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

This is the working sheet for tutorial video <u>https://youtu.be/VDbLmzID42A</u> To view Transfer Force note, visit <u>http://asp.civilbay.com/18-manual/01-manual-sc.aspx#faq25</u> To start program, visit <u>http://asp.civilbay.com/18-manual/05-new-user.aspx#pos-sc</u>

Check next page for brace connection details



# 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 = <b>PASS</b> limit stat	es max ratio = 0.99 PASS				
Brace to Gusset	geometries & weld limitations = <b>PASS</b>	limit states max ratio = 0.40 PASS				
Gusset to Column	geometries & weld limitations = <b>PASS</b>	limit states max ratio = 0.48 PASS				
Gusset to Beam	geometries & weld limitations = <b>PASS</b>	limit states max ratio = 0.27 PASS				
Beam to Column	geometries & weld limitations = <b>PASS</b>	limit states max ratio = <b>0.99 PASS</b>				

Sketch

Vertical Brace Connection

#### Code=AISC 360-16 LRFD







Gusset Plate Interface Forces Cal	culation			
$A_{b} = \begin{pmatrix} V_{c} \\ P_{b} \\ V_{b-c} \\ e_{c} \end{pmatrix}$	H <sub>c</sub> $H_b$	P <sub>bm</sub> _		
Brace Avial Force Load Case 1				
Brace force	P = -76.5 [kips] (T)			
Beam end shear & transfer force	Shear R <sub>b</sub> = 28.8 [kips]	Transfer $A_b = 56.3$	[kips]	
Refer to AISC 15 <sup>th</sup> Page 13-4 and Fig.	13-2 for all charts and definitions of vari	ables and symbols show	vn in calc	ulation below
	e <sub>b</sub> = 3.957 [in]	e <sub>c</sub> = 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 15 <sup>th</sup> Eq. 13-16
	$D = \tan^2 \theta + \left(\frac{\alpha}{\beta}\right)^2$	= 2.152		AISC 15 <sup>th</sup> Eq. 13-24
	$K' = \alpha ( \tan \theta + \frac{\alpha}{\beta})$	= 15.735		AISC 15 <sup>th</sup> Eq. 13-23
	$\overline{\alpha} = [K' \tan \theta + K \left(\frac{\alpha}{\beta}\right)^2] / D$	= 7.815	[in]	AISC 15 <sup>th</sup> Eq. 13-21
	$\overline{\beta}$ = (K' - K tan $\theta$ ) / D	= 7.750	[in]	AISC 15 <sup>th</sup> Eq. 13-22
	r = [( $e_b + \overline{\beta}$ ) <sup>2</sup> + ( $e_c + \overline{\alpha}$ ) <sup>2</sup>	] <sup>0.5</sup> = 17.328	[in]	AISC 15 <sup>th</sup> Eq. 13-6
Brace axial force	P <sub>u</sub> = from user input	= -76.5	[kips]	in tension
Gusset to Column Interface Forces				
Shear force	$V_c = (\overline{\beta} / r) P_u$	= -34.2	[kips]	AISC 15 <sup>th</sup> Eq. 13-2
Axial force	$H_c = (e_c/r)P_u$	= -21.9	[kips]	AISC 15 <sup>th</sup> Eq. 13-3
Moment	$M_c = H_c (\beta - \overline{\beta})$	= 0.00	[kip-ft]	AISC 15 <sup>th</sup> Eq. 13-19
Gusset to Beam Interface Forces				
Shear force	$H_{b} = (\overline{\alpha} / r) P_{u}$	= -34.5	[kips]	AISC 15 <sup>th</sup> Eq. 13-5
Axial force	$V_{b} = (e_{b}/r)P_{u}$	= -17.5	[kips]	AISC 15 <sup>th</sup> Eq. 13-4
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Beam to Column Interface Forces				
Beam to Column Interface Shear F	Force			
Beam end shear reaction	$R_{b} = $ from user input	= 28.8	[kips]	
Brace gusset-beam axial force	V <sub>b</sub> =	= -17.5	[kips]	AISC 15 <sup>th</sup> Eq. 13-4
Beam to column shear force	$V_{b-c} = R_b + V_b$	= 11.3	[kips]	AISC 15 <sup>th</sup> Page 13-4
Beam to Column Interface Axial Fe	orce			
Gusset-column axial force	H <sub>c</sub> =	= -21.9	[kips]	AISC 15 <sup>th</sup> 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 15 <sup>th</sup> 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<sub>bm</sub> - 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<sub>bm</sub> - 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	$\theta$ = from user input	= 47.5	[°]	
Gusset-column axial force	H <sub>c</sub> =	= -21.9	[kips]	AISC 15 <sup>th</sup> 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 15 <sup>th</sup> Page 13-4 and Fig	. 13-2 for all charts ar	nd definitions of varia	bles and symbols show	wn in calcu	lation below
	e <sub>b</sub> = 3.957	[in]	$e_{c} = 4.961$	[in]	
	α = 7.596	[in]	$\beta = 7.750$	[in]	
	$\theta = 47.5$	[°]			
	$K = e_b tan \theta$	- e <sub>c</sub>	= -0.643	[in]	AISC 15 <sup>th</sup> Eq. 13-16
	$D = \tan^2 \theta +$	$-\left(\frac{\alpha}{\beta}\right)^2$	= 2.152		AISC 15 <sup>th</sup> Eq. 13-24
	K' = α ( tan 6	$\theta + \frac{\alpha}{\beta}$	= 15.735		AISC 15 <sup>th</sup> Eq. 13-23
	$\overline{\alpha} = [K' \tan$	$\theta + K \left(\frac{\alpha}{\beta}\right)^2 ] / D$	= 7.815	[in]	AISC 15 <sup>th</sup> Eq. 13-21
	$\overline{\beta}$ = (K' - K	tan θ ) / D	= 7.750	[in]	AISC 15 <sup>th</sup> Eq. 13-22
	$r = [(e_{b} +$	$\overline{\beta}$ ) <sup>2</sup> + (e <sub>c</sub> + $\overline{\alpha}$ ) <sup>2</sup> ] <sup>6</sup>	0.5 = 17.328	[in]	AISC 15 <sup>th</sup> Eq. 13-6
Brace axial force	P <sub>u</sub> = from use	er input	= 76.5	[kips]	in compression

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		n Design	http://asp.civilbay.com/conne	ct	vertical Bra	ce Connecuo	on	VB-Ri
Gusset to Column	Interface Forces							
Shear force		$V_c = (\overline{\beta} / r) F$	u u	= 34.2	[kips]	AISC 15 <sup>th</sup>	Eq. 13-2	
Axial force		$H_{c} = (e_{c} / r)$	Pu	= 21.9	[kips]	AISC 15 <sup>th</sup>	Eq. 13-3	
Moment		$M_c = H_c (\beta - \overline{\beta})$	)	= 0.00	[kip-ft]	AISC 15 <sup>th</sup>	Eq. 13-19	
Gusset to Beam In	terface Forces							
Shear force		$H_{b} = (\overline{\alpha} / r) F$	р ц	= 34.5	[kips]	AISC 15 <sup>th</sup>	Eq. 13-5	
Axial force		$V_{b} = (e_{b} / r)$	Pu	= 17.5	[kips]	AISC 15 <sup>th</sup>	Eq. 13-4	
		$M_{L} = V_{L} (\overline{\alpha} - \alpha)$	ι)	= 0.32	[kip-ft]	AISC 15 <sup>th</sup>	Eq. 13-17	
Moment Beam to Column Ir Beam to Column Ir	iterface Forces							
Moment Beam to Column Ir Beam to Column Ir Beam end shear read	nterface Forces Interface Shear Force	$R_b = $ from use	r input	 = 28.8	[kips]			
Moment Beam to Column Ir Beam to Column Ir Beam end shear read Brace gusset-beam a	aterface Forces Aterface Shear Force ation xial force	$R_b = $ from use $V_b =$	r input	= 28.8 = 17.5	[kips] [kips]	AISC 15 <sup>th</sup>	Eq. 13-4	
Moment Beam to Column Ir Beam to Column Ir Beam end shear read Brace gusset-beam a Beam to column shea	aterface Forces aterface Shear Force ation xial force ar force	$R_b = from use$ $V_b =$ $V_{b-c} = R_b + V_b$	r input	= 28.8 = 17.5 = <b>46.3</b>	[kips] [kips] [kips]	AISC 15 <sup>th</sup> AISC 15 <sup>th</sup>	Eq. 13-4 Page 13-4	
Moment Beam to Column Ir Beam to Column Ir Beam end shear read Brace gusset-beam a Beam to column shea Beam to Column Ir	aterface Forces Aterface Shear Force ation xial force ar force Aterface Axial Force	$R_b = $ from use $V_b =$ $V_{b-c} = R_b + V_b$	r input	= 28.8 = 17.5 = <b>46.3</b>	[kips] [kips] [kips]	AISC 15 <sup>th</sup> AISC 15 <sup>th</sup>	Eq. 13-4 Page 13-4	
Moment Beam to Column Ir Beam to Column Ir Beam end shear read Brace gusset-beam a Beam to column shea Beam to Column Ir Gusset-column axial	aterface Forces aterface Shear Force ation xial force ar force aterface Axial Force force	$R_b = \text{from use}$ $V_b =$ $V_{b-c} = R_b + V_b$ $H_c =$	r input	= 28.8 = 17.5 = <b>46.3</b> = 21.9	[kips] [kips] [kips]	AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup>	Eq. 13-4 Page 13-4 Eq. 13-3	
Moment Beam to Column Ir Beam to Column Ir Beam end shear read Brace gusset-beam a Beam to column shea Beam to Column Ir Gusset-column axial Transfer force from a	aterface Forces Aterface Shear Force ation xial force ar force Aterface Axial Force force djacent bay	$R_{b} = \text{from use}$ $V_{b} =$ $V_{b-c} = R_{b} + V_{b}$ $H_{c} =$ $A_{b} = \text{from use}$	r input r input	= 28.8 = 17.5 = <b>46.3</b> = 21.9 = 56.3	[kips] [kips] [kips] [kips] [kips]	AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup>	Eq. 13-4 Page 13-4 Eq. 13-3	

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

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 $P_{b-c}$  - 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	$\theta$ = from user input	= 47.5	[°]	
Gusset-column axial force	H <sub>c</sub> =	= 21.9	[kips]	AISC 15 <sup>th</sup> 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 <sub>LC1</sub> =-76.5 kips (T)	P <sub>LC2</sub> =76.5 kips (C)	Code=AISC 360-16 LRFD
Result Summary	geometries & weld limitations	i = <b>PASS</b> limit	states max ratio = <b>0.40</b>	PASS

http://asp.civilbay.com/connect

Vertical Brace Connection

VB-Right

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 <sub>min</sub> =	= 0.188	[in]	AISC 15 <sup>th</sup> Table J2.4		
Fillet weld size provided	w =	= 0.250	[in]			
		≥ w <sub>min</sub>	ОК			
Max Fillet Weld Size	_					
Along edge plate thickness	t =	= 0.500	[in]			
Max fillet weld size allowed	$w_{max} = t - \frac{1}{16}$ (2mm)	= 0.438	[in]	AISC 15 <sup>th</sup> J2.2b		
Fillet weld size provided	w =	= 0.250	[in]			
		≤ w <sub>max</sub>	ОК			
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 15 <sup>th</sup> J2.2b		
Min fillet weld length	L =	= 5.000	[in]			
		≥ L <sub>min</sub>	ОК			

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 <sub>g</sub> =	= 7.998	[in <sup>2</sup> ]	
Steel yield strength	F <sub>y</sub> =	= 36.0	[ksi]	
Tensile force required	P <sub>u</sub> =	= 76.5	[kips]	
Tensile yielding strength	$R_n = F_y A_g$	= 287.9	[kips]	AISC 15 <sup>th</sup> Eq D2-1
Resistance factor-LRFD	φ = 0.90			AISC 15 <sup>th</sup> D2 (a)
	φ R <sub>n</sub> =	= 259.1	[kips]	AISC 15 <sup>th</sup> Eq D2-1
	ratio = <b>0.30</b>	> P <sub>u</sub>	ОК	
Double Angle Brace - Tensile Rupt	ure	ratio = 76.5 / 239.9	= 0.32	PASS
Section gross area	$A_{g} = L127x89x13$	= 7.998	[in <sup>2</sup> ]	
Tensile net area	$A_n = A_g$	= 7.998	[in <sup>2</sup> ]	
Length of connection	L =	= 6.000	[in]	
Width of angle leg	W =	= 5.000	[in]	
Eccentricity of connection	$\frac{1}{x}$ = from sect L127x89x13	= 0.906	[in]	
Shear lag factor	$U = \frac{3L^2}{3L^2 + w^2} \left( 1 - \frac{\overline{x}}{L} \right)$	= 0.689		AISC 15 <sup>th</sup> Table D3.1 Case4
Tensile force required	P <sub>u</sub> =	= 76.5	[kips]	
Tensile effective net area	$A_e = A_n U$	= 5.514	[in <sup>2</sup> ]	
Plate tensile strength	F <sub>u</sub> =	= 58.0	[ksi]	
Tensile rupture strength	$R_n = F_u A_e$	= 319.8	[kips]	AISC 15 <sup>th</sup> Eq D2-2
Resistance factor-LRFD	φ = 0.75			AISC 15 <sup>th</sup> D2 (b)
	φ R <sub>n</sub> =	= 239.9	[kips]	AISC 15 <sup>th</sup> Eq D2-2
	ratio = <b>0.32</b>	> P <sub>u</sub>	ОК	

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Vertical Brace Connection

Gusset Plate - Tensile Yield (W	'hitmore)		ratio = 76.	.5 / 268.4	= 0.29	PASS		
Plate Tensile Yielding Check								
Plate size	width b <sub>p</sub> = 11.928	[in]	thickness	st <sub>p</sub> = 0.500	[in]			
Plate yield strength	$F_{y} = 50.0$	[ksi]		۲				
Plate gross area in shear	$A_q = b_p t_p$			= 5.964	[in <sup>2</sup> ]			
Tensile force required	$P_u =$			= 76.5	[kips]			
Plate tensile yielding strength	$R_n = F_y A_g$			= 298.2	[kips]	AISC 15 <sup>th</sup>	Eq J4-1	
Resistance factor-LRFD	φ = 0.90					AISC 15 <sup>th</sup>	Eq J4-1	
	φ R <sub>n</sub> =			= 268.4	[kips]			
	ratio = <b>0.29</b>			> P <sub>u</sub>	ОК			
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	$s t_p = 0.500$	[in]			
Plate tensile strength	$F_{u} = 65.0$	[ksi]		F	-			
Plate net area in tension	$A_{nt} = b_{p}t_{p}$			= 5.964	[in <sup>2</sup> ]			
Tensile force required	P <sub>u</sub> =			= 76.5	[kips]			
Plate tensile rupture strength	$R_n = F_u A_{nt}$			= 387.7	[kips]	AISC 15 <sup>th</sup>	Eq J4-2	
Resistance factor-LRFD	φ = 0.75					AISC 15 <sup>th</sup>	Eq J4-2	
	φ R <sub>n</sub> =			= 290.7	[kips]	AISC 15 <sup>th</sup>	Eq J4-2	
	ratio = <b>0.26</b>			> P <sub>u</sub>	ОК			
	ratio = <b>0.26</b>			> P <sub>u</sub>	OK			
2L Brace to Gusset Plate Weld	ratio = 0.26 Strength		ratio = 76.	> P <sub>u</sub>	ОК = <b>0.40</b>	PASS		
2L Brace to Gusset Plate Weld Fillet Weld Strength Check	ratio = 0.26 Strength		ratio = 76.	> P <sub>u</sub>	ОК = <b>0.40</b>	PASS		
2L Brace to Gusset Plate Weld Fillet Weld Strength Check Fillet weld leg size	ratio = <b>0.26</b> Strength w = <sup>1</sup> / <sub>4</sub>	[in]	ratio = 76. Ioad angle	> P <sub>u</sub> .5 / 189.3 e θ = 0.0	ОК = <b>0.40</b> [°]	PASS		
2L Brace to Gusset Plate Weld Fillet Weld Strength Check Fillet weld leg size Electrode strength	ratio = <b>0.26</b> Strength w = <sup>1</sup> / <sub>4</sub> F <sub>EXX</sub> = 70.0	[in] [ksi]	ratio = 76. load angle strength coeff	> $P_u$ .5 / 189.3 e $\theta$ = 0.0 $C_1$ = 1.00	ок = <b>0.40</b> [°]	PASS AISC 15 <sup>th</sup>	Table 8-3	
2L Brace to Gusset Plate Weld Fillet Weld Strength Check Fillet weld leg size Electrode strength Number of weld line	ratio = <b>0.26</b> <b>Strength</b> $w = \frac{1}{4}$ $F_{EXX} = 70.0$ n = 2 for d	[in] [ksi] louble fillet	ratio = 76. load angle strength coeff	> $P_u$ .5 / 189.3 e $\theta = 0.0$ $C_1 = 1.00$	ок = <b>0.40</b> [°]	PASS AISC 15 <sup>th</sup>	Table 8-3	
2L Brace to Gusset Plate Weld Fillet Weld Strength Check Fillet weld leg size Electrode strength Number of weld line Load angle coefficient	ratio = <b>0.26</b> <b>Strength</b> $w = \frac{1}{4}$ $F_{EXX} = 70.0$ n = 2 for $dC_2 = (1 + 0.0)$	[in] [ksi] louble fillet 5 sin <sup>1.5</sup> θ)	ratio = 76. load angle strength coeff	> $P_u$ .5 / 189.3 e $\theta = 0.0$ C <sub>1</sub> = 1.00 = 1.00	ок = <b>0.40</b> [°]	PASS AISC 15 <sup>th</sup>	Table 8-3 Page 8-9	
2L Brace to Gusset Plate Weld Fillet Weld Strength Check Fillet weld leg size Electrode strength Number of weld line Load angle coefficient Fillet weld shear strength	ratio = <b>0.26</b> <b>Strength</b> $W = \frac{1}{4}$ $F_{EXX} = 70.0$ $n = 2$ for $C_{1}$ $R_{n-w} = 0.6$ ( $C_{1}$	[in] [ksi] louble fillet 5 sin <sup>1.5</sup> θ ) x 70 ksi) 0.7	ratio = 76. load angl strength coeff 07 w n C <sub>2</sub>	> $P_u$ .5 / 189.3 e $\theta = 0.0$ C <sub>1</sub> = 1.00 = 14.85	ОК = <b>0.40</b> [°] [kip/in]	PASS AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup>	Table 8-3 Page 8-9 Eq 8-1	
2L Brace to Gusset Plate Weld Fillet Weld Strength Check Fillet weld leg size Electrode strength Number of weld line Load angle coefficient Fillet weld shear strength Base metal - gusset plate	ratio = <b>0.26</b> <b>Strength</b> $w = \frac{1}{4}$ $F_{EXX} = 70.0$ n = 2 for $cC_2 = (1 + 0.)R_{n-w} = 0.6 (C_1thickness t = 0.500$	[in] [ksi] louble fillet 5 sin <sup>1.5</sup> θ ) x 70 ksi) 0.7 [in]	ratio = 76. load angle strength coeff 07 w n C <sub>2</sub> tensile	> $P_u$ .5 / 189.3 e $\theta = 0.0$ $C_1 = 1.00$ = 14.85 : $F_u = 65.0$	ОК = <b>0.40</b> [°] [kip/in] [ksi]	PASS AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup>	Table 8-3 Page 8-9 Eq 8-1	
2L Brace to Gusset Plate Weld Fillet Weld Strength Check Fillet weld leg size Electrode strength Number of weld line Load angle coefficient Fillet weld shear strength Base metal - gusset plate Base metal - gusset plate is in shear,	ratio = <b>0.26</b> Strength $w = \frac{1}{4}$ $F_{EXX} = 70.0$ $n = 2$ for $C_{1}$ $C_{2} = (1 + 0.0)$ $R_{n-w} = 0.6$ ( $C_{1}$ thickness t = 0.500 <u>shear</u> rupture as per A	[in] [ksi] louble fillet 5 sin <sup>1.5</sup> θ ) x 70 ksi) 0.7 [in] AISC 15 <sup>th</sup> Eq	ratio = 76. load angle strength coeff 07 w n C <sub>2</sub> tensile J4-4 is checked	> $P_u$ .5 / 189.3 e $\theta = 0.0$ C <sub>1</sub> = 1.00 = 14.85 e $F_u = 65.0$	ОК = <b>0.40</b> [°] [kip/in] [ksi]	PASS AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup>	Table 8-3 Page 8-9 Eq 8-1 J2.4	
2L Brace to Gusset Plate Weld Fillet Weld Strength Check Fillet weld leg size Electrode strength Number of weld line Load angle coefficient Fillet weld shear strength Base metal - gusset plate Base metal - gusset plate is in shear, Base metal shear rupture	ratio = <b>0.26</b> Strength $W = \frac{1}{4}$ $F_{EXX} = 70.0$ $n = 2$ for $C_{1}$ $C_{2} = (1 + 0.0)$ $R_{n-w} = 0.6$ ( $C_{1}$ thickness t = 0.500 $\frac{1}{2}$ $\frac{1}{4}$ $C_{2} = (1 + 0.0)$ $C_{1} = 0.0$ $C_{1} = 0.0$ $C_{1} = 0.0$ $C_{2} = 0.0$	[in] [ksi] louble fillet 5 sin <sup>1.5</sup> θ ) x 70 ksi) 0.7 [in] AISC 15 <sup>th</sup> Eq	ratio = 76. load angle strength coeff 07 w n C <sub>2</sub> tensile J4-4 is checked	> $P_u$ .5 / 189.3 e $\theta = 0.0$ C <sub>1</sub> = 1.00 = 14.85 e $F_u = 65.0$ = 19.50	ОК = <b>0.40</b> [°] [kip/in] [ksi] [kip/in]	PASS AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup>	Table 8-3 Page 8-9 Eq 8-1 J2.4 Eq J4-4	
2L Brace to Gusset Plate Weld Fillet Weld Strength Check Fillet weld leg size Electrode strength Number of weld line Load angle coefficient Fillet weld shear strength Base metal - gusset plate Base metal - gusset plate Base metal shear rupture Double fillet linear shear strength	ratio = <b>0.26</b> Strength $w = \frac{1}{4}$ $F_{EXX} = 70.0$ $n = 2$ for $C_{2} = (1 + 0.0)$ $R_{n-w} = 0.6$ ( $C_{1}$ thickness t = 0.500 shear rupture as per $A$ $R_{n-b} = 0.6$ $F_{u}$ t $R_{n} = min$ ( $R$	[in] [ksi] louble fillet 5 sin <sup>1.5</sup> θ ) x 70 ksi) 0.7 [in] AISC 15 <sup>th</sup> Eq <sub>n-w</sub> , R <sub>n-b</sub> )	ratio = 76. load angle strength coeff 07 w n C <sub>2</sub> tensile J4-4 is checked	> $P_u$ .5 / 189.3 e $\theta = 0.0$ C <sub>1</sub> = 1.00 = 14.85 e $F_u = 65.0$ = 19.50 = 14.847	OK = <b>0.40</b> [°] [kip/in] [ksi] [kip/in] [kip/in]	PASS AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup>	Table 8-3 Page 8-9 Eq 8-1 J2.4 Eq J4-4 Eq 9-2	
2L Brace to Gusset Plate Weld Fillet Weld Strength Check Fillet weld leg size Electrode strength Number of weld line Load angle coefficient Fillet weld shear strength Base metal - gusset plate Base metal - gusset plate is in shear, Base metal shear rupture Double fillet linear shear strength Resistance factor-LRFD	ratio = <b>0.26</b> Strength $w = \frac{1}{4}$ $F_{EXX} = 70.0$ $n = 2$ for $C_2 = (1 + 0.0)$ $R_{n-w} = 0.6$ ( $C_1$ thickness t = 0.500 shear rupture as per $A$ $R_{n-b} = 0.6$ $F_u$ t $R_n = min (R_1)$ $\phi = 0.75$	[in] [ksi] louble fillet 5 sin <sup>1.5</sup> θ) x 70 ksi) 0.7 [in] AISC 15 <sup>th</sup> Eq <sub>n-w</sub> , R <sub>n-b</sub> )	ratio = 76. load angle strength coeff 07 w n C <sub>2</sub> tensile	> $P_u$ .5 / 189.3 e $\theta = 0.0$ C 1 = 1.00 = 14.85 e $F_u = 65.0$ = 19.50 = <b>14.847</b>	ОК = 0.40 [°] [kip/in] [ksi] [kip/in] [kip/in]	PASS AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup>	Table 8-3 Page 8-9 Eq 8-1 J2.4 Eq J4-4 Eq 9-2 Eq 8-1	
2L Brace to Gusset Plate Weld Fillet Weld Strength Check Fillet weld leg size Electrode strength Number of weld line Load angle coefficient Fillet weld shear strength Base metal - gusset plate Base metal - gusset plate Base metal - gusset plate Base metal shear rupture Double fillet linear shear strength Resistance factor-LRFD	ratio = 0.26 Strength $w = \frac{1}{4}$ $F_{EXX} = 70.0$ $n = 2 \text{ for } d$ $C_2 = (1 + 0)$ $R_{n-w} = 0.6 (C_1)$ thickness t = 0.500 shear rupture as per A $R_{n-b} = 0.6 F_u t$ $R_n = \min (R_1)$ $\varphi = 0.75$ $\varphi R_n = 0.0000000000000000000000000000000000$	[in] [ksi] louble fillet 5 sin <sup>1.5</sup> θ) x 70 ksi) 0.7 [in] AISC 15 <sup>th</sup> Eq <sub>n-w</sub> , R <sub>n-b</sub> )	ratio = 76. load angle strength coeff 07 w n C <sub>2</sub> tensile J4-4 is checked	> $P_u$ .5 / 189.3 e $\theta = 0.0$ C <sub>1</sub> = 1.00 = 14.85 e $F_u = 65.0$ = 19.50 = 14.847 = 11.135	ОК = <b>0.40</b> [°] [kip/in] [ksi] [kip/in] [kip/in]	PASS AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup>	Table 8-3 Page 8-9 Eq 8-1 J2.4 Eq J4-4 Eq 9-2 Eq 8-1	
2L Brace to Gusset Plate Weld Fillet Weld Strength Check Fillet weld leg size Electrode strength Number of weld line Load angle coefficient Fillet weld shear strength Base metal - gusset plate Base metal - gusset plate is in shear, Base metal shear rupture Double fillet linear shear strength Resistance factor-LRFD Weld resistance required	ratio = 0.26 Strength $w = \frac{1}{4}$ $F_{EXX} = 70.0$ $n = 2 \text{ for } d$ $C_2 = (1 + 0.)$ $R_{n-w} = 0.6 (C_1)$ thickness t = 0.500 shear rupture as per A $R_{n-b} = 0.6 F_u t$ $R_n = \min (R_1)$ $\phi = 0.75$ $\phi R_n = V_u = V_u = V_u = V_u = V_u$	[in] [ksi] louble fillet 5 sin <sup>1.5</sup> θ) x 70 ksi) 0.7 [in] AISC 15 <sup>th</sup> Eq	ratio = 76. load angle strength coeff 07 w n C <sub>2</sub> tensile J4-4 is checked	> $P_u$ .5 / 189.3 e $\theta = 0.0$ C <sub>1</sub> = 1.00 = 14.85 e $F_u = 65.0$ = 19.50 = 14.847 = 11.135 = 76.5	ОК = <b>0.40</b> [°] [kip/in] [ksi] [kip/in] [kip/in] [kip/in]	PASS AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup>	Table 8-3 Page 8-9 Eq 8-1 J2.4 Eq J4-4 Eq 9-2 Eq 8-1	
2L Brace to Gusset Plate Weld         Fillet Weld Strength Check         Fillet weld leg size         Electrode strength         Number of weld line         Load angle coefficient         Fillet weld shear strength         Base metal - gusset plate         Base metal - gusset plate is in shear,         Base metal shear rupture         Double fillet linear shear strength         Resistance factor-LRFD         Weld resistance required         Fillet weld lenoth - double fillet	ratio = <b>0.26</b> Strength $w = \frac{1}{4}$ $F_{EXX} = 70.0$ $n = 2 \text{ for } d$ $C_2 = (1 + 0.0)$ $R_{n-w} = 0.6 (C_1)$ thickness t = 0.500 shear rupture as per 4 $R_{n-b} = 0.6 F_u t$ $R_n = \min (R_1)$ $\phi = 0.75$ $\phi R_n =$ $V_u =$ $L =$	[in] [ksi] louble fillet 5 sin <sup>1.5</sup> θ) x 70 ksi) 0.7 [in] AISC 15 <sup>th</sup> Eq <sub>n-w</sub> , R <sub>n-b</sub> )	ratio = 76. load angle strength coeff 07 w n C <sub>2</sub> tensile J4-4 is checked	> $P_u$ .5 / 189.3 e $\theta = 0.0$ C <sub>1</sub> = 1.00 = 14.85 e $F_u = 65.0$ = 19.50 = 14.847 = 11.135 = 76.5 = 17.000	ОК = 0.40 [°] [kip/in] [kip/in] [kip/in] [kip/in] [kip/in]	PASS AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup>	Table 8-3 Page 8-9 Eq 8-1 J2.4 Eq J4-4 Eq 9-2 Eq 8-1	
2L Brace to Gusset Plate Weld Fillet Weld Strength Check Fillet weld leg size Electrode strength Number of weld line Load angle coefficient Fillet weld shear strength Base metal - gusset plate Base metal - gusset plate is in shear, Base metal shear rupture Double fillet linear shear strength Resistance factor-LRFD Weld resistance required Fillet weld length - double fillet Weld resistance provided	ratio = 0.26 Strength $w = \frac{1}{4}$ $F_{EXX} = 70.0$ $n = 2 \text{ for } d$ $C_2 = (1 + 0.0)$ $R_{n-w} = 0.6 (C_1)$ thickness t = 0.500 shear rupture as per A $R_{n-b} = 0.6 F_u t$ $R_n = \min (R_1)$ $\phi = 0.75$ $\phi R_n =$ $V_u =$ $L =$ $\phi F_n = \phi R_n \times 1$	[in] [ksi] louble fillet 5 sin <sup>1.5</sup> θ) x 70 ksi) 0.7 [in] AISC 15 <sup>th</sup> Eq <sub>n-w</sub> , R <sub>n-b</sub> )	ratio = 76. load angle strength coeff 07 w n C <sub>2</sub> tensile J4-4 is checked	> $P_u$ .5 / 189.3 e $\theta = 0.0$ C <sub>1</sub> = 1.00 = 1.00 = 14.85 e $F_u = 65.0$ = 19.50 = 14.847 = 11.135 = 76.5 = 17.000 = 189.3	ОК = 0.40 [°] [kip/in] [kip/in] [kip/in] [kip/in] [kip/in] [kips] [in]	PASS AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup>	Table 8-3 Page 8-9 Eq 8-1 J2.4 Eq J4-4 Eq 9-2 Eq 8-1	
2L Brace to Gusset Plate Weld         Fillet Weld Strength Check         Fillet weld leg size         Electrode strength         Number of weld line         Load angle coefficient         Fillet weld shear strength         Base metal - gusset plate         Base metal - gusset plate is in shear,         Base metal shear rupture         Double fillet linear shear strength         Resistance factor-LRFD         Weld resistance required         Fillet weld length - double fillet         Weld resistance provided	ratio = <b>0.26</b> Strength $w = \frac{1}{4}$ $F_{EXX} = 70.0$ $n = 2 \text{ for } C$ $C_2 = (1 + 0.0)$ $R_{n-w} = 0.6 (C_1)$ thickness t = 0.500 shear rupture as per A $R_{n-b} = 0.6 F_u t$ $R_n = \min (R)$ $\phi = 0.75$ $\phi R_n =$ $V_u =$ $L =$ $\phi F_n = \phi R_n \times L$ $ratio = 0.40$	[in] [ksi] louble fillet 5 sin <sup>1.5</sup> θ) x 70 ksi) 0.7 [in] AISC 15 <sup>th</sup> Eq <sub>n-w</sub> , R <sub>n-b</sub> )	ratio = 76. load angle strength coeff 07 w n C <sub>2</sub> tensile J4-4 is checked	> $P_u$ .5 / 189.3 e $\theta = 0.0$ C $_1 = 1.00$ = 14.85 e $F_u = 65.0$ = 19.50 = 14.847 = 11.135 = 76.5 = 17.000 = 189.3 > $V_u$	ОК = 0.40 [°] [kip/in] [kip/in] [kip/in] [kip/in] [kip/in] [kip/in] [kips] [in] [Kips] ОК	PASS AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup> AISC 15 <sup>th</sup>	Table 8-3 Page 8-9 Eq 8-1 J2.4 Eq J4-4 Eq 9-2 Eq 8-1	

