The Chevron Effect—Not an Isolated Problem

PATRICK J. FORTNEY and WILLIAM A. THORNTON

ABSTRACT

Vertical braces that connect concentrically to frame beams away from the beam-column joint are referred to as V-type or inverted V-type braced frames, as chevron braced frames or as mid-span braces. The braces are commonly connected to the frame beam using gusset plates. Typically, these gusseted connections are analyzed and designed considering only the effect of the brace forces on the region of the beam within the connection region. This is a reasonable approach when the summation of the vertical components of the brace forces is zero. However, when the vertical components result in a non-zero net vertical force (also referred to as an unbalanced force), analyzing and designing the connection as if it were isolated from the frame may result in a significantly undersized beam, requiring expensive beam web and flange reinforcement. In this paper, the effect of the brace forces on the beam in this type of braced frame configuration is referred to as the *chevron effect*. This paper presents a method for determining the distribution of brace forces within the connection and also the impact of the brace force distribution on the frame beam. The mechanism analysis required by the 2010 AISC *Seismic Provisions for Structural Steel Buildings*, AISC 341-10, is presented, and the discussion illustrates the importance of considering the entire frame when evaluating the impact of the brace forces on the beam.

Keywords: Gusset plates, chevron braces, V-braces, brace forces, analysis, design.

When vertical braces connect concentrically to frame beams away from the beam-column joint, these concentrically configured braces are referred to as V-type or inverted V-type braced frames. It is also common to refer to these types of braced frames as chevron braced frames or mid-span braces. The braces are commonly connected to the frame beam using gusset plates. Typically, these gusseted connections are analyzed and designed considering only the effect of the brace forces on the portion of the beam within the connection region. This is called designing the connection in isolation and is a reasonable approach when the summation of the vertical components of the brace forces is zero. However, when the vertical components result in a non-zero net force, the connection should not be analyzed and designed as if it were isolated from the frame. The beam span and the location of the work point along the span of the beam must be considered in order to fully understand the impact of the brace forces on the frame beam. In this paper, the effect of the brace forces on the beam in this type of braced frame consideration is referred to as the *chevron effect*. This paper presents a method for determining the distribution of brace forces within the connection and also the impact of the brace force distribution on the frame beam.

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To illustrate the chevron effect, the mechanism analysis required by AISC 341-10, *Seismic Provisions for Structural Steel Buildings* (AISC, 2010a) is presented. The discussion illustrates the importance of considering the entire frame when evaluating the impact of the brace forces on the beam and the potentially unconservative results when evaluating the connection as if it were isolated from the frame.

Concentric braced frame structures can be set up in various configurations. Braces can frame to beam-column joints, to various locations along the height of the frame column and to various locations along the span of the frame beam. The discussion presented in this paper focuses on a concentric brace configuration referred to in AISC 341-10 as V-type or inverted V-type configurations, also known as chevron braces or mid-span braces. In the V-type configuration, two braces connect to the top side of the frame beam somewhere along the clear span of the frame beam away from the beam-column joint. In the inverted V-type configuration, two braces connect to the bottom side of the frame beam somewhere along the clear span of the frame beam away from the beam-column joint. In some cases, the configuration is such that the braces form a two-story X-brace in a manner where both V-type and inverted V-type braces connect to the intermediate frame beam level. Figure 1 shows these three types of chevron configurations.

There are two common types of gusseted connections used in the types of brace configurations shown in Figure 1. A combined gusset, which is one plate that is used to connect both braces to the beam, or, when geometry permits, a single gusset can be used to connect each brace to the beam individually. Figures 2a and 2b show these two common types of gusset connections. The discussion presented in this paper focuses on combined gussets. However, it is important to recognize that the issues addressed in this paper apply equally to chevron braces connected with single gussets. The only significant difference between the two types of gussets is how the forces acting at the gusset-to-beam interface are calculated.

Unlike connection design for braces that frame to a beam-column joint, where the Uniform Force Method (UFM) is typically used to distribute brace forces through the connection, the force distribution in a chevron brace connection can be determined using any type of distribution that satisfies static equilibrium. Part 13 of the 14th edition AISC *Steel Construction Manual* (2011b) provides comprehensive guidance how to distribute forces in brace connections for braces that frame to a beam-column joint. However, there is very little published work on how to distribute brace forces in other types of brace configurations, such as V-type and inverted V-type brace configurations (see example problem II.C-5 of the AISC *Design Examples* Manual, v.14.1). A method for doing so is presented in this paper. The impact of the force distribution on the frame beam must also be considered. A thorough treatment on this topic is also presented.

In chevron brace connections, the algebraic sum of the vertical components of the brace forces can have a significant impact on the shear and moment distribution in the frame beam. When the sum of the vertical components of the brace forces is non-zero, the beam shear and moment distribution along the span of the beam are highly dependent on the span of the beam as well as the location of the work point along the span of the beam. Furthermore, the maximum beam shear and moment can be potentially underestimated or overestimated if the impact of the brace forces is evaluated as if the connection is isolated from the frame. Maximum beam shear and moment may also be located outside of the connection region of the beam. This impact on the beam is referred to in this paper as the chevron effect, and will be discussed in detail.

There are various reasons why the summation of the vertical components of the brace forces is non-zero. The most common reason involves mechanism analysis as required in seismic braced frame analysis and design. It is also possible to have a non-zero vertical component summation when braces are permitted to resist gravity loads simultaneous with a lateral load analysis.

This paper presents the following:

1. A procedure for determining an admissible force distribution within the connection.
2. The current typical method for determining beam shear.

