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Course Description

Vertical Bracing Connections – Corner Connection Part 1: Wind and Low-Seismic Systems

October 20, 2014

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

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Learning Objectives

- Become familiar with various methodologies for bracing connection design.
- Develop an understanding of the limit states that must be reviewed for the design of bracing connections.
- Gain an understanding of the bracing connection design process through the presentation of a design example.
- Become familiar with practical design tips and how to apply to common design challenges.

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Bracing Connections and Related Topics

Session 4: Vertical Bracing Connections – Corner Connection Part 1: Wind and Low-Seismic Systems

Presented by
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Roswell, GA

Bracing Connections and Related Topics

By: William Thornton

Course Outline

1. Basic Principles
2. Uniform Force Method
3. Bracing Connection Details and Prying Action
4. **Vertical Bracing Connections – Corner Part 1**
5. Vertical Bracing Connections – Corner Part 2
6. Chevron Gussets for Wind or Low-Seismic
7. Chevron Gussets for High Seismic
8. Additional Connection Topics

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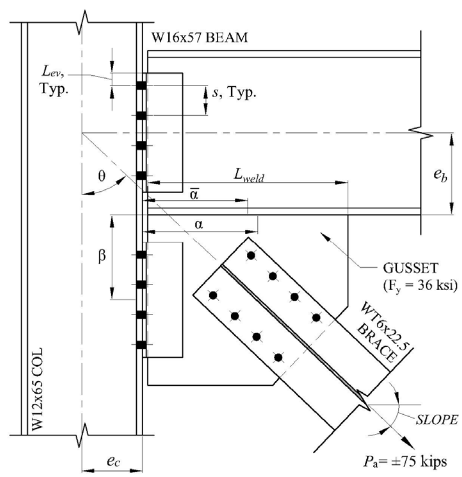
Example of VB Corner Connection (Part I) in Wind and Low-Seismic



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

Given:

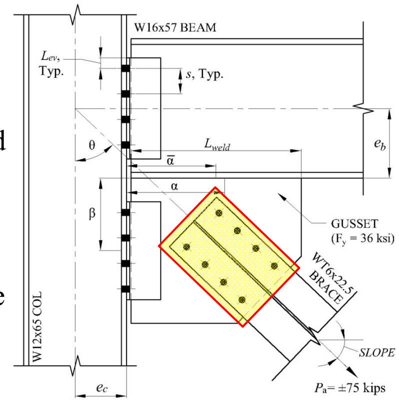
1. AISC 14th Edition, ASD
2. Beam-to-Column and Gusset-to-Column: $n = 4$
3. L_{weld} = 20 in. min. (gusset to beam) with ¼" min. weld
4. ¾" dia. A325-N, STD holes, UNO
5. Double L4x4x3/8" Gr. 36 angles, horizontal SSLT with L_{ev} = 1.25 in. minimum and bolt spacing, s = 3 in.
6. Brace force, P_a = ±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

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

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

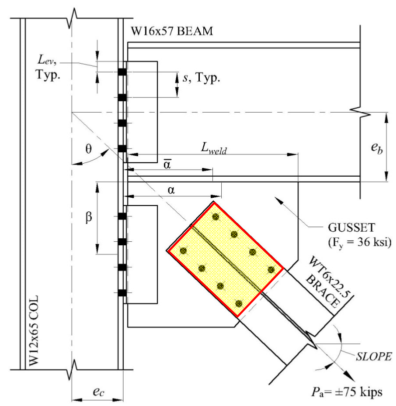
$R_b/\Omega = nr_v/\Omega$ (single shear)

$= 8\times 11.9$ kips/bolt

$= 95.2$ kips ≥ 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	Loading	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.)

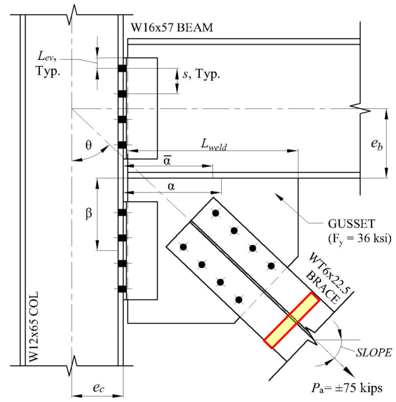
Brace-to-Gusset – WT Gross Tension:

AISC *Specification* Eq. J4-1

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

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

$= 196$ kips > 75 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 taken as $^1/_{16}$" greater than the nominal dimension of the hole. (Spec. D3.2)
- 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 by fasteners or welds (except as in Cases 4, 5 and 6).		$U = 1.0$	——
2	All tension members, except plates and HSS, where the tension load is transmitted to some but not all of the cross-sectional elements by fasteners or longitudinal welds or by longitudinal welds in combination with transverse welds. (Alternatively, for W, M, S and HP, Case 7 may be used. For angles, Case 8 may be used.)		$U = 1 - \bar{x}/l$	
3	load is transmitted only by transverse welds to some but not all of the cross-sectional elements.		and A_n = area of the directly connected elements	——
4	Plates where the tension load is transmitted by longitudinal welds only.		$l \geq 2w \ldots U = 1.0$ $2w > l \geq 1.5w \ldots U = 0.87$ $1.5w > l \geq w \ldots U = 0.75$	
5	Round HSS with a single concentric gusset plate		$l \geq 1.3D \ldots U = 1.0$ $D \leq l < 1.3D \ldots U = 1 - \bar{x}/l$ $\bar{x} = D/\pi$	
6	Rectangular HSS	with a single concentric gusset plate	$l \geq H \ldots U = 1 - \bar{x}/l$ $\bar{x} = \frac{B^2 + 2BH}{4(B+H)}$	
		with two side gusset plates	$l \geq H \ldots U = 1 - \bar{x}/l$ $\bar{x} = \frac{B^2}{4(B+H)}$	
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 3 or more fasteners per line in the direction of loading	$b_f \geq 2/3d \ldots U = 0.90$ $b_f < 2/3d \ldots U = 0.85$	——
		with web connected with 4 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 4 or more fasteners per line in the direction of loading	$U = 0.80$	——
		with 3 fasteners per line in the direction of loading (With fewer than 3 fasteners per line in the direction of loading, use Case 2.)	$U = 0.60$	——

l = length of connection, in. (mm); w = plate width, in. (mm); $\bar{x}$ = eccentricity of connection, in. (mm); B = overall width of rectangular HSS member, measured 90° to the plane of the connection, in. (mm); H = overall height of rectangular HSS member, measured in the plane of the connection, in. (mm)

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

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

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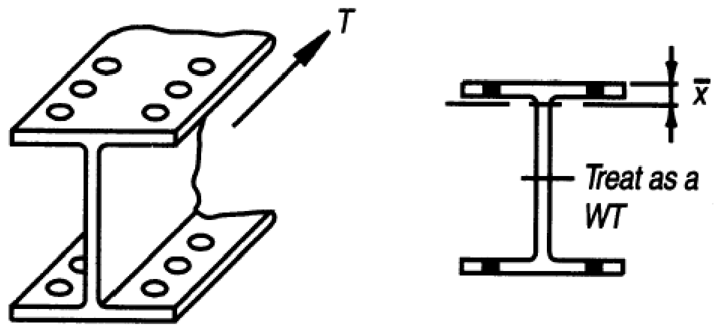
Net Tension
Determination of $\bar{x}$



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

Brace-to-Gusset – WT Net Tension:

AISC *Specification* Eq. J4-2

$\bar{x} = 1.13$ in.

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

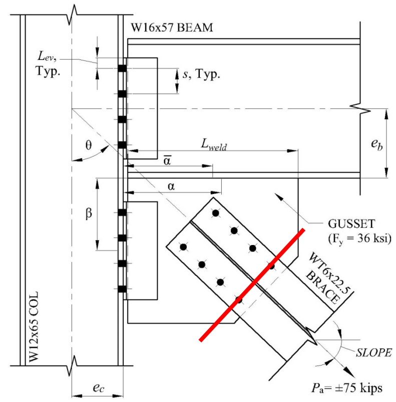
$U = 1 - \bar{x}/l = 1 - (1.13 \text{ in.}/9 \text{ in.}) = 0.874$

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.



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

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

$A_e = U[A_g - t_f(\text{\# lines})(d_h + {}^1\!/_{16}\text{ in.})]$

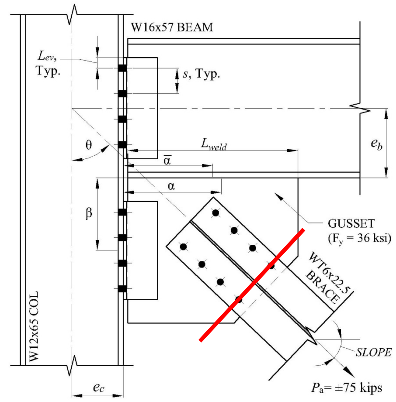
$= 0.874\ [6.56\text{ in.}^2 - (0.575\text{ in.})(2)(7/8\text{ in.})]$

$= 4.85\text{ in.}^2$

$$\frac{R_n}{\Omega} = \frac{F_u A_e}{\Omega}$$

$$= \frac{(65\text{ ksi})(4.85\text{ in.}^2)}{2}$$

$= 158$ kips > 75 kips **o.k.**



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Block Shear

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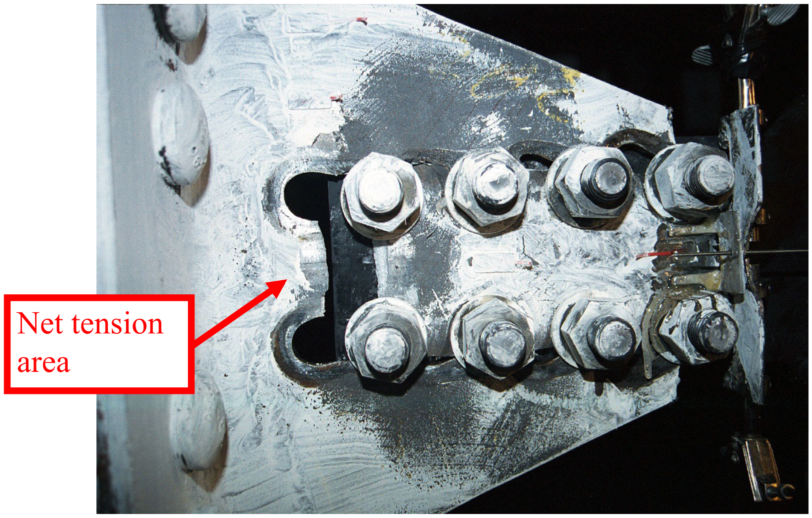
Block Shear



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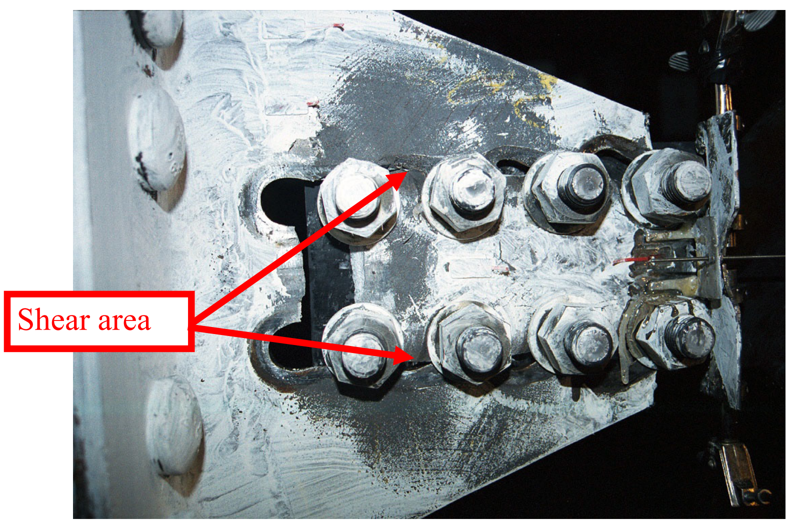
Block Shear



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

Brace-to-Gusset – Block Shear WT:

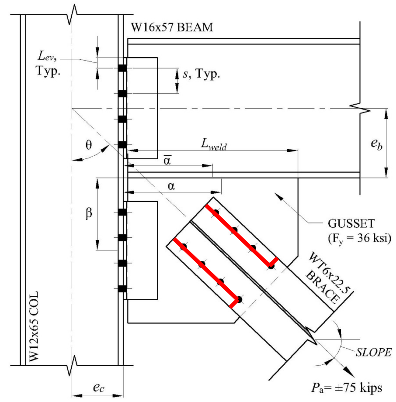
AISC *Spec* Section J4.3 *Block Shear Strength*

$\Omega = 2.00$

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

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



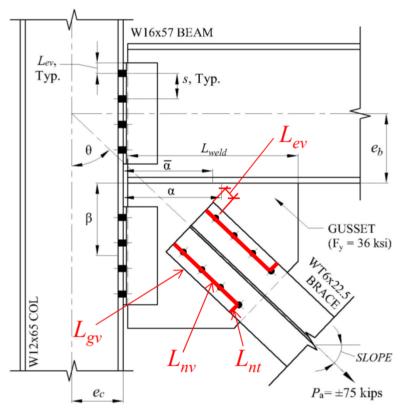
Example VB Corner Connection, I
Brace-to-Gusset (cont.)

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

$L_{gv} = L_{ev} + s(n - 1)$
= 1.5 in. + (3 in.)(4 – 1) = 10.5 in.

$L_{nv} = L_{ev} + s(n - 1) - (n - 0.5)(d_h + {}^{1}/_{16}\text{ in.})$
= 1.5 in. + (3 in.)(4 – 1)
– (4 – 0.5)(⅞ in.)
= 7.44 in.

$L_{nt} = (b_f - \text{gage})/2 - (d_h + {}^{1}/_{16}\text{ in.})/2$
= (8.05 in. – 5.5 in.)/2 – (⅞ in.)/2
= 0.838 in.



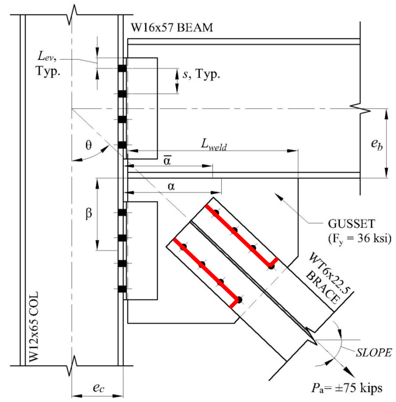
Example VB Corner Connection, I Brace-to-Gusset (cont.)

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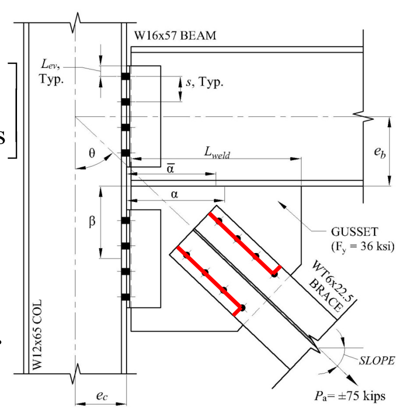
Example VB Corner Connection, I Brace-to-Gusset (cont.)

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:

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

$\Omega = 2.00$

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

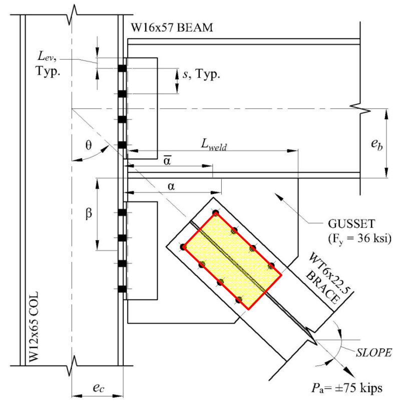
$\leq 0.6F_yA_{gv} + U_{bs}F_uA_{nt}$

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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The lecture will skip to slide 35.

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

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

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

= 1.5 in. + (3 in.)(4 – 1) = 10.5 in.

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

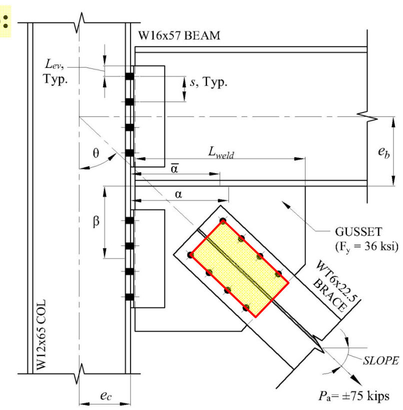
= 1.5 in. + (3 in.)(4 – 1)

– (4 – 0.5)(⅞ in.)

= 7.44 in.

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

= 5.5 in. – (⅞ in.) = 4.63 in.



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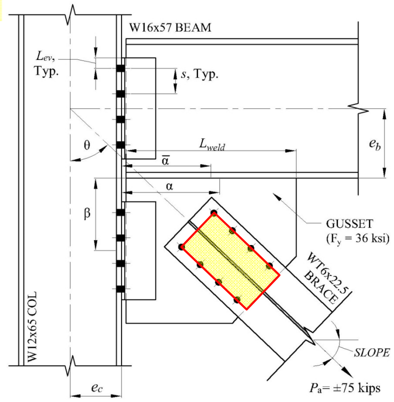
Example VB Corner Connection, I Brace-to-Gusset (cont.)

