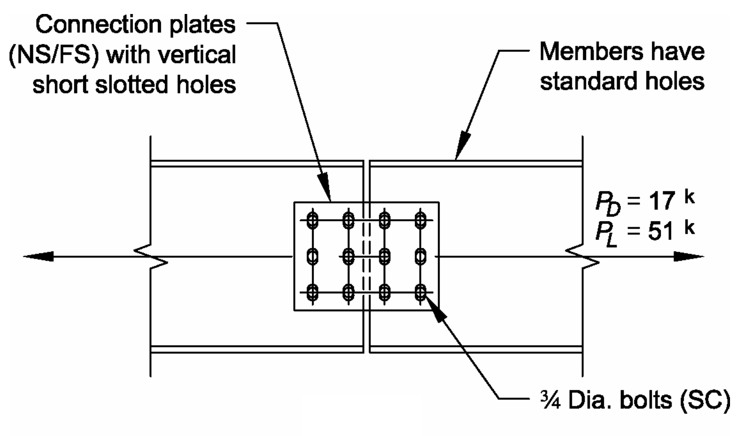
Example J.4 Slip-Critical Connection with Short Slotted Holes

High-strength bolts in slip-critical connections are permitted to be designed to prevent slip either as a serviceability limit state or as a strength limit state. The most common design case is design for slip as a serviceability limit state. The design of slip as a strength limit state should only be applied when bolt slip can result in a connection geometry that will increase the required strength beyond that of a strength limit state, such as bearing or bolt shear. Such considerations occur only when oversized holes or slots parallel to the load are used, and when the slipped geometry increases the demand on the connection. Examples include the case of ponding in flat-roofed long span trusses, or the case of shallow, short lateral bracing.

Given:

Select the number of ¾-in. ASTM A325 slip-critical bolts with a Class A faying surface that are required to support the loads shown when the connection plates have short slots transverse to the load. Select the number of bolts required for slip resistance only.

Assume that the connected pieces have short slots transverse to the load. Use a mean slip coefficient of 0.35, which corresponds to a Class A surface.



Solution:

Calculate the required strength

LRFD	ASD
P_u = 1.2(17 kips) + 1.6(51 kips) = 102 kips	P_a = 17 kips + 51 kips = 68 kips

For standard holes or slots transverse to the direction of the load, a connection can be designed on the basis of the serviceability limit state. For the serviceability limit state:

$$\phi = 1.00 \quad \Omega = 1.50$$

Specification Section J3.8

Find R_n, where:

μ = 0.35 for Class A surface
D_u = 1.13
h_{sc} = 0.85 (short slotted holes)
T_b = 28 kips
N_s = 2, number of slip planes

Table J3.1

$R_n = \mu D_u h_{sc} T_b N$

Eqn.J3-4

R_n = 0.35(1.13)(0.85)(28)(2) = 18.8 kips/bolt

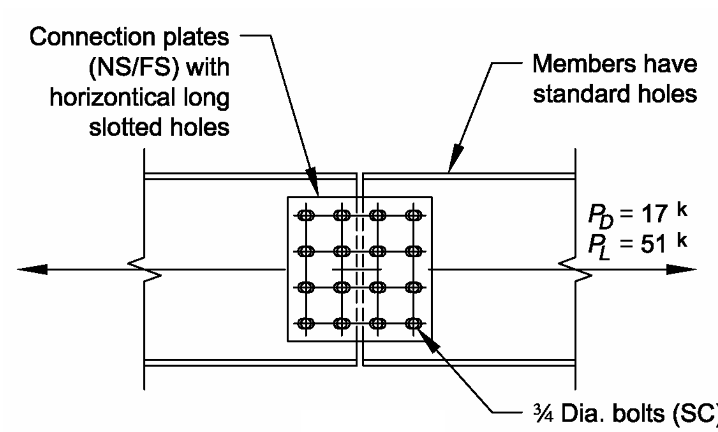
Determine the required number of bolts.

LRFD	ASD
102 kips/1.00(18.8 kips/bolt) = 5.42 bolts	68 kips $/ \dfrac{18.8 \text{ kips/bolt}}{1.50}$ = 5.42 bolts
Use 6 bolts **o.k.**	Use 6 bolts **o.k.**

Manual Table 7-3

Given:

Repeat the problem with the same loads, but assuming that the connected pieces have long slotted holes in the direction of the load and that the deformed geometry of the connection would result in a critical load increase.



Solution:

P_u= 102 kips and P_a = 68 kips per the first solution

For this connection, the designer has determined that oversized holes or slots parallel to the direction of the load will result in a deformed geometry of the connection that creates a critical load case. Therefore, the connection is designed to prevent slip at the required strength level.

$$\phi = 0.85 \quad \Omega = 1.76$$

Specification Section J3.8

In addition, h_{sc} will change because we now have long slotted holes.