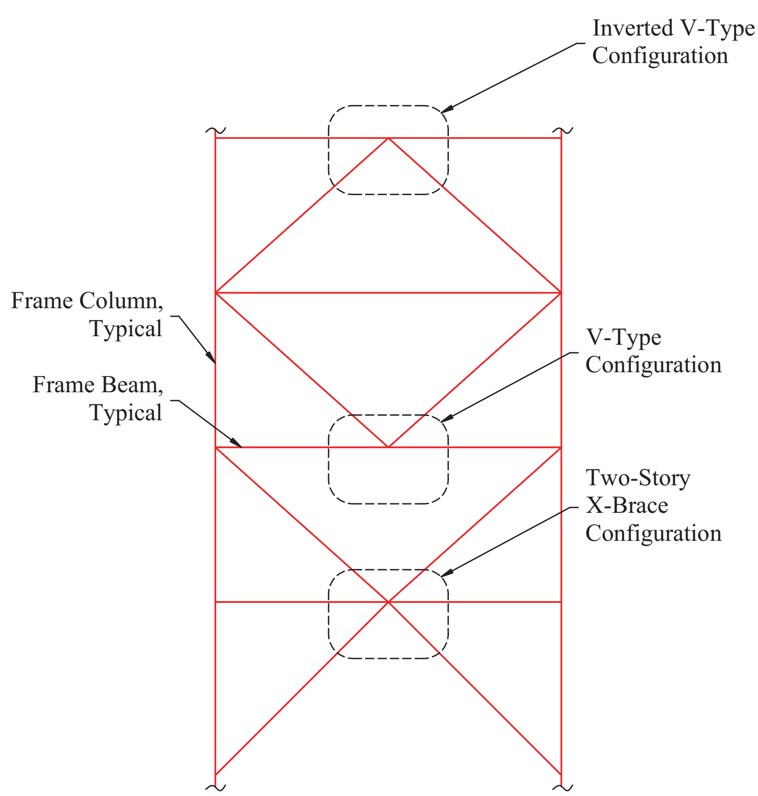


Fig. 1. V-type, inverted V-type and two-story X-braced frame configurations.

3. Distribution of forces acting on the beam-to-gusset interface.
4. Beam shear and moment distribution.
   a. The effect of the span of the beam.
   b. The effect of the location of the work point along the span of the beam.
5. The chevron effect.
6. A rule of thumb for estimating the moment acting at the gusset-to-beam interface.
7. Actual design problem.

Example problems to support the discussion are provided throughout the paper. It's worth noting that in this type of work, the calculated values part of the solutions are typically shown using three significant figures. However, in order to have beam shear and moment diagrams close nicely, the authors have chosen to present values with higher number of significant figures than would typically be presented.

1. AN ADMISSIBLE CHEVRON BRACE FORCE DISTRIBUTION

When generating an admissible force distribution in the connection, a control section must be selected. The method presented in this paper assumes a horizontal control section that is taken at the edge of the gusset that interfaces with the beam.

Horizontal Control Section

This method first evaluates the forces acting on the horizontal edge of the gusset adjacent to the frame beam. This section is referred to as section *a-a*. See Figure 3a for geometry and parameters used. Once the forces acting on section *a-a* are determined, a vertical section located at one-half of the gusset length, L_g, is cut. This section is referred to as section *b-b* (see Figures 3d and 3e). Each half of the gusset is evaluated. For each half-gusset body, the forces acting on the horizontal edge are taken as one-half of the forces acting on section *a-a*. The moment acting on section *a-a* is applied to the horizontal edge of the body as a couple and is taken as $2M_{a\text{-}a}/L_g$, as shown in Figures 3b and 3c. Figures 3d and 3e show the free body diagrams of each the half-gusset bodies.

Note that the analysis considers brace bevels, brace forces and the effects of any eccentricities that may exist in both the horizontal and vertical directions. The eccentricity, Δ, accounts for variations between brace 1 and brace 2 bevels and the vertical components of the brace forces. The eccentricity resulting from the horizontal components delivered by the gusset to the beam flange is accounted for with the parameter e_b. The sign convention used assumes that a brace force component is positive when acting to the right in the horizontal direction and when acting upward in the vertical direction. A clockwise moment is considered to be positive. It is important to recognize that this is not the only way this analysis can be approached.

The equations derived from statics for the forces and moments acting on sections *a-a* and *b-b* using the approach shown in Figure 3 will be derived in their entirety.

Referring to Figure 3a, the vertical eccentricity parameter, Δ can be written as,

$$\Delta = \frac{1}{2}(L_1 - L_2) \tag{1}$$

Note that Δ is positive when to the left of the work point.

Forces Acting on Section a-a

Referring to Figure 3b, equations for the forces and moment acting on section *a-a* can be written using the three equations of equilibrium. In these equations, the subscripts 1 and 2 refer to the brace forces from the left and right braces,

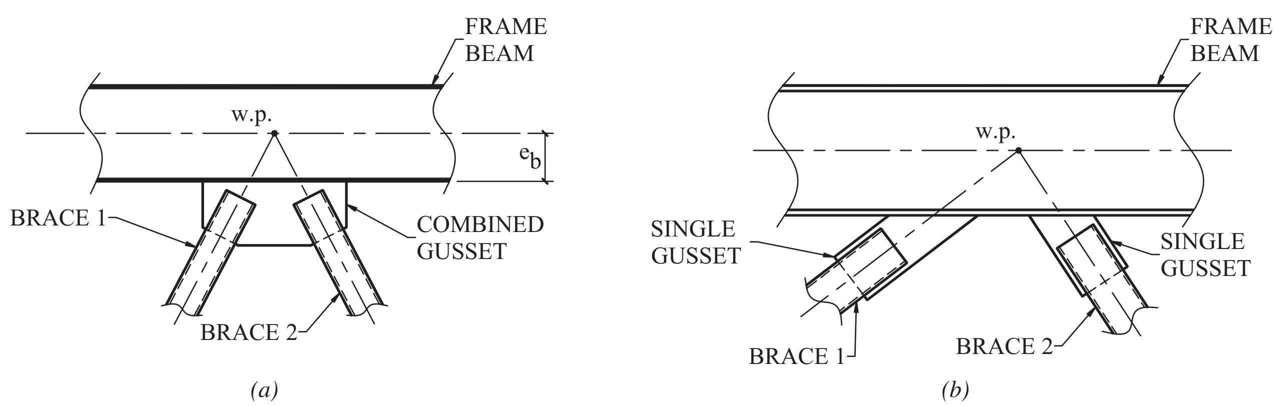


Fig. 2. Representative sketches of combined and single chevron gusset plates: (a) combined gusset; (b) single gusset.

respectively. The subscripts H and V refer to forces acting in the horizontal and vertical directions, respectively. The subscript *w.p.* refers to the work point, and the subscript *a-a* refers to section *a-a* as shown in Figure 3.

$$\begin{aligned}
\sum F_H &= 0 = H_{a-a} + H_1 + H_2 \\
H_{a-a} &= -(H_1 + H_2) \\
\sum F_V &= 0 = V_{a-a} + V_1 + V_2 \\
V_{a-a} &= -(V_1 + V_2) \\
\sum M_{w.p.} &= 0 = -(V_1 + V_2)(\Delta) + (H_1 + H_2)e_b + M_{a-a} \\
M_{a-a} &= (V_1 + V_2)\Delta - (H_1 + H_2)e_b
\end{aligned}$$

