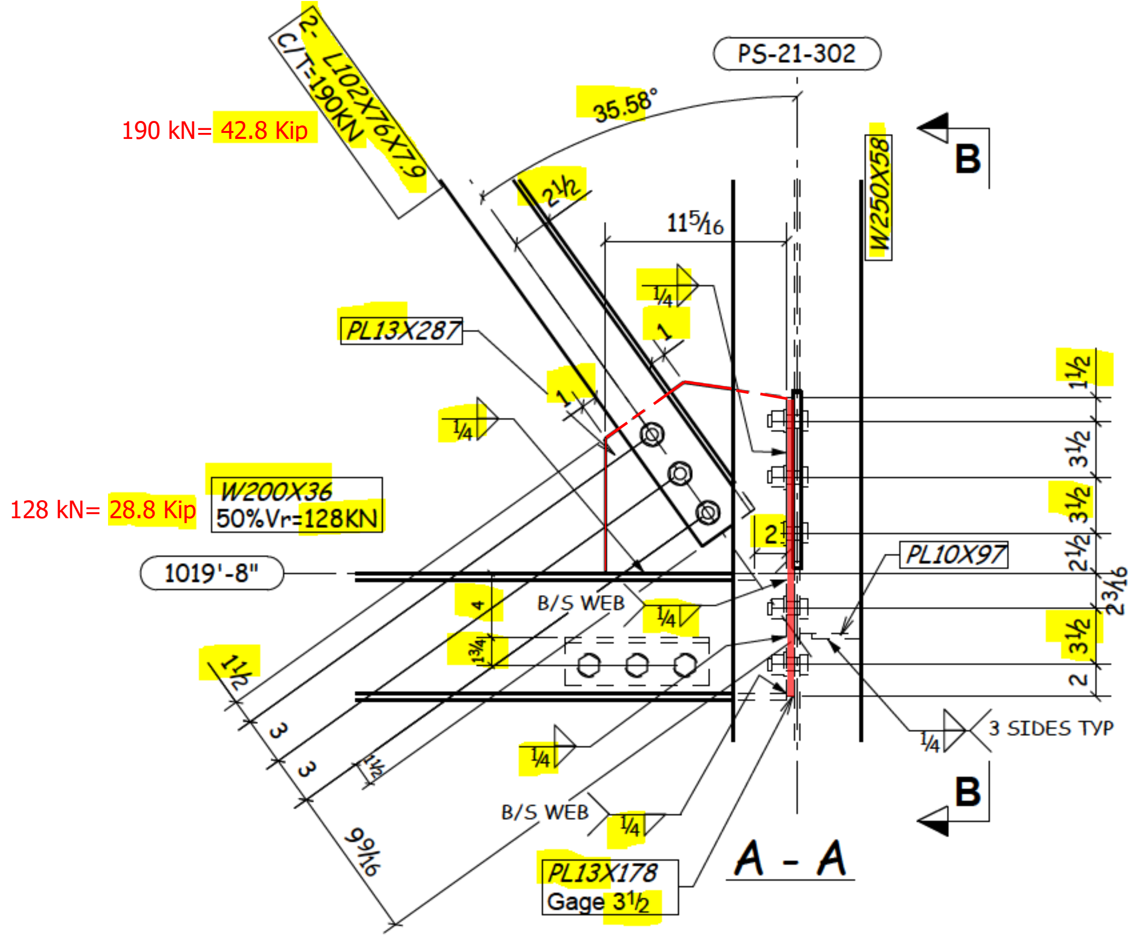
CivilBay Steel Connection Design – Transfer Force or Pass Through Force in Brace Connection – Tutorial

This is the working sheet for tutorial video https://youtu.be/VDbLmzID42A

To view Transfer Force note, visit http://asp.civilbay.com/18-manual/01-manual-sc.aspx#faq25

To start program, visit http://asp.civilbay.com/18-manual/05-new-user.aspx#pos-sc

Check next page for brace connection details



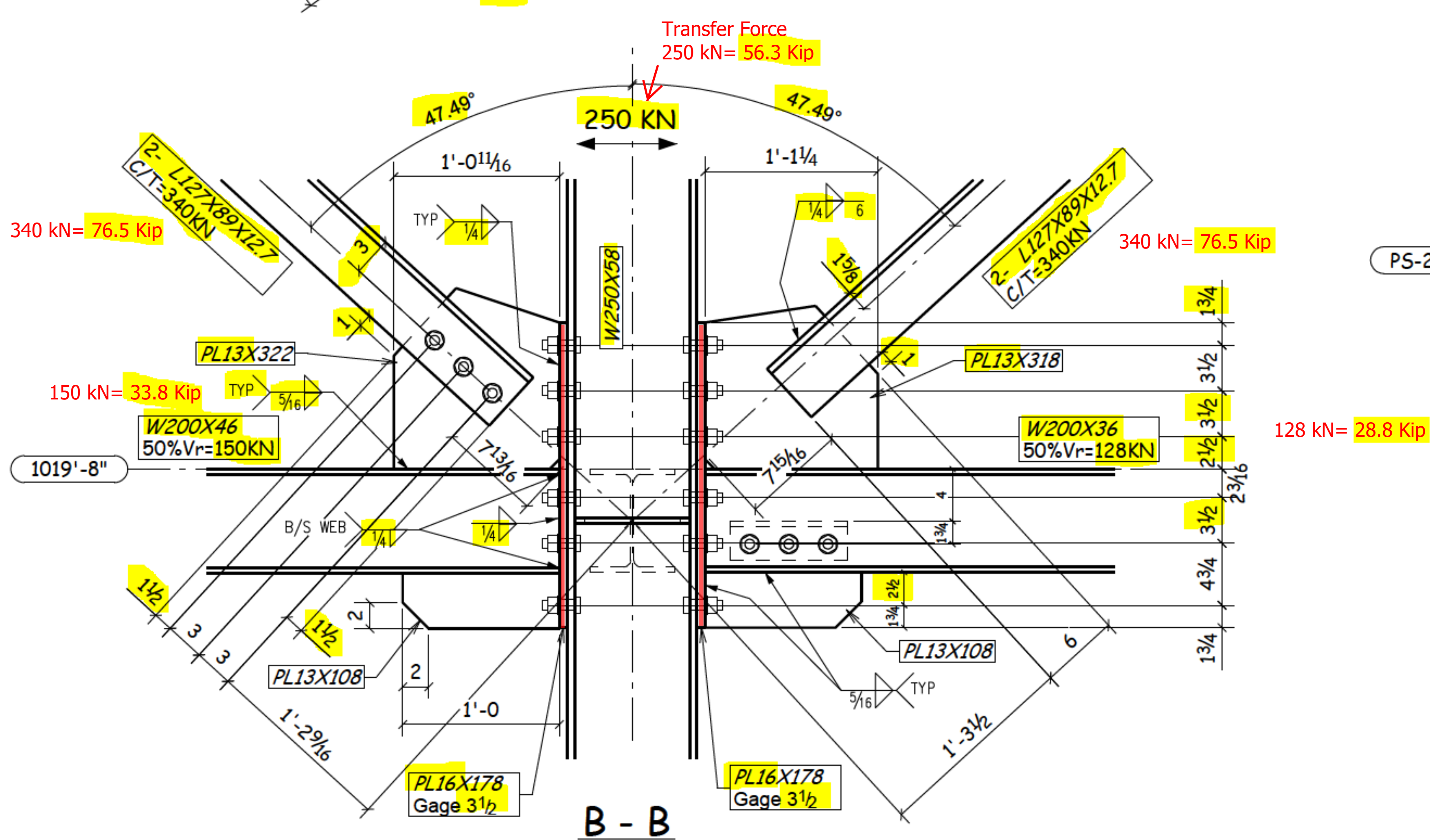
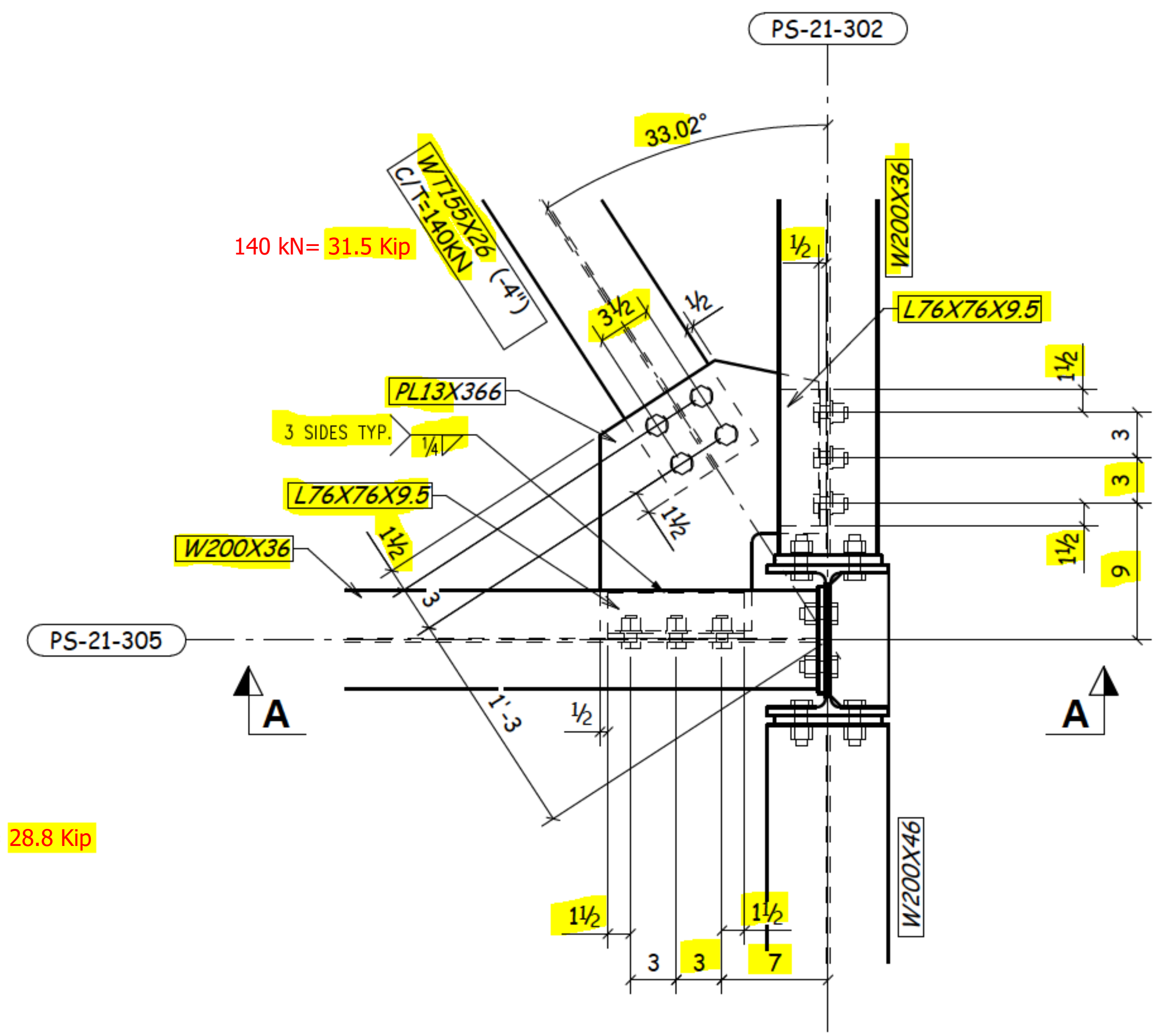


Plate Steel = 350W
Beam & Column Steel = A992-GR.50
Bracing Angle Steel = 300W
Cnx angle steel: 350W
Stiff plates steel: 300W
Bolts : 3/4" A325-N STD Holes



AISC Brace Connection Design Example

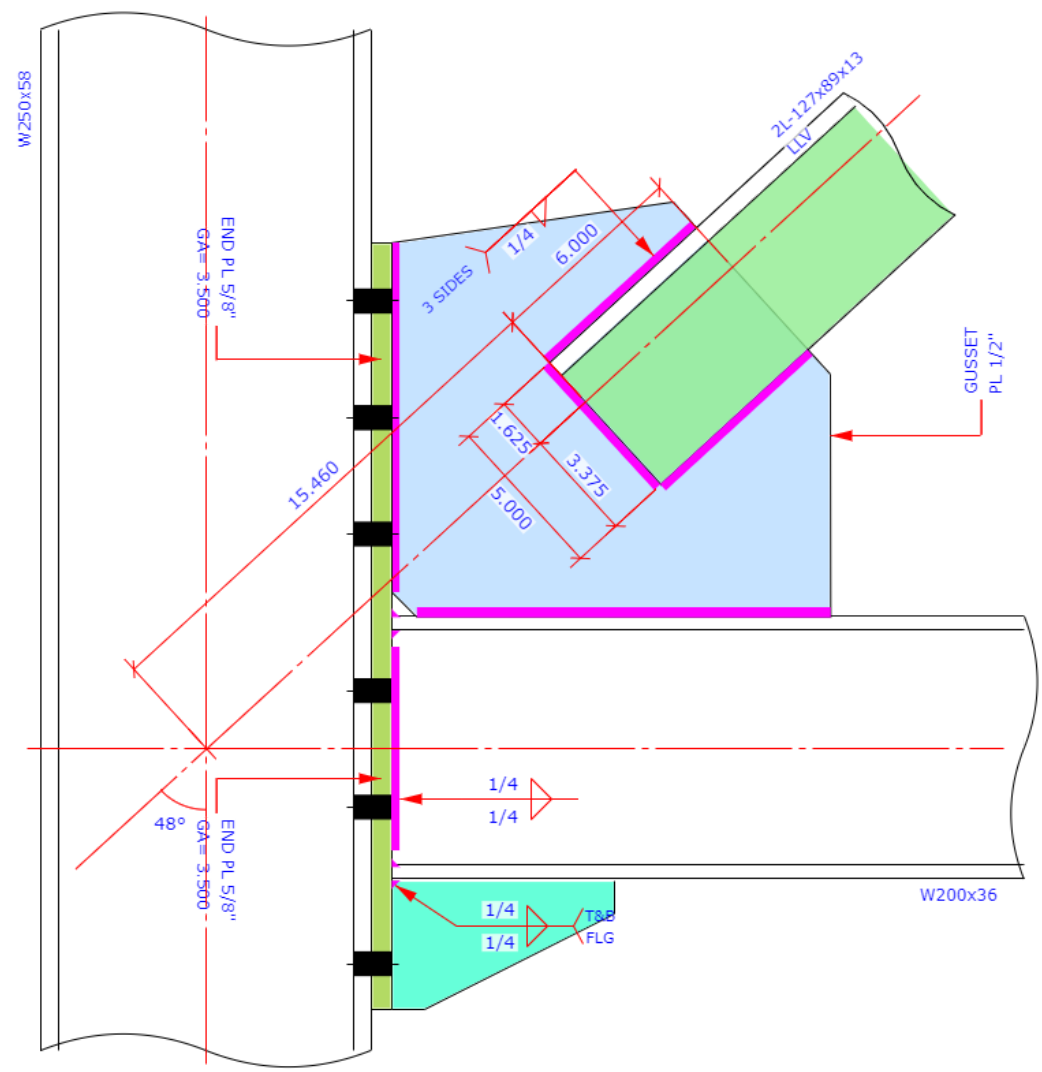
Transfer Force or Pass Through Force

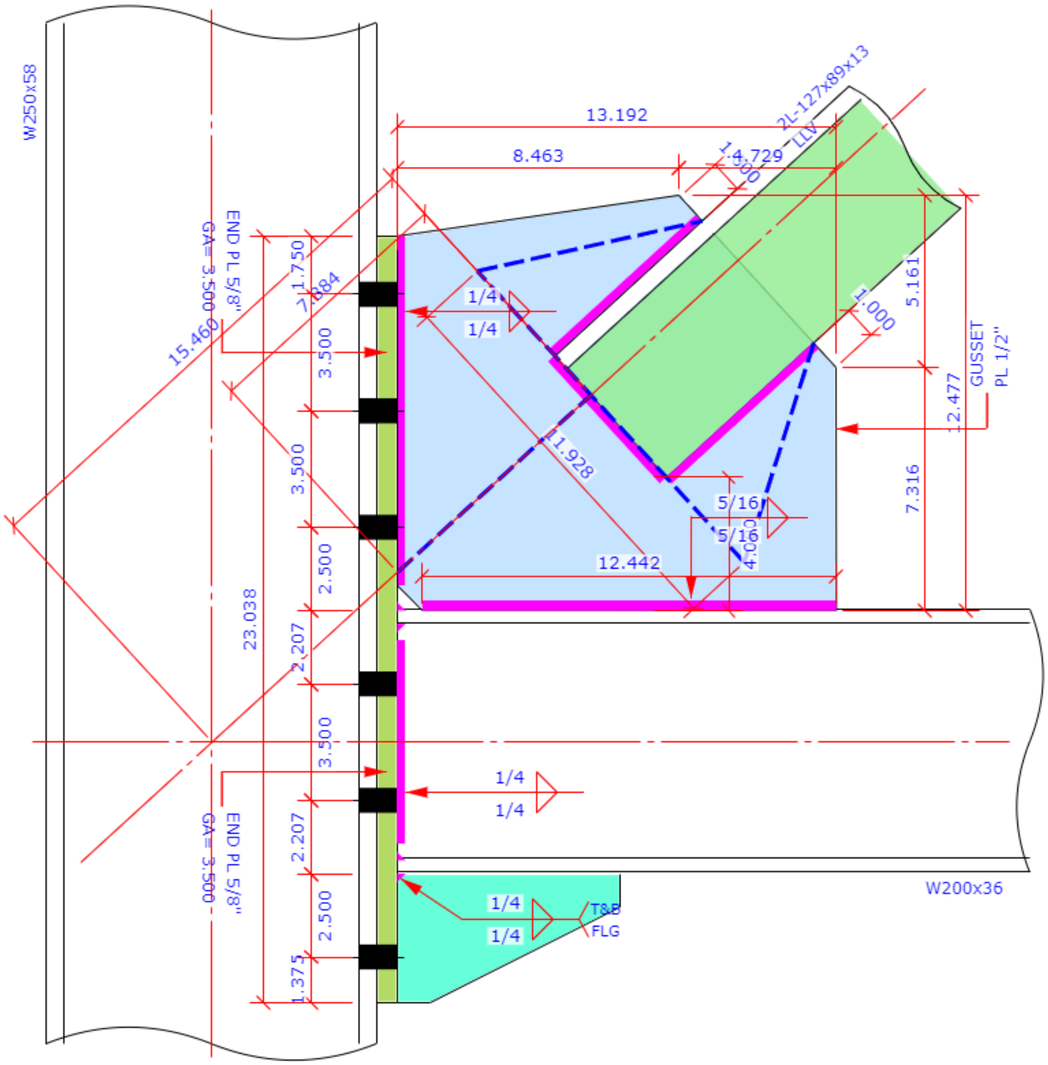
in Brace Connection

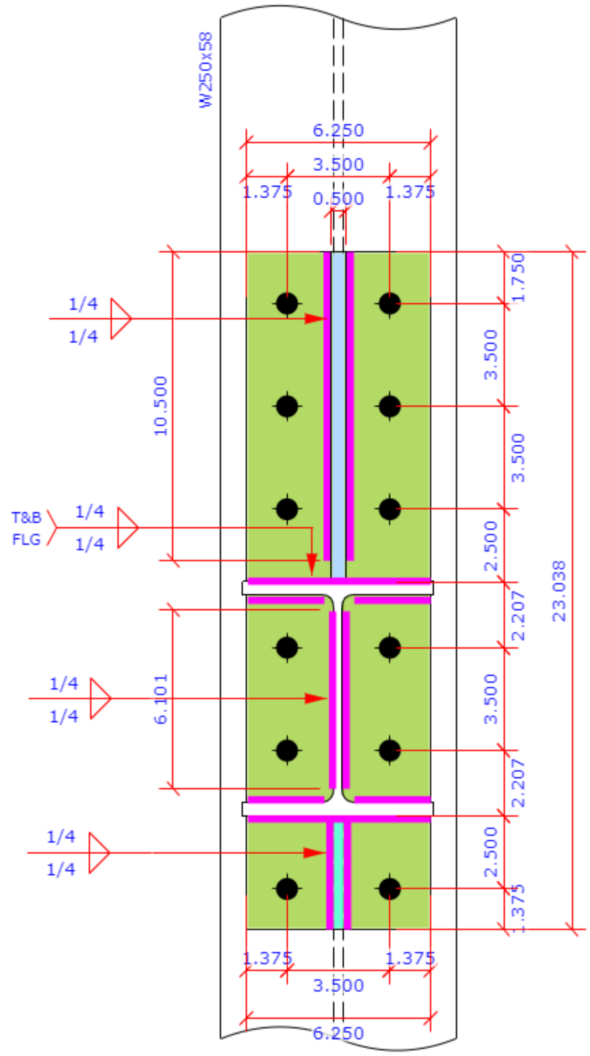
Result Summary - Overall	Vertical Brace Connection	Code=AISC 360-16 LRFD

