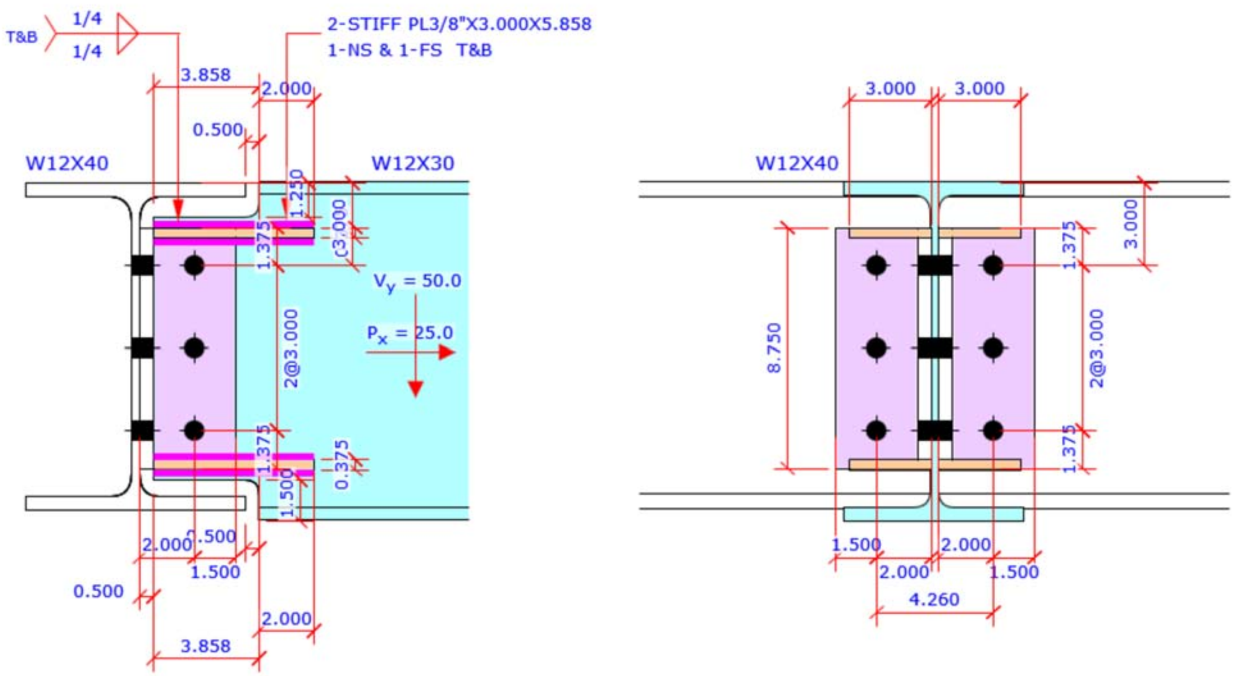
Beam to Girder	Clip Angle Shear Connection	Code=AISC 360-10 LRFD

Result Summary	geometries & weld limitations = **PASS**	limit states max ratio = **0.95** **PASS**

Sketch	Shear Connection	Code=AISC 360-10 LRFD



Members & Components Summary		
Member	Shear Connection	Code=AISC 360-10 LRFD

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

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

W Shape Beam - Tensile Yield		ratio = 25.00 / 111.74	= **0.22**		**PASS**
Gross area subject to tension	A_g =		= 2.483	[in^2]	
Steel yield strength	F_y =		= 50.0	[ksi]	
Tensile force required	P_u =		= **25.00**	[kips]	
Tensile yielding strength	$R_n = F_y A_g$		= 124.15	[kips]	AISC 14th Eq D2-1
Resistance factor-LRFD	ϕ = 0.90				AISC 14th D2 (a)
	$\phi\ R_n$ =		= **111.74**	[kips]	AISC 14th Eq D2-1
	ratio = **0.22**		> P_u	OK	

W Shape Beam - Tensile Rupture			ratio = 25.00 / 87.77	= **0.28**	**PASS**
W beam section	= W12X30				
	d = 12.300	[in]	b_f = 6.520	[in]	
	t_f = 0.440	[in]	t_w = 0.260	[in]	
	A_g = 2.483	[in^2]			
Bolt hole diameter	bolt dia d_b = $^3/_4$	[in]	bolt hole dia d_h = $^7/_8$	[in]	AISC 14th B4.3b
Number of bolt row	n = 3				
W section net area	$A_n = A_g - n\ d_h t_w$		= 1.801	[in^2]	
Shear lag factor	U =		= 1.000		AISC 14th D3
Tensile force required	P_u =		= **25.00**	[kips]	
Tensile effective net area	$A_e = A_n U$		= 1.801	[in^2]	
Plate tensile strength	F_u =		= 65.0	[ksi]	
Tensile rupture strength	$R_n = F_u A_e$		= 117.03	[kips]	AISC 14th Eq D2-2
Resistance factor-LRFD	ϕ = 0.75				AISC 14th D2 (b)
	ϕR_n =		= **87.77**	[kips]	AISC 14th Eq D2-2
	ratio = **0.28**		> P_u	OK	

Beam Web - Shear Yielding			ratio = 50.00 / 74.49	= **0.67**	**PASS**
Plate Shear Yielding Check					
Plate size	width b_p = 9.550	[in]	thickness t_p = 0.260	[in]	
Plate yield strength	F_y = 50.0	[ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$		= 2.483	[in^2]	
Shear force required	V_u =		= **50.00**	[kips]	
Plate shear yielding strength	$R_n = 0.6\ F_y A_{gv}$		= 74.49	[kips]	AISC 14th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00				AISC 14th Eq J4-3
	ϕR_n =		= **74.49**	[kips]	
	ratio = **0.67**		> V_u	OK	

