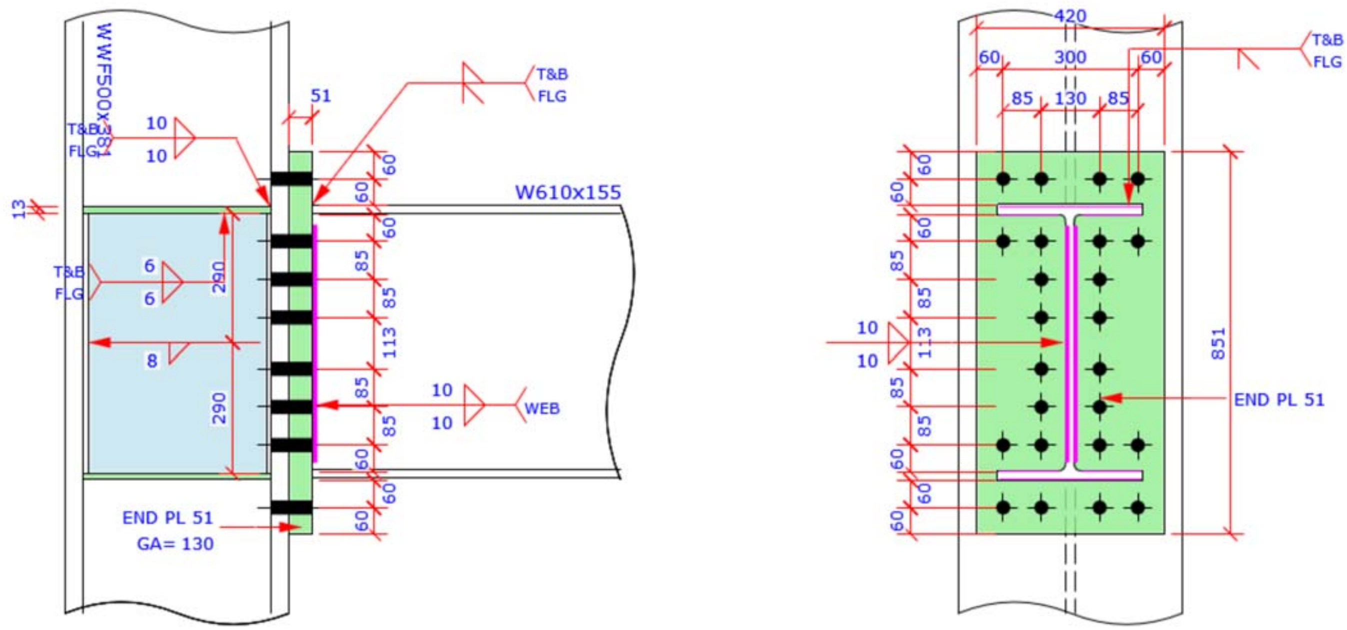
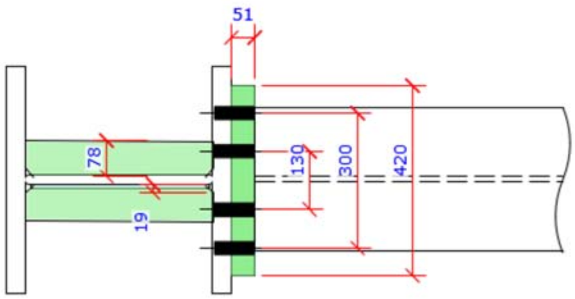
Result Summary - Overall	Moment Connection - Beam to Column	Code=AISC 360-10 LRFD

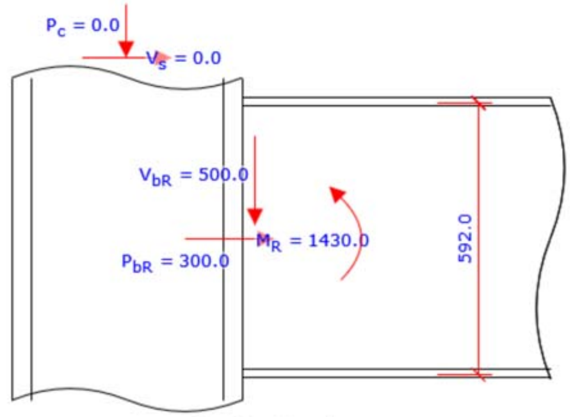
Result Summary - Overall	geometries & weld limitations = **PASS**	limit states max ratio = **0.83**	**PASS**
Right Beam to Column	geometries & weld limitations = **PASS**	limit states max ratio = **0.83**	**PASS**