Find R_n

μ = 0.35 for Class A surface
D_u = 1.13
h_{sc} = 0.70 (long slotted holes)
T_b = 28 kips
N_s = 2, number of slip planes — Table J3.1

$R_n = \mu D_u h_{sc} T_b N_s$

R_n = 0.35(1.13)(0.70)(28)(2) = 15.5 kips/bolt — Specification Eqn. J3-4

Determine the required number of bolts

LRFD	ASD
$\dfrac{102\text{ kips}}{0.85(15.5\text{ kips/bolt})} = 7.73\text{ bolts}$	$\dfrac{68\text{ kips }(1.76)}{15.5\text{ kips/bolt}} = 7.63\text{ bolts}$
Use 8 bolts **o.k.**	Use 8 bolts **o.k.**

Manual
Table 7-4

Example II.C-3 Bracing Connection

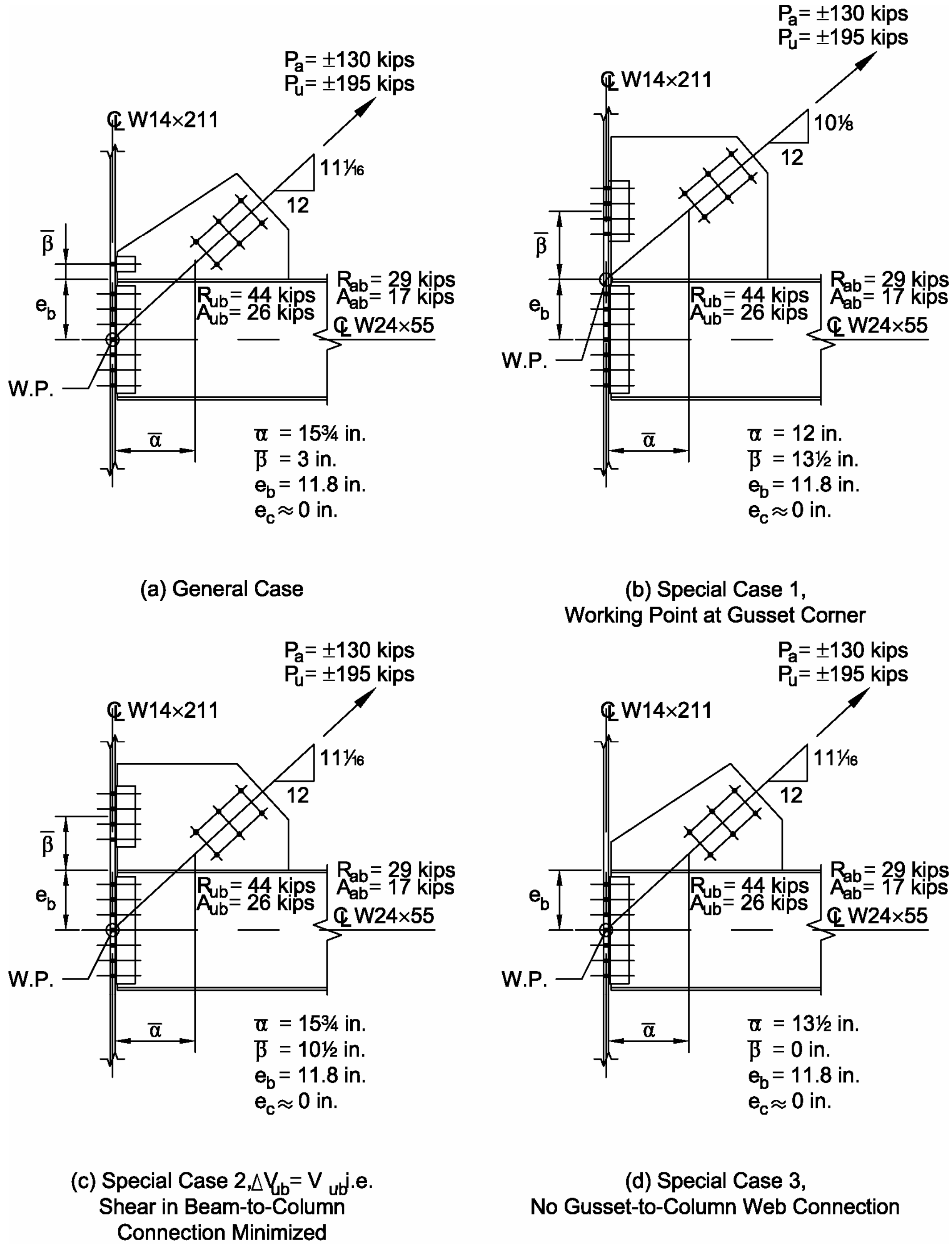
Given:

Each of the four designs shown for the diagonal bracing connection between the W14×68 brace, W24×55 beam, and W14×211 column web have been developed using the Uniform Force Method (the General Case and Special Cases 1, 2, and 3) for the load case of 1.2D + 1.3W for LRFD and D + W for ASD.