Beam Web - Shear Rupture			ratio = 50.00 / 52.66	= **0.95**	**PASS**
Plate Shear Rupture Check					
Bolt hole diameter	bolt dia d_b = $^3/_4$	[in]	bolt hole dia d_h = $^7/_8$	[in]	AISC 14th B4.3b
Number of bolt	n = 3				
Plate size	width b_p = 9.550	[in]	thickness t_p = 0.260	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate net area in shear	$A_{nv} = (b_p - n\ d_h)\ t_p$		= 1.801	[in^2]	
Shear force required	V_u =		= **50.00**	[kips]	
Plate shear rupture strength	$R_n = 0.6\ F_u A_{nv}$		= 70.22	[kips]	AISC 14th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75				AISC 14th Eq J4-4
	ϕR_n =		= **52.66**	[kips]	
	ratio = **0.95**		> V_u	OK	

Beam Web - Bolt Bearing on Beam Web		ratio = 55.90 / 85.56	= **0.65**	**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 $V = 50.00$ [kips]		axial $P = -25.00$	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$		= **55.90**	[kips]	
Resultant force/hor line load angle	$\theta = \tan^{-1} (V / P)$		= 63.43	[°]	
Bolt hole diameter	bolt dia $d_b = 0.750$ [in]		bolt hole dia $d_{bh} = 0.813$	[in]	AISC 14th B4.3b
Bolt hole ver. dimension	$d_v =$		= 0.813	[in]	
Bolt hole hor. dimension	$d_h =$		= 0.813	[in]	
Bolt center to bolt hole edge dist	$d_c = 0.5\ d_{bh}$		= 0.406	[in]	
Bolt no in ver & hor direction	Bolt Row $n_v = 3$		Bolt Col $n_h = 1$		
Bolt spacing	ver $s_v = 3.000$ [in]				
Bolt edge distance	ver $e_v = 1.750$ [in]		hor $e_h = 1.500$	[in]	
Bolt clear dist - bot right corner bolt	$L_{cA} = \min\left(\frac{e_v}{\sin\theta} , \frac{e_h}{\cos\theta} \right) - d_c$		= 1.550	[in]	
Bolt clear dist - right side edge bolt	$L_{cB} = \min\left(\frac{s_v - 0.5d_v}{\sin\theta} , \frac{e_h}{\cos\theta} \right) - d_c$		= 2.494	[in]	

Single Bolt Shear Strength

Bolt shear stress	bolt grade = A325-N		$F_{nv} = 54.0$	[ksi]	AISC 14th Table J3.2
	bolt dia $d_b = 0.750$ [in]		bolt area $A_b = 0.442$	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = 2 \times F_{nv} A_b$		= 47.71	[kips]	AISC 14th Eq J3-1
Bolt bearing on plate	thick $t = 0.260$ [in]		tensile $F_u = 65.0$	[ksi]	
Bolt bearing strength	$R_{n\text{-}br} = 3.0\ d_b\, t\, F_u$		= 38.03	[kips]	AISC 14th Eq J3-6b
Type A - Bolt Group Bottom Right Corner Bolt					
Number of bolt	$n_A = 1$				
Bolt tear out strength	$R_{n\text{-}tA} = 1.5\ L_{cA}\, t\, F_u$		= 39.30	[kips]	AISC 14th Eq J3-6b
Bolt bearing strength	$R_{nA} = \min(R_{n\text{-}tA}, R_{n\text{-}br}, R_{n\text{-}bolt})$		= **38.03**	[kips]	
Type B - Bolt Group Right Side Edge Bolt					
Number of bolt	$n_B = 2$				
Bolt tear out strength	$R_{n\text{-}tB} = 1.5\ L_{cB}\, t\, F_u$		= 63.21	[kips]	AISC 14th Eq J3-6b
Bolt bearing strength	$R_{nB} = \min(R_{n\text{-}tB}, R_{n\text{-}br}, R_{n\text{-}bolt})$		= **38.03**	[kips]	
Bolt bearing strength for all bolts	$R_n = n_A R_{nA} + n_B R_{nB} + n_C R_{nC} + n_D R_{nD}$		= 114.08	[kips]	
Bolt resistance factor-LRFD	$\phi = 0.75$				AISC 14th J3-10
	$\phi R_n =$		= **85.56**	[kips]	
	ratio = **0.65**		> R	OK	

Beam Web - Shear - Block Shear - 1-Side Strip			ratio = 50.00 / 55.77	= **0.90**	**PASS**
Plate Block Shear - Side Strip					
Bolt hole diameter	bolt dia $d_b = {}^3/_4$	[in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 14th B4.3b
Plate thickness	$t_p = 0.260$	[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	$e_v = 1.500$	[in]	$e_h = 1.750$	[in]	
Gross area subject to shear	$A_{gv} = [\ (n_h - 1)\ s_h + e_h\ ]\ t_p$		= 2.015	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [\ (n_h - 1) + 0.5\ ]\ d_h\ t_p$		= 1.446	[in^2]	
Net area subject to tension	$A_{nt} = (\ e_v - 0.5\ d_h\ )\ t_p$		= 0.276	[in^2]	
Block shear strength required	$V_u =$		= **50.00**	[kips]	
Uniform tension stress factor	$U_{bs} = 1.00$				AISC 14th Fig C-J4.2
Bolt shear resistance provided	$R_n = \min\ (0.6F_u A_{nv}\ ,\ 0.6F_y A_{gv}\ ) + U_{bs} F_u A_{nt}$		= 74.36	[kips]	AISC 14th Eq J4-5
Resistance factor-LRFD	$\phi = 0.75$				AISC 14th Eq J4-5
	$\phi\ R_n =$		= **55.77**	[kips]	
	ratio = **0.90**		$> V_u$	OK	