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

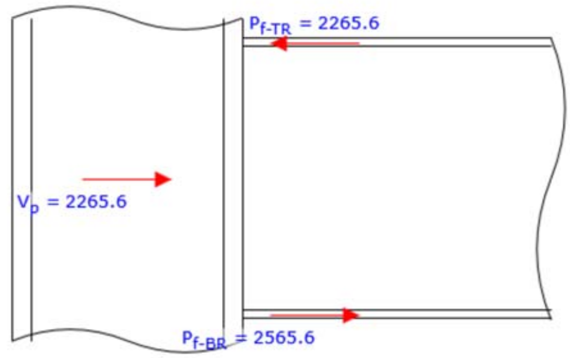


Right Side Beam





Design Load



Beam Flange Force & Panel Zone Shear Vp

Members & Components Summary

Member	Moment Connection	Code=AISC 360-10 LRFD

Column Section

WWF500x381	d =	500.0	[mm]	b_f =	500.0	[mm]
	t_f =	40.0	[mm]	t_w =	20.0	[mm]
	k_{des} =	51.0	[mm]	k_{det} =	51.0	[mm]
	k_1 =	0.0	[mm]	A =	48600	[mm^2]
	S_x =	9010.0	[10^3mm^3]	Z_x =	10100.0	[10^3mm^3]
Steel Grade A992	F_y =	344.8	[MPa]	F_u =	448.2	[MPa]

Right Side Beam Section

W610x155	d =	611.0	[mm]	b_f =	324.0	[mm]
	t_f =	19.0	[mm]	t_w =	12.7	[mm]
	k_{des} =	41.0	[mm]	k_{det} =	41.0	[mm]
	k_1 =	27.0	[mm]	A =	19700	[mm^2]
	S_x =	4220.0	[10^3mm^3]	Z_x =	4730.0	[10^3mm^3]
Steel Grade A992	F_y =	344.8	[MPa]	F_u =	448.2	[MPa]

Beam Flange Force Calc

Beam Flange Force - Right Side Beam

Beam section	d_b = 24.055 [in]	t_{fb} = 0.748	[in]
Flange force moment arm	$d_m = d_b - t_{fb}$	= 23.307	[in]
User input load	axial P_{bR} = -67.44 [kips] in tension	moment M_R = 1054.73	[kip-ft]
Beam flange force - top	$P_{f\text{-}TR} = P_{bR} / 2 + M_R / d_m$	= **509.32**	[kips]
Beam flange force - bottom	$P_{f\text{-}BR} = P_{bR} / 2 - M_R / d_m$	= **-576.77**	[kips]

Panel Zone Shear Force Calc

Column story shear	V_s = from user input	= 0.00	[kips]
Panel zone shear force	$V_p = P_{f\text{-}TR} - P_{f\text{-}TL} - V_s$	= **509.32**	[kips]

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

Result Summary	geometries & weld limitations = **PASS**	limit states max ratio = **0.83**	**PASS**

Geometry Restriction Checks — PASS

Min Bolt Edge Distance - Column Flange

Bolt diameter	d_b =	= 1.250	[in]	
Min edge distance allowed	$L_{e\text{-}min}$ =	= **1.625**	[in]	AISC 14th Table J3.4
Min edge distance in Column Flange	L_e =	= **3.937**	[in]	
		> $L_{e\text{-}min}$	OK	

Min Bolt Spacing - End Plate

Bolt diameter	d_b =	= 1.250	[in]	
Min bolt spacing allowed	$L_{s\text{-}min}$ = 2.667 d_b	= **3.333**	[in]	AISC 14th J3.3
Min Bolt spacing in End Plate	L_s =	= **3.346**	[in]	
		> $L_{s\text{-}min}$	OK	

Min Bolt Edge Distance - End Plate

Bolt diameter	d_b =	= 1.250	[in]	
Min edge distance allowed	$L_{e\text{-}min}$ =	= **1.625**	[in]	AISC 14th Table J3.4
Min edge distance in End Plate	L_e =	= **2.362**	[in]	
		> $L_{e\text{-}min}$	OK	

Max Bolt Edge Distance - End Plate

Connecting plate thickness	t_p =	= 2.000	[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 =	= **2.362**	[in]	
		< $L_{e\text{-}max}$	OK	

Beam Web Fillet Weld Limitation — PASS

Min Fillet Weld Size

Thinner part joined thickness	t =	= 0.500	[in]	
Min fillet weld size allowed	w_{min} =	= **0.188**	[in]	AISC 14th Table J2.4
Fillet weld size provided	w =	= **0.394**	[in]	
		> w_{min}	OK	

Min Fillet Weld Length

Fillet weld size provided	w =	= 0.394	[in]	
Min fillet weld length allowed	L_{min} = 4 x w	= **1.575**	[in]	AISC 14th J2.2b
Min fillet weld length	L = 0.5 d_b - k_b	= **10.414**	[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	ϕ_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 $\phi R_{n\text{-}w}$ x 1.5 x 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	$D_{min} = \phi_v F_{yb} t_{wb} / (\phi R_{n\text{-}w} \times 1.5 \times 2)$	= **5.388**	[1/16 "]	
Fillet weld size provided	D =	= **6.299**	[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 = 1.250 [in]	bolt area A_b = 1.227	[in^2]	
Bolt norminal tensile strength	$P_t = F_t A_b$	= 110.45	[kips]	AISC 14th Eq J3-1
Tension bolt moment arm	h_0 = 26.043 [in]	h_1 = 20.571	[in]	
	h_2 = 17.225 [in]	h_3 = 13.879	[in]	

The following bolt moment without prying action for 12 bolts MRE 1/3-4W/2W bolt pattern is taken from Table 3-12 in Virginia Tech Report to MBMA, Developing and Validating New Bolted End-Plate Moment Connection Configurations July 2015 by Nonish Jain, Matthew Eatherton and Thomas Murray. Thomas Murray is the author of both AISC DG4 and DG16.

