AISC Night School

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

Appendix 1

April 26, 2022 | William A Thornton

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

Brace-to-Gusset – Block Shear Gusset:

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

$\Omega = 2.00$

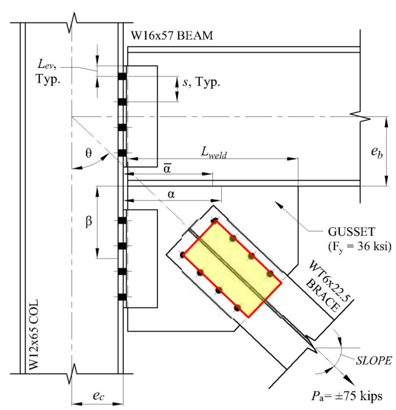
$$R_n = 0.6F_uA_{nv} + U_{bs}F_uA_{nt} \le 0.6F_yA_{gv} + U_{bs}F_uA_{nt}$$

Shear Rupture $= 0.6F_uA_{nv}$

Shear Yield $= 0.6F_yA_{gv}$

Tension Rupture $= F_uA_{nt}$

U_{bs} = 1.0 for Direct Loaded Connections



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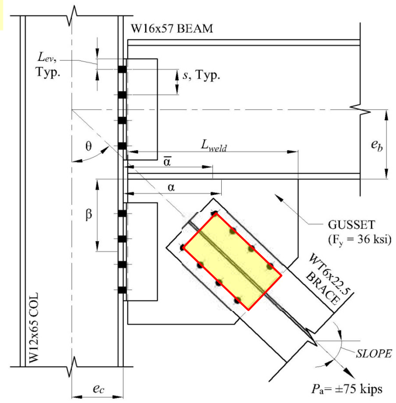
Example VB Corner Connection, I Brace-to-Gusset (cont.)

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

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

$L_{nv} = L_{ev} + s(n-1) - (n-0.5)(d_h + {}^{1}/_{16}\text{ in.})$
$= 1.5\text{ in.} + (3\text{ in.})(4-1) - (4-0.5)(\tfrac{7}{8}\text{ in.})$
$= 7.44\text{ in.}$

$L_{nt} = \text{gage} - (d_h + {}^{1}/_{16}\text{ in.})$
$= 5.5\text{ in.} - (\tfrac{7}{8}\text{ in.}) = 4.63\text{ in.}$



3

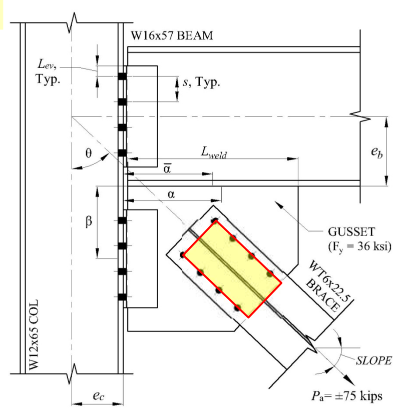
Example VB Corner Connection, I Brace-to-Gusset (cont.)

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.)

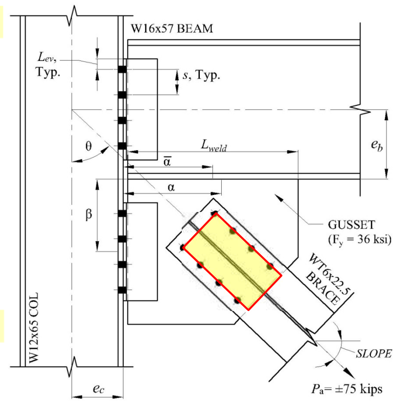
Brace-to-Gusset – Block Shear Gusset (cont.):

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

$$+ [1(58\text{ ksi})(1.74\text{ in.}^2)]$$

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

R_n/Ω = (271.13 kips)/2 = 136 kips > 75 kips **o.k.**



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AISC Night School

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

Appendix 2

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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*

$\Omega = 2.00$

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

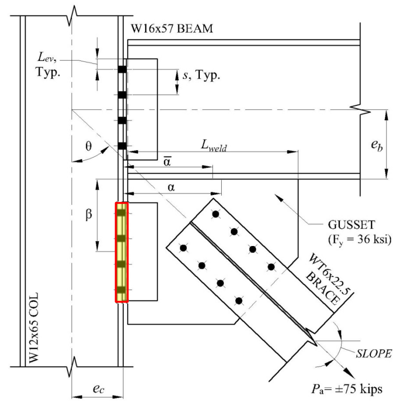
$\leq 0.6F_yA_{gv} + U_{bs}F_uA_{nt}$

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

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

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

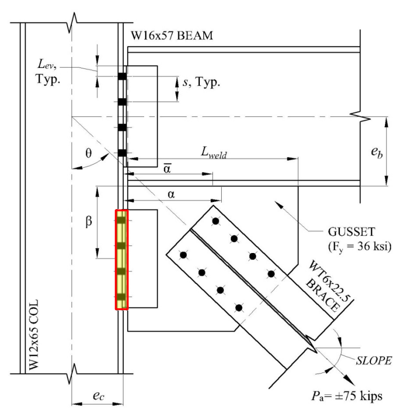
$= 1.25\text{ in.} + (3\text{ in.})(4-1) = 10.3\text{ in.}$