Beam Web - Axial Tearout - Block Shear - Center Strip			ratio = 25.00 / 70.03	= **0.36**	**PASS**
Plate Block Shear - Center Strip					
Bolt hole diameter	bolt dia $d_b = {}^3/_4$	[in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 14th B4.3b
Plate thickness	$t_p = 0.260$	[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]	$s_h = 1.750$	[in]	
Bolt edge dist in ver & hor dir	$e_v = 3.000$	[in]	$e_h = 1.500$	[in]	
Gross area subject to shear	$A_{gv} = [\ (n_h - 1)\ s_h + e_h\ ]\ t_p \times 2$		= 0.780	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5]\ d_h\ t_p \times 2$		= 0.553	[in^2]	
Net area subject to tension when sheared out by center strip	$A_{nt} = (\ n_v - 1)\ (\ s_v - d_h\ )\ t_p$		= 1.105	[in^2]	
Block shear strength required	$V_u =$		= **25.00**	[kips]	
Uniform tension stress factor	$U_{bs} = 1.00$				AISC 14th Fig C-J4.2
Bolt shear resistance provided	$R_n = \min\ (0.6F_u A_{nv}\ ,\ 0.6F_y A_{gv}\ ) + U_{bs} F_u A_{nt}$		= 93.37	[kips]	AISC 14th Eq J4-5
Resistance factor-LRFD	$\phi = 0.75$				AISC 14th Eq J4-5
	$\phi\ R_n =$		= **70.03**	[kips]	
	ratio = **0.36**		$> V_u$	OK	

Coped Beam - Flexural Rupture			ratio = 50.00 / 240.69 = **0.21**		**PASS**
Beam section & cope side	sect = W12X30		cope side = double cope		
Beam top flange cope	depth d_c = 1.250	[in]	length L_c = 3.858	[in]	
Beam bottom flange cope	depth d_c = 1.500	[in]	length L_c = 3.858	[in]	
S_{net} of Coped Beam With Hor Reinforcing Stiffener Plates					
Beam sect W12X30	d = 12.300	[in]	b_f = 6.520	[in]	
	t_f = 0.440	[in]	t_w = 0.260	[in]	
Stiffener plate size	w_p = 3.000	[in]	t_p = 0.375	[in]	
Flange cope depth-top & bot flange	d_{ct} = 1.250	[in]	d_{cb} = 1.500	[in]	
Properties of Coped W Sect With Hor Reinforcing Stiffener Plates					
Top flange	b_{ft} = 6.260	[in]	t_{ft} = 0.375	[in]	
Bottom flange	b_{fb} = 6.260	[in]	t_{fb} = 0.375	[in]	
W sect depth	d = 8.800	[in]	web t_w = 0.260	[in]	
Dist from sect centroid to T&B flange face	x_t = 4.400	[in]	x_b = 4.400	[in]	
Max dist sect centroid to T&B flange face	$x_{max} = \max(x_t, x_b)$		= 4.400	[in]	
W sect moment of inertia	I_x =		= 94.7	[in^4]	
W sect elastic modulus	$S_{net} = I_x / x_{max}$		= **21.52**	[in^3]	
Beam section tensile strength	F_u =		= 65.0	[ksi]	
Distance from face of cope to the point of inflection of beam	e =		= 4.358	[in]	AISC 14th Page 9-6
Beam end shear force	V_u =		= **50.00**	[kips]	
Beam end shear resistance	$R_n = F_u S_{net} / e$		= 320.92	[kips]	AISC 14th Eq 9-4
Resistance factor-LRFD	ϕ = 0.75				AISC 14th Eq 9-4
	ϕR_n =		= **240.69**	[kips]	
	ratio = **0.21**		> V_u	OK	

Coped Beam - Local Web Buckling			ratio = 50.00 / 222.17	= **0.23**	**PASS**
Beam section & cope side	sect = W12X30		cope side = double cope		
Beam top flange cope	depth d_{ct} = 1.250	[in]	length L_{ct} = 3.858	[in]	
Beam bottom flange cope	depth d_{cb} = 1.500	[in]	length L_{cb} = 3.858	[in]	
S_{net} of Coped Beam With Hor Reinforcing Stiffener Plates					
Beam sect W12X30	d = 12.300	[in]	b_f = 6.520	[in]	
	t_f = 0.440	[in]	t_w = 0.260	[in]	
Stiffener plate size	w_p = 3.000	[in]	t_p = 0.375	[in]	
Flange cope depth-top & bot flange	d_{ct} = 1.250	[in]	d_{cb} = 1.500	[in]	
Properties of Coped W Sect With Hor Reinforcing Stiffener Plates					
Top flange	b_{ft} = 6.260	[in]	t_{ft} = 0.375	[in]	
Bottom flange	b_{fb} = 6.260	[in]	t_{fb} = 0.375	[in]	
W sect depth	d = 8.800	[in]	web t_w = 0.260	[in]	
Dist from sect centroid to T&B flange face	x_t = 4.400	[in]	x_b = 4.400	[in]	
Max dist sect centroid to T&B flange face	$x_{max} = \max(x_t, x_b)$		= 4.400	[in]	
W sect moment of inertia	I_x =		= 94.7	[in^4]	
W sect elastic modulus	$S_{net} = I_x / x_{max}$		= **21.52**	[in^3]	
Distance from face of cope to the point of inflection of beam	e =		= 4.358	[in]	AISC 14th Page 9-6
Beam section	depth d = 12.300	[in]	web t_w = 0.260	[in]	
	F_y = 50.0	[ksi]	E = 29000	[ksi]	
	$f_d = 3.5 - 7.5\,(d_{ct} / d)$		= 2.738		AISC 14th Eq 9-13
Reduced beam depth	$h_0 = d - d_{ct} - d_{cb}$		= 9.550	[in]	
Plate local buckling stress	$F_{cr} = 0.62\,\pi\,E\,\dfrac{t_w^2}{L_{ct}\,h_0}\,f_d$		= 283.7	[ksi]	AISC 14th Eq 9-12
	$F_{cr} = F_{cr} \le F_y$		= 50.0	[ksi]	AISC 14th Eq 9-12
Beam end shear force	V_u =		= **50.00**	[kips]	
Beam end shear resistance	$R_n = F_{cr}\,S_{net} / e$		= 246.86	[kips]	AISC 14th Eq 9-6
Resistance factor-LRFD	ϕ = 0.90				AISC 14th Eq 9-6
	$\phi\,R_n$ =		= **222.17**	[kips]	
	ratio = **0.23**		> V_u	OK	

