



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

$$\Omega = 2.00$$

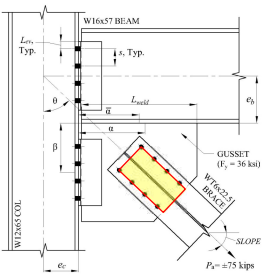
$$R_n = 0.6F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.6F_y A_{gv} + U_{bs} F_u A_{nt}$$

$$\text{Shear Rupture} = 0.6F_u A_{nv}$$

$$\text{Shear Yield} = 0.6F_y A_{gv}$$

$$\text{Tension Rupture} = F_u A_{nt}$$

$U_{bs} = 1.0$ for Direct Loaded Connections



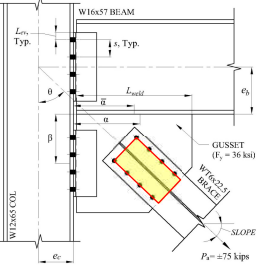
**Example VB Corner Connection, I
 Brace-to-Gusset (cont.)**

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

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

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

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



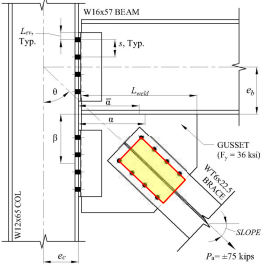
**Example VB Corner Connection, I
 Brace-to-Gusset (cont.)**

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

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

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

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



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

Gusset-to-Column – Block Shear of Angles

OSL (cont.):

$$L_{gt} = L_{ch} = a \text{ from prying calcs}$$

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

$$= [2(4 \text{ in.}) + 0.375 \text{ in.} - 5.5 \text{ in.}]/2$$

$$= 1.44 \text{ in.}$$

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

$$= 1.44 \text{ in.} - 0.5(1 + 1/16 \text{ in.})$$

$$= 0.91 \text{ in.}$$

AISC Specification

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



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

Gusset-to-Column – Block Shear of Angles

OSL (cont.):

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

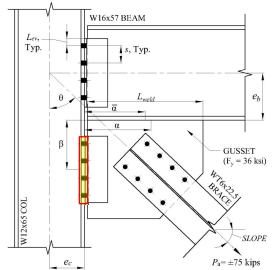
$$= 7.73 \text{ in.}^2$$

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

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

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

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



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

Gusset-to-Column – Block Shear of Angles

OSL (cont.):

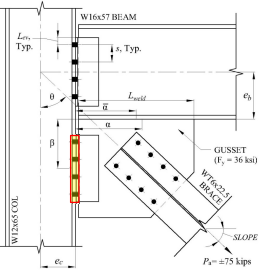
$$R_n = \min \left[\begin{aligned} &0.6(58 \text{ ksi})(5.39 \text{ in.}^2) = 187.57 \text{ kips} \\ &0.6(36 \text{ ksi})(7.73 \text{ in.}^2) = 166.97 \text{ kips} \end{aligned} \right]$$

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

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

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

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



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.)

Table 3-6 (continued)
 Maximum Total Uniform Load, kips $F_y = 50$ ksi
 W-Shapes

Shape	W16x									
	89		77		67		57		50	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
7							282	423	249	372
8							282	394	230	345
9	353	520	300	450			233	350	204	307
10	349	525	290	450	258	386	210	315	184	278
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	158	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)

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

Beam-to-Column – Forces (cont.):

$$V = V_g + V_{b(top)} + V_{H(bot)}$$

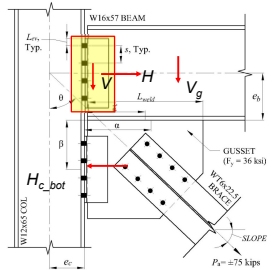
$$= 41.9 \text{ kips} + 0 \text{ kips} + 25.4 \text{ kips}$$

$$= 67.3 \text{ kips}$$

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

$$= \max(18.8 \text{ kips}, 0 \text{ kips} + 0 \text{ kips})$$

$$= 18.8 \text{ kips}$$

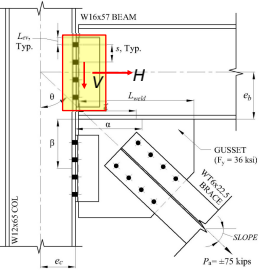


Example VB Corner Connection, I Beam-to-Column (cont.)