In summary, the forces and moments acting on section *a-a* are as given in Equations 2, 3 and 4.

$$H_{a-a} = -(H_1 + H_2) \tag{2}$$

$$V_{a-a} = -(V_1 + V_2) \tag{3}$$

$$M_{a-a} = (V_1 + V_2)\Delta - (H_1 + H_2)e_b \tag{4}$$

Force Acting on Section **b-b** *(left half of the gusset)*

Referring to Figure 3d, equations for the forces and moment acting on section *b-b* at the left half of the gusset can be written using the three equations of equilibrium. As discussed previously, note that the forces acting on the horizontal section of the left half gusset are taken as one-half of the total forces acting on section *a-a*. Also, the moment $M_{a\text{-}a}$ is converted to a couple acting at $L_g/4$ of the gusset on both the left and right halves of the gusset. In the following derivations, the subscript $b1$ refers to forces and moments acting on section *b-b* due to the brace force from brace 1. Refer to Figure 3d.

$$\begin{aligned}
\sum F_H &= 0 \\
&= H_{b1} + H_1 - \frac{1}{2}(H_1 + H_2) \\
H_{b1} &= \frac{1}{2}(H_1 + H_2) - H_1 \\
\sum F_V &= 0 = V_{b1} + V_1 - \frac{1}{2}(V_1 + V_2) + \frac{2M_{a-a}}{L_g} \\
V_{b1} &= \frac{1}{2}(V_1 + V_2) - \frac{2M_{a-a}}{L_g} - V_1 \\
\sum M_b &= 0 \\
&= M_{b1} + H_1\left(e_b + \frac{h}{2}\right) - V_1 \quad + \frac{2M_{a-a}}{L_g}\left(\frac{L_g}{4}\right) \\
&\quad - \frac{1}{2}(V_1 + V_2)\left(\frac{L_g}{4}\right) - \frac{1}{2}(H_1 + H_2)\left(\frac{h}{2}\right)
\end{aligned}$$

$$\begin{aligned}
M_{b1} = {} & \frac{L_g}{8}(V_1 + V_2) + \frac{h}{4}(H_1 + H_2) \\
& - \frac{M_{a-a}}{2} + V_1\Delta - H_1\left(e_b + \frac{h}{2}\right)
\end{aligned}$$

The couple, N_{eq}, of the moment, $M_{a\text{-}a}$, shown in Figures 3d and 3e is given in Equation 5.

$$N_{eq} = \frac{2M_{a-a}}{L_g} \tag{5}$$

In summary, the forces and moment acting on section *b-b* from the perspective of the left half of the gusset are as given in Equations 6, 7 and 8.

$$H_{b1} = \frac{1}{2}(H_1 + H_2) - H_1 \tag{6}$$

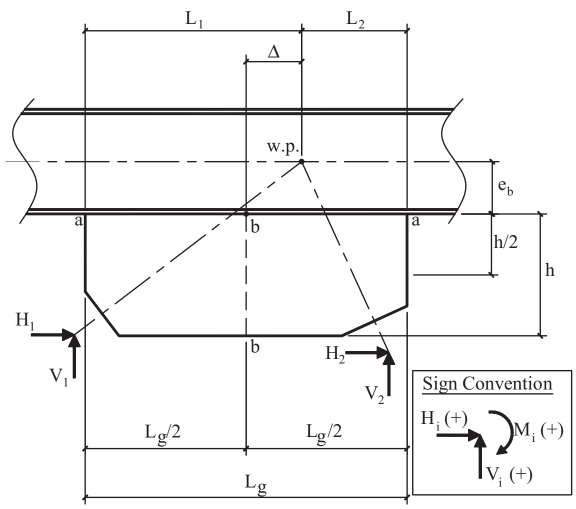
$$V_{b1} = \frac{1}{2}(V_1 + V_2) - \frac{2M_{a-a}}{L_g} - V_1 \tag{7}$$

$$\begin{aligned}
M_{b1} = {} & \frac{L_g}{8}(V_1 + V_2) + \frac{h}{4}(H_1 + H_2) \\
& - \frac{M_{a-a}}{2} + V_1\Delta - H_1\left(e_b + \frac{h}{2}\right)
\end{aligned} \tag{8}$$

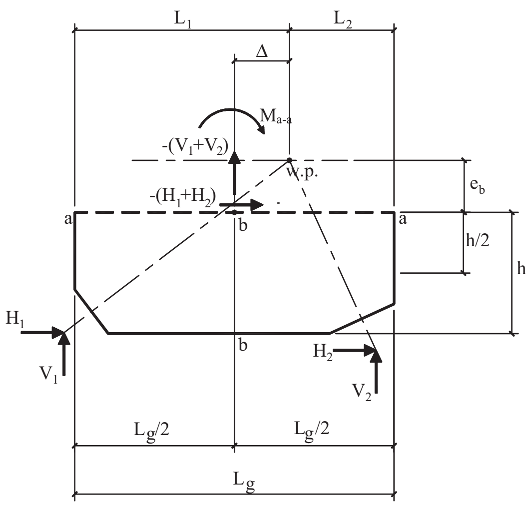
Forces Acting on Section **b-b** *(right half of the gusset)*

Referring to Figure 3e, equations for the forces and moment acting on section *b-b* at the right half of the gusset can be written using the three equations of equilibrium. As discussed previously, note that the forces acting on the horizontal section of the right half gusset are taken as one-half of the total forces acting on section *a-a*. In the following derivations, the subscript $b2$ refers to forces and moments acting on section *b-b* due to the brace force from brace 2. Refer to Figure 3e.

