Night School 28: Vertical Bracing Connections

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AISC Night School

Vertical Bracing Connections, Session 4: Vertical Bracing Corner Connection – Wind and Low-seismic

April 26, 2022 | William A Thornton

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AISC Night School

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Course Description

Vertical Bracing Connections

Vertical Bracing Corner Connection – Wind and Low-seismic
April 26, 2022

The various steps and methodologies used for bracing connection design will be highlighted as the presenter works through a design example. Practical design tips will be discussed as well as addressing common design challenges.

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Learning Objectives

1. Determine forces in a corner connection using the Uniform Force Method.
2. List the limit states that must be reviewed for the design of bracing connections.
3. Describe the bracing connection design process through the presentation of a design example.
4. Determine gusset plate dimensions based on the type of corner connection.

Night School 28: Vertical Bracing Connections

Session 4: **Vertical Bracing Corner Connection – Wind and Low-seismic**

April 26, 2022

William A. Thornton, corporate consultant to Cives Steel

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Bracing Connections and Related Topics

By: William Thornton

Course Outline

1. Basic Principles
2. Uniform Force Method Part 1
3. Bracing Connection Details and Prying Action
4. **Vertical Bracing Corner Connection – Wind and Low-seismic**
5. Uniform Force Method Part 2
6. Vertical Bracing Corner Connection – Seismic
7. Chevron Gussets Connection
8. Other Connection Topics and Case Study

Session Outline

- Example of VB Corner Connection in Wind and Low-Seismic Systems
  - Begin with the brace-to-gusset design
  - Determine gusset dimensions based on brace-to-gusset connection
  - Check gusset-to-column connection
  - Check gusset-to-beam connection
  - Check beam-to-column connection

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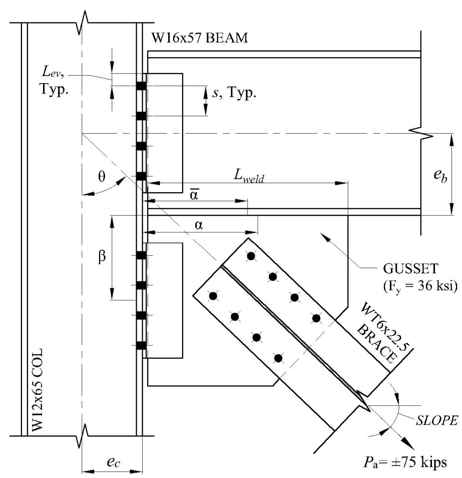
Example of VB Corner Connection (Part I) in Wind and Low-Seismic



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Example of VB Corner Connection (Part I) in Wind and Low-Seismic

Given:

1. AISC 15th Edition, ASD
2. Beam-to-Column and Gusset-to-Column: n = 4
3. L_{weld} = 20 in. min. (gusset to beam) with ¼” min. weld
4. ¾” dia. ASTM F3125 Gr. A325-N, STD holes, UNO
5. Double L4x4x3/8” Gr. 36 angles, horizontal SSLT with L_{ev} = 1.25 in. minimum and bolt spacing, s = 3 in.
6. Brace force, P_a = ±75 kips with slope = 43.6°, θ = 46.4°
7. 3/8” Gusset PL (A36), F_y = 36 ksi
8. WT and W Shapes ASTM A992, F_y = 50 ksi

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Example VB Corner Connection, I Brace-to-Gusset (cont.)

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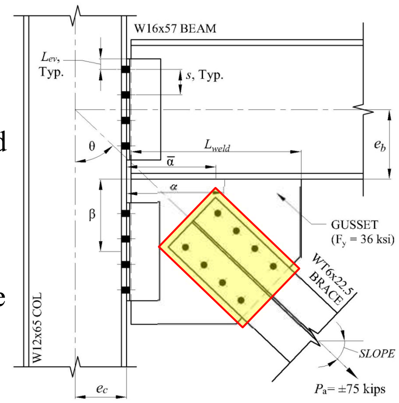
Example VB Corner Connection, I Brace-to-Gusset (cont.)

The example will begin with designing the brace-to-gusset connection. This will help set the geometry of the gusset.

a. Determine the number of bolts required to attach the brace to the gusset
b. Check gross and net tension and block shear for the brace
c. Check bolt bearing and tearout of brace and gusset
d. Check block shear and Whitmore tension and buckling for the gusset



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Example VB Corner Connection, I Brace-to-Gusset (cont.)

Brace-to-Gusset – Bolt Shear:

AISC *Manual* Table 7-1

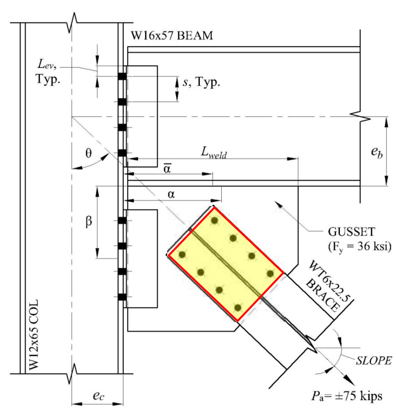
$R_b/\Omega = nr_v/\Omega$ (single shear)

$= 8 \times 11.9$ kips/bolt

$= 95.2 \text{ kips} \geq 75$ kips **o.k.**

Table 7-1
Available Shear
Strength of Bolts, kips

Nominal Bolt Diameter, d, in.					5/8		3/4		7/8		1	
Nominal Bolt Area, in.²					0.307		0.442		0.601		0.785	
ASTM Desig.	Thread Cond.	F_{nv}/Ω (ksi) ASD	ϕF_{nv} (ksi) LRFD	Load-ing	r_n/Ω ASD	ϕr_n LRFD	r_n/Ω ASD	ϕr_n LRFD	r_n/Ω ASD	ϕr_n LRFD	r_n/Ω ASD	ϕr_n LRFD
Group A	N	27.0	40.5	S	8.29	12.4	11.9	17.9	16.2	24.3	21.2	31.8
				D	16.6	24.9	23.9	35.8	32.5	48.7	42.4	63.6
	X	34.0	51.0	S	10.4	15.7	15.0	22.5	20.4	30.7	26.7	40.0
				D	20.9	31.3	30.1	45.1	40.9	61.3	53.4	80.1



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Example VB Corner Connection, I Brace-to-Gusset (cont.)

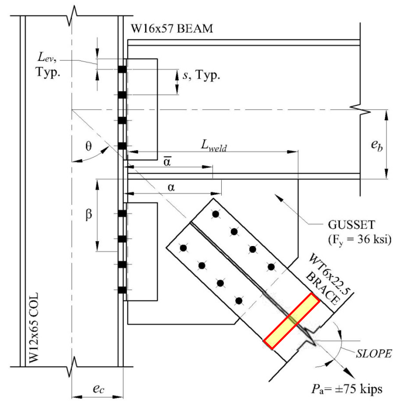
Brace-to-Gusset – WT Gross Tension:

AISC *Specification* Eq. J4-1

$$\frac{R_n}{\Omega} = \frac{F_y A_g}{\Omega}$$

$$= \frac{(50\text{ ksi})(6.56\text{ in.}^2)}{1.67}$$

$$= 196\text{ kips} > 75\text{ kips}$$ **o.k.**



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Net Tension

- The determination of the net section strength must account for
  - the area reduction due to the holes
  - shear lag effects
- The width of the holes is (d_b + 1/16 in.) for standard holes and bolts less than 1.0 in. in diameter. For bolts ≥ 1.0 in dia., width of hole is (d_b + 1/8 in.) – Spec. section J3.2
- The shear lag factor, *U*, is computed using Table D3.1.

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TABLE D3.1
Shear Lag Factors for Connections to Tension Members

Case	Description of Element		Shear Lag Factor, U	Example
1	All tension members where the tension load is transmitted directly to each of the cross sectional elements		$U = 1.0$	—
2	All tension members, except HSS, where the tension load is transmitted to some but not all of the cross sectional elements by fasteners or by longitudinal welds in combination with transverse welds. Alternatively, Case 7 is permitted for W, M, S and HP shapes. (For angles, Case 8 is permitted to be used.)		$U = 1 - \frac{\bar{x}}{l}$	
3	All tension members where the tension load is transmitted only by transverse welds to some but not all of the cross sectional elements.		$U = 1.0$ and A_n = area of the directly connected elements	—
4[a]	Plates, angles, channels with welds at heels, toes, and W shapes with connected elements, where the tension load is transmitted by longitudinal welds only. See Case 2 for definition of $\bar{x}$.		$U = \frac{3l^2}{3l^2 + w^2}\left(1 - \frac{\bar{x}}{l}\right)$	Plate or connected element
5	Round HSS with a single concentric gusset plate through slots in the HSS.		$l \geq 1.3D$, $U = 1.0$ $D \leq l < 1.3D$, $U = 1 - \frac{\bar{x}}{l}$ $\bar{x} = \frac{D}{\pi}$	
6	Rectangular HSS.	with a single concentric gusset plate	$l \geq H$, $U = 1 - \frac{\bar{x}}{l}$ $\bar{x} = \frac{B^2 + 2BH}{4(B+H)}$	
		with two side gusset plates	$l \geq H$, $U = 1 - \frac{\bar{x}}{l}$ $\bar{x} = \frac{B^2}{4(B+H)}$	
7	W, M, S or HP shapes, or tees cut from these shapes. (If U is calculated per Case 2, the larger value is permitted to be used.)	with flange connected with three or more fasteners per line in the direction of loading	$b_f \geq \frac{2}{3}d$, $U = 0.90$ $b_f < \frac{2}{3}d$, $U = 0.85$	—
		with web connected with four or more fasteners per line in the direction of loading	$U = 0.70$	—
8	Single and double angles. (If U is calculated per Case 2, the larger value is permitted to be used.)	with four or more fasteners per line in the direction of loading	$U = 0.80$	—
		with three fasteners per line in the direction of loading (with fewer than three fasteners per line in the direction of loading, use Case 2)	$U = 0.60$	—

B = overall width of rectangular HSS member, measured 90° to the plane of the connection, in. (mm); D = outside diameter of round HSS, in. (mm); H = overall height of rectangular HSS member, measured in the plane of the connection, in. (mm); d = depth of section, in. (mm); for tees, d = depth of the section from which the tee was cut, in. (mm); l = length of connection, in. (mm); w = width of plate, in. (mm); $\bar{x}$ = eccentricity of connection, in. (mm).

[a] $l = \frac{l_1 + l_2}{2}$, where l_1 and l_2 shall not be less than 4 times the weld size.

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TABLE D3.1
Shear Lag Factors for Connections to Tension Members

$$U = 1 - \bar{x}/l$$

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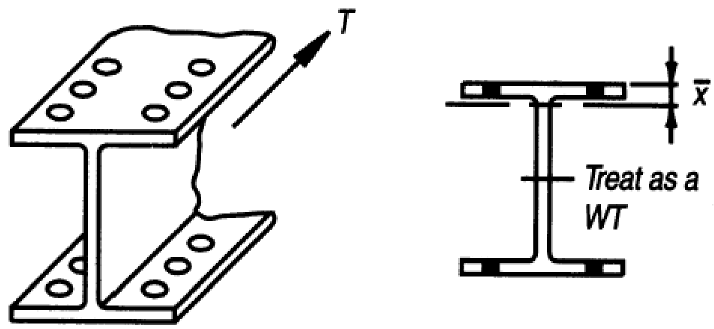
Net Tension

Determination of $\overline{x}$



Example VB Corner Connection, I Brace-to-Gusset (cont.)

Brace-to-Gusset – WT Net Tension:

AISC *Specification* Eq. J4-2

$\overline{x} = 1.13$ in.

$l = s(n - 1) = (3\text{ in.})(4 - 1) = 9$ in.

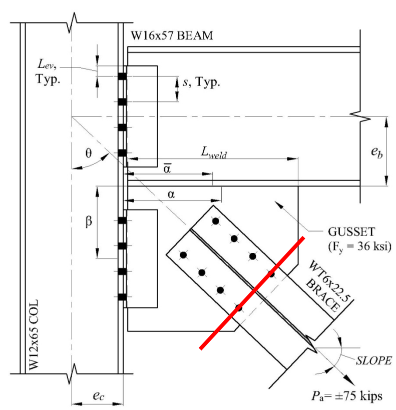
$U = 1 - \overline{x}/l = 1 - (1.13\text{ in.}/9\text{ in.}) = 0.874$

To simplify calculations:

For ¾ in. dia. bolts with STD holes,

$d_h = {}^{13}/_{16}$ in. and

$(d_h + {}^{1}/_{16}\text{ in.}) = ({}^{13}/_{16}\text{ in.} + {}^{1}/_{16}\text{ in.}) = ⅞$ in.

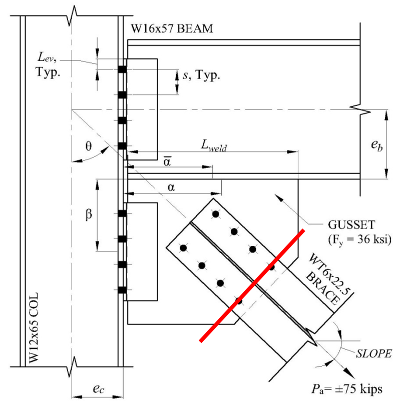


Example VB Corner Connection, I Brace-to-Gusset (cont.)

