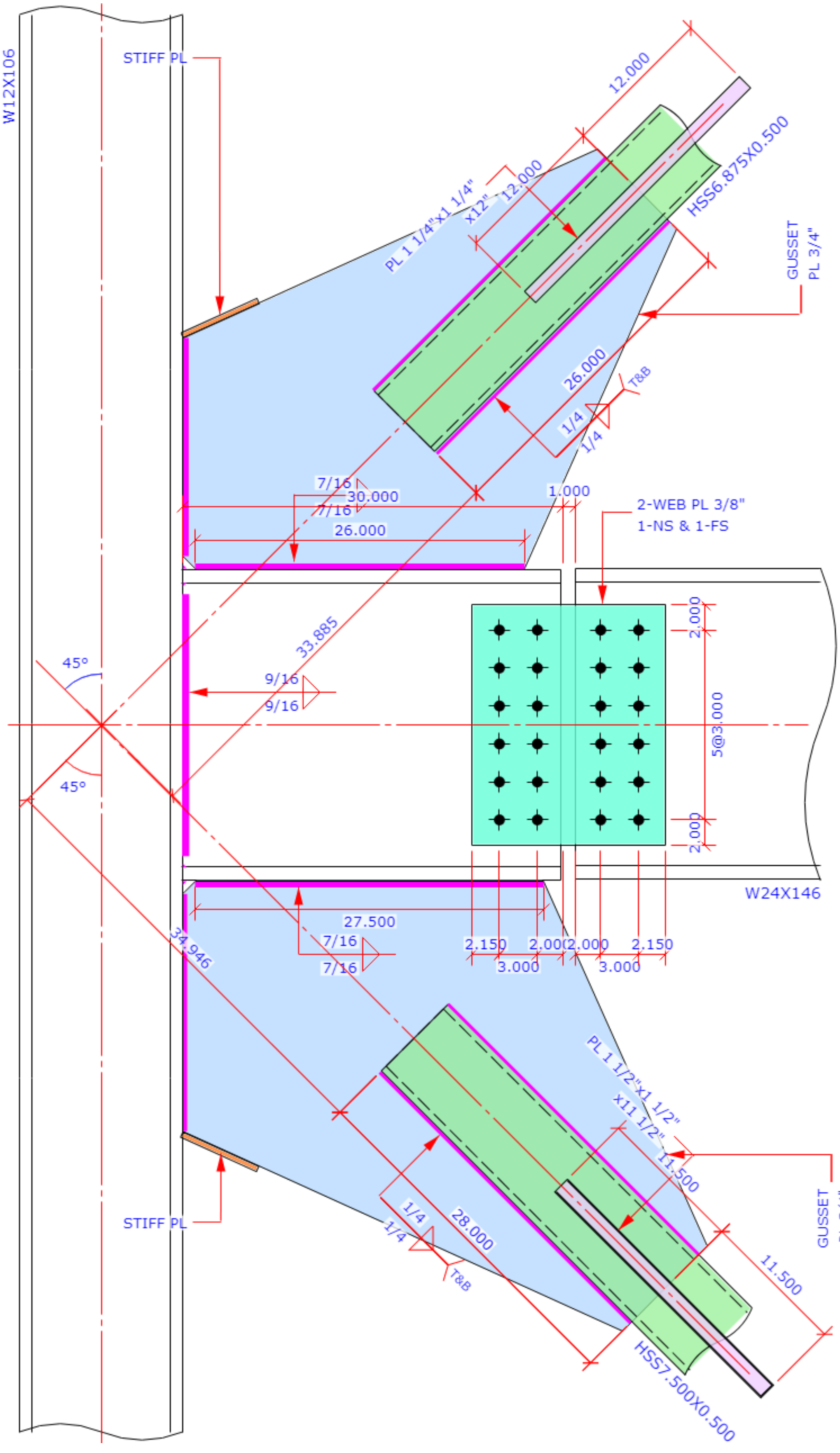
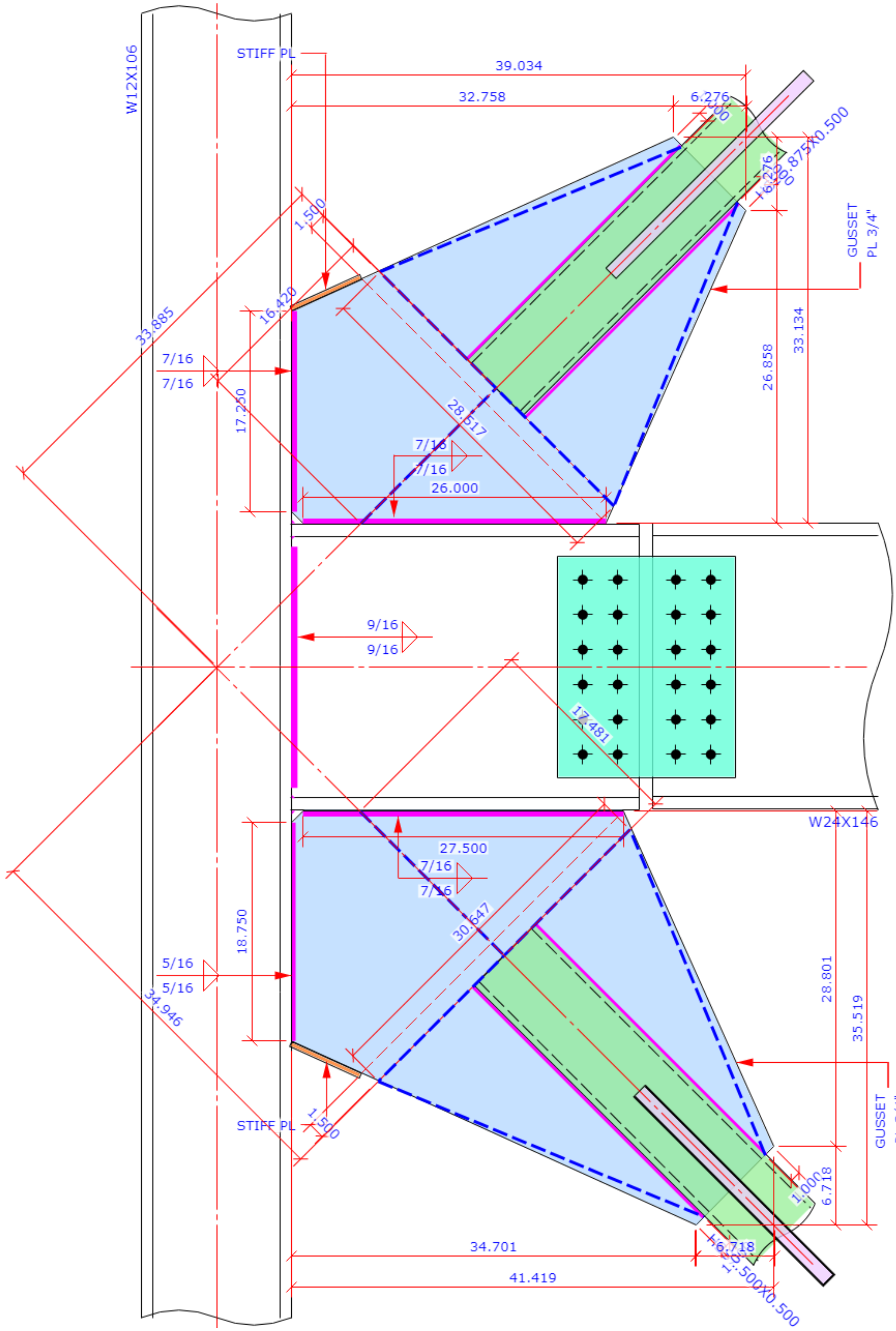
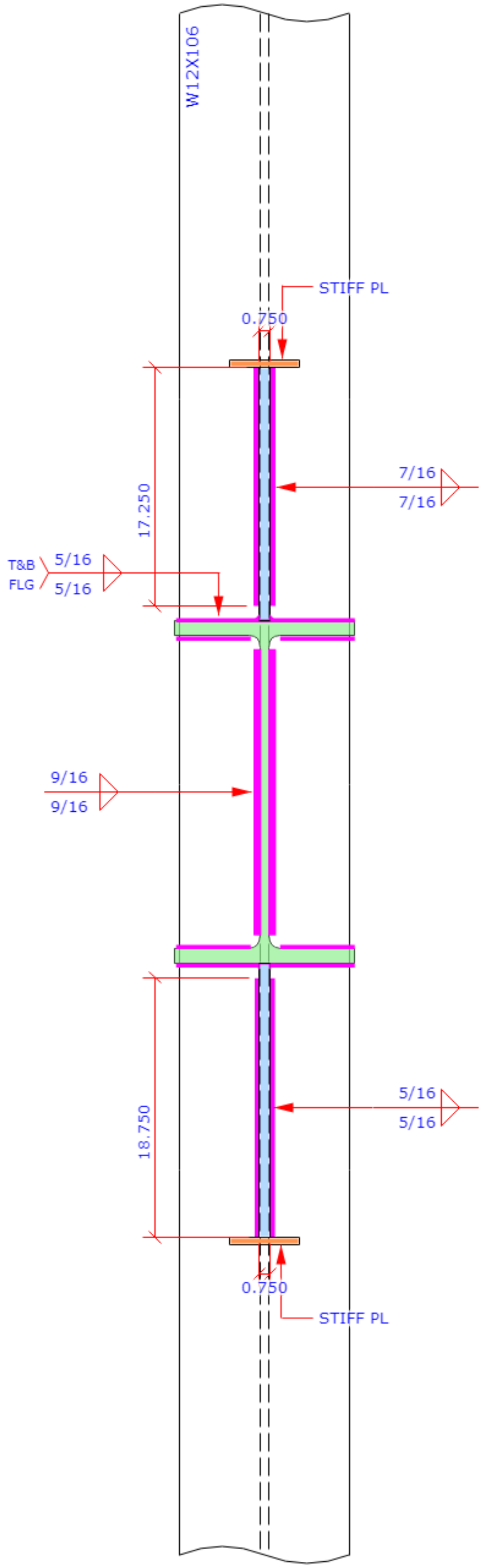
Sketch	Vertical Brace Connection	Code=AISC 360-16 LRFD




W12X106
STIFF PL
39.034
32.758
6.276
GUSSET
PL 3/4"
33.885
16.420
1.500
7/16
7/16
17.250
28.517
26.000
26.858
33.134
9/16
9/16
17.481
W24X146
27.500
18.750
5/16
5/16
34.946
30.647
28.801
35.519
STIFF PL
1.500
34.701
41.419
6.718
1.000



W12X106
STIFF PL
0.750
17.250
7/16
7/16
T&B
FLG
5/16
5/16
9/16
9/16
5/16
5/16
18.750
0.750
STIFF PL

Result Summary - Overall Seismic Braced Frame - SCBF Code=AISC 360-16 LRFD

Result Summary - Overall geometries & weld limitations = **PASS** limit states max ratio = **1.01** **FAIL**

Seismic - SCBF Load Case LC1 & LC2

Top Brace - Brace to Gusset	geometries & weld limitations = **PASS**	limit states max ratio = **0.97** **PASS**
Top Brace - Gusset to Column	geometries & weld limitations = **PASS**	limit states max ratio = **0.82** **PASS**
Top Brace - Gusset to Beam	geometries & weld limitations = **PASS**	limit states max ratio = **0.79** **PASS**
Bot Brace - Brace to Gusset	geometries & weld limitations = **PASS**	limit states max ratio = **1.01** **FAIL**
Bot Brace - Gusset to Column	geometries & weld limitations = **PASS**	limit states max ratio = **0.98** **PASS**
Bot Brace - Gusset to Beam	geometries & weld limitations = **PASS**	limit states max ratio = **0.82** **PASS**
Beam to Column	geometries & weld limitations = **PASS**	limit states max ratio = **0.97** **PASS**
Beam Splice	geometries & weld limitations = **PASS**	limit states max ratio = **0.28** **PASS**

Seismic - SCBF Load Case LC3 & LC4

Top Brace - Gusset to Column	geometries & weld limitations = **PASS**	limit states max ratio = **0.82** **PASS**
Top Brace - Gusset to Beam	geometries & weld limitations = **PASS**	limit states max ratio = **0.79** **PASS**
Bot Brace - Gusset to Column	geometries & weld limitations = **PASS**	limit states max ratio = **0.98** **PASS**
Bot Brace - Gusset to Beam	geometries & weld limitations = **PASS**	limit states max ratio = **0.82** **PASS**
Beam to Column	geometries & weld limitations = **PASS**	limit states max ratio = **0.69** **PASS**
Beam Splice	geometries & weld limitations = **PASS**	limit states max ratio = **0.61** **PASS**

Seismic Calculation Brace Seismic System = SCBF Code=AISC 360-16 LRFD

Seismic Brace Axial Forces Calc & Design Cases Summary

Top Brace Section Properties & Member Data

Brace sect HSS6.875X0.500	Grade = A500 Gr.C Round	$F_y = 46.0$ [ksi]	
Ratio of expected Fy to specified min Fy	$R_y = 1.30$		AISC 341-16 Table A3.1
	$A_g = 9.360$ [in^2]	$r_y = 2.272$ [in]	
	$E = 29000$ [ksi]		
Brace member length & effective length factor K	$L = 144.0$ [in]	$K = 1.00$	

Top Brace Seismic Design Force in Tension

Brace expected yield strength in tension	$P_{et} = R_y F_y A_g$	= 559.7 [kips]	AISC 341-16 F2.6c (1)
Top Brace seismic design force in tension	$P_{s\_t} = P_{et}$	= **-559.7** [kips]	AISC 341-16 F2.6c (1)

Top Brace Seismic Design Force in Compression

Member length L & effective length factor K	L = 144.0	[in]	K = 1.00		
Member radius of gyration & elastic modulus	r = 2.272	[in]	E = 29000	[ksi]	
Member slenderness ratio	KL/r = K x L / r		= 63.37		
Elastic buckling stress	$F_e = \dfrac{\pi^2 E}{(KL/r)^2}$		= 71.27	[ksi]	AISC 15th Eq E3-4
	when $\dfrac{KL}{r} \le 4.71 \left(\dfrac{E}{R_y F_y}\right)^{0.5} = 103.72$				AISC 15th E3
Critical stress	$F_{cr} = 0.658^{(R_y F_y / F_e)} R_y F_y$		= 42.09	[ksi]	AISC 15th Eq E3-2
Brace expected yield strength in compression	$P_{ec} = \min(R_y F_y A_g, 1.14 F_{cr} A_g)$		= 449.1	[kips]	AISC 341-16 F2.3
Brace force in compression	P_c = from user input in load section		= 0.0	[kips]	
Top Brace seismic design force in compression	$P_{s\_ci} = P_{ec}$		= **449.1**	[kips]	AISC 341-16 F2.6c (2)
Top Brace seismic design force in compression - post-buckling	$P_{s\_cii} = 0.3 \times P_{ec}$		= **134.7**	[kips]	AISC 341-16 F2.3 (ii)

Bot Brace Section Properties & Member Data

Brace sect HSS7.500X0.500	Grade = A500 Gr.C Round		F_y = 46.0	[ksi]	
Ratio of expected Fy to specified min Fy	R_y = 1.30				AISC 341-16 Table A3.1
	A_g = 10.300	[in²]	r_y = 2.493	[in]	
	E = 29000	[ksi]			
Brace member length & effective length factor K	L = 144.0	[in]	K = 1.00		

Bot Brace Seismic Design Force in Tension

Brace expected yield strength in tension	$P_{et} = R_y F_y A_g$	= 615.9	[kips]	AISC 341-16 F2.6c (1)
Bot Brace seismic design force in tension	$P_{s\_t} = P_{et}$	= **-615.9**	[kips]	AISC 341-16 F2.6c (1)

Bot Brace Seismic Design Force in Compression

Member length L & effective length factor K	L = 144.0	[in]	K = 1.00		
Member radius of gyration & elastic modulus	r = 2.493	[in]	E = 29000	[ksi]	
Member slenderness ratio	KL/r = K x L / r		= 57.77		
Elastic buckling stress	$F_e = \dfrac{\pi^2 E}{(KL/r)^2}$		= 85.76	[ksi]	AISC 15th Eq E3-4
	when $\dfrac{KL}{r} \le 4.71 \left(\dfrac{E}{R_y F_y}\right)^{0.5} = 103.72$				AISC 15th E3
Critical stress	$F_{cr} = 0.658^{(R_y F_y / F_e)} R_y F_y$		= 44.66	[ksi]	AISC 15th Eq E3-2
Brace expected yield strength in compression	$P_{ec} = \min(R_y F_y A_g, 1.14 F_{cr} A_g)$		= 524.4	[kips]	AISC 341-16 F2.3
Brace force in compression	P_c = from user input in load section		= 0.0	[kips]	

Bot Brace seismic design force in compression	$P_{s\_ci} = P_{ec}$	= **524.4**	[kips]	AISC 341-16 F2.6c (2)
Bot Brace seismic design force in compression - post-buckling	$P_{s\_cii} = 0.3 \times P_{ec}$	= **157.3**	[kips]	AISC 341-16 F2.3 (ii)

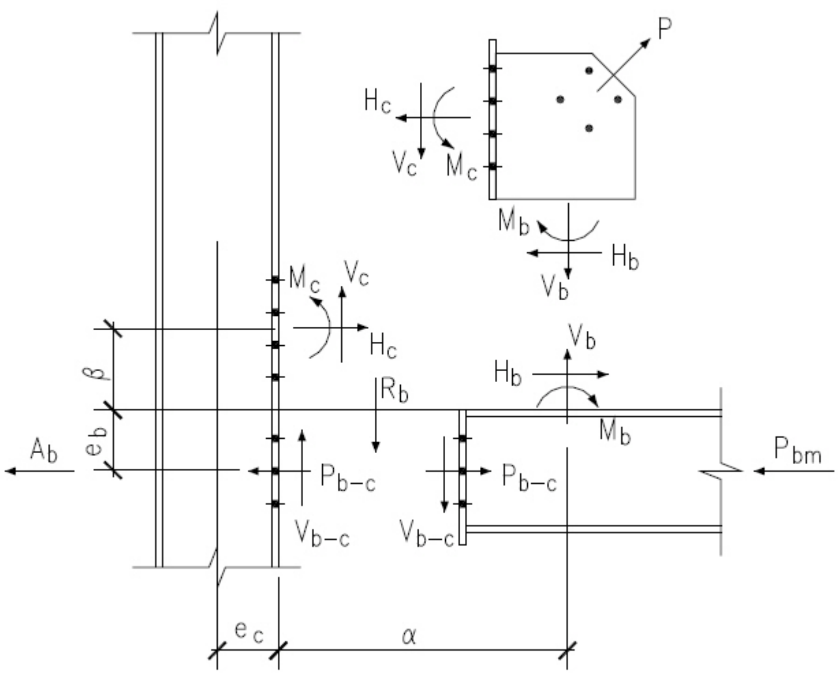
Brace Axial Force Design Cases Summary

Refer to AISC 341-16 F2.3(i), LC1 & LC2 are the load cases in which all braces are assumed to resist forces corresponding to their expected strength in tension P_{s_t} or in compression P_{s_ci}

F2.3(ii), LC3 & LC4 are the load cases in which all braces are assumed to resist forces corresponding to their expected strength in tension P_{s_t} and all braces in compression are assumed to resist their expected compressive post-buckling strength P_{s_cii}

LC1	Top Brace	P_{s_t} =-559.7 kips (T)	Bot Brace	P_{s_ci} =524.4 kips (C)		AISC 341-16 F2.3(i)
LC2	Top Brace	P_{s_ci} =449.1 kips (C)	Bot Brace	P_{s_t} =-615.9 kips (T)		
LC3	Top Brace	P_{s_t} =-559.7 kips (T)	Bot Brace	P_{s_cii} =157.3 kips (C)	post-buckling	AISC 341-16 F2.3(ii)
LC4	Top Brace	P_{s_cii} =134.7 kips (C)	Bot Brace	P_{s_t} =-615.9 kips (T)	post-buckling	

Seismic - SCBF LC1 & LC2 Gusset Interface Forces Calc



Brace - SCBF Load Case LC1

Top and bottom brace force	Top P_{top} = -559.7	[kips] (T)	Bot P_{bot} = 524.4	[kips] (C)	
Beam end shear & transfer force	Shear R_b = 19.9	[kips]	Transfer A_b = -46.3	[kips]	

Top Brace Interface Forces

Refer to AISC 15th Page 13-4 and Fig. 13-2 for all charts and definitions of variables and symbols shown in calculation below

e_b = 12.350	[in]	e_c = 6.450	[in]	
α = 14.000	[in]	β = 9.625	[in]	
θ = 45.0	[°]			
$K = e_b \tan\theta - e_c$		= 5.900	[in]	AISC 15th Eq. 13-16

	$D = \tan^2\theta + (\frac{\alpha}{\beta})^2$	= 3.116		AISC 15th Eq. 13-24
	$K' = \alpha\,(\tan\theta + \frac{\alpha}{\beta})$	= 34.364		AISC 15th Eq. 13-23
	$\bar{\alpha} = [\,K'\tan\theta + K\,(\frac{\alpha}{\beta})^2\,] / D$	= 14.000	[in]	AISC 15th Eq. 13-21
	$\bar{\beta} = (\,K' - K\tan\theta\,) / D$	= 8.100	[in]	AISC 15th Eq. 13-22
	$r = [\,(e_b + \bar{\beta})^2 + (e_c + \bar{\alpha})^2\,]^{0.5}$	= 28.921	[in]	AISC 15th Eq. 13-6
Brace axial force	P_u = from seismic brace force calc	= -559.7	[kips]	in tension
Gusset to Column Interface Forces				
Shear force	$V_c = (\bar{\beta} / r)\,P_u$	= **-156.8**	[kips]	AISC 15th Eq. 13-2
Axial force	$H_c = (e_c / r)\,P_u$	= **-124.8**	[kips]	AISC 15th Eq. 13-3
Moment	$M_c = H_c(\beta - \bar{\beta})$	= **15.86**	[kip-ft]	AISC 15th Eq. 13-19
Gusset to Beam Interface Forces				
Shear force	$H_b = (\bar{\alpha} / r)\,P_u$	= **-270.9**	[kips]	AISC 15th Eq. 13-5
Axial force	$V_b = (e_b / r)\,P_u$	= **-239.0**	[kips]	AISC 15th Eq. 13-4
Moment	$M_b = V_b(\bar{\alpha} - \alpha)$	= **0.00**	[kip-ft]	AISC 15th Eq. 13-17

Bottom Brace Interface Forces

Refer to AISC 15th Page 13-4 and Fig. 13-2 for all charts and definitions of variables and symbols shown in calculation below

	e_b = 12.350 [in]	e_c = 6.450	[in]	
	α = 14.750 [in]	β = 10.375	[in]	
	θ = 45.0 [°]			
	$K = e_b\tan\theta - e_c$	= 5.900	[in]	AISC 15th Eq. 13-16
	$D = \tan^2\theta + (\frac{\alpha}{\beta})^2$	= 3.021		AISC 15th Eq. 13-24
	$K' = \alpha\,(\tan\theta + \frac{\alpha}{\beta})$	= 35.720		AISC 15th Eq. 13-23
	$\bar{\alpha} = [\,K'\tan\theta + K\,(\frac{\alpha}{\beta})^2\,] / D$	= 14.750	[in]	AISC 15th Eq. 13-21
	$\bar{\beta} = (\,K' - K\tan\theta\,) / D$	= 8.850	[in]	AISC 15th Eq. 13-22
	$r = [\,(e_b + \bar{\beta})^2 + (e_c + \bar{\alpha})^2\,]^{0.5}$	= 29.981	[in]	AISC 15th Eq. 13-6
Brace axial force	P_u = from seismic brace force calc	= 524.4	[kips]	in compression
Gusset to Column Interface Forces				
Shear force	$V_c = (\bar{\beta} / r)\,P_u$	= **154.8**	[kips]	AISC 15th Eq. 13-2
Axial force	$H_c = (e_c / r)\,P_u$	= **112.8**	[kips]	AISC 15th Eq. 13-3
Moment	$M_c = H_c(\beta - \bar{\beta})$	= **-14.34**	[kip-ft]	AISC 15th Eq. 13-19
Gusset to Beam Interface Forces				
Shear force	$H_b = (\bar{\alpha} / r)\,P_u$	= **258.0**	[kips]	AISC 15th Eq. 13-5
Axial force	$V_b = (e_b / r)\,P_u$	= **216.0**	[kips]	AISC 15th Eq. 13-4
Moment	$M_b = V_b(\bar{\alpha} - \alpha)$	= **0.00**	[kip-ft]	AISC 15th Eq. 13-17

Beam to Column Interface Forces

Beam to Column Interface Shear Force

Beam end shear reaction	R_b = from user input	= 19.9	[kips]

Top brace gusset-beam axial force	$V_{b\text{-}top} =$	= -239.0	[kips]	AISC 15th Eq. 13-4
Bot brace gusset-beam axial force	$V_{b\text{-}bot} =$	= 216.0	[kips]	AISC 15th Eq. 13-4
Beam to column shear force	$V_{b\text{-}c} = R_b + V_{b\text{-}top} - V_{b\text{-}bot}$	= **-435.1**	[kips]	AISC 15th Page 13-4

Beam to Column Interface Axial Force

Top brace gusset-column axial force	$H_{c\text{-}top} =$	= -124.8	[kips]	AISC 15th Eq. 13-3
Bot brace gusset-column axial force	$H_{c\text{-}bot} =$	= 112.8	[kips]	AISC 15th Eq. 13-3
Transfer force from adjacent bay	$A_b =$ from user input	= -46.3	[kips]	
Beam to column axial force	$P_{b\text{-}c} = (H_{c\text{-}top} + H_{c\text{-}bot}) \times -1 - A_b$	= **58.3**	[kips]	AISC 15th Page 13-4

Beam Member Axial Force

This force is not for use in connection calc. It's output here for user input connection forces equilibrium check only.

P_{bm} - Beam member axial force is different from $P_{b\text{-}c}$ - Beam to column interface axial force as shown above.

P_{bm} - Beam member axial force is from structural analysis output and cannot be used directly in beam end to column connection design as this force is interrupted by brace gusset to beam interface force before beam end reaching the column. This force is actually not needed from user's input for beam end to column connection design.

$P_{b\text{-}c}$ - Beam to column interface axial force is calculated from user's input of brace axial forces and trasnfer force using uniform force method. This force is used in the beam end to column connection design.

P_{bm} - Beam member axial force is not needed for the beam end to column connection design and is calculated here for verification purpose only. If it matches the structural analysis output, that means equilibrium is reached and user's input of brace axial forces and trasnfer force are correct.

Top brace axial force	$P_t =$ from seismic brace force calc	= -559.7	[kips]	in tension
Top brace to ver line angle	$\theta_t =$ from user input	= 45.0	[°]	
Top brace gusset-column axial force	$H_{ct} =$ from calc shown above	= -124.8	[kips]	AISC 15th Eq. 13-3
Bot brace axial force	$P_b =$ from seismic brace force calc	= 524.4	[kips]	in compression
Bot brace to ver line angle	$\theta_b =$ from user input	= 45.0	[°]	
Bot brace gusset-column axial force	$H_{cb} =$ from calc shown above	= 112.8	[kips]	AISC 15th Eq. 13-3
Beam to column interface axial force	$P_{b\text{-}c} =$ from calc shown above	= 58.3	[kips]	AISC 15th Page 13-4
Beam member axial force	$P_{bm} = (H_{ct} - P_t \sin\theta_t) + (H_{cb} - P_b \sin\theta_b) + P_{b\text{-}c}$	= **71.3**	[kips]	in compression

Brace - SCBF Load Case LC2

Top and bottom brace force	Top $P_{top} = 449.1$ [kips] (C)		Bot $P_{bot} = -615.9$	[kips] (T)
Beam end shear & transfer force	Shear $R_b = 19.9$ [kips]		Transfer $A_b = -46.3$	[kips]

Top Brace Interface Forces

Refer to AISC 15th Page 13-4 and Fig. 13-2 for all charts and definitions of variables and symbols shown in calculation below

$e_b = 12.350$ [in] $e_c = 6.450$ [in]

$\alpha = 14.000$ [in] $\beta = 9.625$ [in]

$\theta = 45.0$ [°]

$K = e_b \tan\theta - e_c$	= 5.900	[in]	AISC 15th Eq. 13-16
$D = \tan^2\theta + (\dfrac{\alpha}{\beta})^2$	= 3.116		AISC 15th Eq. 13-24
$K' = \alpha (\tan\theta + \dfrac{\alpha}{\beta})$	= 34.364		AISC 15th Eq. 13-23
$\bar{\alpha} = [K' \tan\theta + K (\dfrac{\alpha}{\beta})^2] / D$	= 14.000	[in]	AISC 15th Eq. 13-21

	$\bar{\beta} = (K' - K \tan\theta) / D$	= 8.100	[in]	AISC 15th Eq. 13-22
	$r = [(e_b + \bar{\beta})^2 + (e_c + \bar{\alpha})^2]^{0.5}$	= 28.921	[in]	AISC 15th Eq. 13-6
Brace axial force	P_u = from seismic brace force calc	= 449.1	[kips]	in compression
Gusset to Column Interface Forces				
Shear force	$V_c = (\bar{\beta} / r) P_u$	= **125.8**	[kips]	AISC 15th Eq. 13-2
Axial force	$H_c = (e_c / r) P_u$	= **100.2**	[kips]	AISC 15th Eq. 13-3
Moment	$M_c = H_c (\beta - \bar{\beta})$	= **-12.73**	[kip-ft]	AISC 15th Eq. 13-19
Gusset to Beam Interface Forces				
Shear force	$H_b = (\bar{\alpha} / r) P_u$	= **217.4**	[kips]	AISC 15th Eq. 13-5
Axial force	$V_b = (e_b / r) P_u$	= **191.8**	[kips]	AISC 15th Eq. 13-4
Moment	$M_b = V_b (\bar{\alpha} - \alpha)$	= **0.00**	[kip-ft]	AISC 15th Eq. 13-17

Bottom Brace Interface Forces

Refer to AISC 15th Page 13-4 and Fig. 13-2 for all charts and definitions of variables and symbols shown in calculation below

e_b = 12.350 [in] e_c = 6.450 [in]

α = 14.750 [in] β = 10.375 [in]

θ = 45.0 [°]

$K = e_b \tan\theta - e_c$	= 5.900	[in]	AISC 15th Eq. 13-16
$D = \tan^2\theta + (\frac{\alpha}{\beta})^2$	= 3.021		AISC 15th Eq. 13-24
$K' = \alpha (\tan\theta + \frac{\alpha}{\beta})$	= 35.720		AISC 15th Eq. 13-23
$\bar{\alpha} = [K' \tan\theta + K (\frac{\alpha}{\beta})^2] / D$	= 14.750	[in]	AISC 15th Eq. 13-21
$\bar{\beta} = (K' - K \tan\theta) / D$	= 8.850	[in]	AISC 15th Eq. 13-22
$r = [(e_b + \bar{\beta})^2 + (e_c + \bar{\alpha})^2]^{0.5}$	= 29.981	[in]	AISC 15th Eq. 13-6

Brace axial force	P_u = from seismic brace force calc	= -615.9	[kips]	in tension
Gusset to Column Interface Forces				
Shear force	$V_c = (\bar{\beta} / r) P_u$	= **-181.8**	[kips]	AISC 15th Eq. 13-2
Axial force	$H_c = (e_c / r) P_u$	= **-132.5**	[kips]	AISC 15th Eq. 13-3
Moment	$M_c = H_c (\beta - \bar{\beta})$	= **16.84**	[kip-ft]	AISC 15th Eq. 13-19
Gusset to Beam Interface Forces				
Shear force	$H_b = (\bar{\alpha} / r) P_u$	= **-303.0**	[kips]	AISC 15th Eq. 13-5
Axial force	$V_b = (e_b / r) P_u$	= **-253.7**	[kips]	AISC 15th Eq. 13-4
Moment	$M_b = V_b (\bar{\alpha} - \alpha)$	= **0.00**	[kip-ft]	AISC 15th Eq. 13-17

Beam to Column Interface Forces

Beam to Column Interface Shear Force				
Beam end shear reaction	R_b = from user input	= 19.9	[kips]	
Top brace gusset-beam axial force	$V_{b\text{-}top}$ =	= 191.8	[kips]	AISC 15th Eq. 13-4
Bot brace gusset-beam axial force	$V_{b\text{-}bot}$ =	= -253.7	[kips]	AISC 15th Eq. 13-4
Beam to column shear force	$V_{b\text{-}c} = R_b + V_{b\text{-}top} - V_{b\text{-}bot}$	= **465.4**	[kips]	AISC 15th Page 13-4
Beam to Column Interface Axial Force				
Top brace gusset-column axial force	$H_{c\text{-}top}$ =	= 100.2	[kips]	AISC 15th Eq. 13-3
Bot brace gusset-column axial force	$H_{c\text{-}bot}$ =	= -132.5	[kips]	AISC 15th Eq. 13-3

Transfer force from adjacent bay	A_b = from user input	= -46.3	[kips]	
Beam to column axial force	$P_{b-c} = (H_{c-top} + H_{c-bot}) \times -1 - A_b$	= **78.6**	[kips]	AISC 15th Page 13-4

Beam Member Axial Force

This force is not for use in connection calc. It's output here for user input connection forces equilibrium check only.

P_{bm} - Beam member axial force is different from P_{b-c} - Beam to column interface axial force as shown above.

P_{bm} - Beam member axial force is from structural analysis output and cannot be used directly in beam end to column connection design as this force is interrupted by brace gusset to beam interface force before beam end reaching the column. This force is actually not needed from user's input for beam end to column connection design.

P_{b-c} - Beam to column interface axial force is calculated from user's input of brace axial forces and trasnfer force using uniform force method. This force is used in the beam end to column connection design.

P_{bm} - Beam member axial force is not needed for the beam end to column connection design and is calculated here for verification purpose only. If it matches the structural analysis output, that means equilibrium is reached and user's input of brace axial forces and trasnfer force are correct.

Top brace axial force	P_t = from seismic brace force calc	= 449.1	[kips]	in compression
Top brace to ver line angle	θ_t = from user input	= 45.0	[°]	
Top brace gusset-column axial force	H_{ct} = from calc shown above	= 100.2	[kips]	AISC 15th Eq. 13-3
Bot brace axial force	P_b = from seismic brace force calc	= -615.9	[kips]	in tension
Bot brace to ver line angle	θ_b = from user input	= 45.0	[°]	
Bot brace gusset-column axial force	H_{cb} = from calc shown above	= -132.5	[kips]	AISC 15th Eq. 13-3
Beam to column interface axial force	P_{b-c} = from calc shown above	= 78.6	[kips]	AISC 15th Page 13-4
Beam member axial force	$P_{bm} = (H_{ct} - P_t \sin\theta_t) + (H_{cb} - P_b \sin\theta_b) + P_{b-c}$	= **71.6**	[kips]	in compression

Top Brace - Brace to Gusset Sect=HSS 6.875 x 0.500 P_1 =-559.7 kips (T) P_2 =449.1 kips (C) Code=AISC 360-16 LRFD

Result Summary geometries & weld limitations = **PASS** limit states max ratio = **0.97** **PASS**

Seismic SCBF Brace Highly Ductile Section Check **PASS**

HSS Section Limiting Width-to-Thickness Ratio Check

Check HSS section limiting width-to-thickness ratio for HSS wall in compression as Highly Ductile section per AISC Seismic Design Manual 3rd Ed Table 1-D — AISC SDM 3rd Table 1-D

CHS sect HSS6.875X0.500	D = 6.875 [in]	t = 0.465	[in]	
HSS sect HSS6.875X0.500	F_y = A500 Gr.C Round	= 46.0	[ksi]	
	E = 29000 [ksi]			
Ratio of expected Fy to specified min Fy	R_y = 1.30			AISC 341-16 Table A3.1
CHS width-to-thickness ratio limit	$\lambda_{hd} = 0.053 \dfrac{E}{R_y F_y}$	= **25.70**		AISC SDM 3rd Table 1-D
CHS width-to-thickness ratio actual	D/t = D/t	= **14.78**		
		$\le \lambda_{hd}$	OK	

Brace Slot Effective Net Area Check — PASS

HSS With Reinforcing Plates Effective Net Area

CHS sect HSS6.875X0.500	$D = 6.875$ [in]	$t = 0.465$ [in]		
	$A_g = 9.360$ [in^2]			
Gusset plate thickness	t_{gp} = from user input	= 0.750	[in]	
HSS cut slot width	$w = t_{gp} + 1/8"$	= 0.875	[in]	
HSS brace net area	$A_{nb} = A_g - 2\,w\,t$	= 8.546	[in^2]	
Reinforcing plate	$w_r = 1.250$ [in]	$t_r = 1.250$ [in]		
Reinforcing plate area	$A_r = w_r \times t_r$	= 1.563	[in^2]	
CHS 1/2 net area $A_1 = 0.5A_{nb}$	$A_1 = 4.273$ [in^2]	$r_1 = 2.040$ [in]		
Reinforce plate	$A_2 = 1.563$ [in^2]	$r_2 = 4.063$ [in]		
Dist to centroid of comb sect	$\bar{x} = \dfrac{A_1 r_1 + A_2 r_2}{A_1 + A_2}$	= 2.582	[in]	
Length of connection	L =	= 26.000	[in]	
Shear lag factor	$U = 1 - \bar{x} / L$	= 0.901		AISC 15th Table D3.1
Total net area	$A_n = A_{nb} + 2 \times A_r$	= 11.671	[in^2]	
Total effective net area	$A_e = U\,A_n$	= **10.512**	[in^2]	

The brace effective net area shall not be less than the brace gross area — AISC 341-16 F2.5b (3)

HSS sect HSS6.875X0.500	A_g = brace gross area	= **9.360**	[in^2]	
Total brace effective net area	$A_e = U\,A_n$	= **10.512**	[in^2]	
		$\geq A_g$	OK	AISC 341-16 F2.5b (3)

The specified minimum yield strength of the reinforce plate shall be at least the specified minimum yield strength of the brace — AISC 341-16 F2.5b (3)(i)

HSS sect HSS6.875X0.500	F_y = A500 Gr.C Round	= **46.0**	[ksi]	
Reinforce plate	F_{yp} = A992	= **50.0**	[ksi]	
		$\geq F_y$	OK	AISC 341-16 F2.5b (3)(i)

Brace Slot to Gusset Plate Weld Limitation Check — PASS

Min Fillet Weld Size

Thinner part joined thickness	t =	= 0.465	[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 =	= **26.000**	[in]	
		$\geq L_{min}$	OK	

Seismic SCBF LC1 Sect=HSS 6.875 x 0.500 P =-559.7 kips (T) ratio = **0.97** **PASS**

HSS Brace Wall - Gusset PL - Shear Yield		ratio = 559.7 / 1735.2	= **0.32**	**PASS**
HSS Brace Wall-Gusset Plate Shear Yielding				
HSS sect HSS6.875X0.500 wall thick	t = 0.465 [in]	F_y = 46.0 [ksi]		
HSS brace wall-gusset overlap length	L = 26.000 [in]			
Ratio of expected Fy to specified min Fy	R_y = 1.30			AISC 341-16 Table A3.1
Beam axial load	P_u = from seismic load calc	= **559.7**	[kips]	
HSS brace wall-gusset shear yielding	$R_n = 0.6\ R_y F_y t\ L$ x 4 walls	= 1735.2	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	ϕR_n =	= **1735.2**	[kips]	
	ratio = **0.32**	> P_u	OK	

HSS Brace Wall - Gusset PL - Shear Rupture		ratio = 559.7 / 1619.1	= **0.35**	**PASS**
HSS Brace Wall-Gusset Plate Shear Yielding				
HSS sect HSS6.875X0.500 wall thick	t = 0.465 [in]	F_u = 62.0 [ksi]		
HSS brace wall-gusset overlap length	L = 26.000 [in]			
Ratio of expected Fu to specified min Fu	R_t = 1.20			AISC 341-16 Table A3.1
Beam axial load	P_u = from seismic load calc	= **559.7**	[kips]	
HSS brace wall-gusset shear rupture	$R_n = 0.6\ R_t F_u t\ L$ x 4 walls	= 2158.8	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-4
	ϕR_n =	= **1619.1**	[kips]	
	ratio = **0.35**	> P_u	OK	

Gusset Plate - Block Shear Rupture		ratio = 559.7 / 1128.9	= **0.50**	**PASS**
Plate Block Shear - Center Strip				
Plate thickness	t_p = 0.750 [in]			
Plate strength	F_y = 50.0 [ksi]	F_u = 65.0 [ksi]		
C shape weld group size	width b = 26.000 [in]	depth d = 6.875 [in]		
Gross area subject to shear	$A_{gv} = b\ t_p \times 2$	= 39.000	[in^2]	
Net area subject to shear	$A_{nv} = A_{gvb}$	= 39.000	[in^2]	
Net area subject to tension	$A_{nt} = d\ t_p$	= 5.156	[in^2]	
Block shear strength required	V_u =	= **559.7**	[kips]	
Uniform tension stress factor	U_{bs} = 1.00			AISC 15th Fig C-J4.2
Bolt shear resistance provided	$R_n = \min(0.6F_u A_{nv}, 0.6F_y A_{gv}) + U_{bs} F_u A_{nt}$	= 1505.2	[kips]	AISC 15th Eq J4-5
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-5
	ϕR_n =	= **1128.9**	[kips]	
	ratio = **0.50**	> V_u	OK	

Gusset Plate - Tensile Yield (Whitmore)			ratio = 559.7 / 963.3	= **0.58**	**PASS**
Plate Tensile Yielding Check					
Plate size	width b_p = 28.541	[in]	thickness t_p = 0.750	[in]	
Plate yield strength	F_y = 50.0	[ksi]			
Plate gross area in shear	$A_g = b_p t_p$		= 21.406	[in^2]	
Tensile force required	P_u =		= **559.7**	[kips]	
Plate tensile yielding strength	$R_n = F_y A_g$		= 1070.3	[kips]	AISC 15th Eq J4-1
Resistance factor-LRFD	ϕ = 0.90				AISC 15th Eq J4-1
	ϕR_n =		= **963.3**	[kips]	
	ratio = **0.58**		> P_u	OK	

Gusset Plate - Tensile Rupture (Whitmore)			ratio = 559.7 / 1043.5	= **0.54**	**PASS**
Plate Tensile Rupture Check					
Plate size	width b_p = 28.541	[in]	thickness t_p = 0.750	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate net area in tension	$A_{nt} = b_p t_p$		= 21.406	[in^2]	
Tensile force required	P_u =		= **559.7**	[kips]	
Plate tensile rupture strength	$R_n = F_u A_{nt}$		= 1391.4	[kips]	AISC 15th Eq J4-2
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J4-2
	ϕR_n =		= **1043.5**	[kips]	AISC 15th Eq J4-2
	ratio = **0.54**		> P_u	OK	

Brace Slot to Gusset Plate Weld Strength ratio = 559.7 / 574.4 = **0.97** **PASS**

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 - brace	thickness t = 0.750 [in]	tensile F_u = 65.0	[ksi]	
Base metal - brace is in shear, <u>shear</u> rupture as per AISC 15th Eq J4-4 is checked				AISC 15th J2.4
Base metal shear rupture	$R_{n\text{-}b} = 0.6\ F_u\ t$	= 29.250	[kip/in]	AISC 15th Eq J4-4
Double fillet linear shear strength	$R_n = \min (R_{n\text{-}w} , R_{n\text{-}b})$	= **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 =	= **559.7**	[kips]	
Fillet weld leg size	$w = 1/4$ [in]	max L = 26.000	[in]	
	when L/w = 104.0 >100 , weld length reduction applied			AISC 15th J2.2b
Weld length reduction factor	$\beta = 1.2 - 0.002\ (L/w) \le 1.0$	= 0.99		AISC 15th Eq J2-1
Weld length used for design	$L = \beta \times L$	= 51.584	[in]	
Fillet weld length - double fillet	L =	= 51.584	[in]	
Shear resistance provided	$\phi F_n = \phi R_n \times L$	= **574.4**	[kips]	
	ratio = **0.97**	> V_u	OK	

