m = d_b + t_p$		= 25.400	[in]	
Flange force - tension	$P_u = P_b / 2 + M / d_m$		= **131.5**	[kips]	
Plate Tensile Rupture Check					
Bolt hole diameter	bolt dia d_b = 7/8	[in]	bolt hole dia d_h = 1	[in]	AISC 15th B4.3b
Number of bolt	n = 2				
Plate size	width b_p = 9.000	[in]	thickness t_p = 1.500	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate net area in tension	$A_{nt} = (b_p - n\, d_h) t_p$		= 10.500	[in^2]	
Tensile force required	P_u =		= **131.5**	[kips]	
Plate tensile rupture strength	$R_n = F_u A_{nt}$		= 682.5	[kips]	AISC 15th Eq J4-2
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J4-2
	ϕR_n =		= **511.9**	[kips]	AISC 15th Eq J4-2
	ratio = **0.26**		> P_u	OK	

Flange Plate - Compression			ratio = 131.5 / 607.5	= **0.22**	**PASS**
Flange force moment arm	$d_m = d_b + t_p$		= 25.400	[in]	
Flange force - compression	$P_u = P_b / 2 + M / d_m$		= **131.5**	[kips]	
Plate Compression Check					
Plate size	width b_p = 9.000 [in]	thickness t_p = 1.500	[in]		
	F_y = 50.0 [ksi]	E = 29000	[ksi]		
Plate gross area in compression	$A_g = b_p t_p$		= 13.500	[in^2]	
Plate radius of gyration	$r = t_p / \sqrt{12}$		= 0.433	[in]	
Plate effective length factor	K =		= 0.65		
Plate unbraced length	L_u =		= 4.500	[in]	
Plate slenderness	$KL/r = 0.65 \times L_u / r$		= 6.75		
Plate compression required	P_u =		= **131.5**	[kips]	
	when $\frac{KL}{r} \leq 25$				AISC 15th J4.4 (a)
Plate compression provided	$R_n = F_y \times A_g$		= 675.0	[kips]	AISC 15th Eq J4-6
Resistance factor-LRFD	ϕ = 0.90				AISC 15th J4.4 (a)
	ϕR_n =		= **607.5**	[kips]	
	ratio = **0.22**		> P_u	OK	

Flange Plate - Slip Critical			ratio = 139.7 / 232.6	= **0.60**	**PASS**
Flange force moment arm	$d_m = d_b$		= 23.900	[in]	
Beam flange force as shear	$V_u = P_b / 2 + M / d_m$		= **139.7**	[kips]	
Bolt dia & bolt pretension	dia d_b = $^7/_8$ [in]	Pretension T_b = 49.0	[kips]		AISC 15th Table J3.1
Surface class	= Class A	Slip coeff. μ = 0.30			AISC 15th J3.8
Min. bolt pretension	D_u = 1.13	Filler factor h_f = 1.00			AISC 15th J3.8
No of bolt row & column	n_r = 2	n_c = 7			
No of slip plane	n_s = 1				
Bolt group eccentricity coefficient	C_{ec} =		= 1.000		
Required shear strength	V_u =		= **139.7**	[kips]	
Slip resistance	$R_n = \mu D_u h_f T_b n_s n_r n_c C_{ec}$		= 232.6	[kips]	AISC 15th Eq J3-4
Resistance factor-LRFD	ϕ = 1.00	for standard size or SSLT hole			AISC 15th J3.8
	ϕR_n =		= **232.6**	[kips]	
	ratio = **0.60**		> V_u	OK	

Web Plate - Bolt Bearing			ratio = 24.5 / 120.0	= **0.20**	**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 = 0.875	[in]	bolt area A_b = 0.601	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$		= 32.5	[kips]	AISC 15th Eq J3-1
Bolt Bearing/TearOut Strength on Plate					
Bolt hole diameter	bolt dia d_b = $^7/_8$	[in]	bolt hole dia d_h = $^{15}/_{16}$	[in]	AISC 15th Table J3.3
Bolt spacing & edge distance	spacing L_s = 3.000	[in]	edge distance L_e = 1.500	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate thickness	t = 0.375	[in]			
Interior Bolt					
Bolt hole edge clear distance	$L_c = L_s - d_h$		= 2.063	[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
	= 60.3 ≤ 51.2		= 51.2	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$		= 32.5	[kips]	
Edge Bolt					
Bolt hole edge clear distance	$L_c = L_e - d_h / 2$		= 1.031	[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
	= 30.2 ≤ 51.2		= 30.2	[kips]	
Bolt strength at edge	$R_{n\text{-}ed} = \min(R_{n\text{-}t\&b\text{-}ed}, R_{n\text{-}bolt})$		= 30.2	[kips]	
Number of bolt	interior n_{in} = 4		edge n_{ed} = 1		
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-}in} + n_{ed} R_{n\text{-}ed}$		= 160.0	[kips]	
Required shear strength	V_u =		= **24.5**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 15th J3-10
	ϕR_n =		= **120.0**	[kips]	
	ratio = **0.20**		> V_u	OK	

Web Plate - Shear Yielding			ratio = 24.5 / 168.8	= **0.15**	**PASS**
Plate Shear Yielding Check					
Plate size	width b_p = 15.000	[in]	thickness t_p = 0.375	[in]	
Plate yield strength	F_y = 50.0	[ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$		= 5.625	[in^2]	
Shear force required	V_u =		= **24.5**	[kips]	
Plate shear yielding strength	$R_n = 0.6\, F_y A_{gv}$		= 168.8	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00				AISC 15th Eq J4-3
	ϕR_n =		= **168.8**	[kips]	
	ratio = **0.15**		> V_u	OK	

Web Plate - Shear Rupture		ratio = 24.5 / 109.7	= **0.22**	**PASS**
Plate Shear Rupture Check				
Bolt hole diameter	bolt dia $d_b = {}^7/_8$ [in]	bolt hole dia $d_h = 1$	[in]	AISC 15th B4.3b
Number of bolt	$n = 5$			
Plate size	width $b_p = 15.000$ [in]	thickness $t_p = 0.375$	[in]	
Plate tensile strength	$F_u = 65.0$ [ksi]			
Plate net area in shear	$A_{nv} = (b_p - n\ d_h)\ t_p$	= 3.750	[in^2]	
Shear force required	$V_u =$	= **24.5**	[kips]	
Plate shear rupture strength	$R_n = 0.6\ F_u A_{nv}$	= 146.3	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	$\phi = 0.75$			AISC 15th Eq J4-4
	$\phi\ R_n =$	= **109.7**	[kips]	
	ratio = **0.22**	$> V_u$	OK	

Web Plate - Slip Critical		ratio = 24.5 / 66.1	= **0.37**	**PASS**
Bolt dia & bolt pretension	dia $d_b = {}^7/_8$ [in]	Pretension $T_b = 39.0$	[kips]	AISC 15th Table J3.1
Surface class	= Class A	Slip coeff. $\mu = 0.30$		AISC 15th J3.8
Min. bolt pretension	$D_u = 1.13$	Filler factor $h_f = 1.00$		AISC 15th J3.8
No of bolt row & column	$n_r = 5$	$n_c = 1$		
No of slip plane	$n_s = 1$			
Bolt group eccentricity coefficient	$C_{ec} =$	= 1.000		
Required shear strength	$V_u =$	= **24.5**	[kips]	
Slip resistance	$R_n = \mu\ D_u h_f T_b n_s n_r n_c C_{ec}$	= 66.1	[kips]	AISC 15th Eq J3-4
Resistance factor-LRFD	$\phi = 1.00$ for standard size or SSLT hole			AISC 15th J3.8
	$\phi\ R_n =$	= **66.1**	[kips]	
	ratio = **0.37**	$> V_u$	OK	

Web Plate - Block Shear - 1-Side Strip		ratio = 24.5 / 135.3	= **0.18**	**PASS**
Plate Block Shear - Side Strip				
Bolt hole diameter	bolt dia $d_b = {}^7/_8$ [in]	bolt hole dia $d_h = 1$	[in]	AISC 15th B4.3b
Plate thickness	$t_p = 0.375$ [in]			
Plate strength	$F_y = 50.0$ [ksi]	$F_u = 65.0$	[ksi]	
Bolt no in ver & hor dir	$n_v = 1$	$n_h = 5$		
Bolt spacing in hor dir	$s_h = 3.000$ [in]			
Bolt edge dist in ver & hor dir	$e_v = 2.500$ [in]	$e_h = 1.500$	[in]	
Gross area subject to shear	$A_{gv} = [\ (n_h - 1)\ s_h + e_h\ ]\ t_p$	= 5.063	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [\ (n_h - 1) + 0.5\ ]\ d_h t_p$	= 3.375	[in^2]	
Net area subject to tension	$A_{nt} = (e_v - 0.5\ d_h)\ t_p$	= 0.750	[in^2]	
Block shear strength required	$V_u =$	= **24.5**	[kips]	
Uniform tension stress factor	$U_{bs} = 1.00$			AISC 15th Fig C-J4.2
Bolt shear resistance provided	$R_n = \min (0.6F_u A_{nv} , 0.6F_y A_{gv}) + U_{bs} F_u A_{nt}$	= 180.4	[kips]	AISC 15th Eq J4-5
Resistance factor-LRFD	$\phi = 0.75$			AISC 15th Eq J4-5
	$\phi\ R_n =$	= **135.3**	[kips]	
	ratio = **0.18**	$> V_u$	OK	

Beam Flange - Bolt Shear		ratio = 139.7 / 530.4	= **0.26**	**PASS**
Flange force moment arm	$d_m = d_b$	= 23.900	[in]	
Beam flange force as shear	$V_u = P_b / 2 + M / d_m$	= **139.7**	[kips]	
Bolt shear stress	bolt grade = A490-X	F_{nv} = 84.0	[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.875 [in]	bolt area A_b = 0.601	[in^2]	
Number of bolt carried shear	n_s = 14.0	shear plane m = 1		
Bolt group eccentricity coefficient	C_{ec} =	= 1.000		
Required shear strength	V_u =	= **139.7**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 707.2	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J3-1
	ϕR_n =	= **530.4**	[kips]	
	ratio = **0.26**	> V_u	OK	