Brace-to-Gusset – WT Net Tension (cont.):

$$A_e = U[A_g - t_f(\text{\# lines})(d_h + {}^{1}/_{16}\text{ in.})]$$
$$= 0.874\,[6.56\text{ in.}^2 - (0.575\text{ in.})(2)(7/8\text{ in.})]$$
$$= 4.85\text{ in.}^2$$

$$\frac{R_n}{\Omega} = \frac{F_u A_e}{\Omega}$$
$$= \frac{(65\text{ ksi})(4.85\text{ in.}^2)}{2}$$
$$= 158\text{ kips} > 75\text{ kips } \textbf{o.k.}$$



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Block Shear

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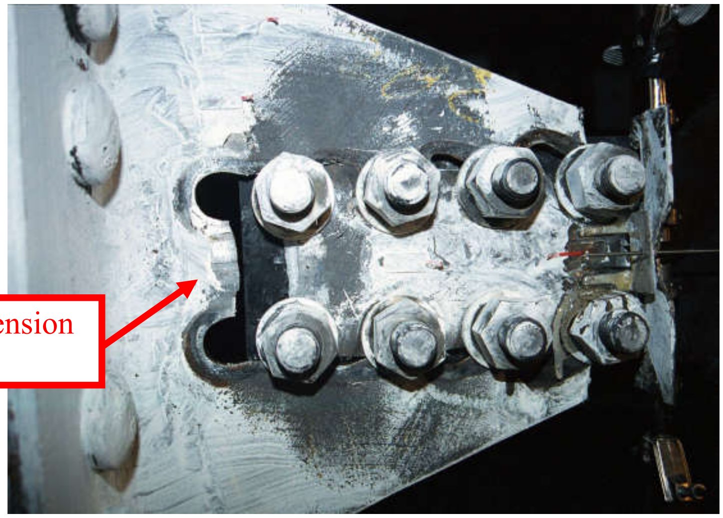
Block Shear



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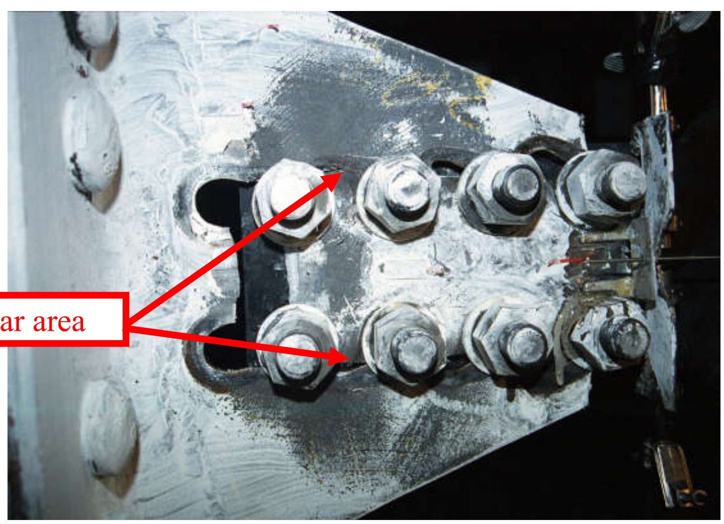
Block Shear



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Example VB Corner Connection, I Brace-to-Gusset (cont.)

Brace-to-Gusset – Block Shear WT:

AISC *Spec* Section J4.3 *Block Shear Strength*

$\Omega = 2.00$

$R_n = 0.6F_uA_{nv} + U_{bs}F_uA_{nt}$

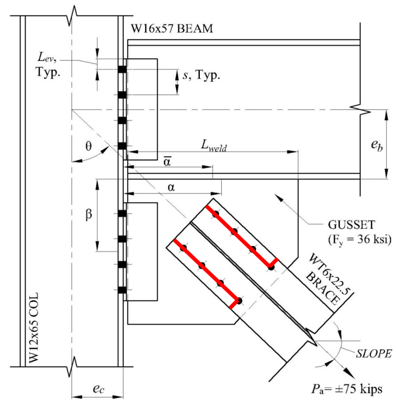
$\leq 0.6F_yA_{gv} + U_{bs}F_uA_{nt}$

Shear Rupture $= 0.6F_uA_{nv}$

Shear Yield $= 0.6F_yA_{gv}$

Tension Rupture $= F_uA_{nt}$

U_{bs} = 1.0 for Direct Loaded Connections



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Example VB Corner Connection, I Brace-to-Gusset (cont.)

Brace-to-Gusset – Block Shear WT (cont.):

$L_{gv} = L_{ev} + s(n - 1)$

$= 1.5 \text{ in.} + (3 \text{ in.})(4 - 1) = 10.5 \text{ in.}$

$L_{nv} = L_{ev} + s(n - 1) - (n - 0.5)(d_h + {}^1/_{16} \text{ in.})$

$= 1.5 \text{ in.} + (3 \text{ in.})(4 - 1)$

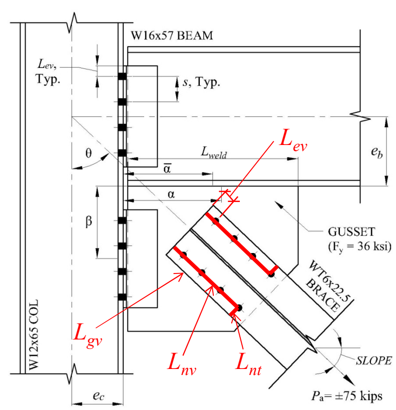
$- (4 - 0.5)(7/8 \text{ in.})$

$= 7.44 \text{ in.}$

$L_{nt} = (b_f - \text{gage})/2 - (d_h + {}^1/_{16} \text{ in.})/2$

$= (8.05 \text{ in.} - 5.5 \text{ in.})/2 - (7/8 \text{ in.})/2$

$= 0.838 \text{ in.}$



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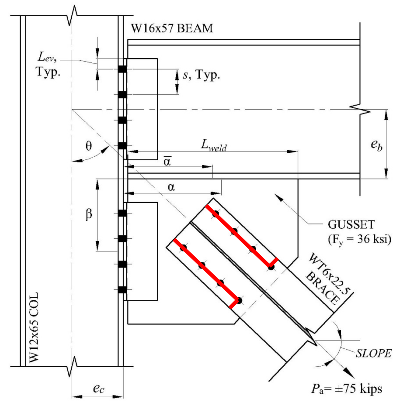
Example VB Corner Connection, I Brace-to-Gusset (cont.)

Brace-to-Gusset – Block Shear WT (cont.):

$$A_{gv} = 2L_{gv}t_f = 2(10.5\text{ in.})(0.575\text{ in.}) = 12.08\text{ in.}^2$$

$$A_{nv} = 2L_{nv}t_f = 2(7.44\text{ in.})(0.575\text{ in.}) = 8.56\text{ in.}^2$$

$$A_{nt} = 2L_{nt}t_f = 2(0.838\text{ in.})(0.575\text{ in.}) = 0.96\text{ in.}^2$$



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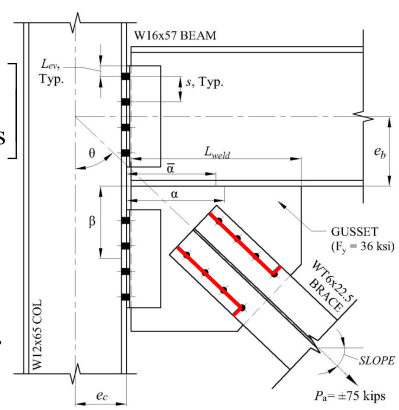
Example VB Corner Connection, I Brace-to-Gusset (cont.)

Brace-to-Gusset – Block Shear WT (cont.):

$$R_n = \min\begin{bmatrix} 0.6(65\text{ ksi})(8.56\text{ in.}^2) = 333.84\text{ kips} \\ 0.6(50\text{ ksi})(12.08\text{ in.}^2) = 362.40\text{ kips} \end{bmatrix} + [1(65\text{ ksi})(0.96\text{ in.}^2)]$$

$$= 333.84\text{ kips} + 62.4\text{ kips} = 396.24\text{ kips}$$

$R_n/\Omega = (396.24\text{ kips})/2 = 198\text{ kips} > 75\text{ kips}$ **o.k.**



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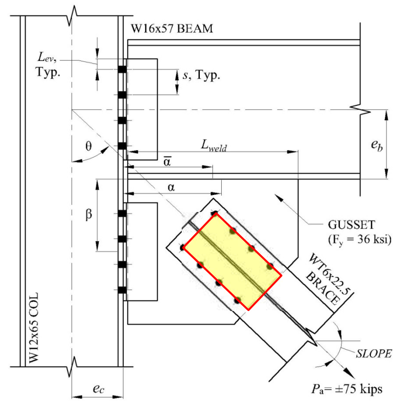
Example VB Corner Connection, I Brace-to-Gusset (cont.)

Brace-to-Gusset – Block Shear Gusset:

These calculations are similar to the block shear check for the WT that was previously shown.

The actual calculations will be contained in Appendix 1 which is available with the handouts for this session.



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Bolt Tearout

Bolt Bearing

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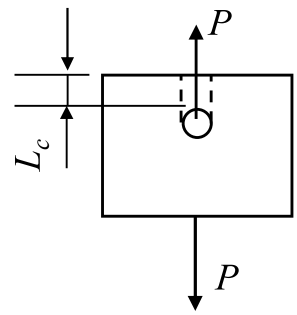
Bolt Bearing and Tearout

Bolt Tearout is easier to understand if you think about a bolt tearing through the material as shown.

There are 2 shear planes; the strength is therefore:

$(2)(0.6)(F_u)(L_c)(t)$

or, $\mathbf{1.2(F_u)(L_c)(t)}$



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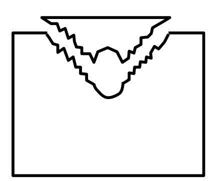
Bolt Bearing and Tearout

In reality, the material will fail as shown, but test results support the equation in the *Specification*.



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Bolt Bearing and Tearout

- For each bolt there are five (5) possible limit states or failure modes that need to be checked
  - Bolt Shear
  - Bolt Bearing on the Main Material
  - Bolt Bearing on the Connection Material
  - Bolt Tearout on the Main Material
  - Bolt Tearout on the Connection Material
- Then the sum of the bolt capacities can be added.

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Example VB Corner Connection, I Brace-to-Gusset (cont.)

Brace-to-Gusset – Bolt Bearing/Tearout:

$R_n = 2.4dtF_u$ AISC *Specification* Eq. J3-6a

$R_n = 1.2l_c tF_u$ AISC *Specification* Eq. J3-6c

AISC Bolt Shear:

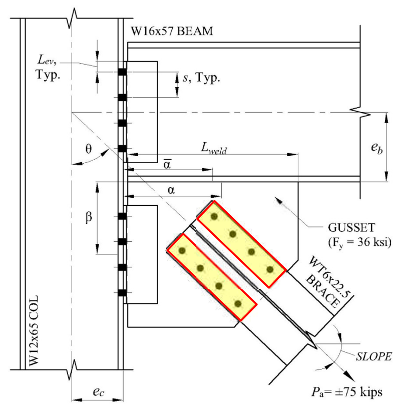
$r_v/\Omega = 11.9$ kips

WT Bolt Bearing:

$R_{brg(WT)}/\Omega = 2.4d_b t_f F_u/\Omega$

$= 2.4(0.75 \text{ in.})(0.575 \text{ in.})(65 \text{ ksi})/2$

$= 33.6$ kips



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Example VB Corner Connection, I Brace-to-Gusset (cont.)

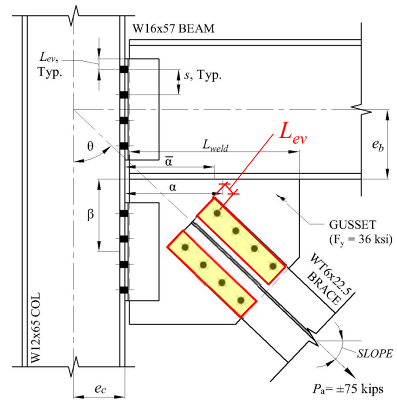
Brace-to-Gusset – Bolt Bearing (cont.):

WT Bolt Tear Out (Edge):

$$R_{tear(WT\_E)}/\Omega = 1.2[L_{ev} - (d_{h\_w}/2)]t_f F_u/\Omega$$
$$= 1.2[1.5 \text{ in.} - (^{13}/_{16} \text{ in.})/2] \times (0.575 \text{ in.})(65 \text{ ksi})/2$$
$$= 24.5 \text{ kips}$$

WT Bolt Tear Out (Center):

$$R_{tear(WT\_C)}/\Omega = 1.2(s - d_{h\_w})t_f F_u/\Omega$$
$$= 1.2(3 \text{ in.} - ^{13}/_{16} \text{ in.})(0.575 \text{ in.}) \times (65 \text{ ksi})/2$$
$$= 49.1 \text{ kips}$$



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Example VB Corner Connection, I Brace-to-Gusset (cont.)

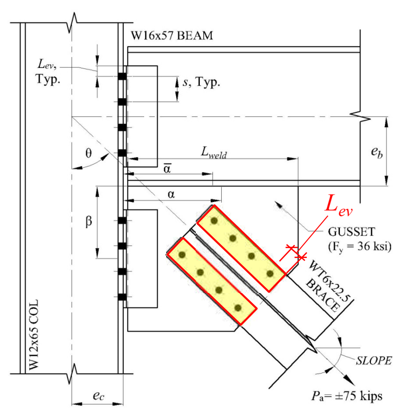
Brace-to-Gusset – Bolt Bearing (cont.):

Plate Bolt Bearing:

$$R_{brg(PL)}/\Omega = 2.4 d_b t_p F_u/\Omega$$
$$= 2.4(0.75 \text{ in.})(0.375 \text{ in.})(58 \text{ ksi})/2$$
$$= 19.6 \text{ kips}$$

Plate Bolt Tear Out (Edge):

$$R_{tear(PL\_E)}/\Omega = 1.2[L_{ev} - (d_{h\_w}/2)]t_p F_u/\Omega$$
$$= 1.2[1.5 \text{ in.} - (^{13}/_{16} \text{ in.})/2] \times (0.375 \text{ in.})(58 \text{ ksi})/2$$
$$= 14.3 \text{ kips}$$



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Example VB Corner Connection, I Brace-to-Gusset (cont.)

Brace-to-Gusset – Bolt Bearing (cont.):

Plate Bolt Tear Out (Center):

$R_{tear(\mathrm{PL\_C})}/\Omega = 1.2(s - d_{h\_w})t_pF_u/\Omega$

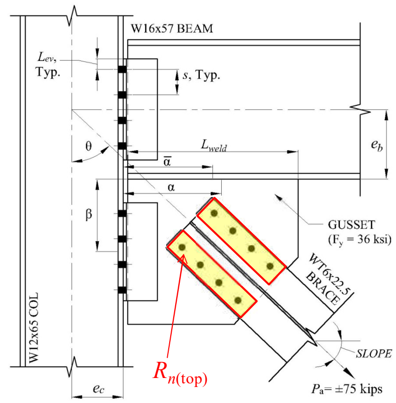
$= 1.2(3\text{ in.} - {}^{13}/_{16}\text{ in.})(0.375\text{ in.})\times(58\text{ ksi})/2$

$= 28.6\text{ kips}$

$R_{n(\mathrm{top})}/\Omega = \min\{r_v/\Omega, R_{brg(\mathrm{WT})}/\Omega, R_{tear(\mathrm{WT\_E})}/\Omega, R_{brg(\mathrm{PL})}/\Omega, R_{tear(\mathrm{PL\_C})}/\Omega)$

$= \min\{11.9\text{ kips}, 33.6\text{ kips}, 24.5\text{ kips}, 19.6\text{ kips}, 28.6\text{ kips}\}$

$= 11.9\text{ kips}$



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Example VB Corner Connection, I Brace-to-Gusset (cont.)

