

Beam to Column

Shear Tab Shear Connection

Code=AISC 360-10 LRFD

**Result Summary**

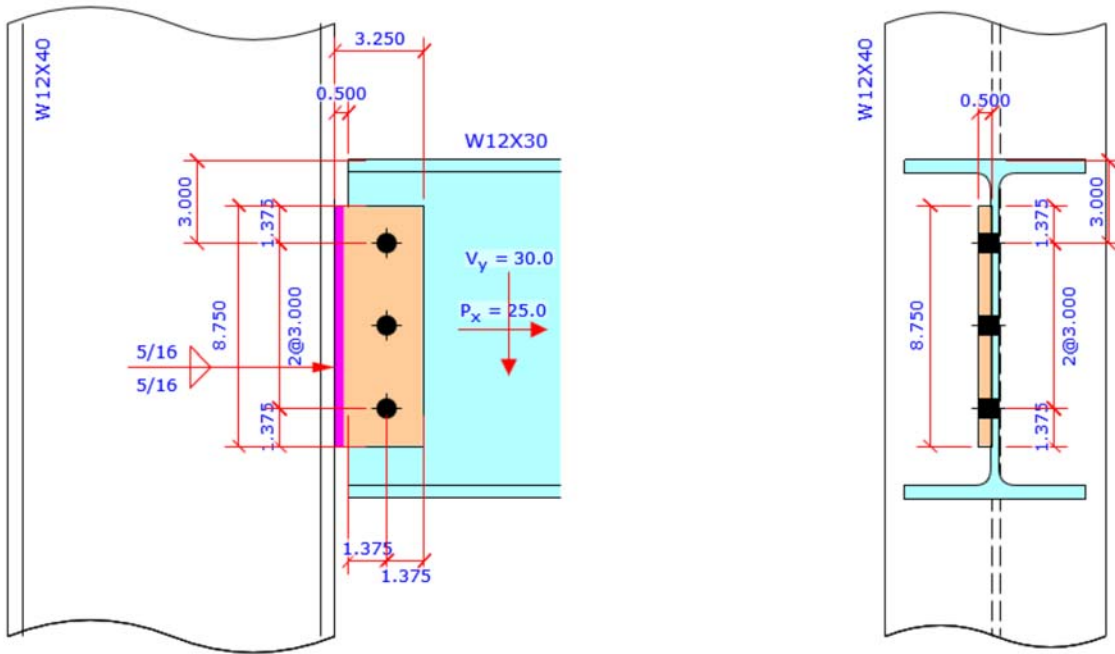
geometries & weld limitations = **PASS**

limit states max ratio = **0.96** **PASS**

Sketch

Shear Connection

Code=AISC 360-10 LRFD



**Members & Components Summary**

Member

Shear Connection

Code=AISC 360-10 LRFD

Geometry Restriction Checks - Shear Tab to Beam Web				PASS
<b>Min Bolt Edge Distance - Shear Tab to Beam Web</b>				
Bolt diameter	$d_b =$	= 0.750	[in]	
Min edge distance allowed	$L_{e-min} =$	= 1.000	[in]	AISC 14 <sup>th</sup> Table J3.4
Min edge distance in Shear Tab to Beam Web	$L_e =$	= 1.375	[in]	
		> $L_{e-min}$		OK
<b>Min Bolt Spacing - Shear Tab to Beam Web</b>				
Bolt diameter	$d_b =$	= 0.750	[in]	
Min bolt spacing allowed	$L_{s-min} = 2.667 d_b$	= 2.000	[in]	AISC 14 <sup>th</sup> J3.3
Min Bolt spacing in Shear Tab to Beam Web	$L_s =$	= 3.000	[in]	
		> $L_{s-min}$		OK
Weld Limitation Check - Shear Tab Weld				PASS
<b>Min Fillet Weld Size</b>				
Thinner part joined thickness	$t =$	= 0.500	[in]	
Min fillet weld size allowed	$w_{min} =$	= 0.188	[in]	AISC 14 <sup>th</sup> Table J2.4
Fillet weld size provided	$w =$	= 0.313	[in]	
		> $w_{min}$		OK
<b>Min Fillet Weld Length</b>				
Fillet weld size provided	$w =$	= 0.313	[in]	
Min fillet weld length allowed	$L_{min} = 4 \times w$	= 1.250	[in]	AISC 14 <sup>th</sup> J2.2b
Min fillet weld length	$L =$	= 8.750	[in]	
		> $L_{min}$		OK
W Shape Beam - Tensile Yield				ratio = 25.00 / 395.55 = 0.06 PASS
Gross area subject to tension	$A_g =$	= 8.790	[in <sup>2</sup> ]	
Steel yield strength	$F_y =$	= 50.0	[ksi]	
Tensile force required	$P_u =$	= 25.00	[kips]	
Tensile yielding strength	$R_n = F_y A_g$	= 439.50	[kips]	AISC 14 <sup>th</sup> Eq D2-1
Resistance factor-LRFD	$\phi = 0.90$			AISC 14 <sup>th</sup> D2 (a)
	$\phi R_n =$	= 395.55	[kips]	AISC 14 <sup>th</sup> Eq D2-1
	ratio = 0.06	> $P_u$		OK