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

$$A_{gv} = 2L_{gv}t_p = 2(10.5\text{ in.})(0.375\text{ in.}) = 7.88\text{ in.}^2$$

$$A_{nv} = 2L_{nv}t_p = 2(7.44\text{ in.})(0.375\text{ in.}) = 5.58\text{ in.}^2$$

$$A_{nt} = L_{nt}t_p = (4.63\text{ in.})(0.375\text{ in.}) = 1.74\text{ in.}^2$$



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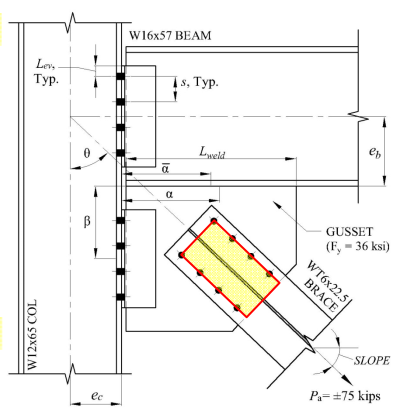
Example VB Corner Connection, I Brace-to-Gusset (cont.)

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

$$R_n = \min\begin{bmatrix} 0.6(58\text{ ksi})(5.58\text{ in.}^2) = 194.18\text{ kips} \\ 0.6(36\text{ ksi})(7.88\text{ in.}^2) = 170.21\text{ kips} \end{bmatrix} + [1(58\text{ ksi})(1.74\text{ in.}^2)]$$

$$= 170.21\text{ kips} + 100.92\text{ kips} = 271.13\text{ kips}$$

$R_n/\Omega = (271.13\text{ kips})/2 = 136\text{ kips} > 75\text{ kips}$ **o.k.**



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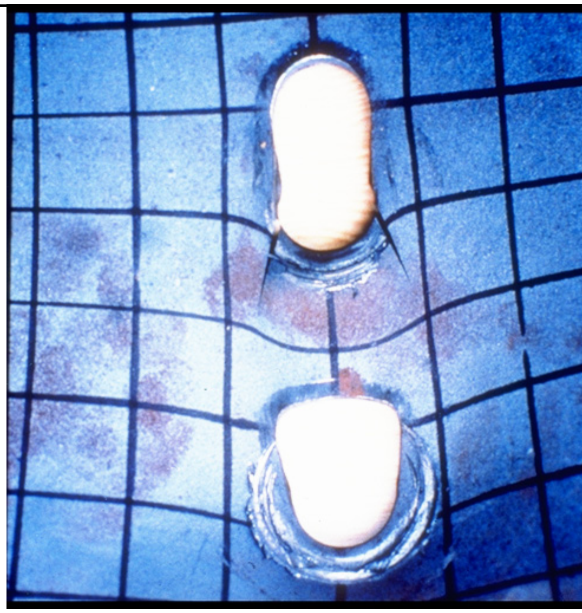

Bolt Tearout

Bolt Bearing

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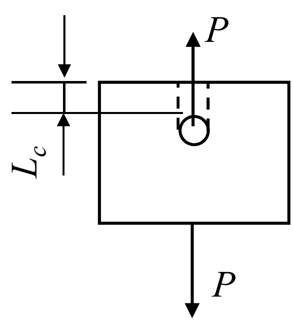
Bolt Bearing and Tearout

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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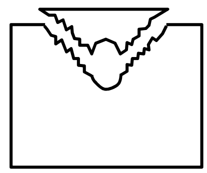
Bolt Bearing and Tearout

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



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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:

AISC *Specification* Eq. J3-6a

AISC Bolt Shear:

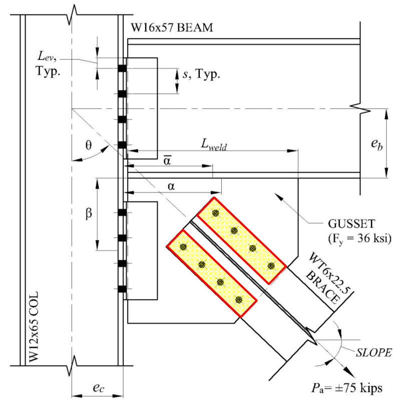
$r_v/\Omega = 11.9$ kips

WT Bolt Bearing:

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

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

$= 33.6$ 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):

$R_{tear(WT\_E)}/\Omega = 1.2[L_{ev} - (d_{h\_w}/2)]t_f F_u/\Omega$

$= 1.2[1.5 \text{ in.} - ({}^{13}/_{16} \text{ in.})/2]$

$\times(0.575 \text{ in.})(65 \text{ ksi})/2$

$= 24.5$ kips

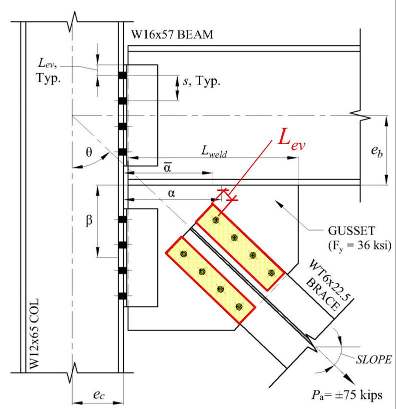
WT Bolt Tear Out (Center):

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

$= 1.2(3 \text{ in.} - {}^{13}/_{16} \text{ in.})(0.575 \text{ in.})$

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

$= 49.1$ kips



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

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

Plate Bolt Bearing:

$R_{brg(PL)}/\Omega = 2.4 d_b t_p F_u/\Omega$

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

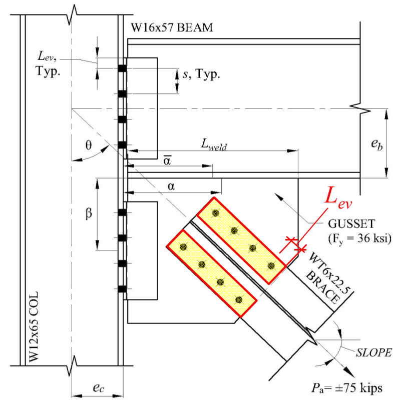
$= 19.6 \text{ kips}$

Plate Bolt Tear Out (Edge):

$R_{tear(PL\_E)}/\Omega = 1.2[L_{ev} - (d_{h\_w}/2)]t_p F_u/\Omega$

$= 1.2[1.5 \text{ in.} - (^{13}/_{16} \text{ in.})/2] \times (0.375 \text{ in.})(58 \text{ ksi})/2$

$= 14.3 \text{ kips}$



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

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

Plate Bolt Tear Out (Center):

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

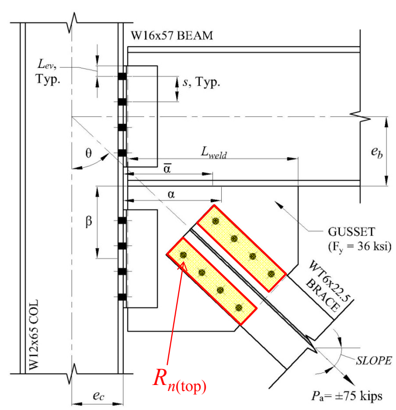
$= 1.2(3 \text{ in.} - {}^{13}/_{16} \text{ in.})(0.375 \text{ in.}) \times (58 \text{ ksi})/2$

$= 28.6 \text{ kips}$

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

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

$= 11.9 \text{ kips}$

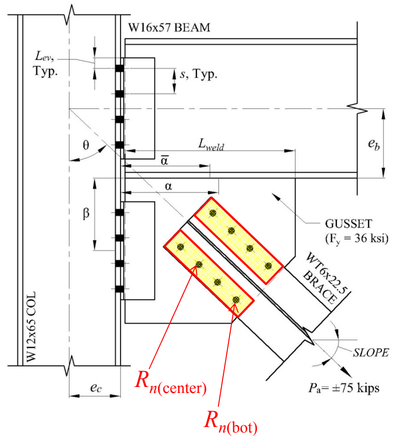


Example VB Corner Connection, I
Brace-to-Gusset (cont.)

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

$$R_{n(\text{center})}/\Omega = \min\{r_v/\Omega,\ R_{brg(\text{WT})}/\Omega,\ R_{tear(\text{WT\_C})}/\Omega,\ R_{brg(\text{PL})}/\Omega,\ R_{tear(\text{PL\_C})}/\Omega\}$$
$$= \min\{11.9 \text{ kips},\ 33.6 \text{ kips},\ 49.1 \text{ kips},\ 19.6 \text{ kips},\ 28.6 \text{ kips}\}$$
$$= 11.9 \text{ kips}$$

$$R_{n(\text{bot})}/\Omega = \min\{r_v/\Omega,\ R_{brg(\text{WT})}/\Omega,\ R_{tear(\text{WT\_C})}/\Omega,\ R_{brg(\text{PL})}/\Omega,\ R_{tear(\text{PL\_E})}/\Omega\}$$
$$= \min\{11.9 \text{ kips},\ 33.6 \text{ kips},\ 49.1 \text{ kips},\ 19.6 \text{ kips},\ 14.3 \text{ kips}\}$$
$$= 11.9 \text{ kips}$$

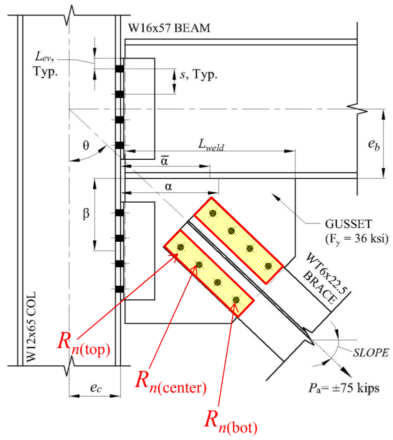


Example VB Corner Connection, I
Brace-to-Gusset (cont.)

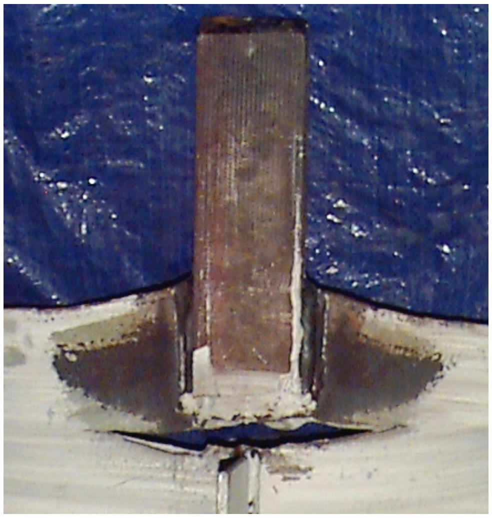
Brace-to-Gusset – Bolt Bearing (cont.):

Total Bolt Strength

$$R_n/\Omega = [R_{n\ (\text{top})}/\Omega + (R_{n(\text{center})}/\Omega)(n-2) + R_{n(\text{bot})}/\Omega](2 \text{ lines})$$
$$= [11.9 \text{ kips} + (11.9 \text{ kips})(4-2) + 11.9 \text{ kips}](2)$$
$$= 95.2 \text{ kips} > 75 \text{ kips } \textbf{o.k.}$$



Whitmore Section



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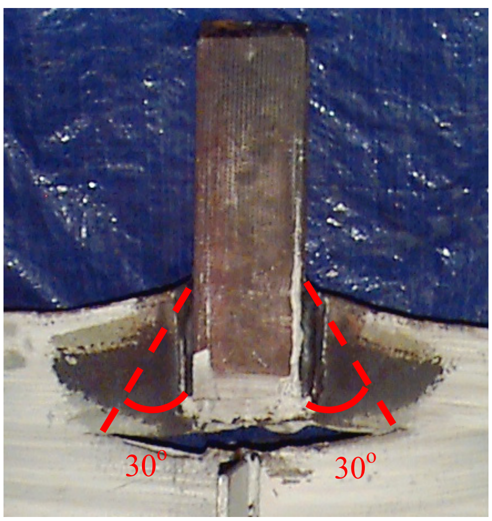
Whitmore Section



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

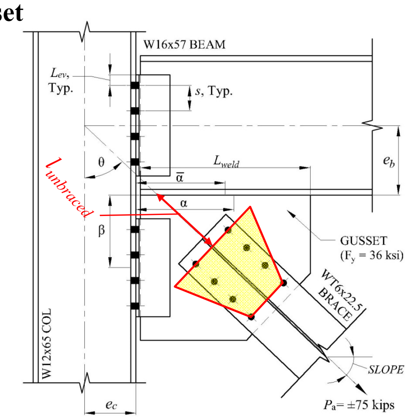
Brace-to-Gusset – Whitmore Section of Gusset Plate:

Whitmore Compression will control over Whitmore Tension by inspection.

$l_{unbraced}$ = 8.81 in. max

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

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



Reference AISC *Manual* Part 9 for Whitmore Section

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

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

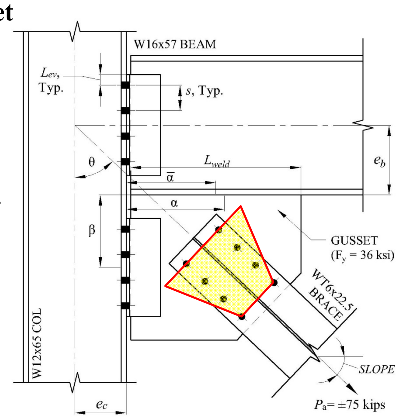
$l_{w(\text{gusset})} = l_w - l_{w(\text{beam})}$
$\quad = 15.9\text{ in.} - 0\text{ in.} = 15.9\text{ in.}$

K = 0.5 from AISC's *Design Examples* (v14.1 is web-based and free at www.aisc.org), Example II.C-2

$Kl_{unbraced}/r = [0.5(8.81\text{ in.})\sqrt{(12)}]/0.375\text{ in.}$
$\quad = 40.7$

AISC *Manual* Table 4-22, $F_{cr(\text{Gr.36})}/\Omega = 19.7$ ksi

AISC *Manual* Table 4-22, $F_{cr(\text{Gr.50})}/\Omega = 26.5$ ksi



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

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

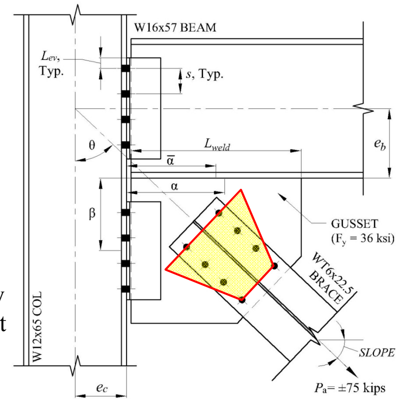
AISC *Manual*

Table 4-22 (continued)
Available Critical Stress for Compression Members

	F_y = 35 ksi			F_y = 36 ksi			F_y = 42 ksi			F_y = 46 ksi			F_y = 50 ksi	
$\frac{KL}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{KL}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{KL}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{KL}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD	$\frac{KL}{r}$	F_{cr}/Ω_c ksi ASD	$\phi_c F_{cr}$ ksi LRFD
41	19.2	28.9	41	19.7	29.7	41	22.7	34.1	41	24.6	37.0	41	26.5	39.8
42	19.2	28.8	42	19.6	29.5	42	22.6	33.9	42	24.5	36.8	42	26.3	39.5
43	19.1	28.7	43	19.6	29.4	43	22.5	33.7	43	24.3	36.6	43	26.2	39.3

From AISC *Design Examples* Example IIC-2: 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.)

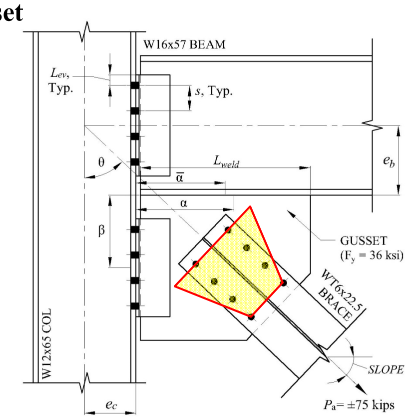


Example VB Corner Connection, I Brace-to-Gusset (cont.)