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 .



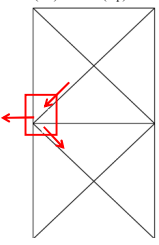
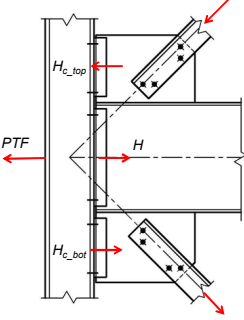
VB Corner Connection

If exact load case not known:

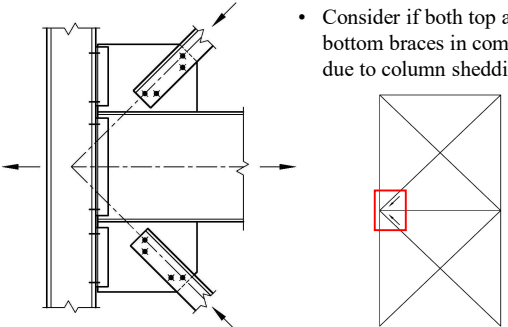
$$H = \max(H_{c(bot)}, H_{c(top)} + PTF)$$

If exact load case is known:


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



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:

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

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

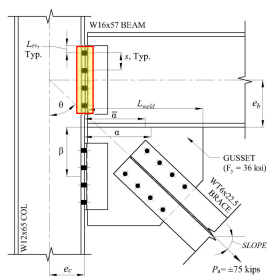

$$= 95.2 \text{ kips} > V = 67.3 \text{ kips o.k.}$$

Check Shear and Tension:

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

$$= 8.41 \text{ kips}$$

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

$$= 2.35 \text{ kips}$$



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

Beam-to-Column – Bolts (cont.):

Check Shear and Tension (cont.):

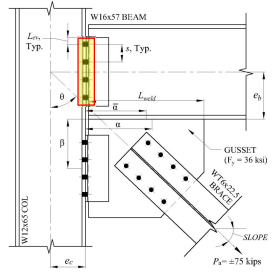

$$A_b = \frac{\pi d_b^2}{4}$$

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

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

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

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

$$= 19.0 \text{ ksi}$$



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

Beam-to-Column – Bolts (cont.):

Check Shear and Tension (cont.):

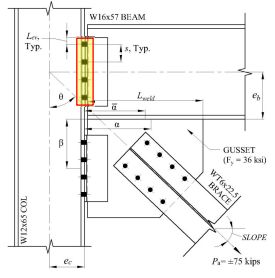

$$F'_{nt} = 1.3F_{nt} - \Omega F_{nt} f_{rv}/F_{nv} \leq F_{nt} \quad (\text{Spec J3-3a})$$

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

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

AISC Specification

TABLE J3.2 Nominal Strength of Fasteners and Threaded Parts, ksi (MPa)		
Description of Fasteners	Nominal Tensile Strength, F_u , ksi (MPa) ⁽¹⁾	Nominal Shear Strength in Bearing-Type Connections, F_u , ksi (MPa) ⁽¹⁾
A307 bolts	45 (310) ⁽¹⁾	27 (188) ⁽¹⁾
Group A (e.g., A325) bolts, when threads are not excluded from shear planes	90 (620)	54 (372)

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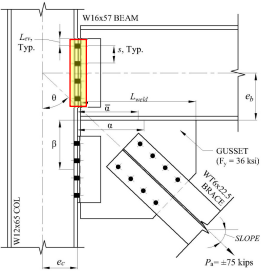
Example VB Corner Connection, I Beam-to-Column (cont.)