For the given values of α and β, determine the interface forces on the gusset-to-column and gusset-to-beam connections for

a. General Case of Figure (a)
b. Special Case 1 of Figure (b)
c. Special Case 2 of Figure (c)
d. Special Case 3 of Figure (d)

Brace Axial Load	P_u = ±195 kips	P_a = ±130 kips
Beam End Reaction	R_u = 44 kips	R_a = 29 kips
Beam Axial Load	A_u = 26 kips	A_a = 17 kips



Material Properties:

Brace	W14×68	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual Tables 2-3 and 2-4
Beam	W24×55	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	
Column	W14×211	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	
Gusset Plate		ASTM A36	F_y = 36 ksi	F_u = 58 ksi	

Geometric Properties:

Brace	W14×68	A=20.0 in.^{2}	d=14.0 in.	t_w=0.415 in.	b_f=10.0 in.	t_f=0.720 in.	Manual Table 1-1
Beam	W24×55	d=23.6 in.	t_w=0.395 in.	b_f=7.01 in.	t_f=0.505 in.	k=1.11	
Column	W14×211	d=15.7 in.	t_w=0.980 in.	b_f=15.8 in.	t_f=1.56 in.		

Solution A (General Case):

Assume $\beta = \bar{\beta} = 3$ in.

$$\alpha = e_b \tan\theta - e_c + \beta\tan\theta = (11.8\text{ in.})\left(\frac{12}{11\frac{1}{16}}\right) - 0 + (3\text{ in.})\left(\frac{12}{11\frac{1}{16}}\right) = 16.1\text{ in.}$$

Since $\alpha \neq \bar{\alpha}$, an eccentricity exists on the gusset-to-beam connection.

Calculate the interface forces

$$r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2} = \sqrt{(16.1\text{ in.} + 0\text{ in.})^2 + (3\text{ in.} + 11.8\text{ in.})^2} = 21.9\text{ in.}$$

On the gusset-to-column connection

LRFD	ASD
$V_{uc} = \dfrac{\beta}{r}P_u = \dfrac{3\text{ in.}}{21.9\text{ in.}}(195\text{ kips}) = 26.7\text{ kips}$ $H_{uc} = \dfrac{e_c}{r}P_u = 0\text{ kips}$	$V_{ac} = \dfrac{\beta}{r}P_a = \dfrac{3\text{ in.}}{21.9\text{ in.}}(130\text{ kips}) = 17.8\text{ kips}$ $H_{ac} = \dfrac{e_c}{r}P_a = 0\text{ kips}$

On the gusset-to-beam connection

LRFD	ASD
$H_{ub} = \dfrac{\alpha}{r}P_u = \dfrac{16.1\text{ in.}}{21.9\text{ in.}}(195\text{ kips}) = 143\text{ kips}$ $V_{ub} = \dfrac{e_b}{r}P_u = \dfrac{11.8\text{ in.}}{21.9\text{ in.}}(195\text{ kips}) = 105\text{ kips}$ $M_{ub} = V_{ub}(\alpha - \bar{\alpha})$ $= \dfrac{(105\text{ kips})(15\frac{3}{4}\text{ in.} - 16.1\text{ in.})}{12\text{ in./ft}}$ = -3.06 kip-ft	$H_{ab} = \dfrac{\alpha}{r}P_a = \dfrac{16.1\text{ in.}}{21.9\text{ in.}}(130\text{ kips}) = 95.6\text{ kips}$ $V_{ab} = \dfrac{e_b}{r}P_a = \dfrac{11.8\text{ in.}}{21.9\text{ in.}}(130\text{ kips}) = 70.0\text{ kips}$ $M_{ab} = V_{ab}(\alpha - \bar{\alpha})$ $= \dfrac{(70.0\text{ kips})(15\frac{3}{4}\text{ in.} - 16.1\text{ in.})}{12\text{ in./ft}}$ = -2.04 kip-ft

In this case, this small moment is negligible.

On the beam-to-column connection, the shear is

LRFD	ASD
$R_{ub} + V_{ub}$ = 44 kips + 105 kips = 149 kips	$R_{ab} + V_{ab}$ = 29 kips + 70 kips = 99 kips

and the axial force is

LRFD	ASD
$A_{ub} + H_{uc}$ = 26 kips ± 0 kips = 26 kips	$A_{ab} + H_{ac}$ = 17 kips ± 0 kips = 17 kips

For a discussion of the sign use between A_{ub} and H_{uc} (A_{ab} an H_{ac} for ASD), refer to AISC (1992).