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

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

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

$= 7.19\text{ in.}$



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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 × 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 1/8	1 1/4	1 1/8 × 1 5/16	1 1/8 × 2½
≥ 1 1/8	*d* + 1/8	*d* + 5/16	(*d* + 1/8) × (*d* + 3/8)	(*d* + 1/8) × 2.5*d*

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

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

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

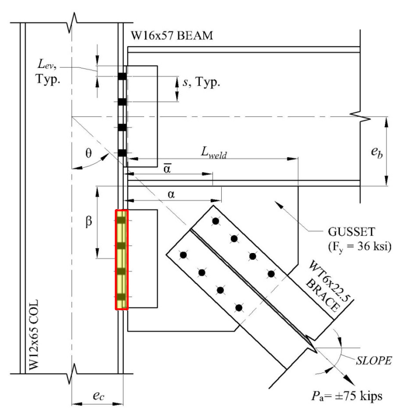
$= 7.73\text{ in.}^2$

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

$= 5.39\text{ in.}^2$

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

$= 0.68\text{ in.}^2$



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

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

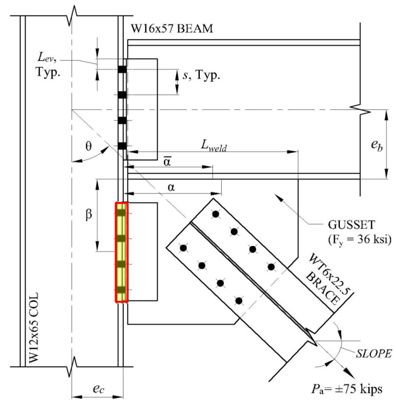
$$R_n = \min\begin{bmatrix} 0.6(58\text{ ksi})(5.39\text{ in.}^2) = 187.57\text{ kips} \\ 0.6(36\text{ ksi})(7.73\text{ in.}^2) = 166.97\text{ kips} \end{bmatrix}$$

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

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

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

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



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

Appendix 3

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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

(UDL projects: not preferred)

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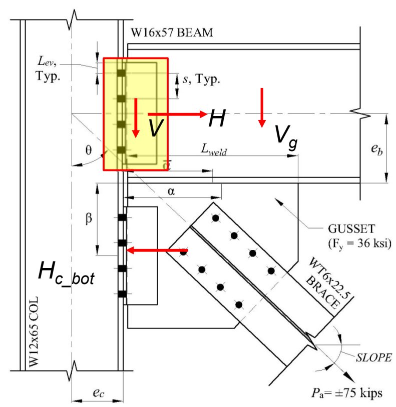
Example VB Corner Connection, I Beam-to-Column (cont.)

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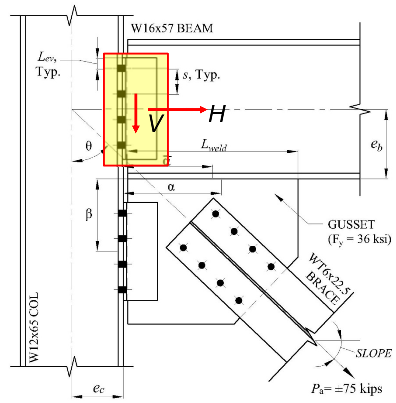
Example VB Corner Connection, I Beam-to-Column (cont.)

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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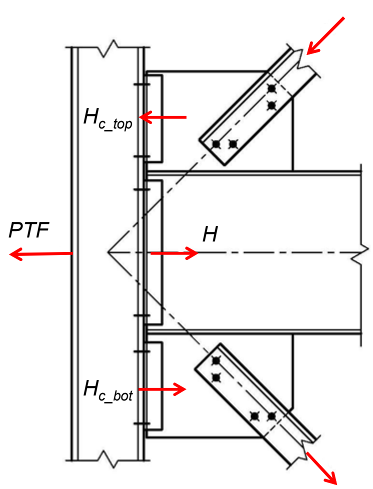
VB Corner Connection

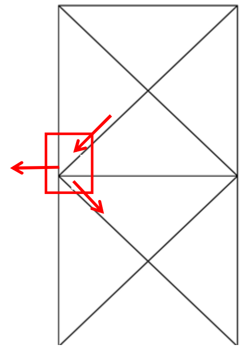


If exact load case not known:

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

If exact load case is known:

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



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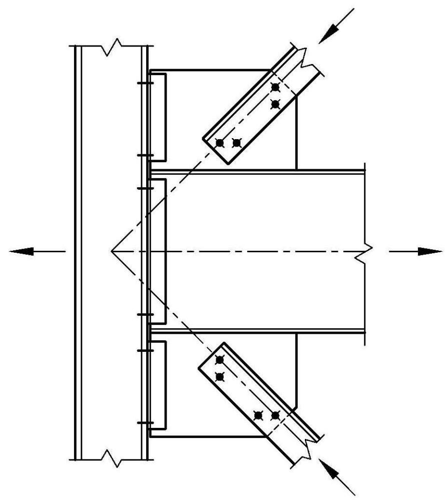
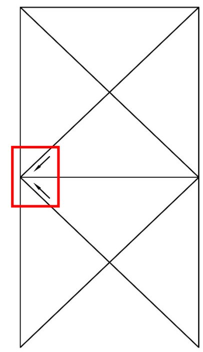
VB Corner Connection