Result Summary - Overall	geometries & weld limitations = **PASS**	limit states max ratio = **0.99**	**PASS**
Brace to Gusset	geometries & weld limitations = **PASS**	limit states max ratio = **0.40**	**PASS**
Gusset to Column	geometries & weld limitations = **PASS**	limit states max ratio = **0.48**	**PASS**
Gusset to Beam	geometries & weld limitations = **PASS**	limit states max ratio = **0.27**	**PASS**
Beam to Column	geometries & weld limitations = **PASS**	limit states max ratio = **0.99**	**PASS**

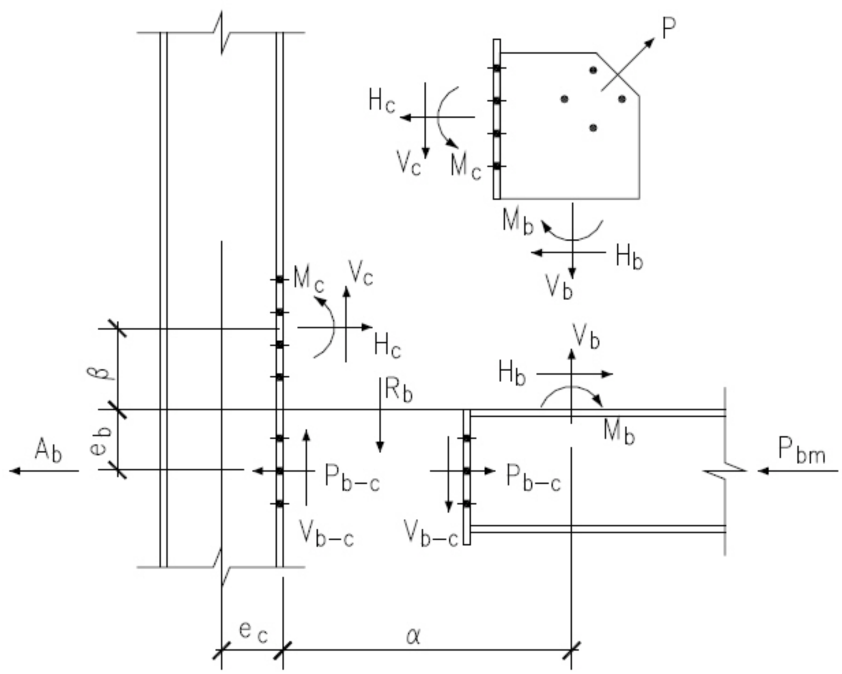
Sketch	Vertical Brace Connection	Code=AISC 360-16 LRFD




W250x58
2L-127x89x13
LLV
13.192
8.463
4.729
1.500
15.460
END PL 5/8"
GA= 3.500
1.750
7.884
3.500
3.500
2.500
2.207
3.500
2.207
2.500
1.375
23.038
1/4
1/4
11.928
5/16
5/16
4.000
12.442
5.161
1.000
GUSSET
PL 1/2"
2.477
7.316
1/4
1/4
END PL 5/8"
GA= 3.500
1/4
1/4
T&B
FLG
W200x36


W250x58
6.250
3.500
1.375
0.500
1.375
1/4
1/4
T&B
FLG
1/4
1/4
10.500
1.750
3.500
3.500
2.500
23.038
2.207
1/4
1/4
6.101
3.500
2.207
1/4
1/4
2.500
1.375
1.375
1.375
3.500
6.250

Gusset Plate Interface Forces Calculation



Brace Axial Force Load Case 1

Brace force $P = -76.5$ [kips] (T)

Beam end shear & transfer force Shear $R_b = 28.8$ [kips] Transfer $A_b = 56.3$ [kips]

Refer to AISC 15th Page 13-4 and Fig. 13-2 for all charts and definitions of variables and symbols shown in calculation below

$e_b = 3.957$ [in] $e_c = 4.961$ [in]

$\alpha = 7.596$ [in] $\beta = 7.750$ [in]

$\theta = 47.5$ [°]

$K = e_b \tan\theta - e_c$ $= -0.643$ [in] AISC 15th Eq. 13-16

$D = \tan^2\theta + \left(\frac{\alpha}{\beta}\right)^2$ $= 2.152$ AISC 15th Eq. 13-24

$K' = \alpha\left(\tan\theta + \frac{\alpha}{\beta}\right)$ $= 15.735$ AISC 15th Eq. 13-23

$\bar{\alpha} = \left[K' \tan\theta + K\left(\frac{\alpha}{\beta}\right)^2\right] / D$ $= 7.815$ [in] AISC 15th Eq. 13-21

$\bar{\beta} = (K' - K\tan\theta) / D$ $= 7.750$ [in] AISC 15th Eq. 13-22

$r = [(e_b + \bar{\beta})^2 + (e_c + \bar{\alpha})^2]^{0.5}$ $= 17.328$ [in] AISC 15th Eq. 13-6

Brace axial force P_u = from user input $= -76.5$ [kips] in tension

Gusset to Column Interface Forces

Shear force $V_c = (\bar{\beta} / r)\, P_u$ = **-34.2** [kips] AISC 15th Eq. 13-2

Axial force $H_c = (e_c / r)\, P_u$ = **-21.9** [kips] AISC 15th Eq. 13-3

Moment $M_c = H_c(\beta - \bar{\beta})$ = **0.00** [kip-ft] AISC 15th Eq. 13-19

Gusset to Beam Interface Forces

Shear force $H_b = (\bar{\alpha} / r)\, P_u$ = **-34.5** [kips] AISC 15th Eq. 13-5

Axial force $V_b = (e_b / r)\, P_u$ = **-17.5** [kips] AISC 15th Eq. 13-4

Moment $M_b = V_b(\bar{\alpha} - \alpha)$ = [illegible] [kip-ft] AISC 15th Eq. 13-17

Moment	$M_b = V_b (\alpha - \alpha)$	= **-0.32**	[kip-ft]	AISC 15th Eq. 13-17

Beam to Column Interface Forces

Beam to Column Interface Shear Force

Beam end shear reaction	R_b = from user input	= 28.8	[kips]	
Brace gusset-beam axial force	V_b =	= -17.5	[kips]	AISC 15th Eq. 13-4
Beam to column shear force	$V_{b-c} = R_b + V_b$	= **11.3**	[kips]	AISC 15th Page 13-4

Beam to Column Interface Axial Force

Gusset-column axial force	H_c =	= -21.9	[kips]	AISC 15th Eq. 13-3
Transfer force from adjacent bay	A_b = from user input	= 56.3	[kips]	
Beam to column axial force	$P_{b-c} = H_c \times -1 - A_b$	= **-34.4**	[kips]	AISC 15th Page 13-4

Beam Member Axial Force

This force is not for use in connection calc. It's output here for user input connection forces equilibrium check only.

P_{bm} - Beam member axial force is different from P_{b-c} - Beam to column interface axial force as shown above.

P_{bm} - Beam member axial force is from structural analysis output and cannot be used directly in beam end to column connection design as this force is interrupted by brace gusset to beam interface force before beam end reaching the column. This force is actually not needed from user's input for beam end to column connection design.

P_{b-c} - Beam to column interface axial force is calculated from user's input of brace axial forces and trasnfer force using uniform force method. This force is used in the beam end to column connection design.

P_{bm} - Beam member axial force is not needed for the beam end to column connection design and is calculated here for verification purpose only. If it matches the structural analysis output, that means equilibrium is reached and user's input of brace axial forces and trasnfer force are correct.

Brace axial force	P = from user input	= -76.5	[kips]	
Brace to ver line angle	θ = from user input	= 47.5	[°]	
Gusset-column axial force	H_c =	= -21.9	[kips]	AISC 15th Eq. 13-3
Beam member axial force	$P_{bm} = (H_c - P \sin\theta) + P_{b-c}$	= **0.1**	[kips]	in compression

Brace Axial Force Load Case 2

Brace force	P = 76.5	[kips] (C)		
Beam end shear & transfer force	Shear R_b = 28.8	[kips]	Transfer A_b = 56.3	[kips]

Refer to AISC 15th Page 13-4 and Fig. 13-2 for all charts and definitions of variables and symbols shown in calculation below

e_b = 3.957	[in]	e_c = 4.961	[in]	
α = 7.596	[in]	β = 7.750	[in]	
θ = 47.5	[°]			

$K = e_b \tan\theta - e_c$	= -0.643	[in]	AISC 15th Eq. 13-16
$D = \tan^2\theta + \left(\frac{\alpha}{\beta}\right)^2$	= 2.152		AISC 15th Eq. 13-24
$K' = \alpha \left(\tan\theta + \frac{\alpha}{\beta} \right)$	= 15.735		AISC 15th Eq. 13-23
$\bar{\alpha} = \left[K' \tan\theta + K \left(\frac{\alpha}{\beta}\right)^2 \right] / D$	= 7.815	[in]	AISC 15th Eq. 13-21
$\bar{\beta} = (K' - K \tan\theta) / D$	= 7.750	[in]	AISC 15th Eq. 13-22
$r = [(e_b + \bar{\beta})^2 + (e_c + \bar{\alpha})^2]^{0.5}$	= 17.328	[in]	AISC 15th Eq. 13-6

Brace axial force	P_u = from user input	= 76.5	[kips]	in compression

Gusset to Column Interface Forces

Shear force	$V_c = (\bar{\beta} / r) P_u$	= **34.2**	[kips]	AISC 15th Eq. 13-2
Axial force	$H_c = (e_c / r) P_u$	= **21.9**	[kips]	AISC 15th Eq. 13-3
Moment	$M_c = H_c (\beta - \bar{\beta})$	= **0.00**	[kip-ft]	AISC 15th Eq. 13-19

Gusset to Beam Interface Forces

Shear force	$H_b = (\bar{\alpha} / r) P_u$	= **34.5**	[kips]	AISC 15th Eq. 13-5
Axial force	$V_b = (e_b / r) P_u$	= **17.5**	[kips]	AISC 15th Eq. 13-4
Moment	$M_b = V_b (\bar{\alpha} - \alpha)$	= **0.32**	[kip-ft]	AISC 15th Eq. 13-17

Beam to Column Interface Forces

Beam to Column Interface Shear Force

Beam end shear reaction	R_b = from user input	= 28.8	[kips]	
Brace gusset-beam axial force	V_b =	= 17.5	[kips]	AISC 15th Eq. 13-4
Beam to column shear force	$V_{b-c} = R_b + V_b$	= **46.3**	[kips]	AISC 15th Page 13-4

Beam to Column Interface Axial Force

Gusset-column axial force	H_c =	= 21.9	[kips]	AISC 15th Eq. 13-3
Transfer force from adjacent bay	A_b = from user input	= 56.3	[kips]	
Beam to column axial force	$P_{b-c} = H_c \times -1 - A_b$	= **-78.2**	[kips]	AISC 15th Page 13-4

Beam Member Axial Force

This force is not for use in connection calc. It's output here for user input connection forces equilibrium check only.

P_{bm} - Beam member axial force is different from P_{b-c} - Beam to column interface axial force as shown above.

P_{bm} - Beam member axial force is from structural analysis output and cannot be used directly in beam end to column connection design as this force is interrupted by brace gusset to beam interface force before beam end reaching the column. This force is actually not needed from user's input for beam end to column connection design.

P_{b-c} - Beam to column interface axial force is calculated from user's input of brace axial forces and trasnfer force using uniform force method. This force is used in the beam end to column connection design.

P_{bm} - Beam member axial force is not needed for the beam end to column connection design and is calculated here for verification purpose only. If it matches the structural analysis output, that means equilibrium is reached and user's input of brace axial forces and trasnfer force are correct.

Brace axial force	P = from user input	= 76.5	[kips]	
Brace to ver line angle	θ = from user input	= 47.5	[°]	
Gusset-column axial force	H_c =	= 21.9	[kips]	AISC 15th Eq. 13-3
Beam member axial force	$P_{bm} = (H_c - P \sin\theta) + P_{b-c}$	= **-112.7**	[kips]	in tension

Top Brace - Brace to Gusset	Sect=2L127x89x13 LLV	P_{LC1} =-76.5 kips (T)	P_{LC2} =76.5 kips (C)	Code=AISC 360-16 LRFD

Result Summary	geometries & weld limitations = **PASS**	limit states max ratio = **0.40**	**PASS**

Brace Weld Limitation Checks - 2L Brace to Gusset Plate					**PASS**
Min Fillet Weld Size					
Thinner part joined thickness	$t =$	= 0.500	[in]		
Min fillet weld size allowed	$w_{min} =$	= **0.188**	[in]	AISC 15th Table J2.4	
Fillet weld size provided	$w =$	= **0.250**	[in]		
		$\geq w_{min}$	OK		
Max Fillet Weld Size					
Along edge plate thickness	$t =$	= 0.500	[in]		
Max fillet weld size allowed	$w_{max} = t - {}^{1}/_{16}$" (2mm)	= **0.438**	[in]	AISC 15th J2.2b	
Fillet weld size provided	$w =$	= **0.250**	[in]		
		$\leq w_{max}$	OK		
Min Fillet Weld Length					
Fillet weld size provided	$w =$	= 0.250	[in]		
Min fillet weld length allowed	$L_{min} = 4 \times w$	= **1.000**	[in]	AISC 15th J2.2b	
Min fillet weld length	$L =$	= **5.000**	[in]		
		$\geq L_{min}$	OK		

Brace Force Load Case 1 Sect=2L127x89x13 LLV P =-76.5 kips (T) ratio = **0.40** **PASS**

Double Angle Brace - Tensile Yield		ratio = 76.5 / 259.1	= **0.30**	**PASS**
Gross area subject to tension	$A_g =$	= 7.998	[in^2]	
Steel yield strength	$F_y =$	= 36.0	[ksi]	
Tensile force required	$P_u =$	= **76.5**	[kips]	
Tensile yielding strength	$R_n = F_y A_g$	= 287.9	[kips]	AISC 15th Eq D2-1
Resistance factor-LRFD	ϕ = 0.90			AISC 15th D2 (a)
	$\phi R_n =$	= **259.1**	[kips]	AISC 15th Eq D2-1
	ratio = **0.30**	$> P_u$	OK	

Double Angle Brace - Tensile Rupture		ratio = 76.5 / 239.9	= **0.32**	**PASS**
Section gross area	A_g = L127x89x13	= 7.998	[in^2]	
Tensile net area	$A_n = A_g$	= 7.998	[in^2]	
Length of connection	$L =$	= 6.000	[in]	
Width of angle leg	$w =$	= 5.000	[in]	
Eccentricity of connection	$\bar{x}$ = from sect L127x89x13	= 0.906	[in]	
Shear lag factor	$U = \frac{3L^2}{3L^2 + w^2}\left(1 - \frac{\bar{x}}{L}\right)$	= 0.689		AISC 15th Table D3.1 Case4
Tensile force required	$P_u =$	= **76.5**	[kips]	
Tensile effective net area	$A_e = A_n U$	= 5.514	[in^2]	
Plate tensile strength	$F_u =$	= 58.0	[ksi]	
Tensile rupture strength	$R_n = F_u A_e$	= 319.8	[kips]	AISC 15th Eq D2-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15th D2 (b)
	$\phi R_n =$	= **239.9**	[kips]	AISC 15th Eq D2-2
	ratio = **0.32**	$> P_u$	OK	

Gusset Plate - Tensile Yield (Whitmore)		ratio = 76.5 / 268.4	= **0.29**	**PASS**	
Plate Tensile Yielding Check					
Plate size	width b_p = 11.928 [in]	thickness t_p = 0.500	[in]		
Plate yield strength	F_y = 50.0 [ksi]				
Plate gross area in shear	$A_g = b_p t_p$	= 5.964	[in^2]		
Tensile force required	P_u =	= **76.5**	[kips]		
Plate tensile yielding strength	$R_n = F_y A_g$	= 298.2	[kips]	AISC 15th Eq J4-1	
Resistance factor-LRFD	ϕ = 0.90			AISC 15th Eq J4-1	
	ϕR_n =	= **268.4**	[kips]		
	ratio = **0.29**	> P_u	OK		

Gusset Plate - Tensile Rupture (Whitmore)		ratio = 76.5 / 290.7	= **0.26**	**PASS**	
Plate Tensile Rupture Check					
Plate size	width b_p = 11.928 [in]	thickness t_p = 0.500	[in]		
Plate tensile strength	F_u = 65.0 [ksi]				
Plate net area in tension	$A_{nt} = b_p t_p$	= 5.964	[in^2]		
Tensile force required	P_u =	= **76.5**	[kips]		
Plate tensile rupture strength	$R_n = F_u A_{nt}$	= 387.7	[kips]	AISC 15th Eq J4-2	
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-2	
	ϕR_n =	= **290.7**	[kips]	AISC 15th Eq J4-2	
	ratio = **0.26**	> P_u	OK		

2L Brace to Gusset Plate Weld Strength		ratio = 76.5 / 189.3	= **0.40**	**PASS**	
Fillet Weld Strength Check					
Fillet weld leg size	$w = {}^1/_4$ [in]	load angle θ = 0.0	[°]		
Electrode strength	F_{EXX} = 70.0 [ksi]	strength coeff C_1 = 1.00		AISC 15th 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 15th Page 8-9	
Fillet weld shear strength	$R_{n\text{-}w} = 0.6 (C_1 \times 70 \text{ ksi}) \, 0.707 \, w \, n \, C_2$	= 14.85	[kip/in]	AISC 15th Eq 8-1	
Base metal - gusset plate	thickness t = 0.500 [in]	tensile F_u = 65.0	[ksi]		
Base metal - gusset plate is in shear, <u>shear</u> rupture as per AISC 15th Eq J4-4 is checked				AISC 15th J2.4	
Base metal shear rupture	$R_{n\text{-}b} = 0.6 F_u t$	= 19.50	[kip/in]	AISC 15th Eq J4-4	
Double fillet linear shear strength	$R_n = \min (R_{n\text{-}w} , R_{n\text{-}b})$	= **14.847**	[kip/in]	AISC 15th Eq 9-2	
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq 8-1	
	ϕR_n =	= **11.135**	[kip/in]		
Weld resistance required	V_u =	= **76.5**	[kips]		
Fillet weld length - double fillet	L =	= 17.000	[in]		
Weld resistance provided	$\phi F_n = \phi R_n \times L$	= **189.3**	[kips]		
	ratio = **0.40**	> V_u	OK		

Brace Force Load Case 2 Sect=2L127x89x13 LLV P =76.5 kips (C) ratio = **0.40** **PASS**

