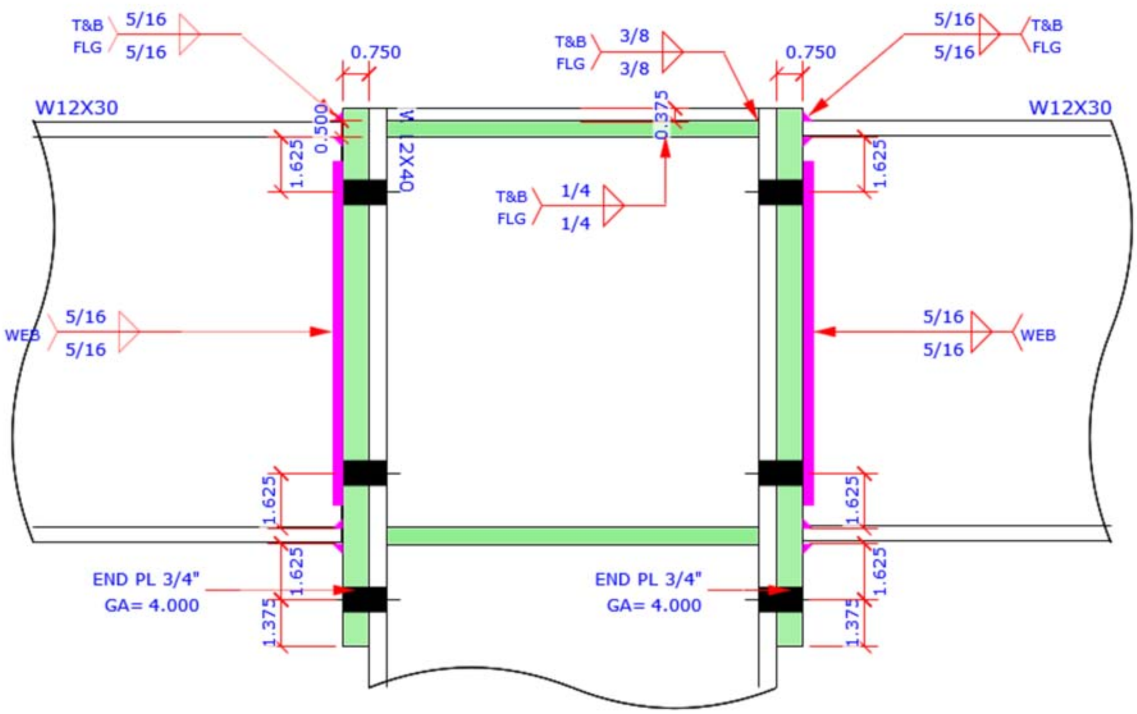
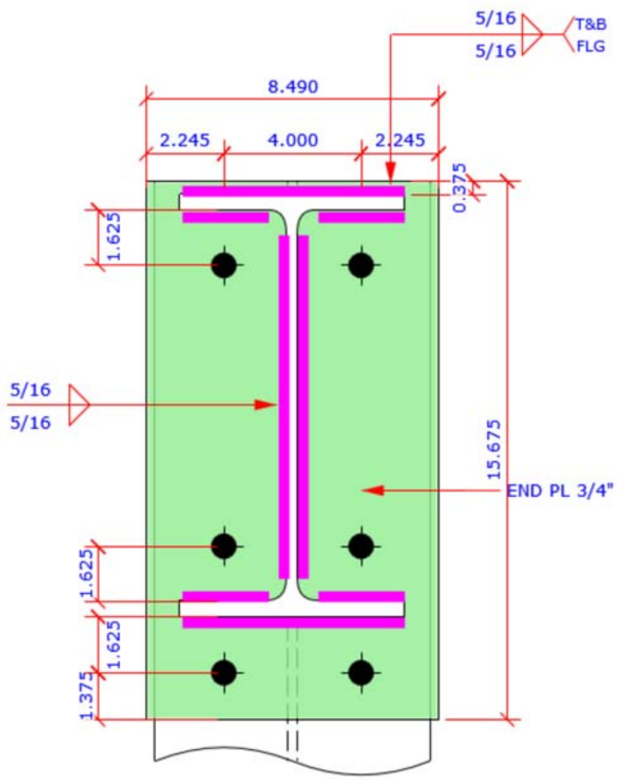
Result Summary - Overall	Moment Connection - Beam to Column	Code=AISC 360-10 LRFD

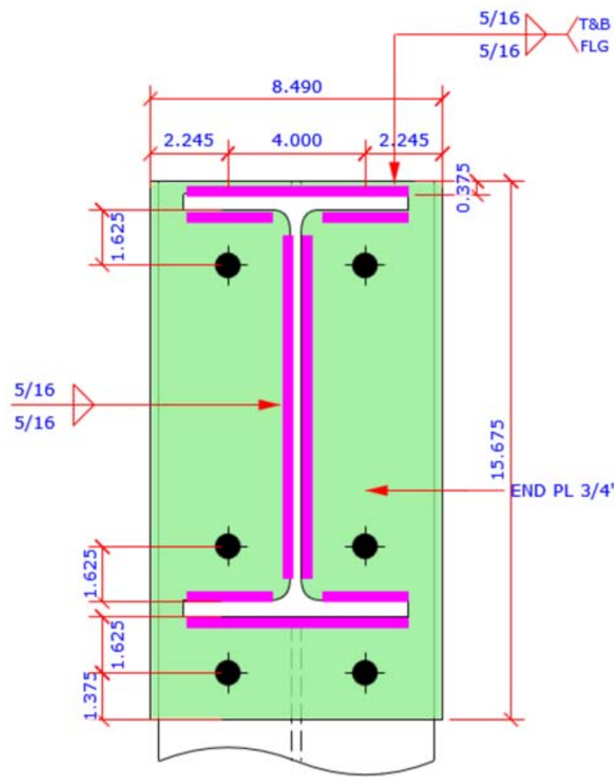
Result Summary - Overall	geometries & weld limitations = **PASS**	limit states max ratio = **0.95**	**PASS**
Right Beam to Column	geometries & weld limitations = **PASS**	limit states max ratio = **0.84**	**PASS**
Left Beam to Column	geometries & weld limitations = **PASS**	limit states max ratio = **0.95**	**PASS**

