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	imit 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







Ver Beam W12X40

Hor Beam W12X30

Member Brace Connection Code=AISC 360-10 LRFD Hor Beam Section $M12X30$ d = 12.300 [in] b _f = 6.520 [in] W12X30 d = 12.400 [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] Z _x = 43.10 [in ³] Steel Grade A992 F _y = 50.0 [ksi] F _u = 65.0 [ksi]	Members & Components Summary						
Hor Beam Section d = 12.300 [in] $b_f = 6.520$ [in] 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]	nber E	race Connection			Code=AISC 360-10 LRFD		
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]	Beam Section						
$\begin{array}{ccccc} t_{f}=0.440 & [in] & t_{w}=0.260 & [in] \\ k_{des}=0.740 & [in] & k_{det}=1.125 & [in] \\ k_{1}=0.750 & [in] & A=8.790 & [in^{2}] \\ \\ S_{x}=38.60 & [in^{3}] & Z_{x}=43.10 & [in^{3}] \\ \end{array}$ Steel Grade A992 $F_{y}=50.0 & [ksi] & F_{u}=65.0 & [ksi] \end{array}$	2X30	d = 12.300	[in]	b _f = 6.520	[in]		
$k_{des} = 0.740$ [in] $k_{det} = 1.125$ [in] $k_1 = 0.750$ [in] $A = 8.790$ [in²] $S_x = 38.60$ [in³] $Z_x = 43.10$ [in³]Steel Grade A992 $F_y = 50.0$ [ksi] $F_u = 65.0$ [ksi]		$t_{f} = 0.440$	[in]	t _w = 0.260	[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]		$k_{des} = 0.740$	[in]	k _{det} = 1.125	[in]		
$S_x = 38.60$ [in ³] $Z_x = 43.10$ [in ³]Steel Grade A992 $F_y = 50.0$ [ksi] $F_u = 65.0$ [ksi]		$k_1 = 0.750$	[in]	A = 8.790	[in ²]		
Steel Grade A992 $F_y = 50.0$ [ksi] $F_u = 65.0$ [ksi]		S _x = 38.60	[in ³]	Z _x = 43.10	[in ³]		
	el Grade A992	$F_{y} = 50.0$	[ksi]	$F_{u} = 65.0$	[ksi]		
Ver Beam Section	Beam Section						
W12X40 $d = 11.900$ [in] $b_f = 8.010$ [in]	2X40	d = 11.900	[in]	b _f = 8.010	[in]		
$t_f = 0.515$ [in] $t_w = 0.295$ [in]		$t_{f} = 0.515$	[in]	t _w = 0.295	[in]		
k _{des} = 1.020 [in] k _{det} = 1.375 [in]		$k_{des} = 1.020$	[in]	k _{det} = 1.375	[in]		
$k_1 = 0.875$ [in] $A = 11.700$ [in ²]		$k_1 = 0.875$	[in]	A = 11.700	[in ²]		
$S_x = 51.50$ [in ³] $Z_x = 57.00$ [in ³]		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]	el Grade A992	$F_{y} = 50.0$	[ksi]	$F_{u} = 65.0$	[ksi]		

Gusset Plate Interface Forces Calculation						
H_{c} V_{c} M_{b} H_{b} H_{c} H_{b} H_{b						
Brace Axial Force Load Case 1						
Brace force	P = -45.00 [kips] (T)					
Refer to AISC 14 th Page 13-4 and Fig. 13-2 t	for all charts and definitions of variables and $e_b = 0.130$ [in] $\alpha = 10.483$ [in] $\theta = 45.0$ [°] $K = e_b \tan\theta - e_c$ $D = \tan^2\theta + (\frac{\alpha}{\beta})^2$ $K' = \alpha (\tan\theta + \frac{\alpha}{\beta})$ $\overline{\alpha} = [K' \tan\theta + K(\frac{\alpha}{\beta})^2] / D$ $\overline{\beta} = (K' - K \tan\theta) / D$	i symbols shou $e_c = 0.148$ $\beta = 14.820$ = -0.017 = 1.500 = 17.897 = 11.923 = 11.941	wn in calcu [in] [in] [in] [in]	AISC 14 th Eq. 13-16 AISC 14 th Eq. 13-24 AISC 14 th Eq. 13-23 AISC 14 th Eq. 13-21 AISC 14 th Eq. 13-21		
	r = [($e_{b} + \overline{\beta}$) ² + ($e_{c} + \overline{\alpha}$) ²] ^{0.5}	= 17.070	[in]	AISC 14 th Eq. 13-6		
Brace axial force Gusset to Ver Beam Interface Forces	P _u = from user input	= -45.00	[kips]	in tension		
Shear force	$V_c = (\overline{\beta} / r) P_u$	= -31.48	[kips]	AISC 14 th Eq. 13-2		
Axial force $H_c = (e_c/r)P_u$ = -0.39 [kips] AISC 14 th Eq. 13-3						
Moment	$M_c = H_c (\beta - \overline{\beta})$	= 0.09	[kip-ft]	AISC 14 th Eq. 13-19		
Gusset to Hor Beam Interface Forces						
Shear force	$H_{b} = (\overline{\alpha} / r) P_{u}$	= -31.43	[kips]	AISC 14 th Eq. 13-5		
Axial force	$V_b = (e_b/r)P_u$	= -0.34	[kips]	AISC 14 th Eq. 13-4		
Moment	$M_{b} = V_{b} (\alpha - \alpha)$	= -0.04	[kip-ft]	AISC 14 th 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 ra	tio = 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-min} =		= 1.000	[in]	AISC 14 th Table J3.4	
Min edge distance in WT Flange Gusset	to L _e =		= 1.250	[in]		
			> L _{e-min}	ОК		
Min Bolt Spacing - WT Flange to Gusset						
Bolt diameter	d _b =		= 0.750	[in]		
Min bolt spacing allowed	$L_{s-min} = 2.667 d_{b}$		= 2.000	[in]	AISC 14 th J3.3	
Min Bolt spacing in WT Flange to Gusset	C L _s =		= 3.000	[in]		
			> L _{s-min}	ОК		

Brace Force Load Case 1 Sect=WT4X12 P =-45.00 kips (T) ratio = 0.6
--

WT Shape Brace - Tensile Yield ratio = 45.00 / 159.30 = 0.28 PASS = 3.540 [in²] Gross area subject to tension $A_g =$ Steel yield strength $F_y =$ = 50.0 [ksi] Tensile force required P_u = = 45.00 [kips] $R_n = F_y A_g$ AISC 14 th Eq D2-1 Tensile yielding strength = 177.00 [kips] AISC 14th D2 (a) Resistance factor-LRFD $\phi = 0.90$ AISC 14th Eq D2-1 $\phi R_n =$ = 159.30 [kips] $> P_u$ ratio = 0.28 ОК

PASS

WT Shape Brace - Tensile Rupt	WT Shape Brace - Tensile Rupture			PASS
Section gross area	$A_g = WT4X12$	= 3.540	[in ²]	
Bolt hole diameter	bolt dia $d_b = \frac{3}{4}$ [in]	bolt hole dia $d_h = \frac{7}{8}$	[in]	AISC 14 th 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 14 th 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 14 th Eq D2-2
Resistance factor-LRFD	φ = 0.75			AISC 14 th D2 (b)
	φ R _n =	= 106.38	[kips]	AISC 14 th Eq D2-2
	ratio = 0.42	> P _u	ОК	

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 14 th 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	$R_n = F_{nv}A_bn_smC_{ec}$	= 95.43	[kips]	AISC 14 th Eq J3-1
Bolt resistance factor-LRFD	φ = 0.75			AISC 14 th Eq J3-1
	φ R _n =	= 71.57	[kips]	
	ratio = 0.63	> V _u	ОК	

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 14 th Table J3.2
	bolt dia $d_b = 0.750$	[in]	bolt area $A_b = 0.442$	[in ²]	
Single bolt shear strength	$R_{n-bolt} = F_{nv}A_b$		= 23.86	[kips]	AISC 14 th Eq J3-1
Bolt Bearing/TearOut Strength of	on Plate				
Bolt hole diameter	bolt dia $d_b = \frac{3}{4}$	[in]	bolt hole dia d _h = ${}^{13}\!/_{16}$	[in]	AISC 14 th 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	$R_{n-t\&b-in} = 1.5 L_c t$	F _u ≤ 3.0 d _b t F	u		AISC 14 th Eq J3-6b
	= 85.31 ≤	58.50	= 58.50	[kips]	
Bolt strength at interior	R _{n-in} = min (R	n-t&b-in,R _{n-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	$R_{n-t\&b-ed} = 1.5 L_c t$	F _u ≤ 3.0 d _b t F	u		AISC 14 th Eq J3-6b
	= 37.78 ≤	58.50	= 37.78	[kips]	
Bolt strength at edge	$R_{n-ed} = min (R)$	n-t&b-ed , R _{n-bolt}) = 23.86	[kips]	
Number of bolt	interior n _{in} = 2		edge n _{ed} = 2		
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n-in}$	+ n _{ed} R _{n-ed}	= 95.43	[kips]	
Required shear strength	V _u =		= 45.00	[kips]	
Bolt resistance factor-LRFD	φ = 0.75				AISC 14 th J3-10
	φ R _n =		= 71.57	[kips]	
	ratio = 0.63		> V _u	ОК	

WT Brace - Bolt Bearing on Gusset Plate rational rationa			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 14 th Table J3.2
	bolt dia d _b = 0.750	[in]	bolt area $A_b = 0.442$	[in ²]	
Single bolt shear strength	$R_{n-bolt} = F_{nv}A_b$		= 23.86	[kips]	AISC 14 th Eq J3-1
Bolt Bearing/TearOut Strength of	on Plate				
Bolt hole diameter	bolt dia $d_b = \frac{3}{4}$	[in]	bolt hole dia d _h = 13 / ₁₆	[in]	AISC 14 th 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	$R_{n-t\&b-in} = 1.5 L_c t$	F _u ≤ 3.0 d _b t F	u		AISC 14 th Eq J3-6b
	= 106.64	≤ 73.13	= 73.13	[kips]	
Bolt strength at interior	R _{n-in} = min (R	n-t&b-in,R _{n-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	$R_{n-t\&b-ed} = 1.5 L_c t$	F _u ≤ 3.0 d _b t F	u		AISC 14 th Eq J3-6b
	= 59.41 ≤	73.13	= 59.41	[kips]	
Bolt strength at edge	$R_{n-ed} = min (R)$	n-t&b-ed , R _{n-bolt}) = 23.86	[kips]	
Number of bolt	interior n _{in} = 2		edge n _{ed} = 2		
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n-in}$	+ n _{ed} R _{n-ed}	= 95.43	[kips]	
Required shear strength	V _u =		= 45.00	[kips]	
Bolt resistance factor-LRFD	φ = 0.75				AISC 14 th J3-10
	$\phi R_n =$		= 71.57	[kips]	
	ratio = 0.63		> V _u	ОК	

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 $d_b = \frac{3}{4}$	[in]	bolt hole dia $d_h = \frac{7}{8}$	[in]	AISC 14 th 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	$A_{gv} = [(n_{h} - 1)]$	l)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 14 th Fig C-J4.2
Bolt shear resistance provided	$R_n = min (0.0)$ $U_{bs}F_u A$	6F _u A _{nv} , 0.6F _y A A _{nt}	$(A_{gv}) + = 68.90$	[kips]	AISC 14 th Eq J4-5
Resistance factor-LRFD	φ = 0.75				AISC 14 th Eq J4-5
	φ R _n =		= 51.68	[kips]	
	ratio = 0.44		> V _u	ОК	

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 14 th Eq J4-1		
Resistance factor-LRFD	φ = 0.90				AISC 14 th Eq J4-1		
	φ R _n =		= 167.94	[kips]			
	ratio = 0.27		> P _u	ОК			
Gusset Plate - Tensile Rupture (Gusset Plate - Tensile Rupture (Whitmore)			= 0.32	PASS		
Plate Tensile Runture Check							