Gusset Plate - Compression (Whitmore)		ratio = 76.5 / 248.1	= **0.31**	**PASS**
Plate Compression Check				
Plate size	width b_p = 11.928 [in]	thickness t_p = 0.500 [in]		
	F_y = 50.0 [ksi]	E = 29000.0 [ksi]		
Plate gross area in compression	$A_g = b_p t_p$	= 5.964 [in^2]		
Plate radius of gyration	$r = t_p / \sqrt{12}$	= 0.144 [in]		
Plate effective length factor	K =	= 0.60		
Plate unbraced length	L_u =	= 7.884 [in]		
Plate slenderness	KL/r = 0.60 x L_u / r	= 32.77		
	when $\frac{KL}{r} > 25$, use Chapter E		AISC 15th J4.4 (b)	
Elastic buckling stress	$F_e = \frac{\pi^2 E}{(KL/r)^2}$	= 266.5 [ksi]	AISC 15th Eq E3-4	
	when $\frac{KL}{r} \leq 4.71 \left(\frac{E}{F_y}\right)^{0.5} = 113.43$		AISC 15th E3 (a)	
Critical stress	$F_{cr} = 0.658^{(F_y / F_e)} F_y$	= 46.2 [ksi]	AISC 15th Eq E3-2	
Plate compression required	P_u =	= **76.5** [kips]		
Plate compression provided	$R_n = F_{cr} \times A_g$	= 275.7 [kips]	AISC 15th Eq E3-1	
Resistance factor-LRFD	ϕ = 0.90		AISC 15th E1	
	ϕR_n =	= **248.1** [kips]		
	ratio = **0.31**	> P_u OK		

2L Brace to Gusset Plate Weld Strength		ratio = 76.5 / 189.3	= **0.40**	**PASS**
Fillet Weld Strength Check				
Fillet weld leg size	w = 1/4 [in]	load angle θ = 0.0 [°]		
Electrode strength	F_{EXX} = 70.0 [ksi]	strength coeff C_1 = 1.00	AISC 15th 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 15th Page 8-9	
Fillet weld shear strength	$R_{n\text{-}w} = 0.6 (C_1 \times 70\ ksi)\ 0.707\ w\ n\ C_2$	= 14.85 [kip/in]	AISC 15th Eq 8-1	
Base metal - gusset plate	thickness t = 0.500 [in]	tensile F_u = 65.0 [ksi]		
Base metal - gusset plate is in shear, shear rupture as per AISC 15th Eq J4-4 is checked			AISC 15th J2.4	
Base metal shear rupture	$R_{n\text{-}b} = 0.6 F_u t$	= 19.50 [kip/in]	AISC 15th Eq J4-4	
Double fillet linear shear strength	$R_n = \min(R_{n\text{-}w}, R_{n\text{-}b})$	= **14.847** [kip/in]	AISC 15th Eq 9-2	
Resistance factor-LRFD	ϕ = 0.75		AISC 15th Eq 8-1	
	ϕR_n =	= **11.135** [kip/in]		
Weld resistance required	V_u =	= **76.5** [kips]		
Fillet weld length - double fillet	L =	= 17.000 [in]		
Weld resistance provided	$\phi F_n = \phi R_n \times L$	= **189.3** [kips]		
	ratio = **0.40**	> V_u OK		

Gusset to Column	End Plate Connection	Code=AISC 360-16 LRFD

Result Summary	geometries & weld limitations = **PASS**	limit states max ratio = **0.48**	**PASS**

Geometry Restriction Checks - End Plate to Column Flange — PASS

Min Bolt Edge Distance - End Plate to Column Flange

Bolt diameter	d_b =	= 0.750	[in]	
Min edge distance allowed	$L_{e\text{-}min}$ =	= **1.000**	[in]	AISC 15th Table J3.4
Min edge distance in End Plate to Column Flange	L_e =	= **1.375**	[in]	
		≥ $L_{e\text{-}min}$	OK	

Min Bolt Spacing - End Plate to Column Flange

Bolt diameter	d_b =	= 0.750	[in]	
Min bolt spacing allowed	$L_{s\text{-}min}$ = 2.667 d_b	= **2.000**	[in]	AISC 15th J3.3
Min Bolt spacing in End Plate to Column Flange	L_s =	= **3.500**	[in]	
		≥ $L_{s\text{-}min}$	OK	

Geometry Restriction Checks - End Plate - Bolt Gage Clearance — PASS

Bolt Gage Entering Clearance Check - Plate Welded to End Plate

Bolt diameter	d_b = 0.750 [in]	gage g = 3.500	[in]		
Bolt entering clearance	c_3 = from AISC manual Table 7-15	= **0.750**	[in]	AISC 15th Table 7-15	
Plate thickness	t = 0.500 [in]	dbl fillet w = 0.250	[in]		
Bolt center clearance distance to fillet toe	c = (g - t - 2 w) / 2	= **1.250**	[in]		
		≥ c_3	OK	AISC 15th Table 7-15	

Geometry Restriction Checks-Column Flg-Bolt Gage Clearance — PASS

Bolt Gage Entering Clearance Check - Bolt on W Shape Flange

Bolt diameter	d_b = 0.750 [in]	gage g = 3.500	[in]	
Bolt entering clearance	c_3 = from AISC manual Table 7-15	= **0.750**	[in]	AISC 15th Table 7-15
W section	t_w = 0.315 [in]	k_1 = 0.748	[in]	
Bolt center clearance distance to fillet toe	c = (g - 2 k_1) / 2	= **1.002**	[in]	
		≥ c_3	OK	AISC 15th Table 7-15

Weld Limitation Checks - Gusset Plate to End Plate — PASS

Min Fillet Weld Size

Thinner part joined thickness	t =	= 0.500	[in]	
Min fillet weld size allowed	w_{min} =	= **0.188**	[in]	AISC 15th Table J2.4
Fillet weld size provided	w =	= **0.250**	[in]	
		≥ w_{min}	OK	

Min Fillet Weld Length

Fillet weld size provided	w =	= 0.250	[in]	
Min fillet weld length allowed	L_{min} = 4 x w	= **1.000**	[in]	AISC 15th J2.2b
Min fillet weld length	L =	= **10.500**	[in]	
		≥ L_{min}	OK	

Brace Force Load Case 1 Gusset plate t=0.500 P =-76.5 kips (T) ratio = **0.48** **PASS**

Gusset Plate - Shear Yielding		ratio = 34.2 / 212.4	= **0.16**	**PASS**
Calculate gusset or stiff plate length outside beam flange and count it as beam web extension to resist shear				
Beam sect depth W200x36	d_b = 7.913 [in]			
Bolt pitch & edge distance	d_1 = 2.500 [in]	e_v = 1.375	[in]	
Beam web extension outside beam flange	$L_e = 2(d_1 + e_v)$ - 2x0.75 in clip	= 6.250	[in]	
Total beam web depth to resist shear	$L = d_b + L_e$	= **14.163**	[in]	
Plate Shear Yielding Check				
Plate size	width b_p = 14.163 [in]	thickness t_p = 0.500	[in]	
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$	= 7.082	[in^2]	
Shear force required	V_u =	= **34.2**	[kips]	
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$	= 212.4	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	ϕR_n =	= **212.4**	[kips]	
	ratio = **0.16**	> V_u	OK	

Gusset Plate - Shear Rupture		ratio = 34.2 / 207.1	= **0.17**	**PASS**
Calculate gusset or stiff plate length outside beam flange and count it as beam web extension to resist shear				
Beam sect depth W200x36	d_b = 7.913 [in]			
Bolt pitch & edge distance	d_1 = 2.500 [in]	e_v = 1.375	[in]	
Beam web extension outside beam flange	$L_e = 2(d_1 + e_v)$ - 2x0.75 in clip	= 6.250	[in]	
Total beam web depth to resist shear	$L = d_b + L_e$	= **14.163**	[in]	
Plate Shear Rupture Check				
Plate size	width b_p = 14.163 [in]	thickness t_p = 0.500	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate net area in shear	$A_{nv} = b_p t_p$	= 7.082	[in^2]	
Shear force in demand	V_u =	= **34.2**	[kips]	
Plate shear rupture strength	$R_n = 0.6 F_u A_{nv}$	= 276.2	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-4
	ϕR_n =	= **207.1**	[kips]	
	ratio = **0.17**	> V_u	OK	

Gusset Plate - Axial Yield			ratio = 21.9 / 263.9	= **0.08**	**PASS**
Plate Tensile Yielding Check					
Plate size	width b_p = 11.727	[in]	thickness t_p = 0.500	[in]	
Plate yield strength	F_y = 50.0	[ksi]			
Plate gross area in shear	$A_g = b_p t_p$		= 5.864	[in^2]	
Tensile force required	P_u =		= **21.9**	[kips]	
Plate tensile yielding strength	$R_n = F_y A_g$		= 293.2	[kips]	AISC 15th Eq J4-1
Resistance factor-LRFD	ϕ = 0.90				AISC 15th Eq J4-1
	ϕR_n =		= **263.9**	[kips]	
	ratio = **0.08**		> P_u	OK	

Gusset Plate - Axial Tensile Rupture			ratio = 21.9 / 285.8	= **0.08**	**PASS**
Plate Axial Tensile Rupture Check					
Plate size	width b_p = 11.727	[in]	thickness t_p = 0.500	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate gross area	$A_n = b_p t_p$		= 5.864	[in^2]	
Shear lag factor	U =		= 1.000		
Tensile force required	P_u =		= **21.9**	[kips]	
Tensile effective net area	$A_e = A_n U$		= 5.864	[in^2]	
Plate tensile strength	F_u =		= 65.0	[ksi]	
Tensile rupture strength	$R_n = F_u A_e$		= 381.1	[kips]	AISC 15th Eq D2-2
Resistance factor-LRFD	ϕ = 0.75				AISC 15th D2 (b)
	ϕR_n =		= **285.8**	[kips]	AISC 15th Eq D2-2
	ratio = **0.08**		> P_u	OK	

End Plate - Shear Yield			ratio = 17.1 / 131.3	= **0.13**	**PASS**
Plate Shear Yielding Check					
Plate size	width b_p = 7.000	[in]	thickness t_p = 0.625	[in]	
Plate yield strength	F_y = 50.0	[ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$		= 4.375	[in^2]	
Shear force required	V_u =		= **17.1**	[kips]	
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$		= 131.3	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00				AISC 15th Eq J4-3
	ϕR_n =		= **131.3**	[kips]	
	ratio = **0.13**		> V_u	OK	

End Plate - Shear Rupture			ratio = 17.1 / 96.0	= **0.18**	**PASS**
Plate Shear Rupture Check					
Bolt hole diameter	bolt dia $d_b = ^3/_4$	[in]	bolt hole dia $d_h = ^7/_8$	[in]	AISC 15th B4.3b
Number of bolt	n = 2				
Plate size	width b_p = 7.000	[in]	thickness t_p = 0.625	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate net area in shear	$A_{nv} = (b_p - n\ d_h)\ t_p$		= 3.281	[in^2]	
Shear force required	V_u =		= **17.1**	[kips]	
Plate shear rupture strength	$R_n = 0.6\ F_u A_{nv}$		= 128.0	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J4-4
	$\phi\ R_n$ =		= **96.0**	[kips]	
	ratio = **0.18**		> V_u	OK	

End Plate - Block Shear - Center Strip			ratio = 34.2 / 223.9	= **0.15**	**PASS**
Plate Block Shear - Center Strip					
Bolt hole diameter	bolt dia $d_b = ^3/_4$	[in]	bolt hole dia $d_h = ^7/_8$	[in]	AISC 15th B4.3b
Plate thickness	t_p = 0.625	[in]			
Plate strength	F_y = 50.0	[ksi]	F_u = 65.0	[ksi]	
Bolt no in ver & hor dir	n_v = 2		n_h = 2		
Bolt spacing in ver & hor dir	s_v = 3.500	[in]	s_h = 3.500	[in]	
Bolt edge dist in ver & hor dir	e_v = 1.375	[in]	e_h = 1.750	[in]	
Gross area subject to shear	$A_{gv} = [\ (n_h - 1)\ s_h + e_h]\ t_p \times 2$		= 6.563	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5]\ d_h t_p \times 2$		= 4.922	[in^2]	
Net area subject to tension					
when sheared out by center strip	$A_{nt} = (n_v - 1)\ (s_v - d_h)\ t_p$		= 1.641	[in^2]	
Block shear strength required	V_u =		= **34.2**	[kips]	
Uniform tension stress factor	U_{bs} = 1.00				AISC 15th Fig C-J4.2
Bolt shear resistance provided	$R_n = \min\ (0.6F_u A_{nv}\ ,\ 0.6F_y A_{gv}) + U_{bs} F_u A_{nt}$		= 298.6	[kips]	AISC 15th Eq J4-5
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J4-5
	$\phi\ R_n$ =		= **223.9**	[kips]	
	ratio = **0.15**		> V_u	OK	

End Plate - Block Shear - 2-Side Strip			ratio = 34.2 / 201.1	= **0.17**	**PASS**
Plate Block Shear - 2 Side Strips					
Bolt hole diameter	bolt dia $d_b = {}^3/_4$	[in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 15th B4.3b
Plate thickness	t_p = 0.625	[in]			
Plate strength	F_y = 50.0	[ksi]	F_u = 65.0	[ksi]	
Bolt no in ver & hor dir	n_v = 2		n_h = 2		
Bolt spacing in ver & hor dir	s_v = 3.500	[in]	s_h = 3.500	[in]	
Bolt edge dist in ver & hor dir	e_v = 1.375	[in]	e_h = 1.750	[in]	
Gross area subject to shear	$A_{gv} = [\ (n_h - 1)\ s_h + e_h\ ]\ t_p \times 2$		= 6.563	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5]\ d_h\ t_p \times 2$		= 4.922	[in^2]	
Net area subject to tension					
when sheared out by 2 side strips	$A_{nt} = (\ e_v - 0.5\ d_h\ )\ t_p \times 2$		= 1.172	[in^2]	
Block shear strength required	V_u =		= **34.2**	[kips]	
Uniform tension stress factor	U_{bs} = 1.00				AISC 15th Fig C-J4.2
Bolt shear resistance provided	$R_n = \min\ (0.6F_u A_{nv}, 0.6F_y A_{gv}) + U_{bs} F_u A_{nt}$		= 268.1	[kips]	AISC 15th Eq J4-5
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J4-5
	$\phi\ R_n$ =		= **201.1**	[kips]	
	ratio = **0.17**		> V_u	OK	

End Plate - Bolt Bearing on End Plate			ratio = 34.2 / 71.6	= **0.48**	**PASS**
Single Bolt Shear Strength					
Bolt shear stress	bolt grade = A325-N		F_{nv} = 54.0	[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.750	[in]	bolt area A_b = 0.442	[in^2]	
Single bolt shear strength	$R_{n\text{-bolt}} = F_{nv} A_b$		= 23.9	[kips]	AISC 15th 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 15th Table J3.3
Bolt spacing & edge distance	spacing L_s = 3.500	[in]	edge distance L_e = 1.750	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate thickness	t = 0.625	[in]			
Interior Bolt					
Bolt hole edge clear distance	$L_c = L_s - d_h$		= 2.688	[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 15th Eq J3-6b
	= 163.8 ≤ 91.4		= 91.4	[kips]	
Bolt strength at interior	$R_{n\text{-in}} = \min(R_{n\text{-t\&b-in}}, R_{n\text{-bolt}})$		= 23.9	[kips]	
Edge Bolt					
Bolt hole edge clear distance	$L_c = L_e - d_h / 2$		= 1.344	[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 15th Eq J3-6b
	= 81.9 ≤ 91.4		= 81.9	[kips]	
Bolt strength at edge	$R_{n\text{-ed}} = \min(R_{n\text{-t\&b-ed}}, R_{n\text{-bolt}})$		= 23.9	[kips]	
Number of bolt	interior n_{in} = 2		edge n_{ed} = 2		
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-in}} + n_{ed} R_{n\text{-ed}}$		= 95.4	[kips]	
Required shear strength	V_u =		= **34.2**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 15th J3.10
	ϕR_n =		= **71.6**	[kips]	
	ratio = **0.48**		> V_u	OK	

End Plate / Column - Bolt Shear			ratio = 34.2 / 71.6	= **0.48**	**PASS**
Bolt group forces	shear V = 34.2	[kips]	axial P = 21.9	[kips]	
Bolt A325-N	dia d_b = 0.750	[in]	A_b = 0.442	[in^2]	
Bolt shear stress	grade = A325-N		F_{nv} = 54.0	[ksi]	AISC 15th Table J3.2
Number of bolt carried shear	n_s = 4.0		shear plane m = 1		
Bolt group eccentricity coefficient	C_{ec} =		= 1.000		
Required shear strength	V_u =		= **34.2**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$		= 95.4	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J3-1
	ϕR_n =		= **71.6**	[kips]	
	ratio = **0.48**		> V_u	OK	

End Plate / Column - Bolt Bearing on Column		ratio = 34.2 / 71.6	= **0.48**	**PASS**

Single Bolt Shear Strength

Bolt shear stress	bolt grade = A325-N		F_{nv} = 54.0	[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.750	[in]	bolt area A_b = 0.442	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$		= 23.9	[kips]	AISC 15th 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 15th Table J3.3
Bolt spacing	spacing L_s = 3.500	[in]			
Plate tensile strength	F_u = 65.0	[ksi]			
Plate thickness	t = 0.531	[in]			

Interior Bolt

Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.688	[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 15th Eq J3-6b
	= 139.1 ≤ 77.7	= 77.7	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$	= 23.9	[kips]	

Number of bolt	interior n_{in} = 4			
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-}in}$	= 95.4	[kips]	
Required shear strength	V_u =	= **34.2**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th J3.10
	ϕR_n =	= **71.6**	[kips]	
	ratio = **0.48**	> V_u	OK	

Bolt Tensile Prying Action on End Plate			ratio = 5.5 / 20.9	= **0.26**	**PASS**
Bolt group forces	shear V = 34.2 [kips]		axial P = -21.9 [kips]		

Single Bolt Tensile Capacity Without Considering Prying

Bolt grade A325-N	dia d_b = 0.750 [in]	area A_b = 0.442	[in^2]	
Nominal tensile/shear stress	F_{nt} = 90.0 [ksi]	F_{nv} = 54.0	[ksi]	AISC 15th Table J3.2
Bolt group shear force	shear V = 34.2 [kips]	no of bolt n = 4		
Shear stress required	$f_{rv} = V / (n A_b)$	= 19.4	[ksi]	
Resistance factor-LRFD	ϕ = 0.75			AISC 15th J3.7
Modified nominal tensile stress	$F'_{nt} = 1.3 F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_{rv} \le F_{nt}$	= **74.0**	[ksi]	AISC 15th Eq J3-3a
Bolt norminal tensile strength	$r_n = F'_{nt} A_b$	= 32.7	[kips]	AISC 15th Eq J3-1
Resistance factor-LRFD	ϕ = 0.75			AISC 15th J3.6
Single bolt tensile capacity	ϕr_n =	= **24.5**	[kips]	

Single Bolt Tensile Capacity After Considering Prying

End plate	width w = 6.250 [in]	bolt gage g = 3.500	[in]	
	web t_w = 0.500 [in]			
Dist from bolt center to plate edge	a = 0.5 (w - g)	= 1.375	[in]	
	$a' = a + 0.5 d_b \le (1.25 b + 0.5 d_b)$	= 1.750	[in]	AISC 15th Eq 9-23
Bolt hole diameter	bolt dia d_b = 0.750 [in]	bolt hole dia d_h = 0.813	[in]	AISC 15th B4.3b
Dist from bolt center to face of web	$b = 0.5(g - t_w)$	= 1.500	[in]	
	$b' = b - 0.5 d_b$	= 1.125	[in]	AISC 15th Eq 9-18
Bolt pitch spacing	s_v = 3.500 [in]			
Bolt tributary length	$p = s_v$ $p \le 2b$ and $p \le s_v$	= 3.000	[in]	AISC 15th Page 9-12
	$\rho = b' / a'$	= 0.643		AISC 15th Eq 9-22
	$\delta = 1 - d_h / p$	= 0.729		AISC 15th Eq 9-20
Tensile capacity per bolt before considering prying	B = from calc shown in above section	= 24.5	[kips]	
Resistance factor-LRFD	ϕ = 0.90			AISC 15th Page 9-12
End plate thickness	t = 0.625 [in]	tensile F_u = 65.0	[ksi]	
Plate thickness req'd to develop bolt tensile capacity without prying	$t_c = (\frac{4 B b'}{\phi p F_u})^{0.5}$	= 0.793	[in]	AISC 15th Eq 9-26a
	$\alpha' = \frac{1}{\delta (1 + \rho)} [(\frac{t_c}{t})^2 - 1]$	= 0.509		AISC 15th Eq 9-28
when $0 \le \alpha' \le 1$	$Q = (\frac{t}{t_c})^2 (1 + \delta \alpha')$	= 0.852		AISC 15th Eq 9-27
Bolt tensile force per bolt in demand	T = from calc shown below	= **5.5**	[kips]	
Tensile strength per bolt after considering prying	$\phi r_n = B \times Q$	= **20.9**	[kips]	AISC 15th Eq 9-27
	ratio = **0.26**	> T	OK	