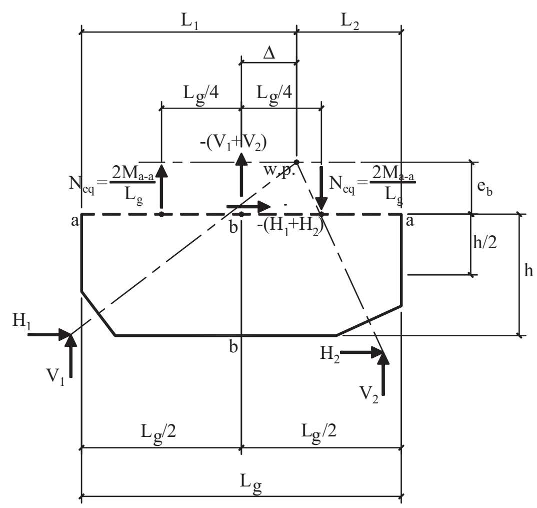
$$\begin{aligned}
\sum F_H &= 0 \\
&= H_{b2} + H_2 - \frac{1}{2}(H_1 + H_2) \\
H_{b2} &= \frac{1}{2}(H_1 + H_2) - H_2 \\
\sum F_V &= 0 \\
&= V_{b2} + V_2 - \frac{2M_{a-a}}{L_g} - \frac{1}{2}(V_1 + V_2) \\
V_{b2} &= \frac{1}{2}(V_1 + V_2) + \frac{2M_{a-a}}{L_g} - V_2
\end{aligned}$$



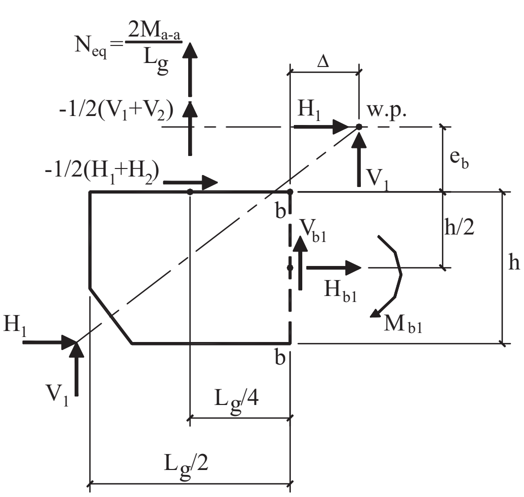
(a) Geometry, parameters and sign convention



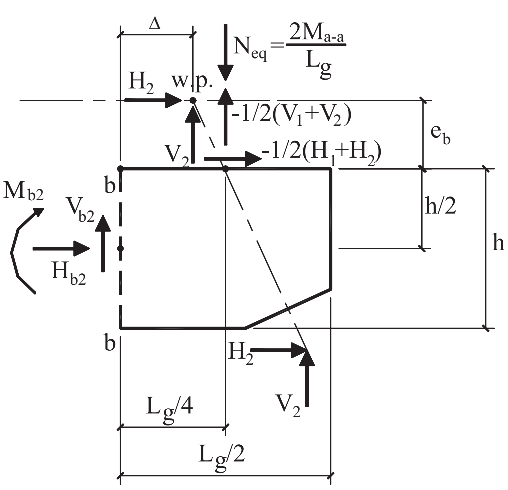
(b) Forces and moment on section a-a



(c) Equivalent forces on section a-a



(d) Forces on left half of gusset



(e) Forces on right half of gusset

Fig. 3. Free body forces on critical horizontal and vertical gusset sections.

$$\sum M_b = 0$$

$$= M_{b2} + H_2\left(e_b + \frac{h}{2}\right) - V_2\Delta + \frac{2M_{a-a}}{L_g}\left(\frac{L}{4}\right)$$

$$+\frac{1}{2}(V_1 + V_2)\left(\frac{L_g}{4}\right) - \frac{1}{2}(H_1 + H_2)\left(\frac{h}{2}\right)$$

$$M_{b2} = -\frac{L_g}{8}(V_1 + V_2) + \frac{h}{4}(H_1 + H_2)$$

$$-\frac{M_{a-a}}{2} + V_2\Delta - H_2\left(e_b + \frac{h}{2}\right)$$

In summary, the forces and moments acting on section *b-b* from the perspective of the right half of the gusset are as given in Equations 9, 10 and 11.

$$H_{b2} = \frac{1}{2}(H_1 + H_2) - H_2 \tag{9}$$

$$V_{b2} = \frac{1}{2}(V_1 + V_2) + \frac{2M_{a-a}}{L_g} - V_2 \tag{10}$$

$$M_{b2} = -\frac{L_g}{8}(V_1 + V_2) + \frac{h}{4}(H_1 + H_2) - \frac{M_{a-a}}{2} + V_2\Delta - H_2\left(e_b + \frac{h}{2}\right) \tag{11}$$

Example 1: Brace Force Distribution

Figure 4 shows the chevron connection geometry and dimensions. The force distributions acting on sections *a-a* and *b-b* will be determined using Equations 1 through 4 and 6 through 8, respectively. Equation 5 will be used to calculate the couple of the moment, $M_{a\text{-}a}$.

The variables for Example 1 are shown below. Note the signs of the component brace forces and the calculation of Δ. Using the assumed sign convention, a component force acting to the right in the horizontal direction, or upward in the vertical direction, is positive. The vertical eccentricity parameter, Δ, is calculated as shown in Equation 1. In this solution, the forces acting on section *b-b* are calculated using the left-half gusset body (Equations 6, 7 and 8). Note that the right-half gusset body (Equations 9, 10 and 11) can be used just as easily giving the same results.

L_g = 70.63 in., L_1 = 44.56 in., L_2 = 26.06 in.

e_b = 9.75 in., h = 22.75 in.