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

$$\frac{P_n}{\Omega_c} = \frac{F_{cr(\text{gusset})}}{\Omega_c} A_{w(\text{gusset})} + \frac{F_{cr(\text{beam})}}{\Omega_c} A_{w(\text{beam})}$$
$$= (19.7 \text{ ksi})(0.375 \text{ in.})(15.9 \text{ in.}) + 0$$
$$= 83 \text{ kips} \geq 75 \text{ kips } \textbf{o.k.}$$

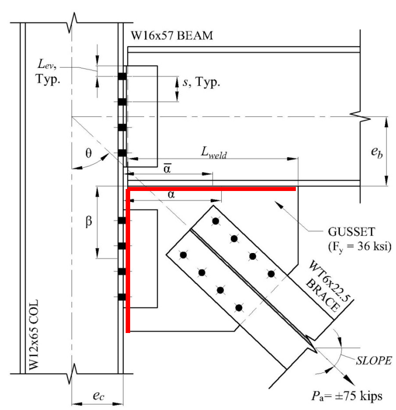
Note: for Whitmore tension, use F_y / Ω in lieu of F_{cr} / Ω



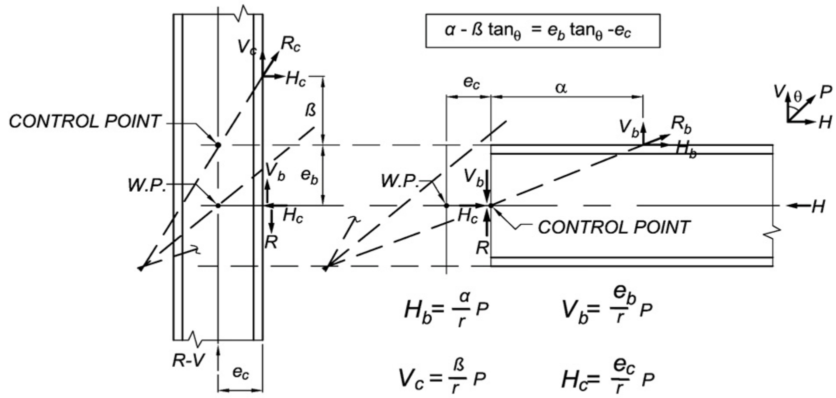
Example VB Corner Connection, I Gusset Forces

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.)

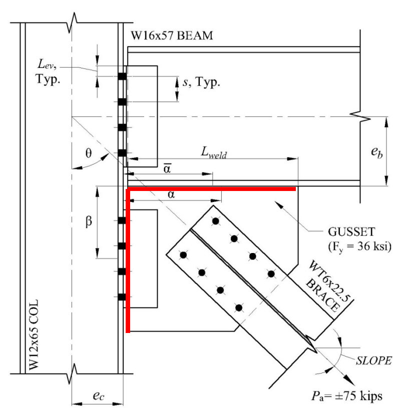
Determine Gusset Forces using UFM:

$\tan\theta = 1.05$

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

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

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



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

Determine Gusset Forces using UFM (cont.):

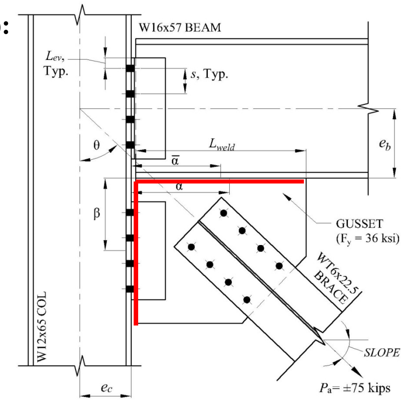
$\bar{\beta}$ = distance to center of gusset-to-column connections

From geometric layout, H_{gusset} = 16.875 in.

Place bolts in approximate center of gusset, $\bar{\beta}$ = 8.5 in.

Let $\beta = \bar{\beta}$

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



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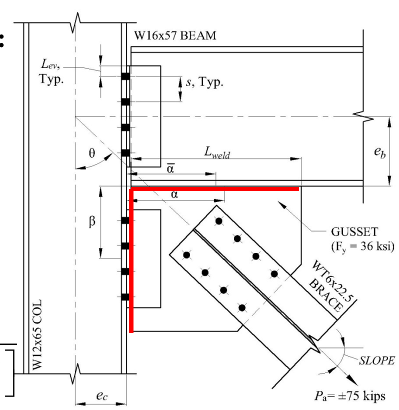
Example VB Corner Connection, I Gusset Forces (cont.)

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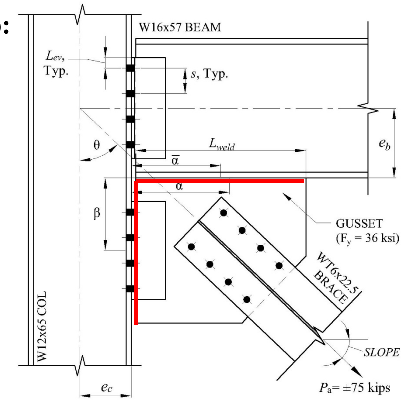
Example VB Corner Connection, I Gusset Forces (cont.)

Determine Gusset Forces using UFM (cont.):

$V_b = P_a(e_b/r)$
$= (75 \text{ kips})(8.2 \text{ in.}/24.2 \text{ in.})$
$= 25.4 \text{ kips}$

$H_b = P_a(\alpha/r)$
$= (75 \text{ kips})(11.5 \text{ in.}/24.2 \text{ in.})$
$= 35.6 \text{ kips}$



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

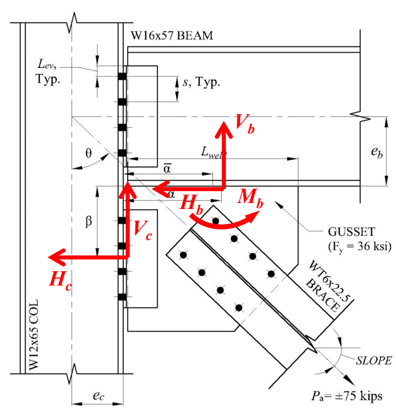
$M_b = V_b(\alpha - \bar{\alpha})$
$= 25.4 \text{ kips}(11.5 \text{ in.} - 10.5 \text{ in.})$
$= 25.4 \text{ kip-in.}$

$V_c = P_a(\beta/r)$
$= 75 \text{ kips}(8.5 \text{ in.}/24.2 \text{ in.})$
$= 26.3 \text{ kips}$

$H_c = P_a(e_c/r)$
$= 75 \text{ kips}(6.05 \text{ in.}/24.2 \text{ in.})$
$= 18.8 \text{ kips}$

(Note: $H_C = 0$ kips for connections to column web, except Special Case IV)

$M_c = 0$ since $\beta = \bar{\beta}$

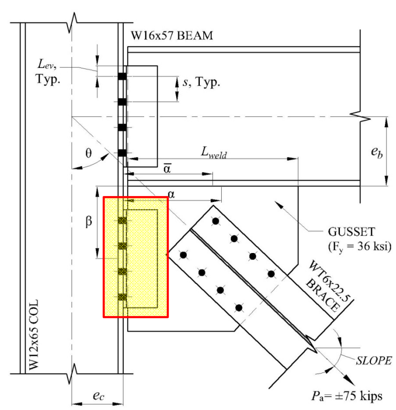


Example VB Corner Connection, I
Gusset-to-Column

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



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.**

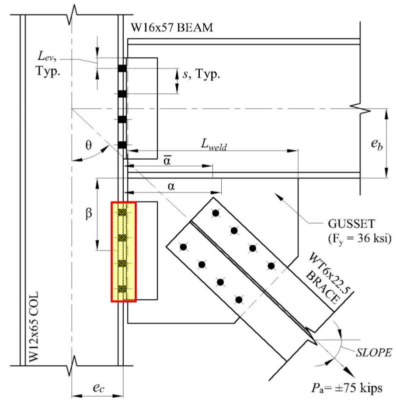
Check Shear and Tension:

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

$= 3.29 \text{ kips}$

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

$= 2.35 \text{ kips}$



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

Gusset-to-Column – Bolts (cont.):

Check Shear and Tension (cont.):

$$A_b = \frac{\pi d_b^2}{4}$$

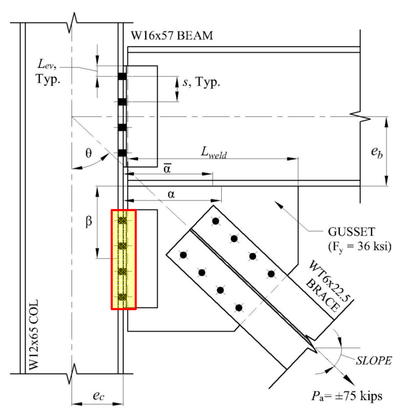
$$= \frac{3.14(0.75 \text{ in.})^2}{4}$$

$$= 0.442 \text{ in.}^2$$

$f_{rv} = V_{bolt}/A_b$

$= 3.29 \text{ kips}/0.442 \text{ in.}^2$

$= 7.44 \text{ ksi}$



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

Gusset-to-Column – Bolts (cont.):

Check Shear and Tension (cont.):

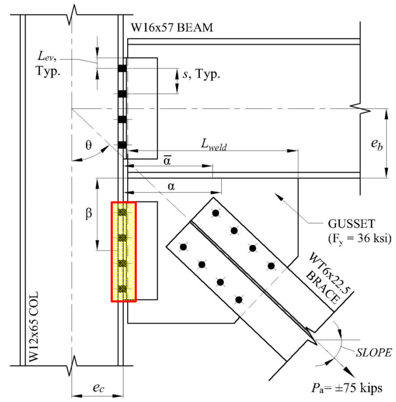
$F'_{nt} = 1.3F_{nt} - \Omega F_{nt} f_{rv}/F_{nv} \le F_{nt}$ (*Spec* J3-3a)

$= 1.3(90\text{ ksi}) - 2(90\text{ ksi})(7.44\text{ ksi})/(54\text{ ksi})$

$= 92.2\text{ ksi} > 90\text{ ksi, use } 90\text{ ksi}$

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

Description of Fasteners	Nominal Tensile Strength, F_{nt}, ksi (MPa)[a]	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)



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

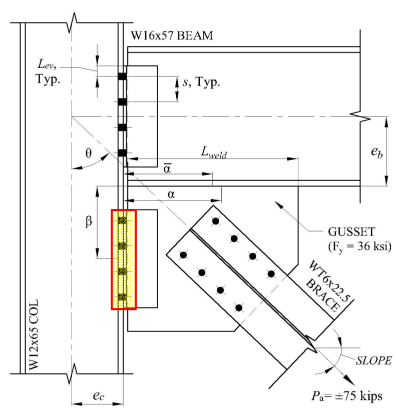
Gusset-to-Column – Bolts (cont.):

Check Shear and Tension (cont.):

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

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

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



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

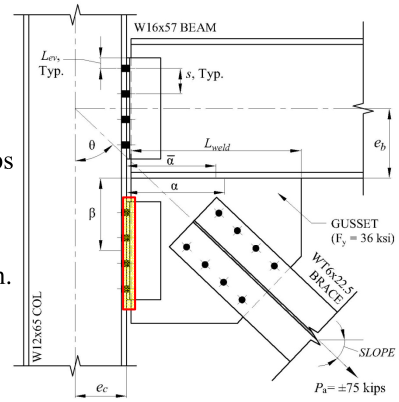
Gusset-to-Column – Prying of Angles:

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:

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

$$b = \text{gage}/2 - t_p/2 - t_a/2$$
$$= 5.5 \text{ in.}/2 - 0.375 \text{ in.}/2 - 0.375/2 = 2.38 \text{ in.}$$

$$b' = b - d_b/2 =$$
$$= 2.38 \text{ in.} - 0.75 \text{ in.}/2 = 2 \text{ in.}$$



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The lecture will skip to slide 75.

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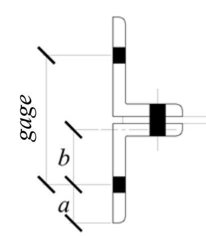
Example VB Corner Connection, I Gusset-to-Column (cont.)

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

$$a = [2(\text{angle OSL}) + t_p - \text{gage}]/2$$
$$= [2(4 \text{ in.}) + 0.375 \text{ in.} - 5.5 \text{ in.}]/2$$
$$= 1.44 \text{ in.}$$

$$a' = a + d_b/2 \leq 1.25b + d_b/2$$
$$= 1.44 \text{ in.} + 0.75 \text{ in.}/2$$
$$\leq 1.25(2.38 \text{ in.}) + 0.75 \text{ in.}/2$$
$$= 1.82 \text{ in.} \leq 3.35 \text{ in.}$$
$$= 1.82 \text{ in.}$$

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



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

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

$p = L/n = 11.5 \text{ in.}/4$

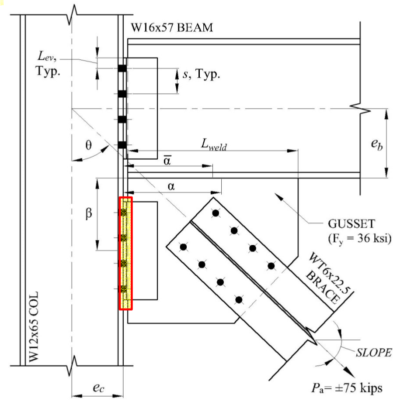
$= 2.88 \text{ in.}$ (tributary area per bolt)

$\delta = 1 - d'/p = 1 - 0.8125 \text{ in.}/2.88 \text{ in.}$

$= 0.717$

$B = r_t/\Omega$

$= 19.9$ kips (bolt available tensile strength from previously)



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

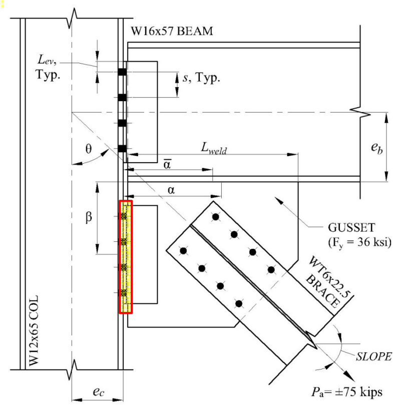
Gusset-to-Column – Prying of Angles (cont.):

$$t_c = \sqrt{\frac{\Omega 4 B b'}{p F_u}} \quad \text{(AISC \textit{Manual} Eq. 9-30a)}$$

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

$= 1.26$ in.

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



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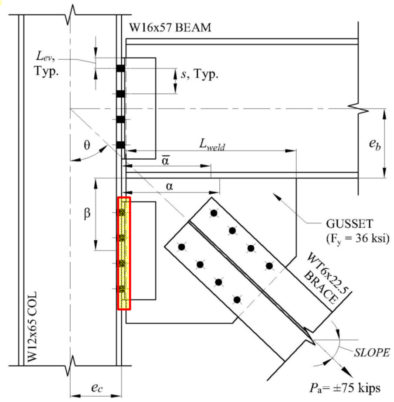
Example VB Corner Connection, I Gusset-to-Column (cont.)

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

$$\alpha' = \frac{1}{\delta(1+\rho)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right] \quad \text{(AISC } \textit{Manual} \text{ Eq. 9-35)}$$

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

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



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

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

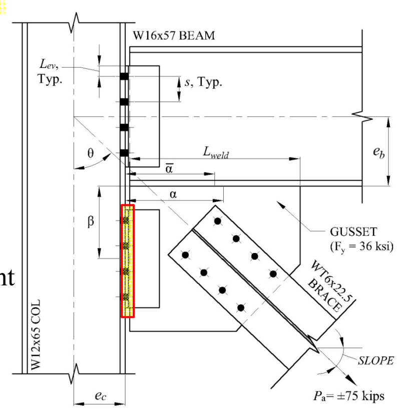
Note:

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

$$Q = 1 \quad \text{(AISC } \textit{Manual} \text{ Eq. 9-32)}$$

When $0 \leq \alpha' < 1$, 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,

$$Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta\alpha') \quad \text{(AISC } \textit{Manual} \text{ Eq. 9-33)}$$



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

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

For the example,

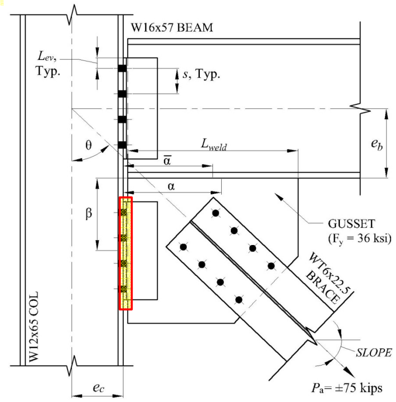
$\alpha' = 6.83 > 1$

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

$$Q = \left(\frac{t}{t_c}\right)^2 (1+\delta) \qquad \text{(AISC } \textit{Manual} \text{ Eq. 9-34)}$$

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

$$= 0.152$$



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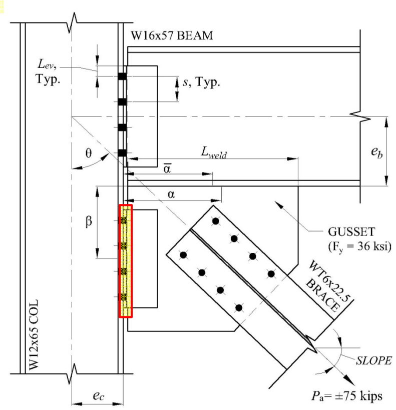
Example VB Corner Connection, I Gusset-to-Column (cont.)