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



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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.**

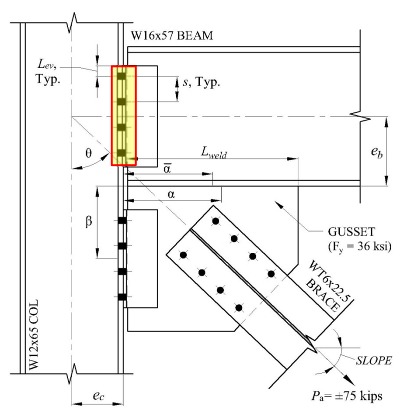
Check Shear and Tension:

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

$= 8.41 \text{ kips}$

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

$= 2.35 \text{ kips}$



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

Beam-to-Column – Bolts (cont.):

Check Shear and Tension (cont.):

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

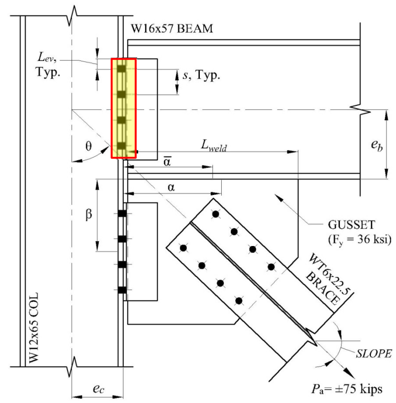
$$= \frac{3.14(0.75\text{ in.})^2}{4}$$

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

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

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

$$= 19.0\text{ ksi}$$



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

Beam-to-Column – Bolts (cont.):

Check Shear and Tension (cont.):

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

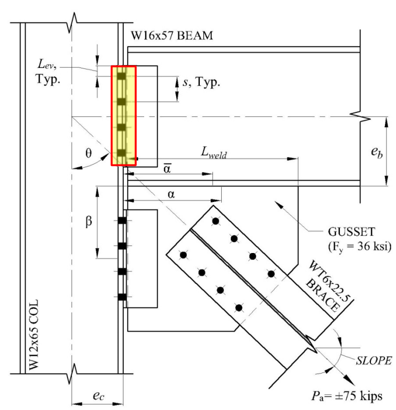
$$= 1.3(90\text{ ksi}) - 2(90\text{ ksi})(19.0\text{ ksi})/(54\text{ ksi})$$

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

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)[c]	27 (186)[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.)

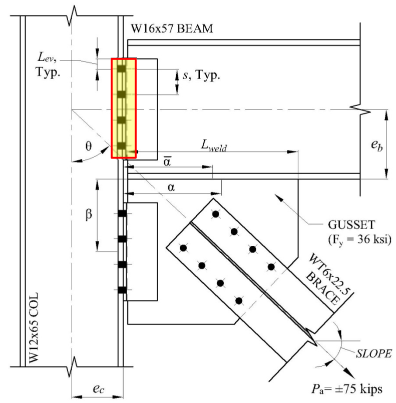
Beam-to-Column – Bolts (cont.):

Check Shear and Tension (cont.):

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

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

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



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

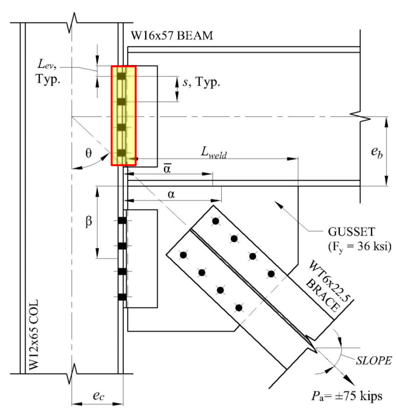
Beam-to-Column – Prying of Angles :

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:

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

$$b = gage/2 - t_w/2 - t_a/2$$
$$= 5.5 \text{ in.}/2 - 0.43 \text{ in.}/2 - 0.375/2 = 2.35 \text{ in.}$$

$$b' = b - d_b/2 =$$
$$= 2.35 \text{ in.} - 0.75 \text{ in.}/2 = 1.97 \text{ in.}$$



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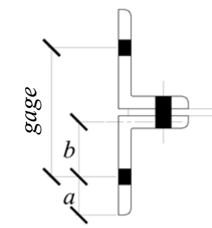
Example VB Corner Connection, I Beam-to-Column (cont.)

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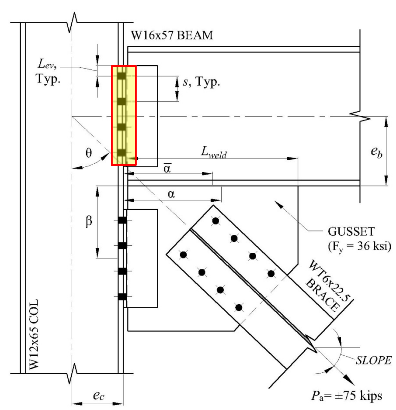
Example VB Corner Connection, I Beam-to-Column (cont.)