$\Delta = \frac{1}{2}(44.56 \text{ in.} - 26.06 \text{ in.}) = 9.25 \text{ in.}$

$H_1 = -338.6$ kips

$H_2 = -163.6$ kips

$V_1 = -225.8$ kips

$V_2 = 150.0$ kips

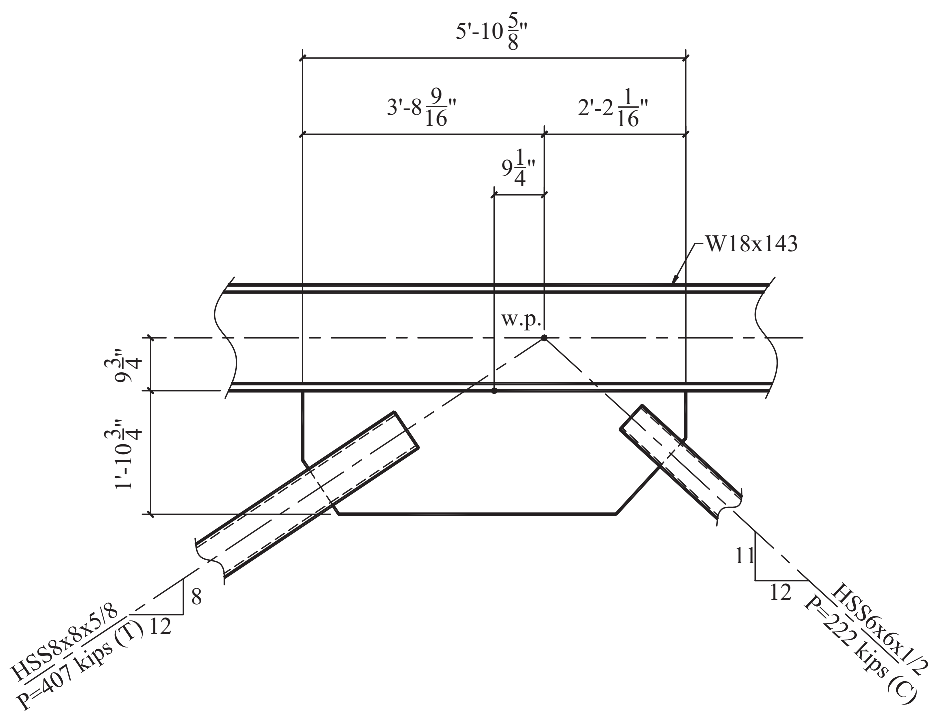


Fig. 4. Geometry and dimensions for Example 1.

Forces acting on section* a-a*:

$$H_{a-a} = -(H_1 + H_2) = -(-338.6 - 163.6)$$
$$= 502 \text{ kips}$$
$$V_{a-a} = -(V_1 + V_2) = -(-225.8 + 150)$$
$$= 75.8 \text{ kips}$$
$$M_{a-a} = (V_1 + V_2)\Delta - (H_1 + H_2)e_b$$
$$= (-225.8 + 150)(9.25) - (-338.6 - 163.6)(9.75)$$
$$= 4{,}195 \text{ kip-in}$$

Forces acting on section* b-b*:

$$H_{b-b} = H_{b1}$$
$$= \frac{1}{2}(H_1 + H_2) - H_1$$
$$= \frac{1}{2}(-338.6 - 163.6) - (-338.6) = 87.5 \text{ kips}$$

$$V_{b-b} = V_{b1}$$
$$= \frac{1}{2}(V_1 + V_2) - \frac{2M_{a-a}}{L} - V_1$$
$$= \frac{1}{2}(-225.8 + 150) - \frac{(2)(4{,}195)}{70.63} - (-225.8)$$
$$= 69.1 \text{ kips}$$
$$M_{b-b} = M_{b1}$$
$$= \frac{L}{8}(V_1 + V_2) + \frac{h}{4}(H_1 + H_2) - \frac{M_{a-a}}{2}$$
$$+ V_1\Delta - H_1\left(e_b + \frac{h}{2}\right)$$
$$= \frac{70.63}{8}(-225.8 + 150) + \frac{22.75}{4}(-338.6 - 163.6)$$
$$- \frac{4{,}195}{2} + (-225.8)(9.25) - (-338.6)(21.13)$$
$$= -559 \text{ kip-in.}$$

The free body diagrams are shown in Figure 5.

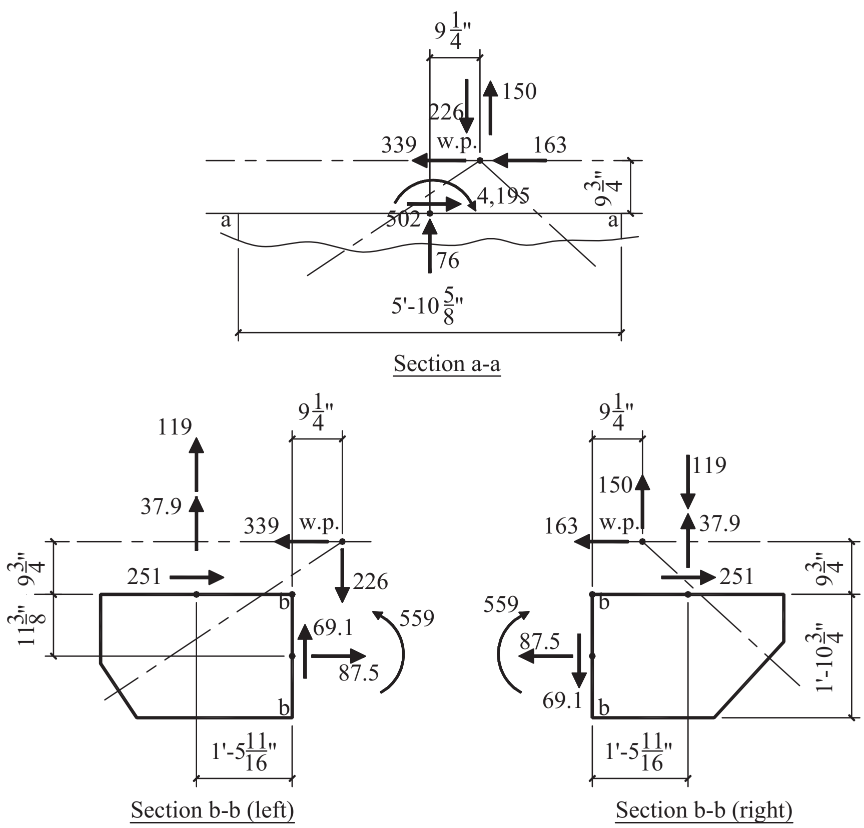


Fig. 5. Free body diagrams for Example 1.