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

The available tensile strength including the effects of prying action, T_{avail}, is:

$$T_{avail} = BQ = (19.9 \text{ kips})(0.152)$$

$$= 3.02 \text{ kips} > T_{bolt} = 2.35 \text{ kips } \textbf{o.k.}$$



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

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

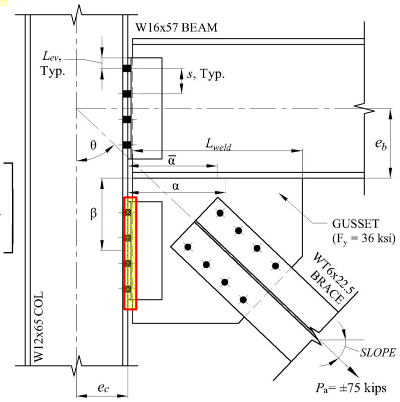
If prying force, q, needs to be determined:

$$\alpha = \left(\frac{1}{\delta}\right)\left[\frac{T}{B}\left(\frac{t_c}{t}\right)^2 - 1\right] \quad \text{(AISC } \textit{Manual} \text{ Eq. 9-29)}$$

$$= \left(\frac{1}{0.717}\right)\left[\left(\frac{2.35\text{ kips}}{19.9\text{ kips}}\right)\left(\frac{1.26\text{ in.}}{0.375\text{ in.}}\right)^2 - 1\right]$$

$$= 0.465 > 0 \text{ and } < 1.0$$

Note: When $\alpha > 1$, the connection is not adequate.



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

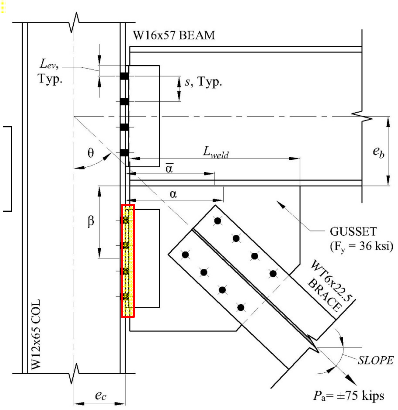
Gusset-to-Column – Prying of Angles (cont.):

$$q = B\left[\delta\alpha\rho\left(\frac{t}{t_c}\right)^2\right] \quad \text{(AISC } \textit{Manual} \text{ Eq. 9-28)}$$

$$= 19.9\text{ kips}\left[(0.717)(0.465)(1.10)\left(\frac{0.375\text{ in.}}{1.26\text{ in.}}\right)^2\right]$$

$$= 0.637\text{ kips}$$

$$B_c = T_{bolt} + q = 2.35\text{ kips} + 0.637\text{ kips}$$
$$= 2.99\text{ kips} \le B = 19.9\text{ kips } \textbf{o.k.}$$



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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:

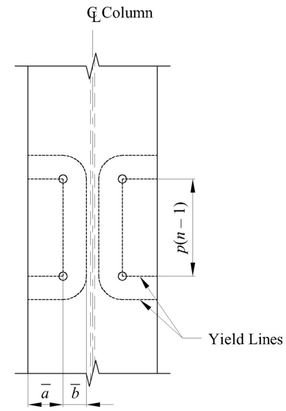
Determine effective flange width

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

(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)

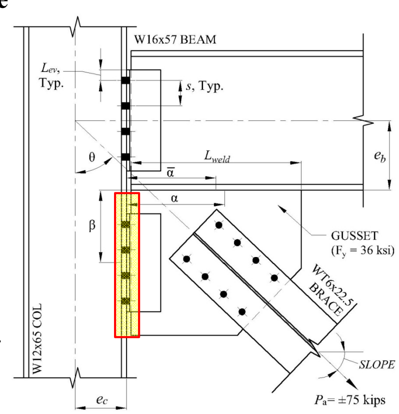
Example VB Corner Connection, I Gusset-to-Column (cont.)

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

$$\bar{b} = (\text{gage} - t_{wc})/2$$
$$= (5.5\text{ in.} - 0.39\text{ in.})/2 = 2.56\text{ in.}$$
$$\bar{a} = (b_{fc} - \text{gage})/2$$
$$= (12\text{ in.} - 5.5\text{ in.})/2 = 3.25\text{ in.}$$

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

Note: use 3" max if at top of column



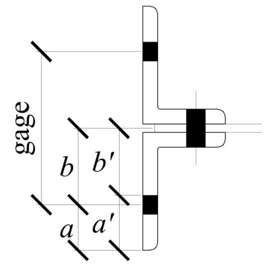
Example VB Corner Connection, I Gusset-to-Column (cont.)

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

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

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

$b' = b - d_b/2 =$
$= 2.55 \text{ in.} - 0.75 \text{ in.}/2 = 2.18 \text{ in.}$



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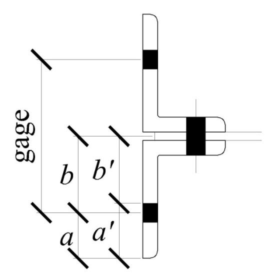
Example VB Corner Connection, I Gusset-to-Column (cont.)

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

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

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

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



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

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

$p = p_{eff} = 5.88$ in.

$\delta = 1 - d'/p = 1 - 0.8125$ in./5.88 in.

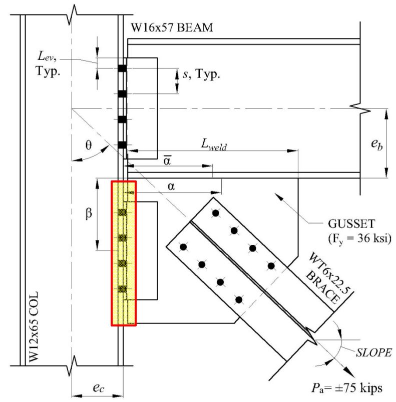
$= 0.862$

$$t_c = \sqrt{\frac{\Omega 4Bb'}{pF_u}} \quad \text{(AISC } \textit{Manual} \text{ Eq. 9-30a)}$$

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

$= 0.871$ in. $> t_{fc} = 0.605$ in.

→ check available strength of bolt including the effects of prying action



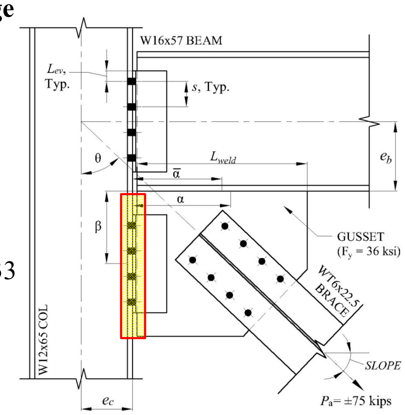
Example VB Corner Connection, I Gusset-to-Column (cont.)

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

$$\alpha' = \frac{1}{\delta(1+\rho)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right] \quad \text{(AISC } \textit{Manual} \text{ Eq. 9-35)}$$

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

$= 0.565 > 0$ and < 1.0 → *Manual* Eq. 9-33 applies



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

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

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

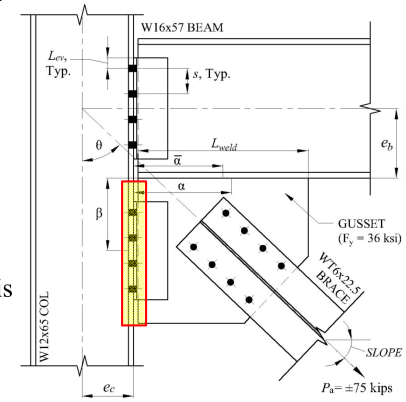
$$= \left(\frac{0.605\text{ in.}}{0.871\text{ in.}}\right)^2 \left[1+(0.862)(0.565)\right]$$

$$= 0.717$$

The available strength of bolt including prying is

$T_{avail} = BQ = (19.9\text{ kips})(0.717)$

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



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

Gusset-to-Column – Bolt Bearing:

AISC *Specification* Eq. J3-6a

AISC Bolt Shear:

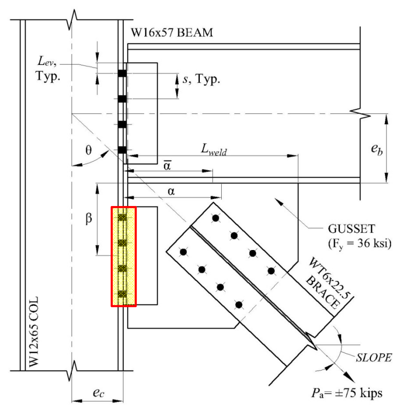
$r_v/\Omega = 11.9\text{ kips}$

Column Bolt Bearing:

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

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

$= 35.4\text{ kips}$



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The lecture will skip to slide 87.

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

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

Column Bolt Tear Out (Center):

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

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

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

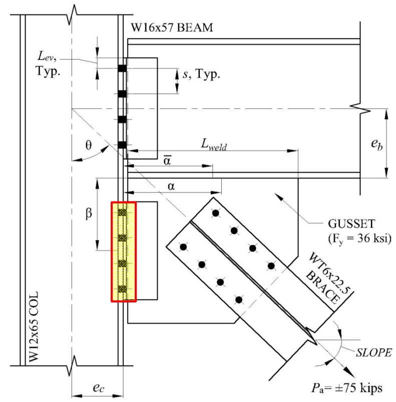
$= 51.6 \text{ kips}$

Angle Bolt Bearing:

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

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

$= 19.6 \text{ kips}$



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

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

Angle Bolt Tear Out (Edge):

$R_{tear(L\_E)}/\Omega = 1.2(L_{ev} - d_{h\_w}/2)t_a F_u/\Omega$

$= 1.2\,(1.25 \text{ in.} - {}^{13}/_{16} \text{ in.}/2)$

$\times(0.375 \text{ in.})(58 \text{ ksi})/2$

$= 11.0 \text{ kips}$

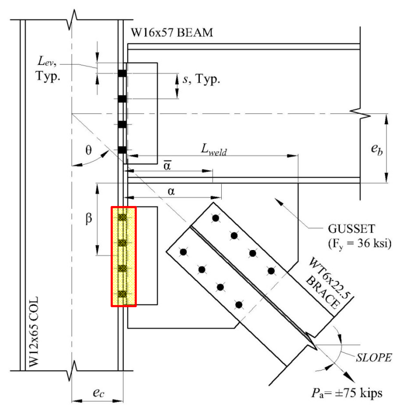
Angle Bolt Tear Out (Center):

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

$= 1.2\,(3 \text{ in.} - {}^{13}/_{16} \text{ in.})(0.375 \text{ in.})$

$\times(58 \text{ ksi})/2$

$= 28.6 \text{ kips}$



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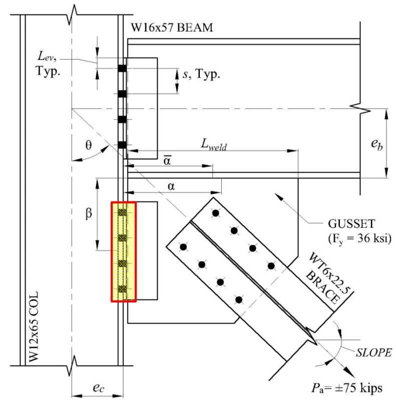
Example VB Corner Connection, I Gusset-to-Column (cont.)

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

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

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



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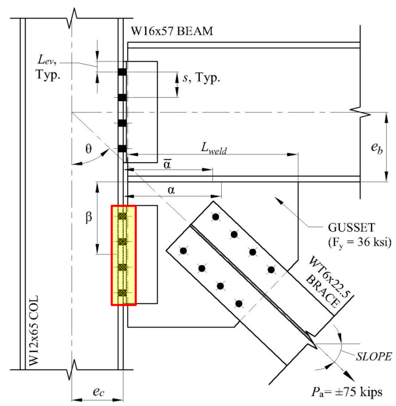
Example VB Corner Connection, I Gusset-to-Column (cont.)

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.)

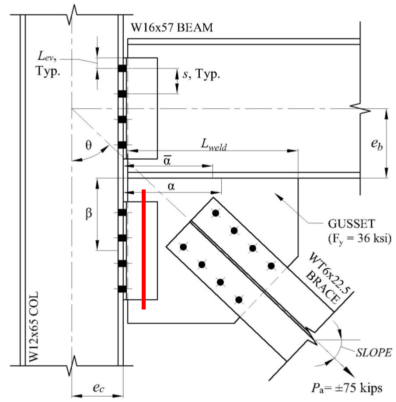
Gusset-to-Column – Angle Gross Shear:

AISC *Specification* Eq. J4-1

$$\frac{R_n}{\Omega} = \frac{0.6F_y A_g}{\Omega}$$

$$= \frac{0.6(36\text{ ksi})(11.5\text{ in.})(0.375\text{ in.})}{1.5}$$

$$= 124\text{ kips} > V_c = 26.3\text{ kips } \textbf{o.k.}$$



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

Gusset-to-Column – Angle Net Shear:

AISC *Specification* Eq. J4-4

$$L_{nv} = L_{eff} - n(d_{h,\text{width}} + {}^{1}/_{16}\text{ in.})$$

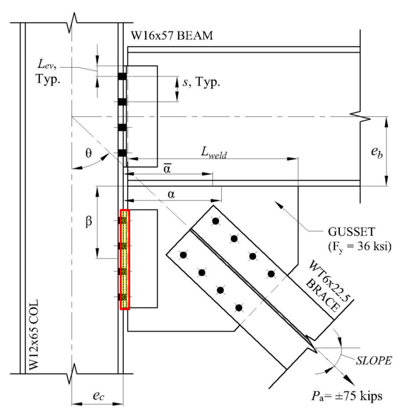
$$= 11.5\text{ in.} - 4({}^{13}/_{16}\text{ in.} + {}^{1}/_{16}\text{ in.})$$

$$= 8\text{ in.}$$

$$\frac{R_n}{\Omega} = \frac{0.6F_u A_{nv}}{\Omega}$$

$$= \frac{0.6(58\text{ ksi})(2)(8\text{ in.})(0.375\text{ in.})}{2}$$

$$= 104\text{ kips} > V_c = 26.3\text{ kips } \textbf{o.k.}$$



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

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

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

$\Omega = 2.00$

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

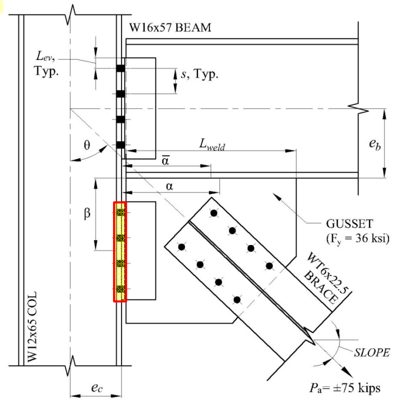
$\leq 0.6F_yA_{gv} + U_{bs}F_uA_{nt}$

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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The lecture will skip to slide 94.

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

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

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

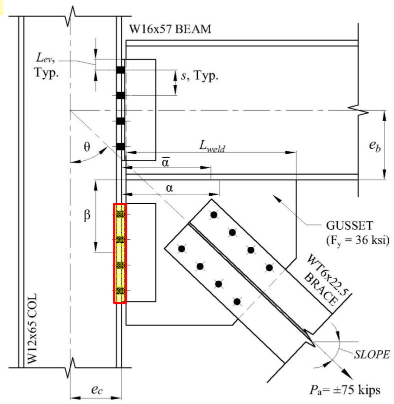
$= 1.25 \text{ in.} + (3 \text{ in.})(4 - 1) = 10.3 \text{ in.}$

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

$= 1.25 \text{ in.} + (3 \text{ in.})(4 - 1)$

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

$= 7.19 \text{ in.}$



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

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

$L_{gt} = L_{eh} = a$ from prying calcs
$= [2(\text{angle OSL}) + t_p - \text{gage}]/2$
$= [2(4\text{ in.}) + 0.375\text{ in.} - 5.5\text{ in.}]/2$
$= 1.44\text{ in.}$

$L_{nt} = L_{eh} - 0.5(\text{hole length} + {}^1/_{16}\text{ in.})$
$= 1.44\text{ in.} - 0.5(1 + {}^1/_{16}\text{ in.})$
$= 0.91\text{ in.}$

AISC *Specification* **TABLE J3.3**
Nominal Hole Dimensions, in.

Bolt Diameter, in.	Hole Dimensions			
	Standard (Dia.)	Oversize (Dia.)	Short-Slot (Width × Length)	Long-Slot (Width × Length)
$^1/_2$	$^9/_{16}$	$^5/_8$	$^9/_{16} \times {}^{11}/_{16}$	$^9/_{16} \times 1^1/_4$
$^5/_8$	$^{11}/_{16}$	$^{13}/_{16}$	$^{11}/_{16} \times {}^7/_8$	$^{11}/_{16} \times 1^9/_{16}$
$^3/_4$	$^{13}/_{16}$	$^{15}/_{16}$	$^{13}/_{16} \times 1$	$^{13}/_{16} \times 1^7/_8$
$^7/_8$	$^{15}/_{16}$	$1^1/_{16}$	$^{15}/_{16} \times 1^1/_8$	$^{15}/_{16} \times 2^3/_{16}$
1	$1^1/_{16}$	$1^1/_4$	$1^1/_{16} \times 1^5/_{16}$	$1^1/_{16} \times 2½$
$\geq 1^1/_8$	$d + {}^1/_{16}$	$d + {}^5/_{16}$	$(d + {}^1/_{16}) \times (d + {}^3/_8)$	$(d + {}^1/_{16}) \times (2.5 \times d)$

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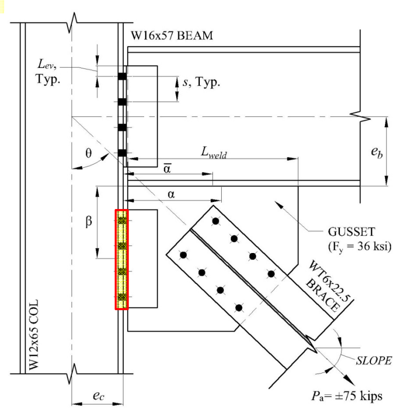
Example VB Corner Connection, I Gusset-to-Column (cont.)