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

$B_c = r_t/\Omega$ = 11.9 kips
(bolt available tensile strength shown previously)



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

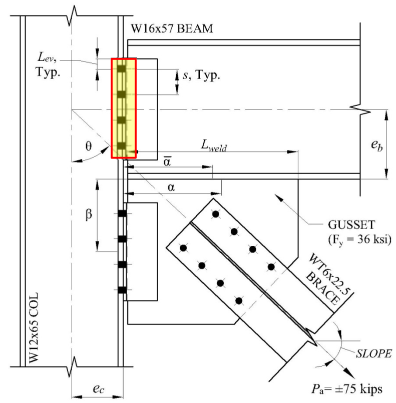
Beam-to-Column – Prying of Angles (cont.):

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

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

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

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



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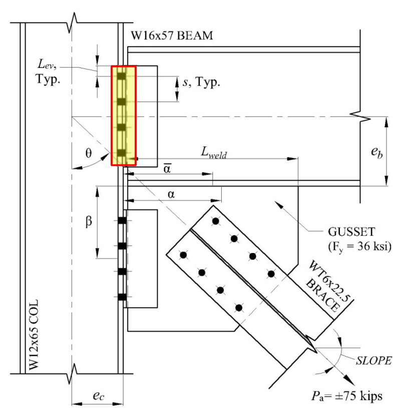
Example VB Corner Connection, I Beam-to-Column (cont.)

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

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

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

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



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

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

Note:

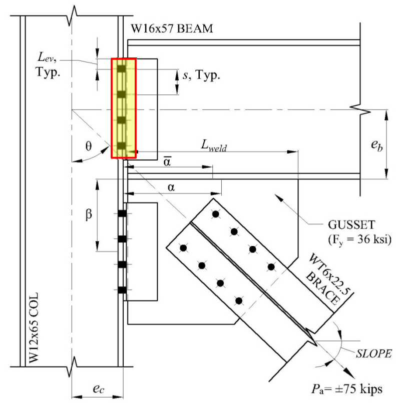
When $\alpha' < 0$, this means that the fitting has sufficient strength and stiffness to develop the full bolt available tensile strength,

$$Q = 1$$

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

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

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



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

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

For the example,

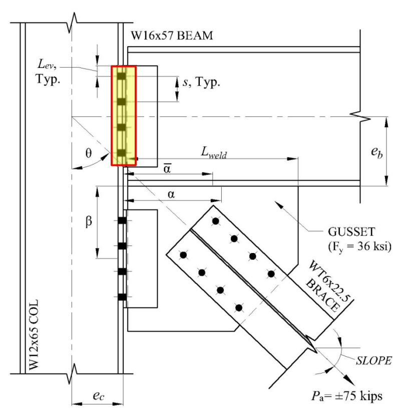
$$\alpha' = 3.83 > 1$$

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

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

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

$$= 0.258$$



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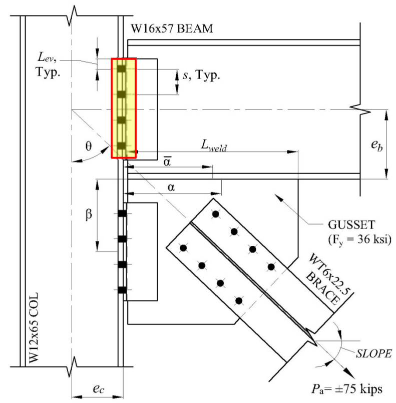
Example VB Corner Connection, I Beam-to-Column (cont.)

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

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

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



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

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

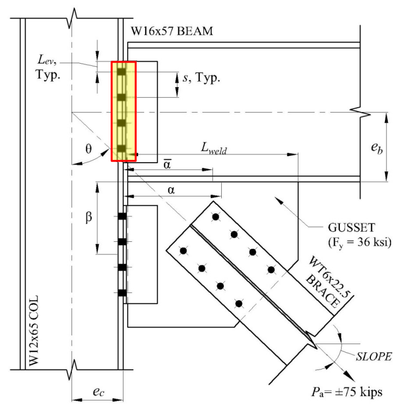
If prying force, q_r, needs to be determined:

$$\alpha = \left(\frac{1}{\delta}\right)\left[\frac{T_r}{B_c}\left(\frac{t_c}{t}\right)^2 - 1\right] \quad \text{(AISC \textit{Manual} Eq. 9-25)}$$

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

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

Note: When α > 1, the connection is not adequate.



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

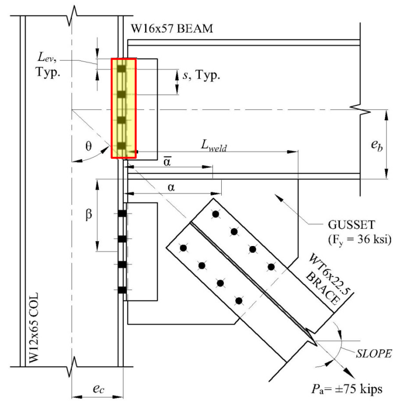
Beam-to-Column – Prying of Angles (cont.):

$$q_r = B_c\left[\delta\alpha\rho\left(\frac{t}{t_c}\right)^2\right] \quad \text{(AISC } \textit{Manual} \text{ Eq. 9-24)}$$

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

$$= 0.599\text{ kips}$$

$$B_r = T_r + q_r = 2.35\text{ kips} + 0.599\text{ kips}$$
$$= 2.95\text{ kips} \le B_c = 11.9\text{ kips } \textbf{o.k.}$$



