Night School 28: Vertical Bracing Connections

Thank you for joining our live webinar. We will begin shortly. Please standby.

Vertical Bracing Connections, Session 5: Uniform Force Method Part 2—Special Case IV and V

May 3, 2022 | William A Thornton

Smarter. Stronger. Steel.

AISC Night School

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Course Description

Vertical Bracing Connections

Session 5: Uniform Force Method Part 2—Special Case IV and V

May 3, 2022

This session will look at two cases of the uniform force method: Special Case IV, and Special Case V. A design example will be presented to demonstrate applicability.

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Learning Objectives

1. Describe the Uniform Force Method – Special Case IV
2. Explain the advantages of Special Case IV
3. Describe the Uniform Force Method – Special Case V
4. Explain the advantages of Special Case V

Night School 28: Vertical Bracing Connections

Session 5: Uniform Force Method Part 2—Special Case IV and V

May 3, 2022

William A. Thornton, corporate consultant to Cives Steel

Smarter. Stronger. Steel.

Vertical Bracing Connections

By: William Thornton, Rafael Sabelli, and Carol Drucker

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Course Outline

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

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Session Outline

- Development of the Uniform Force Method Special Case IV
- Development of the Uniform Force Method Special Case V

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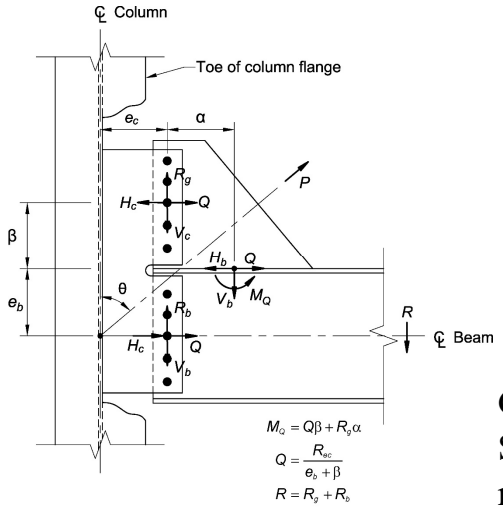
UFM Special Case IV



General notation for forces. See free body diagram on next two slides.

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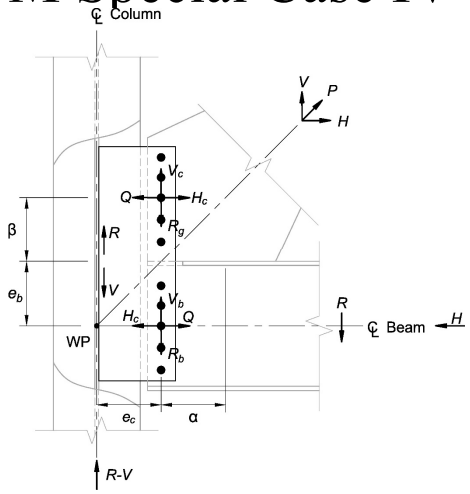
UFM Special Case IV



Free Body Diagram of the Plate
(all forces are shown on the plate)

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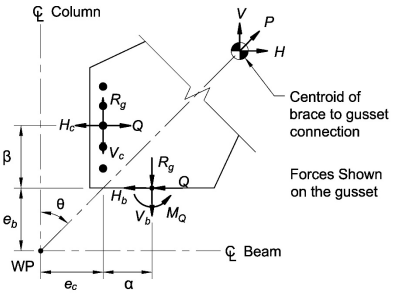
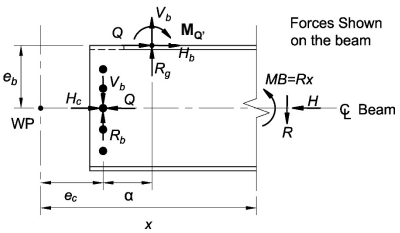
UFM Special Case IV



Free Body Diagram of the gusset



Free Body Diagram of the beam

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UFM Special Case IV

Derivation of Equations

Gusset

$$\circlearrowleft_{+} \quad \Sigma M_{Wp} = 0 = R_g e_c - R_g(e_c + \alpha) - Q\beta + M_Q = 0$$

$$0 = -R_g\alpha - Q\beta + M_Q$$

$$M_Q = Q\beta + R_g\alpha$$

Beam

$$\circlearrowleft_{+} \quad \Sigma M_{Wp} = 0 = -M_Q' - Qe_b + R_b e_c + R_g(e_c + \alpha) - Rx + Rx$$

$$M_Q' = -Qe_b + \mathrm{Re}_c + R_g\alpha$$

$$= -Qe_b + \mathrm{Re}_c + R_g\alpha$$

Using: $\alpha - \beta \tan\theta = e_b \tan\theta - e_c$

and $Q = \dfrac{\mathrm{Re}_c}{e_b + \beta}$

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UFM Special Case IV

Cont'd.

$$M_Q' = \frac{-\mathrm{Re}_c e_b}{e_b + \beta} \mathrm{Re}_c + R_g\alpha$$

$$= \frac{R}{e_b + \beta}(-e_c e_b + e_c(e_b + \beta)) + R_g\alpha$$

$$M_Q' = \frac{R}{e_b + \beta}(-e_c e_b + e_c e_b + e_c\beta) + R_g\alpha$$

$$= \frac{R}{e_b + \beta}(e_c\beta) + R_g\alpha = Q\beta + R_g\alpha = M_Q \rightarrow \text{QED}$$

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Example of Vertical Bracing Corner Connection Special Case IV

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Example Special Case IV

Given:

1. AISC 15th Edition *Manual*, LRFD
2. 1 in. diameter ASTM A325-N bolts
3. STD Holes, $1^1/8$ in. diameter
4. Brace Slope = 45°
5. Plate: ASTM A572 Gr. 50, F_y = 50 ksi, F_u = 65 ksi
6. HSS : ASTM A500 Gr. C, F_y = 50 ksi, F_u = 62 ksi

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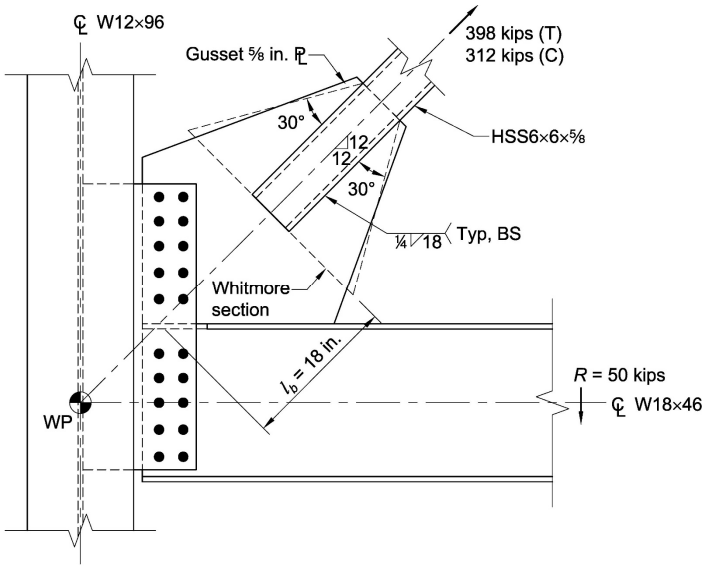
Example Special Case IV



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Example Special Case IV

Weld (*Spec*. Section J2.4)

Use a ¼" fillet weld to keep weld passes to one.

$$\phi R_n = 0.75(0.6)(F_{EXX})t_e l n$$

$$= 0.75(0.6)(70)\left(\frac{D}{16\sqrt{2}}\right) \ln$$

$$= 1.392 D \ln$$

D = no. of 16ths

l = length

n = no. of welds

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Example Special Case IV

Weld cont'd.

$$\phi R_n = 1.392(4)(l)(4) \geq 398$$

$$l \geq \frac{398}{1.392(16)} = 17.9" \rightarrow \text{use 18" of 1/4" fillet weld}$$

HSS Wall

$$\phi R_n = 0.9(0.6)(50)(18)(0.581)(4) = 1129k > 398k \quad -\text{OK}$$

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Example Special Case IV

Gusset Yield and Buckling

Whitmore section

$$l_{whitmore} = (18\tan 30^\circ)(2) + 6 = 26.8in.$$

Whitmore falls outside of gusset

$$l_{\text{whitmore}} \text{ (available)} = 8 + 8 + 6 = 22in.$$

Yield

$$\phi R_n = 0.9(1.0)(50)(22)(t_g) \geq 398k$$

$$t_g \geq 0.4023 \rightarrow \text{try 1/2" plate}$$

Buckling - Buckling length $l_b = 18in.$

$$K = 0.65$$

cont'd.

$$Kl/r = \frac{0.65 \times 18}{0.5}\sqrt{12} = 81.1$$

From *Manual* Table 4-14

$$\phi F_{cr} = 27.5ksi @ Kl/r = 82$$

$$\phi P_{cr} = (27.5)(0.5)(22) = 302k < 312k \quad -NG$$

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Example Special Case IV

Try 5/8" plate

$$Kl/r = \frac{0.65 \times 18}{0.625}\sqrt{12} = 64.8$$

From *Manual* Table 4-14

$$\phi F_{cr} = 33.0 ksi \,@\, Kl/r = 65$$

$$\phi P_{cr} = (33.0)(0.625)(22) = 454k > 312k \quad -\text{OK}$$

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Example of Corner Connection Special Case IV

Brace-to-Gusset – Brace Check – Shear Lag:

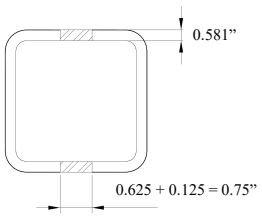
Effective Area:

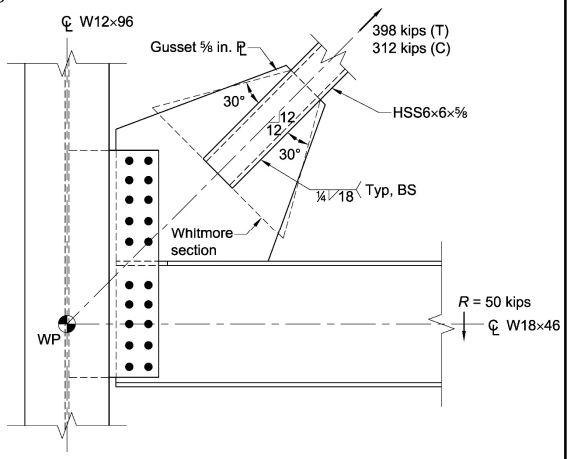
$$A_g = 11.7 \text{ in.}^2$$

$$A_n = A_g - 2t_{HSS\ wall}t_{slot}$$

$$= 11.7 \text{ in.}^2 - 2(0.581 \text{ in.})(0.75 \text{ in.})$$

$$= 10.80 \text{ in.}^2$$





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Example of Corner Connection Special Case IV

Brace-to-Gusset – Shear Lag (cont.):

AISC *Specification* Table D3.1, Case 6,

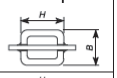
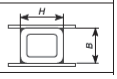
$$\bar{x} = \frac{B^2 + 2BH}{4(B+H)} = \frac{(6.0\text{ in.})^2 + 2(6.0\text{ in.})(6.0\text{ in.})}{4(6.0\text{ in.} + 6.0\text{ in.})}$$

$$= 2.25\text{ in.}$$

$$U = 1 - \frac{\bar{x}}{l_{conn}} = 1 - \frac{2.25\text{ in.}}{18.0\text{ in.}}$$

$$= 0.875$$

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

Case	Description of Element		Shear Lag Factor, *U*	Example
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	$l \geq H \ldots U = 1 - \bar{x}/l$ $\bar{x} = \frac{B^2}{4(B+H)}$	