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2, 9:38 PM AISC Steel Co	onnection Design	http://asp.civi	lbay.com/conne	ct V	ertical Brac	e Connectior	י ו	VB-Rigl
Gusset Plate - Compression (W	/hitmore)		ratio = 76.5	/ 248.1	= 0.31	PASS		
Plate Compression Check								
Plate size	width b <sub>p</sub> = 11.928	[in]	thickness t	<sub>p</sub> = 0.500	[in]			
	$F_{y} = 50.0$	[ksi]	I	E = 29000.0	) [ksi]			
Plate gross area in compression	$A_g = b_p t_p$			= 5.964	[in <sup>2</sup> ]			
Plate radius of gyration	$r = t_p / \sqrt{1}$	2		= 0.144	[in]			
Plate effective length factor	K =			= 0.60				
Plate unbraced length	L <sub>u</sub> =			= 7.884	[in]			
Plate slenderness	KL/r = 0.60 x L	<sub>u</sub> / r		= 32.77				
	when $\frac{KL}{r} > 2$	25 , use Chapte	er E			AISC 15 <sup>th</sup>	l4.4 (b)	
Elastic buckling stress	$F_e = \frac{\pi^2 E}{(KL/r)}$	2		= 266.5	[ksi]	AISC 15 <sup>th</sup> I	Eq E3-4	
	KL when <u> </u>	4.71 ( <mark>E</mark> ) <sup>0.5</sup>	5 = 113.43			AISC 15 <sup>th</sup> I	E3 (a)	
Critical stress	$F_{cr} = 0.658$ <sup>(F</sup>	<sup>F</sup> y <sup>/F</sup> e) Fy		= 46.2	[ksi]	AISC 15 <sup>th</sup> I	Eq E3-2	
Plate compression required	P <sub>u</sub> =			= 76.5	[kips]			
Plate compression provided	$R_n = F_{cr} x A_g$			= 275.7	[kips]	AISC 15 <sup>th</sup>	Eq E3-1	
Resistance factor-LRFD	φ = 0.90					AISC 15 <sup>th</sup>	∃1	
	$\phi R_n =$			= 248.1	[kips]			
	ratio = <b>0.31</b>			> P <sub>u</sub>	OK			
2L Brace to Gusset Plate Weld	Strength		ratio = 76.5	/ 189.3	= 0.40	PASS		
Fillet Weld Strength Check								
Fillet weld leg size	$w = \frac{1}{2}$	[in]	load angle (	9 – 0 0	٢٥٦			
Electrode strength	F <sub>EXX</sub> = 70.0	[ksi]	strength coeff C	1 = 1.00		AISC 15 <sup>th</sup>	Table 8-3	
Number of weld line	n = 2 for do	ouble fillet	5	1				
Load angle coefficient	$C_2 = (1 + 0.5)$	5 sin <sup>1.5</sup> θ)		= 1.00		AISC 15 <sup>th</sup>	Page 8-9	
Fillet weld shear strength	R <sub>n-w</sub> = 0.6 (C <sub>1</sub> )	< 70 ksi) 0.707	wnC <sub>2</sub>	= 14.85	[kip/in]	AISC 15 <sup>th</sup> I	Eq 8-1	
Base metal - gusset plate	thickness t = 0.500	[in]	tensile F	<sub>u</sub> = 65.0	[ksi]			
Base metal - gusset plate is in shear,	shear rupture as per A	ISC 15 <sup>th</sup> Eq J4	-4 is checked			AISC 15 <sup>th</sup>	12.4	
Base metal shear rupture	$R_{n-b} = 0.6 F_{u}t$			= 19.50	[kip/in]	AISC 15 <sup>th</sup>	Eq J4-4	
Double fillet linear shear strength	R <sub>n</sub> = min ( R <sub>n</sub>	<sub>I-w</sub> , R <sub>n-b</sub> )		= 14.847	[kip/in]	AISC 15 <sup>th</sup>	Eq 9-2	
Resistance factor-LRFD	φ = 0.75					AISC 15 <sup>th</sup>	Eq 8-1	
	φ R <sub>n</sub> =			= 11.135	[kip/in]			
Weld resistance required	V <sub>u</sub> =			= 76.5	[kips]			
Fillet weld length - double fillet	L =			= 17.000	[in]			
Weld resistance provided	$\phi F_n = \phi R_n x L$			= 189.3	[kips]			
	ratio = <b>0.40</b>			> V <sub>u</sub>	ОК			

2, 9.00 MINI /	S PM AISC Steel Connection Design http://asp.civilbay.com/connect					
Gusset to Column	End Plate Conn	ection			Code=AISC 360-:	16 LRFD
Result Summary	geometries & weld limita	ations = <b>PASS</b>	limit states m	nax ratio = <b>0</b> .	48 PASS	
Geometry Restriction	on Checks - End Plate to Colu	mn Flange			PASS	
Min Bolt Edge Distand	ce - End Plate to Column Flang	e				
Bolt diameter	d <sub>b</sub> =		= 0.	750 [in]		
Min edge distance allow	red L <sub>e-min</sub> =		= 1.	. <b>000</b> [in]	AISC 15 <sup>th</sup> Ta	ble J3.4
Column Flange	$L_e =$		= 1.	.375 [in]		
			≥ L <sub>e</sub>	e-min OK		
Min Bolt Spacing - En	d Plate to Column Flange					
Bolt diameter	d <sub>b</sub> =		= 0.	750 [in]		
Min bolt spacing allowed	d L <sub>s-min</sub> = 2.667	' d <sub>b</sub>	= 2.	. <b>000</b> [in]	AISC 15 <sup>th</sup> J3	.3
Column Flange	Plate to L <sub>s</sub> =		= 3.	. <b>500</b> [in]		
			≥ L <sub>s</sub>	s-min OK		
Geometry Restriction	on Checks - End Plate - Bolt G	age Clearance			PASS	
Bolt Gage Entering Cl	learance Check - Plate Welded	to End Plate				
Bolt diameter	d = 0.750	) [in]	aaaa a - 3	500 [in]		
Bolt entering clearance	$a_b = 0.750$	AISC manual Table 7-	yaye y = 5.	. <b>750</b> [in]	AISC 15 <sup>th</sup> Ta	ble 7-15
Plate thickness	t = 0.500	) [in]	dbl fillet w = 0.	250 [in]	110010 10	510 / 20
Bolt center clearance di	stance to $C = (q - 1)$	t - 2 w ) / 2	= 1.	250 [in]		
fillet toe		, , _	 ≥ C.	, OK	AISC 15 <sup>th</sup> Tab	le 7-15
Geometry Restrictio	on Checks-Column Flg-Bolt Ga	ige Clearance			PASS	
		ipe riange				
Bolt diameter	$d_{b} = 0.750$	) [in]	gage $g = 3$ .	500 [in]		
Bolt entering clearance	$C_3 = from$	AISC manual Table 7-	15 = <b>0</b> .	. <b>750</b> [in]	AISC 15 <sup>th</sup> Ta	ble 7-15
W section	$t_w = 0.315$	5 [in]	$k_1 = 0.$	748 [in]		
fillet toe	c = (g -	2 k <sub>1</sub> )/2	= 1.	. <b>002</b> [in]		
			≥ c <sub>3</sub>	3 OK	AISC 15 <sup>th</sup> Tab	le 7-15
Weld Limitation Ch	ecks - Gusset Plate to End Pla	ite			PASS	
Min Fillet Weld Size						
Thinner part joined thic	kness t =		= 0.	500 [in]		
Min fillet weld size allow	ved w <sub>min</sub> =		= 0.	. <b>188</b> [in]	AISC 15 <sup>th</sup> Ta	ble J2.4
Fillet weld size provided	i w =		= 0.	. <b>250</b> [in]		
			≥ w	min OK		
Min Fillet Weld Lengt	h					
Fillet weld size provided	i w =		= 0.	250 [in]		
Min fillet weld length all	lowed $L_{min} = 4 \times w$	,	= 1.	. <b>000</b> [in]	AISC 15 <sup>th</sup> J2	.2b
Min fillet weld length	L =		= 10	<b>0.500</b> [in]		
			>	. OK		

AISC Steel Connection Design

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Brace Force Load Case 1	Gusset plate t=0.500	P =-76.5 kips (T) rat	io = <b>0.48</b>	PASS
Gusset Plate - Shear Yielding		ratio = 34.2 / 212.4	= 0.16	PASS
Calculate gusset or stiff plate length o	utside beam flange and count it as b	eam web extension to resist	shear	
Beam sect depth W200x36	d <sub>b</sub> = 7.913 [in]			
Bolt pitch & edge distance	d <sub>1</sub> = 2.500 [in]	e <sub>v</sub> = 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 <sub>p</sub> = 14.163 [in]	thickness $t_p = 0.500$	[in]	
Plate yield strength	F <sub>y</sub> = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$	= 7.082	[in <sup>2</sup> ]	
Shear force required	V <sub>u</sub> =	= 34.2	[kips]	
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$	= 212.4	[kips]	AISC 15 <sup>th</sup> Eq J4-3
Resistance factor-LRFD	$\varphi = 1.00$			AISC 15 <sup>th</sup> Eq J4-3
	φ R <sub>n</sub> =	= 212.4	[kips]	
	ratio = <b>0.16</b>	> V <sub>u</sub>	OK	
Gusset Plate - Shear Rupture		ratio = 34.2 / 207.1	= 0.17	PASS
Calculate gusset or stiff plate length o	utside beam flange and count it as b	eam web extension to resist	shear	
Beam sect depth W200x36	d <sub>b</sub> = 7.913 [in]			
Bolt pitch & edge distance	d <sub>1</sub> = 2.500 [in]	e <sub>v</sub> = 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 <sub>u</sub> = 65.0 [ksi]			
Plate net area in shear	$A_{nv} = b_p t_p$	= 7.082	[in <sup>2</sup> ]	
Shear force in demand	V <sub>u</sub> =	= 34.2	[kips]	
Plate shear rupture strength	$R_n = 0.6 F_u A_{nv}$	= 276.2	[kips]	AISC 15 <sup>th</sup> Eq J4-4
Resistance factor-LRFD	φ = 0.75			AISC 15 <sup>th</sup> Eq J4-4
	$\phi R_n =$	= 207.1	[kips]	

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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 <sup>2</sup> ]		
Tensile force required	P <sub>u</sub> =		= 21.9	[kips]		
Plate tensile yielding strength	$R_n = F_y A_g$		= 293.2	[kips]	AISC 15 <sup>th</sup> Eq J4-1	
Resistance factor-LRFD	φ = 0.90				AISC 15 <sup>th</sup> Eq J4-1	
	φ R <sub>n</sub> =		= 263.9	[kips]		
	ratio = <b>0.08</b>		> P <sub>u</sub>	ОК		
Gusset Plate - Axial Tensile Ru	upture		ratio = 21.9 / 285.8	= 0.08	PASS	
Plate Axial Tensile Rupture Chec	:k					
Plate size	width $b_p = 11.727$	[in]	thickness $t_p = 0.500$	[in]		
Plate tensile strength	F <sub>u</sub> = 65.0	[ksi]	F			
Plate gross area	$A_n = b_p t_p$		= 5.864	[in <sup>2</sup> ]		
Shear lag factor	U =		= 1.000			
Tensile force required	P <sub>u</sub> =		= 21.9	[kips]		
Tensile effective net area	$A_e = A_n U$		= 5.864	[in <sup>2</sup> ]		
Plate tensile strength	F <sub>u</sub> =		= 65.0	[ksi]		
Tensile rupture strength	$R_n = F_u A_e$		= 381.1	[kips]	AISC $15^{th}$ Eq D2-2	
Resistance factor-LRFD	φ = 0.75				AISC 15 <sup>th</sup> D2 (b)	
	φ R <sub>n</sub> =		= 285.8	[kips]	AISC $15^{th}$ Eq D2-2	
	ratio = <b>0.08</b>		> P <sub>u</sub>	ОК		
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 <sup>2</sup> ]		
Shear force required	V <sub>u</sub> =		= 17.1	[kips]		
Plate shear yielding strength	$R_{n} = 0.6 F_{y}A$	A <sub>gv</sub>	= 131.3	[kips]	AISC 15 <sup>th</sup> Eq J4-3	
Resistance factor-LRFD	$\varphi = 1.00$				AISC 15 <sup>th</sup> Eq J4-3	
	φ R <sub>n</sub> =		= 131.3	[kips]		

 $> V_u$ 

OK

ratio = **0.13** 

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End Plate - Shear Rupture			ratio = 17.1 / 96.0	= 0.18	PASS
Plate Shear Rupture Check			,		
Bolt hole diameter	bolt dia $d_1 = \frac{3}{4}$	[in]	bolt hole dia $d_1 = \frac{7}{2}$	[in]	AISC 15 <sup>th</sup> B4 3b
Number of bolt	n = 2	[]	bole hole and a <sub>h</sub> , g	[]	
Plate size	width $b = 7.000$	[in]	thickness $t = 0.6$	25 [in]	
Plate tensile strength	$F_{\rm u} = 65.0$	[ksi]	enterniede ep ere		
Plate net area in shear	A = (b n	d <sub>+</sub> )t <sub>-</sub>	= 3.2	81 [in <sup>2</sup> ]	
Shear force required	V., =	- пи -р	= 17	.1 [kips]	
Plate shear rupture strength	R <sub>n</sub> = 0.6 F <sub>11</sub> A	Any	= 128	3.0 [kips]	AISC 15 <sup>th</sup> Eq J4-4
Resistance factor-LRFD	$\phi = 0.75$	iiv			AISC 15 <sup>th</sup> Eq J4-4
	$\phi R_n =$		= 96	. <b>0</b> [kips]	
	ratio = <b>0.18</b>		> V <sub>u</sub>	OK	
End Plate - Block Shear - Cente	er Strip		ratio = 34.2 / 223.9	9 = <b>0.15</b>	PASS
Plate Block Shear - Center Strip					
Bolt hole diameter	bolt dia d <sub>b</sub> = $\frac{3}{4}$	[in]	bolt hole dia d <sub>h</sub> = $\frac{7}{8}$	[in]	AISC 15 <sup>th</sup> 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 <sub>v</sub> = 2		n <sub>h</sub> = 2		
Bolt spacing in ver & hor dir	$s_v = 3.500$	[in]	s <sub>h</sub> = 3.5	00 [in]	
Bolt edge dist in ver & hor dir	e <sub>v</sub> = 1.375	[in]	e <sub>h</sub> = 1.7	50 [in]	
Gross area subject to shear	A <sub>gv</sub> = [ (n <sub>h</sub> - :	1)s <sub>h</sub> +e <sub>h</sub> ]t <sub>p</sub> x	2 = 6.5	63 [in <sup>2</sup> ]	
Net area subject to shear	$A_{nv} = A_{gv} - [($	n <sub>h</sub> -1)+0.5]d <sub>r</sub>	$t_{p}x^{2} = 4.9$	22 [in <sup>2</sup> ]	
Net area subject to tension					
when sheared out by center strip	$A_{nt} = (n_v - 1)$	)(s <sub>v</sub> -d <sub>h</sub> )t <sub>p</sub>	= 1.6	41 [in <sup>2</sup> ]	
Block shear strength required	V <sub>u</sub> =		= 34	.2 [kips]	
Uniform tension stress factor	$U_{bs} = 1.00$				AISC 15 <sup>th</sup> Fig C-J4.2
Bolt shear resistance provided	$R_n = min (0.$	6F <sub>u</sub> A <sub>nv</sub> , 0.6F <sub>y</sub> A	A <sub>gv</sub> ) + = 298	3.6 [kips]	AISC 15 <sup>th</sup> Eq J4-5
	U <sub>bs</sub> F <sub>u</sub> A	A <sub>nt</sub>			- th
Resistance factor-LRFD	$\phi = 0.75$				AISC 15" Eq J4-5
	$\phi R_n =$		= 22	3.9 [kips]	
	ratio = <b>0.15</b>		> V <sub>u</sub>	UK	