Reinforce Plate to Brace Wall Weld Strength		ratio = 85.9 / 175.7	= **0.49**	**PASS**
Reinforcing plate	w_r = 1.250 [in]	t_r = 1.250 [in]		
	F_y = 50.0 [ksi]			
Ratio of expected Fy to specified min Fy	R_y = 1.10			AISC 341-16 Table A3.1
Reinforcing plate area	$A_r = w_r \times t_r$	= 1.563	$[in^2]$	
Required strength of weld	$P_u = R_y F_y A_r$	= **85.9**	[kips]	

Reinforce Plate to Brace Wall Fillet Weld Length

Longitudinal weld length	L_L = reinforce plate length	= 12.000	[in]	
Transverse weld length	L_T = reinforce plate width	= 1.250	[in]	
Total weld length - single fillet weld	$L_1 = 2 \times L_L + L_T$	= 25.250	[in]	AISC 15th Eq J2-10a
	$L_2 = 0.85 \times 2 \times L_L + 1.5 \times L_T$	= 22.275	[in]	AISC 15th Eq J2-10b
	$L = \max(L_1, L_2)$	= **25.250**	[in]	AISC 15th J2.4 (c)

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 = 1 for single fillet			
Load angle coefficient	$C_2 = (1 + 0.5 \sin^{1.5}\theta)$	= 1.00		AISC 15th Page 8-9
Fillet weld shear strength	$R_{n\text{-}w} = 0.6\,(C_1 \times 70\text{ ksi})\,0.707\,w\,n\,C_2$	= 9.279	[kip/in]	AISC 15th Eq 8-1
Base metal - reinforce plate	thickness t = 1.250 [in]	tensile F_u = 65.0	[ksi]	
Base metal - reinforce plate is in shear, shear rupture as per AISC 15th Eq J4-4 is checked				AISC 15th J2.4
Base metal shear rupture	$R_{n\text{-}b} = 0.6\,F_u\,t$	= 48.750	[kip/in]	AISC 15th Eq J4-4
Single fillet linear shear strength	$R_n = \min(R_{n\text{-}w}, R_{n\text{-}b})$	= **9.279**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq 8-1
	ϕR_n =	= **6.960**	[kip/in]	
Shear resistance required	P_u =	= **85.9**	[kips]	
Fillet weld length - single fillet	L =	= 25.250	[in]	
Shear resistance provided	$\phi F_n = \phi R_n \times L$	= **175.7**	[kips]	
	ratio = **0.49**	> P_u	OK	

Seismic SCBF LC2	Sect=HSS 6.875 x 0.500	P =449.1 kips (C)	ratio = **0.78**	**PASS**

Gusset Plate - Compression (Whitmore)		ratio = 449.1 / 827.9	= **0.54**	**PASS**

Plate Compression Check

Plate size	width b_p = 28.541 [in]	thickness t_p = 0.750	[in]	
	F_y = 50.0 [ksi]	E = 29000	[ksi]	
Plate gross area in compression	$A_g = b_p t_p$	= 21.406	[in^2]	
Plate radius of gyration	$r = t_p / \sqrt{12}$	= 0.217	[in]	
Plate effective length factor	K =	= 0.60		
Plate unbraced length	L_u =	= 16.420	[in]	
Plate slenderness	$KL/r = 0.60 \times L_u / r$	= 45.50		
	when $\frac{KL}{r} > 25$, use Chapter E			AISC 15th J4.4 (b)
Elastic buckling stress	$F_e = \frac{\pi^2 E}{(KL/r)^2}$	= 138.23	[ksi]	AISC 15th Eq E3-4
	when $\frac{KL}{r} \le 4.71 \left(\frac{E}{F_y}\right)^{0.5} = 113.43$			AISC 15th E3 (a)
Critical stress	$F_{cr} = 0.658^{(F_y/F_e)} F_y$	= 42.98	[ksi]	AISC 15th Eq E3-2
Plate compression required	$P_u = P_c = 449.1$	= **449.1**	[kips]	
Plate compression provided	$R_n = F_{cr} \times A_g$	= 919.9	[kips]	AISC 15th Eq E3-1
Resistance factor-LRFD	$\phi = 0.90$			AISC 15th E1
	ϕR_n =	= **827.9**	[kips]	
	ratio = **0.54**	> P_u	OK	

Brace Slot to Gusset Plate Weld Strength	ratio = 449.1 / 574.4	= **0.78**	**PASS**

Fillet Weld Strength Check

Fillet weld leg size	$w = 1/4$	[in]	load angle θ = 0.0	[°]	
Electrode strength	F_{EXX} = 70.0	[ksi]	strength coeff C_1 = 1.00		AISC 15th Table 8-3
Number of weld line	n = 2 for double fillet				
Load angle coefficient	$C_2 = (1 + 0.5 \sin^{1.5} \theta)$		= 1.00		AISC 15th Page 8-9
Fillet weld shear strength	$R_{n-w} = 0.6 (C_1 \times 70 \text{ ksi}) 0.707\ w\ n\ C_2$		= 14.847	[kip/in]	AISC 15th Eq 8-1
Base metal - brace	thickness t = 0.750	[in]	tensile F_u = 65.0	[ksi]	
Base metal - brace is in shear, <u>shear</u> rupture as per AISC 15th Eq J4-4 is checked					AISC 15th J2.4
Base metal shear rupture	$R_{n-b} = 0.6\ F_u\ t$		= 29.250	[kip/in]	AISC 15th Eq J4-4
Double fillet linear shear strength	$R_n = \min (R_{n-w} , R_{n-b})$		= **14.847**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq 8-1
	$\phi\ R_n$ =		= **11.135**	[kip/in]	
Shear resistance required	V_u =		= **449.1**	[kips]	
Fillet weld leg size	$w = 1/4$	[in]	max L = 26.000	[in]	
	when L/w = 104.0 >100 , weld length reduction applied				AISC 15th J2.2b
Weld length reduction factor	$\beta = 1.2 - 0.002\ (L/w) \le 1.0$		= 0.99		AISC 15th Eq J2-1
Weld length used for design	$L = \beta \times L$		= 51.584	[in]	
Fillet weld length - double fillet	L =		= 51.584	[in]	
Shear resistance provided	$\phi\ F_n = \phi\ R_n \times L$		= **574.4**	[kips]	
	ratio = **0.78**		> V_u	OK	

Reinforce Plate to Brace Wall Weld Strength		ratio = 85.9 / 175.7	= **0.49**	**PASS**
Reinforcing plate	w_r = 1.250 [in]	t_r = 1.250 [in]		
	F_y = 50.0 [ksi]			
Ratio of expected Fy to specified min Fy	R_y = 1.10			AISC 341-16 Table A3.1
Reinforcing plate area	$A_r = w_r \times t_r$	= 1.563	[in^2]	
Required strength of weld	$P_u = R_y F_y A_r$	= **85.9**	[kips]	
Reinforce Plate to Brace Wall Fillet Weld Length				
Longitudinal weld length	L_L = reinforce plate length	= 12.000	[in]	
Transverse weld length	L_T = reinforce plate width	= 1.250	[in]	
Total weld length - single fillet weld	$L_1 = 2 \times L_L + L_T$	= 25.250	[in]	AISC 15th Eq J2-10a
	$L_2 = 0.85 \times 2 \times L_L + 1.5 \times L_T$	= 22.275	[in]	AISC 15th Eq J2-10b
	$L = \max(L_1, L_2)$	= **25.250**	[in]	AISC 15th J2.4 (c)
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 = 1 for single fillet			
Load angle coefficient	$C_2 = (1 + 0.5 \sin^{1.5} \theta)$	= 1.00		AISC 15th Page 8-9
Fillet weld shear strength	$R_{n\text{-}w} = 0.6\ (C_1 \times 70\ \text{ksi})\ 0.707\ w\ n\ C_2$	= 9.279	[kip/in]	AISC 15th Eq 8-1
Base metal - reinforce plate	thickness t = 1.250 [in]	tensile F_u = 65.0	[ksi]	
Base metal - reinforce plate is in shear, shear rupture as per AISC 15th Eq J4-4 is checked				AISC 15th J2.4
Base metal shear rupture	$R_{n\text{-}b} = 0.6\ F_u\ t$	= 48.750	[kip/in]	AISC 15th Eq J4-4
Single fillet linear shear strength	$R_n = \min (R_{n\text{-}w}, R_{n\text{-}b})$	= **9.279**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq 8-1
	ϕR_n =	= **6.960**	[kip/in]	
Shear resistance required	P_u =	= **85.9**	[kips]	
Fillet weld length - single fillet	L =	= 25.250	[in]	
Shear resistance provided	$\phi F_n = \phi R_n \times L$	= **175.7**	[kips]	
	ratio = **0.49**	> P_u	OK	

Top Brace - Gusset to Column Direct Weld Connection Code=AISC 360-16 LRFD

Result Summary geometries & weld limitations = **PASS** limit states max ratio = **0.82** **PASS**

Weld Limitation Checks - Gusset to Column **PASS**

Min Fillet Weld Size

Thinner part joined thickness	$t =$	= 0.750	[in]	
Min fillet weld size allowed	$w_{min} =$	= **0.250**	[in]	AISC 15^{th} Table J2.4
Fillet weld size provided	$w =$	= **0.438**	[in]	
		$\geq w_{min}$	OK	

Min Fillet Weld Length

Fillet weld size provided	$w =$	= 0.438	[in]	
Min fillet weld length allowed	$L_{min} = 4 \times w$	= **1.750**	[in]	AISC 15^{th} J2.2b
Min fillet weld length	$L =$	= **17.250**	[in]	
		$\geq L_{min}$	OK	

Seismic SCBF LC1 P_1 =-559.7 kips (T) P_2 =524.4 kips (C) ratio = **0.82** **PASS**

Gusset Plate - Shear Yielding ratio = 156.8 / 388.1 = **0.40** **PASS**

Plate Shear Yielding Check

Plate size	width b_p = 17.250 [in]	thickness t_p = 0.750	[in]	
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$	= 12.938	[in^2]	
Shear force required	$V_u =$	= **156.8**	[kips]	
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$	= 388.1	[kips]	AISC 15^{th} Eq J4-3
Resistance factor-LRFD	$\phi = 1.00$			AISC 15^{th} Eq J4-3
	$\phi R_n =$	= **388.1**	[kips]	
	ratio = **0.40**	$> V_u$	OK	

Gusset Plate - Shear Rupture ratio = 156.8 / 378.4 = **0.41** **PASS**

Plate Shear Rupture Check

Plate size	width b_p = 17.250 [in]	thickness t_p = 0.750	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate net area in shear	$A_{nv} = b_p t_p$	= 12.938	[in^2]	
Shear force in demand	$V_u =$	= **156.8**	[kips]	
Plate shear rupture strength	$R_n = 0.6 F_u A_{nv}$	= 504.6	[kips]	AISC 15^{th} Eq J4-4
Resistance factor-LRFD	$\phi = 0.75$			AISC 15^{th} Eq J4-4
	$\phi R_n =$	= **378.4**	[kips]	
	ratio = **0.41**	$> V_u$	OK	

Gusset Plate - Axial Tensile Yield		ratio = 124.8 / 582.2	= **0.21**		**PASS**
Plate Tensile Yielding Check					
Plate size	width b_p = 17.250 [in]	thickness t_p = 0.750		[in]	
Plate yield strength	F_y = 50.0 [ksi]				
Plate gross area in shear	$A_g = b_p t_p$		= 12.938	[in^2]	
Tensile force required	P_u =		= **124.8**	[kips]	
Plate tensile yielding strength	$R_n = F_y A_g$		= 646.9	[kips]	AISC 15th Eq J4-1
Resistance factor-LRFD	ϕ = 0.90				AISC 15th Eq J4-1
	ϕR_n =		= **582.2**	[kips]	
	ratio = **0.21**		> P_u	OK	

Gusset Plate - Axial Tensile Rupture		ratio = 124.8 / 630.7	= **0.20**		**PASS**
Plate Tensile Rupture Check					
Plate size	width b_p = 17.250 [in]	thickness t_p = 0.750		[in]	
Plate tensile strength	F_u = 65.0 [ksi]				
Plate net area in tension	$A_{nt} = b_p t_p$		= 12.938	[in^2]	
Tensile force required	P_u =		= **124.8**	[kips]	
Plate tensile rupture strength	$R_n = F_u A_{nt}$		= 840.9	[kips]	AISC 15th Eq J4-2
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J4-2
	ϕR_n =		= **630.7**	[kips]	AISC 15th Eq J4-2
	ratio = **0.20**		> P_u	OK	

Gusset Plate - Flexural Yield Interact		ratio =	= **0.25**		**PASS**
Gusset plate	width b_p = 17.250 [in]	thick t_p = 0.750		[in]	
	yield F_y = 50.0 [ksi]				
Shear plate - gross area	$A_g = b_p \times t_p$		= 12.938	[in^2]	
Shear plate - plastic modulus	$Z_p = (b_p \times t_p^2) / 4$		= 55.79	[in^3]	
Flexural strength available	$M_c = \phi F_y Z_p$ ϕ=0.90		= 209.22	[kip-ft]	
Flexural strength required	M_r = from gusset interface forces calc		= 15.86	[kip-ft]	
Axial strength available	P_c = from axial tensile yield check		= 582.2	[kips]	
Axial strength required	P_r = from gusset interface forces calc		= -124.8	[kips]	
Shear strength available	V_c = from shear yielding check		= 388.1	[kips]	
Shear strength required	V_r = from gusset interface forces calc		= 156.8	[kips]	
Flexural yield interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right)^2$		= **0.25**		AISC 15th Eq 10-5
			< 1.0	OK	

Gusset Plate - Flexural Rupture Interact		ratio =	= **0.24**	**PASS**
Gusset plate	width b_p = 17.250 [in]	thick t_p = 0.750 [in]		
	tensile F_u = 65.0 [ksi]			
Net area of plate	$A_n = b_p \times t_p$	= 12.938 [in^2]		
Plastic modulus of net section	$Z_{net} = (b_p \times t_p^2) / 4$	= 55.79 [in^3]		
Flexural strength available	$M_c = \phi F_u Z_{net}$ ϕ=0.75	= 226.66 [kip-ft]		
Flexural strength required	M_r = from gusset interface forces calc	= 15.86 [kip-ft]		
Axial strength available	P_c = from axial tensile rupture check	= 630.7 [kips]		
Axial strength required	P_r = from gusset interface forces calc	= -124.8 [kips]		
Shear strength available	V_c = from shear rupture check	= 378.4 [kips]		
Shear strength required	V_r = from gusset interface forces calc	= 156.8 [kips]		
Flexural rupture interaction	ratio $= \left(\frac{V_r}{V_c} \right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c} \right)^2$	= **0.24**	AISC 15th Eq 10-5	
		< 1.0	OK	

Gusset to Column Weld Strength		ratio = 14.33 / 17.55	= **0.82**	**PASS**
Gusset to Column Interface - Forces				
	shear V_c = 156.8 [kips]	axial H_c = -124.8 [kips] in tension		
	moment M_c = 15.86 [kip-ft]			
Gusset to Column Interface - Weld Length				
Gusset-column fillet weld length	L_{wc} = from user input	= **17.250** [in]		
Gusset to Column Interface - Combined Weld Stress				
Weld stress from axial force	$f_a = H_c / L_{wc}$	= -7.235 [kip/in] in tension		
Weld stress from shear force	$f_v = V_c / L_{wc}$	= 9.090 [kip/in]		
Weld stress from moment force	$f_b = \frac{M}{L^2 / 6}$	= 3.838 [kip/in]		
Weld stress combined - max	$f_{max} = [(f_a - f_b)^2 + f_v^2]^{0.5}$	= **14.326** [kip/in]	AISC 15th Eq 8-11	
Weld resultant load angle	$\theta = \tan^{-1} [(f_b - f_a) / f_v]$	= 50.6 [°]		
Fillet Weld Strength Calc				
Fillet weld leg size	w = $^7/_{16}$ [in]	load angle θ = 50.6 [°]		
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	$C_2 = (1 + 0.5 \sin^{1.5} \theta)$	= 1.34	AISC 15th Page 8-9	
Fillet weld shear strength	$R_{n\text{-}w} = 0.6 (C_1 \times 70 \text{ ksi}) 0.707 \text{ w n } C_2$	= 34.810 [kip/in]	AISC 15th Eq 8-1	
Base metal - gusset plate	thickness t = 0.750 [in]	tensile F_u = 65.0 [ksi]		
Base metal - gusset 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	$R_{n\text{-}b} = 0.6 F_u t$	= 29.250 [kip/in]	AISC 15th Eq J4-4	
Double fillet linear shear strength	$R_n = \min (R_{n\text{-}w} , R_{n\text{-}b})$	= **29.250** [kip/in]	AISC 15th Eq 9-2	
Resistance factor-LRFD	ϕ = 0.75		AISC 15th Eq 8-1	
	ϕR_n =	= 21.938 [kip/in]		
When gusset plate is directly welded to beam or column, apply 1.25 ductility factor to allow adequate force redistribution in the weld group			AISC 15th Page 13-11	
Weld strength used for design after applying ductility factor	$\phi R_n = \phi R_n \times (1/1.25)$	= **17.550** [kip/in]		
	ratio = **0.82**	> f_{max}	OK	

Column Web Local Yielding		ratio = 168.9 / 768.6	= **0.22**	**PASS**

Gusset Edge Equivalent Normal Force

Refer to AISC DG29 Fig. B-1 for formula below to calculate gusset edge equivalent normal force

Gusset edge axial force	$N =$	= -124.8	[kips]	
Gusset edge moment force	$M =$	= 15.86	[kip-ft]	
Gusset edge interface length	$L =$	= 17.250	[in]	
Gusset edge equivalent normal force	$N_e = N - \frac{4\ M}{L}$	= -168.9	[kips]	AISC DG29 Fig B-1

Concentrated force from gusset	$P_u =$			= **168.9**	[kips]	
Column section	$d =$ 12.900	[in]		$t_f =$ 0.990	[in]	
	$t_w =$ 0.610	[in]		$k =$ 1.590	[in]	
	yield $F_y =$ 50.0	[ksi]				
Length of bearing	$l_b =$ Gusset/Column interface length			= 17.250	[in]	
Column web local yielding strength	$R_n = F_y t_w (5\ k + l_b)$			= 768.6	[kips]	AISC 15th Eq J10-2
Resistance factor-LRFD	$\phi =$ 1.00					
	$\phi\ R_n =$			= **768.6**	[kips]	
	ratio = **0.22**			$> P_u$	OK	

Column Web Shear Strength		ratio = 124.8 / 236.1	= **0.53**	**PASS**

W Shape Column Shear Strength Check

W sect W12X106	$d =$ 12.900	[in]	$t_w =$ 0.610	[in]	
	$F_y =$ 50.0	[ksi]			
Gusset to column axial force	$V_u =$ from gusset interface force calc		= **124.8**	[kips]	
Column shear strength	$R_n = 0.6\ F_y\ d\ t_w$		= 236.1	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	$\phi =$ 1.00				AISC 15th Eq J4-3
	$\phi\ R_n =$		= **236.1**	[kips]	
	ratio = **0.53**		$> V_u$	OK	

Seismic SCBF LC2 P_1 =449.1 kips (C) P_2 =-615.9 kips (T) ratio = **0.47** **PASS**

Gusset Plate - Shear Yielding		ratio = 125.8 / 388.1	= **0.32**	**PASS**

Plate Shear Yielding Check

Plate size	width $b_p =$ 17.250	[in]	thickness $t_p =$ 0.750	[in]	
Plate yield strength	$F_y =$ 50.0	[ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$		= 12.938	[in^2]	
Shear force required	$V_u =$		= **125.8**	[kips]	
Plate shear yielding strength	$R_n = 0.6\ F_y A_{gv}$		= 388.1	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	$\phi =$ 1.00				AISC 15th Eq J4-3
	$\phi\ R_n =$		= **388.1**	[kips]	
	ratio = **0.32**		$> V_u$	OK	

Gusset Plate - Shear Rupture		ratio = 125.8 / 378.4	= **0.33**	**PASS**	
Plate Shear Rupture Check					
Plate size	width b_p = 17.250 [in]	thickness t_p = 0.750	[in]		
Plate tensile strength	F_u = 65.0 [ksi]				
Plate net area in shear	$A_{nv} = b_p t_p$	= 12.938	[in^2]		
Shear force in demand	V_u =	= **125.8**	[kips]		
Plate shear rupture strength	$R_n = 0.6\ F_u A_{nv}$	= 504.6	[kips]	AISC 15th Eq J4-4	
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-4	
	ϕR_n =	= **378.4**	[kips]		
	ratio = **0.33**	> V_u	OK		

Gusset Plate - Axial Yield		ratio = 100.2 / 582.2	= **0.17**	**PASS**	
Plate Tensile Yielding Check					
Plate size	width b_p = 17.250 [in]	thickness t_p = 0.750	[in]		
Plate yield strength	F_y = 50.0 [ksi]				
Plate gross area in shear	$A_g = b_p t_p$	= 12.938	[in^2]		
Tensile force required	P_u =	= **100.2**	[kips]		
Plate tensile yielding strength	$R_n = F_y A_g$	= 646.9	[kips]	AISC 15th Eq J4-1	
Resistance factor-LRFD	ϕ = 0.90			AISC 15th Eq J4-1	
	ϕR_n =	= **582.2**	[kips]		
	ratio = **0.17**	> P_u	OK		

Gusset Plate - Flexural Yield Interact		ratio =	= **0.12**	**PASS**
Gusset plate	width b_p = 17.250 [in]	thick t_p = 0.750	[in]	
	yield F_y = 50.0 [ksi]			
Shear plate - gross area	$A_g = b_p \times t_p$	= 12.938	[in^2]	
Shear plate - plastic modulus	$Z_p = (b_p \times t_p^2) / 4$	= 55.79	[in^3]	
Flexural strength available	$M_c = \phi F_y Z_p$ ϕ=0.90	= 209.22	[kip-ft]	
Flexural strength required	M_r = from gusset interface forces calc	= 12.73	[kip-ft]	
Axial strength available	P_c = from axial tensile yield check	= 582.2	[kips]	
Axial strength required	P_r = from gusset interface forces calc	= 100.2	[kips]	
Shear strength available	V_c = from shear yielding check	= 388.1	[kips]	
Shear strength required	V_r = from gusset interface forces calc	= 125.8	[kips]	
Flexural yield interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right)^2$	= **0.12**		AISC 15th Eq 10-5
		< 1.0	OK	

Gusset Plate - Flexural Rupture Interact		ratio =	= **0.11**	**PASS**
Gusset plate	width b_p = 17.250 [in]	thick t_p = 0.750 [in]		
	tensile F_u = 65.0 [ksi]			
Net area of plate	$A_n = b_p \times t_p$	= 12.938 [in^2]		
Plastic modulus of net section	$Z_{net} = (b_p \times t_p^2) / 4$	= 55.79 [in^3]		
Flexural strength available	$M_c = \phi F_u Z_{net}$ ϕ=0.75	= 226.66 [kip-ft]		
Flexural strength required	M_r = from gusset interface forces calc	= 12.73 [kip-ft]		
Shear strength available	V_c = from shear rupture check	= 378.4 [kips]		
Shear strength required	V_r = from gusset interface forces calc	= 125.8 [kips]		
Flexural rupture interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{M_r}{M_c}\right)^2$	= **0.11**		AISC 15th Eq 10-5
		< 1.0	OK	

Gusset to Column Weld Strength		ratio = 7.29 / 15.59	= **0.47**	**PASS**
Gusset to Column Interface - Forces				
	shear V_c = 125.8 [kips]	axial H_c = 100.2 [kips]	in compression	
	moment M_c = 12.73 [kip-ft]			
Gusset to Column Interface - Weld Length				
Gusset-column fillet weld length	L_{wc} = from user input	= **17.250** [in]		
Gusset to Column Interface - Combined Weld Stress				
Weld stress from axial force	$f_a = H_c / L_{wc}$	= 5.809 [kip/in]	in compression	
Weld stress from shear force	$f_v = V_c / L_{wc}$	= 7.293 [kip/in]		
Weld stress from moment force	$f_b = \frac{M}{L^2 / 6}$	= 3.080 [kip/in]		
Weld stress combined - max	$f_{max} = f_v$	= **7.293** [kip/in]		AISC 15th Eq 8-11
Weld resultant load angle	θ = weld only has shear component	= 0.0 [°]		
Fillet Weld Strength Calc				
Fillet weld leg size	w = $^7/_{16}$ [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	$R_{n\text{-}w} = 0.6 (C_1 \times 70 \text{ ksi}) 0.707 w n C_2$	= 25.982 [kip/in]		AISC 15th Eq 8-1
Base metal - gusset plate	thickness t = 0.750 [in]	tensile F_u = 65.0 [ksi]		
Base metal - gusset plate is in shear, shear rupture as per AISC 15th Eq J4-4 is checked				AISC 15th J2.4
Base metal shear rupture	$R_{n\text{-}b} = 0.6 F_u t$	= 29.250 [kip/in]		AISC 15th Eq J4-4
Double fillet linear shear strength	$R_n = \min (R_{n\text{-}w}, R_{n\text{-}b})$	= **25.982** [kip/in]		AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq 8-1
	ϕR_n =	= 19.487 [kip/in]		
When gusset plate is directly welded to beam or column, apply 1.25 ductility factor to allow adequate force redistribution in the weld group				AISC 15th Page 13-11
Weld strength used for design after applying ductility factor	$\phi R_n = \phi R_n \times (1/1.25)$	= **15.589** [kip/in]		
	ratio **0.47**	> f_{max}	OK	

Column Web Local Yielding

ratio = 135.6 / 768.6 = **0.18** **PASS**

Gusset Edge Equivalent Normal Force

Refer to AISC DG29 Fig. B-1 for formula below to calculate gusset edge equivalent normal force

Gusset edge axial force	$N =$	= 100.2	[kips]	
Gusset edge moment force	$M =$	= 12.73	[kip-ft]	
Gusset edge interface length	$L =$	= 17.250	[in]	
Gusset edge equivalent normal force	$N_e = N + \frac{4M}{L}$	= 135.6	[kips]	AISC DG29 Fig B-1

Concentrated force from gusset	$P_u =$		= **135.6**	[kips]	
Column section	$d = 12.900$	[in]	$t_f = 0.990$	[in]	
	$t_w = 0.610$	[in]	$k = 1.590$	[in]	
	yield $F_y = 50.0$	[ksi]			

Length of bearing	l_b = Gusset/Column interface length	= 17.250	[in]	
Column web local yielding strength	$R_n = F_y t_w (5k + l_b)$	= 768.6	[kips]	AISC 15th Eq J10-2
Resistance factor-LRFD	$\phi = 1.00$			
	$\phi R_n =$	= **768.6**	[kips]	
	ratio = **0.18**	> P_u	OK	

Column Web Local Crippling

ratio = 135.6 / 1007.0 = **0.13** **PASS**

Gusset Edge Equivalent Normal Force

Refer to AISC DG29 Fig. B-1 for formula below to calculate gusset edge equivalent normal force

Gusset edge axial force	$N =$	= 100.2	[kips]	
Gusset edge moment force	$M =$	= 12.73	[kip-ft]	
Gusset edge interface length	$L =$	= 17.250	[in]	
Gusset edge equivalent normal force	$N_e = N + \frac{4M}{L}$	= 135.6	[kips]	AISC DG29 Fig B-1

Concentrated force from gusset	$P_u =$		= **135.6**	[kips]	
Column section	$d = 12.900$	[in]	$t_f = 0.990$	[in]	
	$t_w = 0.610$	[in]	$k = 1.590$	[in]	
	yield $F_y = 50.0$	[ksi]	$E = 29000$	[ksi]	

Length of bearing	l_b = Gusset/Column interface length	= 17.250	[in]	
	when $l_N \geq d/2$, use Eq J10-4			AISC 15th Eq J10-4
Column web local crippling strength	$R_n = 0.8\, t_w^2\ [1+3\frac{l_b}{d}(\frac{t_w}{t_f})^{1.5}] \times (\frac{E F_y t_f}{t_w})^{0.5}$	= 1342.7	[kips]	AISC 15th Eq J10-4
Resistance factor-LRFD	$\phi = 0.75$			AISC 15th J10.3
	$\phi R_n =$	= **1007.0**	[kips]	
	ratio = **0.13**	> P_u	OK	

Column Web Shear Strength		ratio = 100.2 / 236.1	= **0.42**	**PASS**

W Shape Column Shear Strength Check

W sect W12X106	d = 12.900 [in]	t_w = 0.610	[in]	
	F_y = 50.0 [ksi]			
Gusset to column axial force	V_u = from gusset interface force calc	= **100.2**	[kips]	
Column shear strength	$R_n = 0.6\ F_y\ d\ t_w$	= 236.1	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	$\phi\ R_n$ =	= **236.1**	[kips]	
	ratio = **0.42**	> V_u	OK	

Top Brace - Gusset to Beam	Direct Weld Connection	Code=AISC 360-16 LRFD

Result Summary	geometries & weld limitations = **PASS**	limit states max ratio = **0.79**	**PASS**

Brace Weld Limitation Checks - Gusset to Beam **PASS**

Min Fillet Weld Size

Thinner part joined thickness	$t =$	= 0.750	[in]	
Min fillet weld size allowed	$w_{min} =$	= **0.250**	[in]	AISC 15th Table J2.4
Fillet weld size provided	$w =$	= **0.438**	[in]	
		$\geq w_{min}$	OK	

Min Fillet Weld Length

Fillet weld size provided	$w =$	= 0.438	[in]	
Min fillet weld length allowed	$L_{min} = 4 \times w$	= **1.750**	[in]	AISC 15th J2.2b
Min fillet weld length	$L =$	= **26.000**	[in]	
		$\geq L_{min}$	OK	

Seismic SCBF LC1

P_1 =-559.7 kips (T) P_2 =524.4 kips (C) ratio = **0.79** **PASS**

Gusset Plate - Shear Yielding ratio = 270.9 / 585.0 = **0.46** **PASS**

Plate Shear Yielding Check

Plate size	width b_p = 26.000 [in]	thickness t_p = 0.750	[in]	
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$	= 19.500	[in^2]	
Shear force required	$V_u =$	= **270.9**	[kips]	
Plate shear yielding strength	$R_n = 0.6\ F_y A_{gv}$	= 585.0	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	$\phi R_n =$	= **585.0**	[kips]	
	ratio = **0.46**	$> V_u$	OK	

Gusset Plate - Shear Rupture ratio = 270.9 / 570.4 = **0.47** **PASS**

Plate Shear Rupture Check

Plate size	width b_p = 26.000 [in]	thickness t_p = 0.750	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate net area in shear	$A_{nv} = b_p t_p$	= 19.500	[in^2]	
Shear force in demand	$V_u =$	= **270.9**	[kips]	
Plate shear rupture strength	$R_n = 0.6\ F_u A_{nv}$	= 760.5	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-4
	$\phi R_n =$	= **570.4**	[kips]	
	ratio = **0.47**	$> V_u$	OK	

Gusset Plate - Axial Tensile Yield		ratio = 239.0 / 877.5	= **0.27**	**PASS**
Plate Tensile Yielding Check				
Plate size	width b_p = 26.000 [in]	thickness t_p = 0.750	[in]	
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_g = b_p t_p$	= 19.500	[in^2]	
Tensile force required	P_u =	= **239.0**	[kips]	
Plate tensile yielding strength	$R_n = F_y A_g$	= 975.0	[kips]	AISC 15th Eq J4-1
Resistance factor-LRFD	ϕ = 0.90			AISC 15th Eq J4-1
	ϕR_n =	= **877.5**	[kips]	
	ratio = **0.27**	> P_u	OK	

Gusset Plate - Axial Tensile Rupture		ratio = 239.0 / 950.6	= **0.25**	**PASS**
Plate Tensile Rupture Check				
Plate size	width b_p = 26.000 [in]	thickness t_p = 0.750	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate net area in tension	$A_{nt} = b_p t_p$	= 19.500	[in^2]	
Tensile force required	P_u =	= **239.0**	[kips]	
Plate tensile rupture strength	$R_n = F_u A_{nt}$	= 1267.5	[kips]	AISC 15th Eq J4-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-2
	ϕR_n =	= **950.6**	[kips]	AISC 15th Eq J4-2
	ratio = **0.25**	> P_u	OK	

Gusset Plate - Flexural Yield Interact		ratio =	= **0.29**	**PASS**
Gusset plate	width b_p = 26.000 [in]	thick t_p = 0.750	[in]	
	yield F_y = 50.0 [ksi]			
Shear plate - gross area	$A_g = b_p \times t_p$	= 19.500	[in^2]	
Shear plate - plastic modulus	$Z_p = (b_p \times t_p^2) / 4$	= 126.75	[in^3]	
Flexural strength available	$M_c = \phi F_y Z_p$ ϕ=0.90	= 475.31	[kip-ft]	
Flexural strength required	M_r = from gusset interface forces calc	= 0.00	[kip-ft]	
Axial strength available	P_c = from axial tensile yield check	= 877.5	[kips]	
Axial strength required	P_r = from gusset interface forces calc	= -239.0	[kips]	
Shear strength available	V_c = from shear yielding check	= 585.0	[kips]	
Shear strength required	V_r = from gusset interface forces calc	= 270.9	[kips]	
Flexural yield interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right)^2$	= **0.29**		AISC 15th Eq 10-5
		< 1.0	OK	

Gusset Plate - Flexural Rupture Interact		ratio =	= **0.29**	**PASS**
Gusset plate	width b_p = 26.000 [in]	thick t_p = 0.750 [in]		
	tensile F_u = 65.0 [ksi]			
Net area of plate	$A_n = b_p \times t_p$	= 19.500 [in^2]		
Plastic modulus of net section	$Z_{net} = (b_p \times t_p^2) / 4$	= 126.75 [in^3]		
Flexural strength available	$M_c = \phi F_u Z_{net}$ ϕ=0.75	= 514.92 [kip-ft]		
Flexural strength required	M_r = from gusset interface forces calc	= 0.00 [kip-ft]		
Axial strength available	P_c = from axial tensile rupture check	= 950.6 [kips]		
Axial strength required	P_r = from gusset interface forces calc	= -239.0 [kips]		
Shear strength available	V_c = from shear rupture check	= 570.4 [kips]		
Shear strength required	V_r = from gusset interface forces calc	= 270.9 [kips]		
Flexural rupture interaction	ratio $= \left(\frac{V_r}{V_c} \right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c} \right)^2$	= **0.29**	AISC 15th Eq 10-5	
		< 1.0	OK	

Gusset to Beam Weld Strength		ratio = 13.89 / 17.55	= **0.79**	**PASS**
Gusset to Beam Interface - Forces				
	shear H_b = 270.9 [kips]	axial V_b = -239.0 [kips] in tension		
	moment M_b = 0.00 [kip-ft]			
Gusset-beam fillet weld length	L_w =	= 26.000 [in]		
Gusset to Beam Interface - Combined Weld Stress				
Weld stress from axial force	$f_a = V_b / L_{wb}$	= -9.192 [kip/in] in tension		
Weld stress from shear force	$f_v = H_b / L_{wb}$	= 10.419 [kip/in]		
Weld stress from moment force	$f_b = \frac{M}{L^2 / 6}$	= 0.000 [kip/in]		
Weld stress combined - max	$f_{max} = [(f_a - f_b)^2 + f_v^2]^{0.5}$	= **13.895** [kip/in]	AISC 15th Eq 8-11	
Weld resultant load angle	$\theta = \tan^{-1} [(f_b - f_a) / f_v]$	= 41.4 [°]		
Fillet Weld Strength Calc				
Fillet weld leg size	w = $^7/_{16}$ [in]	load angle θ = 41.4 [°]		
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	$C_2 = (1 + 0.5 \sin^{1.5} \theta)$	= 1.27	AISC 15th Page 8-9	
Fillet weld shear strength	$R_{n\text{-}w} = 0.6 (C_1 \times 70 \text{ ksi}) 0.707 \text{ w n } C_2$	= 32.973 [kip/in]	AISC 15th Eq 8-1	
Base metal - gusset plate	thickness t = 0.750 [in]	tensile F_u = 65.0 [ksi]		
Base metal - gusset 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	$R_{n\text{-}b} = 0.6 F_u t$	= 29.250 [kip/in]	AISC 15th Eq J4-4	
Double fillet linear shear strength	$R_n = \min (R_{n\text{-}w} , R_{n\text{-}b})$	= **29.250** [kip/in]	AISC 15th Eq 9-2	
Resistance factor-LRFD	ϕ = 0.75		AISC 15th Eq 8-1	
	ϕR_n =	= 21.938 [kip/in]		
When gusset plate is directly welded to beam or column, apply 1.25 ductility factor to allow adequate force redistribution in the weld group			AISC 15th Page 13-11	
Weld strength used for design after applying ductility factor	$\phi R_n = \phi R_n \times (1/1.25)$	= **17.550** [kip/in]		
	ratio = **0.79**	> f_{max}	OK	

Beam Web Local Yielding		ratio = 239.0 / 974.2	= **0.25**	**PASS**
Concentrated force from gusset	P_u =	= **239.0**	[kips]	
Beam section	d = 24.700 [in]	t_f = 1.090	[in]	
	t_w = 0.650 [in]	k = 1.590	[in]	
	yield F_y = 50.0 [ksi]			
Length of bearing	l_b = Gusset/Beam interface length	= 26.000	[in]	
Gusset plate corner clip	clip = from user input	= 1.000	[in]	
Distance from normal force applied point to member end	$l_N = 0.5\ l_b$ + clip	= 14.000	[in]	
	when $l_N \le d$, use AISC 15th Eq J10-3			AISC 15th Eq J10-3
Beam web local yielding strength	$R_n = F_y t_w (2.5\ k + l_b)$	= 974.2	[kips]	AISC 15th Eq J10-3
Resistance factor-LRFD	ϕ = 1.00			
	$\phi\ R_n$ =	= **974.2**	[kips]	
	ratio = **0.25**	> P_u	OK	

Seismic SCBF LC2 P_1 =449.1 kips (C) P_2 =-615.9 kips (T) ratio = **0.54** **PASS**

Gusset Plate - Shear Yielding		ratio = 217.4 / 585.0	= **0.37**	**PASS**
Plate Shear Yielding Check				
Plate size	width b_p = 26.000 [in]	thickness t_p = 0.750	[in]	
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$	= 19.500	[in^2]	
Shear force required	V_u =	= **217.4**	[kips]	
Plate shear yielding strength	$R_n = 0.6\ F_y A_{gv}$	= 585.0	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	$\phi\ R_n$ =	= **585.0**	[kips]	
	ratio = **0.37**	> V_u	OK	

Gusset Plate - Shear Rupture		ratio = 217.4 / 570.4	= **0.38**	**PASS**
Plate Shear Rupture Check				
Plate size	width b_p = 26.000 [in]	thickness t_p = 0.750	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate net area in shear	$A_{nv} = b_p t_p$	= 19.500	[in^2]	
Shear force in demand	V_u =	= **217.4**	[kips]	
Plate shear rupture strength	$R_n = 0.6\ F_u A_{nv}$	= 760.5	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-4
	$\phi\ R_n$ =	= **570.4**	[kips]	
	ratio = **0.38**	> V_u	OK	

Gusset Plate - Axial Yield	ratio = 191.8 / 877.5	= **0.22**	**PASS**		
Plate Tensile Yielding Check					
Plate size	width b_p = 26.000 [in]	thickness t_p = 0.750	[in]		
Plate yield strength	F_y = 50.0 [ksi]				
Plate gross area in shear	$A_g = b_p t_p$	= 19.500	[in^2]		
Tensile force required	P_u =	= **191.8**	[kips]		
Plate tensile yielding strength	$R_n = F_y A_g$	= 975.0	[kips]	AISC 15th Eq J4-1	
Resistance factor-LRFD	ϕ = 0.90			AISC 15th Eq J4-1	
	ϕR_n =	= **877.5**	[kips]		
	ratio = **0.22**	> P_u	OK		

Gusset Plate - Flexural Yield Interact	ratio =	= **0.19**	**PASS**	
Gusset plate	width b_p = 26.000 [in]	thick t_p = 0.750	[in]	
	yield F_y = 50.0 [ksi]			
Shear plate - gross area	$A_g = b_p \times t_p$	= 19.500	[in^2]	
Shear plate - plastic modulus	$Z_p = (b_p \times t_p^2) / 4$	= 126.75	[in^3]	
Flexural strength available	$M_c = \phi F_y Z_p$ ϕ=0.90	= 475.31	[kip-ft]	
Flexural strength required	M_r = from gusset interface forces calc	= 0.00	[kip-ft]	
Axial strength available	P_c = from axial tensile yield check	= 877.5	[kips]	
Axial strength required	P_r = from gusset interface forces calc	= 191.8	[kips]	
Shear strength available	V_c = from shear yielding check	= 585.0	[kips]	
Shear strength required	V_r = from gusset interface forces calc	= 217.4	[kips]	
Flexural yield interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right)^2$	= **0.19**		AISC 15th Eq 10-5
		< 1.0	OK	

Gusset Plate - Flexural Rupture Interact	ratio =	= **0.15**	**PASS**	
Gusset plate	width b_p = 26.000 [in]	thick t_p = 0.750	[in]	
	tensile F_u = 65.0 [ksi]			
Net area of plate	$A_n = b_p \times t_p$	= 19.500	[in^2]	
Plastic modulus of net section	$Z_{net} = (b_p \times t_p^2) / 4$	= 126.75	[in^3]	
Flexural strength available	$M_c = \phi F_u Z_{net}$ ϕ=0.75	= 514.92	[kip-ft]	
Flexural strength required	M_r = from gusset interface forces calc	= 0.00	[kip-ft]	
Shear strength available	V_c = from shear rupture check	= 570.4	[kips]	
Shear strength required	V_r = from gusset interface forces calc	= 217.4	[kips]	
Flexural rupture interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{M_r}{M_c}\right)^2$	= **0.15**		AISC 15th Eq 10-5
		< 1.0	OK	

Gusset to Beam Weld Strength		ratio = 8.36 / 15.59	= **0.54**	**PASS**

Gusset to Beam Interface - Forces

	shear H_b = 217.4 [kips]	axial V_b = 191.8	[kips]	in compression
	moment M_b = 0.00 [kip-ft]			
Gusset-beam fillet weld length	L_w =	= 26.000	[in]	

Gusset to Beam Interface - Combined Weld Stress

Weld stress from axial force	$f_a = V_b / L_{wb}$	= 0.000	[kip/in]	in compression
Weld stress from shear force	$f_v = H_b / L_{wb}$	= 8.362	[kip/in]	
Weld stress from moment force	$f_b = \dfrac{M}{L^2 / 6}$	= 0.000	[kip/in]	
Weld stress combined - max	$f_{max} = f_v$	= **8.362**	[kip/in]	AISC 15th Eq 8-11
Weld resultant load angle	θ = weld only has shear component	= 0.0	[°]	

Fillet Weld Strength Calc

Fillet weld leg size	w = $^7/_{16}$ [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	$C_2 = (1 + 0.5 \sin^{1.5} θ)$	= 1.00		AISC 15th Page 8-9
Fillet weld shear strength	$R_{n-w} = 0.6 (C_1 \times 70 \text{ ksi}) 0.707 \text{ w n } C_2$	= 25.982	[kip/in]	AISC 15th Eq 8-1
Base metal - gusset plate	thickness t = 0.750 [in] tensile F_u = 65.0		[ksi]	