<b>W Shape Beam - Tensile Rupture</b>		ratio = 25.00 / 133.51	= 0.19	<b>PASS</b>
W beam section	= W12X30			
	d = 12.300 [in]	b <sub>f</sub> = 6.520 [in]		
	t <sub>f</sub> = 0.440 [in]	t <sub>w</sub> = 0.260 [in]		
	A <sub>g</sub> = 8.790 [in <sup>2</sup> ]			
Bolt hole diameter	bolt dia d <sub>b</sub> = 3/4 [in]	bolt hole dia d <sub>n</sub> = 7/8 [in]		AISC 14 <sup>th</sup> B4.3b
Number of bolt row	n = 3			
W section net area	A <sub>n</sub> = A <sub>g</sub> - n d <sub>h</sub> t <sub>w</sub>	= 8.108 [in <sup>2</sup> ]		
Shear lag factor min value	U = $\frac{(d - 2 t_f) t_w}{A_g}$	= 0.338		AISC 14 <sup>th</sup> D3
Tensile force required	P <sub>u</sub> =	= 25.00 [kips]		
Tensile effective net area	A <sub>e</sub> = A <sub>n</sub> U	= 2.739 [in <sup>2</sup> ]		
Plate tensile strength	F <sub>u</sub> =	= 65.0 [ksi]		
Tensile rupture strength	R <sub>n</sub> = F <sub>u</sub> A <sub>e</sub>	= 178.01 [kips]		AISC 14 <sup>th</sup> Eq D2-2
Resistance factor-LRFD	φ = 0.75			AISC 14 <sup>th</sup> D2 (b)
	φ R <sub>n</sub> =	= 133.51 [kips]		AISC 14 <sup>th</sup> Eq D2-2
	ratio = 0.19	> P <sub>u</sub>	<b>OK</b>	

<b>Beam Web - Shear Yielding</b>		ratio = 30.00 / 95.94	= 0.31	<b>PASS</b>
<b>Plate Shear Yielding Check</b>				
Plate size	width b <sub>p</sub> = 12.300 [in]	thickness t <sub>p</sub> = 0.260 [in]		
Plate yield strength	F <sub>y</sub> = 50.0 [ksi]			
Plate gross area in shear	A <sub>gv</sub> = b <sub>p</sub> t <sub>p</sub>	= 3.198 [in <sup>2</sup> ]		
Shear force required	V <sub>u</sub> =	= 30.00 [kips]		
Plate shear yielding strength	R <sub>n</sub> = 0.6 F <sub>y</sub> A <sub>gv</sub>	= 95.94 [kips]		AISC 14 <sup>th</sup> Eq J4-3
Resistance factor-LRFD	φ = 1.00			AISC 14 <sup>th</sup> Eq J4-3
	φ R <sub>n</sub> =	= 95.94 [kips]		
	ratio = 0.31	> V <sub>u</sub>	<b>OK</b>	

<b>Beam Web - Shear Rupture</b>		ratio = 30.00 / 73.58	= 0.41	<b>PASS</b>
<b>Plate Shear Rupture Check</b>				
Bolt hole diameter	bolt dia d <sub>b</sub> = 3/4 [in]	bolt hole dia d <sub>n</sub> = 7/8 [in]		AISC 14 <sup>th</sup> B4.3b
Number of bolt	n = 3			
Plate size	width b <sub>p</sub> = 12.300 [in]	thickness t <sub>p</sub> = 0.260 [in]		
Plate tensile strength	F <sub>u</sub> = 65.0 [ksi]			
Plate net area in shear	A <sub>nv</sub> = (b <sub>p</sub> - n d <sub>h</sub> ) t <sub>p</sub>	= 2.516 [in <sup>2</sup> ]		
Shear force required	V <sub>u</sub> =	= 30.00 [kips]		
Plate shear rupture strength	R <sub>n</sub> = 0.6 F <sub>u</sub> A <sub>nv</sub>	= 98.10 [kips]		AISC 14 <sup>th</sup> Eq J4-4
Resistance factor-LRFD	φ = 0.75			AISC 14 <sup>th</sup> Eq J4-4
	φ R <sub>n</sub> =	= 73.58 [kips]		
	ratio = 0.41	> V <sub>u</sub>	<b>OK</b>	

