Note that the signs on the various forces and equations are important.

Calculating the forces on Section a-a—the gusset-to-beam interface (using equations based on Figure 4-6):

LRFD	ASD
Axial $N_u = V_{u1} + V_{u2}$ $= -204 \text{ kips} + 204 \text{ kips}$ $= 0 \text{ kips}$	Axial $N_a = V_{a1} + V_{a2}$ $= -136 \text{ kips} + 136 \text{ kips}$ $= 0 \text{ kips}$
Shear $V_u = H_{u1} - H_{u2}$ $= -204 \text{ kips} - 204 \text{ kips}$ $= -408 \text{ kips}$	Shear $V_a = H_{a1} - H_{a2}$ $= -136 \text{ kips} - 136 \text{ kips}$ $= -272 \text{ kips}$
Moment $M_u = M_{u1} - M_{u2}$ $= -2{,}780 \text{ kip-in.} - 2{,}780 \text{ kip-in.}$ $= -5{,}560 \text{ kip-in.}$	Moment $M_a = M_{a1} - M_{a2}$ $= -1{,}860 \text{ kip-in.} - 1{,}860 \text{ kip-in.}$ $= -3{,}720 \text{ kip-in.}$

The negative signs on the shear and moment indicate that these forces act opposite to the positive directions assumed in Figure 4-6. An important location in the gusset is Section b-b shown in Figure 4-7. The forces on this section are:

LRFD	ASD
Axial $N'_u = ½(H_{u1} + H_{u2})$ $= ½(-204 \text{ kips} + 204 \text{ kips})$ $= 0 \text{ kips}$	Axial $N'_a = ½(H_{a1} + H_{a2})$ $= ½(-136 \text{ kips} + 136 \text{ kips})$ $= 0 \text{ kips}$
Shear $V'_u = ½(V_{u1} - V_{u2}) - \dfrac{2M_u}{L}$ $= ½(-204 \text{ kips} - 204 \text{ kips}) - \dfrac{2(-5{,}560 \text{ kip-in.})}{64.0 \text{ in.}}$ $= -30.3 \text{ kips}$	Shear $V'_a = ½(V_{a1} - V_{a2}) - \dfrac{2M_a}{L}$ $= ½(-136 \text{ kips} - 136 \text{ kips}) - \dfrac{2(-3{,}720 \text{ kip-in.})}{64.0 \text{ in.}}$ $= -19.8 \text{ kips}$
Moment $M'_u = M'_{u1} + M'_{u2}$ $= 676 \text{ kip-in.} + (-676 \text{ kip-in.})$ $= 0$	Moment $M'_a = M'_{a1} + M'_{a2}$ $= 454 \text{ kip-in.} + (-454 \text{ kip-in.})$ $= 0$

These forces are shown in Figure 5-20 acting on the left half of the gusset. V_u' is acting opposite to the direction shown, as indicated by the minus sign. With all interface (Section a-a) and internal (Section b-b) forces known, the connection can now be designed.

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Brace-to-Gusset Connection

This part of the connection should be designed first because it will give a minimum required size of the gusset plate.

Check tension yielding on the brace

The available tensile yielding strength is determined from AISC *Specification* Section J4.1(a), Equation J4-1:

LRFD	ASD
$\phi R_n = \phi F_y A_g$ $= 0.90(46\text{ ksi})(13.5\text{ in.}^2)$ $= 559\text{ kips} > 289\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = \dfrac{F_y A_g}{\Omega}$ $= \dfrac{(46\text{ ksi})(13.5\text{ in.}^2)}{1.67}$ $= 372\text{ kips} > 193\text{ kips}$ **o.k.**

Check tensile rupture on the brace

From AISC *Specification* Equation J4-4, determine the minimum weld length required to provide adequate available shear rupture strength of the brace material. Choose a connection length between the brace and the gusset plate based on the limit state of shear rupture in the brace wall:

LRFD	ASD
$P_u = 0.60\left[\phi F_u tl(4)\right]$ $289\text{ kips} = 0.60(0.75)(58\text{ ksi})(0.465\text{ in.})l(4)$ Therefore, $l = 5.95$ in.	$P_a = 0.60\left[\dfrac{F_u}{\Omega} tl(4)\right]$ $193\text{ kips} = 0.60\left(\dfrac{58\text{ ksi}}{2.00}\right)(0.465\text{ in.})l(4)$ Therefore, $l = 5.96$ in.