Hor Stiffener to Coped Beam Web Fillet Weld Limitation					**PASS**
Min Fillet Weld Size					
Thinner part joined thickness	$t =$		= 0.260	[in]	
Min fillet weld size allowed	$w_{min} =$		= **0.188**	[in]	AISC 14th Table J2.4
Fillet weld size provided	$w =$		= **0.250**	[in]	
			> w_{min}	OK	
Min Fillet Weld Length					
Fillet weld size provided	$w =$		= 0.250	[in]	
Min fillet weld length allowed	$L_{min} = 4 \times w$		= **1.000**	[in]	AISC 14th J2.2b
Min fillet weld length	$L =$		= **5.858**	[in]	
			> L_{min}	OK	
Hor Reinforcing Stiffener Extension Beyond Cope					
To prevent local crippling of the beam web, the longitudinal stiffening must be extended min a distance of d_c beyond the cope					AISC 14th Fig 9-10 (b)
Flange cope depth-top & bot flange	$d_{ct} = 1.250$ [in]		$d_{cb} = 1.500$	[in]	
Max cope depth - top & bot flange	$d_c = \max(d_{ct}, d_{cb})$		= **1.500**	[in]	
Hor stiffener plate extension beyond cope	$L_e =$		= **2.000**	[in]	
			> d_c	OK	

Hor Stiffener to Coped Beam Web Fillet Weld Strength	ratio = 5.22 / 10.97	= **0.48**	**PASS**

Stiffener to Coped Beam Web Weld Line Force Calc

Refer to AISC Design Example v15 Page IIA-78 for the formula used below on how to get the stiffener weld line forces

From S_{net} calc in Coped Beam - Local Web Buckling check above, the properties of stiffener reinforced W section

Reinforced W sect moment of inertia	I_{net} =	= 94.7	[in^4]	
Reinforced stiffener plate area	A_p =	= 2.348	[in^2]	
Dist from centroid of reinforced sect to centroid of stiffener plate	y =	= 4.213	[in]	
First moment of reinforced stiffener plate	$Q = A_p\, y$	= 9.892	[in^3]	
Beam end shear force	V_u =	= **50.00**	[kips]	
Weld line shear stress	$r_{u1} = \dfrac{V_u\, Q}{I_{net}}$	= 5.223	[kip/in]	
Distance from face of cope to the point of inflection of beam	e =	= 4.358	[in]	AISC 14th Page 9-6
Beam web hor coped length	L_c =	= 3.858	[in]	
Hor stiffener plate extension beyond the cope	L_e =	= 2.000	[in]	
Stiffener to beam web weld length	$L_w = L_c + L_e$	= 5.858	[in]	
Weld line shear stress	$r_{u2} = \dfrac{V_u\, e\, Q}{I_{net}\, L_w}$	= 3.886	[kip/in]	
Weld line shear stress - max	$r_u = \max(r_{u1}, r_{u2})$	= **5.223**	[kip/in]	

Fillet Weld Strength Calc

Fillet weld leg size	w = $^1/_4$ [in]	load angle θ = 0.0	[°]	
Electrode strength	F_{EXX} = 70.0 [ksi]	strength coeff C_1 = 1.00		AISC 14th Table 8-3
Number of weld line	n = 2 for double fillet			
Load angle coefficient	$C_2 = (1 + 0.5 \sin^{1.5}\theta)$	= 1.00		AISC 14th Page 8-9
Fillet weld shear strength	$R_{n\text{-}w} = 0.6\,(C_1 \times 70\text{ ksi})\; 0.707\, w\, n\, C_2$	= 14.847	[kip/in]	AISC 14th Eq 8-1
Base metal - stiffener	thickness t = 0.375 [in]	tensile F_u = 65.0	[ksi]	
Base metal - stiffener is in shear, <u>shear</u> rupture as per AISC 14th Eq J4-4 is checked				AISC 14th J2.4
Base metal shear rupture	$R_{n\text{-}b} = 0.6\, F_u\, t$	= 14.625	[kip/in]	AISC 14th Eq J4-4
Double fillet linear shear strength	$R_n = \min(R_{n\text{-}w}, R_{n\text{-}b})$	= **14.625**	[kip/in]	AISC 14th Eq 9-2
Resistance factor-LRFD	φ = 0.75			AISC 14th Eq 8-1
	ϕR_n =	= **10.969**	[kip/in]	
	ratio = **0.48**	> r_u	OK	