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Example of Corner Connection Special Case IV

Brace-to-Gusset – Shear Lag (cont.):

AISC *Specification* Eq. D3-1,

$$A_e = UA_n = 0.875\left(10.8\text{ in.}^2\right)$$

$$= 9.45\text{ in.}^2$$

$$\phi Rn = 0.75(62)(9.45)$$

$$= 439k > 398k \quad -\text{OK}$$

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Determination of Interface Forces

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Example of Corner Connection Special Case IV

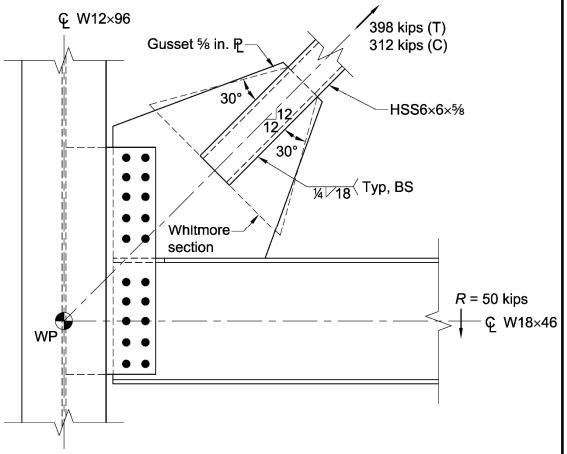
Determination of Interface Forces – Uniform Force Method (Special Case IV):

Using the uniform force method (UFM), with a = 8.875 in. (AISC *Manual* p. 10-103) take,

$$e_c = a + \frac{s_{bolts}}{2} = 8.875 \text{ in.} + \frac{3.0 \text{ in.}}{2}$$
$$= 10.375 \text{ in.}$$

The web connection is treated as a flange connection of an equivalent column at depth of 2(10.375 in.) = 20.75 in., that is a W21 column.

$$e_b = \frac{d_{beam}}{2} = \frac{18.1 \text{ in.}}{2}$$
$$= 9.05 \text{ in.}$$



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Example of Corner Connection
Special Case IV

Determination of Interface Forces:

Using the brace to gusset connection as a guide, 5 rows of bolts will be required for the gusset to shear plate connection.

Strength calculations (not shown) require that there are 2 columns of bolts.

So choose,

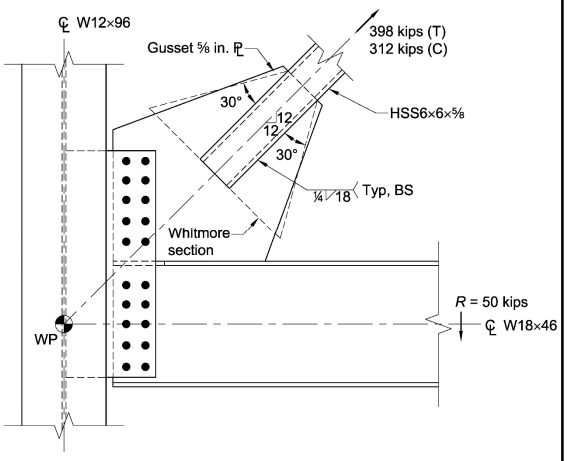
$$\beta = \overline{\beta} = 9.0 \text{ in.}$$

Then,

$$\alpha - \beta \tan\theta = e_b \tan\theta - e_c$$

$$\alpha = (9.05 \text{ in.} + 9.0 \text{ in.}) \tan 45° - 10.375 \text{ in.}$$

$$= 7.675 \text{ in.}$$



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Example of Corner Connection
Special Case IV

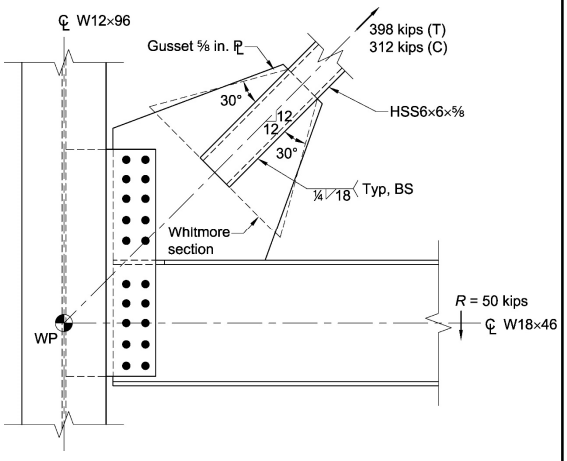
Determination of Interface Forces (cont.):

Choose the length of the gusset to beam connection such that the centroid of this connection falls at

$$\alpha = \overline{\alpha} = 7.675 \text{ in.}$$

Remember that α is measured from the face of the column flange, at

$$e_c = 10.375 \text{ in.}$$



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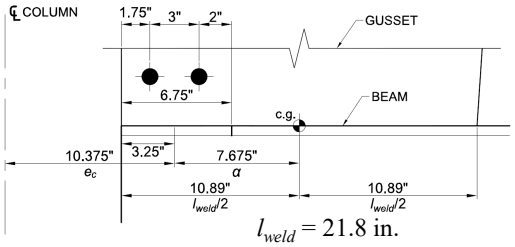
Example of Corner Connection
Special Case IV

Determination of Interface Forces (cont.):

Length of the gusset to beam connection,



$l_{weld} = 21.8$ in.

Distance from beam end to extent of blocking = 1.75 in.+ 3.0 in.+ 2.0 in.
= 6.75 in.

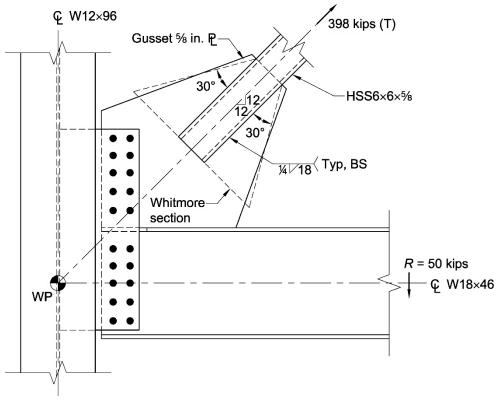
Length of weld beyond block = 21.8 in. – 6.75 in.
= 15.0 in

Example of Corner Connection
Special Case IV

Determination of Interface Forces (cont.): Tension Case

$$r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2}$$
$$= \sqrt{(7.675 \text{ in.} + 10.375 \text{ in.})^2 + (9.0 \text{ in.} + 9.05 \text{ in.})^2}$$
$$= 25.53 \text{ in.}$$

$$\frac{P}{r} = \frac{398 \text{ kips}}{25.53 \text{ in.}} = 15.59 \text{ kips/in.}$$



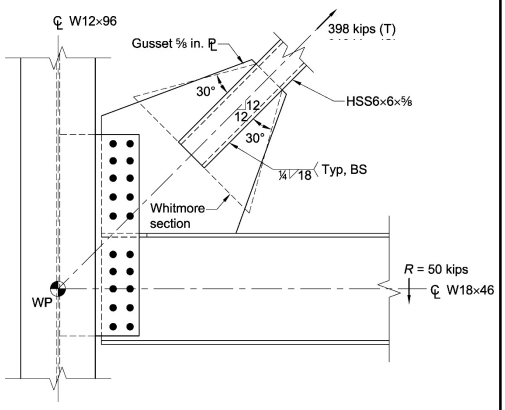
Example of Corner Connection
Special Case IV

Determination of Interface Forces (cont.): Tension Case

$$H_c = e_c\left(\frac{P}{r}\right) = 10.375 \text{ in.}(15.59 \text{ kips/in.})$$
$$= 162 \text{ kips}$$

$$H_b = \alpha\left(\frac{P}{r}\right) = 7.675(15.59 \text{ kips/in.})$$
$$= 120 \text{ kips}$$

$$\Sigma_H = 162 \text{ kips} + 120 \text{ kips}$$
$$= 282 \text{ kips} = 398(\cos 45°)$$
$$= 281 \text{ kips} \quad -\text{OK due to rounding}$$



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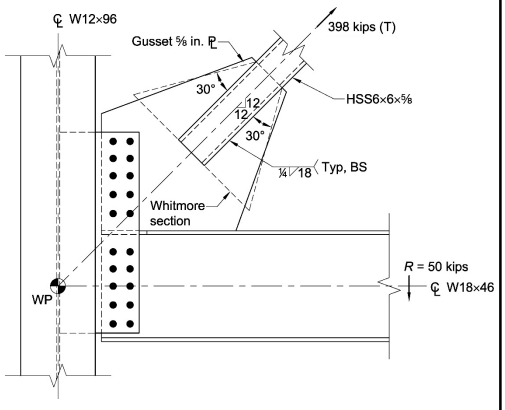
Example of Corner Connection
Special Case IV

Determination of Interface Forces (cont.): Tension Case

$$V_c = \beta\left(\frac{P}{r}\right) = 9.0 \text{ in.}(15.59 \text{ kips/in.})$$
$$= 141 \text{ kips}$$

$$V_b = e_b\left(\frac{P}{r}\right) = 9.05 \text{ in.}(15.59 \text{ kips/in.})$$
$$= 141 \text{ kips}$$

$$\Sigma_V = 141 \text{ kips} + 141 \text{ kips}$$
$$= 282 = 398 \text{ kips}(\sin 45°)$$
$$= 281 \text{ kips} \quad -\text{OK due to rounding}$$



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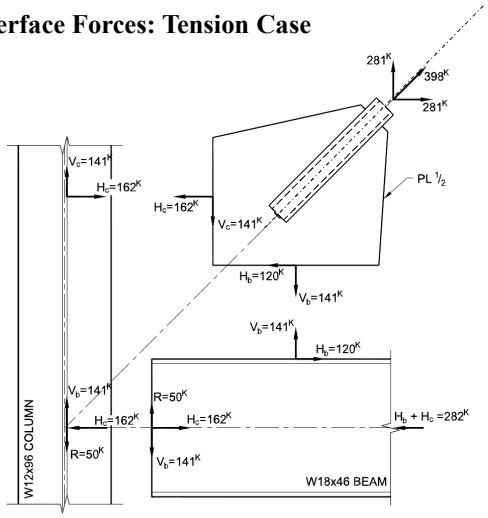
Example of Corner Connection
Special Case IV

Exposition of Interface Forces: Tension Case



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Example of Corner Connection
Special Case IV

Exposition of Interface Forces: Tension Case

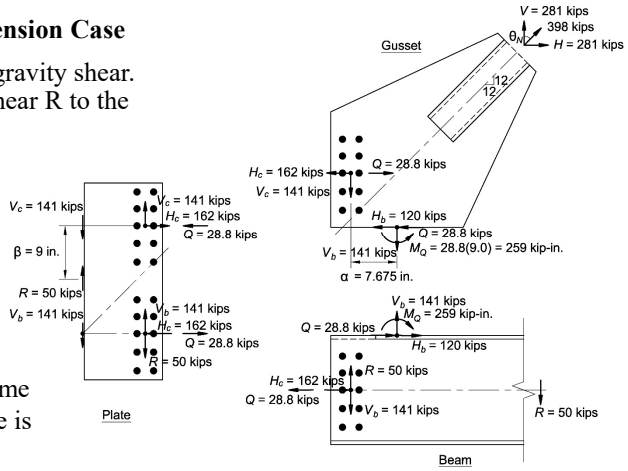
- The force, Q, is due to the beam gravity shear. It is required to move the beam shear R to the face of the column web.

$$M_Q = R\beta + R_g\alpha$$

$$Q = \frac{\mathrm{Re}_c}{e_{b+}\beta}$$

$$R = R_g + R_b$$

These forces always act in the same direction, whether the brace force is tension or compression.



