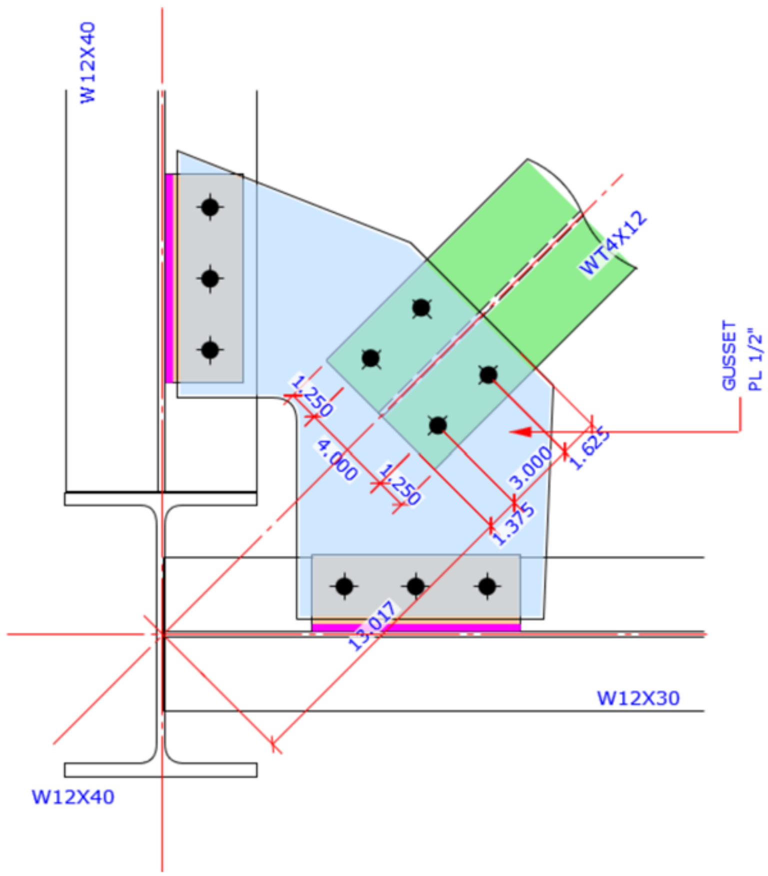
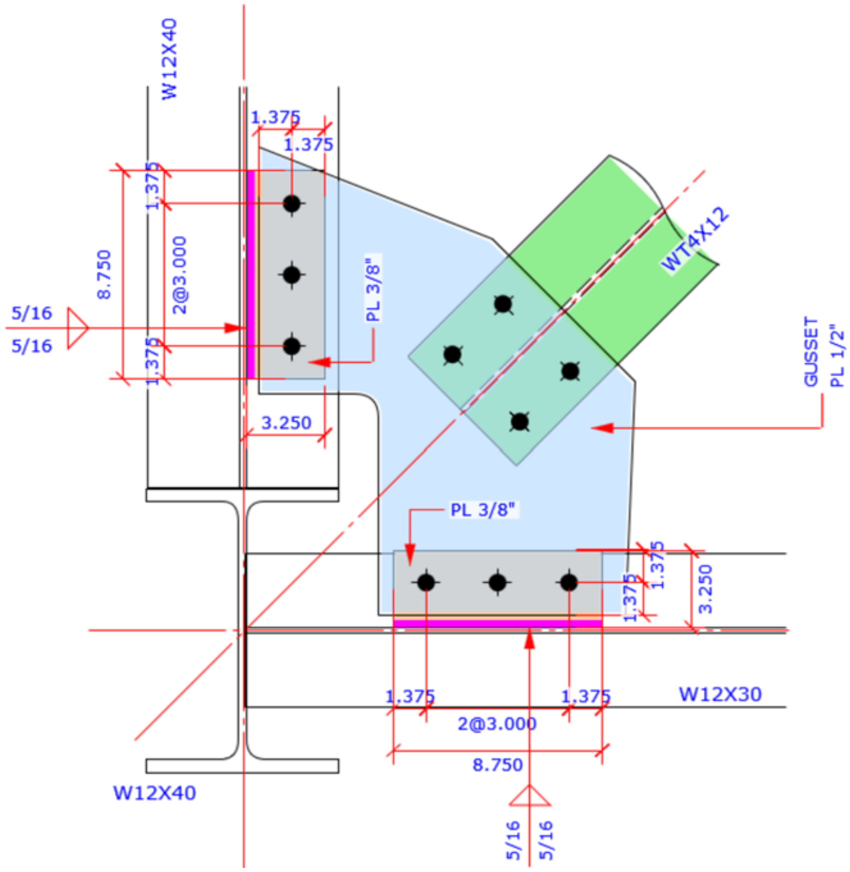
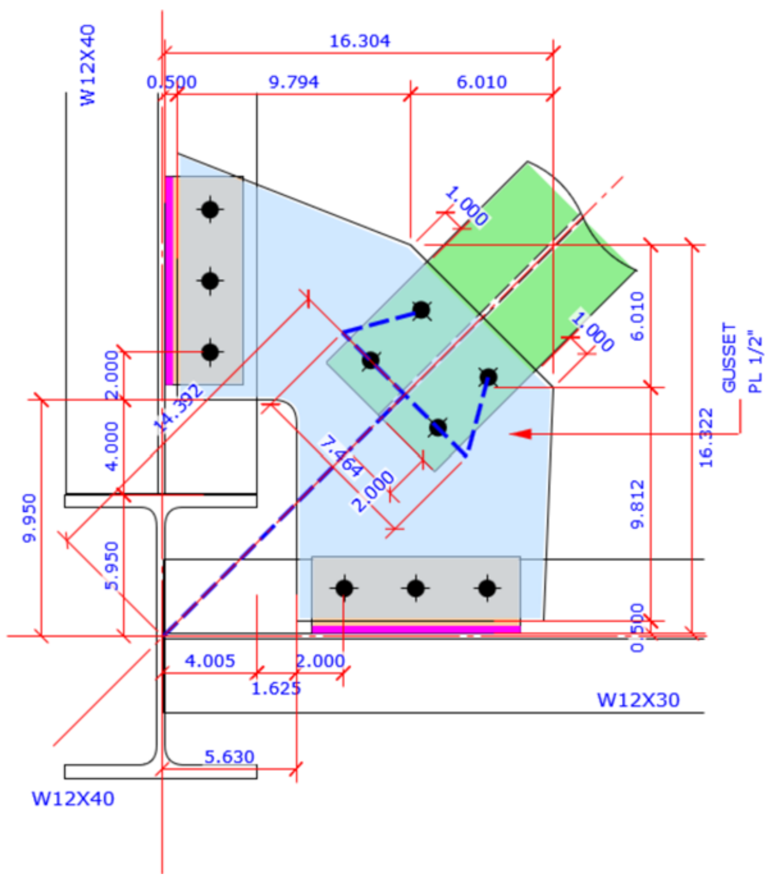
Result Summary - Overall	Horizontal Brace Connection	Code=AISC 360-10 LRFD

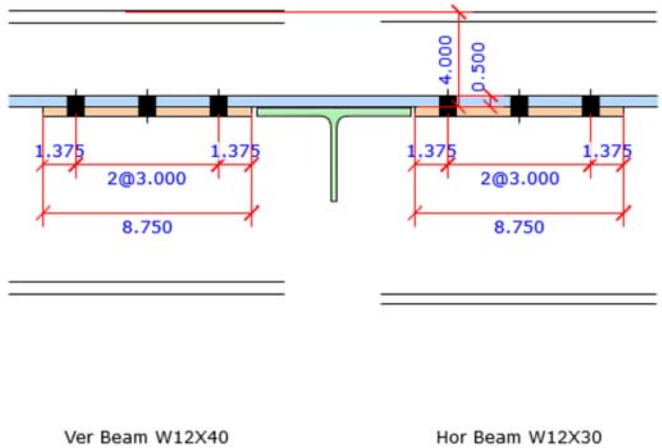
Result Summary - Overall	geometries & weld limitations = **PASS**	limit states max ratio = **0.86**	**PASS**
Brace to Gusset	geometries & weld limitations = **PASS**	limit states max ratio = **0.63**	**PASS**
Gusset to Ver Beam	geometries & weld limitations = **PASS**	limit states max ratio = **0.77**	**PASS**
Gusset to Hor Beam	geometries & weld limitations = **PASS**	limit states max ratio = **0.86**	**PASS**

Sketch	Horizontal Brace Connection	Code=AISC 360-10 LRFD




W12X40
1.375
1.375
1.375
8.750
2@3.000
5/16
5/16
1.375
PL 3/8"
WT4X12
GUSSET
PL 1/2"
3.250
PL 3/8"
1.375
1.375
3.250
1.375
1.375
W12X30
2@3.000
8.750
W12X40
5/16
5/16


W12X40
16.304
0.500
9.794
6.010
1.000
6.010
1.000
GUSSET
PL 1/2"
2.000
4.000
14.392
7.464
2.000
9.950
5.950
9.812
16.322
0.500
4.005
2.000
1.625
W12X30
5.630
W12X40


4.000
0.500
1.375
1.375
2@3.000
8.750
1.375
1.375
2@3.000
8.750
Ver Beam W12X40
Hor Beam W12X30

Members & Components Summary

Member	Brace Connection	Code=AISC 360-10 LRFD

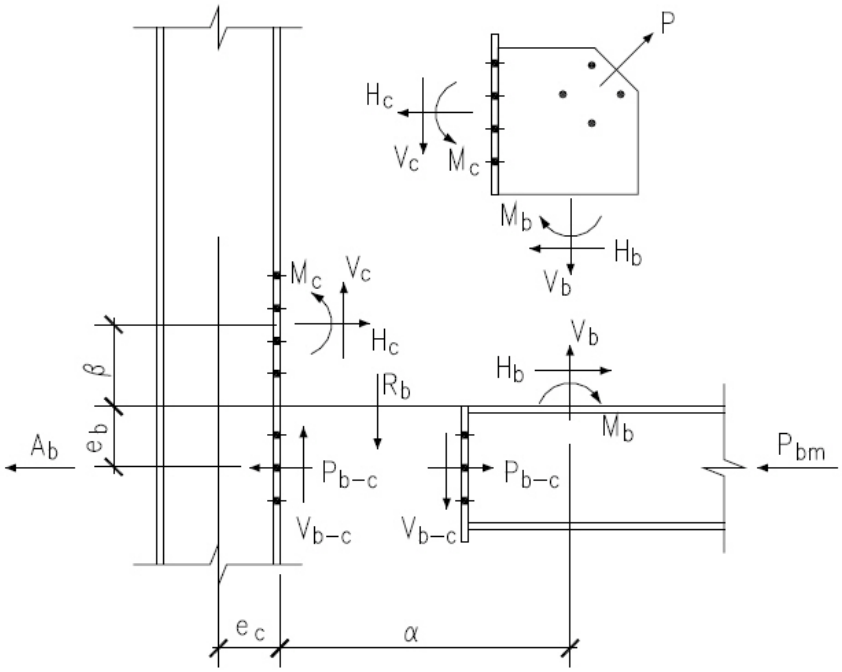
Hor Beam Section

W12X30	d = 12.300	[in]	b_f = 6.520	[in]
	t_f = 0.440	[in]	t_w = 0.260	[in]
	k_{des} = 0.740	[in]	k_{det} = 1.125	[in]
	k_1 = 0.750	[in]	A = 8.790	[in^2]
	S_x = 38.60	[in^3]	Z_x = 43.10	[in^3]
Steel Grade A992	F_y = 50.0	[ksi]	F_u = 65.0	[ksi]

Ver Beam Section

W12X40	d = 11.900	[in]	b_f = 8.010	[in]
	t_f = 0.515	[in]	t_w = 0.295	[in]
	k_{des} = 1.020	[in]	k_{det} = 1.375	[in]
	k_1 = 0.875	[in]	A = 11.700	[in^2]
	S_x = 51.50	[in^3]	Z_x = 57.00	[in^3]
Steel Grade A992	F_y = 50.0	[ksi]	F_u = 65.0	[ksi]

Gusset Plate Interface Forces Calculation



Brace Axial Force Load Case 1

Brace force P = -45.00 [kips] (T)

Refer to AISC 14th Page 13-4 and Fig. 13-2 for all charts and definitions of variables and symbols shown in calculation below

e_b = 0.130 [in]		e_c = 0.148 [in]		
α = 10.483 [in]		β = 14.820 [in]		
θ = 45.0 [°]				
$K = e_b \tan\theta - e_c$	= -0.017	[in]	AISC 14th Eq. 13-16	
$D = \tan^2\theta + (\frac{\alpha}{\beta})^2$	= 1.500		AISC 14th Eq. 13-24	
$K' = \alpha\ (\tan\theta + \frac{\alpha}{\beta})$	= 17.897		AISC 14th Eq. 13-23	
$\bar{\alpha} = [\ K' \tan\theta + K\ (\frac{\alpha}{\beta})^2\ ] / D$	= 11.923	[in]	AISC 14th Eq. 13-21	
$\bar{\beta} = (\ K' - K \tan\theta\ ) / D$	= 11.941	[in]	AISC 14th Eq. 13-22	
$r = [\ (e_b + \bar{\beta})^2 + (e_c + \bar{\alpha})^2\ ]^{0.5}$	= 17.070	[in]	AISC 14th Eq. 13-6	

Brace axial force	P_u = from user input	= -45.00	[kips]	in tension
Gusset to Ver Beam Interface Forces				
Shear force	$V_c = (\ \bar{\beta} / r\ )\ P_u$	= **-31.48**	[kips]	AISC 14th Eq. 13-2
Axial force	$H_c = (\ e_c / r\ )\ P_u$	= **-0.39**	[kips]	AISC 14th Eq. 13-3
Moment	$M_c = H_c (\ \beta - \bar{\beta}\ )$	= **0.09**	[kip-ft]	AISC 14th Eq. 13-19
Gusset to Hor Beam Interface Forces				
Shear force	$H_b = (\ \bar{\alpha} / r\ )\ P_u$	= **-31.43**	[kips]	AISC 14th Eq. 13-5
Axial force	$V_b = (\ e_b / r\ )\ P_u$	= **-0.34**	[kips]	AISC 14th Eq. 13-4
Moment	$M_b = V_b (\ \bar{\alpha} - \alpha\ )$	= **-0.04**	[kip-ft]	AISC 14th Eq. 13-17

Top Brace - Brace to Gusset	Sect=WT4X12	P_{LC1} =-45.00 kips (T)	P_{LC2} =45.00 kips (C)	Code=AISC 360-10 LRFD

Result Summary	geometries & weld limitations = **PASS**	limit states max ratio = **0.63**	**PASS**

Geometry Restriction Checks - WT Flange to Gusset					**PASS**
Min Bolt Edge Distance - WT Flange to Gusset					
Bolt diameter	d_b =	= 0.750	[in]		
Min edge distance allowed	$L_{e\text{-}min}$ =	= **1.000**	[in]	AISC 14th Table J3.4	
Min edge distance in WT Flange to Gusset	L_e =	= **1.250**	[in]		
		> $L_{e\text{-}min}$	OK		
Min Bolt Spacing - WT Flange to Gusset					
Bolt diameter	d_b =	= 0.750	[in]		
Min bolt spacing allowed	$L_{s\text{-}min}$ = 2.667 d_b	= **2.000**	[in]	AISC 14th J3.3	
Min Bolt spacing in WT Flange to Gusset	L_s =	= **3.000**	[in]		
		> $L_{s\text{-}min}$	OK		

Brace Force Load Case 1	Sect=WT4X12	P =-45.00 kips (T)	ratio = **0.63**	**PASS**

WT Shape Brace - Tensile Yield		ratio = 45.00 / 159.30 = **0.28**			**PASS**
Gross area subject to tension	A_g =	= 3.540	[in^2]		
Steel yield strength	F_y =	= 50.0	[ksi]		
Tensile force required	P_u =	= **45.00**	[kips]		
Tensile yielding strength	$R_n = F_y A_g$	= 177.00	[kips]	AISC 14th Eq D2-1	
Resistance factor-LRFD	ϕ = 0.90			AISC 14th D2 (a)	
	ϕR_n =	= **159.30**	[kips]	AISC 14th Eq D2-1	
	ratio = **0.28**	> P_u	OK		