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

$A_{gv} = 2L_{gv}t_a = 2(10.3\text{ in.})(0.375\text{ in.})$
$= 7.73\text{ in.}^2$

$A_{nv} = 2L_{nv}t_a = 2(7.19\text{ in.})(0.375\text{ in.})$
$= 5.39\text{ in.}^2$

$A_{nt} = 2L_{nt}t_a = 2(0.91\text{ in.})(0.375\text{ in.})$
$= 0.68\text{ in.}^2$



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

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

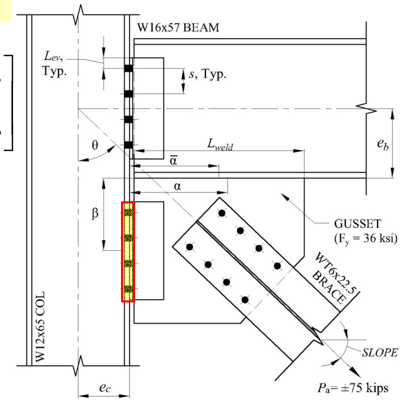
$$R_n = \min\begin{bmatrix} 0.6(58\text{ ksi})(5.39\text{ in.}^2) = 187.57\text{ kips} \\ 0.6(36\text{ ksi})(7.73\text{ in.}^2) = 166.97\text{ kips} \end{bmatrix}$$

$$+ [1(58\text{ ksi})(0.68\text{ in.}^2)]$$

$$= 166.97\text{ kips} + 39.44\text{ kips} = 206.41\text{ kips}$$

$$R_n/\Omega = (206.41\text{ kips})/2$$

$$= 103\text{ kips} > V_c = 26.3\text{ kips } \textbf{o.k.}$$



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

Gusset-to-Column – Block Shear of Gusset:

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

$\Omega = 2.00$

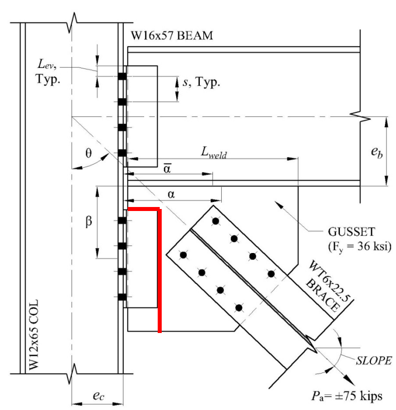
$$R_n = 0.6F_uA_{nv} + U_{bs}F_uA_{nt}$$
$$\leq 0.6F_yA_{gv} + U_{bs}F_uA_{nt}$$

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



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

Gusset-to-Column – Block Shear of Gusset:

Gusset is welded

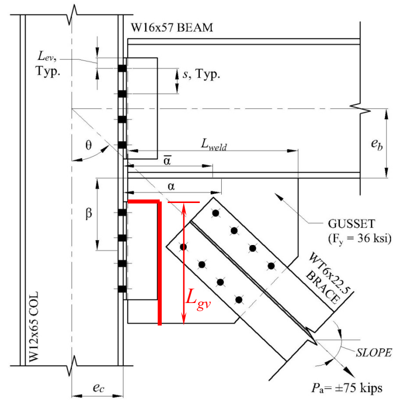
For Shear check:

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.



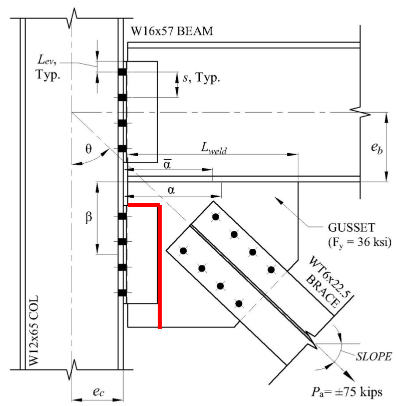
Example VB Corner Connection, I Gusset-to-Column (cont.)

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

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

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

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



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

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

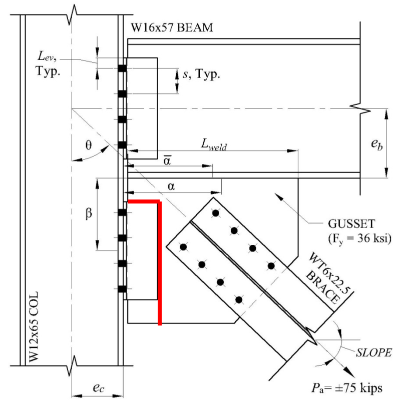
$$R_n = \min \begin{bmatrix} 0.6(58\text{ ksi})(5.34\text{ in.}^2) = 185.83\text{ kips} \\ 0.6(36\text{ ksi})(5.34\text{ in.}^2) = 115.34\text{ kips} \end{bmatrix}$$

$$+ [1(58\text{ ksi})(1.31\text{ in.}^2)]$$

$$= 115.34\text{ kips} + 75.98\text{ kips} = 191.32\text{ kips}$$

$R_n/\Omega = (191.32\text{ kips})/2$

$= 96\text{ kips} > V_c = 26.3\text{ kips}$ **o.k.**



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

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

Gusset is welded

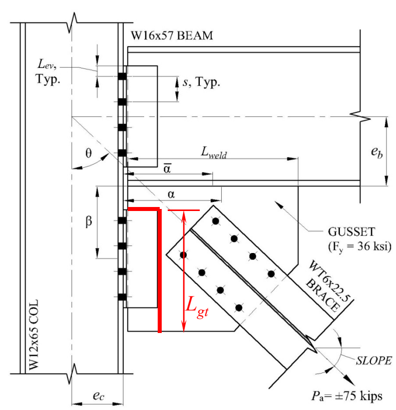
For Axial check:

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

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

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

$L_{nt} = 14.25$ in.



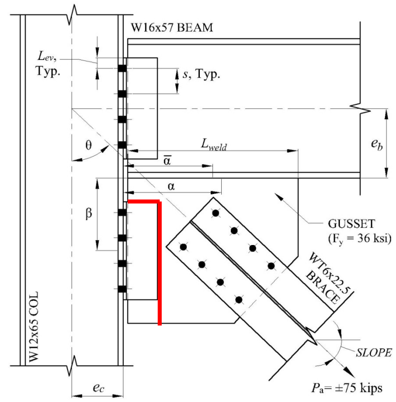
Example VB Corner Connection, I Gusset-to-Column (cont.)

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.):

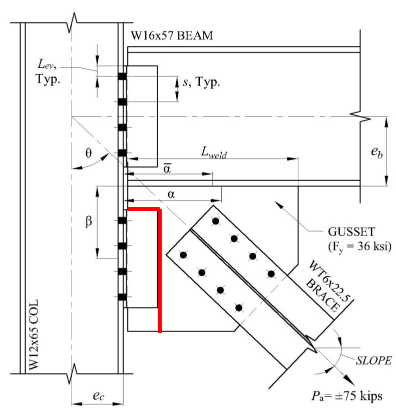
$$R_n = \min\begin{bmatrix} 0.6(58 \text{ ksi})(1.31 \text{ in.}^2) = 45.59 \text{ kips} \\ 0.6(36 \text{ ksi})(1.31 \text{ in.}^2) = 28.30 \text{ kips} \end{bmatrix}$$

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

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

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

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



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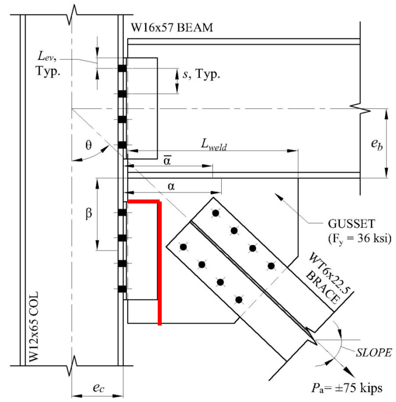
Example VB Corner Connection, I
Gusset-to-Column (cont.)

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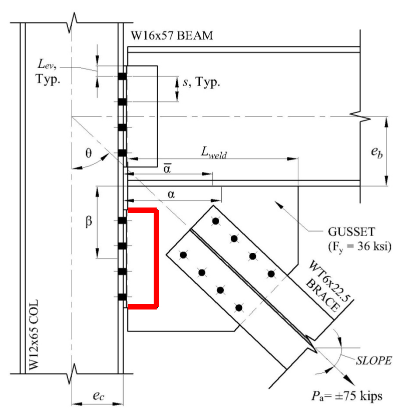
Example VB Corner Connection, I
Gusset-to-Column (cont.)

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 in./11.5 in. = 0.304

x = 0.057

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

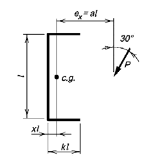
= [4 in. – 0.057(11.5 in.)]/11.5 in.

= 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.)

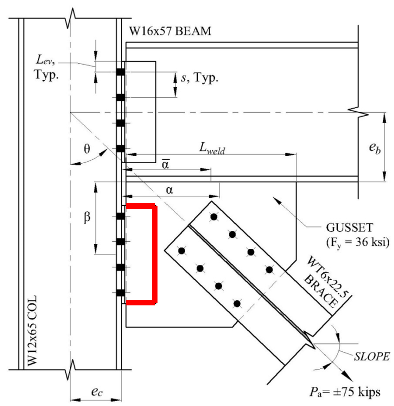
Gusset-to-Column – Size Weld (cont.):

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.):

$D_{min} = P_c\Omega/(2CC_1L)$

$= 32.3 \text{ kips}(2)/((2)(2.96)(1.0)(11.5 \text{ in.}))$

$= 0.949 < 4$ (16th of an inch)

¼” fillet weld **o.k.**

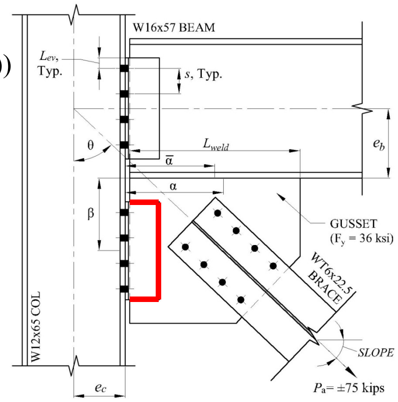
$t_{min} = 2(0.928D_{min})/[0.6(F_u/\Omega)]$

$= 2(0.928)(0.949)/[0.6(58 \text{ ksi}/2)]$

$= 0.101 \text{ in.} > t_p = 0.375$ in. **o.k.**

Note, the above is same as *Manual* Eq 9-3:

$t_{min} = 6.19D/F_u$



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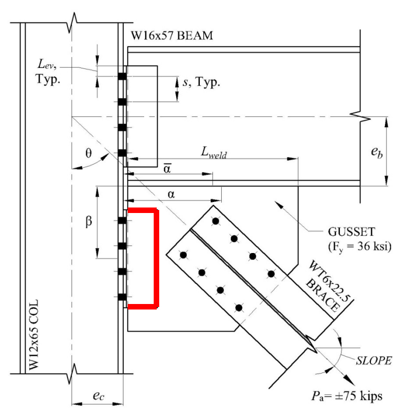
Example VB Corner Connection, I Gusset-to-Column (cont.)

Gusset-to-Column – Size Weld (cont.):

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

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



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Example VB Corner Connection, I
Gusset-to-Beam

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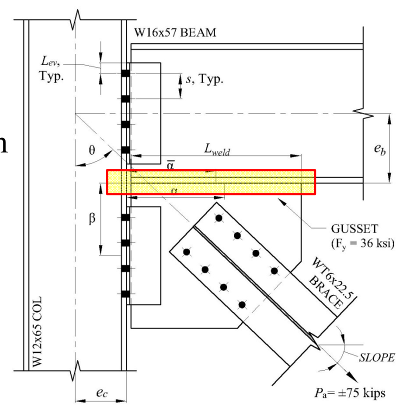
Example VB Corner Connection, I
Gusset-to-Beam

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



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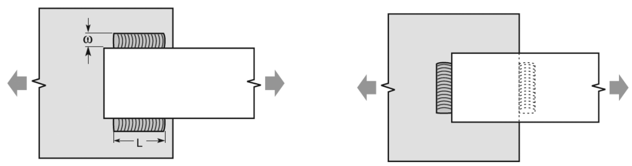
Design of Fillet Welds

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

$$R_n = 0.6F_{EXX}(1.0 + 0.5\sin^{1.5}\theta)A_w$$

F_w = nominal unit stress, ksi
F_{EXX} = electrode classification number, i.e., minimum specified tensile strength, ksi
θ = angle of loading measured from the weld longitudinal axis, degrees



$\theta = 0^o$ $\theta = 90^o$

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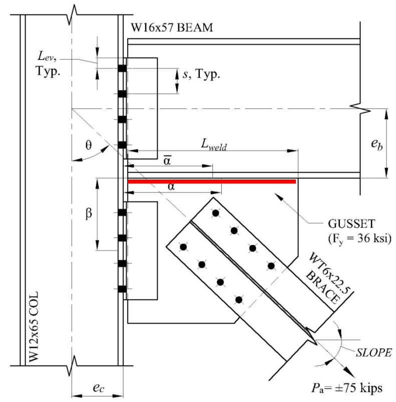
Example VB Corner Connection, I Gusset-to-Beam (cont.)

Gusset-to-Beam – Size Weld:

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

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

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

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

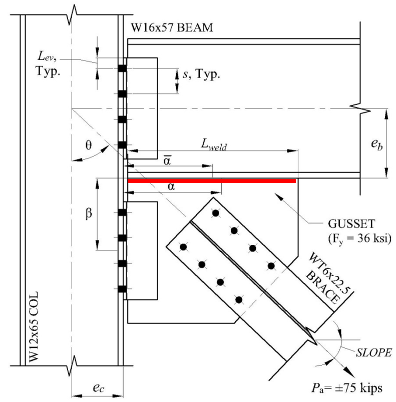
$f_w = 0.60(F_{EXX}/\Omega)(0.707D/16)(1.0 + 0.50 \sin^{1.5}\theta)$
$= 0.928D\,(1.0 + 0.50 \sin^{1.5} 40.6°)$
$= 0.928D(1.26)$

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

Gusset-to-Beam – Size Weld (cont.):

$D_{min} = \max(f_{peak}, 1.25f_{avg}) / [f_w(2 \text{ sides})]$
$= \max(2.34 \text{ kips/in.}, 2.75 \text{ kips/in.}) / [0.928(1.26)(2)]$
$= 1.18 > 4$ (16th on an inch) **o.k.**
(1/4" fillet provided)

Also, per AISC *Specification* Table 2.4, minimum size fillet weld for 3/8" gusset = 3/16" < 1/4" **o.k.**



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

Gusset-to-Beam – Check Gusset:

Shear:

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

$= (1.78 \text{ kips/in.})/[0.6(36 \text{ ksi}/1.5)]$

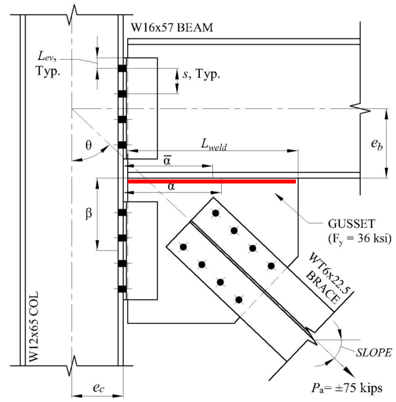
$= 0.124 \text{ in.} < t_p = 0.375 \text{ in.}$ **o.k.**

Tension:

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

$= (1.27 \text{ k/in.} + 0.254 \text{ k/in.}) / (36 \text{ ksi}/1.67)$

$= 0.071 \text{ in.} < t_p = 0.375 \text{ in.}$ **o.k.**



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

Gusset-to-Beam – Beam Web Yielding:

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

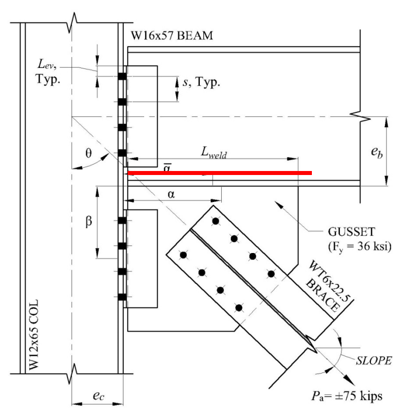
$= 2[25.4 \text{ k-in.}/(20 \text{ in.}/2)] + 25.4 \text{ kips}$

$= 30.5$ 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)

$$\frac{R_n}{\Omega} = \frac{0.8t_w^{\,2}\left[1+3\left(\frac{L_{weld}}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}}}{\Omega}$$

$$= \frac{\left[\frac{0.8(0.43\text{ in.})^2}{2}\left[1+3\left(\frac{20\text{ in.}}{16.4\text{ in.}}\right)\left(\frac{0.43\text{ in.}}{0.715\text{ in.}}\right)^{1.5}\right] \times\sqrt{\frac{(29{,}000\text{ ksi})(50\text{ ksi})(0.715\text{ in.})}{0.43\text{ in.}}}\right]}{2} = 311\text{ kips} > 30.5\text{ kips}\ \textbf{o.k.}$$

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Example VB Corner Connection, I Beam-to-Column

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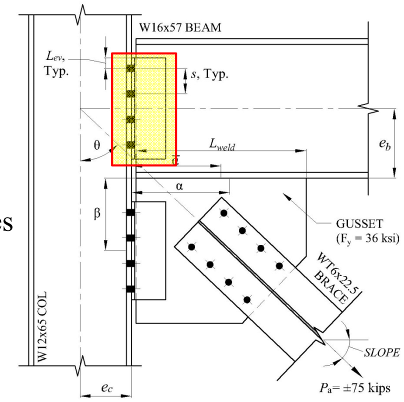
Example VB Corner Connection, I Beam-to-Column

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.)