Base metal - gusset plate is in shear, <u>shear</u> rupture as per AISC 15th Eq J4-4 is checked — AISC 15th J2.4

Base metal shear rupture	$R_{n-b} = 0.6 F_u t$	= 29.250	[kip/in]	AISC 15th Eq J4-4
Double fillet linear shear strength	$R_n = \min (R_{n-w} , R_{n-b})$	= **25.982**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq 8-1
	$ϕ R_n$ =	= 19.487	[kip/in]	

When gusset plate is directly welded to beam or column, apply 1.25 ductility factor to allow adequate force redistribution in the weld group — AISC 15th Page 13-11

Weld strength used for design after applying ductility factor	$ϕ R_n = ϕ R_n \times (1/1.25)$	= **15.589**	[kip/in]	
	ratio = **0.54**	> f_{max}	OK	

Beam Web Local Yielding		ratio = 191.8 / 974.2	= **0.20**	**PASS**

Concentrated force from gusset	P_u =	= **191.8**	[kips]	
Beam section	d = 24.700 [in]	t_f = 1.090	[in]	
	t_w = 0.650 [in]	k = 1.590	[in]	
	yield F_y = 50.0 [ksi]			
Length of bearing	l_b = Gusset/Beam interface length	= 26.000	[in]	
Gusset plate corner clip	clip = from user input	= 1.000	[in]	
Distance from normal force applied point to member end	$l_N = 0.5\, l_b$ + clip	= 14.000	[in]	
	when $l_N \le d$, use AISC 15th Eq J10-3			AISC 15th Eq J10-3
Beam web local yielding strength	$R_n = F_y t_w (2.5 k + l_b)$	= 974.2	[kips]	AISC 15th Eq J10-3
Resistance factor-LRFD	ϕ = 1.00			
	$ϕ R_n$ =	= **974.2**	[kips]	
	ratio = **0.20**	> P_u	OK	

Beam Web Local Crippling		ratio = 191.8 / 970.1	= **0.20**	**PASS**
Concentrated force from gusset	P_u =	= **191.8**	[kips]	
Beam section	d = 24.700 [in]	t_f = 1.090	[in]	
	t_w = 0.650 [in]	k = 1.590	[in]	
	yield F_y = 50.0 [ksi]	E = 29000	[ksi]	
Length of bearing	l_b = Gusset/Beam interface length	= 26.000	[in]	
Gusset plate corner clip	clip = from user input	= 1.000	[in]	
Distance from normal force applied point to member end	$l_N = 0.5\ l_b + \text{clip}$	= 14.000	[in]	
	when $l_N \geq d/2$, use Eq J10-4			AISC 15th Eq J10-4
Beam web local crippling strength	$R_n = 0.8\ t_w^2\ [1+3\frac{l_b}{d}(\frac{t_w}{t_f})^{1.5}] \times (\frac{E\ F_y\ t_f}{t_w})^{0.5}$	= 1293.5	[kips]	AISC 15th Eq J10-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th J10.3
	ϕR_n =	= **970.1**	[kips]	
	ratio = **0.20**	> P_u	OK	

Bot Brace - Brace to Gusset	Sect=HSS 7.500 x 0.500	P_1 =524.4 kips (C)	P_2 =-615.9 kips (T)	Code=AISC 360-16 LRFD

Result Summary	geometries & weld limitations = **PASS**	limit states max ratio = **1.01**	**FAIL**

Seismic SCBF Brace Highly Ductile Section Check **PASS**

HSS Section Limiting Width-to-Thickness Ratio Check

Check HSS section limiting width-to-thickness ratio for HSS wall in compression as Highly Ductile section per AISC Seismic Design Manual 3rd Ed Table 1-D — AISC SDM 3rd Table 1-D

CHS sect HSS7.500X0.500	D = 7.500 [in]	t = 0.465 [in]		
HSS sect HSS7.500X0.500	F_y = A500 Gr.C Round	= 46.0 [ksi]		
	E = 29000 [ksi]			
Ratio of expected Fy to specified min Fy	R_y = 1.30			AISC 341-16 Table A3.1
CHS width-to-thickness ratio limit	$\lambda_{hd} = 0.053 \dfrac{E}{R_y F_y}$	= **25.70**		AISC SDM 3rd Table 1-D
CHS width-to-thickness ratio actual	D/t = D/t	= **16.13**		
		$\leq \lambda_{hd}$	OK	

Brace Slot Effective Net Area Check — PASS

HSS With Reinforcing Plates Effective Net Area

CHS sect HSS7.500X0.500	D = 7.500 [in]	t = 0.465	[in]	
	A_g = 10.300 [in^2]			
Gusset plate thickness	t_{gp} = from user input	= 0.750	[in]	
HSS cut slot width	$w = t_{gp} + 1/8"$	= 0.875	[in]	
HSS brace net area	$A_{nb} = A_g - 2\,w\,t$	= 9.486	[in^2]	
Reinforcing plate	w_r = 1.500 [in]	t_r = 1.500	[in]	
Reinforcing plate area	$A_r = w_r \times t_r$	= 2.250	[in^2]	
CHS 1/2 net area $A_1 = 0.5A_{nb}$	A_1 = 4.743 [in^2]	r_1 = 2.239	[in]	
Reinforce plate	A_2 = 2.250 [in^2]	r_2 = 4.500	[in]	
Dist to centroid of comb sect	$\bar{x} = \dfrac{A_1 r_1 + A_2 r_2}{A_1 + A_2}$	= 2.967	[in]	
Length of connection	L =	= 28.000	[in]	
Shear lag factor	$U = 1 - \bar{x} / L$	= 0.894		AISC 15th Table D3.1
Total net area	$A_n = A_{nb} + 2 \times A_r$	= 13.986	[in^2]	
Total effective net area	$A_e = U\,A_n$	= **12.504**	[in^2]	

The brace effective net area shall not be less than the brace gross area — AISC 341-16 F2.5b (3)

HSS sect HSS7.500X0.500	A_g = brace gross area	= **10.300**	[in^2]	
Total brace effective net area	$A_e = U\,A_n$	= **12.504**	[in^2]	
		$\geq A_g$	OK	AISC 341-16 F2.5b (3)

The specified minimum yield strength of the reinforce plate shall be at least the specified minimum yield strength of the brace — AISC 341-16 F2.5b (3)(i)

HSS sect HSS7.500X0.500	F_y = A500 Gr.C Round	= **46.0**	[ksi]	
Reinforce plate	F_{yp} = A992	= **50.0**	[ksi]	
		$\geq F_y$	OK	AISC 341-16 F2.5b (3)(i)

Brace Slot to Gusset Plate Weld Limitation Check — PASS

Min Fillet Weld Size

Thinner part joined thickness	t =	= 0.465	[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 =	= **28.000**	[in]	
		$\geq L_{min}$	OK	

Seismic SCBF LC1 Sect=HSS 7.500 x 0.500 P =524.4 kips (C) ratio = **0.91** **PASS**

Gusset Plate - Compression (Whitmore)		ratio = 524.4 / 871.7	= **0.60**	**PASS**

Plate Compression Check

Plate size	width b_p = 30.663 [in]	thickness t_p = 0.750	[in]	
	F_y = 50.0 [ksi]	E = 29000	[ksi]	
Plate gross area in compression	$A_g = b_p t_p$	= 22.997	[in^2]	
Plate radius of gyration	$r = t_p / \sqrt{12}$	= 0.217	[in]	
Plate effective length factor	K =	= 0.60		
Plate unbraced length	L_u =	= 17.481	[in]	
Plate slenderness	$KL/r = 0.60 \times L_u / r$	= 48.44		
	when $\frac{KL}{r} > 25$, use Chapter E			AISC 15^{th} J4.4 (b)
Elastic buckling stress	$F_e = \frac{\pi^2 E}{(KL/r)^2}$	= 121.96	[ksi]	AISC 15^{th} Eq E3-4
	when $\frac{KL}{r} \leq 4.71 \left(\frac{E}{F_y}\right)^{0.5} = 113.43$			AISC 15^{th} E3 (a)
Critical stress	$F_{cr} = 0.658^{(F_y / F_e)} F_y$	= 42.12	[ksi]	AISC 15^{th} Eq E3-2
Plate compression required	$P_u = P_c = 524.4$	= **524.4**	[kips]	
Plate compression provided	$R_n = F_{cr} \times A_g$	= 968.5	[kips]	AISC 15^{th} Eq E3-1
Resistance factor-LRFD	ϕ = 0.90			AISC 15^{th} E1
	ϕR_n =	= **871.7**	[kips]	
	ratio = **0.60**	> P_u	OK	

Brace Slot to Gusset Plate Weld Strength		ratio = 524.4 / 608.6	= **0.86**	**PASS**

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 - brace	thickness t = 0.750 [in]	tensile F_u = 65.0	[ksi]	
Base metal - brace is in shear, <u>shear</u> rupture as per AISC 15th Eq J4-4 is checked				AISC 15th J2.4
Base metal shear rupture	$R_{n\text{-}b} = 0.6\ F_u\ t$	= 29.250	[kip/in]	AISC 15th Eq J4-4
Double fillet linear shear strength	$R_n = \min (R_{n\text{-}w} , R_{n\text{-}b})$	= **14.847**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq 8-1
	$\phi\ R_n$ =	= **11.135**	[kip/in]	
Shear resistance required	V_u =	= **524.4**	[kips]	
Fillet weld leg size	$w = {}^{1}/_{4}$ [in]	max L = 28.000	[in]	
	when L/w = 112.0 >100 , weld length reduction applied			AISC 15th J2.2b
Weld length reduction factor	$\beta = 1.2 - 0.002\ (L/w) \leq 1.0$	= 0.98		AISC 15th Eq J2-1
Weld length used for design	$L = \beta \times L$	= 54.656	[in]	
Fillet weld length - double fillet	L =	= 54.656	[in]	
Shear resistance provided	$\phi\ F_n = \phi\ R_n \times L$	= **608.6**	[kips]	
	ratio = **0.86**	> V_u	OK	

Reinforce Plate to Brace Wall Weld Strength		ratio = 123.8 / 136.4	= **0.91**	**PASS**
Reinforcing plate	w_r = 1.500 [in]	t_r = 1.500 [in]		
	F_y = 50.0 [ksi]			
Ratio of expected Fy to specified min Fy	R_y = 1.10			AISC 341-16 Table A3.1
Reinforcing plate area	$A_r = w_r \times t_r$	= 2.250	[in^2]	
Required strength of weld	$P_u = R_y F_y A_r$	= **123.8**	[kips]	
Reinforce Plate to Brace Wall Fillet Weld Length				
Longitudinal weld length	L_L = reinforce plate length	= 11.500	[in]	
Transverse weld length	L_T = reinforce plate width	= 1.500	[in]	
Total weld length - single fillet weld	$L_1 = 2 \times L_L + L_T$	= 24.500	[in]	AISC 15th Eq J2-10a
	$L_2 = 0.85 \times 2 \times L_L + 1.5 \times L_T$	= 21.800	[in]	AISC 15th Eq J2-10b
	$L = \max(L_1, L_2)$	= **24.500**	[in]	AISC 15th J2.4 (c)
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 = 1 for single fillet			
Load angle coefficient	$C_2 = (1 + 0.5 \sin^{1.5} \theta)$	= 1.00		AISC 15th Page 8-9
Fillet weld shear strength	$R_{n\text{-}w} = 0.6\,(C_1 \times 70\text{ ksi})\,0.707\,w\,n\,C_2$	= 7.424	[kip/in]	AISC 15th Eq 8-1
Base metal - reinforce plate	thickness t = 1.500 [in]	tensile F_u = 65.0	[ksi]	
Base metal - reinforce plate is in shear, shear rupture as per AISC 15th Eq J4-4 is checked				AISC 15th J2.4
Base metal shear rupture	$R_{n\text{-}b} = 0.6\,F_u\,t$	= 58.500	[kip/in]	AISC 15th Eq J4-4
Single fillet linear shear strength	$R_n = \min(R_{n\text{-}w}, R_{n\text{-}b})$	= **7.424**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq 8-1
	ϕR_n =	= **5.568**	[kip/in]	
Shear resistance required	P_u =	= **123.8**	[kips]	
Fillet weld length - single fillet	L =	= 24.500	[in]	
Shear resistance provided	$\phi F_n = \phi R_n \times L$	= **136.4**	[kips]	
	ratio = **0.91**	> P_u	OK	

Seismic SCBF LC2 Sect=HSS 7.500 x 0.500 P =-615.9 kips (T) ratio = **1.01** **FAIL**

HSS Brace Wall - Gusset PL - Shear Yield		ratio = 615.9 / 1868.6	= **0.33**	**PASS**
HSS Brace Wall-Gusset Plate Shear Yielding				
HSS sect HSS7.500X0.500 wall thick	t = 0.465 [in]	F_y = 46.0 [ksi]		
HSS brace wall-gusset overlap length	L = 28.000 [in]			
Ratio of expected Fy to specified min Fy	R_y = 1.30			AISC 341-16 Table A3.1
Beam axial load	P_u = from seismic load calc	= **615.9**	[kips]	
HSS brace wall-gusset shear yielding	R_n = 0.6 $R_y F_y$ t L x 4 walls	= 1868.6	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	ϕR_n =	= **1868.6**	[kips]	
	ratio = **0.33**	> P_u	OK	

HSS Brace Wall - Gusset PL - Shear Rupture		ratio = 615.9 / 1743.6	= **0.35**	**PASS**
HSS Brace Wall-Gusset Plate Shear Yielding				
HSS sect HSS7.500X0.500 wall thick	t = 0.465 [in]	F_u = 62.0 [ksi]		
HSS brace wall-gusset overlap length	L = 28.000 [in]			
Ratio of expected Fu to specified min Fu	R_t = 1.20			AISC 341-16 Table A3.1
Beam axial load	P_u = from seismic load calc	= **615.9**	[kips]	
HSS brace wall-gusset shear rupture	R_n = 0.6 $R_t F_u$ t L x 4 walls	= 2324.9	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-4
	ϕR_n =	= **1743.6**	[kips]	
	ratio = **0.35**	> P_u	OK	

Gusset Plate - Block Shear Rupture		ratio = 615.9 / 1219.2	= **0.51**	**PASS**
Plate Block Shear - Center Strip				
Plate thickness	t_p = 0.750 [in]			
Plate strength	F_y = 50.0 [ksi]	F_u = 65.0 [ksi]		
C shape weld group size	width b = 28.000 [in]	depth d = 7.500 [in]		
Gross area subject to shear	$A_{gv} = b\ t_p \times 2$	= 42.000	[in^2]	
Net area subject to shear	$A_{nv} = A_{gvb}$	= 42.000	[in^2]	
Net area subject to tension	$A_{nt} = d\ t_p$	= 5.625	[in^2]	
Block shear strength required	V_u =	= **615.9**	[kips]	
Uniform tension stress factor	U_{bs} = 1.00			AISC 15th Fig C-J4.2
Bolt shear resistance provided	$R_n = \min(0.6F_u A_{nv}, 0.6F_y A_{gv}) + U_{bs} F_u A_{nt}$	= 1625.6	[kips]	AISC 15th Eq J4-5
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-5
	ϕR_n =	= **1219.2**	[kips]	
	ratio = **0.51**	> V_u	OK	

Gusset Plate - Tensile Yield (Whitmore)			ratio = 615.9 / 1034.9	= **0.60**	**PASS**
Plate Tensile Yielding Check					
Plate size	width b_p = 30.663	[in]	thickness t_p = 0.750	[in]	
Plate yield strength	F_y = 50.0	[ksi]			
Plate gross area in shear	$A_g = b_p t_p$		= 22.997	[in^2]	
Tensile force required	P_u =		= **615.9**	[kips]	
Plate tensile yielding strength	$R_n = F_y A_g$		= 1149.9	[kips]	AISC 15th Eq J4-1
Resistance factor-LRFD	ϕ = 0.90				AISC 15th Eq J4-1
	ϕR_n =		= **1034.9**	[kips]	
	ratio = **0.60**		> P_u	OK	

Gusset Plate - Tensile Rupture (Whitmore)			ratio = 615.9 / 1121.1	= **0.55**	**PASS**
Plate Tensile Rupture Check					
Plate size	width b_p = 30.663	[in]	thickness t_p = 0.750	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate net area in tension	$A_{nt} = b_p t_p$		= 22.997	[in^2]	
Tensile force required	P_u =		= **615.9**	[kips]	
Plate tensile rupture strength	$R_n = F_u A_{nt}$		= 1494.8	[kips]	AISC 15th Eq J4-2
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J4-2
	ϕR_n =		= **1121.1**	[kips]	AISC 15th Eq J4-2
	ratio = **0.55**		> P_u	OK	

Brace Slot to Gusset Plate Weld Strength			ratio = 615.9 / 608.6	= **1.01**	**FAIL**

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 - brace	thickness t = 0.750 [in]	tensile F_u = 65.0	[ksi]	
Base metal - brace is in shear, <u>shear</u> rupture as per AISC 15th Eq J4-4 is checked				AISC 15th J2.4
Base metal shear rupture	$R_{n\text{-}b} = 0.6\ F_u\ t$	= 29.250	[kip/in]	AISC 15th Eq J4-4
Double fillet linear shear strength	$R_n = \min(R_{n\text{-}w}, R_{n\text{-}b})$	= **14.847**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq 8-1
	$\phi\ R_n$ =	= **11.135**	[kip/in]	
Shear resistance required	V_u =	= **615.9**	[kips]	
Fillet weld leg size	$w = {}^{1}/_{4}$ [in]	max L = 28.000	[in]	
	when L/w = 112.0 >100 , weld length reduction applied			AISC 15th J2.2b
Weld length reduction factor	$\beta = 1.2 - 0.002\ (L/w) \le 1.0$	= 0.98		AISC 15th Eq J2-1
Weld length used for design	$L = \beta \times L$	= 54.656	[in]	
Fillet weld length - double fillet	L =	= 54.656	[in]	
Shear resistance provided	$\phi\ F_n = \phi\ R_n \times L$	= **608.6**	[kips]	
	ratio = **1.01**	< V_u	**NG**	

Reinforce Plate to Brace Wall Weld Strength		ratio = 123.8 / 136.4	= **0.91**	**PASS**

Reinforcing plate	w_r = 1.500 [in]	t_r = 1.500	[in]	
	F_y = 50.0 [ksi]			
Ratio of expected Fy to specified min Fy	R_y = 1.10			AISC 341-16 Table A3.1
Reinforcing plate area	$A_r = w_r \times t_r$	= 2.250	[in^2]	
Required strength of weld	$P_u = R_y F_y A_r$	= **123.8**	[kips]	

Reinforce Plate to Brace Wall Fillet Weld Length

Longitudinal weld length	L_L = reinforce plate length	= 11.500	[in]	
Transverse weld length	L_T = reinforce plate width	= 1.500	[in]	
Total weld length - single fillet weld	$L_1 = 2 \times L_L + L_T$	= 24.500	[in]	AISC 15th Eq J2-10a
	$L_2 = 0.85 \times 2 \times L_L + 1.5 \times L_T$	= 21.800	[in]	AISC 15th Eq J2-10b
	$L = \max(L_1, L_2)$	= **24.500**	[in]	AISC 15th J2.4 (c)

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 = 1 for single fillet			
Load angle coefficient	$C_2 = (1 + 0.5 \sin^{1.5}\theta)$	= 1.00		AISC 15th Page 8-9
Fillet weld shear strength	$R_{n\text{-}w} = 0.6\,(C_1 \times 70\text{ ksi})\ 0.707\ w\ n\ C_2$	= 7.424	[kip/in]	AISC 15th Eq 8-1
Base metal - reinforce plate	thickness t = 1.500 [in]	tensile F_u = 65.0	[ksi]	
Base metal - reinforce 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	$R_{n\text{-}b} = 0.6\,F_u\,t$	= 58.500	[kip/in]	AISC 15th Eq J4-4
Single fillet linear shear strength	$R_n = \min(R_{n\text{-}w}, R_{n\text{-}b})$	= **7.424**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq 8-1
	ϕR_n =	= **5.568**	[kip/in]	
Shear resistance required	P_u =	= **123.8**	[kips]	
Fillet weld length - single fillet	L =	= 24.500	[in]	
Shear resistance provided	$\phi F_n = \phi R_n \times L$	= **136.4**	[kips]	
	ratio = **0.91**	> P_u	OK	

Bot Brace - Gusset to Column	Direct Weld Connection	Code=AISC 360-16 LRFD

Result Summary	geometries & weld limitations = **PASS**	limit states max ratio = **0.98**	**PASS**

Weld Limitation Checks - Gusset to Column **PASS**

Min Fillet Weld Size

Thinner part joined thickness	$t =$	= 0.750	[in]	
Min fillet weld size allowed	$w_{min} =$	= **0.250**	[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	$L =$	= **18.750**	[in]	
		$\geq L_{min}$	OK	

Seismic SCBF LC1	P_1 =-559.7 kips (T)	P_2 =524.4 kips (C)	ratio = **0.74**	**PASS**

Gusset Plate - Shear Yielding ratio = 154.8 / 421.9 = **0.37** **PASS**

Plate Shear Yielding Check

Plate size	width b_p = 18.750 [in]	thickness t_p = 0.750	[in]	
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$	= 14.063	[in^2]	
Shear force required	$V_u =$	= **154.8**	[kips]	
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$	= 421.9	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	$\phi R_n =$	= **421.9**	[kips]	
	ratio = **0.37**	$> V_u$	OK	

Gusset Plate - Shear Rupture ratio = 154.8 / 411.3 = **0.38** **PASS**

Plate Shear Rupture Check

Plate size	width b_p = 18.750 [in]	thickness t_p = 0.750	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate net area in shear	$A_{nv} = b_p t_p$	= 14.063	[in^2]	
Shear force in demand	$V_u =$	= **154.8**	[kips]	
Plate shear rupture strength	$R_n = 0.6 F_u A_{nv}$	= 548.4	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-4
	$\phi R_n =$	= **411.3**	[kips]	
	ratio = **0.38**	$> V_u$	OK	

Gusset Plate - Axial Tensile Yield			ratio = 112.8 / 632.8	= **0.18**	**PASS**
Plate Tensile Yielding Check					
Plate size	width b_p = 18.750	[in]	thickness t_p = 0.750	[in]	
Plate yield strength	F_y = 50.0	[ksi]			
Plate gross area in shear	$A_g = b_p t_p$		= 14.063	[in^2]	
Tensile force required	P_u =		= **112.8**	[kips]	
Plate tensile yielding strength	$R_n = F_y A_g$		= 703.1	[kips]	AISC 15th Eq J4-1
Resistance factor-LRFD	ϕ = 0.90				AISC 15th Eq J4-1
	ϕR_n =		= **632.8**	[kips]	
	ratio = **0.18**		> P_u	OK	

Gusset Plate - Axial Tensile Rupture			ratio = 112.8 / 685.5	= **0.16**	**PASS**
Plate Tensile Rupture Check					
Plate size	width b_p = 18.750	[in]	thickness t_p = 0.750	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate net area in tension	$A_{nt} = b_p t_p$		= 14.063	[in^2]	
Tensile force required	P_u =		= **112.8**	[kips]	
Plate tensile rupture strength	$R_n = F_u A_{nt}$		= 914.1	[kips]	AISC 15th Eq J4-2
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J4-2
	ϕR_n =		= **685.5**	[kips]	AISC 15th Eq J4-2
	ratio = **0.16**		> P_u	OK	

Gusset Plate - Flexural Yield Interact			ratio =	= **0.15**	**PASS**
Gusset plate	width b_p = 18.750	[in]	thick t_p = 0.750	[in]	
	yield F_y = 50.0	[ksi]			
Shear plate - gross area	$A_g = b_p \times t_p$		= 14.063	[in^2]	
Shear plate - plastic modulus	$Z_p = (b_p \times t_p^2) / 4$		= 65.92	[in^3]	
Flexural strength available	$M_c = \phi F_y Z_p$ ϕ=0.90		= 247.19	[kip-ft]	
Flexural strength required	M_r = from gusset interface forces calc		= 14.34	[kip-ft]	
Axial strength available	P_c = from axial tensile yield check		= 632.8	[kips]	
Axial strength required	P_r = from gusset interface forces calc		= 112.8	[kips]	
Shear strength available	V_c = from shear yielding check		= 421.9	[kips]	
Shear strength required	V_r = from gusset interface forces calc		= 154.8	[kips]	
Flexural yield interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right)^2$		= **0.15**		AISC 15th Eq 10-5
			< 1.0	OK	

Gusset Plate - Flexural Rupture Interact		ratio =	= **0.15**	**PASS**
Gusset plate	width b_p = 18.750 [in]	thick t_p = 0.750 [in]		
	tensile F_u = 65.0 [ksi]			
Net area of plate	$A_n = b_p \times t_p$	= 14.063 [in^2]		
Plastic modulus of net section	$Z_{net} = (b_p \times t_p^2) / 4$	= 65.92 [in^3]		
Flexural strength available	$M_c = \phi F_u Z_{net}$ ϕ=0.75	= 267.79 [kip-ft]		
Flexural strength required	M_r = from gusset interface forces calc	= 14.34 [kip-ft]		
Axial strength available	P_c = from axial tensile rupture check	= 685.5 [kips]		
Axial strength required	P_r = from gusset interface forces calc	= 112.8 [kips]		
Shear strength available	V_c = from shear rupture check	= 411.3 [kips]		
Shear strength required	V_r = from gusset interface forces calc	= 154.8 [kips]		
Flexural rupture interaction	ratio = $\left(\frac{V_r}{V_c} \right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c} \right)^2$	= **0.15**		AISC 15th Eq 10-5
		< 1.0	OK	

Gusset to Column Weld Strength		ratio = 8.26 / 11.14	= **0.74**	**PASS**
Gusset to Column Interface - Forces				
	shear V_c = 154.8 [kips]	axial H_c = 112.8 [kips]	in compression	
	moment M_c = 14.34 [kip-ft]			
Gusset to Column Interface - Weld Length				
Gusset-column fillet weld length	L_{wc} = from user input	= **18.750** [in]		
Gusset to Column Interface - Combined Weld Stress				
Weld stress from axial force	$f_a = H_c / L_{wc}$	= 6.016 [kip/in]	in compression	
Weld stress from shear force	$f_v = V_c / L_{wc}$	= 8.256 [kip/in]		
Weld stress from moment force	$f_b = \frac{M}{L^2 / 6}$	= 2.937 [kip/in]		
Weld stress combined - max	$f_{max} = f_v$	= **8.256** [kip/in]		AISC 15th Eq 8-11
Weld resultant load angle	θ = weld only has shear component	= 0.0 [°]		
Fillet Weld Strength Calc				
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	$R_{n\text{-}w} = 0.6 (C_1 \times 70 \text{ ksi}) 0.707 \text{ w n } C_2$	= 18.559 [kip/in]		AISC 15th Eq 8-1
Base metal - gusset plate	thickness t = 0.750 [in]	tensile F_u = 65.0 [ksi]		
Base metal - gusset plate is in shear, shear rupture as per AISC 15th Eq J4-4 is checked				AISC 15th J2.4
Base metal shear rupture	$R_{n\text{-}b} = 0.6 F_u t$	= 29.250 [kip/in]		AISC 15th Eq J4-4
Double fillet linear shear strength	$R_n = \min (R_{n\text{-}w} , R_{n\text{-}b})$	= **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]		
When gusset plate is directly welded to beam or column, apply 1.25 ductility factor to allow adequate force redistribution in the weld group				AISC 15th Page 13-11
Weld strength used for design after applying ductility factor	$\phi R_n = \phi R_n \times (1/1.25)$	= **11.135** [kip/in]		
	ratio = **0.74**	> f_{max}	OK	

Column Web Local Yielding		ratio = 149.5 / 814.4	= **0.18**	**PASS**	
Gusset Edge Equivalent Normal Force					
Refer to AISC DG29 Fig. B-1 for formula below to calculate gusset edge equivalent normal force					
Gusset edge axial force	$N =$		= 112.8	[kips]	
Gusset edge moment force	$M =$		= 14.34	[kip-ft]	
Gusset edge interface length	$L =$		= 18.750	[in]	
Gusset edge equivalent normal force	$N_e = N + \frac{4\,M}{L}$		= 149.5	[kips]	AISC DG29 Fig B-1
Concentrated force from gusset	$P_u =$		= **149.5**	[kips]	
Column section	$d = 12.900$ [in]		$t_f = 0.990$	[in]	
	$t_w = 0.610$ [in]		$k = 1.590$	[in]	
	yield $F_y = 50.0$ [ksi]				
Length of bearing	$l_b =$ Gusset/Column interface length		= 18.750	[in]	
Column web local yielding strength	$R_n = F_y\, t_w\, (\, 5\, k + l_b\,)$		= 814.4	[kips]	AISC 15th Eq J10-2
Resistance factor-LRFD	$\phi = 1.00$				
	$\phi\, R_n =$		= **814.4**	[kips]	
	ratio = **0.18**		$> P_u$	OK	

Column Web Local Crippling		ratio = 149.5 / 1064.8	= **0.14**	**PASS**	
Gusset Edge Equivalent Normal Force					
Refer to AISC DG29 Fig. B-1 for formula below to calculate gusset edge equivalent normal force					
Gusset edge axial force	$N =$		= 112.8	[kips]	
Gusset edge moment force	$M =$		= 14.34	[kip-ft]	
Gusset edge interface length	$L =$		= 18.750	[in]	
Gusset edge equivalent normal force	$N_e = N + \frac{4\,M}{L}$		= 149.5	[kips]	AISC DG29 Fig B-1
Concentrated force from gusset	$P_u =$		= **149.5**	[kips]	
Column section	$d = 12.900$ [in]		$t_f = 0.990$	[in]	
	$t_w = 0.610$ [in]		$k = 1.590$	[in]	
	yield $F_y = 50.0$ [ksi]		$E = 29000$	[ksi]	
Length of bearing	$l_b =$ Gusset/Column interface length		= 18.750	[in]	
	when $l_N \geq d/2$, use Eq J10-4				AISC 15th Eq J10-4
Column web local crippling strength	$R_n = 0.8\, t_w^2\, [1+3\frac{l_b}{d}(\frac{t_w}{t_f})^{1.5}]\times(\frac{E\,F_y\,t_f}{t_w})^{0.5}$		= 1419.7	[kips]	AISC 15th Eq J10-4
Resistance factor-LRFD	$\phi = 0.75$				AISC 15th J10.3
	$\phi R_n =$		= **1064.8**	[kips]	
	ratio = **0.14**		$> P_u$	OK	

Column Web Shear Strength		ratio = 112.8 / 236.1	= **0.48**	**PASS**

W Shape Column Shear Strength Check

W sect W12X106	d = 12.900 [in]	t_w = 0.610	[in]	
	F_y = 50.0 [ksi]			
Gusset to column axial force	V_u = from gusset interface force calc	= **112.8**	[kips]	
Column shear strength	$R_n = 0.6\ F_y\ d\ t_w$	= 236.1	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	$\phi\ R_n$ =	= **236.1**	[kips]	
	ratio = **0.48**	> V_u	OK	

Seismic SCBF LC2	P_1 =449.1 kips (C)	P_2 =-615.9 kips (T)	ratio = **0.98**	**PASS**

Gusset Plate - Shear Yielding		ratio = 181.8 / 421.9	= **0.43**	**PASS**

Plate Shear Yielding Check

Plate size	width b_p = 18.750 [in]	thickness t_p = 0.750	[in]	
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p\ t_p$	= 14.063	[in^2]	
Shear force required	V_u =	= **181.8**	[kips]	
Plate shear yielding strength	$R_n = 0.6\ F_y\ A_{gv}$	= 421.9	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	$\phi\ R_n$ =	= **421.9**	[kips]	
	ratio = **0.43**	> V_u	OK	

Gusset Plate - Shear Rupture		ratio = 181.8 / 411.3	= **0.44**	**PASS**

Plate Shear Rupture Check

Plate size	width b_p = 18.750 [in]	thickness t_p = 0.750	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate net area in shear	$A_{nv} = b_p\ t_p$	= 14.063	[in^2]	
Shear force in demand	V_u =	= **181.8**	[kips]	
Plate shear rupture strength	$R_n = 0.6\ F_u\ A_{nv}$	= 548.4	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-4
	$\phi\ R_n$ =	= **411.3**	[kips]	
	ratio = **0.44**	> V_u	OK	

Gusset Plate - Axial Yield		ratio = 132.5 / 632.8	= **0.21**	**PASS**

Plate Tensile Yielding Check

Plate size	width b_p = 18.750 [in]	thickness t_p = 0.750	[in]	
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_g = b_p\ t_p$	= 14.063	[in^2]	
Tensile force required	P_u =	= **132.5**	[kips]	
Plate tensile yielding strength	$R_n = F_y\ A_g$	= 703.1	[kips]	AISC 15th Eq J4-1
Resistance factor-LRFD	ϕ = 0.90			AISC 15th Eq J4-1
	$\phi\ R_n$ =	= **632.8**	[kips]	
	ratio = **0.21**	> P_u	OK	

Gusset Plate - Flexural Yield Interact		ratio =	= **0.26**		**PASS**
Gusset plate	width b_p = 18.750 [in]	thick t_p = 0.750	[in]		
	yield F_y = 50.0 [ksi]				
Shear plate - gross area	$A_g = b_p \times t_p$		= 14.063	[in^2]	
Shear plate - plastic modulus	$Z_p = (b_p \times t_p^2) / 4$		= 65.92	[in^3]	
Flexural strength available	$M_c = \phi F_y Z_p$ ϕ=0.90		= 247.19	[kip-ft]	
Flexural strength required	M_r = from gusset interface forces calc		= 16.84	[kip-ft]	
Axial strength available	P_c = from axial tensile yield check		= 632.8	[kips]	
Axial strength required	P_r = from gusset interface forces calc		= -132.5	[kips]	
Shear strength available	V_c = from shear yielding check		= 421.9	[kips]	
Shear strength required	V_r = from gusset interface forces calc		= 181.8	[kips]	
Flexural yield interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right)^2$		= **0.26**		AISC 15th Eq 10-5
			< 1.0	OK	

Gusset Plate - Flexural Rupture Interact		ratio =	= **0.20**		**PASS**
Gusset plate	width b_p = 18.750 [in]	thick t_p = 0.750	[in]		
	tensile F_u = 65.0 [ksi]				
Net area of plate	$A_n = b_p \times t_p$		= 14.063	[in^2]	
Plastic modulus of net section	$Z_{net} = (b_p \times t_p^2) / 4$		= 65.92	[in^3]	
Flexural strength available	$M_c = \phi F_u Z_{net}$ ϕ=0.75		= 267.79	[kip-ft]	
Flexural strength required	M_r = from gusset interface forces calc		= 16.84	[kip-ft]	
Shear strength available	V_c = from shear rupture check		= 411.3	[kips]	
Shear strength required	V_r = from gusset interface forces calc		= 181.8	[kips]	
Flexural rupture interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{M_r}{M_c}\right)^2$		= **0.20**		AISC 15th Eq 10-5
			< 1.0	OK	

Gusset to Column Weld Strength		ratio = 14.30 / 14.64	= **0.98**	**PASS**

Gusset to Column Interface - Forces

	shear V_c = 181.8 [kips]	axial H_c = -132.5	[kips]	in tension
	moment M_c = 16.84 [kip-ft]			

Gusset to Column Interface - Weld Length

Gusset-column fillet weld length	L_{wc} = from user input	= **18.750**	[in]	

Gusset to Column Interface - Combined Weld Stress

Weld stress from axial force	$f_a = H_c / L_{wc}$	= -7.067	[kip/in]	in tension
Weld stress from shear force	$f_v = V_c / L_{wc}$	= 9.696	[kip/in]	
Weld stress from moment force	$f_b = \dfrac{M}{L^2 / 6}$	= 3.449	[kip/in]	
Weld stress combined - max	$f_{max} = [\ (f_a - f_b)^2 + f_v^2\ ]^{0.5}$	= **14.303**	[kip/in]	AISC 15th Eq 8-11
Weld resultant load angle	$\theta = \tan^{-1} [(f_b - f_a) / f_v]$	= 47.3	[°]	

Fillet Weld Strength Calc

Fillet weld leg size	$w = {}^{5}/_{16}$ [in]	load angle θ = 47.3	[°]	
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	$C_2 = (1 + 0.5 \sin^{1.5} \theta)$	= 1.32		AISC 15th Page 8-9
Fillet weld shear strength	$R_{n\text{-}w} = 0.6\ (C_1 \times 70\ \text{ksi})\ 0.707\ w\ n\ C_2$	= 24.408	[kip/in]	AISC 15th Eq 8-1

Base metal - gusset plate	thickness t = 0.750 [in]	tensile F_u = 65.0	[ksi]	
Base metal - gusset plate is in shear, shear rupture as per AISC 15th Eq J4-4 is checked				AISC 15th J2.4
Base metal shear rupture	$R_{n\text{-}b} = 0.6\ F_u\ t$	= 29.250	[kip/in]	AISC 15th Eq J4-4

Double fillet linear shear strength	$R_n = \min\ (R_{n\text{-}w} , R_{n\text{-}b})$	= **24.408**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq 8-1
	$\phi\ R_n$ =	= 18.306	[kip/in]	
When gusset plate is directly welded to beam or column, apply 1.25 ductility factor to allow adequate force redistribution in the weld group				AISC 15th Page 13-11
Weld strength used for design after applying ductility factor	$\phi\ R_n = \phi\ R_n \times (1/1.25)$	= **14.645**	[kip/in]	
	ratio = **0.98**	> f_{max}	OK	

Column Web Local Yielding		ratio = 175.6 / 814.4	= **0.22**	**PASS**
Gusset Edge Equivalent Normal Force				
Refer to AISC DG29 Fig. B-1 for formula below to calculate gusset edge equivalent normal force				
Gusset edge axial force	$N =$	= -132.5	[kips]	
Gusset edge moment force	$M =$	= 16.84	[kip-ft]	
Gusset edge interface length	$L =$	= 18.750	[in]	
Gusset edge equivalent normal force	$N_e = N - \frac{4\ M}{L}$	= -175.6	[kips]	AISC DG29 Fig B-1
Concentrated force from gusset	$P_u =$	= **175.6**	[kips]	
Column section	d = 12.900 [in]	t_f = 0.990	[in]	
	t_w = 0.610 [in]	k = 1.590	[in]	
	yield F_y = 50.0 [ksi]			
Length of bearing	l_b = Gusset/Column interface length	= 18.750	[in]	
Column web local yielding strength	$R_n = F_y t_w (5\ k + l_b)$	= 814.4	[kips]	AISC 15th Eq J10-2
Resistance factor-LRFD	ϕ = 1.00			
	$\phi\ R_n =$	= **814.4**	[kips]	
	ratio = **0.22**	> P_u	OK	

Column Web Shear Strength		ratio = 132.5 / 236.1	= **0.56**	**PASS**
W Shape Column Shear Strength Check				
W sect W12X106	d = 12.900 [in]	t_w = 0.610	[in]	
	F_y = 50.0 [ksi]			
Gusset to column axial force	V_u = from gusset interface force calc	= **132.5**	[kips]	
Column shear strength	$R_n = 0.6\ F_y\ d\ t_w$	= 236.1	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	$\phi\ R_n =$	= **236.1**	[kips]	
	ratio = **0.56**	> V_u	OK	

Bot Brace - Gusset to Beam Direct Weld Connection Code=AISC 360-16 LRFD

Result Summary	geometries & weld limitations = **PASS**	limit states max ratio = **0.82**	**PASS**

Brace Weld Limitation Checks - Gusset to Beam **PASS**

Min Fillet Weld Size

Thinner part joined thickness	t =	= 0.750	[in]	
Min fillet weld size allowed	w_{min} =	= **0.250**	[in]	AISC 15th Table J2.4
Fillet weld size provided	w =	= **0.438**	[in]	
		$\geq w_{min}$	OK	

Min Fillet Weld Length

Fillet weld size provided	w =	= 0.438	[in]	
Min fillet weld length allowed	L_{min} = 4 x w	= **1.750**	[in]	AISC 15th J2.2b
Min fillet weld length	L =	= **27.500**	[in]	
		$\geq L_{min}$	OK	

Seismic SCBF LC1 P_1 =-559.7 kips (T) P_2 =524.4 kips (C) ratio = **0.60** **PASS**

Gusset Plate - Shear Yielding ratio = 258.0 / 618.8 = **0.42** **PASS**

Plate Shear Yielding Check

Plate size	width b_p = 27.500 [in]	thickness t_p = 0.750	[in]		
Plate yield strength	F_y = 50.0 [ksi]				
Plate gross area in shear	$A_{gv} = b_p t_p$	= 20.625	[in^2]		
Shear force required	V_u =	= **258.0**	[kips]		
Plate shear yielding strength	$R_n = 0.6\ F_y A_{gv}$	= 618.8	[kips]	AISC 15th Eq J4-3	
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3	
	ϕR_n =	= **618.8**	[kips]		
	ratio = **0.42**	$> V_u$	OK		

Gusset Plate - Shear Rupture ratio = 258.0 / 603.3 = **0.43** **PASS**

Plate Shear Rupture Check

Plate size	width b_p = 27.500 [in]	thickness t_p = 0.750	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate net area in shear	$A_{nv} = b_p t_p$	= 20.625	[in^2]	
Shear force in demand	V_u =	= **258.0**	[kips]	
Plate shear rupture strength	$R_n = 0.6\ F_u A_{nv}$	= 804.4	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-4
	ϕR_n =	= **603.3**	[kips]	
	ratio = **0.43**	$> V_u$	OK	

Gusset Plate - Axial Tensile Yield		ratio = 216.0 / 928.1	= **0.23**	**PASS**

Plate Tensile Yielding Check

Plate size	width b_p = 27.500 [in]	thickness t_p = 0.750	[in]	
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_g = b_p t_p$	= 20.625	[in^2]	
Tensile force required	P_u =	= **216.0**	[kips]	
Plate tensile yielding strength	$R_n = F_y A_g$	= 1031.3	[kips]	AISC 15th Eq J4-1
Resistance factor-LRFD	ϕ = 0.90			AISC 15th Eq J4-1
	ϕR_n =	= **928.1**	[kips]	
	ratio = **0.23**	> P_u	OK	

Gusset Plate - Flexural Yield Interact		ratio =	= **0.23**	**PASS**

Gusset plate	width b_p = 27.500 [in]	thick t_p = 0.750	[in]	
	yield F_y = 50.0 [ksi]			
Shear plate - gross area	$A_g = b_p \times t_p$	= 20.625	[in^2]	
Shear plate - plastic modulus	$Z_p = (b_p \times t_p^2) / 4$	= 141.80	[in^3]	
Flexural strength available	$M_c = \phi F_y Z_p$ ϕ=0.90	= 531.74	[kip-ft]	
Flexural strength required	M_r = from gusset interface forces calc	= 0.00	[kip-ft]	
Axial strength available	P_c = from axial tensile yield check	= 928.1	[kips]	
Axial strength required	P_r = from gusset interface forces calc	= 216.0	[kips]	
Shear strength available	V_c = from shear yielding check	= 618.8	[kips]	
Shear strength required	V_r = from gusset interface forces calc	= 258.0	[kips]	
Flexural yield interaction	ratio = $\left(\frac{V_r}{V_c} \right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c} \right)^2$	= **0.23**		AISC 15th Eq 10-5
		< 1.0	OK	