WT Shape Brace - Tensile Rupture		ratio = 45.00 / 106.38	= **0.42**		**PASS**
Section gross area	A_g = WT4X12		= 3.540	[in^2]	
Bolt hole diameter	bolt dia d_b = $^3/_4$ [in]	bolt hole dia d_h =	$^7/_8$	[in]	AISC 14th B4.3b
Number of bolt row	n_v = 2	flange t_f =	0.400	[in]	
Tensile net area	$A_n = A_g - n_v d_h t_f$		= 2.840	[in^2]	
No of bolt column	n_h = 2	bolt space s_h =	3.000	[in]	
Length of connection	$L = (n_h -1) s_h$		= 3.000	[in]	
Eccentricity of connection	$\bar{x}$ = from sect WT4X12		= 0.695	[in]	
Shear lag factor	$U = 1 - \bar{x} / L$		= 0.768		AISC 14th Table D3.1
Tensile force required	P_u =		= **45.00**	[kips]	
Tensile effective net area	$A_e = A_n U$		= 2.182	[in^2]	
Plate tensile strength	F_u =		= 65.0	[ksi]	
Tensile rupture strength	$R_n = F_u A_e$		= 141.83	[kips]	AISC 14th Eq D2-2
Resistance factor-LRFD	ϕ = 0.75				AISC 14th D2 (b)
	ϕR_n =		= **106.38**	[kips]	AISC 14th Eq D2-2
	ratio = **0.42**		> P_u	OK	

WT Brace - Bolt Shear		ratio = 45.00 / 71.57	= **0.63**		**PASS**
Bolt shear stress	bolt grade = A325-N	F_{nv} =	54.0	[ksi]	AISC 14th Table J3.2
	bolt dia d_b = 0.750 [in]	bolt area A_b =	0.442	[in^2]	
Number of bolt carried shear	n_s = 4.0	shear plane m =	1		
Bolt group eccentricity coefficient	C_{ec} =		= 1.000		
Required shear strength	V_u =		= **45.00**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$		= 95.43	[kips]	AISC 14th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 14th Eq J3-1
	ϕR_n =		= **71.57**	[kips]	
	ratio = **0.63**		> V_u	OK	

WT Brace - Bolt Bearing on WT Flange		ratio = 45.00 / 71.57	= **0.63**	**PASS**	
Single Bolt Shear Strength					
Bolt shear stress	bolt grade = A325-N	F_{nv} = 54.0	[ksi]	AISC 14th Table J3.2	
	bolt dia d_b = 0.750 [in]	bolt area A_b = 0.442	[in^2]		
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$	= 23.86	[kips]	AISC 14th Eq J3-1	
Bolt Bearing / TearOut Strength on Plate					
Bolt hole diameter	bolt dia d_b = $^3/_4$ [in]	bolt hole dia d_h = $^{13}/_{16}$	[in]	AISC 14th Table J3.3	
Bolt spacing & edge distance	spacing L_s = 3.000 [in]	edge distance L_e = 1.375	[in]		
Plate tensile strength	F_u = 65.0 [ksi]				
Plate thickness	t = 0.400 [in]				
Interior Bolt					
Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.188	[in]		
Bolt tear out/bearing strength	$R_{n\text{-}t\&b\text{-}in} = 1.5\, L_c\, t\, F_u \le 3.0\, d_b\, t\, F_u$			AISC 14th Eq J3-6b	
	= 85.31 ≤ 58.50	= 58.50	[kips]		
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$	= 23.86	[kips]		
Edge Bolt					
Bolt hole edge clear distance	$L_c = L_e - d_h / 2$	= 0.969	[in]		
Bolt tear out/bearing strength	$R_{n\text{-}t\&b\text{-}ed} = 1.5\, L_c\, t\, F_u \le 3.0\, d_b\, t\, F_u$			AISC 14th Eq J3-6b	
	= 37.78 ≤ 58.50	= 37.78	[kips]		
Bolt strength at edge	$R_{n\text{-}ed} = \min(R_{n\text{-}t\&b\text{-}ed}, R_{n\text{-}bolt})$	= 23.86	[kips]		
Number of bolt	interior n_{in} = 2	edge n_{ed} = 2			
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-}in} + n_{ed} R_{n\text{-}ed}$	= 95.43	[kips]		
Required shear strength	V_u =	= **45.00**	[kips]		
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 14th J3-10	
	ϕR_n =	= **71.57**	[kips]		
	ratio = **0.63**	> V_u	OK		

WT Brace - Bolt Bearing on Gusset Plate		ratio = 45.00 / 71.57	= **0.63**	**PASS**	
Single Bolt Shear Strength					
Bolt shear stress	bolt grade = A325-N	F_{nv} = 54.0		[ksi]	AISC 14th Table J3.2
	bolt dia d_b = 0.750 [in]	bolt area A_b = 0.442		[in^2]	
Single bolt shear strength	$R_{n\text{-bolt}} = F_{nv} A_b$		= 23.86	[kips]	AISC 14th Eq J3-1
Bolt Bearing / TearOut Strength on Plate					
Bolt hole diameter	bolt dia $d_b = {}^3/_4$ [in]	bolt hole dia $d_h = {}^{13}/_{16}$		[in]	AISC 14th Table J3.3
Bolt spacing & edge distance	spacing L_s = 3.000 [in]	edge distance L_e = 1.625		[in]	
Plate tensile strength	F_u = 65.0 [ksi]				
Plate thickness	t = 0.500 [in]				
Interior Bolt					
Bolt hole edge clear distance	$L_c = L_s - d_h$		= 2.188	[in]	
Bolt tear out/bearing strength	$R_{n\text{-t\&b-in}} = 1.5\, L_c\, t\, F_u \le 3.0\, d_b\, t\, F_u$				AISC 14th Eq J3-6b
	= 106.64 ≤ 73.13		= 73.13	[kips]	
Bolt strength at interior	$R_{n\text{-in}} = \min(R_{n\text{-t\&b-in}}, R_{n\text{-bolt}})$		= 23.86	[kips]	
Edge Bolt					
Bolt hole edge clear distance	$L_c = L_e - d_h / 2$		= 1.219	[in]	
Bolt tear out/bearing strength	$R_{n\text{-t\&b-ed}} = 1.5\, L_c\, t\, F_u \le 3.0\, d_b\, t\, F_u$				AISC 14th Eq J3-6b
	= 59.41 ≤ 73.13		= 59.41	[kips]	
Bolt strength at edge	$R_{n\text{-ed}} = \min(R_{n\text{-t\&b-ed}}, R_{n\text{-bolt}})$		= 23.86	[kips]	
Number of bolt	interior n_{in} = 2	edge n_{ed} = 2			
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-in}} + n_{ed} R_{n\text{-ed}}$		= 95.43	[kips]	
Required shear strength	V_u =		= **45.00**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 14th J3-10
	ϕR_n =		= **71.57**	[kips]	
	ratio = **0.63**		> V_u	OK	

WT Brace Flange - Block Shear - 1-Side Strip		ratio = 22.50 / 51.68	= **0.44**	**PASS**
Plate Block Shear - Side Strip				
Bolt hole diameter	bolt dia d_b = $^3/_4$ [in]	bolt hole dia d_h = $^7/_8$ [in]		AISC 14th B4.3b
Plate thickness	t_p = 0.400 [in]			
Plate strength	F_y = 50.0 [ksi]	F_u = 65.0 [ksi]		
Bolt no in ver & hor dir	n_v = 1	n_h = 2		
Bolt spacing in hor dir	s_h = 3.000 [in]			
Bolt edge dist in ver & hor dir	e_v = 1.250 [in]	e_h = 1.375 [in]		
Gross area subject to shear	$A_{gv} = [(n_h - 1) s_h + e_h] t_p$	= 1.750 [in^2]		
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5] d_h t_p$	= 1.225 [in^2]		
Net area subject to tension	$A_{nt} = (e_v - 0.5 d_h) t_p$	= 0.325 [in^2]		
Block shear strength required	V_u =	= **22.50** [kips]		
Uniform tension stress factor	U_{bs} = 1.00			AISC 14th Fig C-J4.2
Bolt shear resistance provided	$R_n = \min(0.6F_u A_{nv}, 0.6F_y A_{gv}) + U_{bs} F_u A_{nt}$	= 68.90 [kips]		AISC 14th Eq J4-5
Resistance factor-LRFD	ϕ = 0.75			AISC 14th Eq J4-5
	ϕR_n =	= **51.68** [kips]		
	ratio = **0.44**	> V_u	OK	

Gusset Plate - Tensile Yield (Whitmore)		ratio = 45.00 / 167.94	= **0.27**	**PASS**
Plate Tensile Yielding Check				
Plate size	width b_p = 7.464 [in]	thickness t_p = 0.500 [in]		
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_g = b_p t_p$	= 3.732 [in^2]		
Tensile force required	P_u =	= **45.00** [kips]		
Plate tensile yielding strength	$R_n = F_y A_g$	= 186.60 [kips]		AISC 14th Eq J4-1
Resistance factor-LRFD	ϕ = 0.90			AISC 14th Eq J4-1
	ϕR_n =	= **167.94** [kips]		
	ratio = **0.27**	> P_u	OK	

Gusset Plate - Tensile Rupture (Whitmore)		ratio = 45.00 / 139.28	= **0.32**	**PASS**
Plate Tensile Rupture Check				
Bolt hole diameter	bolt dia d_b = $^3/_4$ [in]	bolt hole dia d_h = $^7/_8$ [in]		AISC 14th B4.3b
Number of bolt	n = 2			
Plate size	width b_p = 7.464 [in]	thickness t_p = 0.500 [in]		
Plate tensile strength	F_u = 65.0 [ksi]			
Plate net area in tension	$A_{nt} = (b_p - n d_h) t_p$	= 2.857 [in^2]		
Tensile force required	P_u =	= **45.00** [kips]		
Plate tensile rupture strength	$R_n = F_u A_{nt}$	= 185.71 [kips]		AISC 14th Eq J4-2
Resistance factor-LRFD	ϕ = 0.75			AISC 14th Eq J4-2
	ϕR_n =	= **139.28** [kips]		AISC 14th Eq J4-2
	ratio = **0.32**	> P_u	OK	

Gusset Plate - Block Shear - Center Strip			ratio = 45.00 / 173.06	= **0.26**	**PASS**
Plate Block Shear - Center Strip					
Bolt hole diameter	bolt dia $d_b = {}^3/_4$	[in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 14th B4.3b
Plate thickness	$t_p = 0.500$	[in]			
Plate strength	$F_y = 50.0$	[ksi]	$F_u = 65.0$	[ksi]	
Bolt no in ver & hor dir	$n_v = 2.0$		$n_h = 2$		
Bolt spacing in hor dir	$s_h = 3.000$	[in]	edge dist $e_h = 1.625$	[in]	
Width of block shear strip	$W_{bs} = 4.000$	[in]			
Gross area subject to shear	$A_{gv} = [\ (n_h - 1)\ s_h + e_h\ ]\ t_p \times 2$		= 4.625	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5]\ d_h\ t_p \times 2$		= 3.313	[in^2]	
Net area subject to tension					
when sheared out by center strip	$A_{nt} = [\ W_{bs} - (n_v - 1)\ d_h\ ]\ t_p$		= 1.563	[in^2]	
Block shear strength required	$V_u =$		= **45.00**	[kips]	
Uniform tension stress factor	$U_{bs} = 1.00$				AISC 14th Fig C-J4.2
Bolt shear resistance provided	$R_n = \min(0.6F_u A_{nv}, 0.6F_y A_{gv}) + U_{bs} F_u A_{nt}$		= 230.75	[kips]	AISC 14th Eq J4-5
Resistance factor-LRFD	$\phi = 0.75$				AISC 14th Eq J4-5
	$\phi R_n =$		= **173.06**	[kips]	
	ratio = **0.26**		$> V_u$	OK	

Brace Force Load Case 2 Sect=WT4X12 P =45.00 kips (C) ratio = **0.63** **PASS**

WT Brace - Bolt Shear			ratio = 45.00 / 71.57	= **0.63**	**PASS**
Bolt shear stress	bolt grade = A325-N		$F_{nv} = 54.0$	[ksi]	AISC 14th Table J3.2
	bolt dia $d_b = 0.750$	[in]	bolt area $A_b = 0.442$	[in^2]	
Number of bolt carried shear	$n_s = 4.0$		shear plane m = 1		
Bolt group eccentricity coefficient	$C_{ec} =$		= 1.000		
Required shear strength	$V_u =$		= **45.00**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$		= 95.43	[kips]	AISC 14th Eq J3-1
Bolt resistance factor-LRFD	$\phi = 0.75$				AISC 14th Eq J3-1
	$\phi R_n =$		= **71.57**	[kips]	
	ratio = **0.63**		$> V_u$	OK	

WT Brace - Bolt Bearing on WT Flange		ratio = 45.00 / 71.57	= **0.63**	**PASS**
Single Bolt Shear Strength				
Bolt shear stress	bolt grade = A325-N	F_{nv} = 54.0 [ksi]		AISC 14th Table J3.2
	bolt dia d_b = 0.750 [in]	bolt area A_b = 0.442 [in^2]		
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$	= 23.86 [kips]		AISC 14th Eq J3-1
Bolt Bearing / TearOut Strength on Plate				
Bolt hole diameter	bolt dia d_b = $^3/_4$ [in]	bolt hole dia d_h = $^{13}/_{16}$ [in]		AISC 14th Table J3.3
Bolt spacing	spacing L_s = 3.000 [in]			
Plate tensile strength	F_u = 65.0 [ksi]			
Plate thickness	t = 0.400 [in]			
Interior Bolt				
Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.188 [in]		
Bolt tear out/bearing strength	$R_{n\text{-}t\&b\text{-}in} = 1.5\, L_c\, t\, F_u \le 3.0\, d_b\, t\, m\, F_u$			AISC 14th Eq J3-6b
	= 85.31 ≤ 58.50	= 58.50 [kips]		
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$	= 23.86 [kips]		
Number of bolt	interior n_{in} = 4			
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-}in}$	= 95.43 [kips]		
Required shear strength	V_u =	= **45.00** [kips]		
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 14th J3-10
	ϕR_n =	= **71.57** [kips]		
	ratio = **0.63**	> V_u	OK	

WT Brace - Bolt Bearing on Gusset Plate		ratio = 45.00 / 71.57	= **0.63**	**PASS**	
Single Bolt Shear Strength					
Bolt shear stress	bolt grade = A325-N	F_{nv} = 54.0	[ksi]		AISC 14th Table J3.2
	bolt dia d_b = 0.750 [in]	bolt area A_b = 0.442	[in^2]		
Single bolt shear strength	$R_{n\text{-bolt}} = F_{nv} A_b$	= 23.86	[kips]		AISC 14th Eq J3-1
Bolt Bearing / TearOut Strength on Plate					
Bolt hole diameter	bolt dia $d_b = {}^3/_4$ [in]	bolt hole dia $d_h = {}^{13}/_{16}$	[in]		AISC 14th Table J3.3
Bolt spacing	spacing L_s = 3.000 [in]				
Plate tensile strength	F_u = 65.0 [ksi]				
Plate thickness	t = 0.500 [in]				
Interior Bolt					
Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.188	[in]		
Bolt tear out/bearing strength	$R_{n\text{-t\&b-in}} = 1.5\ L_c\ t\ F_u \le 3.0\ d_b\ t\ m\ F_u$				AISC 14th Eq J3-6b
	= 106.64 ≤ 73.13	= 73.13	[kips]		
Bolt strength at interior	$R_{n\text{-in}} = \min(R_{n\text{-t\&b-in}}, R_{n\text{-bolt}})$	= 23.86	[kips]		
Number of bolt	interior n_{in} = 4				
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-in}}$	= 95.43	[kips]		
Required shear strength	V_u =	= **45.00**	[kips]		
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 14th J3-10
	ϕR_n =	= **71.57**	[kips]		
	ratio = **0.63**	> V_u	OK		

Gusset Plate - Compression (Whitmore)		ratio = 45.00 / 140.03	= **0.32**	**PASS**

Plate Compression Check

Plate size	width b_p = 7.464 [in]	thickness t_p = 0.500	[in]	
	F_y = 50.0 [ksi]	E = 29000	[ksi]	
Plate gross area in compression	$A_g = b_p t_p$	= 3.732	[in^2]	
Plate radius of gyration	$r = t_p / \sqrt{12}$	= 0.144	[in]	
Plate effective length factor	K =	= 0.50		
Plate unbraced length	L_u =	= 14.392	[in]	
Plate slenderness	KL/r = 0.50 x L_u / r	= 49.86		
	when $\frac{KL}{r} > 25$, use Chapter E			AISC 14th J4.4 (b)
Elastic buckling stress	$F_e = \frac{\pi^2 E}{(KL/r)^2}$	= 115.15	[ksi]	AISC 14th Eq E3-4
	when $\frac{KL}{r} \leq 4.71 \left(\frac{E}{F_y}\right)^{0.5}$ = 113.43			AISC 14th E3 (a)
Critical stress	$F_{cr} = 0.658^{(F_y/F_e)} F_y$	= 41.69	[ksi]	AISC 14th Eq E3-2
Plate compression required	P_u =	= **45.00**	[kips]	
Plate compression provided	$R_n = F_{cr} \times A_g$	= 155.59	[kips]	AISC 14th Eq E3-1
Bolt resistance factor-LRFD	ϕ = 0.90			AISC 14th E1
	ϕR_n =	= **140.03**	[kips]	
	ratio = **0.32**	> P_u	OK	