Sketch	Moment Connection - Beam to Column	Code=AISC 360-10 LRFD

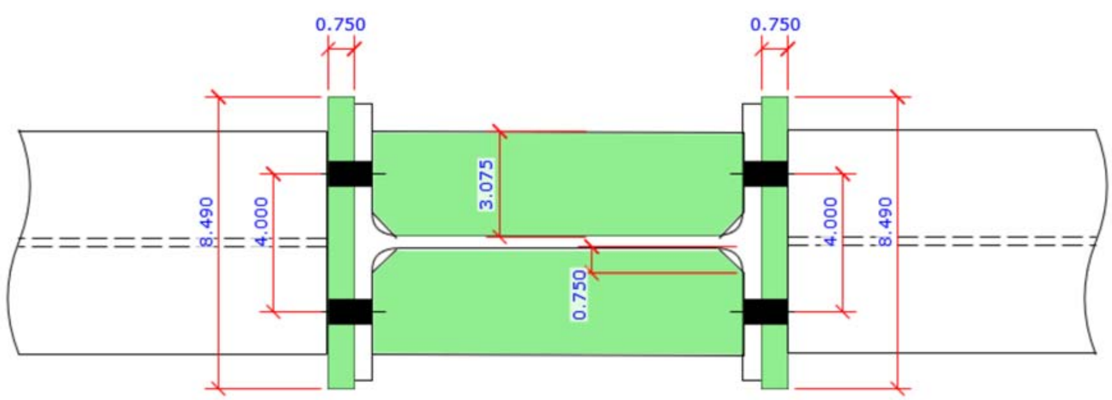


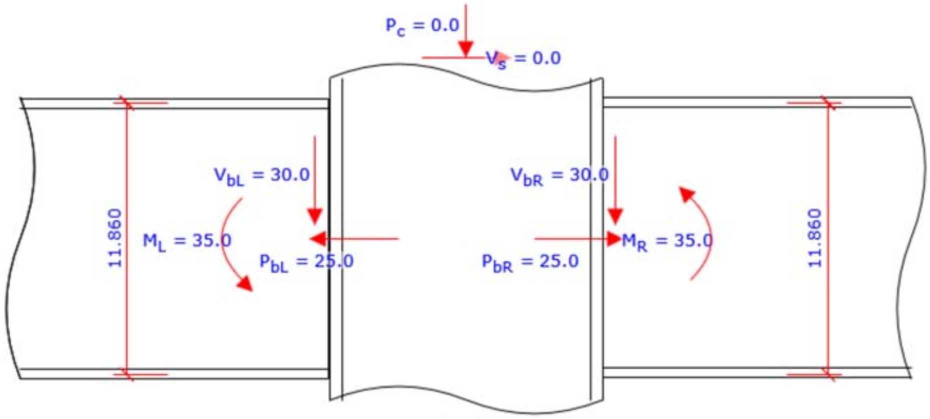


Left Side Beam

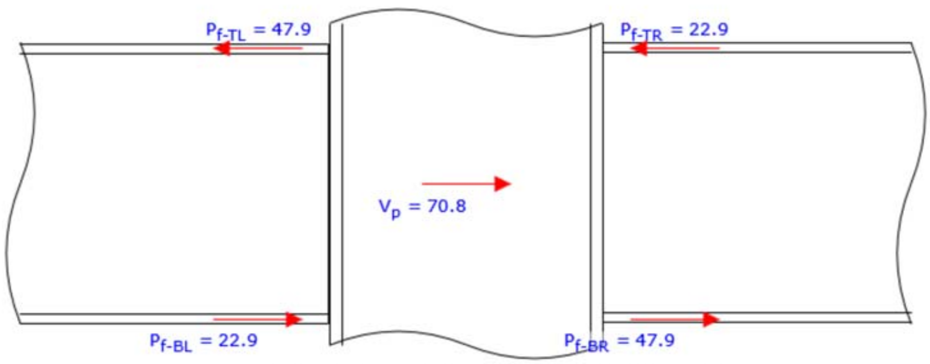


Right Side Beam





Design Load



Beam Flange Force & Panel Zone Shear Vp

Members & Components Summary

Member	Moment Connection	Code=AISC 360-10 LRFD

Column Section

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

Right Side Beam Section

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

Left Side Beam Section

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

Beam Flange Force Calc

Beam Flange Force - Right Side Beam

Beam section	d_b = 12.300	[in]	t_{fb} = 0.440	[in]
Flange force moment arm	$d_m = d_b - t_{fb}$		= 11.860	[in]
User input load	axial P_{bR} = -25.00	[kips]	moment M_R = 35.00	[kip-ft]
	in tension			
Beam flange force - top	$P_{f\text{-}TR} = P_{bR} / 2 + M_R / d_m$		= **22.91**	[kips]
Beam flange force - bottom	$P_{f\text{-}BR} = P_{bR} / 2 - M_R / d_m$		= **-47.91**	[kips]

Beam Flange Force - Left Side Beam

Beam section	d_b = 12.300	[in]	t_{fb} = 0.440	[in]
Flange force moment arm	$d_m = d_b - t_{fb}$		= 11.860	[in]
User input load	axial P_{bL} = -25.00	[kips]	moment M_L = 35.00	[kip-ft]
	in tension			
Beam flange force - top	$P_{f\text{-}TL} = P_{bL} / 2 - M_L / d_m$		= **-47.91**	[kips]
Beam flange force - bottom	$P_{f\text{-}BL} = P_{bL} / 2 + M_L / d_m$		= **22.91**	[kips]

Panel Zone Shear Force Calc

Column story shear	V_s = from user input	= 0.00	[kips]
Panel zone shear force	$V_p = P_{f\text{-}TR} - P_{f\text{-}TL} - V_s$	= **70.83**	[kips]

Right Beam to Column MC Connection Code=AISC 360-10 LRFD

Result Summary geometries & weld limitations = **PASS** limit states max ratio = **0.84** **PASS**

Geometry Restriction Checks					PASS
Min Bolt Edge Distance - Column Flange					
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 Column Flange	L_e =		= **2.005**	[in]	
			> $L_{e\text{-}min}$	OK	
Min Bolt Spacing - End Plate					
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	L_s =		= **3.690**	[in]	
			> $L_{s\text{-}min}$	OK	
Min Bolt Edge Distance - End Plate					
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	L_e =		= **1.375**	[in]	
			> $L_{e\text{-}min}$	OK	
Max Bolt Edge Distance - End Plate					
Connecting plate thickness	t_p =		= 0.750	[in]	
Max edge distance allowed	$L_{e\text{-}max}$ = min (12t , 6")		= **6.000**	[in]	AISC 14th J3.5
Max edge distance in End Plate	L_e =		= **2.245**	[in]	
			< $L_{e\text{-}max}$	OK	

Beam Flange Fillet Weld Limitation					PASS
Min Fillet Weld Size					
Thinner part joined thickness	t =		= 0.440	[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 = 0.5 b_{fb} - k_{1b}		= **2.510**	[in]	
			> L_{min}	OK	

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 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	$L = 0.5\, d_b - k_b$		= **5.025**	[in]	
			> L_{min}	OK	
Min Beam Web to End Plate Fillet Weld Size					
Beam web to end-plate fillet weld in the tension-bolt region to develop the yield strength of the beam web					AISC DG4 Page 9 Item 7
Shear resistance factor-LRFD	$\phi_v = 0.90$				AISC 14[th] G1
Fillet weld shear strength	$\phi\, R_{n\text{-}w}$ =		= 1.392	[kip/in]	AISC 14[th] Eq 8-2a
Fillet weld strength $\phi\, R_{n\text{-}w} \times 1.5 \times 2$ to account for 90° load angle when it's in tension and double fillet					
Min double fillet weld size to match beam web yield strength	$D_{min} = \phi_v F_{yb}\, t_{wb} / (\phi\, R_{n\text{-}w} \times 1.5 \times 2)$		= **2.802**	[1/16 "]	
Fillet weld size provided	D =		= **5.000**	[1/16 "]	
			> D_{min}	OK	

Verify AISC DG4 Bolt No Prying Assumption — AISC DG4 Is Used

Bolt Moment Strength (No Prying)

	bolt grade = A325-N		F_t = 90.0 [ksi]	AISC 14th Table J3.2
	bolt dia d_b = 0.750 [in]		bolt area A_b = 0.442 [in^2]	
Bolt norminal tensile strength	$P_t = F_t A_b$		= 39.76 [kips]	AISC 14th Eq J3-1
Tension bolt moment arm	h_0 = 13.705 [in]		h_1 = 10.015 [in]	
Bolt moment strength (no prying)	$M_{np} = 2 P_t (h_0 + h_1)$		= 157.19 [kip-ft]	AISC DG4 Eq 3.7
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 14th Eq J3-1
	ϕM_{np} =		= **117.89** [kip-ft]	

End Plate Bending Strength

End plate width	b_{plate} = 8.490 [in]		thickness t_p = 0.750 [in]	
Beam flange width	b_{fb} = 6.520 [in]			
Effective end plate width	$b_p = \min (b_{plate} , b_{fb} + 1")$		= 7.520 [in]	AISC DG4 Page 9 item 5
End plate yield strength	F_{yp} = 50.0 [ksi]			