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End Plate - Block Shear - 2-Sid	e Strip		ratio = 34.2 / 201.1	= 0.17	PASS	
Plate Block Shear - 2 Side Strips						
Bolt hole diameter	bolt dia d <sub>b</sub> = $\frac{3}{4}$	[in]	bolt hole dia d <sub>h</sub> = $\frac{7}{8}$	[in]	AISC 15 <sup>th</sup> 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 <sub>v</sub> = 2		n <sub>h</sub> = 2			
Bolt spacing in ver & hor dir	$s_v = 3.500$	[in]	s <sub>h</sub> = 3.500	[in]		
Bolt edge dist in ver & hor dir	e <sub>v</sub> = 1.375	[in]	e <sub>h</sub> = 1.750	[in]		
Gross area subject to shear	A <sub>gv</sub> = [ (n <sub>h</sub> - :	1) $s_h + e_h$ ] $t_p x$	2 = 6.563	[in <sup>2</sup> ]		
Net area subject to shear	$A_{nv} = A_{gv} - [($	n <sub>h</sub> -1)+0.5]d <sub>h</sub>	$t_p x^2 = 4.922$	[in <sup>2</sup> ]		
Net area subject to tension						
when sheared out by 2 side strips	$A_{nt} = (e_v - 0)$	.5 d <sub>h</sub> )t <sub>p</sub> x 2	= 1.172	[in <sup>2</sup> ]		
Block shear strength required	V <sub>u</sub> =		= 34.2	[kips]		
Uniform tension stress factor	$U_{bs} = 1.00$				AISC 15 <sup>th</sup> Fig C-J4	.2
Bolt shear resistance provided	R <sub>n</sub> = min (0.	6F <sub>u</sub> A <sub>nv</sub> , 0.6F <sub>y</sub> A	$(A_{gv}) + = 268.1$	[kips]	AISC 15 <sup>th</sup> Eq J4-5	
Resistance factor-LRFD	$\phi = 0.75$	nt			AISC 15 <sup>th</sup> Eq J4-5	
	φ R <sub>n</sub> =		= 201.1	L [kips]		
	ratio = <b>0.17</b>		> V <sub>u</sub>	ОК		

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Vertical Brace Connection

End Plate - Bolt Bearing on En	d Plate		ratio = 34.2 / 71.6	= 0.48	PASS	
Single Bolt Shear Strength						
Bolt shear stress	bolt grade = A325-N		F <sub>nv</sub> = 54.0	[ksi]	AISC 15 <sup>th</sup>	Table J3.2
	bolt dia $d_b = 0.750$	[in]	bolt area $A_b = 0.442$	[in <sup>2</sup> ]		
Single bolt shear strength	$R_{n-bolt} = F_{nv}A_b$		= 23.9	[kips]	AISC $15^{\text{th}}$	Eq J3-1
Bolt Bearing/TearOut Strength	on Plate					
Bolt hole diameter	bolt dia $d_b = \frac{3}{4}$	[in]	bolt hole dia d <sub>h</sub> = $\frac{13}{16}$	[in]	AISC 15 <sup>th</sup>	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	$R_{n-t\&b-in} = 1.5 L_c t$	F <sub>u</sub> ≤ 3.0 d <sub>b</sub> t F	u		AISC $15^{th}$	Eq J3-6b
	= 163.8 ≤	91.4	= 91.4	[kips]		
Bolt strength at interior	R <sub>n-in</sub> = min ( R	n-t&b-in , R n-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	$R_{n-t\&b-ed} = 1.5 L_c t$	F <sub>u</sub> ≤ 3.0 d <sub>b</sub> t F	u		AISC $15^{\text{th}}$	Eq J3-6b
	= 81.9 ≤ °	91.4	= 81.9	[kips]		
Bolt strength at edge	$R_{n-ed} = min (R)$	n-t&b-ed , R n-bolt	) = 23.9	[kips]		
Number of bolt	interior n <sub>in</sub> = 2		edge n <sub>ed</sub> = 2			
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n-in}$	+ n <sub>ed</sub> R <sub>n-ed</sub>	= 95.4	[kips]		
Required shear strength	V <sub>u</sub> =		= 34.2	[kips]		
Bolt resistance factor-LRFD	φ = 0.75				AISC $15^{\text{th}}$	J3.10
	φ R <sub>n</sub> =		= 71.6	[kips]		
	ratio = <b>0.48</b>		> V	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 <sup>2</sup> ]	
Bolt shear stress	grade = A325-N		$F_{nv} = 54.0$	[ksi]	AISC 15 <sup>th</sup> Table J3.2
Number of bolt carried shear	n <sub>s</sub> = 4.0		shear plane $m = 1$		
Bolt group eccentricity coefficient	C <sub>ec</sub> =		= 1.000		
Required shear strength	V <sub>u</sub> =		= 34.2	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n$	n <sub>s</sub> m C <sub>ec</sub>	= 95.4	[kips]	AISC 15 <sup>th</sup> Eq J3-1
Bolt resistance factor-LRFD	φ = 0.75				AISC 15 <sup>th</sup> Eq J3-1
	φ R <sub>n</sub> =		= 71.6	[kips]	
	ratio = <b>0.48</b>		> V <sub>u</sub>	OK	

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End Plate / C	olumn - Bolt Bea	ring on Column		ratio = 34.2 / 71.6	= 0.48	PASS	
Single Bolt She	ar Strength						
Bolt shear stress	5	bolt grade = A325-N		$F_{nv} = 54.0$	[ksi]	AISC 15 <sup>th</sup> Table	e J3.2
		bolt dia d <sub>b</sub> = $0.750$	[in]	bolt area $A_b = 0.442$	2 [in <sup>2</sup> ]		
Single bolt shear	strength	$R_{n-bolt} = F_{nv}A_b$		= 23.9	[kips]	AISC 15 <sup>th</sup> Eq J	3-1
Bolt Bearing/T	earOut Strength o	on Plate					
Bolt hole diamet	er	bolt dia d <sub>b</sub> = $\frac{3}{4}$	[in]	bolt hole dia $d_h = \frac{13}{16}$	[in]	AISC 15 <sup>th</sup> Table	e J3.3
Bolt spacing		spacing L <sub>s</sub> = 3.500	[in]				
Plate tensile stre	ngth	F <sub>u</sub> = 65.0	[ksi]				
Plate thickness		t = 0.531	[in]				
Interior Bolt							
Bolt hole edge cl	ear distance	$L_c = L_s - d_h$		= 2.688	8 [in]		
Bolt tear out/bea	aring strength	$R_{n-t\&b-in} = 1.5 L_c t$	F <sub>u</sub> ≤ 3.0 d <sub>b</sub> t m	F <sub>u</sub>		AISC 15 <sup>th</sup> Eq J	3-6b
		= 139.1 ≤	77.7	= 77.7	[kips]		
Bolt strength at	interior	R <sub>n-in</sub> = min ( R	<sub>n-t&amp;b-in</sub> , R <sub>n-bolt</sub> )	= 23.9	[kips]		
Number of bolt		interior n <sub>in</sub> = 4					
Bolt bearing stre	ngth for all bolts	$R_n = n_{in} R_{n-in}$	I	= 95.4	[kips]		
Required shear s	strength	V <sub>u</sub> =		= 34.2	[kips]		
Bolt resistance fa	actor-LRFD	φ = 0.75				AISC 15 <sup>th</sup> J3.10	)
		φ R <sub>n</sub> =		= 71.6	[kips]		
		ratio = <b>0.48</b>		> V	ОК		

Bolt Tensile Prving Action on F	nd Plate		ratio = $5.5 / 20.9$	= 0.26	PASS
Bolt group forces	shear V = 34.2	[kips]	axial P = -2	- 0.20	FASS
	_				
Single Bolt Tensile Capacity With	out Considering Pry	ing		<b>)</b> -	
Bolt grade A325-N	dia d <sub>b</sub> = 0.750	[in]	area $A_b = 0.4$	442 [in²]	
Nominal tensile/shear stress	$F_{nt} = 90.0$	[ksi]	F <sub>nv</sub> = 54	.0 [ksi]	AISC 15 <sup>th</sup> 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 <sub>b</sub> )	= 19	.4 [ksi]	
Resistance factor-LRFD	φ = 0.75				AISC 15 <sup>th</sup> J3.7
Modified nominal tensile stress	$F'_{nt} = 1.3 F_{nt}$	$-\frac{F_{nt}}{\phi F_{nv}}f_{rv} \le$	F <sub>nt</sub> = <b>7</b> 4	<b>I.0</b> [ksi]	AISC 15 <sup>th</sup> Eq J3-3a
Bolt norminal tensile strength	$r_n = F'_{nt}A_b$		= 32	.7 [kips]	AISC 15 <sup>th</sup> Eq J3-1
Resistance factor-LRFD	φ = 0.75				AISC 15 <sup>th</sup> J3.6
Single bolt tensile capacity	φ r <sub>n</sub> =		= 24	<b>I.5</b> [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 <sub>b</sub> ≤ (1.25 b	$+ 0.5 d_{b}) = 1.$	750 [in]	AISC 15 <sup>th</sup> Eq 9-23
Bolt hole diameter	bolt dia d <sub>b</sub> = 0.750	[in]	bolt hole dia $d_h = 0.1$	813 [in]	AISC 15 <sup>th</sup> B4.3b
Dist from bolt center to face of web	b = 0.5(g -	t <sub>w</sub> )	= 1.	500 [in]	
	b' = b - 0.5	d <sub>b</sub>	= 1.	125 [in]	AISC 15 <sup>th</sup> Eq 9-18
Bolt pitch spacing	s <sub>v</sub> = 3.500	[in]			
Bolt tributary length	$p = s_v p$	$\leq$ 2b and p $\leq$	s <sub>v</sub> = 3.	000 [in]	AISC 15 <sup>th</sup> Page 9-12
	ρ = b' / a'		= 0.	643	AISC 15 <sup>th</sup> Eq 9-22
	$\delta = 1 - d_h/$	р	= 0.	729	AISC 15 <sup>th</sup> Eq 9-20
Tensile capacity per bolt before	B = from ca	lc shown in ab	ove section = 24	.5 [kips]	
Resistance factor-LRFD	$\phi = 0.90$				AISC 15 <sup>th</sup> Page 9-12
End plate thickness	t = 0.625	[in]	tensile F = 65	0 [ksi]	
Plate thickness regid to develop bolt	4 B I	o' 0.5		[]	th
tensile capacity without prying	$t_c = ( \phi p f$	)	= 0.	793 [in]	AISC 15" Eq 9-26a
	$\alpha' = \frac{1}{\delta (1 + \delta)}$	$\frac{t_c}{\rho}$ [ $(\frac{t_c}{t})^2$	- 1 ] = 0.	509	AISC 15 <sup>th</sup> Eq 9-28
when $0 \le \alpha' \le 1$	$Q = \left(\frac{t}{t}\right)^2$	<sup>2</sup> (1 + δα')	= 0.	852	AISC 15 <sup>th</sup> Eq 9-27
Bolt tensile force per bolt in demand	د T = from ca	lc shown helo	N = 5.	<b>5</b> [kins]	
Tensile strength per bolt after			5.	- [bo]	Moodsth =
considering prying	$\phi r_n = B \times Q$		= 20	<b>9.9</b> [kips]	AISC 15" Eq 9-27
	ratio = <b>0.26</b>		> 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 $n_v = 2$		
3olt tensile force per bolt	T = P / ( n	, n <sub>h</sub> )	= 5.	5 [kips]	
Bolt Tensile Prying Action on C	olumn Flange		ratio = 5.5 / 18.8	= 0.29	PASS
	shear V = 34.2	[kins]	avial P = -2	19 [kins]	

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Single Bolt Tensile Capacity With	out Considering Pryi	ng			
Bolt grade A325-N	dia d <sub>b</sub> = $0.750$	[in]	area $A_b = 0.44$	42 [in <sup>2</sup> ]	
Nominal tensile/shear stress	F <sub>nt</sub> = 90.0	[ksi]	F <sub>nv</sub> = 54.0	0 [ksi]	AISC 15 <sup>th</sup> Table J3.2
Bolt group shear force	shear V = 34.2	[kips]	no of bolt $n = 4$		
Shear stress required	$f_{rv} = V / (n A)$	А <sub>b</sub> )	= 19.4	4 [ksi]	
Resistance factor-LRFD	φ = 0.75				AISC 15 <sup>th</sup> J3.7
Modified nominal tensile stress	$F'_{nt} = 1.3 F_{nt}$	$-\frac{F_{nt}}{\phi F_{nv}}f_{rv} \le F_{nt}$	= 74.	<b>0</b> [ksi]	AISC 15 <sup>th</sup> Eq J3-3a
Bolt norminal tensile strength	$r_n = F'_{nt}A_b$		= 32.7	7 [kips]	AISC 15 <sup>th</sup> Eq J3-1
Resistance factor-LRFD	φ = 0.75				AISC 15 <sup>th</sup> J3.6
Single bolt tensile capacity	φ r <sub>n</sub> =		= 24.	<b>5</b> [kips]	
Single Bolt Tensile Capacity After	Considering Prying				
Column flange as tee	b <sub>f</sub> = 7.992	[in]	bolt gage $g = 3.50$	00 [in]	
	web $t_{w} = 0.315$	[in]			
Dist from bolt center to flange edge	a <sub>cf</sub> = 0.5 ( b <sub>f</sub>	-g)	= 2.24	46 [in]	
End plate	width w = 6.250	[in]	bolt gage $g = 3.50$	00 [in]	
Dist from bolt center to plate edge	$a_{pl} = 0.5$ ( w	-g)	= 1.3	75 [in]	
Dist from bolt center to plate edge	 a = min ( a,	<sub>cf</sub> , a <sub>pl</sub> )	= 1.3	75 [in]	
	a' = a + 0.5	d <sub>b</sub> ≤ (1.25 b + 0	$(.5 d_b) = 1.75$	50 [in]	AISC 15 <sup>th</sup> Eq 9-23
Bolt hole diameter	bolt dia d <sub>b</sub> = 0.750	[in]	bolt hole dia $d_h = 0.83$	13 [in]	AISC 15 <sup>th</sup> B4.3b
Dist from bolt center to face of web	b = 0.5(g - 1)	t <sub>w</sub> )	= 1.59	93 [in]	
	b' = b - 0.5 c	d <sub>b</sub>	= 1.2	18 [in]	AISC 15 <sup>th</sup> Eq 9-18
Bolt pitch spacing	s <sub>v</sub> = 3.500	[in]			
Bolt tributary length	$p = s_v p =$	$\leq$ 2b and p $\leq$ s <sub>v</sub>	= 3.18	85 [in]	AISC 15 <sup>th</sup> Page 9-12
	 ρ = b' / a'		= 0.69	96	AISC 15 <sup>th</sup> Eq 9-22
	$\delta = 1 - d_{\rm h}/$	р	= 0.74	45	AISC 15 <sup>th</sup> Eq 9-20
Tensile capacity per bolt before considering prying	B = from ca	lc shown in above	e section = 24.	5 [kips]	
Resistance factor-LRFD	φ = 0.90				AISC 15 <sup>th</sup> Page 9-12
Column flange thickness	t = 0.531	[in]	tensile $F_u = 65.0$	0 [ksi]	
Plate thickness req'd to develop bolt tensile capacity without prying	$t_c = \left(\frac{4 B b}{\phi p F}\right)$	) <sup>0.5</sup>	= 0.80	01 [in]	AISC 15 <sup>th</sup> Eq 9-26a
	$\alpha' = \frac{1}{\delta (1 + )}$	$\frac{t_c}{\rho}$ [ $(\frac{t_c}{t})^2 - 1$	] = 1.00	08	AISC 15 <sup>th</sup> Eq 9-28
when $\alpha' > 1$	$Q = \left(\frac{t}{t_c}\right)^2$	(1 + δ)	= 0.76	58	AISC 15 <sup>th</sup> Eq 9-27
Bolt tensile force per bolt in demand	T = from ca	lc shown below	= 5.5	[kips]	
Tensile strength per bolt after considering prying	$\phi r_n = B \times Q$		= 18.	<b>8</b> [kips]	AISC 15 <sup>th</sup> Eq 9-27
- 577 5	ratio = <b>0.29</b>		> T	ОК	
Calculate Max Single Bolt Tensile	Load				
Bolt group force	axial $P = 21.9$	[kips]			
Bolt number	Bolt Row n <sub>h</sub> = 2		Bolt Col $n_v = 2$		
Bolt tensile force per bolt	T = P / ( n.,	n <sub>h</sub> )	= 5.5	[kips]	

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Vertical Brace Connection

VB-Right

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 = <b>7.375</b>	[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 15 <sup>th</sup> Eq 8-11
Weld stress load angle	$\theta = \tan^{-1}\left(\frac{f_a}{f_v}\right)$	= 32.6	[°]	
Fillet Weld Strength Calc				
Fillet weld leg size	$w = \frac{1}{4}$ [in]	load angle $\theta$ = 32.6	[°]	
Electrode strength	F <sub>EXX</sub> = 70.0 [ksi]	strength coeff C <sub>1</sub> = 1.00		AISC 15 <sup>th</sup> 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 15 <sup>th</sup> Page 8-9
Fillet weld shear strength	R <sub>n-w</sub> = 0.6 (C <sub>1</sub> x 70 ksi) 0.70	$7 \text{ w n C}_2 = 17.79$	[kip/in]	AISC 15 <sup>th</sup> 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 15 <sup>th</sup> Eq J	J4-4 is checked		AISC 15 <sup>th</sup> J2.4
Base metal shear rupture	$R_{n-b} = 0.6 F_{u}t$	= 19.50	[kip/in]	AISC 15 <sup>th</sup> Eq J4-4
Double fillet linear shear strength	$R_n = min (R_{n-w}, R_{n-b})$	= 17.787	[kip/in]	AISC 15 <sup>th</sup> Eq 9-2
Resistance factor-LRFD	φ = 0.75			AISC 15 <sup>th</sup> Eq 8-1
	φ R <sub>n</sub> =	= 13.340	[kip/in]	
	ratio = <b>0.41</b>	> f <sub>max</sub>	ОК	

Column Web Local Yielding		ratio = 21.9 / 203.3	= 0.11	PASS
Concentrated force on column	P <sub>u</sub> =	= 21.9	[kips]	
Column section	d = 9.921 [in]	$t_{f} = 0.531$	[in]	
	t <sub>w</sub> = 0.315 [in]	k = 1.181	[in]	
	yield $F_y = 50.0$ [ksi]			
Length of bearing	$I_{b} = end plate length$	= 7.000	[in]	
Column web local yielding strength	$R_n = F_y t_w (5 k + I_b)$	= 203.3	[kips]	AISC 15 <sup>th</sup> Eq J10-2
Resistance factor-LRFD	$\varphi = 1.00$			
	φ R <sub>n</sub> =	= 203.3	[kips]	
		_		
	ratio = <b>0.11</b>	> P <sub>u</sub>	UK	
Column Flange Local Bending	ratio = <b>0.11</b>	> P <sub>u</sub> ratio = 21.9 / 79.3	= <b>0.28</b>	PASS
Column Flange Local Bending Concentrated force from gusset	ratio = <b>0.11</b>	<pre>&gt; P<sub>u</sub> ratio = 21.9 / 79.3 = 21.9</pre>	= <b>0.28</b> [kips]	PASS
Column Flange Local Bending Concentrated force from gusset Column w section	ratio = <b>0.11</b> $P_u = t_f = 0.531$ [in]	$P_u$ ratio = 21.9 / 79.3 = <b>21.9</b> yield $F_y$ = 50.0	= <b>0.28</b> [kips] [ksi]	PASS
Column Flange Local Bending Concentrated force from gusset Column w section Column flange local bending strength	ratio = <b>0.11</b> $P_u =$ $t_f = 0.531$ [in] $R_n = 6.25 F_y t_f^2$	$P_u$ ratio = 21.9 / 79.3 = <b>21.9</b> yield $F_y$ = 50.0 = 88.1	= 0.28           [kips]           [ksi]           [kips]	PASS AISC 15 <sup>th</sup> Eq J10-1
Column Flange Local Bending Concentrated force from gusset Column w section Column flange local bending strength Resistance factor-LRFD	ratio = <b>0.11</b> $P_{u} =$ $t_{f} = 0.531  [in]$ $R_{n} = 6.25 F_{y} t_{f}^{2}$ $\phi = 0.90$	$P_u$ ratio = 21.9 / 79.3 = <b>21.9</b> yield $F_y$ = 50.0 = 88.1	<b>= 0.28</b> [kips] [ksi] [kips]	PASS AISC 15 <sup>th</sup> Eq J10-1 AISC 15 <sup>th</sup> J10.1
Column Flange Local Bending Concentrated force from gusset Column w section Column flange local bending strength Resistance factor-LRFD	ratio = <b>0.11</b> $P_{u} =$ $t_{f} = 0.531  [in]$ $R_{n} = 6.25 F_{y} t_{f}^{2}$ $\phi = 0.90$ $\phi R_{n} =$	$P_u$ ratio = 21.9 / 79.3 = 21.9 yield $F_y$ = 50.0 = 88.1 = 79.3	= <b>0.28</b> [kips] [ksi] [kips]	PASS AISC 15 <sup>th</sup> Eq J10-1 AISC 15 <sup>th</sup> J10.1
Column Flange Local Bending Concentrated force from gusset Column w section Column flange local bending strength Resistance factor-LRFD	ratio = <b>0.11</b> $P_{u} =$ $t_{f} = 0.531  [in]$ $R_{n} = 6.25 F_{y} t_{f}^{2}$ $\phi = 0.90$ $\phi R_{n} =$ ratio = <b>0.28</b>	$P_u$ ratio = 21.9 / 79.3 = 21.9 yield $F_y$ = 50.0 = 88.1 = 79.3 > $P_u$		PASS AISC 15 <sup>th</sup> Eq J10-1 AISC 15 <sup>th</sup> J10.1

Brace Force Load Case 2

P =76.5 kips (C) ra

Gusset Plate - Shear Yielding			ratio = 34.2 / 212.4	= 0.16	PASS
Calculate gusset or stiff plate length ou	itside beam flange ar	id count it as bean	n web extension to resist	shear	
Beam sect depth W200x36	d <sub>b</sub> = 7.913	[in]			
Bolt pitch & edge distance	d <sub>1</sub> = 2.500	[in]	e <sub>v</sub> = 1.375	[in]	
Beam web extension outside beam flange	$L_{e} = 2(d_{1} + e)$	$e_{ m v}$ ) - 2x0.75 in clip	o = 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 <sup>2</sup> ]	
Shear force required	V <sub>u</sub> =		= 34.2	[kips]	
Plate shear yielding strength	$R_{n} = 0.6 F_{y}A$	gv	= 212.4	[kips]	AISC 15 <sup>th</sup> Eq J4-3
Resistance factor-LRFD	$\phi = 1.00$				AISC 15 <sup>th</sup> Eq J4-3
	φ R <sub>n</sub> =		= 212.4	[kips]	
	ratio = <b>0.16</b>		> V <sub>u</sub>	OK	
Gusset Plate - Shear Rupture			ratio = 34.2 / 207.1	= 0.17	PASS
Calculate gusset or stiff plate length ou	itside beam flange ar	id count it as bean	n web extension to resist	shear	
Beam sect depth W200x36	d <sub>b</sub> = 7.913	[in]			
Bolt pitch & edge distance	$d_1 = 2.500$	[in]	e <sub>v</sub> = 1.375	[in]	
Beam web extension outside beam flange	$L_{e} = 2(d_{1} + e)$	$e_{ m v}$ ) - 2x0.75 in clip	o = 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	$F_{\mu} = 65.0$	[ksi]	۲		
Plate net area in shear	$A_{nv} = b_p t_p$		= 7.082	[in <sup>2</sup> ]	
Shear force in demand	V <sub>u</sub> =		= 34.2	[kips]	
Plate shear rupture strength	$R_{n} = 0.6 F_{u}A$	nv	= 276.2	[kips]	AISC 15 <sup>th</sup> Eq J4-4
Resistance factor-LRFD	φ = 0.75				AISC 15 <sup>th</sup> Eq J4-4
	φ R <sub>n</sub> =		= 207.1	[kips]	
	ratio = <b>0.17</b>		> V <sub>u</sub>	ОК	
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 <sup>2</sup> ]	
Shear force required	V <sub>u</sub> =		= 17.1	[kips]	
Plate shear yielding strength	$R_{n} = 0.6 F_{y}A$	gv	= 131.3	[kips]	AISC 15 <sup>th</sup> Eq J4-3
Resistance factor-LRFD	$\phi = 1.00$				AISC 15 <sup>th</sup> Eq J4-3
	φ R <sub>n</sub> =		= 131.3	[kips]	