2. CURRENT METHOD USED FOR BEAM SHEAR DETERMINATION

Typically, the shear imparted to the beam by the brace force distribution is evaluated as if the connection is isolated from the frame. The beam span and the location of the work point along the span of the beam are not considered. Consider the joint shown in Figure 6, where the brace bevels are equal, the magnitude of the brace forces are equal and one brace is in tension while the other is in compression. Using the procedure presented previously, the forces acting at the gusset-to-beam interface are given in Figure 7. Without considering the span of the beam or the location of the work point along the span of the beam, the shear in the beam, V_{beam}, is constant between the two points of applied load and would be taken as the $M_{a\text{-}a}$ couple of 60.94 kips ($M_{a\text{-}a}$ = 96.49 k-ft, L_g = 3 ft, 2 in.). The moment in the beam would be taken as one-half of the area under that shear gradient, which would be that given in Equation 12. It's worth noting that evaluating the shear demand on the beam would typically be a consideration to determine if the beam required a web doubler in the connection region. Beam moment in the connection region is not typically considered. The beam moment in the connection region is calculated as given in Equation 12.

$$M_{beam} = \frac{V_{beam} L_g}{4} \tag{12}$$

It is no coincidence that Equation 12 is equivalent to one-half of the summation of the horizontal components of the brace forces times one-half the beam depth as given by Equation 13. Figure 8 shows the beam shear and moment distribution resulting from the forces acting on section *a-a*.

$$M_{beam} = \frac{\sum H_i e_b}{2} \tag{13}$$

With the type of configuration and forces shown in Figures 6 through 8, the impact of the brace forces on the beam is determined to be V_{beam} = 60.94 kips and M_{beam} = 48.24 kip-ft, when the connection is evaluated as if it is isolated from the frame. Is this isolated evaluation adequate? Should the span of the beam, and the location of the work point, be considered? Before addressing these questions, we first have to consider how the forces acting on section *a-a* will be assumed to be distributed for the evaluation of the load effects on the beam.

3. GUSSET-TO-BEAM INTERFACE FORCE DISTRIBUTION

For the analysis and design of the gusset and the gusset-to-beam weld, the normal forces acting on the beam are assumed to be distributed uniformly along the gusset-to-beam interface length. The horizontal forces, $H_{a\text{-}a}$, are assumed to act on the interface eccentrically with a lever arm equal to e_b, as discussed previously. When a normal force, $V_{a\text{-}a}$ is present, the normal force is assumed to be distributed uniformly along the gusset-to-beam interface length. The moment at the interface, $M_{a\text{-}a}$, is assumed to act as distributed tension/compression normal forces equal to the couple of $M_{a\text{-}a}$ (see

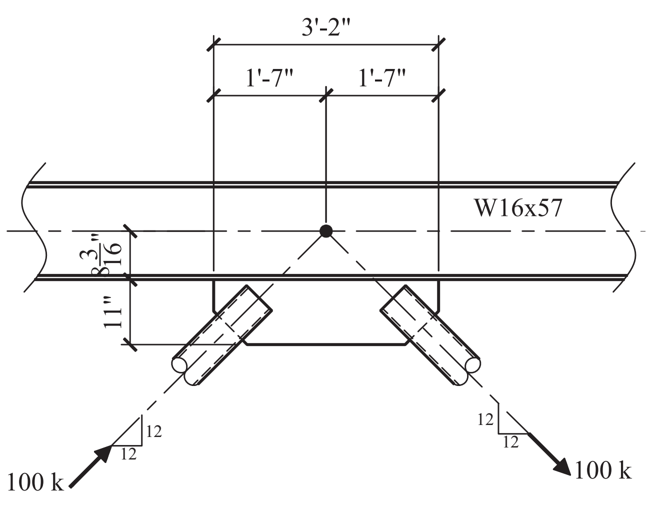


Fig. 6. Equal brace bevels and forces in a compression-tension brace arrangement.

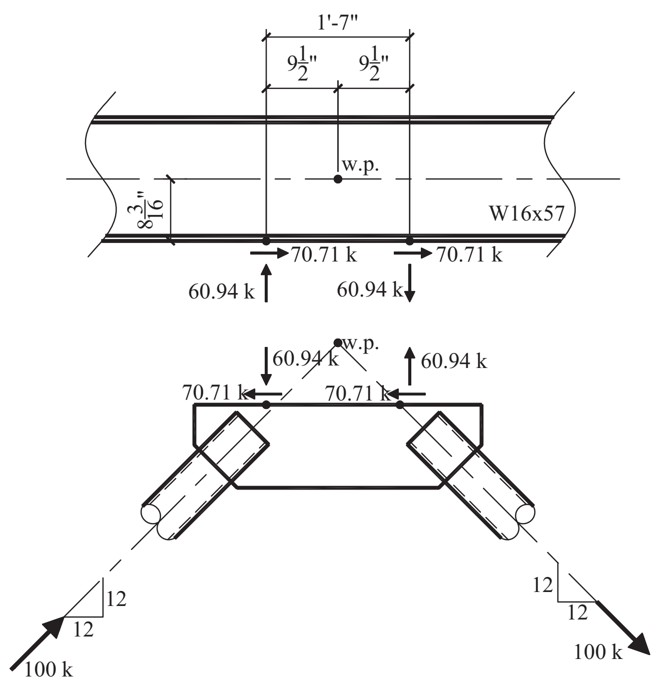


Fig. 7. Forces acting on section a-a *for connection shown in Figure 6.*

Equation 5) divided by one-half the gusset length, $L_g/2$ (see Equation 14). Figure 9 shows the force distributions that are typically assumed for the gusset plate design and the design of the gusset-to-beam weld.

$$n_{eq} = \frac{\dfrac{2M_{a-a}}{L_g}}{\dfrac{L_g}{2}} = \frac{4M_{a-a}}{L_g^2} \tag{14}$$

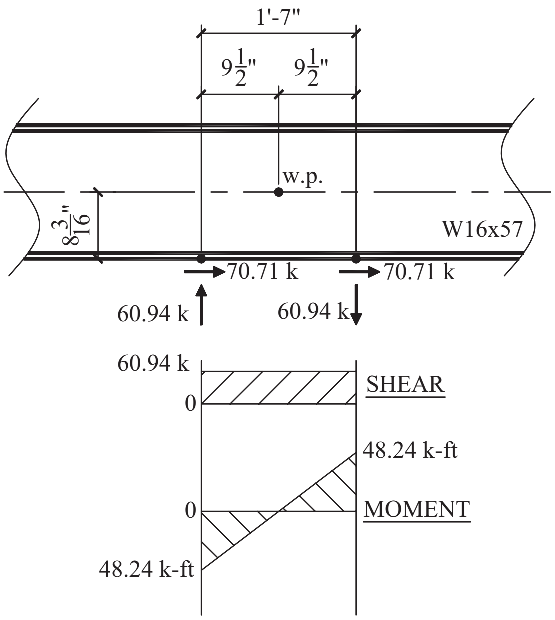


Fig. 8. Beam shear and moment distribution for the isolated connection shown in Figure 6.