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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:

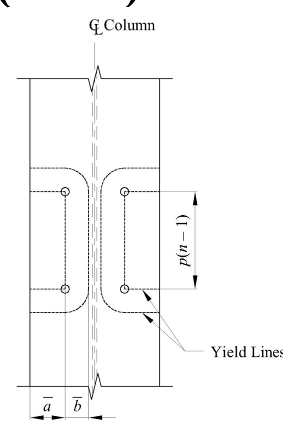
Determine effective flange width

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

(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)

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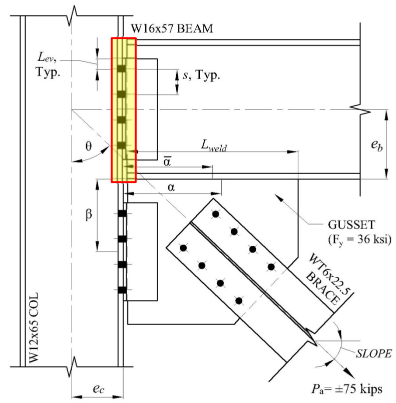
Example VB Corner Connection, I Beam-to-Column (cont.)

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

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

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

Note: use 3" max if at top of column



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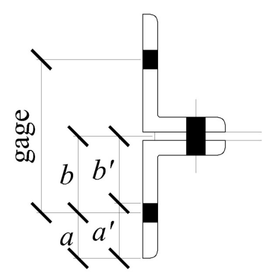
Example VB Corner Connection, I Beam-to-Column (cont.)

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

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

b = gage/2 – t_{wc}/2
= 5.5 in./2 – 0.39 in./2 = 2.56 in.

$b' = b - d_b/2$ =
= 2.56 in. – 0.75 in./2 = 2.18 in.



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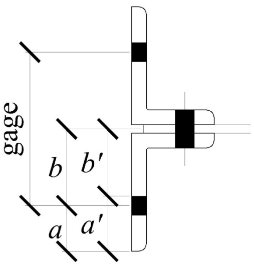
Example VB Corner Connection, I Beam-to-Column (cont.)

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.

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



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

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

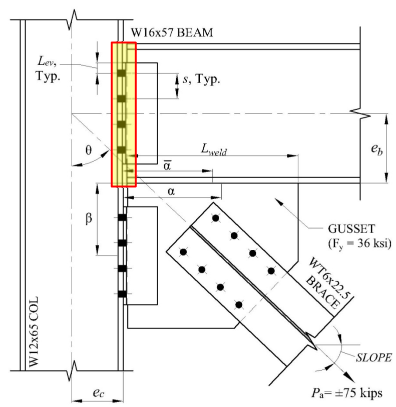
$p = p_{eff} = 5.88 \text{ in.}$
$\delta = 1 - d'/p = 1 - (0.8125 \text{ in.}/5.88 \text{ in.})$
$= 0.862$

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

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

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

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



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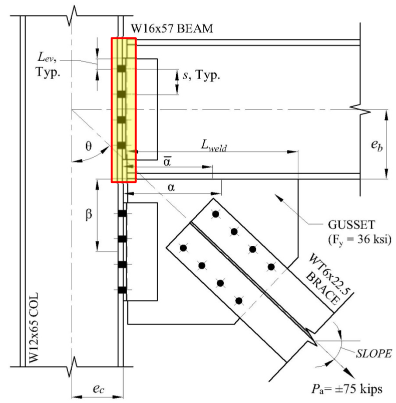
Example VB Corner Connection, I Beam-to-Column (cont.)

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

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

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

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



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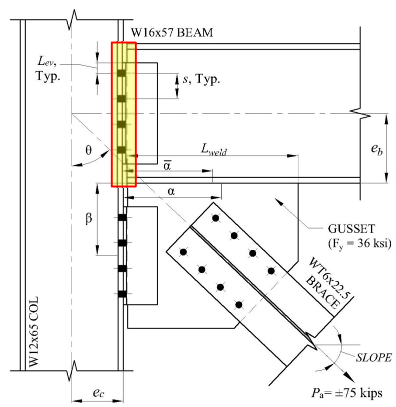
Example VB Corner Connection, I Beam-to-Column (cont.)

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

$T_c = B_c Q = (11.9\text{ kips})(0.896)$

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

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

Beam-to-Column – Bolt Bearing/Tearout:

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

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

AISC Bolt Shear:

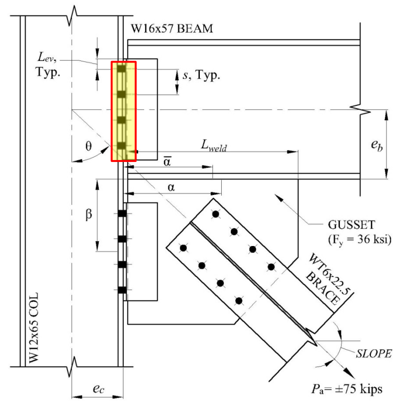
$r_v/\Omega = 11.9$ kips

Column Bolt Bearing:

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

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

$= 35.4$ kips



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

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

Column Bolt Tear Out (Center):

