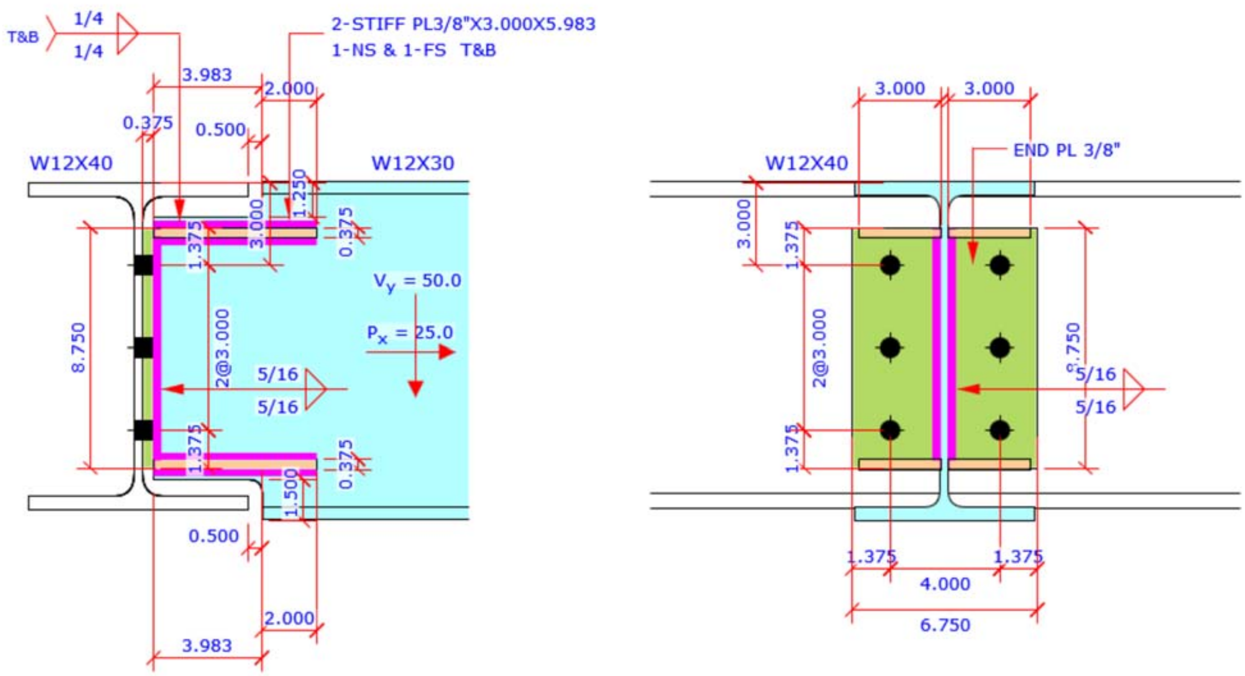
Beam to Girder	End Plate Shear Connection	Code=AISC 360-10 LRFD

Result Summary	geometries & weld limitations = **PASS**	limit states max ratio = **0.90**	**PASS**

Sketch	Shear Connection	Code=AISC 360-10 LRFD



Members & Components Summary

Member	Shear Connection	Code=AISC 360-10 LRFD

End Plate

Plate	W = 6.750	[in]	L = 8.750	[in]
	t = 0.375	[in]		
Steel Grade A992	F_y = 50.0	[ksi]	F_u = 65.0	[ksi]

Bolt end plate bolt

Bolt	dia = 0.750	[in]		
	grade = A325-N		F_u = 120.0	[ksi]
	F_{nt} = 90.0	[ksi]	F_{nv} = 54.0	[ksi]
slip critical	SC = No			

Geometry Restriction Check - End Plate to Girder					**PASS**
Min Bolt Edge Distance - End Plate to Girder					
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 End Plate to Girder	L_e =		= **1.375**	[in]	
			> $L_{e\text{-}min}$	OK	
Min Bolt Spacing - End Plate to Girder					
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 End Plate to Girder	L_s =		= **3.000**	[in]	
			> $L_{s\text{-}min}$	OK	

Weld Limitation Check - Beam Web to End Plate					**PASS**
Min Fillet Weld Size					
Thinner part joined thickness	t =		= 0.260	[in]	
Min fillet weld size allowed	w_{min} =		= **0.188**	[in]	AISC 14th Table J2.4
Fillet weld size provided	w =		= **0.313**	[in]	
			> w_{min}	OK	
Min Fillet Weld Length					
Fillet weld size provided	w =		= 0.313	[in]	
Min fillet weld length allowed	L_{min} = 4 x w		= **1.250**	[in]	AISC 14th J2.2b
Min fillet weld length	L =		= **8.750**	[in]	
			> L_{min}	OK	