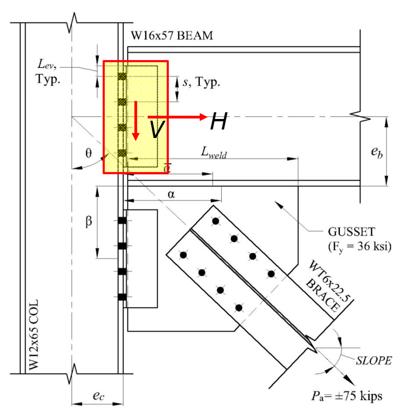
Beam-to-Column – Forces:

$V_{b(top)}$ = 0 kips (no upper brace)

$V_{b(bot)}$ = 25.4 kips (from lower brace)

PTF = Transfer force from adjacent beam into the braced bay
= 0 kips (no transfer force)

V_g = 41.9 kips (50% UDL for span of 25 ft)
(UDL projects: not preferred)



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

W16

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W-Shapes

F_y = 50 ksi

Shape	W16×									
	89		77		67		57		50	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 7							282	423	248	372
8							262	394	230	345
9	353	529	300	450			233	350	204	307
10	349	525	299	450	258	386	210	315	184	276
11	318	477	272	409	236	355	191	286	167	251
12	291	438	250	375	216	325	175	263	153	230
13	269	404	230	346	200	300	161	242	141	212
14	250	375	214	321	185	279	150	225	131	197
15	233	350	200	300	173	260	140	210	122	184
16	218	328	187	281	162	244	131	197	115	173
17	205	309	176	265	153	229	123	185	108	162
18	194	292	166	250	144	217	116	175	102	153
19	184	276	158	237	137	205	110	166	96.6	145
20	175	263	150	225	130	195	105	158	91.8	138
21	166	250	143	214	124	186	99.8	150	87.4	131
22	159	239	136	205	118	177	95.3	143	83.5	125
23	152	228	130	196	113	170	91.1	137	79.8	120
24	146	219	125	188	108	163	87.3	131	76.5	115
25	140	210	120	180	104	156	83.8	126	73.5	110
26	134	202	115	173	99.8	150	80.6	121	70.6	106

50% UDL
= 0.5 (83.8)
= 41.9 kips

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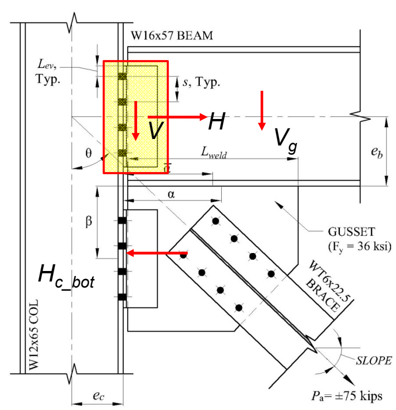
Example VB Corner Connection, I
Beam-to-Column (cont.)

Beam-to-Column – Forces (cont.):

$V = V_g + V_{b(\text{top})} + V_{b(\text{bot})}$
$= 41.9 \text{ kips} + 0 \text{ kips} + 25.4 \text{ kips}$
$= 67.3 \text{ kips}$

$H = \max(H_{c(\text{bot})}, H_{c(\text{top})} + PTF)$
$= \max(18.8 \text{ kips}, 0 \text{ kips} + 0 \text{ kips})$
$= 18.8 \text{ kips}$



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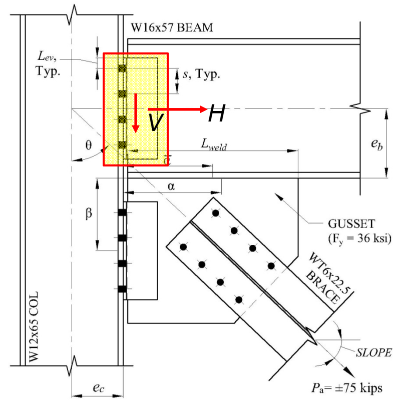
Example VB Corner Connection, I Beam-to-Column (cont.)

Beam-to-Column – Forces (cont.):

Notes:

1. Distortional forces not considered
2. Could use $H = H_{c(bot)} - (H_{c(top)} + PTF)$ if checking exact load case
3. Add opposite H_c if braces are both in compression.
4. Conservatively, the maximum V will be checked with H. However, when the bottom brace is in compression, H is a tension force at the beam and V_b would reduce V_g.



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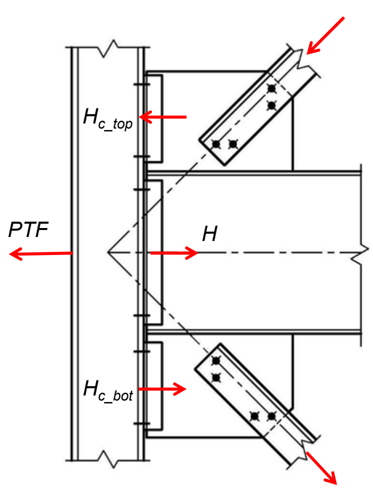
VB Corner Connection

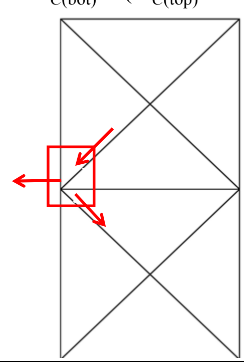


If exact load case not known:

$$H = \max(H_{C(bot)}, H_{C(top)} + PTF)$$

If exact load case is known:

$$H = H_{C(bot)} - (H_{C(top)} + PTF)$$



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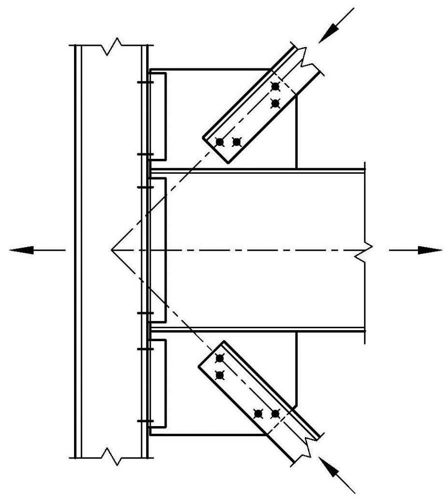
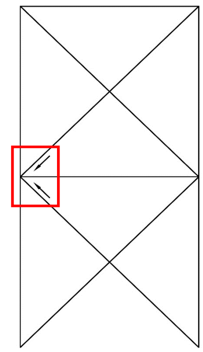
VB Corner Connection



- Consider if both top and bottom braces in compression due to column shedding



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

Beam-to-Column – Bolts:

$R_b/\Omega = 2n(r_v/\Omega)$

$= 2(4)(11.9 \text{ kips})$

$= 95.2 \text{ kips} > V = 67.3 \text{ kips}$ **o.k.**

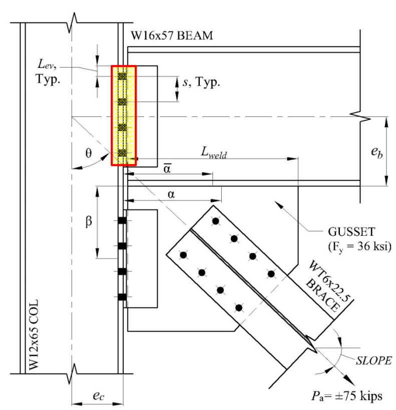
Check Shear and Tension:

$V_{bolt} = V_c/(2n) = 67.3 \text{ kips}/[(2)(4)]$

$= 8.41 \text{ kips}$

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

$= 2.35 \text{ kips}$



The lecture will skip to slide 159.

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

Beam-to-Column – Bolts (cont.):

Check Shear and Tension (cont.):

$$A_b = \frac{\pi d_b^2}{4}$$

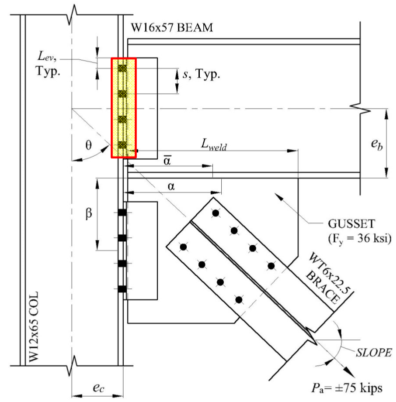
$$= \frac{3.14(0.75 \text{ in.})^2}{4}$$

$$= 0.442 \text{ in.}^2$$

$$f_v = V_{bolt}/A_b$$

$$= 8.41 \text{ kips}/0.442 \text{ in.}^2$$

$$= 19.0 \text{ ksi}$$



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

Beam-to-Column – Bolts (cont.):

Check Shear and Tension (cont.):

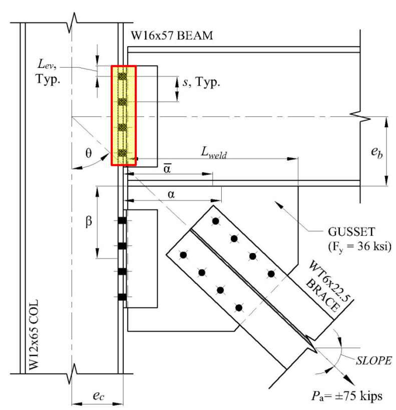
$$F'_{nt} = 1.3F_{nt} - \Omega F_{nt} f_{rv}/F_{nv} \le F_{nt} \quad (\textit{Spec } \text{J3-3a})$$

$$= 1.3(90 \text{ ksi}) - 2(90 \text{ ksi})(19.0 \text{ ksi})/(54 \text{ ksi})$$

$$= 53.7 \text{ ksi} < 90 \text{ ksi, use } 53.7 \text{ ksi}$$

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

Description of Fasteners	Nominal Tensile Strength, F_{nt}, ksi (MPa)[a]	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 Beam-to-Column (cont.)

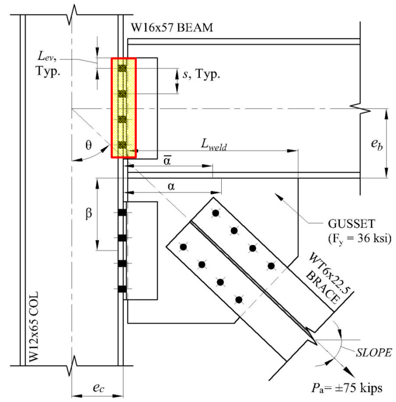
Beam-to-Column – Bolts (cont.):

Check Shear and Tension (cont.):

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

$$= \frac{(53.7 \text{ ksi})(0.442 \text{ in.}^2)}{2}$$

$$= 11.9 \text{ kips} > T_{bolt} = 2.35 \text{ kips } \textbf{o.k.}$$



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

Beam-to-Column – Prying of Angles :

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

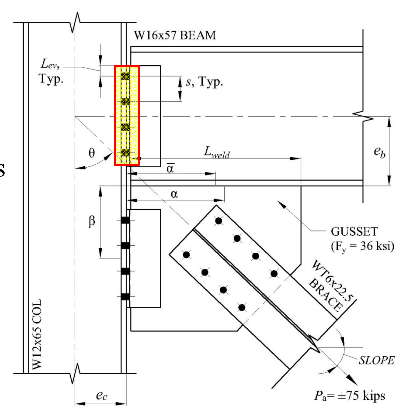
$T_{bolt} = H/(2n)$ = 18.8 kips/[(2)(4)] = 2.35 kips

$b = gage/2 - t_w/2 - t_a/2$

= 5.5 in./2 – 0.43 in./2 – 0.375/2 = 2.35 in.

$b' = b - d_b/2 =$

= 2.35 in. – 0.75 in./2 = 1.97 in.

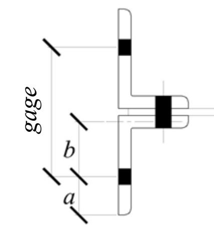


Example VB Corner Connection, I Beam-to-Column (cont.)

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

a = [2(angle OSL) + t_w – gage]/2
= [2(4 in.) + 0.43 in. – 5.5 in.]/2
= 1.47 in.

$a' = a + d_b/2 \leq 1.25b + d_b/2$
= 1.47 in. + 0.75 in./2
≤ 1.25(2.35 in.) + 0.75 in./2
= 1.85 in. ≤ 3.31 in.
= 1.85 in.



$\rho = b'/a'$ = 1.97 in./1.85 in. = 1.06

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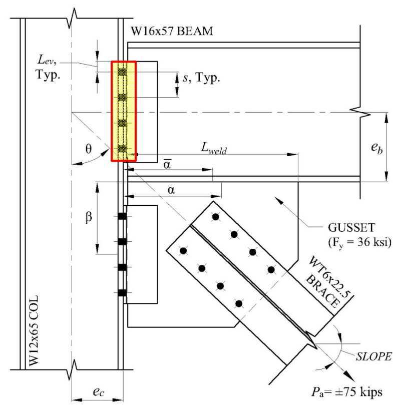
Example VB Corner Connection, I Beam-to-Column (cont.)

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

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

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

$B = r_t/\Omega$ = 11.9 kips
(bolt available tensile strength from previously)



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

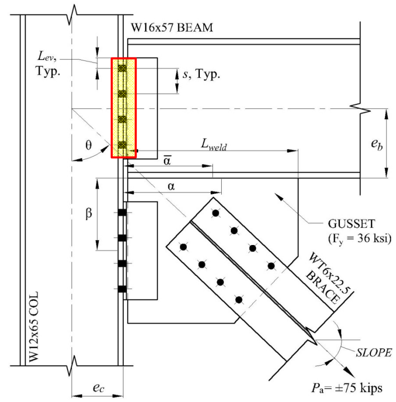
Beam-to-Column – Prying of Angles (cont.):

$$t_c = \sqrt{\frac{\Omega 4Bb'}{pF_u}} \quad \text{(AISC *Manual* Eq. 9-30a)}$$

$$= \sqrt{\frac{1.67(4)(11.9 \text{ kips})(1.97 \text{ in.})}{(2.88 \text{ in.})(58 \text{ ksi})}}$$

$$= 0.968 \text{ in.}$$

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



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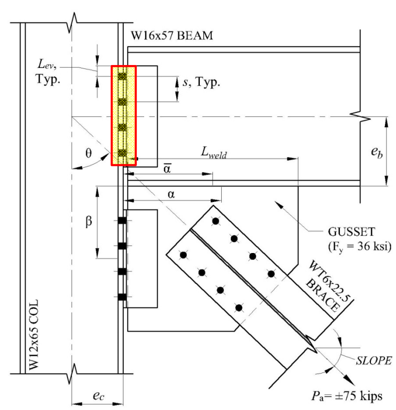
Example VB Corner Connection, I Beam-to-Column (cont.)