<b>Beam Web - Bolt Bearing on Beam Web</b>		ratio = 39.05 / 53.68	= <b>0.73</b>	<b>PASS</b>
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 = 30.00 [kips]		axial P = -25.00 [kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$		= <b>39.05</b> [kips]	
Resultant force/hor line load angle	$\theta = \tan^{-1}(V/P)$		= 50.19 [°]	
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Bolt hole diameter	bolt dia $d_b = 0.750$ [in]		bolt hole dia $d_{bh} = 0.813$ [in]	AISC 14 <sup>th</sup> 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]	
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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 $e_v = 3.000$ [in]		hor $e_h = 1.375$ [in]	
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Bolt clear dist - bot right corner bolt	$L_{cA} = \min\left(\frac{e_v}{\sin \theta}, \frac{e_h}{\cos \theta}\right) - d_c$		= 1.742 [in]	
Bolt clear dist - right side edge bolt	$L_{cB} = \min\left(\frac{s_v - 0.5d_v}{\sin \theta}, \frac{e_h}{\cos \theta}\right) - d_c$		= 1.742 [in]	
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<b>Single Bolt Shear Strength</b>				
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Bolt shear stress	bolt grade = A325-N		$F_{nv} = 54.0$ [ksi]	AISC 14 <sup>th</sup> Table J3.2
	bolt dia $d_b = 0.750$ [in]		bolt area $A_b = 0.442$ [in <sup>2</sup> ]	
Single bolt shear strength	$R_{n-bolt} = F_{nv} A_b$		= 23.86 [kips]	AISC 14 <sup>th</sup> Eq J3-1
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Bolt bearing on plate	thick t = 0.260 [in]		tensile $F_u = 65.0$ [ksi]	
Bolt bearing strength	$R_{n-br} = 3.0 d_b t F_u$		= 38.03 [kips]	AISC 14 <sup>th</sup> Eq J3-6b
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Type A - Bolt Group Bottom Right Corner Bolt				
Number of bolt	$n_A = 1$			
Bolt tear out strength	$R_{n-tA} = 1.5 L_{cA} t F_u$		= 44.15 [kips]	AISC 14 <sup>th</sup> Eq J3-6b
Bolt bearing strength	$R_{nA} = \min(R_{n-tA}, R_{n-br}, R_{n-bolt})$		= <b>23.86</b> [kips]	
<hr/>				
Type B - Bolt Group Right Side Edge Bolt				
Number of bolt	$n_B = 2$			
Bolt tear out strength	$R_{n-tB} = 1.5 L_{cB} t F_u$		= 44.15 [kips]	AISC 14 <sup>th</sup> Eq J3-6b
Bolt bearing strength	$R_{nB} = \min(R_{n-tB}, R_{n-br}, R_{n-bolt})$		= <b>23.86</b> [kips]	
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Bolt bearing strength for all bolts	$R_n = n_A R_{nA} + n_B R_{nB} + n_C R_{nC} + n_D R_{nD}$		= 71.57 [kips]	
Bolt resistance factor-LRFD	$\phi = 0.75$			AISC 14 <sup>th</sup> J3-10
	$\phi R_n =$		= <b>53.68</b> [kips]	
	ratio = <b>0.73</b>		> R	<b>OK</b>

<b>Beam Web - Axial Tearout - Block Shear - Center Strip</b>		ratio = 25.00 / 68.13	= 0.37	<b>PASS</b>
<b>Plate Block Shear - Center Strip</b>				
Bolt hole diameter	bolt dia $d_b = 3/4$ [in]	bolt hole dia $d_h = 7/8$ [in]		AISC 14 <sup>th</sup> B4.3b
Plate thickness	$t_p = 0.260$ [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	$e_v = 3.000$ [in]	$e_h = 1.375$ [in]		
Gross area subject to shear	$A_{gv} = [(n_h - 1) s_h + e_h] t_p \times 2$	= 0.715 [in <sup>2</sup> ]		
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5] d_h t_p \times 2$	= 0.488 [in <sup>2</sup> ]		
Net area subject to tension when sheared out by center strip	$A_{nt} = (n_v - 1) (s_v - d_h) t_p$	= 1.105 [in <sup>2</sup> ]		
Block shear strength required	$V_u =$	= 25.00 [kips]		
Uniform tension stress factor	$U_{bs} = 1.00$			AISC 14 <sup>th</sup> Fig C-J4.2
Bolt shear resistance provided	$R_n = \min(0.6F_u A_{nv}, 0.6F_y A_{gv}) + U_{bs} F_u A_{nt}$	= 90.84 [kips]		AISC 14 <sup>th</sup> Eq J4-5
Resistance factor-LRFD	$\phi = 0.75$			AISC 14 <sup>th</sup> Eq J4-5
	$\phi R_n =$	= 68.13 [kips]		
	ratio = 0.37	> $V_u$	<b>OK</b>	