Gusset Plate - Flexural Rupture Interact		ratio =	= **0.18**	**PASS**

Gusset plate	width b_p = 27.500 [in]	thick t_p = 0.750	[in]	
	tensile F_u = 65.0 [ksi]			
Net area of plate	$A_n = b_p \times t_p$	= 20.625	[in^2]	
Plastic modulus of net section	$Z_{net} = (b_p \times t_p^2) / 4$	= 141.80	[in^3]	
Flexural strength available	$M_c = \phi F_u Z_{net}$ ϕ=0.75	= 576.05	[kip-ft]	
Flexural strength required	M_r = from gusset interface forces calc	= 0.00	[kip-ft]	
Shear strength available	V_c = from shear rupture check	= 603.3	[kips]	
Shear strength required	V_r = from gusset interface forces calc	= 258.0	[kips]	
Flexural rupture interaction	ratio = $\left(\frac{V_r}{V_c} \right)^2 + \left(\frac{M_r}{M_c} \right)^2$	= **0.18**		AISC 15th Eq 10-5
		< 1.0	OK	

Gusset to Beam Weld Strength — ratio = 9.38 / 15.59 = **0.60** PASS

Gusset to Beam Interface - Forces

shear H_b = 258.0 [kips] axial V_b = 216.0 [kips] in compression

moment M_b = 0.00 [kip-ft]

Gusset-beam fillet weld length	L_w =	= 27.500	[in]	
Gusset to Beam Interface - Combined Weld Stress				
Weld stress from axial force	$f_a = V_b / L_{wb}$	= 0.000	[kip/in]	in compression
Weld stress from shear force	$f_v = H_b / L_{wb}$	= 9.382	[kip/in]	
Weld stress from moment force	$f_b = \frac{M}{L^2 / 6}$	= 0.000	[kip/in]	
Weld stress combined - max	$f_{max} = f_v$	= **9.382**	[kip/in]	AISC 15th Eq 8-11
Weld resultant load angle	θ = weld only has shear component	= 0.0	[°]	
Fillet Weld Strength Calc				
Fillet weld leg size	w = 7/16 [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	$R_{n\text{-}w} = 0.6 (C_1 \times 70\ \text{ksi})\ 0.707\ w\ n\ C_2$	= 25.982	[kip/in]	AISC 15th Eq 8-1
Base metal - gusset plate	thickness t = 0.750 [in]	tensile F_u = 65.0	[ksi]	
Base metal - gusset plate is in shear, shear rupture as per AISC 15th Eq J4-4 is checked				AISC 15th J2.4
Base metal shear rupture	$R_{n\text{-}b} = 0.6\ F_u\ t$	= 29.250	[kip/in]	AISC 15th Eq J4-4
Double fillet linear shear strength	$R_n = \min (R_{n\text{-}w} , R_{n\text{-}b})$	= **25.982**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq 8-1
	ϕR_n =	= 19.487	[kip/in]	
When gusset plate is directly welded to beam or column, apply 1.25 ductility factor to allow adequate force redistribution in the weld group				AISC 15th Page 13-11
Weld strength used for design after applying ductility factor	$\phi R_n = \phi R_n \times (1/1.25)$	= **15.589**	[kip/in]	
	ratio = **0.60**	> f_{max}	OK	

Beam Web Local Yielding — ratio = 216.0 / 1022.9 = **0.21** PASS

Concentrated force from gusset	P_u =	= **216.0**	[kips]	
Beam section	d = 24.700 [in]	t_f = 1.090	[in]	
	t_w = 0.650 [in]	k = 1.590	[in]	
	yield F_y = 50.0 [ksi]			
Length of bearing	l_b = Gusset/Beam interface length	= 27.500	[in]	
Gusset plate corner clip	clip = from user input	= 1.000	[in]	
Distance from normal force applied point to member end	$l_N = 0.5\ l_b$ + clip	= 14.750	[in]	
	when $l_N \le d$, use AISC 15th Eq J10-3			AISC 15th Eq J10-3
Beam web local yielding strength	$R_n = F_y\ t_w\ (2.5\ k + l_b)$	= 1022.9	[kips]	AISC 15th Eq J10-3
Resistance factor-LRFD	ϕ = 1.00			
	ϕR_n =	= **1022.9**	[kips]	
	ratio = **0.21**	> P_u	OK	

Beam Web Local Crippling		ratio = 216.0 / 1003.3 = **0.22**		**PASS**
Concentrated force from gusset	P_u =	= **216.0**	[kips]	
Beam section	d = 24.700 [in]	t_f = 1.090	[in]	
	t_w = 0.650 [in]	k = 1.590	[in]	
	yield F_y = 50.0 [ksi]	E = 29000	[ksi]	
Length of bearing	l_b = Gusset/Beam interface length	= 27.500	[in]	
Gusset plate corner clip	clip = from user input	= 1.000	[in]	
Distance from normal force applied point to member end	$l_N = 0.5\, l_b$ + clip	= 14.750	[in]	
	when $l_N \geq d/2$, use Eq J10-4			AISC 15th Eq J10-4
Beam web local crippling strength	$R_n = 0.8\, t_w^2 \left[1+3\frac{l_b}{d}\left(\frac{t_w}{t_f}\right)^{1.5}\right] \times \left(\frac{E F_y t_f}{t_w}\right)^{0.5}$	= 1337.7	[kips]	AISC 15th Eq J10-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th J10.3
	ϕR_n =	= **1003.3**	[kips]	
	ratio = **0.22**	> P_u	OK	

Seismic SCBF LC2 P_1 =449.1 kips (C) P_2 =-615.9 kips (T) ratio = **0.82** **PASS**

Gusset Plate - Shear Yielding		ratio = 303.0 / 618.8 = **0.49**		**PASS**
Plate Shear Yielding Check				
Plate size	width b_p = 27.500 [in]	thickness t_p = 0.750	[in]	
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$	= 20.625	[in^2]	
Shear force required	V_u =	= **303.0**	[kips]	
Plate shear yielding strength	$R_n = 0.6\, F_y A_{gv}$	= 618.8	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	ϕR_n =	= **618.8**	[kips]	
	ratio = **0.49**	> V_u	OK	

Gusset Plate - Shear Rupture		ratio = 303.0 / 603.3 = **0.50**		**PASS**
Plate Shear Rupture Check				
Plate size	width b_p = 27.500 [in]	thickness t_p = 0.750	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate net area in shear	$A_{nv} = b_p t_p$	= 20.625	[in^2]	
Shear force in demand	V_u =	= **303.0**	[kips]	
Plate shear rupture strength	$R_n = 0.6\, F_u A_{nv}$	= 804.4	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-4
	ϕR_n =	= **603.3**	[kips]	
	ratio = **0.50**	> V_u	OK	

Gusset Plate - Axial Yield		ratio = 253.7 / 928.1	= **0.27**		**PASS**
Plate Tensile Yielding Check					
Plate size	width b_p = 27.500 [in]	thickness t_p = 0.750	[in]		
Plate yield strength	F_y = 50.0 [ksi]				
Plate gross area in shear	$A_g = b_p t_p$	= 20.625	[in^2]		
Tensile force required	P_u =	= **253.7**	[kips]		
Plate tensile yielding strength	$R_n = F_y A_g$	= 1031.3	[kips]	AISC 15th Eq J4-1	
Resistance factor-LRFD	ϕ = 0.90			AISC 15th Eq J4-1	
	ϕR_n =	= **928.1**	[kips]		
	ratio = **0.27**	> P_u	OK		

Gusset Plate - Axial Tensile Rupture		ratio = 253.7 / 1005.5	= **0.25**		**PASS**
Plate Tensile Rupture Check					
Plate size	width b_p = 27.500 [in]	thickness t_p = 0.750	[in]		
Plate tensile strength	F_u = 65.0 [ksi]				
Plate net area in tension	$A_{nt} = b_p t_p$	= 20.625	[in^2]		
Tensile force required	P_u =	= **253.7**	[kips]		
Plate tensile rupture strength	$R_n = F_u A_{nt}$	= 1340.6	[kips]	AISC 15th Eq J4-2	
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-2	
	ϕR_n =	= **1005.5**	[kips]	AISC 15th Eq J4-2	
	ratio = **0.25**	> P_u	OK		

Gusset Plate - Flexural Yield Interact		ratio =	= **0.31**		**PASS**
Gusset plate	width b_p = 27.500 [in]	thick t_p = 0.750	[in]		
	yield F_y = 50.0 [ksi]				
Shear plate - gross area	$A_g = b_p \times t_p$	= 20.625	[in^2]		
Shear plate - plastic modulus	$Z_p = (b_p \times t_p^2) / 4$	= 141.80	[in^3]		
Flexural strength available	$M_c = \phi F_y Z_p$ ϕ=0.90	= 531.74	[kip-ft]		
Flexural strength required	M_r = from gusset interface forces calc	= 0.00	[kip-ft]		
Axial strength available	P_c = from axial tensile yield check	= 928.1	[kips]		
Axial strength required	P_r = from gusset interface forces calc	= -253.7	[kips]		
Shear strength available	V_c = from shear yielding check	= 618.8	[kips]		
Shear strength required	V_r = from gusset interface forces calc	= 303.0	[kips]		
Flexural yield interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right)^2$	= **0.31**		AISC 15th Eq 10-5	
		< 1.0	OK		

Gusset Plate - Flexural Rupture Interact ratio = = **0.32** PASS

Gusset plate	width b_p = 27.500 [in]	thick t_p = 0.750 [in]		
	tensile F_u = 65.0 [ksi]			
Net area of plate	$A_n = b_p \times t_p$	= 20.625	[in^2]	
Plastic modulus of net section	$Z_{net} = (b_p \times t_p^2) / 4$	= 141.80	[in^3]	
Flexural strength available	$M_c = \phi F_u Z_{net}$ ϕ=0.75	= 576.05	[kip-ft]	
Flexural strength required	M_r = from gusset interface forces calc	= 0.00	[kip-ft]	
Axial strength available	P_c = from axial tensile rupture check	= 1005.5	[kips]	
Axial strength required	P_r = from gusset interface forces calc	= -253.7	[kips]	
Shear strength available	V_c = from shear rupture check	= 603.3	[kips]	
Shear strength required	V_r = from gusset interface forces calc	= 303.0	[kips]	
Flexural rupture interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right)^2$	= **0.32**		AISC 15th Eq 10-5
		< 1.0	OK	

Gusset to Beam Weld Strength ratio = 14.37 / 17.55 = **0.82** PASS

Gusset to Beam Interface - Forces

	shear H_b = 303.0 [kips]	axial V_b = -253.7 [kips]	in tension	
	moment M_b = 0.00 [kip-ft]			
Gusset-beam fillet weld length	L_w =	= 27.500	[in]	

Gusset to Beam Interface - Combined Weld Stress

Weld stress from axial force	$f_a = V_b / L_{wb}$	= -9.225	[kip/in]	in tension
Weld stress from shear force	$f_v = H_b / L_{wb}$	= 11.018	[kip/in]	
Weld stress from moment force	$f_b = \frac{M}{L^2 / 6}$	= 0.000	[kip/in]	
Weld stress combined - max	$f_{max} = [(f_a - f_b)^2 + f_v^2]^{0.5}$	= **14.370**	[kip/in]	AISC 15th Eq 8-11
Weld resultant load angle	$\theta = \tan^{-1} [(f_b - f_a) / f_v]$	= 39.9	[°]	

Fillet Weld Strength Calc

Fillet weld leg size	w = $^7/_{16}$ [in]	load angle θ = 39.9	[°]	
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	$C_2 = (1 + 0.5 \sin^{1.5} \theta)$	= 1.26		AISC 15th Page 8-9
Fillet weld shear strength	$R_{n\text{-}w} = 0.6 (C_1 \times 70 \text{ ksi}) 0.707 w n C_2$	= 32.665	[kip/in]	AISC 15th Eq 8-1
Base metal - gusset plate	thickness t = 0.750 [in]	tensile F_u = 65.0	[ksi]	

Base metal - gusset plate is in shear, shear rupture as per AISC 15th Eq J4-4 is checked — AISC 15th J2.4

Base metal shear rupture	$R_{n\text{-}b} = 0.6 F_u t$	= 29.250	[kip/in]	AISC 15th Eq J4-4
Double fillet linear shear strength	$R_n = \min (R_{n\text{-}w} , R_{n\text{-}b})$	= **29.250**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq 8-1
	ϕR_n =	= 21.938	[kip/in]	

When gusset plate is directly welded to beam or column, apply 1.25 ductility factor to allow adequate force redistribution in the weld group — AISC 15th Page 13-11

Weld strength used for design after applying ductility factor	$\phi R_n = \phi R_n \times (1/1.25)$	= **17.550**	[kip/in]	
	ratio = **0.82**	> f_{max}	OK	

Beam Web Local Yielding		ratio = 253.7 / 1022.9	= **0.25**	**PASS**
Concentrated force from gusset	P_u =	= **253.7**	[kips]	
Beam section	d = 24.700 [in]	t_f = 1.090	[in]	
	t_w = 0.650 [in]	k = 1.590	[in]	
	yield F_y = 50.0 [ksi]			
Length of bearing	l_b = Gusset/Beam interface length	= 27.500	[in]	
Gusset plate corner clip	clip = from user input	= 1.000	[in]	
Distance from normal force applied point to member end	$l_N = 0.5\, l_b$ + clip	= 14.750	[in]	
	when $l_N \le d$, use AISC 15th Eq J10-3			AISC 15th Eq J10-3
Beam web local yielding strength	$R_n = F_y t_w (2.5\, k + l_b)$	= 1022.9	[kips]	AISC 15th Eq J10-3
Resistance factor-LRFD	ϕ = 1.00			
	ϕR_n =	= **1022.9**	[kips]	
	ratio = **0.25**	> P_u	OK	

Beam to Column	Direct Weld Connection	Code=AISC 360-16 LRFD

Result Summary	geometries & weld limitations = **PASS**	limit states max ratio = **0.97**	**PASS**

Beam Flange Fillet Weld Limitation **PASS**

Min Fillet Weld Size

Thinner part joined thickness	$t =$	= 0.990	[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	$L = 0.5\ b_{fb} - k_{1b}$	= **4.975**	[in]	
		$\geq L_{min}$	OK	

Beam Web Fillet Weld Limitation **PASS**

Min Fillet Weld Size

Thinner part joined thickness	$t =$	= 0.650	[in]	
Min fillet weld size allowed	$w_{min} =$	= **0.250**	[in]	AISC 15th Table J2.4
Fillet weld size provided	$w =$	= **0.563**	[in]	
		$\geq w_{min}$	OK	

Min Fillet Weld Length

Fillet weld size provided	$w =$	= 0.563	[in]	
Min fillet weld length allowed	$L_{min} = 4 \times w$	= **2.250**	[in]	AISC 15th J2.2b
Min fillet weld length	$L = d_b - 2\ k_b$	= **20.700**	[in]	
		$\geq L_{min}$	OK	

Seismic SCBF LC1	shear V = 435.1 kips	axial P = 58.3 kips (C)	ratio = **0.90**	**PASS**

Beam to Column - Beam Shear ratio = 435.1 / 481.7 = **0.90** **PASS**

W Shape Beam Shear Yielding Check

W sect W24X146	$d = 24.700$ [in]	$t_w = 0.650$	[in]	
	$F_y = 50.0$ [ksi]			
Beam to column shear	$V_u =$ from gusset interface force calc	= **435.1**	[kips]	
Beam shear strength	$R_n = 0.6\ F_y\ d\ t_w$	= 481.7	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	$\phi = 1.00$			AISC 15th Eq J4-3
	$\phi\ R_n =$	= **481.7**	[kips]	
	ratio = **0.90**	$> V_u$	OK	

Beam to Column - Beam Stub Compressive Strength ratio = 58.3 / 1935.0 = **0.03** **PASS**

W Shape Compression Check

W sect W24X146	A = 43.000 [in^2]	r_y = 3.022 [in]	
	F_y = 50.0 [ksi]		
W sect effective length factor	K =	= 1.00	
W sect unbraced length	L =	= 30.000 [in]	
W sect slenderness	$KL/r = 1.00 \times L \ / \ r_y$	= 9.93	
Compression force on W sect	P_u =	= **58.3** [kips]	
	when $\frac{KL}{r} \leq 25$		AISC 15th J4.4 (a)
W sect compression provided	$R_n = F_y \times A$	= 2150.0 [kips]	AISC 15th Eq J4-6
Resistance factor-LRFD	ϕ = 0.90		AISC 15th J4.4 (a)
	ϕR_n =	= **1935.0** [kips]	
	ratio = **0.03**	> P_u	OK

Beam W Shape Axial Force Distribution

W Shape Beam Axial Force Distribution

W sect W24X146	d = 24.700 [in]	b_f = 12.900 [in]	
	t_f = 1.090 [in]	t_w = 0.650 [in]	
	A = 43.000 [in^2]		
W shape one side flange area	$A_f = b_f t_f$	= 14.061	[in^2]
W shape web area	$A_w = A - 2 A_f$	= 14.878	[in^2]
W shape axial force	P =	= 58.3	[kips]
Axial force on flange	$P_f = \frac{A_f}{A} P$	= 19.1	[kips]
Axial force on web	$P_w = \frac{A_w}{A} P$	= 20.2	[kips]

Beam to Column - W Shape Flange Weld ratio = NA = **0.00** **PASS**

Beam flange tensile force	P_u = from beam axial force distribution calc	= **19.1**	[kips]

Beam flange force is in compression, the flange weld check is not needed

Beam to Column - W Shape Web Weld ratio = 21.02 / 25.05 = **0.84** **PASS**

Beam section W24X146	d_b = 24.700 [in]	k_b = 2.000	[in]	
Fillet weld length on beam web	$L = d_b - 2\ k_b$	= 20.700	[in]	
Weld Group Forces				
Weld group forces	shear V = 435.1 [kips]	axial P = 20.2	[kips]	in compression
Beam web fillet weld size	w =	= 0.563	[in]	
Combined Weld Stress				
Weld stress from axial force	$f_a = P / L$	= 0.000	[kip/in]	in compression
Weld stress from shear force	$f_v = V / L$	= 21.019	[kip/in]	
Weld stress combined - max	$f_{max} = f_v$	= **21.019**	[kip/in]	AISC 15th Eq 8-11
Weld stress load angle	θ =	= 0.0	[°]	
Fillet Weld Strength Calc				
Fillet weld leg size	w = $^9/_{16}$ [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	$R_{n\text{-}w} = 0.6\ (C_1 \times 70\ \text{ksi})\ 0.707\ w\ n\ C_2$	= 33.406	[kip/in]	AISC 15th Eq 8-1
Base metal - beam web	thickness t = 0.650 [in]	tensile F_u = 65.0	[ksi]	
Base metal - beam web is in tension, tensile rupture as per AISC 15th Eq J4-2 is checked				AISC 15th J2.4
Base metal tensile rupture	$R_{n\text{-}b} = F_u\ t$	= 42.250	[kip/in]	AISC 15th Eq J4-2
Double fillet linear shear strength	$R_n = \min(R_{n\text{-}w}, R_{n\text{-}b})$	= **33.406**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq 8-1
	ϕR_n =	= **25.054**	[kip/in]	
	ratio = **0.84**	> f_{max}	OK	

Column Web Local Yielding - Beam Flange ratio = 19.1 / 275.7 = **0.07** **PASS**

Column Web Local Yielding at Beam Flange Location

Column sect W12X106	k_c = 1.590 [in]	t_{wc} = 0.610	[in]	
	F_{yc} = 50.0 [ksi]			
Beam section W24X146	t_{fb} = 1.090 [in]			
Beam flange force	P_u = from beam axial force distribution calc	= **19.1**	[kips]	
Column web local yielding strength	$R_n = F_{yc}\ t_{wc}\ (5\ k_c + t_{fb})$	= 275.7	[kips]	AISC 15th Eq 4-2
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq 4-2
	ϕR_n =	= **275.7**	[kips]	
	ratio = **0.07**	> P_u	OK	

Column Web Local Yielding - Beam Web	ratio = 20.2 / 631.4	= **0.03**	**PASS**

Column Web Local Yielding at Beam Web Location

Column sect W12X106	k_c = 1.590 [in]	t_{wc} = 0.610	[in]	
	F_{yc} = 50.0 [ksi]			
Beam section W24X146	T_b = 20.700 [in]			
Beam axial force on beam web	P_u = from beam axial force distribution calc	= **20.2**	[kips]	
Column web local yielding strength	$R_n = F_{yc}\, t_{wc}\, T_b$	= 631.4	[kips]	AISC 15th Eq J10-2
Resistance factor-LRFD	ϕ = 1.00			AISC 15th J10.2
	$\phi\, R_n$ =	= **631.4**	[kips]	
	ratio = **0.03**	> P_u	OK	

Column Web Local Crippling - Beam Web	ratio = 58.3 / 1139.9	= **0.05**	**PASS**

Column Web Local Crippling at Beam Web Location

Column sect W12X106	d_c = 12.900 [in]	t_{fc} = 0.990	[in]	
	t_{wc} = 0.610 [in]			
	F_{yc} = 50.0 [ksi]	E = 29000	[ksi]	
Beam section W24X146	T_b = 20.700 [in]			
Beam web compression force	P_u = from beam axial force distribution calc	= **58.3**	[kips]	
Column web local crippling strength	$R_n = 0.8\, t_{wc}^2\, [1+3\frac{T_b}{d_c}(\frac{t_{wc}}{t_{fc}})^{1.5}] \times (\frac{E\, F_{cy}\, t_{fc}}{t_{wc}})^{0.5}$	= 1519.9	[kips]	AISC 15th Eq J10-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th J10.3
	ϕR_n =	= **1139.9**	[kips]	
	ratio = **0.05**	> P_u	OK	

Seismic SCBF LC2 shear V = 465.4 kips axial P = 78.6 kips (C) ratio = **0.97** **PASS**

Beam to Column - Beam Shear	ratio = 465.4 / 481.7	= **0.97**	**PASS**

W Shape Beam Shear Yielding Check

W sect W24X146	d = 24.700 [in]	t_w = 0.650	[in]	
	F_y = 50.0 [ksi]			
Beam to column shear	V_u = from gusset interface force calc	= **465.4**	[kips]	
Beam shear strength	$R_n = 0.6\, F_y\, d\, t_w$	= 481.7	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	$\phi\, R_n$ =	= **481.7**	[kips]	
	ratio = **0.97**	> V_u	OK	

Beam to Column - Beam Stub Compressive Strength ratio = 78.6 / 1935.0 = **0.04** PASS

W Shape Compression Check

W sect W24X146	A = 43.000 [in^2]	r_y = 3.022 [in]	
	F_y = 50.0 [ksi]		
W sect effective length factor	K =	= 1.00	
W sect unbraced length	L =	= 30.000 [in]	
W sect slenderness	$KL/r = 1.00 \times L \; / \; r_y$	= 9.93	
Compression force on W sect	P_u =	= **78.6** [kips]	
	when $\frac{KL}{r} \leq 25$		AISC 15th J4.4 (a)
W sect compression provided	$R_n = F_y \times A$	= 2150.0 [kips]	AISC 15th Eq J4-6
Resistance factor-LRFD	ϕ = 0.90		AISC 15th J4.4 (a)
	ϕR_n =	= **1935.0** [kips]	
	ratio = **0.04**	> P_u OK	

Beam W Shape Axial Force Distribution

W Shape Beam Axial Force Distribution

W sect W24X146	d = 24.700 [in]	b_f = 12.900 [in]
	t_f = 1.090 [in]	t_w = 0.650 [in]
	A = 43.000 [in^2]	
W shape one side flange area	$A_f = b_f t_f$	= 14.061 [in^2]
W shape web area	$A_w = A - 2 A_f$	= 14.878 [in^2]
W shape axial force	P =	= 78.6 [kips]
Axial force on flange	$P_f = \frac{A_f}{A} P$	= 25.7 [kips]
Axial force on web	$P_w = \frac{A_w}{A} P$	= 27.2 [kips]

Beam to Column - W Shape Flange Weld ratio = NA = **0.00** PASS

Beam flange tensile force P_u = from beam axial force distribution calc = **25.7** [kips]

Beam flange force is in compression, the flange weld check is not needed

Beam to Column - W Shape Web Weld		ratio = 22.48 / 25.05	= **0.90**	**PASS**

Beam section W24X146	d_b = 24.700 [in]	k_b = 2.000	[in]	
Fillet weld length on beam web	$L = d_b - 2\ k_b$	= 20.700	[in]	
Weld Group Forces				
Weld group forces	shear V = 465.4 [kips]	axial P = 27.2	[kips]	in compression
Beam web fillet weld size	w =	= 0.563	[in]	
Combined Weld Stress				
Weld stress from axial force	$f_a = P / L$	= 0.000	[kip/in]	in compression
Weld stress from shear force	$f_v = V / L$	= 22.483	[kip/in]	
Weld stress combined - max	$f_{max} = f_v$	= **22.483**	[kip/in]	AISC 15th Eq 8-11
Weld stress load angle	θ =	= 0.0	[°]	
Fillet Weld Strength Calc				
Fillet weld leg size	$w = 9/16$ [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	$R_{n-w} = 0.6\ (C_1 \times 70\ ksi)\ 0.707\ w\ n\ C_2$	= 33.406	[kip/in]	AISC 15th Eq 8-1
Base metal - beam web	thickness t = 0.650 [in]	tensile F_u = 65.0	[ksi]	
Base metal - beam web is in tension, <u>tensile</u> rupture as per AISC 15th Eq J4-2 is checked				AISC 15th J2.4
Base metal tensile rupture	$R_{n-b} = F_u\ t$	= 42.250	[kip/in]	AISC 15th Eq J4-2
Double fillet linear shear strength	$R_n = \min (R_{n-w} , R_{n-b})$	= **33.406**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	φ = 0.75			AISC 15th Eq 8-1
	$\phi\ R_n$ =	= **25.054**	[kip/in]	
	ratio = **0.90**	> f_{max}	OK	

Column Web Local Yielding - Beam Flange		ratio = 25.7 / 275.7	= **0.09**	**PASS**

Column Web Local Yielding at Beam Flange Location

Column sect W12X106	k_c = 1.590 [in]	t_{wc} = 0.610	[in]	
	F_{yc} = 50.0 [ksi]			
Beam section W24X146	t_{fb} = 1.090 [in]			
Beam flange force	P_u = from beam axial force distribution calc	= **25.7**	[kips]	
Column web local yielding strength	$R_n = F_{yc}\ t_{wc}\ (5\ k_c + t_{fb})$	= 275.7	[kips]	AISC 15th Eq 4-2
Resistance factor-LRFD	φ = 1.00			AISC 15th Eq 4-2
	$\phi\ R_n$ =	= **275.7**	[kips]	
	ratio = **0.09**	> P_u	OK	

Column Web Local Yielding - Beam Web	ratio = 27.2 / 631.4	= **0.04**	**PASS**

Column Web Local Yielding at Beam Web Location

Column sect W12X106	k_c = 1.590 [in]	t_{wc} = 0.610	[in]	
	F_{yc} = 50.0 [ksi]			
Beam section W24X146	T_b = 20.700 [in]			
Beam axial force on beam web	P_u = from beam axial force distribution calc	= **27.2**	[kips]	
Column web local yielding strength	$R_n = F_{yc} t_{wc} T_b$	= 631.4	[kips]	AISC 15th Eq J10-2
Resistance factor-LRFD	ϕ = 1.00			AISC 15th J10.2
	ϕR_n =	= **631.4**	[kips]	
	ratio = **0.04**	> P_u	OK	

Column Web Local Crippling - Beam Web	ratio = 78.6 / 1139.9	= **0.07**	**PASS**

Column Web Local Crippling at Beam Web Location

Column sect W12X106	d_c = 12.900 [in]	t_{fc} = 0.990	[in]	
	t_{wc} = 0.610 [in]			
	F_{yc} = 50.0 [ksi]	E = 29000	[ksi]	
Beam section W24X146	T_b = 20.700 [in]			
Beam web compression force	P_u = from beam axial force distribution calc	= **78.6**	[kips]	
Column web local crippling strength	$R_n = 0.8\, t_{wc}^2 \left[1+3\frac{T_b}{d_c}\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right] \times \left(\frac{E\, F_{cy}\, t_{fc}}{t_{wc}}\right)^{0.5}$	= 1519.9	[kips]	AISC 15th Eq J10-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th J10.3
	ϕR_n =	= **1139.9**	[kips]	
	ratio = **0.07**	> P_u	OK	

Beam Splice Calculation	Brace Seismic System = SCBF	Code=AISC 360-16 LRFD

Result Summary	geometries & weld limitations = **PASS**	limit states max ratio = **0.28**	**PASS**

Min Clearance Gap Between Beam Stub & Beam	**PASS**

Refer to AISC Seismic Design Manual 2nd Ed Page 5-235, in order to prevent binding at a 0.025 rad storey drift, the clearance between beam stub and beam at the splice must be at least (0.5 beam depth x 0.025 rad)

Beam sect W24X146 depth	d =	= 24.700	[in]
Min required gap between beam stub and beam	c_{min} = 0.5 d x 0.025 rad	= **0.309**	[in]
Actual gap between beam stub and beam	c = from user input	= **1.000**	[in]
		≥ c_{min}	OK

Splice Plate Thickness Limit for Ductility					**PASS**
Bolt grade	grade = A325-X				
Nominal tensile/shear stress	F_t = 90.0 [ksi]	F_v = 68.0	[ksi]	AISC 15th Table J3.2	
Bolt diameter & area	d_b = 0.875 [in]	Area A_b = 0.601	[in^2]		
Bolt group	row n_v = 6	col n_h = 2			
	row s_v = 3.000 [in]	col s_h = 3.000	[in]		
Bolt group coefficient C'	C' = from AISC 15th Table 7-6 ~ 7-13	= 54.2			
Splice plate thickness & depth	t = 0.375 [in]	d = 19.000	[in]		
	F_y = 50.0 [ksi]				
Bolt group flexural strength	$M_{max} = \frac{F_v}{0.9}(A_b C')$	= 205.08	[kip-ft]	AISC 15th Eq 10-4	
Max allowed splice plate thickness	$t_{max} = \frac{6 M_{max}}{F_y d^2}$	= **0.818**	[in]	AISC 15th Eq 10-3	
Splice plate thickness provided	2t = t x 2 sides	= **0.750**	[in]		
		≤ t_{max}	OK		

Seismic SCBF LC1 shear V = 19.9 kips axial P = 71.3 kips (C) ratio = **0.28** **PASS**

Beam Web - Outer Side Bolt Group Eccentricity - Shear Only			
Shear V - from user input, beam end shear reaction caused by gravity load only			
e_x - hor distance from column face to the beam splice outer side bolt group CG point			
Bolt group forces	shear V = 19.9 [kips]	axial P = 0.0	[kips]
Resultant force to ver Y axis angle	θ = $\tan^{-1}$ (P / V)	= 0.00	[°]
Bolt group row and column	bolt row n_r = 6	bolt col n_c = 2	
Bolt row/column spacing	bolt row s_r = 3.000 [in]	bolt col s_c = 3.000	[in]
Shear force to bolt group CG ecc	e_x =	= 34.500	[in]
Shear force to ver Y axis angle	θ =	= 0.00	[°]
Bolt group coefficient C	C = from AISC 15th Table 7-6 ~ 7-13	= 1.552	
Bolt group eccentricity coefficient	C_{ec} = C / (n_r x n_c)	= **0.129**	

Beam Web - Outer Side Bolt Group - Bolt Shear		ratio = 19.9 / 94.9	= **0.21**	**PASS**
Bolt group forces	shear V = 19.9 [kips]	axial P = 0.0	[kips]	
Bolt group resultant force	R = ($V^2 + P^2$)$^{0.5}$	= **19.9**	[kips]	
Bolt shear stress	bolt grade = A325-X	F_{nv} = 68.0	[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 = 12.0	shear plane m = 2		
Bolt group eccentricity coefficient	C_{ec} = from 'Bolt Group Eccentricity' calc	= 0.129		
Required shear strength	V_u =	= **19.9**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 126.6	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J3-1
	ϕ R_n =	= **94.9**	[kips]	
	ratio = **0.21**	> V_u	OK	

Beam Web - Outer Side Bolt Group - Bolt Bearing		ratio = 19.9 / 94.9	= **0.21**	**PASS**

The bolt group is oriented so that the shear force V is in ver. direction and the axial force P is in hor. direction

Bolt group forces	shear V = 19.9	[kips]	axial P = 0.0	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$		= **19.9**	[kips]	
Single Bolt Shear Strength					
Bolt shear stress	bolt grade = A325-X		F_{nv} = 68.0	[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	$R_{n\text{-bolt}} = 2 \times F_{nv} A_b$		= 81.8	[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 = 0.650	[in]			
Interior Bolt					
Bolt hole edge clear distance	$L_c = L_s - d_h$		= 2.063	[in]	
Bolt tear out/bearing strength	$R_{n\text{-t\&b-in}} = 1.2\, L_c\, t\, F_u \le 2.4\, d_b\, t\, m\, F_u$				AISC 15th Eq J3-6a
	= 104.6 ≤ 88.7		= 88.7	[kips]	
Bolt strength at interior	$R_{n\text{-in}} = \min (R_{n\text{-t\&b-in}} , R_{n\text{-bolt}})$		= 81.8	[kips]	
Number of bolt	interior n_{in} = 12				
Bolt group eccentricity coefficient	C_{ec} = from calc in above section		= 0.129		
Bolt bearing strength for all bolts	$R_n = (n_{in} R_{n\text{-in}}) C_{ec}$		= 126.6	[kips]	
Required shear strength	V_u =		= **19.9**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 15th J3-10
	ϕR_n =		= **94.9**	[kips]	
	ratio = **0.21**		> V_u	OK	

Beam Web - Inner Side Bolt Group Eccentricity - Shear Only

Shear V - from user input, beam end shear reaction caused by gravity load only
e_x - hor distance from column face to the beam splice inner side bolt group CG point

Bolt group forces	shear V = 19.9	[kips]	axial P = 0.0	[kips]	
Resultant force to ver Y axis angle	$\theta = \tan^{-1} (P / V)$		= 0.00	[°]	
Bolt group row and column	bolt row n_r = 6		bolt col n_c = 2		
Bolt row/column spacing	bolt row s_r = 3.000	[in]	bolt col s_c = 3.000	[in]	
Shear force to bolt group CG ecc	e_x =		= 26.500	[in]	
Shear force to ver Y axis angle	θ =		= 0.00	[°]	
Bolt group coefficient C	C = from AISC 15th Table 7-6 ~ 7-13		= 2.007		
Bolt group eccentricity coefficient	$C_{ec} = C / (n_r \times n_c)$		= **0.167**		

Beam Web - Inner Side Bolt Group - Bolt Shear		ratio = 19.9 / 122.9	= **0.16**	**PASS**
Bolt group forces	shear V = 19.9 [kips]	axial P = 0.0	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= **19.9**	[kips]	
Bolt shear stress	bolt grade = A325-X	F_{nv} = 68.0	[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 = 12.0	shear plane m = 2		
Bolt group eccentricity coefficient	C_{ec} = from 'Bolt Group Eccentricity' calc	= 0.167		
Required shear strength	V_u =	= **19.9**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 163.9	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J3-1
	ϕR_n =	= **122.9**	[kips]	
	ratio = **0.16**	> V_u	OK	

Beam Web - Inner Side Bolt Group - Bolt Bearing		ratio = 19.9 / 122.9	= **0.16**	**PASS**
The bolt group is oriented so that the shear force V is in ver. direction and the axial force P is in hor. direction				
Bolt group forces	shear V = 19.9 [kips]	axial P = 0.0	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= **19.9**	[kips]	
Single Bolt Shear Strength				
Bolt shear stress	bolt grade = A325-X	F_{nv} = 68.0	[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	$R_{n\text{-}bolt} = 2 \times F_{nv} A_b$	= 81.8	[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 = 0.650 [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
	= 104.6 ≤ 88.7	= 88.7	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min (R_{n\text{-}t\&b\text{-}in} , R_{n\text{-}bolt})$	= 81.8	[kips]	
Number of bolt	interior n_{in} = 12			
Bolt group eccentricity coefficient	C_{ec} = from calc in above section	= 0.167		
Bolt bearing strength for all bolts	$R_n = (n_{in} R_{n\text{-}in}) C_{ec}$	= 163.9	[kips]	
Required shear strength	V_u =	= **19.9**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th J3-10
	ϕR_n =	= **122.9**	[kips]	
	ratio = **0.16**	> V_u	OK	

Beam Web - Inner Side Bolt Group Eccentricity - Shear + Axial

Shear V - from user input, beam end shear reaction caused by gravity load only
Axial P - from gusset interface forces calc , beam member axial load P_{bm}
e_x - hor distance from the gusset-beam weld line CG point to the beam splice inner side bolt group CG point

Bolt group forces	shear V = 19.9	[kips]	axial P = 71.3	[kips]
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$		= 74.0	[kips]
Resultant force to ver Y axis angle	$\theta = \tan^{-1} (P / V)$		= 74.41	[°]

Bolt group row and column	bolt row n_r = 6		bolt col n_c = 2	
Bolt row/column spacing	bolt row s_r = 3.000	[in]	bolt col s_c = 3.000	[in]
Shear force to bolt group CG ecc	e_x =		= 12.125	[in]
Shear force to ver Y axis angle	θ =		= 74.41	[°]
Bolt group coefficient C	C = from AISC 15th Table 7-6 ~ 7-13		= 8.067	
Bolt group eccentricity coefficient	$C_{ec} = C / (n_r \times n_c)$		= **0.672**	

Beam Web - Inner Side Bolt Group - Bolt Shear ratio = 74.0 / 494.6 = **0.15** **PASS**

Bolt group forces	shear V = 19.9	[kips]	axial P = 71.3	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$		= **74.0**	[kips]	

Bolt shear stress	bolt grade = A325-X		F_{nv} = 68.0	[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 = 12.0		shear plane m = 2		
Bolt group eccentricity coefficient	C_{ec} = from 'Bolt Group Eccentricity' calc		= 0.672		
Required shear strength	V_u =		= **74.0**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$		= 659.5	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J3-1
	ϕR_n =		= **494.6**	[kips]	
	ratio = **0.15**		> V_u	OK	

Beam Web - Inner Side Bolt Group - Bolt Bearing		ratio = 74.0 / 482.1	= **0.15**	**PASS**	

The bolt group is oriented so that the shear force V is in ver. direction and the axial force P is in hor. direction

Bolt group forces	shear V = 19.9	[kips]	axial P = 71.3	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$		= **74.0**	[kips]	

Single Bolt Shear Strength

Bolt shear stress	bolt grade = A325-X		F_{nv} = 68.0	[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	$R_{n\text{-}bolt} = 2 \times F_{nv} A_b$		= 81.8	[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 = 2.000	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate thickness	t = 0.650	[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
	$= 104.6 \le 88.7$		= 88.7	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$		= 81.8	[kips]	

Edge Bolt

Bolt hole edge clear distance	$L_c = L_e - d_h / 2$		= 1.531	[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
	$= 77.6 \le 88.7$		= 77.6	[kips]	
Bolt strength at edge	$R_{n\text{-}ed} = \min(R_{n\text{-}t\&b\text{-}ed}, R_{n\text{-}bolt})$		= 77.6	[kips]	

Number of bolt	interior n_{in} = 6		edge n_{ed} = 6		
Bolt group eccentricity coefficient	C_{ec} = from calc in above section		= 0.672		
Bolt bearing strength for all bolts	$R_n = (n_{in} R_{n\text{-}in} + n_{ed} R_{n\text{-}ed}) C_{ec}$		= 642.8	[kips]	
Required shear strength	V_u =		= **74.0**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 15th J3-10
	ϕR_n =		= **482.1**	[kips]	
	ratio = **0.15**		> V_u	OK	

Beam Web - Outer Side Bolt Group Eccentricity - Shear + Axial

Shear V - from user input, beam end shear reaction caused by gravity load only
Axial P - from gusset interface forces calc , beam member axial load P_{bm}
e_x - hor distance from the gusset-beam weld line CG point to the beam splice outer side bolt group CG point

Bolt group forces	shear V = 19.9 [kips]	axial P = 71.3	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= 74.0	[kips]	
Resultant force to ver Y axis angle	$\theta = \tan^{-1} (P / V)$	= 74.41	[°]	
Bolt group row and column	bolt row n_r = 6	bolt col n_c = 2		
Bolt row/column spacing	bolt row s_r = 3.000 [in]	bolt col s_c = 3.000	[in]	
Shear force to bolt group CG ecc	e_x =	= 20.125	[in]	
Shear force to ver Y axis angle	θ =	= 74.41	[°]	
Bolt group coefficient C	C = from AISC 15th Table 7-6 ~ 7-13	= 6.489		
Bolt group eccentricity coefficient	$C_{ec} = C / (n_r \times n_c)$	= **0.541**		

Beam Web - Outer Side Bolt Group - Bolt Shear ratio = 74.0 / 398.2 = **0.19** **PASS**

Bolt group forces	shear V = 19.9 [kips]	axial P = 71.3	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= **74.0**	[kips]	
Bolt shear stress	bolt grade = A325-X	F_{nv} = 68.0	[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 = 12.0	shear plane m = 2		
Bolt group eccentricity coefficient	C_{ec} = from 'Bolt Group Eccentricity' calc	= 0.541		
Required shear strength	V_u =	= **74.0**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 530.9	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J3-1
	ϕR_n =	= **398.2**	[kips]	
	ratio = **0.19**	> V_u	OK	

Beam Web - Outer Side Bolt Group - Bolt Bearing		ratio = 74.0 / 388.1	= **0.19**	**PASS**	
The bolt group is oriented so that the shear force V is in ver. direction and the axial force P is in hor. direction					
Bolt group forces	shear V = 19.9 [kips]	axial P = 71.3	[kips]		
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= **74.0**	[kips]		
Single Bolt Shear Strength					
Bolt shear stress	bolt grade = A325-X	F_{nv} = 68.0	[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	$R_{n\text{-}bolt} = 2 \times F_{nv} A_b$	= 81.8	[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 = 2.000	[in]		
Plate tensile strength	F_u = 65.0 [ksi]				
Plate thickness	t = 0.650 [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	
	= 104.6 ≤ 88.7	= 88.7	[kips]		
Bolt strength at interior	$R_{n\text{-}in} = \min (R_{n\text{-}t\&b\text{-}in} , R_{n\text{-}bolt})$	= 81.8	[kips]		
Edge Bolt					
Bolt hole edge clear distance	$L_c = L_e - d_h / 2$	= 1.531	[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	
	= 77.6 ≤ 88.7	= 77.6	[kips]		
Bolt strength at edge	$R_{n\text{-}ed} = \min (R_{n\text{-}t\&b\text{-}ed} , R_{n\text{-}bolt})$	= 77.6	[kips]		
Number of bolt	interior n_{in} = 6	edge n_{ed} = 6			
Bolt group eccentricity coefficient	C_{ec} = from calc in above section	= 0.541			
Bolt bearing strength for all bolts	$R_n = (n_{in} R_{n\text{-}in} + n_{ed} R_{n\text{-}ed})\ C_{ec}$	= 517.5	[kips]		
Required shear strength	V_u =	= **74.0**	[kips]		
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th J3-10	
	ϕR_n =	= **388.1**	[kips]		
	ratio = **0.19**	> V_u	OK		

Splice Plate - Outer Side Bolt Group Eccentricity - Shear Only			
Shear V - from user input, beam end shear reaction caused by gravity load only e_x - hor distance from column face to the beam splice outer side bolt group CG point			
Bolt group forces	shear V = 19.9 [kips]	axial P = 0.0	[kips]
Resultant force to ver Y axis angle	$\theta = \tan^{-1} (P / V)$	= 0.00	[°]
Bolt group row and column	bolt row n_r = 6	bolt col n_c = 2	
Bolt row/column spacing	bolt row s_r = 3.000 [in]	bolt col s_c = 3.000	[in]
Shear force to bolt group CG ecc	e_x =	= 34.500	[in]
Shear force to ver Y axis angle	θ =	= 0.00	[°]
Bolt group coefficient C	C = from AISC 15th Table 7-6 ~ 7-13	= 1.552	
Bolt group eccentricity coefficient	$C_{ec} = C / (n_r \times n_c)$	= **0.129**	