Clip Angle - Beam Side - Shear Yielding			ratio = 25.00 / 131.25	= **0.19**	**PASS**
Plate Shear Yielding Check					
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^2]	
Shear force required	V_u =		= **25.00**	[kips]	
Plate shear yielding strength	$R_n = 0.6\ F_y A_{gv}$		= 131.25	[kips]	AISC 14th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00				AISC 14th Eq J4-3
	$\phi\ R_n$ =		= **131.25**	[kips]	
	ratio = **0.19**		> V_u	OK	

Clip Angle - Beam Side - Shear Rupture			ratio = 25.00 / 89.58	= **0.28**	**PASS**
Plate Shear Rupture Check					
Bolt hole diameter	bolt dia $d_b = {}^3/_4$	[in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 14th B4.3b
Number of bolt	n = 3				
Plate size	width b_p = 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^2]	
Shear force required	V_u =		= **25.00**	[kips]	
Plate shear rupture strength	$R_n = 0.6\ F_u A_{nv}$		= 119.44	[kips]	AISC 14th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75				AISC 14th Eq J4-4
	$\phi\ R_n$ =		= **89.58**	[kips]	
	ratio = **0.28**		> V_u	OK	

Clip Angle - Beam Side - Axial Tensile Yield			ratio = 12.50 / 196.88	= **0.06**	**PASS**
Plate Tensile Yielding Check					
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^2]	
Tensile force required	P_u =		= **12.50**	[kips]	
Plate tensile yielding strength	$R_n = F_y A_g$		= 218.75	[kips]	AISC 14th Eq J4-1
Resistance factor-LRFD	ϕ = 0.90				AISC 14th Eq J4-1
	$\phi\ R_n$ =		= **196.88**	[kips]	
	ratio = **0.06**		> P_u	OK	

Clip Angle - Beam Side - Axial Tensile Rupture	ratio = 12.50 / 149.30	= **0.08**	**PASS**			
Plate Tensile Rupture Check						
Bolt hole diameter	bolt dia $d_b = {}^3/_4$	[in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 14th B4.3b
Number of bolt	$n = 3$				
Plate size	width $b_p = 8.750$	[in]	thickness $t_p = 0.500$	[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^2]	
Tensile force required	$P_u =$		= **12.50**	[kips]	
Plate tensile rupture strength	$R_n = F_u A_{nt}$		= 199.06	[kips]	AISC 14th Eq J4-2
Resistance factor-LRFD	$\phi = 0.75$				AISC 14th Eq J4-2
	$\phi R_n =$		= **149.30**	[kips]	AISC 14th Eq J4-2
	ratio = **0.08**		> P_u	OK	

Clip Angle - Beam Side - Bolt Bearing on Clip Angle		ratio = 27.95 / 53.68	= **0.52**	**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 = 50.00 [kips]	axial P = -25.00	[kips]		
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$	= 55.90	[kips]		
Each angle or plate takes	R = 0.50 x R	= **27.95**	[kips]		
Resultant force/hor line load angle	$\theta = \tan^{-1} (V / P)$	= 63.43	[°]		
Bolt hole diameter	bolt dia d_b = 0.750 [in]	bolt hole dia d_{bh} = 0.813	[in]	AISC 14th B4.3b	
Bolt hole ver. dimension	d_v =	= 0.813	[in]		
Bolt hole hor. dimension	d_h =	= 0.813	[in]		
Bolt center to bolt hole edge dist	$d_c = 0.5\ d_{bh}$	= 0.406	[in]		
Bolt no in ver & hor direction	Bolt Row n_v = 3	Bolt Col n_h = 1			
Bolt spacing	ver s_v = 3.000 [in]				
Bolt edge distance	ver e_v = 1.375 [in]	hor e_h = 1.500	[in]		
Bolt clear dist - bot right corner bolt	$L_{cA} = \min\left(\frac{e_v}{\sin\theta}, \frac{e_h}{\cos\theta} \right) - d_c$	= 1.131	[in]		
Bolt clear dist - right side edge bolt	$L_{cB} = \min\left(\frac{s_v - 0.5d_v}{\sin\theta}, \frac{e_h}{\cos\theta} \right) - d_c$	= 2.494	[in]		

Single Bolt Shear Strength

Bolt shear stress	bolt grade = A325-N	F_{nv} = 54.0	[ksi]	AISC 14th Table J3.2
	bolt dia d_b = 0.750 [in]	bolt area A_b = 0.442	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$	= 23.86	[kips]	AISC 14th Eq J3-1
Bolt bearing on plate	thick t = 0.500 [in]	tensile F_u = 65.0	[ksi]	
Bolt bearing strength	$R_{n\text{-}br} = 3.0\ d_b\ t\ F_u$	= 73.13	[kips]	AISC 14th Eq J3-6b
Type A - Bolt Group Bottom Right Corner Bolt				
Number of bolt	n_A = 1			
Bolt tear out strength	$R_{n\text{-}tA} = 1.5\ L_{cA}\ t\ F_u$	= 55.14	[kips]	AISC 14th Eq J3-6b
Bolt bearing strength	$R_{nA} = \min(R_{n\text{-}tA}, R_{n\text{-}br}, R_{n\text{-}bolt})$	= **23.86**	[kips]	
Type B - Bolt Group Right Side Edge Bolt				
Number of bolt	n_B = 2			
Bolt tear out strength	$R_{n\text{-}tB} = 1.5\ L_{cB}\ t\ F_u$	= 121.57	[kips]	AISC 14th Eq J3-6b
Bolt bearing strength	$R_{nB} = \min(R_{n\text{-}tB}, R_{n\text{-}br}, R_{n\text{-}bolt})$	= **23.86**	[kips]	
Bolt bearing strength for all bolts	$R_n = n_A R_{nA} + n_B R_{nB} + n_C R_{nC} + n_D R_{nD}$	= 71.57	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 14th J3-10
	ϕR_n =	= **53.68**	[kips]	
	ratio = **0.52**	> R	OK	

