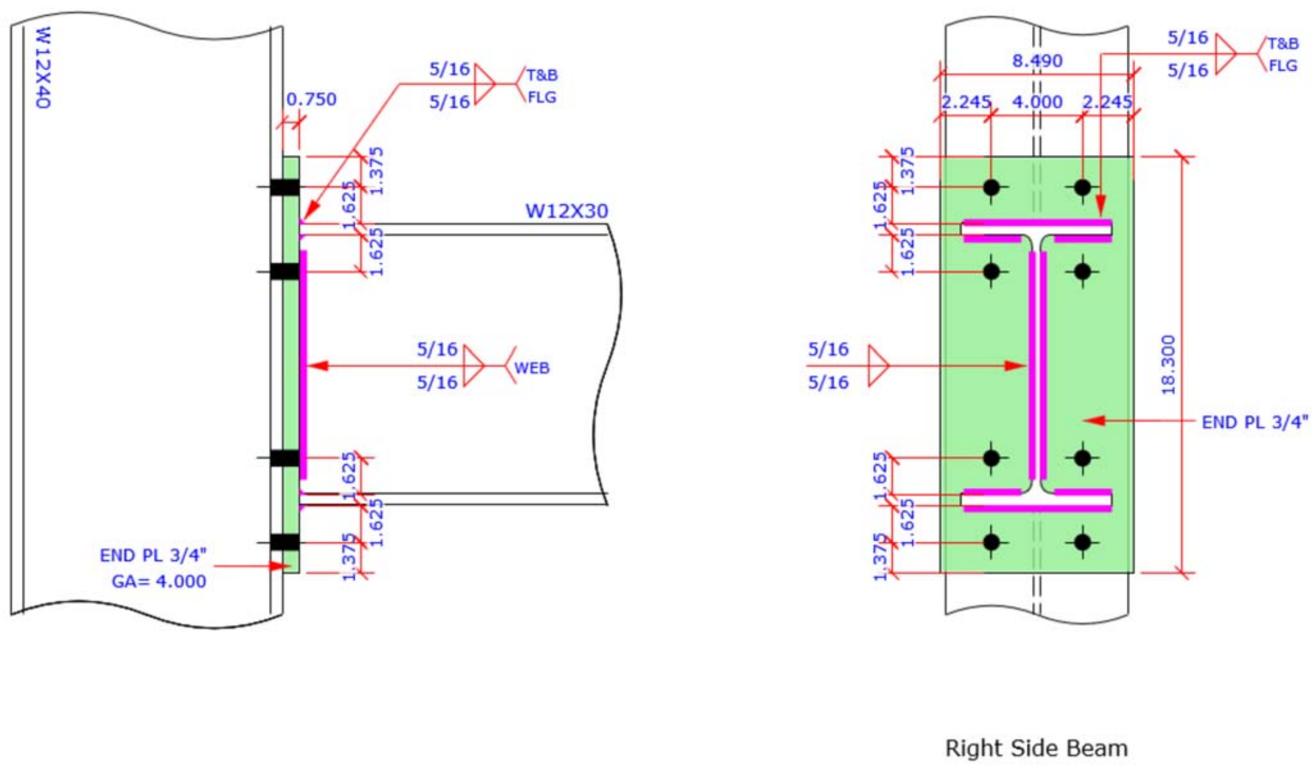
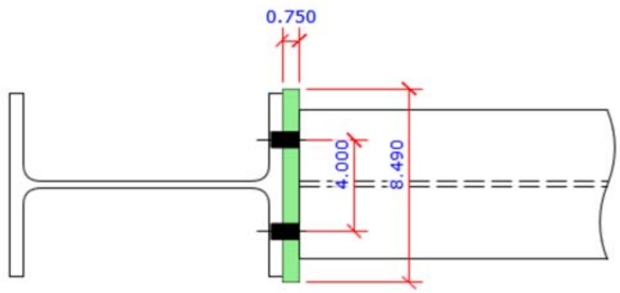
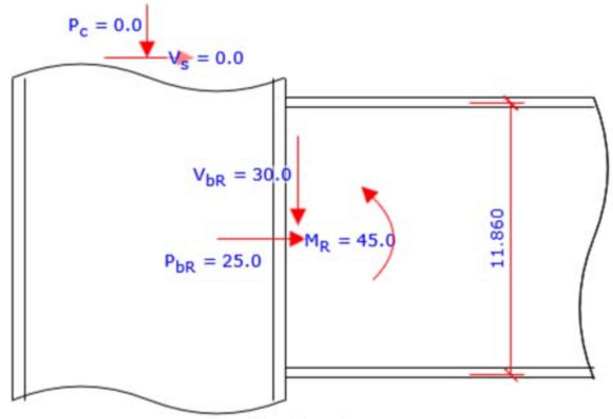
Result Summary - Overall	Moment Connection - Beam to Column	Code=AISC 360-10 LRFD