The forces and moment acting on the gusset-to-beam interface are treated as externally applied loads to evaluate beam shear and moment. The distributions of these interface forces can be uniformly distributed as shown in Figure 9. However, using these distributions can be tedious when evaluating beam shear and moment distribution. To simplify the beam analysis, it is recommended that the resultant force like those shown in Figure 5 be used to evaluate the beam. The following is an example to illustrate the differences between the two methods of beam evaluation (i.e., the distributed method versus the resultant method).

Suppose the connection presented in Example 1 is part of a frame with a beam spanning 25 ft, and the work point is located 15 ft 6 in. from the left support, as shown in Figure 10a. The forces acting on the gusset-to-beam interface can be assumed to be distributed uniformly along the length of the interface, as shown in the loading diagram in Figure 11, or as resultant forces acting at the centroids of the two half gusset bodies, as shown in Figure 12. The beam shear and moment distributions along the length of the beam are shown in Figures 11 and 12 for each method, respectively. As can be seen in the two figures, the beam shear and moment gradients are a little different as a result of the types of loads. Both loading conditions produce the same maximum beam shear, and the maximum beam shears are located at the same locations. The maximum moment, for this example, is about 37% larger when the resultant loads are used and occurs within the connection region. The conservativeness in the maximum moment calculation using the resultant method can be attributed to two facts:

1. The beam end reactions are the same regardless of which method is used. However, the area under the shear gradient is larger using the resultant loads because the length of the gradient interval is longer. For example, the left support reactions using both methods is 28.83 kips, as can be seen in Figures 11 and 12. The

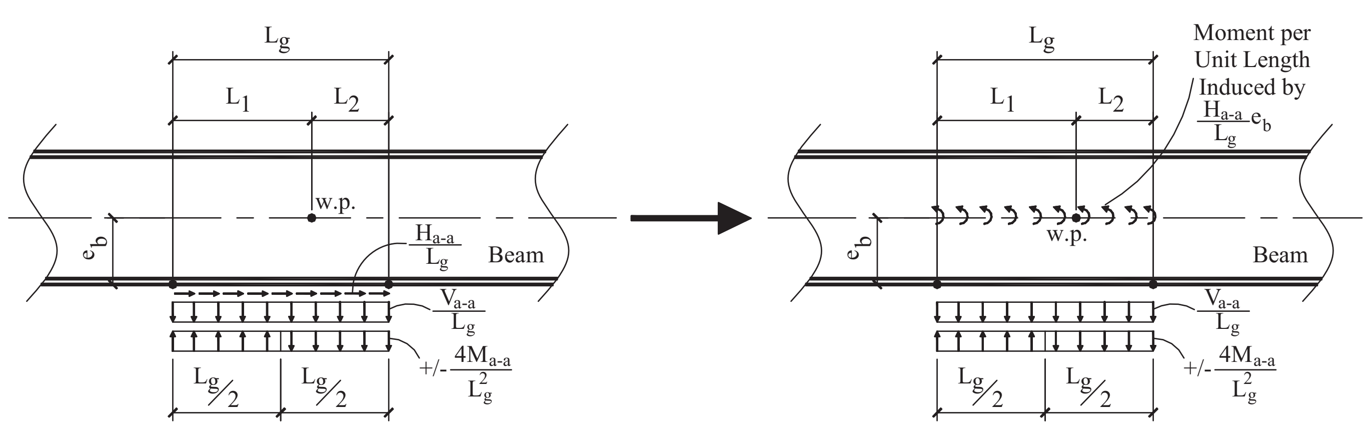
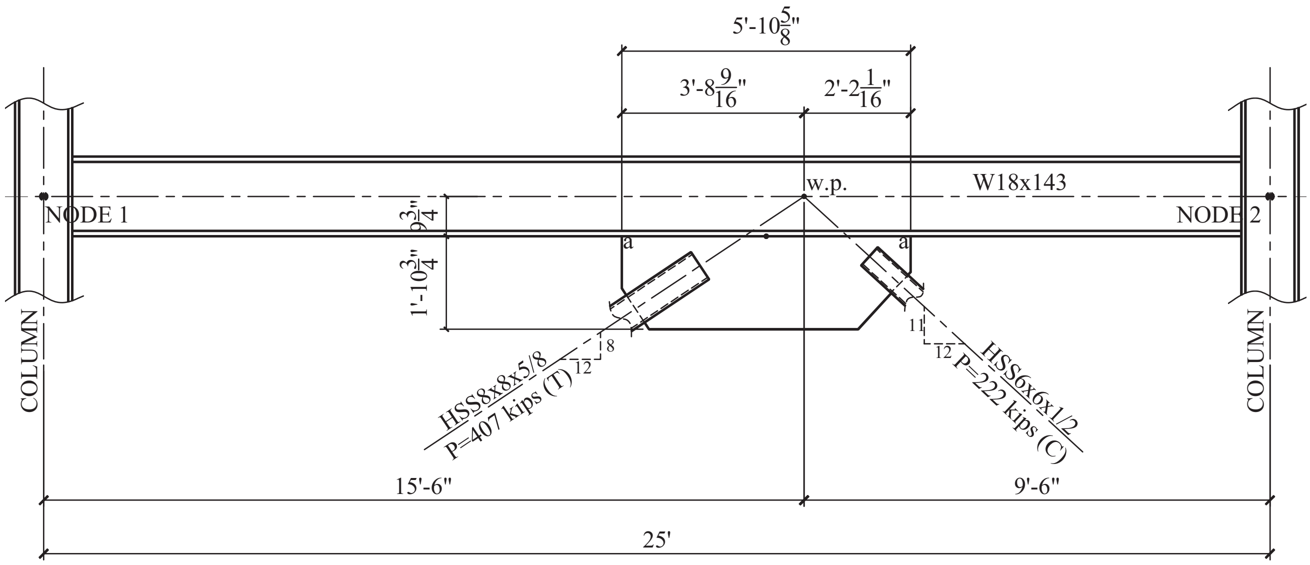
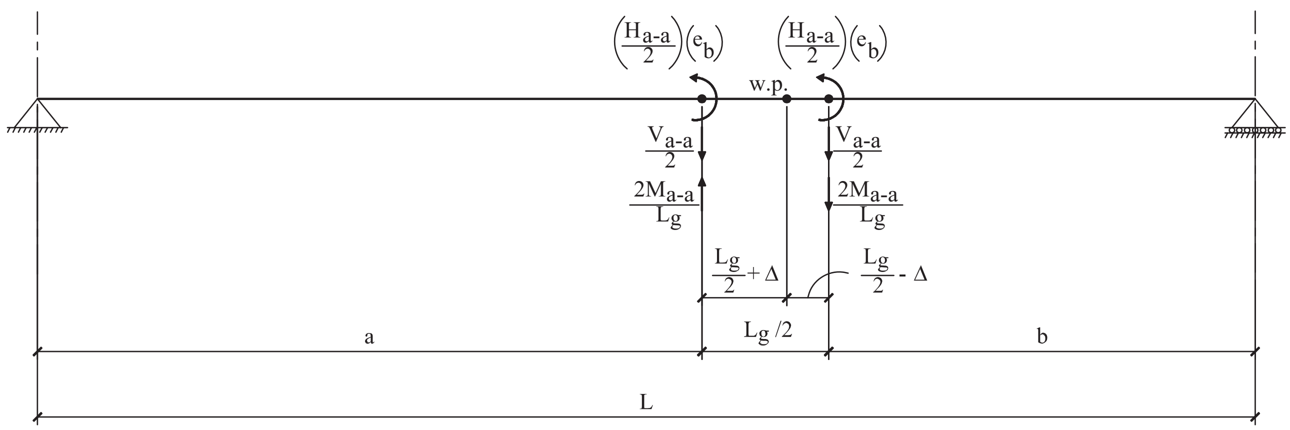