<b>Shear Tab - Shear Yielding</b>		ratio = 30.00 / 131.25	= 0.23	<b>PASS</b>
<b>Plate Shear Yielding Check</b>				
Plate size	width $b_p = 8.750$ [in]	thickness $t_p = 0.500$ [in]		
Plate yield strength	$F_y = 50.0$ [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$	= 4.375 [in <sup>2</sup> ]		
Shear force required	$V_u =$	= 30.00 [kips]		
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$	= 131.25 [kips]		AISC 14 <sup>th</sup> Eq J4-3
Resistance factor-LRFD	$\phi = 1.00$			AISC 14 <sup>th</sup> Eq J4-3
	$\phi R_n =$	= 131.25 [kips]		
	ratio = 0.23	> $V_u$	<b>OK</b>	

<b>Shear Tab - Shear Rupture</b>		ratio = 30.00 / 89.58	= 0.33	<b>PASS</b>
<b>Plate Shear Rupture Check</b>				
Bolt hole diameter	bolt dia $d_b = 3/4$ [in]	bolt hole dia $d_h = 7/8$ [in]		AISC 14 <sup>th</sup> B4.3b
Number of bolt	$n = 3$			
Plate size	width $b_p = 8.750$ [in]	thickness $t_p = 0.500$ [in]		
Plate tensile strength	$F_u = 65.0$ [ksi]			
Plate net area in shear	$A_{nv} = (b_p - n d_h) t_p$	= 3.063 [in <sup>2</sup> ]		
Shear force required	$V_u =$	= 30.00 [kips]		
Plate shear rupture strength	$R_n = 0.6 F_u A_{nv}$	= 119.44 [kips]		AISC 14 <sup>th</sup> Eq J4-4
Resistance factor-LRFD	$\phi = 0.75$			AISC 14 <sup>th</sup> Eq J4-4
	$\phi R_n =$	= 89.58 [kips]		
	ratio = 0.33	> $V_u$	<b>OK</b>	

<b>Shear Tab - Axial Tensile Yield</b>		ratio = 25.00 / 196.88	= 0.13	<b>PASS</b>
<b>Plate Tensile Yielding Check</b>				
Plate size	width $b_p = 8.750$ [in]	thickness $t_p = 0.500$ [in]		
Plate yield strength	$F_y = 50.0$ [ksi]			
Plate gross area in shear	$A_g = b_p t_p$	= 4.375 [in <sup>2</sup> ]		
Tensile force required	$P_u =$	= 25.00 [kips]		
Plate tensile yielding strength	$R_n = F_y A_g$	= 218.75 [kips]		AISC 14 <sup>th</sup> Eq J4-1
Resistance factor-LRFD	$\phi = 0.90$			AISC 14 <sup>th</sup> Eq J4-1
	$\phi R_n =$	= 196.88 [kips]		
	ratio = 0.13	> $P_u$		<b>OK</b>

<b>Shear Tab - Axial Tensile Rupture</b>		ratio = 25.00 / 149.30	= 0.17	<b>PASS</b>
<b>Plate Tensile Rupture Check</b>				
Bolt hole diameter	bolt dia $d_b = 3/4$ [in]	bolt hole dia $d_h = 7/8$ [in]		AISC 14 <sup>th</sup> B4.3b
Number of bolt	$n = 3$			
Plate size	width $b_p = 8.750$ [in]	thickness $t_p = 0.500$ [in]		
Plate tensile strength	$F_u = 65.0$ [ksi]			
Plate net area in tension	$A_{nt} = (b_p - n d_h) t_p$	= 3.063 [in <sup>2</sup> ]		
Tensile force required	$P_u =$	= 25.00 [kips]		
Plate tensile rupture strength	$R_n = F_u A_{nt}$	= 199.06 [kips]		AISC 14 <sup>th</sup> Eq J4-2
Resistance factor-LRFD	$\phi = 0.75$			AISC 14 <sup>th</sup> Eq J4-2
	$\phi R_n =$	= 149.30 [kips]		AISC 14 <sup>th</sup> Eq J4-2
	ratio = 0.17	> $P_u$		<b>OK</b>

<b>Shear Tab - Flexural Yield Interact</b>		ratio =	= 0.12	<b>PASS</b>
Plate width & thick	width $b_p = 8.750$ [in]	thick $t_p = 0.500$ [in]		
	yield $F_y = 50.0$ [ksi]			
Shear plate - gross area	$A_g = b_p \times t_p$	= 4.375 [in <sup>2</sup> ]		
Shear plate - plastic modulus	$Z_p = (b_p \times t_p^2) / 4$	= 9.570 [in <sup>3</sup> ]		
Axial strength available	$P_c =$ from axial tensile yield check	= 196.88 [kips]		
Axial strength required	$P_r =$ from user load input	= 25.00 [kips]		
Shear strength available	$V_c =$ from shear yielding check	= 131.25 [kips]		
Shear strength required	$V_r =$ from user load input	= 30.00 [kips]		
Flexural strength available	$M_c = \phi F_y Z_p \quad \phi=0.90$	= 35.89 [kip-ft]		
Bolt group ecc for shear $V_r$	$e_v =$	= 1.875 [in]		
Bolt group ecc for axial $P_r$	$e_p =$	= 0.000 [in]		
Flexural strength required	$M_r = V_r e_v + P_r e_p$	= 4.69 [kip-ft]		
Flexural yield interaction	ratio = $(\frac{V_r}{V_c})^2 + (\frac{P_r}{P_c} + \frac{M_r}{M_c})^2$	= 0.12		AISC 14 <sup>th</sup> Eq 10-5
		< 1.0		<b>OK</b>