Plate Tensile Rupture Check					
Bolt hole diameter	bolt dia $d_b = \frac{3}{4}$	[in]	bolt hole dia $d_h = \frac{7}{8}$	[in]	AISC 14 th 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 14 th Eq J4-2
Resistance factor-LRFD	φ = 0.75				AISC 14 th Eq J4-2
	φ R _n =		= 139.28	[kips]	AISC 14 th Eq J4-2
	ratio = 0.32		> P _u	ОК	

Gusset Plate - Block Shear - Ce	enter Strip		ratio = 45.00 /	/ 173.06	= 0.26	PASS
Plate Block Shear - Center Strip						
Bolt hole diameter	bolt dia $d_b = \frac{3}{4}$	[in]	bolt hole dia d _h =	= ⁷ / ₈	[in]	AISC 14 th 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	l)s _h +e _h]t _p	x 2 =	= 4.625	[in ²]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_{mv} - n_{mv})]$	n _h -1)+0.5]	d _h t _p x2 =	= 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 14 th Fig C-J4.2
Bolt shear resistance provided	R _n = min (0.4 U _{bs} F _u A	6F _u A _{nv} , 0.6F	_y A _{gv})+ =	= 230.75	[kips]	AISC 14 th Eq J4-5
Resistance factor-LRFD	φ = 0.75					AISC 14 th Eq J4-5
	φ R _n =		=	= 173.06	[kips]	
	ratio = 0.26		;	> V _u	ОК	

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 14 th 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	$R_n = F_{nv}A_bn_sm C_{ec}$	= 95.43	[kips]	AISC 14 th Eq J3-1
Bolt resistance factor-LRFD	φ = 0.75			AISC 14 th Eq J3-1
	φ R _n =	= 71.57	[kips]	
	ratio = 0.63	> V _u	ОК	

WT Brace - Bolt Bearing on WT Flange rati			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 14 th Table J3.2
	bolt dia d _b = 0.750	[in]	bolt area $A_b = 0.442$	2 [in ²]	
Single bolt shear strength	$R_{n-bolt} = F_{nv}A_b$		= 23.80	5 [kips]	AISC 14 th Eq J3-1
Bolt Bearing/TearOut Strength o	on Plate				
Bolt hole diameter	bolt dia $d_b = \frac{3}{4}$	[in]	bolt hole dia d _h = ${}^{13}\!/_{16}$	[in]	AISC 14 th 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	$L_c = L_s - d_h$		= 2.18	3 [in]	
Bolt tear out/bearing strength	$R_{n-t\&b-in} = 1.5 L_c t$	F _u ≤ 3.0 d _b t m	F _u		AISC 14 th Eq J3-6b
	= 85.31 ≤	58.50	= 58.50) [kips]	
Bolt strength at interior	R _{n-in} = min (R	_{n-t&b-in} , R _{n-bolt})	= 23.80	6 [kips]	
Number of bolt	interior n _{in} = 4				
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n-in}$	l	= 95.43	3 [kips]	
Required shear strength	V _u =		= 45.0	0 [kips]	
Bolt resistance factor-LRFD	φ = 0.75				AISC 14 th J3-10
	φ R _n =		= 71.5	7 [kips]	
	ratio = 0.63		> V _u	ОК	

WT Brace - Bolt Bearing on Gu	isset 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 14 th Table J3.2
	bolt dia d _b = 0.750	[in]	bolt area $A_b = 0.442$	[in ²]	
Single bolt shear strength	$R_{n-bolt} = F_{nv}A_b$		= 23.86	[kips]	AISC 14 th Eq J3-1
Bolt Bearing/TearOut Strength o	on Plate				
Bolt hole diameter	bolt dia $d_b = \frac{3}{4}$	[in]	bolt hole dia d _h = ${}^{13}\!/_{16}$	[in]	AISC 14 th 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	$R_{n-t\&b-in} = 1.5 L_c t$	F _u ≤ 3.0 d _b t m	F _u		AISC 14 th Eq J3-6b
	= 106.64	≤ 73.13	= 73.13	[kips]	
Bolt strength at interior	R _{n-in} = min (R	_{n-t&b-in} , R _{n-bolt})	= 23.86	[kips]	
Number of bolt	interior n _{in} = 4				
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n-in}$	1	= 95.43	[kips]	
Required shear strength	V _u =		= 45.00	[kips]	
Bolt resistance factor-LRFD	φ = 0.75				AISC 14 th J3-10
	φ R _n =		= 71.57	[kips]	
	ratio = 0.63		> V _u	ОК	

Gusset Plate - Compression (\	Whitmore)	ratio = 45.00 / 140.03	= 0.32	PASS
Plate Compression Check				
Plate size	 width b = 7.464 [in]	thickness $t_n = 0.500$	[in]	
	$F_v = 50.0$ [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	К =	= 0.50		
Plate unbraced length	L _u =	= 14.392	[in]	
Plate slenderness	$KL/r = 0.50 \times L_u / r$	= 49.86		
	when $\frac{KL}{r}$ > 25 , use Ch	apter E		AISC 14 th J4.4 (b)
Elastic buckling stress	$F_{e} = \frac{\pi^{2} E}{(KL/r)^{2}}$	= 115.15	[ksi]	AISC 14 th Eq E3-4
	when $\frac{KL}{r} \leq 4.71 \left(\frac{E}{F_{y}}\right)$) ^{0.5} = 113.43		AISC 14 th E3 (a)
Critical stress	$F_{cr} = 0.658 (F_y/F_e) F_y$	= 41.69	[ksi]	AISC 14 th 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 14 th Eq E3-1
Bolt resistance factor-LRFD	φ = 0.90			AISC 14 th E1
	φ R _n =	= 140.03	[kips]	
	ratio = 0.32	> P _u	ОК	
Gusset Plate - Block Shear - C	enter Strip	ratio = 45.00 / 173.06	= 0.26	PASS
Plate Block Shear - Center Strip				
Bolt hole diameter	bolt dia $d_b = \frac{3}{4}$ [in]	bolt hole dia $d_h = \frac{7}{8}$	[in]	AISC 14 th 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	$A_{gv} = [(n_h - 1)s_h + e_h]$	$t_p x 2 = 4.625$	[in ²]	
Net area subject to shear	$A_{nv} = A_{qv} - [(n_h - 1) + 0.5]$	$] d_h t_p x^2 = 3.313$	[in ²]	
Net area subject to tension	5			
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 14 th Fig C-J4.2
Bolt shear resistance provided	$R_{n} = \min (0.6F_{u}A_{nv}, 0.6)$ $U_{bs}F_{u}A_{nt}$	$(F_y A_{gv}) + = 230.75$	[kips]	AISC 14 th Eq J4-5
Resistance factor-LRFD	$\phi = 0.75$			AISC 14 th Eq J4-5
	φ R _n =	= 173.06	[kips]	
	ratio = 0.26	> V	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 Chec	ks - Shear Tab to Ver Beam Web		PASS
Min Bolt Edge Distance - She	ar Tab to Ver Beam Web		
Bolt diameter	d _b =	= 0.750 [in]	
Min edge distance allowed	L _{e-min} =	= 1.000 [in]	AISC 14 th Table J3.4
Min edge distance in Shear Tab t Ver Beam Web	L _e =	= 1.375 [in]	
		> L _{e-min} OK	
Min Bolt Spacing - Shear Tab	to Ver Beam Web		
Bolt diameter	d _b =	= 0.750 [in]	
Min bolt spacing allowed	L _{s-min} = 2.667 d _b	= 2.000 [in]	AISC 14 th J3.3
Min Bolt spacing in Shear Tab to Beam Web	Ver L _s =	= 3.000 [in]	
		> L _{s-min} OK	I
Weld Limitation Check - Sh	ear 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 14 th 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 14 th 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] AI	SC 14 th Eq J4-3
Resistance factor-LRFD	$\varphi = 1.00$	AI	SC 14 th Eq J4-3
	φ 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 = $\frac{3}{4}$	[in]	bolt hole dia $d_h = \frac{7}{8}$	[in]	AISC 14 th 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 14 th Eq J4-4
Resistance factor-LRFD	φ = 0.75				AISC 14 th Eq J4-4
	$\phi R_n =$		= 106.03	[kips]	
	ratio = 0.30		> V _u	ОК	

Gusset Plate Leg - Flexural Yieldi	ng		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	$R_n = F_y (t d^2)$	2/6)	= 33.86	[kip-ft]				
Resistance factor-LRFD	φ = 0.90							
	φ R _n =		= 30.47	[kips]				
	ratio = 0.48		> M _u	ОК				
Gusset Plate Leg - Lateral Torsion	nal Buckling		ratio = 14.77 / 147.54	= 0.10	PASS			
Refer to Bo Dowswell's paper 'Design of	Wrap-Around Steel	Gusset Plates' fo	r more details on this limit	state check	< colored and set of the set of t			
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 gusset-l	e from gusset load beam interface lir	d CG to = 10.630	[in]				
Critical moment - gusset leg	$R_{n} = 0.94 $	$EG \frac{dt^3}{L}$	= 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	ОК				

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	$V_{u} = (V^{2} + F)$	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	$R_{n} = 0.6 F_{y} A$	A _{gv}	= 98.44	[kips]	AISC 14 th Eq J4-3
Resistance factor-LRFD	$\phi = 1.00$				AISC 14 th Eq J4-3
	φ R _n =		= 98.44	[kips]	
	ratio = 0.32		> V _u	ОК	
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	$V_{u} = (V^{2} + F)$	²) ^{0.5}	= 31.48	[kips]	
Plate Shear Rupture Check					
Bolt hole diameter	bolt dia $d_b = \frac{3}{4}$	[in]	bolt hole dia d _h = $\frac{7}{8}$	[in]	AISC 14 th 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	$R_{n} = 0.6 F_{u}A$	A _{nv}	= 89.58	[kips]	AISC 14 th Eq J4-4
Resistance factor-LRFD	φ = 0.75				AISC 14 th Eq J4-4
	φ R _n =		= 67.18	[kips]	
	ratio = 0.47		> V _u	ОК	
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 14 th Eq J4-1
Resistance factor-LRFD	φ = 0.90				AISC 14 th Eq J4-1
	φ R _n =		= 147.66	[kips]	
	ratio = 0.00		> P _u	ОК	