Solution B (Special Case 1):

In this case, the centroidal positions of the gusset-edge connections are irrelevant; $\bar{\alpha}$ and $\bar{\beta}$ are given to define the geometry of the connection, but are not needed to determine the gusset edge forces.

The angle of the brace from the vertical is

$$\theta = \tan^{-1}\left(\frac{12}{10\,\frac{1}{8}}\right) = 49.8°$$

The horizontal and vertical components of the brace force are

LRFD	ASD
$H_u = P_u \sin\theta = (195\text{ kips})\sin 49.8° = 149\text{ kips}$ $V_u = P_u \cos\theta = (195\text{ kips})\cos 49.8° = 126\text{ kips}$	$H_a = P_a \sin\theta = (130\text{ kips})\sin 49.8° = 99.3\text{ kips}$ $V_a = P_a \cos\theta = (130\text{ kips})\cos 49.8° = 83.9\text{ kips}$

On the gusset-to-column connection

LRFD	ASD
$V_{uc} = V_u = 126\text{ kips}$ $H_{uc} = 0\text{ kips}$	$V_{ac} = V_a = 83.9\text{ kips}$ $H_{ac} = 0\text{ kips}$

On the gusset-to-beam connection

LRFD	ASD
$V_{ub} = 0\text{ kips}$ $H_{ub} = H_u = 149\text{ kips}$	$V_{ab} = 0\text{ kips}$ $H_{ab} = H_a = 99.3\text{ kips}$

On the beam-to-column connection

LRFD	ASD
R_{ub} = 44 kips (shear) A_{ub} = 26 kips (axial transfer force)	R_{ab} = 29 kips (shear) A_{ab} = 17 kips (axial transfer force)

In addition to the forces on the connection interfaces, the beam is subjected to a moment M_{ub} or or M_{ab}, where

LRFD	ASD
$M_{ub} = H_{ub}e_b = \frac{(149\text{ kips})(11.8\text{ in.})}{12\text{ in./ft}}$ $= 147\text{ kip-ft}$	$M_{ab} = H_{ab}e_b = \frac{(99.3\text{ kips})(11.8\text{ in.})}{12\text{ in./ft}}$ $= 97.6\text{ kip-ft}$

This moment, as well as the beam axial load H_{ub} = 149 kips or H_{ab} = 99.3 kips and the moment and shear in the beam associated with the end reaction R_{ub} or R_{ab}, must be considered in the design of the beam.

Solution C (Special Case 2):

Assume $\beta = \bar{\beta} = 10\,^1\!/_2$ in.

$$\alpha = e_b \tan\theta - e_c + \beta\tan\theta = (11.8\text{ in.})\left(\frac{12}{11\,^1\!/_{16}}\right) - 0 + (10\,^1\!/_2\text{ in.})\left(\frac{12}{11\,^1\!/_{16}}\right) = 24.2\text{ in.}$$

Calculate the interface forces for the general case before applying Special Case 2.

$$r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2} = \sqrt{(24.2\text{ in.} + 0\text{ in.})^2 + (10\,^1\!/_2\text{ in.} + 11.8\text{ in.})^2} = 32.9\text{ in.}$$

On the gusset-to-column connection

LRFD	ASD
$V_{uc} = \dfrac{\beta}{r}P_u = \dfrac{10\,^1\!/_2\text{ in.}}{32.9\text{ in.}}(195\text{ kips}) = 62.2\text{ kips}$ $H_{uc} = \dfrac{e_c}{r}P_u = 0\text{ kips}$	$V_{ac} = \dfrac{\beta}{r}P_a = \dfrac{10\,^1\!/_2\text{ in.}}{32.9\text{ in.}}(130\text{ kips}) = 41.5\text{ kips}$ $H_{ac} = \dfrac{e_c}{r}P_a = 0\text{ kips}$

On the gusset-to-beam connection

LRFD	ASD
$H_{ub} = \dfrac{\alpha}{r}P_u = \dfrac{24.2\text{ in.}}{32.9\text{ in.}}(195\text{ kips}) = 143\text{ kips}$ $V_{ub} = \dfrac{e_b}{r}P_u = \dfrac{11.8\text{ in.}}{32.9\text{ in.}}(195\text{ kips}) = 69.9\text{ kips}$	$H_{ab} = \dfrac{\alpha}{r}P_a = \dfrac{24.2\text{ in.}}{32.9\text{ in.}}(130\text{ kips}) = 95.6\text{ kips}$ $V_{ab} = \dfrac{e_b}{r}P_a = \dfrac{11.8\text{ in.}}{32.9\text{ in.}}(130\text{ kips}) = 46.6\text{ kips}$

On the beam-to-column connection, the shear is

LRFD	ASD
$R_{ub} + V_{ub}$ = 44 kips + 69.9 kips = 114 kips	$R_{ab} + V_{ab}$ = 29 kips + 46.6 kips = 75.6 kips

and the axial force is

LRFD	ASD
$A_{ub} + H_{uc}$ = 26 kips ± 0 kips = 26 kips	$A_{ab} + H_{ac}$ = 17 kips ± 0 kips = 17 kips

Next, applying Special Case 2 with $\Delta V_{ub} = V_{ub}$ = 69.9 kips ($\Delta V_{ab} = V_{ab}$ = 46.6 kips for ASD), calculate the interface forces.