 $> V_u$ 

OK

ratio = **0.13** 

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End Plate - Shear Rupture			ratio = 17.1 / 96.0	) = 0.1	8 PASS	
Plate Shear Rupture Check						
Bolt hole diameter	bolt dia d <sub>b</sub> = $\frac{3}{4}$	[in]	bolt hole dia $d_{h} = \frac{7}{2}$	3 [in]	AISC 15 <sup>th</sup> B4.3b	
Number of bolt	n = 2			<b>-</b> -		
Plate size	width $b_p = 7.000$	[in]	thickness $t_p = 0$ .	625 [in]		
Plate tensile strength	F <sub>u</sub> = 65.0	[ksi]	r			
Plate net area in shear	$A_{nv} = (b_p - n)$	d <sub>h</sub> )t <sub>p</sub>	= 3.	281 [in <sup>2</sup> ]		
Shear force required	V <sub>u</sub> =	·	= 17	<b>7.1</b> [kips]		
Plate shear rupture strength	$R_{n} = 0.6 F_{u}A$	۹ <sub>nv</sub>	= 12	8.0 [kips]	AISC 15 <sup>th</sup> Eq J4-4	
Resistance factor-LRFD	φ = 0.75				AISC 15 <sup>th</sup> Eq J4-4	
	φ R <sub>n</sub> =		= 96	5.0 [kips]		
	ratio = <b>0.18</b>		> V.	OK		
End Plate - Block Shear - Cente	er Strip		ratio = 34.2 / 223	.9 = <b>0.1</b>	15 PASS	
Plate Block Shear - Center Strip						
Bolt hole diameter	bolt dia d <sub>b</sub> = $\frac{3}{4}$	[in]	bolt hole dia $d_h = \frac{7}{4}$	3 [in]	AISC 15 <sup>th</sup> B4.3b	
Plate thickness	t <sub>p</sub> = 0.625	[in]				
Plate strength	$F_{y} = 50.0$	[ksi]	F <sub>u</sub> = 65	.0 [ksi]		
Bolt no in ver & hor dir	n <sub>v</sub> = 2		n <sub>h</sub> = 2			
Bolt spacing in ver & hor dir	s <sub>v</sub> = 3.500	[in]	s <sub>h</sub> = 3.	500 [in]		
Bolt edge dist in ver & hor dir	e <sub>v</sub> = 1.375	[in]	e <sub>h</sub> = 1.	750 [in]		
Gross area subject to shear	A <sub>gv</sub> = [ (n <sub>h</sub> - :	1) $s_h + e_h$ ] $t_p x$	2 = 6.	563 [in <sup>2</sup> ]		
Net area subject to shear	$A_{nv} = A_{gv} - [($	n <sub>h</sub> -1)+0.5]d <sub>r</sub>	$t_p x^2 = 4.$	922 [in <sup>2</sup> ]		
Net area subject to tension						
when sheared out by center strip	$A_{nt} = (n_v - 1)$	)(s <sub>v</sub> -d <sub>h</sub> )t <sub>p</sub>	= 1.	641 [in <sup>2</sup> ]		
Block shear strength required	V <sub>u</sub> =		= 34	<b>I.2</b> [kips]		
Uniform tension stress factor	$U_{bs} = 1.00$				AISC 15 <sup>th</sup> Fig C-J4.2	
Bolt shear resistance provided	$R_n = min (0.$	6F <sub>u</sub> A <sub>nv</sub> , 0.6F <sub>y</sub> A	A <sub>gv</sub> ) + = 29	8.6 [kips]	AISC 15 <sup>th</sup> Eq J4-5	
	U <sub>bs</sub> F <sub>u</sub> A	۹ <sub>nt</sub>			th	
Resistance factor-LRFD	φ = 0.75				AISC 15"' Eq J4-5	
	$\phi R_n =$		= 22	23.9 [kips]		
	ratio = <b>0.15</b>		> V.	UK		

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Vertical Brace Connection

	enneeden Beelgn	mp.//dop.or/		Voltical Blac		v D r tigi
End Plate - Block Shear - 2-Sid	le Strip		ratio = 34.2 / 201.1	= 0.17	PASS	
Plate Block Shear - 2 Side Strips						
Bolt hole diameter	bolt dia $d_b = \frac{3}{4}$	[in]	bolt hole dia d <sub>h</sub> = $\frac{7}{8}$	[in]	AISC 15 <sup>th</sup> 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 <sub>v</sub> = 2		n <sub>h</sub> = 2			
Bolt spacing in ver & hor dir	s <sub>v</sub> = 3.500	[in]	s <sub>h</sub> = 3.500	) [in]		
Bolt edge dist in ver & hor dir	e <sub>v</sub> = 1.375	[in]	e <sub>h</sub> = 1.750	) [in]		
Gross area subject to shear	$A_{gv} = [(n_{h} - 1)]$	1) $s_h + e_h$ ] $t_p x$	2 = 6.563	8 [in <sup>2</sup> ]		
Net area subject to shear	$A_{nv} = A_{gv} - [($	n <sub>h</sub> -1)+0.5]d <sub>1</sub>	$t_{p}x^{2} = 4.922$	2 [in <sup>2</sup> ]		
Net area subject to tension						
when sheared out by 2 side strips	$A_{nt} = (e_v - 0)$	.5 d <sub>h</sub> )t <sub>p</sub> x 2	= 1.172	2 [in <sup>2</sup> ]		
Block shear strength required	V <sub>u</sub> =		= 34.2	[kips]		
Uniform tension stress factor	$U_{bs} = 1.00$				AISC 15 <sup>th</sup> Fig C-J4	.2
Bolt shear resistance provided	$R_n = \min(0.$	6F <sub>u</sub> A <sub>nv</sub> , 0.6F <sub>y</sub>	A <sub>gv</sub> ) + = 268.1	[kips]	AISC 15 <sup>th</sup> Eq J4-5	
Pacistance factor LRED	$U_{bs}F_{u}A$	A <sub>nt</sub>			AISC 15 <sup>th</sup> Eq. 14 E	
	φ = 0.75		- 201 -	I [kins]	AI3C I3 LY J4-3	
	$\psi \kappa_n =$		= 201			
	ratio = <b>0.17</b>		> V <sub>u</sub>	UK		

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Vertical Brace Connection

End Plate - Boit Bearing on El	nd Plate		ratio = 34.2 / 71.6	= 0.48	PASS
Single Bolt Shear Strength					
Bolt shear stress	bolt grade = A325-N		F <sub>nv</sub> = 54.0	[ksi]	AISC 15 <sup>th</sup> Table J3.2
	bolt dia d <sub>b</sub> = $0.750$	[in]	bolt area $A_b = 0.442$	[in <sup>2</sup> ]	
Single bolt shear strength	$R_{n-bolt} = F_{nv} A_b$		= 23.9	[kips]	AISC 15 <sup>th</sup> Eq J3-1
Bolt Bearing/TearOut Strength	on Plate				
Bolt hole diameter	bolt dia $d_b = \frac{3}{4}$	[in]	bolt hole dia d <sub>h</sub> = $\frac{13}{16}$	[in]	AISC 15 <sup>th</sup> Table J3.3
Bolt spacing & edge distance	spacing L <sub>s</sub> = 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	$R_{n-t\&b-in} = 1.5 L_c t$	F <sub>u</sub> ≤ 3.0 d <sub>b</sub> t F <sub>u</sub>	l		AISC 15 <sup>th</sup> Eq J3-6b
	= 163.8 ≤	91.4	= 91.4	[kips]	
Bolt strength at interior	$R_{n-in} = min (R_{n-in})$	<sub>n-t&amp;b-in</sub> , R <sub>n-bolt</sub> )	= 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	$R_{n-t\&b-ed} = 1.5 L_c t$	F <sub>u</sub> ≤ 3.0 d <sub>b</sub> t F <sub>u</sub>	1		AISC 15 <sup>th</sup> Eq J3-6b
	= 81.9 ≤ 9	91.4	= 81.9	[kips]	
Bolt strength at edge	$R_{n-ed} = min (R_{n-ed})$	n-t&b-ed , R n-bolt	= 23.9	[kips]	
Number of bolt	interior n <sub>in</sub> = 2		edge n <sub>ed</sub> = 2		
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n-in}$	+ n <sub>ed</sub> R <sub>n-ed</sub>	= 95.4	[kips]	
Required shear strength	V <sub>u</sub> =		= 34.2	[kips]	
Bolt resistance factor-LRFD	φ = 0.75				AISC 15 <sup>th</sup> J3.10
	φ R <sub>n</sub> =		= 71.6	[kips]	
	ratio = <b>0.48</b>		> V.,	ОК	

End Plate / Column - Bolt Shea	r	ratio = 34.2 / 71.6	= 0.48	PASS
Bolt shear stress	bolt grade = A325-N	F <sub>nv</sub> = 54.0	[ksi]	AISC 15 <sup>th</sup> Table J3.2
	bolt dia d <sub>b</sub> = 0.750 [in]	bolt area $A_b = 0.442$	[in <sup>2</sup> ]	
Number of bolt carried shear	n <sub>s</sub> = 4.0	shear plane $m = 1$		
Bolt group eccentricity coefficient	C <sub>ec</sub> =	= 1.000		
Required shear strength	V <sub>u</sub> =	= 34.2	[kips]	
Bolt shear strength	$R_n = F_{nv} A_b n_s m C_{ec}$	= 95.4	[kips]	AISC 15 <sup>th</sup> Eq J3-1
Bolt resistance factor-LRFD	φ = 0.75			AISC 15 <sup>th</sup> Eq J3-1
	φ R <sub>n</sub> =	= 71.6	[kips]	
	ratio = <b>0.48</b>	> V <sub>u</sub>	OK	

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End Plate / Colum	nn - Bolt Bea	ring on Column		ratio = 34.2 / 71.6	= 0.48	PASS	
Single Bolt Shear S	trength						
Bolt shear stress		bolt grade = A325-N		$F_{nv} = 54.0$	[ksi]	AISC 15 <sup>th</sup> Table J	3.2
		bolt dia d <sub>b</sub> = $0.750$	[in]	bolt area $A_b = 0.442$	2 [in <sup>2</sup> ]		
Single bolt shear stre	ength	$R_{n-bolt} = F_{nv}A_b$		= 23.9	[kips]	AISC 15 <sup>th</sup> Eq J3-	1
Bolt Bearing/TearC	Out Strength o	on Plate					
Bolt hole diameter		bolt dia $d_b = \frac{3}{4}$	[in]	bolt hole dia d <sub>h</sub> = $\frac{13}{16}$	[in]	AISC 15 <sup>th</sup> Table J	3.3
Bolt spacing		spacing L <sub>s</sub> = 3.500	[in]				
Plate tensile strength		$F_{u} = 65.0$	[ksi]				
Plate thickness		t = 0.531	[in]				
Interior Bolt							
Bolt hole edge clear o	listance	$L_c = L_s - d_h$		= 2.688	3 [in]		
Bolt tear out/bearing	strength	$R_{n-t\&b-in} = 1.5 L_c t$	F <sub>u</sub> ≤ 3.0 d <sub>b</sub> t m	Fu		AISC 15 <sup>th</sup> Eq J3-	6b
		= 139.1 ≤	377.7	= 77.7	[kips]		
Bolt strength at inter	ior	R <sub>n-in</sub> = min ( R	<sub>n-t&amp;b-in</sub> , R <sub>n-bolt</sub> )	= 23.9	[kips]		
Number of bolt		interior n <sub>in</sub> = 4					
Bolt bearing strength	for all bolts	$R_n = n_{in} R_{n-in}$	1	= 95.4	[kips]		
Required shear streng	gth	V <sub>u</sub> =		= 34.2	[kips]		
Bolt resistance factor	-LRFD	φ = 0.75				AISC 15 <sup>th</sup> J3.10	
		φ R <sub>n</sub> =		= 71.6	[kips]		
		ratio = <b>0.48</b>		> V	ОК		

AISC Steel Connection Design

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Vertical Brace Connection

Gusset Plate to End Plate Weld	Strength		ratio = 4.64	/ 11.14	= 0.42	PASS
Weld Group Forces						
	shear V = 34.2	[kips]	axial F	9 = 21.9	[kips] in	compression
Gusset-end plate fillet weld length	L = weld len	igth 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 15 <sup>th</sup> Eq 8-11
Weld stress load angle	θ =			= 0.0	[°]	
Fillet Weld Strength Calc						
Fillet weld leg size	$w = \frac{1}{4}$	[in]	load angle 6	0.0	[°]	
Electrode strength	F <sub>EXX</sub> = 70.0	[ksi]	strength coeff C	<sub>1</sub> = 1.00		AISC 15 <sup>th</sup> Table 8-3
Number of weld line	n = 2 for d	ouble fillet				
Load angle coefficient	$C_2 = (1 + 0.$	5 sin <sup>1.5</sup> θ)		= 1.00		AISC 15 <sup>th</sup> Page 8-9
Fillet weld shear strength	$R_{n-w} = 0.6 (C_1)$	x 70 ksi) 0.70	07 w n C <sub>2</sub>	= 14.85	[kip/in]	AISC 15 <sup>th</sup> 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 A	AISC 15 <sup>th</sup> Eq	J4-4 is checked			AISC 15 <sup>th</sup> J2.4
Base metal shear rupture	$R_{n-b} = 0.6 F_{u}t$			= 19.50	[kip/in]	AISC 15 <sup>th</sup> Eq J4-4
Double fillet linear shear strength	R <sub>n</sub> = min ( R	<sub>n-w</sub> , R <sub>n-b</sub> )		= 14.847	[kip/in]	AISC 15 <sup>th</sup> Eq 9-2
Resistance factor-LRFD	φ = 0.75					AISC 15 <sup>th</sup> Eq 8-1
	φ R <sub>n</sub> =			= 11.135	[kip/in]	
	ratio = <b>0.42</b>			> f <sub>max</sub>	ОК	

Column Web Local Yielding		ratio = 21.9 / 203.3	= 0.11	PASS
Concentrated force on column	P <sub>u</sub> =	= 21.9	[kips]	
Column section	d = 9.921 [in]	$t_{f} = 0.531$	[in]	
	t <sub>w</sub> = 0.315 [in]	k = 1.181	[in]	
	yield $F_y = 50.0$ [ksi]			
Length of bearing	- I <sub>b</sub> = end plate length	= 7.000	[in]	
Column web local yielding strength	$R_n = F_y t_w (5 k + I_b)$	= 203.3	[kips]	AISC 15 <sup>th</sup> Eq J10-2
Resistance factor-LRFD	$\varphi = 1.00$			
	φ R <sub>n</sub> =	= 203.3	[kips]	
	ratio = <b>0.11</b>	> P <sub>u</sub>	OK	

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Vertical Brace Connection

Column Web Local Crippling			ratio = 21.9 / 183.1	= 0.12	PASS
Concentrated force on column	P <sub>u</sub> =		= 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	۔ ا <sub>b</sub> = end pla	te length	= 7.000	[in]	
Column web local crippling strength	$R_{n} = 0.8 t_{w}^{2}$	$[1+3\frac{l_{b}}{d}(\frac{t_{w}}{t_{f}})^{1.5}]$	] x = 244.1	[kips]	AISC 15 <sup>th</sup> Eq J10-4
	(EFyt	<sup>f</sup> ) <sup>0.5</sup>			
Resistance factor-LRFD	φ = 0.75				AISC 15 <sup>th</sup> J10.3
	φR <sub>n</sub> =		= 183.1	[kips]	
	ratio = <b>0.12</b>		> P <sub>u</sub>	OK	

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Vertical Brace Connection

Gusset to Beam	Direct Weld Connection	Code=AISC 360-16 LRFD
Result Summary	decompetries & weld limitations = $PASS$	limit states may 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 <sub>min</sub> =	= <b>0.188</b> [in]	AISC 15 <sup>th</sup> Table J2.4
Fillet weld size provided	w =	= <b>0.313</b> [in]	
		≥ w <sub>min</sub> OK	
Min Fillet Weld Length			
Fillet weld size provided	w =	= 0.313 [in]	
Min fillet weld length allowed	$L_{min} = 4 \times w$	= <b>1.250</b> [in]	AISC 15 <sup>th</sup> J2.2b
Min fillet weld length	L =	= <b>12.442</b> [in]	
		≥ L <sub>min</sub> OK	

Brace Force Load Case 1	Gusset plate t=0.500	P =-76.5 kips (T) ra	tio = <b>0.27</b>	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 <sub>y</sub> = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$	= 6.221	[in <sup>2</sup> ]	
Shear force required	V <sub>u</sub> =	= 34.5	[kips]	
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$	= 186.6	[kips]	AISC 15 <sup>th</sup> Eq J4-3
Resistance factor-LRFD	$\varphi = 1.00$			AISC 15 <sup>th</sup> Eq J4-3
	φ R <sub>n</sub> =	= 186.6	[kips]	
	ratio = <b>0.18</b>	> V.,	OK	
		ŭ		
Gusset Plate - Shear Rupture	3	ratio = 34.5 / 182.0	= 0.19	PASS
Gusset Plate - Shear Rupture Plate Shear Rupture Check	3	ratio = 34.5 / 182.0	= 0.19	PASS
Gusset Plate - Shear Rupture Plate Shear Rupture Check Plate size	width b <sub>p</sub> = 12.442 [in]	ratio = 34.5 / 182.0 thickness t <sub>p</sub> = 0.500	= <b>0.19</b> [in]	PASS
Gusset Plate - Shear Rupture Plate Shear Rupture Check Plate size Plate tensile strength	width $b_p = 12.442$ [in] $F_u = 65.0$ [ksi]	ratio = 34.5 / 182.0 thickness t <sub>p</sub> = 0.500	= <b>0.19</b> [in]	PASS
Gusset Plate - Shear Rupture Plate Shear Rupture Check Plate size Plate tensile strength Plate net area in shear	width $b_p = 12.442$ [in] $F_u = 65.0$ [ksi] $A_{nv} = b_p t_p$	ratio = $34.5 / 182.0$ thickness t <sub>p</sub> = 0.500 = 6.221	= <b>0.19</b> [in] [in <sup>2</sup> ]	PASS
Gusset Plate - Shear Rupture Plate Shear Rupture Check Plate size Plate tensile strength Plate net area in shear Shear force in demand	width $b_p = 12.442$ [in] $F_u = 65.0$ [ksi] $A_{nv} = b_p t_p$ $V_u =$	ratio = $34.5 / 182.0$ thickness t <sub>p</sub> = 0.500 = 6.221 = <b>34.5</b>	= <b>0.19</b> [in] [in <sup>2</sup> ] [kips]	PASS
Gusset Plate - Shear Rupture Plate Shear Rupture Check Plate size Plate tensile strength Plate net area in shear Shear force in demand Plate shear rupture strength	width $b_p = 12.442$ [in] $F_u = 65.0$ [ksi] $A_{nv} = b_p t_p$ $V_u =$ $R_n = 0.6 F_u A_{nv}$	ratio = $34.5 / 182.0$ thickness t <sub>p</sub> = 0.500 = $6.221$ = <b>34.5</b> = 242.6	= <b>0.19</b> [in] [in <sup>2</sup> ] [kips] [kips]	PASS AISC 15 <sup>th</sup> Eq J4-4
Gusset Plate - Shear Rupture Plate Shear Rupture Check Plate size Plate tensile strength Plate net area in shear Shear force in demand Plate shear rupture strength Resistance factor-LRFD	width $b_p = 12.442$ [in] $F_u = 65.0$ [ksi] $A_{nv} = b_p t_p$ $V_u =$ $R_n = 0.6 F_u A_{nv}$ $\phi = 0.75$	ratio = $34.5 / 182.0$ thickness t <sub>p</sub> = 0.500 = 6.221 = <b>34.5</b> = 242.6	= <b>0.19</b> [in] [in <sup>2</sup> ] [kips] [kips]	AISC 15 <sup>th</sup> Eq J4-4 AISC 15 <sup>th</sup> Eq J4-4
Gusset Plate - Shear Rupture Plate Shear Rupture Check Plate size Plate tensile strength Plate net area in shear Shear force in demand Plate shear rupture strength Resistance factor-LRFD	width $b_p = 12.442$ [in] $F_u = 65.0$ [ksi] $A_{nv} = b_p t_p$ $V_u =$ $R_n = 0.6 F_u A_{nv}$ $\phi = 0.75$ $\phi R_n =$	ratio = $34.5 / 182.0$ thickness t <sub>p</sub> = 0.500 = $6.221$ = $34.5$ = $242.6$ = $182.0$	= <b>0.19</b> [in] [in <sup>2</sup> ] [kips] [kips] [kips]	PASS AISC 15 <sup>th</sup> Eq J4-4 AISC 15 <sup>th</sup> Eq J4-4

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Vertical Brace Connection

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 <sup>2</sup> ]	
Tensile force required	P <sub>u</sub> =		= 17.5	[kips]	
Plate tensile yielding strength	$R_n = F_y A_g$		= 311.1	[kips]	AISC 15 <sup>th</sup> Eq J4-1
Resistance factor-LRFD	$\varphi = 0.90$				AISC 15 <sup>th</sup> Eq J4-1
	φ R <sub>n</sub> =		= 279.9	[kips]	
	ratio = <b>0.06</b>		> P <sub>u</sub>	OK	
Gusset Plate - Axial Tensile Rupt	ure		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 <sup>2</sup> ]	
Tensile force required	P <sub>u</sub> =		= 17.5	[kips]	
	$R = F \Delta$		= 404.4	[kips]	AISC 15 <sup>th</sup> Ea 14-2
Plate tensile rupture strength	in - i u Ant			r 13	24512
Plate tensile rupture strength Resistance factor-LRFD	$\phi = 0.75$				AISC 15 <sup>th</sup> Eq J4-2
Plate tensile rupture strength Resistance factor-LRFD	$\phi = 0.75$ $\phi R_n =$		= 303.3	[kips]	AISC 15 <sup>th</sup> Eq J4-2 AISC 15 <sup>th</sup> Eq J4-2

Gusset Plate - Flexural Yield Inte	eract	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	$A_g = b_p x t_p$	= 6.221	[in <sup>2</sup> ]	
Shear plate - plastic modulus	$Z_{p} = (b_{p} x t_{p}^{2}) / 4$	= 19.35	[in <sup>3</sup> ]	
Flexural strength available	$M_c = \phi F_y Z_p  \phi=0.9$	= 72.56	[kip-ft]	
Flexural strength required	$M_r = from gusset inte$	rface forces calc = 0.32	[kip-ft]	
Axial strength available	P <sub>c</sub> = from axial tensile	e yield check = 279.9	[kips]	
Axial strength required	$P_r = $ from gusset inte	rface forces calc = -17.5	[kips]	
Shear strength available	V <sub>c</sub> = from shear yield	ng check = 186.6	[kips]	
Shear strength required	$V_r =$ from gusset inte	rface forces calc = 34.5	[kips]	
Flexural yield interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c}\right)^2$	$+\frac{M_r}{M_c})^2 = 0.04$		AISC 15 <sup>th</sup> Eq 10-5
		< 1.0	ОК	