Shear Tab - Flexural Rupture Interact		ratio =	= 0.22	PASS
<b>Plate <math>A_n</math> and <math>Z_{net}</math> Calc</b>				
Bolt hole diameter	bolt dia $d_b = 3/4$ [in]	bolt hole dia $d_h = 7/8$ [in]		AISC 14 <sup>th</sup> B4.3b
Number of bolt	$n = 3$			
Plate size	width $b_p = 8.750$ [in]	thickness $t_p = 0.500$ [in]		
Plate net area	$A_n = (b_p - n d_h) t_p$		= 3.063 [in <sup>2</sup> ]	
Plate net plastic sect modulus	$Z_{net} =$		= 6.850 [in <sup>3</sup> ]	
Plate net elastic sect modulus	$S_{net} =$		= 4.561 [in <sup>3</sup> ]	
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Plate width & thick	width $b_p = 8.750$ [in]	thick $t_p = 0.500$ [in]		
	tensile $F_u = 65.0$ [ksi]			
Axial strength available	$P_c =$ from axial tensile rupture check		= 149.30 [kips]	
Axial strength required	$P_r =$ from user load input		= 25.00 [kips]	
Shear strength available	$V_c =$ from shear rupture check		= 89.58 [kips]	
Shear strength required	$V_r =$ from user load input		= 30.00 [kips]	
Flexural strength available	$M_c = \phi F_u Z_{net}$ $\phi=0.75$		= 27.83 [kip-ft]	AISC 14 <sup>th</sup> Eq 9-4
Bolt group ecc for shear $V_r$	$e_v =$		= 1.875 [in]	
Bolt group ecc for axial $P_r$	$e_p =$		= 0.000 [in]	
Flexural strength required	$M_r = V_r e_v + P_r e_p$		= 4.69 [kip-ft]	
Flexural rupture interaction	$ratio = \left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right)^2$		= 0.22	AISC 14 <sup>th</sup> Eq 10-5
			< 1.0	OK

<b>Shear Tab - Bolt Bearing on Shear Tab</b>		ratio = 39.05 / 53.68	= <b>0.73</b>	<b>PASS</b>
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 = 30.00 [kips]		axial P = -25.00 [kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$		= <b>39.05</b> [kips]	
Resultant force/hor line load angle	$\theta = \tan^{-1}(V/P)$		= 50.19 [°]	
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Bolt hole diameter	bolt dia $d_b = 0.750$ [in]		bolt hole dia $d_{bh} = 0.813$ [in]	AISC 14 <sup>th</sup> 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]	
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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]	
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Bolt clear dist - bot right corner bolt	$L_{cA} = \min\left(\frac{e_v}{\sin \theta}, \frac{e_h}{\cos \theta}\right) - d_c$		= 1.384 [in]	
Bolt clear dist - right side edge bolt	$L_{cB} = \min\left(\frac{s_v - 0.5d_v}{\sin \theta}, \frac{e_h}{\cos \theta}\right) - d_c$		= 1.742 [in]	
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<b>Single Bolt Shear Strength</b>				
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Bolt shear stress	bolt grade = A325-N		$F_{nv} = 54.0$ [ksi]	AISC 14 <sup>th</sup> Table J3.2
	bolt dia $d_b = 0.750$ [in]		bolt area $A_b = 0.442$ [in <sup>2</sup> ]	
Single bolt shear strength	$R_{n-bolt} = F_{nv} A_b$		= 23.86 [kips]	AISC 14 <sup>th</sup> Eq J3-1
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Bolt bearing on plate	thick t = 0.500 [in]		tensile $F_u = 65.0$ [ksi]	
Bolt bearing strength	$R_{n-br} = 3.0 d_b t F_u$		= 73.13 [kips]	AISC 14 <sup>th</sup> Eq J3-6b
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Type A - Bolt Group Bottom Right Corner Bolt				
Number of bolt	$n_A = 1$			
Bolt tear out strength	$R_{n-tA} = 1.5 L_{cA} t F_u$		= 67.45 [kips]	AISC 14 <sup>th</sup> Eq J3-6b
Bolt bearing strength	$R_{nA} = \min(R_{n-tA}, R_{n-br}, R_{n-bolt})$		= <b>23.86</b> [kips]	
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Type B - Bolt Group Right Side Edge Bolt				
Number of bolt	$n_B = 2$			
Bolt tear out strength	$R_{n-tB} = 1.5 L_{cB} t F_u$		= 84.90 [kips]	AISC 14 <sup>th</sup> Eq J3-6b
Bolt bearing strength	$R_{nB} = \min(R_{n-tB}, R_{n-br}, R_{n-bolt})$		= <b>23.86</b> [kips]	
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Bolt bearing strength for all bolts	$R_n = n_A R_{nA} + n_B R_{nB} + n_C R_{nC} + n_D R_{nD}$		= 71.57 [kips]	
Bolt resistance factor-LRFD	$\phi = 0.75$			AISC 14 <sup>th</sup> J3-10
	$\phi R_n =$		= <b>53.68</b> [kips]	
	ratio = <b>0.73</b>		> R	<b>OK</b>