Splice Plate - Outer Side Bolt Group - Bolt Bearing		ratio = 10.0 / 47.5	= **0.21**	**PASS**	

The bolt group is oriented so that the shear force V is in ver. direction and the axial force P is in hor. direction

Bolt group forces	shear V = 10.0	[kips]	axial P = 0.0	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$		= **10.0**	[kips]	
Single Bolt Shear Strength					
Bolt shear stress	bolt grade = A325-X		F_{nv} = 68.0	[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	$R_{n\text{-}bolt} = F_{nv} A_b$		= 40.9	[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 = 0.375	[in]			
Interior Bolt					
Bolt hole edge clear distance	$L_c = L_s - d_h$		= 2.063	[in]	
Bolt tear out/bearing strength	$R_{n\text{-}t\&b\text{-}in} = 1.2\, L_c\, t\, F_u \le 2.4\, d_b\, t\, m\, F_u$				AISC 15th Eq J3-6a
	= 60.3 ≤ 51.2		= 51.2	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$		= 40.9	[kips]	
Number of bolt	interior n_{in} = 12				
Bolt group eccentricity coefficient	C_{ec} = from calc in above section		= 0.129		
Bolt bearing strength for all bolts	$R_n = (n_{in} R_{n\text{-}in}) C_{ec}$		= 63.3	[kips]	
Required shear strength	V_u =		= **10.0**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 15th J3-10
	ϕR_n =		= **47.5**	[kips]	
	ratio = **0.21**		> V_u	OK	

Splice Plate - Compression Buckling

ratio = 35.7 / 256.1 = **0.14** **PASS**

Plate Compression Check

Plate size	width b_p = 19.000 [in]	thickness t_p = 0.375	[in]	
	F_y = 50.0 [ksi]	E = 29000	[ksi]	
Plate gross area in compression	$A_g = b_p t_p$	= 7.125	[in^2]	
Plate radius of gyration	$r = t_p / \sqrt{12}$	= 0.108	[in]	
Plate effective length factor	K =	= 1.20		AISC 15th Table C-A-7.1
Plate unbraced length	L_u =	= 5.000	[in]	
Plate slenderness	$KL/r = 1.20 \times L_u / r$	= 55.43		
	when $\frac{KL}{r} > 25$, use Chapter E			AISC 15th J4.4 (b)
Elastic buckling stress	$F_e = \frac{\pi^2 E}{(KL/r)^2}$	= 93.17	[ksi]	AISC 15th Eq E3-4
	when $\frac{KL}{r} \le 4.71 \left(\frac{E}{F_y}\right)^{0.5} = 113.43$			AISC 15th E3 (a)
Critical stress	$F_{cr} = 0.658^{(F_y/F_e)} F_y$	= 39.94	[ksi]	AISC 15th Eq E3-2
Plate compression required	P_u = 0.5 x beam axial compression P	= **35.7**	[kips]	
Plate compression provided	$R_n = F_{cr} \times A_g$	= 284.6	[kips]	AISC 15th Eq E3-1
Resistance factor-LRFD	ϕ = 0.90			AISC 15th E1
	ϕR_n =	= **256.1**	[kips]	
	ratio = **0.14**	> P_u	OK	

Splice Plate - Shear Yielding

ratio = 19.9 / 427.5 = **0.05** **PASS**

Splice Plate Shear Yielding

Plate size	width b_p = 19.000 [in]	thickness t_p = 0.375	[in]	
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$ x 2 side	= 14.250	[in^2]	
Shear force required	V_u =	= **19.9**	[kips]	
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$	= 427.5	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	ϕR_n =	= **427.5**	[kips]	
	ratio = **0.05**	> V_u	OK	

Splice Plate - Flexural Yielding

ratio = 54.73 / 253.83 = **0.22** **PASS**

Plate width & thick	width b_p = 19.000 [in]	thick t_p = 0.375	[in]	
	yield F_y = 50.0 [ksi]			
Beam end shear reaction caused by gravity load only	V = from user input	= 19.9	[kips]	
Ecc from column face to outer bolt group first bolt line	e =	= 33.000	[in]	
Flexural strength required	M_r = V x e	= **54.73**	[kip-ft]	
Shear plate - plastic modulus	$Z_p = (b_p \times t_p^2) / 4$ x 2 side	= 67.69	[in^3]	
Flexural strength available	$M_c = \phi F_y Z_p$ ϕ=0.90	= 253.83	[kip-ft]	
	ratio = **0.22**	> M_r	OK	

Splice Plate - Shear Rupture

ratio = 10.0 / 142.6 = **0.07** **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 = 6				
Plate size	width b_p = 19.000	[in]	thickness t_p = 0.375	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate net area in shear	$A_{nv} = (b_p - n\ d_h)\ t_p$		= 4.875	[in^2]	
Shear force required	V_u =		= **10.0**	[kips]	
Plate shear rupture strength	$R_n = 0.6\ F_u\ A_{nv}$		= 190.1	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J4-4
	$\phi\ R_n$ =		= **142.6**	[kips]	
	ratio = **0.07**		> V_u	OK	

Splice Plate - Flexural Rupture

ratio = 27.36 / 96.36 = **0.28** **PASS**

Plate A_n and Z_{net} Calc

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 = 6				
Plate size	width b_p = 19.000	[in]	thickness t_p = 0.375	[in]	
Plate net area	$A_n = (b_p - n\ d_h)\ t_p$		= 4.875	[in^2]	
Plate net plastic sect modulus	Z_{net} =		= 23.72	[in^3]	
Plate net elastic sect modulus	S_{net} =		= 16.33	[in^3]	

Plate width & thick	width b_p = 19.000 [in]	thick t_p = 0.375	[in]	
	F_u = 65.0 [ksi]			
Beam end shear reaction caused by gravity load only	V = from user input, x 0.5 for 2 side plate	= 10.0	[kips]	
Ecc from column face to outer bolt group first bolt line	e =	= 33.000	[in]	
Flexural strength required	M_r = V x e	= **27.36**	[kip-ft]	
Shear plate - net plastic modulus	Z_{net} = from above calc	= 23.72	[in^3]	
Flexural strength available	$M_c = \phi\ F_u\ Z_{net}$ ϕ=0.75	= **96.36**	[kip-ft]	
	ratio = **0.28**	> M_r	OK	

Seismic SCBF LC2

shear V = 19.9 kips axial P = 71.6 kips (C) ratio = **0.28** **PASS**

Beam Web - Outer Side Bolt Group Eccentricity - Shear Only

Shear V - from user input, beam end shear reaction caused by gravity load only
e_x - hor distance from column face to the beam splice outer side bolt group CG point

Bolt group forces	shear V = 19.9 [kips]	axial P = 0.0	[kips]	
Resultant force to ver Y axis angle	$\theta = \tan^{-1} (P / V)$	= 0.00	[°]	

Bolt group row and column	bolt row n_r = 6	bolt col n_c = 2		
Bolt row/column spacing	bolt row s_r = 3.000 [in]	bolt col s_c = 3.000	[in]	
Shear force to bolt group CG ecc	e_x =	= 34.500	[in]	
Shear force to ver Y axis angle	θ =	= 0.00	[°]	
Bolt group coefficient C	C = from AISC 15th Table 7-6 ~ 7-13	= 1.552		
Bolt group eccentricity coefficient	$C_{ec} = C / (n_r \times n_c)$	= **0.129**		

Beam Web - Outer Side Bolt Group - Bolt Shear		ratio = 19.9 / 94.9	= **0.21**	**PASS**	
Bolt group forces	shear V = 19.9 [kips]	axial P = 0.0	[kips]		
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= **19.9**	[kips]		
Bolt shear stress	bolt grade = A325-X	F_{nv} = 68.0	[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 = 12.0	shear plane m = 2			
Bolt group eccentricity coefficient	C_{ec} = from 'Bolt Group Eccentricity' calc	= 0.129			
Required shear strength	V_u =	= **19.9**	[kips]		
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 126.6	[kips]	AISC 15th Eq J3-1	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J3-1	
	ϕR_n =	= **94.9**	[kips]		
	ratio = **0.21**	> V_u	OK		

Beam Web - Outer Side Bolt Group - Bolt Bearing		ratio = 19.9 / 94.9	= **0.21**	**PASS**
The bolt group is oriented so that the shear force V is in ver. direction and the axial force P is in hor. direction				
Bolt group forces	shear V = 19.9 [kips]	axial P = 0.0	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= **19.9**	[kips]	
Single Bolt Shear Strength				
Bolt shear stress	bolt grade = A325-X	F_{nv} = 68.0	[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	$R_{n\text{-}bolt} = 2 \times F_{nv} A_b$	= 81.8	[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 = 0.650 [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
	= 104.6 ≤ 88.7	= 88.7	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min (R_{n\text{-}t\&b\text{-}in} , R_{n\text{-}bolt})$	= 81.8	[kips]	
Number of bolt	interior n_{in} = 12			
Bolt group eccentricity coefficient	C_{ec} = from calc in above section	= 0.129		
Bolt bearing strength for all bolts	$R_n = (n_{in} R_{n\text{-}in}) C_{ec}$	= 126.6	[kips]	
Required shear strength	V_u =	= **19.9**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th J3-10
	ϕR_n =	= **94.9**	[kips]	
	ratio = **0.21**	> V_u	OK	

Beam Web - Inner Side Bolt Group Eccentricity - Shear Only

Shear V - from user input, beam end shear reaction caused by gravity load only
e_x - hor distance from column face to the beam splice inner side bolt group CG point

Bolt group forces	shear V = 19.9 [kips]	axial P = 0.0	[kips]	
Resultant force to ver Y axis angle	$\theta = \tan^{-1}$ (P / V)	= 0.00	[°]	
Bolt group row and column	bolt row n_r = 6	bolt col n_c = 2		
Bolt row/column spacing	bolt row s_r = 3.000 [in]	bolt col s_c = 3.000	[in]	
Shear force to bolt group CG ecc	e_x =	= 26.500	[in]	
Shear force to ver Y axis angle	θ =	= 0.00	[°]	
Bolt group coefficient C	C = from AISC 15th Table 7-6 ~ 7-13	= 2.007		
Bolt group eccentricity coefficient	C_{ec} = C / (n_r x n_c)	= **0.167**		

Beam Web - Inner Side Bolt Group - Bolt Shear ratio = 19.9 / 122.9 = **0.16** **PASS**

Bolt group forces	shear V = 19.9 [kips]	axial P = 0.0	[kips]	
Bolt group resultant force	R = ($V^2 + P^2$)$^{0.5}$	= **19.9**	[kips]	
Bolt shear stress	bolt grade = A325-X	F_{nv} = 68.0	[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 = 12.0	shear plane m = 2		
Bolt group eccentricity coefficient	C_{ec} = from 'Bolt Group Eccentricity' calc	= 0.167		
Required shear strength	V_u =	= **19.9**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 163.9	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J3-1
	ϕ R_n =	= **122.9**	[kips]	
	ratio = **0.16**	> V_u	OK	

Beam Web - Inner Side Bolt Group - Bolt Bearing ratio = 19.9 / 122.9 = **0.16** **PASS**

The bolt group is oriented so that the shear force V is in ver. direction and the axial force P is in hor. direction

Bolt group forces	shear V = 19.9	[kips]	axial P = 0.0	[kips]		
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$		= **19.9**	[kips]		

Single Bolt Shear Strength

Bolt shear stress	bolt grade = A325-X		F_{nv} = 68.0	[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	$R_{n\text{-}bolt} = 2 \times F_{nv} A_b$		= 81.8	[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 = 0.650	[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
	$= 104.6 \le 88.7$	= 88.7	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min (R_{n\text{-}t\&b\text{-}in} , R_{n\text{-}bolt})$	= 81.8	[kips]	

Number of bolt	interior n_{in} = 12			
Bolt group eccentricity coefficient	C_{ec} = from calc in above section	= 0.167		
Bolt bearing strength for all bolts	$R_n = (n_{in} R_{n\text{-}in}) C_{ec}$	= 163.9	[kips]	
Required shear strength	V_u =	= **19.9**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th J3-10
	ϕR_n =	= **122.9**	[kips]	
	ratio = **0.16**	$> V_u$	OK	

Beam Web - Inner Side Bolt Group Eccentricity - Shear + Axial

Shear V - from user input, beam end shear reaction caused by gravity load only
Axial P - from gusset interface forces calc , beam member axial load P_{bm}
e_x - hor distance from the gusset-beam weld line CG point to the beam splice inner side bolt group CG point

Bolt group forces	shear V = 19.9	[kips]	axial P = 71.6	[kips]
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$		= 74.3	[kips]
Resultant force to ver Y axis angle	$\theta = \tan^{-1} (P / V)$		= 74.47	[°]

Bolt group row and column	bolt row n_r = 6		bolt col n_c = 2	
Bolt row/column spacing	bolt row s_r = 3.000	[in]	bolt col s_c = 3.000	[in]
Shear force to bolt group CG ecc	e_x =		= 12.125	[in]
Shear force to ver Y axis angle	θ =		= 74.47	[°]
Bolt group coefficient C	C = from AISC 15th Table 7-6 ~ 7-13		= 8.077	
Bolt group eccentricity coefficient	$C_{ec} = C / (n_r \times n_c)$		= **0.673**	

Beam Web - Inner Side Bolt Group - Bolt Shear		ratio = 74.3 / 495.3	= **0.15**	**PASS**
Bolt group forces	shear V = 19.9 [kips]	axial P = 71.6	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= **74.3**	[kips]	
Bolt shear stress	bolt grade = A325-X	F_{nv} = 68.0	[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 = 12.0	shear plane m = 2		
Bolt group eccentricity coefficient	C_{ec} = from 'Bolt Group Eccentricity' calc	= 0.673		
Required shear strength	V_u =	= **74.3**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 660.5	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J3-1
	ϕR_n =	= **495.3**	[kips]	
	ratio = **0.15**	> V_u	OK	

Beam Web - Inner Side Bolt Group - Bolt Bearing		ratio = 74.3 / 482.8	= **0.15**	**PASS**

The bolt group is oriented so that the shear force V is in ver. direction and the axial force P is in hor. direction

Bolt group forces	shear V = 19.9 [kips]	axial P = 71.6	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= **74.3**	[kips]	
Single Bolt Shear Strength				
Bolt shear stress	bolt grade = A325-X	F_{nv} = 68.0	[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	$R_{n\text{-}bolt} = 2 \times F_{nv} A_b$	= 81.8	[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 = 2.000	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate thickness	t = 0.650 [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
	= 104.6 ≤ 88.7	= 88.7	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min (R_{n\text{-}t\&b\text{-}in} , R_{n\text{-}bolt})$	= 81.8	[kips]	
Edge Bolt				
Bolt hole edge clear distance	$L_c = L_e - d_h / 2$	= 1.531	[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
	= 77.6 ≤ 88.7	= 77.6	[kips]	
Bolt strength at edge	$R_{n\text{-}ed} = \min (R_{n\text{-}t\&b\text{-}ed} , R_{n\text{-}bolt})$	= 77.6	[kips]	
Number of bolt	interior n_{in} = 6	edge n_{ed} = 6		
Bolt group eccentricity coefficient	C_{ec} = from calc in above section	= 0.673		
Bolt bearing strength for all bolts	$R_n = (n_{in} R_{n\text{-}in} + n_{ed} R_{n\text{-}ed}) C_{ec}$	= 643.7	[kips]	
Required shear strength	V_u =	= **74.3**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th J3-10
	ϕR_n =	= **482.8**	[kips]	
	ratio = **0.15**	> V_u	OK	

Beam Web - Outer Side Bolt Group Eccentricity - Shear + Axial

Shear V - from user input, beam end shear reaction caused by gravity load only
Axial P - from gusset interface forces calc , beam member axial load P_{bm}
e_x - hor distance from the gusset-beam weld line CG point to the beam splice outer side bolt group CG point

Bolt group forces	shear V = 19.9 [kips]	axial P = 71.6	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= 74.3	[kips]	
Resultant force to ver Y axis angle	$\theta = \tan^{-1} (P / V)$	= 74.47	[°]	
Bolt group row and column	bolt row n_r = 6	bolt col n_c = 2		
Bolt row/column spacing	bolt row s_r = 3.000 [in]	bolt col s_c = 3.000	[in]	
Shear force to bolt group CG ecc	e_x =	= 20.125	[in]	
Shear force to ver Y axis angle	θ =	= 74.47	[°]	
Bolt group coefficient C	C = from AISC 15th Table 7-6 ~ 7-13	= 6.501		
Bolt group eccentricity coefficient	$C_{ec} = C / (n_r \times n_c)$	= **0.542**		

Beam Web - Outer Side Bolt Group - Bolt Shear   ratio = 74.3 / 398.9 = **0.19** **PASS**

Bolt group forces	shear V = 19.9 [kips]	axial P = 71.6	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= **74.3**	[kips]	
Bolt shear stress	bolt grade = A325-X	F_{nv} = 68.0	[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 = 12.0	shear plane m = 2		
Bolt group eccentricity coefficient	C_{ec} = from 'Bolt Group Eccentricity' calc	= 0.542		
Required shear strength	V_u =	= **74.3**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 531.9	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J3-1
	ϕR_n =	= **398.9**	[kips]	
	ratio = **0.19**	> V_u	OK	

Beam Web - Outer Side Bolt Group - Bolt Bearing ratio = 74.3 / 388.8 = **0.19** **PASS**

The bolt group is oriented so that the shear force V is in ver. direction and the axial force P is in hor. direction

Bolt group forces	shear V = 19.9	[kips]	axial P = 71.6	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$		= **74.3**	[kips]	

Single Bolt Shear Strength

Bolt shear stress	bolt grade = A325-X		F_{nv} = 68.0	[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	$R_{n\text{-bolt}} = 2 \times F_{nv} A_b$		= 81.8	[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 = 2.000	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate thickness	t = 0.650	[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
	= 104.6 ≤ 88.7	= 88.7	[kips]	
Bolt strength at interior	$R_{n\text{-in}} = \min (R_{n\text{-t\&b-in}} , R_{n\text{-bolt}})$	= 81.8	[kips]	

Edge Bolt

Bolt hole edge clear distance	$L_c = L_e - d_h / 2$	= 1.531	[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
	= 77.6 ≤ 88.7	= 77.6	[kips]	
Bolt strength at edge	$R_{n\text{-ed}} = \min (R_{n\text{-t\&b-ed}} , R_{n\text{-bolt}})$	= 77.6	[kips]	

Number of bolt	interior n_{in} = 6	edge n_{ed} = 6		
Bolt group eccentricity coefficient	C_{ec} = from calc in above section	= 0.542		
Bolt bearing strength for all bolts	$R_n = (n_{in} R_{n\text{-in}} + n_{ed} R_{n\text{-ed}})\ C_{ec}$	= 518.4	[kips]	
Required shear strength	V_u =	= **74.3**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th J3-10
	$\phi\ R_n$ =	= **388.8**	[kips]	
	ratio = **0.19**	> V_u	OK	

Splice Plate - Outer Side Bolt Group Eccentricity - Shear Only

Shear V - from user input, beam end shear reaction caused by gravity load only
e_x - hor distance from column face to the beam splice outer side bolt group CG point

Bolt group forces	shear V = 19.9 [kips]	axial P = 0.0	[kips]	
Resultant force to ver Y axis angle	$\theta = \tan^{-1} (P / V)$	= 0.00	[°]	
Bolt group row and column	bolt row n_r = 6	bolt col n_c = 2		
Bolt row/column spacing	bolt row s_r = 3.000 [in]	bolt col s_c = 3.000	[in]	
Shear force to bolt group CG ecc	e_x =	= 34.500	[in]	
Shear force to ver Y axis angle	θ =	= 0.00	[°]	
Bolt group coefficient C	C = from AISC 15th Table 7-6 ~ 7-13	= 1.552		
Bolt group eccentricity coefficient	$C_{ec} = C / (n_r \times n_c)$	= **0.129**		

Splice Plate - Outer Side Bolt Group - Bolt Bearing		ratio = 10.0 / 47.5	= **0.21**	**PASS**	
The bolt group is oriented so that the shear force V is in ver. direction and the axial force P is in hor. direction					
Bolt group forces	shear V = 10.0 [kips]	axial P = 0.0	[kips]		
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= **10.0**	[kips]		
Single Bolt Shear Strength					
Bolt shear stress	bolt grade = A325-X	F_{nv} = 68.0	[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	$R_{n\text{-bolt}} = F_{nv} A_b$	= 40.9	[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 = 0.375 [in]				
Interior Bolt					
Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.063	[in]		
Bolt tear out/bearing strength	$R_{n\text{-t\&b-in}} = 1.2\ L_c\ t\ F_u \leq 2.4\ d_b\ t\ m\ F_u$			AISC 15th Eq J3-6a	
	= 60.3 ≤ 51.2	= 51.2	[kips]		
Bolt strength at interior	$R_{n\text{-in}} = \min (R_{n\text{-t\&b-in}} , R_{n\text{-bolt}})$	= 40.9	[kips]		
Number of bolt	interior n_{in} = 12				
Bolt group eccentricity coefficient	C_{ec} = from calc in above section	= 0.129			
Bolt bearing strength for all bolts	$R_n = (n_{in} R_{n\text{-in}}) C_{ec}$	= 63.3	[kips]		
Required shear strength	V_u =	= **10.0**	[kips]		
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th J3-10	
	ϕR_n =	= **47.5**	[kips]		
	ratio = **0.21**	> V_u	OK		

Splice Plate - Compression Buckling

ratio = 35.8 / 256.1 = **0.14** **PASS**

Plate Compression Check

Plate size	width b_p = 19.000 [in]	thickness t_p = 0.375	[in]	
	F_y = 50.0 [ksi]	E = 29000	[ksi]	
Plate gross area in compression	$A_g = b_p t_p$	= 7.125	[in^2]	
Plate radius of gyration	$r = t_p / \sqrt{12}$	= 0.108	[in]	
Plate effective length factor	K =	= 1.20		AISC 15th Table C-A-7.1
Plate unbraced length	L_u =	= 5.000	[in]	
Plate slenderness	$KL/r = 1.20 \times L_u / r$	= 55.43		
	when $\frac{KL}{r} > 25$, use Chapter E			AISC 15th J4.4 (b)
Elastic buckling stress	$F_e = \frac{\pi^2 E}{(KL/r)^2}$	= 93.17	[ksi]	AISC 15th Eq E3-4
	when $\frac{KL}{r} \le 4.71 \left(\frac{E}{F_y}\right)^{0.5} = 113.43$			AISC 15th E3 (a)
Critical stress	$F_{cr} = 0.658^{(F_y/F_e)} F_y$	= 39.94	[ksi]	AISC 15th Eq E3-2
Plate compression required	P_u = 0.5 x beam axial compression P	= **35.8**	[kips]	
Plate compression provided	$R_n = F_{cr} \times A_g$	= 284.6	[kips]	AISC 15th Eq E3-1
Resistance factor-LRFD	ϕ = 0.90			AISC 15th E1
	ϕR_n =	= **256.1**	[kips]	
	ratio = **0.14**	> P_u	OK	

Splice Plate - Shear Yielding

ratio = 19.9 / 427.5 = **0.05** **PASS**

Splice Plate Shear Yielding

Plate size	width b_p = 19.000 [in]	thickness t_p = 0.375	[in]	
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p \times 2$ side	= 14.250	[in^2]	
Shear force required	V_u =	= **19.9**	[kips]	
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$	= 427.5	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	ϕR_n =	= **427.5**	[kips]	
	ratio = **0.05**	> V_u	OK	

Splice Plate - Flexural Yielding

ratio = 54.73 / 253.83 = **0.22** **PASS**

Plate width & thick	width b_p = 19.000 [in]	thick t_p = 0.375	[in]
	yield F_y = 50.0 [ksi]		
Beam end shear reaction caused by gravity load only	V = from user input	= 19.9	[kips]
Ecc from column face to outer bolt group first bolt line	e =	= 33.000	[in]
Flexural strength required	$M_r = V \times e$	= **54.73**	[kip-ft]
Shear plate - plastic modulus	$Z_p = (b_p \times t_p^2) / 4 \times 2$ side	= 67.69	[in^3]
Flexural strength available	$M_c = \phi F_y Z_p$ ϕ=0.90	= 253.83	[kip-ft]
	ratio = **0.22**	> M_r	OK

Splice Plate - Shear Rupture   ratio = 10.0 / 142.6 = **0.07** **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 = 6				
Plate size	width b_p = 19.000	[in]	thickness t_p = 0.375	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate net area in shear	$A_{nv} = (b_p - n\ d_h)\ t_p$		= 4.875	[in^2]	
Shear force required	V_u =		= **10.0**	[kips]	
Plate shear rupture strength	$R_n = 0.6\ F_u A_{nv}$		= 190.1	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J4-4
	$\phi\ R_n$ =		= **142.6**	[kips]	
	ratio = **0.07**		> V_u	OK	

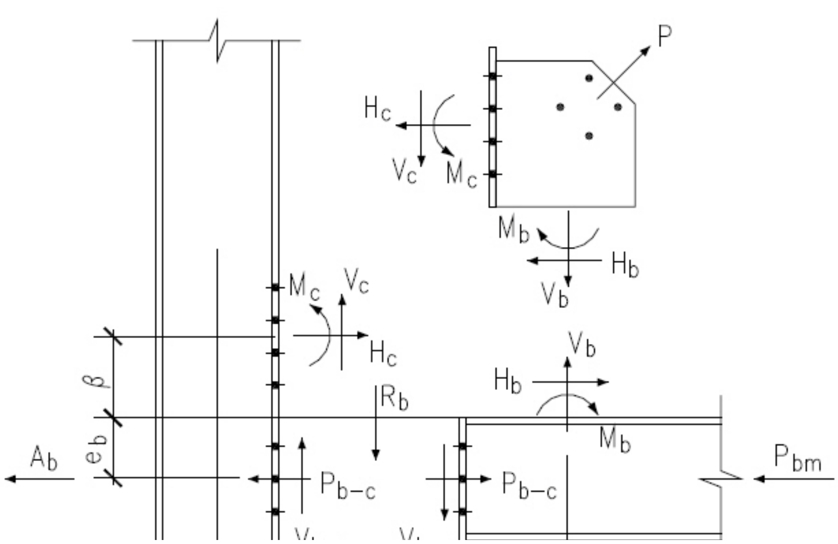
Splice Plate - Flexural Rupture   ratio = 27.36 / 96.36 = **0.28** **PASS**

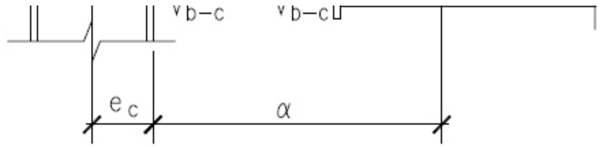
Plate A_n and Z_{net} Calc

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 = 6				
Plate size	width b_p = 19.000	[in]	thickness t_p = 0.375	[in]	
Plate net area	$A_n = (b_p - n\ d_h)\ t_p$		= 4.875	[in^2]	
Plate net plastic sect modulus	Z_{net} =		= 23.72	[in^3]	
Plate net elastic sect modulus	S_{net} =		= 16.33	[in^3]	

Plate width & thick	width b_p = 19.000 [in]	thick t_p = 0.375	[in]	
	F_u = 65.0 [ksi]			
Beam end shear reaction caused by gravity load only	V = from user input, x 0.5 for 2 side plate	= 10.0	[kips]	
Ecc from column face to outer bolt group first bolt line	e =	= 33.000	[in]	
Flexural strength required	M_r = V x e	= **27.36**	[kip-ft]	
Shear plate - net plastic modulus	Z_{net} = from above calc	= 23.72	[in^3]	
Flexural strength available	$M_c = \phi\ F_u Z_{net}$ ϕ=0.75	= **96.36**	[kip-ft]	
	ratio = **0.28**	> M_r	OK	

Seismic - SCBF LC3 & LC4 Gusset Interface Forces Calc





Brace - SCBF Load Case LC3

Top and bottom brace force — Top P_{top} = -559.7 [kips] (T) — Bot P_{bot} = 157.3 [kips] (C)

Beam end shear & transfer force — Shear R_b = 19.9 [kips] — Transfer A_b = -27.6 [kips]

Top Brace Interface Forces

Refer to AISC 15th Page 13-4 and Fig. 13-2 for all charts and definitions of variables and symbols shown in calculation below

e_b = 12.350 [in] — e_c = 6.450 [in]

α = 14.000 [in] — β = 9.625 [in]

θ = 45.0 [°]

	$K = e_b \tan\theta - e_c$	= 5.900	[in]	AISC 15th Eq. 13-16
	$D = \tan^2\theta + (\frac{\alpha}{\beta})^2$	= 3.116		AISC 15th Eq. 13-24
	$K' = \alpha\ (\tan\theta + \frac{\alpha}{\beta})$	= 34.364		AISC 15th Eq. 13-23
	$\bar{\alpha} = [\ K' \tan\theta + K\ (\frac{\alpha}{\beta})^2\ ]\ /\ D$	= 14.000	[in]	AISC 15th Eq. 13-21
	$\bar{\beta} = (\ K' - K \tan\theta\ )\ /\ D$	= 8.100	[in]	AISC 15th Eq. 13-22
	$r = [\ (\ e_b + \bar{\beta}\ )^2 + (\ e_c + \bar{\alpha}\ )^2\ ]^{0.5}$	= 28.921	[in]	AISC 15th Eq. 13-6
Brace axial force	P_u = from seismic brace force calc	= -559.7	[kips]	in tension
Gusset to Column Interface Forces				
Shear force	$V_c = (\ \bar{\beta}\ /\ r\ )\ P_u$	= **-156.8**	[kips]	AISC 15th Eq. 13-2
Axial force	$H_c = (\ e_c\ /\ r\ )\ P_u$	= **-124.8**	[kips]	AISC 15th Eq. 13-3
Moment	$M_c = H_c\ (\ \beta - \bar{\beta}\ )$	= **15.86**	[kip-ft]	AISC 15th Eq. 13-19
Gusset to Beam Interface Forces				
Shear force	$H_b = (\ \bar{\alpha}\ /\ r\ )\ P_u$	= **-270.9**	[kips]	AISC 15th Eq. 13-5
Axial force	$V_b = (\ e_b\ /\ r\ )\ P_u$	= **-239.0**	[kips]	AISC 15th Eq. 13-4
Moment	$M_b = V_b\ (\ \bar{\alpha} - \alpha\ )$	= **0.00**	[kip-ft]	AISC 15th Eq. 13-17

Bottom Brace Interface Forces

Refer to AISC 15th Page 13-4 and Fig. 13-2 for all charts and definitions of variables and symbols shown in calculation below

e_b = 12.350 [in] — e_c = 6.450 [in]

α = 14.750 [in] — β = 10.375 [in]

θ = 45.0 [°]

$K = e_b \tan\theta - e_c$	= 5.900	[in]	AISC 15th Eq. 13-16
$D = \tan^2\theta + (\frac{\alpha}{\beta})^2$	= 3.021		AISC 15th Eq. 13-24
$K' = \alpha\ (\tan\theta + \frac{\alpha}{\beta})$	= 35.720		AISC 15th Eq. 13-23
$\bar{\alpha} = [\ K' \tan\theta + K\ (\frac{\alpha}{\beta})^2\ ]\ /\ D$	= 14.750	[in]	AISC 15th Eq. 13-21

	$\bar{\beta} = (K' - K \tan\theta) / D$	= 8.850	[in]	AISC 15th Eq. 13-22
	$r = [(e_b + \bar{\beta})^2 + (e_c + \bar{\alpha})^2]^{0.5}$	= 29.981	[in]	AISC 15th Eq. 13-6
Brace axial force	P_u = from seismic brace force calc	= 157.3	[kips]	in compression
Gusset to Column Interface Forces				
Shear force	$V_c = (\bar{\beta} / r) P_u$	= **46.4**	[kips]	AISC 15th Eq. 13-2
Axial force	$H_c = (e_c / r) P_u$	= **33.8**	[kips]	AISC 15th Eq. 13-3
Moment	$M_c = H_c (\beta - \bar{\beta})$	= **-4.30**	[kip-ft]	AISC 15th Eq. 13-19
Gusset to Beam Interface Forces				
Shear force	$H_b = (\bar{\alpha} / r) P_u$	= **77.4**	[kips]	AISC 15th Eq. 13-5
Axial force	$V_b = (e_b / r) P_u$	= **64.8**	[kips]	AISC 15th Eq. 13-4
Moment	$M_b = V_b (\bar{\alpha} - \alpha)$	= **0.00**	[kip-ft]	AISC 15th Eq. 13-17

Beam to Column Interface Forces

Beam to Column Interface Shear Force				
Beam end shear reaction	R_b = from user input	= 19.9	[kips]	
Top brace gusset-beam axial force	$V_{b\text{-}top}$ =	= -239.0	[kips]	AISC 15th Eq. 13-4
Bot brace gusset-beam axial force	$V_{b\text{-}bot}$ =	= 64.8	[kips]	AISC 15th Eq. 13-4
Beam to column shear force	$V_{b\text{-}c} = R_b + V_{b\text{-}top} - V_{b\text{-}bot}$	= **-283.9**	[kips]	AISC 15th Page 13-4
Beam to Column Interface Axial Force				
Top brace gusset-column axial force	$H_{c\text{-}top}$ =	= -124.8	[kips]	AISC 15th Eq. 13-3
Bot brace gusset-column axial force	$H_{c\text{-}bot}$ =	= 33.8	[kips]	AISC 15th Eq. 13-3
Transfer force from adjacent bay	A_b = from user input	= -27.6	[kips]	
Beam to column axial force	$P_{b\text{-}c} = (H_{c\text{-}top} + H_{c\text{-}bot}) \times -1 - A_b$	= **118.6**	[kips]	AISC 15th Page 13-4

Beam Member Axial Force

This force is not for use in connection calc. It's output here for user input connection forces equilibrium check only.

P_{bm} - Beam member axial force is different from $P_{b\text{-}c}$ - Beam to column interface axial force as shown above.

P_{bm} - Beam member axial force is from structural analysis output and cannot be used directly in beam end to column connection design as this force is interrupted by brace gusset to beam interface force before beam end reaching the column. This force is actually not needed from user's input for beam end to column connection design.

$P_{b\text{-}c}$ - Beam to column interface axial force is calculated from user's input of brace axial forces and trasnfer force using uniform force method. This force is used in the beam end to column connection design.

P_{bm} - Beam member axial force is not needed for the beam end to column connection design and is calculated here for verification purpose only. If it matches the structural analysis output, that means equilibrium is reached and user's input of brace axial forces and trasnfer force are correct.

Top brace axial force	P_t = from seismic brace force calc	= -559.7	[kips]	in tension
Top brace to ver line angle	θ_t = from user input	= 45.0	[°]	
Top brace gusset-column axial force	H_{ct} = from calc shown above	= -124.8	[kips]	AISC 15th Eq. 13-3
Bot brace axial force	P_b = from seismic brace force calc	= 157.3	[kips]	in compression
Bot brace to ver line angle	θ_b = from user input	= 45.0	[°]	
Bot brace gusset-column axial force	H_{cb} = from calc shown above	= 33.8	[kips]	AISC 15th Eq. 13-3
Beam to column interface axial force	$P_{b\text{-}c}$ = from calc shown above	= 118.6	[kips]	AISC 15th Page 13-4
Beam member axial force	$P_{bm} = (H_{ct} - P_t \sin\theta_t) + (H_{cb} - P_b \sin\theta_b) + P_{b\text{-}c}$	= **312.1**	[kips]	in compression

Brace - SCBF Load Case LC4

Top and bottom brace force	Top P_{top} = 134.7	[kips] (C)	Bot P_{bot} = -615.9	[kips] (T)	
Beam end shear & transfer force	Shear R_b = 19.9	[kips]	Transfer A_b = -27.6	[kips]	

Top Brace Interface Forces

Refer to AISC 15th Page 13-4 and Fig. 13-2 for all charts and definitions of variables and symbols shown in calculation below

e_b = 12.350 [in]	e_c = 6.450	[in]		
α = 14.000 [in]	β = 9.625	[in]		
θ = 45.0 [°]				
$K = e_b \tan\theta - e_c$	= 5.900	[in]	AISC 15^{th} Eq. 13-16	
$D = \tan^2\theta + (\frac{\alpha}{\beta})^2$	= 3.116		AISC 15^{th} Eq. 13-24	
$K' = \alpha (\tan\theta + \frac{\alpha}{\beta})$	= 34.364		AISC 15^{th} Eq. 13-23	
$\bar{\alpha} = [K' \tan\theta + K (\frac{\alpha}{\beta})^2] / D$	= 14.000	[in]	AISC 15^{th} Eq. 13-21	
$\bar{\beta} = (K' - K \tan\theta) / D$	= 8.100	[in]	AISC 15^{th} Eq. 13-22	
$r = [(e_b + \bar{\beta})^2 + (e_c + \bar{\alpha})^2]^{0.5}$	= 28.921	[in]	AISC 15^{th} Eq. 13-6	

Brace axial force	P_u = from seismic brace force calc	= 134.7	[kips]	in compression
Gusset to Column Interface Forces				
Shear force	$V_c = (\bar{\beta} / r) P_u$	= **37.7**	[kips]	AISC 15^{th} Eq. 13-2
Axial force	$H_c = (e_c / r) P_u$	= **30.0**	[kips]	AISC 15^{th} Eq. 13-3
Moment	$M_c = H_c (\beta - \bar{\beta})$	= **-3.82**	[kip-ft]	AISC 15^{th} Eq. 13-19
Gusset to Beam Interface Forces				
Shear force	$H_b = (\bar{\alpha} / r) P_u$	= **65.2**	[kips]	AISC 15^{th} Eq. 13-5
Axial force	$V_b = (e_b / r) P_u$	= **57.5**	[kips]	AISC 15^{th} Eq. 13-4
Moment	$M_b = V_b (\bar{\alpha} - \alpha)$	= **0.00**	[kip-ft]	AISC 15^{th} Eq. 13-17

Bottom Brace Interface Forces

Refer to AISC 15th Page 13-4 and Fig. 13-2 for all charts and definitions of variables and symbols shown in calculation below

e_b = 12.350 [in]	e_c = 6.450	[in]		
α = 14.750 [in]	β = 10.375	[in]		
θ = 45.0 [°]				
$K = e_b \tan\theta - e_c$	= 5.900	[in]	AISC 15^{th} Eq. 13-16	
$D = \tan^2\theta + (\frac{\alpha}{\beta})^2$	= 3.021		AISC 15^{th} Eq. 13-24	
$K' = \alpha (\tan\theta + \frac{\alpha}{\beta})$	= 35.720		AISC 15^{th} Eq. 13-23	
$\bar{\alpha} = [K' \tan\theta + K (\frac{\alpha}{\beta})^2] / D$	= 14.750	[in]	AISC 15^{th} Eq. 13-21	
$\bar{\beta} = (K' - K \tan\theta) / D$	= 8.850	[in]	AISC 15^{th} Eq. 13-22	
$r = [(e_b + \bar{\beta})^2 + (e_c + \bar{\alpha})^2]^{0.5}$	= 29.981	[in]	AISC 15^{th} Eq. 13-6	

Brace axial force	P_u = from seismic brace force calc	= -615.9	[kips]	in tension
Gusset to Column Interface Forces				
Shear force	$V_c = (\bar{\beta} / r) P_u$	= **-181.8**	[kips]	AISC 15^{th} Eq. 13-2

Axial force	$H_c = (e_c / r) P_u$	= **-132.5**	[kips]	AISC 15th Eq. 13-3
Moment	$M_c = H_c (\beta - \bar{\beta})$	= **16.84**	[kip-ft]	AISC 15th Eq. 13-19

Gusset to Beam Interface Forces

Shear force	$H_b = (\bar{\alpha} / r) P_u$	= **-303.0**	[kips]	AISC 15th Eq. 13-5
Axial force	$V_b = (e_b / r) P_u$	= **-253.7**	[kips]	AISC 15th Eq. 13-4
Moment	$M_b = V_b (\bar{\alpha} - \alpha)$	= **0.00**	[kip-ft]	AISC 15th Eq. 13-17

Beam to Column Interface Forces

Beam to Column Interface Shear Force

Beam end shear reaction	R_b = from user input	= 19.9	[kips]	
Top brace gusset-beam axial force	$V_{b\text{-}top}$ =	= 57.5	[kips]	AISC 15th Eq. 13-4
Bot brace gusset-beam axial force	$V_{b\text{-}bot}$ =	= -253.7	[kips]	AISC 15th Eq. 13-4
Beam to column shear force	$V_{b\text{-}c} = R_b + V_{b\text{-}top} - V_{b\text{-}bot}$	= **331.1**	[kips]	AISC 15th Page 13-4

Beam to Column Interface Axial Force

Top brace gusset-column axial force	$H_{c\text{-}top}$ =	= 30.0	[kips]	AISC 15th Eq. 13-3
Bot brace gusset-column axial force	$H_{c\text{-}bot}$ =	= -132.5	[kips]	AISC 15th Eq. 13-3
Transfer force from adjacent bay	A_b = from user input	= -27.6	[kips]	
Beam to column axial force	$P_{b\text{-}c} = (H_{c\text{-}top} + H_{c\text{-}bot}) \times -1 - A_b$	= **130.1**	[kips]	AISC 15th Page 13-4

Beam Member Axial Force

This force is not for use in connection calc. It's output here for user input connection forces equilibrium check only.

P_{bm} - Beam member axial force is different from $P_{b\text{-}c}$ - Beam to column interface axial force as shown above.

P_{bm} - Beam member axial force is from structural analysis output and cannot be used directly in beam end to column connection design as this force is interrupted by brace gusset to beam interface force before beam end reaching the column. This force is actually not needed from user's input for beam end to column connection design.

$P_{b\text{-}c}$ - Beam to column interface axial force is calculated from user's input of brace axial forces and trasnfer force using uniform force method. This force is used in the beam end to column connection design.

P_{bm} - Beam member axial force is not needed for the beam end to column connection design and is calculated here for verification purpose only. If it matches the structural analysis output, that means equilibrium is reached and user's input of brace axial forces and trasnfer force are correct.