Calculate Max Single Bolt Tensile Load

Bolt group force	axial P = 21.9 [kips]			
Bolt number	Bolt Row n_h = 2	Bolt Col n_v = 2		
Bolt tensile force per bolt	$T = P / (n_v n_h)$	= **5.5**	[kips]	

Bolt Tensile Prying Action on Column Flange			ratio = 5.5 / 18.8	= **0.29**	**PASS**
Bolt group forces	shear V = 34.2 [kips]		axial P = -21.9 [kips]		

Single Bolt Tensile Capacity Without Considering Prying

Bolt grade A325-N	dia d_b = 0.750 [in]	area A_b = 0.442	[in^2]		
Nominal tensile/shear stress	F_{nt} = 90.0 [ksi]	F_{nv} = 54.0	[ksi]	AISC 15^{th} Table J3.2
Bolt group shear force	shear V = 34.2 [kips]	no of bolt n = 4		
Shear stress required	$f_{rv} = V / (n A_b)$	= 19.4	[ksi]	
Resistance factor-LRFD	ϕ = 0.75			AISC 15^{th} J3.7
Modified nominal tensile stress	$F'_{nt} = 1.3 F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_{rv} \le F_{nt}$	= **74.0**	[ksi]	AISC 15^{th} Eq J3-3a
Bolt norminal tensile strength	$r_n = F'_{nt} A_b$	= 32.7	[kips]	AISC 15^{th} Eq J3-1
Resistance factor-LRFD	ϕ = 0.75			AISC 15^{th} J3.6
Single bolt tensile capacity	ϕr_n =	= **24.5**	[kips]	

Single Bolt Tensile Capacity After Considering Prying

Column flange as tee	b_f = 7.992 [in]	bolt gage g = 3.500	[in]	
	web t_w = 0.315 [in]			
Dist from bolt center to flange edge	$a_{cf} = 0.5 (b_f - g)$	= 2.246	[in]	
End plate	width w = 6.250 [in]	bolt gage g = 3.500	[in]	
Dist from bolt center to plate edge	$a_{pl} = 0.5 (w - g)$	= 1.375	[in]	
Dist from bolt center to plate edge	$a = \min (a_{cf} , a_{pl})$	= 1.375	[in]	
	$a' = a + 0.5 d_b \le (1.25 b + 0.5 d_b)$	= 1.750	[in]	AISC 15^{th} Eq 9-23
Bolt hole diameter	bolt dia d_b = 0.750 [in]	bolt hole dia d_h = 0.813	[in]	AISC 15^{th} B4.3b
Dist from bolt center to face of web	$b = 0.5(g - t_w)$	= 1.593	[in]	
	$b' = b - 0.5 d_b$	= 1.218	[in]	AISC 15^{th} Eq 9-18
Bolt pitch spacing	s_v = 3.500 [in]			
Bolt tributary length	$p = s_v \quad p \le 2b$ and $p \le s_v$	= 3.185	[in]	AISC 15^{th} Page 9-12
	$\rho = b' / a'$	= 0.696		AISC 15^{th} Eq 9-22
	$\delta = 1 - d_h / p$	= 0.745		AISC 15^{th} Eq 9-20
Tensile capacity per bolt before considering prying	B = from calc shown in above section	= 24.5	[kips]	
Resistance factor-LRFD	ϕ = 0.90			AISC 15^{th} Page 9-12
Column flange thickness	t = 0.531 [in]	tensile F_u = 65.0	[ksi]	
Plate thickness req'd to develop bolt tensile capacity without prying	$t_c = (\frac{4 B b'}{\phi p F_u})^{0.5}$	= 0.801	[in]	AISC 15^{th} Eq 9-26a
	$\alpha' = \frac{1}{\delta (1 + \rho)} [(\frac{t_c}{t})^2 - 1]$	= 1.008		AISC 15^{th} Eq 9-28
when $\alpha' > 1$	$Q = (\frac{t}{t_c})^2 (1 + \delta)$	= 0.768		AISC 15^{th} Eq 9-27
Bolt tensile force per bolt in demand	T = from calc shown below	= **5.5**	[kips]	
Tensile strength per bolt after considering prying	$\phi r_n = B \times Q$	= **18.8**	[kips]	AISC 15^{th} Eq 9-27
	ratio = **0.29**	> T	OK	

Calculate Max Single Bolt Tensile Load

Bolt group force	axial P = 21.9 [kips]			
Bolt number	Bolt Row n_h = 2	Bolt Col n_v = 2		
Bolt tensile force per bolt	$T = P / (n_v n_h)$	= **5.5**	[kips]	

Gusset Plate to End Plate Weld Strength ratio = 5.51 / 13.34 = **0.41** **PASS**

Weld Group Forces

shear V = 34.2 [kips] axial P = -21.9 [kips] in tension

Gusset-end plate fillet weld length L = weld length tributary to bolt group = **7.375** [in]

Combined Weld Stress

Weld stress from axial force	$f_a = P / L$	= -2.97	[kip/in]	in tension
Weld stress from shear force	$f_v = V / L$	= 4.64	[kip/in]	
Weld stress combined - max	$f_{max} = (f_a^2 + f_v^2)^{0.5}$	= **5.507**	[kip/in]	AISC 15th Eq 8-11
Weld stress load angle	$\theta = \tan^{-1} (\frac{f_a}{f_v})$	= 32.6	[°]	

Fillet Weld Strength Calc

Fillet weld leg size	$w = 1/4$ [in]	load angle θ = 32.6	[°]	
Electrode strength	F_{EXX} = 70.0 [ksi]	strength coeff C_1 = 1.00		AISC 15th Table 8-3
Number of weld line	n = 2 for double fillet			
Load angle coefficient	$C_2 = (1 + 0.5 \sin^{1.5} \theta)$	= 1.20		AISC 15th Page 8-9
Fillet weld shear strength	$R_{n\text{-}w} = 0.6 (C_1 \times 70 \text{ ksi}) 0.707 \text{ w n } C_2$	= 17.79	[kip/in]	AISC 15th Eq 8-1

Base metal - gusset plate thickness t = 0.500 [in] tensile F_u = 65.0 [ksi]

Base metal - gusset plate is in shear, <u>shear</u> rupture as per AISC 15th Eq J4-4 is checked AISC 15th J2.4

Base metal shear rupture	$R_{n\text{-}b} = 0.6 F_u t$	= 19.50	[kip/in]	AISC 15th Eq J4-4
Double fillet linear shear strength	$R_n = \min (R_{n\text{-}w} , R_{n\text{-}b})$	= **17.787**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq 8-1
	ϕR_n =	= **13.340**	[kip/in]	
	ratio = **0.41**	> f_{max}	OK	

Column Web Local Yielding ratio = 21.9 / 203.3 = **0.11** **PASS**

Concentrated force on column	P_u =	= **21.9**	[kips]	
Column section	d = 9.921 [in]	t_f = 0.531	[in]	
	t_w = 0.315 [in]	k = 1.181	[in]	
	yield F_y = 50.0 [ksi]			
Length of bearing	l_b = end plate length	= 7.000	[in]	
Column web local yielding strength	$R_n = F_y t_w (5 k + l_b)$	= 203.3	[kips]	AISC 15th Eq J10-2
Resistance factor-LRFD	ϕ = 1.00			
	ϕR_n =	= **203.3**	[kips]	
	ratio = **0.11**	> P_u	OK	

Column Flange Local Bending ratio = 21.9 / 79.3 = **0.28** **PASS**

Concentrated force from gusset	P_u =	= **21.9**	[kips]	
Column w section	t_f = 0.531 [in]	yield F_y = 50.0	[ksi]	
Column flange local bending strength	$R_n = 6.25 F_y t_f^2$	= 88.1	[kips]	AISC 15th Eq J10-1
Resistance factor-LRFD	ϕ = 0.90			AISC 15th J10.1
	ϕR_n =	= **79.3**	[kips]	
	ratio = **0.28**	> P_u	OK	

Brace Force Load Case 2 Gusset plate t=0.500 P =76.5 kips (C) ratio = **0.48** **PASS**

Gusset Plate - Shear Yielding			ratio = 34.2 / 212.4	= **0.16**	**PASS**
Calculate gusset or stiff plate length outside beam flange and count it as beam web extension to resist shear					
Beam sect depth W200x36	d_b = 7.913	[in]			
Bolt pitch & edge distance	d_1 = 2.500	[in]	e_v = 1.375	[in]	
Beam web extension outside beam flange	$L_e = 2(d_1 + e_v)$ - 2x0.75 in clip		= 6.250	[in]	
Total beam web depth to resist shear	$L = d_b + L_e$		= **14.163**	[in]	
Plate Shear Yielding Check					
Plate size	width b_p = 14.163	[in]	thickness t_p = 0.500	[in]	
Plate yield strength	F_y = 50.0	[ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$		= 7.082	[in^2]	
Shear force required	V_u =		= **34.2**	[kips]	
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$		= 212.4	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00				AISC 15th Eq J4-3
	ϕR_n =		= **212.4**	[kips]	
	ratio = **0.16**		> V_u	OK	

Gusset Plate - Shear Rupture			ratio = 34.2 / 207.1	= **0.17**	**PASS**
Calculate gusset or stiff plate length outside beam flange and count it as beam web extension to resist shear					
Beam sect depth W200x36	d_b = 7.913	[in]			
Bolt pitch & edge distance	d_1 = 2.500	[in]	e_v = 1.375	[in]	
Beam web extension outside beam flange	$L_e = 2(d_1 + e_v)$ - 2x0.75 in clip		= 6.250	[in]	
Total beam web depth to resist shear	$L = d_b + L_e$		= **14.163**	[in]	
Plate Shear Rupture Check					
Plate size	width b_p = 14.163	[in]	thickness t_p = 0.500	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate net area in shear	$A_{nv} = b_p t_p$		= 7.082	[in^2]	
Shear force in demand	V_u =		= **34.2**	[kips]	
Plate shear rupture strength	$R_n = 0.6 F_u A_{nv}$		= 276.2	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J4-4
	ϕR_n =		= **207.1**	[kips]	
	ratio = **0.17**		> V_u	OK	

End Plate - Shear Yield			ratio = 17.1 / 131.3	= **0.13**	**PASS**
Plate Shear Yielding Check					
Plate size	width b_p = 7.000	[in]	thickness t_p = 0.625	[in]	
Plate yield strength	F_y = 50.0	[ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$		= 4.375	[in^2]	
Shear force required	V_u =		= **17.1**	[kips]	
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$		= 131.3	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00				AISC 15th Eq J4-3
	ϕR_n =		= **131.3**	[kips]	
	ratio = **0.13**		> V_u	OK	

End Plate - Shear Rupture ratio = 17.1 / 96.0 = **0.18** **PASS**

Plate Shear Rupture Check

Bolt hole diameter	bolt dia $d_b = {}^3/_4$	[in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 15th B4.3b
Number of bolt	n = 2				
Plate size	width b_p = 7.000	[in]	thickness t_p = 0.625	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate net area in shear	$A_{nv} = (b_p - n\, d_h)\, t_p$		= 3.281	[in^2]	
Shear force required	V_u =		= **17.1**	[kips]	
Plate shear rupture strength	$R_n = 0.6\, F_u A_{nv}$		= 128.0	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J4-4
	$\phi\, R_n$ =		= **96.0**	[kips]	
	ratio = **0.18**		> V_u	OK	

End Plate - Block Shear - Center Strip ratio = 34.2 / 223.9 = **0.15** **PASS**

Plate Block Shear - Center Strip

Bolt hole diameter	bolt dia $d_b = {}^3/_4$	[in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 15th B4.3b
Plate thickness	t_p = 0.625	[in]			
Plate strength	F_y = 50.0	[ksi]	F_u = 65.0	[ksi]	
Bolt no in ver & hor dir	n_v = 2		n_h = 2		
Bolt spacing in ver & hor dir	s_v = 3.500	[in]	s_h = 3.500	[in]	
Bolt edge dist in ver & hor dir	e_v = 1.375	[in]	e_h = 1.750	[in]	
Gross area subject to shear	$A_{gv} = [\,(n_h - 1)\, s_h + e_h\,]\, t_p \times 2$		= 6.563	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5]\, d_h t_p \times 2$		= 4.922	[in^2]	
Net area subject to tension when sheared out by center strip	$A_{nt} = (n_v - 1)\, (s_v - d_h)\, t_p$		= 1.641	[in^2]	
Block shear strength required	V_u =		= **34.2**	[kips]	
Uniform tension stress factor	U_{bs} = 1.00				AISC 15th Fig C-J4.2
Bolt shear resistance provided	$R_n = \min\,(0.6F_u A_{nv}, 0.6F_y A_{gv}) + U_{bs} F_u A_{nt}$		= 298.6	[kips]	AISC 15th Eq J4-5
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J4-5
	$\phi\, R_n$ =		= **223.9**	[kips]	
	ratio = **0.15**		> V_u	OK	

End Plate - Block Shear - 2-Side Strip		ratio = 34.2 / 201.1	= **0.17**	**PASS**

Plate Block Shear - 2 Side Strips

Bolt hole diameter	bolt dia d_b = $^3/_4$	[in]	bolt hole dia d_h = $^7/_8$	[in]	AISC 15th B4.3b
Plate thickness	t_p = 0.625	[in]			
Plate strength	F_y = 50.0	[ksi]	F_u = 65.0	[ksi]	
Bolt no in ver & hor dir	n_v = 2		n_h = 2		
Bolt spacing in ver & hor dir	s_v = 3.500	[in]	s_h = 3.500	[in]	
Bolt edge dist in ver & hor dir	e_v = 1.375	[in]	e_h = 1.750	[in]	

Gross area subject to shear	$A_{gv} = [\ (n_h - 1)\ s_h + e_h\ ]\ t_p \times 2$	= 6.563	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5]\ d_h\ t_p \times 2$	= 4.922	[in^2]	
Net area subject to tension				
when sheared out by 2 side strips	$A_{nt} = (\ e_v - 0.5\ d_h\ )\ t_p \times 2$	= 1.172	[in^2]	

Block shear strength required	V_u =	= **34.2**	[kips]	
Uniform tension stress factor	U_{bs} = 1.00			AISC 15th Fig C-J4.2
Bolt shear resistance provided	$R_n = \min\ (0.6F_u A_{nv}, 0.6F_y A_{gv}) + U_{bs} F_u A_{nt}$	= 268.1	[kips]	AISC 15th Eq J4-5
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-5
	$\phi\ R_n$ =	= **201.1**	[kips]	
	ratio = **0.17**	> V_u	OK	

End Plate - Bolt Bearing on End Plate		ratio = 34.2 / 71.6	= **0.48**	**PASS**
Single Bolt Shear Strength				
Bolt shear stress	bolt grade = A325-N	F_{nv} = 54.0	[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.750 [in]	bolt area A_b = 0.442	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$	= 23.9	[kips]	AISC 15th 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 15th Table J3.3
Bolt spacing & edge distance	spacing L_s = 3.500 [in]	edge distance L_e = 1.750	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate thickness	t = 0.625 [in]			
Interior Bolt				
Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.688	[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 15th Eq J3-6b
	= 163.8 ≤ 91.4	= 91.4	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$	= 23.9	[kips]	
Edge Bolt				
Bolt hole edge clear distance	$L_c = L_e - d_h / 2$	= 1.344	[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 15th Eq J3-6b
	= 81.9 ≤ 91.4	= 81.9	[kips]	
Bolt strength at edge	$R_{n\text{-}ed} = \min(R_{n\text{-}t\&b\text{-}ed}, R_{n\text{-}bolt})$	= 23.9	[kips]	
Number of bolt	interior n_{in} = 2	edge n_{ed} = 2		
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-}in} + n_{ed} R_{n\text{-}ed}$	= 95.4	[kips]	
Required shear strength	V_u =	= **34.2**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th J3.10
	ϕR_n =	= **71.6**	[kips]	
	ratio = **0.48**	> V_u	OK	

End Plate / Column - Bolt Shear		ratio = 34.2 / 71.6	= **0.48**	**PASS**
Bolt shear stress	bolt grade = A325-N	F_{nv} = 54.0	[ksi]	AISC 15th 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		
Bolt group eccentricity coefficient	C_{ec} =	= 1.000		
Required shear strength	V_u =	= **34.2**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 95.4	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J3-1
	ϕR_n =	= **71.6**	[kips]	
	ratio = **0.48**	> V_u	OK	

End Plate / Column - Bolt Bearing on Column ratio = 34.2 / 71.6 = **0.48** **PASS**

Single Bolt Shear Strength

Bolt shear stress	bolt grade = A325-N	F_{nv} = 54.0	[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.750 [in]	bolt area A_b = 0.442	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$	= 23.9	[kips]	AISC 15th 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 15th Table J3.3
Bolt spacing	spacing L_s = 3.500 [in]			
Plate tensile strength	F_u = 65.0 [ksi]			
Plate thickness	t = 0.531 [in]			

Interior Bolt

Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.688	[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 15th Eq J3-6b
	= 139.1 ≤ 77.7	= 77.7	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$	= 23.9	[kips]	

Number of bolt	interior n_{in} = 4			
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-}in}$	= 95.4	[kips]	
Required shear strength	V_u =	= **34.2**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th J3.10
	ϕR_n =	= **71.6**	[kips]	
	ratio = **0.48**	> V_u	OK	

Gusset Plate to End Plate Weld Strength			ratio = 4.64 / 11.14	= **0.42**	**PASS**
Weld Group Forces					
	shear V = 34.2 [kips]		axial P = 21.9	[kips]	in compression
Gusset-end plate fillet weld length	L = weld length tributary to bolt group		= **7.375**	[in]	
Combined Weld Stress					
Weld stress from axial force	$f_a = P / L$		= 0.00	[kip/in]	in compression
Weld stress from shear force	$f_v = V / L$		= 4.64	[kip/in]	
Weld stress combined - max	$f_{max} = f_v$		= **4.637**	[kip/in]	AISC 15th Eq 8-11
Weld stress load angle	θ =		= 0.0	[°]	
Fillet Weld Strength Calc					
Fillet weld leg size	$w = ^1/_4$ [in]		load angle θ = 0.0	[°]	
Electrode strength	F_{EXX} = 70.0 [ksi]		strength coeff C_1 = 1.00		AISC 15th 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 15th Page 8-9
Fillet weld shear strength	$R_{n-w} = 0.6 (C_1 \times 70 \text{ ksi}) 0.707\ w\ n\ C_2$		= 14.85	[kip/in]	AISC 15th Eq 8-1
Base metal - gusset plate	thickness t = 0.500 [in]		tensile F_u = 65.0	[ksi]	
Base metal - gusset plate is in shear, shear rupture as per AISC 15th Eq J4-4 is checked					AISC 15th J2.4
Base metal shear rupture	$R_{n-b} = 0.6\ F_u\ t$		= 19.50	[kip/in]	AISC 15th Eq J4-4
Double fillet linear shear strength	$R_n = \min (R_{n-w} , R_{n-b})$		= **14.847**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq 8-1
	$\phi\ R_n$ =		= **11.135**	[kip/in]	
	ratio = **0.42**		> f_{max}	OK	

Column Web Local Yielding			ratio = 21.9 / 203.3	= **0.11**	**PASS**
Concentrated force on column	P_u =		= **21.9**	[kips]	
Column section	d = 9.921 [in]		t_f = 0.531	[in]	
	t_w = 0.315 [in]		k = 1.181	[in]	
	yield F_y = 50.0 [ksi]				
Length of bearing	l_b = end plate length		= 7.000	[in]	
Column web local yielding strength	$R_n = F_y\ t_w (5\ k + l_b)$		= 203.3	[kips]	AISC 15th Eq J10-2
Resistance factor-LRFD	ϕ = 1.00				
	$\phi\ R_n$ =		= **203.3**	[kips]	
	ratio = **0.11**		> P_u	OK	