Beam-to-Column – Bolts (cont.):
 Check Shear and Tension (cont.):

$$\frac{r_t}{\Omega} = \frac{F_t A_b}{\Omega} \quad (\text{AISC Spec J3-2})$$

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

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



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:

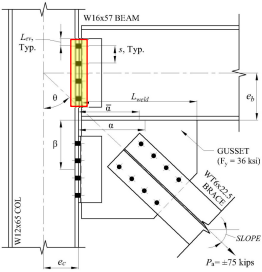
$$T_{bolt} = H/(2n) = 18.8 \text{ kips}/[(2)(4)] = 2.35 \text{ kips}$$

$$b = \text{gage}/2 - t_w/2 - t_f/2$$

$$= 5.5 \text{ in.}/2 - 0.43 \text{ in.}/2 - 0.375/2 = 2.35 \text{ in.}$$

$$b' = b - d_f/2 =$$

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



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

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

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

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

$$= 1.47 \text{ in.}$$

$$a' = a + d_f/2 \leq 1.25b + d_f/2$$

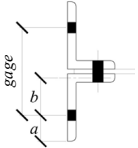
$$= 1.47 \text{ in.} + 0.75 \text{ in.}/2$$

$$\leq 1.25(2.35 \text{ in.}) + 0.75 \text{ in.}/2$$

$$= 1.85 \text{ in.} \leq 3.31 \text{ in.}$$

$$= 1.85 \text{ in.}$$

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



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

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

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

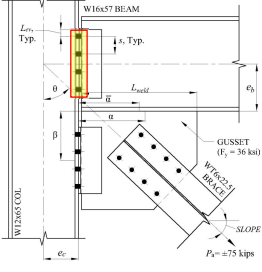
$$= 2.88 \text{ in. (tributary area per bolt)}$$

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

$$= 0.717$$

$$B_c = r_t/\Omega = 11.9 \text{ kips}$$

(bolt available tensile strength shown previously)



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

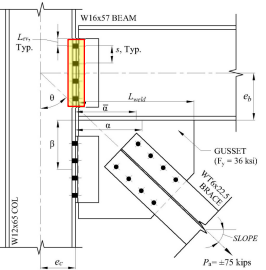
Beam-to-Column – Prying of Angles (cont.):

$$t_c = \sqrt{\frac{Q4B_b b'}{pF_u}} \quad (\text{AISC Manual Eq. 9-26b})$$

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

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

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



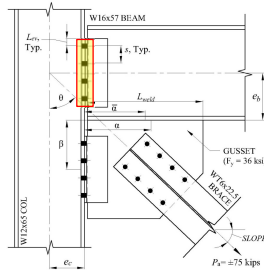
Example VB Corner Connection, I Beam-to-Column (cont.)

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

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

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

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



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

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

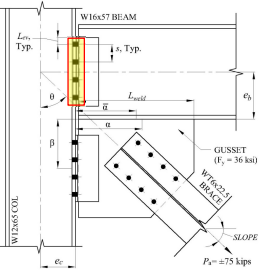
Note:
 When $\alpha' < 0$, this means that the fitting has sufficient strength and stiffness to develop the full bolt available tensile strength,

$$Q = 1$$

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

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

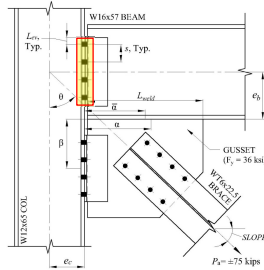
(See AISC Manual Eq. 9-27)



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,
 $Q = \left(\frac{t}{t_c} \right)^2 (1 + \delta)$
 $= \left(\frac{0.375 \text{ in.}}{0.968 \text{ in.}} \right)^2 (1 + 0.717 \text{ in.})$
 $= 0.258$

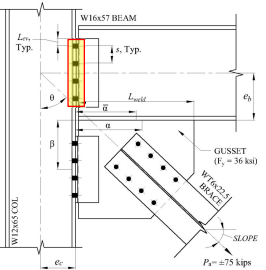


Example VB Corner Connection, I Beam-to-Column (cont.)