Clip Angle - Beam Side - Block Shear - 1-Side Strip	ratio = 25.00 / 101.77	= **0.25**	**PASS**			
Plate Block Shear - Side Strip						
Bolt hole diameter	bolt dia $d_b = {}^3/_4$	[in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 14th B4.3b	
Plate thickness	$t_p = 0.500$	[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	$e_v = 1.500$	[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^2]		
Net area subject to shear	$A_{nv} = A_{gv} - [\ (n_h - 1) + 0.5\ ]\ d_h\ t_p$		= 2.594	[in^2]		
Net area subject to tension	$A_{nt} = (\ e_v - 0.5\ d_h\ )\ t_p$		= 0.531	[in^2]		
Block shear strength required	$V_u =$		= **25.00**	[kips]		
Uniform tension stress factor	$U_{bs} = 1.00$				AISC 14th Fig C-J4.2	
Bolt shear resistance provided	$R_n = \min(0.6F_u A_{nv},\ 0.6F_y A_{gv}) + U_{bs} F_u A_{nt}$		= 135.69	[kips]	AISC 14th Eq J4-5	
Resistance factor-LRFD	$\phi = 0.75$				AISC 14th Eq J4-5	
	$\phi\ R_n =$		= **101.77**	[kips]		
	ratio = **0.25**		> V_u	OK		

Clip Angle - Beam Side-Axial Tearout - Block Shear - Center Strip	ratio = 12.50 / 134.67	= **0.09**	**PASS**			
Plate Block Shear - Center Strip						
Bolt hole diameter	bolt dia $d_b = {}^3/_4$	[in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 14th B4.3b	
Plate thickness	$t_p = 0.500$	[in]				
Plate strength	$F_y = 50.0$	[ksi]	$F_u = 65.0$	[ksi]		
Bolt no in ver & hor dir	$n_v = 3$		$n_h = 1$			
Bolt spacing in ver & hor dir	$s_v = 3.000$	[in]	$s_h = 1.750$	[in]		
Bolt edge dist in ver & hor dir	$e_v = 1.375$	[in]	$e_h = 1.500$	[in]		
Gross area subject to shear	$A_{gv} = [\ (n_h - 1)\ s_h + e_h\ ]\ t_p \times 2$		= 1.500	[in^2]		
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5]\ d_h\ t_p \times 2$		= 1.063	[in^2]		
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^2]		
Block shear strength required	$V_u =$		= **12.50**	[kips]		
Uniform tension stress factor	$U_{bs} = 1.00$				AISC 14th Fig C-J4.2	
Bolt shear resistance provided	$R_n = \min(0.6F_u A_{nv},\ 0.6F_y A_{gv}) + U_{bs} F_u A_{nt}$		= 179.56	[kips]	AISC 14th Eq J4-5	
Resistance factor-LRFD	$\phi = 0.75$				AISC 14th Eq J4-5	
	$\phi\ R_n =$		= **134.67**	[kips]		
	ratio = **0.09**		> V_u	OK		

Clip Angle - Beam Side - Block Shear - Shear / Tensile Interact	ratio =		= **0.07**		**PASS**
Shear block shear strength required	V_u =		= 25.00	[kips]	
Axial block shear strength required	P_u =		= 12.50	[kips]	
Shear block shear strength available	ϕR_{nv} = from calc shown above		= 101.77	[kips]	
Axial block shear strength available	ϕR_{nt} = from calc shown above		= 134.67	[kips]	
Block shear shear/tensile interaction	ratio = $\left(\frac{V_u}{\phi R_{nv}}\right)^2 + \left(\frac{P_u}{\phi R_{nt}}\right)^2$		= **0.07**		AISC 14th Eq 10-5
			< 1.0	OK	

Clip Angle - Girder Side - Shear Rupture		ratio = 25.00 / 89.58	= **0.28**		**PASS**
Plate Shear Rupture Check					
Bolt hole diameter	bolt dia $d_b = {}^3/_4$ [in]	bolt hole dia $d_h = {}^7/_8$		[in]	AISC 14th B4.3b
Number of bolt	n = 3				
Plate size	width b_p = 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^2]	
Shear force required	V_u =		= **25.00**	[kips]	
Plate shear rupture strength	$R_n = 0.6\ F_u A_{nv}$		= 119.44	[kips]	AISC 14th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75				AISC 14th Eq J4-4
	ϕR_n =		= **89.58**	[kips]	
	ratio = **0.28**		> V_u	OK	

Clip Angle - Girder Side - Bolt Bearing on Clip Angle ratio = 25.00 / 53.68 = **0.47** **PASS**

Single Bolt Shear Strength

Bolt shear stress	bolt grade = A325-N	F_{nv} = 54.0	[ksi]	AISC 14th Table J3.2
	bolt dia d_b = 0.750 [in]	bolt area A_b = 0.442	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$	= 23.86	[kips]	AISC 14th Eq J3-1

Bolt Bearing / TearOut Strength on Plate

Bolt hole diameter	bolt dia d_b = $^3/_4$ [in]	bolt hole dia d_h = $^{13}/_{16}$	[in]	AISC 14th Table J3.3
Bolt spacing & edge distance	spacing L_s = 3.000 [in]	edge distance L_e = 1.375	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate thickness	t = 0.500 [in]			