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Vertical Brace Connection

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 <sup>2</sup> ]	
Plastic modulus of net section	$Z_{net} = (b_p x t_p^2) / 4$	= 19.35	[in <sup>3</sup> ]	
Flexural strength available	$M_c = \phi F_u Z_{net} \phi = 0.75$	= 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 rupt	cure 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 ch	eck = 182.0	[kips]	
Shear strength required	$V_r$ = from gusset interface	forces calc = 34.5	[kips]	
Flexural rupture interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right)^2$	$(r_{c})^{2} = 0.04$		AISC 15 <sup>th</sup> Eq 10-5
		< 1.0	ОК	

Gusset to Beam Weld Strengt	h		ratio = 3.18	/ 11.70	= 0.27	PASS
Gusset to Beam Interface - Forc	es					
	shear $H_b = 34.5$	[kips]	axial V	<sub>b</sub> = -17.5	[kips] in	tension
	moment $M_b = 0.32$	[kip-ft]				
Gusset-beam fillet weld length	L <sub>w</sub> =			= 12.442	[in]	
Gusset to Beam Interface - Com	bined Weld Stress					
Weld stress from axial force	$f_a = V_b / L_w$	b		= -1.41	[kip/in]	in tension
Weld stress from shear force	$f_v = H_b / L_w$	b		= 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 <sub>max</sub> = [ (f <sub>a</sub> - f	$_{b})^{2} + f_{v}^{2} ]^{0.5}$		= 3.179	[kip/in]	AISC 15 <sup>th</sup> 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	$w = \frac{5}{16}$	[in]	load angle 6	) = 29.3	[°]	
Electrode strength	F <sub>EXX</sub> = 70.0	[ksi]	strength coeff C	<sub>1</sub> = 1.00		AISC 15 <sup>th</sup> Table 8-3
Number of weld line	n = 2 for c	louble fillet				
Load angle coefficient	$C_2 = (1 + 0)$	.5 sin <sup>1.5</sup> θ)		= 1.17		AISC 15 <sup>th</sup> Page 8-9
Fillet weld shear strength	$R_{n-w} = 0.6 (C_1)$	x 70 ksi) 0.7	07 w n C <sub>2</sub>	= 21.73	[kip/in]	AISC 15 <sup>th</sup> Eq 8-1
Base metal - gusset plate	thickness t = $0.500$	[in]	tensile F	u = 65.0	[ksi]	
Base metal - gusset plate is in shea	ar, <u>shear</u> rupture as per a	AISC 15 <sup>th</sup> Eq	J4-4 is checked			AISC 15 <sup>th</sup> J2.4
Base metal shear rupture	$R_{n-b} = 0.6 F_u t$			= 19.50	[kip/in]	AISC 15 <sup>th</sup> Eq J4-4
Double fillet linear shear strength	R <sub>n</sub> = min ( R	<sub>n-w</sub> , R <sub>n-b</sub> )		= 19.500	[kip/in]	AISC 15 <sup>th</sup> Eq 9-2
Resistance factor-LRFD	φ = 0.75					AISC 15 <sup>th</sup> Eq 8-1
	φ R <sub>n</sub> =			= 14.63	[kip/in]	
When gusset plate is directly welder to allow adequate force redistribution	d to beam or column, an on in the weld group	oply 1.25 duct	tility factor			AISC 15 <sup>th</sup> Page 13-11
Weld strength used for design after applying ductility factor	$\phi R_{n} = \phi R_{n} x$	(1/1.25)		= 11.70	[kip/in]	
	ratio = <b>0.27</b>			> f <sub>max</sub>	ОК	

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Vertical Brace Connection

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 formu	- la below to calculate gusset edge equiv	alent 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 <sub>u</sub> =	= 18.7	[kips]	
Beam section	d = 7.913 [in]	$t_{f} = 0.402$	[in]	
	t <sub>w</sub> = 0.244 [in]	k = 0.906	[in]	
	yield $F_y = 50.0$ [ksi]			
Length of bearing	I <sub>b</sub> = Gusset/Beam interface ler	igth = 12.442	[in]	
Gusset plate corner clip	clip = from user input	= 0.750	[in]	
Distance from normal force applied point to member end	$I_{\rm N} = 0.5 I_{\rm b} + {\rm clip}$	= 6.971	[in]	
	when $I_N \leq d$ , use AISC $15^{th}$ B	Eq J10-3		AISC 15 <sup>th</sup> Eq J10-3
Beam web local yielding strength	$R_{n} = F_{y}t_{w}(2.5 \text{ k} + I_{b})$	= 179.4	[kips]	AISC 15 <sup>th</sup> Eq J10-3
Resistance factor-LRFD	$\phi = 1.00$			
	φ R <sub>n</sub> =	= 179.4	[kips]	
	ratio = <b>0.10</b>	> P <sub>u</sub>	ОК	
Brace Force Load Case 2	Gusset plate t=0.500 P	=76.5 kips (C) rat	io = <b>0.25</b>	PASS

Gusset Plate - Shear Yielding		ratio = 34.5 / 186.6	= 0.18	PASS
Plate Shear Yielding Check				
Plate size		thickness $t_p = 0.500$	[in]	
Plate yield strength	F <sub>y</sub> = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$	= 6.221	[in <sup>2</sup> ]	
Shear force required	V <sub>u</sub> =	= 34.5	[kips]	
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$	= 186.6	[kips]	AISC 15 <sup>th</sup> Eq J4-3
Resistance factor-LRFD	$\varphi = 1.00$			AISC 15 <sup>th</sup> Eq J4-3
	φ R <sub>n</sub> =	= 186.6	[kips]	
	ratio = <b>0.18</b>	> V <sub>u</sub>	OK	

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Vertical Brace Connection

Gusset Plate - Shear Rupture		ratio = 34.5 / 182.0	= 0.19	PASS
Plate Shear Rupture Check				
Plate size	 width b <sub>p</sub> = 12.442 [in]	thickness $t_p = 0.500$	[in]	
Plate tensile strength	F <sub>u</sub> = 65.0 [ksi]			
Plate net area in shear	$A_{nv} = b_p t_p$	= 6.221	[in <sup>2</sup> ]	
Shear force in demand	V <sub>u</sub> =	= 34.5	[kips]	
Plate shear rupture strength	$R_n = 0.6 F_u A_{nv}$	= 242.6	[kips]	AISC 15 <sup>th</sup> Eq J4-4
Resistance factor-LRFD	φ = 0.75			AISC 15 <sup>th</sup> Eq J4-4
	φ R <sub>n</sub> =	= 182.0	[kips]	
	ratio = <b>0.19</b>	> V <sub>u</sub>	OK	

Gusset Plate - Axial Yield			ratio = 17.5 / 279.9	= 0.06	PASS
Plate Tensile Yielding Check					
Plate size	- width b <sub>p</sub> = 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 <sup>2</sup> ]	
Tensile force required	P <sub>u</sub> =		= 17.5	[kips]	
Plate tensile yielding strength	$R_n = F_y A_g$		= 311.1	[kips]	AISC 15 <sup>th</sup> Eq J4-1
Resistance factor-LRFD	φ = 0.90				AISC 15 <sup>th</sup> Eq J4-1
	$\phi R_n =$		= 279.9	[kips]	
	ratio = <b>0.06</b>		> P <sub>u</sub>	ОК	

Gusset Plate - Flexural Yield Inte	eract	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 <sup>2</sup> ]	
Shear plate - plastic modulus	$Z_{p} = (b_{p} x t_{p}^{2}) / 4$	= 19.35	[in <sup>3</sup> ]	
Flexural strength available	$M_c = \phi F_y Z_p  \phi = 0.90$	= 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	d 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 ch	eck = 186.6	[kips]	
Shear strength required	$V_r$ = from gusset interface	forces calc = 34.5	[kips]	
Flexural yield interaction	ratio = $\left(\frac{V_r}{V_c}\right)^2 + \left(\frac{P_r}{P_c} + \frac{M}{M}\right)^2$	$(r_{c})^{2} = 0.04$		AISC 15 <sup>th</sup> Eq 10-5
		< 1.0	ОК	

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Vertical Brace Connection

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 <sub>u</sub> = 65.0 [ksi]			
Net area of plate	$A_n = b_p \times t_p$	= 6.221	[in <sup>2</sup> ]	
Plastic modulus of net section	$Z_{net} = (b_p x t_p^2) / 4$	= 19.35	[in <sup>3</sup> ]	
Flexural strength available	$M_c = \phi F_u Z_{net} \phi = 0.75$	= 78.61	[kip-ft]	
Flexural strength required	$M_r$ = from gusset interface for	ces calc = 0.00	[kip-ft]	
Shear strength available	$V_c =$ from shear rupture check	<pre>&lt; = 182.0</pre>	[kips]	
Shear strength required	$V_r$ = from gusset interface for	ces calc = 34.5	[kips]	
Flexural rupture interaction	ratio = $(\frac{V_r}{V_c})^2 + (\frac{M_r}{M_c})^2$	= 0.04		AISC 15 <sup>th</sup> Eq 10-5
		< 1.0	ОК	

Gusset to Beam Weld Strengt	ı		ratio = 2.77	/ 11.14	= 0.25	PASS		
Gusset to Beam Interface - Forces								
	shear $H_b = 34.5$	[kips]	axial V	<sub>b</sub> = 17.5	[kips] ir	compression		
	moment $M_b = 0.00$	[kip-ft]						
Gusset-beam fillet weld length	L <sub>w</sub> =			= 12.442	[in]			
Gusset to Beam Interface - Com	bined 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 15 <sup>th</sup> Eq 8-11		
Weld resultant load angle	$\theta$ = weld onl	y has shear o	component	= 0.0	[°]			
Fillet Weld Strength Calc								
Fillet weld leg size	$w = \frac{5}{16}$	[in]	load angle	$\theta = 0.0$	[°]			
Electrode strength	F <sub>EXX</sub> = 70.0	[ksi]	strength coeff C	<sub>1</sub> = 1.00		AISC 15 <sup>th</sup> Table 8-3		
Number of weld line	n = 2 for d	ouble fillet						
Load angle coefficient	$C_2 = (1 + 0.5)$	5 sin <sup>1.5</sup> θ)		= 1.00		AISC 15 <sup>th</sup> Page 8-9		
Fillet weld shear strength	$R_{n-w} = 0.6 (C_1)$	x 70 ksi) 0.70	07 w n C <sub>2</sub>	= 18.56	[kip/in]	AISC 15 <sup>th</sup> Eq 8-1		
Base metal - gusset plate	thickness t = 0.500	[in]	tensile F	<sub>u</sub> = 65.0	[ksi]			
Base metal - gusset plate is in shea	r, <u>shear</u> rupture as per A	ISC 15 <sup>th</sup> Eq	J4-4 is checked			AISC 15 <sup>th</sup> J2.4		
Base metal shear rupture	$R_{n-b} = 0.6 F_{u}t$			= 19.50	[kip/in]	AISC 15 <sup>th</sup> Eq J4-4		
Double fillet linear shear strength	 R <sub>n</sub> = min ( R <sub>r</sub>	<sub>n-w</sub> , R <sub>n-b</sub> )		= 18.559	[kip/in]	AISC 15 <sup>th</sup> Eq 9-2		
Resistance factor-LRFD	φ = 0.75					AISC 15 <sup>th</sup> Eq 8-1		
	φ R <sub>n</sub> =			= 13.92	[kip/in]			
When gusset plate is directly welded to allow adequate force redistribution	d to beam or column, ap In in the weld group	ply 1.25 duct	ility factor			AISC 15 <sup>th</sup> Page 13-11		
Weld strength used for design after	$\phi R_n = \phi R_n x$ (	1/1.25)		= 11.14	[kip/in]			
	ratio = <b>0.25</b>			> f <sub>max</sub>	ОК			

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Vertical Brace Connection

Beam Web Local Yielding			ratio = 17.5 / 179.4	= 0.10	PASS
Concentrated force from gusset	P <sub>u</sub> =		= 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	I <sub>b</sub> = Gusset/	/Beam interface ler	gth = 12.442	[in]	
Gusset plate corner clip	clip = from us	ser input	= 0.750	[in]	
Distance from normal force applied point to member end	$I_{\rm N} = 0.5 I_{\rm b} +$	- clip	= 6.971	[in]	
	when $I_N \leq 0$	d , use AISC 15 <sup>th</sup> B	Eq J10-3		AISC 15 <sup>th</sup> Eq J10-3
Beam web local yielding strength	$R_n = F_y t_w (1)$	2.5 k + I <sub>b</sub> )	= 179.4	[kips]	AISC 15 <sup>th</sup> Eq J10-3
Resistance factor-LRFD	φ = 1.00				
	φ R <sub>n</sub> =		= 179.4	[kips]	
	ratio = <b>0.10</b>		> P <sub>u</sub>	OK	
Beam Web Local Crippling			ratio = 17.5 / 178.4	= 0.10	PASS
Concentrated force from gusset	P <sub>u</sub> =		= 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	$I_b = Gusset/$	/Beam interface ler	gth = 12.442	[in]	
Gusset plate corner clip	clip = from us	ser input	= 0.750	[in]	
Distance from normal force applied point to member end	$I_{\rm N} = 0.5 I_{\rm b} +$	- clip	= 6.971	[in]	
	when $ _{N} \ge 0$	d/2 , use Eq J10-4			AISC 15 <sup>th</sup> 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 = 237.8	[kips]	AISC 15 <sup>th</sup> Eq J10-4
	$\left(\frac{E F_{y} t}{t_{w}}\right)$	<sup>f</sup> ) <sup>0.5</sup>			
Resistance factor-LRFD	φ = 0.75				AISC 15 <sup>th</sup> J10.3
	$\phi R_n =$		= 178.4	[kips]	
	ratio = <b>0.10</b>		> P <sub>u</sub>	OK	

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Vertical Brace Connection

Beam to Column	End Plate Connection		Co	ode=AISC 360-16 LRFD
Result Summary	geometries & weld limitations = <b>PASS</b>	limit states max ra	tio = <b>0.99</b>	PASS
Geometry Restriction Chec	k - End Plate to Column Flange			PASS
Min Bolt Edge Distance - End	Plate to Column Flange			
Bolt diameter	d <sub>b</sub> =	= 0.750	[in]	
Min edge distance allowed	L <sub>e-min</sub> =	= 1.000	[in]	AISC 15 <sup>th</sup> Table J3.4
Min edge distance in End Plate to	L <sub>e</sub> =	= 1.375	[in]	
Column Hange		≥ L <sub>e-min</sub>	OK	
Min Bolt Spacing - End Plate	to Column Flange			
Bolt diameter	d <sub>b</sub> =	= 0.750	[in]	
Min bolt spacing allowed	$L_{c,min} = 2.667  d_{b}$	= 2.000	[in]	AISC 15 <sup>th</sup> J3.3
Min Bolt spacing in End Plate to		= 3.500	[in]	
Column Flange	-s	>   .	OK	
		⊑ ⊑s-min		
Geometry Restriction Chec	k - End Plate-Bolt Gage Clearance			PASS
Bolt Gage Entering Clearance	Check - Plate Welded to End Plate			
Bolt diameter	d <sub>b</sub> = 0.750 [in]	gage g = 3.500	[in]	
Bolt entering clearance	$c_3 =$ from AISC manual Table 7-15	= 0.750	[in]	AISC 15 <sup>th</sup> 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]	
		≥ c <sub>3</sub>	ОК	AISC 15 <sup>th</sup> Table 7-15
Geometry Restriction Chec	k - Column Flange-Bolt Gage Clearance			PASS
Bolt Gage Entering Clearance	Check - Bolt on W Shape Flange			
Bolt diameter	d <sub>b</sub> = 0.750 [in]	gage g = 3.500	[in]	
Bolt entering clearance	c <sub>3</sub> = from AISC manual Table 7-15	= 0.750	[in]	AISC 15 <sup>th</sup> Table 7-15
W section	t <sub>w</sub> = 0.315 [in]	$k_1 = 0.748$	[in]	
Bolt center clearance distance to	$c = (g - 2k_1) / 2$	= 1.002	[in]	
		≥ c <sub>3</sub>	ОК	AISC 15 <sup>th</sup> Table 7-15
Beam Flange Fillet Weld Lii	mitation			PASS
Min Fillet Weld Size				
Thinner part joined thickness	 t =	= 0.402	[in]	
Min fillet weld size allowed	w <sub>min</sub> =	= 0.188	[in]	AISC 15 <sup>th</sup> Table J2.4
Fillet weld size provided	 w =	= 0.250	 [in]	
		≥ w <sub>min</sub>	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 15 <sup>th</sup> J2.2b
Min fillet weld length	$L = 0.5 b_{f} - k_{1}$	= 2.574	[in]	
		≥ L <sub>min</sub>	OK	

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Vertical Brace Connection

VB-Right

Beam Web Fillet Weld Limitation				PASS
Min Fillet Weld Size				
Thinner part joined thickness	t =	= 0.244	[in]	
Min fillet weld size allowed	w <sub>min</sub> =	= 0.125	[in]	AISC 15 <sup>th</sup> Table J2.4
Fillet weld size provided	w =	= 0.250	[in]	
		≥ w <sub>min</sub>	ОК	
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 15 <sup>th</sup> J2.2b
Min fillet weld length	L = d - 2 k	= 6.101	[in]	
		≥ L <sub>min</sub>	ОК	

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 ou	tside beam flange and count it as beam	n web extension to resist	shear	
Beam sect depth W200x36	d <sub>b</sub> = 7.913 [in]			
Bolt pitch & edge distance	d <sub>1</sub> = 2.500 [in]	e <sub>v</sub> = 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 <sub>p</sub> = 14.163 [in]	thickness $t_p = 0.244$	[in]	
Plate yield strength	F <sub>y</sub> = 50.0 [ksi]			
Plate gross area in shear	$A_{gv} = b_p t_p$	= 3.456	[in <sup>2</sup> ]	
Shear force required	V <sub>u</sub> =	= 11.3	[kips]	
Plate shear yielding strength	$R_n = 0.6 F_y A_{gv}$	= 103.7	[kips]	AISC 15 <sup>th</sup> Eq J4-3
Resistance factor-LRFD	$\varphi = 1.00$			AISC 15 <sup>th</sup> Eq J4-3
	φ R <sub>n</sub> =	= 103.7	[kips]	
	ratio = <b>0.11</b>	> V <sub>u</sub>	OK	

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Vertical Brace Connection

Beam Web - Shear Rupture		ratio = 11.3 / 101.1	= 0.11	PASS
Calculate gusset or stiff plate length c	outside beam flange and count it as bean	n web extension to resist	shear	
Beam sect depth W200x36	d <sub>b</sub> = 7.913 [in]			
Bolt pitch & edge distance	d <sub>1</sub> = 2.500 [in]	e <sub>v</sub> = 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 <sub>p</sub> = 14.163 [in]	thickness t <sub>p</sub> = 0.244	[in]	
Plate tensile strength	$F_{u} = 65.0$ [ksi]	r		
Plate net area in shear	$A_{nv} = b_p t_p$	= 3.456	[in <sup>2</sup> ]	
Shear force in demand	V <sub>u</sub> =	= 11.3	[kips]	
Plate shear rupture strength	$R_n = 0.6 F_u A_{nv}$	= 134.8	[kips]	AISC 15 <sup>th</sup> Eq J4-4
Resistance factor-LRFD	φ = 0.75			AISC 15 <sup>th</sup> Eq J4-4
	φ R <sub>n</sub> =	= 101.1	[kips]	
	ratio = <b>0.11</b>	> V <sub>u</sub>	ОК	
Beam - Tensile Yielding		ratio = 34.4 / 318.8	= 0.11	PASS
Gross area subject to tension	A <sub>g</sub> =	= 7.084	[in <sup>2</sup> ]	
Steel yield strength	F <sub>y</sub> =	= 50.0	[ksi]	
Tensile force required	P <sub>u</sub> =	= 34.4	[kips]	
Tensile yielding strength	$R_n = F_y A_g$	= 354.2	[kips]	AISC 15 <sup>th</sup> Eq D2-1
Resistance factor-LRFD	$\varphi = 0.90$			AISC 15 <sup>th</sup> D2 (a)
	$\phi R_n =$	= 318.8	[kips]	AISC 15 <sup>th</sup> Eq D2-1
	ratio = <b>0.11</b>	> P <sub>u</sub>	ОК	
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 <sup>2</sup> ]	
Shear lag factor	U =	= 1.000		
Tensile force required	P <sub>u</sub> =	= 34.4	[kips]	
Tensile effective net area	$A_e = A_n U$	= 7.084	[in <sup>2</sup> ]	
Plate tensile strength	F <sub>u</sub> =	= 65.0	[ksi]	
Tensile rupture strength	$R_n = F_u A_e$	= 460.5	[kips]	AISC 15 <sup>th</sup> Eq D2-2
Resistance factor-LRFD	φ = 0.75			AISC 15 <sup>th</sup> D2 (b)
	φ R <sub>n</sub> =	= 345.3	[kips]	AISC 15 <sup>th</sup> Eq D2-2
	ratio = <b>0.10</b>	> P <sub>u</sub>	OK	

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Vertical Brace Connection

2, 9.30 F M AISC Steel C		http://asp.civilba	ay.com/connect			VD-INI
End Plate - Shear Yield			ratio = 5.7 / 293.7	= 0.02	PASS	
Plate Shear Yielding Check						
Plate size	 width b = 15.663	[in]	thickness $t_{-} = 0.625$	[in]		
Plate vield strength	$F_{\rm u} = 50.0$	[ksi]	encertaine ep ereze	[]		
Plate gross area in shear	$A_{av} = b_{a} t_{a}$		= 9,789	[in <sup>2</sup> ]		
Shear force required	V =		= 5.7	[kips]		
Plate shear vielding strength	R = 0.6 F. A		= 293.7	[kips]	AISC 15 <sup>th</sup> Ea J4-3	
Resistance factor-LRFD	$\Phi = 1.00$	]v		[	AISC 15 <sup>th</sup> Eq J4-3	
	$\phi R_n =$		= 293.7	[kips]		
	ratio = <b>0.02</b>		> V.,	OK		
			u	_		
End Plate - Shear Rupture			ratio = 5.7 / 222.4	= 0.03	PASS	
Plate Shear Rupture Check						
Bolt hole diameter	bolt dia d <sub>b</sub> = $\frac{3}{4}$	[in] t	bolt hole dia $d_h = \frac{7}{8}$	[in]	AISC 15 <sup>th</sup> 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 <sub>u</sub> = 65.0	[ksi]	·			
Plate net area in shear	$A_{nv} = (b_p - nc)$	l <sub>h</sub> )t <sub>p</sub>	= 7.602	[in <sup>2</sup> ]		
Shear force required	V <sub>u</sub> =		= 5.7	[kips]		
Plate shear rupture strength	$R_{n} = 0.6 F_{u}A_{n}$	۱v	= 296.5	[kips]	AISC 15 <sup>th</sup> Eq J4-4	
Resistance factor-LRFD	φ = 0.75				AISC 15 <sup>th</sup> Eq J4-4	
	φ R <sub>n</sub> =		= 222.4	[kips]		
	ratio = <b>0.03</b>		> V <sub>u</sub>	ОК		
End Plate - Block Shear - Cen	ter Strip		ratio = 11.3 / 402.2	= 0.03	PASS	
Plate Block Shear - Center Strip						
Bolt hole diameter	bolt dia d $= \frac{3}{2}$	[in] ł	polt hole dia d $-\frac{7}{4}$	[in]	AISC 15 <sup>th</sup> B4 3b	
Plate thickness	$t_{b} = 0.625$	[in]	hole those that the mass of th	[]	AI3C 13 D4.50	
Plate strength	$r_p = 50.023$	[wsi]	F = 65.0	[ksi]		
Bolt no in ver & hor dir	$r_y = 30.0$	[K31]	$n_{\rm u} = 4$	[KSI]		
Bolt spacing in ver & hor dir	$n_v = 2$	[in]	$n_{h} = 4$	[in]		
Bolt edge dist in ver & hor dir	$s_v = 3.300$ e = 1.375	[in]	$s_h = 5.300$ $e_r = 1.375$	[in]		
		[]	c <sub>h</sub> = 1.575	[]		
Gross area subject to shear	$A_{gv} = [(n_{h} - 1)]$	$s_h + e_h ] t_p x 2$	= 14.844	4 [in <sup>2</sup> ]		
Net area subject to shear	$A_{nv} = A_{gv} - [(n + 1) + 1]$	<sub>h</sub> -1)+0.5]d <sub>h</sub> t <sub>p</sub>	x2 = 11.010	5 [in <sup>2</sup> ]		
Net area subject to tension						
when sheared out by center strip	$A_{nt} = (n_v - 1)$	(s <sub>v</sub> -d <sub>h</sub> )t <sub>p</sub>	= 1.641	[in <sup>2</sup> ]		
Block shear strength required	V <sub>u</sub> =		= 11.3	[kips]		
Uniform tension stress factor	$U_{bs} = 1.00$				AISC 15 <sup>th</sup> Fig C-J4.	2
Bolt shear resistance provided	R <sub>n</sub> = min (0.6	F <sub>u</sub> A <sub>nv</sub> , 0.6F <sub>y</sub> A <sub>av</sub>	,)+ = 536.3	[kips]	AISC 15 <sup>th</sup> Eq J4-5	
	U <sub>bs</sub> F <sub>u</sub> A	nt				
Resistance factor-LRFD	φ = 0.75				AISC 15 <sup>th</sup> Eq J4-5	
	$\phi R_n =$		= 402.2	[kips]		
	ratio = <b>0.03</b>		> V <sub>u</sub>	OK		