Column Web Local Crippling		ratio = 21.9 / 183.1	= **0.12**	**PASS**
Concentrated force on column	P_u =	= **21.9**	[kips]	
Column section	d = 9.921 [in]	t_f = 0.531	[in]	
	t_w = 0.315 [in]	k = 1.181	[in]	
	yield F_y = 50.0 [ksi]	E = 29000.0	[ksi]	
Length of bearing	l_b = end plate length	= 7.000	[in]	
Column web local crippling strength	$R_n = 0.8\, t_w^2\, [1+3\frac{l_b}{d}(\frac{t_w}{t_f})^{1.5}] \times (\frac{E F_y t_f}{t_w})^{0.5}$	= 244.1	[kips]	AISC 15th Eq J10-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th J10.3
	ϕR_n =	= **183.1**	[kips]	
	ratio = **0.12**	> P_u	OK	

Gusset to Beam	Direct Weld Connection	Code=AISC 360-16 LRFD

Result Summary	geometries & weld limitations = **PASS**	limit states max ratio = **0.27**	**PASS**

Brace Weld Limitation Checks - Gusset to Beam **PASS**

Min Fillet Weld Size

Thinner part joined thickness	$t =$	= 0.402	[in]	
Min fillet weld size allowed	$w_{min} =$	= **0.188**	[in]	AISC 15th Table J2.4
Fillet weld size provided	$w =$	= **0.313**	[in]	
		$\geq w_{min}$	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 15th J2.2b
Min fillet weld length	$L =$	= **12.442**	[in]	
		$\geq L_{min}$	OK	

Brace Force Load Case 1 Gusset plate t=0.500 P =-76.5 kips (T) ratio = **0.27** **PASS**

Gusset Plate - Shear Yielding ratio = 34.5 / 186.6 = **0.18** **PASS**

Plate Shear Yielding Check

Plate size	width b_p = 12.442 [in]	thickness t_p = 0.500	[in]	
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$	= 6.221	[in^2]	
Shear force required	$V_u =$	= **34.5**	[kips]	
Plate shear yielding strength	$R_n = 0.6\, F_y A_{gv}$	= 186.6	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	$\phi R_n =$	= **186.6**	[kips]	
	ratio = **0.18**	$> V_u$	OK	

Gusset Plate - Shear Rupture ratio = 34.5 / 182.0 = **0.19** **PASS**

Plate Shear Rupture Check

Plate size	width b_p = 12.442 [in]	thickness t_p = 0.500	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate net area in shear	$A_{nv} = b_p t_p$	= 6.221	[in^2]	
Shear force in demand	$V_u =$	= **34.5**	[kips]	
Plate shear rupture strength	$R_n = 0.6\, F_u A_{nv}$	= 242.6	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-4
	$\phi R_n =$	= **182.0**	[kips]	
	ratio = **0.19**	$> V_u$	OK	

Gusset Plate - Axial Tensile Yield		ratio = 17.5 / 279.9	= **0.06**	**PASS**
Plate Tensile Yielding Check				
Plate size	width b_p = 12.442 [in]	thickness t_p = 0.500	[in]	
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_g = b_p t_p$	= 6.221	[in^2]	
Tensile force required	P_u =	= **17.5**	[kips]	
Plate tensile yielding strength	$R_n = F_y A_g$	= 311.1	[kips]	AISC 15th Eq J4-1
Resistance factor-LRFD	ϕ = 0.90			AISC 15th Eq J4-1
	ϕR_n =	= **279.9**	[kips]	
	ratio = **0.06**	> P_u	OK	

Gusset Plate - Axial Tensile Rupture		ratio = 17.5 / 303.3	= **0.06**	**PASS**
Plate Tensile Rupture Check				
Plate size	width b_p = 12.442 [in]	thickness t_p = 0.500	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate net area in tension	$A_{nt} = b_p t_p$	= 6.221	[in^2]	
Tensile force required	P_u =	= **17.5**	[kips]	
Plate tensile rupture strength	$R_n = F_u A_{nt}$	= 404.4	[kips]	AISC 15th Eq J4-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-2
	ϕR_n =	= **303.3**	[kips]	AISC 15th Eq J4-2
	ratio = **0.06**	> P_u	OK	

Gusset Plate - Flexural Yield Interact		ratio =	= **0.04**	**PASS**
Gusset plate	width b_p = 12.442 [in]	thick t_p = 0.500	[in]	
	yield F_y = 50.0 [ksi]			
Shear plate - gross area	$A_g = b_p \times t_p$	= 6.221	[in^2]	
Shear plate - plastic modulus	$Z_p = (b_p \times t_p^2) / 4$	= 19.35	[in^3]	
Flexural strength available	$M_c = \phi F_y Z_p$ ϕ=0.90	= 72.56	[kip-ft]	
Flexural strength required	M_r = from gusset interface forces calc	= 0.32	[kip-ft]	
Axial strength available	P_c = from axial tensile yield check	= 279.9	[kips]	
Axial strength required	P_r = from gusset interface forces calc	= -17.5	[kips]	
Shear strength available	V_c = from shear yielding check	= 186.6	[kips]	
Shear strength required	V_r = from gusset interface forces calc	= 34.5	[kips]	
Flexural yield interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right)^2$	= **0.04**		AISC 15th Eq 10-5
		< 1.0	OK	

Gusset Plate - Flexural Rupture Interact		ratio =	= **0.04**	**PASS**	
Gusset plate	width b_p = 12.442 [in]		thick t_p = 0.500	[in]	
	tensile F_u = 65.0 [ksi]				
Net area of plate	$A_n = b_p \times t_p$		= 6.221	[in^2]	
Plastic modulus of net section	$Z_{net} = (b_p \times t_p^2) / 4$		= 19.35	[in^3]	
Flexural strength available	$M_c = \phi F_u Z_{net}$ ϕ=0.75		= 78.61	[kip-ft]	
Flexural strength required	M_r = from gusset interface forces calc		= 0.32	[kip-ft]	
Axial strength available	P_c = from axial tensile rupture check		= 303.3	[kips]	
Axial strength required	P_r = from gusset interface forces calc		= -17.5	[kips]	
Shear strength available	V_c = from shear rupture check		= 182.0	[kips]	
Shear strength required	V_r = from gusset interface forces calc		= 34.5	[kips]	
Flexural rupture interaction	ratio $= \left(\frac{V_r}{V_c} \right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c} \right)^2$		= **0.04**		AISC 15th Eq 10-5
			< 1.0	OK	

Gusset to Beam Weld Strength		ratio = 3.18 / 11.70	= **0.27**	**PASS**	
Gusset to Beam Interface - Forces					
	shear H_b = 34.5 [kips]		axial V_b = -17.5	[kips]	in tension
	moment M_b = 0.32 [kip-ft]				
Gusset-beam fillet weld length	L_w =		= 12.442	[in]	
Gusset to Beam Interface - Combined Weld Stress					
Weld stress from axial force	$f_a = V_b / L_{wb}$		= -1.41	[kip/in]	in tension
Weld stress from shear force	$f_v = H_b / L_{wb}$		= 2.77	[kip/in]	
Weld stress from moment force	$f_b = \frac{M}{L^2 / 6}$		= 0.15	[kip/in]	
Weld stress combined - max	$f_{max} = [(f_a - f_b)^2 + f_v^2]^{0.5}$		= **3.179**	[kip/in]	AISC 15th Eq 8-11
Weld resultant load angle	$\theta = \tan^{-1} [(f_b - f_a) / f_v]$		= 29.3	[°]	
Fillet Weld Strength Calc					
Fillet weld leg size	w = $^5/_{16}$ [in]		load angle θ = 29.3	[°]	
Electrode strength	F_{EXX} = 70.0 [ksi]		strength coeff C_1 = 1.00		AISC 15th Table 8-3
Number of weld line	n = 2 for double fillet				
Load angle coefficient	$C_2 = (1 + 0.5 \sin^{1.5} \theta)$		= 1.17		AISC 15th Page 8-9
Fillet weld shear strength	$R_{n-w} = 0.6 (C_1 \times 70$ ksi$) 0.707\ w\ n\ C_2$		= 21.73	[kip/in]	AISC 15th Eq 8-1
Base metal - gusset plate	thickness t = 0.500 [in]		tensile F_u = 65.0	[ksi]	
Base metal - gusset plate is in shear, shear rupture as per AISC 15th Eq J4-4 is checked					AISC 15th J2.4
Base metal shear rupture	$R_{n-b} = 0.6 F_u t$		= 19.50	[kip/in]	AISC 15th Eq J4-4
Double fillet linear shear strength	$R_n = \min (R_{n-w} , R_{n-b})$		= **19.500**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq 8-1
	ϕR_n =		= 14.63	[kip/in]	
When gusset plate is directly welded to beam or column, apply 1.25 ductility factor to allow adequate force redistribution in the weld group					AISC 15th Page 13-11
Weld strength used for design after applying ductility factor	$\phi R_n = \phi R_n \times (1/1.25)$		= **11.70**	[kip/in]	
	ratio = **0.27**		> f_{max}	OK	

Beam Web Local Yielding		ratio = 18.7 / 179.4	= **0.10**	**PASS**

Gusset Edge Equivalent Normal Force

Refer to AISC DG29 Fig. B-1 for formula below to calculate gusset edge equivalent normal force

Gusset edge axial force	N =	= -17.5	[kips]	
Gusset edge moment force	M =	= 0.32	[kip-ft]	
Gusset edge interface length	L =	= 12.442	[in]	
Gusset edge equivalent normal force	$N_e = N - \frac{4\,M}{L}$	= -18.7	[kips]	AISC DG29 Fig B-1

Concentrated force from gusset	P_u =	= **18.7**	[kips]	
Beam section	d = 7.913 [in]	t_f = 0.402	[in]	
	t_w = 0.244 [in]	k = 0.906	[in]	
	yield F_y = 50.0 [ksi]			

Length of bearing	l_b = Gusset/Beam interface length	= 12.442	[in]	
Gusset plate corner clip	clip = from user input	= 0.750	[in]	
Distance from normal force applied point to member end	$l_N = 0.5\, l_b$ + clip	= 6.971	[in]	
	when $l_N \le d$, use AISC 15th Eq J10-3			AISC 15th Eq J10-3
Beam web local yielding strength	$R_n = F_y t_w (2.5\,k + l_b)$	= 179.4	[kips]	AISC 15th Eq J10-3
Resistance factor-LRFD	ϕ = 1.00			
	$\phi\, R_n$ =	= **179.4**	[kips]	
	ratio = **0.10**	> P_u	OK	

Brace Force Load Case 2	Gusset plate t=0.500	P =76.5 kips (C)	ratio = **0.25**	**PASS**

Gusset Plate - Shear Yielding		ratio = 34.5 / 186.6	= **0.18**	**PASS**

Plate Shear Yielding Check

Plate size	width b_p = 12.442 [in]	thickness t_p = 0.500	[in]	
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$	= 6.221	[in^2]	
Shear force required	V_u =	= **34.5**	[kips]	
Plate shear yielding strength	$R_n = 0.6\, F_y A_{gv}$	= 186.6	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	$\phi\, R_n$ =	= **186.6**	[kips]	
	ratio = **0.18**	> V_u	OK	

Gusset Plate - Shear Rupture		ratio = 34.5 / 182.0	= **0.19**	**PASS**
Plate Shear Rupture Check				
Plate size	width b_p = 12.442 [in]	thickness t_p = 0.500	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate net area in shear	$A_{nv} = b_p t_p$	= 6.221	[in^2]	
Shear force in demand	V_u =	= **34.5**	[kips]	
Plate shear rupture strength	$R_n = 0.6\, F_u A_{nv}$	= 242.6	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J4-4
	$\phi\, R_n$ =	= **182.0**	[kips]	
	ratio = **0.19**	> V_u	OK	

Gusset Plate - Axial Yield		ratio = 17.5 / 279.9	= **0.06**	**PASS**
Plate Tensile Yielding Check				
Plate size	width b_p = 12.442 [in]	thickness t_p = 0.500	[in]	
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_g = b_p t_p$	= 6.221	[in^2]	
Tensile force required	P_u =	= **17.5**	[kips]	
Plate tensile yielding strength	$R_n = F_y A_g$	= 311.1	[kips]	AISC 15th Eq J4-1
Resistance factor-LRFD	ϕ = 0.90			AISC 15th Eq J4-1
	$\phi\, R_n$ =	= **279.9**	[kips]	
	ratio = **0.06**	> P_u	OK	

Gusset Plate - Flexural Yield Interact		ratio =	= **0.04**	**PASS**
Gusset plate	width b_p = 12.442 [in]	thick t_p = 0.500	[in]	
	yield F_y = 50.0 [ksi]			
Shear plate - gross area	$A_g = b_p \times t_p$	= 6.221	[in^2]	
Shear plate - plastic modulus	$Z_p = (b_p \times t_p^2) / 4$	= 19.35	[in^3]	
Flexural strength available	$M_c = \phi\, F_y Z_p$ ϕ=0.90	= 72.56	[kip-ft]	
Flexural strength required	M_r = from gusset interface forces calc	= 0.00	[kip-ft]	
Axial strength available	P_c = from axial tensile yield check	= 279.9	[kips]	
Axial strength required	P_r = from gusset interface forces calc	= 17.5	[kips]	
Shear strength available	V_c = from shear yielding check	= 186.6	[kips]	
Shear strength required	V_r = from gusset interface forces calc	= 34.5	[kips]	
Flexural yield interaction	ratio $= \left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right)^2$	= **0.04**		AISC 15th Eq 10-5
		< 1.0	OK	

Gusset Plate - Flexural Rupture Interact		ratio =	= **0.04**	**PASS**
Gusset plate	width b_p = 12.442 [in]	thick t_p = 0.500 [in]		
	tensile F_u = 65.0 [ksi]			
Net area of plate	$A_n = b_p \times t_p$	= 6.221 [in^2]		
Plastic modulus of net section	$Z_{net} = (b_p \times t_p^2) / 4$	= 19.35 [in^3]		
Flexural strength available	$M_c = \phi F_u Z_{net}$ ϕ=0.75	= 78.61 [kip-ft]		
Flexural strength required	M_r = from gusset interface forces calc	= 0.00 [kip-ft]		
Shear strength available	V_c = from shear rupture check	= 182.0 [kips]		
Shear strength required	V_r = from gusset interface forces calc	= 34.5 [kips]		
Flexural rupture interaction	ratio = $\left(\frac{V_r}{V_c} \right)^2 + \left(\frac{M_r}{M_c} \right)^2$	= **0.04**	AISC 15th Eq 10-5	
		< 1.0	OK	

Gusset to Beam Weld Strength		ratio = 2.77 / 11.14	= **0.25**	**PASS**
Gusset to Beam Interface - Forces				
	shear H_b = 34.5 [kips]	axial V_b = 17.5 [kips]	in compression	
	moment M_b = 0.00 [kip-ft]			
Gusset-beam fillet weld length	L_w =	= 12.442 [in]		
Gusset to Beam Interface - Combined Weld Stress				
Weld stress from axial force	$f_a = V_b / L_{wb}$	= 0.00 [kip/in]	in compression	
Weld stress from shear force	$f_v = H_b / L_{wb}$	= 2.77 [kip/in]		
Weld stress from moment force	$f_b = \frac{M}{L^2 / 6}$	= 0.00 [kip/in]		
Weld stress combined - max	$f_{max} = f_v$	= **2.773** [kip/in]	AISC 15th Eq 8-11	
Weld resultant load angle	θ = weld only has shear component	= 0.0 [°]		
Fillet Weld Strength Calc				
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 15th 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 15th Page 8-9	
Fillet weld shear strength	$R_{n\text{-}w} = 0.6 (C_1 \times 70 \text{ ksi}) 0.707 \text{ w n } C_2$	= 18.56 [kip/in]	AISC 15th Eq 8-1	
Base metal - gusset plate	thickness t = 0.500 [in]	tensile F_u = 65.0 [ksi]		
Base metal - gusset plate is in shear, shear rupture as per AISC 15th Eq J4-4 is checked			AISC 15th J2.4	
Base metal shear rupture	$R_{n\text{-}b} = 0.6 F_u t$	= 19.50 [kip/in]	AISC 15th Eq J4-4	
Double fillet linear shear strength	$R_n = \min (R_{n\text{-}w} , R_{n\text{-}b})$	= **18.559** [kip/in]	AISC 15th Eq 9-2	
Resistance factor-LRFD	ϕ = 0.75		AISC 15th Eq 8-1	
	ϕR_n =	= 13.92 [kip/in]		
When gusset plate is directly welded to beam or column, apply 1.25 ductility factor to allow adequate force redistribution in the weld group			AISC 15th Page 13-11	
Weld strength used for design after applying ductility factor	$\phi R_n = \phi R_n \times (1/1.25)$	= **11.14** [kip/in]		
	ratio = **0.25**	> f_{max}	OK	

Beam Web Local Yielding ratio = 17.5 / 179.4 = **0.10** **PASS**

Concentrated force from gusset	P_u =	= **17.5**	[kips]	
Beam section	d = 7.913 [in]	t_f = 0.402	[in]	
	t_w = 0.244 [in]	k = 0.906	[in]	
	yield F_y = 50.0 [ksi]			
Length of bearing	l_b = Gusset/Beam interface length	= 12.442	[in]	
Gusset plate corner clip	clip = from user input	= 0.750	[in]	
Distance from normal force applied point to member end	$l_N = 0.5\ l_b$ + clip	= 6.971	[in]	
	when $l_N \le d$, use AISC 15th Eq J10-3			AISC 15th Eq J10-3
Beam web local yielding strength	$R_n = F_y t_w (2.5\ k + l_b)$	= 179.4	[kips]	AISC 15th Eq J10-3
Resistance factor-LRFD	ϕ = 1.00			
	$\phi\ R_n$ =	= **179.4**	[kips]	
	ratio = **0.10**	> P_u	OK	

Beam Web Local Crippling ratio = 17.5 / 178.4 = **0.10** **PASS**

Concentrated force from gusset	P_u =	= **17.5**	[kips]	
Beam section	d = 7.913 [in]	t_f = 0.402	[in]	
	t_w = 0.244 [in]	k = 0.906	[in]	
	yield F_y = 50.0 [ksi]	E = 29000.0	[ksi]	
Length of bearing	l_b = Gusset/Beam interface length	= 12.442	[in]	
Gusset plate corner clip	clip = from user input	= 0.750	[in]	
Distance from normal force applied point to member end	$l_N = 0.5\ l_b$ + clip	= 6.971	[in]	
	when $l_N \ge d/2$, use Eq J10-4			AISC 15th Eq J10-4
Beam web local crippling strength	$R_n = 0.8\ t_w^2\ [1+3\frac{l_b}{d}(\frac{t_w}{t_f})^{1.5}]\ x\ (\frac{E\ F_y t_f}{t_w})^{0.5}$	= 237.8	[kips]	AISC 15th Eq J10-4
Resistance factor-LRFD	ϕ = 0.75			AISC 15th J10.3
	ϕR_n =	= **178.4**	[kips]	
	ratio = **0.10**	> P_u	OK	

Beam to Column	End Plate Connection	Code=AISC 360-16 LRFD

Result Summary	geometries & weld limitations = **PASS**	limit states max ratio = **0.99**	**PASS**

Geometry Restriction Check - End Plate to Column Flange — PASS

Min Bolt Edge Distance - End Plate to Column Flange

Bolt diameter	d_b =	= 0.750	[in]	
Min edge distance allowed	$L_{e\text{-}min}$ =	= **1.000**	[in]	AISC 15th Table J3.4
Min edge distance in End Plate to Column Flange	L_e =	= **1.375**	[in]	
		$\geq L_{e\text{-}min}$	OK	

Min Bolt Spacing - End Plate to Column Flange

Bolt diameter	d_b =	= 0.750	[in]	
Min bolt spacing allowed	$L_{s\text{-}min} = 2.667\ d_b$	= **2.000**	[in]	AISC 15th J3.3
Min Bolt spacing in End Plate to Column Flange	L_s =	= **3.500**	[in]	
		$\geq L_{s\text{-}min}$	OK	