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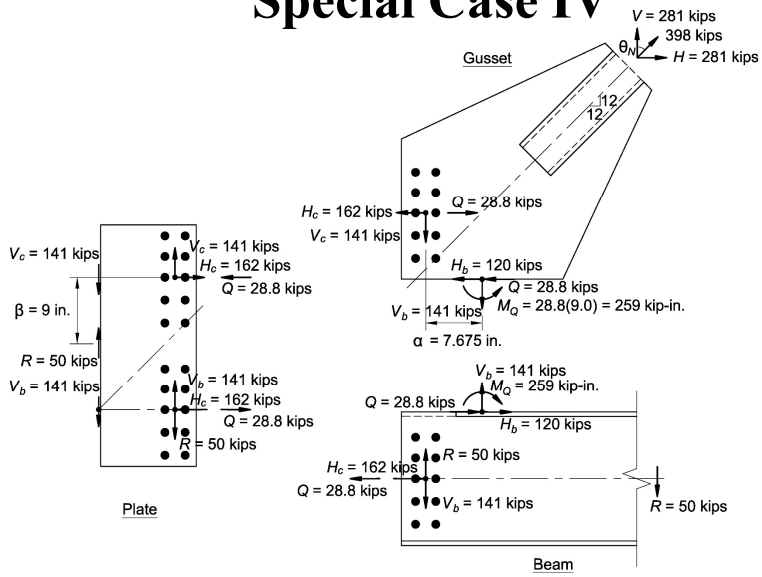
Example of Corner Connection
Special Case IV



Interface forces for brace tension of 398k

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Example of Corner Connection
Special Case IV

Distribution of Forces (UFM + Effect of *R*): Brace Tension

The beam reaction acts at the face of the column web.

$$R(10.375\text{ in.}) = Q(18.01\text{ in.})$$

$$Q = \frac{50\text{ kips}(10.375\text{ in.})}{18.01\text{ in.}}$$

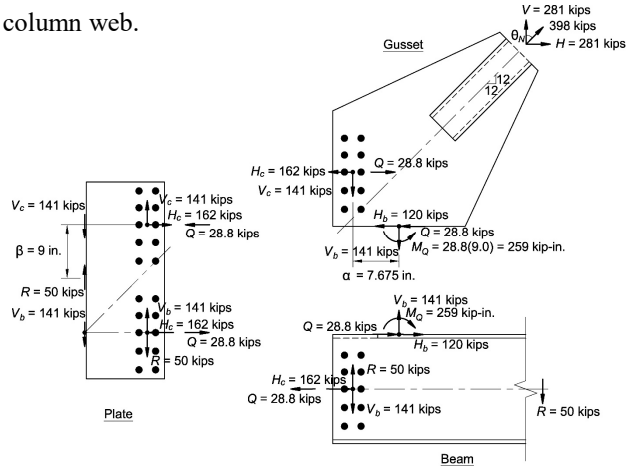
$$= 28.8\text{ kips}$$

$$R_g = 0 \qquad R_b = R$$

$$M_Q = Q\beta + R_g\alpha$$

$$= 28.8(9) + 0(7.675)$$

$$= 259k - in$$



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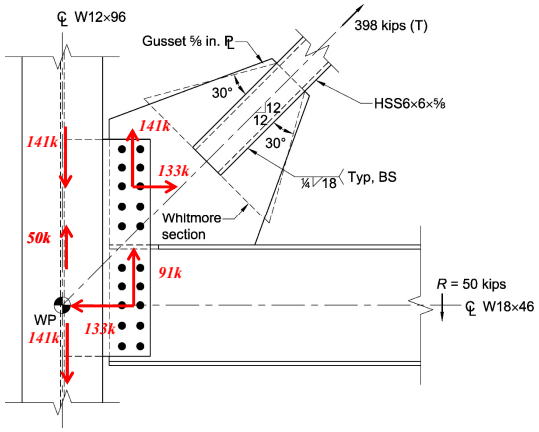
Example of Corner Connection
Special Case IV



Net Interface Forces on Plate (Tension)

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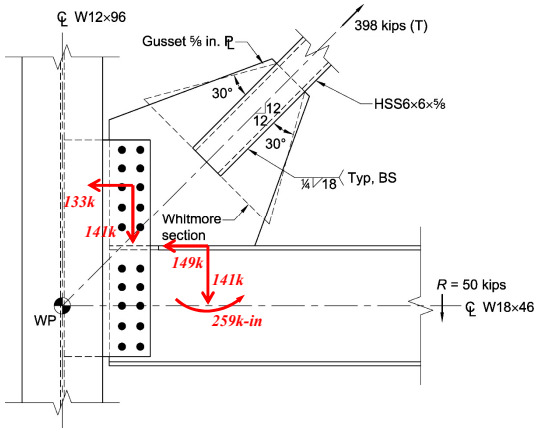
Example of Corner Connection
Special Case IV



Net Interface Forces on Gusset (Tension)

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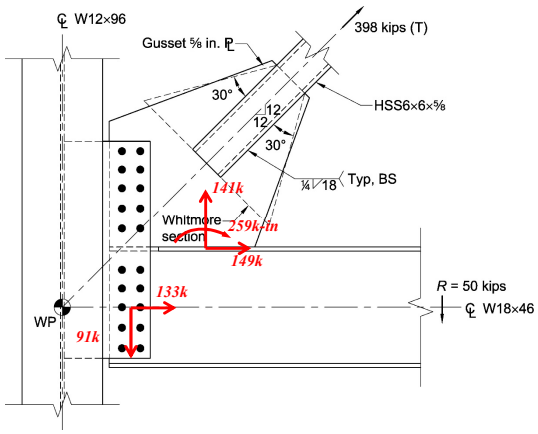
Example of Corner Connection
Special Case IV



Net Interface Forces on Beam (Tension)

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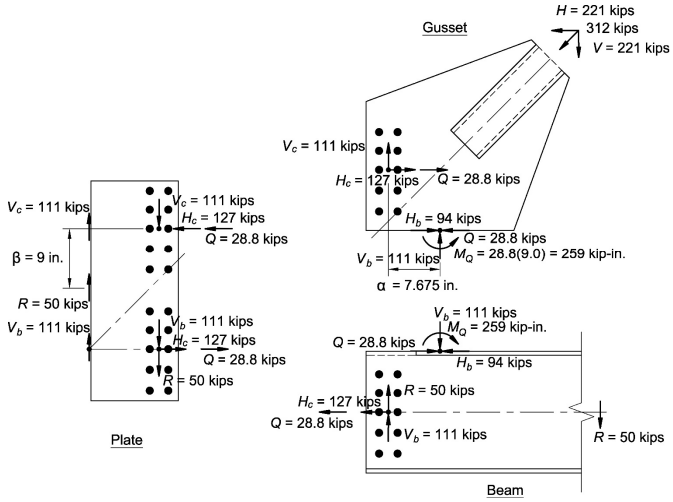
Example of Corner Connection
Special Case IV



Interface Forces Brace Compression

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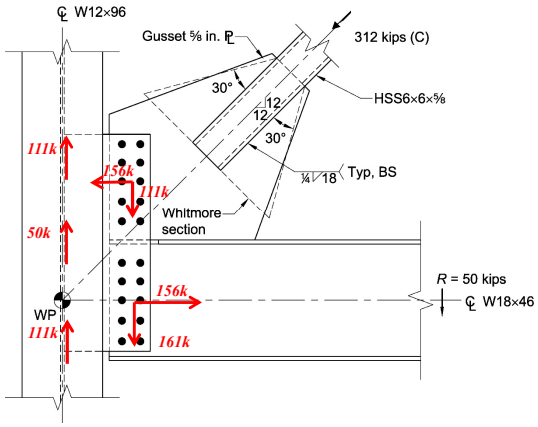
Example of Corner Connection
Special Case IV



Net Interface Forces on Plate (Compression)

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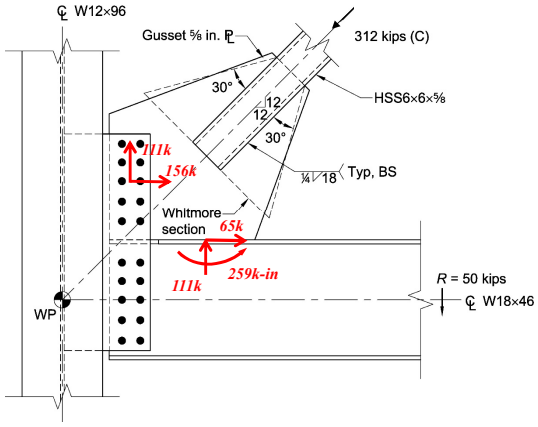
Example of Corner Connection
Special Case IV



Net Interface Forces on Gusset (Compression)

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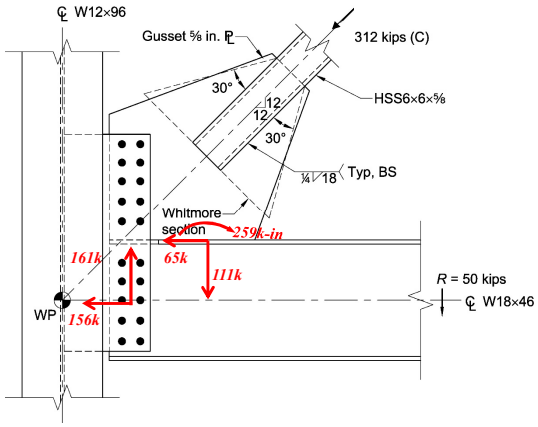
Example of Corner Connection
Special Case IV



Net Interface Forces on Beam (Compression)

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Example of Corner Connection Gusset-to-Column

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Example of Corner Connection
Special Case IV

Plate: Gusset-to-Column – Bolt Shear: Table 7-1

Brace Tension H=133k, V=141k

Brace Compression H=156k, V=111k

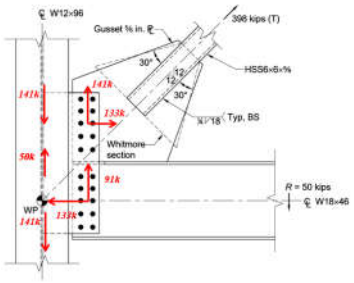
Tension controls

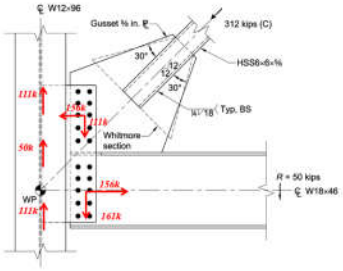
$$R = \sqrt{133^2 + 141^2} = 194k$$

$$r_{bolt} = \frac{R}{n} = \frac{194 \text{ kips}}{10 \text{ bolts}}$$

$$= 19.4 \text{ kips/bolt} < \phi r_v = 31.8 \text{ kips/bolt} \quad -\text{OK}$$

All bolts see 19.4 kips





47

Example of Corner Connection
Special Case IV

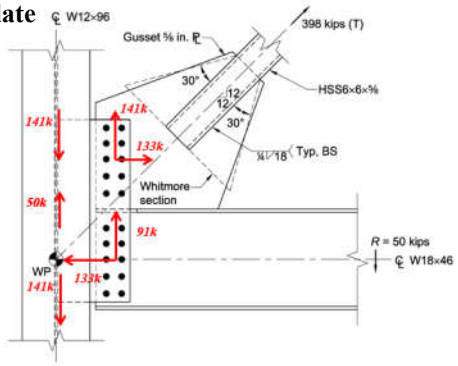
Plate: Gusset to Column Bearing and Tearout of Plate