Gusset Plate - Block Shear - Center Strip		ratio = 45.00 / 173.06	= **0.26**	**PASS**

Plate Block Shear - Center Strip

Bolt hole diameter	bolt dia d_b = 3/4 [in]	bolt hole dia d_h = 7/8	[in]	AISC 14th B4.3b
Plate thickness	t_p = 0.500 [in]			
Plate strength	F_y = 50.0 [ksi]	F_u = 65.0	[ksi]	
Bolt no in ver & hor dir	n_v = 2.0	n_h = 2		
Bolt spacing in hor dir	s_h = 3.000 [in]	edge dist e_h = 1.625	[in]	
Width of block shear strip	W_{bs} = 4.000 [in]			
Gross area subject to shear	$A_{gv} = [(n_h - 1) s_h + e_h] t_p \times 2$	= 4.625	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5] d_h t_p \times 2$	= 3.313	[in^2]	
Net area subject to tension when sheared out by center strip	$A_{nt} = [W_{bs} - (n_v - 1) d_h] t_p$	= 1.563	[in^2]	
Block shear strength required	V_u =	= **45.00**	[kips]	
Uniform tension stress factor	U_{bs} = 1.00			AISC 14th Fig C-J4.2
Bolt shear resistance provided	$R_n = \min(0.6F_u A_{nv}, 0.6F_y A_{gv}) + U_{bs} F_u A_{nt}$	= 230.75	[kips]	AISC 14th Eq J4-5
Resistance factor-LRFD	ϕ = 0.75			AISC 14th Eq J4-5
	ϕR_n =	= **173.06**	[kips]	
	ratio = **0.26**	> V_u	OK	

Gusset to Ver Beam	Shear Tab Connection	Code=AISC 360-10 LRFD

Result Summary geometries & weld limitations = **PASS** limit states max ratio = **0.77** **PASS**

Geometry Restriction Checks - Shear Tab to Ver Beam Web — PASS

Min Bolt Edge Distance - Shear Tab to Ver Beam Web

Bolt diameter	d_b =	= 0.750	[in]	
Min edge distance allowed	$L_{e\text{-}min}$ =	= **1.000**	[in]	AISC 14th Table J3.4
Min edge distance in Shear Tab to Ver Beam Web	L_e =	= **1.375**	[in]	
		> $L_{e\text{-}min}$	OK	

Min Bolt Spacing - Shear Tab to Ver Beam Web

Bolt diameter	d_b =	= 0.750	[in]	
Min bolt spacing allowed	$L_{s\text{-}min} = 2.667\ d_b$	= **2.000**	[in]	AISC 14th J3.3
Min Bolt spacing in Shear Tab to Ver Beam Web	L_s =	= **3.000**	[in]	
		> $L_{s\text{-}min}$	OK	

Weld Limitation Check - Shear Tab Weld — PASS

Min Fillet Weld Size

Thinner part joined thickness	t =	= 0.295	[in]	
Min fillet weld size allowed	w_{min} =	= **0.188**	[in]	AISC 14th 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 14th J2.2b
Min fillet weld length	L =	= **8.750**	[in]	
		> L_{min}	OK	

Brace Force Load Case 1 Gusset plate t=0.500 P =-45.00 kips (T) ratio = **0.77** **PASS**

Gusset Plate - Shear Yielding — ratio = 31.48 / 148.13 = **0.21** PASS

Plate Shear Yielding Check

Plate size	width b_p = 9.875 [in]	thickness t_p = 0.500	[in]	
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$	= 4.938	[in^2]	
Shear force required	V_u =	= **31.48**	[kips]	
Plate shear yielding strength	$R_n = 0.6\ F_y A_{gv}$	= 148.13	[kips]	AISC 14th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 14th Eq J4-3
	$\phi\ R_n$ =	= **148.13**	[kips]	
	ratio = **0.21**	> V_u	OK	

Gusset Plate - Shear Rupture			ratio = 31.48 / 106.03	= **0.30**	**PASS**
Plate Shear Rupture Check					
Bolt hole diameter	bolt dia $d_b = {}^3/_4$	[in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 14th B4.3b
Number of bolt	n = 3				
Plate size	width b_p = 9.875	[in]	thickness t_p = 0.500	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate net area in shear	$A_{nv} = (b_p - n\ d_h)\ t_p$		= 3.625	[in^2]	
Shear force required	V_u =		= **31.48**	[kips]	
Plate shear rupture strength	$R_n = 0.6\ F_u A_{nv}$		= 141.38	[kips]	AISC 14th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75				AISC 14th Eq J4-4
	$\phi\ R_n$ =		= **106.03**	[kips]	
	ratio = **0.30**		> V_u	OK	

Gusset Plate Leg - Flexural Yielding			ratio = 14.77 / 30.47	= **0.48**	**PASS**

Refer to Bo Dowswell's paper 'Design of Wrap-Around Steel Gusset Plates' for more details on this limit state check

Shear on guseet leg & moment arm	shear V = 31.48	[kips]	ecc e = 5.630	[in]
Moment on gusset plate leg	M_u = V e		= **14.77**	[kip-ft]
Gusset plate leg size	width d = 9.875	[in]	thick t = 0.500	[in]
Gusset plate steel strength	F_y = 50.0	[ksi]		
Moment on gusset plate leg	$R_n = F_y (t\ d^2 / 6)$		= 33.86	[kip-ft]
Resistance factor-LRFD	ϕ = 0.90			
	$\phi\ R_n$ =		= **30.47**	[kips]
	ratio = **0.48**		> M_u	OK

Gusset Plate Leg - Lateral Torsional Buckling			ratio = 14.77 / 147.54	= **0.10**	**PASS**

Refer to Bo Dowswell's paper 'Design of Wrap-Around Steel Gusset Plates' for more details on this limit state check

Shear on guseet leg & moment arm	shear P = 31.48	[kips]	ecc e = 5.630	[in]	
Moment on gusset plate leg	M_u = P e		= **14.77**	[kip-ft]	
Gusset plate leg size	width d = 9.875	[in]	thick t = 0.500	[in]	
Gusset plate steel strength	E = 29000	[ksi]	G = 11200	[ksi]	
	F_y = 50.0	[ksi]			
Gusset leg buckiling length	L = distance from gusset load CG to gusset-beam interface line		= 10.630	[in]	
Critical moment - gusset leg	$R_n = 0.94 \sqrt{E\ G}\ \frac{d\ t^3}{L}$		= 163.93	[kip-ft]	Dowswell Paper Eq 9
Resistance factor-LRFD	ϕ = 0.90				
	$\phi\ R_n$ =		= **147.54**	[kip-ft]	
	ratio = **0.10**		> M_u	OK	

Shear Tab - Shear Yielding			ratio = 31.48 / 98.44	= **0.32**	**PASS**
Applied shear/axial forces	shear V = 31.48	[kips]	axial P = -0.39	[kips]	
Resultant shear force	$V_u = (V^2 + P^2)^{0.5}$		= **31.48**	[kips]	
Plate Shear Yielding Check					
Plate size	width b_p = 8.750	[in]	thickness t_p = 0.375	[in]	
Plate yield strength	F_y = 50.0	[ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$		= 3.281	[in^2]	
Shear force required	V_u =		= **31.48**	[kips]	
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$		= 98.44	[kips]	AISC 14th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00				AISC 14th Eq J4-3
	ϕR_n =		= **98.44**	[kips]	
	ratio = **0.32**		> V_u	OK	

Shear Tab - Shear Rupture			ratio = 31.48 / 67.18	= **0.47**	**PASS**
Applied shear/axial forces	shear V = 31.48	[kips]	axial P = -0.39	[kips]	
Resultant shear force	$V_u = (V^2 + P^2)^{0.5}$		= **31.48**	[kips]	
Plate Shear Rupture Check					
Bolt hole diameter	bolt dia $d_b = {}^3/_4$	[in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 14th B4.3b
Number of bolt	n = 3				
Plate size	width b_p = 8.750	[in]	thickness t_p = 0.375	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate net area in shear	$A_{nv} = (b_p - n\, d_h)\, t_p$		= 2.297	[in^2]	
Shear force required	V_u =		= **31.48**	[kips]	
Plate shear rupture strength	$R_n = 0.6 F_u A_{nv}$		= 89.58	[kips]	AISC 14th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75				AISC 14th Eq J4-4
	ϕR_n =		= **67.18**	[kips]	
	ratio = **0.47**		> V_u	OK	

Shear Tab - Axial Tensile Yield			ratio = 0.39 / 147.66	= **0.00**	**PASS**
Plate Tensile Yielding Check					
Plate size	width b_p = 8.750	[in]	thickness t_p = 0.375	[in]	
Plate yield strength	F_y = 50.0	[ksi]			
Plate gross area in shear	$A_g = b_p t_p$		= 3.281	[in^2]	
Tensile force required	P_u =		= **0.39**	[kips]	
Plate tensile yielding strength	$R_n = F_y A_g$		= 164.06	[kips]	AISC 14th Eq J4-1
Resistance factor-LRFD	ϕ = 0.90				AISC 14th Eq J4-1
	ϕR_n =		= **147.66**	[kips]	
	ratio = **0.00**		> P_u	OK	

Shear Tab - Axial Tensile Rupture			ratio = 0.39 / 111.97	= **0.00**	**PASS**
Plate Tensile Rupture Check					
Bolt hole diameter	bolt dia $d_b = 3/4$	[in]	bolt hole dia $d_h = 7/8$	[in]	AISC 14th B4.3b
Number of bolt	$n = 3$				
Plate size	width $b_p = 8.750$	[in]	thickness $t_p = 0.375$	[in]	
Plate tensile strength	$F_u = 65.0$	[ksi]			
Plate net area in tension	$A_{nt} = (b_p - n\ d_h)\ t_p$		= 2.297	[in^2]	
Tensile force required	$P_u =$		= **0.39**	[kips]	
Plate tensile rupture strength	$R_n = F_u A_{nt}$		= 149.30	[kips]	AISC 14th Eq J4-2
Resistance factor-LRFD	$\phi = 0.75$				AISC 14th Eq J4-2
	$\phi R_n =$		= **111.97**	[kips]	AISC 14th Eq J4-2
	ratio = **0.00**		> P_u	OK	

Shear Tab - Flexural Yield Interact			ratio =	= **0.14**	**PASS**
Plate width & thick	width $b_p = 8.750$	[in]	thick $t_p = 0.375$	[in]	
	yield $F_y = 50.0$	[ksi]			
Shear plate - gross area	$A_g = b_p \times t_p$		= 3.281	[in^2]	
Shear plate - plastic modulus	$Z_p = (b_p \times t_p^2) / 4$		= 7.178	[in^3]	
Axial strength available	P_c = from axial tensile yield check		= 147.66	[kips]	
Axial strength required	P_r = from gusset interface forces calc		= 0.39	[kips]	
Shear strength available	V_c = from shear yielding check		= 98.44	[kips]	
Shear strength required	V_r = from gusset interface forces calc		= 31.48	[kips]	
Flexural strength available	$M_c = \phi F_y Z_p \quad \phi=0.90$		= 26.92	[kip-ft]	
Flexural strength required	M_r = from gusset interface forces calc		= 4.92	[kip-ft]	
Flexural yield interaction	ratio $= \left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right)^2$		= **0.14**		AISC 14th Eq 10-5
			< 1.0	OK	

Shear Tab - Flexural Rupture Interact	ratio =	= **0.28**	**PASS**

Plate A_n and Z_{net} Calc

Bolt hole diameter	bolt dia $d_b = {}^{3}/_{4}$	[in]	bolt hole dia $d_h = {}^{7}/_{8}$		[in]	AISC 14th B4.3b
Number of bolt	$n = 3$					
Plate size	width $b_p = 8.750$	[in]	thickness $t_p =$	0.375	[in]	
Plate net area	$A_n = (b_p - n\ d_h)\ t_p$			= 2.297	[in^2]	
Plate net plastic sect modulus	$Z_{net} =$			= 5.137	[in^3]	
Plate net elastic sect modulus	$S_{net} =$			= 3.421	[in^3]	

Plate width & thick	width $b_p = 8.750$	[in]	thick $t_p =$	0.375	[in]	
	tensile $F_u = 65.0$	[ksi]				
Axial strength available	P_c = from axial tensile rupture check			= 111.97	[kips]	
Axial strength required	P_r = from gusset interface forces calc			= 0.39	[kips]	
Shear strength available	V_c = from shear rupture check			= 67.18	[kips]	
Shear strength required	V_r = from gusset interface forces calc			= 31.48	[kips]	
Flexural strength available	$M_c = \phi\ F_u\ Z_{net}$ ϕ=0.75			= 20.87	[kip-ft]	AISC 14th Eq 9-4
Flexural strength required	M_r = from gusset interface forces calc			= 4.92	[kip-ft]	
Flexural rupture interaction	$\text{ratio} = \left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right)^2$			= **0.28**		AISC 14th Eq 10-5
				< 1.0	OK	

Shear Tab - Bolt Bearing on Shear Tab		ratio = 31.48 / 53.68	= **0.59**	**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 = 31.48 [kips]	axial P = -0.39	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= **31.48**	[kips]	
Resultant force/hor line load angle	$\theta = \tan^{-1} (V / P)$	= 89.29	[°]	
Bolt hole diameter	bolt dia d_b = 0.750 [in]	bolt hole dia d_{bh} = 0.813	[in]	AISC 14[th] B4.3b
Bolt hole ver. dimension	d_v =	= 0.813	[in]	
Bolt hole hor. dimension	d_h =	= 0.813	[in]	
Bolt center to bolt hole edge dist	$d_c = 0.5\, d_{bh}$	= 0.406	[in]	
Bolt no in ver & hor direction	Bolt Row n_v = 3	Bolt Col n_h = 1		
Bolt spacing	ver s_v = 3.000 [in]			
Bolt edge distance	ver e_v = 1.375 [in]	hor e_h = 1.375	[in]	
Bolt clear dist - bot right corner bolt	$L_{cA} = \min\left(\dfrac{e_v}{\sin\theta}, \dfrac{e_h}{\cos\theta} \right) - d_c$	= 0.969	[in]	
Bolt clear dist - right side edge bolt	$L_{cB} = \min\left(\dfrac{s_v - 0.5 d_v}{\sin\theta}, \dfrac{e_h}{\cos\theta} \right) - d_c$	= 2.188	[in]	

Single Bolt Shear Strength

Bolt shear stress	bolt grade = A325-N	F_{nv} = 54.0	[ksi]	AISC 14[th] Table J3.2
	bolt dia d_b = 0.750 [in]	bolt area A_b = 0.442	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$	= 23.86	[kips]	AISC 14[th] Eq J3-1
Bolt bearing on plate	thick t = 0.375 [in]	tensile F_u = 65.0	[ksi]	
Bolt bearing strength	$R_{n\text{-}br} = 3.0\, d_b\, t\, F_u$	= 54.84	[kips]	AISC 14[th] Eq J3-6b
Type A - Bolt Group Bottom Right Corner Bolt				
Number of bolt	n_A = 1			
Bolt tear out strength	$R_{n\text{-}tA} = 1.5\, L_{cA}\, t\, F_u$	= 35.42	[kips]	AISC 14[th] Eq J3-6b
Bolt bearing strength	$R_{nA} = \min(R_{n\text{-}tA}, R_{n\text{-}br}, R_{n\text{-}bolt})$	= **23.86**	[kips]	
Type B - Bolt Group Right Side Edge Bolt				
Number of bolt	n_B = 2			
Bolt tear out strength	$R_{n\text{-}tB} = 1.5\, L_{cB}\, t\, F_u$	= 79.99	[kips]	AISC 14[th] Eq J3-6b
Bolt bearing strength	$R_{nB} = \min(R_{n\text{-}tB}, R_{n\text{-}br}, R_{n\text{-}bolt})$	= **23.86**	[kips]	
Bolt bearing strength for all bolts	$R_n = n_A R_{nA} + n_B R_{nB} + n_C R_{nC} + n_D R_{nD}$	= 71.57	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 14[th] J3-10
	ϕR_n =	= **53.68**	[kips]	
	ratio = **0.59**	> R	OK	