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Vertical Brace Connection

End Plate - Block Shear - 2-Sid	e Strip		ratio = 11	.3 / 379.3	= 0.03	PASS
Plate Block Shear - 2 Side Strips						
Bolt hole diameter	bolt dia d <sub>b</sub> = $\frac{3}{4}$	[in]	bolt hole dia	$d_{h} = \frac{7}{8}$	[in]	AISC 15 <sup>th</sup> B4.3b
Plate thickness	t <sub>p</sub> = 0.625	[in]				
Plate strength	$F_{y} = 50.0$	[ksi]		$F_{u} = 65.0$	[ksi]	
Bolt no in ver & hor dir	n <sub>v</sub> = 2			n <sub>h</sub> = 4		
Bolt spacing in ver & hor dir	s <sub>v</sub> = 3.500	[in]		s <sub>h</sub> = 3.500	[in]	
Bolt edge dist in ver & hor dir	e <sub>v</sub> = 1.375	[in]		e <sub>h</sub> = 1.375	[in]	
Gross area subject to shear	A <sub>gv</sub> = [ (n <sub>h</sub> - 1	l)s <sub>h</sub> +e <sub>h</sub> ]t <sub>p</sub> x	2	= 14.844	[in <sup>2</sup> ]	
Net area subject to shear	$A_{nv} = A_{gv} - [(n_{nv} - n_{nv})]$	n <sub>h</sub> -1)+0.5]d <sub>h</sub>	t <sub>p</sub> x2	= 11.016	[in <sup>2</sup> ]	
Net area subject to tension						
when sheared out by 2 side strips	$A_{nt} = (e_v - 0.)$	5d <sub>h</sub> )t <sub>p</sub> x2		= 1.172	[in <sup>2</sup> ]	
Block shear strength required	V <sub>u</sub> =			= 11.3	[kips]	
Uniform tension stress factor	$U_{bs} = 1.00$					AISC 15 <sup>th</sup> Fig C-J4.2
Bolt shear resistance provided	$R_n = min (0.4)$ $U_{bs}F_u A$	6F <sub>u</sub> A <sub>nv</sub> , 0.6F <sub>y</sub> A A <sub>nt</sub>	( <sub>gv</sub> ) +	= 505.8	[kips]	AISC 15 <sup>th</sup> Eq J4-5
Resistance factor-LRFD	$\varphi = 0.75$					AISC 15 <sup>th</sup> Eq J4-5
	φ R <sub>n</sub> =			= 379.3	[kips]	
	ratio = <b>0.03</b>			> V <sub>u</sub>	ОК	

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Vertical Brace Connection

End Plate - Bolt Bearing on Er	nd Plate		ratio = 11.3 / 143.1	= 0.08	PASS
Single Bolt Shear Strength					
Bolt shear stress	bolt grade = A325-N		F <sub>nv</sub> = 54.0	[ksi]	AISC 15 <sup>th</sup> Table J3.2
	bolt dia d <sub>b</sub> = $0.750$	[in]	bolt area $A_b = 0.442$	[in <sup>2</sup> ]	
Single bolt shear strength	$R_{n-bolt} = F_{nv} A_b$		= 23.9	[kips]	AISC 15 <sup>th</sup> Eq J3-1
Bolt Bearing/TearOut Strength	on Plate				
Bolt hole diameter	bolt dia d <sub>b</sub> = $\frac{3}{4}$	[in]	bolt hole dia $d_h = \frac{13}{16}$	[in]	AISC 15 <sup>th</sup> Table J3.3
Bolt spacing & edge distance	spacing L <sub>s</sub> = 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	$R_{n-t\&b-in} = 1.5 L_c t$	F <sub>u</sub> ≤ 3.0 d <sub>b</sub> t F	L		AISC 15 <sup>th</sup> Eq J3-6b
	= 163.8 ≤	91.4	= 91.4	[kips]	
Bolt strength at interior	$R_{n-in} = min (R_{n-in})$	<sub>n-t&amp;b-in</sub> , R <sub>n-bolt</sub> )	= 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	$R_{n-t\&b-ed} = 1.5 L_c t$	F <sub>u</sub> ≤ 3.0 d <sub>b</sub> t F	L		AISC 15 <sup>th</sup> Eq J3-6b
	= 59.0 ≤ 9	91.4	= 59.0	[kips]	
Bolt strength at edge	$R_{n-ed} = min (R_{n-ed})$	n-t&b-ed , R n-bolt	) = 23.9	[kips]	
Number of bolt	interior n <sub>in</sub> = 6		edge n <sub>ed</sub> = 2		
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n-in}$	+ n <sub>ed</sub> R <sub>n-ed</sub>	= 190.9	[kips]	
Required shear strength	V <sub>u</sub> =		= 11.3	[kips]	
Bolt resistance factor-LRFD	φ = 0.75				AISC 15 <sup>th</sup> J3.10
	φ R <sub>n</sub> =		= 143.1	[kips]	
	ratio = <b>0.08</b>		> V <sub>u</sub>	OK	

End Plate / Column - Bolt Shear		ratio = 11.3 / 143.1	= 0.08	PASS
Bolt A325-N	dia d <sub>b</sub> = 0.750 [in]	$A_{b} = 0.442$	[in <sup>2</sup> ]	
Bolt shear stress	grade = A325-N	$F_{nv} = 54.0$	[ksi]	AISC 15 <sup>th</sup> Table J3.2
Number of bolt carried shear	n <sub>s</sub> = 8.0	shear plane $m = 1$		
Bolt group eccentricity coefficient	C <sub>ec</sub> =	= 1.000		
Required shear strength	V <sub>u</sub> =	= 11.3	[kips]	
Bolt shear strength	$R_n = F_{nv}A_bn_smC_{ec}$	= 190.9	[kips]	AISC 15 <sup>th</sup> Eq J3-1
Bolt resistance factor-LRFD	φ = 0.75			AISC 15 <sup>th</sup> Eq J3-1
	φ R <sub>n</sub> =	= 143.1	[kips]	
	ratio = <b>0.08</b>	> V <sub>u</sub>	OK	

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							-
End Plate / Column - Bolt Bea		ratio = 11.3 / 143	3.1	= 0.08	PASS		
Single Bolt Shear Strength							
Bolt shear stress	bolt grade = A325-N		$F_{nv} = 5$	4.0	[ksi]	AISC 15 <sup>th</sup>	Table J3.2
	bolt dia d $_{b}$ = 0.750	[in]	bolt area $A_b = 0$	.442	[in <sup>2</sup> ]		
Single bolt shear strength	$R_{n-bolt} = F_{nv}A_b$		= 2	3.9	[kips]	AISC 15 <sup>th</sup>	Eq J3-1
Bolt Bearing/TearOut Strength o	on Plate						
Bolt hole diameter	bolt dia $d_b = \frac{3}{4}$	[in]	bolt hole dia $d_h = \frac{12}{3}$	<sup>3</sup> / <sub>16</sub>	[in]	AISC 15 <sup>th</sup>	Table J3.3
Bolt spacing	spacing L <sub>s</sub> = 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	$R_{n-t\&b-in} = 1.5 L_{c}t$	F <sub>u</sub> ≤ 3.0 d <sub>b</sub> t m	F <sub>u</sub>			AISC $15^{\text{th}}$	Eq J3-6b
	= 139.1 ≤	77.7	= 7	7.7	[kips]		
Bolt strength at interior	$R_{n-in} = min (R_{n-in})$	<sub>n-t&amp;b-in</sub> , R <sub>n-bolt</sub> )	= 2	3.9	[kips]		
Number of bolt	interior n <sub>in</sub> = 8						
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n-in}$		= 1	90.9	[kips]		
Required shear strength	V <sub>u</sub> =		= 1	1.3	[kips]		
Bolt resistance factor-LRFD	φ = 0.75					AISC $15^{th}$	J3.10
	φ R <sub>n</sub> =		= 1	43.1	[kips]		
	ratio = <b>0.08</b>		> V	′u	ОК		

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Vertical Brace Connection

VB-Right

Bolt Tensile Prving Action on E	nd Plate		ratio = 4.3 / 23.0	б :	= 0.18	PASS	
Bolt group forces	shear V = 11.3	[kips]	axial P = -	-34.4 [k	ips]	1455	
					1-1		
Single Bolt Tensile Capacity With	out Considering Pryi	ng			2-		
Bolt grade A325-N	dia d <sub>b</sub> = 0.750	[in]	area A <sub>b</sub> = 0	).442 [ir	1 <sup>2</sup> ]	and a sth	
Nominal tensile/shear stress	$F_{nt} = 90.0$	[ksi]	$F_{nv} = 5$	54.0 [k	sij	AISC 15"	Table J3.2
Bolt group shear force	shear V = $11.3$	[kips]	no of bolt $n = 8$	3	- : 7		
Snear stress required	$r_{rv} = v / (n)$	ч <sub>b</sub> )	= :	3.2 [K	SIJ		12 7
	$\varphi = 0.75$	Fat				AISC 15	72.7
Modified nominal tensile stress	$F'_{nt} = 1.3 F_{nt}$	$\phi F_{nv} f_{rv}$	≤ F <sub>nt</sub> = 9	<b>90.0</b> [k	si]	AISC 15 <sup>th</sup>	Eq J3-3a
Bolt norminal tensile strength	$r_n = F'_{nt} A_b$		= 3	39.8 [k	ips]	AISC 15 <sup>th</sup>	Eq J3-1
Resistance factor-LRFD	φ = 0.75					AISC 15 <sup>th</sup>	J3.6
Single bolt tensile capacity	φ r <sub>n</sub> =		= 2	<b>29.8</b> [k	ips]		
Single Bolt Tensile Capacity After	Considering Prying						
End plate	width $w = 6.250$	[in]	bolt gage g = 3	3.500 [ir	ן ו		
	web $t_{w} = 0.244$	[in]					
Dist from bolt center to plate edge	 a = 0.5 (w -	a)	= 1	1.375 [ir	าไ		
	a' = a + 0.5	d <sub>h</sub> ≤ (1.25	$b + 0.5 d_{\rm b}$ ) = 1	1.750 [ir	1]	AISC 15 <sup>th</sup>	Eq 9-23
				-	_		
Bolt hole diameter	bolt dia $d_b = 0.750$	[in]	bolt hole dia $d_h = 0$	0.813 [ir	ן -	AISC 15 <sup>th</sup>	B4.3b
Dist from bolt center to face of web	b = 0.5(g - 1)	t <sub>w</sub> )	= 1	1.628 [ir	ן י	and a sth	
	b' = b - 0.5	d <sub>b</sub>	= ]	1.253 [ir	ו	AISC 15"	Eq 9-18
Bolt pitch spacing	$s_v = 3.500$	[in]					
Bolt tributary length	$p = s_v p$	≤ 2b and p	$\leq s_v = 3$	3.256 [ir	ו]	AISC 15 <sup>th</sup>	Page 9-12
	ρ = b' / a'		= (	0.716		AISC 15 <sup>th</sup>	Eq 9-22
	$\delta = 1 - d_h/$	р	= (	0.750		AISC $15^{\text{th}}$	Eq 9-20
Tensile capacity per bolt before considering prying	B = from ca	lc shown in	above section = 2	29.8 [k	ips]		
Resistance factor-LRFD	φ = 0.90					AISC 15 <sup>th</sup>	Page 9-12
End plate thickness	t = 0.625	[in]	tensile $F_u = 6$	55.0 [k	si]		
Plate thickness req'd to develop bolt	$t_c = \left(\frac{4 B b}{\Phi n F}\right)$	') <sup>0.5</sup>	= (	).886 [ir	ן ו	AISC 15 <sup>th</sup>	Eq 9-26a
······································	1	u t <sub>c</sub>	2				
	$\alpha' = \frac{1}{\delta (1 + 1)}$	ρ) [( <u>t</u> ]	2 - 1 ] = (	0.783		AISC 15 <sup>th</sup>	Eq 9-28
when $0 \le \alpha' \le 1$	$Q = \left(\frac{t}{t_c}\right)^2$	(1 + δ α')	= (	0.790		AISC 15 <sup>th</sup>	Eq 9-27
Bolt tensile force per bolt in demand	T = from ca	lc shown be	ow = 4	<b>4.3</b> [k	ips]		
Tensile strength per bolt after considering prving	$\phi r_n = B \times Q$		= 2	<b>23.6</b> [k	ips]	AISC 15 <sup>th</sup>	Eq 9-27
	ratio = <b>0.18</b>		٦ <	г 📔	ОК		
Calculate Max Single Bolt Tensile	Load						
Bolt group force	axial P = 34.4	[kips]					
Bolt number	Bolt Row n <sub>h</sub> = 2		Bolt Col n <sub>v</sub> = 4	1			
Bolt tensile force per bolt	 T = P / ( n <sub>v</sub>	n <sub>h</sub> )	= 4	<b>4.3</b> [k	ips]		
Bolt Tensile Prving Action on C	olumn Flange		ratio = 4.3 / 18.8	8 =	= 0.23	PASS	
Bolt group forces	shear V = 11.3	[kips]	axial P = -	-34.4 [k	ips]		

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Vertical Brace Connection

VB-Right

	, <u>,</u>					
Bolt grade A325-N	dia d <sub>b</sub> = 0.750 [	[in]	area A <sub>b</sub>	= 0.442	[in <sup>2</sup> ]	
Nominal tensile/shear stress	F <sub>nt</sub> = 90.0 [	[ksi]	F <sub>nv</sub> :	= 54.0	[ksi]	AISC 15 <sup>th</sup> 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 <sup>th</sup> J3.7
Modified nominal tensile stress	F' <sub>nt</sub> = 1.3 F <sub>nt</sub>	F <sub>nt</sub> φF <sub>nv</sub> f <sub>rv</sub> ≤ F <sub>n</sub>	: :	= 90.0	[ksi]	AISC 15 <sup>th</sup> Eq J3-3a
Bolt norminal tensile strength	$r_n = F'_{nt}A_b$		:	= 39.8	[kips]	AISC 15 <sup>th</sup> Eq J3-1
Resistance factor-LRFD	φ = 0.75					AISC 15 <sup>th</sup> J3.6
Single bolt tensile capacity	φ r <sub>n</sub> =		:	= 29.8	[kips]	
Single Bolt Tensile Capacity After	Considering Prying					
Column flange as tee	b <sub>f</sub> = 7.992 [	[in]	bolt gage g :	= 3.500	[in]	
	web t <sub>w</sub> = 0.315 [	[in]				
Dist from bolt center to flange edge	a <sub>cf</sub> = 0.5 ( b <sub>f</sub> -	g )	:	= 2.246	[in]	
End plata		[in]	holt acces	- 2 500	[in]	
Enu place	width $W = 6.250$ [	[III] a )	bolt gage g :	- 1 275	[II]	
Dist nom boit center to plate edge	$a_{pl} = 0.5 (W - 0)$	y)	:	- 1.3/3	[111]	
Dist from bolt center to plate edge	a = min ( a <sub>cf</sub> ,	, a <sub>pl</sub> )	:	= 1.375	[in]	
	a' = a + 0.5 d	<sub>b</sub> ≤ (1.25 b + 0	).5 d <sub>b</sub> )	= 1.750	[in]	AISC 15 <sup>th</sup> Eq 9-23
Bolt hole diameter	bolt dia d <sub>b</sub> = 0.750 [	[in]	bolt hole dia d <sub>h</sub>	= 0.813	[in]	AISC 15 <sup>th</sup> B4.3b
Dist from bolt center to face of web	$b = 0.5(g - t_w)$	,)	:	= 1.593	[in]	
	b' = b - 0.5 d <sub>b</sub>	)	:	= 1.218	[in]	AISC 15 <sup>th</sup> Eq 9-18
Bolt pitch spacing		[in]				
Bolt tributary length	$p = s_v  p \le c_v$	2b and $p \leq s_v$	:	= 3.185	[in]	AISC 15 <sup>th</sup> Page 9-12
				0.000		
	$\rho = D / a$			- 0 745		AISC $15^{\text{th}}$ Eq 9-22
Tensile capacity per bolt before	B = from calc	shown in abov	e section	- 29.8	[kine]	AISC 15 Lq 9-20
considering prying		5110 WIT IIT 0000		- 29.0	[Kib2]	
Resistance ractor-LKFD	$\varphi = 0.90$	[in]	toncilo E	- 65 0	[kei]	AISC IS Page 9-1.
Diata thickness reald to douglon bolt	נ = 0.551 נ 4 B h'	[III]	tensne r <sub>u</sub>	- 05.0	[גאו]	
tensile capacity without prying	$t_c = (\frac{1}{\phi p F_u})$	-) <sup>0.5</sup>	:	= 0.883	[in]	AISC 15 <sup>th</sup> Eq 9-26a
	$\alpha' = \frac{1}{\delta (1 + \rho)}$	$\frac{t_c}{t}$ [ ( $\frac{t_c}{t}$ ) <sup>2</sup> - 1	] -	= 1.397		AISC 15 <sup>th</sup> Eq 9-28
when $\alpha' > 1$	$Q = \left(\frac{t}{t_c}\right)^2 (1)$	1 + δ)		= 0.631		AISC 15 <sup>th</sup> Eq 9-27
Bolt tensile force per bolt in demand	T = from calc	shown below		= 4.3	[kips]	
Tensile strength per bolt after	φ r_ = Β x Ο		:	= 18.8	[kips]	AISC 15 <sup>th</sup> Ea 9-27
considering prying	ratio - 0 22					
				~ 1		
Calculate Max Single Bolt Tensile	Load					
Bolt group force	axial P = 34.4 [	[kips]				
Bolt number	Bolt Row $n_h = 2$		Bolt Col n <sub>v</sub> :	= 4		
Bolt tensile force per bolt	$T = P / (n_{\perp}n_{\perp})$	ь)	:	= 4.3	[kips]	

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Beam Flange Weld Strength			ratio = 2.9	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]								
Been earlier W20020		[:-]		k 0.000	[:]					
Beam Section W200x36	$a_{b} = 7.913$	[IN] [in]		$k_{b} = 0.906$	[IN]					
	$D_{\rm fb} = 0.490$	[III]		к <sub>1b</sub> – 0.551	[111]					
Ver weld length on beam web	$L_{w1} = d_{b} - 2 k$	b		= 6.101	[in]					
Bolt pitch & edge distance	d <sub>1</sub> = 2.500	[in]		e <sub>v</sub> =1.375	[in]					
Ver weld length outside flange-weld line on stiff plate	$L_{e} = 2(d_{1} +$	e <sub>v</sub> )-2x0.7	5 in clip	= 6.250	[in]					
Ver weld length - total	$L_w = L_{w1} + L$	e		= 12.351	[in]					
Fillet weld length on beam flange	$L_{f} = (2b_{fb} - 2)$	2k <sub>1b</sub> )/2 as	double fillet	= 5.945	[in]					
Fillet weld length - total	$L = L_w + 2$	L <sub>f</sub>		= 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	e flange dbl	fillet weld length	= 5.945	[in]					
Beam flange fillet weld size	w =			= 0.250	[in]					
Combined Weld Stress										
Weld stress from axial <u>tensile</u> 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	$f_{max} = (f_a^2 + f_a)$	f <sup>2</sup> ) <sup>0.5</sup>		= 2.931	[kip/in]	AISC 15 <sup>th</sup> Eq 8-11				
Weld stress load angle	$\theta = \tan^{-1}($	f <sub>a</sub> f <sub>v</sub> )		= 80.8	[°]					
Fillet Weld Strength Calc										
Fillet weld leg size	$w = \frac{1}{4}$	[in]	load angl	eθ= 80.8	[°]					
Electrode strength	$F_{EXX} = 70.0$	[ksi]	strength coeff	C <sub>1</sub> = 1.00		AISC 15 <sup>th</sup> Table 8-3				
Number of weld line	n = 2 for c	double fillet								
Load angle coefficient	$C_2 = (1 + 0)$	.5 sin <sup>1.5</sup> θ)		= 1.49		AISC 15 <sup>th</sup> Page 8-9				
Fillet weld shear strength	$R_{n-w} = 0.6 (C_1)$	x 70 ksi) 0.3	707 w n C <sub>2</sub>	= 22.13	[kip/in]	AISC 15 <sup>th</sup> Eq 8-1				
Base metal - beam web th	nickness t = 0.402	[in]	tensile	e F <sub>u</sub> = 65.0	[ksi]					
Base metal - beam web is in shear, <u>she</u>	<u>ar</u> rupture as per AI	SC 15 <sup>th</sup> Eq.	J4-4 is checked			AISC 15 <sup>th</sup> J2.4				
Base metal shear rupture	$R_{n-b} = 0.6 F_u t$			= 15.68	[kip/in]	AISC 15 <sup>th</sup> Eq J4-4				
Double fillet linear shear strength	R <sub>n</sub> = min ( R	<sub>n-w</sub> , R <sub>n-b</sub> )		= 15.678	[kip/in]	AISC 15 <sup>th</sup> Eq 9-2				
Resistance factor-LRFD	φ = 0.75					AISC 15 <sup>th</sup> Eq 8-1				
	φ R <sub>n</sub> =			= 11.759	[kip/in]					
	ratio = <b>0.25</b>			> f <sub>max</sub>	ОК					

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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 fro	m Gusset Plate Inte	erface Forces Ca	lc above						
Axial force	axial $P = -34.4$	[kips]				in tension			
Shear force	shear V = 11.3	[kips]							
Beam section W200x36	d <sub>b</sub> = 7.913	[in]	k	<sub>b</sub> = 0.906	[in]				
	$b_{fb} = 6.496$	[in]	k <sub>1</sub>	<sub>.b</sub> = 0.551	[in]				
Ver weld length on beam web	$L_{w1} = d_{b} - 2 k$	< <sub>b</sub>		= 6.101	[in]				
Bolt pitch & edge distance	$d_1 = 2.500$	[in]	e	<sub>v</sub> = 1.375	[in]				
Ver weld length outside flange-weld line on stiff plate	$L_{e} = 2(d_{1} +$	e <sub>v</sub> ) - 2x0.75 in	clip	= 6.250	[in]				
Ver weld length - total	$L_w = L_{w1} + L_{w1}$	e		= 12.351	[in]				
Fillet weld length on beam flange	$L_{f} = (2b_{fb} -$	2k <sub>1b</sub> ) / 2 as do	uble fillet	= 5.945	[in]				
Fillet weld length - total	$L = L_w + 2$	L <sub>f</sub>		= 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	$w = \frac{1}{4}$	[in]	load angle	θ = 0.0	[°]				
Electrode strength	F <sub>EXX</sub> = 70.0	[ksi]	strength coeff C	<sub>1</sub> = 1.00		AISC 15 <sup>th</sup> Table 8-3			
Number of weld line	n = 2 for a	double fillet							
Load angle coefficient	$C_2 = (1 + 0)$	.5 sin <sup>1.5</sup> θ)		= 1.00		AISC 15 <sup>th</sup> Page 8-9			
Fillet weld shear strength	$R_{n-w} = 0.6 (C_1)$	<sub>L</sub> x 70 ksi) 0.707	'wnC <sub>2</sub>	= 14.85	[kip/in]	AISC 15 <sup>th</sup> Eq 8-1			
Base metal - beam web th	nickness t = 0.244	[in]	tensile F	<sub>u</sub> = 65.0	[ksi]				
Base metal - beam web is in shear, <u>shea</u>	<u>ar</u> rupture as per AI	ISC 15 <sup>th</sup> Eq J4-4	4 is checked			AISC 15 <sup>th</sup> J2.4			
Base metal shear rupture	$R_{n-b} = 0.6 F_{u}t$	t		= 9.52	[kip/in]	AISC 15 <sup>th</sup> Eq J4-4			
Double fillet linear shear strength	R <sub>n</sub> = min ( R	R <sub>n-w</sub> , R <sub>n-b</sub> )		= 9.516	[kip/in]	AISC 15 <sup>th</sup> Eq 9-2			
Resistance factor-LRFD	φ = 0.75					AISC 15 <sup>th</sup> Eq 8-1			
	$\phi R_n =$			= 7.137	[kip/in]				
Weld resistance required	V <sub>u</sub> =			= 5.8	[kips]				
Fillet weld length - double fillet	L =			= 12.351	[in]				
Weld resistance provided	$\phi F_n = \phi R_n x$	L		= 88.1	[kips]				
	ratio = <b>0.07</b>			> V <sub>u</sub>	OK				