Bolt moment strength (no prying)	$M_{np} = 2\, P_t (\, 2\, h_0 + 2\, h_1 + h_2 + h_3\,)$	= 2288.68	[kip-ft]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 14th Eq J3-1
	$\phi\, M_{np}$ =	= **1716.51**	[kip-ft]	

End Plate Bending Strength

End plate width	b_{plate} = 16.535 [in]	thickness t_p = 2.000	[in]	
Beam flange width	b_{fb} = 12.756 [in]			
Effective end plate width	$b_p = \min(\, b_{plate}\,,\, b_{fb} + 1"\,)$	= 13.756	[in]	AISC DG4 Page 9 item 5
End plate yield strength	F_{yp} = 50.0 [ksi]			

The following end plate yield line for 12 bolts MRE 1/3-4W/2W bolt pattern is taken from Table 3-12 in Virginia Tech Report to MBMA, Developing and Validating New Bolted End-Plate Moment Connection Configurations July 2015 by Nonish Jain, Matthew Eatherton and Thomas Murray. Thomas Murray is the author of both AISC DG4 and DG16.

formulas for calculating yield-line parameters

$$s = \frac{1}{2}\sqrt{b_p\, g} \qquad \text{use } p_{fi} = s \text{ when } p_{fi} > s$$

$$Y_p = \frac{b_p}{2}\left[\frac{h_0}{p_{fo}} + \frac{h_1}{p_{fi}} + \frac{h_3}{s} - 1\right] + \frac{2}{g}\left[h_1\left(p_{fi} + 1.5\, p_b\right) + h_3\left(s + 0.5\, p_b\right)\right] + \frac{g}{2}$$

Tension bolt moment arm	h_0 = 26.043 [in]	h_1 = 20.571	[in]	
	h_2 = 17.225 [in]	h_3 = 13.879	[in]	
	g = 5.118 [in]	p_b = 3.346	[in]	
	p_{fi} = 2.362 [in]	p_{fo} = 2.362	[in]	
	s = 4.195 [in]	Y_p = 245.33	[in]	
Flexure resistance factor-LRFD	ϕ_b = 0.90			AISC 14th F1 (1)
End plate bending strength	$\phi_b M_{pl} = \phi_b F_{yp} t_p^2\ Y_p$	= **3680.00**	[kip-ft]	
Check thick end plate condition	$\phi_b M_{pl}$ >= 1.11 X ϕ M_{np}			AISC DG4 Eq 3.33
	ratio = **0.52** thick plate			

Column Flange Bending Strength

The column flange yield line for MRE 1/3 bolt pattern is taken from Table 8.13 of Virginia Tech Emmet Sumner's PhD thesis - Unified Design of Extended End-Plate Moment Connections Subject to Cyclic Loading June, 2003. This thesis was approved by Thomas M. Murray who is the author of AISC DG4 and DG16.

formulas for calculating yield-line parameters

Bolt Moment Strength (No Prying)		ratio = 576.77 / 883.77 = **0.65**		**PASS**
	bolt grade = A325-N	F_t = 90.0	[ksi]	AISC 14th Table J3.2
	bolt dia d_b = 1.250 [in]	bolt area A_b = 1.227	[in^2]	
Bolt norminal tensile strength	$P_t = F_t A_b$	= 110.45	[kips]	AISC 14th Eq J3-1
Tension bolt moment arm	h_0 = 26.043 [in]	h_1 = 20.571	[in]	
	h_2 = 17.225 [in]	h_3 = 13.879	[in]	
Flange force moment arm	$d_m = d_b - t_{fb}$	= 23.307	[in]	
Flange force required in tension	$P_{uf\_t} = P_u / 2 - M_u / d_m$	= **576.77**	[kips]	
Flange force resistance by bolt	$F_n = 2 P_t (2h_0 + 2h_1 + h_2 + h_3) / d_m$	= 1178.36	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 14th Eq J3-1
	ϕF_n =	= **883.77**	[kips]	AISC DG4 Eq 3.7
	ratio = **0.65**	> P_{uf_t}	OK	

Bolt Shear Strength		ratio = 112.41 / 596.41 = **0.19**		**PASS**
Bolt shear stress	bolt grade = A325-N	F_{nv} = 54.0	[ksi]	AISC 14th Table J3.2
	bolt dia d_b = 1.250 [in]	bolt area A_b = 1.227	[in^2]	
Number of bolt carried shear	n_s = 12.0	shear plane m = 1		
Required shear strength	V_u =	= **112.41**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 795.22	[kips]	AISC 14th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 14th Eq J3-1
	ϕR_n =	= **596.41**	[kips]	
	ratio = **0.19**	> V_u	OK	

