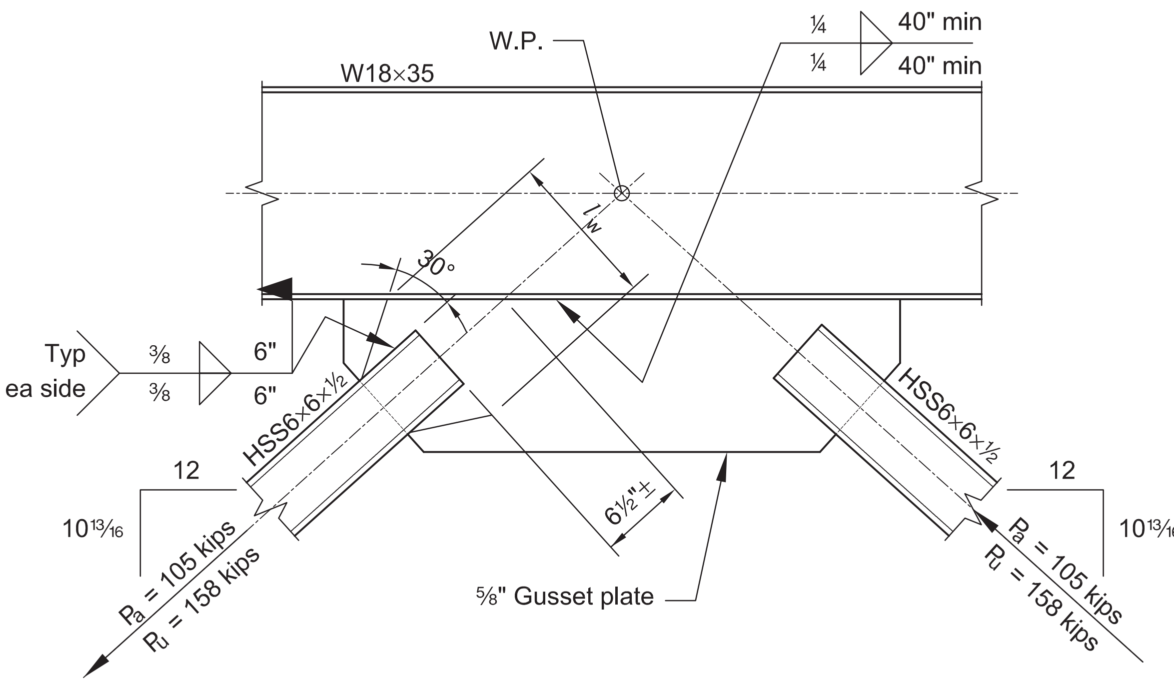
Example II.C-5 HSS Chevron Brace Connection

Given:

Check the HSS braces for tension and compression and design the connection including welding, shear lag requirements and check to see if stiffening is required.

Use E70 electrodes.



Material Properties:

Beam	W18×35	ASTM A992	F_y = 50 ksi	F_u = 65 ksi	Manual Tables 2-3 and 2-4
Brace	HSS6×6×½	ASTM A500 Gr.B	F_y = 46 ksi	F_u = 58 ksi	
Gusset Plate		ASTM A36	F_y = 36 ksi	F_u = 58 ksi	

Geometric Properties:

Beam	W18×35	d = 17.7 in.	t_w = 0.300 in.	k_{des} = 0.827 in.		Manual Tables 1-1 and 1-12
Brace	HSS6×6×½	H = 6.00 in.	B = 6.00 in.	A = 9.74 in.^{2}	t = 0.465 in.	

Solution:

Determine the required brace to gusset weld size.

Manual Part 8

LRFD	ASD
$D_{req} = \dfrac{R_u}{1.392l}$ $= \dfrac{158 \text{ kips}}{1.392(4)(6.00 \text{ in.})}$ $= 4.73$ sixteenths Increase the weld size by $\frac{1}{16}$ in. to account	$D_{req} = \dfrac{R_a}{0.928l}$ $= \dfrac{105 \text{ kips}}{0.928(4)(6.00 \text{ in.})}$ $= 4.71$ sixteenths Increase the weld size by $\frac{1}{16}$ in. to account

LRFD	ASD
for slot in the HSS D_{req} = 4.73 sixteenths + 1.00 sixteenths = 5.73 sixteenths	for the slot in the HSS D_{req} = 4.71 sixteenths + 1.00 sixteenth = 5.71 sixteenths

The minimum weld size for this connection is 3⁄16 in. — Manual Table J2.4

Use ⅜ in. fillet welds

Determine the minimum gusset plate thickness to match the required shear rupture strength of the fillet welds.

Manual Part 9

$$t_{\min} = \frac{6.19D}{F_u} = \frac{6.19(4.73 \text{ sixteenths})}{58 \text{ ksi}} = 0.505 \text{ in.}$$

Try a ⅝ in. thick gusset plate.