See AISC DG4 Table 3.1 for all formulas to derive the following parameters — AISC DG4 Table 3.1

Tension bolt moment arm	h_0 = 13.705 [in]		h_1 = 10.015 [in]	
	g = 4.000 [in]			
	p_{fi} = 1.625 [in]		p_{fo} = 1.625 [in]	
	s = 2.742 [in]		Y_p = 88.61 [in]	
Flexure resistance factor-LRFD	ϕ_b = 0.90			AISC 14th F1 (1)
End plate bending strength	$\phi_b M_{pl} = \phi_b F_{yp} t_p^2 \ Y_p$		= **186.90** [kip-ft]	AISC DG4 Table 3.1
Check thick end plate condition	$\phi_b M_{pl}$ >= 1.11 X ϕ M_{np}			AISC DG4 Eq 3.33
	ratio = **0.70** thick plate			

Column Flange Bending Strength

See AISC DG4 Table 3.4 for all formulas to derive the following parameters — AISC DG4 Table 3.4

Tension bolt moment arm	h_0 = 13.705 [in]		h_1 = 10.015 [in]	
*** Stiffened Column Flange Case ***				
Column section	b_{fc} = 8.010 [in]		t_{fc} = 0.515 [in]	
	F_{yc} = 50.0 [ksi]		bolt gage g = 4.000 [in]	
	s = 2.830 [in]		c = 3.690 [in]	
Stiffener plate thickness	t_s = 0.500 [in]			
	p_{si} = 1.595 [in]		p_{so} = 1.595 [in]	
	Y_c = 145.6 [in]			
Flexure resistance factor-LRFD	ϕ_b = 0.90			AISC 14th F1 (1)
Column flange bending strength	$\phi_b M_{cf} = \phi_b F_{yc} t_{fc}^2 \ Y_c$		= **144.82** [kip-ft]	AISC DG4 Table 3.4
Check thick column flange condition	$\phi_b M_{cf}$ >= 1.11 X ϕ M_{np}			AISC DG4 Eq 3.35
	ratio = **0.90** thick plate			

The thick end plate and column flange conditions are met. AISG DG4 is used and no bolt prying is considered — AISC DG4 Eq 3.33 & 3.35

Bolt Moment Strength (No Prying)		ratio = 47.91 / 119.28	= **0.40**	**PASS**
	bolt grade = A325-N	F_t = 90.0	[ksi]	AISC 14th Table J3.2
	bolt dia d_b = 0.750 [in]	bolt area A_b = 0.442	[in^2]	
Bolt norminal tensile strength	$P_t = F_t A_b$	= 39.76	[kips]	AISC 14th Eq J3-1
Tension bolt moment arm	h_0 = 13.705 [in]	h_1 = 10.015	[in]	
Flange force moment arm	$d_m = d_b - t_{fb}$	= 11.860	[in]	
Flange force required in tension	$P_{uf\_t} = P_u / 2 - M_u / d_m$	= **47.91**	[kips]	
Flange force resistance by bolt	$F_n = 2 P_t (h_0 + h_1) / d_m$	= 159.04	[kips]	AISC DG4 Eq 3.7
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 14th Eq J3-1
	ϕF_n =	= **119.28**	[kips]	AISC DG4 Eq 3.7
	ratio = **0.40**	> P_{uf_t}	OK	

Bolt Shear Strength		ratio = 30.00 / 35.78	= **0.84**	**PASS**
Bolt shear stress	bolt grade = A325-N	F_{nv} = 54.0	[ksi]	AISC 14th Table J3.2
	bolt dia d_b = 0.750 [in]	bolt area A_b = 0.442	[in^2]	
Number of bolt carried shear	n_s = 2.0	shear plane m = 1		
Required shear strength	V_u =	= **30.00**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 47.71	[kips]	AISC 14th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 14th Eq J3-1
	ϕR_n =	= **35.78**	[kips]	
	ratio = **0.84**	> V_u	OK	

Bolt Bearing / TearOut Strength on End Plate		ratio = 30.00 / 35.78	= **0.84**	**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 edge distance	edge L_e = 1.375 [in]				
Plate tensile strength	F_u = 65.0 [ksi]				
Plate thickness	t = 0.750 [in]				
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
	= 70.84 ≤ 109.69		= 70.84	[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	edge n_{ed} = 2				
Bolt bearing strength for all bolts	$R_n = n_{ed} R_{n\text{-}ed}$		= 47.71	[kips]	
Required shear strength	V_u =		= **30.00**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 14th J3-10
	ϕR_n =		= **35.78**	[kips]	
	ratio = **0.84**		> V_u	OK	

Bolt Bearing / TearOut Strength on Column Flange		ratio = 30.00 / 35.78	= **0.84**	**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 = 3.690 [in]			
Plate tensile strength	F_u = 65.0 [ksi]			
Plate thickness	t = 0.515 [in]			
Interior Bolt				
Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.878	[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
	= 144.49 ≤ 75.32	= 75.32	[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} = 2			
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-in}}$	= 47.71	[kips]	
Required shear strength	V_u =	= **30.00**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 14th J3-10
	ϕR_n =	= **35.78**	[kips]	
	ratio = **0.84**	> V_u	OK	

End Plate Flexural Yielding		ratio = 47.91 / 189.11	= **0.25**	**PASS**
End Plate Bending Strength				
End plate width	b_{plate} = 8.490 [in]	thickness t_p = 0.750 [in]		
Beam flange width	b_{fb} = 6.520 [in]			
Effective end plate width	$b_p = \min(b_{plate}, b_{fb} + 1")$	= 7.520 [in]		AISC DG4 Page 9 item 5
End plate yield strength	F_{yp} = 50.0 [ksi]			
See AISC DG4 Table 3.1 for all formulas to derive the following parameters				AISC DG4 Table 3.1
Tension bolt moment arm	h_0 = 13.705 [in]	h_1 = 10.015 [in]		
	g = 4.000 [in]			
	p_{fi} = 1.625 [in]	p_{fo} = 1.625 [in]		
	s = 2.742 [in]	Y_p = 88.61 [in]		
Flexure resistance factor-LRFD	ϕ_b = 0.90			AISC 14th F1 (1)
End plate bending strength	$\phi_b M_{pl} = \phi_b F_{yp} t_p^2 Y_p$	= **186.90** [kip-ft]		AISC DG4 Table 3.1
Flange force moment arm	$d_m = d_b - t_{fb}$	= 11.860 [in]		
Flange force required in tension	$P_{uf\_t} = P_u / 2 - M_u / d_m$	= **47.91** [kips]		
Flange force provided by end plate bending	$\phi R_{pl} = \phi M_{pl} / d_m$	= **189.11** [kips]		AISC DG4 Eq 3.10
	ratio = **0.25**	> P_{uf_t}	OK	

End Plate Shear Yielding		ratio = 23.96 / 169.20	= **0.14**	**PASS**
Flange force required in tension	$P_{uf\_t} = P_u / 2 - M_u / d_m$	= **23.96** [kips]		
End plate width	b_{plate} = 8.490 [in]	thickness t_p = 0.750 [in]		
Beam flange width	b_{fb} = 6.520 [in]			
Effective end plate width	$b_p = \min(b_{plate}, b_{fb} + 1")$	= 7.520 [in]		AISC DG4 Page 9 item 5
Plate Shear Yielding Check				
Plate size	width b_p = 7.520 [in]	thickness t_p = 0.750 [in]		
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$	= 5.640 [in^2]		
Shear force required	0.5 P_{uf_t} =	= **23.96** [kips]		
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$	= 169.20 [kips]		AISC 14th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 14th Eq J4-3
	ϕR_n =	= **169.20** [kips]		
	ratio = **0.14**	> 0.5 P_{uf_t}	OK	