AISC Specification J3.10

Tearout $\phi r_{to} = 0.75(1.2)(F_u)(l_c)(t_g)$

Bearing $\phi r_{pb} = 0.75(2.4)(F_u)(d_b)(t_g)$

Because of Special Case IV, all bolt groups are concentrically loaded



Net Interface Forces on Plate (Tension)

48

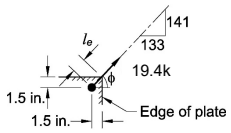
Example of Corner Connection
Special Case IV

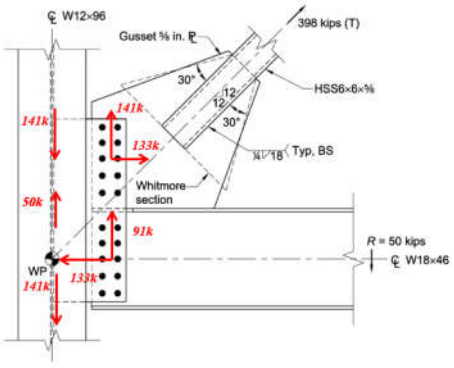
The critical bolt is the top right bolt

$$\phi = \tan^{-1}\frac{141}{133} = 46.7^\circ$$

Since $46.7^\circ > 45^\circ$, the distance to horizontal edge of plate controls

$$\cos\phi = \frac{1.5}{l_e} \rightarrow l_e = \frac{1.5}{\cos 43.3^\circ} = 2.06"$$





Net Interface Forces on Plate (Tension)

49

Example of Corner Connection
Special Case IV

Tearout

$$l_c = l_e - \frac{1}{2}d_h$$

$$= 2.06 - \frac{1}{2}(1.125) = 1.50"$$

$$\phi r_{to} = 0.75(1.2)(65)(1.50)(t_p) \geq 19.4$$

$$t_p \geq 0.221 \rightarrow 3/8 \text{ plate ok, so far}$$

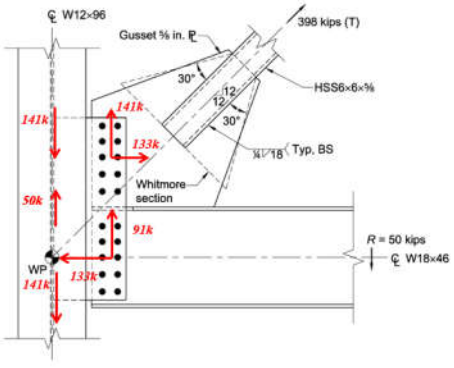
Bearing

$$\phi r_{pb} = 0.75(2.4)(65)(1.0)(0.375)$$

$$= 43.9 > 19.4k \quad -\text{OK}$$

3/8 plate ok, so far

Remember, gusset is 5/8 plate



Net Interface Forces on Plate (Tension)

50

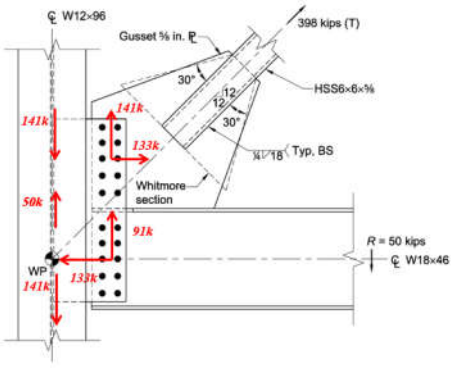
Example of Corner Connection
Special Case IV

Plate: Gusset-to-Column– Block Shear – Shear Plate (Vertical Load):

From AISC *Specification* Section J4.3

$$A_{gt} = \left[\left(n_h - 1\right)s_h + L_{eh}\right]t_{pl}$$
$$= \left[\left(2-1\right)\left(3.0\text{ in.}\right)+1.50\text{ in.}\right]\left(0.375\text{in.}\right)$$
$$= 1.6875\text{ in.}^2$$

$$A_{nt} = A_{gt} - \left(n_h - 0.5\right)d_{hole}t_{pl}$$
$$= 1.6875\text{ in.}^2 - \left(2-0.5\right)\left(1.1875\text{ in.}\right)\left(0.375\text{ in.}\right)$$
$$= 1.019\text{ in.}^2$$



Net Interface Forces on Plate (Tension)

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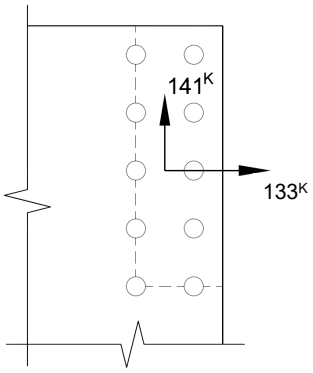
Example of Corner Connection
Special Case IV

Plate: Gusset-to-Column– Block Shear – Shear Plate (Vertical Load) (cont.):

From AISC *Specification* Eq. J4-3,

$$A_{gv} = \left[\left(n_v - 1\right)s_v + L_{ev}\right]t_{pl}$$
$$= \left[\left(5-1\right)\left(3.0\text{ in.}\right)+1.50\text{ in.}\right]\left(0.375\text{in.}\right)$$
$$= 5.0625\text{ in.}^2$$

$$A_{nv} = A_{gv} - \left(n_v - 0.5\right)d_{hole}t_{pl}$$
$$= 5.0625\text{ in.}^2 - \left(5-0.5\right)\left(1.25\text{ in.}\right)\left(0.375\text{ in.}\right)$$
$$= 2.95\text{ in.}^2$$



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Example of Corner Connection
Special Case IV

Plate: Gusset-to-Column– Block Shear – Shear Plate (Vertical Load) (cont.):

From AISC *Specification* Eq. J4-5,

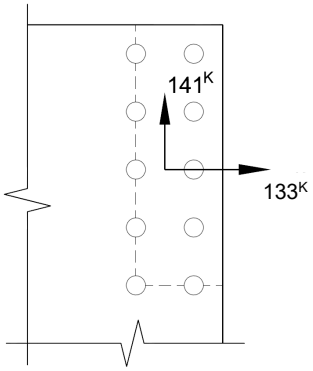
$$\phi R_n = \phi\left[\min\left\{0.6F_u A_{nv},\ 0.6F_y A_{gv}\right\} + U_{bs}F_u A_{nt}\right.$$

$$U_{bs}F_u A_{nt} = 1.0\left(65\text{ ksi}\right)\left(1.019\text{ in.}^2\right) = 66.2\text{ kips}$$

$$0.6F_y A_{gv} = 0.6\left(50\text{ ksi}\right)\left(5.0625\text{ in.}^2\right) = 152\text{ kips}$$

$$0.6F_u A_{nv} = 0.6\left(65\text{ ksi}\right)\left(3.06\text{ in.}^2\right) = 119\text{ kips}$$

$$\phi R_{bsv} = 0.75\left(119\text{ kips} + 66.2\text{ kips}\right)$$
$$= 138\text{ kips} < V_c = 141\text{ kips} \quad \textbf{– NG}$$



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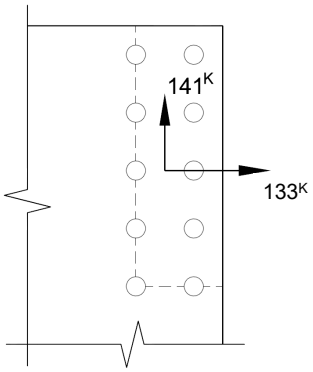
Example of Corner Connection
Special Case IV

Plate: Gusset-to-Column– Block Shear – Shear Plate (Vertical Load) (cont.):

Try a 1/2 plate

$$\phi R_{bsv} = 134x\frac{0.5}{0.375} = 179k > 141k \quad \text{– OK}$$

Continue with 1/2 plate



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Example of Corner Connection
Special Case IV

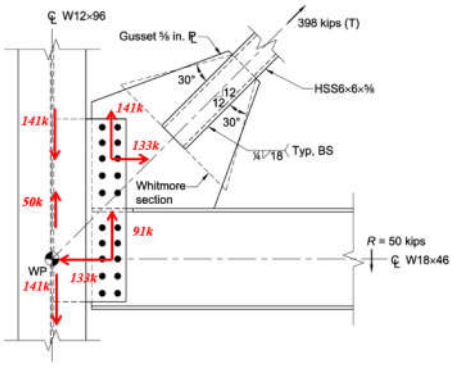
Plate: Gusset-to-Column– Block Shear –

Shear Plate (Horizontal Load):

From AISC *Specification* Eq. J4-5,

$$A_{gt} = \left[\left(n_v - 1\right)s_v + L_{ev}\right]t_{pl}$$
$$= \left[\left(5-1\right)\left(3.0\text{ in.}\right)+1.50\text{ in.}\right]\left(0.50\text{in.}\right)$$
$$= 6.75\text{ in.}^2$$

$$A_{nt} = A_{gt} - \left(n_v - 0.5\right)d_{hole}t_{pl}$$
$$= 6.75\text{ in.}^2 - \left(5-0.5\right)\left(1.875\text{ in.}\right)\left(0.50\text{ in.}\right)$$
$$= 4.08\text{ in.}^2$$



Net Interface Forces on Plate (Tension)

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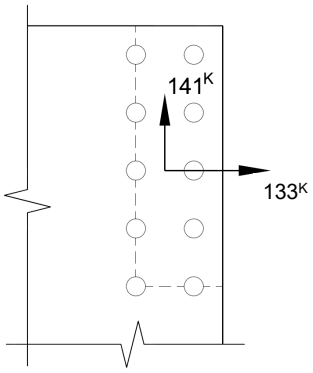
Example of Corner Connection
Special Case IV

Gusset-to-Column– Block Shear – Shear Plate (Horizontal Load) (cont.):

From AISC *Specification* Eq. J4-5,

$$A_{gv} = \left[\left(n_h - 1\right)s_h + L_{eh}\right]t_{pl}$$
$$= \left[\left(2-1\right)\left(3.0\text{ in.}\right)+1.50\text{ in.}\right]\left(0.50\text{in.}\right)$$
$$= 2.25\text{ in.}^2$$

$$A_{nv} = A_{gv} - \left(n_h - 0.5\right)d_{hole}t_{pl}$$
$$= 2.25\text{ in.}^2 - \left(2-0.5\right)\left(1.1875\text{ in.}\right)\left(0.50\text{ in.}\right)$$
$$= 1.359\text{ in.}^2$$



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Example of Corner Connection
Special Case IV

Plate: Gusset-to-Column– Block Shear – Shear Plate (Horizontal Load) (cont.):

From AISC *Specification* Eq. J4-5,

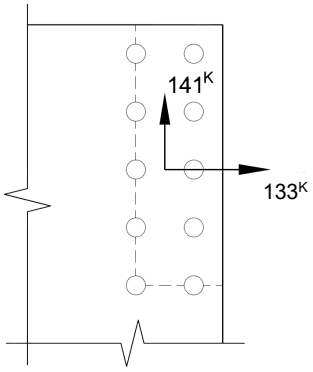
$$\phi R_n = \phi\left[\min\left\{0.6F_u A_{nv},\ 0.6F_y A_{gv}\right\} + U_{bs}F_u A\right.$$

$$U_{bs}F_u A_{nt} = 1.0\left(65\text{ ksi}\right)\left(4.08\text{ in.}^2\right) = 265\text{ kips}$$

$$0.6F_y A_{gv} = 0.6\left(50\text{ ksi}\right)\left(2.25\text{ in.}^2\right) = 67.5\text{ kips}$$

$$0.6F_u A_{nv} = 0.6\left(65\text{ ksi}\right)\left(1.359\text{ in.}^2\right) = 53.0\text{ kips}$$

$$\phi R_{bsh} = 0.75\left(53.0\text{ kips} + 265\text{ kips}\right)$$
$$= 238\text{ kips} > H_c = 133\text{ kips} \quad -\text{OK}$$



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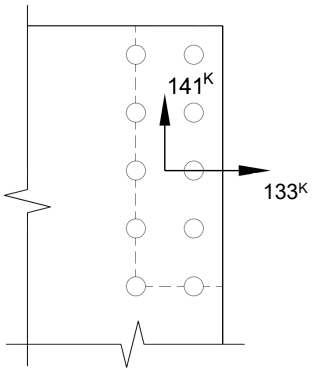
Example of Corner Connection
Special Case IV

Gusset-to-Column – Block Shear – Shear Plate:

Interaction of vertical and horizontal loading,

$$\left(\frac{V_c}{\phi R_{bsv}}\right)^2 + \left(\frac{H_c}{\phi R_{bsh}}\right)^2 = \left(\frac{141\text{ kips}}{179\text{ kips}}\right)^2 + \left(\frac{133\text{ kips}}{238\text{ kips}}\right)^2$$
$$= 0.93 < 1 \quad -\text{OK}$$

A 1/2” minimum shear plate is required.