Shear Tab - Beam Side - Block Shear - 1-Side Strip	ratio = 31.48 / 74.04	= **0.43**	**PASS**			
Plate Block Shear - Side Strip						
Bolt hole diameter	bolt dia $d_b = {}^3/_4$	[in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 14th B4.3b	
Plate thickness	$t_p = 0.375$	[in]				
Plate strength	$F_y = 50.0$	[ksi]	$F_u = 65.0$	[ksi]		
Bolt no in ver & hor dir	$n_v = 1$		$n_h = 3$			
Bolt spacing in hor dir	$s_h = 3.000$	[in]				
Bolt edge dist in ver & hor dir	$e_v = 1.375$	[in]	$e_h = 1.375$	[in]		
Gross area subject to shear	$A_{gv} = [\ (n_h - 1)\ s_h + e_h\ ]\ t_p$		= 2.766	[in^2]		
Net area subject to shear	$A_{nv} = A_{gv} - [\ (n_h - 1) + 0.5\ ]\ d_h\ t_p$		= 1.945	[in^2]		
Net area subject to tension	$A_{nt} = (\ e_v - 0.5\ d_h\ )\ t_p$		= 0.352	[in^2]		
Block shear strength required	$V_u =$		= **31.48**	[kips]		
Uniform tension stress factor	$U_{bs} = 1.00$				AISC 14th Fig C-J4.2	
Bolt shear resistance provided	$R_n = \min\ (0.6F_u A_{nv}\ ,\ 0.6F_y A_{gv}\ ) + U_{bs} F_u A_{nt}$		= 98.72	[kips]	AISC 14th Eq J4-5	
Resistance factor-LRFD	$\phi = 0.75$				AISC 14th Eq J4-5	
	$\phi\ R_n =$		= **74.04**	[kips]		
	ratio = **0.43**		$> V_u$	OK		

Shear Tab - Beam Side-Axial Tearout - Block Shear - Center Strip	ratio = 0.39 / 98.26	= **0.00**	**PASS**			
Plate Block Shear - Center Strip						
Bolt hole diameter	bolt dia $d_b = {}^3/_4$	[in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 14th B4.3b	
Plate thickness	$t_p = 0.375$	[in]				
Plate strength	$F_y = 50.0$	[ksi]	$F_u = 65.0$	[ksi]		
Bolt no in ver & hor dir	$n_v = 3$		$n_h = 1$			
Bolt spacing in ver & hor dir	$s_v = 3.000$	[in]	$s_h = 3.000$	[in]		
Bolt edge dist in ver & hor dir	$e_v = 1.375$	[in]	$e_h = 1.375$	[in]		
Gross area subject to shear	$A_{gv} = [\ (n_h - 1)\ s_h + e_h\ ]\ t_p \times 2$		= 1.031	[in^2]		
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5]\ d_h\ t_p \times 2$		= 0.703	[in^2]		
Net area subject to tension when sheared out by center strip	$A_{nt} = (\ n_v - 1)\ (\ s_v - d_h\ )\ t_p$		= 1.594	[in^2]		
Block shear strength required	$V_u =$		= **0.39**	[kips]		
Uniform tension stress factor	$U_{bs} = 1.00$				AISC 14th Fig C-J4.2	
Bolt shear resistance provided	$R_n = \min\ (0.6F_u A_{nv}\ ,\ 0.6F_y A_{gv}\ ) + U_{bs} F_u A_{nt}$		= 131.02	[kips]	AISC 14th Eq J4-5	
Resistance factor-LRFD	$\phi = 0.75$				AISC 14th Eq J4-5	
	$\phi\ R_n =$		= **98.26**	[kips]		
	ratio = **0.00**		$> V_u$	OK		

Shear Tab - Block Shear - Shear / Tensile Interact	ratio =		= **0.18**	**PASS**
Shear block shear strength required	V_u =	= 31.48	[kips]	
Axial block shear strength required	P_u =	= 0.39	[kips]	
Shear block shear strength available	ϕR_{nv} = from calc shown above	= 74.04	[kips]	
Axial block shear strength available	ϕR_{nt} = from calc shown above	= 98.26	[kips]	
Block shear shear/tensile interaction	ratio = $\left(\frac{V_u}{\phi R_{nv}} \right)^2 + \left(\frac{P_u}{\phi R_{nt}} \right)^2$	= **0.18**		AISC 14th Eq 10-5
		< 1.0	OK	

Shear Tab - Lateral Stability / Stabilizer Plate		ratio = 31.48 / 556.65	= **0.06**	**PASS**
Applied shear/axial forces	shear V = 31.48 [kips]	axial P = -0.39	[kips]	
Resultant shear force	$V_u = (V^2 + P^2)^{0.5}$	= **31.48**	[kips]	
Distance from support to the first line of bolts	a =	= 1.875	[in]	
Plate thickness & depth	t_p = 0.375 [in]	L = 8.750	[in]	
Shear resistance provided	$R_n = 1500\,\pi \frac{L\,t_p^3}{a^2}$	= 618.50	[kips]	AISC 14th Eq 10-6
Resistance factor-LRFD	ϕ = 0.90			AISC 14th Eq 10-6
	ϕR_n =	= **556.65**	[kips]	
	ratio = **0.06**	> V_u	OK	

Shear Tab - Plate Flexural Buckling		ratio = 31.48 / 82.10	= **0.38**	**PASS**
Shear tab size	depth = 8.750 [in]	thick = 0.375	[in]	
Plate buckling model	c = dist from support to fisrt bolt line	= 1.875	[in]	AISC 14th Fig. 9-3
	h_0 = shear tab depth	= 8.750	[in]	
	t_w = shear tab thick	= 0.375	[in]	
Shear tab steel yield stress	F_y = 50.0 [ksi]			
Plate buckling factor	$\lambda = \frac{h_0 \sqrt{F_y}}{10\, t_w \sqrt{475 + 280\,(h_0 / c)^2}}$	= 0.204		AISC 14th Eq 9-18
Plate buckling factor	Q =	= 1.000		AISC 14th Eq 9-15
Plate critical buckling stress	$F_{cr} = Q\,F_y$	= 50.0	[ksi]	AISC 14th Eq 9-14
Shear force in demand	V_u =	= **31.48**	[kips]	
Shear tab net elastic modulus	S_{net} =	= 3.421	[in^3]	
Shear force to bolt group CG ecc	a =	= 1.875	[in]	
Shear resistance	$R_n = F_{cr}\,S_{net} / a$	= 91.22	[kips]	AISC 14th Eq 9-19
Resistance factor-LRFD	ϕ = 0.90			AISC 14th Eq 9-19
	ϕR_n =	= **82.10**	[kips]	
	ratio = **0.38**	> V_u	OK	

Bolt Group Eccentricity

Bolt group forces	shear V = 31.48	[kips]	axial P = 0.39	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$		= 31.48	[kips]	
Resultant force to ver Y axis angle	$\theta = \tan^{-1} (P / V)$		= 0.71	[°]	
Bolt group row and column	bolt row n_r = 3		bolt col n_c = 1		
Bolt row spacing	bolt row s_r = 3.000	[in]			
Shear force to bolt group CG ecc	e_x =		= 1.875	[in]	
Shear force to ver Y axis angle	θ =		= 0.71	[°]	
Bolt group coefficient C	C = from AISC 14th Table 7-6 ~ 7-13		= 2.288		
Bolt group eccentricity coefficient	$C_{ec} = C / (n_r \times n_c)$		= **0.763**		

Shear Tab / Beam Web - Bolt Shear

ratio = 31.48 / 40.96 = **0.77** **PASS**

Bolt group forces	shear V = 31.48	[kips]	axial P = -0.39	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$		= **31.48**	[kips]	
Bolt shear stress	bolt grade = A325-N		F_{nv} = 54.0	[ksi]	AISC 14th Table J3.2
	bolt dia d_b = 0.750	[in]	bolt area A_b = 0.442	[in^2]	
Number of bolt carried shear	n_s = 3.0		shear plane m = 1		
Bolt group eccentricity coefficient	C_{ec} = from 'Bolt Group Eccentricity' calc		= 0.763		
Required shear strength	V_u =		= **31.48**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$		= 54.61	[kips]	AISC 14th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 14th Eq J3-1
	ϕR_n =		= **40.96**	[kips]	
	ratio = **0.77**		> V_u	OK	

Shear Tab to Ver Beam Web Weld Strength			ratio = 5.90 / 10.97	= **0.54**	**PASS**
Weld Group Forces					
	Shear V = 31.48 [kips]		Axial P = -0.39	[kips]	in tension
Shear force to bolt group CG ecc	e_x =		= 1.875	[in]	
Moment due to eccentric shear V	$M = V \times e_x$		= 4.92	[kip-ft]	
Shear tab weld length	L =		= 8.750	[in]	
Combined Weld Stress					
Weld stress from axial force	$f_a = P / L$		= -0.045	[kip/in]	in tension
Weld stress from shear force	$f_v = V / L$		= 3.598	[kip/in]	
Weld stress from moment force	$f_b = \dfrac{M}{L^2 / 6}$		= 4.626	[kip/in]	
Weld stress combined - max	$f_{max} = [\ (f_a - f_b)^2 + f_v^2\ ]^{0.5}$		= **5.895**	[kip/in]	AISC 14th Eq 8-11
Weld stress load angle	$\theta = \tan^{-1} (\dfrac{f_a - f_b}{f_v})$		= 52.4	[°]	
Fillet Weld Strength Calc					
Fillet weld leg size	$w = {}^5/_{16}$ [in]	load angle θ	= 52.4	[°]	
Electrode strength	F_{EXX} = 70.0 [ksi]	strength coeff C_1	= 1.00		AISC 14th Table 8-3
Number of weld line	n = 2 for double fillet				
Load angle coefficient	$C_2 = (1 + 0.5 \sin^{1.5} \theta)$		= 1.35		AISC 14th Page 8-9
Fillet weld shear strength	$R_{n\text{-}w} = 0.6 (C_1 \times 70 \text{ ksi}) 0.707\ w\ n\ C_2$		= 25.102	[kip/in]	AISC 14th Eq 8-1
Base metal - shear tab	thickness t = 0.375 [in]	tensile F_u	= 65.0	[ksi]	
Base metal - shear tab is in shear, shear rupture as per AISC 14th Eq J4-4 is checked					AISC 14th J2.4
Base metal shear rupture	$R_{n\text{-}b} = 0.6\ F_u\ t$		= 14.625	[kip/in]	AISC 14th Eq J4-4
Double fillet linear shear strength	$R_n = \min (R_{n\text{-}w}, R_{n\text{-}b})$		= **14.625**	[kip/in]	AISC 14th Eq 9-2
Resistance factor-LRFD	φ = 0.75				AISC 14th Eq 8-1
	$\phi\ R_n$ =		= **10.969**	[kip/in]	
	ratio = **0.54**		> f_{max}	OK	

Brace Force Load Case 2 Gusset plate t=0.500 P =45.00 kips (C) ratio = **0.77** **PASS**

Gusset Plate - Shear Yielding			ratio = 31.48 / 148.13	= **0.21**	**PASS**
Plate Shear Yielding Check					
Plate size	width b_p = 9.875 [in]	thickness t_p	= 0.500	[in]	
Plate yield strength	F_y = 50.0 [ksi]				
Plate gross area in shear	$A_{gv} = b_p\ t_p$		= 4.938	[in^2]	
Shear force required	V_u =		= **31.48**	[kips]	
Plate shear yielding strength	$R_n = 0.6\ F_y\ A_{gv}$		= 148.13	[kips]	AISC 14th Eq J4-3
Resistance factor-LRFD	φ = 1.00				AISC 14th Eq J4-3
	$\phi\ R_n$ =		= **148.13**	[kips]	
	ratio = **0.21**		> V_u	OK	

Gusset Plate - Shear Rupture			ratio = 31.48 / 106.03	= **0.30**	**PASS**

Plate Shear Rupture Check

Bolt hole diameter	bolt dia $d_b = 3/4$	[in]	bolt hole dia $d_h = 7/8$	[in]	AISC 14th B4.3b
Number of bolt	$n = 3$				
Plate size	width $b_p = 9.875$	[in]	thickness $t_p = 0.500$	[in]	
Plate tensile strength	$F_u = 65.0$	[ksi]			
Plate net area in shear	$A_{nv} = (b_p - n\ d_h)\ t_p$		= 3.625	[in^2]	
Shear force required	$V_u =$		= **31.48**	[kips]	
Plate shear rupture strength	$R_n = 0.6\ F_u A_{nv}$		= 141.38	[kips]	AISC 14th Eq J4-4
Resistance factor-LRFD	$\phi = 0.75$				AISC 14th Eq J4-4
	$\phi\ R_n =$		= **106.03**	[kips]	
	ratio = **0.30**		> V_u	OK	

Gusset Plate Leg - Flexural Yielding			ratio = 14.77 / 30.47	= **0.48**	**PASS**

Refer to Bo Dowswell's paper 'Design of Wrap-Around Steel Gusset Plates' for more details on this limit state check

Shear on guseet leg & moment arm	shear V = 31.48	[kips]	ecc e = 5.630	[in]	
Moment on gusset plate leg	$M_u = V\ e$		= **14.77**	[kip-ft]	
Gusset plate leg size	width d = 9.875	[in]	thick t = 0.500	[in]	
Gusset plate steel strength	$F_y = 50.0$	[ksi]			
Moment on gusset plate leg	$R_n = F_y (t\ d^2 / 6)$		= 33.86	[kip-ft]	
Resistance factor-LRFD	$\phi = 0.90$				
	$\phi\ R_n =$		= **30.47**	[kips]	
	ratio = **0.48**		> M_u	OK	

Gusset Plate Leg - Lateral Torsional Buckling			ratio = 14.77 / 147.54	= **0.10**	**PASS**

Refer to Bo Dowswell's paper 'Design of Wrap-Around Steel Gusset Plates' for more details on this limit state check

Shear on guseet leg & moment arm	shear P = 31.48	[kips]	ecc e = 5.630	[in]	
Moment on gusset plate leg	$M_u = P\ e$		= **14.77**	[kip-ft]	
Gusset plate leg size	width d = 9.875	[in]	thick t = 0.500	[in]	
Gusset plate steel strength	E = 29000	[ksi]	G = 11200	[ksi]	
	$F_y = 50.0$	[ksi]			
Gusset leg buckiling length	L = distance from gusset load CG to gusset-beam interface line		= 10.630	[in]	
Critical moment - gusset leg	$R_n = 0.94 \sqrt{E\ G}\ \frac{d\ t^3}{L}$		= 163.93	[kip-ft]	Dowswell Paper Eq 9
Resistance factor-LRFD	$\phi = 0.90$				
	$\phi\ R_n =$		= **147.54**	[kip-ft]	
	ratio = **0.10**		> M_u	OK	

Shear Tab - Shear Yielding		ratio = 31.48 / 98.44	= **0.32**	**PASS**
Applied shear/axial forces	shear $V = 31.48$ [kips]	axial $P = 0.39$	[kips]	
Resultant shear force	$V_u = (V^2 + P^2)^{0.5}$	= **31.48**	[kips]	
Plate Shear Yielding Check				
Plate size	width $b_p = 8.750$ [in]	thickness $t_p = 0.375$	[in]	
Plate yield strength	$F_y = 50.0$ [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$	= 3.281	[in^2]	
Shear force required	$V_u =$	= **31.48**	[kips]	
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$	= 98.44	[kips]	AISC 14th Eq J4-3
Resistance factor-LRFD	$\phi = 1.00$			AISC 14th Eq J4-3
	$\phi R_n =$	= **98.44**	[kips]	
	ratio = **0.32**	$> V_u$	OK	

Shear Tab - Shear Rupture		ratio = 31.48 / 67.18	= **0.47**	**PASS**
Applied shear/axial forces	shear $V = 31.48$ [kips]	axial $P = 0.39$	[kips]	
Resultant shear force	$V_u = (V^2 + P^2)^{0.5}$	= **31.48**	[kips]	
Plate Shear Rupture Check				
Bolt hole diameter	bolt dia $d_b = {}^3/_4$ [in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 14th B4.3b
Number of bolt	$n = 3$			
Plate size	width $b_p = 8.750$ [in]	thickness $t_p = 0.375$	[in]	
Plate tensile strength	$F_u = 65.0$ [ksi]			
Plate net area in shear	$A_{nv} = (b_p - n\, d_h)\, t_p$	= 2.297	[in^2]	
Shear force required	$V_u =$	= **31.48**	[kips]	
Plate shear rupture strength	$R_n = 0.6 F_u A_{nv}$	= 89.58	[kips]	AISC 14th Eq J4-4
Resistance factor-LRFD	$\phi = 0.75$			AISC 14th Eq J4-4
	$\phi R_n =$	= **67.18**	[kips]	
	ratio = **0.47**	$> V_u$	OK	