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

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

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



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

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

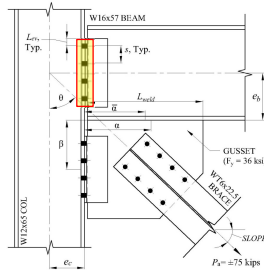
If prying force, q_r , needs to be determined:

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

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

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

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



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

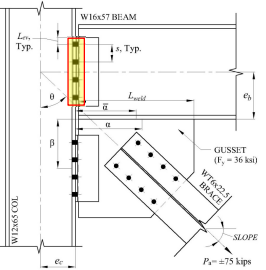
Beam-to-Column – Prying of Angles (cont.):

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

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

$$= 0.599 \text{ kips}$$

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



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

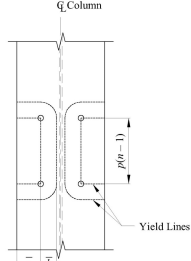
Beam-to-Column – Prying of Column Flange:

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
 $p_{eff} = [p(n-1) + \pi \bar{b} + 2\bar{a}] / n$
 (Ref Akbar Tamboli, *Handbook of Steel Connection Design and Details*, 2nd Ed.)

$$p = \text{spacing} = 3 \text{ in.}$$

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



(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.):

$$\bar{b} = (\text{gage} - t_{wc}) / 2$$

$$= (5.5 \text{ in.} - 0.39 \text{ in.}) / 2 = 2.56 \text{ in.}$$

$$\bar{a} = (b_{fc} - \text{gage}) / 2$$

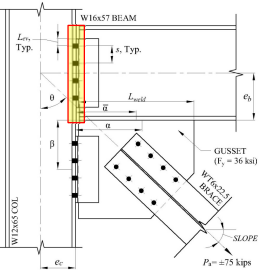
$$= (12 \text{ in.} - 5.5 \text{ in.}) / 2 = 3.25 \text{ in.}$$

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

$$= [(3 \text{ in.})(4-1) + (3.14)(2.56 \text{ in.}) + (2)(3.25 \text{ in.})] / 4$$

$$= 5.88 \text{ in.}$$

Note: use 3" max if at top of column



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

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

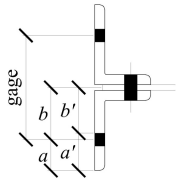
$$T_{bolt} = H_c / 2n = 18.8 \text{ kips} / [(2)(4)] = 2.35 \text{ kips}$$

$$b = \text{gage} / 2 - t_{wc} / 2$$

$$= 5.5 \text{ in.} / 2 - 0.39 \text{ in.} / 2 = 2.56 \text{ in.}$$

$$b' = b - d_p / 2 =$$

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



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

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

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

$$= \min\{(12 \text{ in.} - 5.5 \text{ in.})/2, 1.47 \text{ in.}\}$$

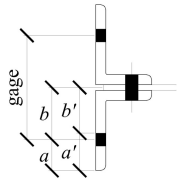
$$= 1.47 \text{ in.}$$

$$a' = a + d_p / 2 \leq 1.25b + d_p / 2$$

$$= 1.47 \text{ in.} + 0.75 \text{ in.} / 2 \leq 1.25(2.56 \text{ in.}) + 0.75 \text{ in.} / 2$$

$$= 1.85 \text{ in.} < 3.57 \text{ in.} \text{ Use } 1.85 \text{ in.}$$

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



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

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

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

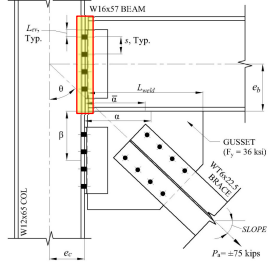
$$\delta = 1 - d' / p = 1 - (0.8125 \text{ in.} / 5.88 \text{ in.}) = 0.862$$

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

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

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

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



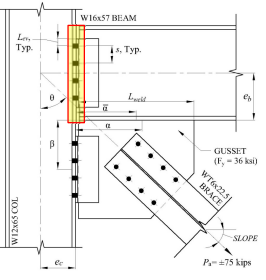
Example VB Corner Connection, I Beam-to-Column (cont.)