Beam Flange - Bolt Bearing		ratio = 143.8 / 530.4	= **0.27**	**PASS**
Flange force moment arm	$d_m = d_b - t_{fb}$	= 23.220	[in]	
Beam flange force as shear	$V_u = P_b / 2 + M / d_m$	= **143.8**	[kips]	
Single Bolt Shear Strength				
Bolt shear stress	bolt grade = A490-X	F_{nv} = 84.0	[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.875 [in]	bolt area A_b = 0.601	[in^2]	
Single bolt shear strength	$R_{n\text{-bolt}} = F_{nv} A_b$	= 50.5	[kips]	AISC 15th Eq J3-1
Bolt Bearing/TearOut Strength on Plate				
Bolt hole diameter	bolt dia $d_b = {}^7/_8$ [in]	bolt hole dia $d_h = {}^{15}/_{16}$	[in]	AISC 15th Table J3.3
Bolt spacing & edge distance	spacing L_s = 3.000 [in]	edge distance L_e = 3.750	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate thickness	t = 0.680 [in]			
Interior Bolt				
Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.063	[in]	
Bolt tear out/bearing strength	$R_{n\text{-t\&b-in}} = 1.2 L_c t F_u \le 2.4 d_b t F_u$			AISC 15th Eq J3-6a
	= 109.4 ≤ 92.8	= 92.8	[kips]	
Bolt strength at interior	$R_{n\text{-in}} = \min(R_{n\text{-t\&b-in}}, R_{n\text{-bolt}})$	= 50.5	[kips]	
Edge Bolt				
Bolt hole edge clear distance	$L_c = L_e - d_h / 2$	= 3.281	[in]	
Bolt tear out/bearing strength	$R_{n\text{-t\&b-ed}} = 1.2 L_c t F_u \le 2.4 d_b t F_u$			AISC 15th Eq J3-6a
	= 174.0 ≤ 92.8	= 92.8	[kips]	
Bolt strength at edge	$R_{n\text{-ed}} = \min(R_{n\text{-t\&b-ed}}, R_{n\text{-bolt}})$	= 50.5	[kips]	
Number of bolt	interior n_{in} = 12	edge n_{ed} = 2		
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-in}} + n_{ed} R_{n\text{-ed}}$	= 707.2	[kips]	
Required shear strength	V_u =	= **143.8**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th J3-10
	ϕR_n =	= **530.4**	[kips]	
	ratio = **0.27**	> V_u	OK	

Beam Flange - Block Shear - 2 Side Strips		ratio = 143.8 / 689.2	= **0.21**	**PASS**
Flange force moment arm	$d_m = d_b - t_{fb}$	= 23.220	[in]	
Beam flange force - tension	$V_u = P_b / 2 + M / d_m$	= **143.8**	[kips]	

Plate Block Shear - 2 Side Strips						
Bolt hole diameter	bolt dia $d_b = 7/8$	[in]	bolt hole dia $d_h = 1$	[in]	AISC 15th B4.3b
Plate thickness	$t_p = 0.680$	[in]			
Plate strength	$F_y = 50.0$	[ksi]	$F_u = 65.0$	[ksi]	
Bolt no in ver & hor dir	$n_v = 2$		$n_h = 7$		
Bolt spacing in ver & hor dir	$s_v = 5.500$	[in]	$s_h = 3.000$	[in]	
Bolt edge dist in ver & hor dir	$e_v = 1.745$	[in]	$e_h = 3.750$	[in]	

Gross area subject to shear	$A_{gv} = [\ (n_h - 1)\ s_h + e_h\ ]\ t_p \times 2$	= 29.580	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5]\ d_h\ t_p \times 2$	= 20.740	[in^2]	
Net area subject to tension when sheared out by 2 side strips	$A_{nt} = (\ e_v - 0.5\ d_h\ )\ t_p \times 2$	= 1.693	[in^2]	

Block shear strength required	$V_u =$	= **143.8**	[kips]	
Uniform tension stress factor	$U_{bs} = 1.00$			AISC 15th Fig C-J4.2
Bolt shear resistance provided	$R_n = \min(0.6F_u A_{nv}, 0.6F_y A_{gv}) + U_{bs} F_u A_{nt}$	= 918.9	[kips]	AISC 15th Eq J4-5
Resistance factor-LRFD	$\phi = 0.75$			AISC 15th Eq J4-5
	$\phi R_n =$	= **689.2**	[kips]	
	ratio = **0.21**	> V_u	OK	

Beam Web - Bolt Shear			ratio = 24.5 / 121.8	= **0.20**	**PASS**
Bolt shear stress	bolt grade = A325-N		$F_{nv} = 54.0$	[ksi]	AISC 15th Table J3.2
	bolt dia $d_b = 0.875$	[in]	bolt area $A_b = 0.601$	[in^2]	
Number of bolt carried shear	$n_s = 5.0$		shear plane m = 1		
Bolt group eccentricity coefficient	$C_{ec} =$		= 1.000		
Required shear strength	$V_u =$		= **24.5**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$		= 162.4	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	$\phi = 0.75$				AISC 15th Eq J3-1
	$\phi R_n =$		= **121.8**	[kips]	
	ratio = **0.20**		> V_u	OK	

Beam Web - Bolt Bearing — ratio = 24.5 / 121.8 = **0.20** **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 = 0.875 [in]	bolt area A_b = 0.601	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$	= 32.5	[kips]	AISC 15th Eq J3-1

Bolt Bearing/TearOut Strength on Plate

Bolt hole diameter	bolt dia d_b = $^7/_8$ [in]	bolt hole dia d_h = $^{15}/_{16}$	[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 = 0.440 [in]			

Interior Bolt

Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.063	[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
	= 70.8 ≤ 60.1	= 60.1	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$	= 32.5	[kips]	

Number of bolt	interior n_{in} = 5			
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-}in}$	= 162.4	[kips]	
Required shear strength	V_u =	= **24.5**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th J3-10
	ϕR_n =	= **121.8**	[kips]	
	ratio = **0.20**	> V_u	OK	

Beam Web - Shear Yielding — ratio = 24.5 / 315.5 = **0.08** **PASS**

Plate Shear Yielding Check

Plate size	width b_p = 23.900 [in]	thickness t_p = 0.440	[in]	
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$	= 10.516	[in^2]	
Shear force required	V_u =	= **24.5**	[kips]	
Plate shear yielding strength	$R_n = 0.6\ F_y A_{gv}$	= 315.5	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	ϕR_n =	= **315.5**	[kips]	
	ratio = **0.08**	> V_u	OK	

Beam Web - Shear Rupture — ratio = 24.5 / 243.2 = **0.10** **PASS**

Plate Shear Rupture Check

Bolt hole diameter	bolt dia d_b = $^7/_8$ [in]	bolt hole dia d_h = 1	[in]	AISC 15th B4.3b
Number of bolt	n = 5			
Plate size	width b_p = 23.900 [in]	thickness t_p = 0.440	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate net area in shear	$A_{nv} = (b_p - n\ d_h)\ t_p$	= 8.316	[in^2]	
Shear force required	V_u =	= **24.5**	[kips]	
Plate shear rupture strength	$R_n = 0.6\ F_u A_{nv}$	= 324.3	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-4
	ϕR_n =	= **243.2**	[kips]	
	ratio = **0.10**	> V_u	OK	

Beam Flange With Holes - Beam Flexural Rupture					**Not Applicable**
Beam sect W24X76	b_f = 8.990	[in]	t_f = 0.680	[in]	
	S_x = 176.00	[in^3]			
	F_y = 50.0	[ksi]	F_u = 65.0	[ksi]	
Gross area of tension flange	$A_{fg} = b_f t_f$		= 6.113	[in^2]	
Bolt hole diameter	bolt dia $d_b = {}^7/_8$	[in]	bolt hole dia d_h = 1	[in]	AISC 15th B4.3b
Number of bolt	n = 2				
Net area of tension flange	$A_{fn} = (b_f - n\ d_h)\ t_f$		= 4.753	[in^2]	
	Y_t = 1.0				
	When $F_u A_{fn} \geq Y_t F_y A_{fg}$ this limit state doesn't apply				AISC 15th Eq F13.1 (a)

Column Flange Bending		ratio = 131.5 / 482.7	= **0.27**	**PASS**	
Flange force moment arm	$d_m = d_b + t_p$		= 25.400	[in]	
Flange force required - tension	$P_{uf\_t} = P_u / 2 - M_u / d_m$		= **131.5**	[kips]	
Column flange thickness	t_{fc} = 1.310 [in]	yield F_{yc} = 50.0		[ksi]	
Top column condition	it's not a top column case	C_t = 1.0			AISC 15th J10.1
Column flange tensile resistance	$R_n = C_t 6.25\ F_{yc} t_{fc}^2$		= 536.3	[kips]	AISC 15th Eq J10-1
Resistance factor-LRFD	ϕ = 0.90				AISC 15th J10.1
	$\phi\ R_n$ =		= **482.7**	[kips]	
	ratio = **0.27**		> P_{uf_t}	OK	