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
	$\phi\ R_n$ =		= **74.49**	[kips]	
	ratio = **0.67**		> V_u	OK	

Beam Web - Shear Rupture			ratio = 50.00 / 72.63	= **0.69**	**PASS**
Plate Shear Rupture Check					
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	$A_{nv} = b_p t_p$		= 2.483	[in^2]	
Shear force in demand	V_u =		= **50.00**	[kips]	
Plate shear rupture strength	$R_n = 0.6 F_u A_{nv}$		= 96.84	[kips]	AISC 14th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75				AISC 14th Eq J4-4
	ϕR_n =		= **72.63**	[kips]	
	ratio = **0.69**		> V_u	OK	

Beam Web - Tensile Yielding		ratio = 25.00 / 102.38	= **0.24**	**PASS**
End Plate Direct Connect Length Calc				
Beam web-end plate connect length	L =	= 8.750	[in]	
Beam web thickness	t_w =	= 0.260	[in]	
Gross area subject to tension	$A_g = L t_w$	= 2.275	[in^2]	
Gross area subject to tension	A_g =	= 2.275	[in^2]	
Steel yield strength	F_y =	= 50.0	[ksi]	
Tensile force required	P_u =	= **25.00**	[kips]	
Tensile yielding strength	$R_n = F_y A_g$	= 113.75	[kips]	AISC 14th Eq D2-1
Resistance factor-LRFD	ϕ = 0.90			AISC 14th D2 (a)
	ϕR_n =	= **102.38**	[kips]	AISC 14th Eq D2-1
	ratio = **0.24**	> P_u	OK	

Beam Web - Tensile Rupture			ratio = 25.00 / 102.98	= **0.24**	**PASS**
End Plate Direct Connect Length Calc					
Beam web-end plate weld length	L =		= 8.750	[in]	
Beam web-end plate fillet weld size	w =		= 0.313	[in]	
Beam web-end plate connect length	$L_w = L - 2w$		= 8.125	[in]	
Plate Tensile Rupture Check					
Plate size	width b_p = 8.125 [in]		thickness t_p = 0.260	[in]	
Plate tensile strength	F_u = 65.0 [ksi]				
Plate net area in tension	$A_{nt} = b_p t_p$		= 2.113	[in^2]	
Tensile force in demand	P_u =		= **25.00**	[kips]	
Plate tensile rupture strength	$R_n = F_u A_{nt}$		= 137.31	[kips]	AISC 14th Eq J4-2
Resistance factor-LRFD	ϕ = 0.75				AISC 14th Eq J4-2
	ϕR_n =		= **102.98**	[kips]	AISC 14th Eq J4-2
	ratio = **0.24**		> P_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 = 3.983 [in]		
Beam bottom flange cope	depth d_c = 1.500 [in]	length L_c = 3.983 [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 14^{th} 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 14^{th} Eq 9-4	
Resistance factor-LRFD	ϕ = 0.75		AISC 14^{th} 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} = 3.983	[in]	
Beam bottom flange cope	depth d_{cb} = 1.500	[in]	length L_{cb} = 3.983	[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	$F_{cr} = 0.62\,\pi\,E\,\dfrac{t_w^2}{L_{ct}\,h_0}\,f_d$		= 274.8	[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 x w		= **1.000**	[in]	AISC 14th J2.2b
Min fillet weld length	L =		= **5.983**	[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 =	= 3.983	[in]	
Hor stiffener plate extension beyond the cope	L_e =	= 2.000	[in]	
Stiffener to beam web weld length	$L_w = L_c + L_e$	= 5.983	[in]	
Weld line shear stress	$r_{u2} = \dfrac{V_u\, e\, Q}{I_{net}\, L_w}$	= 3.804	[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
	$\phi\, R_n$ =	= **10.969**	[kip/in]	
	ratio = **0.48**	> r_u	OK	

End Plate - Shear Yield			ratio = 25.00 / 98.44	= **0.25**	**PASS**
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 =		= **25.00**	[kips]	
Plate shear yielding strength	$R_n = 0.6\ F_y A_{gv}$		= 98.44	[kips]	AISC 14th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00				AISC 14th Eq J4-3
	$\phi\ R_n$ =		= **98.44**	[kips]	
	ratio = **0.25**		> V_u	OK	