<b>Shear Tab - Beam Side - Block Shear - 1-Side Strip</b>		ratio = 30.00 / 98.72	= 0.30	<b>PASS</b>
<b>Plate Block Shear - Side Strip</b>				
Bolt hole diameter	bolt dia $d_b = 3/4$ [in]	bolt hole dia $d_h = 7/8$ [in]		AISC 14 <sup>th</sup> 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	$n_v = 1$	$n_h = 3$		
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	$A_{gv} = [ (n_h - 1) s_h + e_h ] t_p$	= 3.688 [in <sup>2</sup> ]		
Net area subject to shear	$A_{nv} = A_{gv} - [ (n_h - 1) + 0.5 ] d_h t_p$	= 2.594 [in <sup>2</sup> ]		
Net area subject to tension	$A_{nt} = ( e_v - 0.5 d_h ) t_p$	= 0.469 [in <sup>2</sup> ]		
Block shear strength required	$V_u =$	= 30.00 [kips]		
Uniform tension stress factor	$U_{bs} = 1.00$			AISC 14 <sup>th</sup> Fig C-J4.2
Bolt shear resistance provided	$R_n = \min ( 0.6F_u A_{nv} , 0.6F_y A_{gv} ) +$ $U_{bs} F_u A_{nt}$	= 131.63 [kips]		AISC 14 <sup>th</sup> Eq J4-5
Resistance factor-LRFD	$\phi = 0.75$			AISC 14 <sup>th</sup> Eq J4-5
	$\phi R_n =$	= 98.72 [kips]		
	ratio = 0.30	> $V_u$	<b>OK</b>	

<b>Shear Tab - Beam Side-Axial Tearout - Block Shear - Center Strip</b>		ratio = 25.00 / 131.02	= 0.19	<b>PASS</b>
<b>Plate Block Shear - Center Strip</b>				
Bolt hole diameter	bolt dia $d_b = 3/4$ [in]	bolt hole dia $d_h = 7/8$ [in]		AISC 14 <sup>th</sup> 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	$n_v = 3$	$n_h = 1$		
Bolt spacing in ver & hor dir	$s_v = 3.000$ [in]	$s_h = 3.000$ [in]		
Bolt edge dist in ver & hor dir	$e_v = 1.375$ [in]	$e_h = 1.375$ [in]		
Gross area subject to shear	$A_{gv} = [ (n_h - 1) s_h + e_h ] t_p \times 2$	= 1.375 [in <sup>2</sup> ]		
Net area subject to shear	$A_{nv} = A_{gv} - [ (n_h - 1) + 0.5 ] d_h t_p \times 2$	= 0.938 [in <sup>2</sup> ]		
Net area subject to tension	when sheared out by center strip $A_{nt} = ( n_v - 1 ) ( s_v - d_h ) t_p$	= 2.125 [in <sup>2</sup> ]		
Block shear strength required	$V_u =$	= 25.00 [kips]		
Uniform tension stress factor	$U_{bs} = 1.00$			AISC 14 <sup>th</sup> Fig C-J4.2
Bolt shear resistance provided	$R_n = \min ( 0.6F_u A_{nv} , 0.6F_y A_{gv} ) +$ $U_{bs} F_u A_{nt}$	= 174.69 [kips]		AISC 14 <sup>th</sup> Eq J4-5
Resistance factor-LRFD	$\phi = 0.75$			AISC 14 <sup>th</sup> Eq J4-5
	$\phi R_n =$	= 131.02 [kips]		
	ratio = 0.19	> $V_u$	<b>OK</b>	