Column Web Yielding		ratio = 131.5 / 1011.1	= **0.13**	**PASS**	
Column web thickness	t_{wc} = 0.830 [in]	yield F_y = 50.0		[ksi]	
Doubler plate thickness	t_{dp} = 0.500 [in]	yield F_{ydp} = 50.0		[ksi]	
Equivalent web thickness when considering doubler plate	$t_{w-eq} = t_{wc} + t_{dp} \times F_{ydp} / F_y \times 2$ sides		= 1.830	[in]	
Flange force moment arm	$d_m = d_b + t_p$		= 25.400	[in]	
Flange force in demand	$P_{uf} = \max (P_{uf\_t} , P_{uf\_c})$		= **131.5**	[kips]	AISC DG13 Eq 4.2-1
Column section	d_c = 15.200 [in]	t_{fc} = 1.310		[in]	
	t_{wc} = 0.830 [in]	k_c = 1.910		[in]	
Column yield strength	F_{yc} = 50.0 [ksi]				
Top column condition	it's not a top column case				AISC 15th J10.2 (a)
Flange plate fillet weld size	w = 0.000 [in]	flange plate t_p = 1.500		[in]	
Length of bearing	$N = t_p + 2\ w$		= 1.500	[in]	AISC DG4 Eq 3.24
Column web yielding strength	$R_n = (5\ k_c + N)\ F_{yc} t_{w-eq}$		= 1011.1	[kips]	AISC 15th Eq J10-2
Resistance factor-LRFD	ϕ = 1.00				AISC 15th J10.2
	$\phi\ R_n$ =		= **1011.1**	[kips]	
	ratio = **0.13**		> P_{uf}	OK	

Column Web Buckling		ratio = 131.5 / 1878.3	= **0.07**	**PASS**
Flange force moment arm	$d_m = d_b + t_p$	= 25.400	[in]	
Flange force required in compression	$P_{uf\_c} = P_u / 2 - M_u / d_m$	= **131.5**	[kips]	
Column section	d_c = 15.200 [in]	t_{fc} = 1.310	[in]	
	t_{wc} = 0.830 [in]	k_c = 1.910	[in]	
	$h = d_c - 2\,k_c$	= 11.380	[in]	
Column yield strength	F_{yc} = 50.0 [ksi]	E_c = 29000	[ksi]	
Top column condition	it's not a top column case	C_t = 1.0		AISC 15th J10.5
Column web buckling strength	$R_{n1} = \dfrac{C_t\, 24\, t_{wc}^3 \sqrt{E_c F_{yc}}}{h}$	= 1452.1	[kips]	AISC 15th Eq J10-8
Doubler plate thickness	t_{dp} = 0.500 [in]	yield F_{ydp} = 50.0	[ksi]	
Doubler plate buckling strength	$R_{n2} = \dfrac{2\, C_t\, 24\, t_{dp}^3 \sqrt{E_c F_{ydp}}}{h}$	= 634.9	[kips]	AISC 15th Eq J10-8
Total buckling strength	$R_n = R_{n1} + R_{n2}$	= 2087.0	[kips]	
Resistance factor-LRFD	ϕ = 0.90			AISC 15th J10.5
	$\phi\, R_n$ =	= **1878.3**	[kips]	
	ratio = **0.07**	> P_{uf_c}	OK	

Column Web Crippling		ratio = 131.5 / 1344.2	= **0.10**	**PASS**
Flange force moment arm	$d_m = d_b + t_p$	= 25.400	[in]	
Flange force required in compression	$P_{uf\_c} = P_u / 2 - M_u / d_m$	= **131.5**	[kips]	
Column section	d_c = 15.200 [in]	t_{fc} = 1.310	[in]	
	t_{wc} = 0.830 [in]	k_c = 1.910	[in]	
Column yield strength	F_{yc} = 50.0 [ksi]	E_c = 29000	[ksi]	
Flange plate fillet weld size	w = 0.000 [in]	flange plate t_p = 1.500	[in]	
Length of bearing	$l_b = t_p + 2\,w$	= 1.500	[in]	
Distance from top of column to top of beam flange	$d_{end\text{-}flg}$ =	= 4.000	[in]	
Top column condition	it's not a top column case, use Eq J10-4			AISC 15th J10.3 (a)
Column web crippling strength	$R_{n1} = 0.8\, t_{wc}^2 \left[1+3\dfrac{l_b}{d_c}\left(\dfrac{t_{wc}}{t_{fc}}\right)^{1.5}\right] \times \left(\dfrac{E_c F_{yc} t_{fc}}{t_{wc}}\right)^{0.5}$	= 958.2	[kips]	AISC 15th Eq J10-4
Doubler plate thickness	t_{dp} = 0.500 [in]	yield F_{ydp} = 50.0	[ksi]	
Doubler plate crippling strength	$R_{n2} = 0.8\, t_{dp}^2 \left[1+3\dfrac{l_b}{d_c}\left(\dfrac{t_{dp}}{t_{fc}}\right)^{1.5}\right] \times \left(\dfrac{E_c F_{ydp} t_{fc}}{t_{dp}}\right)^{0.5}$	= 834.1	[kips]	AISC 15th Eq J10-4
Total crippling strength	$R_n = R_{n1} + R_{n2}$	= 1792.3	[kips]	
Resistance factor-LRFD	ϕ = 0.75			AISC 15th J10.3
	$\phi\, R_n$ =	= **1344.2**	[kips]	
	ratio = **0.10**	> P_{uf_c}	OK	

Column Panel Zone Shear		ratio = 104.0 / 421.8	= **0.25**	**PASS**
Panel zone shear force	$V_p = P_{f\text{-}TR} - P_{f\text{-}TL} - V_s$	= **104.0**	[kips]	
Column W14X176	d_c = 15.200 [in]	b_{cf} = 15.700	[in]	
	t_{cf} = 1.310 [in]	t_{cw} = 0.830	[in]	
	A_c = 51.800 [in^2]	F_{cy} = 50.0	[ksi]	
Beam W24X76	d_b = 23.900 [in]	t_{bf} = 0.680	[in]	
Flange plate thickness	t_p = from user input	= 1.500	[in]	
Moment arm between flange plates	$d_m = d + t_p$	= 26.900	[in]	AISC 358-16 Eq 7.6-7
Column axial compression	P_r = from user input	= 249.0	[kips]	
Column axial yield strength	$P_y = F_{cy} A_c$	= 2590.0	[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	$R_n = 0.6\, F_{cy}\, d_c\, t_{cw} \left(1 + \frac{3\, b_{cf}\, t_{cf}^2}{d_m\, d_c\, t_{cw}}\right)$	= 468.6	[kips]	AISC 15th Eq J10-11
Resistance factor-LRFD	ϕ = 0.90			AISC 15th J10.6
	$\phi\, R_n$ =	= **421.8**	[kips]	
	ratio = **0.25**	> V_p	OK	

Seismic Material & Geometry Limitations — PASS

Check Max Beam Yield Stress

Condition : beam max material $F_y \le F_{y\_max}$ = 50 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 $F_y \le F_{y\_max}$ = 65 ksi — AISC 341-16 A3.1

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

$\le F_{y\_max}$ OK

Check Max Beam Depth

Condition : beam depth shall be limited to W36 maximum — AISC 358-16 7.3.1 (2)

Beam W24X76 depth d = = **23.900** [in]

≤ W36 OK

Check Max Beam Weight

Condition : beam weight shall be limited to 150 lb/ft maximum — AISC 358-16 7.3.1 (3)

Beam W24X76 weight wt = = **75** [lb/ft]

≤ 150 OK

Check Max Beam Flange Thickness

Condition : beam flange thickness shall be limited to 1 in maximum — AISC 358-16 7.3.1 (4)

Beam W24X76 flange thickness t_f = = **0.680** [in]

$\le t_f$ OK

Check Min Beam Clear Span to Depth Ratio

Condition : beam clear span-to-depth ratio ≥ 9 — AISC 358-16 7.3.1 (5)

Beam W24X76 clear span L = 345.0 [in] depth d = 23.900 [in]

Beam clear span to depth ratio L/d = L / d = **14.44**

≥ 9 OK

Check Max Column Depth

Condition : column depth shall be limited to W36 maximum — AISC 358-16 7.3.2 (3)

Column W14X176 depth d = = **15.200** [in]

≤ W36 OK

Check Flange Plate Bolt Hole Type

Condition : Flange plate bolt hole type shall be STD

Flange plate bolt hole type = STD + STD

OK AISC 358-16 7.5.4

Check Web Plate Bolt Hole Type

Condition : Web plate bolt hole type shall be STD or SSLT

Web plate bolt hole type = STD + STD

OK AISC 341-16 D2.2 (3)

Seismic Geometry SMF Column Flange Thickness **PASS**

Check SMF Column Condition Without Continuity Plate

SMF column flange thcikness shall satisfy AISC 341-16 Eq E3-8 , otherwise continuity plate is required AISC 341-16 E3.6f.1

Siffener palte is provided as continuity plate, so this check is not required.

OK AISC 341-16 E3.6f.1

Seismic Geometry SMF Column Stiffener Plate Thickness **PASS**

Check SMF Column Min Stiffener Plate Thickness

SMF column minimum stiffener plate thcikness shall satisfy requirements in AISC 341-16 E3.6f.2(b) AISC 341-16 E3.6f.2(b)

Right beam W24X76 flange thickness	t_{fb1} =	= 0.680	[in]		
Left beam W24X76 flange thickness	t_{fb2} =	= 0.680	[in]		
Thicker beam flange thickness	t_{fb} = max (t_{fb1} , t_{fb2})	= 0.680	[in]		
Min stiffener plate thickness	t_{min} = t_{fb} x 0.75	= **0.510**	[in]		
Stiffener plate thickness	t_s = from user input	= **1.125**	[in]		
		≥ t_{min}		OK	AISC 341-16 E3.6f.2(b)

Seismic Width to Thickness Ratio — **PASS**

Check Beam Flange

Beam half flange width	$b = 4.495$ [in]	$t_f = 0.680$ [in]	
Beam yield strength	$F_y = 50.0$ [ksi]	$E = 29000$ [ksi]	
Width-to-thickness ratio-allow	$\lambda_{hd} = 0.30 \sqrt{E / F_y}$	= **7.22**	
Width-to-thickness ratio-actual	$b/t_f = b / t_f$	= **6.61**	
		$\leq \lambda_{hd}$	OK AISC 341-16 Table D1.1