Bolt Bearing / TearOut Strength on End Plate			ratio = 112.41 / 596.41	= **0.19**	**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 = 1.250	[in]	bolt area A_b = 1.227	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$		= 66.27	[kips]	AISC 14th Eq J3-1
Bolt Bearing / TearOut Strength on Plate					
Bolt hole diameter	bolt dia $d_b = 1\,^1/_4$	[in]	bolt hole dia $d_h = 1\,^5/_{16}$	[in]	AISC 14th Table J3.3
Bolt spacing & edge distance	spacing L_s = 3.346	[in]	edge distance L_e = 2.362	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate thickness	t = 2.000	[in]			
Interior Bolt					
Bolt hole edge clear distance	$L_c = L_s - d_h$		= 2.034	[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
	= 396.53 ≤ 487.50		= 396.53	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$		= 66.27	[kips]	
Edge Bolt					
Bolt hole edge clear distance	$L_c = L_e - d_h / 2$		= 1.706	[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
	= 332.62 ≤ 487.50		= 332.62	[kips]	
Bolt strength at edge	$R_{n\text{-}ed} = \min(R_{n\text{-}t\&b\text{-}ed}, R_{n\text{-}bolt})$		= 66.27	[kips]	
Number of bolt	interior n_{in} = 8		edge n_{ed} = 4		
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-}in} + n_{ed} R_{n\text{-}ed}$		= 795.22	[kips]	
Required shear strength	V_u =		= **112.41**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 14th J3-10
	ϕR_n =		= **596.41**	[kips]	
	ratio = **0.19**		> V_u	OK	

Bolt Bearing / TearOut Strength on Column Flange		ratio = 112.41 / 596.41	= **0.19**	**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 = 1.250 [in]	bolt area A_b = 1.227	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$	= 66.27	[kips]	AISC 14th Eq J3-1
Bolt Bearing / TearOut Strength on Plate				
Bolt hole diameter	bolt dia $d_b = 1\,^1/_4$ [in]	bolt hole dia $d_h = 1\,^5/_{16}$	[in]	AISC 14th Table J3.3
Bolt spacing	spacing L_s = 3.346 [in]			
Plate tensile strength	F_u = 65.0 [ksi]			
Plate thickness	t = 1.575 [in]			
Interior Bolt				
Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.034	[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
	= 312.27 ≤ 383.91	= 312.27	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$	= 66.27	[kips]	
Number of bolt	interior n_{in} = 12			
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-}in}$	= 795.22	[kips]	
Required shear strength	V_u =	= **112.41**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 14th J3-10
	ϕR_n =	= **596.41**	[kips]	
	ratio = **0.19**	> V_u	OK	

End Plate Flexural Yielding ratio = 576.77 / 1894.71 = **0.30** **PASS**

End Plate Bending Strength

End plate width	b_{plate} = 16.535 [in]	thickness t_p = 2.000 [in]		
Beam flange width	b_{fb} = 12.756 [in]			
Effective end plate width	$b_p = \min(b_{plate}, b_{fb} + 1")$	= 13.756 [in]	AISC DG4 Page 9 item 5	
End plate yield strength	F_{yp} = 50.0 [ksi]			

The following end plate yield line for 12 bolts MRE 1/3-4W/2W bolt pattern is taken from Table 3-12 in Virginia Tech Report to MBMA, Developing and Validating New Bolted End-Plate Moment Connection Configurations July 2015 by Nonish Jain, Matthew Eatherton and Thomas Murray. Thomas Murray is the author of both AISC DG4 and DG16.

formulas for calculating yield-line parameters

$$s = \frac{1}{2}\sqrt{b_p\, g} \qquad \text{use } p_{fi} = s \text{ when } p_{fi} > s$$

$$Y_p = \frac{b_p}{2}\left[\frac{h_0}{p_{fo}} + \frac{h_1}{p_{fi}} + \frac{h_3}{s} - 1\right] + \frac{2}{g}\left[h_1\left(p_{fi} + 1.5\, p_b\right) + h_3\left(s + 0.5\, p_b\right)\right] + \frac{g}{2}$$

Tension bolt moment arm	h_0 = 26.043 [in]	h_1 = 20.571 [in]	
	h_2 = 17.225 [in]	h_3 = 13.879 [in]	
	g = 5.118 [in]	p_b = 3.346 [in]	
	p_{fi} = 2.362 [in]	p_{fo} = 2.362 [in]	
	s = 4.195 [in]	Y_p = 245.33 [in]	
Flexure resistance factor-LRFD	ϕ_b = 0.90		AISC 14th F1 (1)
End plate bending strength	$\phi_b M_{pl} = \phi_b F_{yp} t_p^2\, Y_p$	= **3680.00** [kip-ft]	
Flange force moment arm	$d_m = d_b - t_{fb}$	= 23.307 [in]	
Flange force required in tension	$P_{uf\_t} = P_u / 2 - M_u / d_m$	= **576.77** [kips]	
Flange force provided by end plate bending	$\phi R_{pl} = \phi M_{pl} / d_m$	= **1894.71** [kips]	AISC DG4 Eq 3.10
	ratio = **0.30**	> P_{uf_t} OK	