<b>Shear Tab - Block Shear - Shear/Tensile Interact</b>		ratio =	= 0.13	<b>PASS</b>
Shear block shear strength required	$V_u =$	= 30.00	[kips]	
Axial block shear strength required	$P_u =$	= 25.00	[kips]	
Shear block shear strength available	$\phi R_{nv} =$ from calc shown above	= 98.72	[kips]	
Axial block shear strength available	$\phi R_{nt} =$ from calc shown above	= 131.02	[kips]	
Block shear shear/tensile interaction	$\text{ratio} = \left( \frac{V_u}{\phi R_{nv}} \right)^2 + \left( \frac{P_u}{\phi R_{nt}} \right)^2$	= 0.13		AISC 14 <sup>th</sup> Eq 10-5
		< 1.0	<b>OK</b>	

<b>Shear Tab - Lateral Stability / Stabilizer Plate</b>		ratio = 39.05 / 1319.47	= 0.03	<b>PASS</b>
Applied shear/axial forces	shear $V = 30.00$ [kips]	axial $P = -25.00$	[kips]	
Resultant shear force	$V_u = (V^2 + P^2)^{0.5}$	= 39.05	[kips]	
Distance from support to the first line of bolts	$a =$	= 1.875	[in]	
Plate thickness & depth	$t_p = 0.500$ [in]	$L = 8.750$	[in]	
Shear resistance provided	$R_n = 1500 \pi \frac{L t_p^3}{a^2}$	= 1466.08	[kips]	AISC 14 <sup>th</sup> Eq 10-6
Resistance factor-LRFD	$\phi = 0.90$			AISC 14 <sup>th</sup> Eq 10-6
	$\phi R_n =$	= 1319.47	[kips]	
	ratio = 0.03	> $V_u$	<b>OK</b>	

<b>Shear Tab - Plate Flexural Buckling</b>		ratio = 30.00 / 109.47	= 0.27	<b>PASS</b>
Shear tab size	depth = 8.750 [in]	thick = 0.500	[in]	
Plate buckling model	$c =$ dist from support to first bolt line	= 1.875	[in]	AISC 14 <sup>th</sup> Fig. 9-3
	$h_0 =$ shear tab depth	= 8.750	[in]	
	$t_w =$ shear tab thick	= 0.500	[in]	
Shear tab steel yield stress	$F_y = 50.0$ [ksi]			
Plate buckling factor	$\lambda = \frac{h_0 \sqrt{F_y}}{10 t_w \sqrt{475 + 280 (h_0/c)^2}}$	= 0.153		AISC 14 <sup>th</sup> Eq 9-18
Plate buckling factor	$Q =$	= 1.000		AISC 14 <sup>th</sup> Eq 9-15
Plate critical buckling stress	$F_{cr} = Q F_y$	= 50.0	[ksi]	AISC 14 <sup>th</sup> Eq 9-14
Shear force in demand	$V_u =$	= 30.00	[kips]	
Shear tab net elastic modulus	$S_{net} =$	= 4.561	[in <sup>3</sup> ]	
Shear force to bolt group CG ecc	$a =$	= 1.875	[in]	
Shear resistance	$R_n = F_{cr} S_{net} / a$	= 121.63	[kips]	AISC 14 <sup>th</sup> Eq 9-19
Resistance factor-LRFD	$\phi = 0.90$			AISC 14 <sup>th</sup> Eq 9-19
	$\phi R_n =$	= 109.47	[kips]	
	ratio = 0.27	> $V_u$	<b>OK</b>	

<b>Bolt Group Eccentricity</b>			
Bolt group forces	shear $V = 30.00$ [kips]	axial $P = 25.00$ [kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	$= 39.05$ [kips]	
Resultant force to ver Y axis angle	$\theta = \tan^{-1}(P/V)$	$= 39.81$ [°]	
<hr/>			
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	$\theta =$	$= 39.81$ [°]	
Bolt group coefficient C	$C =$ from AISC 14 <sup>th</sup> Table 7-6 ~ 7-13	$= 2.267$	
Bolt group eccentricity coefficient	$C_{ec} = C / (n_r \times n_c)$	$= 0.756$	
<hr/>			
<b>Shear Tab / Beam Web - Bolt Shear</b>		ratio = $39.05 / 40.58$	<b>= 0.96 PASS</b>
Bolt group forces	shear $V = 30.00$ [kips]	axial $P = -25.00$ [kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	$= 39.05$ [kips]	
<hr/>			
Bolt shear stress	bolt grade = A325-N	$F_{nv} = 54.0$ [ksi]	AISC 14 <sup>th</sup> Table J3.2
	bolt dia $d_b = 0.750$ [in]	bolt area $A_b = 0.442$ [in <sup>2</sup> ]	
Number of bolt carried shear	$n_s = 3.0$	shear plane $m = 1$	
Bolt group eccentricity coefficient	$C_{ec} =$ from 'Bolt Group Eccentricity' calc	$= 0.756$	
Required shear strength	$V_u =$	$= 39.05$ [kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	$= 54.11$ [kips]	AISC 14 <sup>th</sup> Eq J3-1
Bolt resistance factor-LRFD	$\phi = 0.75$		AISC 14 <sup>th</sup> Eq J3-1
	$\phi R_n =$	$= 40.58$ [kips]	
	ratio = <b>0.96</b>	$> V_u$	<b>OK</b>