If a 5⁄16-in. fillet weld is assumed with four lines of welds, AISC *Specification* Equation J2-4 and Table J2.5 give:

LRFD	ASD
$\phi P_n = \phi 0.60 F_{EXX}\left(\dfrac{1}{\sqrt{2}}\right)w(4l) \ge P_u$ $l \ge \dfrac{P_u}{\phi 0.60 F_{EXX}\left(\dfrac{1}{\sqrt{2}}\right)w(4)}$ $\ge \dfrac{289\text{ kips}}{0.75(0.60)(70\text{ ksi})\left(\dfrac{1}{\sqrt{2}}\right)(5/16\text{ in.})(4)}$ ≥ 10.4 in.	$\dfrac{P_n}{\Omega} = \dfrac{0.60 F_{EXX}\left(\dfrac{1}{\sqrt{2}}\right)w(4l)}{\Omega} \ge P_a$ $l \ge \dfrac{\Omega P_a}{0.60 F_{EXX}\left(\dfrac{1}{\sqrt{2}}\right)w(4)}$ $\ge \dfrac{2.00(193\text{ kips})}{0.60(70\text{ ksi})\left(\dfrac{1}{\sqrt{2}}\right)(5/16\text{ in.})(4)}$ ≥ 10.4 in.

Note that in this application, the actual weld size used would be 5⁄16 in. because of the slot gap; that is, with ¼-in. fillet welds called out on the drawing, the fabricator would increase the weld size to 5⁄16 in. in order to compensate for the gap between the brace and the gusset (see AWS D1.1 clause 5.22.1). This is a single pass fillet whereas the specified 5⁄16-in. fillet weld will be a multi-pass 3⁄8-in. fillet weld.

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From the AISC *Manual* Part 8, Equations 8-2a and 8-2b, this calculation could more simply be written:

LRFD	ASD
$\phi P_n = 1.392Dl$ $289\text{ kips} = 1.392(5)(4l)$ $l = 10.4\text{ in.}$	$\dfrac{P_n}{\Omega} = 0.928Dl$ $193\text{ kips} = 0.928(5)(4l)$ $l = 10.4\text{ in.}$

Use a 12-in.-long 5⁄16-in. fillet weld. With l = 12.0 in., determine the shear lag factor, U.

The symbol, d_{slot}, used in the following is the width of the slot in the HSS brace, which is taken as the gusset thickness plus 1⁄16 in. on either side. A ¾-in.-thick gusset plate is assumed and later checked.

$$
\begin{aligned}
A_g &= 13.5\text{ in.}^2 \\
A_n &= A_g - 2td_{slot} \quad (d_{slot} = \textit{width of slot}) \\
&= 13.5\text{ in.}^2 - 2(0.465\text{ in.})(\tfrac{3}{4}\text{ in.} + \tfrac{1}{16}\text{ in.} + \tfrac{1}{16}\text{ in.}) \\
&= 12.7\text{ in.}^2
\end{aligned}
$$

From AISC *Specification* Table D3.1, Case 6 with a single concentric gusset plate:

$$
\begin{aligned}
\bar{x} &= \frac{B^2 + 2BH}{4(B+H)} \qquad (B = H = 8.00\text{ in.}) \\
&= \frac{(8.00\text{ in.})^2 + 2(8.00\text{ in.})(8.00\text{ in.})}{4(8.00\text{ in.} + 8.00\text{ in.})} \\
&= 3.00\text{ in.}
\end{aligned}
$$

$$
\begin{aligned}
U &= 1 - \left(\frac{\bar{x}}{l}\right) \\
&= 1 - \left(\frac{3.00\text{ in.}}{12.0\text{ in.}}\right) \\
&= 0.750
\end{aligned}
$$