Brace-to-Gusset – Bolt Bearing (cont.):

$R_{n(\mathrm{center})}/\Omega = \min\{r_v/\Omega, R_{brg(\mathrm{WT})}/\Omega, R_{tear(\mathrm{WT\_C})}/\Omega, R_{brg(\mathrm{PL})}/\Omega, R_{tear(\mathrm{PL\_C})}/\Omega\}$

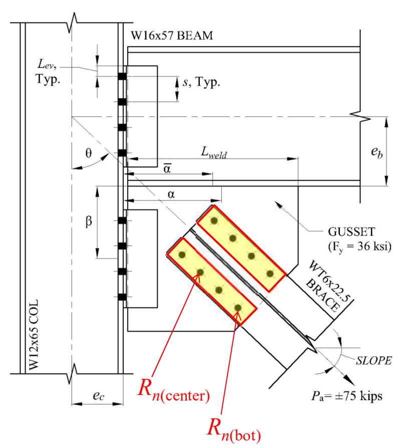
$= \min\{11.9\text{ kips}, 33.6\text{ kips}, 49.1\text{ kips}, 19.6\text{ kips}, 28.6\text{ kips}\}$

$= 11.9\text{ kips}$

$R_{n(\mathrm{bot})}/\Omega = \min\{r_v/\Omega, R_{brg(\mathrm{WT})}/\Omega, R_{tear(\mathrm{WT\_C})}/\Omega, R_{brg(\mathrm{PL})}/\Omega, R_{tear(\mathrm{PL\_E})}/\Omega\}$

$= \min\{11.9\text{ kips}, 33.6\text{ kips}, 49.1\text{ kips}, 19.6\text{ kips}, 14.3\text{ kips}\}$

$= 11.9\text{ kips}$



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Example VB Corner Connection, I Brace-to-Gusset (cont.)

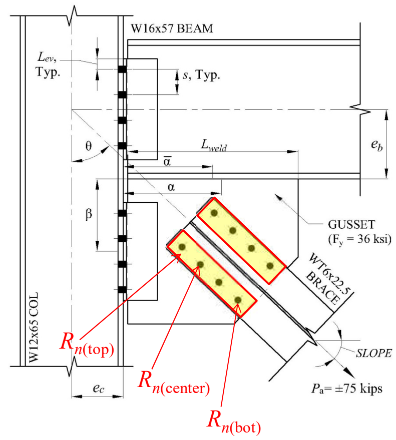
Brace-to-Gusset – Bolt Bearing (cont.):

Total Bolt Strength

$$R_n/\Omega = [R_{n\,(top)}/\Omega + (R_{n(center)}/\Omega)(n-2) + R_{n(bot)}/\Omega](2\text{ lines})$$

$$= [11.9\text{ kips} + (11.9\text{ kips})(4-2) + 11.9\text{ kips}](2)$$

$$= 95.2\text{ kips} > 75\text{ kips } \textbf{o.k.}$$



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Whitmore Section

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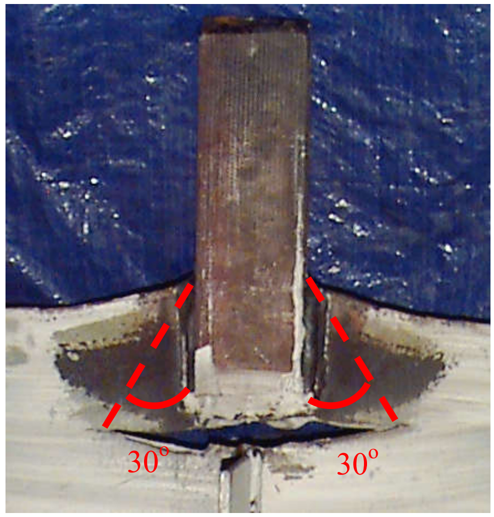
Whitmore Section



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Example VB Corner Connection, I Brace-to-Gusset (cont.)

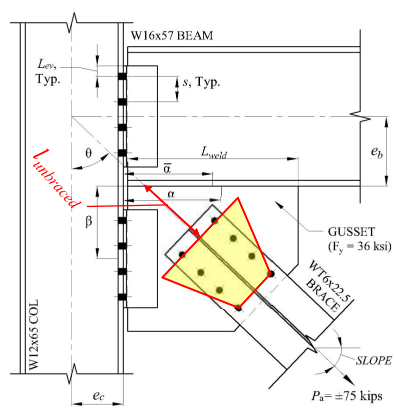
Brace-to-Gusset – Whitmore Section of Gusset Plate:

Whitmore Compression will control over Whitmore Tension by inspection.

$l_{unbraced}$ = 8.81 in. max

$l_w = 2(l_{conn})\tan 30° + gage$
$= 2(9 \text{ in.})\tan 30° + 5.5$ in. min.
$= 15.9$ in.

$l_{w(\text{beam})}$ = length of Whitmore in beam
= 0 in. from geometric layout



Reference AISC *Manual* Part 9 for Whitmore Section

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Example VB Corner Connection, I Brace-to-Gusset (cont.)

Brace-to-Gusset – Whitmore Section of Gusset Plate (cont.):

$l_{w(\text{gusset})} = l_w - l_{w(\text{beam})}$

$= 15.9 \text{ in.} - 0 \text{ in.} = 15.9 \text{ in.}$

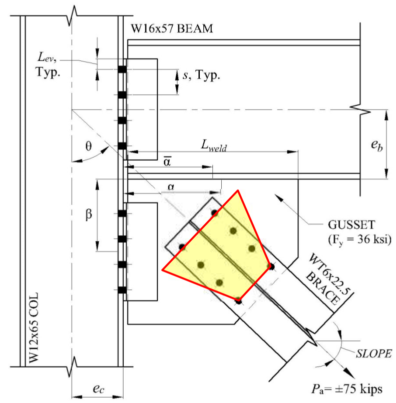
$K = 0.5$ from AISC's *Design Guide 29* - Chapter 5 Design Examples

$Kl_{unbraced}/r = [0.5(8.81 \text{ in.})\sqrt{(12)}]/0.375 \text{ in.}$

$= 40.7$

AISC *Manual* Table 4-14, $F_{cr(\text{Gr.36})}/\Omega = 19.7$ ksi

AISC *Manual* Table 4-14, $F_{cr(\text{Gr.50})}/\Omega = 26.5$ ksi



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Example VB Corner Connection, I Brace-to-Gusset (cont.)

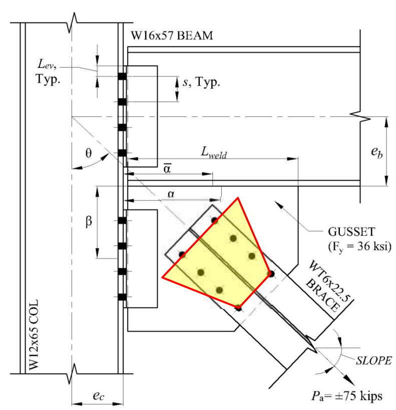
Brace-to-Gusset – Whitmore Section of Gusset Plate (cont.):

Table 4-14 (continued)
Available Critical Stress for Compression Members

L_c/r	F_y = 35 ksi		F_y = 36 ksi		F_y = 46 ksi		F_y = 50 ksi		F_y = 65 ksi		F_y = 70 ksi	
	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	F_{cr}/Ω_c	$\phi_c F_{cr}$
	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi	ksi
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
41	19.2	28.9	19.7	29.7	24.6	37.0	26.5	39.8	33.2	49.9	35.3	53.0
42	19.2	28.8	19.6	29.5	24.5	36.8	26.3	39.5	32.9	49.5	35.0	52.6
43	19.1	28.7	19.6	29.4	24.3	36.6	26.2	39.3	32.6	49.1	34.7	52.1

From AISC *Design Guide 29*: The effective length factor K has been established as 0.5 by full scale tests on bracing connections (Gross, 1990). It assumes that the gusset plate is supported on both connected edges.

(Alternately, see: Dowswell, Bo (2006), "Effective Length Factors for Gusset Plate Buckling," AISC *Engineering Journal*, 2nd Qtr.)



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Example VB Corner Connection, I
Brace-to-Gusset (cont.)

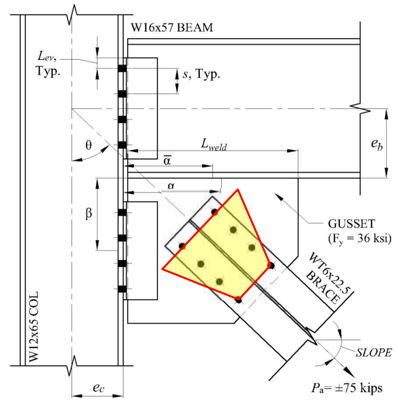
Brace-to-Gusset – Whitmore Section of Gusset Plate (cont.):

$$\frac{P_n}{\Omega_c} = \frac{F_{cr(\text{gusset})}}{\Omega_c} A_{w(\text{gusset})} + \frac{F_{cr(\text{beam})}}{\Omega_c} A_{w(\text{beam})}$$

$$= (19.7 \text{ ksi})(0.375 \text{ in.})(15.9 \text{ in.}) + 0$$

$$= 83 \text{ kips} \geq 75 \text{ kips } \textbf{o.k.}$$

Note: for Whitmore tension, use F_y/Ω in lieu of F_{cr}/Ω



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Example VB Corner Connection, I
Gusset Forces

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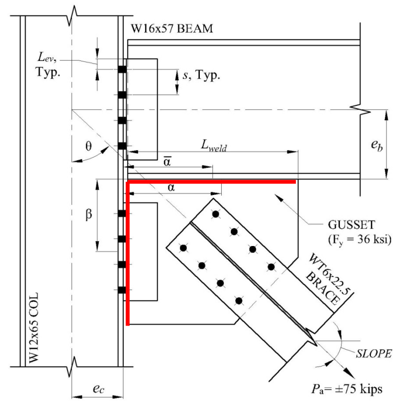
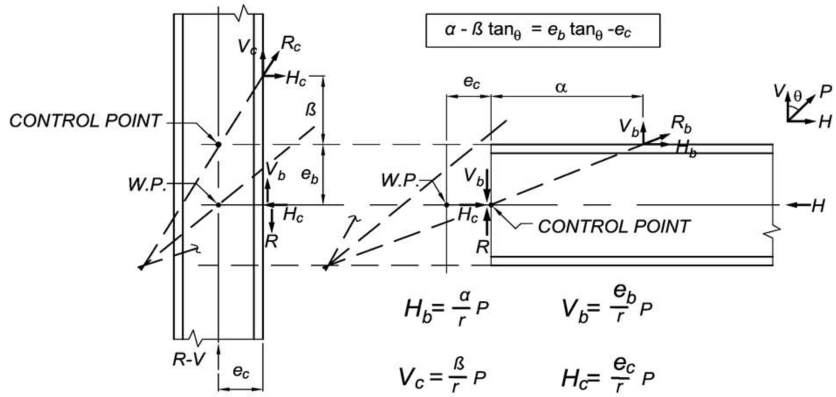
Example VB Corner Connection, I
Gusset Forces

Next, the gusset forces, V_b, H_b, M_b, V_c, H_c, and M_c, will be determined using the Uniform Force Method.



Geometry of UFM
All Parts are in Equilibrium



$$H_b = \frac{\alpha}{r} P \qquad V_b = \frac{e_b}{r} P$$

$$V_c = \frac{\beta}{r} P \qquad H_c = \frac{e_c}{r} P$$

$$r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2}$$

Example VB Corner Connection, I
Gusset Forces (cont.)

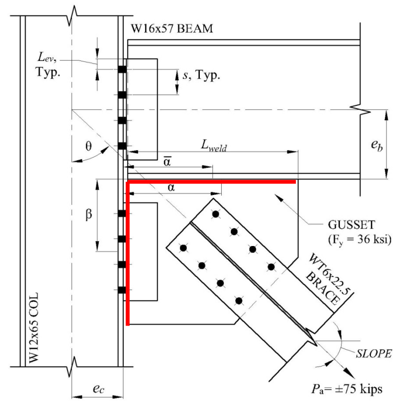
Determine Gusset Forces using UFM:

$\tan\theta = 1.05$

$e_b = d_B/2 = (16.4 \text{ in.})/2 = 8.2 \text{ in.}$

$e_c = d_c/2 = (12.1 \text{ in.})/2 = 6.05 \text{ in.}$

(note: $e_c = 0.0$ in. for connections into column web, except Special Case IV)



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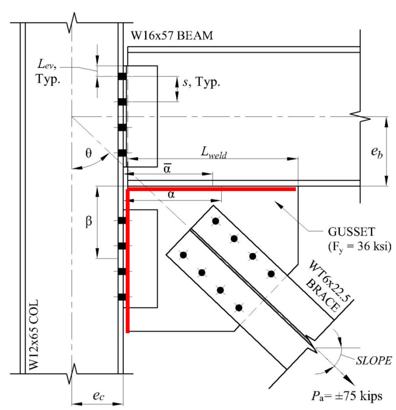
Example VB Corner Connection, I
Gusset Forces (cont.)

β = distance to center of gusset-to-column connections

From geometric layout, $H_{gusset} = 16.875$ in.
Place bolts in approximate center of gusset,
$\beta = 8.5$ in.
Let $\bar{\beta} = \beta$

$$\alpha = e_b \tan\theta - e_c + \beta \tan\theta$$
$$= (8.2 \text{ in.} \times 1.05) - 6.05 \text{ in.} + (8.5 \text{ in.} \times 1.05)$$
$$= 11.5 \text{ in.}$$



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Example VB Corner Connection, I Gusset Forces (cont.)