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

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

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

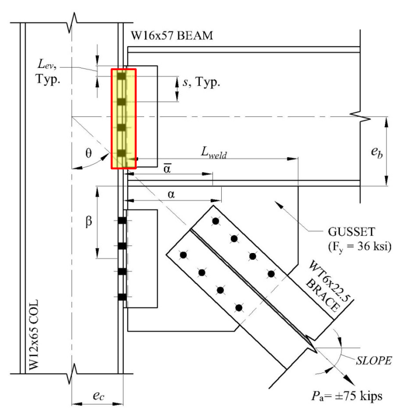
$= 51.6$ kips

Angle Bolt Bearing:

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

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

$= 19.6$ kips



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

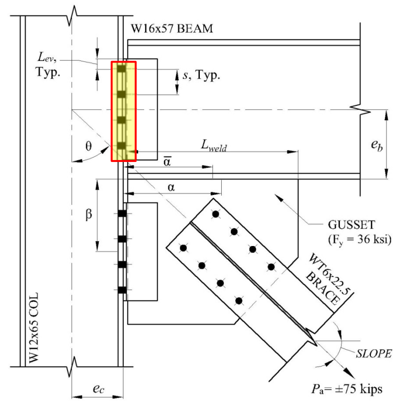
Beam-to-Column – Bolt Bearing (cont.):

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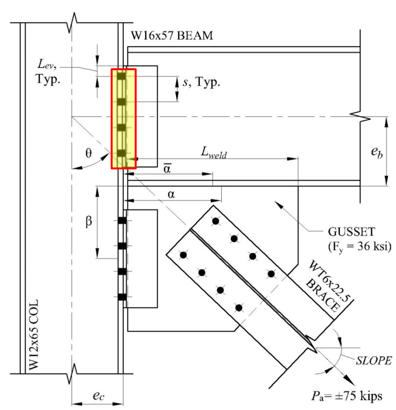
Example VB Corner Connection, I Beam-to-Column (cont.)

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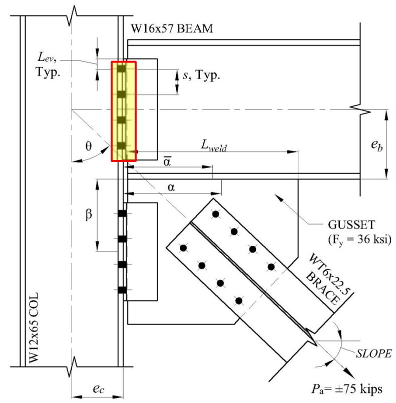
Example VB Corner Connection, I Beam-to-Column (cont.)

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

$R_n/\Omega = [R_{n(\text{top})}/\Omega + (R_{n(\text{center})}/\Omega)(n-2) + R_{n(\text{bot})}/\Omega](2 \text{ lines})$
$= [11.9 \text{ kips} + (11.9 \text{ kips})(4-2) + 11.0 \text{ kips}](2)$
$= 93.4 \text{ kips} > V = 67.3 \text{ kips}$ **o.k.**



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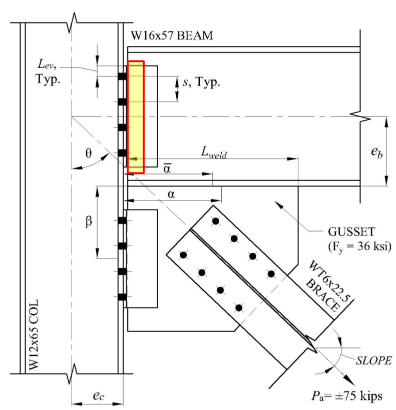
Example VB Corner Connection, I Beam-to-Column (cont.)

Beam-to-Column – Angle Gross Shear:

AISC *Specification* Eq. J4-3

$$\frac{R_n}{\Omega} = \frac{0.6F_yA_g}{\Omega}$$
$$= \frac{0.6(36 \text{ ksi})(11.5 \text{ in.})(0.375 \text{ in.})}{1.5}$$
$$= 124 \text{ kips} > V = 67.3 \text{ kips} \textbf{ o.k.}$$



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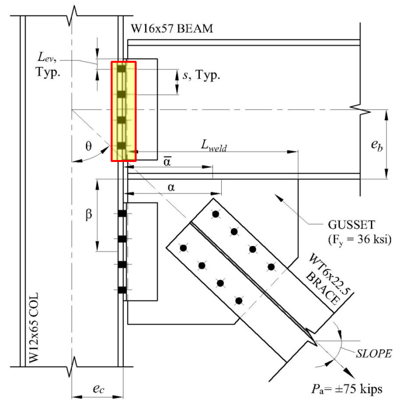
Example VB Corner Connection, I
Beam-to-Column (cont.)