Interior Bolt

Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.188	[in]	
Bolt tear out/bearing strength	$R_{n\text{-}t\&b\text{-}in} = 1.5 L_c t F_u \le 3.0 d_b t F_u$			AISC 14th Eq J3-6b
	= 106.64 ≤ 73.13	= 73.13	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$	= 23.86	[kips]	

Edge Bolt

Bolt hole edge clear distance	$L_c = L_e - d_h / 2$	= 0.969	[in]	
Bolt tear out/bearing strength	$R_{n\text{-}t\&b\text{-}ed} = 1.5 L_c t F_u \le 3.0 d_b t F_u$			AISC 14th Eq J3-6b
	= 47.23 ≤ 73.13	= 47.23	[kips]	
Bolt strength at edge	$R_{n\text{-}ed} = \min(R_{n\text{-}t\&b\text{-}ed}, R_{n\text{-}bolt})$	= 23.86	[kips]	

Number of bolt	interior n_{in} = 2	edge n_{ed} = 1		
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-}in} + n_{ed} R_{n\text{-}ed}$	= 71.57	[kips]	
Required shear strength	V_u =	= **25.00**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 14th J3-10
	ϕR_n =	= **53.68**	[kips]	
	ratio = **0.47**	> V_u	OK	

Clip Angle - Girder Side - Block Shear - 1-Side Strip			ratio = 25.00 / 101.77	= **0.25**	**PASS**
Plate Block Shear - Side Strip					
Bolt hole diameter	bolt dia $d_b = {}^3/_4$	[in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 14th B4.3b
Plate thickness	t_p = 0.500	[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 = 1.500	[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^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5] d_h t_p$		= 2.594	[in^2]	
Net area subject to tension	$A_{nt} = (e_v - 0.5 d_h) t_p$		= 0.531	[in^2]	
Block shear strength required	V_u =		= **25.00**	[kips]	
Uniform tension stress factor	U_{bs} = 1.00				AISC 14th Fig C-J4.2
Bolt shear resistance provided	$R_n = \min(0.6F_u A_{nv}, 0.6F_y A_{gv}) + U_{bs} F_u A_{nt}$		= 135.69	[kips]	AISC 14th Eq J4-5
Resistance factor-LRFD	ϕ = 0.75				AISC 14th Eq J4-5
	ϕR_n =		= **101.77**	[kips]	
	ratio = **0.25**		> V_u	OK	

Clip Angle / Beam Web - Bolt Shear			ratio = 55.90 / 107.35	= **0.52**	**PASS**
Bolt group forces	shear V = 50.00	[kips]	axial P = -25.00	[kips]	
Bolt group resultant force	$R = (V^2 + P^2)^{0.5}$		= **55.90**	[kips]	
Bolt shear stress	bolt grade = A325-N		F_{nv} = 54.0	[ksi]	AISC 14th Table J3.2
	bolt dia d_b = 0.750	[in]	bolt area A_b = 0.442	[in^2]	
Number of bolt carried shear	n_s = 3.0		shear plane m = 2		
Beam Side Bolt Group Eccentricity					
Eccentricity in double angle connection can be neglected when bolt group has single vertical row of bolt and the distance from face of angle OSL to bolt group CG is less than 3 inch					AISC 14th Page 10-8
Bolt group eccentricity coefficient	C_{ec} =		= 1.000		AISC 14th Page 10-8
Required shear strength	V_u =		= **55.90**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$		= 143.14	[kips]	AISC 14th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 14th Eq J3-1
	ϕR_n =		= **107.35**	[kips]	
	ratio = **0.52**		> V_u	OK	

Clip Angle / Girder - Bolt Shear			ratio = 25.00 / 53.68	= **0.47**	**PASS**
Bolt group forces	shear V = 25.00	[kips]	axial P = 12.50	[kips]	
Bolt shear stress	grade = A325-N		F_{nv} = 54.0	[ksi]	AISC 14th Table J3.2
	bolt dia d_b = 0.750	[in]	bolt area A_b = 0.442	[in^2]	
Number of bolt carried shear	n_s = 3.0		shear plane m = 1		
Bolt group eccentricity coefficient	C_{ec} =		= 1.000		
Required shear strength	V_u =		= **25.00**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$		= 71.57	[kips]	AISC 14th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 14th Eq J3-1
	ϕR_n =		= **53.68**	[kips]	
	ratio = **0.47**		> V_u	OK	

Clip Angle / Girder - Bolt Bearing on Girder			ratio = 50.00 / 107.35	= **0.47**	**PASS**
Single Bolt Shear Strength					
Bolt shear stress	bolt grade = A325-N		F_{nv} = 54.0	[ksi]	AISC 14th Table J3.2
	bolt dia d_b = 0.750	[in]	bolt area A_b = 0.442	[in^2]	
Single bolt shear strength	$R_{n\text{-bolt}} = F_{nv} A_b$		= 23.86	[kips]	AISC 14th Eq J3-1
Bolt Bearing / TearOut Strength on Plate					
Bolt hole diameter	bolt dia $d_b = {}^{3}/_{4}$	[in]	bolt hole dia $d_h = {}^{13}/_{16}$	[in]	AISC 14th Table J3.3
Bolt spacing	spacing L_s = 3.000	[in]			
Plate tensile strength	F_u = 65.0	[ksi]			
Plate thickness	t = 0.295	[in]			
Interior Bolt					
Bolt hole edge clear distance	$L_c = L_s - d_h$		= 2.188	[in]	
Bolt tear out/bearing strength	$R_{n\text{-t\&b-in}} = 1.5\, L_c\, t\, F_u \le 3.0\, d_b\, t\, m\, F_u$				AISC 14th Eq J3-6b
	= 62.92 ≤ 43.14		= 43.14	[kips]	
Bolt strength at interior	$R_{n\text{-in}} = \min(R_{n\text{-t\&b-in}}, R_{n\text{-bolt}})$		= 23.86	[kips]	
Number of bolt	interior n_{in} = 6				
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-in}}$		= 143.14	[kips]	
Required shear strength	V_u =		= **50.00**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 14th J3-10
	ϕR_n =		= **107.35**	[kips]	
	ratio = **0.47**		> V_u	OK	