Top brace axial force	P_t = from seismic brace force calc	= 134.7	[kips]	in compression
Top brace to ver line angle	θ_t = from user input	= 45.0	[°]	
Top brace gusset-column axial force	H_{ct} = from calc shown above	= 30.0	[kips]	AISC 15th Eq. 13-3
Bot brace axial force	P_b = from seismic brace force calc	= -615.9	[kips]	in tension
Bot brace to ver line angle	θ_b = from user input	= 45.0	[°]	
Bot brace gusset-column axial force	H_{cb} = from calc shown above	= -132.5	[kips]	AISC 15th Eq. 13-3
Beam to column interface axial force	$P_{b\text{-}c}$ = from calc shown above	= 130.1	[kips]	AISC 15th Page 13-4
Beam member axial force	$P_{bm} = (H_{ct} - P_t \sin\theta_t) + (H_{cb} - P_b \sin\theta_b) + P_{b\text{-}c}$	= **312.7**	[kips]	in compression

Top Brace - Gusset to Column Direct Weld Connection Code=AISC 360-16 LRFD

Result Summary geometries & weld limitations = **PASS** limit states max ratio = **0.82** **PASS**

Weld Limitation Checks - Gusset to Column **PASS**

Min Fillet Weld Size

Thinner part joined thickness	t =	= 0.750	[in]	
Min fillet weld size allowed	w_{min} =	= **0.250**	[in]	AISC 15th Table J2.4
Fillet weld size provided	w =	= **0.438**	[in]	
		$\geq w_{min}$	OK	

Min Fillet Weld Length

Fillet weld size provided	w =	= 0.438	[in]	
Min fillet weld length allowed	$L_{min} = 4 \times w$	= **1.750**	[in]	AISC 15th J2.2b
Min fillet weld length	L =	= **17.250**	[in]	
		$\geq L_{min}$	OK	

Seismic SCBF LC3 P_1 =-559.7 kips (T) P_2 =157.3 kips (C) ratio = **0.82** **PASS**

Gusset Plate - Shear Yielding ratio = 156.8 / 388.1 = **0.40** **PASS**

Plate Shear Yielding Check

Plate size	width b_p = 17.250 [in]	thickness t_p = 0.750	[in]		
Plate yield strength	F_y = 50.0 [ksi]				
Plate gross area in shear	$A_{gv} = b_p t_p$	= 12.938	[in^2]		
Shear force required	V_u =	= **156.8**	[kips]		
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$	= 388.1	[kips]	AISC 15th Eq J4-3	
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3	
	ϕR_n =	= **388.1**	[kips]		
	ratio = **0.40**	$> V_u$	OK		

Gusset Plate - Shear Rupture ratio = 156.8 / 378.4 = **0.41** **PASS**

Plate Shear Rupture Check

Plate size	width b_p = 17.250 [in]	thickness t_p = 0.750	[in]		
Plate tensile strength	F_u = 65.0 [ksi]				
Plate net area in shear	$A_{nv} = b_p t_p$	= 12.938	[in^2]		
Shear force in demand	V_u =	= **156.8**	[kips]		
Plate shear rupture strength	$R_n = 0.6 F_u A_{nv}$	= 504.6	[kips]	AISC 15th Eq J4-4	
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-4	
	ϕR_n =	= **378.4**	[kips]		
	ratio = **0.41**	$> V_u$	OK		

Gusset Plate - Axial Tensile Yield		ratio = 124.8 / 582.2	= **0.21**		**PASS**
Plate Tensile Yielding Check					
Plate size	width b_p = 17.250 [in]	thickness t_p = 0.750		[in]	
Plate yield strength	F_y = 50.0 [ksi]				
Plate gross area in shear	$A_g = b_p t_p$		= 12.938	[in^2]	
Tensile force required	P_u =		= **124.8**	[kips]	
Plate tensile yielding strength	$R_n = F_y A_g$		= 646.9	[kips]	AISC 15th Eq J4-1
Resistance factor-LRFD	ϕ = 0.90				AISC 15th Eq J4-1
	ϕR_n =		= **582.2**	[kips]	
	ratio = **0.21**		> P_u	OK	

Gusset Plate - Axial Tensile Rupture		ratio = 124.8 / 630.7	= **0.20**		**PASS**
Plate Tensile Rupture Check					
Plate size	width b_p = 17.250 [in]	thickness t_p = 0.750		[in]	
Plate tensile strength	F_u = 65.0 [ksi]				
Plate net area in tension	$A_{nt} = b_p t_p$		= 12.938	[in^2]	
Tensile force required	P_u =		= **124.8**	[kips]	
Plate tensile rupture strength	$R_n = F_u A_{nt}$		= 840.9	[kips]	AISC 15th Eq J4-2
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J4-2
	ϕR_n =		= **630.7**	[kips]	AISC 15th Eq J4-2
	ratio = **0.20**		> P_u	OK	

Gusset Plate - Flexural Yield Interact		ratio =	= **0.25**		**PASS**
Gusset plate	width b_p = 17.250 [in]	thick t_p = 0.750		[in]	
	yield F_y = 50.0 [ksi]				
Shear plate - gross area	$A_g = b_p \times t_p$		= 12.938	[in^2]	
Shear plate - plastic modulus	$Z_p = (b_p \times t_p^2) / 4$		= 55.79	[in^3]	
Flexural strength available	$M_c = \phi F_y Z_p$ ϕ=0.90		= 209.22	[kip-ft]	
Flexural strength required	M_r = from gusset interface forces calc		= 15.86	[kip-ft]	
Axial strength available	P_c = from axial tensile yield check		= 582.2	[kips]	
Axial strength required	P_r = from gusset interface forces calc		= -124.8	[kips]	
Shear strength available	V_c = from shear yielding check		= 388.1	[kips]	
Shear strength required	V_r = from gusset interface forces calc		= 156.8	[kips]	
Flexural yield interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right)^2$		= **0.25**		AISC 15th Eq 10-5
			< 1.0	OK	

Gusset Plate - Flexural Rupture Interact		ratio =	= **0.24**	**PASS**
Gusset plate	width b_p = 17.250 [in]	thick t_p = 0.750 [in]		
	tensile F_u = 65.0 [ksi]			
Net area of plate	$A_n = b_p \times t_p$	= 12.938 [in^2]		
Plastic modulus of net section	$Z_{net} = (b_p \times t_p^2) / 4$	= 55.79 [in^3]		
Flexural strength available	$M_c = \phi F_u Z_{net}$ ϕ=0.75	= 226.66 [kip-ft]		
Flexural strength required	M_r = from gusset interface forces calc	= 15.86 [kip-ft]		
Axial strength available	P_c = from axial tensile rupture check	= 630.7 [kips]		
Axial strength required	P_r = from gusset interface forces calc	= -124.8 [kips]		
Shear strength available	V_c = from shear rupture check	= 378.4 [kips]		
Shear strength required	V_r = from gusset interface forces calc	= 156.8 [kips]		
Flexural rupture interaction	ratio $= \left(\frac{V_r}{V_c} \right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c} \right)^2$	= **0.24**	AISC 15^{th} Eq 10-5	
		< 1.0	OK	

Gusset to Column Weld Strength		ratio = 14.33 / 17.55	= **0.82**	**PASS**
Gusset to Column Interface - Forces				
	shear V_c = 156.8 [kips]	axial H_c = -124.8 [kips] in tension		
	moment M_c = 15.86 [kip-ft]			
Gusset to Column Interface - Weld Length				
Gusset-column fillet weld length	L_{wc} = from user input	= **17.250** [in]		
Gusset to Column Interface - Combined Weld Stress				
Weld stress from axial force	$f_a = H_c / L_{wc}$	= -7.235 [kip/in]	in tension	
Weld stress from shear force	$f_v = V_c / L_{wc}$	= 9.090 [kip/in]		
Weld stress from moment force	$f_b = \frac{M}{L^2 / 6}$	= 3.838 [kip/in]		
Weld stress combined - max	$f_{max} = [(f_a - f_b)^2 + f_v^2]^{0.5}$	= **14.326** [kip/in]	AISC 15^{th} Eq 8-11	
Weld resultant load angle	$\theta = \tan^{-1} [(f_b - f_a) / f_v]$	= 50.6 [°]		
Fillet Weld Strength Calc				
Fillet weld leg size	$w = {}^{7}/_{16}$ [in]	load angle θ = 50.6 [°]		
Electrode strength	F_{EXX} = 70.0 [ksi]	strength coeff C_1 = 1.00	AISC 15^{th} Table 8-3	
Number of weld line	n = 2 for double fillet			
Load angle coefficient	$C_2 = (1 + 0.5 \sin^{1.5} \theta)$	= 1.34	AISC 15^{th} Page 8-9	
Fillet weld shear strength	$R_{n-w} = 0.6 (C_1 \times 70\ ksi)\ 0.707\ w\ n\ C_2$	= 34.810 [kip/in]	AISC 15^{th} Eq 8-1	
Base metal - gusset plate	thickness t = 0.750 [in]	tensile F_u = 65.0 [ksi]		
Base metal - gusset plate is in shear, shear rupture as per AISC 15^{th} Eq J4-4 is checked			AISC 15^{th} J2.4	
Base metal shear rupture	$R_{n-b} = 0.6\ F_u\ t$	= 29.250 [kip/in]	AISC 15^{th} Eq J4-4	
Double fillet linear shear strength	$R_n = \min (R_{n-w} , R_{n-b})$	= **29.250** [kip/in]	AISC 15^{th} Eq 9-2	
Resistance factor-LRFD	ϕ = 0.75		AISC 15^{th} Eq 8-1	
	ϕR_n =	= 21.938 [kip/in]		
When gusset plate is directly welded to beam or column, apply 1.25 ductility factor to allow adequate force redistribution in the weld group			AISC 15^{th} Page 13-11	
Weld strength used for design after applying ductility factor	$\phi R_n = \phi R_n \times (1/1.25)$	= **17.550** [kip/in]		
	ratio = **0.82**	> f_{max}	OK	

Column Web Local Yielding		ratio = 168.9 / 768.6	= **0.22**	**PASS**
Gusset Edge Equivalent Normal Force				
Refer to AISC DG29 Fig. B-1 for formula below to calculate gusset edge equivalent normal force				
Gusset edge axial force	$N =$	= -124.8	[kips]	
Gusset edge moment force	$M =$	= 15.86	[kip-ft]	
Gusset edge interface length	$L =$	= 17.250	[in]	
Gusset edge equivalent normal force	$N_e = N - \frac{4\,M}{L}$	= -168.9	[kips]	AISC DG29 Fig B-1
Concentrated force from gusset	$P_u =$	= **168.9**	[kips]	
Column section	d = 12.900 [in]	t_f = 0.990	[in]	
	t_w = 0.610 [in]	k = 1.590	[in]	
	yield F_y = 50.0 [ksi]			
Length of bearing	l_b = Gusset/Column interface length	= 17.250	[in]	
Column web local yielding strength	$R_n = F_y\, t_w\, (\, 5\, k + l_b\,)$	= 768.6	[kips]	AISC 15th Eq J10-2
Resistance factor-LRFD	ϕ = 1.00			
	$\phi\, R_n =$	= **768.6**	[kips]	
	ratio = **0.22**	$> P_u$	OK	

Column Web Shear Strength		ratio = 124.8 / 236.1	= **0.53**	**PASS**
W Shape Column Shear Strength Check				
W sect W12X106	d = 12.900 [in]	t_w = 0.610	[in]	
	F_y = 50.0 [ksi]			
Gusset to column axial force	V_u = from gusset interface force calc	= **124.8**	[kips]	
Column shear strength	$R_n = 0.6\, F_y\, d\, t_w$	= 236.1	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	$\phi\, R_n =$	= **236.1**	[kips]	
	ratio = **0.53**	$> V_u$	OK	

Seismic SCBF LC4 P_1 =134.7 kips (C) P_2 =-615.9 kips (T) ratio = **0.14** **PASS**

Gusset Plate - Shear Yielding		ratio = 37.7 / 388.1	= **0.10**	**PASS**
Plate Shear Yielding Check				
Plate size	width b_p = 17.250 [in]	thickness t_p = 0.750	[in]	
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p\, t_p$	= 12.938	[in^2]	
Shear force required	$V_u =$	= **37.7**	[kips]	
Plate shear yielding strength	$R_n = 0.6\, F_y\, A_{gv}$	= 388.1	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	$\phi\, R_n =$	= **388.1**	[kips]	
	ratio = **0.10**	$> V_u$	OK	

Gusset Plate - Shear Rupture		ratio = 37.7 / 378.4	= **0.10**	**PASS**

Plate Shear Rupture Check

Plate size	width b_p = 17.250 [in]	thickness t_p = 0.750	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate net area in shear	$A_{nv} = b_p t_p$	= 12.938	[in^2]	
Shear force in demand	V_u =	= **37.7**	[kips]	
Plate shear rupture strength	$R_n = 0.6\, F_u A_{nv}$	= 504.6	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-4
	ϕR_n =	= **378.4**	[kips]	
	ratio = **0.10**	> V_u	OK	

Gusset Plate - Axial Yield		ratio = 30.0 / 582.2	= **0.05**	**PASS**

Plate Tensile Yielding Check

Plate size	width b_p = 17.250 [in]	thickness t_p = 0.750	[in]	
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_g = b_p t_p$	= 12.938	[in^2]	
Tensile force required	P_u =	= **30.0**	[kips]	
Plate tensile yielding strength	$R_n = F_y A_g$	= 646.9	[kips]	AISC 15th Eq J4-1
Resistance factor-LRFD	ϕ = 0.90			AISC 15th Eq J4-1
	ϕR_n =	= **582.2**	[kips]	
	ratio = **0.05**	> P_u	OK	

Gusset Plate - Flexural Yield Interact		ratio =	= **0.01**	**PASS**

Gusset plate	width b_p = 17.250 [in]	thick t_p = 0.750	[in]	
	yield F_y = 50.0 [ksi]			
Shear plate - gross area	$A_g = b_p \times t_p$	= 12.938	[in^2]	
Shear plate - plastic modulus	$Z_p = (b_p \times t_p^2) / 4$	= 55.79	[in^3]	
Flexural strength available	$M_c = \phi F_y Z_p$ ϕ=0.90	= 209.22	[kip-ft]	
Flexural strength required	M_r = from gusset interface forces calc	= 3.82	[kip-ft]	
Axial strength available	P_c = from axial tensile yield check	= 582.2	[kips]	
Axial strength required	P_r = from gusset interface forces calc	= 30.0	[kips]	
Shear strength available	V_c = from shear yielding check	= 388.1	[kips]	
Shear strength required	V_r = from gusset interface forces calc	= 37.7	[kips]	
Flexural yield interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right)^2$	= **0.01**		AISC 15th Eq 10-5
		< 1.0	OK	

Gusset Plate - Flexural Rupture Interact		ratio =	= **0.01**	**PASS**
Gusset plate	width b_p = 17.250 [in]	thick t_p = 0.750 [in]		
	tensile F_u = 65.0 [ksi]			
Net area of plate	$A_n = b_p \times t_p$	= 12.938 [in^2]		
Plastic modulus of net section	$Z_{net} = (b_p \times t_p^2) / 4$	= 55.79 [in^3]		
Flexural strength available	$M_c = \phi F_u Z_{net}$ ϕ=0.75	= 226.66 [kip-ft]		
Flexural strength required	M_r = from gusset interface forces calc	= 3.82 [kip-ft]		
Shear strength available	V_c = from shear rupture check	= 378.4 [kips]		
Shear strength required	V_r = from gusset interface forces calc	= 37.7 [kips]		
Flexural rupture interaction	ratio $= \left(\frac{V_r}{V_c} \right)^2 + \left(\frac{M_r}{M_c} \right)^2$	= **0.01**	AISC 15th Eq 10-5	
		< 1.0	OK	

Gusset to Column Weld Strength		ratio = 2.19 / 15.59	= **0.14**	**PASS**
Gusset to Column Interface - Forces				
	shear V_c = 37.7 [kips]	axial H_c = 30.0 [kips]	in compression	
	moment M_c = 3.82 [kip-ft]			
Gusset to Column Interface - Weld Length				
Gusset-column fillet weld length	L_{wc} = from user input	= **17.250** [in]		
Gusset to Column Interface - Combined Weld Stress				
Weld stress from axial force	$f_a = H_c / L_{wc}$	= 1.739 [kip/in]	in compression	
Weld stress from shear force	$f_v = V_c / L_{wc}$	= 2.186 [kip/in]		
Weld stress from moment force	$f_b = \frac{M}{L^2 / 6}$	= 0.924 [kip/in]		
Weld stress combined - max	$f_{max} = f_v$	= **2.186** [kip/in]	AISC 15th Eq 8-11	
Weld resultant load angle	θ = weld only has shear component	= 0.0 [°]		
Fillet Weld Strength Calc				
Fillet weld leg size	$w = ^7/_{16}$ [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	$R_{n-w} = 0.6 (C_1 \times 70 \text{ ksi}) 0.707 \text{ w n } C_2$	= 25.982 [kip/in]	AISC 15th Eq 8-1	
Base metal - gusset plate	thickness t = 0.750 [in]	tensile F_u = 65.0 [ksi]		
Base metal - gusset plate is in shear, <u>shear</u> rupture as per AISC 15th Eq J4-4 is checked			AISC 15th J2.4	
Base metal shear rupture	$R_{n-b} = 0.6 F_u t$	= 29.250 [kip/in]	AISC 15th Eq J4-4	
Double fillet linear shear strength	$R_n = \min (R_{n-w} , R_{n-b})$	= **25.982** [kip/in]	AISC 15th Eq 9-2	
Resistance factor-LRFD	ϕ = 0.75		AISC 15th Eq 8-1	
	ϕR_n =	= 19.487 [kip/in]		
When gusset plate is directly welded to beam or column, apply 1.25 ductility factor to allow adequate force redistribution in the weld group			AISC 15th Page 13-11	
Weld strength used for design after applying ductility factor	$\phi R_n = \phi R_n \times (1/1.25)$	= **15.589** [kip/in]		
	ratio **0.14**	> f_{max}	OK	

Column Web Local Yielding

ratio = 40.6 / 768.6 = **0.05** **PASS**

Gusset Edge Equivalent Normal Force

Refer to AISC DG29 Fig. B-1 for formula below to calculate gusset edge equivalent normal force

Gusset edge axial force	N =	= 30.0	[kips]	
Gusset edge moment force	M =	= 3.82	[kip-ft]	
Gusset edge interface length	L =	= 17.250	[in]	
Gusset edge equivalent normal force	$N_e = N + \frac{4\,M}{L}$	= 40.6	[kips]	AISC DG29 Fig B-1
Concentrated force from gusset	P_u =	= **40.6**	[kips]	
Column section	d = 12.900 [in]	t_f = 0.990	[in]	
	t_w = 0.610 [in]	k = 1.590	[in]	
	yield F_y = 50.0 [ksi]			
Length of bearing	l_b = Gusset/Column interface length	= 17.250	[in]	
Column web local yielding strength	$R_n = F_y t_w (5\,k + l_b)$	= 768.6	[kips]	AISC 15th Eq J10-2
Resistance factor-LRFD	ϕ = 1.00			
	$\phi\,R_n$ =	= **768.6**	[kips]	
	ratio = **0.05**	> P_u	OK	

Column Web Local Crippling

ratio = 40.6 / 1007.0 = **0.04** **PASS**

Gusset Edge Equivalent Normal Force

Refer to AISC DG29 Fig. B-1 for formula below to calculate gusset edge equivalent normal force

Gusset edge axial force	N =	= 30.0	[kips]	
Gusset edge moment force	M =	= 3.82	[kip-ft]	
Gusset edge interface length	L =	= 17.250	[in]	
Gusset edge equivalent normal force	$N_e = N + \frac{4\,M}{L}$	= 40.6	[kips]	AISC DG29 Fig B-1
Concentrated force from gusset	P_u =	= **40.6**	[kips]	
Column section	d = 12.900 [in]	t_f = 0.990	[in]	
	t_w = 0.610 [in]	k = 1.590	[in]	
	yield F_y = 50.0 [ksi]	E = 29000	[ksi]	
Length of bearing	l_b = Gusset/Column interface length	= 17.250	[in]	
	when $l_N \geq d/2$, use Eq J10-4			AISC 15th Eq J10-4
Column web local crippling strength	$R_n = 0.8\,t_w^2\,[1+3\frac{l_b}{d}(\frac{t_w}{t_f})^{1.5}]\times(\frac{E\,F_y t_f}{t_w})^{0.5}$	= 1342.7	[kips]	AISC 15th Eq J10-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th J10.3
	ϕR_n =	= **1007.0**	[kips]	
	ratio = **0.04**	> P_u	OK	

Column Web Shear Strength		ratio = 30.0 / 236.1	= **0.13**	**PASS**

W Shape Column Shear Strength Check

W sect W12X106	d = 12.900 [in]	t_w = 0.610	[in]	
	F_y = 50.0 [ksi]			
Gusset to column axial force	V_u = from gusset interface force calc	= **30.0**	[kips]	
Column shear strength	$R_n = 0.6\ F_y\ d\ t_w$	= 236.1	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	$\phi\ R_n$ =	= **236.1**	[kips]	
	ratio = **0.13**	> V_u	OK	

Top Brace - Gusset to Beam Direct Weld Connection Code=AISC 360-16 LRFD

Result Summary	geometries & weld limitations = **PASS**	limit states max ratio = **0.79**	**PASS**

Brace Weld Limitation Checks - Gusset to Beam **PASS**

Min Fillet Weld Size

Thinner part joined thickness	$t =$	= 0.750	[in]	
Min fillet weld size allowed	$w_{min} =$	= **0.250**	[in]	AISC 15th Table J2.4
Fillet weld size provided	$w =$	= **0.438**	[in]	
		$\geq w_{min}$	OK	

Min Fillet Weld Length

Fillet weld size provided	$w =$	= 0.438	[in]	
Min fillet weld length allowed	$L_{min} = 4 \times w$	= **1.750**	[in]	AISC 15th J2.2b
Min fillet weld length	$L =$	= **26.000**	[in]	
		$\geq L_{min}$	OK	

Seismic SCBF LC3 P_1 =-559.7 kips (T) P_2 =157.3 kips (C) ratio = **0.79** **PASS**

Gusset Plate - Shear Yielding ratio = 270.9 / 585.0 = **0.46** **PASS**

Plate Shear Yielding Check

Plate size	width b_p = 26.000 [in]	thickness t_p = 0.750	[in]	
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$	= 19.500	[in^2]	
Shear force required	$V_u =$	= **270.9**	[kips]	
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$	= 585.0	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	$\phi R_n =$	= **585.0**	[kips]	
	ratio = **0.46**	$> V_u$	OK	

Gusset Plate - Shear Rupture ratio = 270.9 / 570.4 = **0.47** **PASS**

Plate Shear Rupture Check

Plate size	width b_p = 26.000 [in]	thickness t_p = 0.750	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate net area in shear	$A_{nv} = b_p t_p$	= 19.500	[in^2]	
Shear force in demand	$V_u =$	= **270.9**	[kips]	
Plate shear rupture strength	$R_n = 0.6 F_u A_{nv}$	= 760.5	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-4
	$\phi R_n =$	= **570.4**	[kips]	
	ratio = **0.47**	$> V_u$	OK	

Gusset Plate - Axial Tensile Yield		ratio = 239.0 / 877.5	= **0.27**		**PASS**
Plate Tensile Yielding Check					
Plate size	width b_p = 26.000 [in]	thickness t_p = 0.750	[in]		
Plate yield strength	F_y = 50.0 [ksi]				
Plate gross area in shear	$A_g = b_p t_p$	= 19.500	[in^2]		
Tensile force required	P_u =	= **239.0**	[kips]		
Plate tensile yielding strength	$R_n = F_y A_g$	= 975.0	[kips]	AISC 15th Eq J4-1	
Resistance factor-LRFD	ϕ = 0.90			AISC 15th Eq J4-1	
	ϕR_n =	= **877.5**	[kips]		
	ratio = **0.27**	> P_u	OK		

Gusset Plate - Axial Tensile Rupture		ratio = 239.0 / 950.6	= **0.25**		**PASS**
Plate Tensile Rupture Check					
Plate size	width b_p = 26.000 [in]	thickness t_p = 0.750	[in]		
Plate tensile strength	F_u = 65.0 [ksi]				
Plate net area in tension	$A_{nt} = b_p t_p$	= 19.500	[in^2]		
Tensile force required	P_u =	= **239.0**	[kips]		
Plate tensile rupture strength	$R_n = F_u A_{nt}$	= 1267.5	[kips]	AISC 15th Eq J4-2	
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-2	
	ϕR_n =	= **950.6**	[kips]	AISC 15th Eq J4-2	
	ratio = **0.25**	> P_u	OK		

Gusset Plate - Flexural Yield Interact		ratio =	= **0.29**		**PASS**
Gusset plate	width b_p = 26.000 [in]	thick t_p = 0.750	[in]		
	yield F_y = 50.0 [ksi]				
Shear plate - gross area	$A_g = b_p \times t_p$	= 19.500	[in^2]		
Shear plate - plastic modulus	$Z_p = (b_p \times t_p^2) / 4$	= 126.75	[in^3]		
Flexural strength available	$M_c = \phi F_y Z_p$ ϕ=0.90	= 475.31	[kip-ft]		
Flexural strength required	M_r = from gusset interface forces calc	= 0.00	[kip-ft]		
Axial strength available	P_c = from axial tensile yield check	= 877.5	[kips]		
Axial strength required	P_r = from gusset interface forces calc	= -239.0	[kips]		
Shear strength available	V_c = from shear yielding check	= 585.0	[kips]		
Shear strength required	V_r = from gusset interface forces calc	= 270.9	[kips]		
Flexural yield interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right)^2$	= **0.29**		AISC 15th Eq 10-5	
		< 1.0	OK		

Gusset Plate - Flexural Rupture Interact		ratio =	= **0.29**	**PASS**
Gusset plate	width b_p = 26.000 [in]	thick t_p = 0.750 [in]		
	tensile F_u = 65.0 [ksi]			
Net area of plate	$A_n = b_p \times t_p$	= 19.500 [in^2]		
Plastic modulus of net section	$Z_{net} = (b_p \times t_p^2) / 4$	= 126.75 [in^3]		
Flexural strength available	$M_c = \phi F_u Z_{net}$ ϕ=0.75	= 514.92 [kip-ft]		
Flexural strength required	M_r = from gusset interface forces calc	= 0.00 [kip-ft]		
Axial strength available	P_c = from axial tensile rupture check	= 950.6 [kips]		
Axial strength required	P_r = from gusset interface forces calc	= -239.0 [kips]		
Shear strength available	V_c = from shear rupture check	= 570.4 [kips]		
Shear strength required	V_r = from gusset interface forces calc	= 270.9 [kips]		
Flexural rupture interaction	ratio $= (\frac{V_r}{V_c})^2 + (\frac{P_r}{P_c} + \frac{M_r}{M_c})^2$	= **0.29**	AISC 15th Eq 10-5	
		< 1.0 OK		

Gusset to Beam Weld Strength		ratio = 13.89 / 17.55	= **0.79**	**PASS**
Gusset to Beam Interface - Forces				
	shear H_b = 270.9 [kips]	axial V_b = -239.0 [kips] in tension		
	moment M_b = 0.00 [kip-ft]			
Gusset-beam fillet weld length	L_w =	= 26.000 [in]		
Gusset to Beam Interface - Combined Weld Stress				
Weld stress from axial force	$f_a = V_b / L_{wb}$	= -9.192 [kip/in] in tension		
Weld stress from shear force	$f_v = H_b / L_{wb}$	= 10.419 [kip/in]		
Weld stress from moment force	$f_b = \frac{M}{L^2 / 6}$	= 0.000 [kip/in]		
Weld stress combined - max	$f_{max} = [(f_a - f_b)^2 + f_v^2]^{0.5}$	= **13.895** [kip/in]	AISC 15th Eq 8-11	
Weld resultant load angle	$\theta = \tan^{-1} [(f_b - f_a) / f_v]$	= 41.4 [°]		
Fillet Weld Strength Calc				
Fillet weld leg size	w = $^7/_{16}$ [in]	load angle θ = 41.4 [°]		
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	$C_2 = (1 + 0.5 \sin^{1.5} \theta)$	= 1.27	AISC 15th Page 8-9	
Fillet weld shear strength	$R_{n\text{-}w} = 0.6 (C_1 \times 70 \text{ ksi}) 0.707 \text{ w n } C_2$	= 32.973 [kip/in]	AISC 15th Eq 8-1	
Base metal - gusset plate	thickness t = 0.750 [in]	tensile F_u = 65.0 [ksi]		
Base metal - gusset 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	$R_{n\text{-}b} = 0.6 F_u t$	= 29.250 [kip/in]	AISC 15th Eq J4-4	
Double fillet linear shear strength	$R_n = \min (R_{n\text{-}w} , R_{n\text{-}b})$	= **29.250** [kip/in]	AISC 15th Eq 9-2	
Resistance factor-LRFD	ϕ = 0.75		AISC 15th Eq 8-1	
	ϕR_n =	= 21.938 [kip/in]		
When gusset plate is directly welded to beam or column, apply 1.25 ductility factor to allow adequate force redistribution in the weld group			AISC 15th Page 13-11	
Weld strength used for design after applying ductility factor	$\phi R_n = \phi R_n \times (1/1.25)$	= **17.550** [kip/in]		
	ratio = **0.79**	> f_{max} OK		

Beam Web Local Yielding		ratio = 239.0 / 974.2	= **0.25**	**PASS**
Concentrated force from gusset	P_u =		= **239.0** [kips]	
Beam section	d = 24.700 [in]		t_f = 1.090 [in]	
	t_w = 0.650 [in]		k = 1.590 [in]	
	yield F_y = 50.0 [ksi]			
Length of bearing	l_b = Gusset/Beam interface length		= 26.000 [in]	
Gusset plate corner clip	clip = from user input		= 1.000 [in]	
Distance from normal force applied point to member end	$l_N = 0.5\ l_b$ + clip		= 14.000 [in]	
	when $l_N \le d$, use AISC 15th Eq J10-3			AISC 15th Eq J10-3
Beam web local yielding strength	$R_n = F_y t_w (2.5\ k + l_b)$		= 974.2 [kips]	AISC 15th Eq J10-3
Resistance factor-LRFD	ϕ = 1.00			
	ϕR_n =		= **974.2** [kips]	
	ratio = **0.25**		> P_u	OK

Seismic SCBF LC4 P_1 =134.7 kips (C) P_2 =-615.9 kips (T) ratio = **0.16** **PASS**

Gusset Plate - Shear Yielding		ratio = 65.2 / 585.0	= **0.11**	**PASS**
Plate Shear Yielding Check				
Plate size	width b_p = 26.000 [in]	thickness t_p = 0.750 [in]		
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$		= 19.500 [in^2]	
Shear force required	V_u =		= **65.2** [kips]	
Plate shear yielding strength	$R_n = 0.6\ F_y A_{gv}$		= 585.0 [kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	ϕR_n =		= **585.0** [kips]	
	ratio = **0.11**		> V_u	OK

Gusset Plate - Shear Rupture		ratio = 65.2 / 570.4	= **0.11**	**PASS**
Plate Shear Rupture Check				
Plate size	width b_p = 26.000 [in]	thickness t_p = 0.750 [in]		
Plate tensile strength	F_u = 65.0 [ksi]			
Plate net area in shear	$A_{nv} = b_p t_p$		= 19.500 [in^2]	
Shear force in demand	V_u =		= **65.2** [kips]	
Plate shear rupture strength	$R_n = 0.6\ F_u A_{nv}$		= 760.5 [kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-4
	ϕR_n =		= **570.4** [kips]	
	ratio = **0.11**		> V_u	OK

Gusset Plate - Axial Yield		ratio = 57.5 / 877.5	= **0.07**		**PASS**
Plate Tensile Yielding Check					
Plate size	width b_p = 26.000 [in]	thickness t_p = 0.750	[in]		
Plate yield strength	F_y = 50.0 [ksi]				
Plate gross area in shear	$A_g = b_p t_p$		= 19.500	[in^2]	
Tensile force required	P_u =		= **57.5**	[kips]	
Plate tensile yielding strength	$R_n = F_y A_g$		= 975.0	[kips]	AISC 15th Eq J4-1
Resistance factor-LRFD	ϕ = 0.90				AISC 15th Eq J4-1
	ϕR_n =		= **877.5**	[kips]	
	ratio = **0.07**		> P_u	OK	

Gusset Plate - Flexural Yield Interact		ratio =	= **0.02**		**PASS**
Gusset plate	width b_p = 26.000 [in]	thick t_p = 0.750	[in]		
	yield F_y = 50.0 [ksi]				
Shear plate - gross area	$A_g = b_p \times t_p$		= 19.500	[in^2]	
Shear plate - plastic modulus	$Z_p = (b_p \times t_p^2) / 4$		= 126.75	[in^3]	
Flexural strength available	$M_c = \phi F_y Z_p$ ϕ=0.90		= 475.31	[kip-ft]	
Flexural strength required	M_r = from gusset interface forces calc		= 0.00	[kip-ft]	
Axial strength available	P_c = from axial tensile yield check		= 877.5	[kips]	
Axial strength required	P_r = from gusset interface forces calc		= 57.5	[kips]	
Shear strength available	V_c = from shear yielding check		= 585.0	[kips]	
Shear strength required	V_r = from gusset interface forces calc		= 65.2	[kips]	
Flexural yield interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right)^2$		= **0.02**		AISC 15th Eq 10-5
			< 1.0	OK	

Gusset Plate - Flexural Rupture Interact		ratio =	= **0.01**		**PASS**
Gusset plate	width b_p = 26.000 [in]	thick t_p = 0.750	[in]		
	tensile F_u = 65.0 [ksi]				
Net area of plate	$A_n = b_p \times t_p$		= 19.500	[in^2]	
Plastic modulus of net section	$Z_{net} = (b_p \times t_p^2) / 4$		= 126.75	[in^3]	
Flexural strength available	$M_c = \phi F_u Z_{net}$ ϕ=0.75		= 514.92	[kip-ft]	
Flexural strength required	M_r = from gusset interface forces calc		= 0.00	[kip-ft]	
Shear strength available	V_c = from shear rupture check		= 570.4	[kips]	
Shear strength required	V_r = from gusset interface forces calc		= 65.2	[kips]	
Flexural rupture interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{M_r}{M_c}\right)^2$		= **0.01**		AISC 15th Eq 10-5
			< 1.0	OK	

Gusset to Beam Weld Strength — ratio = 2.51 / 15.59 = **0.16** PASS

Gusset to Beam Interface - Forces

	shear H_b = 65.2	[kips]	axial V_b = 57.5	[kips]	in compression
	moment M_b = 0.00	[kip-ft]			
Gusset-beam fillet weld length	L_w =		= 26.000	[in]	

Gusset to Beam Interface - Combined Weld Stress

Weld stress from axial force	$f_a = V_b / L_{wb}$	= 0.000	[kip/in]	in compression
Weld stress from shear force	$f_v = H_b / L_{wb}$	= 2.508	[kip/in]	
Weld stress from moment force	$f_b = \dfrac{M}{L^2 / 6}$	= 0.000	[kip/in]	
Weld stress combined - max	$f_{max} = f_v$	= **2.508**	[kip/in]	AISC 15th Eq 8-11
Weld resultant load angle	θ = weld only has shear component	= 0.0	[°]	

Fillet Weld Strength Calc

Fillet weld leg size	w = $^7/_{16}$	[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	$R_{n\text{-}w} = 0.6\,(C_1 \times 70\text{ ksi})\ 0.707\ w\ n\ C_2$		= 25.982	[kip/in]	AISC 15th Eq 8-1	

Base metal - gusset plate	thickness t = 0.750	[in]	tensile F_u = 65.0	[ksi]	
Base metal - gusset 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	$R_{n\text{-}b} = 0.6\ F_u\ t$		= 29.250	[kip/in]	AISC 15th Eq J4-4

Double fillet linear shear strength	$R_n = \min(R_{n\text{-}w}, R_{n\text{-}b})$	= **25.982**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq 8-1
	ϕR_n =	= 19.487	[kip/in]	
When gusset plate is directly welded to beam or column, apply 1.25 ductility factor to allow adequate force redistribution in the weld group				AISC 15th Page 13-11
Weld strength used for design after applying ductility factor	$\phi R_n = \phi R_n \times (1/1.25)$	= **15.589**	[kip/in]	
	ratio = **0.16**	> f_{max}	OK	

Beam Web Local Yielding — ratio = 57.5 / 974.2 = **0.06** PASS

Concentrated force from gusset	P_u =		= **57.5**	[kips]	
Beam section	d = 24.700	[in]	t_f = 1.090	[in]	
	t_w = 0.650	[in]	k = 1.590	[in]	
	yield F_y = 50.0	[ksi]			

Length of bearing	l_b = Gusset/Beam interface length	= 26.000	[in]	
Gusset plate corner clip	clip = from user input	= 1.000	[in]	
Distance from normal force applied point to member end	$l_N = 0.5\ l_b$ + clip	= 14.000	[in]	
	when $l_N \le d$, use AISC 15th Eq J10-3			AISC 15th Eq J10-3
Beam web local yielding strength	$R_n = F_y\, t_w\, (2.5\ k + l_b)$	= 974.2	[kips]	AISC 15th Eq J10-3
Resistance factor-LRFD	ϕ = 1.00			
	ϕR_n =	= **974.2**	[kips]	
	ratio = **0.06**	> P_u	OK	

Beam Web Local Crippling		ratio = 57.5 / 970.1	= **0.06**	**PASS**	
Concentrated force from gusset	P_u =		= **57.5**	[kips]	
Beam section	d = 24.700 [in]		t_f = 1.090	[in]	
	t_w = 0.650 [in]		k = 1.590	[in]	
	yield F_y = 50.0 [ksi]		E = 29000	[ksi]	
Length of bearing	l_b = Gusset/Beam interface length		= 26.000	[in]	
Gusset plate corner clip	clip = from user input		= 1.000	[in]	
Distance from normal force applied point to member end	$l_N = 0.5\ l_b$ + clip		= 14.000	[in]	
	when $l_N \geq d/2$, use Eq J10-4				AISC 15th Eq J10-4
Beam web local crippling strength	$R_n = 0.8\ t_w^2\ [1+3\frac{l_b}{d}(\frac{t_w}{t_f})^{1.5}] \times (\frac{E F_y t_f}{t_w})^{0.5}$		= 1293.5	[kips]	AISC 15th Eq J10-4
Resistance factor-LRFD	ϕ = 0.75				AISC 15th J10.3
	ϕR_n =		= **970.1**	[kips]	
	ratio = **0.06**		> P_u	OK	

Bot Brace - Gusset to Column Direct Weld Connection Code=AISC 360-16 LRFD

Result Summary geometries & weld limitations = **PASS** limit states max ratio = **0.98** **PASS**

Weld Limitation Checks - Gusset to Column **PASS**

Min Fillet Weld Size

Thinner part joined thickness	$t =$	= 0.750	[in]	
Min fillet weld size allowed	$w_{min} =$	= **0.250**	[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	$L =$	= **18.750**	[in]	
		$\geq L_{min}$	OK	

Seismic SCBF LC3 P_1 =-559.7 kips (T) P_2 =157.3 kips (C) ratio = **0.22** **PASS**

Gusset Plate - Shear Yielding ratio = 46.4 / 421.9 = **0.11** **PASS**

Plate Shear Yielding Check

Plate size	width b_p = 18.750 [in]	thickness t_p = 0.750	[in]	
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$	= 14.063	[in^2]	
Shear force required	$V_u =$	= **46.4**	[kips]	
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$	= 421.9	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	$\phi R_n =$	= **421.9**	[kips]	
	ratio = **0.11**	$> V_u$	OK	

Gusset Plate - Shear Rupture ratio = 46.4 / 411.3 = **0.11** **PASS**

Plate Shear Rupture Check

Plate size	width b_p = 18.750 [in]	thickness t_p = 0.750	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate net area in shear	$A_{nv} = b_p t_p$	= 14.063	[in^2]	
Shear force in demand	$V_u =$	= **46.4**	[kips]	
Plate shear rupture strength	$R_n = 0.6 F_u A_{nv}$	= 548.4	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-4
	$\phi R_n =$	= **411.3**	[kips]	
	ratio = **0.11**	$> V_u$	OK	

Gusset Plate - Axial Tensile Yield		ratio = 33.8 / 632.8	= **0.05**	**PASS**
Plate Tensile Yielding Check				
Plate size	width b_p = 18.750 [in]	thickness t_p = 0.750	[in]	
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_g = b_p t_p$	= 14.063	[in^2]	
Tensile force required	P_u =	= **33.8**	[kips]	
Plate tensile yielding strength	$R_n = F_y A_g$	= 703.1	[kips]	AISC 15th Eq J4-1
Resistance factor-LRFD	ϕ = 0.90			AISC 15th Eq J4-1
	ϕR_n =	= **632.8**	[kips]	
	ratio = **0.05**	> P_u	OK	

Gusset Plate - Axial Tensile Rupture		ratio = 33.8 / 685.5	= **0.05**	**PASS**
Plate Tensile Rupture Check				
Plate size	width b_p = 18.750 [in]	thickness t_p = 0.750	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate net area in tension	$A_{nt} = b_p t_p$	= 14.063	[in^2]	
Tensile force required	P_u =	= **33.8**	[kips]	
Plate tensile rupture strength	$R_n = F_u A_{nt}$	= 914.1	[kips]	AISC 15th Eq J4-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-2
	ϕR_n =	= **685.5**	[kips]	AISC 15th Eq J4-2
	ratio = **0.05**	> P_u	OK	

Gusset Plate - Flexural Yield Interact		ratio =	= **0.01**	**PASS**
Gusset plate	width b_p = 18.750 [in]	thick t_p = 0.750	[in]	
	yield F_y = 50.0 [ksi]			
Shear plate - gross area	$A_g = b_p \times t_p$	= 14.063	[in^2]	
Shear plate - plastic modulus	$Z_p = (b_p \times t_p^2) / 4$	= 65.92	[in^3]	
Flexural strength available	$M_c = \phi F_y Z_p$ ϕ=0.90	= 247.19	[kip-ft]	
Flexural strength required	M_r = from gusset interface forces calc	= 4.30	[kip-ft]	
Axial strength available	P_c = from axial tensile yield check	= 632.8	[kips]	
Axial strength required	P_r = from gusset interface forces calc	= 33.8	[kips]	
Shear strength available	V_c = from shear yielding check	= 421.9	[kips]	
Shear strength required	V_r = from gusset interface forces calc	= 46.4	[kips]	
Flexural yield interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right)^2$	= **0.01**		AISC 15th Eq 10-5
		< 1.0	OK	

Gusset Plate - Flexural Rupture Interact		ratio =	= **0.01**	**PASS**
Gusset plate	width b_p = 18.750 [in]	thick t_p = 0.750 [in]		
	tensile F_u = 65.0 [ksi]			
Net area of plate	$A_n = b_p \times t_p$	= 14.063 [in^2]		
Plastic modulus of net section	$Z_{net} = (b_p \times t_p^2) / 4$	= 65.92 [in^3]		
Flexural strength available	$M_c = \phi F_u Z_{net}$ ϕ=0.75	= 267.79 [kip-ft]		
Flexural strength required	M_r = from gusset interface forces calc	= 4.30 [kip-ft]		
Axial strength available	P_c = from axial tensile rupture check	= 685.5 [kips]		
Axial strength required	P_r = from gusset interface forces calc	= 33.8 [kips]		
Shear strength available	V_c = from shear rupture check	= 411.3 [kips]		
Shear strength required	V_r = from gusset interface forces calc	= 46.4 [kips]		
Flexural rupture interaction	ratio = $\left(\frac{V_r}{V_c} \right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c} \right)^2$	= **0.01**		AISC 15^{th} Eq 10-5
		< 1.0	OK	