On the gusset-to-column connection (where V_{uc} is replaced by $V_{uc} + \Delta V_{ub}$) or (where V_{ac} is replaced by $V_{ac} + \Delta V_{ab}$ for ASD)

LRFD	ASD
$V_{uc} = 62.2\text{ kips} + 69.9\text{ kips} = 132\text{ kips}$ $H_{uc} = 0\text{ kips}$ (unchanged)	$V_{ac} = 41.5\text{ kips} + 46.6\text{ kips} = 88.1\text{ kips}$ $H_{ac} = 0\text{ kips}$ (unchanged)

On the gusset-to-beam connection (where V_{ub} is replaced by $V_{ub} - \Delta V_{ub}$) or (where V_{ab} is replaced by $V_{ab} - \Delta V_{ab}$)

LRFD	ASD
$H_{ub} = 143\text{ kips}$ (unchanged) $V_{ub} = 69.9\text{ kips} - 69.9\text{ kips} = 0\text{ kips}$ $M_{ub} = (\Delta V_{ub})\alpha = \dfrac{(69.9\text{ kips})(24.2\text{ in.})}{12\text{in./ft}}$ $= 141\text{ kip-ft}$	$H_{ab} = 95.6\text{ kips}$ (unchanged) $V_{ab} = 46.6\text{ kips} - 46.6\text{ kips} = 0\text{ kips}$ $M_{ab} = (\Delta V_{ab})\alpha = \dfrac{(46.6\text{ kips})(24.2\text{ in.})}{12\text{in./ft}}$ $= 94.0\text{ kip-ft}$

On the beam-to-column connection, the shear is

LRFD	ASD
$R_{ub} + \Delta V_{ub} - \Delta V_{ub}$ $= 44\text{ kips} + 69.9\text{ kips} - 69.9\text{ kips}$ $= 44\text{ kips}$	$R_{ab} + \Delta V_{ab} - \Delta V_{ab}$ $= 29\text{ kips} + 46.6\text{ kips} - 46.6\text{ kips}$ $= 29\text{ kips}$

and the axial force is

LRFD	ASD
$A_{ub} + H_{uc} = 26\text{ kips} \pm 0\text{ kips} = 26\text{ kips}$	$A_{ab} + H_{ac} = 17\text{ kips} \pm 0\text{ kips} = 17\text{ kips}$

Solution D (Special Case 3):

Set $\beta = \bar{\beta} = 0$ in.

$$\alpha = e_b \tan\theta = (11.8\text{ in.})\left(\frac{12}{11\frac{1}{16}}\right) = 12.8\text{ in.}$$

Since, $\alpha \neq \bar{\alpha}$, an eccentricity exists on the gusset-to-beam connection.

Calculate the interface forces

$$r = \sqrt{\alpha^2 + e_b{}^2} = \sqrt{(12.8\text{ in.})^2 + (11.8\text{ in.})^2} = 17.4\text{ in.}$$

On the gusset-to-beam connection

LRFD	ASD
$H_{ub} = \dfrac{\alpha}{r}P_u = \dfrac{12.8\text{ in.}}{17.4\text{ in.}}(195\text{ kips}) = 143\text{ kips}$ $V_{ub} = \dfrac{e_b}{r}P_u = \dfrac{11.8\text{ in.}}{17.4\text{ in.}}(195\text{ kips}) = 132\text{ kips}$ $M_{ub} = V_{ub}\left(\alpha - \bar{\alpha}\right)$ $= \dfrac{(132\text{ kips})(12.8\text{ in.} - 13\frac{1}{2}\text{ in.})}{12\text{ in./ft}}$ = -7.70 kip-ft	$H_{ab} = \dfrac{\alpha}{r}P_a = \dfrac{12.8\text{ in.}}{17.4\text{ in.}}(130\text{ kips}) = 95.6\text{ kips}$ $V_{ab} = \dfrac{e_b}{r}P_a = \dfrac{11.8\text{ in.}}{17.4\text{ in.}}(130\text{ kips}) = 88.2\text{ kips}$ $M_{ab} = V_{ab}\left(\alpha - \bar{\alpha}\right)$ $= \dfrac{(88.2\text{ kips})(12.8\text{ in.} - 13\frac{1}{2}\text{ in.})}{12\text{ in./ft}}$ = -5.15 kip-ft

In this case, this small moment is negligible.