End Plate Shear Rupture		ratio = 23.96 / 126.58	= **0.19**	**PASS**
Flange force required in tension	$P_{uf\_t} = P_u / 2 - M_u / d_m$	= **23.96**	[kips]	
End plate width	b_{plate} = 8.490 [in]	thickness t_p = 0.750	[in]	
Beam flange width	b_{fb} = 6.520 [in]			
Effective end plate width	$b_p = \min(b_{plate}, b_{fb} + 1")$	= 7.520	[in]	AISC DG4 Page 9 item 5
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 = 2			
Plate size	width b_p = 7.520 [in]	thickness t_p = 0.750	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate net area in shear	$A_{nv} = (b_p - n\, d_h)\, t_p$	= 4.328	[in²]	
Shear force required	$0.5\, P_{uf\_t}$ =	= **23.96**	[kips]	
Plate shear rupture strength	$R_n = 0.6\, F_u A_{nv}$	= 168.77	[kips]	AISC 14th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75			AISC 14th Eq J4-4
	ϕR_n =	= **126.58**	[kips]	
	ratio = **0.19**	> $0.5\, P_{uf\_t}$	OK	

Beam Flange Weld Strength		ratio = 47.91 / 120.47	= **0.40**	**PASS**
Flange force required in tension	$P_{uf\_t} = P_u / 2 - M_u / d_m$	= **47.91**	[kips]	
Fillet weld length - double fillet	$L = [b_{fb} + (b_{fb} - 2k_{1b})] / 2$ as dbl fillet	= 5.770	[in]	
Fillet Weld Strength Check				
Fillet weld leg size	w = 5/16 [in]	load angle θ = 90.0	[°]	
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	$C_2 = (1 + 0.5 \sin^{1.5}\theta)$	= 1.50		AISC 14th Page 8-9
Fillet weld shear strength	$R_{n\text{-}w} = 0.6\,(C_1 \times 70\text{ ksi})\, 0.707\, w\, n\, C_2$	= 27.838	[kip/in]	AISC 14th Eq 8-1
Base metal - beam flange	thickness t = 0.440 [in]	tensile F_u = 65.0	[ksi]	
Base metal - beam flange is in tension, tensile rupture as per AISC 14th Eq J4-2 is checked				AISC 14th J2.4
Base metal tensile rupture	$R_{n\text{-}b} = F_u\, t$	= 28.600	[kip/in]	AISC 14th Eq J4-2
Double fillet linear shear strength	$R_n = \min(R_{n\text{-}w}, R_{n\text{-}b})$	= **27.838**	[kip/in]	AISC 14th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75			AISC 14th Eq 8-1
	ϕR_n =	= **20.879**	[kip/in]	
Shear resistance required	P_{uf_t} =	= **47.91**	[kips]	
Fillet weld length - double fillet	L =	= 5.770	[in]	
Shear resistance provided	$\phi F_n = \phi R_n \times L$	= **120.47**	[kips]	
	ratio = **0.40**	> P_{uf_t}	OK	

Beam Web Weld Strength				ratio = 30.00 / 38.22	= **0.79**	**PASS**
Beam Web Effective Weld Length Calc						
Beam section	d_b = 12.300	[in]	t_{fb} = 0.440	[in]		
	k_b = 1.125	[in]				
Bolt diameter	d_{bolt} = 0.750	[in]	bolt inner pitch p_{fi} = 1.625	[in]		
Effective weld length case 1	$L_1 = 0.5\ d_b - k_b$		= 5.025	[in]	AISC DG4 Page 38	
Effective weld length case 2	$L_2 = d_b - 2t_{fb} - p_{fi} - 2\ d_{bolt}$		= 8.295	[in]	AISC DG4 Page 38	
Fillet weld length - double fillet	$L = \min(L_1, L_2)$		= **5.025**	[in]		
Fillet Weld Strength Check						
Fillet weld leg size	$w = {}^{5}/_{16}$	[in]	load angle θ = 0.0	[°]		
Electrode strength	F_{EXX} = 70.0	[ksi]	strength coeff C_1 = 1.00		AISC 14th Table 8-3	
Number of weld line	n = 2 for double fillet					
Load angle coefficient	$C_2 = (1 + 0.5 \sin^{1.5}\theta)$		= 1.00		AISC 14th Page 8-9	
Fillet weld shear strength	$R_{n\text{-}w} = 0.6\ (C_1 \times 70\ \text{ksi})\ 0.707\ w\ n\ C_2$		= 18.559	[kip/in]	AISC 14th Eq 8-1	
Base metal - 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	$R_{n\text{-}b} = 0.6\ F_u\ t$		= 10.140	[kip/in]	AISC 14th Eq J4-4	
Double fillet linear shear strength	$R_n = \min(R_{n\text{-}w}, R_{n\text{-}b})$		= **10.140**	[kip/in]	AISC 14th Eq 9-2	
Resistance factor-LRFD	ϕ = 0.75				AISC 14th Eq 8-1	
	$\phi\ R_n$ =		= **7.605**	[kip/in]		
Shear resistance required	V_u =		= **30.00**	[kips]		
Fillet weld length - double fillet	L =		= 5.025	[in]		
Shear resistance provided	$\phi\ F_n = \phi\ R_n \times L$		= **38.22**	[kips]		
	ratio = **0.79**		> V_u	OK		

Column Flexural Yielding		ratio = 47.91 / 146.53	= **0.33**	**PASS**

Column Flange Bending Strength

See AISC DG4 Table 3.4 for all formulas to derive the following parameters				AISC DG4 Table 3.4
Tension bolt moment arm	h_0 = 13.705 [in]	h_1 = 10.015 [in]		
*** Stiffened Column Flange Case ***				
Column section	b_{fc} = 8.010 [in]	t_{fc} = 0.515 [in]		
	F_{yc} = 50.0 [ksi]	bolt gage g = 4.000 [in]		
	s = 2.830 [in]	c = 3.690 [in]		
Stiffener plate thickness	t_s = 0.500 [in]			
	p_{si} = 1.595 [in]	p_{so} = 1.595 [in]		
	Y_c = 145.6 [in]			
Flexure resistance factor-LRFD	ϕ_b = 0.90			AISC 14th F1 (1)
Column flange bending strength	$\phi_b M_{cf} = \phi_b F_{yc} t_{fc}^2 \ Y_c$	= **144.82**	[kip-ft]	AISC DG4 Table 3.4
Flange force moment arm	$d_m = d_b - t_{fb}$	= 11.860	[in]	
Flange force required in tension	$P_{uf\_t} = P_u \ / 2 - M_u \ / d_m$	= **47.91**	[kips]	
Flange force provided by column flange bending	$\phi R_{cf} = \phi M_{cf} / d_m$	= **146.53**	[kips]	AISC DG4 Eq 3.21
	ratio = **0.33**	> P_{uf_t}	OK	

Column Web Yielding		ratio = 47.91 / 64.05	= **0.75**	**PASS**
Flange force moment arm	$d_m = d_b - t_{fb}$	= 11.860	[in]	
Flange force in demand	P_{uf} = max (P_{uf_t} , P_{uf_c})	= **47.91**	[kips]	AISC DG13 Eq 4.2-1
Column section	d_c = 11.900 [in]	t_{fc} = 0.515 [in]		
	t_{wc} = 0.295 [in]	k_c = 1.020 [in]		
Column yield strength	F_{yc} = 50.0 [ksi]			
Distance from to top of column to top of beam flange	d_{end} = 0.375 [in]			
Top column reduction factor	C_t = 0.5			AISC DG4 Eq 3.24
Beam flange fillet weld size	w = 0.313 [in]	beam flange t_{fb} = 0.440 [in]		
Length of bearing	$N = t_{fb} + 2\ w$	= 1.065	[in]	AISC DG4 Eq 3.24
End plate thickness	t_p = 0.750 [in]			
Column web yielding strength	$R_n = C_t (6\ k_c + N + 2\ t_p) F_{yc} t_{wc}$	= 64.05	[kips]	AISC DG4 Eq 3.24
Resistance factor-LRFD	ϕ = 1.00			AISC 14th J10.2
	ϕR_n =	= **64.05**	[kips]	
	ratio = **0.75**	> P_{uf}	OK	