Check Beam Web

Clear dist between beam flange	$h_b = 22.540$ [in]	$t_w = 0.440$ [in]	
Beam yield strength	$F_{yb} = 50.0$ [ksi]	$E = 29000$ [ksi]	
Resistance factor for compression	$\phi_c = 0.90$	$A_b = 22.400$ [in^2]	
Nominal axial yield strength	$P_{yb} = F_{yb} A_b$	= 1120.0 [kips]	
Beam axial compression	P_{ub} = from user input	= 0.0 [kips]	
Ratio of req'd to available strength	$C_{ab} = P_{ub} / (\phi_c \times P_{yb})$	= 0.00	
Width-to-thickness ratio-allow	$\lambda_{hd} =$	= **59.00**	
Width-to-thickness ratio-actual	$h/t_w = h_b / t_w$	= **51.23**	
		$\leq \lambda_{hd}$	OK AISC 341-16 Table D1.1

Check Column Flange

Column half flange width	$b = 7.850$ [in]	$t_f = 1.310$ [in]	
Column yield strength	$F_y = 50.0$ [ksi]	$E = 29000$ [ksi]	
Width-to-thickness ratio-allow	$\lambda_{hd} = 0.30 \sqrt{E / F_y}$	= **7.22**	
Width-to-thickness ratio-actual	$b/t_f = b / t_f$	= **5.99**	
		$\leq \lambda_{hd}$	OK AISC 341-16 Table D1.1

Check Column Web

Clear dist between column flange	$h_c = 12.580$ [in]	$t_w = 0.830$ [in]	
Column yield strength	$F_{yc} = 50.0$ [ksi]	$E = 29000$ [ksi]	
Resistance factor for compression	$\phi_c = 0.90$	$A_c = 51.800$ [in^2]	
Nominal axial yield strength	$P_{yc} = F_{yc} A_c$	= 2590.0 [kips]	
Column axial compression	P_{uc} = from user input	= 249.0 [kips]	
Ratio of req'd to available strength	$C_{ac} = P_{uc} / (\phi_c \times P_{yc})$	= 0.11	
Width-to-thickness ratio-allow	$\lambda_{hd} =$	= **53.14**	
Width-to-thickness ratio-actual	$h/t_w = h_c / t_w$	= **15.16**	
		$\leq \lambda_{hd}$	OK AISC 341-16 Table D1.1

Seismic Weld Limitation — PASS

Check Beam Web Weld Material

Condition : Web weld electrode must be E70 or E80

Beam web weld electrode material	=	E70XX		
			OK	AISC 341-16 A3.4b

Check Beam Flange Weld Material

Condition : Flange weld electrode must be E70 or E80

Beam flange weld electrode material	=	E70XX		
			OK	AISC 341-16 A3.4b

Check Beam Flange Weld Type

Condition : Flange weld type shall be CJP weld

Beam flange to end plate weld	=	CJP		
			OK	AISC 358-16 7.5.2

Check Column Stiff PL to Column Flange Weld Type

Condition : Continuity plate (column stiff) shall be welded to column flanges using CJP weld

Column stiff to column flange weld	=	CJP		
			OK	AISC 341-16 E3.6f (3)

Seismic Flange Plate Bolt Limitation — FAIL

Check Length of Bolt Group

Condition : Length of bolt group L ≤ beam depth — AISC 358-16 7.5.4

Beam depth	d = 23.900 [in]			
Bolt row & spacing	n = 7	s = 3.000 [in]		
Length of bolt group	$L = (n-1)\,s$	= 18.000 [in]		
		$\le d$	OK	AISC 358-16 7.5.4

Check Check Max Bolt Diameter

Condition : Max bolt diameter $d_b \le$ 1.125 inch

Bolt diameter	= 0.875			
			OK	AISC 358-16 7.5.4

Check Max Bolt Dia to Prevent Beam Flange Tensile Rupture

Ratio of expected/min yield & tensile strength	R_y = 1.1	R_t = 1.2		
Beam sect W24X76	b_f = 8.990 [in]			
Beam sect strength	F_y = 50.0 [ksi]	F_u = 65.0 [ksi]		
Flange plate nax bolt diameter	$d_{max} = \frac{b_f}{2}\left(1 - \frac{R_y F_y}{R_t F_u}\right) - \frac{1}{8}$	= 1.200 [in]		AISC 358-16 Eq 7.6-2
Flange plate bolt diameter	d_b =	= 0.875 [in]		
		$\le d_{max}$	OK	

Check Bolt Grade Material

Condition : Bolt grade must be A490-X or A490M-X

Bolt grade	= A490-X			
			OK	AISC 358-16 7.5.4

Check Bolt Clear Edge Dist at Beam Flange

Bolt and bolt hole dia	d_b = 0.875 [in]	d_h = 0.938 [in]		
Bolt edge dist at beam flange	L_e = 3.750 [in]			
Bolt clear edge dist	$L_{ce} = L_e - 0.5\,d_h$	= 3.281 [in]		
		$\ge 2d_b$	OK	AISC 358-16 7.6 Step 5

Check Bolt Clear Edge Dist at Flange Plate

Bolt and bolt hole dia	d_b = 0.875 [in]	d_h = 0.938 [in]		
Bolt edge dist at flange plate	L_e = 1.750 [in]			
Bolt clear edge dist	$L_{ce} = L_e - 0.5\,d_h$	= 1.281 [in]		
		$< 2d_b$	NG	AISC 358-16 7.6 Step 5

Check Bolt Clear Dist at Bolt Spacing

Bolt and bolt hole dia	d_b = 0.875 [in]	d_h = 0.938 [in]		
Bolt spacing at flange plate	L_s = 3.000 [in]			
Bolt clear dist spacing	$L_{cs} = L_s - d_h$	= 2.063 [in]		
		$\ge 2d_b$	OK	AISC 358-16 7.6 Step 5

Seismic Column Beam Moment Ratio — PASS

Seismic column beam moment ratio check as per AISC 341-16 E3.4a — AISC 341-16 E3.4a

Column W14X176	A_c = 51.800 [in^2]	Z_c = 320.00 [in^3]		
	d_c = 15.200 [in]	F_{yc} = 50.0 [ksi]		
Column axial force	P_{uc} = from user input	= 249.0	[kips]	
LRFD-ASD force adjustment factor	α = for LRFD	= 1.0		AISC 341-16 D1.2a (b)
Half of column height above/below beam	h_t = 75.0 [in]	h_b = 84.0 [in]		
Beam W24X76 depth	d_b = 23.900 [in]			
Top column flexural strength	$M_t = Z_c\,(F_{yc} - \alpha\,P_{uc} / A_c)\dfrac{h_t}{h_t - 0.5d_b}$	= 1433.6	[kip-ft]	AISC 341-16 Eq E3-2
Bot column flexural strength	$M_b = Z_c\,(F_{yc} - \alpha\,P_{uc} / A_c)\dfrac{h_b}{h_b - 0.5d_b}$	= 1405.0	[kip-ft]	
Total column flexural strength	$M_{pc} = M_t + M_b$	= **2838.6**	[kip-ft]	AISC 341-16 Eq E3-2
Beam W24X76	Z_b = 200.00 [in^3]	F_{yb} = 50.0 [ksi]		
Beam ratio of expected/min yield stress	R_{yb} = 1.10			AISC 341-16 Table A3.1

see Seismic Moment and Beam Flange Force Calc section on how below seismic moments are derived

Seismic M at right column face	M_R =	= 1239.2	[kip-ft]	
Seismic M at left column face	M_L =	= 1185.3	[kip-ft]	
Seismic V at right column face	V_R =	= 100.9	[kips]	
Seismic V at left column face	V_L =	= 67.8	[kips]	
Expected flexural strength of beam	$M_{pb} = M_R + M_L + (V_R + V_L) \times 0.5\,d_c$	= **2531.4**	[kip-ft]	AISC 341-16 Eq E3-3
Seismic column beam moment ratio	$= M_{pc} / M_{pb}$	= 1.12		AISC 341-16 Eq E3-1
		≥ 1.0	OK	

Seismic Flange PL Thickness AISC 358-10 7.6 Step 10 — PASS

Check Flange Plate Material

Condition : Flange plate material must be A36 or A572

Flange plate material = A572 Gr.50

OK — AISC 358-16 7.5.1

Check Min Flange Plate Thickness

Flange plate force	F_{pr} = See calc in Seismic Moment and Beam Flange Force Calc	= 560.0	[kips]	
Flange plate	F_y = 50.0 [ksi]	b_{fp} = 9.000 [in]		
Resistance factor-ductile limit state	ϕ_d = 1.00			AISC 358-16 2.4.1
Min flange plate thickness	$t_{min} = \dfrac{F_{pr}}{\phi_d\,F_y\,b_{fp}}$	= 1.244	[in]	AISC 358-16 Eq 7.6-9
Flange plate thickness	t =	= 1.500	[in]	
		≥ t_{min}	OK	

Seismic Flange PL Tensile Yielding		ratio = 560.0 / 675.0	= **0.83**	**PASS**	
Flange force moment arm	$d_m = d_b + t_p$		= 25.400	[in]	
Flange force - tension	$P_u = P_b / 2 + M / d_m$		= **560.0**	[kips]	

Plate Tensile Yielding Check

Plate size	width b_p = 9.000	[in]	thickness t_p = 1.500	[in]	
Plate yield strength	F_y = 50.0	[ksi]			
Plate gross area in shear	$A_g = b_p t_p$		= 13.500	[in^2]	
Tensile force required	P_u =		= **560.0**	[kips]	
Plate tensile yielding strength	$R_n = F_y A_g$		= 675.0	[kips]	AISC 15th Eq J4-1
Resistance factor-LRFD	ϕ_d = 1.00				AISC 358-16 2.4.1
	ϕR_n =		= **675.0**	[kips]	
	ratio = **0.83**		> P_u	OK	

Seismic Flange PL Tensile Rupture AISC 358-10 7.6 - Step 11		ratio = 560.0 / 614.3	= **0.91**	**PASS**	
Flange force moment arm	$d_m = d_b + t_p$		= 25.400	[in]	
Flange force - tension	$P_u = P_b / 2 + M / d_m$		= **560.0**	[kips]	