On the beam-to-column connection, the shear is

LRFD	ASD
$R_{ub} + V_{ub}$ = 44 kips + 132 kips = 176 kips	$R_{ab} + V_{ab}$ = 29 kips + 88.2 kips = 117 kips

And the axial force is

LRFD	ASD
$A_{ub} + H_{uc}$ = 26 kips ± 0 kips = 26 kips	$A_{ab} + H_{ac}$ = 17 kips ± 0 kips = 17 kips

Note: From the foregoing results, designs by Special Case 3 and the General Case of the Uniform Force Method provide the more economical designs. Additionally, note that designs by Special Case 1 and Special Case 2 result in moments on the beam and/ or column that must be considered.

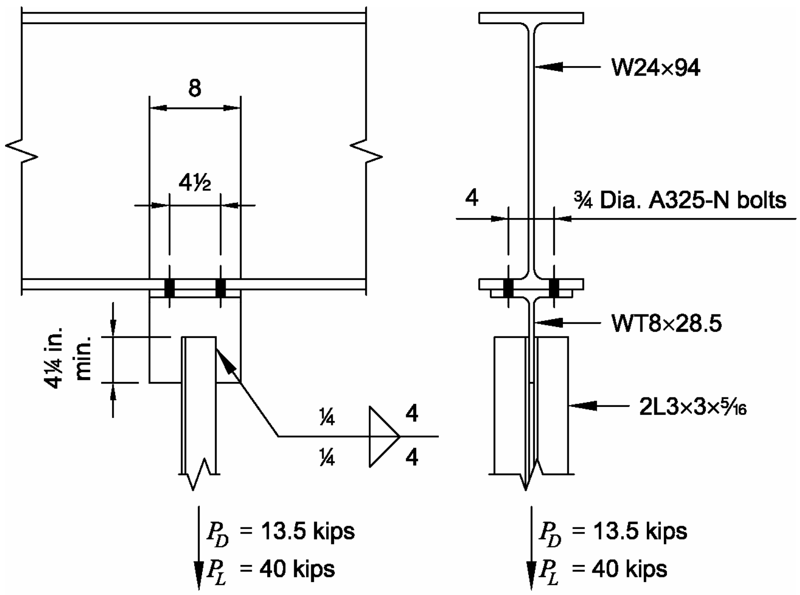
Example II.D-1 Prying Action in Tees and in Single Angles

Given:

Design a WT tension-hanger connection between a 2L3×3×5⁄16 tension member and a W24×94 beam connection to support the following loads:

P_D = 13.5 kips
P_L = 40 kips

Use ¾–in. diameter ASTM A325-N bolts and 70 ksi electrodes.



Material Properties:

Hanger	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual Table 2-3
Beam W24×94	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	
Angles 2L3×3×5⁄16	ASTM A36	F_y = 36 ksi	F_u = 58 ksi	

Geometric Properties:

Beam W24×94 d = 24.3 in. t_w = 0.515 in. b_f = 9.07 in. t_f = 0.875 in.
Angles 2L3×3×5⁄16 A = 3.55 in.^{2} $\bar{x}$ = 0.860 in.

Manual Tables 1-1, 1-7, and 1-15

Solution:

LRFD	ASD
$P_u = 1.2(13.5 \text{ kips}) + 1.6(40 \text{ kips}) = 80.2 \text{ kips}$	$P_a = 13.5 \text{ kips} + 40 \text{ kips} = 53.5 \text{ kips}$

Check tension yielding of angles Eqn. D2-1

$R_n = F_y A_g = (36\text{ ksi})(3.55\text{ in.}^2) = 128\text{ kips}$

LRFD	ASD
$\phi R_n = 0.90(128\text{ kips})$ $= 115\text{ kips} > 80.2\text{ kips}$	$R_n / \Omega = \dfrac{128\text{ kips}}{1.67}$ $= 76.6\text{ kips} > 53.5\text{ kips}$
Try ¼–in. fillet welds $L_{min} = \dfrac{P_u}{1.392D} = \dfrac{80.2\text{ kips}}{1.392(4\text{ sixteenths})}$ $= 14.4\text{ in.}$ Use four 4-in. welds (16 in. total), one at each toe and heel of each angle.	Try ¼–in. fillet welds $L_{min} = \dfrac{P_a}{0.928D} = \dfrac{53.5\text{ kips}}{0.928(4\text{ sixteenths})}$ $= 14.4\text{ in.}$ Use four 4-in. welds (16 in. total), one at each toe and heel of each angle.