Gusset to Column Weld Strength		ratio = 2.47 / 11.14	= **0.22**	**PASS**
Gusset to Column Interface - Forces				
	shear V_c = 46.4 [kips]	axial H_c = 33.8 [kips]	in compression	
	moment M_c = 4.30 [kip-ft]			
Gusset to Column Interface - Weld Length				
Gusset-column fillet weld length	L_{wc} = from user input	= **18.750** [in]		
Gusset to Column Interface - Combined Weld Stress				
Weld stress from axial force	$f_a = H_c / L_{wc}$	= 1.803 [kip/in]	in compression	
Weld stress from shear force	$f_v = V_c / L_{wc}$	= 2.475 [kip/in]		
Weld stress from moment force	$f_b = \frac{M}{L^2 / 6}$	= 0.881 [kip/in]		
Weld stress combined - max	$f_{max} = f_v$	= **2.475** [kip/in]		AISC 15^{th} Eq 8-11
Weld resultant load angle	θ = weld only has shear component	= 0.0 [°]		
Fillet Weld Strength Calc				
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 15^{th} Table 8-3
Number of weld line	n = 2 for double fillet			
Load angle coefficient	$C_2 = (1 + 0.5 \sin^{1.5} \theta)$	= 1.00		AISC 15^{th} Page 8-9
Fillet weld shear strength	$R_{n\text{-}w} = 0.6 (C_1 \times 70 \text{ ksi}) \, 0.707 \, w \, n \, C_2$	= 18.559 [kip/in]		AISC 15^{th} Eq 8-1
Base metal - gusset plate	thickness t = 0.750 [in]	tensile F_u = 65.0 [ksi]		
Base metal - gusset plate is in shear, <u>shear</u> rupture as per AISC 15^{th} Eq J4-4 is checked				AISC 15^{th} J2.4
Base metal shear rupture	$R_{n\text{-}b} = 0.6 F_u t$	= 29.250 [kip/in]		AISC 15^{th} Eq J4-4
Double fillet linear shear strength	$R_n = \min (R_{n\text{-}w} , R_{n\text{-}b})$	= **18.559** [kip/in]		AISC 15^{th} Eq 9-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15^{th} Eq 8-1
	ϕR_n =	= 13.919 [kip/in]		
When gusset plate is directly welded to beam or column, apply 1.25 ductility factor to allow adequate force redistribution in the weld group				AISC 15^{th} Page 13-11
Weld strength used for design after applying ductility factor	$\phi R_n = \phi R_n \times (1/1.25)$	= **11.135** [kip/in]		
	ratio = **0.22**	> f_{max}	OK	

Column Web Local Yielding		ratio = 44.8 / 814.4	= **0.06**	**PASS**
Gusset Edge Equivalent Normal Force				
Refer to AISC DG29 Fig. B-1 for formula below to calculate gusset edge equivalent normal force				
Gusset edge axial force	$N =$	= 33.8	[kips]	
Gusset edge moment force	$M =$	= 4.30	[kip-ft]	
Gusset edge interface length	$L =$	= 18.750	[in]	
Gusset edge equivalent normal force	$N_e = N + \frac{4\,M}{L}$	= 44.8	[kips]	AISC DG29 Fig B-1
Concentrated force from gusset	$P_u =$	= **44.8**	[kips]	
Column section	d = 12.900 [in]	t_f = 0.990	[in]	
	t_w = 0.610 [in]	k = 1.590	[in]	
	yield F_y = 50.0 [ksi]			
Length of bearing	l_b = Gusset/Column interface length	= 18.750	[in]	
Column web local yielding strength	$R_n = F_y t_w (5\,k + l_b)$	= 814.4	[kips]	AISC 15th Eq J10-2
Resistance factor-LRFD	ϕ = 1.00			
	$\phi\,R_n =$	= **814.4**	[kips]	
	ratio = **0.06**	> P_u	OK	

Column Web Local Crippling		ratio = 44.8 / 1064.8	= **0.04**	**PASS**
Gusset Edge Equivalent Normal Force				
Refer to AISC DG29 Fig. B-1 for formula below to calculate gusset edge equivalent normal force				
Gusset edge axial force	$N =$	= 33.8	[kips]	
Gusset edge moment force	$M =$	= 4.30	[kip-ft]	
Gusset edge interface length	$L =$	= 18.750	[in]	
Gusset edge equivalent normal force	$N_e = N + \frac{4\,M}{L}$	= 44.8	[kips]	AISC DG29 Fig B-1
Concentrated force from gusset	$P_u =$	= **44.8**	[kips]	
Column section	d = 12.900 [in]	t_f = 0.990	[in]	
	t_w = 0.610 [in]	k = 1.590	[in]	
	yield F_y = 50.0 [ksi]	E = 29000	[ksi]	
Length of bearing	l_b = Gusset/Column interface length	= 18.750	[in]	
	when $l_N \geq d/2$, use Eq J10-4			AISC 15th Eq J10-4
Column web local crippling strength	$R_n = 0.8\, t_w^2\ [1+3\frac{l_b}{d}(\frac{t_w}{t_f})^{1.5}] \times (\frac{E\,F_y t_f}{t_w})^{0.5}$	= 1419.7	[kips]	AISC 15th Eq J10-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th J10.3
	$\phi R_n =$	= **1064.8**	[kips]	
	ratio = **0.04**	> P_u	OK	

Column Web Shear Strength		ratio = 33.8 / 236.1	= **0.14**	**PASS**

W Shape Column Shear Strength Check

W sect W12X106	d = 12.900 [in]	t_w = 0.610 [in]		
	F_y = 50.0 [ksi]			
Gusset to column axial force	V_u = from gusset interface force calc	= **33.8**	[kips]	
Column shear strength	$R_n = 0.6\ F_y\ d\ t_w$	= 236.1	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	$\phi\ R_n$ =	= **236.1**	[kips]	
	ratio = **0.14**	> V_u	OK	

Seismic SCBF LC4 P_1 =134.7 kips (C) P_2 =-615.9 kips (T) ratio = **0.98** **PASS**

Gusset Plate - Shear Yielding		ratio = 181.8 / 421.9	= **0.43**	**PASS**

Plate Shear Yielding Check

Plate size	width b_p = 18.750 [in]	thickness t_p = 0.750 [in]		
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p\ t_p$	= 14.063	[in^2]	
Shear force required	V_u =	= **181.8**	[kips]	
Plate shear yielding strength	$R_n = 0.6\ F_y\ A_{gv}$	= 421.9	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	$\phi\ R_n$ =	= **421.9**	[kips]	
	ratio = **0.43**	> V_u	OK	

Gusset Plate - Shear Rupture		ratio = 181.8 / 411.3	= **0.44**	**PASS**

Plate Shear Rupture Check

Plate size	width b_p = 18.750 [in]	thickness t_p = 0.750 [in]		
Plate tensile strength	F_u = 65.0 [ksi]			
Plate net area in shear	$A_{nv} = b_p\ t_p$	= 14.063	[in^2]	
Shear force in demand	V_u =	= **181.8**	[kips]	
Plate shear rupture strength	$R_n = 0.6\ F_u\ A_{nv}$	= 548.4	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-4
	$\phi\ R_n$ =	= **411.3**	[kips]	
	ratio = **0.44**	> V_u	OK	

Gusset Plate - Axial Yield		ratio = 132.5 / 632.8	= **0.21**	**PASS**

Plate Tensile Yielding Check

Plate size	width b_p = 18.750 [in]	thickness t_p = 0.750 [in]		
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_g = b_p\ t_p$	= 14.063	[in^2]	
Tensile force required	P_u =	= **132.5**	[kips]	
Plate tensile yielding strength	$R_n = F_y\ A_g$	= 703.1	[kips]	AISC 15th Eq J4-1
Resistance factor-LRFD	ϕ = 0.90			AISC 15th Eq J4-1
	$\phi\ R_n$ =	= **632.8**	[kips]	
	ratio = **0.21**	> P_u	OK	

Gusset Plate - Flexural Yield Interact		ratio =	**= 0.26**	**PASS**
Gusset plate	width b_p = 18.750 [in]	thick t_p = 0.750	[in]	
	yield F_y = 50.0 [ksi]			
Shear plate - gross area	$A_g = b_p \times t_p$	= 14.063	[in^2]	
Shear plate - plastic modulus	$Z_p = (b_p \times t_p^2) / 4$	= 65.92	[in^3]	
Flexural strength available	$M_c = \phi F_y Z_p$ ϕ=0.90	= 247.19	[kip-ft]	
Flexural strength required	M_r = from gusset interface forces calc	= 16.84	[kip-ft]	
Axial strength available	P_c = from axial tensile yield check	= 632.8	[kips]	
Axial strength required	P_r = from gusset interface forces calc	= -132.5	[kips]	
Shear strength available	V_c = from shear yielding check	= 421.9	[kips]	
Shear strength required	V_r = from gusset interface forces calc	= 181.8	[kips]	
Flexural yield interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right)^2$	= **0.26**		AISC 15th Eq 10-5
		< 1.0	OK	

Gusset Plate - Flexural Rupture Interact		ratio =	**= 0.20**	**PASS**
Gusset plate	width b_p = 18.750 [in]	thick t_p = 0.750	[in]	
	tensile F_u = 65.0 [ksi]			
Net area of plate	$A_n = b_p \times t_p$	= 14.063	[in^2]	
Plastic modulus of net section	$Z_{net} = (b_p \times t_p^2) / 4$	= 65.92	[in^3]	
Flexural strength available	$M_c = \phi F_u Z_{net}$ ϕ=0.75	= 267.79	[kip-ft]	
Flexural strength required	M_r = from gusset interface forces calc	= 16.84	[kip-ft]	
Shear strength available	V_c = from shear rupture check	= 411.3	[kips]	
Shear strength required	V_r = from gusset interface forces calc	= 181.8	[kips]	
Flexural rupture interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{M_r}{M_c}\right)^2$	= **0.20**		AISC 15th Eq 10-5
		< 1.0	OK	

Gusset to Column Weld Strength		ratio = 14.30 / 14.64	= **0.98**	**PASS**

Gusset to Column Interface - Forces

	shear V_c = 181.8 [kips]	axial H_c = -132.5	[kips]	in tension
	moment M_c = 16.84 [kip-ft]			

Gusset to Column Interface - Weld Length

Gusset-column fillet weld length	L_{wc} = from user input	= **18.750**	[in]	

Gusset to Column Interface - Combined Weld Stress

Weld stress from axial force	$f_a = H_c / L_{wc}$	= -7.067	[kip/in]	in tension
Weld stress from shear force	$f_v = V_c / L_{wc}$	= 9.696	[kip/in]	
Weld stress from moment force	$f_b = \dfrac{M}{L^2 / 6}$	= 3.449	[kip/in]	
Weld stress combined - max	$f_{max} = [\ (f_a - f_b)^2 + f_v^2\ ]^{0.5}$	= **14.303**	[kip/in]	AISC 15th Eq 8-11
Weld resultant load angle	$\theta = \tan^{-1} [(f_b - f_a) / f_v]$	= 47.3	[°]	

Fillet Weld Strength Calc

Fillet weld leg size	$w = 5/16$ [in]	load angle θ = 47.3	[°]	
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	$C_2 = (1 + 0.5 \sin^{1.5} \theta)$	= 1.32		AISC 15th Page 8-9
Fillet weld shear strength	$R_{n-w} = 0.6\ (C_1 \times 70\ \text{ksi})\ 0.707\ w\ n\ C_2$	= 24.408	[kip/in]	AISC 15th Eq 8-1

Base metal - gusset plate	thickness t = 0.750 [in]	tensile F_u = 65.0	[ksi]	

Base metal - gusset plate is in shear, shear rupture as per AISC 15th Eq J4-4 is checked — AISC 15th J2.4

Base metal shear rupture	$R_{n-b} = 0.6\ F_u\ t$	= 29.250	[kip/in]	AISC 15th Eq J4-4

Double fillet linear shear strength	$R_n = \min (R_{n-w} , R_{n-b})$	= **24.408**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq 8-1
	$\phi\ R_n$ =	= 18.306	[kip/in]	

When gusset plate is directly welded to beam or column, apply 1.25 ductility factor to allow adequate force redistribution in the weld group — AISC 15th Page 13-11

Weld strength used for design after applying ductility factor	$\phi\ R_n = \phi\ R_n \times (1/1.25)$	= **14.645**	[kip/in]	
	ratio = **0.98**	> f_{max}	OK	

Column Web Local Yielding		ratio = 175.6 / 814.4	= **0.22**	**PASS**
Gusset Edge Equivalent Normal Force				
Refer to AISC DG29 Fig. B-1 for formula below to calculate gusset edge equivalent normal force				
Gusset edge axial force	$N =$	= -132.5	[kips]	
Gusset edge moment force	$M =$	= 16.84	[kip-ft]	
Gusset edge interface length	$L =$	= 18.750	[in]	
Gusset edge equivalent normal force	$N_e = N - \frac{4\,M}{L}$	= -175.6	[kips]	AISC DG29 Fig B-1
Concentrated force from gusset	$P_u =$	= **175.6**	[kips]	
Column section	d = 12.900 [in]	t_f = 0.990	[in]	
	t_w = 0.610 [in]	k = 1.590	[in]	
	yield F_y = 50.0 [ksi]			
Length of bearing	l_b = Gusset/Column interface length	= 18.750	[in]	
Column web local yielding strength	$R_n = F_y\, t_w\, (\, 5\,k + l_b\,)$	= 814.4	[kips]	AISC 15th Eq J10-2
Resistance factor-LRFD	ϕ = 1.00			
	$\phi\, R_n$ =	= **814.4**	[kips]	
	ratio = **0.22**	> P_u	OK	

Column Web Shear Strength		ratio = 132.5 / 236.1	= **0.56**	**PASS**
W Shape Column Shear Strength Check				
W sect W12X106	d = 12.900 [in]	t_w = 0.610	[in]	
	F_y = 50.0 [ksi]			
Gusset to column axial force	V_u = from gusset interface force calc	= **132.5**	[kips]	
Column shear strength	$R_n = 0.6\, F_y\, d\, t_w$	= 236.1	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	$\phi\, R_n$ =	= **236.1**	[kips]	
	ratio = **0.56**	> V_u	OK	

Bot Brace - Gusset to Beam Direct Weld Connection Code=AISC 360-16 LRFD

Result Summary	geometries & weld limitations = **PASS**	limit states max ratio = **0.82**	**PASS**

Brace Weld Limitation Checks - Gusset to Beam **PASS**

Min Fillet Weld Size

Thinner part joined thickness	$t =$	= 0.750	[in]	
Min fillet weld size allowed	$w_{min} =$	= **0.250**	[in]	AISC 15th Table J2.4
Fillet weld size provided	$w =$	= **0.438**	[in]	
		$\geq w_{min}$	OK	

Min Fillet Weld Length

Fillet weld size provided	$w =$	= 0.438	[in]	
Min fillet weld length allowed	$L_{min} = 4 \times w$	= **1.750**	[in]	AISC 15th J2.2b
Min fillet weld length	$L =$	= **27.500**	[in]	
		$\geq L_{min}$	OK	

Seismic SCBF LC3 P_1 =-559.7 kips (T) P_2 =157.3 kips (C) ratio = **0.18** **PASS**

Gusset Plate - Shear Yielding ratio = 77.4 / 618.8 = **0.13** **PASS**

Plate Shear Yielding Check

Plate size	width b_p = 27.500 [in]	thickness t_p = 0.750	[in]		
Plate yield strength	F_y = 50.0 [ksi]				
Plate gross area in shear	$A_{gv} = b_p t_p$	= 20.625	[in^2]		
Shear force required	$V_u =$	= **77.4**	[kips]		
Plate shear yielding strength	$R_n = 0.6\ F_y A_{gv}$	= 618.8	[kips]	AISC 15th Eq J4-3	
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3	
	$\phi R_n =$	= **618.8**	[kips]		
	ratio = **0.13**	$> V_u$	OK		

Gusset Plate - Shear Rupture ratio = 77.4 / 603.3 = **0.13** **PASS**

Plate Shear Rupture Check

Plate size	width b_p = 27.500 [in]	thickness t_p = 0.750	[in]		
Plate tensile strength	F_u = 65.0 [ksi]				
Plate net area in shear	$A_{nv} = b_p t_p$	= 20.625	[in^2]		
Shear force in demand	$V_u =$	= **77.4**	[kips]		
Plate shear rupture strength	$R_n = 0.6\ F_u A_{nv}$	= 804.4	[kips]	AISC 15th Eq J4-4	
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-4	
	$\phi R_n =$	= **603.3**	[kips]		
	ratio = **0.13**	$> V_u$	OK		

Gusset Plate - Axial Tensile Yield		ratio = 64.8 / 928.1	= **0.07**	**PASS**
Plate Tensile Yielding Check				
Plate size	width b_p = 27.500 [in]	thickness t_p = 0.750	[in]	
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_g = b_p t_p$	= 20.625	[in^2]	
Tensile force required	P_u =	= **64.8**	[kips]	
Plate tensile yielding strength	$R_n = F_y A_g$	= 1031.3	[kips]	AISC 15th Eq J4-1
Resistance factor-LRFD	ϕ = 0.90			AISC 15th Eq J4-1
	ϕR_n =	= **928.1**	[kips]	
	ratio = **0.07**	> P_u	OK	

Gusset Plate - Flexural Yield Interact		ratio =	= **0.02**	**PASS**
Gusset plate	width b_p = 27.500 [in]	thick t_p = 0.750	[in]	
	yield F_y = 50.0 [ksi]			
Shear plate - gross area	$A_g = b_p \times t_p$	= 20.625	[in^2]	
Shear plate - plastic modulus	$Z_p = (b_p \times t_p^2) / 4$	= 141.80	[in^3]	
Flexural strength available	$M_c = \phi F_y Z_p$ ϕ=0.90	= 531.74	[kip-ft]	
Flexural strength required	M_r = from gusset interface forces calc	= 0.00	[kip-ft]	
Axial strength available	P_c = from axial tensile yield check	= 928.1	[kips]	
Axial strength required	P_r = from gusset interface forces calc	= 64.8	[kips]	
Shear strength available	V_c = from shear yielding check	= 618.8	[kips]	
Shear strength required	V_r = from gusset interface forces calc	= 77.4	[kips]	
Flexural yield interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right)^2$	= **0.02**		AISC 15th Eq 10-5
		< 1.0	OK	

Gusset Plate - Flexural Rupture Interact		ratio =	= **0.02**	**PASS**
Gusset plate	width b_p = 27.500 [in]	thick t_p = 0.750	[in]	
	tensile F_u = 65.0 [ksi]			
Net area of plate	$A_n = b_p \times t_p$	= 20.625	[in^2]	
Plastic modulus of net section	$Z_{net} = (b_p \times t_p^2) / 4$	= 141.80	[in^3]	
Flexural strength available	$M_c = \phi F_u Z_{net}$ ϕ=0.75	= 576.05	[kip-ft]	
Flexural strength required	M_r = from gusset interface forces calc	= 0.00	[kip-ft]	
Shear strength available	V_c = from shear rupture check	= 603.3	[kips]	
Shear strength required	V_r = from gusset interface forces calc	= 77.4	[kips]	
Flexural rupture interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{M_r}{M_c}\right)^2$	= **0.02**		AISC 15th Eq 10-5
		< 1.0	OK	

Gusset to Beam Weld Strength		ratio = 2.81 / 15.59	= **0.18**	**PASS**

Gusset to Beam Interface - Forces

	shear H_b = 77.4	[kips]	axial V_b = 64.8	[kips]	in compression
	moment M_b = 0.00	[kip-ft]			
Gusset-beam fillet weld length	L_w =		= 27.500	[in]	

Gusset to Beam Interface - Combined Weld Stress

Weld stress from axial force	$f_a = V_b / L_{wb}$	= 0.000	[kip/in]	in compression
Weld stress from shear force	$f_v = H_b / L_{wb}$	= 2.815	[kip/in]	
Weld stress from moment force	$f_b = \dfrac{M}{L^2 / 6}$	= 0.000	[kip/in]	
Weld stress combined - max	$f_{max} = f_v$	= **2.815**	[kip/in]	AISC 15th Eq 8-11
Weld resultant load angle	θ = weld only has shear component	= 0.0	[°]	

Fillet Weld Strength Calc

Fillet weld leg size	w = 7/16	[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	$R_{n-w} = 0.6 (C_1 \times 70 \text{ ksi}) 0.707 \text{ w n } C_2$		= 25.982	[kip/in]	AISC 15th Eq 8-1

Base metal - gusset plate	thickness t = 0.750	[in]	tensile F_u = 65.0	[ksi]	

Base metal - gusset plate is in shear, <u>shear</u> rupture as per AISC 15th Eq J4-4 is checked — AISC 15th J2.4

Base metal shear rupture	$R_{n-b} = 0.6 F_u t$	= 29.250	[kip/in]	AISC 15th Eq J4-4
Double fillet linear shear strength	$R_n = \min (R_{n-w} , R_{n-b})$	= **25.982**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq 8-1
	ϕ R_n =	= 19.487	[kip/in]	

When gusset plate is directly welded to beam or column, apply 1.25 ductility factor to allow adequate force redistribution in the weld group — AISC 15th Page 13-11

Weld strength used for design after applying ductility factor	$\phi R_n = \phi R_n \times (1/1.25)$	= **15.589**	[kip/in]	
	ratio = **0.18**	> f_{max}	OK	

Beam Web Local Yielding		ratio = 64.8 / 1022.9	= **0.06**	**PASS**

Concentrated force from gusset	P_u =		= **64.8**	[kips]	
Beam section	d = 24.700	[in]	t_f = 1.090	[in]	
	t_w = 0.650	[in]	k = 1.590	[in]	
	yield F_y = 50.0	[ksi]			

Length of bearing	l_b = Gusset/Beam interface length	= 27.500	[in]	
Gusset plate corner clip	clip = from user input	= 1.000	[in]	
Distance from normal force applied point to member end	$l_N = 0.5\, l_b$ + clip	= 14.750	[in]	
	when $l_N \le d$, use AISC 15th Eq J10-3			AISC 15th Eq J10-3
Beam web local yielding strength	$R_n = F_y t_w (2.5 k + l_b)$	= 1022.9	[kips]	AISC 15th Eq J10-3
Resistance factor-LRFD	ϕ = 1.00			
	ϕ R_n =	= **1022.9**	[kips]	
	ratio = **0.06**	> P_u	OK	

Beam Web Local Crippling		ratio = 64.8 / 1003.3	= **0.06**	**PASS**
Concentrated force from gusset	P_u =	= **64.8**	[kips]	
Beam section	d = 24.700 [in]	t_f = 1.090	[in]	
	t_w = 0.650 [in]	k = 1.590	[in]	
	yield F_y = 50.0 [ksi]	E = 29000	[ksi]	
Length of bearing	l_b = Gusset/Beam interface length	= 27.500	[in]	
Gusset plate corner clip	clip = from user input	= 1.000	[in]	
Distance from normal force applied point to member end	$l_N = 0.5\ l_b$ + clip	= 14.750	[in]	
	when $l_N \geq d/2$, use Eq J10-4			AISC 15th Eq J10-4
Beam web local crippling strength	$R_n = 0.8\ t_w^2\ [1+3\frac{l_b}{d}(\frac{t_w}{t_f})^{1.5}]\times(\frac{E\ F_y t_f}{t_w})^{0.5}$	= 1337.7	[kips]	AISC 15th Eq J10-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th J10.3
	ϕR_n =	= **1003.3**	[kips]	
	ratio = **0.06**	> P_u	OK	

Seismic SCBF LC4 P_1 =134.7 kips (C) P_2 =-615.9 kips (T) ratio = **0.82** **PASS**

Gusset Plate - Shear Yielding		ratio = 303.0 / 618.8	= **0.49**	**PASS**
Plate Shear Yielding Check				
Plate size	width b_p = 27.500 [in]	thickness t_p = 0.750	[in]	
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$	= 20.625	[in^2]	
Shear force required	V_u =	= **303.0**	[kips]	
Plate shear yielding strength	$R_n = 0.6\ F_y A_{gv}$	= 618.8	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	$\phi\ R_n$ =	= **618.8**	[kips]	
	ratio = **0.49**	> V_u	OK	

Gusset Plate - Shear Rupture		ratio = 303.0 / 603.3	= **0.50**	**PASS**
Plate Shear Rupture Check				
Plate size	width b_p = 27.500 [in]	thickness t_p = 0.750	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate net area in shear	$A_{nv} = b_p t_p$	= 20.625	[in^2]	
Shear force in demand	V_u =	= **303.0**	[kips]	
Plate shear rupture strength	$R_n = 0.6\ F_u A_{nv}$	= 804.4	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-4
	$\phi\ R_n$ =	= **603.3**	[kips]	
	ratio = **0.50**	> V_u	OK	

Gusset Plate - Axial Yield		ratio = 253.7 / 928.1	= **0.27**	**PASS**
Plate Tensile Yielding Check				
Plate size	width b_p = 27.500 [in]	thickness t_p = 0.750	[in]	
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_g = b_p t_p$	= 20.625	[in^2]	
Tensile force required	P_u =	= **253.7**	[kips]	
Plate tensile yielding strength	$R_n = F_y A_g$	= 1031.3	[kips]	AISC 15th Eq J4-1
Resistance factor-LRFD	ϕ = 0.90			AISC 15th Eq J4-1
	ϕR_n =	= **928.1**	[kips]	
	ratio = **0.27**	> P_u	OK	

Gusset Plate - Axial Tensile Rupture		ratio = 253.7 / 1005.5	= **0.25**	**PASS**
Plate Tensile Rupture Check				
Plate size	width b_p = 27.500 [in]	thickness t_p = 0.750	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate net area in tension	$A_{nt} = b_p t_p$	= 20.625	[in^2]	
Tensile force required	P_u =	= **253.7**	[kips]	
Plate tensile rupture strength	$R_n = F_u A_{nt}$	= 1340.6	[kips]	AISC 15th Eq J4-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-2
	ϕR_n =	= **1005.5**	[kips]	AISC 15th Eq J4-2
	ratio = **0.25**	> P_u	OK	

Gusset Plate - Flexural Yield Interact		ratio =	= **0.31**	**PASS**
Gusset plate	width b_p = 27.500 [in]	thick t_p = 0.750	[in]	
	yield F_y = 50.0 [ksi]			
Shear plate - gross area	$A_g = b_p \times t_p$	= 20.625	[in^2]	
Shear plate - plastic modulus	$Z_p = (b_p \times t_p^2) / 4$	= 141.80	[in^3]	
Flexural strength available	$M_c = \phi F_y Z_p$ ϕ=0.90	= 531.74	[kip-ft]	
Flexural strength required	M_r = from gusset interface forces calc	= 0.00	[kip-ft]	
Axial strength available	P_c = from axial tensile yield check	= 928.1	[kips]	
Axial strength required	P_r = from gusset interface forces calc	= -253.7	[kips]	
Shear strength available	V_c = from shear yielding check	= 618.8	[kips]	
Shear strength required	V_r = from gusset interface forces calc	= 303.0	[kips]	
Flexural yield interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right)^2$	= **0.31**		AISC 15th Eq 10-5
		< 1.0	OK	

Gusset Plate - Flexural Rupture Interact ratio = = **0.32** **PASS**

Gusset plate	width b_p = 27.500 [in]	thick t_p = 0.750 [in]		
	tensile F_u = 65.0 [ksi]			
Net area of plate	$A_n = b_p \times t_p$	= 20.625	[in^2]	
Plastic modulus of net section	$Z_{net} = (b_p \times t_p^2) / 4$	= 141.80	[in^3]	
Flexural strength available	$M_c = \phi F_u Z_{net}$ ϕ=0.75	= 576.05	[kip-ft]	
Flexural strength required	M_r = from gusset interface forces calc	= 0.00	[kip-ft]	
Axial strength available	P_c = from axial tensile rupture check	= 1005.5	[kips]	
Axial strength required	P_r = from gusset interface forces calc	= -253.7	[kips]	
Shear strength available	V_c = from shear rupture check	= 603.3	[kips]	
Shear strength required	V_r = from gusset interface forces calc	= 303.0	[kips]	
Flexural rupture interaction	$\text{ratio} = \left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right)^2$	= **0.32**		AISC 15th Eq 10-5
		< 1.0	OK	

Gusset to Beam Weld Strength ratio = 14.37 / 17.55 = **0.82** **PASS**

Gusset to Beam Interface - Forces

	shear H_b = 303.0 [kips]	axial V_b = -253.7 [kips]	in tension	
	moment M_b = 0.00 [kip-ft]			
Gusset-beam fillet weld length	L_w =	= 27.500	[in]	

Gusset to Beam Interface - Combined Weld Stress

Weld stress from axial force	$f_a = V_b / L_{wb}$	= -9.225	[kip/in]	in tension
Weld stress from shear force	$f_v = H_b / L_{wb}$	= 11.018	[kip/in]	
Weld stress from moment force	$f_b = \frac{M}{L^2 / 6}$	= 0.000	[kip/in]	
Weld stress combined - max	$f_{max} = [(f_a - f_b)^2 + f_v^2]^{0.5}$	= **14.370**	[kip/in]	AISC 15th Eq 8-11
Weld resultant load angle	$\theta = \tan^{-1} [(f_b - f_a) / f_v]$	= 39.9	[°]	

Fillet Weld Strength Calc

Fillet weld leg size	$w = 7/16$ [in]	load angle θ = 39.9	[°]	
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	$C_2 = (1 + 0.5 \sin^{1.5} \theta)$	= 1.26		AISC 15th Page 8-9
Fillet weld shear strength	$R_{n\text{-}w} = 0.6 (C_1 \times 70 \text{ ksi}) \, 0.707 \, w \, n \, C_2$	= 32.665	[kip/in]	AISC 15th Eq 8-1
Base metal - gusset plate	thickness t = 0.750 [in]	tensile F_u = 65.0	[ksi]	

Base metal - gusset plate is in shear, shear rupture as per AISC 15th Eq J4-4 is checked   AISC 15th J2.4

Base metal shear rupture	$R_{n\text{-}b} = 0.6 F_u t$	= 29.250	[kip/in]	AISC 15th Eq J4-4
Double fillet linear shear strength	$R_n = \min (R_{n\text{-}w} , R_{n\text{-}b})$	= **29.250**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	$\phi = 0.75$			AISC 15th Eq 8-1
	ϕR_n =	= 21.938	[kip/in]	

When gusset plate is directly welded to beam or column, apply 1.25 ductility factor to allow adequate force redistribution in the weld group   AISC 15th Page 13-11

Weld strength used for design after applying ductility factor	$\phi R_n = \phi R_n \times (1/1.25)$	= **17.550**	[kip/in]	
	ratio = **0.82**	> f_{max}	OK	

Beam Web Local Yielding		ratio = 253.7 / 1022.9	= **0.25**	**PASS**
Concentrated force from gusset	P_u =	= **253.7**	[kips]	
Beam section	d = 24.700 [in]	t_f = 1.090	[in]	
	t_w = 0.650 [in]	k = 1.590	[in]	
	yield F_y = 50.0 [ksi]			
Length of bearing	l_b = Gusset/Beam interface length	= 27.500	[in]	
Gusset plate corner clip	clip = from user input	= 1.000	[in]	
Distance from normal force applied point to member end	$l_N = 0.5\ l_b$ + clip	= 14.750	[in]	
	when $l_N \le d$, use AISC 15th Eq J10-3			AISC 15th Eq J10-3
Beam web local yielding strength	$R_n = F_y t_w (2.5\ k + l_b)$	= 1022.9	[kips]	AISC 15th Eq J10-3
Resistance factor-LRFD	ϕ = 1.00			
	$\phi\ R_n$ =	= **1022.9**	[kips]	
	ratio = **0.25**	> P_u	OK	

Beam to Column Direct Weld Connection Code=AISC 360-16 LRFD

Result Summary geometries & weld limitations = **PASS** limit states max ratio = **0.69** **PASS**

Beam Flange Fillet Weld Limitation **PASS**

Min Fillet Weld Size

Thinner part joined thickness	t =	= 0.990	[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	$L = 0.5\, b_{fb} - k_{1b}$	= **4.975**	[in]	
		≥ L_{min}	OK	

Beam Web Fillet Weld Limitation **PASS**

Min Fillet Weld Size

Thinner part joined thickness	t =	= 0.650	[in]	
Min fillet weld size allowed	w_{min} =	= **0.250**	[in]	AISC 15th Table J2.4
Fillet weld size provided	w =	= **0.563**	[in]	
		≥ w_{min}	OK	

Min Fillet Weld Length

Fillet weld size provided	w =	= 0.563	[in]	
Min fillet weld length allowed	$L_{min} = 4 \times w$	= **2.250**	[in]	AISC 15th J2.2b
Min fillet weld length	$L = d_b - 2\, k_b$	= **20.700**	[in]	
		≥ L_{min}	OK	

Seismic SCBF LC3 shear V = 283.9 kips axial P = 118.6 kips (C) ratio = **0.59** **PASS**

Beam to Column - Beam Shear ratio = 283.9 / 481.7 = **0.59** **PASS**

W Shape Beam Shear Yielding Check

W sect W24X146	d = 24.700 [in]	t_w = 0.650	[in]	
	F_y = 50.0 [ksi]			
Beam to column shear	V_u = from gusset interface force calc	= **283.9**	[kips]	
Beam shear strength	$R_n = 0.6\, F_y\, d\, t_w$	= 481.7	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	$\phi\, R_n$ =	= **481.7**	[kips]	
	ratio = **0.59**	> V_u	OK	

Beam to Column - Beam Stub Compressive Strength ratio = 118.6 / 1935.0 = **0.06** **PASS**

W Shape Compression Check

W sect W24X146	A = 43.000 [in^2]	r_y = 3.022 [in]		
	F_y = 50.0 [ksi]			
W sect effective length factor	K =	= 1.00		
W sect unbraced length	L =	= 30.000	[in]	
W sect slenderness	$KL/r = 1.00 \times L\ /\ r_y$	= 9.93		
Compression force on W sect	P_u =	= **118.6**	[kips]	
	when $\frac{KL}{r} \le 25$			AISC 15th J4.4 (a)
W sect compression provided	$R_n = F_y \times A$	= 2150.0	[kips]	AISC 15th Eq J4-6
Resistance factor-LRFD	ϕ = 0.90			AISC 15th J4.4 (a)
	$\phi\ R_n$ =	= **1935.0**	[kips]	
	ratio = **0.06**	> P_u	OK	

Beam W Shape Axial Force Distribution

W Shape Beam Axial Force Distribution

W sect W24X146	d = 24.700 [in]	b_f = 12.900	[in]
	t_f = 1.090 [in]	t_w = 0.650	[in]
	A = 43.000 [in^2]		
W shape one side flange area	$A_f = b_f t_f$	= 14.061	[in^2]
W shape web area	$A_w = A - 2\ A_f$	= 14.878	[in^2]
W shape axial force	P =	= 118.6	[kips]
Axial force on flange	$P_f = \frac{A_f}{A} P$	= 38.8	[kips]
Axial force on web	$P_w = \frac{A_w}{A} P$	= 41.0	[kips]

Beam to Column - W Shape Flange Weld ratio = NA = **0.00** **PASS**

Beam flange tensile force	P_u = from beam axial force distribution calc	= **38.8**	[kips]

Beam flange force is in compression, the flange weld check is not needed

Beam to Column - W Shape Web Weld		ratio = 13.71 / 25.05	= **0.55**	**PASS**
Beam section W24X146	d_b = 24.700 [in]	k_b = 2.000	[in]	
Fillet weld length on beam web	$L = d_b - 2\ k_b$	= 20.700	[in]	
Weld Group Forces				
Weld group forces	shear V = 283.9 [kips]	axial P = 41.0	[kips]	in compression
Beam web fillet weld size	w =	= 0.563	[in]	
Combined Weld Stress				
Weld stress from axial force	$f_a = P / L$	= 0.000	[kip/in]	in compression
Weld stress from shear force	$f_v = V / L$	= 13.715	[kip/in]	
Weld stress combined - max	$f_{max} = f_v$	= **13.715**	[kip/in]	AISC 15th Eq 8-11
Weld stress load angle	θ =	= 0.0	[°]	
Fillet Weld Strength Calc				
Fillet weld leg size	w = $^{9}/_{16}$ [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	$R_{n\text{-}w} = 0.6\ (C_1 \times 70\text{ ksi})\ 0.707\ w\ n\ C_2$	= 33.406	[kip/in]	AISC 15th Eq 8-1
Base metal - beam web	thickness t = 0.650 [in]	tensile F_u = 65.0	[ksi]	
Base metal - beam web is in tension, <u>tensile</u> rupture as per AISC 15th Eq J4-2 is checked				AISC 15th J2.4
Base metal tensile rupture	$R_{n\text{-}b} = F_u\ t$	= 42.250	[kip/in]	AISC 15th Eq J4-2
Double fillet linear shear strength	$R_n = \min(R_{n\text{-}w}, R_{n\text{-}b})$	= **33.406**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq 8-1
	ϕR_n =	= **25.054**	[kip/in]	
	ratio = **0.55**	> f_{max}	OK	

Column Web Local Yielding - Beam Flange		ratio = 38.8 / 275.7	= **0.14**	**PASS**
Column Web Local Yielding at Beam Flange Location				
Column sect W12X106	k_c = 1.590 [in]	t_{wc} = 0.610	[in]	
	F_{yc} = 50.0 [ksi]			
Beam section W24X146	t_{fb} = 1.090 [in]			
Beam flange force	P_u = from beam axial force distribution calc	= **38.8**	[kips]	
Column web local yielding strength	$R_n = F_{yc}\ t_{wc}\ (5\ k_c + t_{fb})$	= 275.7	[kips]	AISC 15th Eq 4-2
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq 4-2
	ϕR_n =	= **275.7**	[kips]	
	ratio = **0.14**	> P_u	OK	

Column Web Local Yielding - Beam Web ratio = 41.0 / 631.4 = **0.06** **PASS**

Column Web Local Yielding at Beam Web Location

Column sect W12X106	k_c = 1.590 [in]	t_{wc} = 0.610	[in]	
	F_{yc} = 50.0 [ksi]			
Beam section W24X146	T_b = 20.700 [in]			
Beam axial force on beam web	P_u = from beam axial force distribution calc	= **41.0**	[kips]	
Column web local yielding strength	$R_n = F_{yc}\, t_{wc}\, T_b$	= 631.4	[kips]	AISC 15th Eq J10-2
Resistance factor-LRFD	ϕ = 1.00			AISC 15th J10.2
	ϕR_n =	= **631.4**	[kips]	
	ratio = **0.06**	> P_u	OK	

Column Web Local Crippling - Beam Web ratio = 118.6 / 1139.9 = **0.10** **PASS**

Column Web Local Crippling at Beam Web Location

Column sect W12X106	d_c = 12.900 [in]	t_{fc} = 0.990	[in]	
	t_{wc} = 0.610 [in]			
	F_{yc} = 50.0 [ksi]	E = 29000	[ksi]	
Beam section W24X146	T_b = 20.700 [in]			
Beam web compression force	P_u = from beam axial force distribution calc	= **118.6**	[kips]	
Column web local crippling strength	$R_n = 0.8\, t_{wc}^2 \left[1+3\frac{T_b}{d_c}\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right] \times \left(\frac{E\, F_{cy}\, t_{fc}}{t_{wc}}\right)^{0.5}$	= 1519.9	[kips]	AISC 15th Eq J10-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th J10.3
	ϕR_n =	= **1139.9**	[kips]	
	ratio = **0.10**	> P_u	OK	

Seismic SCBF LC4 shear V = 331.1 kips axial P = 130.1 kips (C) ratio = **0.69** **PASS**

Beam to Column - Beam Shear ratio = 331.1 / 481.7 = **0.69** **PASS**

W Shape Beam Shear Yielding Check

W sect W24X146	d = 24.700 [in]	t_w = 0.650	[in]	
	F_y = 50.0 [ksi]			
Beam to column shear	V_u = from gusset interface force calc	= **331.1**	[kips]	
Beam shear strength	$R_n = 0.6\, F_y\, d\, t_w$	= 481.7	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	ϕR_n =	= **481.7**	[kips]	
	ratio = **0.69**	> V_u	OK	

Beam to Column - Beam Stub Compressive Strength ratio = 130.1 / 1935.0 = **0.07** PASS

W Shape Compression Check

W sect W24X146	A = 43.000 [in^2]	r_y = 3.022 [in]	
	F_y = 50.0 [ksi]		
W sect effective length factor	K =	= 1.00	
W sect unbraced length	L =	= 30.000 [in]	
W sect slenderness	$KL/r = 1.00 \times L \; / \; r_y$	= 9.93	
Compression force on W sect	P_u =	= **130.1** [kips]	
	when $\frac{KL}{r} \leq 25$		AISC 15th J4.4 (a)
W sect compression provided	$R_n = F_y \times A$	= 2150.0 [kips]	AISC 15th Eq J4-6
Resistance factor-LRFD	ϕ = 0.90		AISC 15th J4.4 (a)
	ϕR_n =	= **1935.0** [kips]	
	ratio = **0.07**	> P_u OK	

Beam W Shape Axial Force Distribution

W Shape Beam Axial Force Distribution

W sect W24X146	d = 24.700 [in]	b_f = 12.900 [in]	
	t_f = 1.090 [in]	t_w = 0.650 [in]	
	A = 43.000 [in^2]		
W shape one side flange area	$A_f = b_f t_f$	= 14.061 [in^2]	
W shape web area	$A_w = A - 2 A_f$	= 14.878 [in^2]	
W shape axial force	P =	= 130.1 [kips]	
Axial force on flange	$P_f = \frac{A_f}{A} P$	= 42.5 [kips]	
Axial force on web	$P_w = \frac{A_w}{A} P$	= 45.0 [kips]	

Beam to Column - W Shape Flange Weld ratio = NA = **0.00** PASS

Beam flange tensile force P_u = from beam axial force distribution calc = **42.5** [kips]

Beam flange force is in compression, the flange weld check is not needed

Beam to Column - W Shape Web Weld ratio = 16.00 / 25.05 = **0.64** **PASS**

Beam section W24X146	d_b = 24.700 [in]	k_b = 2.000	[in]
Fillet weld length on beam web	$L = d_b - 2\ k_b$	= 20.700	[in]

Weld Group Forces

Weld group forces	shear V = 331.1 [kips]	axial P = 45.0	[kips] in compression
Beam web fillet weld size	w =	= 0.563	[in]

Combined Weld Stress

Weld stress from axial force	$f_a = P / L$	= 0.000	[kip/in]	in compression
Weld stress from shear force	$f_v = V / L$	= 15.995	[kip/in]	
Weld stress combined - max	$f_{max} = f_v$	= **15.995**	[kip/in]	AISC 15th Eq 8-11
Weld stress load angle	θ =	= 0.0	[°]	

Fillet Weld Strength Calc

Fillet weld leg size	w = 9/16 [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	$R_{n\text{-}w} = 0.6\ (C_1 \times 70\ \text{ksi})\ 0.707\ w\ n\ C_2$	= 33.406	[kip/in]	AISC 15th Eq 8-1

Base metal - beam web	thickness t = 0.650 [in]	tensile F_u = 65.0	[ksi]	

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

Base metal tensile rupture	$R_{n\text{-}b} = F_u\ t$	= 42.250	[kip/in]	AISC 15th Eq J4-2

Double fillet linear shear strength	$R_n = \min (R_{n\text{-}w} , R_{n\text{-}b})$	= **33.406**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq 8-1
	$\phi\ R_n$ =	= **25.054**	[kip/in]	
	ratio = **0.64**	> f_{max}	OK	

Column Web Local Yielding - Beam Flange ratio = 42.5 / 275.7 = **0.15** **PASS**

Column Web Local Yielding at Beam Flange Location

Column sect W12X106	k_c = 1.590 [in]	t_{wc} = 0.610	[in]
	F_{yc} = 50.0 [ksi]		
Beam section W24X146	t_{fb} = 1.090 [in]		

Beam flange force	P_u = from beam axial force distribution calc	= **42.5**	[kips]	
Column web local yielding strength	$R_n = F_{yc}\ t_{wc}\ (5\ k_c + t_{fb})$	= 275.7	[kips]	AISC 15th Eq 4-2
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq 4-2
	$\phi\ R_n$ =	= **275.7**	[kips]	
	ratio = **0.15**	> P_u	OK	

Column Web Local Yielding - Beam Web		ratio = 45.0 / 631.4	= **0.07**		**PASS**
Column Web Local Yielding at Beam Web Location					
Column sect W12X106	k_c = 1.590 [in]	t_{wc} = 0.610	[in]		
	F_{yc} = 50.0 [ksi]				
Beam section W24X146	T_b = 20.700 [in]				
Beam axial force on beam web	P_u = from beam axial force distribution calc	= **45.0**	[kips]		
Column web local yielding strength	$R_n = F_{yc}\, t_{wc}\, T_b$	= 631.4	[kips]	AISC 15th Eq J10-2	
Resistance factor-LRFD	ϕ = 1.00			AISC 15th J10.2	
	$\phi\, R_n$ =	= **631.4**	[kips]		
	ratio = **0.07**	> P_u	OK		