Shear Tab - Axial Tensile Rupture			ratio = 0.39 ,	/ 111.97	= 0.00	PASS
Plate Tensile Rupture Check						
Bolt hole diameter	bolt dia $d_b = \frac{3}{4}$	[in]	bolt hole dia d _r	$= \frac{7}{8}$	[in]	AISC 14 th 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 14 th Eq J4-2
Resistance factor-LRFD	φ = 0.75					AISC 14 th Eq J4-2
	φ R _n =			= 111.97	[kips]	AISC 14 th Eq J4-2
	ratio = 0.00			> P _u	ОК	
Shear Tab - Flexural Yield Inter	ract		ratio =		= 0.14	PASS
Plate width & thick	width $b_p = 8.750$	[in]	thick t _r	= 0.375	[in]	
Plate width & thick	width $b_p = 8.750$ yield $F_y = 50.0$	[in] [ksi]	thick t _r	,= 0.375	[in]	
Plate width & thick Shear plate - gross area	width $b_p = 8.750$ yield $F_y = 50.0$ $A_g = b_p x t_p$	[in] [ksi]	thick t _r	= 0.375 = 3.281	[in] [in ²]	
Plate width & thick Shear plate - gross area Shear plate - plastic modulus	width $b_p = 8.750$ yield $F_y = 50.0$ $A_g = b_p x t_p$ $Z_p = (b_p x t_p^2)$	[in] [ksi] 2) / 4	thick t _r	= 0.375 = 3.281 = 7.178	[in] [in ²] [in ³]	
Plate width & thick Shear plate - gross area Shear plate - plastic modulus Axial strength available	width $b_p = 8.750$ yield $F_y = 50.0$ $A_g = b_p x t_p$ $Z_p = (b_p x t_p^2)$ $P_c = from ax$	[in] [ksi] 2) / 4 ial tensile yield c	thick t _r :heck	= 0.375 = 3.281 = 7.178 = 147.66	[in] [in ²] [in ³] [kips]	
Plate width & thick Shear plate - gross area Shear plate - plastic modulus Axial strength available Axial strength required	width $b_p = 8.750$ yield $F_y = 50.0$ $A_g = b_p x t_p$ $Z_p = (b_p x t_p^2)$ $P_c = from ax$ $P_r = from gu$	[in] [ksi] 2) / 4 ial tensile yield c sset interface for	thick t _r heck rces calc	= 0.375 = 3.281 = 7.178 = 147.66 = 0.39	[in] [in ²] [in ³] [kips]	
Plate width & thick Shear plate - gross area Shear plate - plastic modulus Axial strength available Axial strength required Shear strength available	width $b_p = 8.750$ yield $F_y = 50.0$ $A_g = b_p x t_p$ $Z_p = (b_p x t_p^2)$ $P_c = from ax$ $P_r = from gu$ $V_c = from show$	[in] [ksi] 2) / 4 ial tensile yield c sset interface for ear yielding chec	thick t _r :heck rces calc :k	= 0.375 = 3.281 = 7.178 = 147.66 = 0.39 = 98.44	[in] [in ²] [in ³] [kips] [kips] [kips]	
Plate width & thick Shear plate - gross area Shear plate - plastic modulus Axial strength available Axial strength required Shear strength required	width $b_p = 8.750$ yield $F_y = 50.0$ $A_g = b_p x t_p$ $Z_p = (b_p x t_p^2)$ $P_c = from ax$ $P_r = from gu$ $V_c = from showned the second $	[in] [ksi] 2) / 4 ial tensile yield c sset interface for ear yielding chec sset interface for	thick t _r check rces calc ck rces calc	= 0.375 = 3.281 = 7.178 = 147.66 = 0.39 = 98.44 = 31.48	[in] [in ²] [in ³] [kips] [kips] [kips]	
Plate width & thick Shear plate - gross area Shear plate - plastic modulus Axial strength available Axial strength required Shear strength available Shear strength required Flexural strength available	width $b_p = 8.750$ yield $F_y = 50.0$ $A_g = b_p \times t_p$ $Z_p = (b_p \times t_p^2)$ $P_c = \text{from ax}$ $P_r = \text{from gu}$ $V_c = \text{from shu}$ $V_r = \text{from gu}$ $M_c = \phi F_y Z_p$	[in] [ksi] $\frac{2}{2}$) / 4 ial tensile yield of sset interface for ear yielding check sset interface for $\phi=0.90$	thick t _r check rces calc ck rces calc	= 0.375 = 3.281 = 7.178 = 147.66 = 0.39 = 98.44 = 31.48 = 26.92	[in] [in ²] [kips] [kips] [kips] [kips] [kips]	
Plate width & thick Shear plate - gross area Shear plate - plastic modulus Axial strength available Axial strength required Shear strength required Flexural strength available Flexural strength required	width $b_p = 8.750$ yield $F_y = 50.0$ $A_g = b_p x t_p$ $Z_p = (b_p x t_f^2)$ $P_c = \text{from ax}$ $P_r = \text{from gu}$ $V_c = \text{from gu}$ $M_c = \phi F_y Z_p$ $M_r = \text{from gu}$	[in] [ksi] $\frac{2}{5}$) / 4 ial tensile yield of sset interface for ear yielding check sset interface for $\phi=0.90$ sset interface for	thick t _r check rces calc ck rces calc rces calc	= 0.375 = 3.281 = 7.178 = 147.66 = 0.39 = 98.44 = 31.48 = 26.92 = 4.92	[in] [in ²] [kips] [kips] [kips] [kips] [kips] [kip-ft]	
Plate width & thick Shear plate - gross area Shear plate - plastic modulus Axial strength available Axial strength required Shear strength available Shear strength required Flexural strength required Flexural yield interaction	width $b_p = 8.750$ yield $F_y = 50.0$ $A_g = b_p \times t_p$ $Z_p = (b_p \times t_p^2)$ $P_c = \text{from ax}$ $P_r = \text{from gu}$ $V_c = \text{from gu}$ $V_c = \text{from gu}$ $M_c = \phi F_y Z_p$ $M_r = \text{from gu}$ ratio = $(\frac{V_r}{V_c})^2$	[in] [ksi] $\frac{2}{5}$) / 4 ial tensile yield of sset interface for ear yielding check sset interface for $\phi = 0.90$ sset interface for $\phi = 0.90$ $\phi = (\frac{P_r}{P_c} + \frac{M_r}{M_c})$	thick t _r theck rces calc tk rces calc rces calc	= 0.375 = 3.281 = 7.178 = 147.66 = 0.39 = 98.44 = 31.48 = 26.92 = 4.92 = 0.14	[in] [in ²] [kips] [kips] [kips] [kips] [kip-ft] [kip-ft]	AISC 14 th Eq 10-5

Shear Tab - Flexural Rupture I	nteract		ratio =		= 0.28	PASS
Plate A _n and Z _{net} Calc						
Bolt hole diameter	bolt dia $d_b = \frac{3}{4}$	[in]	bolt hole dia d	$h = \frac{7}{8}$	[in]	AISC 14 th 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 ax	kial tensile rupti	ure check	= 111.97	[kips]	
Axial strength required	P _r = from gu	usset interface f	forces calc	= 0.39	[kips]	
Shear strength available	$V_c = $ from sh	near rupture che	eck	= 67.18	[kips]	
Shear strength required	V _r = from gu	usset interface f	forces calc	= 31.48	[kips]	
Flexural strength available	$M_c = \phi F_u Z_n$	_{et} φ=0.75		= 20.87	[kip-ft]	AISC 14 th Eq 9-4
Flexural strength required	M _r = from gu	usset interface f	forces calc	= 4.92	[kip-ft]	
Flexural rupture interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2$	$P_{\rm c}^2$ + ($\frac{P_{\rm r}}{P_{\rm c}}$ + $\frac{M_{\rm r}}{M_{\rm c}}$	r_) ²	= 0.28		AISC 14 th Eq 10-5
				< 1.0	OK	

Shear Tab - Bolt Bearing on Sh	ear Tab	ratio = 31.48 / 53.68	3 = 0.59	PASS
The bolt group is oriented so that the	e shear force V is in ver. dire	ction and the axial force P is in hor	direction	
Bolt group forces	shear V = 31.48 [kip	s] axial P = -0.39	Ə [kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= 31.4	8 [kips]	
Resultant force/hor line load angle	$\theta = \tan^{-1} (V / P)$	= 89.2	9 [°]	
Bolt hole diameter	bolt dia $d_b = 0.750$ [in]	bolt hole dia d _{bh} = 0.81	3 [in]	AISC 14 th B4.3b
Bolt hole ver. dimension	d _v =	= 0.81	3 [in]	
Bolt hole hor. dimension	d _h =	= 0.81	3 [in]	
Bolt center to bolt hole edge dist	$d_c = 0.5 d_{bh}$	= 0.40	6 [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 = 1.37	5 [in]	
Bolt clear dist - bot right corner bolt	$L_{cA} = min \left(\frac{e_v}{\sin \theta} \right)$	$(\frac{e_{h}}{\cos \theta}) - d_{c} = 0.96$	9 [in]	
Bolt clear dist - right side edge bolt	$L_{cB} = \min(\frac{s_v - 0.5}{\sin \theta})$	$\frac{d_v}{d_v}$, $\frac{e_h}{\cos \theta}$) - d_c = 2.18	8 [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 $A_b = 0.44$	2 [in ²]	
Single bolt shear strength	$R_{n-bolt} = F_{nv} A_b$	= 23.8	6 [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	$R_{n-br} = 3.0 d_b t F_u$	= 54.8	4 [kips]	AISC 14 th Eq J3-6b
Type A - Bolt Group Bottom Right Co	rner Bolt			
Number of bolt	n _A = 1			
Bolt tear out strength	$R_{n-tA} = 1.5 L_{cA} t F_{u}$	= 35.4	2 [kips]	AISC 14 th Eq J3-6b
Bolt bearing strength	$R_{nA} = min (R_{n-tA}, R_{n-tA})$	R_{n-br}, R_{n-bolt} = 23.8	6 [kips]	
Type B - Bolt Group Right Side Edge	Bolt			
Number of bolt	n _B = 2			
Bolt tear out strength	$R_{n-tB} = 1.5 L_{cB} t F_{u}$	= 79.9	9 [kips]	AISC 14 th Eq J3-6b
Bolt bearing strength	$R_{nB} = min (R_{n-tB}, R_{n-tB})$	R_{n-br}, R_{n-bolt} = 23.8	6 [kips]	
Bolt bearing strength for all bolts	$R_n = n_A R_{nA} + n_B R_{nA}$	$R_{nB} + n_C R_{nC} + n_D R_{nD} = 71.5$	7 [kips]	
Bolt resistance factor-LRFD	φ = 0.75			AISC 14 th J3-10
	φ R _n =	= 53.6	8 [kips]	
	ratio = 0.59	> R	ОК	