Check tension rupture of angles

Calculate the effective net area

$U = 1 - \dfrac{\bar{x}}{L} = 1 - \dfrac{0.860\text{ in.}}{4\text{ in.}} = 0.785$ Table D3.1 Case 2

$A_e = A_n U = (3.55\text{ in.}^2)(0.785) = 2.80\text{ in.}^2$ Eqn. D2-2

LRFD	ASD
$\phi = 0.75$ $R_n = F_u A_e = (58\text{ ksi})(2.80\text{ in.}^2) = 163\text{ kips}$ $\phi_t R_n = 0.75(163)$ $= 122\text{ kips} > 80\text{ kips}$ **o.k.**	$\Omega = 2.00$ $R_n = F_u A_e = (58\text{ ksi})(2.80\text{ in.}^2) = 163\text{ kips}$ $R_n / \Omega_t = \dfrac{163}{2.00}$. $= 81.5\text{ kips} > 53.5\text{ kips}$ **o.k.**
Select a preliminary WT *using beam gage* $g = 4\text{ in.}$ With four ¾–in. diameter ASTM A325-N bolts, $T_u = r_{ut} = \dfrac{P_u}{n} = \dfrac{80\text{ kips}}{4} = 20\text{ kips/bolt}$ $B = \phi r_n = 29.8\text{ kips} > 20\text{ kips}$ **o.k.**	*Select a preliminary* WT *using beam gage* $g = 4\text{ in.}$ With four ¾–in. diameter ASTM A325-N bolts, $T_a = r_{at} = \dfrac{P_a}{n} = \dfrac{53.5\text{ kips}}{4} = 13.4\text{ kips/bolt}$ $B = r_n / \Omega = 19.9\text{ kips} > 13.4\text{ kips}$ **o.k.**

Manual Table 7-2

With four bolts, the maximum effective length is $2g$ = 8 in. Thus, there are 4 in. of tee length tributary to each pair of bolts and

LRFD	ASD
$\dfrac{2\text{ bolts}(20\text{ kips/bolt})}{4\text{ in.}} = 10.0\text{ kips/in.}$	$\dfrac{2\text{ bolts}(13.4\text{ kips/bolt})}{4\text{ in.}} = 6.7\text{ kips/in.}$

The minimum depth WT that can be used is equal to the sum of the weld length plus the weld size plus the k-dimension for the selected section. From Manual Table 1-8 with an assumed b = 4 in./2 = 2 in., $t_0 \approx 1\frac{1}{16}$ in., and $d_{min} = 4\text{ in.} + \frac{1}{4}\text{ in.} + k \approx 6\text{ in.}$, appropriate selections include:

WT6×39.5 WT8×28.5
WT7×34 WT9×30

Try WT8×28.5; b_f = 7.12 in., t_f = 0.715 in., t_w = 0.430 in. Manual Table 1-8

Check prying action Manual Part 9

$$b = \frac{g - t_w}{2} = \frac{(4\text{ in.} - 0.430\text{ in.})}{2} = 1.79\text{ in.} > 1\tfrac{1}{4}\text{-in. entering and tightening clearance, o.k.}$$

$$a = \frac{b_f - g}{2} = \frac{(7.12\text{ in.} - 4\text{ in.})}{2} = 1.56\text{ in.}$$

Since a = 1.56 in. < 1.25b = 2.24 in., use a = 1.56 in.

$$b' = b - \frac{d}{2} = 1.79\text{ in.} - \left(\frac{\frac{3}{4}\text{ in.}}{2}\right) = 1.42\text{ in.}$$

$$a' = a + \frac{d}{2} = 1.56\text{ in.} + \left(\frac{\frac{3}{4}\text{ in.}}{2}\right) = 1.94\text{ in.}$$

$$\rho = \frac{b'}{a'} = \frac{1.42\text{ in.}}{1.94\text{ in.}} = 0.732$$

LRFD	ASD
$\beta = \dfrac{1}{\rho}\left(\dfrac{B}{T} - 1\right) = \dfrac{1}{0.732}\left(\dfrac{29.8\text{ kips/bolt}}{20\text{ kips}} - 1\right)$ $= 0.669$	$\beta = \dfrac{1}{\rho}\left(\dfrac{B}{T} - 1\right) = \dfrac{1}{0.732}\left(\dfrac{19.9\text{ kips/bolt}}{13.4\text{ kips}} - 1\right)$ $= 0.663$

$$\delta = 1 - \frac{d'}{p} = 1 - \left(\frac{\frac{13}{16}\text{ in.}}{4\text{ in.}}\right) = 0.797$$