Plate Tensile Rupture Check

Bolt hole diameter	bolt dia d_b = $^7/_8$	[in]	bolt hole dia d_h = 1	[in]	AISC 15th B4.3b
Number of bolt	n = 2				
Plate size	width b_p = 9.000	[in]	thickness t_p = 1.500	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate net area in tension	$A_{nt} = (b_p - n\, d_h)\, t_p$		= 10.500	[in^2]	
Tensile force required	P_u =		= **560.0**	[kips]	
Plate tensile rupture strength	$R_n = F_u A_{nt}$		= 682.5	[kips]	AISC 15th Eq J4-2
Resistance factor-LRFD	ϕ_n = 0.90				AISC 358-16 2.4.1
	ϕR_n =		= **614.3**	[kips]	AISC 15th Eq J4-2
	ratio = **0.91**		> P_u	OK	

Seismic Flange PL Block Shear-2 Side Strips			ratio = 560.0 / 1937.5	= **0.29**	**PASS**	
Flange force moment arm	$d_m = d_b + t_p$			= 25.400	[in]	
Flange force - tension	$P_u = P_b / 2 + M / d_m$			= **560.0**	[kips]	
Plate Block Shear - 2 Side Strips						
Bolt hole diameter	bolt dia $d_b = {}^7/_8$	[in]	bolt hole dia $d_h = 1$		[in]	AISC 15th B4.3b
Plate thickness	$t_p = 1.500$	[in]				
Plate strength	$F_y = 50.0$	[ksi]	$F_u = 65.0$		[ksi]	
Bolt no in ver & hor dir	$n_v = 2$		$n_h = 7$			
Bolt spacing in ver & hor dir	$s_v = 5.500$	[in]	$s_h = 3.000$		[in]	
Bolt edge dist in ver & hor dir	$e_v = 1.750$	[in]	$e_h = 1.750$		[in]	
Gross area subject to shear	$A_{gv} = [\ (n_h - 1)\ s_h + e_h\ ]\ t_p \times 2$			= 59.250	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5]\ d_h\ t_p \times 2$			= 39.750	[in^2]	
Net area subject to tension when sheared out by 2 side strips	$A_{nt} = (\ e_v - 0.5\ d_h\ )\ t_p \times 2$			= 3.750	[in^2]	
Block shear strength required	$V_u =$			= **560.0**	[kips]	
Uniform tension stress factor	$U_{bs} = 1.00$					AISC 15th Fig C-J4.2
Ratio of expected Fy/Fu to specified min Fy/Fu	$R_y = 1.10$		$R_t = 1.20$			AISC 341-16 Table A3.1
Bolt shear resistance provided	$R_n = \min\ (0.6\ R_t F_u A_{nv},\ 0.6\ R_y F_y A_{gv}) + U_{bs} R_t F_u A_{nt}$			= 2152.8	[kips]	AISC 15th Eq J4-5
Resistance factor-LRFD	$\phi_n = 0.90$					AISC 358-16 2.4.1
	$\phi\ R_n =$			= **1937.5**	[kips]	
	ratio = **0.29**			> V_u	OK	

Seismic Flange PL Block Shear-Center Strip			ratio = 560.0 / 2148.1	= **0.26**	**PASS**
Flange force moment arm	$d_m = d_b + t_p$		= 25.400	[in]	
Flange force - tension	$P_u = P_b / 2 + M / d_m$		= **560.0**	[kips]	

Plate Block Shear - Center Strip

Bolt hole diameter	bolt dia $d_b = {}^7/_8$ [in]	bolt hole dia $d_h = 1$		[in]	AISC 15th B4.3b
Plate thickness	$t_p = 1.500$ [in]				
Plate strength	$F_y = 50.0$ [ksi]	$F_u = 65.0$		[ksi]	
Bolt no in ver & hor dir	$n_v = 2$	$n_h = 7$			
Bolt spacing in ver & hor dir	$s_v = 5.500$ [in]	$s_h = 3.000$		[in]	
Bolt edge dist in ver & hor dir	$e_v = 1.750$ [in]	$e_h = 1.750$		[in]	
Gross area subject to shear	$A_{gv} = [\ (n_h - 1)\ s_h + e_h\ ]\ t_p \times 2$		= 59.250	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5]\ d_h\ t_p \times 2$		= 39.750	[in^2]	
Net area subject to tension when sheared out by center strip	$A_{nt} = (\ n_v - 1)\ (\ s_v - d_h)\ t_p$		= 6.750	[in^2]	
Block shear strength required	$V_u =$		= **560.0**	[kips]	
Uniform tension stress factor	$U_{bs} = 1.00$				AISC 15th Fig C-J4.2
Ratio of expected Fy/Fu to specified min Fy/Fu	$R_y = 1.10$	$R_t = 1.20$			AISC 341-16 Table A3.1
Bolt shear resistance provided	$R_n = \min\ (0.6\ R_t F_u A_{nv}\ ,\ 0.6\ R_y F_y A_{gv}\ ) + U_{bs} R_t F_u A_{nt}$		= 2386.8	[kips]	AISC 15th Eq J4-5
Resistance factor-LRFD	$\phi_n = 0.90$				AISC 358-16 2.4.1
	$\phi\ R_n =$		= **2148.1**	[kips]	
	ratio = **0.26**		> V_u	OK	

Seismic Flange Plate - Bolt Shear			ratio = 560.0 / 636.4	= **0.88**	**PASS**
Beam flange force as shear	$V_u =$ See calc in Seismic Moment and Beam Flange Force Calc		= **560.0**	[kips]	
Bolt shear stress	bolt grade = A490-X	$F_{nv} = 84.0$		[ksi]	AISC 15th Table J3.2
	bolt dia $d_b = 0.875$ [in]	bolt area $A_b = 0.601$		[in^2]	
Number of bolt carried shear	$n_s = 14.0$	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		= **560.0**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m\ C_{ec}$		= 707.2	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	$\phi_n = 0.90$				AISC 358-16 2.4.1
	$\phi_n R_n =$		= **636.4**	[kips]	
	ratio = **0.88**		> V_u	OK	

Seismic Flange Plate - Bolt Bearing ratio = 560.0 / 636.4 = **0.88** **PASS**

Beam flange force as shear	V_u = See calc in Seismic Moment and Beam Flange Force Calc	= **560.0**	[kips]	

Single Bolt Shear Strength

Bolt shear stress	bolt grade = A490-X	F_{nv} = 84.0	[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.875 [in]	bolt area A_b = 0.601	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$	= 50.5	[kips]	AISC 15th Eq J3-1

Bolt Bearing/TearOut Strength on Plate

Bolt hole diameter	bolt dia d_b = $^7/_8$ [in]	bolt hole dia d_h = $^{15}/_{16}$	[in]	AISC 15th Table J3.3
Bolt spacing & edge distance	spacing L_s = 3.000 [in]	edge distance L_e = 1.750	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate thickness	t = 1.500 [in]			

Interior Bolt

Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.063	[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
	= 241.3 ≤ 204.8	= 204.8	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$	= 50.5	[kips]	

Edge Bolt

Bolt hole edge clear distance	$L_c = L_e - d_h / 2$	= 1.281	[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
	= 149.9 ≤ 204.8	= 149.9	[kips]	
Bolt strength at edge	$R_{n\text{-}ed} = \min(R_{n\text{-}t\&b\text{-}ed}, R_{n\text{-}bolt})$	= 50.5	[kips]	

Number of bolt	interior n_{in} = 12	edge n_{ed} = 2		
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-}in} + n_{ed} R_{n\text{-}ed}$	= 707.2	[kips]	
Required shear strength	V_u = see Seismic Moment and Beam Flange Force Calc	= **560.0**	[kips]	
Bolt resistance factor-LRFD	ϕ_n = 0.90			AISC 358-16 2.4.1
	$\phi_n R_n$ =	= **636.4**	[kips]	
	ratio = **0.88**	> V_u	OK	

Seismic Beam Flange Block Shear-2 Side Strip - Step 12		ratio = 560.0 / 909.7	= **0.62**	**PASS**
Flange force moment arm	$d_m = d_b - t_{fb}$	= 23.220	[in]	
Beam flange force - tension	$V_u = P_b / 2 + M / d_m$	= **560.0**	[kips]	

Plate Block Shear - 2 Side Strips

Bolt hole diameter	bolt dia $d_b = 7/8$	[in]	bolt hole dia $d_h = 1$	[in]	AISC 15th B4.3b
Plate thickness	$t_p = 0.680$	[in]			
Plate strength	$F_y = 50.0$	[ksi]	$F_u = 65.0$	[ksi]	
Bolt no in ver & hor dir	$n_v = 2$		$n_h = 7$		
Bolt spacing in ver & hor dir	$s_v = 5.500$	[in]	$s_h = 3.000$	[in]	
Bolt edge dist in ver & hor dir	$e_v = 1.745$	[in]	$e_h = 3.750$	[in]	

Gross area subject to shear	$A_{gv} = [(n_h - 1) s_h + e_h] t_p \times 2$	= 29.580	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5] d_h t_p \times 2$	= 20.740	[in^2]	
Net area subject to tension when sheared out by 2 side strips	$A_{nt} = (e_v - 0.5 d_h) t_p \times 2$	= 1.693	[in^2]	
Block shear strength required	$V_u =$	= **560.0**	[kips]	
Uniform tension stress factor	$U_{bs} = 1.00$			AISC 15th Fig C-J4.2
Ratio of expected Fy/Fu to specified min Fy/Fu	$R_y = 1.10$	$R_t = 1.10$		AISC 341-16 Table A3.1
Bolt shear resistance provided	$R_n = \min(0.6 R_t F_u A_{nv}, 0.6 R_y F_y A_{gv}) + U_{bs} R_t F_u A_{nt}$	= 1010.8	[kips]	AISC 15th Eq J4-5
Resistance factor-LRFD	$\phi_n = 0.90$			AISC 358-16 2.4.1
	$\phi R_n =$	= **909.7**	[kips]	
	ratio = **0.62**	> V_u	OK	