From AISC *Specification* Equation D3-1:

$$
\begin{aligned}
A_e &= A_n U && (\textit{Spec}.\text{ Eq. D3-1}) \\
&= (12.7\text{ in.}^2)(0.750) \\
&= 9.53\text{ in.}^2
\end{aligned}
$$

From AISC *Specification* Section J4.1(b), the available tensile rupture strength of the brace is:

LRFD	ASD
$\phi R_n = \phi F_u A_e$ $= 0.75(58.0\text{ ksi})(9.53\text{ in.}^2)$ $= 415\text{ kips} > 289\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = \dfrac{F_u A_e}{2.00}$ $= \dfrac{(58.0\text{ ksi})(9.53\text{ in.}^2)}{2.00}$ $= 276\text{ kips} > 193\text{ kips}$ **o.k.**

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Check block shear rupture on the gusset plate

Assume a gusset plate thickness of ¾ in. The available strength for the limit state of block shear rupture is given in AISC *Specification* Section J4.3 as follows:

$$R_n = 0.60F_u A_{nv} + U_{bs}F_u A_{nt} \leq 0.60F_y A_{gv} + U_{bs}F_u A_{nt} \qquad (Spec.\ \text{Eq. J4-5})$$

Shear yielding component:

$$\begin{aligned} A_{gv} &= A_{nv} \\ &= 2t_g l \\ &= (2)(\tfrac{3}{4}\text{ in.})(12.0\text{ in.}) \\ &= 18.0\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} 0.60F_y A_{gv} &= 0.60(50\text{ ksi})(18.0\text{ in.}^2) \\ &= 540\text{ kips} \end{aligned}$$

Shear rupture component:

$$\begin{aligned} 0.60F_u A_{nv} &= 0.60(65\text{ ksi})(18.0\text{ in.}^2) \\ &= 702\text{ kips} \end{aligned}$$

Tension rupture component:

$U_{bs} = 1$ from AISC *Specification* Section J4.3 because the bolts are uniformly loaded

$$\begin{aligned} A_{gt} &= A_{nt} \\ &= t_g B \\ &= (\tfrac{3}{4}\text{ in.})(8.00\text{ in.}) \\ &= 6.00\text{ in.}^2 \end{aligned}$$

$$\begin{aligned} U_{bs}F_u A_{nt} &= 1(65\text{ ksi})(6.00\text{ in.}^2) \\ &= 390\text{ kips} \end{aligned}$$

The available strength for the limit state of block shear rupture is determined as follows:

$$\begin{aligned} 0.60F_u A_{nv} + U_{bs}F_u A_{nt} &= 702\text{ kips} + 390\text{ kips} \\ &= 1{,}090\text{ kips} \end{aligned}$$

$$\begin{aligned} 0.60F_y A_{gv} + U_{bs}F_u A_{nt} &= 540\text{ kips} + 390\text{ kips} \\ &= 930\text{ kips} \end{aligned}$$

Therefore, $R_n = 930$ kips.

LRFD	ASD
$\phi R_n = 0.75(930\text{ kips})$ $= 698\text{ kips} > 289\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = \dfrac{930\text{ kips}}{2.00}$ $= 465\text{ kips} > 193\text{ kips}$ **o.k.**

If the braces had different loads, block shear would need to be checked for each brace.