Shear Tab - Flexural Yield Interact		ratio =	= **0.14**	**PASS**
Plate width & thick	width $b_p = 8.750$ [in]	thick $t_p = 0.375$	[in]	
	yield $F_y = 50.0$ [ksi]			
Shear plate - gross area	$A_g = b_p \times t_p$	= 3.281	[in^2]	
Shear plate - plastic modulus	$Z_p = (b_p \times t_p^2) / 4$	= 7.178	[in^3]	
Axial strength available	P_c = from axial tensile yield check	= 147.66	[kips]	
Axial strength required	P_r = from gusset interface forces calc	= 0.39	[kips]	
Shear strength available	V_c = from shear yielding check	= 98.44	[kips]	
Shear strength required	V_r = from gusset interface forces calc	= 31.48	[kips]	
Flexural strength available	$M_c = \phi F_y Z_p$ ϕ=0.90	= 26.92	[kip-ft]	
Flexural strength required	M_r = from gusset interface forces calc	= 4.92	[kip-ft]	
Flexural yield interaction	ratio $= \left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right)^2$	= **0.14**		AISC 14th Eq 10-5
		< 1.0	OK	

Shear Tab - Flexural Rupture Interact	ratio =	= **0.28**	**PASS**

Plate A_n and Z_{net} Calc

Bolt hole diameter	bolt dia $d_b = {}^3/_4$	[in]	bolt hole dia $d_h = {}^7/_8$	[in]		AISC 14th B4.3b
Number of bolt	$n = 3$					
Plate size	width $b_p = 8.750$	[in]	thickness $t_p = 0.375$	[in]		
Plate net area	$A_n = (b_p - n\ d_h)\ t_p$		= 2.297	[in²]		
Plate net plastic sect modulus	$Z_{net} =$		= 5.137	[in³]		
Plate net elastic sect modulus	$S_{net} =$		= 3.421	[in³]		

Plate width & thick	width $b_p = 8.750$ [in]	thick $t_p = 0.375$	[in]		
	tensile $F_u = 65.0$ [ksi]				
Axial strength available	P_c = from axial tensile rupture check	= 111.97	[kips]		
Axial strength required	P_r = from gusset interface forces calc	= 0.39	[kips]		
Shear strength available	V_c = from shear rupture check	= 67.18	[kips]		
Shear strength required	V_r = from gusset interface forces calc	= 31.48	[kips]		
Flexural strength available	$M_c = \phi\ F_u\ Z_{net}$ ϕ=0.75	= 20.87	[kip-ft]		AISC 14th Eq 9-4
Flexural strength required	M_r = from gusset interface forces calc	= 4.92	[kip-ft]		
Flexural rupture interaction	$\text{ratio} = \left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right)^2$	= **0.28**			AISC 14th Eq 10-5
		< 1.0	OK		

Shear Tab - Bolt Bearing on Shear Tab	ratio = 31.48 / 53.68	= **0.59**	**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 = 31.48	[kips]	axial P = 0.39	[kips]		
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$		= **31.48**	[kips]		
Resultant force/hor line load angle	$\theta = \tan^{-1}(V / P)$		= 89.29	[°]		
Bolt hole diameter	bolt dia d_b = 0.750	[in]	bolt hole dia d_{bh} = 0.813	[in]	AISC 14th B4.3b	
Bolt hole ver. dimension	d_v =		= 0.813	[in]		
Bolt hole hor. dimension	d_h =		= 0.813	[in]		
Bolt center to bolt hole edge dist	$d_c = 0.5\ d_{bh}$		= 0.406	[in]		
Bolt no in ver & hor direction	Bolt Row n_v = 3		Bolt Col n_h = 1			
Bolt spacing	ver s_v = 3.000	[in]				
Bolt edge distance	ver e_v = 1.375	[in]	hor e_h = 1.375	[in]		
Bolt clear dist - bot right corner bolt	$L_{cA} = \min\left(\dfrac{e_v}{\sin\theta}, \dfrac{e_h}{\cos\theta}\right) - d_c$		= 0.969	[in]		
Bolt clear dist - right side edge bolt	$L_{cB} = \min\left(\dfrac{s_v - 0.5d_v}{\sin\theta}, \dfrac{e_h}{\cos\theta}\right) - d_c$		= 2.188	[in]		

Single Bolt Shear Strength

Bolt shear stress	bolt grade = A325-N		F_{nv} = 54.0	[ksi]	AISC 14th Table J3.2
	bolt dia d_b = 0.750	[in]	bolt area A_b = 0.442	[in²]	
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$		= 23.86	[kips]	AISC 14th Eq J3-1
Bolt bearing on plate	thick t = 0.375	[in]	tensile F_u = 65.0	[ksi]	
Bolt bearing strength	$R_{n\text{-}br} = 3.0\ d_b\ t\ F_u$		= 54.84	[kips]	AISC 14th Eq J3-6b

Type A - Bolt Group Bottom Right Corner Bolt

Number of bolt	n_A = 1			
Bolt tear out strength	$R_{n\text{-}tA} = 1.5\ L_{cA}\ t\ F_u$	= 35.42	[kips]	AISC 14th Eq J3-6b
Bolt bearing strength	$R_{nA} = \min(R_{n\text{-}tA}, R_{n\text{-}br}, R_{n\text{-}bolt})$	= **23.86**	[kips]	

Type B - Bolt Group Right Side Edge Bolt

Number of bolt	n_B = 2			
Bolt tear out strength	$R_{n\text{-}tB} = 1.5\ L_{cB}\ t\ F_u$	= 79.99	[kips]	AISC 14th Eq J3-6b
Bolt bearing strength	$R_{nB} = \min(R_{n\text{-}tB}, R_{n\text{-}br}, R_{n\text{-}bolt})$	= **23.86**	[kips]	

Bolt bearing strength for all bolts	$R_n = n_A R_{nA} + n_B R_{nB} + n_C R_{nC} + n_D R_{nD}$	= 71.57	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 14th J3-10
	ϕR_n =	= **53.68**	[kips]	
	ratio = **0.59**	> R	OK	

Shear Tab - Beam Side - Block Shear - 1-Side Strip		ratio = 31.48 / 74.04	= **0.43**	**PASS**
Plate Block Shear - Side Strip				
Bolt hole diameter	bolt dia $d_b = {}^3/_4$ [in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 14th B4.3b
Plate thickness	$t_p = 0.375$ [in]			
Plate strength	$F_y = 50.0$ [ksi]	$F_u = 65.0$	[ksi]	
Bolt no in ver & hor dir	$n_v = 1$	$n_h = 3$		
Bolt spacing in hor dir	$s_h = 3.000$ [in]			
Bolt edge dist in ver & hor dir	$e_v = 1.375$ [in]	$e_h = 1.375$	[in]	
Gross area subject to shear	$A_{gv} = [\ (n_h - 1)\ s_h + e_h\ ]\ t_p$	= 2.766	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [\ (n_h - 1) + 0.5\ ]\ d_h\ t_p$	= 1.945	[in^2]	
Net area subject to tension	$A_{nt} = (\ e_v - 0.5\ d_h\ )\ t_p$	= 0.352	[in^2]	
Block shear strength required	$V_u =$	= **31.48**	[kips]	
Uniform tension stress factor	$U_{bs} = 1.00$			AISC 14th Fig C-J4.2
Bolt shear resistance provided	$R_n = \min\ (0.6F_u A_{nv},\ 0.6F_y A_{gv}) + U_{bs} F_u A_{nt}$	= 98.72	[kips]	AISC 14th Eq J4-5
Resistance factor-LRFD	$\phi = 0.75$			AISC 14th Eq J4-5
	$\phi\ R_n =$	= **74.04**	[kips]	
	ratio = **0.43**	$> V_u$	OK	

Shear Tab - Lateral Stability / Stabilizer Plate		ratio = 31.48 / 556.65	= **0.06**	**PASS**
Applied shear/axial forces	shear V = 31.48 [kips]	axial P = 0.39	[kips]	
Resultant shear force	$V_u = (\ V^2 + P^2\ )^{0.5}$	= **31.48**	[kips]	
Distance from support to the first line of bolts	a =	= 1.875	[in]	
Plate thickness & depth	$t_p = 0.375$ [in]	L = 8.750	[in]	
Shear resistance provided	$R_n = 1500\ \pi \frac{L\ t_p^3}{a^2}$	= 618.50	[kips]	AISC 14th Eq 10-6
Resistance factor-LRFD	$\phi = 0.90$			AISC 14th Eq 10-6
	$\phi\ R_n =$	= **556.65**	[kips]	
	ratio = **0.06**	$> V_u$	OK	

Shear Tab - Plate Flexural Buckling		ratio = 31.48 / 82.10	= **0.38**	**PASS**
Shear tab size	depth = 8.750 [in]	thick = 0.375	[in]	
Plate buckling model	c = dist from support to fisrt bolt line	= 1.875	[in]	AISC 14th Fig. 9-3
	h_0 = shear tab depth	= 8.750	[in]	
	t_w = shear tab thick	= 0.375	[in]	
Shear tab steel yield stress	F_y = 50.0 [ksi]			
Plate buckling factor	$\lambda = \dfrac{h_0 \sqrt{F_y}}{10\, t_w \sqrt{475 + 280\, (h_0/c)^2}}$	= 0.204		AISC 14th Eq 9-18
Plate buckling factor	Q =	= 1.000		AISC 14th Eq 9-15
Plate critical buckling stress	$F_{cr} = Q\, F_y$	= 50.0	[ksi]	AISC 14th Eq 9-14
Shear force in demand	V_u =	= **31.48**	[kips]	
Shear tab net elastic modulus	S_{net} =	= 3.421	[in^3]	
Shear force to bolt group CG ecc	a =	= 1.875	[in]	
Shear resistance	$R_n = F_{cr}\, S_{net} / a$	= 91.22	[kips]	AISC 14th Eq 9-19
Resistance factor-LRFD	ϕ = 0.90			AISC 14th Eq 9-19
	$\phi\, R_n$ =	= **82.10**	[kips]	
	ratio = **0.38**	> V_u	OK	

Shear Tab - Plate Shear / Axial Compression Interact	ratio =		= **0.15**	**PASS**	
Shear Tab Forces					
	Shear V = 31.48 [kips]	Axial P = 0.39	[kips]	in compression	
Shear tab size	depth h_p = 8.750 [in]	thickness t_p = 0.375	[in]		
Shear tab material strength	F_y = 50.0 [ksi]	E = 29000	[ksi]		
Plate Compressive Capacity					
Plate gross area in compression	$A_g = h_p t_p$	= 3.281	[in^2]		
Plate radius of gyration	$r = t_p / \sqrt{12}$	= 0.108	[in]		
Plate effective length factor	K =	= 1.00			
Plate unbraced length	L_u =	= 1.875	[in]		
Plate slenderness	$KL/r = 1.00 \times L_u / r$	= 17.32			
	when $\frac{KL}{r} \leq 25$			AISC 14th J4.4 (a)	
Plate compression provided	$P_n = F_y \times A_g$	= 164.06	[kips]	AISC 14th Eq J4-6	
Axial compression force	P_u = from user input	= **0.39**	[kips]		
Plate Flexural Buckling Capacity					
Plate buckling model	c = dist from support to fisrt bolt line	= 1.875	[in]	AISC 14th Fig. 9-3	
	h_0 = shear tab depth	= 8.750	[in]		
	t_w = shear tab thick	= 0.375	[in]		
Plate buckling factor	$\lambda = \dfrac{h_0 \sqrt{F_y}}{10\, t_w \sqrt{475 + 280\, (h_0 / c)^2}}$	= 0.204		AISC 14th Eq 9-18	
Plate buckling factor	Q =	= 1.000		AISC 14th Eq 9-15	
Plate critical buckling stress	$F_{cr} = Q\, F_y$	= 50.0	[ksi]	AISC 14th Eq 9-14	
Shear tab net elastic modulus	S_{net} =	= 3.421	[in^3]		
Shear force to bolt group CG ecc	a =	= 1.875	[in]		
Shear resistance	$V_n = F_{cr} S_{net} / a$	= **91.22**	[kips]	AISC 14th Eq 9-19	
Shear force	V_u = from user input	= **31.48**	[kips]		
Resistance factor-LRFD	ϕ = 0.90			AISC 14th Eq 10-5	
Shear-axial interaction	ratio = $\left(\dfrac{V_r}{\phi\, V_n}\right)^2 + \left(\dfrac{P_r}{\phi\, P_n}\right)^2$	= **0.15**		AISC 14th Eq 10-5	
		< 1.0	OK		

Shear Tab - Compression Buckling		ratio = 0.39 / 147.66	= **0.00**	**PASS**
Plate Compression Check				
Plate size	width b_p = 8.750 [in]	thickness t_p = 0.375	[in]	
	F_y = 50.0 [ksi]	E = 29000	[ksi]	
Plate gross area in compression	$A_g = b_p t_p$	= 3.281	[in^2]	
Plate radius of gyration	$r = t_p / \sqrt{12}$	= 0.108	[in]	
Plate effective length factor	K =	= 1.00		
Plate unbraced length	L_u =	= 1.875	[in]	
Plate slenderness	$KL/r = 1.00 \times L_u / r$	= 17.32		
Plate compression required	P_u =	= **0.39**	[kips]	
	when $\frac{KL}{r} \leq 25$			AISC 14th J4.4 (a)
Plate compression provided	$R_n = F_y \times A_g$	= 164.06	[kips]	AISC 14th Eq J4-6
Bolt resistance factor-LRFD	ϕ = 0.90			AISC 14th J4.4 (a)
	ϕR_n =	= **147.66**	[kips]	
	ratio = **0.00**	> P_u	OK	

Bolt Group Eccentricity				
Bolt group forces	shear V = 31.48 [kips]	axial P = 0.39	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= 31.48	[kips]	
Resultant force to ver Y axis angle	$\theta = \tan^{-1} (P / V)$	= 0.71	[°]	
Bolt group row and column	bolt row n_r = 3	bolt col n_c = 1		
Bolt row spacing	bolt row s_r = 3.000 [in]			
Shear force to bolt group CG ecc	e_x =	= 1.875	[in]	
Shear force to ver Y axis angle	θ =	= 0.71	[°]	
Bolt group coefficient C	C = from AISC 14th Table 7-6 ~ 7-13	= 2.288		
Bolt group eccentricity coefficient	$C_{ec} = C / (n_r \times n_c)$	= **0.763**		

Shear Tab / Beam Web - Bolt Shear		ratio = 31.48 / 40.96	= **0.77**	**PASS**
Bolt group forces	shear V = 31.48 [kips]	axial P = 0.39	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= **31.48**	[kips]	
Bolt shear stress	bolt grade = A325-N	F_{nv} = 54.0	[ksi]	AISC 14th Table J3.2
	bolt dia d_b = 0.750 [in]	bolt area A_b = 0.442	[in^2]	
Number of bolt carried shear	n_s = 3.0	shear plane m = 1		
Bolt group eccentricity coefficient	C_{ec} = from 'Bolt Group Eccentricity' calc	= 0.763		
Required shear strength	V_u =	= **31.48**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 54.61	[kips]	AISC 14th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 14th Eq J3-1
	ϕR_n =	= **40.96**	[kips]	
	ratio = **0.77**	> V_u	OK	

Shear Tab to Ver Beam Web Weld Strength		ratio = 31.48 / 89.28	= **0.35**	**PASS**
Weld Group Forces				
	Shear V = 31.48 [kips]	Axial P = 0.00 [kips] in compression		
Shear force to bolt group CG ecc	e_x =	= 1.875 [in]		
Shear tab weld length	L =	= 8.750 [in]		
Shear force in demand	V_u = from user input	= **31.48** [kips]		
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 14th 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 14th Page 8-9	
Fillet weld shear strength	r_w = 0.6 (C_1 x 70 ksi) 0.707 w n C_2	= 18.559 [kip/in]	AISC 14th Eq 8-1	
Base metal - shear tab	thickness t = 0.375 [in]	tensile F_u = 65.0 [ksi]		
Base metal - shear tab is in shear, shear rupture as per AISC 14th Eq J4-4 is checked			AISC 14th J2.4	
Base metal shear rupture	$r_b = 0.6\, F_u\, t$	= 14.625 [kip/in]	AISC 14th Eq J4-4	
Weld stress reduction factor due to less base metal strength	$C_3 = r_b / r_w$ when $r_b < r_w$	= 0.788		
Table 8-4 Coefficient C for Eccentrically Loaded Weld Group			AISC 14th Table 8-4	
	$a = e_x / L$	= 0.21		
	C = C value in Table 8-4 when k=0	= 3.453		
Weld coefficients	C = 3.453	C_1 = 1.000		
		C_3 = 0.788		
Weld size & length	D = 5.000 [1/16]	L = 8.750 [in]		
Weld strength	$R_n = C\, C_1\, C_3\, D\, L$	= 119.04 [kips]	AISC 14th Table 8-4	
Resistance factor-LRFD	ϕ = 0.75			
	ϕR_n =	= **89.28** [kips]		
	ratio = **0.35**	> V_u	OK	