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

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

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

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



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

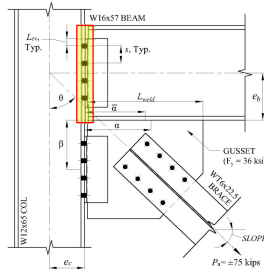
Beam-to-Column – Prying of Column Flange (cont.):

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

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

$$= 0.896$$

The available strength of bolt including prying is
 $T_c = B_c Q = (11.9 \text{ kips})(0.896)$
 $= 10.7 \text{ kips} > T_{bolt} = 2.35 \text{ kips o.k.}$



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

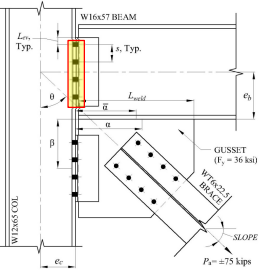
Beam-to-Column – Bolt Bearing/Tearout:

$$R_n = 2.4d_t F_u \quad \text{AISC Specification Eq. J3-6a}$$

$$R_n = 1.2l_c t F_u \quad \text{AISC Specification Eq. J3-6c}$$

AISC Bolt Shear:
 $r_s/\Omega = 11.9 \text{ kips}$

Column Bolt Bearing:
 $R_{brg(col)}/\Omega = 2.4d_b t_f F_u / \Omega$
 $= 2.4(0.75 \text{ in.})(0.605 \text{ in.})(65 \text{ ksi})/2$
 $= 35.4 \text{ kips}$



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

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

Column Bolt Tear Out (Center):

$$R_{tear(col_c)}/\Omega = 1.2(s - d_{h_w}) t_f F_u / \Omega$$

$$= 1.2 (3 \text{ in.} - 13/16 \text{ in.}) \times (0.605 \text{ in.})(65 \text{ ksi})/2$$

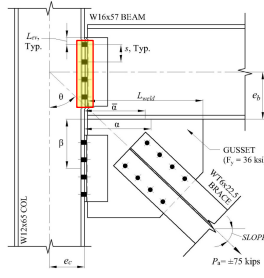
$$= 51.6 \text{ kips}$$

Angle Bolt Bearing:

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

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

$$= 19.6 \text{ kips}$$

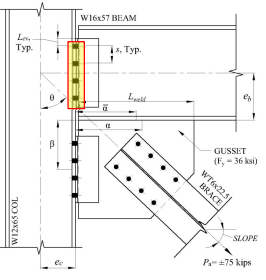


Example VB Corner Connection, I Beam-to-Column (cont.)

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

Angle Bolt Tear Out (Edge):
 $R_{tear(L_E)}/\Omega = 1.2(L_{ev} - d_{h_w}/2)t_a F_u/\Omega$
 $= 1.2(1.25 \text{ in.} - \frac{13}{16} \text{ in.}/2)$
 $\times (0.375 \text{ in.})(58 \text{ ksi})/2$
 $= 11.0 \text{ kips}$

Angle Bolt Tear Out (Center):
 $R_{tear(L_C)}/\Omega = 1.2(s - d_{h_w})t_a F_u/\Omega$
 $= 1.2(3 \text{ in.} - \frac{13}{16} \text{ in.})(0.375 \text{ in.})$
 $\times (58 \text{ ksi})/2$
 $= 28.6 \text{ kips}$

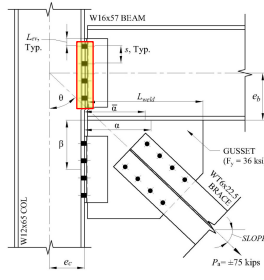


Example VB Corner Connection, I Beam-to-Column (cont.)