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

$$\alpha' = \frac{1}{\delta(1+\rho)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right] \quad \text{(AISC *Manual* Eq. 9-35)}$$

$$= \frac{1}{0.717(1+1.06)}\left[\left(\frac{0.968 \text{ in.}}{0.375 \text{ in.}}\right)^2 - 1\right]$$

$$= 3.83 \text{ in.}$$



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

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

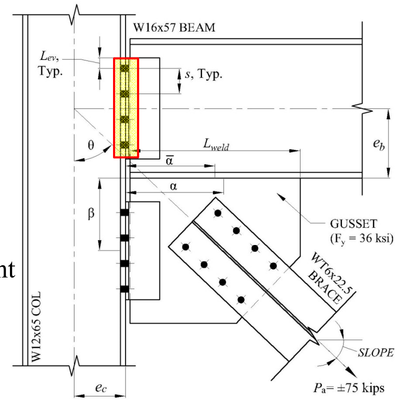
Note:

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

$$Q = 1 \qquad \text{(AISC } \textit{Manual} \text{ Eq. 9-32)}$$

When 0 ≤ α' < 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,

$$Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta\alpha') \qquad \text{(AISC } \textit{Manual} \text{ Eq. 9-33)}$$



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

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

For the example,

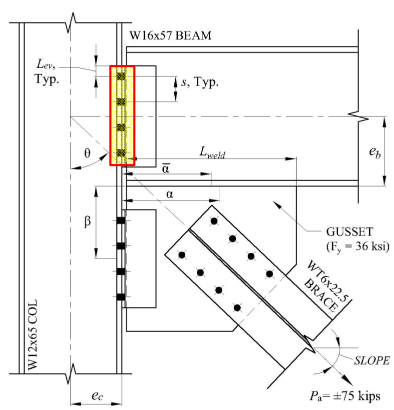
$$\alpha' = 3.83 > 1$$

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

$$Q = \left(\frac{t}{t_c}\right)^2 (1 + \delta) \qquad \text{(AISC } \textit{Manual} \text{ Eq. 9-34)}$$

$$= \left(\frac{0.375 \text{ in.}}{0.968 \text{ in.}}\right)^2 (1 + 0.717 \text{ in.})$$

$$= 0.258$$



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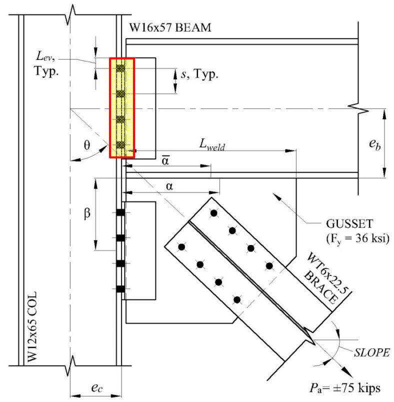
Example VB Corner Connection, I Beam-to-Column (cont.)

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

The available tensile strength including the effects of prying action, T_{avail}, is:

$$T_{avail} = BQ = (11.9\text{ kips})(0.258)$$
$$= 3.07\text{ kips} > T_{bolt} = 2.35\text{ kips } \textbf{o.k.}$$



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

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

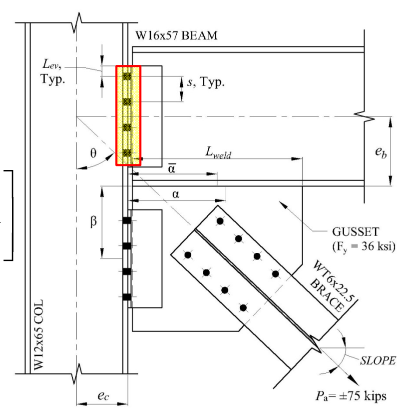
If prying force, q, needs to be determined:

$$\alpha = \left(\frac{1}{\delta}\right)\left[\frac{T}{B}\left(\frac{t_c}{t}\right)^2 - 1\right] \quad \text{(AISC } \textit{Manual} \text{ Eq. 9-29)}$$

$$= \left(\frac{1}{0.717}\right)\left[\left(\frac{2.35\text{ kips}}{11.9\text{ kips}}\right)\left(\frac{0.968\text{ in.}}{0.375\text{ in.}}\right)^2 - 1\right]$$

$$= 0.441 > 0 \text{ and } < 1.0$$

Note: When $\alpha > 1$, the connection is not adequate.



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

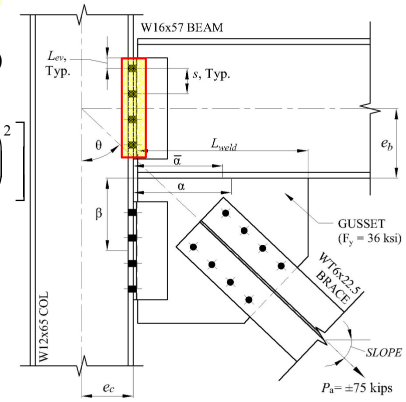
Beam-to-Column – Prying of Angles (cont.):

$$q = B\left[\delta\alpha\rho\left(\frac{t}{t_c}\right)^2\right] \quad \text{(AISC } \textit{Manual} \text{ Eq. 9-28)}$$

$$= 11.9 \text{ kips}\left[(0.717)(0.441)(1.06)\left(\frac{0.375 \text{ in.}}{0.968 \text{ in.}}\right)^2\right]$$

$$= 0.599 \text{ kips}$$

$$B_c = T_{bolt} + q = 2.35 \text{ kips} + 0.599 \text{ kips}$$
$$= 2.95 \text{ kips} \leq B = 11.9 \text{ kips } \textbf{o.k.}$$



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

Beam-to-Column – Prying of Column Flange:

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

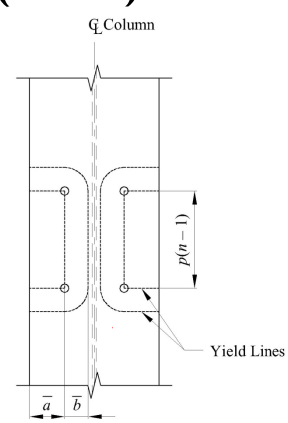
Determine effective flange width

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

(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)

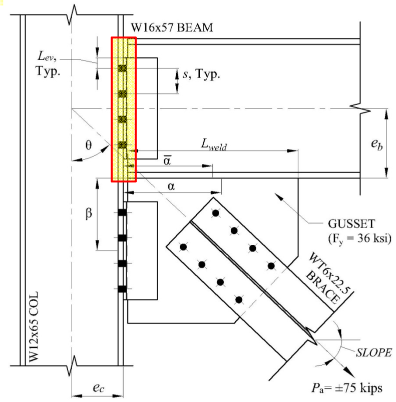
Example VB Corner Connection, I Beam-to-Column (cont.)

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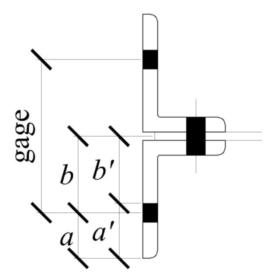
Example VB Corner Connection, I Beam-to-Column (cont.)

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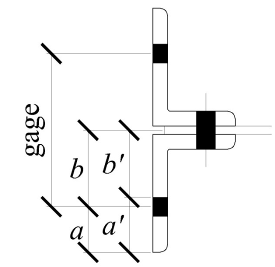
Example VB Corner Connection, I Beam-to-Column (cont.)

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

$a = \min\{(b_{fc} - \text{gage})/2, a \text{ from angle calcs}\}$
$= \min\{(12 \text{ in.} - 5.5 \text{ in.})/2, 1.47 \text{ in.}\}$
$= 1.47 \text{ in.}$



$a' = a + d_b/2 \leq 1.25b + d_b/2$
$= 1.47 \text{ in.} + 0.75 \text{ in.}/2 \leq 1.25(2.56 \text{ in.}) + 0.75 \text{ in.}/2$
$= 1.85 \text{ in.} < 3.57 \text{ in.}$ Use 1.85 in.

$\rho = b'/a' = 2.18 \text{ in.}/1.85 \text{ in.} = 1.18$

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

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

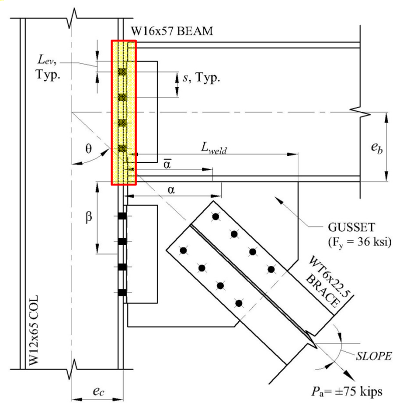
$p = p_{eff} = 5.88 \text{ in.}$
$\delta = 1 - d'/p = 1 - (0.8125 \text{ in.}/5.88 \text{ in.})$
$= 0.862$

$$t_c = \sqrt{\frac{\Omega 4Bb'}{pF_u}}$$ (AISC *Manual* Eq. 9-30a)

$$= \sqrt{\frac{1.67(4)(11.9 \text{ kips})(2.18 \text{ in.})}{(5.88 \text{ in.})(65 \text{ ksi})}}$$

$= 0.673 \text{ in.} > t_f = 0.605 \text{ in.}$

→ check available strength of bolt including the effects of prying action



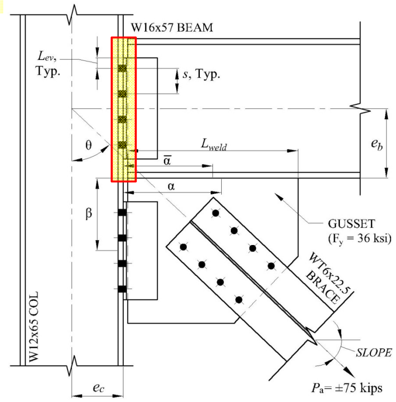
Example VB Corner Connection, I Beam-to-Column (cont.)

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

$$\alpha' = \frac{1}{\delta(1+\rho)}\left[\left(\frac{t_c}{t}\right)^2 - 1\right] \quad \text{(AISC \textit{Manual} Eq. 9-35)}$$

$$= \frac{1}{0.862(1+1.18)}\left[\left(\frac{0.673 \text{ in.}}{0.605 \text{ in.}}\right)^2 - 1\right]$$

$$= 0.126 > 0 \text{ and } < 1.0 \rightarrow \text{ Eq. 9.33 applies}$$



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

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

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

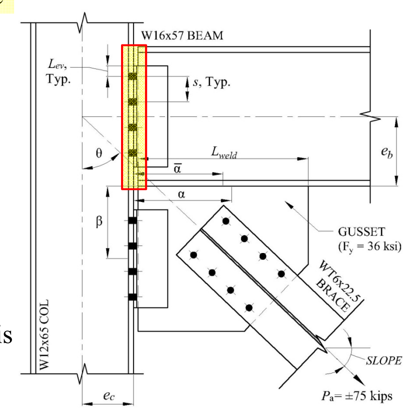
$$= \left(\frac{0.605 \text{ in.}}{0.673 \text{ in.}}\right)^2 \left[1+(0.862)(0.126)\right]$$

$$= 0.896$$

The available strength of bolt including prying is

$$T_{avail} = BQ = (11.9 \text{ kips})(0.896)$$

$$= 10.7 \text{ kips} > T_{bolt} = 2.35 \text{ kips } \textbf{o.k.}$$



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

Beam-to-Column – Bolt Bearing/Tearout:

AISC *Specification* Eq. J3-6a

AISC Bolt Shear:

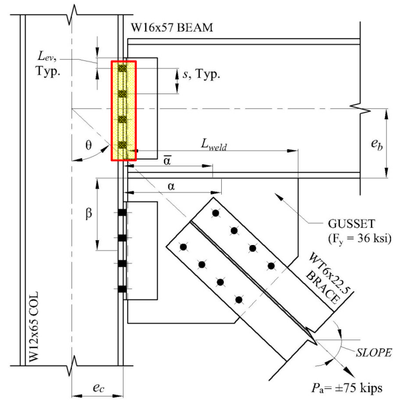
$r_v/\Omega = 11.9$ kips

Column Bolt Bearing:

$R_{brg(col)}/\Omega = 2.4d_bt_fF_u/\Omega$

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

$= 35.4$ kips



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

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

Column Bolt Tear Out (Center):

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

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

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

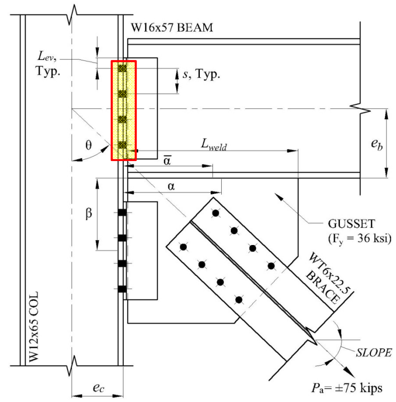
$= 51.6$ kips

Angle Bolt Bearing:

$R_{brg(L)}/\Omega = 2.4d_bt_aF_u/\Omega$

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

$= 19.6$ kips



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

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

Angle Bolt Tear Out (Edge):

$R_{tear(L\_E)}/\Omega = 1.2(L_{ev} - d_{h\_w}/2)t_a F_u/\Omega$

$= 1.2(1.25 \text{ in.} - {}^{13}/_{16} \text{ in.}/2)$

$\times(0.375 \text{ in.})(58 \text{ ksi})/2$

$= 11.0 \text{ kips}$

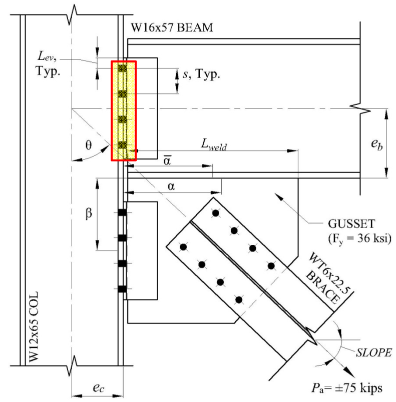
Angle Bolt Tear Out (Center):

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

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

$\times(58 \text{ ksi})/2$

$= 28.6 \text{ kips}$



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

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

$R_{n(top)}/\Omega = \min\{r_v/\Omega, R_{brg(col)}/\Omega, R_{tear(col\_C)}/\Omega,$

$R_{brg(L)}/\Omega, R_{tear(L\_C)}/\Omega\}$

$= \min\{11.9 \text{ kips}, 35.4 \text{ kips}, 51.6 \text{ kips},$

$19.6 \text{ kips}, 28.6 \text{ kips}\}$

$= 11.9 \text{ kips}$

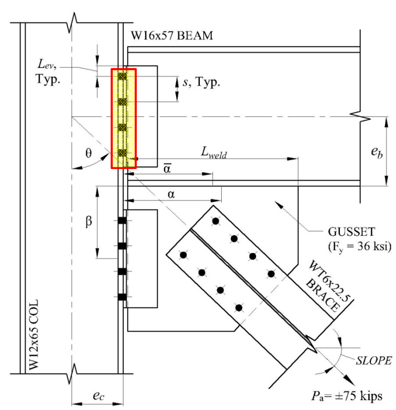
$R_{n(center)}/\Omega = \min\{r_v/\Omega, R_{brg(col)}/\Omega, R_{tear(col\_C)}/\Omega,$

$R_{brg(L)}/\Omega, R_{tear(L\_C)}/\Omega\}$

$= \min\{11.9 \text{ kips}, 35.4 \text{ kips}, 51.6 \text{ kips},$

$19.6 \text{ kips}, 28.6 \text{ kips}\}$

$= 11.9 \text{ kips}$



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

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

$$R_{n(\text{bot})}/\Omega = \min\{r_v/\Omega,\ R_{brg(\text{col})}/\Omega,\ R_{tear(\text{col\_C})}/\Omega,\ R_{brg(\text{L})}/\Omega,\ R_{tear(\text{L\_E})}/\Omega\}$$

$$= \min\{11.9 \text{ kips},\ 35.4 \text{ kips},\ 51.6 \text{ kips},\ 19.6 \text{ kips},\ 11.0 \text{ kips}\}$$

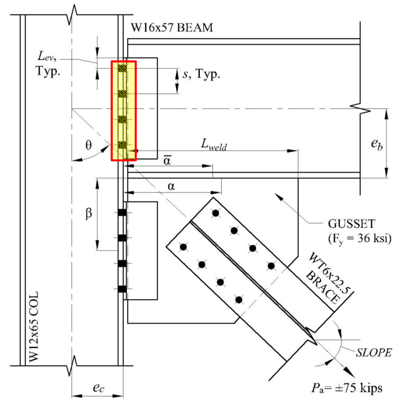
$$= 11.0 \text{ kips}$$

Total Bolt Strength

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

$$= [11.9 \text{ kips} + (11.9 \text{ kips})(4-2) + 11.0 \text{ kips}](2)$$

$$= 93.4 \text{ kips} > V = 67.3 \text{ kips } \textbf{o.k.}$$



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

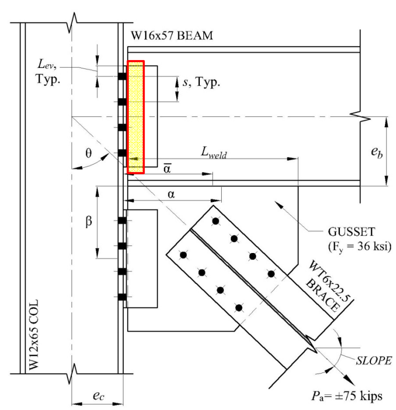
Beam-to-Column – Angle Gross Shear:

AISC *Specification* Eq. J4-1

$$\frac{R_n}{\Omega} = \frac{0.6F_yA_g}{\Omega}$$

$$= \frac{0.6(36 \text{ ksi})(11.5 \text{ in.})(0.375 \text{ in.})}{1.5}$$

$$= 124 \text{ kips} > V = 67.3 \text{ kips } \textbf{o.k.}$$



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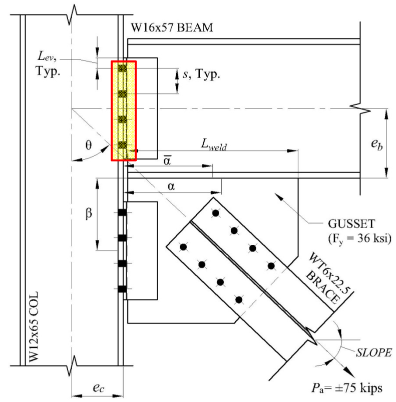
Example VB Corner Connection, I Beam-to-Column (cont.)