Seismic Flange Plate - Compression - Step 13		ratio = 560.0 / 607.5	= **0.92**	**PASS**
Flange force moment arm	$d_m = d_b + t_p$	= 25.400	[in]	
Flange force - compression	$P_u = P_b / 2 + M / d_m$	= **560.0**	[kips]	

Plate Compression Check

Plate size	width $b_p = 9.000$	[in]	thickness $t_p = 1.500$	[in]	
	$F_y = 50.0$	[ksi]	$E = 29000$	[ksi]	
Plate gross area in compression	$A_g = b_p t_p$		= 13.500	[in^2]	

Plate radius of gyration	$r = t_p / \sqrt{12}$	= 0.433	[in]	
Plate effective length factor	$K =$	= 0.65		
Plate unbraced length	$L_u =$	= 4.500	[in]	
Plate slenderness	$KL/r = 0.65 \times L_u / r$	= 6.75		
Plate compression required	$P_u =$	= **560.0**	[kips]	
	when $\frac{KL}{r} \le 25$			AISC 15th J4.4 (a)
Plate compression provided	$R_n = F_y \times A_g$	= 675.0	[kips]	AISC 15th Eq J4-6
Resistance factor-LRFD	$\phi_n = 0.90$			AISC 358-16 2.4.1
	$\phi R_n =$	= **607.5**	[kips]	
	ratio = **0.92**	> P_u	OK	

Seismic Beam Section Shear Strength - Step 14		ratio = 67.8 / 315.5	= **0.21**	**PASS**	
W Shape Beam Shear Yielding Check					
W sect W24X76	d = 23.900 [in]	t_w = 0.440	[in]		
	F_y = 50.0 [ksi]				
Beam to column shear	V_u = from calc above Seismic Moment and Beam Flange Force Calc	= **67.8**	[kips]		
Beam shear strength	$R_n = 0.6\, F_y\, d\, t_w$	= 315.5	[kips]	AISC 15th Eq J4-3	
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3	
	$\phi\, R_n$ =	= **315.5**	[kips]		
	ratio = **0.21**	> V_u	OK		

Seismic Beam Flange - Bolt Shear		ratio = 560.0 / 636.4	= **0.88**	**PASS**
Flange force moment arm	$d_m = d_b$	= 23.900	[in]	
Beam flange force as shear	$V_u = P_b / 2 + M / d_m$	= **560.0**	[kips]	
Bolt shear stress	bolt grade = A490-X	F_{nv} = 84.0	[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.875 [in]	bolt area A_b = 0.601	[in^2]	
Number of bolt carried shear	n_s = 14.0	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	= **560.0**	[kips]	
Bolt shear strength	$R_n = F_{nv}\, A_b\, n_s\, m\, C_{ec}$	= 707.2	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	ϕ_n = 0.90			AISC 358-16 2.4.1
	$\phi_n\, R_n$ =	= **636.4**	[kips]	
	ratio = **0.88**	> V_u	OK	

Seismic Beam Flange - Bolt Bearing		ratio = 560.0 / 636.4	= **0.88**	**PASS**
Flange force moment arm	$d_m = d_b - t_{fb}$	= 23.220	[in]	
Beam flange force as shear	$V_u = P_b / 2 + M / d_m$	= **560.0**	[kips]	

Single Bolt Shear Strength

Bolt shear stress	bolt grade = A490-X	F_{nv} = 84.0	[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.875 [in]	bolt area A_b = 0.601	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$	= 50.5	[kips]	AISC 15th Eq J3-1

Bolt Bearing/TearOut Strength on Plate

Bolt hole diameter	bolt dia d_b = $^7/_8$ [in]	bolt hole dia d_h = $^{15}/_{16}$	[in]	AISC 15th Table J3.3
Bolt spacing & edge distance	spacing L_s = 3.000 [in]	edge distance L_e = 3.750	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate thickness	t = 0.680 [in]			

Interior Bolt

Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.063	[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
	$= 109.4 \le 92.8$	= 92.8	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$	= 50.5	[kips]	

Edge Bolt

Bolt hole edge clear distance	$L_c = L_e - d_h / 2$	= 3.281	[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
	$= 174.0 \le 92.8$	= 92.8	[kips]	
Bolt strength at edge	$R_{n\text{-}ed} = \min(R_{n\text{-}t\&b\text{-}ed}, R_{n\text{-}bolt})$	= 50.5	[kips]	

Number of bolt	interior n_{in} = 12	edge n_{ed} = 2		
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-}in} + n_{ed} R_{n\text{-}ed}$	= 707.2	[kips]	
Required shear strength	V_u = see Seismic Moment and Beam Flange Force Calc	= **560.0**	[kips]	
Bolt resistance factor-LRFD	ϕ_n = 0.90			AISC 358-16 2.4.1
	$\phi_n R_n$ =	= **636.4**	[kips]	
	ratio = **0.88**	> V_u	OK	

Seismic Beam Web - Bolt Shear		ratio = 67.8 / 146.1	= **0.46**	**PASS**
Bolt shear stress	bolt grade = A325-N	F_{nv} = 54.0	[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.875 [in]	bolt area A_b = 0.601	[in^2]	
Number of bolt carried shear	n_s = 5.0	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	= **67.8**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 162.4	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	ϕ_n = 0.90			AISC 358-16 2.4.1
	$\phi_n R_n$ =	= **146.1**	[kips]	
	ratio = **0.46**	> V_u	OK	

Seismic Beam Web - Bolt Bearing			ratio = 67.8 / 146.1	= **0.46**	**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 = 0.875	[in]	bolt area A_b = 0.601	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$		= 32.5	[kips]	AISC 15th Eq J3-1
Bolt Bearing/TearOut Strength on Plate					
Bolt hole diameter	bolt dia d_b = $^7/_8$	[in]	bolt hole dia d_h = $^{15}/_{16}$	[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 = 0.440	[in]			
Interior Bolt					
Bolt hole edge clear distance	$L_c = L_s - d_h$		= 2.063	[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
	= 70.8 ≤ 60.1		= 60.1	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$		= 32.5	[kips]	
Number of bolt	interior n_{in} = 5				
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-}in}$		= 162.4	[kips]	
Required shear strength	V_u = see Seismic Moment and Beam Flange Force Calc		= **67.8**	[kips]	
Bolt resistance factor-LRFD	ϕ_n = 0.90				AISC 358-16 2.4.1
	$\phi_n R_n$ =		= **146.1**	[kips]	
	ratio = **0.46**		> V_u	OK	

Seismic Beam Web - Shear Yielding			ratio = 67.8 / 315.5	= **0.21**	**PASS**
Plate Shear Yielding Check					
Plate size	width b_p = 23.900	[in]	thickness t_p = 0.440	[in]	
Plate yield strength	F_y = 50.0	[ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$		= 10.516	[in^2]	
Shear force required	V_u =		= **67.8**	[kips]	
Plate shear yielding strength	$R_n = 0.6\, F_y A_{gv}$		= 315.5	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00				AISC 15th Eq J4-3
	ϕR_n =		= **315.5**	[kips]	
	ratio = **0.21**		> V_u	OK	

Seismic Beam Web - Shear Rupture			ratio = 67.8 / 243.2	= **0.28**	**PASS**
Plate Shear Rupture Check					
Bolt hole diameter	bolt dia $d_b = 7/8$	[in]	bolt hole dia $d_h = 1$	[in]	AISC 15th B4.3b
Number of bolt	$n = 5$				
Plate size	width $b_p = 23.900$	[in]	thickness $t_p = 0.440$	[in]	
Plate tensile strength	$F_u = 65.0$	[ksi]			
Plate net area in shear	$A_{nv} = (b_p - n\ d_h)\ t_p$		= 8.316	[in^2]	
Shear force required	$V_u =$		= **67.8**	[kips]	
Plate shear rupture strength	$R_n = 0.6\ F_u A_{nv}$		= 324.3	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	$\phi = 0.75$				AISC 15th Eq J4-4
	$\phi\ R_n =$		= **243.2**	[kips]	
	ratio = **0.28**		$> V_u$	OK	

Seismic Web Plate - Bolt Bearing			ratio = 67.8 / 144.0	= **0.47**	**PASS**
Single Bolt Shear Strength					
Bolt shear stress	bolt grade = A325-N		$F_{nv} = 54.0$	[ksi]	AISC 15th Table J3.2
	bolt dia $d_b = 0.875$	[in]	bolt area $A_b = 0.601$	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$		= 32.5	[kips]	AISC 15th Eq J3-1
Bolt Bearing/TearOut Strength on Plate					
Bolt hole diameter	bolt dia $d_b = 7/8$	[in]	bolt hole dia $d_h = 15/16$	[in]	AISC 15th Table J3.3
Bolt spacing & edge distance	spacing $L_s = 3.000$	[in]	edge distance $L_e = 1.500$	[in]	
Plate tensile strength	$F_u = 65.0$	[ksi]			
Plate thickness	$t = 0.375$	[in]			
Interior Bolt					
Bolt hole edge clear distance	$L_c = L_s - d_h$		= 2.063	[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
	$= 60.3 \le 51.2$		= 51.2	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min (R_{n\text{-}t\&b\text{-}in} , R_{n\text{-}bolt})$		= 32.5	[kips]	
Edge Bolt					
Bolt hole edge clear distance	$L_c = L_e - d_h / 2$		= 1.031	[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
	$= 30.2 \le 51.2$		= 30.2	[kips]	
Bolt strength at edge	$R_{n\text{-}ed} = \min (R_{n\text{-}t\&b\text{-}ed} , R_{n\text{-}bolt})$		= 30.2	[kips]	
Number of bolt	interior $n_{in} = 4$		edge $n_{ed} = 1$		
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-}in} + n_{ed} R_{n\text{-}ed}$		= 160.0	[kips]	
Required shear strength	V_u = see Seismic Moment and Beam Flange Force Calc		= **67.8**	[kips]	
Bolt resistance factor-LRFD	$\phi_n = 0.90$				AISC 358-16 2.4.1
	$\phi_n R_n =$		= **144.0**	[kips]	
	ratio = **0.47**		$> V_u$	OK	