End Plate Shear Yielding ratio = 288.39 / 825.36 = **0.35** **PASS**

Flange force required in tension	$P_{uf\_t} = P_u / 2 - M_u / d_m$	= **288.39** [kips]	
End plate width	b_{plate} = 16.535 [in]	thickness t_p = 2.000 [in]	
Beam flange width	b_{fb} = 12.756 [in]		
Effective end plate width	$b_p = \min(b_{plate}, b_{fb} + 1")$	= 13.756 [in]	AISC DG4 Page 9 item 5

Plate Shear Yielding Check

Plate size	width b_p = 13.756 [in]	thickness t_p = 2.000 [in]	
Plate yield strength	F_y = 50.0 [ksi]		
Plate gross area in shear	$A_{gv} = b_p t_p$	= 27.512 [in^2]	
Shear force required	0.5 P_{uf_t} =	= **288.39** [kips]	
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$	= 825.36 [kips]	AISC 14th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00		AISC 14th Eq J4-3
	ϕR_n =	= **825.36** [kips]	
	ratio = **0.35**	> 0.5 P_{uf_t} OK	

End Plate Shear Rupture		ratio = 288.39 / 643.85 = **0.45**		**PASS**
Flange force required in tension	$P_{uf\_t} = P_u / 2 - M_u / d_m$	= **288.39**	[kips]	
End plate width	b_{plate} = 16.535 [in]	thickness t_p = 2.000	[in]	
Beam flange width	b_{fb} = 12.756 [in]			
Effective end plate width	$b_p = \min(b_{plate}, b_{fb} + 1")$	= 13.756	[in]	AISC DG4 Page 9 item 5
Plate Shear Rupture Check				
Bolt hole diameter	bolt dia $d_b = 1^1/_4$ [in]	bolt hole dia $d_h = 1^3/_8$	[in]	AISC 14th B4.3b
Number of bolt	n = 2			
Plate size	width b_p = 13.756 [in]	thickness t_p = 2.000	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate net area in shear	$A_{nv} = (b_p - n\, d_h)\, t_p$	= 22.012	[in^2]	
Shear force required	$0.5\, P_{uf\_t}$ =	= **288.39**	[kips]	
Plate shear rupture strength	$R_n = 0.6\, F_u A_{nv}$	= 858.47	[kips]	AISC 14th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75			AISC 14th Eq J4-4
	ϕR_n =	= **643.85**	[kips]	
	ratio = **0.45**	> $0.5\, P_{uf\_t}$	OK	

Beam Web Weld Strength		ratio = 112.41 / 152.30 = **0.74**	**PASS**

Beam Web Effective Weld Length Calc

Beam section	d_b = 24.055 [in]	t_{fb} = 0.748	[in]	
	k_b = 1.614 [in]			
Bolt diameter	d_{bolt} = 1.250 [in]	bolt inner pitch p_{fi} = 2.362	[in]	
Effective weld length case 1	$L_1 = 0.5\ d_b - k_b$	= 10.414	[in]	AISC DG4 Page 38
Effective weld length case 2	$L_2 = d_b - 2t_{fb} - p_{fi} - 2\ d_{bolt}$	= 17.697	[in]	AISC DG4 Page 38
Fillet weld length - double fillet	$L = \min(L_1, L_2)$	= **10.414**	[in]	

Fillet Weld Strength Check

Fillet weld leg size	$w = {}^3/_8$ [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	$R_{n\text{-}w} = 0.6\ (C_1 \times 70\ \text{ksi})\ 0.707\ w\ n\ C_2$	= 23.381	[kip/in]	AISC 14th Eq 8-1
Base metal - beam web	thickness t = 0.500 [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	$R_{n\text{-}b} = 0.6\ F_u\ t$	= 19.500	[kip/in]	AISC 14th Eq J4-4
Double fillet linear shear strength	$R_n = \min(R_{n\text{-}w}, R_{n\text{-}b})$	= **19.500**	[kip/in]	AISC 14th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75			AISC 14th Eq 8-1
	$\phi\ R_n$ =	= **14.625**	[kip/in]	
Shear resistance required	V_u =	= **112.41**	[kips]	
Fillet weld length - double fillet	L =	= 10.414	[in]	
Shear resistance provided	$\phi\ F_n = \phi\ R_n \times L$	= **152.30**	[kips]	
	ratio = **0.74**	> V_u	OK	

Column Flexural Yielding	ratio = 576.77 / 2099.72 = **0.27**	**PASS**

Column Flange Bending Strength

The column flange yield line for MRE 1/3 bolt pattern is taken from Table 8.13 of Virginia Tech Emmet Sumner's PhD thesis - Unified Design of Extended End-Plate Moment Connections Subject to Cyclic Loading June, 2003. This thesis was approved by Thomas M. Murray who is the author of AISC DG4 and DG16.