Check the gusset plate for tensile yielding on the Whitmore section

From AISC *Manual* Part 9, the width of the Whitmore section is:

$$\begin{aligned} l_w &= B + 2l\tan 30^\circ \\ &= 8.00\text{ in.} + 2(12.0\text{ in.})\tan 30^\circ \\ &= 21.9\text{ in.} \end{aligned}$$

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It can be seen from Figure 5-20 that about 2.00 in. of the Whitmore section is in the beam web. The web thickness, $t_w = 0.570$ in. < $t_g = ¾$ in.; therefore, the portion of the Whitmore area in the web is accounted for.

$$A_w = (21.9 \text{ in.} - 2.00 \text{ in.})(¾ \text{ in.}) + (2.00 \text{ in.})(0.570 \text{ in.})$$
$$= 16.1 \text{ in.}^2$$

From AISC *Specification* Section J4.1(a):

LRFD	ASD
$\phi R_n = 0.90 F_y A_w$ $= 0.90(50 \text{ ksi})(16.1 \text{ in.}^2)$ $= 725 \text{ kips} > 289 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = \dfrac{F_y A_w}{1.67}$ $= \dfrac{(50 \text{ ksi})(16.1 \text{ in.}^2)}{1.67}$ $= 482 \text{ kips} > 193 \text{ kips}$ **o.k.**

Check the gusset plate for buckling on the Whitmore section

The available compressive strength of the gusset plate based on the limit state of flexural buckling is determined from AISC *Specification* Section J4.4, using an effective length factor, K, of 0.65 according to Dowswell (2006, 2009, 2012). As shown in Figure 5-20, the unbraced length of the gusset plate is 8.00 in.

$$\frac{KL}{r} = \frac{0.65(8.00 \text{ in.})}{\left(\dfrac{¾ \text{ in.}}{\sqrt{12}}\right)}$$
$$= 24.0$$

Because $\dfrac{KL}{r} < 25.0$, according to AISC *Specification* Section J4.4, the strength in compression is the same as that for tension yielding, which was previously checked.

Gusset-to-Beam Connection (Section a-a)

The required strengths are:

LRFD	ASD
Required shear strength, $V_u = 408$ kips Required axial strength, $N_u = 0$ kips Required flexural strength, $M_u = 5{,}560$ kip-in.	Required shear strength, $V_a = 272$ kips Required axial strength, $N_a = 0$ kips Required flexural strength, $M_a = 3{,}720$ kip-in.

Check the gusset plate for tensile yielding and shear yielding along the beam flange

In this example, to allow combining the required normal strength and flexural strength, stresses will be used to check shear yielding and tensile yielding limit states on the gusset plate. Shear yielding is checked using AISC *Specification* Section J4.2 and tensile yielding is checked using Section J4.1:

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LRFD	ASD
Shear yielding stress: $f_{uv} = \frac{V_u}{A_g}$ $= \frac{408 \text{ kips}}{(3/4 \text{ in.})(64.0 \text{ in.})}$ $= 8.50 \text{ ksi} < 1.00(0.60)(50 \text{ ksi}) = 30 \text{ ksi}$ **o.k.**	Shear yielding stress: $f_{av} = \frac{V_a}{A_g}$ $= \frac{272 \text{ kips}}{(3/4 \text{ in.})(64.0 \text{ in.})}$ $= 5.67 \text{ ksi} < \frac{0.60(50 \text{ ksi})}{1.50} = 20 \text{ ksi}$ **o.k.**
Tensile yielding stress: $f_{ua} = \frac{N_u}{A_g}$ $= \frac{0 \text{ kips}}{(3/4 \text{ in.})(64.0 \text{ in.})}$ $= 0 \text{ ksi}$ $f_{ub} = \frac{M_u}{Z}$ $= \frac{5{,}560 \text{ kip-in.}}{(3/4 \text{ in.})(64.0 \text{ in.})^2/4}$ $= 7.24 \text{ ksi}$	Tensile yielding stress: $f_{aa} = \frac{N_a}{A_g}$ $= \frac{0 \text{ kips}}{(3/4 \text{ in.})(64.0 \text{ in.})}$ $= 0 \text{ ksi}$ $f_{ab} = \frac{M_a}{Z}$ $= \frac{3{,}720 \text{ kip-in.}}{(3/4 \text{ in.})(64.0 \text{ in.})^2/4}$ $= 4.84 \text{ ksi}$
The total normal stress $= f_{un}$ $f_{un} = f_{ua} + f_{ub}$ $= 0 \text{ ksi} + 7.24 \text{ ksi}$ $= 7.24 \text{ ksi} < 0.90(50 \text{ ksi}) = 45.0 \text{ ksi}$ **o.k.**	The total normal stress $= f_{an}$ $f_{an} = f_{aa} + f_{ab}$ $= 0 \text{ ksi} + 4.84 \text{ ksi}$ $= 4.84 \text{ ksi} < \frac{50 \text{ ksi}}{1.67} = 29.9 \text{ ksi}$ **o.k.**