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

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

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

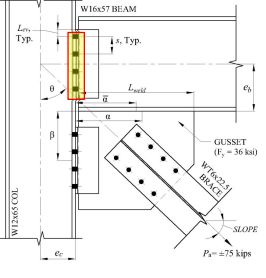


Example VB Corner Connection, I Beam-to-Column (cont.)

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

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

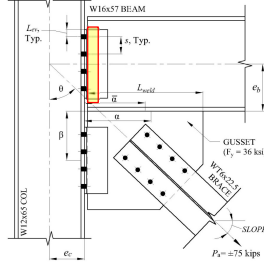
Total Bolt Strength
 $R_n/\Omega = [R_{n(top)}/\Omega + (R_{n(center)}/\Omega)(n - 2)$
 $+ R_{n(bot)}/\Omega](2 \text{ lines})$
 $= [11.9 \text{ kips} + (11.9 \text{ kips})(4 - 2)$
 $+ 11.0 \text{ kips}](2)$
 $= 93.4 \text{ kips} > V = 67.3 \text{ kips o.k.}$



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

Beam-to-Column – Angle Gross Shear:

AISC Specification Eq. J4-3
 $\frac{R_n}{\Omega} = \frac{0.6F_y A_g}{\Omega}$
 $= \frac{0.6(36 \text{ ksi})(11.5 \text{ in.})(0.375 \text{ in.})}{1.5}$
 $= 124 \text{ kips} > V = 67.3 \text{ kips o.k.}$



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

Beam-to-Column – Angle Net Shear:

AISC Specification Eq. J4-4

$$L_{nv} = L_{eff} - n(d_h \text{ width} + 1/16 \text{ in.})$$

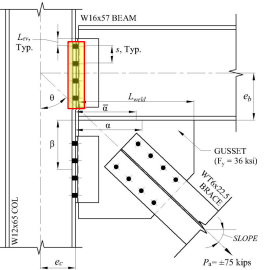
$$= 11.5 \text{ in.} - 4(1 3/16 \text{ in.} + 1/16 \text{ in.})$$

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

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

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

$$= 104 \text{ kips} > V = 67.3 \text{ kips o.k.}$$



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

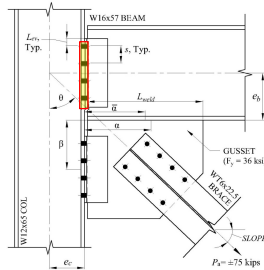
$$\Omega = 2.00$$

$$R_n = 0.6F_u A_{nv} + U_{bs} F_u A_{nt}$$

$$\leq 0.6F_y A_{gv} + U_{bs} F_u A_{nt}$$

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



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

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

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

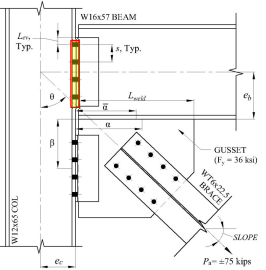
$$= 10.3 \text{ in.}$$

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

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

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

$$= 7.19 \text{ in.}$$



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

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

$$L_{gt} = L_{ch} = a \text{ from prying calcs}$$

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

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

$$= 1.47 \text{ in.}$$

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

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

$$= 0.934 \text{ in.}$$

AISC Specification

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



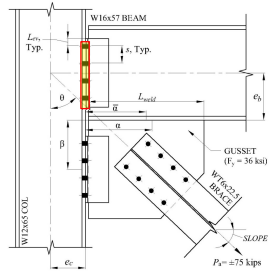
Example VB Corner Connection, I Beam-to-Column (cont.)

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

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

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

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



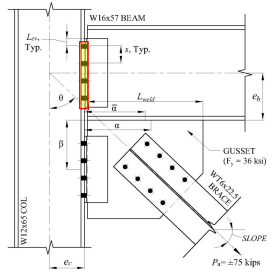
Example VB Corner Connection, I Gusset-to-Column (cont.)