Geometry Restriction Check - End Plate-Bolt Gage Clearance — PASS

Bolt Gage Entering Clearance Check - Plate Welded to End Plate

Bolt diameter	d_b = 0.750 [in]	gage g = 3.500	[in]	
Bolt entering clearance	c_3 = from AISC manual Table 7-15	= **0.750**	[in]	AISC 15th Table 7-15
Plate thickness	t = 0.244 [in]	dbl fillet w = 0.250	[in]	
Bolt center clearance distance to fillet toe	c = (g - t - 2 w) / 2	= **1.378**	[in]	
		$\geq c_3$	OK	AISC 15th Table 7-15

Geometry Restriction Check - Column Flange-Bolt Gage Clearance — PASS

Bolt Gage Entering Clearance Check - Bolt on W Shape Flange

Bolt diameter	d_b = 0.750 [in]	gage g = 3.500	[in]	
Bolt entering clearance	c_3 = from AISC manual Table 7-15	= **0.750**	[in]	AISC 15th Table 7-15
W section	t_w = 0.315 [in]	k_1 = 0.748	[in]	
Bolt center clearance distance to fillet toe	$c = (g - 2\ k_1) / 2$	= **1.002**	[in]	
		$\geq c_3$	OK	AISC 15th Table 7-15

Beam Flange Fillet Weld Limitation — PASS

Min Fillet Weld Size

Thinner part joined thickness	t =	= 0.402	[in]	
Min fillet weld size allowed	w_{min} =	= **0.188**	[in]	AISC 15th Table J2.4
Fillet weld size provided	w =	= **0.250**	[in]	
		$\geq w_{min}$	OK	

Min Fillet Weld Length

Fillet weld size provided	w =	= 0.250	[in]	
Min fillet weld length allowed	L_{min} = 4 x w	= **1.000**	[in]	AISC 15th J2.2b
Min fillet weld length	$L = 0.5\ b_f - k_1$	= **2.574**	[in]	
		$\geq L_{min}$	OK	

Beam Web Fillet Weld Limitation					**PASS**
Min Fillet Weld Size					
Thinner part joined thickness	t =	= 0.244	[in]		
Min fillet weld size allowed	w_{min} =	= **0.125**	[in]	AISC 15th Table J2.4	
Fillet weld size provided	w =	= **0.250**	[in]		
		≥ w_{min}	OK		
Min Fillet Weld Length					
Fillet weld size provided	w =	= 0.250	[in]		
Min fillet weld length allowed	L_{min} = 4 x w	= **1.000**	[in]	AISC 15th J2.2b	
Min fillet weld length	L = d - 2 k	= **6.101**	[in]		
		≥ L_{min}	OK		

Brace Force Load Case 1 shear V = 11.3 kips axial P = -34.4 kips (T) ratio = **0.43** **PASS**

Beam Web - Shear Yielding			ratio = 11.3 / 103.7	= **0.11**	**PASS**
Calculate gusset or stiff plate length outside beam flange and count it as beam web extension to resist shear					
Beam sect depth W200x36	d_b = 7.913 [in]				
Bolt pitch & edge distance	d_1 = 2.500 [in]	e_v = 1.375	[in]		
Beam web extension outside beam flange	L_e = 2(d_1 +e_v) - 2x0.75 in clip	= 6.250	[in]		
Total beam web depth to resist shear	L = d_b + L_e	= **14.163**	[in]		
Plate Shear Yielding Check					
Plate size	width b_p = 14.163 [in]	thickness t_p = 0.244	[in]		
Plate yield strength	F_y = 50.0 [ksi]				
Plate gross area in shear	A_{gv} = $b_p t_p$	= 3.456	[in^2]		
Shear force required	V_u =	= **11.3**	[kips]		
Plate shear yielding strength	R_n = 0.6 $F_y A_{gv}$	= 103.7	[kips]	AISC 15th Eq J4-3	
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3	
	ϕ R_n =	= **103.7**	[kips]		
	ratio = **0.11**	> V_u	OK		

Beam Web - Shear Rupture		ratio = 11.3 / 101.1	= **0.11**	**PASS**
Calculate gusset or stiff plate length outside beam flange and count it as beam web extension to resist shear				
Beam sect depth W200x36	d_b = 7.913 [in]			
Bolt pitch & edge distance	d_1 = 2.500 [in]	e_v = 1.375 [in]		
Beam web extension outside beam flange	$L_e = 2(d_1 + e_v)$ - 2x0.75 in clip	= 6.250 [in]		
Total beam web depth to resist shear	$L = d_b + L_e$	= **14.163** [in]		
Plate Shear Rupture Check				
Plate size	width b_p = 14.163 [in]	thickness t_p = 0.244 [in]		
Plate tensile strength	F_u = 65.0 [ksi]			
Plate net area in shear	$A_{nv} = b_p t_p$	= 3.456 [in^2]		
Shear force in demand	V_u =	= **11.3** [kips]		
Plate shear rupture strength	$R_n = 0.6 F_u A_{nv}$	= 134.8 [kips]	AISC 15th Eq J4-4	
Resistance factor-LRFD	ϕ = 0.75		AISC 15th Eq J4-4	
	ϕR_n =	= **101.1** [kips]		
	ratio = **0.11**	> V_u	OK	

Beam - Tensile Yielding		ratio = 34.4 / 318.8	= **0.11**	**PASS**
Gross area subject to tension	A_g =	= 7.084 [in^2]		
Steel yield strength	F_y =	= 50.0 [ksi]		
Tensile force required	P_u =	= **34.4** [kips]		
Tensile yielding strength	$R_n = F_y A_g$	= 354.2 [kips]	AISC 15th Eq D2-1	
Resistance factor-LRFD	ϕ = 0.90		AISC 15th D2 (a)	
	ϕR_n =	= **318.8** [kips]	AISC 15th Eq D2-1	
	ratio = **0.11**	> P_u	OK	

Beam - Tensile Rupture		ratio = 34.4 / 345.3	= **0.10**	**PASS**
W beam section	= W200x36			
W section net area	$A_n = A_g$	= 7.084 [in^2]		
Shear lag factor	U =	= 1.000		
Tensile force required	P_u =	= **34.4** [kips]		
Tensile effective net area	$A_e = A_n U$	= 7.084 [in^2]		
Plate tensile strength	F_u =	= 65.0 [ksi]		
Tensile rupture strength	$R_n = F_u A_e$	= 460.5 [kips]	AISC 15th Eq D2-2	
Resistance factor-LRFD	ϕ = 0.75		AISC 15th D2 (b)	
	ϕR_n =	= **345.3** [kips]	AISC 15th Eq D2-2	
	ratio = **0.10**	> P_u	OK	

End Plate - Shear Yield			ratio = 5.7 / 293.7	= **0.02**	**PASS**
Plate Shear Yielding Check					
Plate size	width b_p = 15.663	[in]	thickness t_p = 0.625	[in]	
Plate yield strength	F_y = 50.0	[ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$		= 9.789	[in^2]	
Shear force required	V_u =		= **5.7**	[kips]	
Plate shear yielding strength	$R_n = 0.6\ F_y A_{gv}$		= 293.7	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00				AISC 15th Eq J4-3
	ϕR_n =		= **293.7**	[kips]	
	ratio = **0.02**		> V_u	OK	

End Plate - Shear Rupture			ratio = 5.7 / 222.4	= **0.03**	**PASS**
Plate Shear Rupture Check					
Bolt hole diameter	bolt dia $d_b = 3/4$	[in]	bolt hole dia $d_h = 7/8$	[in]	AISC 15th B4.3b
Number of bolt	n = 4				
Plate size	width b_p = 15.663	[in]	thickness t_p = 0.625	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate net area in shear	$A_{nv} = (b_p - n\ d_h)\ t_p$		= 7.602	[in^2]	
Shear force required	V_u =		= **5.7**	[kips]	
Plate shear rupture strength	$R_n = 0.6\ F_u A_{nv}$		= 296.5	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J4-4
	ϕR_n =		= **222.4**	[kips]	
	ratio = **0.03**		> V_u	OK	

End Plate - Block Shear - Center Strip			ratio = 11.3 / 402.2	= **0.03**	**PASS**
Plate Block Shear - Center Strip					
Bolt hole diameter	bolt dia $d_b = 3/4$	[in]	bolt hole dia $d_h = 7/8$	[in]	AISC 15th B4.3b
Plate thickness	t_p = 0.625	[in]			
Plate strength	F_y = 50.0	[ksi]	F_u = 65.0	[ksi]	
Bolt no in ver & hor dir	n_v = 2		n_h = 4		
Bolt spacing in ver & hor dir	s_v = 3.500	[in]	s_h = 3.500	[in]	
Bolt edge dist in ver & hor dir	e_v = 1.375	[in]	e_h = 1.375	[in]	
Gross area subject to shear	$A_{gv} = [\ (n_h - 1)\ s_h + e_h]\ t_p \times 2$		= 14.844	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5]\ d_h t_p \times 2$		= 11.016	[in^2]	
Net area subject to tension					
when sheared out by center strip	$A_{nt} = (n_v - 1)\ (s_v - d_h)\ t_p$		= 1.641	[in^2]	
Block shear strength required	V_u =		= **11.3**	[kips]	
Uniform tension stress factor	U_{bs} = 1.00				AISC 15th Fig C-J4.2
Bolt shear resistance provided	$R_n = \min(0.6F_u A_{nv}, 0.6F_y A_{gv}) + U_{bs} F_u A_{nt}$		= 536.3	[kips]	AISC 15th Eq J4-5
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J4-5
	ϕR_n =		= **402.2**	[kips]	
	ratio = **0.03**		> V_u	OK	

End Plate - Block Shear - 2-Side Strip			ratio = 11.3 / 379.3	= **0.03**	**PASS**	
Plate Block Shear - 2 Side Strips						
Bolt hole diameter	bolt dia $d_b = {}^3/_4$	[in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 15th B4.3b	
Plate thickness	$t_p = 0.625$	[in]				
Plate strength	$F_y = 50.0$	[ksi]	$F_u = 65.0$	[ksi]		
Bolt no in ver & hor dir	$n_v = 2$		$n_h = 4$			
Bolt spacing in ver & hor dir	$s_v = 3.500$	[in]	$s_h = 3.500$	[in]		
Bolt edge dist in ver & hor dir	$e_v = 1.375$	[in]	$e_h = 1.375$	[in]		
Gross area subject to shear	$A_{gv} = [\ (n_h - 1)\ s_h + e_h\ ]\ t_p \times 2$		= 14.844	[in^2]		
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5]\ d_h\ t_p \times 2$		= 11.016	[in^2]		
Net area subject to tension						
when sheared out by 2 side strips	$A_{nt} = (\ e_v - 0.5\ d_h\ )\ t_p \times 2$		= 1.172	[in^2]		
Block shear strength required	$V_u =$		= **11.3**	[kips]		
Uniform tension stress factor	$U_{bs} = 1.00$				AISC 15th Fig C-J4.2	
Bolt shear resistance provided	$R_n = \min\ (0.6F_u A_{nv},\ 0.6F_y A_{gv}) + U_{bs} F_u A_{nt}$		= 505.8	[kips]	AISC 15th Eq J4-5	
Resistance factor-LRFD	$\phi = 0.75$				AISC 15th Eq J4-5	
	$\phi R_n =$		= **379.3**	[kips]		
	ratio = **0.03**		> V_u	OK		

End Plate - Bolt Bearing on End Plate		ratio = 11.3 / 143.1	= **0.08**	**PASS**

Single Bolt Shear Strength

Bolt shear stress	bolt grade = A325-N	F_{nv} = 54.0	[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.750 [in]	bolt area A_b = 0.442	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$	= 23.9	[kips]	AISC 15th 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 15th Table J3.3
Bolt spacing & edge distance	spacing L_s = 3.500 [in]	edge distance L_e = 1.375	[in]	
Plate tensile strength	F_u = 65.0 [ksi]			
Plate thickness	t = 0.625 [in]			

Interior Bolt

Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.688	[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 15th Eq J3-6b
	= 163.8 ≤ 91.4	= 91.4	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$	= 23.9	[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 15th Eq J3-6b
	= 59.0 ≤ 91.4	= 59.0	[kips]	
Bolt strength at edge	$R_{n\text{-}ed} = \min(R_{n\text{-}t\&b\text{-}ed}, R_{n\text{-}bolt})$	= 23.9	[kips]	

Number of bolt	interior n_{in} = 6	edge n_{ed} = 2		
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-}in} + n_{ed} R_{n\text{-}ed}$	= 190.9	[kips]	
Required shear strength	V_u =	= **11.3**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th J3.10
	ϕR_n =	= **143.1**	[kips]	
	ratio = **0.08**	> V_u	OK	

End Plate / Column - Bolt Shear		ratio = 11.3 / 143.1	= **0.08**	**PASS**
Bolt A325-N	dia d_b = 0.750 [in]	A_b = 0.442	[in^2]	
Bolt shear stress	grade = A325-N	F_{nv} = 54.0	[ksi]	AISC 15th Table J3.2
Number of bolt carried shear	n_s = 8.0	shear plane m = 1		
Bolt group eccentricity coefficient	C_{ec} =	= 1.000		
Required shear strength	V_u =	= **11.3**	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 190.9	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq J3-1
	ϕR_n =	= **143.1**	[kips]	
	ratio = **0.08**	> V_u	OK	

End Plate / Column - Bolt Bearing on Column		ratio = 11.3 / 143.1	= **0.08**	**PASS**

Single Bolt Shear Strength

Bolt shear stress	bolt grade = A325-N	F_{nv} = 54.0	[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.750 [in]	bolt area A_b = 0.442	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$	= 23.9	[kips]	AISC 15th 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 15th Table J3.3
Bolt spacing	spacing L_s = 3.500 [in]			
Plate tensile strength	F_u = 65.0 [ksi]			
Plate thickness	t = 0.531 [in]			

Interior Bolt

Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.688	[in]	
Bolt tear out/bearing strength	$R_{n\text{-}t\&b\text{-}in} = 1.5\ L_c\ t\ F_u \leq 3.0\ d_b\ t\ m\ F_u$			AISC 15th Eq J3-6b
	= 139.1 ≤ 77.7	= 77.7	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$	= 23.9	[kips]	

Number of bolt	interior n_{in} = 8			
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-}in}$	= 190.9	[kips]	
Required shear strength	V_u =	= **11.3**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th J3.10
	ϕR_n =	= **143.1**	[kips]	
	ratio = **0.08**	> V_u	OK	

Bolt Tensile Prying Action on End Plate ratio = 4.3 / 23.6 = **0.18** **PASS**

Bolt group forces	shear V = 11.3	[kips]	axial P = -34.4	[kips]		

Single Bolt Tensile Capacity Without Considering Prying

Bolt grade A325-N	dia d_b = 0.750	[in]	area A_b = 0.442	[in^2]		
Nominal tensile/shear stress	F_{nt} = 90.0	[ksi]	F_{nv} = 54.0	[ksi]	AISC 15^{th} Table J3.2	
Bolt group shear force	shear V = 11.3	[kips]	no of bolt n = 8			
Shear stress required	$f_{rv} = V / (n A_b)$		= 3.2	[ksi]		
Resistance factor-LRFD	ϕ = 0.75				AISC 15^{th} J3.7	
Modified nominal tensile stress	$F'_{nt} = 1.3 F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_{rv} \le F_{nt}$		= **90.0**	[ksi]	AISC 15^{th} Eq J3-3a	
Bolt norminal tensile strength	$r_n = F'_{nt} A_b$		= 39.8	[kips]	AISC 15^{th} Eq J3-1	
Resistance factor-LRFD	ϕ = 0.75				AISC 15^{th} J3.6	
Single bolt tensile capacity	ϕr_n =		= **29.8**	[kips]		

Single Bolt Tensile Capacity After Considering Prying

End plate	width w = 6.250	[in]	bolt gage g = 3.500	[in]		
	web t_w = 0.244	[in]				
Dist from bolt center to plate edge	a = 0.5 (w - g)		= 1.375	[in]		
	$a' = a + 0.5 d_b \le (1.25 b + 0.5 d_b)$		= 1.750	[in]	AISC 15^{th} Eq 9-23	
Bolt hole diameter	bolt dia d_b = 0.750	[in]	bolt hole dia d_h = 0.813	[in]	AISC 15^{th} B4.3b	
Dist from bolt center to face of web	$b = 0.5(g - t_w)$		= 1.628	[in]		
	$b' = b - 0.5 d_b$		= 1.253	[in]	AISC 15^{th} Eq 9-18	
Bolt pitch spacing	s_v = 3.500	[in]				
Bolt tributary length	$p = s_v$ $p \le 2b$ and $p \le s_v$		= 3.256	[in]	AISC 15^{th} Page 9-12	
	$\rho = b' / a'$		= 0.716		AISC 15^{th} Eq 9-22	
	$\delta = 1 - d_h / p$		= 0.750		AISC 15^{th} Eq 9-20	
Tensile capacity per bolt before considering prying	B = from calc shown in above section		= 29.8	[kips]		
Resistance factor-LRFD	ϕ = 0.90				AISC 15^{th} Page 9-12	
End plate thickness	t = 0.625	[in]	tensile F_u = 65.0	[ksi]		
Plate thickness req'd to develop bolt tensile capacity without prying	$t_c = (\frac{4 B b'}{\phi p F_u})^{0.5}$		= 0.886	[in]	AISC 15^{th} Eq 9-26a	
	$\alpha' = \frac{1}{\delta (1 + \rho)} [(\frac{t_c}{t})^2 - 1]$		= 0.783		AISC 15^{th} Eq 9-28	
when $0 \le \alpha' \le 1$	$Q = (\frac{t}{t_c})^2 (1 + \delta \alpha')$		= 0.790		AISC 15^{th} Eq 9-27	
Bolt tensile force per bolt in demand	T = from calc shown below		= **4.3**	[kips]		
Tensile strength per bolt after considering prying	$\phi r_n = B \times Q$		= **23.6**	[kips]	AISC 15^{th} Eq 9-27	
	ratio = **0.18**		> T	OK		

Calculate Max Single Bolt Tensile Load

Bolt group force	axial P = 34.4	[kips]				
Bolt number	Bolt Row n_h = 2		Bolt Col n_v = 4			
Bolt tensile force per bolt	$T = P / (n_v n_h)$		= **4.3**	[kips]		

Bolt Tensile Prying Action on Column Flange ratio = 4.3 / 18.8 = **0.23** **PASS**

Bolt group forces	shear V = 11.3	[kips]	axial P = -34.4	[kips]		

Single Bolt Tensile Capacity Without Considering Prying

Bolt grade A325-N	dia d_b = 0.750 [in]	area A_b = 0.442	[in^2]	
Nominal tensile/shear stress	F_{nt} = 90.0 [ksi]	F_{nv} = 54.0	[ksi]	AISC 15th Table J3.2
Bolt group shear force	shear V = 11.3 [kips]	no of bolt n = 8		
Shear stress required	$f_{rv} = V / (n A_b)$	= 3.2	[ksi]	
Resistance factor-LRFD	ϕ = 0.75			AISC 15th J3.7
Modified nominal tensile stress	$F'_{nt} = 1.3 F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_{rv} \le F_{nt}$	= **90.0**	[ksi]	AISC 15th Eq J3-3a
Bolt norminal tensile strength	$r_n = F'_{nt} A_b$	= 39.8	[kips]	AISC 15th Eq J3-1
Resistance factor-LRFD	ϕ = 0.75			AISC 15th J3.6
Single bolt tensile capacity	ϕr_n =	= **29.8**	[kips]	