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Example of Corner Connection
Special Case IV

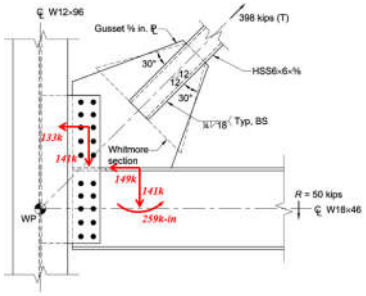
Gusset: Gusset-to-Column – Block Shear –

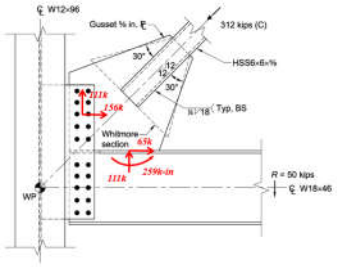
The forces on the gusset act as shown.

For Brace Tension:

Only horizontal block shear is a limit state.

The gusset plate is 5/8". The shear plate has been checked for this load; therefore, 5/8" gusset is ok.





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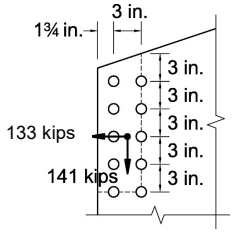
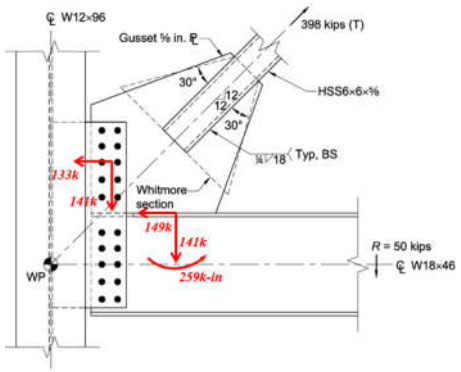
Example of Corner Connection
Special Case IV

Gusset: Gusset-to-Column – Block Shear –

The limit state was ok for the ½" plate. Therefore, 5/8" gusset ok for block shear.





Net Interface Forces on Gusset (Tension)

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Example of Corner Connection
Special Case IV
Beam-to-Column

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Example of Corner Connection
Special Case IV

Plate: Beam-to-Column Connection:

Tension Case:

H=133k V=91k R=161k

Compression Case:

H=156k V=161k R=224k -- Controls

Bearing

$\phi r_{Pb} = 0.75(2.4)F_u(d_b)(t_w)$

$= 0.75(2.4)(65)(1.0)(0.5)$

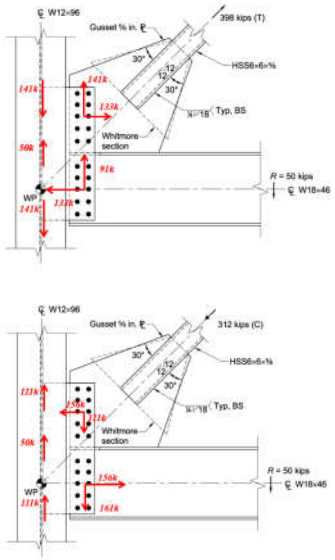
$= 58.5k > 224k/10 = 22.4k$ – OK

Tearout - worst case

$\phi V_{to} = 0.75(1.2)(F_u)(l_c)(t_w)$

$= 0.75(1.2)(65)(1.50 - 0.5(1.125)(0.50)$

$= 27.4k > 22.4k$ – OK



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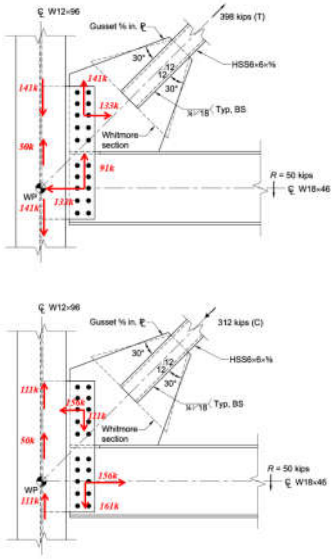
Example of Corner Connection
Special Case IV

Plate: Beam-to-Column Connection:

Block shear is not a limit state for the brace tension case.

For the brace compression case, block shear is a limit state for the vertical plate force of 161k and the horizontal force of 156k.



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Example of Corner Connection
Special Case IV

Plate: Beam-to-Column – Block Shear:

Vertical force, $V = 161$ kips

$$A_{gv} = (13.5)(0.5) = 6.75 \text{ in.}^2$$

$$A_{nv} = 6.75 - 4.5(1.1875)(0.5) = 4.08 \text{ in.}^2$$

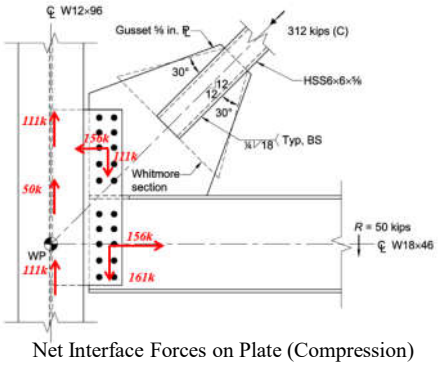
$$A_{nt} = (4.5 - 1.5(1.1875))(0.5) = 1.36 \text{ in.}^2$$

$$F_u A_{nt} = 65(1.36) = 88.4 \text{ kips}$$

$$0.6F_y A_{gv} = 0.6(50)(6.75) = 203 \text{ kips}$$

$$0.6F_u A_{nv} = 0.6(65)(4.08) = 159 \text{ kips} \leftarrow$$

$$\phi R_n = 0.75[1.0(88.4) + 159] = 186 \text{ kips} \quad > 161 \text{ kips} \quad \text{– OK}$$



Net Interface Forces on Plate (Compression)

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Example of Corner Connection
Special Case IV

Plate: Beam-to-Column – Block Shear:

Horizontal force, $H = 156$ kips

$$A_{gv} = (4.5)(0.5) = 2.25 \text{ in.}^2$$

$$A_{nv} = 2.25 - 1.5(1.1875)(0.5) = 1.36 \text{ in.}^2$$

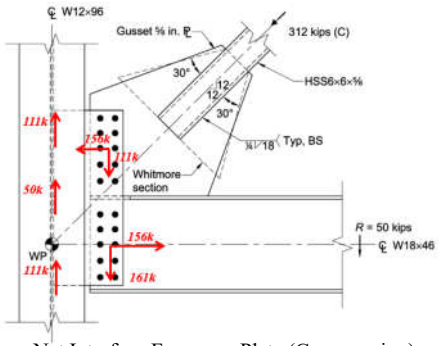
$$A_{nt} = (13.5 - 4.5(1.1875))(0.5) = 4.08 \text{ in.}^2$$

$$F_u A_{nt} = 65(4.08) = 265 \text{ kips}$$

$$0.6F_y A_{gv} = 0.6(50)(2.25) = 67.5 \text{ kips}$$

$$0.6F_u A_{nv} = 0.6(65)(1.36) = 53.0 \text{ kips}$$

$$\phi R_n = 0.75[1.0(265) + 53.0] = 239 \text{ kips} \quad > 156 \text{ kips} \quad \text{– OK}$$



Net Interface Forces on Plate (Compression)

65

Example of Corner Connection
Special Case IV

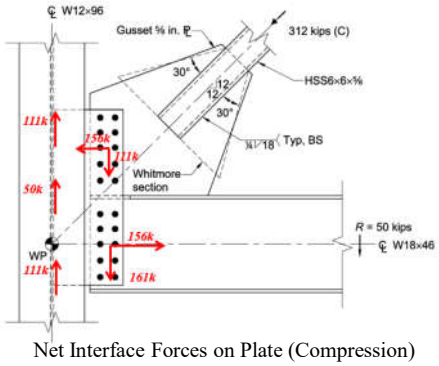
Plate: Beam-to-Column – Block Shear:

Interaction

$$\left(\frac{161}{186}\right)^2 + \left(\frac{156}{239}\right)^2 = 1.18 \quad > 1 \quad \text{– NG}$$

$$1.18\left(\frac{0.5}{t_p}\right)^2 \le 1$$

$$\rightarrow t_p \ge 0.543 \text{ in.} \quad \text{-- Use a 5/8-in. plate.}$$



Net Interface Forces on Plate (Compression)

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Example of Corner Connection
Special Case IV

Beams: Beam-to-Column – Beam Web Shear:

AISC *Manual* Table 3-2,

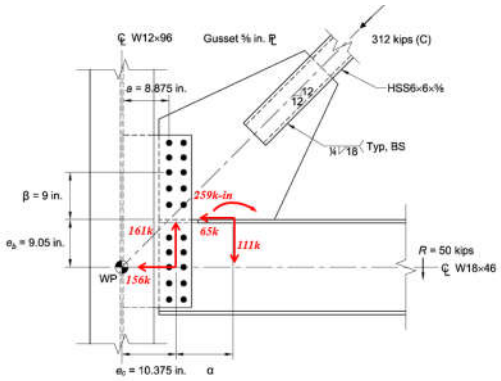
$\phi V_n = 195$ kips > 161 kips, --OK

Table 3-2 (continued)
W-Shapes
Selection by Z_x

$F_y = 50$ ksi

Z_x

Shape	Z_x	M_{ax}/Ω_b kip-ft	$\phi_b M_{px}$ kip-ft	M_{ay}/Ω_b kip-ft	$\phi_b M_{py}$ kip-ft	BF/Ω_b kips	$\phi_b BF$ kips	L_p	L_r	I_x	V_{ax}/Ω_v kips	$\phi_v V_{ax}$ kips
W21×44	95.4	238	358	143	214	11.1	16.8	4.45	13.0	843	145	217
W18×46	90.7	226	340	138	207	9.63	14.6	4.56	13.7	712	130	195



Net Interface Forces on Beam (Compression)

Example of Corner Connection
Special Case IV

Beam: Beam-to-Column – Bearing and Tearout of Beam Web: Tension

Web is critical since: $t_w = 0.360$ in. $< t_p = 0.50$ in.

There is no force acting towards the end of the beam.

Therefore, tearout does not apply.