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

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

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

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



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

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

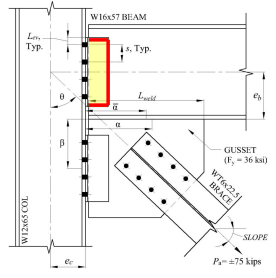
AISC Spec Section J4.3 Block Shear Strength

$$\Omega = 2.00$$

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

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



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

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

For Axial check:

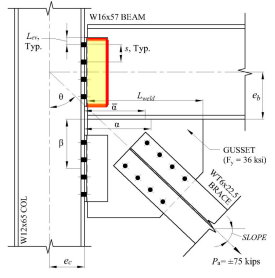
$$L_{gv} = k = 3.5 \text{ in.}$$

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

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

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

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



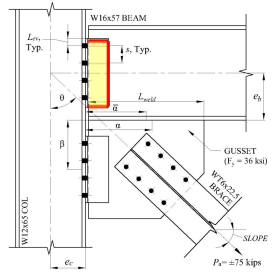
Example VB Corner Connection, I Beam-to-Column (cont.)

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

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

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

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



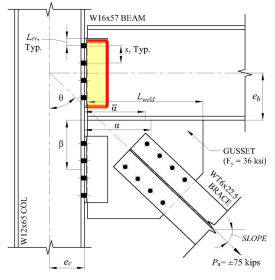
Example VB Corner Connection, I Beam-to-Column (cont.)

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

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

$$= 90.30 \text{ kips} + 321.75 \text{ kips} = 412.05 \text{ kips}$$

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



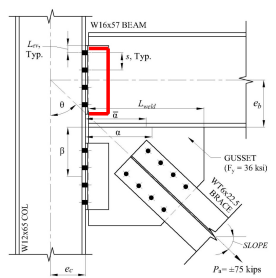
Example VB Corner Connection, I Beam-to-Column (cont.)

Beam-to-Column – Size Weld:

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

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

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



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

Beam-to-Column – Size Weld (cont.):

$$kl = 3.5 \text{ in.}$$

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

$$x = 0.057$$

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

$$C = 2.87 \text{ (AISC Manual Table 8-8 with Angle} = 15^\circ, \text{ round down)}$$

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

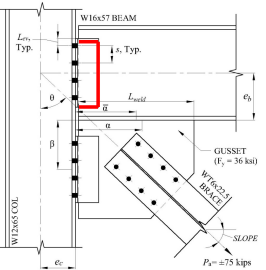
a	k											
	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2
0.00	1.98	2.07	2.11	2.26	4.30	4.65	5.19	5.74	6.28	6.80	7.37	8.46
0.10	1.90	2.25	2.67	3.41	3.95	4.50	5.03	5.60	6.15	6.70	7.24	8.34
0.15	1.84	2.30	2.79	3.30	3.81	4.33	4.88	5.39	5.92	6.45	6.98	8.08
0.20	1.78	2.21	2.68	3.21	3.72	4.25	4.75	5.25	5.76	6.28	6.80	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	7.05



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)

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.):

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

$$= 69.9 \text{ kips} / ((2)(2.87)(1.0)(11.5 \text{ in.}))$$

$$= 2.12 < 4 \text{ (16th of an inch)}$$

1/4" fillet weld o.k.

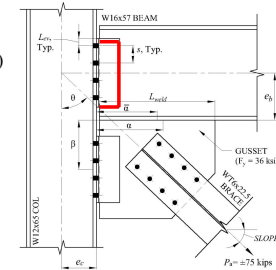
$$t_{min} = 2(0.928D) / (0.6F_u / \Omega)$$

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

$$= 0.202 \text{ in.} > t_w = 0.43 \text{ in. o.k.}$$

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

$$t_{min} = 6.19D / F_u$$



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

Beam-to-Column – Size Weld (cont.):
 If $t_{min} < t_p$, no reduction necessary.

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