Determine Gusset Forces using UFM (cont.):

$$\bar{\alpha} = \frac{L_{weld}}{2} + 0.5 \text{ in. (setback for angles)}$$

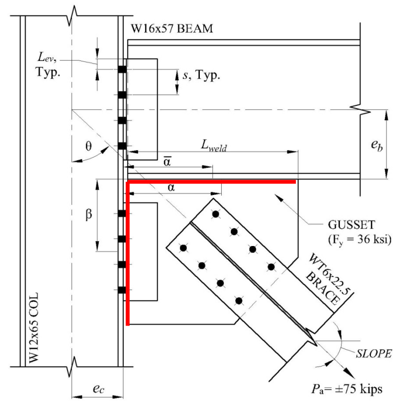
$$= \frac{20 \text{ in.}}{2} + 0.5 \text{ in.}$$

$$= 10.5 \text{ in. min.} \rightarrow \text{ does not equal } \alpha$$

$$r = \sqrt{\left[(\alpha + e_c)^2 + (\beta + e_b)^2\right]}$$

$$= \sqrt{\left[(11.5 \text{ in.} + 6.05 \text{ in.})^2 + (8.5 \text{ in.} + 8.2 \text{ in.})^2\right]}$$

$$= 24.2 \text{ in.}$$



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Example VB Corner Connection, I Gusset Forces (cont.)

Determine Gusset Forces using UFM (cont.):

$$V_b = P_a(e_b/r)$$

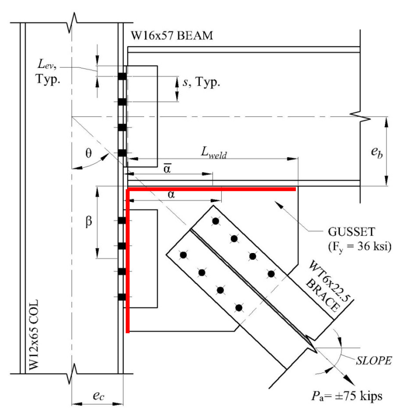
$$= (75 \text{ kips})(8.2 \text{ in.}/24.2 \text{ in.})$$

$$= 25.4 \text{ kips}$$

$$H_b = P_a(\alpha/r)$$

$$= (75 \text{ kips})(11.5 \text{ in.}/24.2 \text{ in.})$$

$$= 35.6 \text{ kips}$$



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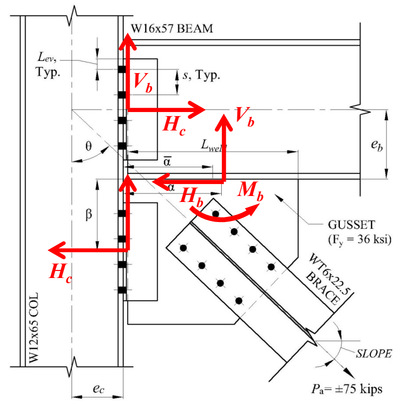
Example VB Corner Connection, I
Gusset Forces (cont.)

$$M_b = V_b(\alpha - \bar{\alpha})$$
$$= 25.4 \text{ kips}(11.5 \text{ in.} - 10.5 \text{ in.})$$
$$= 25.4 \text{ kip-in.}$$
$$V_c = P_a(\beta/r)$$
$$= 75 \text{ kips}(8.5 \text{ in.}/24.2 \text{ in.})$$
$$= 26.3 \text{ kips}$$
$$H_c = P_a(e_c/r)$$
$$= 75 \text{ kips}(6.05 \text{ in.}/24.2 \text{ in.})$$
$$= 18.8 \text{ kips}$$
$$M_c = 0 \text{ since } \bar{\beta} = \beta$$

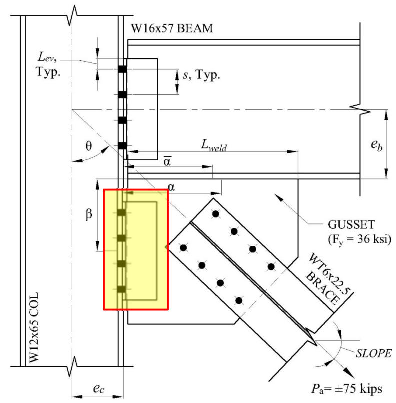


Example VB Corner Connection, I
Gusset-to-Column

Example VB Corner Connection, I Gusset-to-Column

The gusset-to-column will be designed using the gusset forces previously determined.

a. Check bolts at column flange
b. Check prying of angles and column flange
c. Check bolt bearing and tearout of angles and column flange
d. Check gross and net shear and block shear for angles
e. Check angle welds



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Example VB Corner Connection, I Gusset-to-Column (cont.)

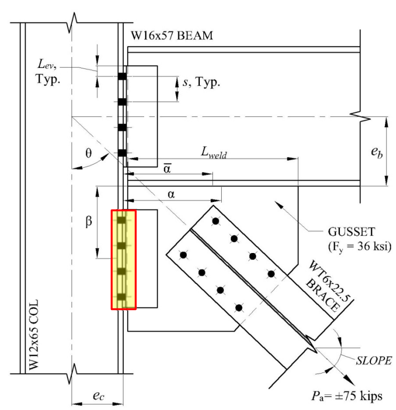
Gusset-to-Column – Bolts:

$R_b/\Omega = 2n(r_v/\Omega)$
$= 2(4)(11.9 \text{ kips})$
$= 95.2 \text{ kips} > V_c = 26.3 \text{ kips}$ **o.k.**

Check Shear and Tension:

$V_{bolt} = V_c/(2n) = 26.3 \text{ kips}/[(2)(4)]$
$= 3.29 \text{ kips}$

$T_{bolt} = H_c/(2n) = 18.8 \text{ kips}/[(2)(4)]$
$= 2.35 \text{ kips}$



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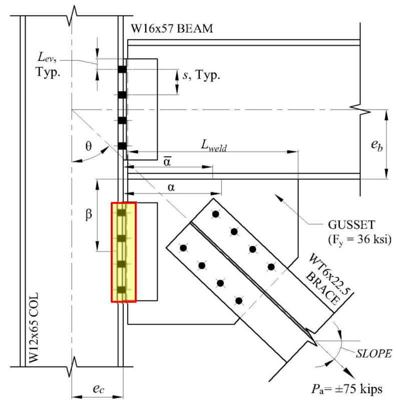
Example VB Corner Connection, I
Gusset-to-Column (cont.)

Gusset-to-Column – Bolts (cont.):

Check Shear and Tension (cont.):

$$A_b = \frac{\pi d_b^{\,2}}{4}$$
$$= \frac{3.14(0.75\text{ in.})^2}{4}$$
$$= 0.442\text{ in.}^2$$

$$f_{rv} = V_{bolt}/A_b$$
$$= 3.29\text{ kips}/0.442\text{ in.}^2$$
$$= 7.44\text{ ksi}$$



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Example VB Corner Connection, I
Gusset-to-Column (cont.)

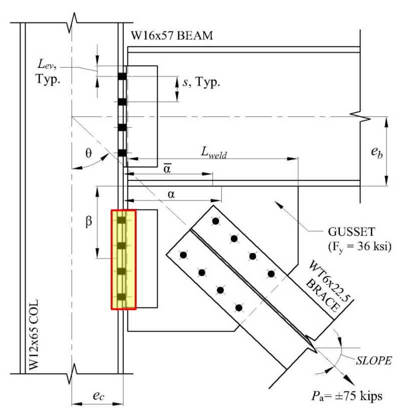
Gusset-to-Column – Bolts (cont.):

Check Shear and Tension (cont.):

$$F'_{nt} = 1.3F_{nt} - \Omega F_{nt} f_{rv}/F_{nv} \leq F_{nt} \quad (\textit{Spec}\text{ J3-3b})$$
$$= 1.3(90\text{ ksi}) - 2(90\text{ ksi})(7.44\text{ ksi})/(54\text{ ksi})$$
$$= 92.2\text{ ksi} > 90\text{ ksi, use 90 ksi}$$

AISC *Specification* **TABLE J3.2**
Nominal Strength of Fasteners and Threaded Parts, ksi (MPa)

Description of Fasteners	Nominal Tensile Strength, F_{nt}, ksi (MPa)[b]	Nominal Shear Strength in Bearing-Type Connections, F_{nv}, ksi (MPa)[b]
A307 bolts	45 (310)	27 (188)[c] [d]
Group A (e.g., A325) bolts, when threads are not excluded from shear planes	90 (620)	54 (372)



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Example VB Corner Connection, I Gusset-to-Column (cont.)

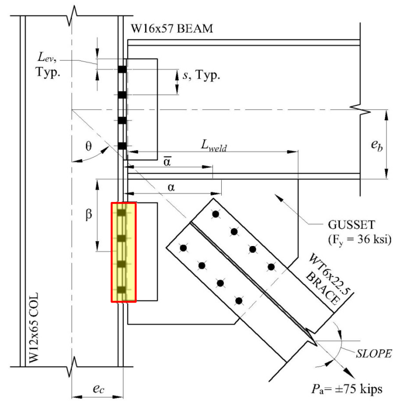
Gusset-to-Column – Bolts (cont.):

Check Shear and Tension (cont.):

$$\frac{r_t}{\Omega} = \frac{F'_{nt} A_b}{\Omega} \quad \text{(AISC } \textit{Spec} \text{ J3-2)}$$

$$= \frac{(90 \text{ ksi})(0.442 \text{ in.}^2)}{2}$$

$$= 19.9 \text{ kips} > T_{bolt} = 2.35 \text{ kips } \textbf{o.k.}$$



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Example VB Corner Connection, I Gusset-to-Column (cont.)

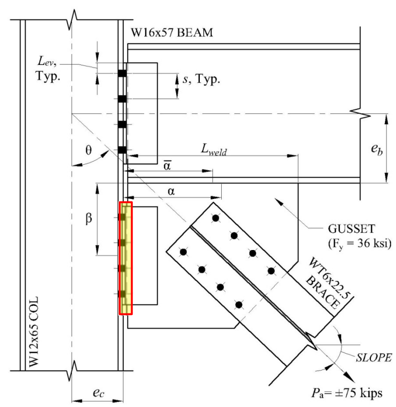
Gusset-to-Column – Prying of Angles:

For connections to column webs, there is no tension force on the angle since H_c = 0 kips. For this example, which is to a column flange, however:

$T_{bolt} = H_c/(2n)$ = 18.8 kips/[(2)(4)] = 2.35 kips

b = gage/2 – t_p/2 – t_a/2
= 5.5 in./2 – 0.375 in./2 – 0.375/2 = 2.38 in.

$b' = b - d_b/2$ =
= 2.38 in. – 0.75 in./2 = 2 in.



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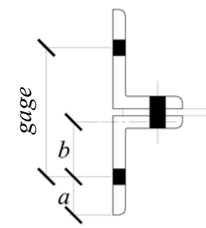
Example VB Corner Connection, I Gusset-to-Column (cont.)

Gusset-to-Column – Prying of Angles (cont.):

a = [2(angle OSL) + t_p – gage]/2
= [2(4 in.) + 0.375 in. – 5.5 in.]/2
= 1.44 in.

$a' = a + d_b/2 \le 1.25b + d_b/2$
= 1.44 in. + 0.75 in./2 ≤ 1.25(2.38 in.) + 0.75 in./2
= 1.82 in. ≤ 3.35 in.
= 1.82 in.

$\rho = b'/a'$ = 2 in./1.82 in. = 1.10



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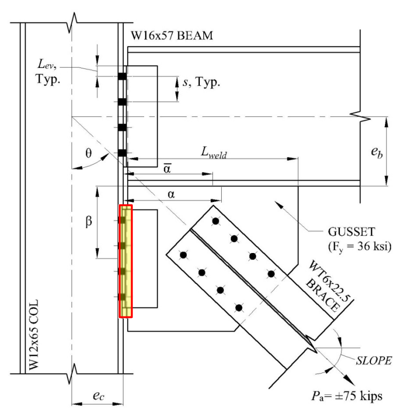
Example VB Corner Connection, I Gusset-to-Column (cont.)

Gusset-to-Column – Prying of Angles (cont.):

$p = L/n$ = 11.5 in./4
= 2.88 in. (tributary length per bolt)

$\delta = 1 - d'/p$ = 1 – 0.8125 in./2.88 in.
= 0.717

$B_c = r_t/\Omega$
=19.9 kips (bolt available tensile strength from previously)



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Example VB Corner Connection, I Gusset-to-Column (cont.)

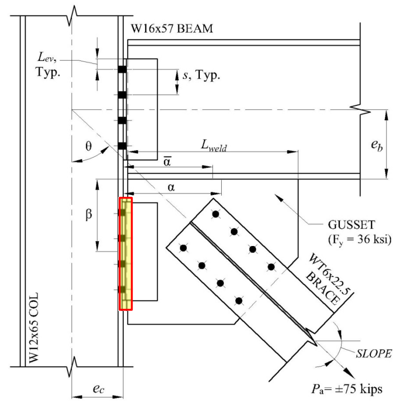
Gusset-to-Column – Prying of Angles (cont.):

$$t_c = \sqrt{\frac{\Omega 4 B_c b'}{pF_u}} \quad \text{(AISC \textit{Manual} Eq. 9-26b)}$$

$$= \sqrt{\frac{1.67(4)(19.9\text{ kips})(2\text{ in.})}{(2.88\text{ in.})(58\text{ ksi})}}$$

$$= 1.26\text{ in.}$$

t_c= minimum thickness of angles to develop full available tensile strength of bolt with no prying



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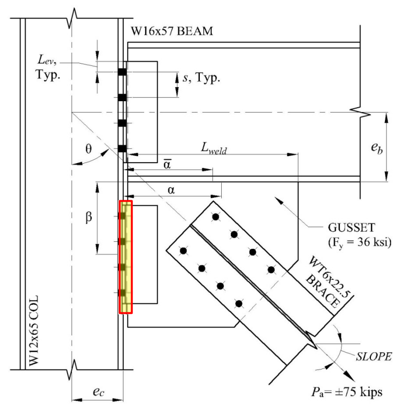
Example VB Corner Connection, I Gusset-to-Column (cont.)