Gusset to Hor Beam	Shear Tab Connection	Code=AISC 360-10 LRFD

Result Summary	geometries & weld limitations = **PASS**	limit states max ratio = **0.86**	**PASS**

Geometry Restriction Checks - Shear Tab to Hor Beam Web				**PASS**
Min Bolt Edge Distance - Shear Tab to Hor Beam Web				
Bolt diameter	d_b =	= 0.750	[in]	
Min edge distance allowed	$L_{e\text{-}min}$ =	= **1.000**	[in]	AISC 14th Table J3.4
Min edge distance in Shear Tab to Hor Beam Web	L_e =	= **1.375**	[in]	
		> $L_{e\text{-}min}$	OK	
Min Bolt Spacing - Shear Tab to Hor Beam Web				
Bolt diameter	d_b =	= 0.750	[in]	
Min bolt spacing allowed	$L_{s\text{-}min}$ = 2.667 d_b	= **2.000**	[in]	AISC 14th J3.3
Min Bolt spacing in Shear Tab to Hor Beam Web	L_s =	= **3.000**	[in]	
		> $L_{s\text{-}min}$	OK	

Weld Limitation Check - Shear Tab Weld				**PASS**
Min Fillet Weld Size				
Thinner part joined thickness	t =	= 0.260	[in]	
Min fillet weld size allowed	w_{min} =	= **0.188**	[in]	AISC 14th 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 x w	= **1.250**	[in]	AISC 14th J2.2b
Min fillet weld length	L =	= **8.750**	[in]	
		> L_{min}	OK	

Brace Force Load Case 1	Gusset plate t=0.500	P =-45.00 kips (T)	ratio = **0.86**	**PASS**

Gusset Plate - Shear Yielding			ratio = 31.43 / 148.13	= **0.21**		**PASS**
Plate Shear Yielding Check						
Plate size	width b_p = 9.875	[in]	thickness t_p = 0.500	[in]		
Plate yield strength	F_y = 50.0	[ksi]				
Plate gross area in shear	$A_{gv} = b_p t_p$		= 4.938	[in^2]		
Shear force required	V_u =		= **31.43**	[kips]		
Plate shear yielding strength	$R_n = 0.6\ F_y A_{gv}$		= 148.13	[kips]	AISC 14th Eq J4-3	
Resistance factor-LRFD	ϕ = 1.00				AISC 14th Eq J4-3	
	$\phi\ R_n$ =		= **148.13**	[kips]		
	ratio = **0.21**		> V_u	OK		

Gusset Plate - Shear Rupture			ratio = 31.43 / 106.03	= **0.30**	**PASS**
Plate Shear Rupture Check					
Bolt hole diameter	bolt dia $d_b = 3/4$	[in]	bolt hole dia $d_h = 7/8$	[in]	AISC 14th B4.3b
Number of bolt	$n = 3$				
Plate size	width $b_p = 9.875$	[in]	thickness $t_p = 0.500$	[in]	
Plate tensile strength	$F_u = 65.0$	[ksi]			
Plate net area in shear	$A_{nv} = (b_p - n\,d_h)\,t_p$		= 3.625	[in^2]	
Shear force required	$V_u =$		= **31.43**	[kips]	
Plate shear rupture strength	$R_n = 0.6\,F_u A_{nv}$		= 141.38	[kips]	AISC 14th Eq J4-4
Resistance factor-LRFD	$\phi = 0.75$				AISC 14th Eq J4-4
	$\phi R_n =$		= **106.03**	[kips]	
	ratio = **0.30**		$> V_u$	OK	

Gusset Plate Leg - Flexural Yielding			ratio = 26.06 / 30.47	= **0.86**	**PASS**
Refer to Bo Dowswell's paper 'Design of Wrap-Around Steel Gusset Plates' for more details on this limit state check					
Shear on guseet leg & moment arm	shear V = 31.43	[kips]	ecc e = 9.950	[in]	
Moment on gusset plate leg	$M_u = V\,e$		= **26.06**	[kip-ft]	
Gusset plate leg size	width d = 9.875	[in]	thick t = 0.500	[in]	
Gusset plate steel strength	$F_y = 50.0$	[ksi]			
Moment on gusset plate leg	$R_n = F_y\,(t\,d^2 / 6)$		= 33.86	[kip-ft]	
Resistance factor-LRFD	$\phi = 0.90$				
	$\phi R_n =$		= **30.47**	[kips]	
	ratio = **0.86**		$> M_u$	OK	

Gusset Plate Leg - Lateral Torsional Buckling			ratio = 26.06 / 156.76	= **0.17**	**PASS**
Refer to Bo Dowswell's paper 'Design of Wrap-Around Steel Gusset Plates' for more details on this limit state check					
Shear on guseet leg & moment arm	shear P = 31.43	[kips]	ecc e = 9.950	[in]	
Moment on gusset plate leg	$M_u = P\,e$		= **26.06**	[kip-ft]	
Gusset plate leg size	width d = 9.875	[in]	thick t = 0.500	[in]	
Gusset plate steel strength	E = 29000	[ksi]	G = 11200	[ksi]	
	$F_y = 50.0$	[ksi]			
Gusset leg buckiling length	L = distance from gusset load CG to gusset-beam interface line		= 10.005	[in]	
Critical moment - gusset leg	$R_n = 0.94 \sqrt{E\,G}\,\dfrac{d\,t^3}{L}$		= 174.17	[kip-ft]	Dowswell Paper Eq 9
Resistance factor-LRFD	$\phi = 0.90$				
	$\phi R_n =$		= **156.76**	[kip-ft]	
	ratio = **0.17**		$> M_u$	OK	

Shear Tab - Shear Yielding			ratio = 31.43 / 98.44 = **0.32**		**PASS**
Applied shear/axial forces	shear V = 31.43	[kips]	axial P = -0.34	[kips]	
Resultant shear force	$V_u = (V^2 + P^2)^{0.5}$		= **31.43**	[kips]	
Plate Shear Yielding Check					
Plate size	width b_p = 8.750	[in]	thickness t_p = 0.375	[in]	
Plate yield strength	F_y = 50.0	[ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$		= 3.281	[in^2]	
Shear force required	V_u =		= **31.43**	[kips]	
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$		= 98.44	[kips]	AISC 14th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00				AISC 14th Eq J4-3
	ϕR_n =		= **98.44**	[kips]	
	ratio = **0.32**		> V_u	OK	

Shear Tab - Shear Rupture			ratio = 31.43 / 67.18 = **0.47**		**PASS**
Applied shear/axial forces	shear V = 31.43	[kips]	axial P = -0.34	[kips]	
Resultant shear force	$V_u = (V^2 + P^2)^{0.5}$		= **31.43**	[kips]	
Plate Shear Rupture Check					
Bolt hole diameter	bolt dia $d_b = {}^3/_4$	[in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 14th B4.3b
Number of bolt	n = 3				
Plate size	width b_p = 8.750	[in]	thickness t_p = 0.375	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate net area in shear	$A_{nv} = (b_p - n\, d_h)\, t_p$		= 2.297	[in^2]	
Shear force required	V_u =		= **31.43**	[kips]	
Plate shear rupture strength	$R_n = 0.6 F_u A_{nv}$		= 89.58	[kips]	AISC 14th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75				AISC 14th Eq J4-4
	ϕR_n =		= **67.18**	[kips]	
	ratio = **0.47**		> V_u	OK	

Shear Tab - Axial Tensile Yield			ratio = 0.34 / 147.66 = **0.00**		**PASS**
Plate Tensile Yielding Check					
Plate size	width b_p = 8.750	[in]	thickness t_p = 0.375	[in]	
Plate yield strength	F_y = 50.0	[ksi]			
Plate gross area in shear	$A_g = b_p t_p$		= 3.281	[in^2]	
Tensile force required	P_u =		= **0.34**	[kips]	
Plate tensile yielding strength	$R_n = F_y A_g$		= 164.06	[kips]	AISC 14th Eq J4-1
Resistance factor-LRFD	ϕ = 0.90				AISC 14th Eq J4-1
	ϕR_n =		= **147.66**	[kips]	
	ratio = **0.00**		> P_u	OK	

Shear Tab - Axial Tensile Rupture			ratio = 0.34 / 111.97	= **0.00**	**PASS**
Plate Tensile Rupture Check					
Bolt hole diameter	bolt dia $d_b = {}^3/_4$	[in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 14th B4.3b
Number of bolt	n = 3				
Plate size	width b_p = 8.750	[in]	thickness t_p = 0.375	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate net area in tension	$A_{nt} = (b_p - n\ d_h)\ t_p$		= 2.297	[in²]	
Tensile force required	P_u =		= **0.34**	[kips]	
Plate tensile rupture strength	$R_n = F_u A_{nt}$		= 149.30	[kips]	AISC 14th Eq J4-2
Resistance factor-LRFD	ϕ = 0.75				AISC 14th Eq J4-2
	ϕR_n =		= **111.97**	[kips]	AISC 14th Eq J4-2
	ratio = **0.00**		> P_u	OK	

Shear Tab - Flexural Yield Interact			ratio =	= **0.14**	**PASS**
Plate width & thick	width b_p = 8.750	[in]	thick t_p = 0.375	[in]	
	yield F_y = 50.0	[ksi]			
Shear plate - gross area	$A_g = b_p \times t_p$		= 3.281	[in²]	
Shear plate - plastic modulus	$Z_p = (b_p \times t_p^2) / 4$		= 7.178	[in³]	
Axial strength available	P_c = from axial tensile yield check		= 147.66	[kips]	
Axial strength required	P_r = from gusset interface forces calc		= 0.34	[kips]	
Shear strength available	V_c = from shear yielding check		= 98.44	[kips]	
Shear strength required	V_r = from gusset interface forces calc		= 31.43	[kips]	
Flexural strength available	$M_c = \phi F_y Z_p$ ϕ=0.90		= 26.92	[kip-ft]	
Flexural strength required	M_r = from gusset interface forces calc		= 4.91	[kip-ft]	
Flexural yield interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right)^2$		= **0.14**		AISC 14th Eq 10-5
			< 1.0	OK	

Shear Tab - Flexural Rupture Interact			ratio =	$= \mathbf{0.28}$	**PASS**	
Plate A_n and Z_{net} Calc						
Bolt hole diameter	bolt dia $d_b = {}^3/_4$	[in]	bolt hole dia $d_h = {}^7/_8$		[in]	AISC 14th B4.3b
Number of bolt	$n = 3$					
Plate size	width $b_p = 8.750$	[in]	thickness $t_p = 0.375$		[in]	
Plate net area	$A_n = (b_p - n\ d_h)\ t_p$			$= 2.297$	[in^2]	
Plate net plastic sect modulus	$Z_{net} =$			$= 5.137$	[in^3]	
Plate net elastic sect modulus	$S_{net} =$			$= 3.421$	[in^3]	
Plate width & thick	width $b_p = 8.750$	[in]	thick $t_p = 0.375$		[in]	
	tensile $F_u = 65.0$	[ksi]				
Axial strength available	$P_c =$ from axial tensile rupture check			$= 111.97$	[kips]	
Axial strength required	$P_r =$ from gusset interface forces calc			$= 0.34$	[kips]	
Shear strength available	$V_c =$ from shear rupture check			$= 67.18$	[kips]	
Shear strength required	$V_r =$ from gusset interface forces calc			$= 31.43$	[kips]	
Flexural strength available	$M_c = \phi\ F_u\ Z_{net}$ $\phi=0.75$			$= 20.87$	[kip-ft]	AISC 14th Eq 9-4
Flexural strength required	$M_r =$ from gusset interface forces calc			$= 4.91$	[kip-ft]	
Flexural rupture interaction	$\text{ratio} = \left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right)^2$			$= \mathbf{0.28}$		AISC 14th Eq 10-5
				< 1.0	OK	

Shear Tab - Bolt Bearing on Shear Tab		ratio = 31.43 / 53.68	= **0.59**	**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 = 31.43 [kips]	axial P = -0.34	[kips]		
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= **31.43**	[kips]		
Resultant force/hor line load angle	$\theta = \tan^{-1} (V / P)$	= 89.38	[°]		
Bolt hole diameter	bolt dia d_b = 0.750 [in]	bolt hole dia d_{bh} = 0.813	[in]	AISC 14th B4.3b	
Bolt hole ver. dimension	d_v =	= 0.813	[in]		
Bolt hole hor. dimension	d_h =	= 0.813	[in]		
Bolt center to bolt hole edge dist	$d_c = 0.5\ d_{bh}$	= 0.406	[in]		
Bolt no in ver & hor direction	Bolt Row n_v = 3	Bolt Col n_h = 1			
Bolt spacing	ver s_v = 3.000 [in]				
Bolt edge distance	ver e_v = 1.375 [in]	hor e_h = 1.375	[in]		
Bolt clear dist - bot right corner bolt	$L_{cA} = \min \left(\frac{e_v}{\sin \theta}, \frac{e_h}{\cos \theta} \right) - d_c$	= 0.969	[in]		
Bolt clear dist - right side edge bolt	$L_{cB} = \min \left(\frac{s_v - 0.5 d_v}{\sin \theta}, \frac{e_h}{\cos \theta} \right) - d_c$	= 2.188	[in]		
Single Bolt Shear Strength					
Bolt shear stress	bolt grade = A325-N	F_{nv} = 54.0	[ksi]	AISC 14th Table J3.2	
	bolt dia d_b = 0.750 [in]	bolt area A_b = 0.442	[in²]		
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$	= 23.86	[kips]	AISC 14th Eq J3-1	
Bolt bearing on plate	thick t = 0.375 [in]	tensile F_u = 65.0	[ksi]		
Bolt bearing strength	$R_{n\text{-}br} = 3.0\ d_b\ t\ F_u$	= 54.84	[kips]	AISC 14th Eq J3-6b	
Type A - Bolt Group Bottom Right Corner Bolt					
Number of bolt	$n_A = 1$				
Bolt tear out strength	$R_{n\text{-}tA} = 1.5\ L_{cA}\ t\ F_u$	= 35.42	[kips]	AISC 14th Eq J3-6b	
Bolt bearing strength	$R_{nA} = \min (R_{n\text{-}tA}, R_{n\text{-}br}, R_{n\text{-}bolt})$	= **23.86**	[kips]		
Type B - Bolt Group Right Side Edge Bolt					
Number of bolt	$n_B = 2$				
Bolt tear out strength	$R_{n\text{-}tB} = 1.5\ L_{cB}\ t\ F_u$	= 79.99	[kips]	AISC 14th Eq J3-6b	
Bolt bearing strength	$R_{nB} = \min (R_{n\text{-}tB}, R_{n\text{-}br}, R_{n\text{-}bolt})$	= **23.86**	[kips]		
Bolt bearing strength for all bolts	$R_n = n_A R_{nA} + n_B R_{nB} + n_C R_{nC} + n_D R_{nD}$	= 71.57	[kips]		
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 14th J3-10	
	ϕR_n =	= **53.68**	[kips]		
	ratio = **0.59**	> R	OK		