Clip Angle / Girder - Angle Leg Bending		ratio = 1.11 / 2.67	= **0.42**	**PASS**	
Angle leg on beam	width b = 8.750	[in]	thickness t = 0.500	[in]	
	tensile F_u = 65.0	[ksi]	bolt gage g = 2.000	[in]	
Beam web thickness	t_p = 0.260	[in]			

The angle leg bending moment is derived based on the assumption that the 2L legs to form a single span L=2d beam with both ends fixed and tensile point load 2P imposed at mid span of this beam, so the moment M=(1/8) x 2P x 2d = 0.5 P d

1/2 beam span - distance from bolt center to gusset plate center	$d = g + 0.5\, t_p$	= 2.130	[in]	
Axial tensile load on single angle	P =	= 12.50	[kips]	
Moment in demand	$M_r = 0.5\, P\, d$	= **1.11**	[kip-ft]	
Moment capacity	$M_n = (t^2 b) / 4 \times F_u$	= 2.96	[kip-ft]	AISC 14th Eq 15-21
Resistance factor-LRFD	ϕ = 0.90			AISC 14th Eq 15-21
	ϕM_n =	= **2.67**	[kip-ft]	
	ratio = **0.42**	> M_r	OK	

Bolt Tensile Prying Action on Clip Angle ratio = 4.17 / 13.35 = **0.31** **PASS**

For 2L clip angle, all loads to be x 0.5 for single angle

Bolt group forces shear V = 25.00 [kips] axial P = -12.50 [kips]

Single Bolt Tensile Capacity Without Considering Prying

Bolt grade grade = A325-N

Nominal tensile/shear stress F_{nt} = 90.0 [ksi] F_{nv} = 54.0 [ksi] AISC 14th Table J3.2

bolt dia d_b = 0.750 [in] bolt area A_b = 0.442 [in^2]

Bolt group shear force shear V = 25.00 [kips] no of bolt n = 3

Shear stress required $f_{rv} = V / (n A_b)$ = 18.86 [ksi]

Resistance factor-LRFD ϕ = 0.75 AISC 14th J3.7

Modified nominal tensile stress $F'_{nt} = 1.3 F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_{rv} \le F_{nt}$ = **75.08** [ksi] AISC 14th Eq J3-3a

Bolt norminal tensile strength $r_n = F'_{nt} A_b$ = 33.17 [kips] AISC 14th Eq J3-1

Resistance factor-LRFD ϕ = 0.75 AISC 14th J3.6

Single bolt tensile capacity ϕr_n = = **24.88** [kips]

Single Bolt Tensile Capacity After Considering Prying

Clip angle leg width L = 3.500 [in] bolt gage g = 2.000 [in]

leg t = 0.500 [in]

Dist from bolt center to leg edge $a = L - g$ = 1.500 [in]

$a' = a + 0.5 d_b \le (1.25 b + 0.5 d_b)$ = 1.875 [in] AISC 14th Eq 9-27

Bolt hole diameter bolt dia d_b = 0.750 [in] bolt hole dia d_h = 0.813 [in] AISC 14th B4.3b

Dist from bolt center to centerline of angle leg $b = g - 0.5 t$ = 1.750 [in]

$b' = b - 0.5 d_b$ = 1.375 [in] AISC 14th Eq 9-21

Angle length L = 8.750 [in] Bolt Col n_v = 3

Bolt spacing s_v = 3.000

Bolt tributary length $p = L / n_v$ $p \le 2b$ and $p \le s_v$ = 2.917 [in] AISC 14th Page 9-11

$\rho = b' / a'$ = 0.733 AISC 14th Eq 9-26

$\delta = 1 - d_h / p$ = 0.721 AISC 14th Eq 9-24

Tensile capacity per bolt before considering prying B = from calc shown in above section = 24.88 [kips]

Resistance factor-LRFD ϕ = 0.90 AISC 14th Page 9-10

Clip angle leg thickness t = 0.500 [in] tensile F_u = 65.0 [ksi]

Plate thickness req'd to develop bolt tensile capacity without prying $t_c = \left(\frac{4 B b'}{\phi p F_u} \right)^{0.5}$ = 0.896 [in] AISC 14th Eq 9-30a

$\alpha' = \frac{1}{\delta (1 + \rho)} \left[\left(\frac{t_c}{t} \right)^2 - 1 \right]$ = 1.765 AISC 14th Eq 9-35

when $\alpha' > 1$ $Q = \left(\frac{t}{t_c} \right)^2 (1 + \delta)$ = 0.537 AISC 14th Eq 9-34

Bolt tensile force per bolt in demand T = from calc shown below = **4.17** [kips]

Tensile strength per bolt after considering prying $\phi r_n = B \times Q$ = **13.35** [kips] AISC 14th Eq 9-31

ratio = **0.31** > T OK

Calculate Max Single Bolt Tensile Load

For 2L clip angle, all loads to be x 0.5 for single angle

Bolt group force axial P = 12.50 [kips]

Bolt number Bolt Row n = 1 Bolt Col n = 3