Determine the minimum HSS brace thickness to match the required shear rupture strength of the fillet welds.

$$t_{\min} = \frac{3.09D}{F_u} = \frac{3.09(4.73 \text{ sixteenths})}{58 \text{ ksi}} = 0.252 \text{ in.} < \tfrac{1}{2} \text{ in.} \qquad \textbf{o.k.}$$

Check gusset plate buckling (compression brace)

$$r = \frac{t_p}{\sqrt{12}} = \frac{0.625 \text{ in.}}{\sqrt{12}} = 0.180 \text{ in.}$$

From the figure, the distance l_1 = 6.50 in.

Since the gusset is attached by one edge only, the buckling mode could be a sidesway type as shown in Commentary Table C-C2.2. In this case, use K = 1.2.

$$\frac{Kl_1}{r} = \frac{1.2\left(6.50 \text{ in.}\right)}{0.180 \text{ in.}} = 43.3$$

Limiting slenderness ratio $4.71\sqrt{\frac{E}{F_y}} = 4.71\sqrt{\frac{29{,}000 \text{ ksi}}{36 \text{ ksi}}} = 134 > 43.3$

$$F_e = \frac{\pi^2 E}{\left(\frac{Kl_1}{r}\right)^2} = \frac{\pi^2\left(29{,}000 \text{ ksi}\right)}{\left(43.3\right)^2} = 153 \text{ ksi}$$ Eqn. E3-4

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right]F_y = \left[0.658^{\frac{36 \text{ ksi}}{153 \text{ ksi}}}\right]\left(36 \text{ ksi}\right) = 32.6 \text{ ksi}$$ Eqn. E3-2

$$l_w = B + 2\left[(\text{connection length})\tan 30°\right] = 6.00 \text{ in.} + 2\left(6.00 \text{ in.}\right)\tan 30° = 12.9 \text{ in.}$$

Note: Here, the Whitmore section is assumed to be entirely in the gusset. The Whitmore section can spread across the joint into adjacent connected material of equal or greater thickness or adjacent connected material of lesser thickness provided that a rational analysis is performed.

$$A_w = l_w t_p = (12.9 \text{ in.})(0.625 \text{ in.}) = 8.06 \text{ in.}^2$$

Eqn. E3-1

$$P_n = F_{cr} A_w = (32.6 \text{ ksi})(8.06 \text{ in.}^2) = 263 \text{ kips}$$

LRFD	ASD
$\phi P_n = 0.90(263 \text{ kips}) = 237 \text{ kips}$	$P_n / \Omega = \dfrac{263 \text{ kips}}{1.67} = 157 \text{ kips}$
237 kips > 158 kips **o.k.**	157 kips > 105 kips **o.k.**

Check tension yielding of gusset plate (tension brace)

From above, A_w = 8.06 in.^{2}

$$R_n = F_y A_w = (36 \text{ ksi})(8.06 \text{ in.}^2) = 290 \text{ kips}$$

Eqn. J4-1

LRFD	ASD
$\phi R_n = 0.90(290 \text{ kips}) = 261 \text{ kips}$	$R_n / \Omega = \dfrac{290 \text{ kips}}{1.67} = 174 \text{ kips}$
261 kips > 158 kips **o.k.**	174 kips > 105 kips **o.k.**

Check the available tensile yield strength of the HSS brace

$$R_n = F_y A_g = (46 \text{ ksi})(9.74 \text{ in.}^2) = 448 \text{ kips}$$

Eqn. J4-1

LRFD	ASD
$\phi R_n = 0.9(448 \text{ kips}) = 403 \text{ kips} > 158 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = \dfrac{(448 \text{ kips})}{1.67} = 268 \text{ kips} > 105 \text{ kips}$ **o.k.**

Check the available tensile rupture strength of the HSS brace

$$\bar{x} = \frac{B^2 + 2BH}{4(B+H)} = \frac{(6.00 \text{ in.})^2 + 2(6.00 \text{ in.})(6.00 \text{ in.})}{4(6.00 \text{ in.} + 6.00 \text{ in.})} = 2.25 \text{ in.}$$

Table D3.1
Case 6

$$U = 1 - \frac{\bar{x}}{L_w} = 1 - \frac{2.25 \text{ in.}}{6.00 \text{ in.}} = 0.625$$

Allowing for a $^1\!/_{16}$ in. gap in fit-up between the HSS and the gusset plate,

$$A_n = A_g - 2\left(t_p + {}^1\!/_{16} \text{ in.}\right)t = 9.74 \text{ in.}^2 - 2\left(0.625 \text{ in.} + {}^1\!/_{16} \text{ in.}\right)(0.465 \text{ in.}) = 9.10 \text{ in.}^2$$

$$A_e = UA_n = 0.625(9.10 \text{ in.}^2) = 5.69 \text{ in.}^2$$

Eqn. D3-1

$$R_n = F_u A_e = (58 \text{ ksi})(5.69 \text{ in.}^2) = 330 \text{ kips}$$

Eqn. J4-2

LRFD	ASD
$\phi R_n = 0.75(330 \text{ kips}) = 248 \text{ kips}$ 248 kips > 158 kips **o.k.**	$R_n / \Omega = \dfrac{330 \text{ kips}}{2.00} = 165 \text{ kips}$ 165 kips > 105 kips **o.k.**