formulas for calculating yield-line parameters

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

$$Y_c = \frac{b_{fc}}{2}\left[\frac{h_1}{p_{si}} + \frac{h_3}{s} + h_0\left(\frac{1}{p_{so}} + \frac{1}{s}\right)\right] + \frac{2}{g}\left[h_1\left(p_{si} + \frac{3}{2}p_b\right) + h_3\left(s + \frac{p_b}{2}\right) + h_0\left(p_{so} + s\right)\right] + \frac{g}{2}$$

Tension bolt moment arm	h_0 = 26.043	[in]	h_1 = 20.571	[in]	
	h_2 = 17.225	[in]	h_3 = 13.879	[in]	

*** Stiffened Column Flange Case ***

Column section	b_{fc} = 19.685	[in]	t_{fc} = 1.575	[in]	
	F_{yc} = 50.0	[ksi]	g = 5.118	[in]	
	s = 5.019	[in]	c = 5.472	[in]	
Stiffener plate thickness	t_s = 0.500	[in]			
	p_{si} = 2.486	[in]	p_{so} = 2.486	[in]	
	p_b = 3.346	[in]	Y_c = 438.4	[in]	
Flexure resistance factor-LRFD	ϕ_b = 0.90				AISC 14th F1 (1)
Column flange bending strength	$\phi_b M_{cf} = \phi_b F_{yc} t_{fc}^2\ Y_c$		= **4078.19**	[kip-ft]	

Flange force moment arm	$d_m = d_b - t_{fb}$	= 23.307	[in]	
Flange force required in tension	$P_{uf\_t} = P_u\ / 2 - M_u\ / d_m$	= **576.77**	[kips]	
Flange force provided by column flange bending	$\phi R_{cf} = \phi M_{cf} / d_m$	= **2099.72**	[kips]	AISC DG4 Eq 3.21
	ratio = **0.27**	> P_{uf_t}	OK	

Column Web Yielding		ratio = 576.77 / 975.85 = **0.59**		**PASS**
Column web thickness	t_{wc} = 0.787 [in]	yield F_y = 50.0 [ksi]		
Doubler plate thickness	t_{dp} = 0.375 [in]	yield F_{ydp} = 50.0 [ksi]		
Equivalent web thickness when considering doubler plate	$t_{w\text{-}eq} = t_{wc} + t_{dp} \times F_{ydp} / F_y$	= 1.162	[in]	
Flange force moment arm	$d_m = d_b - t_{fb}$	= 23.307	[in]	
Flange force in demand	$P_{uf} = \max(P_{uf\_t}, P_{uf\_c})$	= **576.77**	[kips]	AISC DG13 Eq 4.2-1
Column section	d_c = 19.685 [in]	t_{fc} = 1.575	[in]	
	t_{wc} = 0.787 [in]	k_c = 2.008	[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 = 0.000 [in]	beam flange t_{fb} = 0.748	[in]	
Length of bearing	$N = t_{fb} + 2w$	= 0.748	[in]	AISC DG4 Eq 3.24
End plate thickness	t_p = 2.000 [in]			
Column web yielding strength	$R_n = C_t (6 k_c + N + 2 t_p) F_{yc} t_{w\text{-}eq}$	= 975.85	[kips]	AISC DG4 Eq 3.24
Resistance factor-LRFD	ϕ = 1.00			AISC 14th J10.2
	ϕR_n =	= **975.85**	[kips]	
	ratio = **0.59**	> P_{uf}	OK	

Column Web Buckling		ratio = 509.32 / 896.67 = **0.57**		**PASS**
Flange force moment arm	$d_m = d_b - t_{fb}$	= 23.307	[in]	
Flange force required in compression	$P_{uf\_c} = P_u / 2 - M_u / d_m$	= **509.32**	[kips]	
Column section	d_c = 19.685 [in]	t_{fc} = 1.575	[in]	
	t_{wc} = 0.787 [in]	k_c = 2.008	[in]	
	$h = d_c - 2 k_c$	= 15.669	[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	$R_{n1} = \dfrac{C_t 24 t_{wc}^3 \sqrt{E_c F_{yc}}}{h}$	= 899.04	[kips]	AISC 14th Eq J10-8
Doubler plate thickness	t_{dp} = 0.375 [in]	yield F_{ydp} = 50.0	[ksi]	
Doubler plate buckling strength	$R_{n2} = \dfrac{C_t 24 t_{dp}^3 \sqrt{E_c F_{ydp}}}{h}$	= 97.26	[kips]	AISC 14th Eq J10-8
Total buckling strength	$R_n = R_{n1} + R_{n2}$	= 996.30	[kips]	
Resistance factor-LRFD	ϕ = 0.90			AISC 14th J10.5
	ϕR_n =	= **896.67**	[kips]	
	ratio = **0.57**	> P_{uf_c}	OK	