Result Summary - Overall	geometries & weld limitations = **PASS**	limit states max ratio = **0.79**	**PASS**
Right Beam to Column	geometries & weld limitations = **PASS**	limit states max ratio = **0.79**	**PASS**

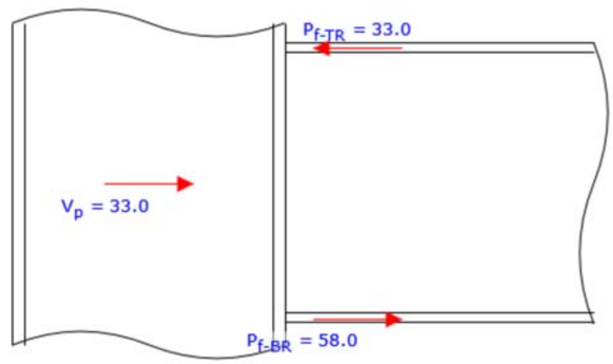
Sketch	Moment Connection - Beam to Column	Code=AISC 360-10 LRFD







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]

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 = 45.00	[kip-ft]
	in tension			
Beam flange force - top	$P_{f\text{-}TR} = P_{bR} / 2 + M_R / d_m$		= **33.03**	[kips]
Beam flange force - bottom	$P_{f\text{-}BR} = P_{bR} / 2 - M_R / d_m$		= **-58.03**	[kips]

Panel Zone Shear Force Calc

Column story shear	V_s = from user input	= 0.00	[kips]
Panel zone shear force	$V_p = P_{f\text{-}TR} - P_{f\text{-}TL} - V_s$	= **33.03**	[kips]

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

Result Summary	geometries & weld limitations = **PASS**	limit states max ratio = **0.79**	**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	$L_{min} = 4 \times w$	= **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 DG16 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
	$\phi\ M_{np}$ =		= **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	$\phi_b M_{pl} >= 1.11 \times \phi\ M_{np}$				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]	
*** Unstiffened 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]	
	p_{fi} = 1.625	[in]	p_{fo} = 1.625	[in]	
	Y_c = 92.7	[in]			
Flexure resistance factor-LRFD	ϕ_b = 0.90				AISC 14th F1 (1)
Column flange bending strength	$\phi_b M_{cf} = \phi_b F_{yc} t_{fc}^2\ Y_c$		= **92.21**	[kip-ft]	AISC DG4 Table 3.4
Check thick column flange condition	$\phi_b M_{cf} >= 1.11 \times \phi\ M_{np}$				AISC DG4 Eq 3.35
	ratio = **1.42**	thin plate			

The thick end plate and column flange conditions are NOT met. AISG DG16 is used and bolt prying is considered — AISC DG4 Eq 3.33 & 3.35