Beam-to-Column – Angle Net Shear:

AISC *Specification* Eq. J4-4

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

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



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

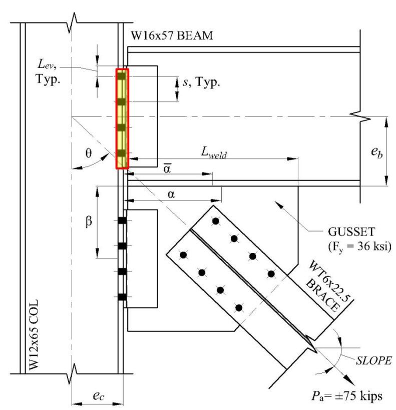
Beam-to-Column – Block Shear of Angles OSL:

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

$$\Omega = 2.00$$
$$R_n = 0.6F_uA_{nv} + U_{bs}F_uA_{nt}$$
$$\leq 0.6F_yA_{gv} + U_{bs}F_uA_{nt}$$

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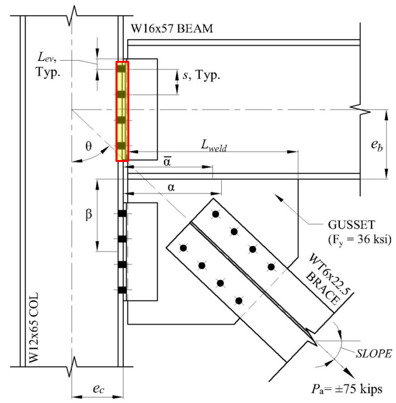
Example VB Corner Connection, I Beam-to-Column (cont.)

Beam-to-Column – Block Shear of 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
$= [2(\text{angle OSL}) + t_w - \text{gage}]/2$
$= [2(4\text{ in.}) + 0.43\text{ in.} - 5.5\text{ in.}]/2$
$= 1.47\text{ in.}$

$L_{nt} = L_{eh} - 0.5(\text{hole length} + {}^1/_{16}\text{ in.})$
$= 1.47\text{ in.} - 0.5(1\text{ in.} + {}^1/_{16}\text{ in.})$
$= 0.934\text{ in.}$

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 × 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 1/8	1 1/4	1 1/8 × 1 5/16	1 1/8 × 2 1/2
≥ 1 1/8	d + 1/8	d + 5/16	(d + 1/8) × (d + 3/8)	(d + 1/8) × 2.5d

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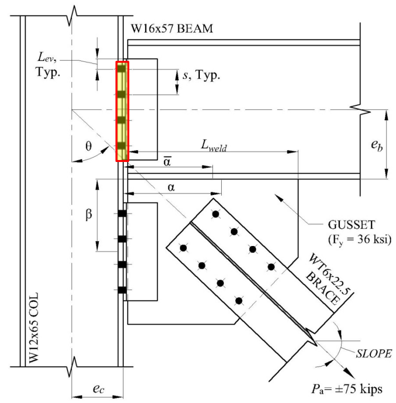
Example VB Corner Connection, I
Beam-to-Column (cont.)

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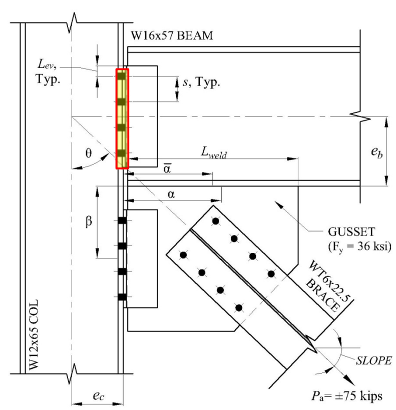
Example VB Corner Connection, I
Gusset-to-Column (cont.)

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

$$R_n = \min\begin{bmatrix} 0.6(58 \text{ ksi})(5.39 \text{ in.}^2) = 187.57 \text{ kips} \\ 0.6(36 \text{ ksi})(7.73 \text{ in.}^2) = 166.97 \text{ kips} \end{bmatrix} + [1(58 \text{ ksi})(0.701 \text{ in.}^2)]$$

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

$$R_n/\Omega = (207.63 \text{ kips})/2 = 104 \text{ kips} > V = 67.3 \text{ kips o.k.}$$



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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*

$\Omega = 2.00$

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

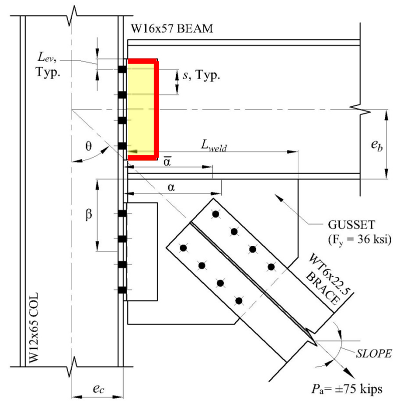
$\leq 0.6F_yA_{gv} + U_{bs}F_uA_{nt}$

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:

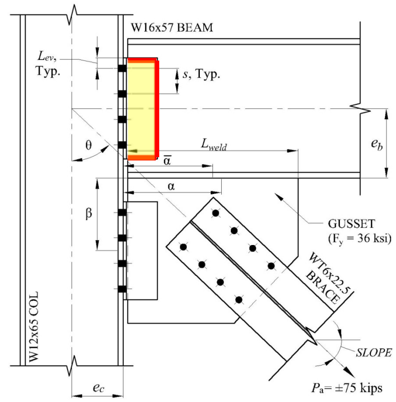
$L_{gv} = k = 3.5$ in.