Column Web Buckling		ratio = 22.91 / 33.86	= **0.68**	**PASS**
Flange force moment arm	$d_m = d_b - t_{fb}$	= 11.860 [in]		
Flange force required in compression	$P_{uf\_c} = P_u / 2 - M_u / d_m$	= **22.91** [kips]		
Column section	d_c = 11.900 [in]	t_{fc} = 0.515 [in]		
	t_{wc} = 0.295 [in]	k_c = 1.020 [in]		
	$h = d_c - 2\,k_c$	= 9.860 [in]		
Column yield strength	F_{yc} = 50.0 [ksi]	E_c = 29000 [ksi]		
Distance from top of beam flange to top of column	$d_{end\text{-}flg}$ = 0.375 [in]	beam flange t_{fb} = 0.440 [in]		
Distance from center of flange force to top of column	$d_{end\text{-}F} = d_{end\text{-}flg} + 0.5\,t_{fb}$	= 0.595 [in]		
	$d_{end\text{-}F} < d_c/2$, R_n reduced by 50%		AISC 14th J10.5	
Top column reduction factor	C_t = 0.5		AISC 14th J10.5	
Column web buckling strength	$R_n = \dfrac{C_t\, 24\, t_{wc}^3 \sqrt{E_c F_{yc}}}{h}$	= 37.62 [kips]	AISC 14th Eq J10-8	
Resistance factor-LRFD	ϕ = 0.90		AISC 14th J10.5	
	$\phi\, R_n$ =	= **33.86** [kips]		
	ratio = **0.68**	> P_{uf_c}	OK	

Column Web Crippling		ratio = 22.91 / 53.46	= **0.43**	**PASS**
Flange force moment arm	$d_m = d_b - t_{fb}$	= 11.860 [in]		
Flange force required in compression	$P_{uf\_c} = P_u / 2 - M_u / d_m$	= **22.91** [kips]		
Column section	d_c = 11.900 [in]	t_{fc} = 0.515 [in]		
	t_{wc} = 0.295 [in]	k_c = 1.020 [in]		
Column yield strength	F_{yc} = 50.0 [ksi]	E_c = 29000 [ksi]		
Beam flange fillet weld size	w = 0.313 [in]	beam flange t_{fb} = 0.440 [in]		
End plate thickness	t_p = 0.750 [in]			
Length of bearing	$l_b = t_{fb} + 2\,w + 2\,t_p$	= 2.565 [in]		
Distance from top of column to top of beam flange	$d_{end\text{-}flg}$ =	= 0.375 [in]		
Distance from top of column to center of flange force	$d_{end\text{-}F} = d_{end\text{-}flg} + 0.5\,t_{fb}$	= 0.595 [in]		
	$d_{end\text{-}F} < d_c/2$ and l_b / d_c =0.22 > 0.2, use Eq J10-5b		AISC 14th Eq J10-5b	
Column web crippling strength	$R_n = 0.4 t_{wc}^2 \left[1+\left(\dfrac{4\,l_b}{d_c}-0.2\right)\left(\dfrac{t_{wc}}{t_{fc}}\right)^{1.5}\right] \times \left(\dfrac{E_c F_{yc} t_{fc}}{t_{wc}}\right)^{0.5}$	= 71.28 [kips]	AISC 14th Eq J10-5b	
Resistance factor-LRFD	ϕ = 0.75		AISC 14th J10.3	
	$\phi\, R_n$ =	= **53.46** [kips]		
	ratio = **0.43**	> P_{uf_c}	OK	

Column Panel Zone Shear	ratio = 70.83 / 94.78	= **0.75**		**PASS**
Panel zone shear force	$V_p = P_{f\text{-}TR} - P_{f\text{-}TL} - V_s$	= **70.83**	[kips]	
Column section	d_c = 11.900 [in]	t_{wc} = 0.295	[in]	
	A_c = 11.700 [in^2]	F_{yc} = 50.0	[ksi]	
Column axial compression - user input	P_r =	= 0.00	[kips]	
	$P_c = P_y = F_{yc} A_c$	= 585.00	[kips]	AISC 14th J10.6
	$P_r \leq 0.4\ P_c$, use Eq J10-9			AISC 14th Eq J10-9
Web panel zone capacity	$R_n = 0.6\ F_{yc}\, d_c\, t_{wc}$	= 105.32	[kips]	AISC 14th Eq J10-9
Resistance factor-LRFD	ϕ = 0.90			AISC 14th J10.6
	$\phi\ R_n$ =	= **94.78**	[kips]	
	ratio = **0.75**	> V_p	OK	

Left Beam to Column	MC Connection	Code=AISC 360-10 LRFD

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

Geometry Restriction Checks — PASS

Min Bolt Edge Distance - Column Flange

Bolt diameter	d_b =	= 0.750	[in]	
Min edge distance allowed	L_{e-min} =	= **1.000**	[in]	AISC 14th Table J3.4
Min edge distance in Column Flange	L_e =	= **2.005**	[in]	
		> L_{e-min}	OK	

Min Bolt Spacing - End Plate

Bolt diameter	d_b =	= 0.750	[in]	
Min bolt spacing allowed	$L_{s-min} = 2.667\ d_b$	= **2.000**	[in]	AISC 14th J3.3
Min Bolt spacing in End Plate	L_s =	= **3.690**	[in]	
		> L_{s-min}	OK	

Min Bolt Edge Distance - End Plate

Bolt diameter	d_b =	= 0.750	[in]	
Min edge distance allowed	L_{e-min} =	= **1.000**	[in]	AISC 14th Table J3.4
Min edge distance in End Plate	L_e =	= **1.375**	[in]	
		> L_{e-min}	OK	

Max Bolt Edge Distance - End Plate

Connecting plate thickness	t_p =	= 0.750	[in]	
Max edge distance allowed	L_{e-max} = min (12t , 6")	= **6.000**	[in]	AISC 14th J3.5
Max edge distance in End Plate	L_e =	= **2.245**	[in]	
		< L_{e-max}	OK	

Beam Flange Fillet Weld Limitation — PASS

Min Fillet Weld Size

Thinner part joined thickness	t =	= 0.440	[in]	
Min fillet weld size allowed	w_{min} =	= **0.188**	[in]	AISC 14th Table J2.4
Fillet weld size provided	w =	= **0.313**	[in]	
		> w_{min}	OK	

Min Fillet Weld Length

Fillet weld size provided	w =	= 0.313	[in]	
Min fillet weld length allowed	$L_{min} = 4 \times w$	= **1.250**	[in]	AISC 14th J2.2b
Min fillet weld length	$L = 0.5\ b_{fb} - k_{1b}$	= **2.510**	[in]	
		> L_{min}	OK	