Bolt Prying Force

Bolt tensile stress	bolt grade = A325-N	F_t = 90.0	[ksi]	AISC 14th Table J3.2
	bolt dia $d_b = 3/4$ [in]	bolt hole dia $d_h = 13/16$	[in]	
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_{py} = 50.0 [ksi]			
Bolt pitch	p_{fi} = 1.625 [in]	p_{fo} = 1.625	[in]	
	$w' = b_p / 2 - d_h$	= 2.948	[in]	AISC DG16 Eq 2-12
Inside bolt prying force	$Q_{m\_i} = \frac{w' t_p^2}{4 a_i} \left[F_{py}^2 - 3 \left(\frac{F'_i}{w' t_p} \right)^2 \right]^{0.5}$	= **5.25**	[kips]	AISC DG16 Eq 2-11
	$a_i = 3.682 (t_p / d_b)^3 - 0.085$	= 3.597	[in]	AISC DG16 Eq 2-13
	$F'_i = \frac{t_p^2 F_{py} \left(0.85 \frac{b_p}{2} + 0.8 w'\right) + \frac{\pi d_b^3 F_t}{8}}{4 p_{fi}}$	= 26.33	[kips]	AISC DG16 Eq 2-14
Outside bolt prying force	$Q_{m\_o} = \frac{w' t_p^2}{4 a_o} \left[F_{py}^2 - 3 \left(\frac{F'_o}{w' t_p} \right)^2 \right]^{0.5}$	= **13.73**	[kips]	AISC DG16 Eq 2-15
Outside bolt ver edge dist	e_v =	= 1.375	[in]	
	$a_o = 3.682 (t_p / d_b)^3 - 0.085 \le e_v$	= 1.375	[in]	AISC DG16 Eq 2-16
	$F'_o = \frac{t_p^2 F_{py} \left(0.85 \frac{b_p}{2} + 0.8 w'\right) + \frac{\pi d_b^3 F_t}{8}}{4 p_{fo}}$	= 26.33	[kips]	AISC DG16 Eq 2-17

Bolt Moment Strength (With Prying)		ratio = 58.03 / 92.25	= **0.63**	**PASS**
	bolt grade = A325-N	F_t = 90.0	[ksi]	AISC 14[th] 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 14[th] 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	$P_{uf\_t} = P_u / 2 - M_u / d_m$	= **58.03**	[kips]	
Min bolt pretension	$T_b = 0.7 P_t$	= 28.00	[kips]	AISC 14[th] Table J3.1
Bolt prying force	Q_{m_i} = 5.25 [kips]	Q_{m_o} = 13.73	[kips]	
Bolt moment - with prying action	$M_{n1} = 2(P_t - Q_{m\_o})h_0 + 2(P_t - Q_{m\_i})h_1$	= 117.06	[kip-ft]	AISC DG16 Table 4-2
	$M_{n2} = 2(P_t - Q_{m\_o})h_0 + 2T_b h_1$	= 106.20	[kip-ft]	
	$M_{n3} = 2(P_t - Q_{m\_i}) h_1 + 2T_b h_0$	= 121.56	[kip-ft]	
	$M_{n4} = 2T_b (h_0 + h_1)$	= 110.69	[kip-ft]	
	$M_n = \max(M_{n1}, M_{n2}, M_{n3}, M_{n4})$	= 121.56	[kip-ft]	
Flange force resistance by bolt	$F_n = M_n / d_m$	= 123.00	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 14[th] Eq J3-1
	ϕF_n =	= **92.25**	[kips]	
	ratio = **0.63**	> 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 14[th] Table J3.2
	bolt dia d_b = 0.750 [in]	bolt area A_b = 0.442	[in^2]	
Number of bolt carried shear	n_s = 4.0	shear plane m = 1		
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 14[th] Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 14[th] 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	$R_{n\text{-t\&b-in}} = 1.5\, L_c\, t\, F_u \le 3.0\, d_b\, t\, F_u$			AISC 14th Eq J3-6b
	= 210.42 ≤ 109.69	= 109.69	[kips]	
Bolt strength at interior	$R_{n\text{-in}} = \min(R_{n\text{-t\&b-in}}, R_{n\text{-bolt}})$	= 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
	= 70.84 ≤ 109.69	= 70.84	[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} = 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	$R_{n\text{-}t\&b\text{-}in} = 1.5\, L_c\, t\, F_u \le 3.0\, d_b\, t\, m\, F_u$			AISC 14th Eq J3-6b
	= 144.49 ≤ 75.32	= 75.32	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$	= 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 = 58.03 / 189.11	= **0.31**	**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	$P_{uf\_t} = P_u / 2 - M_u / d_m$	= **58.03**	[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 = **0.31**	> P_{uf_t}	OK	

End Plate Shear Yielding		ratio = 29.02 / 169.20	= **0.17**	**PASS**
Flange force required in tension	$P_{uf\_t} = P_u / 2 - M_u / d_m$	= **29.02**	[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} =	= **29.02**	[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 = **0.17**	> 0.5 P_{uf_t}	OK	

End Plate Shear Rupture			ratio = 29.02 / 126.58	= **0.23**	**PASS**
Flange force required in tension	$P_{uf\_t} = P_u / 2 - M_u / d_m$		= **29.02**	[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	$0.5\, P_{uf\_t}$ =		= **29.02**	[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 = **0.23**		> $0.5\, P_{uf\_t}$	OK	