Design weld at gusset-to-beam flange connection

The 408-kip (LRFD) or 272-kip (ASD) shear force is to be delivered to the center of the beam. This is a function of the required flexural strength, 5,560 kip-in (LRFD) or 3,720 kip-in. (ASD). The effective eccentricity of the shear force is:

LRFD	ASD
$e = \frac{M_u}{V_u}$ $= \frac{5{,}560 \text{ kip-in.}}{408 \text{ kips}}$ $= 13.6 \text{ in.}$	$e = \frac{M_a}{V_a}$ $= \frac{3{,}720 \text{ kip-in.}}{272 \text{ kips}}$ $= 13.7 \text{ in.}$

Therefore, from AISC *Manual* Table 8-4 for Angle = 0, $k = 0$, $a = 13.6 \text{ in.}/64.0 \text{ in.} = 0.213$, by interpolation $C = 3.46$. With $C_1 =$ 1.0, the number of sixteenths of weld required with a ductility factor of 1.25 (as discussed in AISC *Manual* Part 13) is determined from AISC *Manual* Equation 8-13:

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LRFD	ASD
$D_{req'd} = \dfrac{P_u}{\phi C C_1 l}$ $= \dfrac{(408 \text{ kips})(1.25)}{(0.75)(3.46)(1.0)(64.0 \text{ in.})}$ $= 3.07$	$D_{req'd} = \dfrac{\Omega P_a}{C C_1 l}$ $= \dfrac{2.00(272 \text{ kips})(1.25)}{(3.46)(1.0)(64.0 \text{ in.})}$ $= 3.07$

Use a ¼-in. double-sided fillet weld.

An alternative method to determine the required weld size that does not require the use of an AISC *Manual* table follows. See Appendix B for a discussion of the method. This method is similar to the method of Hewitt and Thornton (2004) except that it allows the use of the increased strength of fillet welds loaded at an angle relative to the weld longitudinal axis.

The required shear strength is $V_u = 408$ kips (LRFD) or $V_a = 272$ kips (ASD). The normal force and the moment can be combined to generate a maximum equivalent normal force (see Appendix B, Figure B-1):