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 =		= **0.313**	[in]	
			> w_{min}	OK	
Min Fillet Weld Length					
Fillet weld size provided	w =		= 0.313	[in]	
Min fillet weld length allowed	$L_{min} = 4 \times w$		= **1.250**	[in]	AISC 14th J2.2b
Min fillet weld length	$L = 0.5\, d_b - k_b$		= **5.025**	[in]	
			> L_{min}	OK	
Min Beam Web to End Plate Fillet Weld Size					
Beam web to end-plate fillet weld in the tension-bolt region to develop the yield strength of the beam web					AISC DG4 Page 9 Item 7
Shear resistance factor-LRFD	$\phi_v = 0.90$				AISC 14th G1
Fillet weld shear strength	$\phi\, R_{n\text{-}w}$ =		= 1.392	[kip/in]	AISC 14th Eq 8-2a
Fillet weld strength $\phi\, R_{n\text{-}w} \times 1.5 \times 2$ to account for 90° load angle when it's in tension and double fillet					
Min double fillet weld size to match beam web yield strength	$D_{min} = \phi_v F_{yb}\, t_{wb} / (\phi\, R_{n\text{-}w} \times 1.5 \times 2)$		= **2.802**	[1/16 "]	
Fillet weld size provided	D =		= **5.000**	[1/16 "]	
			> D_{min}	OK	

Verify AISC DG4 Bolt No Prying Assumption					**AISC DG4 Is Used**
Bolt Moment Strength (No Prying)					
	bolt grade = A325-N		F_t = 90.0	[ksi]	AISC 14th Table J3.2
	bolt dia d_b = 0.750	[in]	bolt area A_b = 0.442	[in^2]	
Bolt norminal tensile strength	$P_t = F_t A_b$		= 39.76	[kips]	AISC 14th Eq J3-1
Tension bolt moment arm	h_1 = 10.015	[in]			
Bolt moment strength (no prying)	$M_{np} = 2\, P_t h_1$		= 66.37	[kip-ft]	AISC DG16 Table 3-2
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 14th Eq J3-1
	$\phi\, M_{np}$ =		= **49.78**	[kip-ft]	
End Plate Bending Strength					
End plate width	b_{plate} = 8.490	[in]	thickness t_p = 0.750	[in]	
Beam flange width	b_{fb} = 6.520	[in]			
Effective end plate width	$b_p = \min(\, b_{plate}\,,\, b_{fb} + 1"\,)$		= 7.520	[in]	AISC DG4 Page 9 item 5
End plate yield strength	F_{yp} = 50.0	[ksi]			
See AISC DG16 Table 3-2 for all formulas to derive the following parameters					
Tension bolt moment arm	h_1 = 10.015	[in]			
	g = 4.000	[in]			AISC DG16 Table 3-2
	p_{fi} = 1.625	[in]	p_b = 3.000	[in]	
	s = 2.742	[in]	Y_p = 58.77	[in]	
Flexure resistance factor-LRFD	ϕ_b = 0.90				AISC 14th F1 (1)
End plate bending strength	$\phi_b M_{pl} = \phi_b F_{yp} t_p^2\ Y_p$		= **123.98**	[kip-ft]	AISC DG16 Table 3-2
Check thick end plate condition	$\phi_b M_{pl}$ >= 1.11 X ϕ M_{np}				AISC DG4 Eq 3.33
	ratio = **0.45**	thick plate			

Column Flange Bending Strength

Use AISC DG16 Table 3-2 2 bolts end-plate yield-line formula to derive similar formula for column flange

formulas for calculating yield-line parameters

$$s = \frac{1}{2}\sqrt{b_{fc}\, g}$$

$$Y_c = \frac{b_{fc}}{2}\left(\frac{h_1}{p_{si}} + \frac{h_1}{s}\right) + \frac{2}{g}\left[\, h_1\,(\, p_{si} + s\,)\,\right]$$

Tension bolt moment arm	h_1 = 10.015	[in]			
*** Stiffened Column Flange Case ***					
Column section	b_{fc} = 8.010	[in]	t_{fc} = 0.515	[in]	
	F_{yc} = 50.0	[ksi]	g = 4.000	[in]	
Stiffener plate thickness	t_s = 0.500	[in]			
	p_{si} = 1.595	[in]	p_b = 3.000	[in]	
	s = 2.830	[in]	Y_c = 61.5	[in]	
Flexure resistance factor-LRFD	ϕ_b = 0.90				AISC 14th F1 (1)
Column flange bending strength	$\phi_b M_{cf} = \phi_b F_{yc} t_{fc}^2\ Y_c$		= **61.15**	[kip-ft]	

Bolt Moment Strength (No Prying)		ratio = 47.91 / 50.36	= **0.95**		**PASS**
	bolt grade = A325-N	F_t = 90.0		[ksi]	AISC 14th Table J3.2
	bolt dia d_b = 0.750 [in]	bolt area A_b = 0.442		[in^2]	
Bolt norminal tensile strength	$P_t = F_t A_b$	= 39.76		[kips]	AISC 14th Eq J3-1
Tension bolt moment arm	h_1 = 10.015 [in]				
Flange force moment arm	$d_m = d_b - t_{fb}$	= 11.860		[in]	
Flange force required in tension	$P_{uf\_t} = P_u / 2 - M_u / d_m$	= **47.91**		[kips]	
Flange force resistance by bolt	$F_n = 2 P_t h_1 / d_m$	= 67.15		[kips]	AISC DG16 Table 3-2
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 14th Eq J3-1
	ϕF_n =	= **50.36**		[kips]	AISC DG4 Eq 3.7
	ratio = **0.95**	> P_{uf_t}		OK	

Bolt Shear Strength		ratio = 30.00 / 71.57	= **0.42**		**PASS**
Bolt shear stress	bolt grade = A325-N	F_{nv} = 54.0		[ksi]	AISC 14th Table J3.2
	bolt dia d_b = 0.750 [in]	bolt area A_b = 0.442		[in^2]	
Number of bolt carried shear	n_s = 4.0	shear plane m = 1			
Required shear strength	V_u =	= **30.00**		[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 95.43		[kips]	AISC 14th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 14th Eq J3-1
	ϕR_n =	= **71.57**		[kips]	
	ratio = **0.42**	> V_u		OK	

Bolt Bearing / TearOut Strength on End Plate		ratio = 30.00 / 71.57	= **0.42**	**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 = 3.690 [in]	edge distance L_e = 1.375	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate thickness	t = 0.750 [in]			
Interior Bolt				
Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.878	[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
	= 210.42 ≤ 109.69	= 109.69	[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
	= 70.84 ≤ 109.69	= 70.84	[kips]	
Bolt strength at edge	$R_{n\text{-}ed} = \min(R_{n\text{-}t\&b\text{-}ed}, R_{n\text{-}bolt})$	= 23.86	[kips]	
Number of bolt	interior n_{in} = 2	edge n_{ed} = 2		
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-}in} + n_{ed} R_{n\text{-}ed}$	= 95.43	[kips]	
Required shear strength	V_u =	= **30.00**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 14th J3-10
	ϕR_n =	= **71.57**	[kips]	
	ratio = **0.42**	> V_u	OK	

Bolt Bearing / TearOut Strength on Column Flange		ratio = 30.00 / 71.57	= **0.42**	**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 = 3.690 [in]				
Plate tensile strength	F_u = 65.0 [ksi]				
Plate thickness	t = 0.515 [in]				
Interior Bolt					
Bolt hole edge clear distance	$L_c = L_s - d_h$		= 2.878	[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
	= 144.49 ≤ 75.32		= 75.32	[kips]	
Bolt strength at interior	$R_{n\text{-in}} = \min(R_{n\text{-t\&b-in}}, R_{n\text{-bolt}})$		= 23.86	[kips]	
Number of bolt	interior n_{in} = 4				
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-in}}$		= 95.43	[kips]	
Required shear strength	V_u =		= **30.00**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 14th J3-10
	ϕR_n =		= **71.57**	[kips]	
	ratio = **0.42**		> V_u	OK	