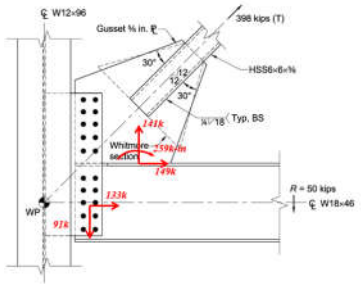
Shear per bolt:

$$R_u = \sqrt{133^2 + 91^2} = 161k$$

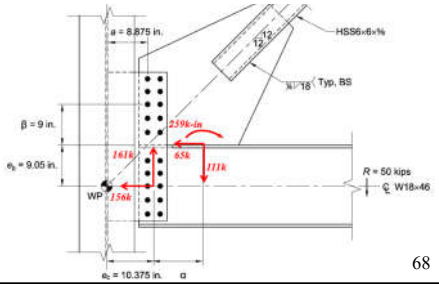
$$r_{bolt} = \frac{R_u}{n_{bolts}} = \frac{161}{10} = 16.1k \text{ / bolt}$$

Bearing:

$$\phi r_{Pb} = 0.75(2.4)F_u(d_b)(t_w)$$
$$= 0.75(2.4)(65)(1.0)(0.36)$$
$$= 48.9k > 16.1k \quad -\text{OK}$$



Net Interface Forces on Beam (Tension)



Example of Corner Connection
Special Case IV

Beam: Beam-to-Column – Bearing and Tearout

Compression controls

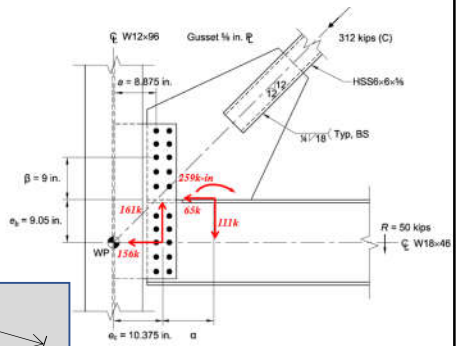
$V = 161$ kips $\quad H = 156$ kips $\quad R = 224$ kips

Shear per bolt

$224/10 = 22.4$ kips $\quad < 31.8$ kips $\quad$ – OK

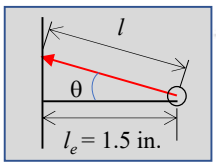
Bearing

$$\phi r_n = 0.75(2.4)(65)(1.0)(0.360)$$
$$= 42.1 \text{ kips} > 22.4 \text{ kips} \quad \text{– OK}$$



Tearout

$\tan\theta = (161/156 = 1.032) \rightarrow \theta = 45.9°$



$\cos\theta = (1.5/l) \rightarrow l = 2.16$ in. $\quad \rightarrow l_c = 2.16 - 0.5(1.1875) = 1.57$ in.

$\phi r_n = 0.75(1.2)(65)(0.360)(1.57) = 33.1$ kips $\quad > 22.4$ kips $\quad$ – OK

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Example of Corner Connection
Special Case IV

Beam: Beam-to-Column – Web Block Shear

Horizontal force, $H = 156$ kips

$A_{gv} = 2(4.5)(0.360) = 3.24$ in.^{2}

$A_{nv} = 2\left[4.5 - 1.5(1.1875)\right](0.360) = 1.96$ in.^{2}

$A_{nt} = \left(12 - 4(1.1875)\right)(0.360) = 2.61$ in.^{2}

$F_u A_{nt} = 65(2.61) = 170$ kips

$0.6F_y A_{gv} = 0.6(50)(3.24) = 97.2$ kips

$0.6F_u A_{nv} = 0.6(65)(1.96) = 76.4$ kips

$\phi R_n = 0.75\left[1.0(170) + 76.4\right] = 185$ kips $\quad > 156$ kips $\quad$ – OK

70

Example of Corner Connection
Special Case IV

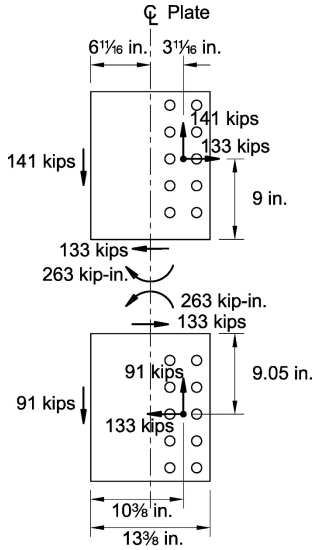

Plate Section Forces – Tension Case

$V = 133 \text{ kips} \quad N_e = \dfrac{4(263)}{13.375} = 78.7 \text{ kips}$

$\phi V_n = 1.0(0.6)(50)(13.375)(0.625) = 251 \text{ kips}$

$> 133 \text{ kips}$ -- OK

$\phi N_n = 0.9(1.0)(50)(13.375)(0.625) = 376 \text{ kips}$

$> 78.7 \text{ kips}$ -- OK

$\left(\dfrac{133}{251}\right)^2 + \left(\dfrac{78.7}{376}\right)^2 = 0.32 \quad < 1.0 \quad - \text{OK}$

71

Example of Corner Connection
Special Case IV

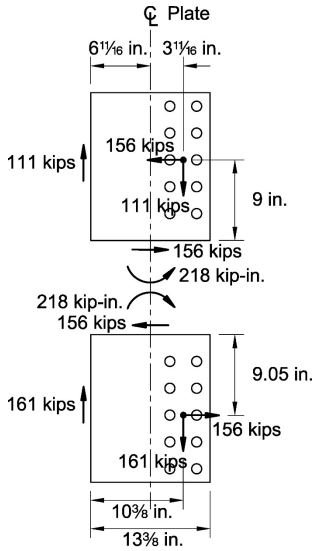

Plate Section Forces – Compression Case

$V = 156 \text{ kips} \quad N_e = \dfrac{4(218)}{13.375} = 65.2 \text{ kips}$

$\phi V_n = 1.0(0.6)(50)(13.375)(0.625) = 251 \text{ kips}$

$> 156 \text{ kips}$ -- OK

$\phi N_n = 0.9(1.0)(50)(13.375)(0.625) = 376 \text{ kips}$

$> 65.2 \text{ kips}$ -- OK

$\left(\dfrac{156}{251}\right)^2 + \left(\dfrac{65.2}{376}\right)^2 = 0.42 \quad < 1.0 \quad - \text{OK}$

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Example of Corner Connection
Special Case IV
Gusset-to-Beam

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Example of Corner Connection
Special Case IV

Beam: Gusset-to-Beam Connection – Weld:

Brace Tension:

Shear: H =149 kips

Normal: $V_b = 141$ kips

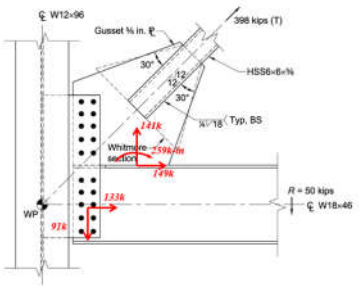
Moment: $M_F = 28.8 \text{ kips}(9.0 \text{ in.}) = 259 \text{ kip-in.}$

Brace Compression:

Shear: H=65.2 kips

Normal: V=111 kips

Moment: M= 259 kip-in.



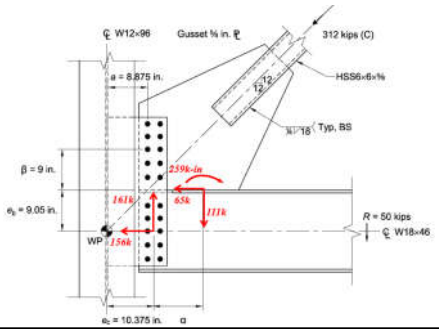
Brace Tension Controls.

Equivalent normal force,

$$N_e = V_b + \frac{4M_F}{L_{weld}}$$

$$= 141 \text{ kips} + \frac{4(259 \text{ kip-in.})}{21.8 \text{ in.}}$$

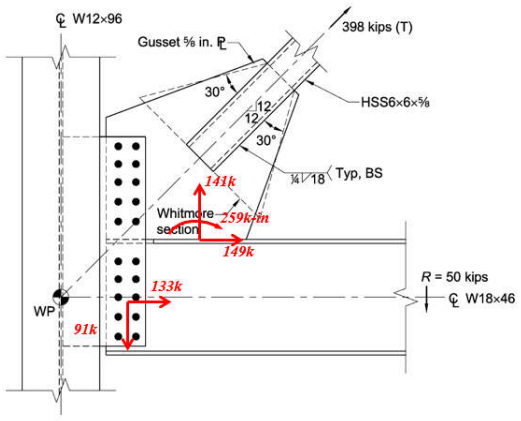
$$= 189 \text{ kips}$$



Example of Corner Connection
Special Case IV

Gusset-to-Beam Connection – Weld (cont.):

$$R_u = \sqrt{V^2 + N^2}$$
$$= \sqrt{(149 \text{ kips})^2 + (189 \text{ kips})^2}$$
$$= 241 \text{ kips}$$
$$\theta = \tan^{-1}\left(\frac{N_e}{V}\right) = \tan^{-1}\left(\frac{189 \text{ kips}}{149 \text{ kips}}\right)$$
$$= 51.75°$$
$$\mu = 1.0 + 0.50 \sin^{1.5}\theta$$
$$= 1.0 + 0.50 \sin^{1.5} 51.75°$$
$$= 1.35$$



Net Interface Forces on Beam (Tension)

75

Example of Corner Connection
Special Case IV

Gusset-to-Beam Connection – Weld (cont.):

$$D_{req} = \frac{R_u}{1.392 \mu l_{weld} (2 \text{ lines})}$$
$$= \frac{(241 \text{ kips})}{1.392(1.35)(21.8 \text{ in.})(2 \text{ lines})}$$
$$= 2.94 \text{ sixteenths} \rightarrow 3/16 \text{ but weld min. is 1/4 per Table J2.4}$$

Use ¼ in. fillet welds for 15.0 in.

Due to block on beam, weld single sided in this section.

$2.94 \text{ sixteenths}(2) = 5.88 \text{ sixteenths}$ **- Use 3 / 8 in. fillet welds for 6.75 in.**

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Example of Corner Connection
Special Case IV

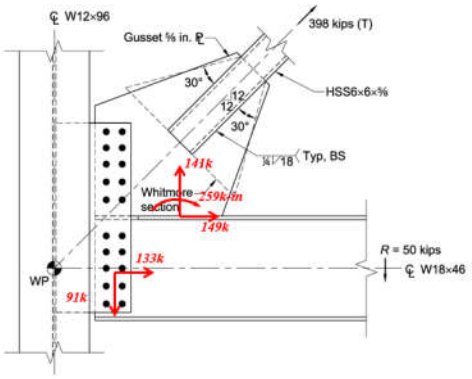
Gusset-to-Beam Connection – Gusset Checks:

AISC *Specification* Eq. J4-4 – Shear Rupture

$$\phi R_n = \phi 0.60 F_u A_{gv}$$
$$= 0.75(0.60)(65\text{ ksi})(0.625\text{ in.})(21.8\text{ in.})$$
$$= 398\text{ kips} > V = 149\text{ kips} \quad -\text{OK}$$

AISC *Specification* Eq. J4-2 – Tension Rupture

$$\phi R_n = \phi F_u A_g$$
$$= 0.75(65\text{ ksi})(0.625\text{ in.})(21.8\text{ in.})$$
$$= 664\text{ kips} > N_e = 189\text{ kips} \quad -\text{OK}$$



Net Interface Forces on Beam (Tension)

77

Example of Corner Connection
Special Case IV

Beam: Gusset-to-Beam Connection – Beam Checks:

AISC *Specification* Eq. J10-3 – Web Local Yielding

$$\phi R_n = \phi F_{yw} t_w (2.5k + l_b)$$
$$= 1.0(50\text{ ksi})(0.360\text{ in.})[2.5(1.01\text{ in.}) + 21.8\text{ in.}]$$
$$= 438\text{ kips} > N_e = 189\text{ kips} \quad -\text{OK}$$