Beam Flange Weld Strength			ratio = 58.03 / 120.47	= **0.48**	**PASS**
Flange force required in tension	$P_{uf\_t} = P_u / 2 - M_u / d_m$		= **58.03**	[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} =		= **58.03**	[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 = **0.48**		> 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	$R_{n-w} = 0.6\ (C_1 \times 70\ \text{ksi})\ 0.707\ w\ n\ C_2$	= 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	$R_{n-b} = 0.6\ F_u\ t$	= 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	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 = 58.03 / 93.30	= **0.62**	**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]		
*** Unstiffened 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]		
	p_{fi} = 1.625 [in]	p_{fo} = 1.625	[in]		
	Y_c = 92.7 [in]				
Flexure resistance factor-LRFD	ϕ_b = 0.90				AISC 14th F1 (1)
Column flange bending strength	$\phi_b M_{cf} = \phi_b F_{yc} t_{fc}^2 \ Y_c$	= **92.21**	[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	$P_{uf\_t} = P_u \ / \ 2 - M_u \ / \ d_m$	= **58.03**	[kips]		
Flange force provided by column flange bending	$\phi R_{cf} = \phi M_{cf} / d_m$	= **93.30**	[kips]		AISC DG4 Eq 3.21
	ratio = **0.62**	> P_{uf_t}	OK		

Column Web Yielding		ratio = 58.03 / 128.10	= **0.45**	**PASS**	
Flange force moment arm	$d_m = d_b - t_{fb}$	= 11.860	[in]		
Flange force in demand	$P_{uf} = \max \ (P_{uf\_t} \ , P_{uf\_c} \ )$	= **58.03**	[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]				
Top column condition	it's not a top column case	C_t = 1.0			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	$R_n = C_t (6 \ k_c + N + 2 \ t_p) \ F_{yc} t_{wc}$	= 128.10	[kips]		AISC DG4 Eq 3.24
Resistance factor-LRFD	ϕ = 1.00				AISC 14th J10.2
	ϕR_n =	= **128.10**	[kips]		
	ratio = **0.45**	> P_{uf}	OK		

Column Web Buckling		ratio = 33.03 / 67.72	= **0.49**	**PASS**
Flange force moment arm	$d_m = d_b - t_{fb}$	= 11.860	[in]	
Flange force required in compression	$P_{uf\_c} = P_u / 2 - M_u / d_m$	= **33.03**	[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]	
Top column condition	it's not a top column case	C_t = 1.0		AISC 14th J10.5
Column web buckling strength	$R_n = \dfrac{C_t\, 24\, t_{wc}^3 \sqrt{E_c F_{yc}}}{h}$	= 75.25	[kips]	AISC 14th Eq J10-8
Resistance factor-LRFD	ϕ = 0.90			AISC 14th J10.5
	ϕR_n =	= **67.72**	[kips]	
	ratio = **0.49**	> P_{uf_c}	OK	

Column Web Crippling		ratio = 33.03 / 106.36	= **0.31**	**PASS**
Flange force moment arm	$d_m = d_b - t_{fb}$	= 11.860	[in]	
Flange force required in compression	$P_{uf\_c} = P_u / 2 - M_u / d_m$	= **33.03**	[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	$d_{end\text{-}flg}$ =	= 4.000	[in]	
Top column condition	it's not a top column case, use Eq J10-4			AISC 14th J10.3 (a)
Column web crippling strength	$R_n = 0.8\, t_{wc}^2 \left[1+3\dfrac{l_b}{d_c}\left(\dfrac{t_{wc}}{t_{fc}}\right)^{1.5}\right] \times \left(\dfrac{E_c F_{yc} t_{fc}}{t_{wc}}\right)^{0.5}$	= 141.82	[kips]	AISC 14th Eq J10-4
Resistance factor-LRFD	ϕ = 0.75			AISC 14th J10.3
	ϕR_n =	= **106.36**	[kips]	
	ratio = **0.31**	> P_{uf_c}	OK	

Column Panel Zone Shear		ratio = 33.03 / 94.78	= **0.35**	**PASS**
Panel zone shear force	$V_p = P_{f\text{-}TR} - P_{f\text{-}TL} - V_s$	= **33.03**	[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 = **0.35**	> V_p	OK	