End Plate - Shear Rupture			ratio = 25.00 / 67.18	= **0.37**	**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 = 0.375	[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 =		= **25.00**	[kips]	
Plate shear rupture strength	$R_n = 0.6\ F_u A_{nv}$		= 89.58	[kips]	AISC 14th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75				AISC 14th Eq J4-4
	$\phi\ R_n$ =		= **67.18**	[kips]	
	ratio = **0.37**		> V_u	OK	

End Plate - Block Shear - Center Strip			ratio = 50.00 / 170.93	= **0.29**	**PASS**
Plate Block Shear - Center Strip					
Bolt hole diameter	bolt dia d_b = $^3/_4$	[in]	bolt hole dia d_h = $^7/_8$	[in]	AISC 14th B4.3b
Plate thickness	t_p = 0.375	[in]			
Plate strength	F_y = 50.0	[ksi]	F_u = 65.0	[ksi]	
Bolt no in ver & hor dir	n_v = 2		n_h = 3		
Bolt spacing in ver & hor dir	s_v = 4.000	[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	$A_{gv} = [\ (n_h - 1)\ s_h + e_h\ ]\ t_p \times 2$		= 5.531	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5]\ d_h t_p \times 2$		= 3.891	[in^2]	
Net area subject to tension when sheared out by center strip	$A_{nt} = (\ n_v - 1)\ (\ s_v - d_h\ )\ t_p$		= 1.172	[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	$R_n = \min\ (0.6F_u A_{nv}\ ,\ 0.6F_y A_{gv}\ ) + U_{bs} F_u A_{nt}$		= 227.91	[kips]	AISC 14th Eq J4-5
Resistance factor-LRFD	ϕ = 0.75				AISC 14th Eq J4-5
	$\phi\ R_n$ =		= **170.93**	[kips]	
	ratio = **0.29**		> V_u	OK	

End Plate - Block Shear - 2-Side Strip			ratio = 50.00 / 148.08	= **0.34**	**PASS**
Plate Block Shear - 2 Side Strips					
Bolt hole diameter	bolt dia $d_b = {}^{3}/_{4}$	[in]	bolt hole dia $d_h = {}^{7}/_{8}$	[in]	AISC 14th B4.3b
Plate thickness	$t_p = 0.375$	[in]			
Plate strength	$F_y = 50.0$	[ksi]	$F_u = 65.0$	[ksi]	
Bolt no in ver & hor dir	$n_v = 2$		$n_h = 3$		
Bolt spacing in ver & hor dir	$s_v = 4.000$	[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	$A_{gv} = [\ (n_h - 1)\ s_h + e_h\ ]\ t_p \times 2$		= 5.531	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5]\ d_h\ t_p \times 2$		= 3.891	[in^2]	
Net area subject to tension when sheared out by 2 side strips	$A_{nt} = (\ e_v - 0.5\ d_h\ )\ t_p \times 2$		= 0.703	[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	$R_n = \min\ (0.6F_u A_{nv}\ ,\ 0.6F_y A_{gv}\ ) + U_{bs} F_u A_{nt}$		= 197.44	[kips]	AISC 14th Eq J4-5
Resistance factor-LRFD	$\phi = 0.75$				AISC 14th Eq J4-5
	$\phi\ R_n =$		= **148.08**	[kips]	
	ratio = **0.34**		$> V_u$	OK	

End Plate - Bolt Bearing on End Plate			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 & 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 = 0.375	[in]			
Interior Bolt					
Bolt hole edge clear distance	$L_c = L_s - d_h$		= 2.188	[in]	
Bolt tear out/bearing strength	$R_{n\text{-t\&b-in}} = 1.5\, L_c\, t\, F_u \le 3.0\, d_b\, t\, F_u$				AISC 14th Eq J3-6b
	= 79.98 ≤ 54.84		= 54.84	[kips]	
Bolt strength at interior	$R_{n\text{-in}} = \min(R_{n\text{-t\&b-in}}, R_{n\text{-bolt}})$		= 23.86	[kips]	
Edge Bolt					
Bolt hole edge clear distance	$L_c = L_e - d_h / 2$		= 0.969	[in]	
Bolt tear out/bearing strength	$R_{n\text{-t\&b-ed}} = 1.5\, L_c\, t\, F_u \le 3.0\, d_b\, t\, F_u$				AISC 14th Eq J3-6b
	= 35.42 ≤ 54.84		= 35.42	[kips]	
Bolt strength at edge	$R_{n\text{-ed}} = \min(R_{n\text{-t\&b-ed}}, R_{n\text{-bolt}})$		= 23.86	[kips]	
Number of bolt	interior n_{in} = 4		edge n_{ed} = 2		
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-in}} + n_{ed} R_{n\text{-ed}}$		= 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	