Seismic Web Plate - Shear Yielding		ratio = 67.8 / 168.8	= **0.40**	**PASS**

Plate Shear Yielding Check

Plate size	width b_p = 15.000 [in]	thickness t_p = 0.375	[in]	
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$	= 5.625	[in^2]	
Shear force required	V_u =	= **67.8**	[kips]	
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$	= 168.8	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	ϕR_n =	= **168.8**	[kips]	
	ratio = **0.40**	> V_u	OK	

Seismic Web Plate - Shear Rupture		ratio = 67.8 / 109.7	= **0.62**	**PASS**

Plate Shear Rupture Check

Bolt hole diameter	bolt dia $d_b = {}^7/_8$ [in]	bolt hole dia d_h = 1	[in]	AISC 15th B4.3b
Number of bolt	n = 5			
Plate size	width b_p = 15.000 [in]	thickness t_p = 0.375	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate net area in shear	$A_{nv} = (b_p - n\ d_h)\ t_p$	= 3.750	[in^2]	
Shear force required	V_u =	= **67.8**	[kips]	
Plate shear rupture strength	$R_n = 0.6 F_u A_{nv}$	= 146.3	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-4
	ϕR_n =	= **109.7**	[kips]	
	ratio = **0.62**	> V_u	OK	

Seismic Web Plate - Block Shear - 1-Side Strip		ratio = 67.8 / 194.8	= **0.35**	**PASS**

Plate Block Shear - Side Strip

Bolt hole diameter	bolt dia $d_b = {}^7/_8$ [in]	bolt hole dia d_h = 1	[in]	AISC 15th B4.3b
Plate thickness	t_p = 0.375 [in]			
Plate strength	F_y = 50.0 [ksi]	F_u = 65.0	[ksi]	
Bolt no in ver & hor dir	n_v = 1	n_h = 5		
Bolt spacing in hor dir	s_h = 3.000 [in]			
Bolt edge dist in ver & hor dir	e_v = 2.500 [in]	e_h = 1.500	[in]	
Gross area subject to shear	$A_{gv} = [(n_h - 1)\ s_h + e_h]\ t_p$	= 5.063	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5]\ d_h t_p$	= 3.375	[in^2]	
Net area subject to tension	$A_{nt} = (e_v - 0.5\ d_h)\ t_p$	= 0.750	[in^2]	
Block shear strength required	V_u =	= **67.8**	[kips]	
Uniform tension stress factor	U_{bs} = 1.00			AISC 15th Fig C-J4.2
Ratio of expected Fy/Fu to specified min Fy/Fu	R_y = 1.10	R_t = 1.20		AISC 341-16 Table A3.1
Bolt shear resistance provided	$R_n = \min (0.6 R_t F_u A_{nv}, 0.6 R_y F_y A_{gv}) + U_{bs} R_t F_u A_{nt}$	= 216.5	[kips]	AISC 15th Eq J4-5
Resistance factor-LRFD	ϕ_n = 0.90			AISC 358-16 2.4.1
	ϕR_n =	= **194.8**	[kips]	
	ratio = **0.35**	> V_u	OK	

Seismic Web Plate Weld Strength ratio = 67.8 / 167.0 = **0.41** **PASS**

Shear force in demand	V_u =	= **67.8**	[kips]	
Fillet weld length - double fillet	L =	= 15.000	[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 - web plate	thickness t = 0.375 [in] tensile F_u = 65.0		[ksi]	
Base metal - web 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	$R_{n\text{-}b} = 0.6\ F_u\ t$	= 14.625	[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	$R_n = \min \left(R_{n\text{-}w} , R_{n\text{-}b} \times \frac{0.90}{0.75} \right)$	= **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	V_u =	= **67.8**	[kips]	
Fillet weld length - double fillet	L =	= 15.000	[in]	
Shear resistance provided	$\phi F_n = \phi R_n \times L$	= **167.0**	[kips]	
	ratio = **0.41**	> V_u	OK	

Seismic Column Panel Zone Thickness **PASS**

Check Column Web Thickness

Condition : column web thickness $t_{wc} \geq (d_z + w_z) /90$ — AISC 341-16 Eq E3-7

Beam sect W24X76	d_b = 23.900 [in]	t_{fb} = 0.680	[in]	
Beam clear dist between flanges	$d_z = d_b - 2\ t_{fb}$	= 22.540	[in]	

Column sect W24X76	d_c = 15.200 [in]	t_{fc} = 1.310	[in]	
Column width of panel zone between flanges	$w_z = d_c - 2\ t_{fc}$	= 12.580	[in]	

Column web thickness required	$t_{min} = (d_z + w_z) /90$	= **0.390**	[in]	AISC 341-16 Eq E3-7
Doubler plate No & thickness	No = 2	t_{db} = 0.500	[in]	
Column web thickness	t_{wc} = 0.830 [in]			

When the doubler plates are plug welded to the column web, the total panel zone thickness is the sum of doubler plates and column web — AISC 341-16 E3.6e (2)

Total panel zone thickness	$t = t_{wc} + 2 \times t_{db}$	= **1.830**	[in]	
		$\geq t_{min}$	OK	AISC 341-16 E3.6e (2)

Seismic Column Flange Bending		560.0 / 482.7		**N/A**
Flange force moment arm	$d_m = d_b + t_p$	= 25.400	[in]	
Flange force required - tension	$P_{uf\_t} = P_u / 2 - M_u / d_m$	= **560.0**	[kips]	
Column flange thickness	t_{fc} = 1.310 [in]	yield F_{yc} = 50.0	[ksi]	
Top column condition	it's not a top column case	C_t = 1.0		AISC 15th J10.1
Column flange tensile resistance	$R_n = C_t 6.25 F_{yc} t_{fc}^2$	= 536.3	[kips]	AISC 15th Eq J10-1
Resistance factor-LRFD	ϕ = 0.90			AISC 15th J10.1
	ϕR_n =	= **482.7**	[kips]	
Unbalanced force to be resisted by transverse stiffeners	$R_s = P_{uf\_t} - \phi R_n$	= **77.3**	[kips]	

Seismic Column Web Yielding		ratio = 560.0 / 1011.1	= **0.55**	**PASS**
Column web thickness	t_{wc} = 0.830 [in]	yield F_y = 50.0	[ksi]	
Doubler plate thickness	t_{dp} = 0.500 [in]	yield F_{ydp} = 50.0	[ksi]	
Equivalent web thickness when considering doubler plate	$t_{w\text{-}eq} = t_{wc} + t_{dp} \times F_{ydp} / F_y \times 2$ sides	= 1.830	[in]	
Flange force moment arm	$d_m = d_b + t_p$	= 25.400	[in]	
Seismic flange force	P_{uf_c} = see Seismic Moment and Beam Flange Force Calc	= **560.0**	[kips]	
Column section	d_c = 15.200 [in]	t_{fc} = 1.310	[in]	
	t_{wc} = 0.830 [in]	k_c = 1.910	[in]	
Column yield strength	F_{yc} = 50.0 [ksi]			
Top column condition	it's not a top column case			AISC 15th J10.2 (a)
Flange plate fillet weld size	w = 0.000 [in]	flange plate t_p = 1.500	[in]	
Length of bearing	$N = t_p + 2w$	= 1.500	[in]	AISC DG4 Eq 3.24
Column web yielding strength	$R_n = (5 k_c + N) F_{yc} t_{w\text{-}eq}$	= 1011.1	[kips]	AISC 15th Eq J10-2
Resistance factor-LRFD	ϕ = 1.00			AISC 341-16 E3.6e (1)
	ϕR_n =	= **1011.1**	[kips]	
	ratio = **0.55**	> P_{uf}	OK	

Seismic Column Web Buckling		ratio = 560.0 / 2087.0	= **0.27**	**PASS**
Flange force moment arm	$d_m = d_b + t_p$	= 25.400	[in]	
Seismic flange force	P_{uf_c} = see Seismic Moment and Beam Flange Force Calc	= **560.0**	[kips]	
Column section	d_c = 15.200 [in]	t_{fc} = 1.310	[in]	
	t_{wc} = 0.830 [in]	k_c = 1.910	[in]	
	$h = d_c - 2\,k_c$	= 11.380	[in]	
Column yield strength	F_{yc} = 50.0 [ksi]	E_c = 29000	[ksi]	
Top column condition	it's not a top column case	C_t = 1.0		AISC 15th J10.5
Column web buckling strength	$R_{n1} = \dfrac{C_t\,24\,t_{wc}^3\,\sqrt{E_c\,F_{yc}}}{h}$	= 1452.1	[kips]	AISC 15th Eq J10-8
Doubler plate thickness	t_{dp} = 0.500 [in]	yield F_{ydp} = 50.0	[ksi]	
Doubler plate buckling strength	$R_{n2} = \dfrac{2\,C_t\,24\,t_{dp}^3\,\sqrt{E_c\,F_{ydp}}}{h}$	= 634.9	[kips]	AISC 15th Eq J10-8
Total buckling strength	$R_n = R_{n1} + R_{n2}$	= 2087.0	[kips]	
Resistance factor-LRFD	ϕ = 1.00			AISC 341-16 E3.6e (1)
	$\phi\,R_n$ =	= **2087.0**	[kips]	
	ratio = **0.27**	> P_{uf_c}	OK	