Column Web Crippling		ratio = 509.32 / 1020.57 = **0.50**	**PASS**
Flange force moment arm	$d_m = d_b - t_{fb}$	= 23.307 [in]	
Flange force required in compression	$P_{uf\_c} = P_u / 2 - M_u / d_m$	= **509.32** [kips]	
Column section	d_c = 19.685 [in]	t_{fc} = 1.575 [in]	
	t_{wc} = 0.787 [in]	k_c = 2.008 [in]	
Column yield strength	F_{yc} = 50.0 [ksi]	E_c = 29000 [ksi]	
Beam flange fillet weld size	w = 0.000 [in]	beam flange t_{fb} = 0.748 [in]	
End plate thickness	t_p = 2.000 [in]		
Length of bearing	$l_b = t_{fb} + 2\,w + 2\,t_p$	= 4.748 [in]	
Distance from top of column to top of beam flange	$d_{end\text{-}flg}$ =	= 0.374 [in]	
Top column condition	it's not a top column case, use Eq J10-4		AISC 14th J10.3 (a)
Column web crippling strength	$R_{n1} = 0.8\, t_{wc}^2 \left[1+3\frac{l_b}{d_c}\left(\frac{t_{wc}}{t_{fc}}\right)^{1.5}\right] \times \left(\frac{E_c F_{yc} t_{fc}}{t_{wc}}\right)^{0.5}$	= 1059.80 [kips]	AISC 14th Eq J10-4
Doubler plate thickness	t_{dp} = 0.375 [in]	yield F_{ydp} = 50.0 [ksi]	
Doubler plate crippling strength	$R_{n2} = 0.8\, t_{dp}^2 \left[1+3\frac{l_b}{d_c}\left(\frac{t_{dp}}{t_{fc}}\right)^{1.5}\right] \times \left(\frac{E_c F_{ydp} t_{fc}}{t_{dp}}\right)^{0.5}$	= 300.97 [kips]	AISC 14th Eq J10-4
Total crippling strength	$R_n = R_{n1} + R_{n2}$	= 1360.76 [kips]	
Resistance factor-LRFD	ϕ = 0.75		AISC 14th J10.3
	$\phi\, R_n$ =	= **1020.57**[kips]	
	ratio = **0.50**	> P_{uf_c} OK	

Column Panel Zone Shear		509.32 / 418.29	**N / A**
Panel zone shear force	$V_p = P_{f\text{-}TR} - P_{f\text{-}TL} - V_s$	= **509.32** [kips]	
Column section	d_c = 19.685 [in]	t_{wc} = 0.787 [in]	
	A_c = 75.330 [in^2]	F_{yc} = 50.0 [ksi]	
Column axial compression - user input	P_r =	= 0.00 [kips]	
	$P_c = P_y = F_{yc} A_c$	= 3766.50 [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	$R_n = 0.6\, F_{yc}\, d_c\, t_{wc}$	= 464.76 [kips]	AISC 14th Eq J10-9
Resistance factor-LRFD	ϕ = 0.90		AISC 14th J10.6
	$\phi\, R_n$ =	= **418.29** [kips]	
Unbalanced force to be resisted by doubler plate	$V_{dp} = R_p - \phi\, R_n$	= **91.03** [kips]	

Doubler Geometry Restriction				**PASS**
Min Doubler Thickness to Allow for Proper Beveling of Plate				AISC DG13 Eq 4.4-4
Column section	t_{fc} = 1.575 [in]	k_c = 2.008	[in]	
Allowed fillet encroachment	r_e = 0.188 [in]			AISC 14th Fig 10-3
Min doubler plate thickness	$t_{pmin} = k_c - t_{fc} - r_e$	= **0.245**	[in]	AISC DG13 Eq 4.4-4
Doubler plate thickness	t_{dp} =	= **0.375**	[in]	
		> t_{pmin}	OK	
Min Doubler Thickness to Avoid Shear Buckling of Plate				AISC DG13 Eq 4.4-5
Column section	d_c = 19.685 [in]	k_c = 2.008	[in]	
	$h = d_c - 2\ k_c$	= 15.669	[in]	
Doubler plate steel	F_y = 50.0 [ksi]	E = 29000	[ksi]	
Min doubler plate thickness	$t_{pmin} = h / 2.24 \sqrt{E / F_y}$	= **0.290**	[in]	AISC 14th G2.1 (a)
Doubler plate thickness	t_{dp} =	= **0.375**	[in]	
		> t_{pmin}	OK	

Doubler Plate Shear Yield		ratio = 91.03 / 221.46	= **0.41**	**PASS**
Doubler plate thickness	t_{dp} = 0.375 [in]	yield strength F_y = 50.0	[ksi]	
Number of doubler	N_{dbl} = 1 for one side			
Column depth	d_c = 19.685 [in]			
Area of web	$A_w = t_{dp} \times d_c$	= 7.382	[in^2]	AISC DG13 Eq 4.4-1
Shear coefficient	C_v = 1.0			AISC 14th Eq G2-2
Doubler plate shear in demand	R_{dp} = unbalanced force from column panel zone shear calc	= **91.03**	[kips]	
Doubler plate shear strength	$R_n = N_{dbl}\ 0.6\ F_y\ A_w\ C_v$	= 221.46	[kips]	AISC 14th Eq G2-1
Resistance factor-LRFD	ϕ = 1.00			AISC 14th G2.1 (a)
	$\phi\ R_n$ =	= **221.46**	[kips]	
	ratio = **0.41**	> R_{dp}	OK	