End Plate / Girder - Bolt Shear			ratio = 50.00 / 107.35	= **0.47**	**PASS**
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 = 6.0		shear plane m = 1		
Bolt group eccentricity coefficient	C_{ec} =		= 1.000		
Required shear strength	V_u =		= **50.00**	[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 = **0.47**		> V_u	OK	

End Plate / 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	

Bolt Tensile Prying Action on End Plate		ratio = 4.17 / 6.91	= **0.60**	**PASS**
Bolt group forces	shear V = 50.00 [kips]	axial P = -25.00 [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 = 50.00 [kips]	no of bolt n = 6		
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 normal 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

End plate	width w = 6.750 [in]	bolt gage g = 4.000	[in]	
	web t_w = 0.260 [in]			
Dist from bolt center to plate edge	$a = 0.5 (w - g)$	= 1.375	[in]	
	$a' = a + 0.5 d_b \le (1.25 b + 0.5 d_b)$	= 1.750	[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 face of web	$b = 0.5(g - t_w)$	= 1.870	[in]	
	$b' = b - 0.5 d_b$	= 1.495	[in]	AISC 14th Eq 9-21
Bolt pitch spacing	s_v = 3.000			
Bolt tributary length	$p = s_v$ $p \le 2b$ and $p \le s_v$	= 2.917	[in]	AISC 14th Page 9-11
	$\rho = b' / a'$	= 0.854		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
End plate thickness	t = 0.375 [in]	tensile F_u = 65.0	[ksi]	
Plate thickness req'd to develop bolt tensile capacity without prying	$t_c = \left(\frac{4 B b'}{\phi p F_u} \right)^{0.5}$	= 0.934	[in]	AISC 14th Eq 9-30a
	$\alpha' = \frac{1}{\delta (1 + \rho)} \left[\left(\frac{t_c}{t} \right)^2 - 1 \right]$	= 3.887		AISC 14th Eq 9-35
when $\alpha' > 1$	$Q = \left(\frac{t}{t_c} \right)^2 (1 + \delta)$	= 0.278		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	$\phi r_n = B \times Q$	= **6.91**	[kips]	AISC 14th Eq 9-31
	ratio = **0.60**	> T	OK	

Calculate Max Single Bolt Tensile Load

Bolt group force	axial P = 25.00 [kips]			
Bolt number	Bolt Row n_h = 2	Bolt Col n_v = 3		
Bolt tensile force per bolt	$T = P / (n_v n_h)$	= **4.17**	[kips]	

Beam Web to End Plate Weld Strength		ratio = 6.88 / 7.61	= **0.90**	**PASS**

Weld Group Forces

	shear V = 50.00 [kips]	axial P = -25.00	[kips]	in tension
Beam web-end plate weld length	L =	= 8.750	[in]	
Beam web-end plate fillet weld size	w =	= 0.313	[in]	
Beam web-end plate weld length used for design	$L_w = L - 2w$	= 8.125	[in]	

Combined Weld Stress

Weld stress from axial force	$f_a = P / L$	= -3.077	[kip/in]	in tension
Weld stress from shear force	$f_v = V / L$	= 6.154	[kip/in]	
Weld stress combined - max	$f_{max} = (f_a^2 + f_v^2)^{0.5}$	= **6.880**	[kip/in]	AISC 14th Eq 8-11
Weld stress load angle	$\theta = \tan^{-1} (\frac{f_a}{f_v})$	= 26.6	[°]	

Fillet Weld Strength Calc

Fillet weld leg size	$w = {}^{5}/_{16}$ [in]	load angle θ = 26.6	[°]	
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	$C_2 = (1 + 0.5 \sin^{1.5} \theta)$	= 1.15		AISC 14th Page 8-9
Fillet weld shear strength	$R_{n-w} = 0.6 (C_1 \times 70 \text{ ksi}) 0.707 \, w \, n \, C_2$	= 21.334	[kip/in]	AISC 14th Eq 8-1
Base metal - beam web	thickness t = 0.260 [in]	tensile F_u = 65.0	[ksi]	

Base metal - beam web is in shear, shear rupture as per AISC 14th Eq J4-4 is checked — AISC 14th J2.4

Base metal shear rupture	$R_{n-b} = 0.6 F_u t$	= 10.140	[kip/in]	AISC 14th Eq J4-4
Double fillet linear shear strength	$R_n = \min (R_{n-w} , R_{n-b})$	= **10.140**	[kip/in]	AISC 14th Eq 9-2
Resistance factor-LRFD	$\phi = 0.75$			AISC 14th Eq 8-1
	$\phi R_n =$	= **7.605**	[kip/in]	
	ratio = **0.90**	> f_{max}	OK	