LRFD	ASD
$N_{max} = \lvert N_u \rvert + \left\lvert \dfrac{2M_u}{L/2} \right\rvert$ $= \lvert N_u \rvert + \left\lvert \dfrac{4M_u}{L} \right\rvert$ $= 0 \text{ kips} + \left\lvert \dfrac{4(-5{,}560 \text{ kip-in.})}{64 \text{ in.}} \right\rvert$ $= 348 \text{ kips}$ and a minimum equivalent normal force: $N_{min} = \left\lvert \lvert N_u \rvert - \left\lvert \dfrac{4M_u}{L} \right\rvert \right\rvert$ $= \left\lvert \lvert 0 \text{ kips} \rvert - \left\lvert \dfrac{4(-5{,}560 \text{ kip-in.})}{64 \text{ in.}} \right\rvert \right\rvert$ $= 348 \text{ kips}$ The peak weld resultant force: $R_{peak} = \sqrt{V_u^2 + N_{max}^2}$ $= \sqrt{(408 \text{ kips})^2 + (348 \text{ kips})^2}$ $= 536 \text{ kips}$ The average weld resultant force: $R_{avg} = \sqrt{V_u^2 + \left(\dfrac{N_{max} + N_{min}}{2}\right)^2}$ $= \sqrt{(408 \text{ kips})^2 + \left(\dfrac{348 \text{ kips} + 348 \text{ kips}}{2}\right)^2}$ $= 536 \text{ kips}$	$N_{max} = \lvert N_a \rvert + \left\lvert \dfrac{2M_a}{L/2} \right\rvert$ $= \lvert N_a \rvert + \left\lvert \dfrac{4M_a}{L} \right\rvert$ $= 0 \text{ kips} + \left\lvert \dfrac{4(-3{,}720 \text{ kip-in.})}{64.0 \text{ in.}} \right\rvert$ $= 233 \text{ kips}$ and a minimum equivalent normal force: $N_{min} = \left\lvert \lvert N_a \rvert - \left\lvert \dfrac{4M_a}{L} \right\rvert \right\rvert$ $= \left\lvert \lvert 0 \text{ kips} \rvert - \left\lvert \dfrac{4(-3{,}720 \text{ kip-in.})}{64.0 \text{ in.}} \right\rvert \right\rvert$ $= 233 \text{ kips}$ The peak weld resultant force: $R_{peak} = \sqrt{V_a^2 + N_{max}^2}$ $= \sqrt{(272 \text{ kips})^2 + (233 \text{ kips})^2}$ $= 358 \text{ kips}$ The average weld resultant force: $R_{avg} = \sqrt{V_a^2 + \left(\dfrac{N_{max} + N_{min}}{2}\right)^2}$ $= \sqrt{(272 \text{ kips})^2 + \left(\dfrac{233 \text{ kips} + 233 \text{ kips}}{2}\right)^2}$ $= 358 \text{ kips}$

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Because $1.25R_{avg} > R_{peak}$, use $1.25R_{avg}$ to size the weld. From AISC *Manual* Equation 8-2, including the directional strength increase of the fillet weld provided in AISC *Specification* Equation J2-4:

LRFD	ASD
$\theta = \tan^{-1}\left(\dfrac{N_{max}}{V_u}\right)$ $= \tan^{-1}\left(\dfrac{348 \text{ kips}}{408 \text{ kips}}\right)$ $= 40.5°$	$\theta = \tan^{-1}\left(\dfrac{N_{max}}{V_a}\right)$ $= \tan^{-1}\left(\dfrac{233 \text{ kips}}{272 \text{ kips}}\right)$ $= 40.6°$

LRFD	ASD
$D_{req'd} = \dfrac{1.25R_{avg}}{1.392\left(1.0+0.50\sin^{1.5}\theta\right)(2L)}$ $= \dfrac{1.25(536 \text{ kips})}{1.392\left(1.0+0.50\sin^{1.5} 40.5°\right)\left[2(64.0 \text{ in.})\right]}$ $= 2.98$ % difference between methods $\left[\dfrac{(2.98-3.07)}{3.07}\right]\times 100\% = 2.93\%$	$D_{req'd} = \dfrac{1.25R_{avg}}{0.928\left(1.0+0.50\sin^{1.5}\theta\right)(2L)}$ $= \dfrac{1.25(358 \text{ kips})}{0.928\left(1.0+0.50\sin^{1.5} 40.6°\right)\left[2(64.0 \text{ in.})\right]}$ $= 2.98$ % difference between methods $\left[\dfrac{(2.98-3.07)}{3.07}\right]\times 100\% = 2.93\%$

Note that the directional strength increase of $1.0 + 0.50 \sin^{1.5} 40.5° = 1.26$ indicates a 26% increase over the strength of a longitudinally loaded fillet weld.

This alternative method produces a result that is 2 to 3% smaller than the result of using the AISC *Manual* Table 8-4 method, which is essentially the same answer. The required weld size is ¼ in., which is the AISC minimum weld size from AISC *Specification* Table J2-4. A discussion on the alternative method is given in Appendix B.