Gusset-to-Column – Prying of Angles (cont.):

$$\alpha' = \frac{1}{\delta(1+\rho)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right] \quad \text{(AISC \textit{Manual} Eq. 9-28)}$$

$$= \frac{1}{0.717(1+1.10)}\left[\left(\frac{1.26\text{ in.}}{0.375\text{ in.}}\right)^2 - 1\right]$$

$$= 6.83\text{ in.}$$



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Example VB Corner Connection, I Gusset-to-Column (cont.)

Gusset-to-Column – Prying of Angles (cont.):

Note:

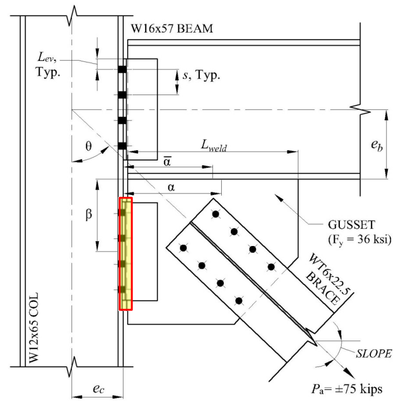
When $\alpha' < 0$, this means that the fitting has sufficient strength and stiffness to develop the full bolt available tensile strength,

$$Q = 1$$

When $0 \le \alpha' \le 1$, this means that the fitting has sufficient strength and stiffness to develop the full bolt available tensile strength, but insufficient stiffness to prevent prying action,

$$Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta\alpha')$$

(see AISC *Manual* Eq. 9-27)



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Example VB Corner Connection, I Gusset-to-Column (cont.)

Gusset-to-Column – Prying of Angles (cont.):

For the example,

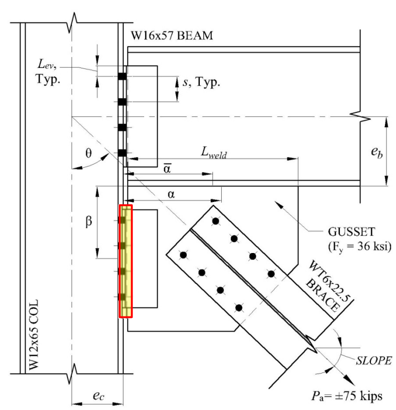
$$\alpha' = 6.83 > 1$$

This means that the angle has insufficient strength to develop the full bolt available tensile strength. Therefore,

$$Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta)$$

$$= \left(\frac{0.375 \text{ in.}}{1.26 \text{ in.}}\right)^2 (1 + 0.717 \text{ in.})$$

$$= 0.152$$



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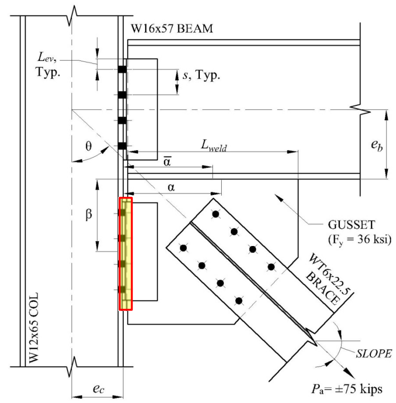
Example VB Corner Connection, I
Gusset-to-Column (cont.)

Gusset-to-Column – Prying of Angles (cont.):

The available tensile strength including the effects of prying action, T_c, is:

$$T_c = B_c Q = (19.9 \text{ kips})(0.152)$$
$$= 3.02 \text{ kips} > T_{bolt} = 2.35 \text{ kips } \textbf{o.k.}$$



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Example VB Corner Connection, I
Gusset-to-Column (cont.)

Gusset-to-Column – Prying of Column Flange:

For connections to column webs, there is no tension force on the angle since H_c = 0 kips. For this example, which is to a column flange, use the following criteria:

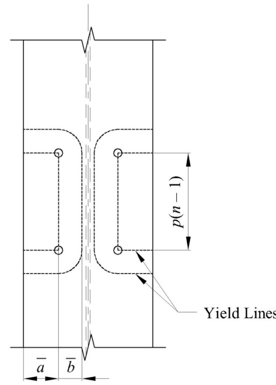
Determine effective flange width

$$p_{eff} = \left[p(n-1) + \pi \bar{b} + 2\bar{a} \right] / n$$

(Ref Akbar Tamboli, *Handbook of Steel Connection Design and Details,* 2nd Ed.)

p = spacing = 3 in.

Note: $\bar{a} = a$ and $\bar{b} = b$



(A.P. Mann and L.J. Morris, "Limit Design of Extended End-Plate Connections," *Journal of the Structural Division, ASCE*, Vol. 105, ST3, March 1979)

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Example VB Corner Connection, I Gusset-to-Column (cont.)

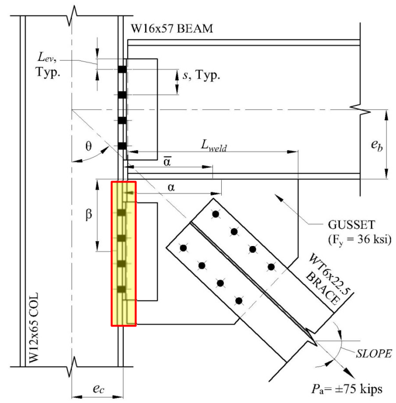
Gusset-to-Column – Prying of Column Flange (cont.):

$\bar{b} = (\text{gage} - t_{wc})/2$
$= (5.5\text{ in.} - 0.39\text{ in.})/2 = 2.56\text{ in.}$

$\bar{a} = (b_{fc} - \text{gage})/2$
$= (12\text{ in.} - 5.5\text{ in.})/2 = 3.25\text{ in.}$

$p_{eff} = [p(n-1) + \pi\bar{b} + 2\bar{a}]/n$
$= [(3\text{ in.})(4-1) + (3.14)(2.56\text{ in.}) + (2)(3.25\text{ in.})]/4$
$= 5.88\text{ in.}$

Note: use 3” max if at top of column



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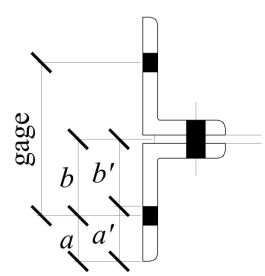
Example VB Corner Connection, I Gusset-to-Column (cont.)

Gusset-to-Column – Prying of Column Flange (cont.):

$T_{bolt} = H_c/(2n) = 18.8\text{ kips}/[(2)(4)] = 2.35\text{ kips}$

$b = \text{gage}/2 - t_{wc}/2$
$= 5.5\text{ in.}/2 - 0.39\text{ in.}/2 = 2.56\text{ in.}$

$b' = b - d_b/2 =$
$= 2.56\text{ in.} - 0.75\text{ in.}/2 = 2.18\text{ in.}$



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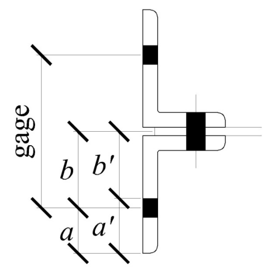
Example VB Corner Connection, I Gusset-to-Column (cont.)

Gusset-to-Column – Prying of Column Flange (cont.):

$a = \min\{(b_{fc} - \text{gage})/2,\ a \text{ from angle calcs}\}$
$= \min\{(12\text{ in.} - 5.5\text{ in.})/2,\ 1.44\text{ in.}\}$
$= 1.44\text{ in.}$

$a' = a + d_b/2 \le 1.25b + d_b/2$
$= 1.44\text{ in.} + 0.75\text{ in.}/2 \le 1.25(2.56\text{ in.}) + 0.75\text{ in.}/2$
$= 1.82\text{ in.} < 3.58\text{ in.}$ Use 1.82 in.

$\rho = b'/a' = 2.18\text{ in.}/1.82\text{ in.} = 1.20$



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Example VB Corner Connection, I Gusset-to-Column (cont.)

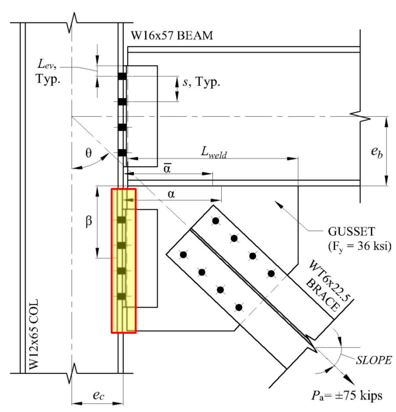
Gusset-to-Column – Prying of Column Flange (cont.):

$p = p_{eff} = 5.88\text{ in.}$
$\delta = 1 - d'/p = 1 - 0.8125\text{ in.}/5.88\text{ in.}$
$= 0.862$

$$t_c = \sqrt{\frac{\Omega 4 B_c b'}{pF_u}}$$ (AISC *Manual* Eq. 9-26b)

$$= \sqrt{\frac{1.67(4)(19.9\text{ kips})(2.18\text{ in.})}{(5.88\text{ in.})(65\text{ ksi})}}$$

$= 0.871\text{ in.}$
→ check available strength of bolt including the effects of prying action



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Example VB Corner Connection, I Gusset-to-Column (cont.)

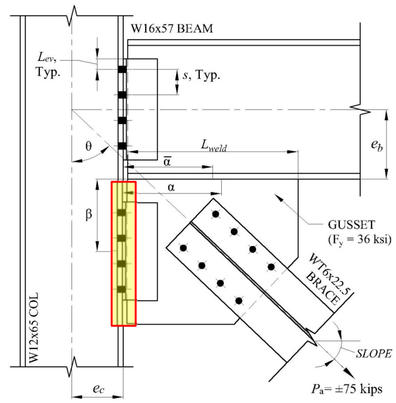
Gusset-to-Column – Prying of Column Flange (cont.):

$$\alpha' = \frac{1}{\delta(1+\rho)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right] \quad \text{(AISC } \textit{Manual} \text{ Eq. 9-28)}$$

$$= \frac{1}{0.862(1+1.20)}\left[\left(\frac{0.871 \text{ in.}}{0.605 \text{ in.}}\right)^2 - 1\right]$$

$$= 0.565 > 0 \text{ and } < 1.0$$

→ use $Q = \left(\frac{t}{t_c}\right)^2 (1+\delta\alpha')$



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Example VB Corner Connection, I Gusset-to-Column (cont.)

Gusset-to-Column – Prying of Column Flange (cont.):

$$Q = \left(\frac{t}{t_c}\right)^2 (1+\delta\alpha')$$

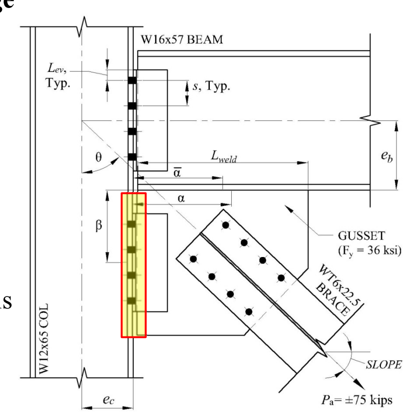
$$= \left(\frac{0.605 \text{ in.}}{0.871 \text{ in.}}\right)^2 \left[1+(0.862)(0.565)\right]$$

$$= 0.717$$

The available strength of bolt including prying is

$T_c = B_c Q = (19.9 \text{ kips})(0.717)$

$= 14.3 \text{ kips} > T_{bolt} = 2.35 \text{ kips}$ **o.k.**



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Example VB Corner Connection, I Gusset-to-Column (cont.)

Gusset-to-Column – Bolt Bearing:

$R_n = 2.4dtF_u$ AISC *Specification* Eq. J3-6a

$R_n = 1.2l_c tF_u$ AISC *Specification* Eq. J3-6c

AISC Bolt Shear:

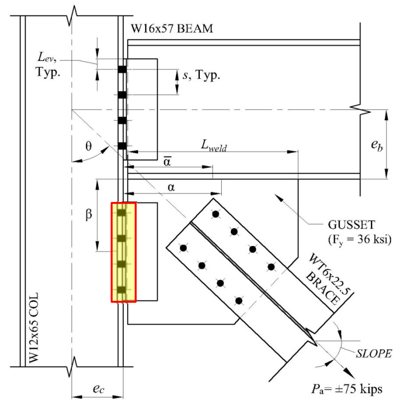
$r_v/\Omega = 11.9$ kips

Column Bolt Bearing:

$R_{brg(col)}/\Omega = 2.4d_b t_f F_u/\Omega$

$= 2.4(0.75 \text{ in.})(0.605 \text{ in.})(65 \text{ ksi})/2$

$= 35.4$ kips



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Example VB Corner Connection, I Gusset-to-Column (cont.)

Gusset-to-Column – Bolt Bearing (cont.):

Column Bolt Tear Out (Center):

$R_{tear(col\_C)}/\Omega = 1.2(s - d_{h\_w})t_f F_u/\Omega$

$= 1.2(3 \text{ in.} - {}^{13}/_{16} \text{ in.})$

$\times(0.605 \text{ in.})(65 \text{ ksi})/2$

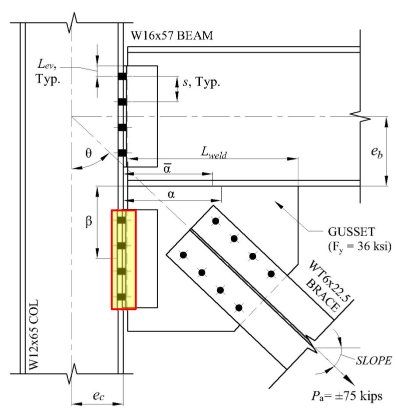
$= 51.6$ kips

Angle Bolt Bearing:

$R_{brg(L)}/\Omega = 2.4d_b t_a F_u/\Omega$

$= 2.4(0.75 \text{ in.})(0.375 \text{ in.})(58 \text{ ksi})/2$

$= 19.6$ kips



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Example VB Corner Connection, I Gusset-to-Column (cont.)