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Vertical Brace Connection

Column Web Local Yielding		ratio = 34.4 / 296.4	= 0.12	PASS
Concentrated force on column	P <sub>u</sub> =	= 34.4	[kips]	
Column section	d = 9.921 [in]	$t_{f} = 0.531$	[in]	
	t <sub>w</sub> = 0.315 [in]	k = 1.181	[in]	
	yield $F_y = 50.0$ [ksi]			
Length of bearing	I <sub>b</sub> = end plate length	= 12.913	[in]	
Column web local yielding strength	$R_n = F_y t_w (5 k + I_b)$	= 296.4	[kips]	AISC 15 <sup>th</sup> Eq J10-2
Resistance factor-LRFD	$\varphi = 1.00$			
	φ R <sub>n</sub> =	= 296.4	[kips]	
	ratio = <b>0.12</b>	> P <sub>u</sub>	OK	
Column Flange Local Bending		ratio = 34.4 / 79.3	= 0.43	PASS
Concentrated force from gusset	P <sub>u</sub> =	= 34.4	[kips]	
Column w section	t <sub>f</sub> = 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 15 <sup>th</sup> Eq J10-1
Resistance factor-LRFD	$\varphi = 0.90$			AISC 15 <sup>th</sup> J10.1
	ф В =	= 79.3	[kips]	
	$\Psi$ $\kappa_n$ –			
	ratio = <b>0.43</b>	> P <sub>u</sub>	OK	
Brace Force Load Case 2	ratio = <b>0.43</b> shear V = 46.3 kips axial P = -78	> P <sub>u</sub> 8.2 kips (T) rat	OK tio = <b>0.99</b>	PASS
Brace Force Load Case 2 Beam Web - Shear Yielding	ratio = <b>0.43</b> shear V = 46.3 kips axial P = -78	> P <sub>u</sub> 8.2 kips (T) rat ratio = 46.3 / 103.7	OK tio = 0.99 = 0.45	PASS
Brace Force Load Case 2 Beam Web - Shear Yielding Calculate gusset or stiff plate length o	ratio = <b>0.43</b> shear V = 46.3 kips axial P = -78	> P <sub>u</sub> 8.2 kips (T) rat ratio = 46.3 / 103.7 m web extension to resist	OK tio = <b>0.99</b> = <b>0.45</b> shear	PASS
Brace Force Load Case 2 Beam Web - Shear Yielding Calculate gusset or stiff plate length of Beam sect depth W200x36	ratio = <b>0.43</b> shear V = 46.3 kips axial P = -78 putside beam flange and count it as bear $d_b = 7.913$ [in]	$P_u$ 8.2 kips (T) rat ratio = 46.3 / 103.7 m web extension to resist	OK tio = <b>0.99</b> = <b>0.45</b> shear	PASS
Brace Force Load Case 2 Beam Web - Shear Yielding Calculate gusset or stiff plate length of Beam sect depth W200x36 Bolt pitch & edge distance	ratio = <b>0.43</b> shear V = 46.3 kips axial P = -78 putside beam flange and count it as bear $d_b = 7.913$ [in] $d_1 = 2.500$ [in]	$> P_u$ 8.2 kips (T) ratio ratio = 46.3 / 103.7 m web extension to resist $e_v = 1.375$	OK tio = <b>0.99</b> = <b>0.45</b> shear [in]	PASS
Brace Force Load Case 2 Beam Web - Shear Yielding Calculate gusset or stiff plate length of Beam sect depth W200x36 Bolt pitch & edge distance Beam web extension outside beam flange	ratio = <b>0.43</b> shear V = 46.3 kips axial P = -78 putside beam flange and count it as bear $d_b = 7.913$ [in] $d_1 = 2.500$ [in] $L_e = 2(d_1 + e_v) - 2x0.75$ in clip	$P_{u}$ 8.2 kips (T) ratio ratio = 46.3 / 103.7 m web extension to resist $e_{v} = 1.375$ p = 6.250	OK tio = <b>0.99</b> = <b>0.45</b> shear [in] [in]	PASS
Brace Force Load Case 2 Beam Web - Shear Yielding Calculate gusset or stiff plate length of Beam sect depth W200x36 Bolt pitch & edge distance Beam web extension outside beam flange Total beam web depth to resist shear	ratio = <b>0.43</b> shear V = 46.3 kips axial P = -76 putside beam flange and count it as beam $d_b = 7.913$ [in] $d_1 = 2.500$ [in] $L_e = 2(d_1 + e_v) - 2x0.75$ in clip $L = d_b + L_e$	> $P_u$ 8.2 kips (T) rat ratio = 46.3 / 103.7 m web extension to resist $e_v = 1.375$ p = 6.250 = <b>14.163</b>	OK tio = <b>0.99</b> = <b>0.45</b> shear [in] [in]	PASS
Brace Force Load Case 2 Beam Web - Shear Yielding Calculate gusset or stiff plate length of Beam sect depth W200x36 Bolt pitch & edge distance Beam web extension outside beam flange Total beam web depth to resist shear Plate Shear Yielding Check	$q_{1}r_{n} = ratio = 0.43$ shear V = 46.3 kips axial P = -78 butside beam flange and count it as bear $d_{b} = 7.913  [in]$ $d_{1} = 2.500  [in]$ $L_{e} = 2(d_{1}+e_{v}) - 2x0.75 \text{ in clip}$ $L = d_{b} + L_{e}$	> $P_u$ 8.2 kips (T) rat ratio = 46.3 / 103.7 m web extension to resist $e_v = 1.375$ p = 6.250 = 14.163	OK tio = <b>0.99</b> = <b>0.45</b> shear [in] [in] [in]	PASS
Brace Force Load Case 2 Beam Web - Shear Yielding Calculate gusset or stiff plate length of Beam sect depth W200x36 Bolt pitch & edge distance Beam web extension outside beam flange Total beam web depth to resist shear Plate Shear Yielding Check Plate size	ratio = <b>0.43</b> shear V = 46.3 kips axial P = -78 putside beam flange and count it as bear $d_b = 7.913$ [in] $d_1 = 2.500$ [in] $L_e = 2(d_1 + e_v) - 2x0.75$ in clip $L = d_b + L_e$ width $b_p = 14.163$ [in]	$P_{u}$ 8.2 kips (T) ratio ratio = 46.3 / 103.7 m web extension to resist $e_{v} = 1.375$ p = 6.250 = 14.163 thickness t <sub>p</sub> = 0.244	OK tio = <b>0.99</b> = <b>0.45</b> shear [in] [in] [in]	PASS
Brace Force Load Case 2 Beam Web - Shear Yielding Calculate gusset or stiff plate length of Beam sect depth W200x36 Bolt pitch & edge distance Beam web extension outside beam flange Total beam web depth to resist shear Plate Shear Yielding Check Plate size Plate yield strength	ratio = <b>0.43</b> shear V = 46.3 kips axial P = -78 putside beam flange and count it as bear $d_b = 7.913$ [in] $d_1 = 2.500$ [in] $L_e = 2(d_1 + e_v) - 2x0.75$ in clip $L = d_b + L_e$ width $b_p = 14.163$ [in] $F_y = 50.0$ [ksi]	$> P_u$ 8.2 kips (T) ratio ratio = 46.3 / 103.7 m web extension to resist $e_v = 1.375$ p = 6.250 = 14.163 thickness $t_p = 0.244$	OK tio = <b>0.99</b> = <b>0.45</b> shear [in] [in] [in]	PASS
Brace Force Load Case 2 Beam Web - Shear Yielding Calculate gusset or stiff plate length of Beam sect depth W200x36 Bolt pitch & edge distance Beam web extension outside beam flange Total beam web depth to resist shear Plate Shear Yielding Check Plate size Plate yield strength Plate gross area in shear	ratio = <b>0.43</b> shear V = 46.3 kips axial P = -78 butside beam flange and count it as beam $d_b = 7.913$ [in] $d_1 = 2.500$ [in] $L_e = 2(d_1 + e_v) - 2x0.75$ in clip $L = d_b + L_e$ width $b_p = 14.163$ [in] $F_y = 50.0$ [ksi] $A_{gv} = b_p t_p$	> $P_u$ 8.2 kips (T) ratio ratio = 46.3 / 103.7 m web extension to resist $e_v = 1.375$ p = 6.250 = 14.163 thickness $t_p = 0.244$ = 3.456	OK tio = <b>0.99</b> = <b>0.45</b> shear [in] [in] [in] [in]	PASS
Brace Force Load Case 2 Beam Web - Shear Yielding Calculate gusset or stiff plate length of Beam sect depth W200x36 Bolt pitch & edge distance Beam web extension outside beam flange Total beam web depth to resist shear Plate Shear Yielding Check Plate size Plate yield strength Plate gross area in shear Shear force required	ratio = <b>0.43</b> shear V = 46.3 kips axial P = -78 butside beam flange and count it as bear $d_b = 7.913$ [in] $d_1 = 2.500$ [in] $L_e = 2(d_1 + e_v) - 2x0.75$ in clip $L = d_b + L_e$ width $b_p = 14.163$ [in] $F_v = 50.0$ [ksi] $A_{gv} = b_p t_p$ $V_u =$	> $P_u$ 8.2 kips (T) ratio ratio = 46.3 / 103.7 m web extension to resist $e_v = 1.375$ p = 6.250 = 14.163 thickness $t_p = 0.244$ = 3.456 = 46.3	OK tio = <b>0.99</b> = <b>0.45</b> shear [in] [in] [in] [in] [in2] [kips]	PASS
Brace Force Load Case 2 Beam Web - Shear Yielding Calculate gusset or stiff plate length of Beam sect depth W200x36 Bolt pitch & edge distance Beam web extension outside beam flange Total beam web depth to resist shear Plate Shear Yielding Check Plate size Plate yield strength Plate gross area in shear Shear force required Plate shear yielding strength	$\varphi \cdot r_{n} =$ ratio = <b>0.43</b> shear V = 46.3 kips axial P = -78 butside beam flange and count it as bear $d_{b} = 7.913  [in]$ $d_{1} = 2.500  [in]$ $L_{e} = 2(d_{1}+e_{v}) - 2x0.75 \text{ in clip}$ $L = d_{b} + L_{e}$ width $b_{p} = 14.163  [in]$ $F_{y} = 50.0  [ksi]$ $A_{gv} = b_{p}t_{p}$ $V_{u} =$ $R_{n} = 0.6 F_{y}A_{gv}$	$> P_u$ 8.2 kips (T) rat ratio = 46.3 / 103.7 ratio = 46.3 / 103.7 ratio = 46.3 / 103.7 ratio = 46.350 ratio = 14.163 ratio = 46.3 ratio = 46.3 ratio = 103.7 ratio = 103.7	OK tio = <b>0.99</b> = <b>0.45</b> shear [in] [in] [in] [in] [in] [in] [kips] [kips]	PASS PASS
Brace Force Load Case 2 Beam Web - Shear Yielding Calculate gusset or stiff plate length of Beam sect depth W200x36 Bolt pitch & edge distance Beam web extension outside beam flange Total beam web depth to resist shear Plate Shear Yielding Check Plate size Plate yield strength Plate gross area in shear Shear force required Plate shear yielding strength Resistance factor-LRFD	ratio = <b>0.43</b> shear V = 46.3 kips axial P = -78 butside beam flange and count it as bear $d_b = 7.913$ [in] $d_1 = 2.500$ [in] $L_e = 2(d_1 + e_v) - 2x0.75$ in clip $L = d_b + L_e$ width $b_p = 14.163$ [in] $F_y = 50.0$ [ksi] $A_{gv} = b_p t_p$ $V_u =$ $R_n = 0.6 F_y A_{gv}$ $\phi = 1.00$	$> P_u$ 8.2 kips (T) ratio = 46.3 / 103.7 meb extension to resist p e_v = 1.375 p = 6.250 = 14.163 thickness t_p = 0.244 = 3.456 = 46.3 = 103.7	OK tio = <b>0.99</b> = <b>0.45</b> shear [in] [in] [in] [in] [in] [kips] [kips]	PASS PASS
Brace Force Load Case 2 Beam Web - Shear Yielding Calculate gusset or stiff plate length of Beam sect depth W200x36 Bolt pitch & edge distance Beam web extension outside beam flange Total beam web depth to resist shear Plate Shear Yielding Check Plate size Plate yield strength Plate gross area in shear Shear force required Plate shear yielding strength Resistance factor-LRFD	ratio = <b>0.43</b> shear V = 46.3 kips axial P = -78 putside beam flange and count it as bear $d_b = 7.913$ [in] $d_1 = 2.500$ [in] $L_e = 2(d_1 + e_v) - 2x0.75$ in clip $L = d_b + L_e$ width $b_p = 14.163$ [in] $F_y = 50.0$ [ksi] $A_{gv} = b_p t_p$ $V_u =$ $R_n = 0.6 F_v A_{gv}$ $\phi = 1.00$ $\phi R_n =$	$> P_u$ 8.2 kips (T) rat ratio = 46.3 / 103.7 ratio = 46.3 / 103.7 ratio = 46.3 / 103.7 ratio = 46.3 ratio = 46.3 ratio = 103.7 ratio = 103.7 ratio = 103.7	OK tio = <b>0.99</b> = <b>0.45</b> shear [in] [in] [in] [in] [kips] [kips] [kips]	PASS PASS

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Beam Web - Shear Rupture		ratio = 46.3 / 101.1	= 0.46	PASS
Calculate gusset or stiff plate length of	outside beam flange and count it as bean	n web extension to resist	shear	
Beam sect depth W200x36	d <sub>b</sub> = 7.913 [in]			
Bolt pitch & edge distance	d <sub>1</sub> = 2.500 [in]	e <sub>v</sub> = 1.375	[in]	
Beam web extension outside beam flange	$L_e = 2(d_1 + e_v) - 2x0.75$ in clip	o = 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 <sub>p</sub> = 14.163 [in]	thickness $t_p = 0.244$	[in]	
Plate tensile strength	F <sub>u</sub> = 65.0 [ksi]	F		
Plate net area in shear	$A_{nv} = b_p t_p$	= 3.456	[in <sup>2</sup> ]	
Shear force in demand	V <sub>u</sub> =	= 46.3	[kips]	
Plate shear rupture strength	$R_n = 0.6 F_u A_{nv}$	= 134.8	[kips]	AISC 15 <sup>th</sup> Eq J4-4
Resistance factor-LRFD	φ = 0.75			AISC 15 <sup>th</sup> Eq J4-4
	φ R <sub>n</sub> =	= 101.1	[kips]	
	ratio = <b>0.46</b>	> V <sub>u</sub>	ОК	
Beam - Tensile Yielding		ratio = 78.2 / 318.8	= 0.25	PASS
Gross area subject to tension	A <sub>g</sub> =	= 7.084	[in <sup>2</sup> ]	
Steel yield strength	F <sub>y</sub> =	= 50.0	[ksi]	
Tensile force required	P <sub>u</sub> =	= 78.2	[kips]	
Tensile yielding strength	$R_n = F_y A_g$	= 354.2	[kips]	AISC 15 <sup>th</sup> Eq D2-1
Resistance factor-LRFD	$\varphi = 0.90$			AISC 15 <sup>th</sup> D2 (a)
	φ R <sub>n</sub> =	= 318.8	[kips]	AISC 15 <sup>th</sup> Eq D2-1
	ratio = <b>0.25</b>	> P <sub>u</sub>	ОК	
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 <sup>2</sup> ]	
Shear lag factor	U =	= 1.000		
Tensile force required	P <sub>u</sub> =	= 78.2	[kips]	
Tensile effective net area	$A_e = A_n U$	= 7.084	[in <sup>2</sup> ]	
Plate tensile strength	F <sub>u</sub> =	= 65.0	[ksi]	
Tensile rupture strength	$R_n = F_u A_e$	= 460.5	[kips]	AISC 15 <sup>th</sup> Eq D2-2
Resistance factor-LRFD	φ = 0.75			AISC 15 <sup>th</sup> D2 (b)
	$\phi R_n =$	= 345.3	[kips]	AISC 15 <sup>th</sup> Eq D2-2
	ratio = <b>0.23</b>	> P <sub>u</sub>	ОК	

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	Sonneotion Beolgn					
End Plate - Shear Yield			ratio = 23.2 / 293.7	= 0.08	PASS	
Plate Shear Yielding Check						
Plate size	width b = 15.663	[in]	thickness t <sub>n</sub> = 0.625	[in]		
Plate yield strength	$F_{y} = 50.0$	[ksi]	μ μ			
Plate gross area in shear	$A_{av} = b_{a}t_{a}$		= 9.789	[in <sup>2</sup> ]		
Shear force required	уч рр V=		= 23.2	[kips]		
Plate shear yielding strength	$R_{n} = 0.6 F_{v}A_{n}$	W	= 293.7	[kips]	AISC 15 <sup>th</sup> Eq J4-3	
Resistance factor-LRFD	$\phi = 1.00$	•			AISC 15 <sup>th</sup> Eq J4-3	
	$\phi R_n =$		= 293.7	[kips]		
	ratio = <b>0.08</b>		> V <sub>u</sub>	ОК		
End Disto - Shoar Punturo			$r_{2} = 22.2 / 222.4$	- 0 10	DASS	
Plate Shear Punture Check			1010 = 23.2 / 222.4	= 0.10	PASS	
Bolt hole diameter	bolt dia d <sub>b</sub> = $\frac{3}{4}$	[in] b	olt hole dia $d_h = \frac{7}{8}$	[in]	AISC 15 <sup>th</sup> 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	$A_{nv} = (b_p - nd)$	<sub>h</sub> )t <sub>p</sub>	= 7.602	[in <sup>2</sup> ]		
Shear force required	V <sub>u</sub> =		= 23.2	[kips]		
Plate shear rupture strength	$R_{n} = 0.6 F_{u}A_{n}$	v	= 296.5	[kips]	AISC 15 <sup>th</sup> Eq J4-4	
Resistance factor-LRFD	φ = 0.75				AISC 15 <sup>th</sup> Eq J4-4	
	φ R <sub>n</sub> =		= 222.4	[kips]		
	ratio = <b>0.10</b>		> V <sub>u</sub>	OK		
End Plate - Block Shear - Cent	ter Strip		ratio = 46.3 / 402.2	= 0.12	PASS	
Plate Block Shear - Center Strip						
Bolt hole diameter	bolt dia $d_{h} = \frac{3}{4}$	[in] b	olt hole dia $d_{h} = \frac{7}{2}$	[in]	AISC 15 <sup>th</sup> B4.3b	
Plate thickness	$t_{\mu} = 0.625$	[in]	8			
Plate strength	ہ F <sub></sub> = 50.0	[ksi]	F., = 65.0	[ksi]		
Bolt no in ver & hor dir	y n <sub></sub> = 2		n <sub>b</sub> = 4			
Bolt spacing in ver & hor dir	s. = 3.500	[in]	s <sub>k</sub> = 3.500	[in]		
Bolt edge dist in ver & hor dir	e= 1.375	[in]	e <sub>⊾</sub> = 1.375	[in]		
	v		11			
Gross area subject to shear	A <sub>gv</sub> = [ (n <sub>h</sub> - 1)	$s_h + e_h ] t_p x 2$	= 14.84	4 [in <sup>2</sup> ]		
Net area subject to shear	$A_{nv} = A_{gv} - [(n_{f})]$	<sub>n</sub> -1)+0.5]d <sub>h</sub> t <sub>p</sub>	x2 = 11.01	6 [in <sup>2</sup> ]		
Net area subject to tension						
when sheared out by center strip	$A_{nt} = (n_v - 1)$	(s <sub>v</sub> -d <sub>h</sub> )t <sub>p</sub>	= 1.641	[in <sup>2</sup> ]		
Block shear strength required	V <sub>u</sub> =		= 46.3	[kips]		
Uniform tension stress factor	$U_{bs} = 1.00$				AISC 15 <sup>th</sup> Fig C-J4.	2
Bolt shear resistance provided	R <sub>n</sub> = min (0.6F	<sup>F</sup> uA <sub>nv</sub> , 0.6F <sub>y</sub> A <sub>gv</sub>	) + = 536.3	[kips]	AISC 15 <sup>th</sup> Eq J4-5	
	U <sub>bs</sub> F <sub>u</sub> A <sub>n</sub>	t				
Resistance factor-LRFD	φ = 0.75				AISC 15 <sup>th</sup> Eq J4-5	
	φ R <sub>n</sub> =		= 402.2	[kips]		
	ratio = <b>0.12</b>		> V <sub>u</sub>	OK		

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	onnection Design	nup.//as	p.civiibay.com/com				v D-i tigi
End Plate - Block Shear - 2-Sig	le Strip		ratio = 46	.3 / 379.3	= 0.12	PASS	
Plate Block Shear - 2 Side Strips							
Bolt hole diameter	bolt dia $d_b = \frac{3}{4}$	[in]	bolt hole dia	$d_{h} = \frac{7}{8}$	[in]	AISC 15 <sup>th</sup> 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 <sub>v</sub> = 2			n <sub>h</sub> = 4			
Bolt spacing in ver & hor dir	$s_v = 3.500$	[in]		s <sub>h</sub> = 3.500	[in]		
Bolt edge dist in ver & hor dir	e <sub>v</sub> = 1.375	[in]		e <sub>h</sub> = 1.375	[in]		
Gross area subject to shear	$A_{gv} = [(n_{h} - 1)]$	1) s <sub>h</sub> +e <sub>h</sub>	t <sub>p</sub> x2	= 14.844	[in <sup>2</sup> ]		
Net area subject to shear	$A_{nv} = A_{gv} - [(n_{mv} - n_{mv} -$	n <sub>h</sub> -1)+0.	5]d <sub>h</sub> t <sub>p</sub> x2	= 11.016	[in <sup>2</sup> ]		
Net area subject to tension							
when sheared out by 2 side strips	$A_{nt} = (e_v - 0.$	.5 d <sub>h</sub> )t <sub>p</sub> x	2	= 1.172	[in <sup>2</sup> ]		
Block shear strength required	V <sub>u</sub> =			= 46.3	[kips]		
Uniform tension stress factor	$U_{bs} = 1.00$					AISC 15 <sup>th</sup> Fig C-J4	.2
Bolt shear resistance provided	R <sub>n</sub> = min (0. U <sub>hs</sub> F <sub>u</sub> A	6F <sub>u</sub> A <sub>nv</sub> ,0	6F <sub>y</sub> A <sub>gv</sub> ) +	= 505.8	[kips]	AISC 15 <sup>th</sup> Eq J4-5	
Resistance factor-LRFD	$\varphi = 0.75$					AISC 15 <sup>th</sup> Eq J4-5	
	φ R <sub>n</sub> =			= 379.3	[kips]		
	ratio = <b>0.12</b>			> V <sub>u</sub>	OK		