<b>Shear Tab to Column Flange Weld Strength</b>		ratio = 8.03 / 14.63	= 0.55	<b>PASS</b>
<b>Weld Group Forces</b>				
	Shear V = 30.00 [kips]		Axial P = -25.00 [kips]	in tension
Shear force to bolt group CG ecc	$e_x =$		= 1.875 [in]	
Moment due to eccentric shear V	$M = V \times e_x$		= 4.69 [kip-ft]	
Shear tab weld length	L =		= 8.750 [in]	
<b>Combined Weld Stress</b>				
Weld stress from axial force	$f_a = P / L$		= -2.857 [kip/in]	in tension
Weld stress from shear force	$f_v = V / L$		= 3.429 [kip/in]	
Weld stress from moment force	$f_b = \frac{M}{L^2 / 6}$		= 4.408 [kip/in]	
Weld stress combined - max	$f_{max} = [ (f_a - f_b)^2 + f_v^2 ]^{0.5}$		= <b>8.034</b> [kip/in]	AISC 14 <sup>th</sup> Eq 8-11
Weld stress load angle	$\theta = \tan^{-1} \left( \frac{f_a - f_b}{f_v} \right)$		= 64.7 [°]	
<b>Fillet Weld Strength Calc</b>				
Fillet weld leg size	w = $\frac{5}{16}$ [in]		load angle $\theta = 64.7$ [°]	
Electrode strength	$F_{EXX} = 70.0$ [ksi]		strength coeff $C_1 = 1.00$	AISC 14 <sup>th</sup> Table 8-3
Number of weld line	n = 2 for double fillet			
Load angle coefficient	$C_2 = ( 1 + 0.5 \sin^{1.5} \theta )$		= 1.43	AISC 14 <sup>th</sup> Page 8-9
Fillet weld shear strength	$R_{n-w} = 0.6 ( C_1 \times 70 \text{ ksi} ) 0.707 w n C_2$		= 26.539 [kip/in]	AISC 14 <sup>th</sup> Eq 8-1
Base metal - shear tab	thickness t = 0.500 [in]		tensile $F_u = 65.0$ [ksi]	
Base metal - shear tab is in shear, <u>shear</u> rupture as per AISC 14 <sup>th</sup> Eq J4-4 is checked				AISC 14 <sup>th</sup> J2.4
Base metal shear rupture	$R_{n-b} = 0.6 F_u t$		= 19.500 [kip/in]	AISC 14 <sup>th</sup> Eq J4-4
Double fillet linear shear strength	$R_n = \min ( R_{n-w}, R_{n-b} )$		= <b>19.500</b> [kip/in]	AISC 14 <sup>th</sup> Eq 9-2
Resistance factor-LRFD	$\phi = 0.75$			AISC 14 <sup>th</sup> Eq 8-1
	$\phi R_n =$		= <b>14.625</b> [kip/in]	
	ratio = <b>0.55</b>		> $f_{max}$	<b>OK</b>

<b>Column Web Local Yielding</b>		ratio = 25.00 / 204.29	= 0.12	<b>PASS</b>
Concentrated force from gusset	$P_u =$		= <b>25.00</b> [kips]	
Column section	d = 11.900 [in]		$t_f = 0.515$ [in]	
	$t_w = 0.295$ [in]		k = 1.020 [in]	
	yield $F_y = 50.0$ [ksi]			
Length of bearing	$l_b = \text{clip angle length}$		= 8.750 [in]	
Column web local yielding strength	$R_n = F_y t_w ( 5 k + l_b )$		= 204.29 [kips]	AISC 14 <sup>th</sup> Eq J10-2
Resistance factor-LRFD	$\phi = 1.00$			
	$\phi R_n =$		= <b>204.29</b> [kips]	
	ratio = <b>0.12</b>		> $P_u$	<b>OK</b>

<b>Column Flange Local Bending</b>		ratio = 25.00 / 74.59	= 0.34	<b>PASS</b>
Concentrated force from gusset	$P_u =$		= 25.00 [kips]	
Column w section	$t_f = 0.515$ [in]		yield $F_y = 50.0$ [ksi]	
Column flange local bending strength	$R_n = 6.25 F_y t_f^2$		= 82.88 [kips]	AISC 14 <sup>th</sup> Eq J10-1
Resistance factor-LRFD	$\phi = 0.90$			AISC 14 <sup>th</sup> J10.1
	$\phi R_n =$		= 74.59 [kips]	
	ratio = 0.34		> $P_u$	<b>OK</b>