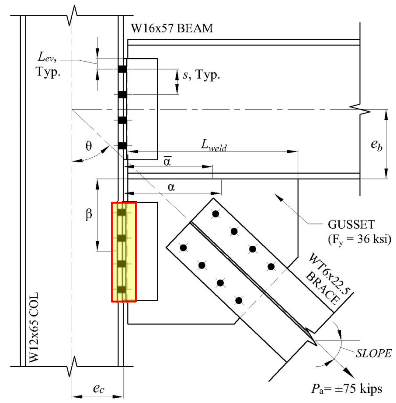
Gusset-to-Column – Bolt Bearing (cont.):

Angle Bolt Tear Out (Edge):

$R_{tear(L\_E)}/\Omega = 1.2(L_{ev} - d_{h\_w}/2)t_a F_u/\Omega$
$= 1.2\ (1.25\ \text{in.} - {}^{13}/_{16}\ \text{in.}/2)$
$\times(0.375\ \text{in.})(58\ \text{ksi})/2$
$= 11.0\ \text{kips}$

Angle Bolt Tear Out (Center):

$R_{tear(L\_C)}/\Omega = 1.2(s - d_{h\_w})t_a F_u/\Omega$
$= 1.2\ (3\ \text{in.} - {}^{13}/_{16}\ \text{in.})(0.375\ \text{in.})$
$\times(58\ \text{ksi})/2$
$= 28.6\ \text{kips}$



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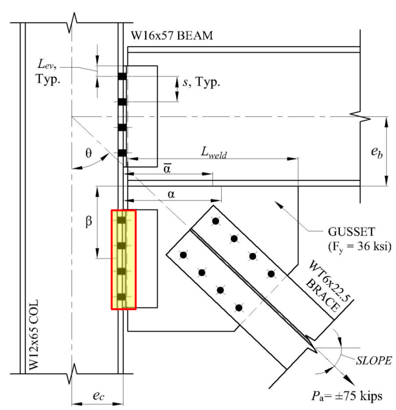
Example VB Corner Connection, I Gusset-to-Column (cont.)

Gusset-to-Column – Bolt Bearing (cont.):

$R_{n(top)}/\Omega = \min\{r_v/\Omega, R_{brg(col)}/\Omega, R_{tear(col\_C)}/\Omega,$
$R_{brg(L)}/\Omega, R_{tear(L\_C)}/\Omega\}$
$= \min\{11.9\ \text{kips}, 35.4\ \text{kips}, 51.6\ \text{kips},$
$19.6\ \text{kips}, 28.6\ \text{kips}\}$
$= 11.9\ \text{kips}$

$R_{n(center)}/\Omega = \min\{r_v/\Omega, R_{brg(col)}/\Omega, R_{tear(col\_C)}/\Omega,$
$R_{brg(L)}/\Omega, R_{tear(L\_C)}/\Omega\}$
$= \min\{11.9\ \text{kips}, 35.4\ \text{kips}, 51.6\ \text{kips},$
$19.6\ \text{kips}, 28.6\ \text{kips}\}$
$= 11.9\ \text{kips}$



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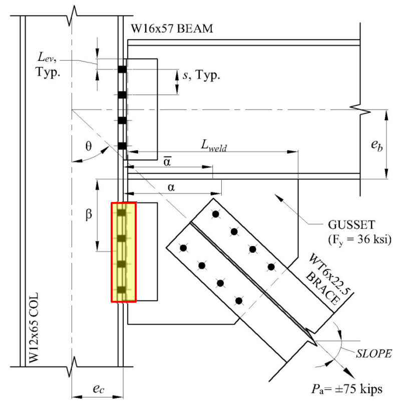
Example VB Corner Connection, I Gusset-to-Column (cont.)

Gusset-to-Column – Bolt Bearing (cont.):

$$R_{n(\text{bot})}/\Omega = \min\{r_v/\Omega, R_{brg(\text{col})}/\Omega, R_{tear(\text{col\_C})}/\Omega, R_{brg(\text{L})}/\Omega, R_{tear(\text{L\_E})}/\Omega\}$$
$$= \min\{11.9 \text{ kips}, 35.4 \text{ kips}, 51.6 \text{ kips}, 19.6 \text{ kips}, 11.0 \text{ kips}\}$$
$$= 11.0 \text{ kips}$$

Total Bolt Strength

$$R_n/\Omega = [R_{n(\text{top})}/\Omega + (R_{n(\text{center})}/\Omega)(n-2) + R_{n(\text{bot})}/\Omega](2 \text{ lines})$$
$$= [11.9 \text{ kips} + (11.9 \text{ kips})(4-2) + 11.0 \text{ kips}](2)$$
$$= 93.4 \text{ kips} > V_c = 26.3 \text{ kips} \textbf{ o.k.}$$



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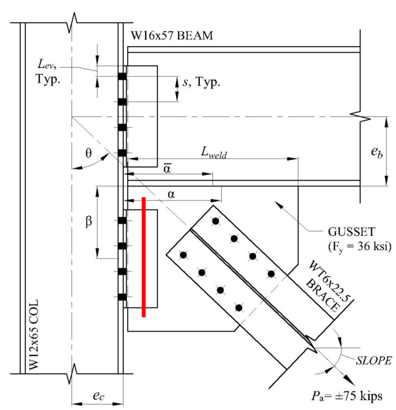
Example VB Corner Connection, I Gusset-to-Column (cont.)

Gusset-to-Column – Angle Gross Shear:

AISC *Specification* Eq. J4-3

$$\frac{R_n}{\Omega} = \frac{0.6F_yA_g}{\Omega}$$
$$= \frac{0.6(36 \text{ ksi})(11.5 \text{ in.})(0.375 \text{ in.})}{1.5}$$
$$= 124 \text{ kips} > V_c = 26.3 \text{ kips} \textbf{ o.k.}$$



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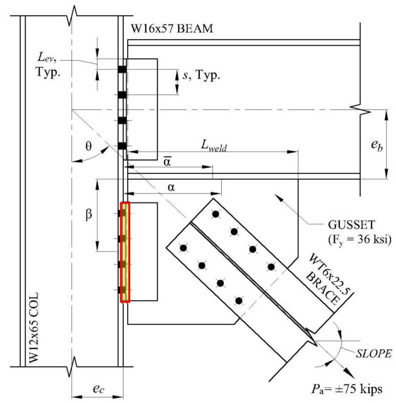
Example VB Corner Connection, I Gusset-to-Column (cont.)

Gusset-to-Column – Angle Net Shear:

AISC *Specification* Eq. J4-4

$$L_{nv} = L_{eff} - n(d_{h,\text{width}} + 1/16 \text{ in.})$$
$$= 11.5 \text{ in.} - 4(13/16 \text{ in.} + 1/16 \text{ in.})$$
$$= 8 \text{ in.}$$

$$\frac{R_n}{\Omega} = \frac{0.6F_uA_{nv}}{\Omega}$$
$$= \frac{0.6(58 \text{ ksi})(2)(8 \text{ in.})(0.375 \text{ in.})}{2}$$
$$= 104 \text{ kips} > V_c = 26.3 \text{ kips} \ \textbf{o.k.}$$



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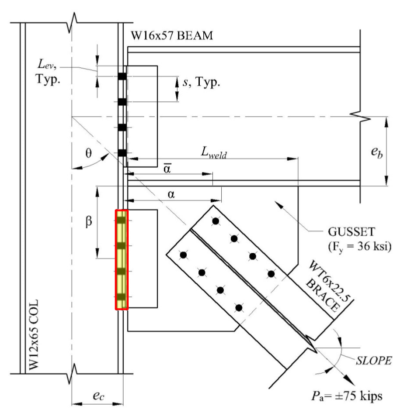
Example VB Corner Connection, I Gusset-to-Column (cont.)

Gusset-to-Column – Block Shear of Angles OSL:

Calculations for block shear of angles are similar to those done in this example for the WT brace to gusset connection.

The actual calculations will be contained in Appendix 2 which is available with the handouts for this session.



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Example VB Corner Connection, I
Gusset-to-Column (cont.)

Gusset-to-Column – Block Shear of Gusset for vertical force (V_c):
AISC *Spec* Section J4.3 *Block Shear Strength*

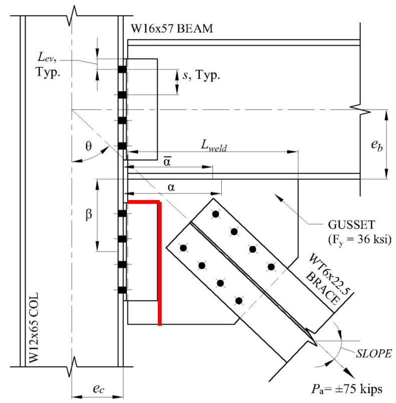
$\Omega = 2.00$

$R_n = 0.6F_uA_{nv} + U_{bs}F_uA_{nt}$

$\leq 0.6F_yA_{gv} + U_{bs}F_uA_{nt}$

Shear Rupture $= 0.6F_uA_{nv}$
Shear Yield $= 0.6F_yA_{gv}$
Tension Rupture $= F_uA_{nt}$

U_{bs} = 1.0 for Direct Loaded Connections



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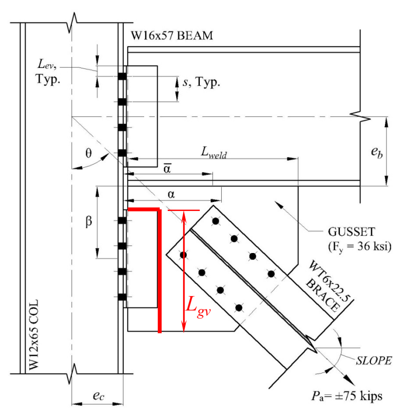
Example VB Corner Connection, I
Gusset-to-Column (cont.)

Gusset-to-Column – Block Shear of Gusset:
Gusset is welded

For Shear check:

L_{gv} = 14.25 in. (from geometric layout)
$L_{gt} = k$ = 3.5 in.
L_{nv} = 14.25 in.
$L_{nt} = k$ = 3.5 in.



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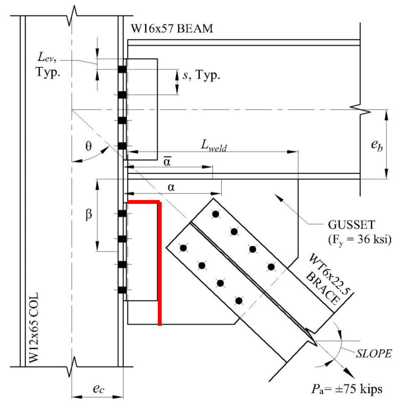
Example VB Corner Connection, I
Gusset-to-Column (cont.)

Gusset-to-Column – Block Shear of Gusset (cont.):

$A_{gv} = L_{gv}t_p = (14.25\text{ in.})(0.375\text{ in.})$
$= 5.34\text{ in.}^2$

$A_{nv} = L_{nv}t_p = (14.25\text{ in.})(0.375\text{ in.})$
$= 5.34\text{ in.}^2$

$A_{nt} = L_{nt}t_p = (3.5\text{ in.})(0.375\text{ in.})$
$= 1.31\text{ in.}^2$



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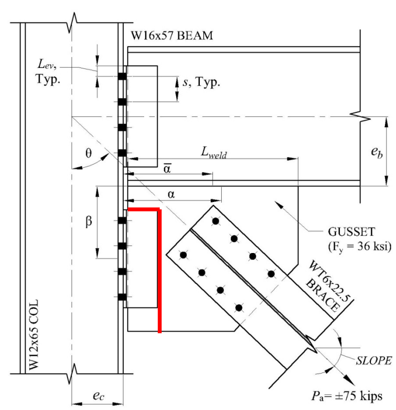
Example VB Corner Connection, I
Gusset-to-Column (cont.)

Gusset-to-Column – Block Shear of Gusset (cont.):

$$R_n = \min\begin{bmatrix} 0.6(58\text{ ksi})(5.34\text{ in.}^2) = 185.83\text{ kips} \\ 0.6(36\text{ ksi})(5.34\text{ in.}^2) = 115.34\text{ kips} \end{bmatrix}$$

$+ [1(58\text{ ksi})(1.31\text{ in.}^2)]$
$= 115.34\text{ kips} + 75.98\text{ kips} = 191.32\text{ kips}$

$R_n/\Omega = (191.32\text{ kips})/2$
$= 96\text{ kips} > V_c = 26.3\text{ kips}$ **o.k.**



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Example VB Corner Connection, I
Gusset-to-Column (cont.)

Gusset-to-Column – Block Shear of Gusset for horizontal force (H_c):

Gusset is welded

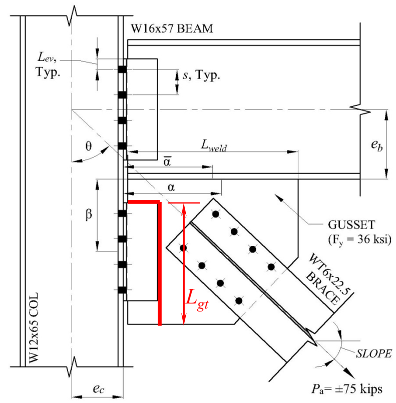
For Axial check:

$L_{gv} = k = 3.5$ in.

$L_{gt} = 14.25$ in. (from geometric layout)

$L_{nv} = k = 3.5$ in.

$L_{nt} = 14.25$ in.



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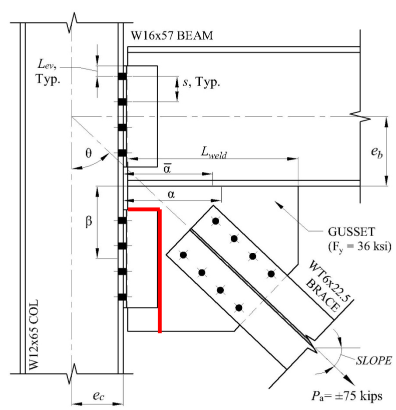
Example VB Corner Connection, I
Gusset-to-Column (cont.)

Gusset-to-Column – Block Shear of Gusset (cont.):

$A_{gv} = L_{gv}t_p = (3.5 \text{ in.})(0.375 \text{ in.})$
$= 1.31 \text{ in.}^2$

$A_{nv} = L_{nv}t_p = (3.5 \text{ in.})(0.375 \text{ in.})$
$= 1.31 \text{ in.}^2$

$A_{nt} = L_{nt}t_p = (14.25 \text{ in.})(0.375 \text{ in.})$
$= 5.34 \text{ in.}^2$



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Example VB Corner Connection, I Gusset-to-Column (cont.)