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 $d_b = \frac{3}{4}$	[in]	bolt hole dia $d_h = \frac{7}{8}$	[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	$A_{gv} = [(n_{h} - 1)]$	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 14 th Fig C-J4.2
Bolt shear resistance provided	$R_n = min (0.$	6F _u A _{nv} , 0.6F _y	A _{gv}) + = 98.72	[kips]	AISC 14 th Eq J4-5
Resistance factor-I RFD	U _{bs} F _u λ Φ = 0.75	۹ _{nt}			AISC 14 th Ea 14-5
	φ = 0.75 φ R =		= 74.04	[kins]	
	ratio = 0.43		> V	OK	
				_	
Shear Tab - Beam Side-Axial T	earout - Block Shea	r - Center Str	ip ratio = 0.39 / 98.26	= 0.00	PASS
Diata Diack Shoar Contor Strin					
Bolt hole diameter	bolt dia d _b = $\frac{3}{4}$	[in]	bolt hole dia d _h = $\frac{7}{8}$	[in]	AISC 14 th B4.3b
Bolt hole diameter	bolt dia $d_{b} = \frac{3}{4}$ $t_{p} = 0.375$	[in] [in]	bolt hole dia d _h = $\frac{7}{8}$	[in]	AISC 14 th B4.3b
Bolt hole diameter Plate thickness Plate strength	bolt dia $d_b = {}^{3/}_{4}$ $t_p = 0.375$ $F_y = 50.0$	[in] [in] [ksi]	bolt hole dia d _h = $\frac{7}{8}$ F _u = 65.0	[in] [ksi]	AISC 14 th B4.3b
Bolt hole diameter Plate thickness Plate strength Bolt no in ver & hor dir	bolt dia $d_{b} = \frac{3}{4}$ $t_{p} = 0.375$ $F_{y} = 50.0$ $n_{v} = 3$	[in] [in] [ksi]	bolt hole dia d _h = $\frac{7}{8}$ F _u = 65.0 n _h = 1	[in] [ksi]	AISC 14 th B4.3b
Bolt hole diameter Plate thickness Plate strength Bolt no in ver & hor dir Bolt spacing in ver & hor dir	bolt dia $d_b = \frac{3}{4}$ $t_p = 0.375$ $F_y = 50.0$ $n_v = 3$ $s_v = 3.000$	[in] [in] [ksi] [in]	bolt hole dia d _h = $\frac{7}{8}$ F _u = 65.0 n _h = 1 s _h = 3.000	[in] [ksi] [in]	AISC 14 th B4.3b
Bolt hole diameter Plate thickness Plate strength Bolt no in ver & hor dir Bolt spacing in ver & hor dir Bolt edge dist in ver & hor dir	bolt dia $d_b = \frac{3}{4}$ $t_p = 0.375$ $F_y = 50.0$ $n_v = 3$ $s_v = 3.000$ $e_v = 1.375$	[in] [in] [ksi] [in] [in]	bolt hole dia d _h = $\frac{7}{8}$ F _u = 65.0 n _h = 1 s _h = 3.000 e _h = 1.375	[in] [ksi] [in] [in]	AISC 14 th B4.3b
Bolt hole diameter Plate thickness Plate strength Bolt no in ver & hor dir Bolt spacing in ver & hor dir Bolt edge dist in ver & hor dir Gross area subject to shear	bolt dia $d_b = \frac{3}{4}$ $t_p = 0.375$ $F_y = 50.0$ $n_v = 3$ $s_v = 3.000$ $e_v = 1.375$ $A_{gv} = [(n_h - 1)^2 + 10^2 +$	[in] [in] [ksi] [in] [in] 1) s _h + e _h] t _p >	bolt hole dia $d_{h} = \frac{7}{8}$ $F_{u} = 65.0$ $n_{h} = 1$ $s_{h} = 3.000$ $e_{h} = 1.375$ x = 2 x = 1.031	[in] [ksi] [in] [in] [in ²]	AISC 14 th B4.3b
Bolt hole diameter Plate thickness Plate strength Bolt no in ver & hor dir Bolt spacing in ver & hor dir Bolt edge dist in ver & hor dir Gross area subject to shear Net area subject to shear	bolt dia $d_b = \frac{3}{4}$ $t_p = 0.375$ $F_y = 50.0$ $n_v = 3$ $s_v = 3.000$ $e_v = 1.375$ $A_{gv} = [(n_h - 1) - 1]$ $A_{nv} = A_{gv} - [(n_v - 1) - 1]$	[in] [in] [ksi] [in] 1) s _h + e _h] t _p > n _h - 1)+ 0.5] d	bolt hole dia $d_h = \frac{7}{8}$ $F_u = 65.0$ $n_h = 1$ $s_h = 3.000$ $e_h = 1.375$ x = 2 $h_h t_p x = 0.703$	[in] [ksi] [in] [in ²] [in ²]	AISC 14 th B4.3b
Bolt hole diameter Plate thickness Plate strength Bolt no in ver & hor dir Bolt spacing in ver & hor dir Bolt edge dist in ver & hor dir Gross area subject to shear Net area subject to shear Net area subject to tension	bolt dia $d_b = \frac{3}{4}$ $t_p = 0.375$ $F_y = 50.0$ $n_v = 3$ $s_v = 3.000$ $e_v = 1.375$ $A_{gv} = [(n_h - 1) A_{nv} = A_{gv} - [(n_v - 1) A_{nv} = A_{gv} - [(n_v - 1) A_{nv} = A_{gv} - [(n_v - 1) A_{nv} = A_{gv} - [((n_v - 1) A_{nv} + A_{gv} - [((n_v - 1) A_{nv} = A_{gv} - [((n_v - 1) A_{nv} = A_{gv} - [((n_v - 1) A_{nv} - ((n_v - 1) A$	[in] [in] [ksi] [in] [in] 1) s _h + e _h] t _p > n _h - 1)+ 0.5] d	bolt hole dia $d_h = \frac{7}{8}$ $F_u = 65.0$ $n_h = 1$ $s_h = 3.000$ $e_h = 1.375$ x = 2 $a_h t_p x = 0.703$	[in] [ksi] [in] [in] [in ²] [in ²]	AISC 14 th B4.3b
Bolt hole diameter Plate thickness Plate strength Bolt no in ver & hor dir Bolt spacing in ver & hor dir Bolt edge dist in ver & hor dir Gross area subject to shear Net area subject to shear Net area subject to tension when sheared out by center strip	bolt dia $d_b = \frac{3}{4}$ $t_p = 0.375$ $F_y = 50.0$ $n_v = 3$ $s_v = 3.000$ $e_v = 1.375$ $A_{gv} = [(n_h - A_{nv} - A_{gv} - [(A_{nv} - A_{nv} - A_{gv} - (A_{nv} - A_{gv} - A_{gv} - A_{gv} - (A_{nv} - A_{gv} $	[in] [in] [ksi] [in] 1) s _h + e _h] t _p > n _h - 1)+ 0.5] d) (s _v - d _h) t _p	bolt hole dia $d_h = \frac{7}{8}$ $F_u = 65.0$ $n_h = 1$ $s_h = 3.000$ $e_h = 1.375$ x = 2 $a_h t_p x = 0.703$ = 1.594	[in] [ksi] [in] [in ²] [in ²]	AISC 14 th B4.3b
Bolt hole diameter Plate thickness Plate strength Bolt no in ver & hor dir Bolt spacing in ver & hor dir Bolt edge dist in ver & hor dir Gross area subject to shear Net area subject to shear Net area subject to tension when sheared out by center strip Block shear strength required	bolt dia $d_b = \frac{3}{4}$ $t_p = 0.375$ $F_y = 50.0$ $n_v = 3$ $s_v = 3.000$ $e_v = 1.375$ $A_{gv} = [(n_h - 1) + 2n_{gv} - [(n_{gv} - 1) + 2n_{gv} - 1)]$ $A_{nv} = A_{gv} - [(n_{gv} - 1) + 2n_{gv} - 1)]$ $V_u = 0$	[in] [in] [ksi] [in] 1) s _h + e _h] t _p > n _h - 1)+ 0.5] d) (s _v - d _h) t _p	bolt hole dia $d_h = \frac{7}{8}$ $F_u = 65.0$ $n_h = 1$ $s_h = 3.000$ $e_h = 1.375$ 4 2 = 1.031 $a_h t_p x 2 = 0.703$ = 1.594 = 0.39	[in] [ksi] [in] [in ²] [in ²] [in ²] [kips]	AISC 14 th B4.3b
Bolt hole diameter Plate thickness Plate strength Bolt no in ver & hor dir Bolt spacing in ver & hor dir Bolt edge dist in ver & hor dir Gross area subject to shear Net area subject to shear Net area subject to tension when sheared out by center strip Block shear strength required Uniform tension stress factor	bolt dia $d_b = \frac{3}{4}$ $t_p = 0.375$ $F_y = 50.0$ $n_v = 3$ $s_v = 3.000$ $e_v = 1.375$ $A_{gv} = [(n_h - 1)^2 + (n_v - 1)^2]$ $A_{nv} = A_{gv} - [(n_v - 1)^2 + (n_v - 1)^2]$ $V_u = U_{bs} = 1.00$	[in] [in] [ksi] [in] 1) s _h + e _h] t _p , n _h - 1) + 0.5] d) (s _v - d _h) t _p	bolt hole dia $d_h = \frac{7}{8}$ $F_u = 65.0$ $n_h = 1$ $s_h = 3.000$ $e_h = 1.375$ x = 2 $h_h t_p x = 0.703$ = 1.594 = 0.39	[in] [ksi] [in] [in ²] [in ²] [in ²] [kips]	AISC 14 th B4.3b
Bolt hole diameter Plate thickness Plate strength Bolt no in ver & hor dir Bolt spacing in ver & hor dir Bolt edge dist in ver & hor dir Gross area subject to shear Net area subject to shear Net area subject to tension when sheared out by center strip Block shear strength required Uniform tension stress factor Bolt shear resistance provided	bolt dia $d_b = \frac{3}{4}$ $t_p = 0.375$ $F_y = 50.0$ $n_v = 3$ $s_v = 3.000$ $e_v = 1.375$ $A_{gv} = [(n_h - 1) + 2n_{gv} - [(n_h - 1) + 2n_{gv} - 1])$ $A_{nv} = A_{gv} - [(n_h - 1) + 2n_{gv} - 1])$ $V_u = 0$ $V_u = 0$ V	[in] [in] [ksi] [in] [in] 1) $s_h + e_h] t_p >$ $n_h - 1) + 0.5] d$) ($s_v - d_h$) t_p $6F_u A_{nv} , 0.6F_y$	bolt hole dia $d_h = \frac{7}{8}$ $F_u = 65.0$ $n_h = 1$ $s_h = 3.000$ $e_h = 1.375$ x = 1.031 = 1.594 = 0.39 A_{gv}) + = 131.02	[in] [ksi] [in] [in ²] [in ²] [kips]	AISC 14 th B4.3b AISC 14 th Fig C-J4.2 AISC 14 th Eq J4-5
Bolt hole diameter Plate thickness Plate strength Bolt no in ver & hor dir Bolt spacing in ver & hor dir Bolt edge dist in ver & hor dir Gross area subject to shear Net area subject to shear Net area subject to tension when sheared out by center strip Block shear strength required Uniform tension stress factor Bolt shear resistance provided Resistance factor-LRFD	bolt dia $d_b = \frac{3}{4}$ $t_p = 0.375$ $F_y = 50.0$ $n_v = 3$ $s_v = 3.000$ $e_v = 1.375$ $A_{gv} = [(n_h - 1)^2 + (n_v -$	<pre>[in] [in] [ksi] [in] [in] 1) s_h + e_h] t_p, n_h - 1) + 0.5] d) (s_v - d_h) t_p 6F_u A_{nv}, 0.6F_y A_{nt}</pre>	bolt hole dia $d_h = \frac{7}{8}$ $F_u = 65.0$ $n_h = 1$ $s_h = 3.000$ $e_h = 1.375$ x = 1.031 = 1.594 = 0.39 $A_{gv}) + = 131.02$	[in] [ksi] [in] [in ²] [in ²] [kips] [kips]	AISC 14 th B4.3b AISC 14 th Fig C-J4.2 AISC 14 th Eq J4-5 AISC 14 th Eq J4-5
Bolt hole diameter Plate thickness Plate strength Bolt no in ver & hor dir Bolt spacing in ver & hor dir Bolt edge dist in ver & hor dir Gross area subject to shear Net area subject to shear Net area subject to tension when sheared out by center strip Block shear strength required Uniform tension stress factor Bolt shear resistance provided Resistance factor-LRFD	bolt dia $d_b = \frac{3}{4}$ $t_p = 0.375$ $F_y = 50.0$ $n_v = 3$ $s_v = 3.000$ $e_v = 1.375$ $A_{gv} = [(n_h - 1) + 2n_{gv} - [(n_h - 1) + 2n_{gv} - 1])$ $A_{nv} = A_{gv} - [(n_h - 1) + 2n_{gv} - 1])$ $V_u = 0$ $V_u = 0$ $V_u = 0$ $V_u = 0$ $V_{us} = 1.00$ $R_n = min (0.0)$ $U_{bs} F_u A$ $\phi = 0.75$ $\phi R_n = 0$	[in] [in] [ksi] [in] 1) $s_h + e_h$] t_p $n_h - 1) + 0.5$] d) ($s_v - d_h$) t_p $6F_u A_{nv}$, 0.6 F_y A_{nt}	bolt hole dia $d_h = \frac{7}{8}$ $F_u = 65.0$ $n_h = 1$ $s_h = 3.000$ $e_h = 1.375$ x = 1.031 = 1.594 = 0.39 A_{gv}) + $= 131.02$ = 98.26	[in] [ksi] [in] [in ²] [in ²] [kips] [kips]	AISC 14 th B4.3b AISC 14 th Fig C-J4.2 AISC 14 th Eq J4-5 AISC 14 th Eq J4-5

Shear Tab - Block Shear - Shear/Te	nsile 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 14 th Eq 10-5
			< 1.0	ОК	

Shear Tab - Lateral Stability / S	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 14 th Eq 10-6
Resistance factor-LRFD	φ = 0.90			AISC 14 th Eq 10-6
	φ R _n =	= 556.65	[kips]	
	ratio = 0.06	> V _u	ОК	

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 fisr	t bolt line = 1.875	[in]	AISC 14 th 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	$\lambda = \frac{h_0 \sqrt{F_y}}{10 t_w \sqrt{475 + 280} (h)}$	$\frac{1}{(0^{1}/c)^{2}}$ = 0.204		AISC 14 th Eq 9-18
Plate buckling factor	Q =	= 1.000		AISC 14 th Eq 9-15
Plate critical buckling stress	$F_{cr} = Q F_{y}$	= 50.0	[ksi]	AISC 14 th 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 14 th Eq 9-19
Resistance factor-LRFD	φ = 0.90			AISC 14 th Eq 9-19
	φ R _n =	= 82.10	[kips]	
	ratio = 0.38	> V _u	ОК	

E.