Single Bolt Tensile Capacity After Considering Prying

Column flange as tee	b_f = 7.992 [in]	bolt gage g = 3.500	[in]	
	web t_w = 0.315 [in]			
Dist from bolt center to flange edge	$a_{cf} = 0.5 (b_f - g)$	= 2.246	[in]	
End plate	width w = 6.250 [in]	bolt gage g = 3.500	[in]	
Dist from bolt center to plate edge	$a_{pl} = 0.5 (w - g)$	= 1.375	[in]	
Dist from bolt center to plate edge	$a = \min (a_{cf}, a_{pl})$	= 1.375	[in]	
	$a' = a + 0.5 d_b \le (1.25 b + 0.5 d_b)$	= 1.750	[in]	AISC 15th Eq 9-23
Bolt hole diameter	bolt dia d_b = 0.750 [in]	bolt hole dia d_h = 0.813	[in]	AISC 15th B4.3b
Dist from bolt center to face of web	$b = 0.5(g - t_w)$	= 1.593	[in]	
	$b' = b - 0.5 d_b$	= 1.218	[in]	AISC 15th Eq 9-18
Bolt pitch spacing	s_v = 3.500 [in]			
Bolt tributary length	$p = s_v \quad p \le 2b$ and $p \le s_v$	= 3.185	[in]	AISC 15th Page 9-12
	$\rho = b' / a'$	= 0.696		AISC 15th Eq 9-22
	$\delta = 1 - d_h / p$	= 0.745		AISC 15th Eq 9-20
Tensile capacity per bolt before considering prying	B = from calc shown in above section	= 29.8	[kips]	
Resistance factor-LRFD	ϕ = 0.90			AISC 15th Page 9-12
Column flange thickness	t = 0.531 [in]	tensile F_u = 65.0	[ksi]	
Plate thickness req'd to develop bolt tensile capacity without prying	$t_c = (\frac{4 B b'}{\phi p F_u})^{0.5}$	= 0.883	[in]	AISC 15th Eq 9-26a
	$\alpha' = \frac{1}{\delta (1 + \rho)} [(\frac{t_c}{t})^2 - 1]$	= 1.397		AISC 15th Eq 9-28
when $\alpha' > 1$	$Q = (\frac{t}{t_c})^2 (1 + \delta)$	= 0.631		AISC 15th Eq 9-27
Bolt tensile force per bolt in demand	T = from calc shown below	= **4.3**	[kips]	
Tensile strength per bolt after considering prying	$\phi r_n = B \times Q$	= **18.8**	[kips]	AISC 15th Eq 9-27
	ratio = **0.23**	> T	OK	

Calculate Max Single Bolt Tensile Load

Bolt group force	axial P = 34.4 [kips]			
Bolt number	Bolt Row n_h = 2	Bolt Col n_v = 4		
Bolt tensile force per bolt	$T = P / (n_v n_h)$	= **4.3**	[kips]	

Beam Flange Weld Strength	ratio = 2.93 / 11.76	= **0.25**	**PASS**

Assume beam T&B flange weld line takes 100% of tensile force and beam web weld line takes no tensile force. Shear force is shared by both flange and web weld lines. The shear force taken by beam flange weld line is calculated based on the beam flange weld length / total weld length ratio

Beam-column interface forces taken from Gusset Plate Interface Forces Calc above

Axial force	axial P = -34.4	[kips]			in tension
Shear force	shear V = 11.3	[kips]			
Beam section W200x36	d_b = 7.913	[in]	k_b = 0.906	[in]	
	b_{fb} = 6.496	[in]	k_{1b} = 0.551	[in]	
Ver weld length on beam web	$L_{w1} = d_b - 2\,k_b$		= 6.101	[in]	
Bolt pitch & edge distance	d_1 = 2.500	[in]	e_v = 1.375	[in]	
Ver weld length outside flange-weld line on stiff plate	$L_e = 2(\,d_1 + e_v\,)$ - 2x0.75 in clip		= 6.250	[in]	
Ver weld length - total	$L_w = L_{w1} + L_e$		= 12.351	[in]	
Fillet weld length on beam flange	$L_f = (2b_{fb} - 2k_{1b})\,/\,2$ as double fillet		= 5.945	[in]	
Fillet weld length - total	$L = L_w + 2\,L_f$		= 24.241	[in]	
Shear force taken by weld at one side beam flange	$V_u = \frac{L_f}{L} \times V$		= **2.8**	[kips]	
Tensile force taken by weld at one side beam flange	$P_u = 0.5P$		= **-17.2**	[kips]	
Beam flange weld length	L = one side flange dbl fillet weld length		= **5.945**	[in]	
Beam flange fillet weld size	w =		= 0.250	[in]	

Combined Weld Stress

Weld stress from axial tensile force	$f_a = P_u\,/\,L$		= -2.89	[kip/in]	
Weld stress from shear force	$f_v = V_u\,/\,L$		= 0.47	[kip/in]	
Weld stress combined - max	$f_{max} = (\,f_a^2 + f_v^2\,)^{0.5}$		= **2.931**	[kip/in]	AISC 15th Eq 8-11
Weld stress load angle	$\theta = \tan^{-1}\left(\frac{f_a}{f_v}\right)$		= 80.8	[°]	

Fillet Weld Strength Calc

Fillet weld leg size	$w = {}^1/_4$	[in]	load angle θ = 80.8	[°]	
Electrode strength	F_{EXX} = 70.0	[ksi]	strength coeff C_1 = 1.00		AISC 15th Table 8-3
Number of weld line	n = 2 for double fillet				
Load angle coefficient	$C_2 = (\,1 + 0.5 \sin^{1.5}\theta\,)$		= 1.49		AISC 15th Page 8-9
Fillet weld shear strength	$R_{n\text{-}w} = 0.6\,(C_1 \times 70\text{ ksi})\,0.707\,w\,n\,C_2$		= 22.13	[kip/in]	AISC 15th Eq 8-1
Base metal - beam web	thickness t = 0.402	[in]	tensile F_u = 65.0	[ksi]	
Base metal - beam web is in shear, shear rupture as per AISC 15th Eq J4-4 is checked					AISC 15th J2.4
Base metal shear rupture	$R_{n\text{-}b} = 0.6\,F_u\,t$		= 15.68	[kip/in]	AISC 15th Eq J4-4
Double fillet linear shear strength	$R_n = \min(\,R_{n\text{-}w}, R_{n\text{-}b}\,)$		= **15.678**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq 8-1
	$\phi\,R_n$ =		= **11.759**	[kip/in]	
	ratio = **0.25**		> f_{max}	OK	

Beam Web Weld Strength		ratio = 5.8 / 88.1	= **0.07**	**PASS**

Assume beam T&B flange weld line takes 100% of tensile force and beam web weld line takes no tensile force. Shear force is shared by both flange and web weld lines. The shear force taken by beam web weld line is calculated based on the beam web weld length / total weld length ratio

Beam-column interface forces taken from Gusset Plate Interface Forces Calc above

Axial force	axial P = -34.4	[kips]			in tension
Shear force	shear V = 11.3	[kips]			
Beam section W200x36	d_b = 7.913	[in]	k_b = 0.906	[in]	
	b_{fb} = 6.496	[in]	k_{1b} = 0.551	[in]	
Ver weld length on beam web	$L_{w1} = d_b - 2\ k_b$		= 6.101	[in]	
Bolt pitch & edge distance	d_1 = 2.500	[in]	e_v = 1.375	[in]	
Ver weld length outside flange-weld line on stiff plate	$L_e = 2(\ d_1 + e_v\ )$ - 2x0.75 in clip		= 6.250	[in]	
Ver weld length - total	$L_w = L_{w1} + L_e$		= 12.351	[in]	
Fillet weld length on beam flange	$L_f = (2b_{fb} - 2k_{1b})\ /\ 2$ as double fillet		= 5.945	[in]	
Fillet weld length - total	$L = L_w + 2\ L_f$		= 24.241	[in]	
Shear force taken by weld at beam web	$V_u = \frac{L_w}{L} \times V$		= **5.8**	[kips]	

Fillet Weld Strength Check

Fillet weld leg size	w = $^1/_4$	[in]	load angle θ = 0.0	[°]	
Electrode strength	F_{EXX} = 70.0	[ksi]	strength coeff C_1 = 1.00		AISC 15th 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 15th Page 8-9
Fillet weld shear strength	$R_{n\text{-}w} = 0.6\ (C_1 \times 70\ \text{ksi})\ 0.707\ w\ n\ C_2$		= 14.85	[kip/in]	AISC 15th Eq 8-1
Base metal - beam web	thickness t = 0.244	[in]	tensile F_u = 65.0	[ksi]	
Base metal - beam web is in shear, <u>shear</u> rupture as per AISC 15th Eq J4-4 is checked					AISC 15th J2.4
Base metal shear rupture	$R_{n\text{-}b} = 0.6\ F_u\ t$		= 9.52	[kip/in]	AISC 15th Eq J4-4
Double fillet linear shear strength	$R_n = \min(\ R_{n\text{-}w}\ ,\ R_{n\text{-}b}\ )$		= **9.516**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq 8-1
	$\phi\ R_n$ =		= **7.137**	[kip/in]	
Weld resistance required	V_u =		= **5.8**	[kips]	
Fillet weld length - double fillet	L =		= 12.351	[in]	
Weld resistance provided	$\phi\ F_n = \phi\ R_n \times L$		= **88.1**	[kips]	
	ratio = **0.07**		> V_u	OK	

Column Web Local Yielding		ratio = 34.4 / 296.4	= **0.12**	**PASS**
Concentrated force on column	P_u =	= **34.4**	[kips]	
Column section	d = 9.921 [in]	t_f = 0.531	[in]	
	t_w = 0.315 [in]	k = 1.181	[in]	
	yield F_y = 50.0 [ksi]			
Length of bearing	l_b = end plate length	= 12.913	[in]	
Column web local yielding strength	$R_n = F_y t_w (5 k + l_b)$	= 296.4	[kips]	AISC 15th Eq J10-2
Resistance factor-LRFD	ϕ = 1.00			
	ϕR_n =	= **296.4**	[kips]	
	ratio = **0.12**	> P_u	OK	

Column Flange Local Bending		ratio = 34.4 / 79.3	= **0.43**	**PASS**
Concentrated force from gusset	P_u =	= **34.4**	[kips]	
Column w section	t_f = 0.531 [in]	yield F_y = 50.0	[ksi]	
Column flange local bending strength	$R_n = 6.25 F_y t_f^2$	= 88.1	[kips]	AISC 15th Eq J10-1
Resistance factor-LRFD	ϕ = 0.90			AISC 15th J10.1
	ϕR_n =	= **79.3**	[kips]	
	ratio = **0.43**	> P_u	OK	

Brace Force Load Case 2 shear V = 46.3 kips axial P = -78.2 kips (T) ratio = **0.99** **PASS**

Beam Web - Shear Yielding		ratio = 46.3 / 103.7	= **0.45**	**PASS**
Calculate gusset or stiff plate length outside beam flange and count it as beam web extension to resist shear				
Beam sect depth W200x36	d_b = 7.913 [in]			
Bolt pitch & edge distance	d_1 = 2.500 [in]	e_v = 1.375	[in]	
Beam web extension outside beam flange	$L_e = 2(d_1 + e_v)$ - 2x0.75 in clip	= 6.250	[in]	
Total beam web depth to resist shear	$L = d_b + L_e$	= **14.163**	[in]	
Plate Shear Yielding Check				
Plate size	width b_p = 14.163 [in]	thickness t_p = 0.244	[in]	
Plate yield strength	F_y = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$	= 3.456	[in^2]	
Shear force required	V_u =	= **46.3**	[kips]	
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$	= 103.7	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00			AISC 15th Eq J4-3
	ϕR_n =	= **103.7**	[kips]	
	ratio = **0.45**	> V_u	OK	

Beam Web - Shear Rupture		ratio = 46.3 / 101.1	= **0.46**	**PASS**
Calculate gusset or stiff plate length outside beam flange and count it as beam web extension to resist shear				
Beam sect depth W200x36	d_b = 7.913 [in]			
Bolt pitch & edge distance	d_1 = 2.500 [in]	e_v = 1.375 [in]		
Beam web extension outside beam flange	$L_e = 2(d_1 + e_v) - 2x0.75$ in clip	= 6.250 [in]		
Total beam web depth to resist shear	$L = d_b + L_e$	= **14.163** [in]		
Plate Shear Rupture Check				
Plate size	width b_p = 14.163 [in]	thickness t_p = 0.244 [in]		
Plate tensile strength	F_u = 65.0 [ksi]			
Plate net area in shear	$A_{nv} = b_p t_p$	= 3.456 [in^2]		
Shear force in demand	V_u =	= **46.3** [kips]		
Plate shear rupture strength	$R_n = 0.6 F_u A_{nv}$	= 134.8 [kips]	AISC 15th Eq J4-4	
Resistance factor-LRFD	ϕ = 0.75		AISC 15th Eq J4-4	
	ϕR_n =	= **101.1** [kips]		
	ratio = **0.46**	> V_u	OK	

Beam - Tensile Yielding		ratio = 78.2 / 318.8	= **0.25**	**PASS**
Gross area subject to tension	A_g =	= 7.084 [in^2]		
Steel yield strength	F_y =	= 50.0 [ksi]		
Tensile force required	P_u =	= **78.2** [kips]		
Tensile yielding strength	$R_n = F_y A_g$	= 354.2 [kips]	AISC 15th Eq D2-1	
Resistance factor-LRFD	ϕ = 0.90		AISC 15th D2 (a)	
	ϕR_n =	= **318.8** [kips]	AISC 15th Eq D2-1	
	ratio = **0.25**	> P_u	OK	

Beam - Tensile Rupture		ratio = 78.2 / 345.3	= **0.23**	**PASS**
W beam section	= W200x36			
W section net area	$A_n = A_g$	= 7.084 [in^2]		
Shear lag factor	U =	= 1.000		
Tensile force required	P_u =	= **78.2** [kips]		
Tensile effective net area	$A_e = A_n U$	= 7.084 [in^2]		
Plate tensile strength	F_u =	= 65.0 [ksi]		
Tensile rupture strength	$R_n = F_u A_e$	= 460.5 [kips]	AISC 15th Eq D2-2	
Resistance factor-LRFD	ϕ = 0.75		AISC 15th D2 (b)	
	ϕR_n =	= **345.3** [kips]	AISC 15th Eq D2-2	
	ratio = **0.23**	> P_u	OK	

End Plate - Shear Yield			ratio = 23.2 / 293.7	= **0.08**	**PASS**
Plate Shear Yielding Check					
Plate size	width b_p = 15.663	[in]	thickness t_p = 0.625	[in]	
Plate yield strength	F_y = 50.0	[ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$		= 9.789	[in^2]	
Shear force required	V_u =		= **23.2**	[kips]	
Plate shear yielding strength	$R_n = 0.6\ F_y A_{gv}$		= 293.7	[kips]	AISC 15th Eq J4-3
Resistance factor-LRFD	ϕ = 1.00				AISC 15th Eq J4-3
	$\phi\ R_n$ =		= **293.7**	[kips]	
	ratio = **0.08**		> V_u	OK	

End Plate - Shear Rupture			ratio = 23.2 / 222.4	= **0.10**	**PASS**
Plate Shear Rupture Check					
Bolt hole diameter	bolt dia $d_b = {}^3/_4$	[in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 15th B4.3b
Number of bolt	n = 4				
Plate size	width b_p = 15.663	[in]	thickness t_p = 0.625	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate net area in shear	$A_{nv} = (\ b_p - n\ d_h\ )\ t_p$		= 7.602	[in^2]	
Shear force required	V_u =		= **23.2**	[kips]	
Plate shear rupture strength	$R_n = 0.6\ F_u A_{nv}$		= 296.5	[kips]	AISC 15th Eq J4-4
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J4-4
	$\phi\ R_n$ =		= **222.4**	[kips]	
	ratio = **0.10**		> V_u	OK	

End Plate - Block Shear - Center Strip			ratio = 46.3 / 402.2	= **0.12**	**PASS**
Plate Block Shear - Center Strip					
Bolt hole diameter	bolt dia $d_b = {}^3/_4$	[in]	bolt hole dia $d_h = {}^7/_8$	[in]	AISC 15th B4.3b
Plate thickness	t_p = 0.625	[in]			
Plate strength	F_y = 50.0	[ksi]	F_u = 65.0	[ksi]	
Bolt no in ver & hor dir	n_v = 2		n_h = 4		
Bolt spacing in ver & hor dir	s_v = 3.500	[in]	s_h = 3.500	[in]	
Bolt edge dist in ver & hor dir	e_v = 1.375	[in]	e_h = 1.375	[in]	
Gross area subject to shear	$A_{gv} = [\ (n_h - 1)\ s_h + e_h\ ]\ t_p \times 2$		= 14.844	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5]\ d_h t_p \times 2$		= 11.016	[in^2]	
Net area subject to tension when sheared out by center strip	$A_{nt} = (\ n_v - 1)\ (\ s_v - d_h\ )\ t_p$		= 1.641	[in^2]	
Block shear strength required	V_u =		= **46.3**	[kips]	
Uniform tension stress factor	U_{bs} = 1.00				AISC 15th Fig C-J4.2
Bolt shear resistance provided	$R_n = \min\ (0.6F_u A_{nv},\ 0.6F_y A_{gv}) + U_{bs} F_u A_{nt}$		= 536.3	[kips]	AISC 15th Eq J4-5
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J4-5
	$\phi\ R_n$ =		= **402.2**	[kips]	
	ratio = **0.12**		> V_u	OK	

End Plate - Block Shear - 2-Side Strip			ratio = 46.3 / 379.3	= **0.12**	**PASS**
Plate Block Shear - 2 Side Strips					
Bolt hole diameter	bolt dia $d_b = ^3/_4$	[in]	bolt hole dia $d_h = ^7/_8$	[in]	AISC 15th B4.3b
Plate thickness	$t_p = 0.625$	[in]			
Plate strength	$F_y = 50.0$	[ksi]	$F_u = 65.0$	[ksi]	
Bolt no in ver & hor dir	$n_v = 2$		$n_h = 4$		
Bolt spacing in ver & hor dir	$s_v = 3.500$	[in]	$s_h = 3.500$	[in]	
Bolt edge dist in ver & hor dir	$e_v = 1.375$	[in]	$e_h = 1.375$	[in]	
Gross area subject to shear	$A_{gv} = [\ (n_h - 1)\ s_h + e_h\ ]\ t_p \times 2$		= 14.844	[in^2]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_h - 1) + 0.5]\ d_h\ t_p \times 2$		= 11.016	[in^2]	
Net area subject to tension					
when sheared out by 2 side strips	$A_{nt} = (\ e_v - 0.5\ d_h\ )\ t_p \times 2$		= 1.172	[in^2]	
Block shear strength required	$V_u =$		= **46.3**	[kips]	
Uniform tension stress factor	$U_{bs} = 1.00$				AISC 15th Fig C-J4.2
Bolt shear resistance provided	$R_n = \min(0.6F_u A_{nv},\ 0.6F_y A_{gv}) + U_{bs} F_u A_{nt}$		= 505.8	[kips]	AISC 15th Eq J4-5
Resistance factor-LRFD	$\phi = 0.75$				AISC 15th Eq J4-5
	$\phi\ R_n =$		= **379.3**	[kips]	
	ratio = **0.12**		> V_u	OK	

End Plate - Bolt Bearing on End Plate		ratio = 46.3 / 143.1	= **0.32**	**PASS**

Single Bolt Shear Strength

Bolt shear stress	bolt grade = A325-N		F_{nv} = 54.0	[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.750	[in]	bolt area A_b = 0.442	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$		= 23.9	[kips]	AISC 15th 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 15th Table J3.3
Bolt spacing & edge distance	spacing L_s = 3.500	[in]	edge distance L_e = 1.375	[in]	
Plate tensile strength	F_u = 65.0	[ksi]			
Plate thickness	t = 0.625	[in]			

Interior Bolt

Bolt hole edge clear distance	$L_c = L_s - d_h$	= 2.688	[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 15th Eq J3-6b
	= 163.8 ≤ 91.4	= 91.4	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$	= 23.9	[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 15th Eq J3-6b
	= 59.0 ≤ 91.4	= 59.0	[kips]	
Bolt strength at edge	$R_{n\text{-}ed} = \min(R_{n\text{-}t\&b\text{-}ed}, R_{n\text{-}bolt})$	= 23.9	[kips]	