Gusset-to-Column – Block Shear of Gusset (cont.):

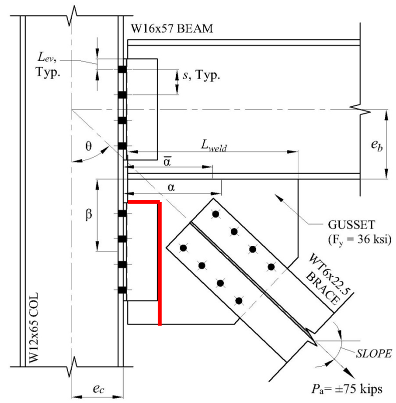
$$R_n = \min\begin{bmatrix} 0.6(58\text{ ksi})(1.31\text{ in.}^2) = 45.59\text{ kips} \\ 0.6(36\text{ ksi})(1.31\text{ in.}^2) = 28.30\text{ kips}\end{bmatrix}$$

$$+ [1(58\text{ ksi})(5.34\text{ in.}^2)]$$

$$= 28.30\text{ kips} + 309.72\text{ kips} = 338.02\text{ kips}$$

$R_n/\Omega = (338.02\text{ kips})/2$

$= 169\text{ kips} > H_c = 18.8\text{ kips}$ **o.k.**



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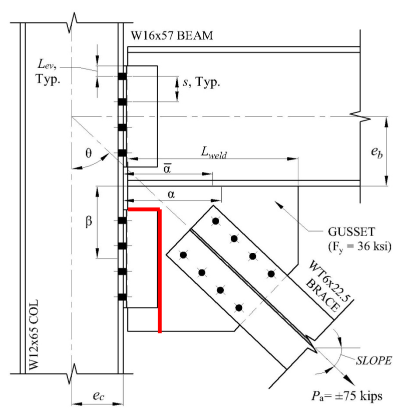
Example VB Corner Connection, I Gusset-to-Column (cont.)

Gusset-to-Column – Block Shear of Gusset (cont.):

Unity Check (Sum of Squares):

$$\left(\frac{V_c}{R_{nv}/\Omega}\right)^2 + \left(\frac{H_c}{R_{nt}/\Omega}\right)^2 \le 1.0$$

$$\left(\frac{26.3\text{ kips}}{96\text{ kips}}\right)^2 + \left(\frac{18.8\text{ kips}}{169\text{ kips}}\right)^2 = 0.09 < 1.0 \quad \textbf{o.k.}$$



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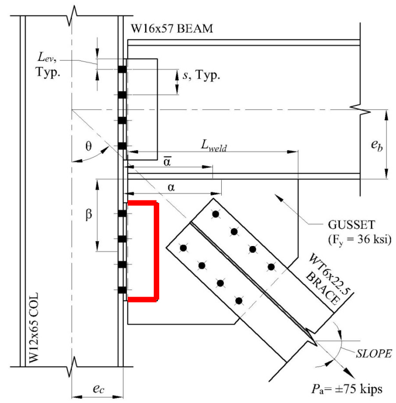
Example VB Corner Connection, I
Gusset-to-Column (cont.)

Gusset-to-Column – Size Weld:

$$P_c = \sqrt{(H_c)^2 + (V_c)^2}$$
$$= \sqrt{(18.8\text{ kips})^2 + (26.3\text{ kips})^2}$$
$$= 32.3\text{ kips}$$

$$\theta = \tan^{-1}(H_c/V_c) = \tan^{-1}(18.8\text{ kips}/26.3\text{ kips})$$
$$= 35.6°$$

$$l = s(n-1) + 2L_{ev}$$
$$= (3\text{ in.})(4-1) + 2(1.25\text{ in.})$$
$$= 11.5\text{ in.}$$



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Example VB Corner Connection, I
Gusset-to-Column (cont.)

Gusset-to-Column – Size Weld (cont.):

$kl = 3.5$ in.

$k = kl/l = 3.5\text{ in.}/11.5\text{ in.} = 0.304$

$x = 0.057$

$a = [(al - xl]/l$

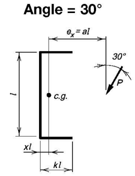
$= [4\text{ in.} - 0.057(11.5\text{ in.})]/11.5\text{ in.}$

$= 0.29$

$C = 2.96$ (AISC *Manual* Table 8-8 with Angle = 30°)

AISC *Manual*

Table 8-8 (continued)
Coefficients, *C*,
for Eccentrically Loaded Weld Groups
Angle = 30°



a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2
0.00	2.18	2.70	3.21	3.73	4.24	4.76	5.27	5.78	6.30	6.81	7.33	8.35
0.10	2.02	2.57	3.10	3.62	4.14	4.67	5.19	5.71	6.23	6.75	7.28	8.32
0.15	1.92	2.43	2.95	3.47	3.98	4.49	5.00	5.52	6.03	6.54	7.05	8.09
0.20	1.82	2.29	2.79	3.29	3.78	4.28	4.77	5.27	5.77	6.27	6.77	7.78
0.25	1.71	2.15	2.62	3.10	3.58	4.06	4.53	5.01	5.49	5.97	6.46	7.45
0.30	1.61	2.01	2.45	2.91	3.37	3.83	4.29	4.75	5.21	5.68	6.15	7.11
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424

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Example VB Corner Connection, I Gusset-to-Column (cont.)

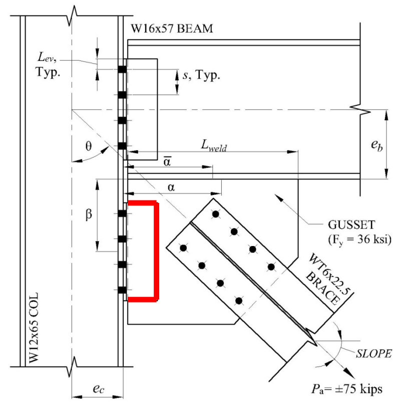
Gusset-to-Column – Size Weld (cont.):

C_1 = 1.0 for E70 (AISC *Manual* Table 8-3)

AISC *Manual*

Table 8-3
Electrode Strength Coefficient, C_1

Electrode	F_{EXX} (ksi)	C_1
E60	60	0.857
E70	70	1.00
E80	80	1.03
E90	90	1.16
E100	100	1.21
E110	110	1.34



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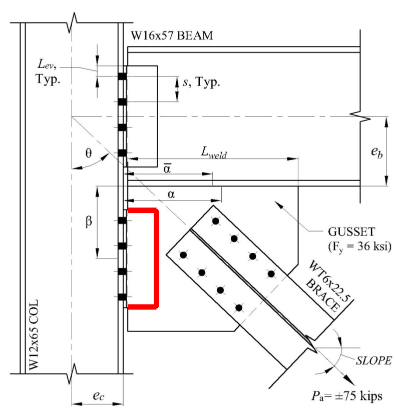
Example VB Corner Connection, I Gusset-to-Column (cont.)

Gusset-to-Column – Size Weld (cont.):

$D_{min} = P_c\Omega/(2CC_1L)$
= 32.3 kips(2)/((2)(2.96)(1.0)(11.5 in.))
= 0.949 < 4 (16th of an inch)
¼" fillet weld **o.k.**

$t_{min} = 2(0.928D_{min})/[0.6(F_u/\Omega)]$
= 2(0.928)(0.949)/[0.6(58 ksi/2)]
= 0.101 in. < t_p = 0.375 in. **o.k.**

Note, the above is same as *Manual* Eq 9-3:
$t_{min} = 6.19D/F_u$



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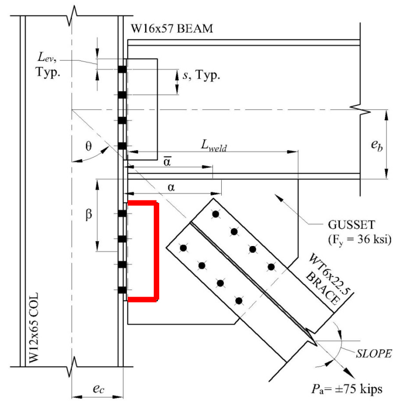
Example VB Corner Connection, I Gusset-to-Column (cont.)

Gusset-to-Column – Size Weld (cont.):

If $t_{min} < t_p$, no reduction necessary.

If $t_{min} > t_p$, $\dfrac{R_w}{\Omega} = \left(\dfrac{R_n}{\Omega}\right)\left(\dfrac{t_p}{t_{min}}\right)$



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Example VB Corner Connection, I Gusset-to-Beam

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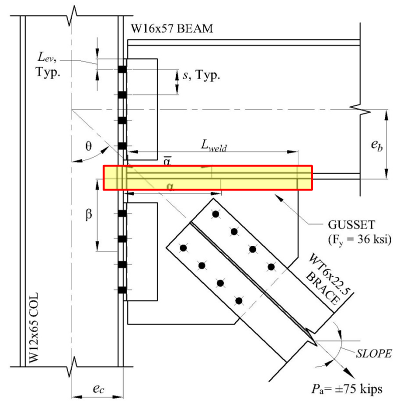
Example VB Corner Connection, I
Gusset-to-Beam

The gusset-to-beam will be designed by doing the following:

a. Size weld between gusset and beam
b. Check gusset for shear and tension at weld line
c. Check beam web yielding
d. Check local beam web crippling



Design of Fillet Welds

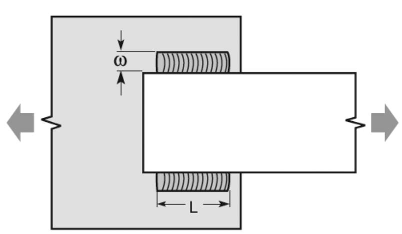
Section J2.4(b) allows the directional strength increase for <u>linear</u> weld groups.

$R_n = F_{nw} A_{we}$ (AISC *Specification* Eq. J2-4)

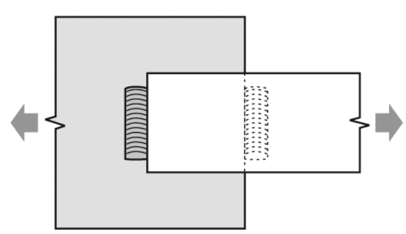
$F_{nw} = 0.6F_{EXX}(1.0 + 0.5\sin^{1.5}\theta)$ (AISC *Specification* Eq. J2-5)

F_{EXX} = electrode classification number, i.e., minimum specified tensile strength, ksi

θ = angle of loading measured from the weld longitudinal axis, degrees



θ = 0°



θ = 90°

Design of Fillet Welds

Again, this is more commonly written:

For ASD: $R_n/\Omega = 0.928(1.0 + 0.5\sin^{1.5}\theta)DL$

For LRFD: $\phi R_n = 1.392(1.0 + 0.5\sin^{1.5}\theta)DL$

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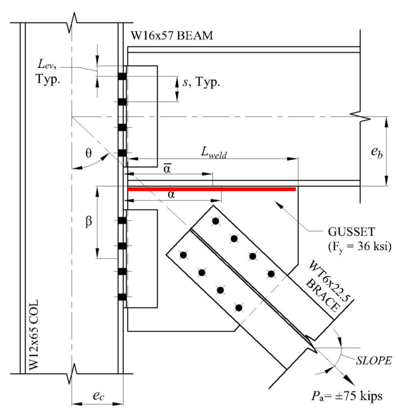
Example VB Corner Connection, I Gusset-to-Beam (cont.)

Gusset-to-Beam – Size Weld:

$f_x = H_b/L_{weld} = 35.6 \text{ kips}/20 \text{ in.}$
$= 1.78 \text{ kips/in.}$

$f_y = V_b/L_{weld} = 25.4 \text{ kips}/20 \text{ in.}$
$= 1.27 \text{ kips/in.}$

$f'_y = 4M_b/L_{weld}^2$
$= 4(25.4 \text{ kip-in.})/(20 \text{ in.})^2$
$= 0.254 \text{ kips/in.}$



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Example VB Corner Connection, I Gusset-to-Beam (cont.)

Gusset-to-Beam – Size Weld (cont.):

$$f_{peak} = \sqrt{(f_x)^2 + (f_y + f'_y)^2}$$
$$= \sqrt{(1.78 \text{ k/in.})^2 + (1.27 \text{ k/in.} + 0.254 \text{ k/in.})^2}$$
$$= 2.34 \text{ kips/in.}$$

$$1.25 f_{avg} = (1.25)\left(\frac{1}{2}\right)\left[\sqrt{(f_x)^2 + (f_y + f'_y)^2} + \sqrt{(f_x)^2 + (f_y - f'_y)^2}\right]$$
$$= (1.25)\left(\frac{1}{2}\right)\left[\sqrt{(1.78 \text{ k/in.})^2 + (1.27 \text{ k/in.} + 0.254 \text{ k/in.})^2} + \sqrt{(1.78 \text{ k/in.})^2 + (1.27 \text{ k/in.} - 0.254 \text{ k/in.})^2}\right]$$
$$= 2.75 \text{ kips/in.}$$

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Example VB Corner Connection, I Gusset-to-Beam (cont.)

Gusset-to-Beam – Size Weld (cont.):

$$\theta = \tan^{-1}[(f_y + f'_y)/f_x)]$$
$$= \tan^{-1}[(1.27 \text{ kips/in.} + 0.254 \text{ kips/in.})/1.78 \text{ kips/in.}]$$
$$= 40.6°$$

$$f_w = 0.60(F_{EXX}/\Omega)(0.707D/16)(1.0 + 0.50 \sin^{1.5}\theta)$$
$$= 0.928D\,(1.0 + 0.50 \sin^{1.5} 40.6°)$$
$$= 0.928D(1.26)$$

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Example VB Corner Connection, I Gusset-to-Beam (cont.)

Gusset-to-Beam – Size Weld (cont.):

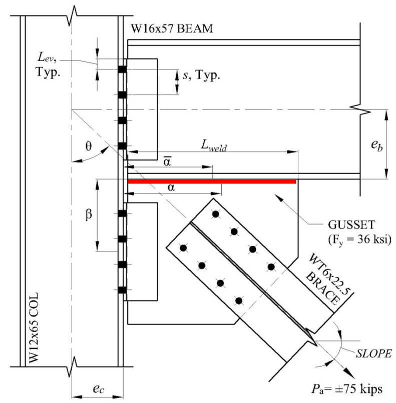
D_{min} = max(f_{peak}, 1.25f_{avg}) / [f_w(2 sides)]

= max(2.34 kips/in., 2.75 kips/in.) / [0.928(1.26)(2)]

= 1.18 < 4 (16[th] on an inch) **o.k.**

(1/4" fillet provided)

Also, per AISC *Specification* Table J2.4, minimum size fillet weld for 3/8" gusset = 3/16" < 1/4" **o.k.**



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Example VB Corner Connection, I Gusset-to-Beam (cont.)