Column Web Local Crippling - Beam Web		ratio = 130.1 / 1139.9	= **0.11**		**PASS**
Column Web Local Crippling at Beam Web Location					
Column sect W12X106	d_c = 12.900 [in]	t_{fc} = 0.990	[in]		
	t_{wc} = 0.610 [in]				
	F_{yc} = 50.0 [ksi]	E = 29000	[ksi]		
Beam section W24X146	T_b = 20.700 [in]				
Beam web compression force	P_u = from beam axial force distribution calc	= **130.1**	[kips]		
Column web local crippling strength	$R_n = 0.8\, t_{wc}^2 \left[1+3\frac{T_b}{d_c}\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right] \times \left(\frac{E\, F_{cy}\, t_{fc}}{t_{wc}}\right)^{0.5}$	= 1519.9	[kips]	AISC 15th Eq J10-4	
Resistance factor-LRFD	ϕ = 0.75			AISC 15th J10.3	
	ϕR_n =	= **1139.9**	[kips]		
	ratio = **0.11**	> P_u	OK		

Beam Splice Calculation Brace Seismic System = SCBF Code=AISC 360-16 LRFD

Result Summary	geometries & weld limitations = **PASS**	limit states max ratio = **0.61**	**PASS**

Min Clearance Gap Between Beam Stub & Beam **PASS**

Refer to AISC Seismic Design Manual 2nd Ed Page 5-235, in order to prevent binding at a 0.025 rad storey drift, the clearance between beam stub and beam at the splice must be at least (0.5 beam depth x 0.025 rad)

Beam sect W24X146 depth	d =	= 24.700	[in]
Min required gap between beam stub and beam	c_{min} = 0.5 d x 0.025 rad	= **0.309**	[in]
Actual gap between beam stub and beam	c = from user input	= **1.000**	[in]
		≥ c_{min}	OK

Splice Plate Thickness Limit for Ductility					**PASS**
Bolt grade	grade = A325-X				
Nominal tensile/shear stress	F_t = 90.0	[ksi]	F_v = 68.0	[ksi]	AISC 15th Table J3.2
Bolt diameter & area	d_b = 0.875	[in]	Area A_b = 0.601	[in^2]	
Bolt group	row n_v = 6		col n_h = 2		
	row s_v = 3.000	[in]	col s_h = 3.000	[in]	
Bolt group coefficient C'	C' = from AISC 15th Table 7-6 ~ 7-13		= 54.2		
Splice plate thickness & depth	t = 0.375	[in]	d = 19.000	[in]	
	F_y = 50.0	[ksi]			
Bolt group flexural strength	$M_{max} = \frac{F_v}{0.9}(A_b C')$		= 205.08	[kip-ft]	AISC 15th Eq 10-4
Max allowed splice plate thickness	$t_{max} = \frac{6 M_{max}}{F_y d^2}$		= **0.818**	[in]	AISC 15th Eq 10-3
Splice plate thickness provided	2t = t x 2 sides		= **0.750**	[in]	
			≤ t_{max}	OK	

Seismic SCBF LC3 shear V = 19.9 kips axial P = 312.1 kips (C) ratio = **0.61** **PASS**

Beam Web - Outer Side Bolt Group Eccentricity - Shear Only

Shear V - from user input, beam end shear reaction caused by gravity load only
e_x - hor distance from column face to the beam splice outer side bolt group CG point

Bolt group forces	shear V = 19.9	[kips]	axial P = 0.0	[kips]
Resultant force to ver Y axis angle	θ = $\tan^{-1}$ (P / V)		= 0.00	[°]
Bolt group row and column	bolt row n_r = 6		bolt col n_c = 2	
Bolt row/column spacing	bolt row s_r = 3.000	[in]	bolt col s_c = 3.000	[in]
Shear force to bolt group CG ecc	e_x =		= 34.500	[in]
Shear force to ver Y axis angle	θ =		= 0.00	[°]
Bolt group coefficient C	C = from AISC 15th Table 7-6 ~ 7-13		= 1.552	
Bolt group eccentricity coefficient	C_{ec} = C / (n_r x n_c)		= **0.129**	

Beam Web - Outer Side Bolt Group - Bolt Shear		ratio = 19.9 / 94.9	= **0.21**	**PASS**	
Bolt group forces	shear V = 19.9	[kips]	axial P = 0.0	[kips]	
Bolt group resultant force	R = $(V^2 + P^2)^{0.5}$		= **19.9**	[kips]	
Bolt shear stress	bolt grade = A325-X		F_{nv} = 68.0	[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 = 12.0		shear plane m = 2		
Bolt group eccentricity coefficient	C_{ec} = from 'Bolt Group Eccentricity' calc		= 0.129		
Required shear strength	V_u =		= **19.9**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$		= 126.6	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J3-1
	ϕ R_n =		= **94.9**	[kips]	
	ratio = **0.21**		> V_u	OK	

Beam Web - Outer Side Bolt Group - Bolt Bearing ratio = 19.9 / 94.9 = **0.21** **PASS**

The bolt group is oriented so that the shear force V is in ver. direction and the axial force P is in hor. direction

Bolt group forces	shear V = 19.9	[kips]	axial P = 0.0	[kips]		
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$		= **19.9**	[kips]		

Single Bolt Shear Strength

Bolt shear stress	bolt grade = A325-X		F_{nv} = 68.0	[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	$R_{n\text{-bolt}} = 2 \times F_{nv} A_b$		= 81.8	[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 = 0.650	[in]			

Interior Bolt

Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.063	[in]	
Bolt tear out/bearing strength	$R_{n\text{-t\&b-in}} = 1.2\, L_c\, t\, F_u \le 2.4\, d_b\, t\, m\, F_u$			AISC 15th Eq J3-6a
	= 104.6 ≤ 88.7	= 88.7	[kips]	
Bolt strength at interior	$R_{n\text{-in}} = \min(R_{n\text{-t\&b-in}}, R_{n\text{-bolt}})$	= 81.8	[kips]	

Number of bolt	interior n_{in} = 12			
Bolt group eccentricity coefficient	C_{ec} = from calc in above section	= 0.129		
Bolt bearing strength for all bolts	$R_n = (n_{in} R_{n\text{-in}})\, C_{ec}$	= 126.6	[kips]	
Required shear strength	V_u =	= **19.9**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th J3-10
	ϕR_n =	= **94.9**	[kips]	
	ratio = **0.21**	> V_u	OK	

Beam Web - Inner Side Bolt Group Eccentricity - Shear Only

Shear V - from user input, beam end shear reaction caused by gravity load only
e_x - hor distance from column face to the beam splice inner side bolt group CG point

Bolt group forces	shear V = 19.9	[kips]	axial P = 0.0	[kips]
Resultant force to ver Y axis angle	$\theta = \tan^{-1}(P / V)$		= 0.00	[°]

Bolt group row and column	bolt row n_r = 6		bolt col n_c = 2	
Bolt row/column spacing	bolt row s_r = 3.000	[in]	bolt col s_c = 3.000	[in]
Shear force to bolt group CG ecc	e_x =		= 26.500	[in]
Shear force to ver Y axis angle	θ =		= 0.00	[°]
Bolt group coefficient C	C = from AISC 15th Table 7-6 ~ 7-13		= 2.007	
Bolt group eccentricity coefficient	$C_{ec} = C / (n_r \times n_c)$		= **0.167**	

Beam Web - Inner Side Bolt Group - Bolt Shear ratio = 19.9 / 122.9 = **0.16** **PASS**

Bolt group forces	shear V = 19.9 [kips]	axial P = 0.0	[kips]		
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= **19.9**	[kips]		
Bolt shear stress	bolt grade = A325-X	F_{nv} = 68.0	[ksi]	AISC 15^{th} 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 = 12.0	shear plane m = 2			
Bolt group eccentricity coefficient	C_{ec} = from 'Bolt Group Eccentricity' calc	= 0.167			
Required shear strength	V_u =	= **19.9**	[kips]		
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 163.9	[kips]	AISC 15^{th} Eq J3-1	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15^{th} Eq J3-1	
	ϕR_n =	= **122.9**	[kips]		
	ratio = **0.16**	$> V_u$	OK		

Beam Web - Inner Side Bolt Group - Bolt Bearing ratio = 19.9 / 122.9 = **0.16** **PASS**

The bolt group is oriented so that the shear force V is in ver. direction and the axial force P is in hor. direction

Bolt group forces	shear V = 19.9 [kips]	axial P = 0.0	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= **19.9**	[kips]	

Single Bolt Shear Strength

Bolt shear stress	bolt grade = A325-X	F_{nv} = 68.0	[ksi]	AISC 15^{th} Table J3.2
	bolt dia d_b = 0.875 [in]	bolt area A_b = 0.601	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = 2 \times F_{nv} A_b$	= 81.8	[kips]	AISC 15^{th} 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 15^{th} Table J3.3
Bolt spacing	spacing L_s = 3.000 [in]			
Plate tensile strength	F_u = 65.0 [ksi]			
Plate thickness	t = 0.650 [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 15^{th} Eq J3-6a
	= 104.6 ≤ 88.7	= 88.7	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min (R_{n\text{-}t\&b\text{-}in} , R_{n\text{-}bolt})$	= 81.8	[kips]	

Number of bolt	interior n_{in} = 12			
Bolt group eccentricity coefficient	C_{ec} = from calc in above section	= 0.167		
Bolt bearing strength for all bolts	$R_n = (n_{in} R_{n\text{-}in}) C_{ec}$	= 163.9	[kips]	
Required shear strength	V_u =	= **19.9**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15^{th} J3-10
	ϕR_n =	= **122.9**	[kips]	
	ratio = **0.16**	$> V_u$	OK	

Beam Web - Inner Side Bolt Group Eccentricity - Shear + Axial

Shear V - from user input, beam end shear reaction caused by gravity load only
Axial P - from gusset interface forces calc , beam member axial load P_{bm}
e_x - hor distance from the gusset-beam weld line CG point to the beam splice inner side bolt group CG point

Bolt group forces	shear V = 19.9 [kips]	axial P = 312.1	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= 312.7	[kips]	
Resultant force to ver Y axis angle	$\theta = \tan^{-1} (P / V)$	= 86.35	[°]	
Bolt group row and column	bolt row n_r = 6	bolt col n_c = 2		
Bolt row/column spacing	bolt row s_r = 3.000 [in]	bolt col s_c = 3.000	[in]	
Shear force to bolt group CG ecc	e_x =	= 12.125	[in]	
Shear force to ver Y axis angle	θ =	= 86.35	[°]	
Bolt group coefficient C	C = from AISC 15th Table 7-6 ~ 7-13	= 10.595		
Bolt group eccentricity coefficient	$C_{ec} = C / (n_r \times n_c)$	= **0.883**		

Beam Web - Inner Side Bolt Group - Bolt Shear ratio = 312.7 / 649.9 = **0.48** **PASS**

Bolt group forces	shear V = 19.9 [kips]	axial P = 312.1	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= **312.7**	[kips]	
Bolt shear stress	bolt grade = A325-X	F_{nv} = 68.0	[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 = 12.0	shear plane m = 2		
Bolt group eccentricity coefficient	C_{ec} = from 'Bolt Group Eccentricity' calc	= 0.883		
Required shear strength	V_u =	= **312.7**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 866.5	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J3-1
	ϕR_n =	= **649.9**	[kips]	
	ratio = **0.48**	> V_u	OK	

Beam Web - Inner Side Bolt Group - Bolt Bearing		ratio = 312.7 / 633.4	= **0.49**	**PASS**	
The bolt group is oriented so that the shear force V is in ver. direction and the axial force P is in hor. direction					
Bolt group forces	shear V = 19.9 [kips]	axial P = 312.1	[kips]		
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= **312.7**	[kips]		
Single Bolt Shear Strength					
Bolt shear stress	bolt grade = A325-X	F_{nv} = 68.0	[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	$R_{n\text{-}bolt} = 2 \times F_{nv} A_b$	= 81.8	[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 = 2.000	[in]		
Plate tensile strength	F_u = 65.0 [ksi]				
Plate thickness	t = 0.650 [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	
	= 104.6 ≤ 88.7	= 88.7	[kips]		
Bolt strength at interior	$R_{n\text{-}in} = \min (R_{n\text{-}t\&b\text{-}in} , R_{n\text{-}bolt})$	= 81.8	[kips]		
Edge Bolt					
Bolt hole edge clear distance	$L_c = L_e - d_h / 2$	= 1.531	[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	
	= 77.6 ≤ 88.7	= 77.6	[kips]		
Bolt strength at edge	$R_{n\text{-}ed} = \min (R_{n\text{-}t\&b\text{-}ed} , R_{n\text{-}bolt})$	= 77.6	[kips]		
Number of bolt	interior n_{in} = 6	edge n_{ed} = 6			
Bolt group eccentricity coefficient	C_{ec} = from calc in above section	= 0.883			
Bolt bearing strength for all bolts	$R_n = (n_{in} R_{n\text{-}in} + n_{ed} R_{n\text{-}ed})\ C_{ec}$	= 844.6	[kips]		
Required shear strength	V_u =	= **312.7**	[kips]		
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th J3-10	
	$\phi\ R_n$ =	= **633.4**	[kips]		
	ratio = **0.49**	> V_u	OK		

Beam Web - Outer Side Bolt Group Eccentricity - Shear + Axial

Shear V - from user input, beam end shear reaction caused by gravity load only
Axial P - from gusset interface forces calc , beam member axial load P_{bm}
e_x - hor distance from the gusset-beam weld line CG point to the beam splice outer side bolt group CG point

Bolt group forces	shear V = 19.9 [kips]	axial P = 312.1	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= 312.7	[kips]	
Resultant force to ver Y axis angle	$\theta = \tan^{-1} (P / V)$	= 86.35	[°]	
Bolt group row and column	bolt row n_r = 6	bolt col n_c = 2		
Bolt row/column spacing	bolt row s_r = 3.000 [in]	bolt col s_c = 3.000	[in]	
Shear force to bolt group CG ecc	e_x =	= 20.125	[in]	
Shear force to ver Y axis angle	θ =	= 86.35	[°]	
Bolt group coefficient C	C = from AISC 15th Table 7-6 ~ 7-13	= 9.943		
Bolt group eccentricity coefficient	$C_{ec} = C / (n_r \times n_c)$	= **0.829**		

Beam Web - Outer Side Bolt Group - Bolt Shear ratio = 312.7 / 610.2 = **0.51** **PASS**

Bolt group forces	shear V = 19.9 [kips]	axial P = 312.1	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= **312.7**	[kips]	
Bolt shear stress	bolt grade = A325-X	F_{nv} = 68.0	[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 = 12.0	shear plane m = 2		
Bolt group eccentricity coefficient	C_{ec} = from 'Bolt Group Eccentricity' calc	= 0.829		
Required shear strength	V_u =	= **312.7**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 813.5	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J3-1
	ϕR_n =	= **610.2**	[kips]	
	ratio = **0.51**	> V_u	OK	

Beam Web - Outer Side Bolt Group - Bolt Bearing ratio = 312.7 / 594.7 = **0.53** **PASS**

The bolt group is oriented so that the shear force V is in ver. direction and the axial force P is in hor. direction

Bolt group forces	shear V = 19.9 [kips]		axial P = 312.1	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$		= **312.7**	[kips]	

Single Bolt Shear Strength

Bolt shear stress	bolt grade = A325-X		F_{nv} = 68.0	[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	$R_{n\text{-bolt}} = 2 \times F_{nv} A_b$		= 81.8	[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 = 2.000	[in]	
Plate tensile strength	F_u = 65.0 [ksi]				
Plate thickness	t = 0.650 [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
	= 104.6 ≤ 88.7		= 88.7	[kips]	
Bolt strength at interior	$R_{n\text{-in}} = \min(R_{n\text{-t\&b-in}}, R_{n\text{-bolt}})$		= 81.8	[kips]	

Edge Bolt

Bolt hole edge clear distance	$L_c = L_e - d_h / 2$		= 1.531	[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
	= 77.6 ≤ 88.7		= 77.6	[kips]	
Bolt strength at edge	$R_{n\text{-ed}} = \min(R_{n\text{-t\&b-ed}}, R_{n\text{-bolt}})$		= 77.6	[kips]	

Number of bolt	interior n_{in} = 6		edge n_{ed} = 6		
Bolt group eccentricity coefficient	C_{ec} = from calc in above section		= 0.829		
Bolt bearing strength for all bolts	$R_n = (n_{in} R_{n\text{-in}} + n_{ed} R_{n\text{-ed}})\, C_{ec}$		= 792.9	[kips]	
Required shear strength	V_u =		= **312.7**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 15th J3-10
	ϕR_n =		= **594.7**	[kips]	
	ratio = **0.53**		> V_u	OK	

Splice Plate - Outer Side Bolt Group Eccentricity - Shear Only

Shear V - from user input, beam end shear reaction caused by gravity load only
e_x - hor distance from column face to the beam splice outer side bolt group CG point

Bolt group forces	shear V = 19.9 [kips]		axial P = 0.0	[kips]	
Resultant force to ver Y axis angle	$\theta = \tan^{-1}(P / V)$		= 0.00	[°]	

Bolt group row and column	bolt row n_r = 6		bolt col n_c = 2		
Bolt row/column spacing	bolt row s_r = 3.000 [in]		bolt col s_c = 3.000	[in]	
Shear force to bolt group CG ecc	e_x =		= 34.500	[in]	
Shear force to ver Y axis angle	θ =		= 0.00	[°]	
Bolt group coefficient C	C = from AISC 15th Table 7-6 ~ 7-13		= 1.552		
Bolt group eccentricity coefficient	$C_{ec} = C / (n_r \times n_c)$		= **0.129**		

Splice Plate - Outer Side Bolt Group - Bolt Bearing		ratio = 10.0 / 47.5	= **0.21**	**PASS**	

The bolt group is oriented so that the shear force V is in ver. direction and the axial force P is in hor. direction

Bolt group forces	shear V = 10.0	[kips]	axial P = 0.0	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$		= **10.0**	[kips]	

Single Bolt Shear Strength

Bolt shear stress	bolt grade = A325-X		F_{nv} = 68.0	[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	$R_{n\text{-}bolt} = F_{nv} A_b$		= 40.9	[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 = 0.375	[in]			

Interior Bolt

Bolt hole edge clear distance	$L_c = L_s - d_h$		= 2.063	[in]	
Bolt tear out/bearing strength	$R_{n\text{-}t\&b\text{-}in} = 1.2\, L_c\, t\, F_u \le 2.4\, d_b\, t\, m\, F_u$				AISC 15th Eq J3-6a
	= 60.3 ≤ 51.2		= 51.2	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$		= 40.9	[kips]	

Number of bolt	interior n_{in} = 12				
Bolt group eccentricity coefficient	C_{ec} = from calc in above section		= 0.129		
Bolt bearing strength for all bolts	$R_n = (n_{in} R_{n\text{-}in})\, C_{ec}$		= 63.3	[kips]	
Required shear strength	V_u =		= **10.0**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 15th J3-10
	ϕR_n =		= **47.5**	[kips]	
	ratio = **0.21**		> V_u	OK	

Splice Plate - Compression Buckling   ratio = 156.1 / 256.1 = **0.61** **PASS**

Plate Compression Check

Plate size	width b_p = 19.000 [in]	thickness t_p = 0.375	[in]	
	F_y = 50.0 [ksi]	E = 29000	[ksi]	
Plate gross area in compression	$A_g = b_p t_p$	= 7.125	[in^2]	
Plate radius of gyration	$r = t_p / \sqrt{12}$	= 0.108	[in]	
Plate effective length factor	K =	= 1.20		AISC 15th Table C-A-7.1
Plate unbraced length	L_u =	= 5.000	[in]	
Plate slenderness	$KL/r = 1.20 \times L_u / r$	= 55.43		
	when $\frac{KL}{r} > 25$, use Chapter E			AISC 15th J4.4 (b)
Elastic buckling stress	$F_e = \frac{\pi^2 E}{(KL/r)^2}$	= 93.17	[ksi]	AISC 15th Eq E3-4
	when $\frac{KL}{r} \le 4.71 \left(\frac{E}{F_y}\right)^{0.5} = 113.43$			AISC 15th E3 (a)
Critical stress	$F_{cr} = 0.658^{(F_y / F_e)} F_y$	= 39.94	[ksi]	AISC 15th Eq E3-2
Plate compression required	P_u = 0.5 x beam axial compression P	= **156.1**	[kips]	
Plate compression provided	$R_n = F_{cr} \times A_g$	= 284.6	[kips]	AISC 15th Eq E3-1
Resistance factor-LRFD	ϕ = 0.90			AISC 15th E1
	ϕR_n =	= **256.1**	[kips]	
	ratio = **0.61**	> P_u	OK	

Splice Plate - Shear Yielding   ratio = 19.9 / 427.5 = **0.05** **PASS**

Splice Plate Shear Yielding

Plate size	width b_p = 19.000 [in]	thickness t_p = 0.375	[in]	
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p \times 2$ side	= 14.250	[in^2]	
Shear force required	V_u =	= **19.9**	[kips]	
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$	= 427.5	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	ϕR_n =	= **427.5**	[kips]	
	ratio = **0.05**	> V_u	OK	

Splice Plate - Flexural Yielding   ratio = 54.73 / 253.83 = **0.22** **PASS**

Plate width & thick	width b_p = 19.000 [in]	thick t_p = 0.375	[in]	
	yield F_y = 50.0 [ksi]			
Beam end shear reaction caused by gravity load only	V = from user input	= 19.9	[kips]	
Ecc from column face to outer bolt group first bolt line	e =	= 33.000	[in]	
Flexural strength required	$M_r = V \times e$	= **54.73**	[kip-ft]	
Shear plate - plastic modulus	$Z_p = (b_p \times t_p^2) / 4 \times 2$ side	= 67.69	[in^3]	
Flexural strength available	$M_c = \phi F_y Z_p$ ϕ=0.90	= 253.83	[kip-ft]	
	ratio = **0.22**	> M_r	OK	

Splice Plate - Shear Rupture		ratio = 10.0 / 142.6	= **0.07**	**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 = 6			
Plate size	width b_p = 19.000 [in]	thickness t_p = 0.375	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate net area in shear	$A_{nv} = (b_p - n\, d_h)\, t_p$	= 4.875	[in^2]	
Shear force required	V_u =	= **10.0**	[kips]	
Plate shear rupture strength	$R_n = 0.6\, F_u A_{nv}$	= 190.1	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-4
	$\phi\, R_n$ =	= **142.6**	[kips]	
	ratio = **0.07**	> V_u	OK	

Splice Plate - Flexural Rupture		ratio = 27.36 / 96.36	= **0.28**	**PASS**

Plate A_n and Z_{net} Calc

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 = 6			
Plate size	width b_p = 19.000 [in]	thickness t_p = 0.375	[in]	
Plate net area	$A_n = (b_p - n\, d_h)\, t_p$	= 4.875	[in^2]	
Plate net plastic sect modulus	Z_{net} =	= 23.72	[in^3]	
Plate net elastic sect modulus	S_{net} =	= 16.33	[in^3]	
Plate width & thick	width b_p = 19.000 [in]	thick t_p = 0.375	[in]	
	F_u = 65.0 [ksi]			
Beam end shear reaction caused by gravity load only	V = from user input, x 0.5 for 2 side plate	= 10.0	[kips]	
Ecc from column face to outer bolt group first bolt line	e =	= 33.000	[in]	
Flexural strength required	M_r = V x e	= **27.36**	[kip-ft]	
Shear plate - net plastic modulus	Z_{net} = from above calc	= 23.72	[in^3]	
Flexural strength available	$M_c = \phi\, F_u\, Z_{net}$ ϕ=0.75	= **96.36**	[kip-ft]	
	ratio = **0.28**	> M_r	OK	

Seismic SCBF LC4	shear V = 19.9 kips	axial P = 312.7 kips (C)	ratio = **0.61**	**PASS**

Beam Web - Outer Side Bolt Group Eccentricity - Shear Only

Shear V - from user input, beam end shear reaction caused by gravity load only
e_x - hor distance from column face to the beam splice outer side bolt group CG point

Bolt group forces	shear V = 19.9 [kips]	axial P = 0.0	[kips]	
Resultant force to ver Y axis angle	$\theta = \tan^{-1} (P / V)$	= 0.00	[°]	
Bolt group row and column	bolt row n_r = 6	bolt col n_c = 2		
Bolt row/column spacing	bolt row s_r = 3.000 [in]	bolt col s_c = 3.000	[in]	
Shear force to bolt group CG ecc	e_x =	= 34.500	[in]	
Shear force to ver Y axis angle	θ =	= 0.00	[°]	
Bolt group coefficient C	C = from AISC 15th Table 7-6 ~ 7-13	= 1.552		
Bolt group eccentricity coefficient	$C_{ec} = C / (n_r \times n_c)$	= **0.129**		

Beam Web - Outer Side Bolt Group - Bolt Shear		ratio = 19.9 / 94.9	= **0.21**	**PASS**	
Bolt group forces	shear V = 19.9 [kips]	axial P = 0.0	[kips]		
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= **19.9**	[kips]		
Bolt shear stress	bolt grade = A325-X	F_{nv} = 68.0	[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 = 12.0	shear plane m = 2			
Bolt group eccentricity coefficient	C_{ec} = from 'Bolt Group Eccentricity' calc	= 0.129			
Required shear strength	V_u =	= **19.9**	[kips]		
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 126.6	[kips]	AISC 15th Eq J3-1	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J3-1	
	ϕR_n =	= **94.9**	[kips]		
	ratio = **0.21**	> V_u	OK		

Beam Web - Outer Side Bolt Group - Bolt Bearing		ratio = 19.9 / 94.9	= **0.21**	**PASS**
The bolt group is oriented so that the shear force V is in ver. direction and the axial force P is in hor. direction				
Bolt group forces	shear V = 19.9 [kips]	axial P = 0.0	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= **19.9**	[kips]	
Single Bolt Shear Strength				
Bolt shear stress	bolt grade = A325-X	F_{nv} = 68.0	[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	$R_{n\text{-}bolt} = 2 \times F_{nv} A_b$	= 81.8	[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 = 0.650 [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
	= 104.6 ≤ 88.7	= 88.7	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min (R_{n\text{-}t\&b\text{-}in} , R_{n\text{-}bolt})$	= 81.8	[kips]	
Number of bolt	interior n_{in} = 12			
Bolt group eccentricity coefficient	C_{ec} = from calc in above section	= 0.129		
Bolt bearing strength for all bolts	$R_n = (n_{in} R_{n\text{-}in}) C_{ec}$	= 126.6	[kips]	
Required shear strength	V_u =	= **19.9**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th J3-10
	ϕR_n =	= **94.9**	[kips]	
	ratio = **0.21**	> V_u	OK	

Beam Web - Inner Side Bolt Group Eccentricity - Shear Only

Shear V - from user input, beam end shear reaction caused by gravity load only
e_x - hor distance from column face to the beam splice inner side bolt group CG point

Bolt group forces	shear V = 19.9	[kips]	axial P = 0.0	[kips]
Resultant force to ver Y axis angle	$\theta = \tan^{-1}(P / V)$		= 0.00	[°]
Bolt group row and column	bolt row n_r = 6		bolt col n_c = 2	
Bolt row/column spacing	bolt row s_r = 3.000	[in]	bolt col s_c = 3.000	[in]
Shear force to bolt group CG ecc	e_x =		= 26.500	[in]
Shear force to ver Y axis angle	θ =		= 0.00	[°]
Bolt group coefficient C	C = from AISC 15th Table 7-6 ~ 7-13		= 2.007	
Bolt group eccentricity coefficient	$C_{ec} = C / (n_r \times n_c)$		= **0.167**	

Beam Web - Inner Side Bolt Group - Bolt Shear ratio = 19.9 / 122.9 = **0.16** **PASS**

Bolt group forces	shear V = 19.9	[kips]	axial P = 0.0	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$		= **19.9**	[kips]	
Bolt shear stress	bolt grade = A325-X		F_{nv} = 68.0	[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 = 12.0		shear plane m = 2		
Bolt group eccentricity coefficient	C_{ec} = from 'Bolt Group Eccentricity' calc		= 0.167		
Required shear strength	V_u =		= **19.9**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$		= 163.9	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J3-1
	ϕR_n =		= **122.9**	[kips]	
	ratio = **0.16**		> V_u	OK	

Beam Web - Inner Side Bolt Group - Bolt Bearing	ratio = 19.9 / 122.9	= **0.16**	**PASS**

The bolt group is oriented so that the shear force V is in ver. direction and the axial force P is in hor. direction

Bolt group forces	shear V = 19.9	[kips]	axial P = 0.0	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$		= **19.9**	[kips]	
Single Bolt Shear Strength					
Bolt shear stress	bolt grade = A325-X		F_{nv} = 68.0	[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	$R_{n\text{-}bolt} = 2 \times F_{nv} A_b$		= 81.8	[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 = 0.650	[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
	= 104.6 ≤ 88.7		= 88.7	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min (R_{n\text{-}t\&b\text{-}in} , R_{n\text{-}bolt})$		= 81.8	[kips]	
Number of bolt	interior n_{in} = 12				
Bolt group eccentricity coefficient	C_{ec} = from calc in above section		= 0.167		
Bolt bearing strength for all bolts	$R_n = (n_{in} R_{n\text{-}in}) C_{ec}$		= 163.9	[kips]	
Required shear strength	V_u =		= **19.9**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 15th J3-10
	ϕR_n =		= **122.9**	[kips]	
	ratio = **0.16**		> V_u	OK	

Beam Web - Inner Side Bolt Group Eccentricity - Shear + Axial

Shear V - from user input, beam end shear reaction caused by gravity load only
Axial P - from gusset interface forces calc , beam member axial load P_{bm}
e_x - hor distance from the gusset-beam weld line CG point to the beam splice inner side bolt group CG point

Bolt group forces	shear V = 19.9	[kips]	axial P = 312.7	[kips]
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$		= 313.3	[kips]
Resultant force to ver Y axis angle	$\theta = \tan^{-1} (P / V)$		= 86.36	[°]
Bolt group row and column	bolt row n_r = 6		bolt col n_c = 2	
Bolt row/column spacing	bolt row s_r = 3.000	[in]	bolt col s_c = 3.000	[in]
Shear force to bolt group CG ecc	e_x =		= 12.125	[in]
Shear force to ver Y axis angle	θ =		= 86.36	[°]
Bolt group coefficient C	C = from AISC 15th Table 7-6 ~ 7-13		= 10.597	
Bolt group eccentricity coefficient	$C_{ec} = C / (n_r \times n_c)$		= **0.883**	

Beam Web - Inner Side Bolt Group - Bolt Shear		ratio = 313.3 / 649.9	= **0.48**	**PASS**
Bolt group forces	shear V = 19.9 [kips]	axial P = 312.7	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= **313.3**	[kips]	
Bolt shear stress	bolt grade = A325-X	F_{nv} = 68.0	[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 = 12.0	shear plane m = 2		
Bolt group eccentricity coefficient	C_{ec} = from 'Bolt Group Eccentricity' calc	= 0.883		
Required shear strength	V_u =	= **313.3**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 866.5	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J3-1
	ϕR_n =	= **649.9**	[kips]	
	ratio = **0.48**	> V_u	OK	

Beam Web - Inner Side Bolt Group - Bolt Bearing		ratio = 313.3 / 633.4	= **0.49**	**PASS**

The bolt group is oriented so that the shear force V is in ver. direction and the axial force P is in hor. direction

Bolt group forces	shear V = 19.9 [kips]	axial P = 312.7	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= **313.3**	[kips]	
Single Bolt Shear Strength				
Bolt shear stress	bolt grade = A325-X	F_{nv} = 68.0	[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	$R_{n\text{-}bolt} = 2 \times F_{nv} A_b$	= 81.8	[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 = 2.000	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate thickness	t = 0.650 [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
	= 104.6 ≤ 88.7	= 88.7	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$	= 81.8	[kips]	
Edge Bolt				
Bolt hole edge clear distance	$L_c = L_e - d_h / 2$	= 1.531	[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
	= 77.6 ≤ 88.7	= 77.6	[kips]	
Bolt strength at edge	$R_{n\text{-}ed} = \min(R_{n\text{-}t\&b\text{-}ed}, R_{n\text{-}bolt})$	= 77.6	[kips]	
Number of bolt	interior n_{in} = 6	edge n_{ed} = 6		
Bolt group eccentricity coefficient	C_{ec} = from calc in above section	= 0.883		
Bolt bearing strength for all bolts	$R_n = (n_{in} R_{n\text{-}in} + n_{ed} R_{n\text{-}ed}) C_{ec}$	= 844.6	[kips]	
Required shear strength	V_u =	= **313.3**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th J3-10
	ϕR_n =	= **633.4**	[kips]	
	ratio = **0.49**	> V_u	OK	

Beam Web - Outer Side Bolt Group Eccentricity - Shear + Axial

Shear V - from user input, beam end shear reaction caused by gravity load only
Axial P - from gusset interface forces calc , beam member axial load P_{bm}
e_x - hor distance from the gusset-beam weld line CG point to the beam splice outer side bolt group CG point

Bolt group forces	shear V = 19.9 [kips]	axial P = 312.7	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= 313.3	[kips]	
Resultant force to ver Y axis angle	$\theta = \tan^{-1} (P / V)$	= 86.36	[°]	
Bolt group row and column	bolt row n_r = 6	bolt col n_c = 2		
Bolt row/column spacing	bolt row s_r = 3.000 [in]	bolt col s_c = 3.000	[in]	
Shear force to bolt group CG ecc	e_x =	= 20.125	[in]	
Shear force to ver Y axis angle	θ =	= 86.36	[°]	
Bolt group coefficient C	C = from AISC 15th Table 7-6 ~ 7-13	= 9.946		
Bolt group eccentricity coefficient	$C_{ec} = C / (n_r \times n_c)$	= **0.829**		

Beam Web - Outer Side Bolt Group - Bolt Shear — ratio = 313.3 / 610.2 = **0.51** **PASS**

Bolt group forces	shear V = 19.9 [kips]	axial P = 312.7	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= **313.3**	[kips]	
Bolt shear stress	bolt grade = A325-X	F_{nv} = 68.0	[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 = 12.0	shear plane m = 2		
Bolt group eccentricity coefficient	C_{ec} = from 'Bolt Group Eccentricity' calc	= 0.829		
Required shear strength	V_u =	= **313.3**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 813.5	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J3-1
	ϕR_n =	= **610.2**	[kips]	
	ratio = **0.51**	> V_u	OK	

Beam Web - Outer Side Bolt Group - Bolt Bearing		ratio = 313.3 / 594.7	= **0.53**	**PASS**	
The bolt group is oriented so that the shear force V is in ver. direction and the axial force P is in hor. direction					
Bolt group forces	shear V = 19.9 [kips]	axial P = 312.7	[kips]		
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= **313.3**	[kips]		
Single Bolt Shear Strength					
Bolt shear stress	bolt grade = A325-X	F_{nv} = 68.0	[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	$R_{n\text{-}bolt} = 2 \times F_{nv} A_b$	= 81.8	[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 = 2.000	[in]		
Plate tensile strength	F_u = 65.0 [ksi]				
Plate thickness	t = 0.650 [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	
	= 104.6 ≤ 88.7	= 88.7	[kips]		
Bolt strength at interior	$R_{n\text{-}in} = \min (R_{n\text{-}t\&b\text{-}in} , R_{n\text{-}bolt})$	= 81.8	[kips]		
Edge Bolt					
Bolt hole edge clear distance	$L_c = L_e - d_h / 2$	= 1.531	[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	
	= 77.6 ≤ 88.7	= 77.6	[kips]		
Bolt strength at edge	$R_{n\text{-}ed} = \min (R_{n\text{-}t\&b\text{-}ed} , R_{n\text{-}bolt})$	= 77.6	[kips]		
Number of bolt	interior n_{in} = 6	edge n_{ed} = 6			
Bolt group eccentricity coefficient	C_{ec} = from calc in above section	= 0.829			
Bolt bearing strength for all bolts	$R_n = (n_{in} R_{n\text{-}in} + n_{ed} R_{n\text{-}ed}) C_{ec}$	= 792.9	[kips]		
Required shear strength	V_u =	= **313.3**	[kips]		
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th J3-10	
	ϕR_n =	= **594.7**	[kips]		
	ratio = **0.53**	> V_u	OK		

Splice Plate - Outer Side Bolt Group Eccentricity - Shear Only				
Shear V - from user input, beam end shear reaction caused by gravity load only				
e_x - hor distance from column face to the beam splice outer side bolt group CG point				
Bolt group forces	shear V = 19.9 [kips]	axial P = 0.0	[kips]	
Resultant force to ver Y axis angle	$\theta = \tan^{-1} (P / V)$	= 0.00	[°]	
Bolt group row and column	bolt row n_r = 6	bolt col n_c = 2		
Bolt row/column spacing	bolt row s_r = 3.000 [in]	bolt col s_c = 3.000	[in]	
Shear force to bolt group CG ecc	e_x =	= 34.500	[in]	
Shear force to ver Y axis angle	θ =	= 0.00	[°]	
Bolt group coefficient C	C = from AISC 15th Table 7-6 ~ 7-13	= 1.552		
Bolt group eccentricity coefficient	$C_{ec} = C / (n_r \times n_c)$	= **0.129**		

Splice Plate - Outer Side Bolt Group - Bolt Bearing		ratio = 10.0 / 47.5	= **0.21**	**PASS**	

The bolt group is oriented so that the shear force V is in ver. direction and the axial force P is in hor. direction

Bolt group forces	shear V = 10.0 [kips]	axial P = 0.0	[kips]		
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= **10.0**	[kips]		

Single Bolt Shear Strength

Bolt shear stress	bolt grade = A325-X	F_{nv} = 68.0	[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	$R_{n\text{-}bolt} = F_{nv} A_b$	= 40.9	[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 = 0.375 [in]			

Interior Bolt

Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.063	[in]	
Bolt tear out/bearing strength	$R_{n\text{-}t\&b\text{-}in} = 1.2\ L_c\ t\ F_u \le 2.4\ d_b\ t\ m\ F_u$			AISC 15th Eq J3-6a
	$= 60.3 \le 51.2$	= 51.2	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min (R_{n\text{-}t\&b\text{-}in} , R_{n\text{-}bolt})$	= 40.9	[kips]	

Number of bolt	interior n_{in} = 12			
Bolt group eccentricity coefficient	C_{ec} = from calc in above section	= 0.129		
Bolt bearing strength for all bolts	$R_n = (n_{in} R_{n\text{-}in})\ C_{ec}$	= 63.3	[kips]	
Required shear strength	V_u =	= **10.0**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th J3-10
	$\phi\ R_n$ =	= **47.5**	[kips]	
	ratio = **0.21**	> V_u	OK	

Splice Plate - Compression Buckling		ratio = 156.4 / 256.1	= **0.61**	**PASS**
Plate Compression Check				
Plate size	width b_p = 19.000 [in]	thickness t_p = 0.375 [in]		
	F_y = 50.0 [ksi]	E = 29000 [ksi]		
Plate gross area in compression	$A_g = b_p t_p$	= 7.125 [in^2]		
Plate radius of gyration	$r = t_p / \sqrt{12}$	= 0.108 [in]		
Plate effective length factor	K =	= 1.20	AISC 15th Table C-A-7.1	
Plate unbraced length	L_u =	= 5.000 [in]		
Plate slenderness	$KL/r = 1.20 \times L_u / r$	= 55.43		
	when $\frac{KL}{r} > 25$, use Chapter E		AISC 15th J4.4 (b)	
Elastic buckling stress	$F_e = \frac{\pi^2 E}{(KL/r)^2}$	= 93.17 [ksi]	AISC 15th Eq E3-4	
	when $\frac{KL}{r} \le 4.71 \left(\frac{E}{F_y}\right)^{0.5} = 113.43$		AISC 15th E3 (a)	
Critical stress	$F_{cr} = 0.658^{(F_y/F_e)} F_y$	= 39.94 [ksi]	AISC 15th Eq E3-2	
Plate compression required	P_u = 0.5 x beam axial compression P	= **156.4** [kips]		
Plate compression provided	$R_n = F_{cr} \times A_g$	= 284.6 [kips]	AISC 15th Eq E3-1	
Resistance factor-LRFD	ϕ = 0.90		AISC 15th E1	
	ϕR_n =	= **256.1** [kips]		
	ratio = **0.61**	> P_u OK		

Splice Plate - Shear Yielding		ratio = 19.9 / 427.5	= **0.05**	**PASS**
Splice Plate Shear Yielding				
Plate size	width b_p = 19.000 [in]	thickness t_p = 0.375 [in]		
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p \times 2$ side	= 14.250 [in^2]		
Shear force required	V_u =	= **19.9** [kips]		
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$	= 427.5 [kips]	AISC 15th Eq J4-3	
Resistance factor-LRFD	ϕ = 1.00		AISC 15th Eq J4-3	
	ϕR_n =	= **427.5** [kips]		
	ratio = **0.05**	> V_u OK		

Splice Plate - Flexural Yielding		ratio = 54.73 / 253.83	= **0.22**	**PASS**
Plate width & thick	width b_p = 19.000 [in]	thick t_p = 0.375 [in]		
	yield F_y = 50.0 [ksi]			
Beam end shear reaction caused by gravity load only	V = from user input	= 19.9 [kips]		
Ecc from column face to outer bolt group first bolt line	e =	= 33.000 [in]		
Flexural strength required	$M_r = V \times e$	= **54.73** [kip-ft]		
Shear plate - plastic modulus	$Z_p = (b_p \times t_p^2) / 4 \times 2$ side	= 67.69 [in^3]		
Flexural strength available	$M_c = \phi F_y Z_p$ ϕ=0.90	= 253.83 [kip-ft]		
	ratio = **0.22**	> M_r OK		

Splice Plate - Shear Rupture		ratio = 10.0 / 142.6	= **0.07**	**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 = 6				
Plate size	width b_p = 19.000 [in]	thickness t_p = 0.375		[in]	
Plate tensile strength	F_u = 65.0 [ksi]				
Plate net area in shear	$A_{nv} = (b_p - n\ d_h)\ t_p$		= 4.875	[in^2]	
Shear force required	V_u =		= **10.0**	[kips]	
Plate shear rupture strength	$R_n = 0.6\ F_u A_{nv}$		= 190.1	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J4-4
	$\phi\ R_n$ =		= **142.6**	[kips]	
	ratio = **0.07**		> V_u	OK	

Splice Plate - Flexural Rupture		ratio = 27.36 / 96.36	= **0.28**	**PASS**

Plate A_n and Z_{net} Calc

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 = 6				
Plate size	width b_p = 19.000 [in]	thickness t_p = 0.375		[in]	
Plate net area	$A_n = (b_p - n\ d_h)\ t_p$		= 4.875	[in^2]	
Plate net plastic sect modulus	Z_{net} =		= 23.72	[in^3]	
Plate net elastic sect modulus	S_{net} =		= 16.33	[in^3]	

Plate width & thick	width b_p = 19.000 [in]	thick t_p = 0.375		[in]
	F_u = 65.0 [ksi]			
Beam end shear reaction caused by gravity load only	V = from user input, x 0.5 for 2 side plate		= 10.0	[kips]
Ecc from column face to outer bolt group first bolt line	e =		= 33.000	[in]
Flexural strength required	M_r = V x e		= **27.36**	[kip-ft]
Shear plate - net plastic modulus	Z_{net} = from above calc		= 23.72	[in^3]
Flexural strength available	$M_c = \phi\ F_u Z_{net}$	ϕ=0.75	= **96.36**	[kip-ft]
	ratio = **0.28**		> M_r	OK