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 14 th Table 7	7-6 ~ 7-13 = 2.288		
Bolt group eccentricity coefficient	$C_{ec} = C / (n_r x n_c)$	= 0.763		
Shear Tab / Beam Web - Bolt S	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 14 th 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 Eccen	tricity' calc = 0.763		
Required shear strength	V _u =	= 31.48	[kips]	
Bolt shear strength	$R_n = F_{nv}A_bn_smC_{ec}$	= 54.61	[kips]	AISC 14 th Eq J3-1
Bolt resistance factor-LRFD	φ = 0.75			AISC 14 th Eq J3-1
	φ R _n =	= 40.96	[kips]	

Shear Tab to Ver Beam Web W	/eld Strength	ratio = 5.90 / 10.97	= 0.5	4 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	$f_{b} = \frac{M}{L^{2}/6}$	= 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 14 th Eq 8-11
Weld stress load angle	$\theta = \tan^{-1} \left(\frac{f_a - f_b}{f_v} \right)$	= 52.4	[°]	
Fillet Weld Strength Calc				
Fillet weld leg size	$w = \frac{5}{16}$ [in]	load angle $\theta = 52.4$	[°]	
Electrode strength	F _{EXX} = 70.0 [ksi]	strength coeff C ₁ = 1.00		AISC 14 th Table 8-3
Number of weld line	n = 2 for double fillet			
Load angle coefficient	$C_2 = (1 + 0.5 \sin^{1.5} \theta)$	= 1.35		AISC 14 th Page 8-9
Fillet weld shear strength	R _{n-w} = 0.6 (C ₁ x 70 ksi) 0.	707 w n C_2 = 25.102	[kip/in]	AISC 14 th 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>s</u>	<u>hear</u> rupture as per AISC 14 th Eq J	4-4 is checked		AISC 14 th J2.4
Base metal shear rupture	$R_{n-b} = 0.6 F_{u}t$	= 14.625	[kip/in]	AISC 14 th Eq J4-4
Double fillet linear shear strength	$R_{n} = min (R_{n-w}, R_{n-b})$	= 14.625	[kip/in]	AISC 14 th Eq 9-2
Resistance factor-LRFD	φ = 0.75			AISC 14 th Eq 8-1
	φ R _n =	= 10.969	[kip/in]	
	ratio = 0.54	> f _{max}	ОК	

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 14 th Eq J4-3
Resistance factor-LRFD	φ = 1.00			AISC 14 th Eq J4-3
	φ 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 = $\frac{3}{4}$	[in]	bolt hole dia $d_h = \frac{7}{8}$	[in]	AISC 14 th 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 14 th Eq J4-4
Resistance factor-LRFD	φ = 0.75				AISC 14 th Eq J4-4
	$\phi R_n =$		= 106.03	[kips]	
	ratio = 0.30		> V _u	ОК	

Gusset Plate Leg - Flexural Yieldi	ng		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	$R_n = F_y (t d^2)$	2/6)	= 33.86	[kip-ft]		
Resistance factor-LRFD	φ = 0.90					
	φ R _n =		= 30.47	[kips]		
	ratio = 0.48		> M _u	ОК		
Gusset Plate Leg - Lateral Torsion	nal Buckling		ratio = 14.77 / 147.54	= 0.10	PASS	
Refer to Bo Dowswell's paper 'Design of	Wrap-Around Steel	Gusset Plates' fo	r more details on this limit	state check	< colored and set of the set of t	
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 gusset-l	e from gusset load beam interface lir	d CG to = 10.630	[in]		
Critical moment - gusset leg	$R_n = 0.94 $	$EG \frac{dt^3}{L}$	= 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	ОК		

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 _v = 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 14 th Eq J4-3
Resistance factor-LRFD	$\varphi = 1.00$			AISC 14 th Eq J4-3
	φ R _n =	= 98.44	[kips]	
	ratio = 0.32	> V _u	ОК	
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 $d_b = \frac{3}{4}$ [in]	bolt hole dia d _h = $\frac{7}{8}$	[in]	AISC 14 th 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	$A_{nv} = (b_p - nd_h)t_p$	= 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 14 th Eq J4-4
Resistance factor-LRFD	φ = 0.75			AISC 14 th Eq J4-4
	$\phi R_n =$	= 67.18	[kips]	
	ratio = 0.47	> V _u	OK	
Shear Tab - Flexural Yield Inte	eract	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	$Z_{p} = (b_{p} x t_{p}^{2}) / 4$	= 7.178	[in ³]	
Axial strength available	$P_c =$ from axial tensile yie	eld check = 147.66	[kips]	
Axial strength required	$P_r = $ from gusset interface	e forces calc = 0.39	[kips]	
Shear strength available	V_c = from shear yielding c	check = 98.44	[kips]	
Shear strength required	$V_r =$ from gusset interface	e forces calc = 31.48	[kips]	
Flexural strength available	$M_c = \phi F_y Z_p \phi=0.90$	= 26.92	[kip-ft]	
Flexural strength required	$M_r = $ from gusset interface	e forces calc = 4.92	[kip-ft]	
Flexural yield interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{P_r}{P_c}\right)^2$	$(\frac{M_r}{M_c})^2 = 0.14$		AISC 14 th Eq 10-5
		< 1.0	OK	

Shear Tab - Flexural Rupture I	nteract		ratio =		= 0.28	PASS
Plate A _n and Z _{net} Calc						
Bolt hole diameter	bolt dia $d_b = \frac{3}{4}$	[in]	bolt hole dia d	$h = \frac{7}{8}$	[in]	AISC 14 th 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 ax	cial tensile rupt	ure check	= 111.97	[kips]	
Axial strength required	P _r = from gu	usset interface f	forces calc	= 0.39	[kips]	
Shear strength available	$V_c = $ from sh	near rupture che	eck	= 67.18	[kips]	
Shear strength required	V _r = from gu	usset interface f	forces calc	= 31.48	[kips]	
Flexural strength available	$M_c = \phi F_u Z_n$	_{et} φ=0.75		= 20.87	[kip-ft]	AISC 14 th Eq 9-4
Flexural strength required	M _r = from gu	usset interface f	forces calc	= 4.92	[kip-ft]	
Flexural rupture interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2$	$\frac{P_r}{P_c} + \left(\frac{P_r}{M_c} + \frac{M_r}{M_c}\right)$	r_) ² c	= 0.28		AISC 14 th Eq 10-5
				< 1.0	OK	

Shear Tab - Bolt Bearing on She	ear Tab	ratio = 31.48 / 53.68	= 0.59	PASS
The bolt group is oriented so that the	shear force V is in ver. direction	on and the axial force P is in hor. d	irection	
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	$L_{cA} = min \left(\frac{e_v}{\sin \theta} \right), -$	$\frac{e_{h}}{\cos \theta}) - d_{c} = 0.969$	[in]	
Bolt clear dist - right side edge bolt	$L_{cB} = \min \left(\frac{s_v - 0.5d_v}{\sin \theta} \right)$	$r_{\rm r} = \frac{e_{\rm h}}{\cos \theta} - d_{\rm c} = 2.188$	[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	$R_{n-bolt} = F_{nv}A_b$	= 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	$R_{n-br} = 3.0 d_b t F_u$	= 54.84	[kips]	AISC 14 th Eq J3-6b
Type A - Bolt Group Bottom Right Cor	ner Bolt			
Number of bolt	n _A = 1			
Bolt tear out strength	$R_{n-tA} = 1.5 L_{cA} t F_{u}$	= 35.42	[kips]	AISC 14 th Eq J3-6b
Bolt bearing strength	$R_{nA} = min (R_{n-tA}, R_{n-tA})$	_{br} , R _{n-bolt}) = 23.86	[kips]	
Type B - Bolt Group Right Side Edge I	Bolt			
Number of bolt	n _B = 2			
Bolt tear out strength	$R_{n-tB} = 1.5 L_{cB} t F_{u}$	= 79.99	[kips]	AISC 14 th Eq J3-6b
Bolt bearing strength	$R_{nB} = min (R_{n-tB}, R_{n})$	br, R _{n-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
	φ R _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 $d_b = \frac{3}{4}$	[in]	bolt hole dia d _h = $\frac{7}{8}$	[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 =		= 31.48	[kips]	
Uniform tension stress factor	$U_{bs} = 1.00$				AISC 14 th Fig C-J4.2
Bolt shear resistance provided	R _n = min (0. U _{bs} F _u A	6F _u A _{nv} , 0.6F _y A _{nt}	(A _{gv}) + = 98.72	[kips]	AISC 14 th Eq J4-5
Resistance factor-LRFD	$\varphi = 0.75$	iic.			AISC 14 th Eq J4-5
	φ R _n =		= 74.04	[kips]	
	ratio = 0.43		> V _u	ОК	

Shear Tab - Lateral Stability / St	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 14 th Eq 10-6
Resistance factor-LRFD	φ = 0.90			AISC 14 th Eq 10-6
	φ R _n =	= 556.65	[kips]	
	ratio = 0.06	> V _u	ОК	

Shear Tab - Plate Flexural Buckling		ra	tio = 31.48 / 82.10	= 0.38	PASS	
Shear tab size	depth = 8.750	[in]	thick $= 0.375$	5 [in]		
Plate buckling model	c = dist fro	m support to fisrt bolt	line = 1.875	5 [in]	AISC 14 th	Fig. 9-3
	h ₀ = shear t	ab depth	= 8.750) [in]		
	t _w = shear t	ab thick	= 0.375	5 [in]		
Shear tab steel yield stress	$F_{y} = 50.0$	[ksi]				
Plate buckling factor	$\lambda = \frac{10 t_{w}}{10 t_{w}}$	$\frac{h_0 \sqrt{F_y}}{\sqrt{475 + 280} (h_0/c)}$	= 0.204	ŀ	AISC 14 th	Eq 9-18
Plate buckling factor	Q =		= 1.000)	AISC 14 th	Eq 9-15
Plate critical buckling stress	$F_{cr} = Q F_{y}$		= 50.0	[ksi]	AISC 14 th	Eq 9-14
Shear force in demand	V _u =		= 31.4	B [kips]		
Shear tab net elastic modulus	S _{net} =		= 3.421	[in ³]		
Shear force to bolt group CG ecc	a =		= 1.875	5 [in]		
Shear resistance	$R_n = F_{cr}S_{net}$	/а	= 91.22	2 [kips]	AISC 14 th	Eq 9-19
Resistance factor-LRFD	φ = 0.90				AISC 14 th	Eq 9-19
	$\phi R_n =$		= 82.1	D [kips]		
	ratio = 0.38		> V _u	ОК		

Shear Tab - Plate Shear/Axial Compression Interact		ct	ratio =		I5 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 / $	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 x	L _u / r	= 17.32		
	when $\frac{KL}{r} \leq$	25			AISC 14 th J4.4 (a)
Plate compression provided	$P_n = F_y \times A_g$		= 164.06	[kips]	AISC 14 th Eq J4-6
Axial compression force	P _u = from user input =		= 0.39	[kips]	
Plate Flexural Buckling Capacity					
Plate buckling model	c = dist fro	m support to fi	srt bolt line = 1.875	[in]	AISC 14 th Fig. 9-3
	h ₀ = shear ta	ab depth	= 8.750	[in]	
	t _w = shear ta	ab thick	= 0.375	[in]	
Plate buckling factor	$\lambda = \frac{10 t_{w}}{10 t_{w}}$	$\frac{h_0 \sqrt{F_y}}{\sqrt{475 + 280}}$ (= 0.204		AISC 14 th Eq 9-18
Plate buckling factor	Q =		= 1.000		AISC 14 th Eq 9-15
Plate critical buckling stress	$F_{cr} = Q F_{y}$		= 50.0	[ksi]	AISC 14 th 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	$V_n = F_{cr} S_{net}$	/ а	= 91.22	[kips]	AISC 14 th Eq 9-19
Shear force	V _u = from us	ser input	= 31.48	[kips]	
Resistance factor-LRFD	φ = 0.90				AISC 14 th Eq 10-5
Shear-axial interaction	ratio = $\left(\frac{V_r}{\phi V_r}\right)$	$-)^2 + \left(\frac{P_r}{\phi P_r}\right)^2$) ² = 0.15		AISC 14 th Eq 10-5
		- 1	< 1.0	ОК	