End Plate Flexural Yielding		ratio = 47.91 / 125.44 = **0.38**		**PASS**
End Plate Bending Strength				
End plate width	b_{plate} = 8.490 [in]	thickness t_p = 0.750 [in]		
Beam flange width	b_{fb} = 6.520 [in]			
Effective end plate width	$b_p = \min(b_{plate}, b_{fb} + 1")$	= 7.520	[in]	AISC DG4 Page 9 item 5
End plate yield strength	F_{yp} = 50.0 [ksi]			
See AISC DG16 Table 3-2 for all formulas to derive the following parameters				
Tension bolt moment arm	h_1 = 10.015 [in]			
	g = 4.000 [in]			AISC DG16 Table 3-2
	p_{fi} = 1.625 [in]	p_b = 3.000 [in]		
	s = 2.742 [in]	Y_p = 58.77 [in]		
Flexure resistance factor-LRFD	ϕ_b = 0.90			AISC 14th F1 (1)
End plate bending strength	$\phi_b M_{pl} = \phi_b F_{yp} t_p^2 Y_p$	= **123.98**	[kip-ft]	AISC DG16 Table 3-2
Flange force moment arm	$d_m = d_b - t_{fb}$	= 11.860	[in]	
Flange force required in tension	$P_{uf\_t} = P_u / 2 - M_u / d_m$	= **47.91**	[kips]	
Flange force provided by end plate bending	$\phi R_{pl} = \phi M_{pl} / d_m$	= **125.44**	[kips]	AISC DG4 Eq 3.10
	ratio = **0.38**	> P_{uf_t}	OK	

End Plate Shear Yielding		ratio = 47.91 / 169.20 = **0.28**		**PASS**
Flange force required in tension	$P_{uf\_t} = P_u / 2 - M_u / d_m$	= **47.91**	[kips]	
End plate width	b_{plate} = 8.490 [in]	thickness t_p = 0.750 [in]		
Beam flange width	b_{fb} = 6.520 [in]			
Effective end plate width	$b_p = \min(b_{plate}, b_{fb} + 1")$	= 7.520	[in]	AISC DG4 Page 9 item 5
Plate Shear Yielding Check				
Plate size	width b_p = 7.520 [in]	thickness t_p = 0.750 [in]		
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$	= 5.640	[in^2]	
Shear force required	P_{uf_t} =	= **47.91**	[kips]	
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$	= 169.20	[kips]	AISC 14th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 14th Eq J4-3
	ϕR_n =	= **169.20**	[kips]	
	ratio = **0.28**	> P_{uf_t}	OK	

End Plate Shear Rupture		ratio = 47.91 / 126.58	= **0.38**	**PASS**
Flange force required in tension	$P_{uf\_t} = P_u$ / 2 - M_u / d_m	= **47.91**	[kips]	
End plate width	b_{plate} = 8.490 [in]	thickness t_p = 0.750	[in]	
Beam flange width	b_{fb} = 6.520 [in]			
Effective end plate width	b_p = min (b_{plate} , b_{fb} + 1")	= 7.520	[in]	AISC DG4 Page 9 item 5
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 = 2			
Plate size	width b_p = 7.520 [in]	thickness t_p = 0.750	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate net area in shear	A_{nv} = (b_p - n d_h) t_p	= 4.328	[in^2]	
Shear force required	P_{uf_t} =	= **47.91**	[kips]	
Plate shear rupture strength	R_n = 0.6 F_u A_{nv}	= 168.77	[kips]	AISC 14th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75			AISC 14th Eq J4-4
	ϕ R_n =	= **126.58**	[kips]	
	ratio = **0.38**	> P_{uf_t}	OK	

Beam Flange Weld Strength		ratio = 47.91 / 120.47	= **0.40**	**PASS**
Flange force required in tension	$P_{uf\_t} = P_u$ / 2 - M_u / d_m	= **47.91**	[kips]	
Fillet weld length - double fillet	L = [b_{fb} + (b_{fb} - 2k_{1b})] /2 as dbl fillet	= 5.770	[in]	
Fillet Weld Strength Check				
Fillet weld leg size	w = 5/16 [in]	load angle θ = 90.0	[°]	
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 $\sin^{1.5}\theta$)	= 1.50		AISC 14th Page 8-9
Fillet weld shear strength	R_{n-w} = 0.6 (C_1 x 70 ksi) 0.707 w n C_2	= 27.838	[kip/in]	AISC 14th Eq 8-1
Base metal - beam flange	thickness t = 0.440 [in]	tensile F_u = 65.0	[ksi]	
Base metal - beam flange is in tension, tensile rupture as per AISC 14th Eq J4-2 is checked				AISC 14th J2.4
Base metal tensile rupture	R_{n-b} = F_u t	= 28.600	[kip/in]	AISC 14th Eq J4-2
Double fillet linear shear strength	R_n = min (R_{n-w} , R_{n-b})	= **27.838**	[kip/in]	AISC 14th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75			AISC 14th Eq 8-1
	ϕ R_n =	= **20.879**	[kip/in]	
Shear resistance required	P_{uf_t} =	= **47.91**	[kips]	
Fillet weld length - double fillet	L =	= 5.770	[in]	
Shear resistance provided	ϕ F_n = ϕ R_n x L	= **120.47**	[kips]	
	ratio = **0.40**	> P_{uf_t}	OK	

Beam Web Weld Strength			ratio = 30.00 / 38.22	= **0.79**	**PASS**

Beam Web Effective Weld Length Calc

Beam section	d_b = 12.300 [in]		t_{fb} = 0.440	[in]	
	k_b = 1.125 [in]				
Bolt diameter	d_{bolt} = 0.750 [in]		bolt inner pitch p_{fi} = 1.625	[in]	
Effective weld length case 1	$L_1 = 0.5\ d_b - k_b$		= 5.025	[in]	AISC DG4 Page 38
Effective weld length case 2	$L_2 = d_b - 2t_{fb} - p_{fi} - 2\ d_{bolt}$		= 8.295	[in]	AISC DG4 Page 38
Fillet weld length - double fillet	$L = \min(L_1, L_2)$		= **5.025**	[in]	

Fillet Weld Strength Check

Fillet weld leg size	$w = {}^{5}/_{16}$ [in]		load angle θ = 0.0	[°]	
Electrode strength	F_{EXX} = 70.0 [ksi]		strength coeff C_1 = 1.00		AISC 14th Table 8-3
Number of weld line	n = 2 for double fillet				
Load angle coefficient	$C_2 = (1 + 0.5 \sin^{1.5} \theta)$		= 1.00		AISC 14th Page 8-9
Fillet weld shear strength	$R_{n\text{-}w} = 0.6\ (C_1 \times 70\ \text{ksi})\ 0.707\ w\ n\ C_2$		= 18.559	[kip/in]	AISC 14th Eq 8-1
Base metal - beam web	thickness t = 0.260 [in]		tensile F_u = 65.0	[ksi]	
Base metal - beam web is in shear, <u>shear</u> rupture as per AISC 14th Eq J4-4 is checked					AISC 14th J2.4
Base metal shear rupture	$R_{n\text{-}b} = 0.6\ F_u\ t$		= 10.140	[kip/in]	AISC 14th Eq J4-4
Double fillet linear shear strength	$R_n = \min(R_{n\text{-}w}, R_{n\text{-}b})$		= **10.140**	[kip/in]	AISC 14th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75				AISC 14th Eq 8-1
	$\phi\ R_n$ =		= **7.605**	[kip/in]	
Shear resistance required	V_u =		= **30.00**	[kips]	
Fillet weld length - double fillet	L =		= 5.025	[in]	
Shear resistance provided	$\phi\ F_n = \phi\ R_n \times L$		= **38.22**	[kips]	
	ratio = **0.79**		> V_u	OK	