Since $\beta < 1.0$,

LRFD	ASD
$\alpha' = \frac{1}{\delta}\left(\frac{\beta}{1-\beta}\right) \leq 1.0$	$\alpha' = \frac{1}{\delta}\left(\frac{\beta}{1-\beta}\right) \leq 1.0$
$= \frac{1}{0.797}\left(\frac{0.669}{1-0.669}\right)$	$= \frac{1}{0.797}\left(\frac{0.663}{1-0.663}\right)$
$= 2.54 \;\therefore\; \alpha' = 1.0$	$= 2.47 \;\therefore\; \alpha' = 1.0$
$t_{\min} = \sqrt{\frac{4.44Tb'}{pF_u(1+\delta\alpha')}}$	$t_{\min} = \sqrt{\frac{6.66Tb'}{pF_u(1+\delta\alpha')}}$
$= \sqrt{\frac{4.44(20\text{ kips/bolt})(1.42\text{ in.})}{(4\text{ in.})(65\text{ ksi})[1+(0.797)(1.0)]}}$	$= \sqrt{\frac{6.66(13.4\text{ kips/bolt})(1.42\text{ in.})}{(4\text{ in.})(65\text{ ksi})[1+(0.797)(1.0)]}}$
$= 0.521\text{ in.} < t_f = 0.715\text{ in.}$ **o.k.**	$= 0.521\text{ in.} < t_f = 0.715\text{ in.}$ **o.k.**

Check tension yielding of the tee stem on the Whitmore section

Manual Part 9

The effective width of the tee stem (which cannot exceed the actual width of 8 in.) is

$$L_w = 3\text{ in.} + 2(4\text{ in.})(\tan 30^\circ) \leq 8\text{ in.}$$

$$= 7.62\text{ in.}$$

and the nominal strength is determined as

LRFD	ASD
$\phi = 0.90$	$\Omega = 1.67$
$R_n = F_y A_{geff} = (50\text{ ksi})(7.62\text{ in.})(0.430\text{ in.}) = 164\text{ kips}$	$R_n = F_y A_{geff} = (50\text{ ksi})(7.62\text{ in.})(0.430\text{ in.}) = 164\text{ kips}$
$\phi R_n = 0.90(164) = 147\text{ kips} > 80\text{ kips}$ **o.k.**	$R_n / \Omega = \frac{164}{1.67} = 98.2\text{ kips} > 53.5\text{ kips}$ **o.k.**

Eqn. D2-1

Manual Part 9

Check shear rupture of the base metal along the toe and heel of each weld line

$$t_{min} = \frac{6.19D}{F_u}$$

$$= 6.19\left(\frac{4}{65\text{ ksi}}\right)$$

$$= 0.381\text{ in.} < 0.430\text{ in.}$$ **o.k.**

Check block shear rupture of the tee stem

Since the angles are welded to the WT-hanger the gross area shear yielding will control.

$$A_{gv} = (2\text{ welds})(4\text{ in.})(0.430\text{ in.}) = 3.44\text{ in.}^2$$

Tension stress is uniform, therefore U_{bs} = 1.0.

Table D3.1

$$A_{nt} = A_g = 1.0(3 \text{ in.})(0.430 \text{ in.}) = 1.29 \text{ in.}^2$$

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

$$R_n = 0.6F_yA_{gv} + U_{bs}F_uA_{nt} = 0.6(50 \text{ ksi})(3.44 \text{ in.}^2) + 1.0(65 \text{ ksi})(1.29 \text{ in.}^2) = 187 \text{ kips}$$

Eqn. J4-5

LRFD	ASD
$\phi = 0.75$ ϕR_n = 0.75(187) = 140 kips > 80 kips **o.k.**	$\Omega = 2.00$ $R_n / \Omega = \dfrac{187}{2.00}$ = 93.5 kips > 53.5 kips **o.k.**

Note: Alternately, a WT tension hanger could be selected with a flange thickness to reduce the effect of prying action to an insignificant amount, i.e., $q_u \approx 0$. Assumsing $b' = 1.42$ in.

LRFD	ASD
$t_{\min} = \sqrt{\dfrac{4.44Tb'}{pF_u}}$ $= \sqrt{\dfrac{4.44(20 \text{ kips/bolt})(1.42 \text{ in.})}{(4 \text{ in./bolt})(65 \text{ ksi})}}$ = 0.696 in. Try WT9×35.5 t_f = 0.810 in. > 0.696 in. **o.k.** t_w = 0.495 in. > 0.430 in. **o.k.** b_f = 7.64 in. > 7.12 in. **o.k.**	$t_{\min} = \sqrt{\dfrac{6.66Tb'}{pF_u}}$ $= \sqrt{\dfrac{6.66(13.4 \text{ kips/bolt})(1.42 \text{ in.})}{(4 \text{ in./bolt})(65 \text{ ksi})}}$ = 0.698 in. Try WT9×35.5 t_f = 0.810 in. > 0.698 in. **o.k.** t_w = 0.495 in. > 0.430 in. **o.k.** b_f = 7.64 in. > 7.12 in. **o.k.**

Manual Table 1-8