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End Plate - Bolt Bearing on Er	nd Plate		ratio = 46.3 / 143.1	= 0.32	PASS
Single Bolt Shear Strength					
Bolt shear stress	bolt grade = A325-N		F <sub>nv</sub> = 54.0	[ksi]	AISC 15 <sup>th</sup> Table J3.2
	bolt dia $d_b = 0.750$	[in]	bolt area $A_b = 0.442$	[in <sup>2</sup> ]	
Single bolt shear strength	$R_{n-bolt} = F_{nv}A_b$		= 23.9	[kips]	AISC 15 <sup>th</sup> Eq J3-1
Bolt Bearing/TearOut Strength	on Plate				
Bolt hole diameter	bolt dia d <sub>b</sub> = $\frac{3}{4}$	[in]	bolt hole dia $d_h = \frac{13}{16}$	[in]	AISC 15 <sup>th</sup> 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	$R_{n-t\&b-in} = 1.5 L_c t$	$F_u \le 3.0 d_b t f$	- u		AISC 15 <sup>th</sup> Eq J3-6b
	= 163.8 ≤	91.4	= 91.4	[kips]	
Bolt strength at interior	R <sub>n-in</sub> = min ( R	<sub>n-t&amp;b-in</sub> , R <sub>n-bolt</sub>	) = 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	$R_{n-t\&b-ed} = 1.5 L_c t$	$F_u \le 3.0 d_b t f$	- u		AISC 15 <sup>th</sup> Eq J3-6b
	= 59.0 ≤	91.4	= 59.0	[kips]	
Bolt strength at edge	$R_{n-ed} = min (R)$	n-t&b-ed , R n-bol	= 23.9	[kips]	
Number of bolt	interior n <sub>in</sub> = 6		edge n <sub>ed</sub> = 2		
Bolt bearing strength for all bolts	$R_n = n_{in} R_{n-in}$	+ n <sub>ed</sub> R <sub>n-ed</sub>	= 190.9	[kips]	
Required shear strength	V <sub>u</sub> =		= 46.3	[kips]	
Bolt resistance factor-LRFD	φ = 0.75				AISC 15 <sup>th</sup> J3.10
	φ R <sub>n</sub> =		= 143.1	[kips]	
	ratio = <b>0.32</b>		> V <sub>u</sub>	ОК	

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Vertical Brace Connection

	-	· ·	•		-
End Plate / Column - Bolt She	ar		ratio = 46.3 / 139.2	= 0.33	PASS
Bolt A325-N	dia $d_{b} = 0.750$	[in]	$A_{b} = 0.442$	[in <sup>2</sup> ]	
Bolt shear stress	grade = A325-N		$F_{nv} = 54.0$	[ksi]	AISC 15 <sup>th</sup> Table J3.2
Modified shear stress after considering combined tensile stress	F' <sub>nv</sub> = from calo	c shown below	= 52.5	[ksi]	AISC 15 <sup>th</sup> J3.7
Number of bolt carried shear	n <sub>s</sub> = 8.0		shear plane $m = 1$		
Bolt group eccentricity coefficient	C <sub>ec</sub> =		= 1.000		
Required shear strength	V <sub>u</sub> =		= 46.3	[kips]	
Bolt shear strength	$R_n = F'_{nv} A_b n$	<sub>s</sub> m C <sub>ec</sub>	= 185.5	[kips]	AISC 15 <sup>th</sup> Eq J3-1
Bolt resistance factor-LRFD	φ = 0.75				AISC 15 <sup>th</sup> Eq J3-1
	φ R <sub>n</sub> =		= 139.2	[kips]	
	ratio = <b>0.33</b>		> V <sub>u</sub>	OK	
Check If Modified Shear Stress F	-' <sub>nv</sub> Shall Be Used				
Bolt group force	axial P = 78.2	[kips]			
Bolt grade	grade = A325-N				
Nominal shear stress	F <sub>nt</sub> = 90.0	[ksi]			AISC 15 <sup>th</sup> Table J3.2
	bolt dia $d_b = 0.750$	[in]	bolt area $A_b = 0.442$	[in <sup>2</sup> ]	
Bolt number	Bolt Row $n_v = 2$		Bolt Col n <sub>h</sub> = 4		
Tensile stress required	$f_{rt} = P / (n_v r)$	n <sub>h</sub> A <sub>b</sub> )	= 22.1	[ksi]	
Resistance factor-LRFD	φ = 0.75				AISC 15 <sup>th</sup> J3.7
Check tensile stress ratio limit	$= \frac{f_{rt}}{\Phi F_{nt}}$		= 0.33		
			> 0.3		AISC 15 <sup>th</sup> J3.7
Combined tensile/shear effect shall	be considered, modified	tensile stress F'	<sub>nv</sub> shall be used		
Calc Modified Shear Stress F' <sub>nv</sub> C	Considering Shear Effe	ct			
Bolt group force	axial P = 78.2	[kips]			
Bolt grade	grade = A325-N				
Nominal tensile/shear stress	F <sub>nt</sub> = 90.0	[ksi]	$F_{nv} = 54.0$	[ksi]	AISC 15 <sup>th</sup> Table J3.2
	bolt dia $d_b = 0.750$	[in]	bolt area $A_b = 0.442$	[in <sup>2</sup> ]	
Bolt number	Bolt Row $n_v = 2$		Bolt Col $n_h = 4$		
Tensile stress required	$f_{rt} = P / (n_v r)$	ι <sub>h</sub> A <sub>b</sub> )	= 22.1	[ksi]	
Resistance factor-LRFD	φ = 0.75				AISC 15 <sup>th</sup> J3.7
Modified nominal shear stress	F' <sub>nv</sub> = 1.3 F <sub>nv</sub> -	F <sub>nv</sub> φF <sub>nt</sub> f <sub>rt</sub> ≤ F <sub>nv</sub>	= 52.5	[ksi]	AISC 15 <sup>th</sup> Eq J3-3a

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Vertical Brace Connection

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End Plate / Column - Bolt	Bearing on Column		ratio = 46.3 / 143.1	= 0.32	2 PASS	
Single Bolt Shear Strength						
Bolt shear stress	bolt grade = A325-N		F <sub>nv</sub> = 54.0	) [ksi]	AISC 15 <sup>th</sup> Table J	13.2
	bolt dia $d_b = 0.750$	[in]	bolt area $A_b = 0.44$	42 [in <sup>2</sup> ]		
Single bolt shear strength	$R_{n-bolt} = F_{nv}A_b$		= 23.9	9 [kips]	AISC 15 <sup>th</sup> Eq J3-	1
Bolt Bearing/TearOut Stren	gth on Plate					
Bolt hole diameter	bolt dia d <sub>b</sub> = $\frac{3}{4}$	[in]	bolt hole dia d <sub>h</sub> = $\frac{13}{1}$	<sub>6</sub> [in]	AISC 15 <sup>th</sup> Table J	13.3
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	$L_c = L_s - d_h$		= 2.68	38 [in]		
Bolt tear out/bearing strength	$R_{n-t\&b-in} = 1.5 L_{c}t$	F <sub>u</sub> ≤ 3.0	d <sub>b</sub> tmF <sub>u</sub>		AISC 15 <sup>th</sup> Eq J3-	6b
	= 139.1 ≤	77.7	= 77.7	7 [kips]		
Bolt strength at interior	$R_{n-in} = min (R_{n-in})$	<sub>n-t&amp;b-in</sub> , R	(n-bolt) = 23.9	9 [kips]		
Number of bolt	interior n <sub>in</sub> = 8					
Bolt bearing strength for all bo	ts $R_n = n_{in} R_{n-in}$		= 190	.9 [kips]		
Required shear strength	V <sub>u</sub> =		= 46.	3 [kips]		
Bolt resistance factor-LRFD	φ = 0.75				AISC 15 <sup>th</sup> J3.10	
	φ R <sub>n</sub> =		= 143	<b>3.1</b> [kips]		
	ratio = <b>0.32</b>		> V <sub>u</sub>	ОК		

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Vertical Brace Connection

VB-Right

Polt Toncilo Drving Action on F	nd Plata		ratio = 0.8 /	22.2	- 0.42	DASS	
Bolt group forces	shoar $V = 46.3$	[kinc]		23.2 2 78 2	= <b>0.42</b>	PASS	
		[kib2]		70.2	[kib2]		
Single Bolt Tensile Capacity With	out Considering Pryi	ng					
Bolt grade A325-N	dia d <sub>b</sub> = 0.750	[in]	area A	<sub>b</sub> = 0.442	[in <sup>2</sup> ]		
Nominal tensile/shear stress	F <sub>nt</sub> = 90.0	[ksi]	Fn	v = 54.0	[ksi]	AISC 15 <sup>th</sup>	Table J3.2
Bolt group shear force	shear V = 46.3	[kips]	no of bolt r	า = 8			
Shear stress required	f <sub>rv</sub> = V / ( n /	۹ <sub>b</sub> )		= 13.1	[ksi]		
Resistance factor-LRFD	φ = 0.75					AISC 15 <sup>th</sup>	J3.7
Modified nominal tensile stress	$F'_{nt} = 1.3 F_{nt}$	− F <sub>nt</sub> φF <sub>nv</sub> f <sub>rv</sub> ≤	F <sub>nt</sub>	= 87.9	[ksi]	AISC 15 <sup>th</sup>	Eq J3-3a
Bolt norminal tensile strength	$r_n = F'_{nt}A_b$			= 38.8	[kips]	AISC 15 <sup>th</sup>	Eq J3-1
Resistance factor-LRFD	φ = 0.75					AISC 15 <sup>th</sup>	J3.6
Single bolt tensile capacity	φ r <sub>n</sub> =			= 29.1	[kips]		
Single Bolt Tensile Capacity After	Considering Prying						
End plate	width w = 6.250	[in]	bolt gage o	g = 3.500	[in]		
	web t <sub>w</sub> = 0.244	[in]		-			
Dist from bolt center to plate edge	a = 0.5 (w -	- g)		= 1.375	[in]	th	
	a' = a + 0.5	d <sub>b</sub> ≤ (1.25 b	+ 0.5 d <sub>b</sub> )	= 1.750	[in]	AISC 15"	Eq 9-23
Bolt hole diameter	bolt dia d $_{b}$ = 0.750	[in]	bolt hole dia d	<sub>h</sub> = 0.813	[in]	AISC 15 <sup>th</sup>	B4.3b
Dist from bolt center to face of web	b = 0.5(g -	t <sub>w</sub> )		= 1.628	[in]		
	b' = b - 0.5	d <sub>b</sub>		= 1.253	[in]	AISC 15 <sup>th</sup>	Eq 9-18
Bolt pitch spacing		[in]					
Bolt tributary length	$\mathbf{p} = \mathbf{s}_{y} + \mathbf{p}_{z}$	$\leq 2b$ and $p \leq$	S.,	= 3.256	[in]	AISC 15 <sup>th</sup>	Page 9-12
			- V				
	ρ = b' / a'			= 0.716		AISC 15 <sup>th</sup>	Eq 9-22
To all a second the second all had second	$\delta = 1 - d_h/$	р		= 0.750		AISC 15 <sup>th</sup>	Eq 9-20
considering prying	B = from ca	lc shown in a	bove section	= 29.1	[kips]		
Resistance factor-LRFD	φ = 0.90					AISC 15 <sup>th</sup>	Page 9-12
End plate thickness	t = 0.625	[in]	tensile F	<sub>u</sub> = 65.0	[ksi]		
Plate thickness req'd to develop bolt tensile capacity without prying	$t_c = \left(\frac{4 B b}{\phi p F}\right)$	) <sup>0.5</sup>		= 0.875	[in]	AISC 15 <sup>th</sup>	Eq 9-26a
	$\alpha' = \frac{1}{\delta (1 + 1)^2}$	$\frac{t_c}{\rho}$ [ $(\frac{t_c}{t})^2$	-1]	= 0.747		AISC 15 <sup>th</sup>	Eq 9-28
when $0 \le \alpha' \le 1$	$Q = \left(\frac{t}{t_c}\right)^2$	<sup>2</sup> (1 + δ α' )		= 0.795		AISC 15 <sup>th</sup>	Eq 9-27
Bolt tensile force per bolt in demand	T = from ca	lc shown belo	w	= 9.8	[kips]		
Tensile strength per bolt after considering prying	$\phi r_n = B \times Q$			= 23.2	[kips]	AISC 15 <sup>th</sup>	Eq 9-27
	ratio = <b>0.42</b>			> T	OK		
Calculate Max Single Bolt Tensile	Load						
Bolt group force	axial P = 78.2	[kips]					
Bolt number	Bolt Row $n_{\rm b} = 2$		Bolt Col n	<sub>v</sub> = 4			
Bolt tensile force per bolt	 T = P/(n <sub>v</sub>	n <sub>h</sub> )		= <b>9.8</b>	[kips]		
Polt Tongila During Artists				10.0	- 0.50	DACO	
Bolt aroun forces	short V = 46.2	[kinc]		10.0	= <b>U.52</b>	PASS	
Doit group roites	siical v - 40.3	[vih2]	axidi h	/0.2	[vih2]		

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Vertical Brace Connection

Single Bolt Tensile Capacity With	out Considering Prying	9			
Bolt grade A325-N	dia d <sub>b</sub> = 0.750 [	[in]	area A <sub>b</sub> = 0.442	[in <sup>2</sup> ]	
Nominal tensile/shear stress	F <sub>nt</sub> = 90.0 [	[ksi]	$F_{nv} = 54.0$	[ksi]	AISC 15 <sup>th</sup> 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 15 <sup>th</sup> J3.7
Modified nominal tensile stress	F' <sub>nt</sub> = 1.3 F <sub>nt</sub>	$\frac{F_{nt}}{\phi F_{nv}} f_{rv} \le F_{nt}$	= 87.9	[ksi]	AISC 15 <sup>th</sup> Eq J3-3a
Bolt norminal tensile strength	$r_n = F'_{nt} A_b$		= 38.8	[kips]	AISC 15 <sup>th</sup> Eq J3-1
Resistance factor-LRFD	φ = 0.75				AISC 15 <sup>th</sup> J3.6
Single bolt tensile capacity	φ r <sub>n</sub> =		= 29.1	[kips]	
Single Bolt Tensile Capacity After	r Considering Prying				
Column flange as tee	b <sub>f</sub> = 7.992 [	[in]	bolt gage g = 3.500	[in]	
	web t <sub>w</sub> = 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 <sub>pl</sub> = 0.5 ( w -	g )	= 1.375	[in]	
Dist from holt center to plate edge		a .)	= 1 375	[in]	
	a' = a + 0.5 d	<sub>b</sub> ≤ (1.25 b + 0.5 d	(10,75) = 1.750	[in]	AISC 15 <sup>th</sup> Eq 9-23
Bolt hole diameter	bolt dia $d_{\rm b} = 0.750$ [	in] bolt	hole dia $d_{\rm b} = 0.813$	[in]	AISC 15 <sup>th</sup> B4.3b
Dist from bolt center to face of web	b = 0.5(a - t)	)	= 1 593	[in]	
	$b' = b - 0.5 d_{\rm b}$	, ,	= 1.218	[in]	AISC 15 <sup>th</sup> Eq 9-18
Bolt pitch spacing	 s, = 3.500 [	, in]			
Bolt tributary length	p = s, p ≤	2b and p ≤ s,	= 3.185	[in]	AISC 15 <sup>th</sup> Page 9-12
		· v			
	$\rho = b' / a'$		= 0.696		AISC $15^{\text{th}}$ Eq 9-22
Tensile capacity per bolt before	$o = 1 - u_h / p$		= 0.745	fldere 1	AISC 15 Eq 9-20
considering prying	B = from calc	shown in above se	= 29.1	[kips]	*10
Resistance factor-LRFD	φ = 0.90				AISC 15 <sup>th</sup> 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	$t_c = \left(\frac{4 B D}{\phi p F_u}\right)$	) <sup>0.5</sup>	= 0.872	[in]	AISC 15 <sup>th</sup> Eq 9-26a
	$\alpha' = \frac{1}{\delta (1 + \rho)}$	$\frac{1}{t} \left[ \left( \frac{t_c}{t} \right)^2 - 1 \right]$	= 1.345		AISC 15 <sup>th</sup> Eq 9-28
when $\alpha' > 1$	$Q = \left(\frac{t}{t_c}\right)^2 \left(\frac{t}{t_c}\right)^2$	1 + δ)	= 0.646		AISC 15 <sup>th</sup> Eq 9-27
Bolt tensile force per bolt in demand	T = from calc	shown below	= 9.8	[kips]	
Tensile strength per bolt after	$\phi r_n = B \times Q$		= 18.8	[kips]	AISC 15 <sup>th</sup> Eq 9-27
e	ratio = <b>0.52</b>		> T	OK	
Calculate Max Single Bolt Tensile	Load				
Bolt group force	axial P = 78.2 [	[kips]			
Bolt number	Bolt Row $n_{1} = 2$		Bolt Col n = 4		
Bolt tensile force per bolt	T = P / (n n)	.)	= 9.8	[kins]	
	/ (		510	[bo]	

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Beam Flange Weld Strength			ratio = 6.8	35 / 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 W200v36	d – 7.913	[in]		k - 0.906	[in]		
Beam Section w200x50	$b_{fb} = 6.496$	[in]		$k_{b} = 0.551$	[in]		
		[]					
Ver weld length on beam web	$L_{w1} = d_{b} - 2 k$	b		= 6.101	[IN]		
Bolt pitch & edge distance	d <sub>1</sub> = 2.500	[in]		e <sub>v</sub> =1.375	[in]		
Ver weld length outside flange-weld line on stiff plate	$L_{e} = 2(d_{1} +$	e <sub>v</sub> ) - 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 <sub>f</sub> = (2b <sub>fb</sub> - 2	2k <sub>1b</sub> )/2 as c	louble fillet	= 5.945	[in]		
Fillet weld length - total	$L = L_w + 2$	L <sub>f</sub>		= 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	e flange dbl fi	llet 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	$f_{max} = (f_a^2 + f_a)$	f <sup>2</sup> <sub>v</sub> ) <sup>0.5</sup>		= 6.849	[kip/in]	AISC 15 <sup>th</sup> Eq 8-11	
Weld stress load angle	$\theta = \tan^{-1}($	f <sub>a</sub> f <sub>v</sub> )		= 73.8	[°]		
Fillet Weld Strength Calc							
Fillet weld leg size	$w = \frac{1}{4}$	[in]	load ang	le θ = 73.8	[°]		
Electrode strength	F <sub>EXX</sub> = 70.0	[ksi]	strength coeff	<sup>c</sup> C <sub>1</sub> = 1.00		AISC 15 <sup>th</sup> Table 8-3	
Number of weld line	n = 2 for c	louble fillet					
Load angle coefficient	$C_2 = (1 + 0.)$	.5 sin <sup>1.5</sup> θ)		= 1.47		AISC 15 <sup>th</sup> Page 8-9	
Fillet weld shear strength	$R_{n-w} = 0.6 (C_1)$	x 70 ksi) 0.70	07 w n C <sub>2</sub>	= 21.83	[kip/in]	AISC 15 <sup>th</sup> Eq 8-1	
Base metal - beam web th	ickness t = 0.402	[in]	tensile	e F <sub>u</sub> = 65.0	[ksi]		
Base metal - beam web is in shear, shear rupture as per AISC 15 <sup>th</sup> Eq J4-4 is checked						AISC 15 <sup>th</sup> J2.4	
Base metal shear rupture	$R_{n-b} = 0.6 F_{u}t$			= 15.68	[kip/in]	AISC 15 <sup>th</sup> Eq J4-4	
Double fillet linear shear strength	R <sub>n</sub> = min ( R	<sub>n-w</sub> , R <sub>n-b</sub> )		= 15.678	[kip/in]	AISC 15 <sup>th</sup> Eq 9-2	
Resistance factor-LRFD	φ = 0.75					AISC 15 <sup>th</sup> Eq 8-1	
	$\phi R_n =$			= 11.759	[kip/in]		
	ratio = <b>0.58</b>			> f <sub>max</sub>	ОК		

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Beam Web Weld Strength			ratio = 23.6	5 / 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 <sub>b</sub> = 7.913	[in]	ł	k <sub>b</sub> = 0.906	[in]		
	b <sub>fb</sub> = 6.496	[in]	k	<sub>1b</sub> = 0.551	[in]		
Ver weld length on beam web	L <sub>w1</sub> = d <sub>b</sub> - 2 k	ь		= 6.101	[in]		
Bolt pitch & edge distance	d <sub>1</sub> = 2.500	[in]		e <sub>v</sub> = 1.375	[in]		
Ver weld length outside flange-weld line on stiff plate	$L_{e} = 2(d_{1} +$	e <sub>v</sub> ) - 2x0.75 i	n clip	= 6.250	[in]		
Ver weld length - total	$L_w = L_{w1} + L$	e		= 12.351	[in]		
Fillet weld length on beam flange	$L_{f} = (2b_{fb} - 2)$	2k <sub>1b</sub> )/2 as de	ouble fillet	= 5.945	[in]		
Fillet weld length - total	$L = L_w + 2$	L <sub>f</sub>		= 24.241	[in]		
Shear force taken by weld at beam web	$V_u = \frac{L_w}{L} \times V$			= 23.6	[kips]		
Fillet Weld Strength Check							
Fillet weld leg size	$w = \frac{1}{4}$	[in]	load angle	$\theta = 0.0$	[°]		
Electrode strength	F <sub>EXX</sub> = 70.0	[ksi]	strength coeff (	$C_1 = 1.00$		AISC 15 <sup>th</sup> Table 8-3	
Number of weld line	n = 2 for c	double fillet					
Load angle coefficient	$C_2 = (1 + 0)$	.5 sin <sup>1.5</sup> θ)		= 1.00		AISC 15 <sup>th</sup> Page 8-9	
Fillet weld shear strength	$R_{n-w} = 0.6 (C_1)$	x 70 ksi) 0.70	7 w n C <sub>2</sub>	= 14.85	[kip/in]	AISC 15 <sup>th</sup> Eq 8-1	
Base metal - beam webthickness t = 0.244 [in]tensile $F_u = 65.0$ [ksi]							
Base metal - beam web is in shear, <u>she</u>	<u>ar</u> rupture as per AI	SC 15 <sup>th</sup> Eq J4	-4 is checked			AISC 15 <sup>th</sup> J2.4	
Base metal shear rupture	$R_{n-b} = 0.6 F_{u}t$	:		= 9.52	[kip/in]	AISC 15 <sup>th</sup> Eq J4-4	
Double fillet linear shear strength	R <sub>n</sub> = min ( R	<sub>n-w</sub> , R <sub>n-b</sub> )		= 9.516	[kip/in]	AISC 15 <sup>th</sup> Eq 9-2	
Resistance factor-LRFD	φ = 0.75					AISC 15 <sup>th</sup> Eq 8-1	
	φ R <sub>n</sub> =			= 7.137	[kip/in]		
Weld resistance required	V <sub>u</sub> =			= 23.6	[kips]		
Fillet weld length - double fillet	L =			= 12.351	[in]		
Weld resistance provided	$\phi F_n = \phi R_n x I$	L		= 88.1	[kips]		
	ratio = <b>0.27</b>			> V <sub>u</sub>	OK		

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Vertical Brace Connection

Column Web Local Yielding			ratio = 78.2 / 296.4	= 0.26	PASS
Concentrated force on column	P <sub>u</sub> =		= 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	$I_{b}$ = 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 15 <sup>th</sup> Eq J10-2
Resistance factor-LRFD	$\varphi = 1.00$				
	φ R <sub>n</sub> =		= 296.4	[kips]	
	ratio = <b>0.26</b>		> P <sub>u</sub>	OK	
Column Flange Local Bending			ratio = 78.2 / 79.3	= 0.99	PASS
Concentrated force from gusset	P <sub>u</sub> =		= 78.2	[kips]	
Column w section	$t_{f} = 0.531$	[in]	yield $F_y = 50.0$	[ksi]	
Column flange local bending strength	$R_{n} = 6.25 F_{y} t$	-2 f	= 88.1	[kips]	AISC 15 <sup>th</sup> Eq J10-1
Resistance factor-LRFD	φ = 0.90				AISC 15 <sup>th</sup> J10.1
	φ R <sub>n</sub> =		= 79.3	[kips]	
	ratio = <b>0.99</b>		> P <sub>u</sub>	OK	