Shear Tab - Beam Side - Block Shear - 1-Side Strip	ratio = 31.43 / 74.04	= **0.42**	**PASS**		
Plate Block Shear - Side Strip					
Bolt hole diameter	bolt dia $d_b = {}^3/_4$	[in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 14[th] B4.3b
Plate thickness	$t_p = 0.375$	[in]			
Plate strength	$F_y = 50.0$	[ksi]	$F_u = 65.0$	[ksi]	
Bolt no in ver & hor dir	$n_v = 1$		$n_h = 3$		
Bolt spacing in hor dir	$s_h = 3.000$	[in]			
Bolt edge dist in ver & hor dir	$e_v = 1.375$	[in]	$e_h = 1.375$	[in]	
Gross area subject to shear	$A_{gv} = [\ (n_h - 1)\ s_h + e_h\ ]\ t_p$		= 2.766	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [\ (n_h - 1) + 0.5\ ]\ d_h\, t_p$		= 1.945	[in^2]	
Net area subject to tension	$A_{nt} = (\ e_v - 0.5\ d_h\ )\ t_p$		= 0.352	[in^2]	
Block shear strength required	$V_u =$		= **31.43**	[kips]	
Uniform tension stress factor	$U_{bs} = 1.00$				AISC 14[th] Fig C-J4.2
Bolt shear resistance provided	$R_n = \min\ (0.6F_u A_{nv}\ ,\ 0.6F_y A_{gv}\ ) + U_{bs} F_u A_{nt}$		= 98.72	[kips]	AISC 14[th] Eq J4-5
Resistance factor-LRFD	$\phi = 0.75$				AISC 14[th] Eq J4-5
	$\phi\ R_n =$		= **74.04**	[kips]	
	ratio = **0.42**		$> V_u$	OK	

Shear Tab - Beam Side-Axial Tearout - Block Shear - Center Strip	ratio = 0.34 / 98.26	= **0.00**	**PASS**		
Plate Block Shear - Center Strip					
Bolt hole diameter	bolt dia $d_b = {}^3/_4$	[in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 14[th] B4.3b
Plate thickness	$t_p = 0.375$	[in]			
Plate strength	$F_y = 50.0$	[ksi]	$F_u = 65.0$	[ksi]	
Bolt no in ver & hor dir	$n_v = 3$		$n_h = 1$		
Bolt spacing in ver & hor dir	$s_v = 3.000$	[in]	$s_h = 3.000$	[in]	
Bolt edge dist in ver & hor dir	$e_v = 1.375$	[in]	$e_h = 1.375$	[in]	
Gross area subject to shear	$A_{gv} = [\ (n_h - 1)\ s_h + e_h\ ]\ t_p \times 2$		= 1.031	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5]\ d_h\, t_p \times 2$		= 0.703	[in^2]	
Net area subject to tension when sheared out by center strip	$A_{nt} = (\ n_v - 1)\ (\ s_v - d_h\ )\ t_p$		= 1.594	[in^2]	
Block shear strength required	$V_u =$		= **0.34**	[kips]	
Uniform tension stress factor	$U_{bs} = 1.00$				AISC 14[th] Fig C-J4.2
Bolt shear resistance provided	$R_n = \min\ (0.6F_u A_{nv}\ ,\ 0.6F_y A_{gv}\ ) + U_{bs} F_u A_{nt}$		= 131.02	[kips]	AISC 14[th] Eq J4-5
Resistance factor-LRFD	$\phi = 0.75$				AISC 14[th] Eq J4-5
	$\phi\ R_n =$		= **98.26**	[kips]	
	ratio = **0.00**		$> V_u$	OK	

Shear Tab - Block Shear - Shear / Tensile Interact	ratio =		= **0.18**	**PASS**	
Shear block shear strength required	V_u =		= 31.43	[kips]	
Axial block shear strength required	P_u =		= 0.34	[kips]	
Shear block shear strength available	ϕR_{nv} = from calc shown above		= 74.04	[kips]	
Axial block shear strength available	ϕR_{nt} = from calc shown above		= 98.26	[kips]	
Block shear shear/tensile interaction	ratio = $\left(\frac{V_u}{\phi R_{nv}}\right)^2 + \left(\frac{P_u}{\phi R_{nt}}\right)^2$		= **0.18**		AISC 14th Eq 10-5
			< 1.0	OK	

Shear Tab - Lateral Stability / Stabilizer Plate	ratio = 31.43 / 556.65		= **0.06**	**PASS**	
Applied shear/axial forces	shear V = 31.43 [kips]		axial P = -0.34	[kips]	
Resultant shear force	$V_u = (V^2 + P^2)^{0.5}$		= **31.43**	[kips]	
Distance from support to the first line of bolts	a =		= 1.875	[in]	
Plate thickness & depth	t_p = 0.375 [in]		L = 8.750	[in]	
Shear resistance provided	$R_n = 1500\,\pi \frac{L\,t_p^3}{a^2}$		= 618.50	[kips]	AISC 14th Eq 10-6
Resistance factor-LRFD	ϕ = 0.90				AISC 14th Eq 10-6
	ϕR_n =		= **556.65**	[kips]	
	ratio = **0.06**		> V_u	OK	

Shear Tab - Plate Flexural Buckling	ratio = 31.43 / 82.10		= **0.38**	**PASS**	
Shear tab size	depth = 8.750 [in]		thick = 0.375	[in]	
Plate buckling model	c = dist from support to fisrt bolt line		= 1.875	[in]	AISC 14th Fig. 9-3
	h_0 = shear tab depth		= 8.750	[in]	
	t_w = shear tab thick		= 0.375	[in]	
Shear tab steel yield stress	F_y = 50.0 [ksi]				
Plate buckling factor	$\lambda = \frac{h_0 \sqrt{F_y}}{10\,t_w \sqrt{475 + 280\,(h_0/c)^2}}$		= 0.204		AISC 14th Eq 9-18
Plate buckling factor	Q =		= 1.000		AISC 14th Eq 9-15
Plate critical buckling stress	$F_{cr} = Q\,F_y$		= 50.0	[ksi]	AISC 14th Eq 9-14
Shear force in demand	V_u =		= **31.43**	[kips]	
Shear tab net elastic modulus	S_{net} =		= 3.421	[in^3]	
Shear force to bolt group CG ecc	a =		= 1.875	[in]	
Shear resistance	$R_n = F_{cr}\,S_{net} / a$		= 91.22	[kips]	AISC 14th Eq 9-19
Resistance factor-LRFD	ϕ = 0.90				AISC 14th Eq 9-19
	ϕR_n =		= **82.10**	[kips]	
	ratio = **0.38**		> V_u	OK	

Bolt Group Eccentricity

Bolt group forces	shear V = 31.43 [kips]	axial P = 0.34	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= 31.43	[kips]	
Resultant force to ver Y axis angle	$\theta = \tan^{-1} (P / V)$	= 0.62	[°]	
Bolt group row and column	bolt row n_r = 3	bolt col n_c = 1		
Bolt row spacing	bolt row s_r = 3.000 [in]			
Shear force to bolt group CG ecc	e_x =	= 1.875	[in]	
Shear force to ver Y axis angle	θ =	= 0.62	[°]	
Bolt group coefficient C	C = from AISC 14th Table 7-6 ~ 7-13	= 2.288		
Bolt group eccentricity coefficient	$C_{ec} = C / (n_r \times n_c)$	= **0.763**		

Shear Tab / Beam Web - Bolt Shear

ratio = 31.43 / 40.96 = **0.77** **PASS**

Bolt group forces	shear V = 31.43 [kips]	axial P = -0.34	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= **31.43**	[kips]	
Bolt shear stress	bolt grade = A325-N	F_{nv} = 54.0	[ksi]	AISC 14th Table J3.2
	bolt dia d_b = 0.750 [in]	bolt area A_b = 0.442	[in^2]	
Number of bolt carried shear	n_s = 3.0	shear plane m = 1		
Bolt group eccentricity coefficient	C_{ec} = from 'Bolt Group Eccentricity' calc	= 0.763		
Required shear strength	V_u =	= **31.43**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 54.61	[kips]	AISC 14th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 14th Eq J3-1
	ϕR_n =	= **40.96**	[kips]	
	ratio = **0.77**	> V_u	OK	

Shear Tab to Hor Beam Web Weld Strength			ratio = 5.88 / 10.97	= **0.54**	**PASS**
Weld Group Forces					
	Shear V = 31.43 [kips]		Axial P = -0.34	[kips]	in tension
Shear force to bolt group CG ecc	$e_x =$		= 1.875	[in]	
Moment due to eccentric shear V	$M = V \times e_x$		= 4.91	[kip-ft]	
Shear tab weld length	$L =$		= 8.750	[in]	
Combined Weld Stress					
Weld stress from axial force	$f_a = P / L$		= -0.039	[kip/in]	in tension
Weld stress from shear force	$f_v = V / L$		= 3.592	[kip/in]	
Weld stress from moment force	$f_b = \frac{M}{L^2 / 6}$		= 4.618	[kip/in]	
Weld stress combined - max	$f_{max} = [\ (f_a - f_b)^2 + f_v^2\ ]^{0.5}$		= **5.881**	[kip/in]	AISC 14th Eq 8-11
Weld stress load angle	$\theta = \tan^{-1} (\frac{f_a - f_b}{f_v})$		= 52.4	[°]	
Fillet Weld Strength Calc					
Fillet weld leg size	$w = {}^{5}/_{16}$ [in]	load angle θ	= 52.4	[°]	
Electrode strength	$F_{EXX} = 70.0$ [ksi]	strength coeff C_1	= 1.00		AISC 14th Table 8-3
Number of weld line	n = 2 for double fillet				
Load angle coefficient	$C_2 = (1 + 0.5 \sin^{1.5} \theta)$		= 1.35		AISC 14th Page 8-9
Fillet weld shear strength	$R_{n-w} = 0.6 (C_1 \times 70 \text{ ksi}) 0.707\ w\ n\ C_2$		= 25.097	[kip/in]	AISC 14th Eq 8-1
Base metal - shear tab	thickness t = 0.375 [in]	tensile F_u	= 65.0	[ksi]	
Base metal - shear tab is in shear, <u>shear</u> rupture as per AISC 14th Eq J4-4 is checked					AISC 14th J2.4
Base metal shear rupture	$R_{n-b} = 0.6\ F_u\ t$		= 14.625	[kip/in]	AISC 14th Eq J4-4
Double fillet linear shear strength	$R_n = \min (R_{n-w}, R_{n-b})$		= **14.625**	[kip/in]	AISC 14th Eq 9-2
Resistance factor-LRFD	$\phi = 0.75$				AISC 14th Eq 8-1
	$\phi\ R_n =$		= **10.969**	[kip/in]	
	ratio = **0.54**		> f_{max}	OK	

Brace Force Load Case 2 Gusset plate t=0.500 P =45.00 kips (C) ratio = **0.86** **PASS**

Gusset Plate - Shear Yielding			ratio = 31.43 / 148.13	= **0.21**	**PASS**
Plate Shear Yielding Check					
Plate size	width $b_p = 9.875$ [in]	thickness t_p	= 0.500	[in]	
Plate yield strength	$F_y = 50.0$ [ksi]				
Plate gross area in shear	$A_{gv} = b_p\ t_p$		= 4.938	[in^2]	
Shear force required	$V_u =$		= **31.43**	[kips]	
Plate shear yielding strength	$R_n = 0.6\ F_y\ A_{gv}$		= 148.13	[kips]	AISC 14th Eq J4-3
Resistance factor-LRFD	$\phi = 1.00$				AISC 14th Eq J4-3
	$\phi\ R_n =$		= **148.13**	[kips]	
	ratio = **0.21**		> V_u	OK	

Gusset Plate - Shear Rupture			ratio = 31.43 / 106.03	= **0.30**	**PASS**

Plate Shear Rupture Check

Bolt hole diameter	bolt dia $d_b = {}^3/_4$	[in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 14th B4.3b
Number of bolt	$n = 3$				
Plate size	width $b_p = 9.875$	[in]	thickness $t_p = 0.500$	[in]	
Plate tensile strength	$F_u = 65.0$	[ksi]			
Plate net area in shear	$A_{nv} = (b_p - n\ d_h)\ t_p$		= 3.625	[in^2]	
Shear force required	$V_u =$		= **31.43**	[kips]	
Plate shear rupture strength	$R_n = 0.6\ F_u A_{nv}$		= 141.38	[kips]	AISC 14th Eq J4-4
Resistance factor-LRFD	$\phi = 0.75$				AISC 14th Eq J4-4
	$\phi\ R_n =$		= **106.03**	[kips]	
	ratio = **0.30**		> V_u	OK	

Gusset Plate Leg - Flexural Yielding			ratio = 26.06 / 30.47	= **0.86**	**PASS**

Refer to Bo Dowswell's paper 'Design of Wrap-Around Steel Gusset Plates' for more details on this limit state check

Shear on guseet leg & moment arm	shear V = 31.43	[kips]	ecc e = 9.950	[in]	
Moment on gusset plate leg	$M_u = V\ e$		= **26.06**	[kip-ft]	
Gusset plate leg size	width d = 9.875	[in]	thick t = 0.500	[in]	
Gusset plate steel strength	$F_y = 50.0$	[ksi]			
Moment on gusset plate leg	$R_n = F_y (t\ d^2 / 6)$		= 33.86	[kip-ft]	
Resistance factor-LRFD	$\phi = 0.90$				
	$\phi\ R_n =$		= **30.47**	[kips]	
	ratio = **0.86**		> M_u	OK	

Gusset Plate Leg - Lateral Torsional Buckling			ratio = 26.06 / 156.76	= **0.17**	**PASS**

Refer to Bo Dowswell's paper 'Design of Wrap-Around Steel Gusset Plates' for more details on this limit state check

Shear on guseet leg & moment arm	shear P = 31.43	[kips]	ecc e = 9.950	[in]	
Moment on gusset plate leg	$M_u = P\ e$		= **26.06**	[kip-ft]	
Gusset plate leg size	width d = 9.875	[in]	thick t = 0.500	[in]	
Gusset plate steel strength	E = 29000	[ksi]	G = 11200	[ksi]	
	$F_y = 50.0$	[ksi]			
Gusset leg buckiling length	L = distance from gusset load CG to gusset-beam interface line		= 10.005	[in]	
Critical moment - gusset leg	$R_n = 0.94 \sqrt{E\ G}\ \frac{d\ t^3}{L}$		= 174.17	[kip-ft]	Dowswell Paper Eq 9
Resistance factor-LRFD	$\phi = 0.90$				
	$\phi\ R_n =$		= **156.76**	[kip-ft]	
	ratio = **0.17**		> M_u	OK	

Shear Tab - Shear Yielding		ratio = 31.43 / 98.44	= **0.32**	**PASS**
Applied shear/axial forces	shear V = 31.43 [kips]	axial P = 0.34 [kips]		
Resultant shear force	$V_u = (V^2 + P^2)^{0.5}$	= **31.43** [kips]		
Plate Shear Yielding Check				
Plate size	width b_p = 8.750 [in]	thickness t_p = 0.375 [in]		
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$	= 3.281 [in^2]		
Shear force required	V_u =	= **31.43** [kips]		
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$	= 98.44 [kips]	AISC 14th Eq J4-3	
Resistance factor-LRFD	ϕ = 1.00		AISC 14th Eq J4-3	
	ϕR_n =	= **98.44** [kips]		
	ratio = **0.32**	> V_u	OK	

Shear Tab - Shear Rupture		ratio = 31.43 / 67.18	= **0.47**	**PASS**
Applied shear/axial forces	shear V = 31.43 [kips]	axial P = 0.34 [kips]		
Resultant shear force	$V_u = (V^2 + P^2)^{0.5}$	= **31.43** [kips]		
Plate Shear Rupture Check				
Bolt hole diameter	bolt dia $d_b = {}^3/_4$ [in]	bolt hole dia $d_h = {}^7/_8$ [in]	AISC 14th B4.3b	
Number of bolt	n = 3			
Plate size	width b_p = 8.750 [in]	thickness t_p = 0.375 [in]		
Plate tensile strength	F_u = 65.0 [ksi]			
Plate net area in shear	$A_{nv} = (b_p - n\ d_h)\ t_p$	= 2.297 [in^2]		
Shear force required	V_u =	= **31.43** [kips]		
Plate shear rupture strength	$R_n = 0.6 F_u A_{nv}$	= 89.58 [kips]	AISC 14th Eq J4-4	
Resistance factor-LRFD	ϕ = 0.75		AISC 14th Eq J4-4	
	ϕR_n =	= **67.18** [kips]		
	ratio = **0.47**	> V_u	OK	

Shear Tab - Flexural Yield Interact		ratio =	= **0.14**	**PASS**
Plate width & thick	width b_p = 8.750 [in]	thick t_p = 0.375 [in]		
	yield F_y = 50.0 [ksi]			
Shear plate - gross area	$A_g = b_p \times t_p$	= 3.281 [in^2]		
Shear plate - plastic modulus	$Z_p = (b_p \times t_p^2) / 4$	= 7.178 [in^3]		
Axial strength available	P_c = from axial tensile yield check	= 147.66 [kips]		
Axial strength required	P_r = from gusset interface forces calc	= 0.34 [kips]		
Shear strength available	V_c = from shear yielding check	= 98.44 [kips]		
Shear strength required	V_r = from gusset interface forces calc	= 31.43 [kips]		
Flexural strength available	$M_c = \phi F_y Z_p$ ϕ=0.90	= 26.92 [kip-ft]		
Flexural strength required	M_r = from gusset interface forces calc	= 4.91 [kip-ft]		
Flexural yield interaction	ratio = $(\frac{V_r}{V_c})^2 + (\frac{P_r}{P_c} + \frac{M_r}{M_c})^2$	= **0.14**	AISC 14th Eq 10-5	
		< 1.0	OK	