Shear Tab - Compression Buck	ling	ratio = 0.39 / 147.66	= 0.00	PASS
Plate Compression Check				
Plate size	 width b = 8 750 [in]	thickness t = 0.375	[in]	
	F = 50.0 [ksi]	F = 29000	[ksi]	
Plate gross area in compression	$A_{a} = b_{a} t_{a}$	= 3.281	[in ²]	
	g *p*p			
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 x L _u / r	= 17.32		
Plate compression required	P _u =	= 0.39	[kips]	
	when $\frac{KL}{r} \leq 25$			AISC 14 th J4.4 (a)
Plate compression provided	$R_n = F_y \times A_g$	= 164.06	[kips]	AISC 14 th Eq J4-6
Bolt resistance factor-LRFD	φ = 0.90			AISC 14 th J4.4 (a)
	φ R _n =	= 147.66	[kips]	
	ratio = 0.00	> P _u	ОК	
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	[°]	
		halt cal m 1		
Bolt group row and column	bolt row $n_r = 3$	Doit coi $n_c = 1$		
Boil row spacing	boit row $s_r = 5.000$ [iii]			
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 14 th Tab	le 7-6 ~ 7-13 = 2.288		
Bolt group eccentricity coefficient	$C_{ec} = C / (n_r x n_c)$	= 0.763		
Shoar Tab / Boam Wob Bolt	Shoar	ratio = 31.48 / 40.96	- 0 77	DASS
Bolt group forces	shear $V = 31.48$ [kins]	axial P = 0.39	- 0.77	FASS
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= 31 48	[kips]	
		- 51.40	[Kib2]	
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]	
Number of bolt carried shear	n _s = 3.0	shear plane $m = 1$		
Bolt group eccentricity coefficient	C _{ec} = from 'Bolt Group Eco	centricity' calc = 0.763		
Required shear strength	V _u =	= 31.48	[kips]	
Bolt shear strength	$R_n = F_{nv}A_bn_smC_{ec}$	= 54.61	[kips]	AISC 14 th Eq J3-1
Bolt resistance factor-LRFD	φ = 0.75			AISC 14 th Eq J3-1
	φ R _n =	= 40.96	[kips]	
	ratio = 0.77	> V _u	OK	

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Shear Tab to Ver Beam Web We	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	$w = \frac{5}{16}$ [in]	load angle $\theta = 0.0$	[°]	
Electrode strength	F _{EXX} = 70.0 [ksi]	strength coeff C ₁ = 1.00		AISC 14 th Table 8-3
Number of weld line	n = 2 for double fi	llet		
Load angle coefficient	$C_2 = (1 + 0.5 \sin^{1.5})$	θ) = 1.00		AISC 14 th Page 8-9
Fillet weld shear strength	r _w = 0.6 (C ₁ x 70 ks	i) 0.707 w n C ₂ = 18.559	[kip/in]	AISC 14 th 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>she</u>	<u>ear</u> rupture as per AISC 14 th	Eq J4-4 is checked		AISC 14 th J2.4
Base metal shear rupture	$r_{b} = 0.6 F_{u} t$	= 14.625	[kip/in]	AISC 14 th 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 Eccentrical	ly Loaded Weld Group			AISC 14 th 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 14 th Table 8-4
Resistance factor-LRFD	φ = 0.75			
	φ R _n =	= 89.28	[kips]	
	ratio = 0.35	> V _u	ОК	

Gusset to Hor Beam	Shear Tab Connection Coo			de=AISC 360-10 LRFD	
Result Summary	geometries & weld limitations = PASS	limit states max rat	io = 0.86	PASS	
Geometry Restriction Check	cs - Shear Tab to Hor Beam Web			PASS	
Min Bolt Edge Distance - Shea	r Tab to Hor Beam Web				
Bolt diameter	d_b =	= 0.750	[in]		
Min edge distance allowed	L _{e-min} =	= 1.000	[in]	AISC 14 th Table J3.4	
Min edge distance in Shear Tab to Hor Beam Web	L _e =	= 1.375	[in]		
		> L _{e-min}	ОК		
Min Bolt Spacing - Shear Tab	to Hor Beam Web				
Bolt diameter	d _b =	= 0.750	[in]		
Min bolt spacing allowed	$L_{s-min} = 2.667 d_{b}$	= 2.000	[in]	AISC 14 th J3.3	
Min Bolt spacing in Shear Tab to Beam Web	Hor L _s =	= 3.000	[in]		
		> L _{s-min}	ОК		
Weld Limitation Check - She	ear 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 14 th Table J2.4	
Fillet weld size provided	w =	= 0.313	[in]		
		> w _{min}	ОК		
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 14 th J2.2b	
Min fillet weld length	L =	= 8.750	[in]		
		> L _{min}	ОК		

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 [i	n]
Plate yield strength	F _y = 50.0 [ksi]		
Plate gross area in shear	$A_{gv} = b_p t_p$	= 4.938 [i	n ²]
Shear force required	V _u =	= 31.43 [k	kips]
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$	= 148.13 [k	kips] AISC 14 th Eq J4-3
Resistance factor-LRFD	$\phi = 1.00$		AISC 14 th Eq J4-3
	φ R _n =	= 148.13 [k	kips]
	ratio = 0.21	> V _u	ОК

Gusset Plate - Shear Rupture			ratio = 31.43 / 106.03	= 0.30	PASS
Plate Shear Rupture Check					
Bolt hole diameter	bolt dia $d_b = \frac{3}{4}$	[in]	bolt hole dia $d_h = \frac{7}{8}$	[in]	AISC 14 th 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 14 th Eq J4-4
Resistance factor-LRFD	φ = 0.75				AISC 14 th Eq J4-4
	φ R _n =		= 106.03	[kips]	
	ratio = 0.30		> V _u	ОК	

Gusset Plate Leg - Flexural Yieldi	ng		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 checl	<
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	$R_n = F_y (t d^2)$	2/6)	= 33.86	[kip-ft]	
Resistance factor-LRFD	φ = 0.90				
	φ R _n =		= 30.47	[kips]	
	ratio = 0.86		> M _u	ОК	
Gusset Plate Leg - Lateral Torsion	nal 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 checl	K
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	e from gusset load	CG to = 10.005	[in]	
	gusset-	beam interface line	1		
Critical moment - gusset leg	$R_n = 0.94 $	$\overline{EG} = \frac{dt^3}{L}$	= 174.17	[kip-ft]	Dowswell Paper Eq 9
Resistance factor-LRFD	φ = 0.90				
	φ R _n =		= 156.76	[kip-ft]	
	ratio = 0.17		> M _u	ОК	

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	$V_{u} = (V^{2} + F)$	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	$R_{n} = 0.6 F_{y} A$	A _{gv}	= 98.44	[kips]	AISC 14 th Eq J4-3
Resistance factor-LRFD	$\phi = 1.00$				AISC 14 th Eq J4-3
	φ R _n =		= 98.44	[kips]	
	ratio = 0.32		> V _u	ОК	
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	$V_{u} = (V^{2} + F)$	²) ^{0.5}	= 31.43	[kips]	
Plate Shear Rupture Check					
Bolt hole diameter	bolt dia d _b = $\frac{3}{4}$	[in]	bolt hole dia $d_h = \frac{7}{8}$	[in]	AISC 14 th 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	$R_{n} = 0.6 F_{u}A$	A _{nv}	= 89.58	[kips]	AISC 14 th Eq J4-4
Resistance factor-LRFD	φ = 0.75				AISC 14 th Eq J4-4
	$\phi R_n =$		= 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 14 th Eq J4-1
Resistance factor-LRFD	φ = 0.90				AISC 14 th Eq J4-1
	$\phi R_n =$		= 147.66	[kips]	
	ratio = 0.00		> P _u	ОК	

Shear Tab - Axial Tensile Rupt	ure		ratio = 0.34	/ 111.97	= 0.00	PASS	
Plate Tensile Rupture Check							
Bolt hole diameter	bolt dia $d_b = \frac{3}{4}$	[in]	bolt hole dia d	$h = \frac{7}{8}$	[in]	AISC 14 th	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 =			= 0.34	[kips]		
Plate tensile rupture strength	$R_n = F_u A_{nt}$			= 149.30	[kips]	AISC 14^{th}	Eq J4-2
Resistance factor-LRFD	φ = 0.75					AISC 14^{th}	Eq J4-2
	φ R _n =			= 111.97	[kips]	AISC 14^{th}	Eq J4-2
	ratio = 0.00			$> P_u$	ОК		
Shear Tab - Flexural Yield Inte	eract		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	$Z_p = (b_p \times t_p^2)$	²)/4		= 7.178	[in ³]		
Axial strength available	$P_c = $ from ax	ial tensile yield o	check	= 147.66	[kips]		
Axial strength required	P _r = from gu	sset interface fo	rces calc	= 0.34	[kips]		
Shear strength available	$V_c = $ from sh	ear yielding che	ck	= 98.44	[kips]		
Shear strength required	V _r = from gu	sset interface fo	rces 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 gu	sset interface fo	rces 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 14 th	Eq 10-5

ОК

< 1.0

Shear Tab - Flexural Rupture I	nteract		ratio =		= 0.28	PASS
Plate A _n and Z _{net} Calc						
Bolt hole diameter	bolt dia $d_b = \frac{3}{4}$	[in]	bolt hole dia d	$h = \frac{7}{8}$	[in]	AISC 14 th 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 ax	cial tensile rupt	ure check	= 111.97	[kips]	
Axial strength required	P _r = from gu	usset interface f	forces calc	= 0.34	[kips]	
Shear strength available	$V_c = $ from sh	near rupture che	eck	= 67.18	[kips]	
Shear strength required	V _r = from gu	usset interface f	forces calc	= 31.43	[kips]	
Flexural strength available	$M_c = \phi F_u Z_n$	_{et} φ=0.75		= 20.87	[kip-ft]	AISC 14 th Eq 9-4
Flexural strength required	M _r = from gu	usset interface f	forces calc	= 4.91	[kip-ft]	
Flexural rupture interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2$	$\frac{P_r}{P_c} + \left(\frac{P_r}{M_c} + \frac{M_r}{M_c}\right)$	r_) ² c	= 0.28		AISC 14 th Eq 10-5
				< 1.0	OK	

Shear Tab - Bolt Bearing on She	ear 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. di	rection	
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	$L_{cA} = \min\left(\frac{e_v}{\sin\theta}, \frac{\theta}{\cos\theta}\right)$	$\frac{\theta_{h}}{\theta_{c}}$) - d _c = 0.969	[in]	
Bolt clear dist - right side edge bolt	$L_{cB} = \min \left(\frac{s_v - 0.5d_v}{\sin \theta} \right),$	$\frac{e_{h}}{\cos \theta}) - d_{c} = 2.188$	[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	$R_{n-bolt} = F_{nv}A_b$	= 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	$R_{n-br} = 3.0 d_b t F_u$	= 54.84	[kips]	AISC 14 th Eq J3-6b
Type A - Bolt Group Bottom Right Co	rner Bolt			
Number of bolt	n _A = 1			
Bolt tear out strength	$R_{n-tA} = 1.5 L_{cA} t F_{u}$	= 35.42	[kips]	AISC 14 th Eq J3-6b
Bolt bearing strength	$R_{nA} = min (R_{n-tA}, R_{n-br})$, R _{n-bolt}) = 23.86	[kips]	
Type B - Bolt Group Right Side Edge	Bolt			
Number of bolt	n _B = 2			
Bolt tear out strength	$R_{n-tB} = 1.5 L_{cB} t F_{u}$	= 79.99	[kips]	AISC 14 th Eq J3-6b
Bolt bearing strength	$R_{nB} = min (R_{n-tB}, R_{n-br})$, R _{n-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
	φ R _n =	= 53.68	[kips]	
	ratio = 0.59	> R	ОК	