Beam-to-Column – Angle Net Shear:

AISC *Specification* Eq. J4-4

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

$$\frac{R_n}{\Omega} = \frac{0.6F_u A_{nv}}{\Omega}$$
$$= \frac{0.6(58\text{ ksi})(2)(8\text{ in.})(0.375\text{ in.})}{2}$$
$$= 104\text{ kips} > V = 67.3\text{ kips } \textbf{o.k.}$$



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

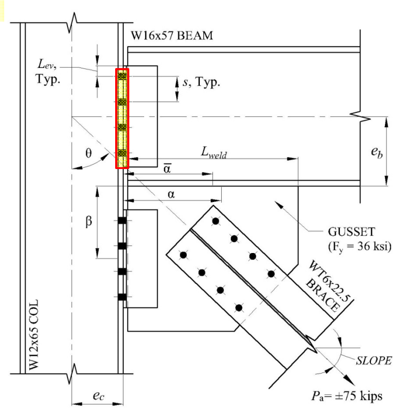
Beam-to-Column – Block Shear of Angles OSL:

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

$$\Omega = 2.00$$
$$R_n = 0.6F_u A_{nv} + U_{bs} F_u A_{nt}$$
$$\leq 0.6F_y A_{gv} + U_{bs} F_u A_{nt}$$

Shear Rupture $= 0.6F_u A_{nv}$
Shear Yield $= 0.6F_y A_{gv}$
Tension Rupture $= F_u A_{nt}$

U_{bs} = 1.0 for Direct Loaded Connections



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

Beam-to-Column – Block Shear of Angles OSL (cont.):

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

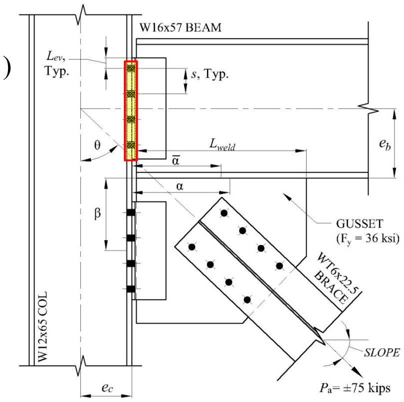
$= 10.3 \text{ in.}$

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

$= 1.25 \text{ in.} + (3 \text{ in.})(4 - 1)$

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

$= 7.19 \text{ in.}$



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

Beam-to-Column – Block Shear of Angles OSL (cont.):

$L_{gt} = L_{eh} = a$ from prying calcs

$= [2(\text{angle OSL}) + t_w - \text{gage}]/2$

$= [2(4 \text{ in.}) + 0.43 \text{ in.} - 5.5 \text{ in.}]/2$

$= 1.47 \text{ in.}$

$L_{nt} = L_{eh} - 0.5(\text{hole length} + {}^1/_{16} \text{ in.})$

$= 1.47 \text{ in.} - 0.5(1 \text{ in.} + {}^1/_{16} \text{ in.})$

$= 0.934 \text{ in.}$

TABLE J3.3
Nominal Hole Dimensions, in.

Bolt Diameter, in.	Hole Dimensions			
	Standard (Dia.)	Oversize (Dia.)	Short-Slot (Width × Length)	Long-Slot (Width × Length)
1/2	9/16	5/8	9/16 × 11/16	9/16 × 1 1/4
5/8	11/16	13/16	11/16 × 7/8	11/16 × 1 9/16
3/4	13/16	15/16	13/16 × 1	13/16 × 1 7/8
7/8	15/16	1 1/16	15/16 × 1 1/8	15/16 × 2 3/16
1	1 1/16	1 1/4	1 1/16 × 1 5/16	1 1/16 × 2½
≥ 1 1/8	d + 1/16	d + 5/16	(d + 1/16) × (d + 3/8)	(d + 1/16) × (2.5 × d)

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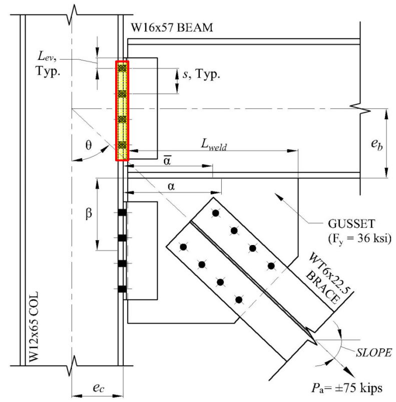
Example VB Corner Connection, I Beam-to-Column (cont.)

Beam-to-Column – Block Shear of Angles OSL (cont.):

$$A_{gv} = 2L_{gv}t_a = 2(10.3\text{ in.})(0.375\text{ in.}) = 7.73\text{ in.}^2$$

$$A_{nv} = 2L_{nv}t_a = 2(7.19\text{ in.})(0.375\text{ in.}) = 5.39\text{ in.}^2$$

$$A_{nt} = 2L_{nt}t_a = 2(0.934\text{ in.})(0.375\text{ in.}) = 0.701\text{ in.}^2$$



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

Beam-to-Column – Block Shear of Angles OSL (cont.):

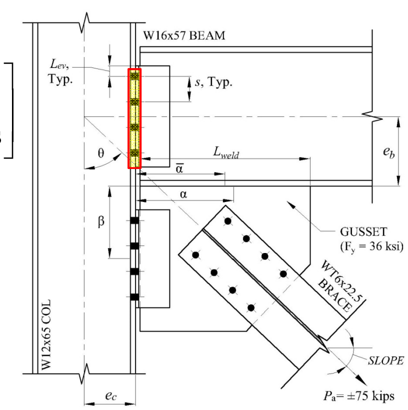
$$R_n = \min\begin{bmatrix} 0.6(58\text{ ksi})(5.39\text{ in.}^2) = 187.57\text{ kips} \\ 0.6(36\text{ ksi})(7.73\text{ in.}^2) = 166.97\text{ kips} \end{bmatrix}$$

$$+ [1(58\text{ ksi})(0.701\text{ in.}^2)]$$

$$= 166.97\text{ kips} + 40.66\text{ kips} = 207.63\text{ kips}$$

$$R_n/\Omega = (207.63\text{ kips})/2$$

$= 104$ kips $> V = 67.3$ kips **o.k.**



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

Beam-to-Column – Block Shear of Beam Web, Axial:

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

$\Omega = 2.00$

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

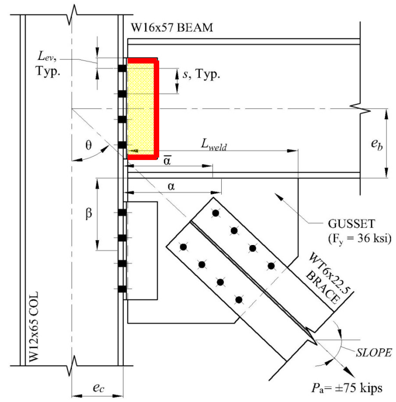
$\leq 0.6F_yA_{gv} + U_{bs}F_uA_{nt}$

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 Beam-to-Column (cont.)

Beam-to-Column – Block Shear of Beam Web, Axial (cont.):

For Axial check:

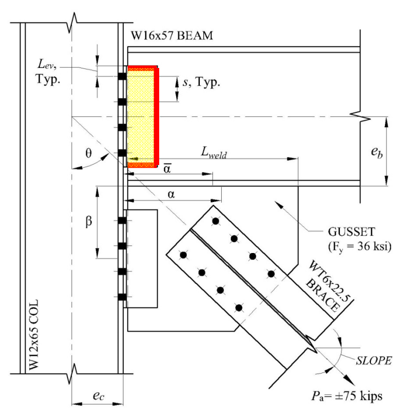
$L_{gv} = k = 3.5$ in.

$L_{gt} = 11.5$ in. (length of angle)

$L_{nv} = L_{gv} = 3.5$ in. (since welded)

$L_{nt} = L_{gt} = 11.5$ in. (since welded)

Note: The beam is checked for axial load only. Block shear due to shear load is not applicable since top flange is not coped.



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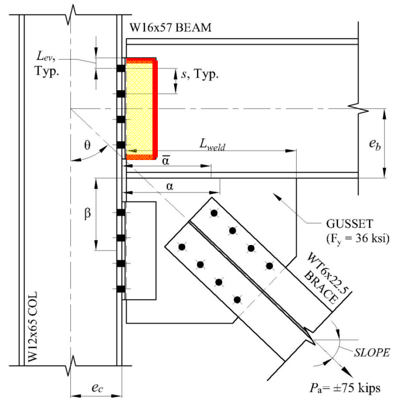
Example VB Corner Connection, I Beam-to-Column (cont.)

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

$A_{gv} = 2L_{gv}t_w = 2(3.5\text{ in.})(0.43\text{ in.})$
$= 3.01\text{ in.}^2$

$A_{nv} = 2L_{nv}t_w = 2(3.5\text{ in.})(0.43\text{ in.})$
$= 3.01\text{ in.}^2$

$A_{nt} = L_{nt}t_w = (11.5\text{ in.})(0.43\text{ in.})$
$= 4.95\text{ in.}^2$



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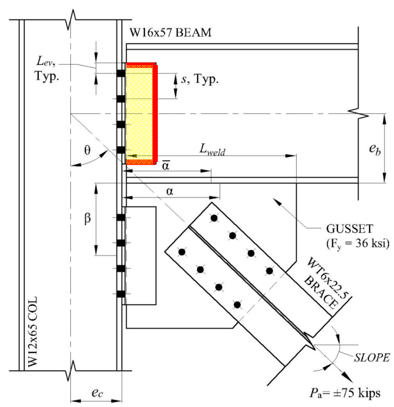
Example VB Corner Connection, I Beam-to-Column (cont.)

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

$$R_n = \min\begin{bmatrix} 0.6(65\text{ ksi})(3.01\text{ in.}^2) = 117.39\text{ kips} \\ 0.6(50\text{ ksi})(3.01\text{ in.}^2) = 90.30\text{ kips} \end{bmatrix}$$
$$+ [1(65\text{ ksi})(4.95\text{ in.}^2)]$$
$$= 90.30\text{ kips} + 321.75\text{ kips} = 412.05\text{ kips}$$

$R_n/\Omega = (412.05\text{ kips})/2$
$= 206\text{ kips} > H = 18.8\text{ kips}$ **o.k.**



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

Beam-to-Column – Size Weld:

$$P = \sqrt{V^2 + H^2}$$

$$= \sqrt{(67.3\text{ kips})^2 + (18.8\text{ kips})^2}$$

$$= 69.9\text{ kips}$$

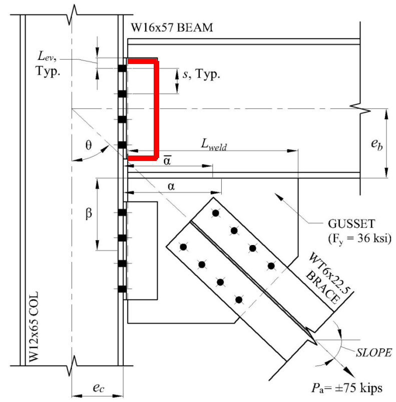
$\theta = \tan^{-1}(H/V) = \tan^{-1}(18.8\text{ kips}/67.3\text{ kips})$

$= 15.6°$

$l = s(n - 1) + 2L_{ev}$

$= (3\text{ in.})(4 - 1) + 2(1.25\text{ in.})$

$= 11.5\text{ in.}$



The lecture will skip to slide 168.

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

Beam-to-Column – Size Weld (cont.):

$kl = 3.5\text{ in.}$

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

$x = 0.057$

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

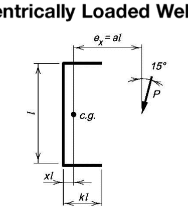
$= [4\text{ in.} - (0.057)(11.5\text{ in.})]/11.5\text{ in.}$

$= 0.29$

$C = 2.87$ (AISC *Manual* Table 8-8 with Angle = 15°, round down)

AISC *Manual*

Table 8-8 (continued)
Coefficients, *C*, for Eccentrically Loaded Weld Groups



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	1.98	2.47	3.01	3.56	4.10	4.65	5.19	5.74	6.28	6.83	7.37	8.46
0.10	1.90	2.35	2.87	3.41	3.95	4.50	5.05	5.60	6.15	6.70	7.24	8.34
0.15	1.84	2.30	2.79	3.30	3.81	4.33	4.86	5.39	5.92	6.45	6.98	8.06
0.20	1.76	2.21	2.68	3.16	3.65	4.15	4.65	5.16	5.67	6.18	6.69	7.72
0.25	1.65	2.08	2.54	3.00	3.47	3.94	4.42	4.91	5.39	5.89	6.38	7.38
0.30	1.55	1.95	2.39	2.82	3.27	3.72	4.18	4.64	5.11	5.58	6.06	7.03

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

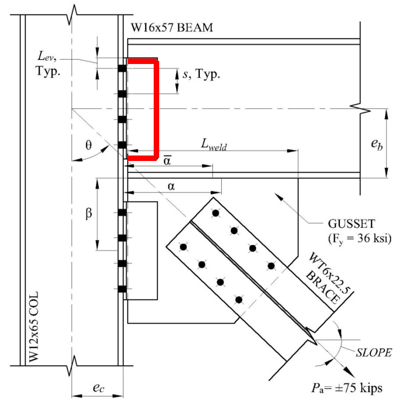
Beam-to-Column – Size Weld (cont.):

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



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

Beam-to-Column – Size Weld (cont.):

$D_{min} = P_c\Omega/(2CC_1L)$

$= 69.9$ kips$(2)/((2)(2.87)(1.0)(11.5$ in.$))$

$= 2.12 < 4$ (16th of an inch)

¼" fillet weld o.k.

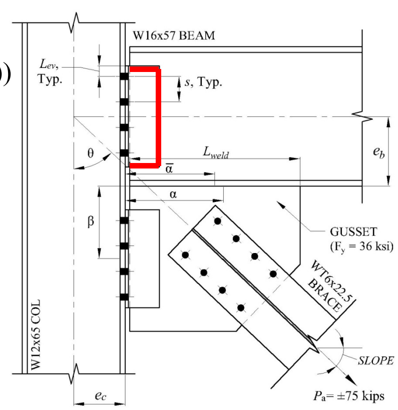
$t_{min} = 2(0.928D)/(0.6F_u/\Omega)$

$= 2(0.928)(2.12)/[(0.6)(65\text{ ksi}/2)]$

$= 0.202$ in. $> t_w = 0.43$ in. **o.k.**

Note, the above is same as *Manual* Eq 9-3:

$t_{min} = 6.19D/F_u$

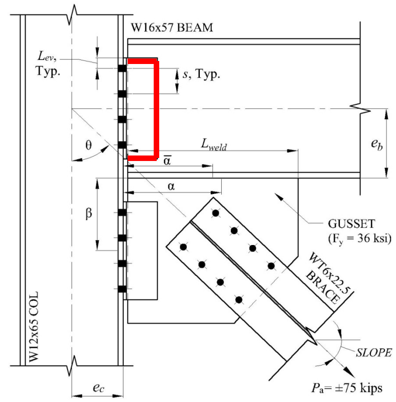


Example VB Corner Connection, I Beam-to-Column (cont.)

Beam-to-Column – Size Weld (cont.):

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

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



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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.

Practical Connection Design

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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 minimum W18 beams.
- Use same depth beams either side of the column when transfer forces exist.

Economy of Vertical Bracing Connections

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

Summary

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

Questions?

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Within 2 business days…

- You will receive an email on how to report 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!

Individual Webinar Registrants

CEU/PDH Certificates

Within 2 business days…

- New reporting site (URL will be provided in the forthcoming email).
- Username: Same as AISC website username.
- Password: Same as AISC website password.

8-Session Registrants

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

8-Session Registrants

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Quiz and Attendance records: Posted Tuesday mornings. www.aisc.org/nightschool - click on Current Course Details.

Reasons for quiz:

- EEU – must take all quizzes and final to receive EEU
- CEUs/PDHS – If you watch a recorded session you must take quiz for CEUs/PDHs.
- REINFORCEMENT – Reinforce what you learned tonight. Get more out of the course.

NOTE: If you attend the live presentation, you do not have to take the quizzes to receive CEUs/PDHs.

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Thank You

Please give us your feedback!
Survey at conclusion of webinar.

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