Doubler to Column Flange Fillet Weld Limitation				**PASS**
Min Fillet Weld Size				
Thinner part joined thickness	t =	= 0.375	[in]	
Min fillet weld size allowed	w_{min} =	= **0.188**	[in]	AISC 14th Table J2.4
Fillet weld size provided	w =	= **0.315**	[in]	
		> w_{min}	OK	
Min Fillet Weld Length				
Fillet weld size provided	w =	= 0.315	[in]	
Min fillet weld length allowed	$L_{min} = 4 \times w$	= **1.260**	[in]	AISC 14th J2.2b
Min fillet weld length	$L = d_b + (3\ k_c + t_p) \times 2$ Ends	= **40.103**	[in]	
		> L_{min}	OK	

Doubler to Column Web Fillet Weld Limitation					**PASS**
Min Fillet Weld Size					
Thinner part joined thickness	t =	= 0.375	[in]		
Min fillet weld size allowed	w_{min} =	= **0.188**	[in]	AISC 14th Table J2.4	
Fillet weld size provided	w =	= **0.315**	[in]		
		> w_{min}	OK		
Min Fillet Weld Length					
Fillet weld size provided	w =	= 0.315	[in]		
Min fillet weld length allowed	$L_{min} = 4 \times w$	= **1.260**	[in]	AISC 14th J2.2b	
Min fillet weld length	$L = d_c - 2 \times k_c$	= **15.669**	[in]		
		> L_{min}	OK		

Doubler Weld Strength at Column Flange		ratio = 91.03 / 281.30	= **0.32**	**PASS**	
Doubler Plate to Column Flange Weld Length Calc					
Beam section	d_b = 24.055 [in]	column section k_c = 2.008	[in]		
End plate thickness	t_p = 2.000 [in]				
Doubler plate to column flange weld length - single fillet	$L = d_b + (3\ k_c + t_p) \times 2$ ends	= **40.103**	[in]	AISC DG13 Fig 4-3	
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 = 1 for single fillet				
Load angle coefficient	$C_2 = (1 + 0.5 \sin^{1.5}\theta)$	= 1.00		AISC 14th Page 8-9	
Fillet weld shear strength	$R_{n\text{-}w} = 0.6\,(C_1 \times 70\text{ ksi})\ 0.707\ w\ n\ C_2$	= 9.352	[kip/in]	AISC 14th Eq 8-1	
Base metal - doubler plate	thickness t = 0.375 [in]	tensile F_u = 65.0	[ksi]		
Base metal - doubler plate is in shear, shear rupture as per AISC 14th Eq J4-4 is checked				AISC 14th J2.4	
Base metal shear rupture	$R_{n\text{-}b} = 0.6\ F_u\ t$	= 14.625	[kip/in]	AISC 14th Eq J4-4	
Single fillet linear shear strength	$R_n = \min(R_{n\text{-}w}, R_{n\text{-}b})$	= **9.352**	[kip/in]	AISC 14th Eq 9-2	
Resistance factor-LRFD	ϕ = 0.75			AISC 14th Eq 8-1	
	$\phi\ R_n$ =	= **7.014**	[kip/in]		
Shear resistance required	R_{dp} =	= **91.03**	[kips]		
Fillet weld length - single fillet	L =	= 40.103	[in]		
Shear resistance provided	$\phi\ F_n = \phi\ R_n \times L$	= **281.30**	[kips]		
	ratio = **0.32**	> R_{dp}	OK		

Doubler Weld Strength at Column Web		ratio = 91.03 / 109.91	= **0.83**	**PASS**	
Doubler Plate to Column Web Weld Length Calc					
Column section	d_c = 19.685 [in]	k_c = 2.008 [in]			
Doubler plate to column web weld length - single fillet	$L = d_c - 2 \times k_c$	= **15.669** [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 = 1 for single fillet				
Load angle coefficient	$C_2 = (1 + 0.5 \sin^{1.5} \theta)$	= 1.00		AISC 14th Page 8-9	
Fillet weld shear strength	$R_{n\text{-}w} = 0.6 (C_1 \times 70 \text{ ksi}) 0.707\ w\ n\ C_2$	= 9.352 [kip/in]		AISC 14th Eq 8-1	
Base metal - doubler plate	thickness t = 0.375 [in]	tensile F_u = 65.0 [ksi]			
Base metal - doubler plate 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	
Single fillet linear shear strength	$R_n = \min (R_{n\text{-}w} , R_{n\text{-}b})$	= **9.352** [kip/in]		AISC 14th Eq 9-2	
Resistance factor-LRFD	ϕ = 0.75			AISC 14th Eq 8-1	
	ϕR_n =	= **7.014** [kip/in]			
Shear resistance required	R_{dp} =	= **91.03** [kips]			
Fillet weld length - single fillet	L =	= 15.669 [in]			
Shear resistance provided	$\phi F_n = \phi R_n \times L$	= **109.91** [kips]			
	ratio = **0.83**	> R_{dp}	OK		