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 $d_b = \frac{3}{4}$	[in]	bolt hole dia $d_h = \frac{7}{8}$	[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	$A_{nv} = A_{gv} - [$ (r	n _h -1)+0.5]c	$i_{h}t_{p} = 1.945$	[in ²]	
Net area subject to tension	$A_{nt} = (e_v - 0.5)$	5 d _h)t _p	= 0.352	[in ²]	
Block shear strength required	V =		= 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 (0.6	SF _u A _{nv} , 0.6F _y A	_{gv}) + = 98.72	[kips]	AISC 14 th Eq J4-5
Resistance factor-LRFD	$U_{bs}F_{u}A$ $\Phi = 0.75$	nt			AISC 14 th Eq J4-5
	$\phi R_n =$		= 74.04	[kips]	·
	ratio = 0.42		> V	ОК	
Shear Tab - Beam Side-Axial 1	earout - Block Shear	- Center Strip	ratio = 0.34 / 98.26	= 0.00	PASS
Plate Block Snear - Center Strip					
Bolt hole diameter	bolt dia d _b = $\frac{3}{4}$	[in]	bolt hole dia d _h = $\frac{7}{8}$	[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. = 1 375	[in]	
Gross area subject to shear			c _h 11373	[]	
Gross area subject to shear	A _{gv} = [(n _h - 1)s _h +e _h]t _p x	2 = 1.031	[in ²]	
Net area subject to shear	A _{gv} = [(n _h - 1) A _{nv} = A _{gv} - [(n)s _h +e _h]t _p x h _h -1)+0.5]d _h	$c_h = 1.031$ $t_p x 2 = 0.703$	[in ²] [in ²]	
Net area subject to shear Net area subject to tension	A _{gv} = [(n _h - 1 A _{nv} = A _{gv} - [(n)s _h +e _h]t _p x: n _h -1)+0.5]d _h	$c_h = 1.031$ $t_p x^2 = 0.703$	[in ²] [in ²]	
Net area subject to shear Net area subject to tension when sheared out by center strip	$A_{gv} = [(n_{h} - 1) A_{nv} = A_{gv} - [(n_{h} - 1) A_{nt} = (n_{v} - 1)]$) s _h + e _h] t _p x : m _h - 1)+ 0.5] d _h (s _v - d _h) t _p	$c_h = 1.031$ $t_p \times 2 = 0.703$ = 1.594	[in ²] [in ²] [in ²]	
Net area subject to shear Net area subject to tension when sheared out by center strip Block shear strength required	$A_{gv} = [(n_{h} - 1) \\ A_{nv} = A_{gv} - [(n_{h} - 1) \\ A_{nt} = (n_{v} - 1) \\ V_{u} = 0$) s _h + e _h] t _p x : n _h - 1)+ 0.5] d _h (s _v - d _h) t _p	$c_h = 1.031$ $t_p \times 2 = 0.703$ = 1.594 = 0.34	[in ²] [in ²] [in ²] [kips]	
Net area subject to shear Net area subject to tension when sheared out by center strip Block shear strength required Uniform tension stress factor	$A_{gv} = [(n_{h} - 1) \\ A_{nv} = A_{gv} - [(n_{h} - 1) \\ A_{nv} = A_{gv} - [(n_{h} - 1) \\ A_{nt} = (n_{h} - 1) \\ V_{u} = \\ U_{bs} = 1.00$) s _h + e _h] t _p x : n _h - 1)+ 0.5] d _h (s _v - d _h) t _p	$t_p x^2 = 1.031$ = 1.594 = 0.34	[in] [in ²] [in ²] [kips]	AISC 14 th Fig C-J4.2
Net area subject to shear Net area subject to tension when sheared out by center strip Block shear strength required Uniform tension stress factor Bolt shear resistance provided	$A_{gv} = [(n_{h} - 1) \\ A_{nv} = A_{gv} - [(n_{h} - 1) \\ A_{nv} = A_{gv} - [(n_{h} - 1) \\ A_{nt} = (n_{v} - 1) \\ V_{u} = \\ U_{bs} = 1.00 \\ R_{n} = \min(0.6 \\ U_{bs} F_{u} A$) s _h + e _h] t _p x : n _h - 1)+ 0.5] d _h (s _v - d _h) t _p GF _u A _{nv} , 0.6F _y A	$(t_{p} = 1.031)$ $(t_{p} \times 2) = 0.703$ $(t_{p} \times 2) = 0.703$ $(t_{p} \times 2) = 0.34$ $(t_{p} \times 2) = 0.34$ $(t_{p} \times 2) = 131.02$	[in ²] [in ²] [in ²] [kips] [kips]	AISC 14 th Fig C-J4.2 AISC 14 th Eq J4-5
Net area subject to shear Net area subject to shear Net area subject to tension when sheared out by center strip Block shear strength required Uniform tension stress factor Bolt shear resistance provided Resistance factor-LRFD	$A_{gv} = [(n_{h} - 1) \\ A_{nv} = A_{gv} - [(n_{h} - 1) \\ A_{nt} = (n_{v} - 1) \\ V_{u} = \\ U_{bs} = 1.00 \\ R_{n} = \min(0.6 \\ U_{bs} F_{u} A \\ \varphi = 0.75$) s _h + e _h] t _p x : n _h - 1)+ 0.5] d _h (s _v - d _h) t _p GF _u A _{nv} , 0.6F _y A	$c_h = 1.031$ $t_p \times 2 = 0.703$ = 1.594 = 0.34 $g_v) + = 131.02$	[in ²] [in ²] [in ²] [kips] [kips]	AISC 14 th Fig C-J4.2 AISC 14 th Eq J4-5 AISC 14 th Eq J4-5
Net area subject to shear Net area subject to tension when sheared out by center strip Block shear strength required Uniform tension stress factor Bolt shear resistance provided Resistance factor-LRFD	$A_{gv} = [(n_{h} - 1) \\ A_{nv} = A_{gv} - [(n_{h} - 1) \\ A_{nv} = A_{gv} - [(n_{h} - 1) \\ V_{u} = (n_{v} - 1) \\ V_{u} = U_{bs} = 1.00 \\ R_{n} = \min(0.6 \\ U_{bs} F_{u} A \\ \phi = 0.75 \\ \phi R_{n} =$) s _h + e _h] t _p x : n _h - 1)+ 0.5] d _h (s _v - d _h) t _p GF _u A _{nv} , 0.6F _y A	$(t_{p} \times 2) = 1.031$ $(t_{p} \times 2) = 0.703$ = 1.594 = 0.34 $(g_{v}) + = 131.02$ = 98.26	[in ²] [in ²] [in ²] [kips] [kips]	AISC 14 th Fig C-J4.2 AISC 14 th Eq J4-5 AISC 14 th Eq J4-5

Shear Tab - Block Shear - Shear/Te	nsile 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 14 th Eq 10-5
			< 1.0	ОК	

Shear Tab - Lateral Stability / St	tabilizer 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	$V_{u} = (V^{2} + P^{2})$	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	$R_n = 1500 \pi - a$	$\frac{t_p^3}{a^2}$	= 618.50	[kips]	AISC 14 th Eq 10-6
Resistance factor-LRFD	φ = 0.90				AISC 14 th Eq 10-6
	φ R _n =		= 556.65	[kips]	
	ratio = 0.06		> V _u	ОК	

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 14 th 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	$\lambda = \frac{h_0 \sqrt{10}}{10 t_w \sqrt{475 + 2}}$	$\frac{F_{y}}{80 (h_{0}/c)^{2}} = 0.204$		AISC 14 th Eq 9-18
Plate buckling factor	Q =	= 1.000		AISC 14 th Eq 9-15
Plate critical buckling stress	$F_{cr} = Q F_{y}$	= 50.0	[ksi]	AISC 14 th 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 14 th Eq 9-19
Resistance factor-LRFD	φ = 0.90			AISC 14 th Eq 9-19
	φ R _n =	= 82.10	[kips]	
	ratio = 0.38	> V _u	OK	

E.

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 14 th Table	7-6 ~ 7-13 = 2.288		
Bolt group eccentricity coefficient	$C_{ec} = C / (n_r x n_c)$	= 0.763		
Shear Tab / Beam Web - Bolt S	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 14 th 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 Eccer	ntricity' calc = 0.763		
Required shear strength	V _u =	= 31.43	[kips]	
Bolt shear strength	$R_n = F_{nv}A_bn_sm C_{ec}$	= 54.61	[kips]	AISC 14 th Eq J3-1
Bolt resistance factor-LRFD	φ = 0.75			AISC 14 th Eq J3-1
	φ R _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] ii	n 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 14 th Eq 8-11
Weld stress load angle	$\theta = \tan^{-1} \left(\frac{f_a - f_b}{f_v} \right)$	= 52.4	[°]	
Fillet Weld Strength Calc				
Fillet weld leg size	$w = \frac{5}{16}$ [in]	load angle $\theta = 52.4$	[°]	
Electrode strength	F _{EXX} = 70.0 [ksi]	strength coeff C ₁ = 1.00		AISC 14 th Table 8-3
Number of weld line	n = 2 for double fill	let		
Load angle coefficient	$C_2 = (1 + 0.5 \sin^{1.5} 6)$	Ə) = 1.35		AISC 14 th Page 8-9
Fillet weld shear strength	R _{n-w} = 0.6 (C ₁ x 70 ksi)	$0.707 \text{ w n C}_2 = 25.097$	[kip/in]	AISC 14 th 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>s</u>	<u>hear</u> rupture as per AISC 14 th E	q J4-4 is checked		AISC 14 th J2.4
Base metal shear rupture	$R_{n-b} = 0.6 F_{u}t$	= 14.625	[kip/in]	AISC 14 th Eq J4-4
Double fillet linear shear strength	$R_n = min (R_{n-w}, R_{n-t})$) = 14.625	[kip/in]	AISC 14 th Eq 9-2
Resistance factor-LRFD	φ = 0.75			AISC 14 th Eq 8-1
	φ R _n =	= 10.969	[kip/in]	
	ratio = 0.54	> f _{max}	ОК	

Brace Force Load Case 2 Gusset plate t=0.500

0 P =45.00 kips (C)

ratio = 0.86 PASS

ratio = 31.43 / 148.13 **Gusset Plate - Shear Yielding** = 0.21 PASS Plate Shear Yielding Check width $b_p = 9.875$ [in] Plate size thickness $t_p = 0.500$ [in] $F_{y} = 50.0$ Plate yield strength [ksi] Plate gross area in shear $A_{gv} = b_p t_p$ = 4.938 [in²] Shear force required V _u = = 31.43 [kips] AISC 14th Eq J4-3 Plate shear yielding strength $R_n = 0.6 F_y A_{gv}$ = 148.13 [kips] AISC 14th Eq J4-3 $\phi = 1.00$ Resistance factor-LRFD = 148.13 [kips] $\phi R_n =$ ОК ratio = 0.21 $> V_u$

Gusset Plate - Shear Rupture			ratio = 31.43 / 106.03	= 0.30	PASS
Plate Shear Rupture Check					
Bolt hole diameter	bolt dia $d_b = \frac{3}{4}$	[in]	bolt hole dia $d_h = \frac{7}{8}$	[in]	AISC 14 th 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 14 th Eq J4-4
Resistance factor-LRFD	φ = 0.75				AISC 14 th Eq J4-4
	φ R _n =		= 106.03	[kips]	
	ratio = 0.30		> V _u	ОК	

Gusset Plate Leg - Flexural Yieldi	ng		ratio = 26.06 / 30.47	= 0.86	PASS		
Refer to Bo Dowswell's paper 'Design of	Wrap-Around Steel	Gusset Plates' fo	r more details on this limit	state chec	k		
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	$R_n = F_y (t d^2)$	2/6)	= 33.86	[kip-ft]			
Resistance factor-LRFD	φ = 0.90						
	φ R _n =		= 30.47	[kips]			
	ratio = 0.86		> M _u	ОК			
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' fo	r more details on this limit	state chec	k		
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	e from gusset load	d CG to = 10.005	[in]			
	gusset-	beam interface lir	ie				
Critical moment - gusset leg	$R_n = 0.94 $	$EG \frac{dt^3}{L}$	= 174.17	[kip-ft]	Dowswell Paper Eq 9		
Resistance factor-LRFD	φ = 0.90						
	$\phi R_n =$		= 156.76	[kip-ft]			
	ratio = 0.17		> M _u	ОК			