Column Flexural Yielding		ratio = 47.91 / 61.87	= **0.77**	**PASS**

Column Flange Bending Strength

Use AISC DG16 Table 3-2 2 bolts end-plate yield-line formula to derive similar formula for column flange

formulas for calculating yield-line parameters

$$s = \frac{1}{2}\sqrt{b_{fc}\, g}$$

$$Y_c = \frac{b_{fc}}{2}\left(\frac{h_1}{p_{si}} + \frac{h_1}{s}\right) + \frac{2}{g}\left[h_1\left(p_{si} + s\right)\right]$$

Tension bolt moment arm	h_1 = 10.015 [in]				
*** Stiffened Column Flange Case ***					
Column section	b_{fc} = 8.010 [in]	t_{fc} = 0.515	[in]		
	F_{yc} = 50.0 [ksi]	g = 4.000	[in]		
Stiffener plate thickness	t_s = 0.500 [in]				
	p_{si} = 1.595 [in]	p_b = 3.000	[in]		
	s = 2.830 [in]	Y_c = 61.5	[in]		
Flexure resistance factor-LRFD	ϕ_b = 0.90			AISC 14th F1 (1)	
Column flange bending strength	$\phi_b M_{cf} = \phi_b F_{yc} t_{fc}^2 \; Y_c$	= **61.15**	[kip-ft]		
Flange force moment arm	$d_m = d_b - t_{fb}$	= 11.860	[in]		
Flange force required in tension	$P_{uf\_t} = P_u \;/\; 2 - M_u \;/\; d_m$	= **47.91**	[kips]		
Flange force provided by column flange bending	$\phi\, R_{cf} = \phi\, M_{cf} / \, d_m$	= **61.87**	[kips]	AISC DG4 Eq 3.21	
	ratio = **0.77**	> P_{uf_t}	OK		

Column Web Yielding		ratio = 47.91 / 54.76	= **0.87**	**PASS**

Flange force moment arm	$d_m = d_b - t_{fb}$	= 11.860	[in]	
Flange force in demand	$P_{uf} = \max\left(P_{uf\_t}\;,\; P_{uf\_c}\right)$	= **47.91**	[kips]	AISC DG13 Eq 4.2-1
Column section	d_c = 11.900 [in]	t_{fc} = 0.515	[in]	
	t_{wc} = 0.295 [in]	k_c = 1.020	[in]	
Column yield strength	F_{yc} = 50.0 [ksi]			
Distance from to top of column to top of beam flange	d_{end} = 0.375 [in]			
Top column reduction factor	C_t = 0.5			AISC DG4 Eq 3.24
Beam flange fillet weld size	w = 0.313 [in]	beam flange t_{fb} = 0.440	[in]	
Length of bearing	$N = t_{fb} + 2\, w$	= 1.065	[in]	AISC DG4 Eq 3.24
End plate thickness	t_p = 0.750 [in]			
Column web yielding strength	$R_n = C_t\left(5.5\, k_c + N + t_p\right) F_{yc}\, t_{wc}$	= 54.76	[kips]	AISC DG4 Eq 3.24
Resistance factor-LRFD	ϕ = 1.00			AISC 14th J10.2
	$\phi\, R_n$ =	= **54.76**	[kips]	
	ratio = **0.87**	> P_{uf}	OK	

Column Web Buckling		ratio = 22.91 / 33.86	= **0.68**	**PASS**
Flange force moment arm	$d_m = d_b - t_{fb}$	= 11.860	[in]	
Flange force required in compression	$P_{uf\_c} = P_u / 2 - M_u / d_m$	= **22.91**	[kips]	
Column section	d_c = 11.900 [in]	t_{fc} = 0.515	[in]	
	t_{wc} = 0.295 [in]	k_c = 1.020	[in]	
	$h = d_c - 2\ k_c$	= 9.860	[in]	
Column yield strength	F_{yc} = 50.0 [ksi]	E_c = 29000	[ksi]	
Distance from top of beam flange to top of column	$d_{end\text{-}flg}$ = 0.375 [in]	beam flange t_{fb} = 0.440	[in]	
Distance from center of flange force to top of column	$d_{end\text{-}F} = d_{end\text{-}flg} + 0.5\ t_{fb}$	= 0.595	[in]	
	$d_{end\text{-}F} < d_c/2$, R_n reduced by 50%			AISC 14th J10.5
Top column reduction factor	$C_t = 0.5$			AISC 14th J10.5
Column web buckling strength	$R_n = \dfrac{C_t\, 24\, t_{wc}^3 \sqrt{E_c F_{yc}}}{h}$	= 37.62	[kips]	AISC 14th Eq J10-8
Resistance factor-LRFD	$\phi = 0.90$			AISC 14th J10.5
	$\phi\ R_n =$	= **33.86**	[kips]	
	ratio = **0.68**	> P_{uf_c}	OK	

Column Web Crippling		ratio = 22.91 / 49.78	= **0.46**	**PASS**
Flange force moment arm	$d_m = d_b - t_{fb}$	= 11.860	[in]	
Flange force required in compression	$P_{uf\_c} = P_u / 2 - M_u / d_m$	= **22.91**	[kips]	
Column section	d_c = 11.900 [in]	t_{fc} = 0.515	[in]	
	t_{wc} = 0.295 [in]	k_c = 1.020	[in]	
Column yield strength	F_{yc} = 50.0 [ksi]	E_c = 29000	[ksi]	
Beam flange fillet weld size	w = 0.313 [in]	beam flange t_{fb} = 0.440	[in]	
End plate thickness	t_p = 0.750 [in]			
Length of bearing	$l_b = t_{fb} + 2\ w + t_p$	= 1.815	[in]	
Distance from top of column to top of beam flange	$d_{end\text{-}flg}$ =	= 0.375	[in]	
Distance from top of column to center of flange force	$d_{end\text{-}F} = d_{end\text{-}flg} + 0.5\ t_{fb}$	= 0.595	[in]	
	$d_{end\text{-}F} < d_c/2$ and $l_b / d_c = 0.15 \le 0.2$, use Eq J10-5a			AISC 14th Eq J10-5a
Column web crippling strength	$R_n = 0.4\ t_{wc}^2\ [1+3\dfrac{l_b}{d_c}(\dfrac{t_{wc}}{t_{fc}})^{1.5}]\text{x}\ (\dfrac{E_c F_{yc} t_{fc}}{t_{wc}})^{0.5}$	= 66.37	[kips]	AISC 14th Eq J10-5a
Resistance factor-LRFD	$\phi = 0.75$			AISC 14th J10.3
	$\phi\ R_n =$	= **49.78**	[kips]	
	ratio = **0.46**	> P_{uf_c}	OK	

Column Panel Zone Shear		ratio = 70.83 / 94.78	= **0.75**	**PASS**
Panel zone shear force	$V_p = P_{f\text{-}TR} - P_{f\text{-}TL} - V_s$	= **70.83**	[kips]	
Column section	d_c = 11.900 [in]	t_{wc} = 0.295	[in]	
	A_c = 11.700 [in^2]	F_{yc} = 50.0	[ksi]	
Column axial compression - user input	P_r =	= 0.00	[kips]	
	$P_c = P_y = F_{yc} A_c$	= 585.00	[kips]	AISC 14th J10.6
	$P_r \le 0.4\ P_c$, use Eq J10-9			AISC 14th Eq J10-9
Web panel zone capacity	$R_n = 0.6\ F_{yc}\ d_c\ t_{wc}$	= 105.32	[kips]	AISC 14th Eq J10-9
Resistance factor-LRFD	ϕ = 0.90			AISC 14th J10.6
	$\phi\ R_n$ =	= **94.78**	[kips]	
	ratio = **0.75**	> V_p	OK	