$L_{gt} = 11.5$ 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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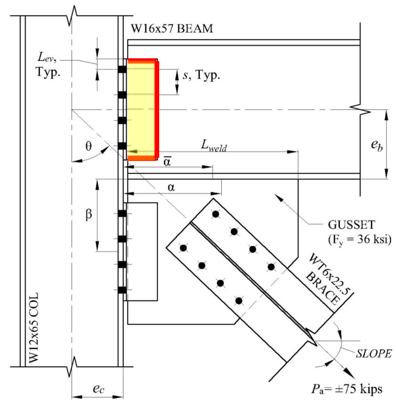
Example VB Corner Connection, I Beam-to-Column (cont.)

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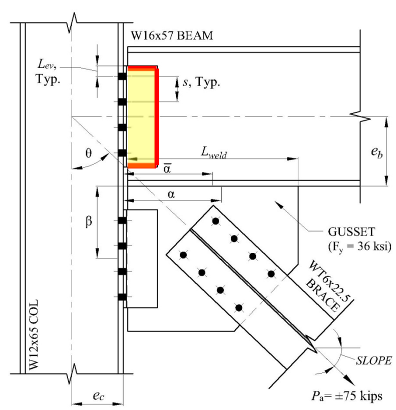
Example VB Corner Connection, I Beam-to-Column (cont.)

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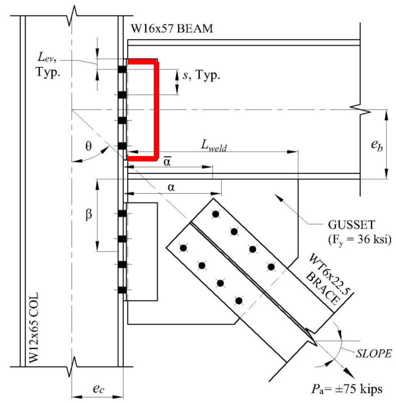
Example VB Corner Connection, I Beam-to-Column (cont.)

Beam-to-Column – Size Weld:

$$P = \sqrt{V^2 + H^2}$$
$$= \sqrt{(67.3\text{ kips})^2 + (18.8\text{ kips})^2}$$
$$= 69.9\text{ kips}$$

$$\theta = \tan^{-1}(H/V) = \tan^{-1}(18.8\text{ kips}/67.3\text{ kips})$$
$$= 15.6°$$

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



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

Beam-to-Column – Size Weld (cont.):

$kl = 3.5$ in.

$k = kl/l = 3.5$ in./11.5 in. = 0.304

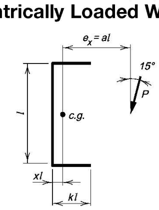
$x = 0.057$

$a = [(al+xl) - xl)]/l$
$= [4\text{ in.} - (0.057)(11.5\text{ in.})]/11.5\text{ in.}$
$= 0.29$

$C = 2.87$ (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.)

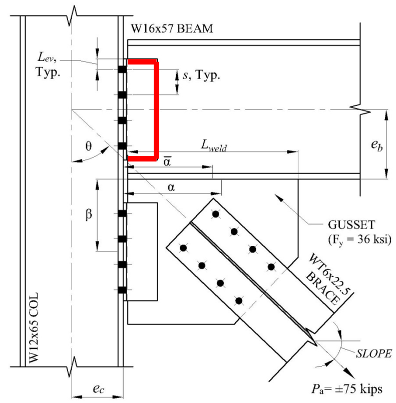
Beam-to-Column – Size Weld (cont.):

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



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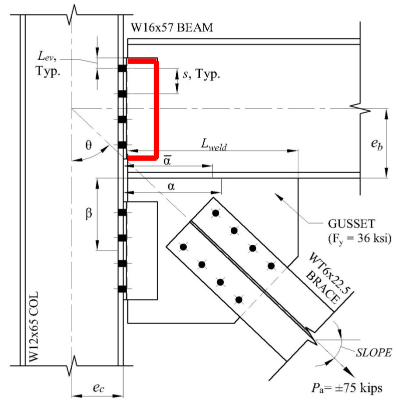
Example VB Corner Connection, I Beam-to-Column (cont.)

Beam-to-Column – Size Weld (cont.):

$D_{min} = P_c\Omega/(2CC_1L)$
$= 69.9 \text{ kips}(2)/((2)(2.87)(1.0)(11.5 \text{ in.}))$
$= 2.12 < 4$ (16^{th} of an inch)
¼” fillet weld o.k.

$t_{min} = 2(0.928D)/(0.6F_u/\Omega)$
$= 2(0.928)(2.12)/[(0.6)(65 \text{ ksi}/2)]$
$= 0.202 \text{ in.} > t_w = 0.43 \text{ in.}$ **o.k.**

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



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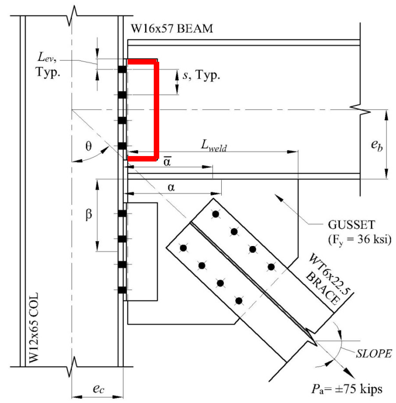
Example VB Corner Connection, I Beam-to-Column (cont.)

Beam-to-Column – Size Weld (cont.):

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

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



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