Number of bolt	interior n_{in} = 6	edge n_{ed} = 2		
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-}in} + n_{ed} R_{n\text{-}ed}$	= 190.9	[kips]	
Required shear strength	V_u =	= **46.3**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75			AISC 15th J3.10
	$\phi\ R_n$ =	= **143.1**	[kips]	
	ratio = **0.32**	> V_u	OK	

End Plate / Column - Bolt Shear		ratio = 46.3 / 139.2	= **0.33**		**PASS**
Bolt A325-N	dia d_b = 0.750 [in]		A_b = 0.442	[in^2]	
Bolt shear stress	grade = A325-N		F_{nv} = 54.0	[ksi]	AISC 15th Table J3.2
Modified shear stress after considering combined tensile stress	F'_{nv} = from calc shown below		= 52.5	[ksi]	AISC 15th J3.7
Number of bolt carried shear	n_s = 8.0	shear plane m = 1			
Bolt group eccentricity coefficient	C_{ec} =		= 1.000		
Required shear strength	V_u =		= **46.3**	[kips]	
Bolt shear strength	$R_n = F'_{nv} A_b n_s m C_{ec}$		= 185.5	[kips]	AISC 15th Eq J3-1
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq J3-1
	ϕR_n =		= **139.2**	[kips]	
	ratio = **0.33**		> V_u	OK	

Check If Modified Shear Stress F'_{nv} Shall Be Used

Bolt group force	axial P = 78.2 [kips]				
Bolt grade	grade = A325-N				
Nominal shear stress	F_{nt} = 90.0 [ksi]				AISC 15th Table J3.2
	bolt dia d_b = 0.750 [in]	bolt area A_b = 0.442		[in^2]	
Bolt number	Bolt Row n_v = 2	Bolt Col n_h = 4			
Tensile stress required	$f_{rt} = P / (n_v n_h A_b)$		= 22.1	[ksi]	
Resistance factor-LRFD	ϕ = 0.75				AISC 15th J3.7
Check tensile stress ratio limit	$= \dfrac{f_{rt}}{\phi F_{nt}}$		= **0.33**		
			> 0.3		AISC 15th J3.7

Combined tensile/shear effect shall be considered, modified tensile stress F'_{nv} shall be used

Calc Modified Shear Stress F'_{nv} Considering Shear Effect

Bolt group force	axial P = 78.2 [kips]				
Bolt grade	grade = A325-N				
Nominal tensile/shear stress	F_{nt} = 90.0 [ksi]		F_{nv} = 54.0	[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.750 [in]	bolt area A_b = 0.442		[in^2]	
Bolt number	Bolt Row n_v = 2	Bolt Col n_h = 4			
Tensile stress required	$f_{rt} = P / (n_v n_h A_b)$		= 22.1	[ksi]	
Resistance factor-LRFD	ϕ = 0.75				AISC 15th J3.7
Modified nominal shear stress	$F'_{nv} = 1.3 F_{nv} - \dfrac{F_{nv}}{\phi F_{nt}} f_{rt} \le F_{nv}$		= **52.5**	[ksi]	AISC 15th Eq J3-3a

End Plate / Column - Bolt Bearing on Column		ratio = 46.3 / 143.1	= **0.32**	**PASS**	
Single Bolt Shear Strength					
Bolt shear stress	bolt grade = A325-N		F_{nv} = 54.0	[ksi]	AISC 15th Table J3.2
	bolt dia d_b = 0.750 [in]		bolt area A_b = 0.442	[in^2]	
Single bolt shear strength	$R_{n\text{-}bolt} = F_{nv} A_b$		= 23.9	[kips]	AISC 15th 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 15th Table J3.3
Bolt spacing	spacing L_s = 3.500 [in]				
Plate tensile strength	F_u = 65.0 [ksi]				
Plate thickness	t = 0.531 [in]				
Interior Bolt					
Bolt hole edge clear distance	$L_c = L_s - d_h$		= 2.688	[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 15th Eq J3-6b
	= 139.1 ≤ 77.7		= 77.7	[kips]	
Bolt strength at interior	$R_{n\text{-}in} = \min(R_{n\text{-}t\&b\text{-}in}, R_{n\text{-}bolt})$		= 23.9	[kips]	
Number of bolt	interior n_{in} = 8				
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n\text{-}in}$		= 190.9	[kips]	
Required shear strength	V_u =		= **46.3**	[kips]	
Bolt resistance factor-LRFD	ϕ = 0.75				AISC 15th J3.10
	ϕR_n =		= **143.1**	[kips]	
	ratio = **0.32**		> V_u	OK	

Bolt Tensile Prying Action on End Plate		ratio = 9.8 / 23.2	= **0.42**	**PASS**	
Bolt group forces	shear V = 46.3 [kips]	axial P = -78.2	[kips]		

Single Bolt Tensile Capacity Without Considering Prying

Bolt grade A325-N	dia d_b = 0.750 [in]	area A_b = 0.442	[in^2]	
Nominal tensile/shear stress	F_{nt} = 90.0 [ksi]	F_{nv} = 54.0	[ksi]	AISC 15th Table J3.2
Bolt group shear force	shear V = 46.3 [kips]	no of bolt n = 8		
Shear stress required	$f_{rv} = V / (n A_b)$	= 13.1	[ksi]	
Resistance factor-LRFD	ϕ = 0.75			AISC 15th J3.7
Modified nominal tensile stress	$F'_{nt} = 1.3 F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_{rv} \le F_{nt}$	= **87.9**	[ksi]	AISC 15th Eq J3-3a
Bolt norminal tensile strength	$r_n = F'_{nt} A_b$	= 38.8	[kips]	AISC 15th Eq J3-1
Resistance factor-LRFD	ϕ = 0.75			AISC 15th J3.6
Single bolt tensile capacity	ϕr_n =	= **29.1**	[kips]	

Single Bolt Tensile Capacity After Considering Prying

End plate	width w = 6.250 [in]	bolt gage g = 3.500	[in]	
	web t_w = 0.244 [in]			
Dist from bolt center to plate edge	$a = 0.5 (w - g)$	= 1.375	[in]	
	$a' = a + 0.5 d_b \le (1.25 b + 0.5 d_b)$	= 1.750	[in]	AISC 15th Eq 9-23
Bolt hole diameter	bolt dia d_b = 0.750 [in]	bolt hole dia d_h = 0.813	[in]	AISC 15th B4.3b
Dist from bolt center to face of web	$b = 0.5(g - t_w)$	= 1.628	[in]	
	$b' = b - 0.5 d_b$	= 1.253	[in]	AISC 15th Eq 9-18
Bolt pitch spacing	s_v = 3.500 [in]			
Bolt tributary length	$p = s_v \quad p \le 2b$ and $p \le s_v$	= 3.256	[in]	AISC 15th Page 9-12
	$\rho = b' / a'$	= 0.716		AISC 15th Eq 9-22
	$\delta = 1 - d_h / p$	= 0.750		AISC 15th Eq 9-20
Tensile capacity per bolt before considering prying	B = from calc shown in above section	= 29.1	[kips]	
Resistance factor-LRFD	ϕ = 0.90			AISC 15th Page 9-12
End plate thickness	t = 0.625 [in]	tensile F_u = 65.0	[ksi]	
Plate thickness req'd to develop bolt tensile capacity without prying	$t_c = \left(\frac{4 B b'}{\phi p F_u} \right)^{0.5}$	= 0.875	[in]	AISC 15th Eq 9-26a
	$\alpha' = \frac{1}{\delta (1 + \rho)} \left[\left(\frac{t_c}{t} \right)^2 - 1 \right]$	= 0.747		AISC 15th Eq 9-28
when $0 \le \alpha' \le 1$	$Q = \left(\frac{t}{t_c} \right)^2 (1 + \delta \alpha')$	= 0.795		AISC 15th Eq 9-27
Bolt tensile force per bolt in demand	T = from calc shown below	= **9.8**	[kips]	
Tensile strength per bolt after considering prying	$\phi r_n = B \times Q$	= **23.2**	[kips]	AISC 15th Eq 9-27
	ratio = **0.42**	> T	OK	

Calculate Max Single Bolt Tensile Load

Bolt group force	axial P = 78.2 [kips]			
Bolt number	Bolt Row n_h = 2	Bolt Col n_v = 4		
Bolt tensile force per bolt	$T = P / (n_v n_h)$	= **9.8**	[kips]	

Bolt Tensile Prying Action on Column Flange		ratio = 9.8 / 18.8	= **0.52**	**PASS**	
Bolt group forces	shear V = 46.3 [kips]	axial P = -78.2	[kips]		

Single Bolt Tensile Capacity Without Considering Prying

Bolt grade A325-N	dia d_b = 0.750 [in]	area A_b = 0.442	[in^2]	
Nominal tensile/shear stress	F_{nt} = 90.0 [ksi]	F_{nv} = 54.0	[ksi]	AISC 15th Table J3.2
Bolt group shear force	shear V = 46.3 [kips]	no of bolt n = 8		
Shear stress required	$f_{rv} = V / (n A_b)$	= 13.1	[ksi]	
Resistance factor-LRFD	ϕ = 0.75			AISC 15th J3.7
Modified nominal tensile stress	$F'_{nt} = 1.3 F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_{rv} \le F_{nt}$	= **87.9**	[ksi]	AISC 15th Eq J3-3a
Bolt norminal tensile strength	$r_n = F'_{nt} A_b$	= 38.8	[kips]	AISC 15th Eq J3-1
Resistance factor-LRFD	ϕ = 0.75			AISC 15th J3.6
Single bolt tensile capacity	ϕr_n =	= **29.1**	[kips]	

Single Bolt Tensile Capacity After Considering Prying

Column flange as tee	b_f = 7.992 [in]	bolt gage g = 3.500	[in]	
	web t_w = 0.315 [in]			
Dist from bolt center to flange edge	$a_{cf} = 0.5 (b_f - g)$	= 2.246	[in]	
End plate	width w = 6.250 [in]	bolt gage g = 3.500	[in]	
Dist from bolt center to plate edge	$a_{pl} = 0.5 (w - g)$	= 1.375	[in]	
Dist from bolt center to plate edge	$a = \min (a_{cf} , a_{pl})$	= 1.375	[in]	
	$a' = a + 0.5 d_b \le (1.25 b + 0.5 d_b)$	= 1.750	[in]	AISC 15th Eq 9-23
Bolt hole diameter	bolt dia d_b = 0.750 [in]	bolt hole dia d_h = 0.813	[in]	AISC 15th B4.3b
Dist from bolt center to face of web	$b = 0.5(g - t_w)$	= 1.593	[in]	
	$b' = b - 0.5 d_b$	= 1.218	[in]	AISC 15th Eq 9-18
Bolt pitch spacing	s_v = 3.500 [in]			
Bolt tributary length	$p = s_v \quad p \le 2b$ and $p \le s_v$	= 3.185	[in]	AISC 15th Page 9-12
	$\rho = b' / a'$	= 0.696		AISC 15th Eq 9-22
	$\delta = 1 - d_h / p$	= 0.745		AISC 15th Eq 9-20
Tensile capacity per bolt before considering prying	B = from calc shown in above section	= 29.1	[kips]	
Resistance factor-LRFD	ϕ = 0.90			AISC 15th Page 9-12
Column flange thickness	t = 0.531 [in]	tensile F_u = 65.0	[ksi]	
Plate thickness req'd to develop bolt tensile capacity without prying	$t_c = (\frac{4 B b'}{\phi p F_u})^{0.5}$	= 0.872	[in]	AISC 15th Eq 9-26a
	$\alpha' = \frac{1}{\delta (1 + \rho)} [(\frac{t_c}{t})^2 - 1]$	= 1.345		AISC 15th Eq 9-28
when $\alpha' > 1$	$Q = (\frac{t}{t_c})^2 (1 + \delta)$	= 0.646		AISC 15th Eq 9-27
Bolt tensile force per bolt in demand	T = from calc shown below	= **9.8**	[kips]	
Tensile strength per bolt after considering prying	$\phi r_n = B \times Q$	= **18.8**	[kips]	AISC 15th Eq 9-27
	ratio = **0.52**	> T	OK	

Calculate Max Single Bolt Tensile Load

Bolt group force	axial P = 78.2 [kips]			
Bolt number	Bolt Row n_h = 2	Bolt Col n_v = 4		
Bolt tensile force per bolt	$T = P / (n_v n_h)$	= **9.8**	[kips]	

Beam Flange Weld Strength	ratio = 6.85 / 11.76	= **0.58**	**PASS**

Assume beam T&B flange weld line takes 100% of tensile force and beam web weld line takes no tensile force. Shear force is shared by both flange and web weld lines. The shear force taken by beam flange weld line is calculated based on the beam flange weld length / total weld length ratio

Beam-column interface forces taken from Gusset Plate Interface Forces Calc above

Axial force	axial P = -78.2	[kips]			in tension
Shear force	shear V = 46.3	[kips]			
Beam section W200x36	d_b = 7.913	[in]	k_b = 0.906	[in]	
	b_{fb} = 6.496	[in]	k_{1b} = 0.551	[in]	
Ver weld length on beam web	$L_{w1} = d_b - 2\ k_b$		= 6.101	[in]	
Bolt pitch & edge distance	d_1 = 2.500	[in]	e_v = 1.375	[in]	
Ver weld length outside flange-weld line on stiff plate	$L_e = 2(\ d_1 + e_v\ )$ - 2x0.75 in clip		= 6.250	[in]	
Ver weld length - total	$L_w = L_{w1} + L_e$		= 12.351	[in]	
Fillet weld length on beam flange	$L_f = (2b_{fb} - 2k_{1b})\ /\ 2$ as double fillet		= 5.945	[in]	
Fillet weld length - total	$L = L_w + 2\ L_f$		= 24.241	[in]	
Shear force taken by weld at one side beam flange	$V_u = \frac{L_f}{L} \times V$		= **11.4**	[kips]	
Tensile force taken by weld at one side beam flange	$P_u = 0.5P$		= **-39.1**	[kips]	
Beam flange weld length	L = one side flange dbl fillet weld length		= **5.945**	[in]	
Beam flange fillet weld size	w =		= 0.250	[in]	

Combined Weld Stress

Weld stress from axial tensile force	$f_a = P_u\ /\ L$	= -6.58	[kip/in]	
Weld stress from shear force	$f_v = V_u\ /\ L$	= 1.91	[kip/in]	
Weld stress combined - max	$f_{max} = (\ f_a^2 + f_v^2\ )^{0.5}$	= **6.849**	[kip/in]	AISC 15th Eq 8-11
Weld stress load angle	$\theta = \tan^{-1}(\ \frac{f_a}{f_v}\ )$	= 73.8	[°]	

Fillet Weld Strength Calc

Fillet weld leg size	$w = {}^1/_4$	[in]	load angle θ = 73.8	[°]		
Electrode strength	F_{EXX} = 70.0	[ksi]	strength coeff C_1 = 1.00		AISC 15th Table 8-3	
Number of weld line	n = 2 for double fillet					
Load angle coefficient	$C_2 = (\ 1 + 0.5 \sin^{1.5}\theta\ )$		= 1.47		AISC 15th Page 8-9	
Fillet weld shear strength	$R_{n\text{-}w} = 0.6\ (C_1 \times 70\text{ ksi})\ 0.707\ w\ n\ C_2$		= 21.83	[kip/in]	AISC 15th Eq 8-1	
Base metal - beam web	thickness t = 0.402	[in]	tensile F_u = 65.0	[ksi]		

Base metal - beam web is in shear, shear rupture as per AISC 15th Eq J4-4 is checked — AISC 15th J2.4

Base metal shear rupture	$R_{n\text{-}b} = 0.6\ F_u\ t$	= 15.68	[kip/in]	AISC 15th Eq J4-4
Double fillet linear shear strength	$R_n = \min(\ R_{n\text{-}w},\ R_{n\text{-}b}\ )$	= **15.678**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75			AISC 15th Eq 8-1
	$\phi\ R_n$ =	= **11.759**	[kip/in]	
	ratio = **0.58**	> f_{max}	OK	

Beam Web Weld Strength	ratio = 23.6 / 88.1	= **0.27**	**PASS**

Assume beam T&B flange weld line takes 100% of tensile force and beam web weld line takes no tensile force. Shear force is shared by both flange and web weld lines. The shear force taken by beam web weld line is calculated based on the beam web weld length / total weld length ratio

Beam-column interface forces taken from Gusset Plate Interface Forces Calc above

Axial force	axial P = -78.2	[kips]			in tension
Shear force	shear V = 46.3	[kips]			
Beam section W200x36	d_b = 7.913	[in]	k_b = 0.906	[in]	
	b_{fb} = 6.496	[in]	k_{1b} = 0.551	[in]	
Ver weld length on beam web	$L_{w1} = d_b - 2\ k_b$		= 6.101	[in]	
Bolt pitch & edge distance	d_1 = 2.500	[in]	e_v = 1.375	[in]	
Ver weld length outside flange-weld line on stiff plate	$L_e = 2(\ d_1 + e_v\ )$ - 2x0.75 in clip		= 6.250	[in]	
Ver weld length - total	$L_w = L_{w1} + L_e$		= 12.351	[in]	
Fillet weld length on beam flange	$L_f = (2b_{fb} - 2k_{1b})\ /\ 2$ as double fillet		= 5.945	[in]	
Fillet weld length - total	$L = L_w + 2\ L_f$		= 24.241	[in]	
Shear force taken by weld at beam web	$V_u = \dfrac{L_w}{L} \times V$		= **23.6**	[kips]	

Fillet Weld Strength Check

Fillet weld leg size	w = $^1/_4$	[in]	load angle θ = 0.0	[°]	
Electrode strength	F_{EXX} = 70.0	[ksi]	strength coeff C_1 = 1.00		AISC 15th 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 15th Page 8-9
Fillet weld shear strength	$R_{n\text{-}w} = 0.6\ (C_1 \times 70$ ksi$)\ 0.707\ w\ n\ C_2$		= 14.85	[kip/in]	AISC 15th Eq 8-1
Base metal - beam web	thickness t = 0.244	[in]	tensile F_u = 65.0	[ksi]	
Base metal - beam web is in shear, <u>shear</u> rupture as per AISC 15th Eq J4-4 is checked					AISC 15th J2.4
Base metal shear rupture	$R_{n\text{-}b} = 0.6\ F_u\ t$		= 9.52	[kip/in]	AISC 15th Eq J4-4
Double fillet linear shear strength	$R_n = \min\ (\ R_{n\text{-}w}\ ,\ R_{n\text{-}b}\ )$		= **9.516**	[kip/in]	AISC 15th Eq 9-2
Resistance factor-LRFD	ϕ = 0.75				AISC 15th Eq 8-1
	$\phi\ R_n$ =		= **7.137**	[kip/in]	
Weld resistance required	V_u =		= **23.6**	[kips]	
Fillet weld length - double fillet	L =		= 12.351	[in]	
Weld resistance provided	$\phi\ F_n = \phi\ R_n \times L$		= **88.1**	[kips]	
	ratio = **0.27**		> V_u	OK	

Column Web Local Yielding			ratio = 78.2 / 296.4	= **0.26**	**PASS**
Concentrated force on column	P_u =		= **78.2**	[kips]	
Column section	d = 9.921	[in]	t_f = 0.531	[in]	
	t_w = 0.315	[in]	k = 1.181	[in]	
	yield F_y = 50.0	[ksi]			
Length of bearing	l_b = end plate length		= 12.913	[in]	
Column web local yielding strength	$R_n = F_y t_w (5 k + l_b)$		= 296.4	[kips]	AISC 15th Eq J10-2
Resistance factor-LRFD	ϕ = 1.00				
	ϕR_n =		= **296.4**	[kips]	
	ratio = **0.26**		> P_u	OK	

Column Flange Local Bending			ratio = 78.2 / 79.3	= **0.99**	**PASS**
Concentrated force from gusset	P_u =		= **78.2**	[kips]	
Column w section	t_f = 0.531	[in]	yield F_y = 50.0	[ksi]	
Column flange local bending strength	$R_n = 6.25 F_y t_f^2$		= 88.1	[kips]	AISC 15th Eq J10-1
Resistance factor-LRFD	ϕ = 0.90				AISC 15th J10.1
	ϕR_n =		= **79.3**	[kips]	
	ratio = **0.99**		> P_u	OK	