Shear Tab - Flexural Rupture Interact		ratio =		= **0.28**	**PASS**

Plate A_n and Z_{net} Calc

Bolt hole diameter	bolt dia $d_b = {}^3/_4$ [in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 14th B4.3b
Number of bolt	n = 3			
Plate size	width b_p = 8.750 [in]	thickness t_p = 0.375	[in]	
Plate net area	$A_n = (b_p - n\ d_h)\ t_p$	= 2.297	[in^2]	
Plate net plastic sect modulus	Z_{net} =	= 5.137	[in^3]	
Plate net elastic sect modulus	S_{net} =	= 3.421	[in^3]	

Plate width & thick	width b_p = 8.750 [in]	thick t_p = 0.375	[in]	
	tensile F_u = 65.0 [ksi]			
Axial strength available	P_c = from axial tensile rupture check	= 111.97	[kips]	
Axial strength required	P_r = from gusset interface forces calc	= 0.34	[kips]	
Shear strength available	V_c = from shear rupture check	= 67.18	[kips]	
Shear strength required	V_r = from gusset interface forces calc	= 31.43	[kips]	
Flexural strength available	$M_c = \phi\ F_u\ Z_{net}$ ϕ=0.75	= 20.87	[kip-ft]	AISC 14th Eq 9-4
Flexural strength required	M_r = from gusset interface forces calc	= 4.91	[kip-ft]	
Flexural rupture interaction	ratio $= \left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right)^2$	= **0.28**		AISC 14th Eq 10-5
		< 1.0	OK	

Shear Tab - Bolt Bearing on Shear Tab	ratio = 31.43 / 53.68	= **0.59**	**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 = 31.43 [kips]	axial P = 0.34	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= **31.43**	[kips]	
Resultant force/hor line load angle	$\theta = \tan^{-1} (V / P)$	= 89.38	[°]	
Bolt hole diameter	bolt dia d_b = 0.750 [in]	bolt hole dia d_{bh} = 0.813	[in]	AISC 14[th] B4.3b
Bolt hole ver. dimension	d_v =	= 0.813	[in]	
Bolt hole hor. dimension	d_h =	= 0.813	[in]	
Bolt center to bolt hole edge dist	$d_c = 0.5\, d_{bh}$	= 0.406	[in]	
Bolt no in ver & hor direction	Bolt Row n_v = 3	Bolt Col n_h = 1		
Bolt spacing	ver s_v = 3.000 [in]			
Bolt edge distance	ver e_v = 1.375 [in]	hor e_h = 1.375	[in]	
Bolt clear dist - bot right corner bolt	$L_{cA} = \min\left(\frac{e_v}{\sin\theta}, \frac{e_h}{\cos\theta} \right) - d_c$	= 0.969	[in]	
Bolt clear dist - right side edge bolt	$L_{cB} = \min\left(\frac{s_v - 0.5 d_v}{\sin\theta}, \frac{e_h}{\cos\theta} \right) - d_c$	= 2.188	[in]	

Single Bolt Shear Strength

Bolt shear stress	bolt grade = A325-N	F_{nv} = 54.0	[ksi]	AISC 14[th] Table J3.2
	bolt dia d_b = 0.750 [in]	bolt area A_b = 0.442	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$	= 23.86	[kips]	AISC 14[th] Eq J3-1
Bolt bearing on plate	thick t = 0.375 [in]	tensile F_u = 65.0	[ksi]	
Bolt bearing strength	$R_{n\text{-}br} = 3.0\, d_b\, t\, F_u$	= 54.84	[kips]	AISC 14[th] Eq J3-6b
Type A - Bolt Group Bottom Right Corner Bolt				
Number of bolt	$n_A = 1$			
Bolt tear out strength	$R_{n\text{-}tA} = 1.5\, L_{cA}\, t\, F_u$	= 35.42	[kips]	AISC 14[th] Eq J3-6b
Bolt bearing strength	$R_{nA} = \min(R_{n\text{-}tA}, R_{n\text{-}br}, R_{n\text{-}bolt})$	= **23.86**	[kips]	
Type B - Bolt Group Right Side Edge Bolt				
Number of bolt	$n_B = 2$			
Bolt tear out strength	$R_{n\text{-}tB} = 1.5\, L_{cB}\, t\, F_u$	= 79.99	[kips]	AISC 14[th] Eq J3-6b
Bolt bearing strength	$R_{nB} = \min(R_{n\text{-}tB}, R_{n\text{-}br}, R_{n\text{-}bolt})$	= **23.86**	[kips]	
Bolt bearing strength for all bolts	$R_n = n_A R_{nA} + n_B R_{nB} + n_C R_{nC} + n_D R_{nD}$	= 71.57	[kips]	
Bolt resistance factor-LRFD	$\phi = 0.75$			AISC 14[th] J3-10
	ϕR_n =	= **53.68**	[kips]	
	ratio = **0.59**	> R	OK	

Shear Tab - Beam Side - Block Shear - 1-Side Strip			ratio = 31.43 / 74.04	= **0.42**	**PASS**	
Plate Block Shear - Side Strip						
Bolt hole diameter	bolt dia $d_b = {}^3/_4$	[in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 14th B4.3b	
Plate thickness	t_p = 0.375	[in]				
Plate strength	F_y = 50.0	[ksi]	F_u = 65.0	[ksi]		
Bolt no in ver & hor dir	n_v = 1		n_h = 3			
Bolt spacing in hor dir	s_h = 3.000	[in]				
Bolt edge dist in ver & hor dir	e_v = 1.375	[in]	e_h = 1.375	[in]		
Gross area subject to shear	$A_{gv} = [\ (n_h - 1)\ s_h + e_h\ ]\ t_p$		= 2.766	[in^2]		
Net area subject to shear	$A_{nv} = A_{gv} - [\ (n_h - 1) + 0.5\ ]\ d_h\ t_p$		= 1.945	[in^2]		
Net area subject to tension	$A_{nt} = (\ e_v - 0.5\ d_h\ )\ t_p$		= 0.352	[in^2]		
Block shear strength required	V_u =		= **31.43**	[kips]		
Uniform tension stress factor	U_{bs} = 1.00				AISC 14th Fig C-J4.2	
Bolt shear resistance provided	$R_n = \min\ (0.6F_u A_{nv}\ ,\ 0.6F_y A_{gv}\ ) + U_{bs} F_u A_{nt}$		= 98.72	[kips]	AISC 14th Eq J4-5	
Resistance factor-LRFD	ϕ = 0.75				AISC 14th Eq J4-5	
	$\phi\ R_n$ =		= **74.04**	[kips]		
	ratio = **0.42**		> V_u	OK		

Shear Tab - Lateral Stability / Stabilizer Plate			ratio = 31.43 / 556.65	= **0.06**	**PASS**	
Applied shear/axial forces	shear V = 31.43	[kips]	axial P = 0.34	[kips]		
Resultant shear force	$V_u = (\ V^2 + P^2\ )^{0.5}$		= **31.43**	[kips]		
Distance from support to the first line of bolts	a =		= 1.875	[in]		
Plate thickness & depth	t_p = 0.375	[in]	L = 8.750	[in]		
Shear resistance provided	$R_n = 1500\ \pi \dfrac{L\ t_p^3}{a^2}$		= 618.50	[kips]	AISC 14th Eq 10-6	
Resistance factor-LRFD	ϕ = 0.90				AISC 14th Eq 10-6	
	$\phi\ R_n$ =		= **556.65**	[kips]		
	ratio = **0.06**		> V_u	OK		

Shear Tab - Plate Flexural Buckling		ratio = 31.43 / 82.10	= **0.38**	**PASS**
Shear tab size	depth = 8.750 [in]	thick = 0.375	[in]	
Plate buckling model	c = dist from support to fisrt bolt line	= 1.875	[in]	AISC 14th Fig. 9-3
	h_0 = shear tab depth	= 8.750	[in]	
	t_w = shear tab thick	= 0.375	[in]	
Shear tab steel yield stress	F_y = 50.0 [ksi]			
Plate buckling factor	$\lambda = \dfrac{h_0\sqrt{F_y}}{10\,t_w\sqrt{475+280\,(h_0/c)^2}}$	= 0.204		AISC 14th Eq 9-18
Plate buckling factor	Q =	= 1.000		AISC 14th Eq 9-15
Plate critical buckling stress	$F_{cr} = Q\,F_y$	= 50.0	[ksi]	AISC 14th Eq 9-14
Shear force in demand	V_u =	= **31.43**	[kips]	
Shear tab net elastic modulus	S_{net} =	= 3.421	[in^3]	
Shear force to bolt group CG ecc	a =	= 1.875	[in]	
Shear resistance	$R_n = F_{cr}\,S_{net} / a$	= 91.22	[kips]	AISC 14th Eq 9-19
Resistance factor-LRFD	ϕ = 0.90			AISC 14th Eq 9-19
	$\phi\,R_n$ =	= **82.10**	[kips]	
	ratio = **0.38**	> V_u	OK	

Shear Tab - Plate Shear / Axial Compression Interact		ratio =	= **0.15**		**PASS**
Shear Tab Forces					
	Shear V = 31.43 [kips]	Axial P = 0.34	[kips]	in compression	
Shear tab size	depth h_p = 8.750 [in]	thickness t_p = 0.375	[in]		
Shear tab material strength	F_y = 50.0 [ksi]	E = 29000	[ksi]		
Plate Compressive Capacity					
Plate gross area in compression	$A_g = h_p t_p$	= 3.281	[in^2]		
Plate radius of gyration	$r = t_p / \sqrt{12}$	= 0.108	[in]		
Plate effective length factor	K =	= 1.00			
Plate unbraced length	L_u =	= 1.875	[in]		
Plate slenderness	$KL/r = 1.00 \times L_u / r$	= 17.32			
	when $\frac{KL}{r} \leq 25$			AISC 14th J4.4 (a)	
Plate compression provided	$P_n = F_y \times A_g$	= 164.06	[kips]	AISC 14th Eq J4-6	
Axial compression force	P_u = from user input	= **0.34**	[kips]		
Plate Flexural Buckling Capacity					
Plate buckling model	c = dist from support to fisrt bolt line	= 1.875	[in]	AISC 14th Fig. 9-3	
	h_0 = shear tab depth	= 8.750	[in]		
	t_w = shear tab thick	= 0.375	[in]		
Plate buckling factor	$\lambda = \frac{h_0 \sqrt{F_y}}{10\, t_w \sqrt{475 + 280\,(h_0/c)^2}}$	= 0.204		AISC 14th Eq 9-18	
Plate buckling factor	Q =	= 1.000		AISC 14th Eq 9-15	
Plate critical buckling stress	$F_{cr} = Q F_y$	= 50.0	[ksi]	AISC 14th Eq 9-14	
Shear tab net elastic modulus	S_{net} =	= 3.421	[in^3]		
Shear force to bolt group CG ecc	a =	= 1.875	[in]		
Shear resistance	$V_n = F_{cr} S_{net} / a$	= **91.22**	[kips]	AISC 14th Eq 9-19	
Shear force	V_u = from user input	= **31.43**	[kips]		
Resistance factor-LRFD	$\phi = 0.90$			AISC 14th Eq 10-5	
Shear-axial interaction	$\text{ratio} = \left(\frac{V_r}{\phi V_n}\right)^2 + \left(\frac{P_r}{\phi P_n}\right)^2$	= **0.15**		AISC 14th Eq 10-5	
		< 1.0	OK		

Shear Tab - Compression Buckling		ratio = 0.34 / 147.66	= **0.00**	**PASS**
Plate Compression Check				
Plate size	width b_p = 8.750 [in]	thickness t_p = 0.375 [in]		
	F_y = 50.0 [ksi]	E = 29000 [ksi]		
Plate gross area in compression	$A_g = b_p t_p$	= 3.281 [in^2]		
Plate radius of gyration	$r = t_p / \sqrt{12}$	= 0.108 [in]		
Plate effective length factor	K =	= 1.00		
Plate unbraced length	L_u =	= 1.875 [in]		
Plate slenderness	$KL/r = 1.00 \times L_u / r$	= 17.32		
Plate compression required	P_u =	= **0.34** [kips]		
	when $\frac{KL}{r} \leq 25$		AISC 14th J4.4 (a)	
Plate compression provided	$R_n = F_y \times A_g$	= 164.06 [kips]	AISC 14th Eq J4-6	
Bolt resistance factor-LRFD	ϕ = 0.90		AISC 14th J4.4 (a)	
	ϕR_n =	= **147.66** [kips]		
	ratio = **0.00**	> P_u	OK	

Bolt Group Eccentricity			
Bolt group forces	shear V = 31.43 [kips]	axial P = 0.34 [kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= 31.43 [kips]	
Resultant force to ver Y axis angle	$\theta = \tan^{-1} (P / V)$	= 0.62 [°]	
Bolt group row and column	bolt row n_r = 3	bolt col n_c = 1	
Bolt row spacing	bolt row s_r = 3.000 [in]		
Shear force to bolt group CG ecc	e_x =	= 1.875 [in]	
Shear force to ver Y axis angle	θ =	= 0.62 [°]	
Bolt group coefficient C	C = from AISC 14th Table 7-6 ~ 7-13	= 2.288	
Bolt group eccentricity coefficient	$C_{ec} = C / (n_r \times n_c)$	= **0.763**	

Shear Tab / Beam Web - Bolt Shear		ratio = 31.43 / 40.96	= **0.77**	**PASS**
Bolt group forces	shear V = 31.43 [kips]	axial P = 0.34 [kips]		
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= **31.43** [kips]		
Bolt shear stress	bolt grade = A325-N	F_{nv} = 54.0 [ksi]	AISC 14th Table J3.2	
	bolt dia d_b = 0.750 [in]	bolt area A_b = 0.442 [in^2]		
Number of bolt carried shear	n_s = 3.0	shear plane m = 1		
Bolt group eccentricity coefficient	C_{ec} = from 'Bolt Group Eccentricity' calc	= 0.763		
Required shear strength	V_u =	= **31.43** [kips]		
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 54.61 [kips]	AISC 14th Eq J3-1	
Bolt resistance factor-LRFD	ϕ = 0.75		AISC 14th Eq J3-1	
	ϕR_n =	= **40.96** [kips]		
	ratio = **0.77**	> V_u	OK	

Shear Tab to Hor Beam Web Weld Strength		ratio = 31.43 / 89.28	= **0.35**	**PASS**
Weld Group Forces				
	Shear V = 31.43 [kips]	Axial P = 0.00 [kips] in compression		
Shear force to bolt group CG ecc	e_x =	= 1.875 [in]		
Shear tab weld length	L =	= 8.750 [in]		
Shear force in demand	V_u = from user input	= **31.43** [kips]		
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 14th 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 14th Page 8-9	
Fillet weld shear strength	$r_w = 0.6\,(C_1 \times 70\text{ ksi})\,0.707\,w\,n\,C_2$	= 18.559 [kip/in]	AISC 14th Eq 8-1	
Base metal - shear tab	thickness t = 0.375 [in]	tensile F_u = 65.0 [ksi]		
Base metal - shear tab is in shear, shear rupture as per AISC 14th Eq J4-4 is checked			AISC 14th J2.4	
Base metal shear rupture	$r_b = 0.6\,F_u\,t$	= 14.625 [kip/in]	AISC 14th Eq J4-4	
Weld stress reduction factor due to less base metal strength	$C_3 = r_b / r_w$ when $r_b < r_w$	= 0.788		
Table 8-4 Coefficient C for Eccentrically Loaded Weld Group			AISC 14th Table 8-4	
	$a = e_x / L$	= 0.21		
	C = C value in Table 8-4 when k=0	= 3.453		
Weld coefficients	C = 3.453	C_1 = 1.000		
		C_3 = 0.788		
Weld size & length	D = 5.000 [1/16]	L = 8.750 [in]		
Weld strength	$R_n = C\,C_1\,C_3\,D\,L$	= 119.04 [kips]	AISC 14th Table 8-4	
Resistance factor-LRFD	ϕ = 0.75			
	ϕR_n =	= **89.28** [kips]		
	ratio = **0.35**	> V_u	OK	