Check gusset internal strength (Section b-b)

As mentioned previously, the prime symbol after the force quantity indicates that it acts on Section b-b through the gusset plate. The control section for this chevron gusset is the horizontal Section a-a. Section b-b, which is an internal gusset section, plus the beam cross section above the gusset Section b-b, must be able to transfer the brace vertical components, V_1 and V_2, across this section. The forces H', N' and M' are a direct result of the forces H, N and M assumed on the control Section a-a. See Figures 4-5, 4-6 and 4-7.

The required strengths are:

LRFD	ASD
Required shear strength, $V'_u = -30.3$ kips Required axial strength, $N'_u = 0$ kips Required flexural strength, $M'_u = 0$ kip-in.	Required shear strength, $V'_a = -19.8$ kips Required axial strength, $N'_a = 0$ kips Required flexural strength, $M'_a = 0$ kip-in.

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Check the gusset plate for shear yielding at Section b-b

From AISC *Specification* Section J4.2(a), the available shear yielding strength of the gusset plate is:

LRFD	ASD
$\phi R_n = \phi 0.60 F_y A_{gv}$ $= 1.00(0.60)(50\text{ ksi})(\tfrac{3}{4}\text{ in.})(18.0\text{ in.})$ $= 405\text{ kips} > 30.3\text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = \dfrac{0.60 F_y A_{gv}}{\Omega}$ $= \dfrac{0.60(50\text{ ksi})(\tfrac{3}{4}\text{ in.})(18.0\text{ in.})}{1.50}$ $= 270\text{ kips} > 19.8\text{ kips}$ **o.k.**

Because N' and M' are zero, there are no further checks.

If N' and M' were not zero, it is possible for a compressive stress to exist on the gusset free edge at Section b-b. In this case, the gusset should be checked for buckling under this stress. The procedure given in the AISC *Manual* on pages 9-8 and 9-9 for buckling of double-coped beams can be used. This *Manual* procedure is developed into a gusset free-edge buckling method in Appendix C, Section C.4, where it is referred to as the admissible force maintenance method (AFMM).

Check gusset stresses on Section b-b for a hypothetical load case

As an example, consider that both braces of Figure 5-19 are in compression simultaneously. Then $P_1 = P_2$, and from Figure 4-5:

LRFD	ASD
$M_{u1} = (-204\text{ kips})(13.65\text{ in.})$ $= -2{,}780\text{ kip-in.}$ $M_{u2} = (-204\text{ kips})(13.65\text{ in.})$ $= -2{,}780\text{ kip-in.}$ $M'_{u1} = \tfrac{1}{8}(-204\text{ kips})(64.0\text{ in.}) - \tfrac{1}{4}(-204\text{ kips})(18.0\text{ in.})$ $- \tfrac{1}{2}(-2{,}780\text{ kip-in.})$ $= 676\text{ kip-in.}$ $M'_{u2} = \tfrac{1}{8}(-204\text{ kips})(64.0\text{ in.}) - \tfrac{1}{4}(-204\text{ kips})(18.0\text{ in.})$ $- \tfrac{1}{2}(-2{,}780\text{ kip-in.})$ $= 676\text{ kip-in.}$	$M_{a1} = (-136\text{ kips})(13.65\text{ in.})$ $= -1{,}860\text{ kip-in.}$ $M_{a2} = (-136\text{ kips})(13.65\text{ in.})$ $= -1{,}860\text{ kip-in.}$ $M'_{a1} = \tfrac{1}{8}(-136\text{ kips})(64.0\text{ in.})$ $- \tfrac{1}{4}(-136\text{ kips})(18.0\text{ in.}) - \tfrac{1}{2}(-1{,}860\text{ kip-in.})$ $= 454\text{ kip-in.}$ $M'_{a2} = \tfrac{1}{8}(-136\text{ kips})(64.0\text{ in.})$ $- \tfrac{1}{4}(-136\text{ kips})(18.0\text{ in.}) - \tfrac{1}{2}(-1{,}860\text{ kip-in.})$ $= 454\text{ kip-in.}$