AISC *Specification* Eq. J10-4 – Web Local Crippling

$$\phi R_n = \phi 0.80 t_w^2 \left[1 + 3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}}$$
$$= \phi 0.80(0.360)^2 \left[1 + 3\left(\frac{21.8\text{ in.}}{18.1\text{ in.}}\right)\left(\frac{0.360\text{ in.}}{0.605\text{ in.}}\right)^{1.5}\right]\sqrt{\frac{29{,}000\text{ ksi}(50\text{ ksi})(0.605\text{ in.})}{0.360\text{ in.}}}$$
$$= 323\text{ kips} > N_e = 189\text{ kips} \quad -\text{OK}$$

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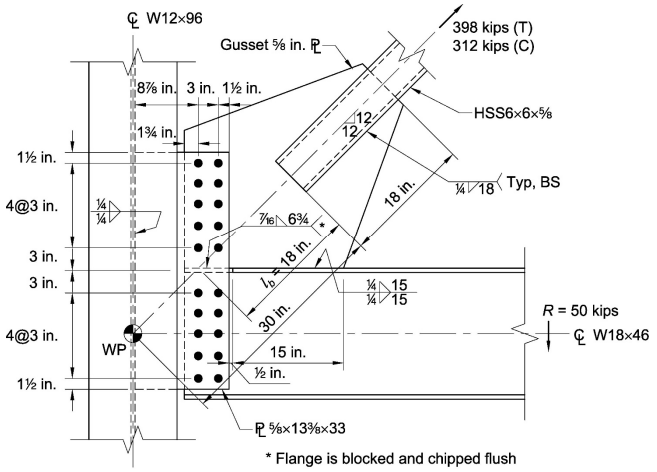
Example of Corner Connection
Special Case IV



Final Design

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UFM Special Case V – Bypass Method

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UFM Special Case V – Bypass Method

- "Design for Local Web Shear at Brace Connections: An Adaptation of the Uniform Force Method"

  by Sabelli, Saxey, Li, and Thornton

  AISC *Engineering Journal*, 2021, Vol. 58, No. 4, pp. 223-265

  Visit aisc.org/ej for access.

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UFM Special Case V – Bypass Method

- When beam shear is high due to UFM component, V_b, and gravity shear, R…
- If Special Case II cannot reduce beam shear enough to eliminate doubler…

  …try the Bypass Method.

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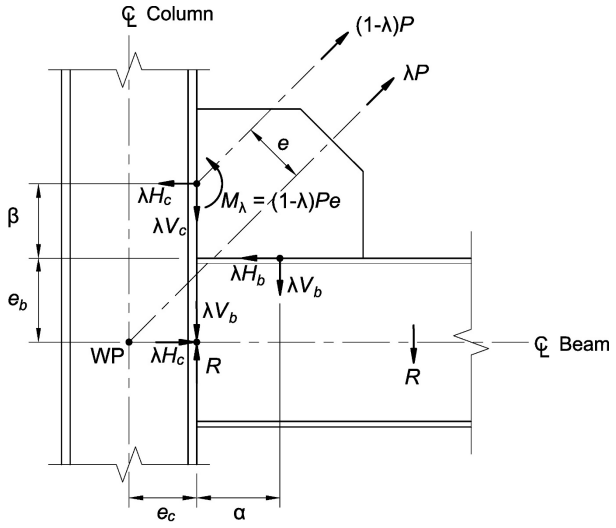
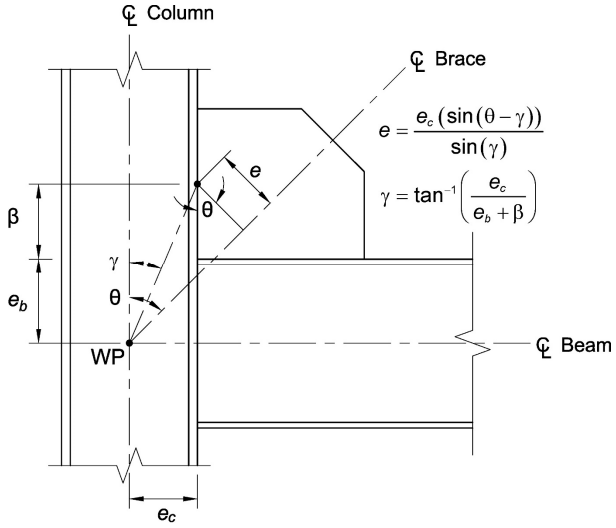
UFM Special Case V – Bypass Method



83

Calculation of Eccentricity, *e*



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UFM Special Case V – Bypass Method

When $R + |V_b| > \phi R_n$

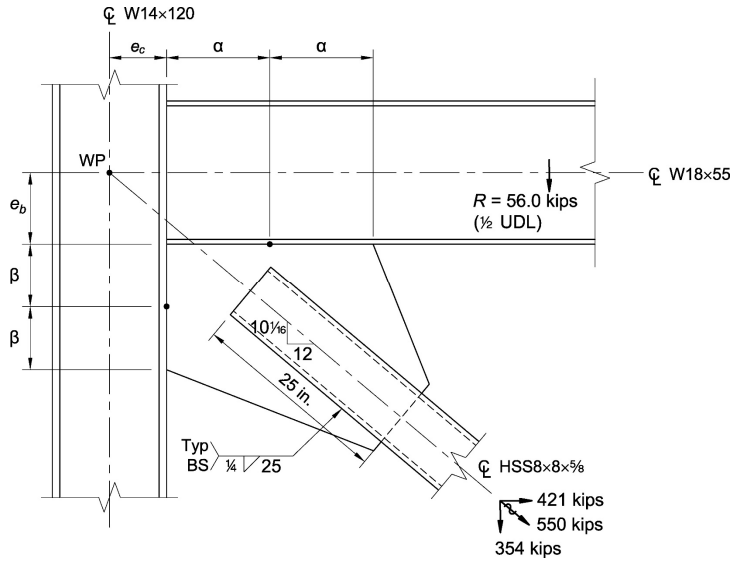
$$\text{set } \lambda = \frac{\phi R_n - R}{|V_b|}$$

where

ϕR_n is the beam web strength or the web connection strength, whichever controls

85

Special Case V – Example



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Special Case V – Example

Using the UFM

$e_b = 9.05$ in. $\quad e_c = 7.25$ in. $\quad \tan\theta = \dfrac{12}{10.0625} = 1.1925 \rightarrow \theta = 50°$

$\alpha - \beta \tan\theta = e_b \tan\theta - e_c$

Choose $\beta = 8$, then $\alpha = (8 + 9.05)(1.1925) - 7.25 = 13.1$ in.

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

$$= \sqrt{(13.1 + 7.25)^2 + (8 + 9.05)^2} = 26.55 \text{ in.}$$

$$\frac{P}{r} = \frac{550}{26.55} = 20.72 \frac{\text{kips}}{\text{in.}}$$

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Special Case V – Example

$$H_b = \alpha \frac{P}{r} = (13.1)(20.72) = 271 \text{ kips}$$

$$H_c = e_c \frac{P}{r} = (7.25)(20.72) = 150 \text{ kips}$$

$$\sum = 421 \text{ kips (OK)}$$

$$V_c = \beta \frac{P}{r} = (8)(20.72) = 166 \text{ kips}$$

$$V_b = e_b \frac{P}{r} = (9.05)(20.72) = 187 \text{ kips}$$

$$\sum = 353 \text{ kips (OK)}$$

Placing these forces on the connection…

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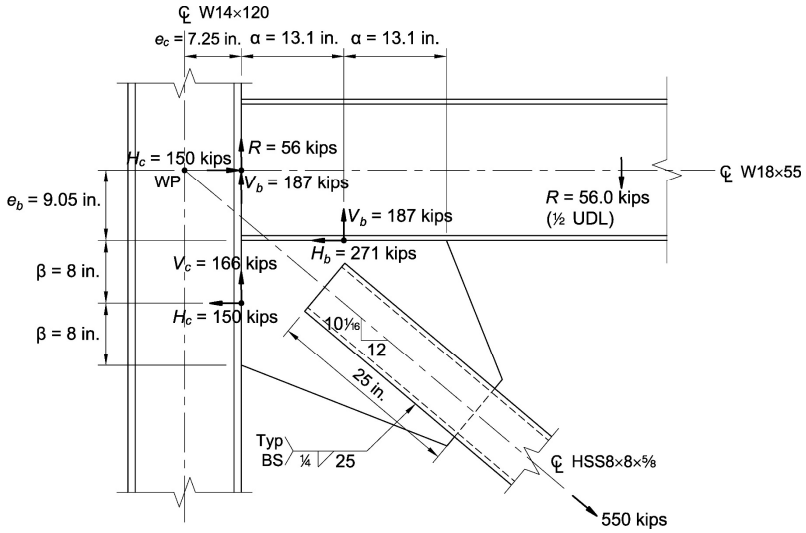
Special Case V – Example



89

Special Case V – Example

The shear capacity at the beam

$\phi V_n = 212$ kips

The shear demand is

$187 + 56 = 243$ kips > 212 kips -- No Good

Could try Special Case 2 to reduce the shear by introduction of

$\Delta V_b = 243 - 212 = 31$ kips

This will work for this problem

-- but --

I want to demonstrate **Special Case 5**.

90

Special Case V – Example

Special Case 5…

Set $\lambda = \dfrac{\phi R_n - R}{|V_b|} = \dfrac{212 - 56}{187} = 0.834$

If the brace force, $P = 550$, is reduced to

$$\lambda P = (0.834)(550) = 459 \text{ kips}$$

the beam shear will be reduced to

$$(187)(0.834) = 156 \text{ kips} \quad < \phi R_n = 212 \text{ kips} \quad - \text{OK}$$

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Special Case V – Example

The brace force, $P = 550$, is reduced to

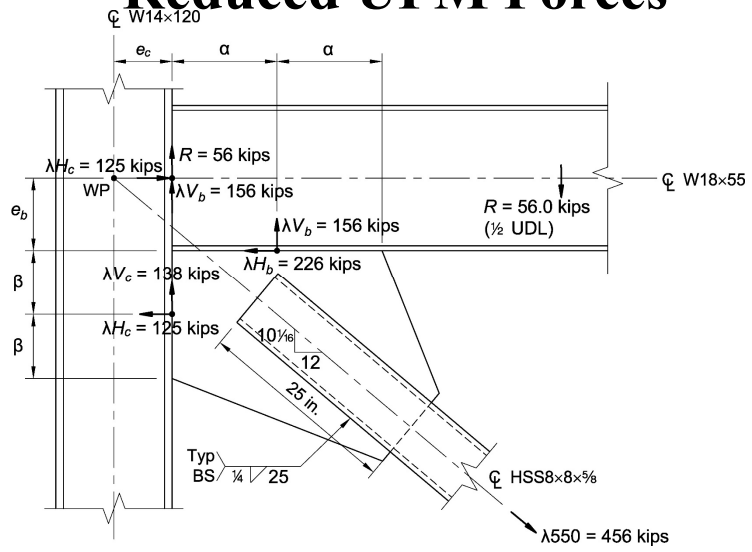
$$\lambda P = (0.834)(550) = 459 \text{ kips}$$

This brace force, $\lambda P = 459$ kips, is used in the UFD.

All the UFM forces are reduced by $\lambda = 0.834$ as shown in the next figure.