Calculate interface forces

Design the gusset-to-beam connection as if each brace were the only brace and locate each brace's connection centroid at the ideal centroid locations to avoid inducing a moment on the gusset-beam interface, similarly to uniform force method special case 3.

$$e_b = \frac{d}{2} = \frac{17.7 \text{ in.}}{2} = 8.85 \text{ in.}$$

$$\theta = \tan^{-1}\left(\frac{12}{10\,{}^{13}\!/_{16}}\right) = 48.0°$$

Let $\bar{\alpha} = \alpha = e_b \tan\theta = (8.85 \text{ in.})\tan 48.0° = 9.83 \text{ in.} \rightarrow$ Use 10.0 in.

$$\beta = e_c = 0$$

$$r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2} = \sqrt{(10.0 \text{ in.} + 0)^2 + (0 + 8.85 \text{ in.})^2} = 13.4 \text{ in.}$$

LRFD	ASD
$H_{ub} = \dfrac{\alpha P_u}{r} = \dfrac{(10.0 \text{ in.})(158 \text{ kips})}{13.4 \text{ in.}} = 118 \text{ kips}$ $V_{ub} = \dfrac{e_b P_u}{r} = \dfrac{(8.85 \text{ in.})(158 \text{ kips})}{13.4 \text{ in.}} = 104 \text{ kips}$	$H_{ab} = \dfrac{\alpha P_a}{r} = \dfrac{(10.0 \text{ in.})(105 \text{ kips})}{13.4 \text{ in.}} = 78.4 \text{ kips}$ $V_{ab} = \dfrac{e_b P_a}{r} = \dfrac{(8.85 \text{ in.})(105 \text{ kips})}{13.4 \text{ in.}} = 69.3 \text{ kips}$

Determine required gusset-to-beam weld size

The weld length is twice the horizontal distance from the work point to the centroid of the gusset-to-beam connection, α, for each brace. Therefore, $l = 2\alpha = 2(10.0 \text{ in.}) = 20.0 \text{ in.}$

Since the gusset straddles the work line of each brace, the weld is uniformly loaded. Therefore, the available strength is the average required strength and the fillet weld should be designed for 1.25 times the average strength. Manual Part 13

LRFD	ASD
$D_{req'd} = \dfrac{1.25 P_u}{1.392 l} = \dfrac{1.25(158 \text{ kips})}{1.392(20.0 \text{ in.})(2)} = 3.55$	$D_{req'd} = \dfrac{1.25 P_a}{0.928 l} = \dfrac{1.25(105 \text{ kips})}{0.928(20.0 \text{ in.})(2)} = 3.54$

The minimum fillet weld size is ¼ in. The required weld size is also ¼ in., use a ¼ in. fillet weld 40 in. long on each side of the gusset plate. Table J2.4

Check gusset thickness (against weld size required for strength)

Manual Part 9

$$t_{\min} = \frac{6.19D}{F_u} = \frac{6.19(3.55 \text{ sixteenths})}{58 \text{ ksi}} = 0.379 \text{ in.} < \tfrac{5}{8} \text{ in.} \quad \textbf{o.k.}$$

Check local web yielding of the beam

Eqn. J10-2

$$R_n = (N + 5k)F_y t_w = [20.0 \text{ in.} + 5(0.827 \text{ in.})](50 \text{ ksi})(0.300 \text{ in.}) = 362 \text{ kips}$$

LRFD	ASD
$R_u = 158 \text{ kips}(\cos\ 48.0°) = 106 \text{ kips}$ $\phi = 1.00$ $\phi R_n = 1.00(362 \text{ kips}) = 362 \text{ kips}$ 362 kips > 106 kips **o.k.**	$R_a = 105 \text{ kips}(\cos\ 48.0°) = 70.3 \text{ kips}$ $\Omega = 1.50$ $R_n / \Omega = \dfrac{362 \text{ kips}}{1.50} = 241 \text{ kips}$ 241 kips > 70.3 kips **o.k.**

Check web crippling

Eqn. J10-4

$$R_n = 0.80 t_w^{\,2}\left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}}$$

$$= 0.80(0.300 \text{ in.})^2\left[1 + 3\left(\frac{20.0 \text{ in.}}{17.7 \text{ in.}}\right)\left(\frac{0.300 \text{ in.}}{0.425 \text{ in.}}\right)^{1.5}\right]\sqrt{\frac{(29{,}000 \text{ ksi})(50 \text{ ksi})(0.425 \text{ in.})}{(0.300 \text{ in.})}}$$

$$= 311 \text{ kips}$$

LRFD	ASD
$\phi = 0.75$ $\phi R_n = 0.75(311 \text{ kips}) = 233 \text{ kips}$ 233 kips > 106 kips **o.k.**	$\Omega = 2.00$ $\dfrac{R_n}{\Omega} = \dfrac{311 \text{ kips}}{2.00} = 156 \text{ kips}$ 156 kips > 70.3 kips **o.k.**