From Figure 4-6, the forces on Section a-a are:

LRFD	ASD
$N_u = -204\text{ kips} + (-204\text{ kips})$ $= -408\text{ kips}$ $V_u = -204\text{ kips} - (-204\text{ kips})$ $= 0\text{ kips}$ $M_u = -2{,}780\text{ kip-in.} - (-2{,}780\text{ kip-in.})$ $= 0\text{ kip-in.}$	$N_a = -136\text{ kips} + (-136\text{ kips})$ $= -272\text{ kips}$ $V_a = -136\text{ kips} - (-136\text{ kips})$ $= 0\text{ kips}$ $M_a = -1{,}860\text{ kip-in.} - (-1{,}860\text{ kip-in.})$ $= 0\text{ kip-in.}$

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And those on Section b-b from Figure 4-7 are:

LRFD	ASD
$N' = \frac{1}{2}\left[-204 \text{ kips} + (-204 \text{ kips})\right]$ $= -204 \text{ kips}$ $V' = \frac{1}{2}\left[-204 \text{ kips} - (-204 \text{ kips})\right] - \frac{2}{64 \text{ in.}}(0)$ $= 0 \text{ kips}$ $M' = 676 \text{ kip-in.} + 676 \text{ kip-in.}$ $= 1{,}350 \text{ kip-in.}$	$N' = \frac{1}{2}\left[-136 \text{ kips} + (-136 \text{ kips})\right]$ $= -136 \text{ kips}$ $V' = \frac{1}{2}\left[-136 \text{ kips} - (-136 \text{ kips})\right] - \frac{2}{64 \text{ in.}}(0)$ $= 0 \text{ kips}$ $M' = 454 \text{ kip-in.} + 454 \text{ kip-in.}$ $= 908 \text{ kip-in.}$

The entire Section b-b is under a uniform compression of N' and an additional compressive force due to M'. The worst case equivalent normal force (compression) can be calculated (see Appendix B, Figure B-1) as:

LRFD	ASD
$N_{ue} = 204 \text{ kips} + \left(\frac{1{,}350 \text{ kip-in.}}{9.00 \text{ in.}}\right)(2)$ $= 504 \text{ kips}$	$N_{ae} = 136 \text{ kips} + \left(\frac{908 \text{ kip-in.}}{9.00 \text{ in.}}\right)(2)$ $= 338 \text{ kips}$

Note that this equivalent normal force will produce the correct gusset maximum normal stress but is conservative for buckling because the compressive portion of the stress due to M' acts over the upper half of gusset Section b-b, because M' is greater than zero. The dimensions a and b, scaled or calculated from Figure 5-19, are $a = 52$ in. and $b = 18$ in. From Appendix C, Section C.4, λ and Q can be determined as follows:

$$\lambda = \frac{(b/t_w)\sqrt{F_y}}{5\sqrt{475 + \dfrac{1{,}120}{(a/b)^2}}} \tag{C-7}$$

$$= \frac{(18.0 \text{ in.}/\tfrac{3}{4} \text{ in.})\sqrt{50 \text{ ksi}}}{5\sqrt{475 + \dfrac{1{,}120}{(52.0 \text{ in.}/18.0 \text{ in.})^2}}}$$

$$= 1.38$$

Since $0.70 < \lambda \le 1.41$:

$$Q = 1.34 - 0.486\lambda$$
$$= 1.34 - 0.486(1.38)$$
$$= 0.670$$

LRFD	ASD
$\phi F_{cr} = 0.90(0.670)(50 \text{ ksi})$ $= 30.2 \text{ ksi}$	$\frac{F_{cr}}{\Omega} = \frac{(0.670)(50 \text{ ksi})}{1.67}$ $= 20.1 \text{ ksi}$

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