92

Special Case V – Example Reduced UFM Forces



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Special Case V – Example

The remainder of the brace force is applied to the gusset-to-column connection as

$$\overline{P} = (1-\lambda)P$$
$$= (1-0.834)(550) = 91.3 \text{ kips}$$

This force is eccentric to the W.P. and causes a moment

$$\overline{P}e = (91.3)(8.42) = 769 \text{ kips}$$

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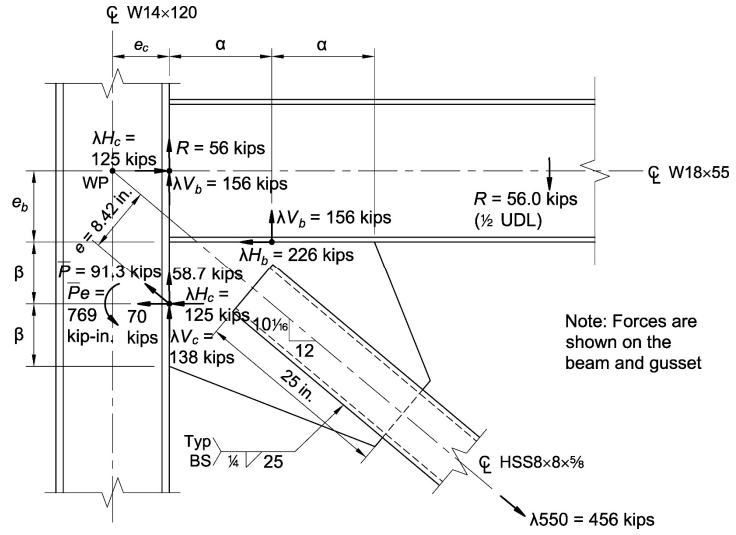
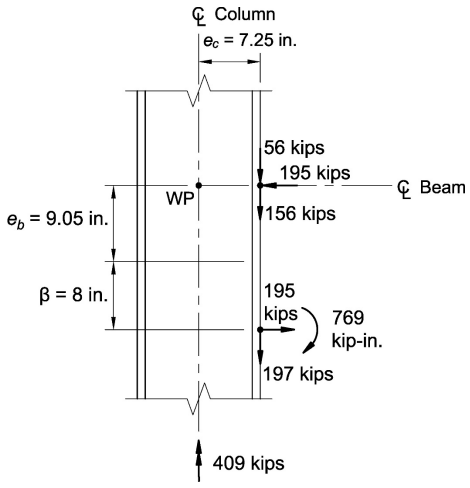
Special Case V – Example
Special Case V Force Distribution



95

Special Case V – Example
Free Body Diagram



Free Body Diagram for the column

Beam shear is applied at column flange.
There is a moment: 56kx7.25

Beam shear is applied at column flange.

There is a moment:

$56 kips \; x 7.25 = 406 k-in \rightarrow$ ignored

$\overset{+}{\rightarrow} \sum F_x = 0 = 195 - 195 = 0$

$\overset{+}{\uparrow} \sum F_y = 0 = -56 - 156 - 197 + 409 = 0$

$\overset{+}{\circlearrowleft} \sum M_{wp} = 0 = -769 - (156 + 197)7.25 + 195(9.05 + 8) = -3.5 \cong 0$ – OK

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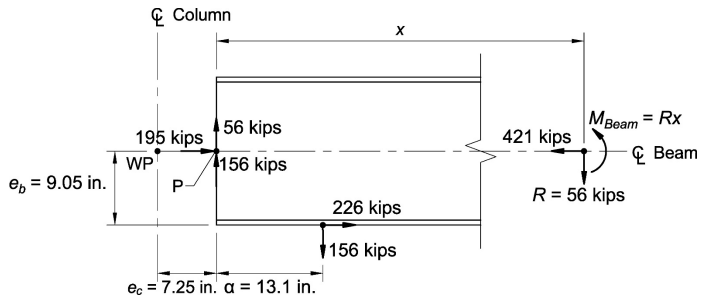
Special Case V – Example
Free Body Diagram



$\rightarrow +\ \sum F_x=0=195+226-426=0$

$\uparrow +\ \sum F_y=0=56+156-56-156=0$

$\circlearrowleft +\ \sum M_p=0=-156(13.1)+226(9.05)-R_x+R_x$

$=0=-2044+2045\cong 0$ – OK

Free Body Diagram for the beam

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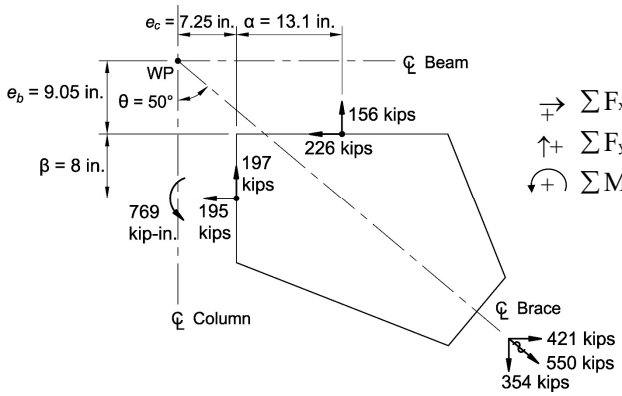
Special Case V – Example
Free Body Diagram



$\rightarrow +\ \sum F_x=0=-226-195+421=0$

$\uparrow +\ \sum F_y=0=156+197-354-1\cong 0 \rightarrow$ O.K.

$\circlearrowleft +\ \sum M_{wp}=0=0+156(7.25+13.1)-226(9.05)197(7.25)$

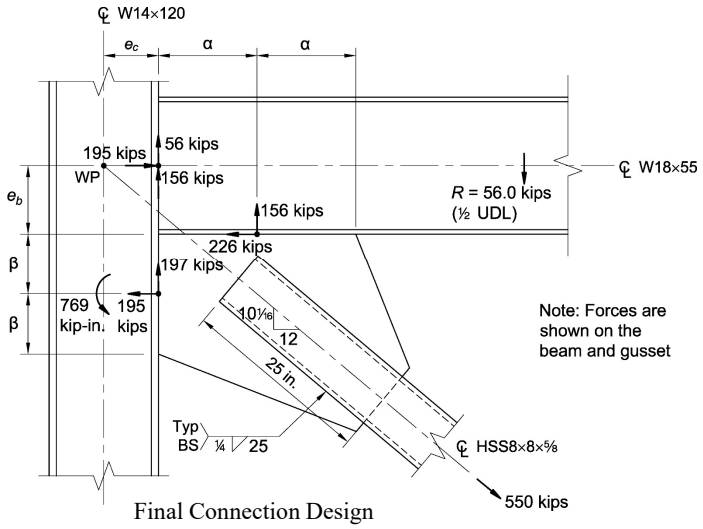
$-195(9.05+8)+769$

$=3175-2045+1428-3325+769=2\cong 0$ – OK

Free Body Diagram for the gusset

98

Special Case V – Example



Final Connection Design

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The forces on all interfaces are now known.

The connection design can now be completed.

Summary

Summary

- Uniform Force Method – Special Case IV
  - Provides concentrically loaded bolt groups
- Uniform Force Method – Special Case V
  - With heavy columns and light beams, provides an alternate method to avoid doubler plates

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Thank you!

AISC | Questions

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Individual Session Registrants

PDH Certificates

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

8-Session Registrants

PDH Certificates

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

8-Session Registrants

Access to the quiz

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

Quiz and attendance records

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

Reasons for quiz

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

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

8-Session Registrants

Access to the recording

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

PDHs via recording

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

8-Session Registrants

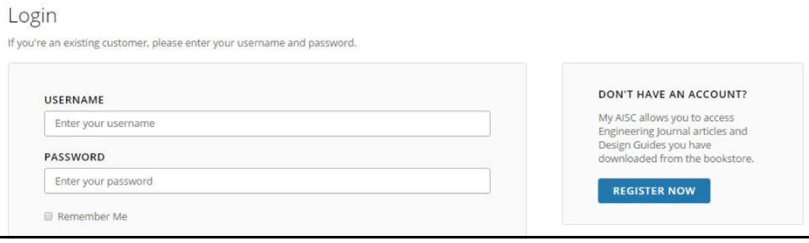
Night School Resources

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

8-Session Registrants

Night School Resources

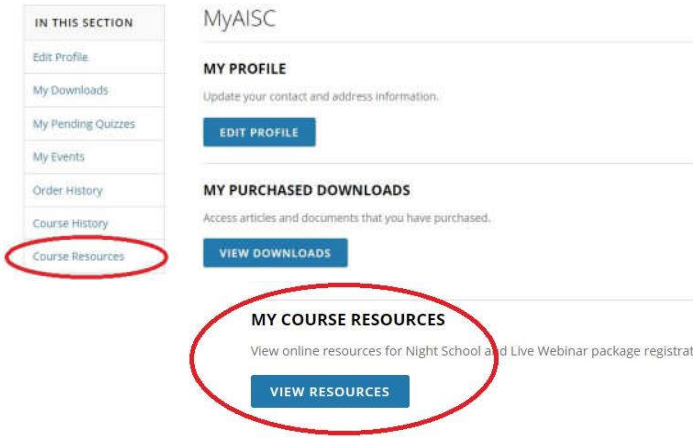
Go to www.aisc.org and sign in.



8-Session Registrants

Night School Resources

Go to www.aisc.org and sign in.



8-Session Registrants

Night School Resources

Course Resources

Event	Start Date
NS 13 8-Session Package-Night School 13 - Design of Industrial Buildings	1/30/2017 7:00:00 PM
NS 14 8-Session Package-Night School 14 - Fundamentals of Stability	6/5/2017 7:00:00 PM

8-Session Registrants

Night School Resources

Night School 13: Design of Industrial Buildings

8-SESSION PACKAGE RESOURCES

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

8-Session Registrants

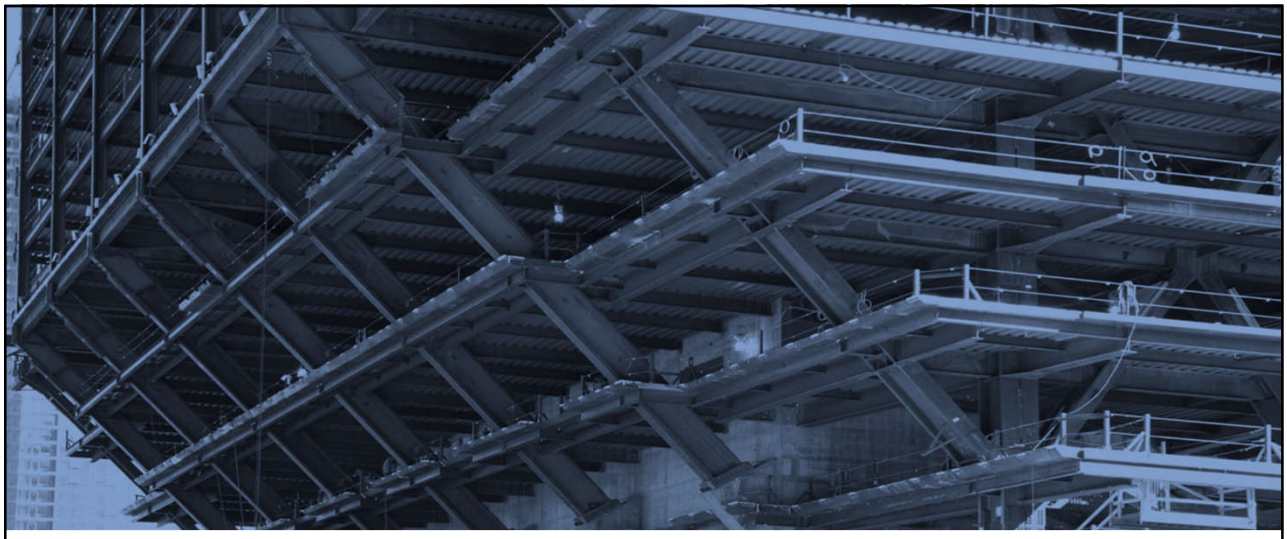
Night School Resources

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

8-Session Registrants

Night School Resources

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



AISC | Thank you

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