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 _v = 50.0 [ksi]	۲		
Plate gross area in shear	$A_{qv} = b_p t_p$	= 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 14 th Eq J4-3
Resistance factor-LRFD	φ = 1.00			AISC 14 th Eq J4-3
	φ R _n =	= 98.44	[kips]	
	ratio = 0.32	> V _u	ОК	
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 $d_b = \frac{3}{4}$ [in]	bolt hole dia $d_h = \frac{7}{8}$	[in]	AISC 14 th 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	$A_{nv} = (b_{p} - nd_{h})t_{p}$	= 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 14 th Eq J4-4
Resistance factor-LRFD	φ = 0.75			AISC 14 th Eq J4-4
	φ R _n =	= 67.18	[kips]	
	ratio = 0.47	> V _u	ОК	
Shear Tab - Flexural Yield Inte	ract	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 for	orces calc = 0.34	[kips]	
Shear strength available	$V_c =$ from shear yielding che	eck = 98.44	[kips]	
Shear strength required	$V_r =$ from gusset interface for	orces calc = 31.43	[kips]	
Flexural strength available	$M_c = \phi F_y Z_p \phi=0.90$	= 26.92	[kip-ft]	
Flexural strength required	$M_r = from gusset interface from from the set of the $	orces 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 14 th Eq 10-5
		< 1.0	OK	

Shear Tab - Flexural Rupture I	nteract		ratio =		= 0.28	PASS
Plate A _n and Z _{net} Calc						
Bolt hole diameter	bolt dia $d_b = \frac{3}{4}$	[in]	bolt hole dia d	$h = \frac{7}{8}$	[in]	AISC 14 th 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 ax	cial tensile rupt	ure check	= 111.97	[kips]	
Axial strength required	P _r = from gu	usset interface f	forces calc	= 0.34	[kips]	
Shear strength available	$V_c = $ from sh	near rupture che	eck	= 67.18	[kips]	
Shear strength required	V _r = from gu	usset interface f	forces calc	= 31.43	[kips]	
Flexural strength available	$M_c = \phi F_u Z_n$	_{et} φ=0.75		= 20.87	[kip-ft]	AISC 14 th Eq 9-4
Flexural strength required	M _r = from gu	usset interface f	forces calc	= 4.91	[kip-ft]	
Flexural rupture interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2$	$\frac{P_r}{P_c} + \left(\frac{P_r}{M_c} + \frac{M_r}{M_c}\right)$	r_) ² c	= 0.28		AISC 14 th Eq 10-5
				< 1.0	OK	

Shear Tab - Bolt Bearing on Sh	ear Tab	rat	io = 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 [l	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	$\theta = \tan^{-1} (V / I)$	P)	= 89.38	[°]			
Bolt hole diameter	bolt dia d _b = 0.750 [i	in] bolt h	ole 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 [i	in]					
Bolt edge distance	ver e _v = 1.375 [i	in]	hor $e_{h} = 1.375$	[in]			
Bolt clear dist - bot right corner bolt	$L_{cA} = min \left(\frac{e_v}{sin} \right)$	$\frac{e_h}{\theta}$, $\frac{e_h}{\cos \theta}$) - d _c	= 0.969	[in]			
Bolt clear dist - right side edge bolt	$L_{cB} = min \left(\frac{S_v - b}{sir} \right)$	$\frac{0.5d_v}{n \theta}, \frac{e_h}{\cos \theta}) - c$	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 [i	in] b	olt area $A_b = 0.442$	[in ²]			
Single bolt shear strength	$R_{n-bolt} = F_{nv}A_b$		= 23.86	[kips]	AISC 14 th Eq J3-1		
Bolt bearing on plate	thick t = 0.375 [i	in]	tensile $F_u = 65.0$	[ksi]			
Bolt bearing strength	$R_{n-br} = 3.0 d_b t F_u$	I	= 54.84	[kips]	AISC 14 th Eq J3-6b		
Type A - Bolt Group Bottom Right Co	rner Bolt						
Number of bolt	n _A = 1						
Bolt tear out strength	$R_{n-tA} = 1.5 L_{cA} t F$	u	= 35.42	[kips]	AISC 14 th Eq J3-6b		
Bolt bearing strength	$R_{nA} = min (R_{n-tA})$	$A, R_{n-br}, R_{n-bolt})$	= 23.86	[kips]			
Type B - Bolt Group Right Side Edge	Bolt						
Number of bolt	n _B = 2						
Bolt tear out strength	$R_{n-tB} = 1.5 L_{cB} t F$	u	= 79.99	[kips]	AISC 14 th Eq J3-6b		
Bolt bearing strength	$R_{nB} = min (R_{n-tE})$	$_{B}$, R_{n-br} , R_{n-bolt})	= 23.86	[kips]			
Bolt bearing strength for all bolts	$R_{n} = n_{A}R_{nA} + n$	$_{B}R_{nB} + n_{C}R_{nC} + n_{C}$	$_{\rm D} R_{\rm nD} = 71.57$	[kips]			
Bolt resistance factor-LRFD	φ = 0.75				AISC 14 th J3-10		
	$\phi R_n =$		= 53.68	[kips]			
	ratio = 0.59		> R	ОК			

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 $d_b = \frac{3}{4}$	[in]	bolt hole dia $d_h = \frac{7}{8}$	[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 - 2	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. U _{bs} F _u A	6F _u A _{nv} , 0.6F _y A _{nt}	A _{gv}) + = 98.72	[kips]	AISC 14 th Eq J4-5
Resistance factor-LRFD	φ = 0.75				AISC 14 th Eq J4-5
	φ R _n =		= 74.04	[kips]	
	ratio = 0.42		> V _u	ОК	

Shear Tab - Lateral Stability / St	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 14 th Eq 10-6
Resistance factor-LRFD	φ = 0.90			AISC 14 th Eq 10-6
	φ R _n =	= 556.65	[kips]	
	ratio = 0.06	> V _u	ОК	

Shear Tab - Plate Flexural Buckling		rat	tio = 31.43 / 82.10	= 0.38	PASS	
Shear tab size	depth = 8.750	[in]	thick = 0.375	[in]		
Plate buckling model	c = dist fro	m support to fisrt bolt	line = 1.875	[in]	AISC 14^{th}	Fig. 9-3
	h ₀ = shear t	ab depth	= 8.750	[in]		
	t _w = shear t	ab thick	= 0.375	[in]		
Shear tab steel yield stress	$F_{y} = 50.0$	[ksi]				
Plate buckling factor	$\lambda = \frac{10 t_{w}}{10 t_{w}}$	$\frac{h_0 \sqrt{F_y}}{\sqrt{475 + 280} (h_0/c)}$	= 0.204		AISC 14 th	Eq 9-18
Plate buckling factor	Q =		= 1.000		AISC 14 th	Eq 9-15
Plate critical buckling stress	$F_{cr} = Q F_{y}$		= 50.0	[ksi]	AISC 14 th	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	$R_n = F_{cr}S_{net}$	/а	= 91.22	[kips]	AISC 14^{th}	Eq 9-19
Resistance factor-LRFD	φ = 0.90				AISC 14 th	Eq 9-19
	$\phi R_n =$		= 82.10	[kips]		
	ratio = 0.38		$> V_u$	ОК		

Shear Tab - Plate Shear/Axial Compression Interact		ct	ratio =	= 0.1	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 / $	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 x	L _u / r	= 17.32		
	when $\frac{KL}{r} \leq$	25			AISC 14 th J4.4 (a)
Plate compression provided	$P_n = F_y x A_g$		= 164.06	[kips]	AISC 14 th Eq J4-6
Axial compression force	P _u = from us	ser input	= 0.34	[kips]	
Plate Flexural Buckling Capacity					
Plate buckling model	c = dist from support to fisrt bolt line = 1.87			[in]	AISC 14 th Fig. 9-3
	h ₀ = shear ta	ab depth	= 8.750	[in]	
	t _w = shear ta	ab thick	= 0.375	[in]	
Plate buckling factor	$\lambda = \frac{10 t_{w}}{10 t_{w}}$	$\frac{h_0 \sqrt{F_y}}{\sqrt{475 + 280}}$	= 0.204		AISC 14 th Eq 9-18
Plate buckling factor	Q =		= 1.000		AISC 14 th Eq 9-15
Plate critical buckling stress	$F_{cr} = Q F_{y}$		= 50.0	[ksi]	AISC 14 th 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	$V_n = F_{cr} S_{net}$	/ а	= 91.22	[kips]	AISC 14 th Eq 9-19
Shear force	V _u = from us	ser input	= 31.43	[kips]	
Resistance factor-LRFD	φ = 0.90				AISC 14 th Eq 10-5
Shear-axial interaction	ratio = $\left(\frac{V_r}{\phi V_r}\right)$	$-)^2 + \left(\frac{P_r}{\phi P_n}\right)$) ² = 0.15		AISC 14 th Eq 10-5
			< 1.0	ОК	

Shear Tab - Compression Buck	ling	ratio = 0.34 / 147.66	= 0.00	PASS
Plate Compression Check				
Plate size	 width b_ = 8.750 [in]	thickness t = 0.375	[in]	
	$F_{v} = 50.0$ [ksi]	E = 29000	[ksi]	
Plate gross area in compression	$A_{g} = b_{p}t_{p}$	= 3.281	[in ²]	
	урр 			
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 14 th J4.4 (a)
Plate compression provided	$R_n = F_y \times A_g$	= 164.06	[kips]	AISC 14 th Eq J4-6
Bolt resistance factor-LRFD	φ = 0.90			AISC 14 th J4.4 (a)
	$\phi R_n =$	= 147.66	[kips]	
	ratio = 0.00	> P _u	ОК	
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	$\frac{1}{1}$	bolt col $n = 1$		
Bolt row spacing	bolt row $n_r = 3$			
	2010 OF 01000 []			
Shear force to bolt group CG ecc	e _x =	= 1.8/5	[IN]	
Shear force to ver Y axis angle	θ =	= 0.62	[°]	
Bolt group coefficient C	C = from AISC 14 th Table	e 7-6 ~ 7-13 = 2.288		
Bolt group eccentricity coefficient	$C_{ec} = C / (n_r x n_c)$	= 0.763		
Shear Tab / Beam Web - Bolt S	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]	
		F F4.0	[]	ATCC 14 th Table 12.2
Bolt shear stress	Dolt grade = $A325 - N$	$F_{nv} = 54.0$	[KSI]	AISC 14 th Table J3.2
Number of bolt convict share	boit dia $d_b = 0.750$ [in]	boit area $A_b = 0.442$	fiu 1	
Rolt group accentricity coefficient	$\Pi_s = 3.0$	shear plane $m = 1$		
Bolt group eccentricity coefficient			[kinc]	
Bolt chear strength	v_{u} – P – F A n m C	- 54 61	[kips]	AISC 14 th Eq. 13-1
Bolt resistance factor-I DED	$\sigma_{n} = \sigma_{nv} \sigma_{b} \sigma_{s} m c_{ec}$	- 54.01	רגואסן	AISC 14 th Eq. 13-1
	φ = 0.75 d R =	- 10 04	[kins]	
	ratio = 0.77	- 40.90 > V	OK	
		∕ ⊻u	On	

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Shear Tab to Hor Beam Web We	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	$w = \frac{5}{16}$ [in] load	angle $\theta = 0.0$	[°]	
Electrode strength	F _{EXX} = 70.0 [ksi] strength o	$coeff C_1 = 1.00$		AISC 14 th Table 8-3
Number of weld line	n = 2 for dou	ıble fillet			
Load angle coefficient	$C_2 = (1 + 0.5)$	sin ^{1.5} θ)	= 1.00		AISC 14 th Page 8-9
Fillet weld shear strength	$r_{w} = 0.6 (C_{1} \times T_{1})$	70 ksi) 0.707 w n C ₂	= 18.559	[kip/in]	AISC 14 th Eq 8-1
Base metal - shear tab	thickness t = 0.375 [in] te	nsile $F_u = 65.0$	[ksi]	
Base metal - shear tab is in shear, <u>sh</u>	ear rupture as per AISC	14 th Eq J4-4 is checke	d		AISC 14 th J2.4
Base metal shear rupture	$r_{b} = 0.6 F_{u}t$		= 14.625	[kip/in]	AISC 14 th Eq J4-4
Weld stress reduction factor due to less base metal strength	$C_3 = r_b / r_w$ wi	hen $r_b < r_w$	= 0.788		
Table 8-4 Coefficient C for Eccentrical	ly Loaded Weld Group				AISC 14 th 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 14 th Table 8-4
Resistance factor-LRFD	φ = 0.75				
	φ R _n =		= 89.28	[kips]	
	ratio = 0.35		> V _u	ОК	