Gusset-to-Beam – Check Gusset:

Shear:

$t_{min(\text{gusset})} = f_x/[0.6(F_y/\Omega)]$

= (1.78 kips/in.)/[0.6(36 ksi/1.5)]

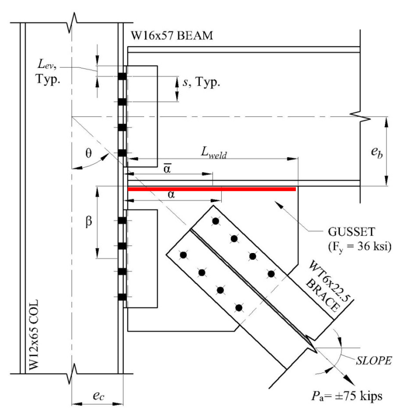
= 0.124 in. < t_p = 0.375 in. **o.k.**

Tension:

$t_{min(\text{gusset})} = (f_y + f'_y)/(F_y/\Omega)$

= (1.27 k/in. + 0.254 k/in.) / (36 ksi/1.67)

= 0.071 in. < t_p = 0.375 in. **o.k.**



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Example VB Corner Connection, I Gusset-to-Beam (cont.)

Gusset-to-Beam – Beam Web Yielding:

$P_{equival} = 2[M_b/(L_{weld}/2)] + V_b$

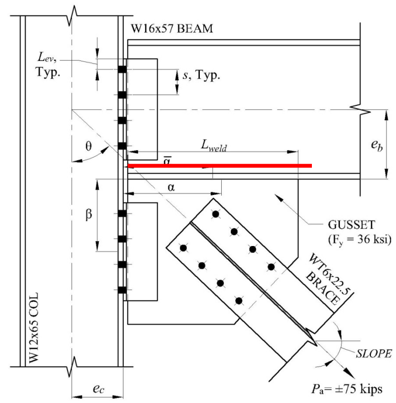
$= 2[25.4 \text{ k-in.}/(20 \text{ in.}/2)] + 25.4 \text{ kips}$

$= 30.5$ kips for full weld length

$R_n/\Omega = t_w(L_{weld} + 2.5k)(F_y/\Omega)$

$= (0.43 \text{ in.})[20 \text{ in.} + 2.5(1.12 \text{ in.})] \times (50 \text{ ksi}/1.5)$

$= 327 \text{ kips} > 30.5 \text{ kips}$ **o.k.**



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Example VB Corner Connection, I Gusset-to-Beam (cont.)

Gusset-to-Beam – Local Beam Web Crippling:

$P_{equival} = 30.5$ kips for full weld length (from beam web yielding calcs)

$$\frac{R_n}{\Omega} = \frac{0.80 t_w^2 \left[1 + 3\left(\frac{L_{weld}}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}}}{\Omega} Q_f$$

AISC *Specification* Eq. J10-4

$$= \frac{\left[\frac{0.80(0.43 \text{ in.})^2}{2}\left[1 + 3\left(\frac{20 \text{ in.}}{16.4 \text{ in.}}\right)\left(\frac{0.43 \text{ in.}}{0.715 \text{ in.}}\right)^{1.5}\right] \times \sqrt{\frac{(29{,}000 \text{ ksi})(50 \text{ ksi})(0.715 \text{ in.})}{0.43 \text{ in.}}}\right]}{2}(1.0)$$

$= 311 \text{ kips} > 30.5 \text{ kips}$ **o.k.**

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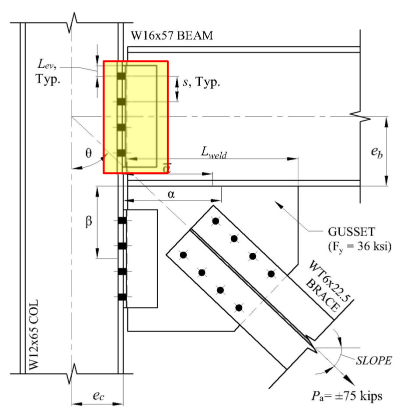
Example VB Corner Connection, I Beam-to-Column

Example VB Corner Connection, I Beam-to-Column

Finally, the beam-to-column will be designed, checking the beam end as a shear and axially loaded end connection.

a. Check bolts at column flange
b. Check prying of angles and column flanges
c. Check bolt bearing and tearout of angles and column flange
d. Check gross and net section shear and block shear for angles
e. Check block shear of beam
f. Check angle welds



Example VB Corner Connection, I Beam-to-Column (cont.)

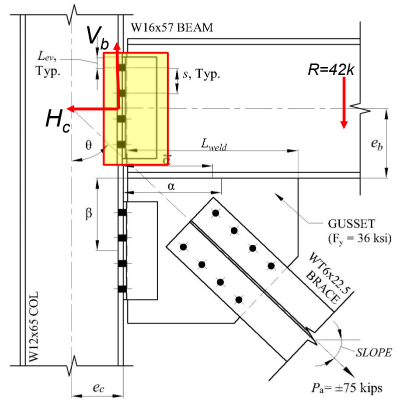
Beam-to-Column – Forces:

V_b = 25.4 kips

R = 42 kips

Calculations for this connection (shown in yellow) are similar to those done in this example for the gusset to column connection.

The actual calculations will be contained in Appendix 3 which is available with the handouts for this session.



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Vertical Brace Example

If either the gusset-to-column connection or the beam-to-column cannot be made to work economically, it may be necessary to change the force distribution. This can be done by:

a. Changing the dimensions of the gusset plate.
b. Using ΔV_b to change the distribution of the vertical brace component – Special Case II.
c. Try using Special Case V (Bypass Method) which will be discussed next week (Session 5)

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Economy of Vertical Bracing Connections

- Use the Uniform Force Method.
- Use W12 or W14 columns as a minimum where possible.
- Use W18 beams as a minimum.
- Use same depth beams either side of the column when transfer forces exist.

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Economy of Vertical Bracing Connections

- Consider net section capacity when sizing brace members.
- Provide actual loads (tension and compression).

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Summary

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Summary

- Example of a corner vertical bracing connection was reviewed for a Wind and Low-Seismic condition
  - Began the example with the brace-to-gusset design
  - Gusset dimensions were determined based on the brace-to-gusset connection
  - The gusset-to-column connection including prying action was checked
  - Followed by the gusset-to-beam connection
  - Finally, the beam-to-column connection, also including prying action, was checked

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Questions?

Thank you!

AISC | Questions

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Individual Session Registrants

PDH Certificates

- All WFH individuals associated with a group registration will be issued a certificate.
- All individuals attending at your connection: you will receive an email on how to report their attendance from: registration@aisc.org.
  - Be on the lookout: Check your spam filter! Check your junk folder!
  - Completely fill out online form. Don't forget to check the boxes next to each attendee's name!

8-Session Registrants

PDH Certificates

One certificate will be issued at the conclusion of all 8 sessions.

8-Session Registrants

Access to the quiz

Information for accessing the quiz will be emailed to you by Wednesday. It will contain a link to access the quiz. EMAIL COMES FROM NIGHTSCHOOL@AISC.ORG.

Quiz and attendance records

Posted Friday mornings. www.aisc.org/nightschool -- Click on Current Course Details.

Reasons for quiz

- EEU – You must take all quizzes and the final exam to receive EEU.
- PDHs – If you watch a recorded session, you must pass quiz for PDHs.
- REINFORCEMENT – Reinforce what you learn tonight. Get more out of the course.

Note: If you attend the live presentation, you do not have to take the quizzes to receive PDHs

8-Session Registrants

Access to the recording

Information for accessing the recording will be emailed to you by Wednesday. The recording will be available for four weeks. (For 8-session registrants only.) EMAIL COMES FROM NIGHTSCHOOL@AISC.ORG.

PDHs via recording

If you watch a recorded session, you must take *and pass* the quiz for PDHs.

8-Session Registrants

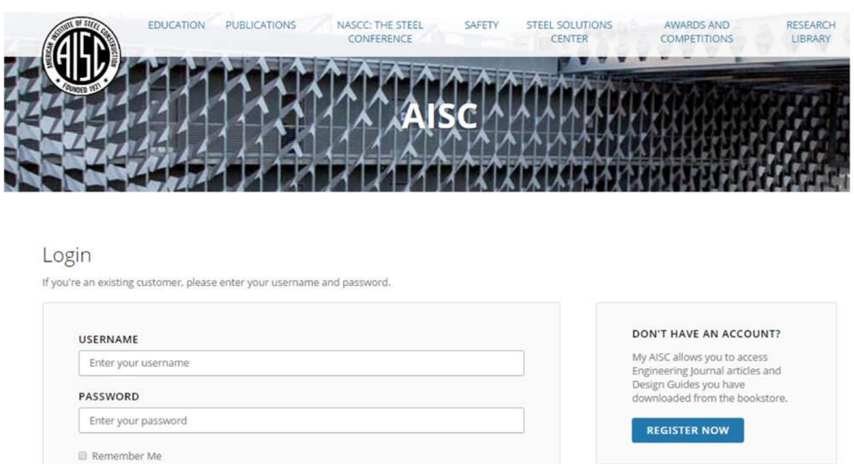
Night School Resources

Find all your handouts, quizzes and quiz scores, recording access, and attendance information all in one place!

8-Session Registrants

Night School Resources

Go to www.aisc.org and sign in.



8-Session Registrants

Night School Resources

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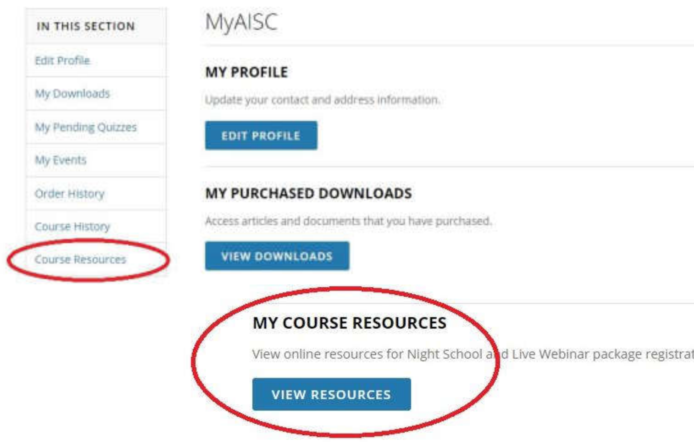
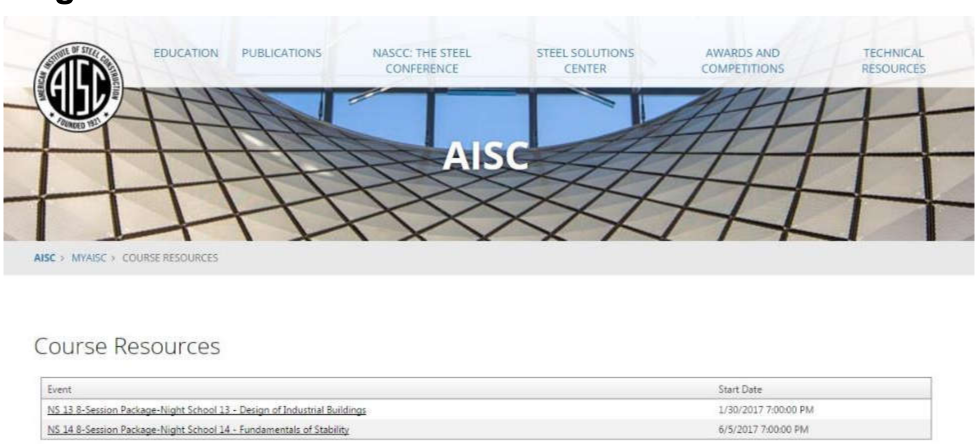


8-Session Registrants

Night School Resources



8-Session Registrants

Night School Resources

Night School 13: Design of Industrial Buildings

8-SESSION PACKAGE RESOURCES

Event	Date	Handouts	Video	Quiz	Attendance
NS13 - Design Criteria	1/30/2017 7:00:00 PM	Handouts	View Passcode: NS13DSN	Pass Score: 80	Pending
NS13 - Economic Considerations	2/6/2017 7:00:00 PM	Handouts	Available 02/08/2017 5pm EST	Available 02/08/2017 5pm EST	Pending
NS13 - Lateral Load Systems and Details	2/13/2017 7:00:00 PM	Handouts	Available 02/15/2017 5pm EST	Available 02/15/2017 5pm EST	Pending
NS13 - Preliminary Design Procedures	2/27/2017 7:00:00 PM	Handouts	Available 03/01/2017 5pm EST	Available 03/01/2017 5pm EST	Pending
NS13 - Crane Girder Design and Frame Analysis	3/6/2017 7:00:00 PM	Handouts	Available 03/08/2017 5pm EST	Available 03/08/2017 5pm EST	Pending
NS13 - Frame Member and Connection Design	3/13/2017 7:00:00 PM	Handouts	Available 03/15/2017 5pm EST	Available 03/15/2017 5pm EST	Pending
NS13 - Transfer Crane Girder & Longitudinal Bldg Bracing Dsn	3/27/2017 7:00:00 PM	Handouts	Available 03/29/2017 5pm EST	Available 03/29/2017 5pm EST	Pending
NS13 - Building Envelope and Bracing Design	4/3/2017 7:00:00 PM	Handouts	Available 04/05/2017 5pm EST	Available 04/05/2017 5pm EST	Pending

8-Session Registrants

Night School Resources

- Weekly “quiz and recording” email.
- Weekly updates of the master quiz and attendance record, found at www.aisc.org/nightschool28. Scroll down to Quiz and Attendance records.
    - Updated on Friday mornings.

8-Session Registrants

Night School Resources

- Webinar connection information
  - Reminder email sent out Monday mornings
- Links to handouts also found here

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