Seismic Column Web Crippling		ratio = 560.0 / 1792.3 = **0.31**		**PASS**
Flange force moment arm	$d_m = d_b + t_p$	= 25.400	[in]	
Seismic flange force	P_{uf_c} = see Seismic Moment and Beam Flange Force Calc	= **560.0**	[kips]	
Column section	d_c = 15.200 [in]	t_{fc} = 1.310	[in]	
	t_{wc} = 0.830 [in]	k_c = 1.910	[in]	
Column yield strength	F_{yc} = 50.0 [ksi]	E_c = 29000	[ksi]	
Flange plate fillet weld size	w = 0.000 [in]	flange plate t_p = 1.500	[in]	
Length of bearing	$l_b = t_p + 2w$	= 1.500	[in]	
Distance from top of column to top of beam flange	$d_{end\text{-}flg}$ =	= 4.000	[in]	
Top column condition	it's not a top column case, use Eq J10-4			AISC 15th J10.3 (a)
Column web crippling strength	$R_{n1} = 0.8\, t_{wc}^2 \left[1+3\frac{l_b}{d_c}\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right] \times \left(\frac{E_c F_{yc} t_{fc}}{t_{wc}}\right)^{0.5}$	= 958.2	[kips]	AISC 15th Eq J10-4
Doubler plate thickness	t_{dp} = 0.500 [in]	yield F_{ydp} = 50.0	[ksi]	
Doubler plate crippling strength	$R_{n2} = 0.8\, t_{dp}^2 \left[1+3\frac{l_b}{d_c}\left(\frac{t_{dp}}{t_{fc}}\right)^{1.5}\right] \times \left(\frac{E_c F_{ydp} t_{fc}}{t_{dp}}\right)^{0.5}$	= 834.1	[kips]	AISC 15th Eq J10-4
Total crippling strength	$R_n = R_{n1} + R_{n2}$	= 1792.3	[kips]	
Resistance factor-LRFD	ϕ = 1.00			AISC 341-16 E3.6e (1)
	ϕR_n =	= **1792.3**	[kips]	
	ratio = **0.31**	> P_{uf_c}	OK	

Seismic Column Panel Zone Shear		836.9 / 468.6		**N/A**
Seismic panel zone shear in demand	V_p = see Seismic Moment and Beam Flange Force Calc	= **836.9**	[kips]	
Column W14X176	d_c = 15.200 [in]	b_{cf} = 15.700	[in]	
	t_{cf} = 1.310 [in]	t_{cw} = 0.830	[in]	
	A_c = 51.800 [in^2]	F_{cy} = 50.0	[ksi]	
Beam W24X76	d_b = 23.900 [in]	t_{bf} = 0.680	[in]	
Flange plate thickness	t_p = from user input	= 1.500	[in]	
Moment arm between flange plates	$d_m = d + t_p$	= 26.900	[in]	AISC 358-16 Eq 7.6-7
Column axial compression	P_r = from user input	= 249.0	[kips]	
Column axial yield strength	$P_y = F_{cy} A_c$	= 2590.0	[kips]	AISC 15th J10.6 (b)
LRFD-ASD force adjustment factor	α = for LRFD	= 1.0		AISC 341-16 D1.2a (b)
	when $\alpha P_r \leq 0.75\ P_y$, use Eq J10-11			AISC 15th Eq J10-11
Column web panel zone capacity	$R_n = 0.6\ F_{cy}\ d_c\ t_{cw}\ (1 + \dfrac{3\ b_{cf}\ t_{cf}^2}{d_m\ d_c\ t_{cw}})$	= 468.6	[kips]	AISC 15th Eq J10-11
Resistance factor-LRFD	ϕ_v = 1.00			AISC 341-16 E3.6e (1)
	$\phi_v R_n$ =	= **468.6**	[kips]	
Unbalanced force to be resisted by doubler plate	$V_{dp} = R_p - \phi R_n$	= **368.3**	[kips]	

Seismic Stiffener Geometry Restriction				**PASS**
Stiffener plate width	b_s = 6.000 [in]	depth d_s = 12.580	[in]	
Stiffener plate thickness	t_s = 1.125 [in]			
Column flange thickness	t_{fc} = 1.310 [in]	column depth d_c = 15.200	[in]	
Beam flange thickness	t_{fb} = 0.680 [in]			
Min Stiffener Plate Thickness				
Min stiffener plate thickness	t_{smin} = max (t_{fb} / 2 , b_s / 16)	= **0.375**	[in]	AISC 15th J10.8 (2)
Stiffener plate thickness	t_s =	= **1.125**	[in]	
		$\geq t_{smin}$	OK	
Min Stiffener Plate Depth				
Min stiffener plate depth	d_{smin} = (d_c - 2 t_{fc}) / 2	= **6.290**	[in]	AISC 15th J10.8 (3)
Stiffener plate depth	d_s =	= **12.580**	[in]	
		$\geq d_{smin}$	OK	

Seismic Stiffener Yield at Column Flange		ratio = 77.3 / 455.6	= **0.17**	**PASS**
Stiffener plate width	b_s = 5.500 [in]	thickness t_s = 1.125	[in]	
Stiffener plate corner clip	clip =	= 1.000	[in]	AISC 15th Page 8-18
Stiffener plate yield strength	F_y =	= 50.0	[ksi]	
Stiffener plate cross sect area	A_{st} = (b_s - clip) t_s	= 5.063	[in^2]	
Trans stiffener strength required	R_s =	= **77.3**	[kips]	
Trans stiffener strength provided	$R_n = F_y \times 2 \times A_{st}$	= 506.3	[kips]	AISC 15th Eq J4-1
Bolt resistance factor-LRFD	ϕ = 0.90			AISC 15th Eq J4-1
	ϕR_n =	= **455.6**	[kips]	
	ratio = **0.17**	$> R_s$	OK	

Seismic Stiffener Shear at Column Web		ratio = 77.3 / 714.2	= **0.11**	**PASS**
Stiffener plate depth	d_s = 12.580 [in]	thickness t_s = 1.125	[in]	
Stiffener plate corner clip	clip =	= 1.000	[in]	AISC 15th Page 8-18
Stiffener plate yield strength	F_y =	= 50.0	[ksi]	
Stiffener plate cross sect area	$A_{gv} = (d_s - 2 \times clip) t_s$	= 11.903	[in^2]	
Trans stiffener strength required	R_s =	= **77.3**	[kips]	
Trans stiffener strength provided	$R_n = 2 \times 0.6 \times F_y \times A_{gv}$	= 714.2	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	ϕR_n =	= **714.2**	[kips]	
	ratio = **0.11**	> R_s	OK	

Seismic Stiffener to Column Web Fillet Weld Limitation				**PASS**
Min Fillet Weld Size				
Thinner part joined thickness	t =	= 0.830	[in]	
Min fillet weld size allowed	w_{min} =	= **0.313**	[in]	AISC 15th Table J2.4
Fillet weld size provided	w =	= **0.313**	[in]	
		≥ w_{min}	OK	
Min Fillet Weld Length				
Fillet weld size provided	w =	= 0.313	[in]	
Min fillet weld length allowed	$L_{min} = 4 \times w$	= **1.250**	[in]	AISC 15th J2.2b
Min fillet weld length	$L = d_c - 2 \times t_{fc} - 2 \times clip$	= **10.580**	[in]	
		≥ L_{min}	OK	

Seismic Stiffener Weld Strength at Column Web

ratio = 77.3 / 294.5 = **0.26** **PASS**

Stiffener to Column Web Weld Length Calc

Column section	d_c = 15.200 [in]	t_{fc} = 1.310 [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	= **21.160** [in]	
Trans stiffener strength required	R_s =	= **77.3** [kips]	

Fillet Weld Strength Check

Fillet weld leg size	$w = {}^{5}/_{16}$ [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	$R_{n\text{-}w} = 0.6 (C_1 \times 70 \text{ ksi}) 0.707\ w\ n\ C_2$	= 18.559 [kip/in]	AISC 15th Eq 8-1
Base metal - stiffener plate	thickness t = 1.125 [in]	tensile F_u = 65.0 [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	$R_{n\text{-}b} = 0.6\ F_u\ t$	= 43.875 [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	$R_n = \min \left(R_{n\text{-}w} , R_{n\text{-}b} \times \frac{0.90}{0.75} \right)$	= **18.559** [kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75		AISC 15th Eq 8-1
	ϕR_n =	= **13.919** [kip/in]	
Shear resistance required	R_s =	= **77.3** [kips]	
Fillet weld length - double fillet	L =	= 21.160 [in]	
Shear resistance provided	$\phi F_n = \phi R_n \times L$	= **294.5** [kips]	
	ratio = **0.26**	> R_s OK	

Seismic Doubler Plate Thickness

PASS

Check Doubler Plate Thickness

Condition : Doubler plate thickness $t_{db} \geq (d_z + w_z) /90$ — AISC 341-16 Eq E3-7

Beam sect W24X76	d_b = 23.900 [in]	t_{fb} = 0.680 [in]	
Beam clear dist between flanges	$d_z = d_b - 2\ t_{fb}$	= 22.540 [in]	
Column sect W24X76	d_c = 15.200 [in]	t_{fc} = 1.310 [in]	
Column width of panel zone between flanges	$w_z = d_c - 2\ t_{fc}$	= 12.580 [in]	
Doubler plate thickness required	$t = (d_z + w_z) /90$	= **0.390** [in]	AISC 341-16 Eq E3-7
Doubler plate thickness	t_{db} =	= **0.500** [in]	
		≥ t OK	AISC 341-16 E3.6e (2)

Seismic Doubler Plate Shear Yield		ratio = 368.3 / 456.0	= **0.81**	**PASS**
Doubler plate thickness	t_{dp} = 0.500 [in]	yield strength F_y = 50.0	[ksi]	
Number of doubler	N_{dbl} = 2 for two side			
Column depth	d_c = 15.200 [in]			
Area of web	$A_w = t_{dp} \times d_c$	= 7.600	[in^2]	AISC DG13 Eq 4.4-1
Shear coefficient	C_v = 1.0			AISC 15th Eq G2-2
Doubler plate shear in demand	R_{dp} = unbalanced force from column panel zone shear calc	= **368.3**	[kips]	
Doubler plate shear strength	$R_n = N_{dbl}\, 0.6\, F_y A_w C_v$	= 456.0	[kips]	AISC 15th Eq G2-1
Resistance factor-LRFD	ϕ = 1.00			AISC 15th G2.1 (a)
	ϕR_n =	= **456.0**	[kips]	
	ratio = **0.81**	> R_{dp}	OK	