Fig. 9. Interface forces uniformly distributed.



(a)



(b)

Fig. 10. (a) Gusset connection shown in Figure 5 in context of a frame; (b) beam model: resultant force loading and boundary conditions.

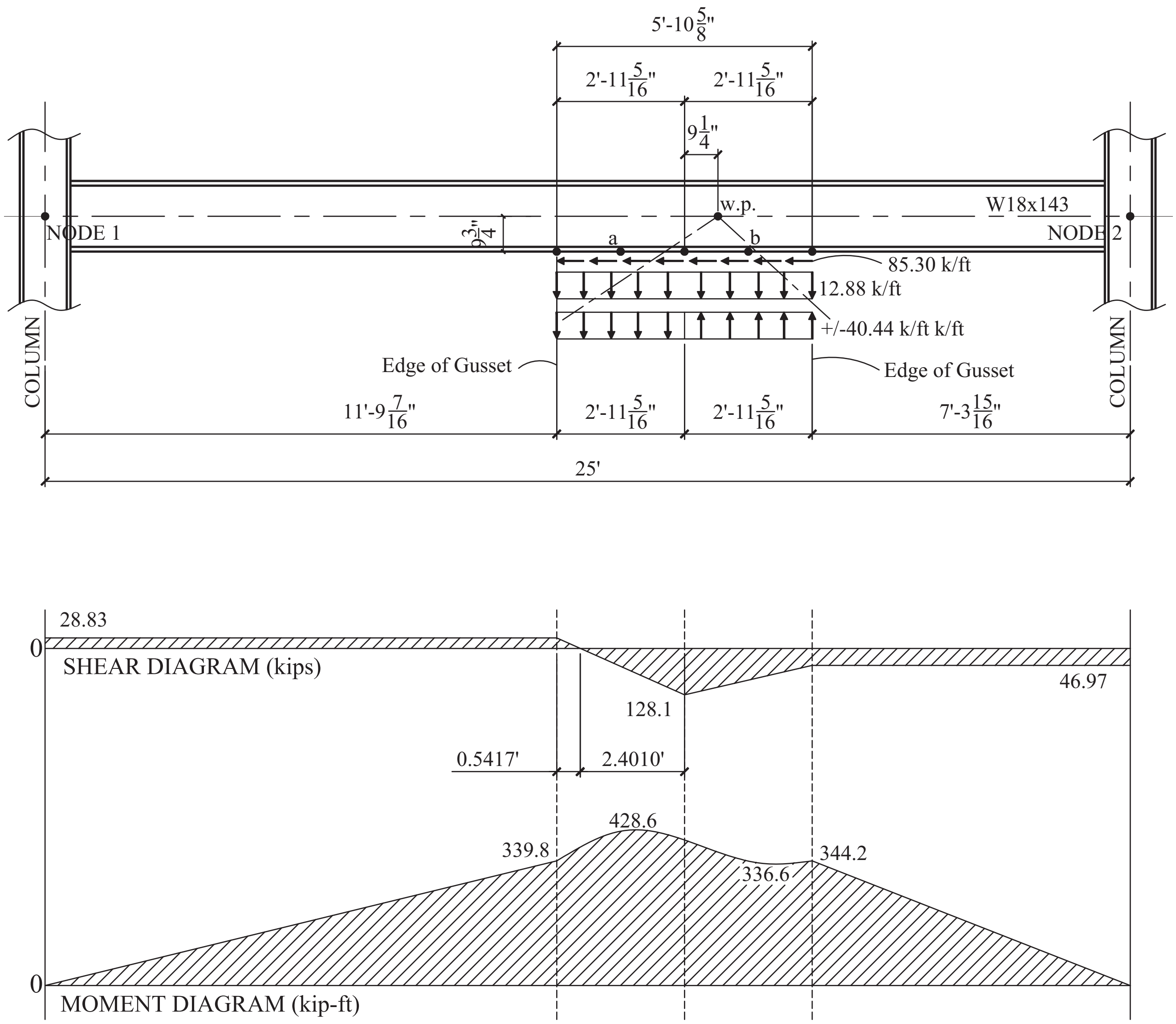


Fig. 11. Beam shear and moment with uniformly distributed loads acting on interface.

distance from the left support to the change in loading in Figure 11 is the distance to the left edge of the gusset (11 ft 9 7⁄16 in.) assuming distributed load; the distance from the left support to the change in loading in Figure 12 is the distance to the centroid of the left half gusset body (13 ft 3 1⁄8 in.). The difference between the two distances is ¼ of the gusset length, L_g. Thus, the longer the gusset, the more conservative the moment calculation will be when using resulting loads.

2. The concentrated moment using the resultant method is more conservative relative to distributing the moment over the entire length of the gusset.

Given that using resultant loads gives the same beam end reactions and maximum beam shears (for most cases), that the calculated moment will always be conservative and that the resultant load method is far simpler relative to assuming distributed loads, resultant loads will be used throughout this discussion and in the example problems presented, except for beam web doubler plate detailing where the distributed loads are used. Figure 10b shows the general beam model that will be used to determine beam shear and moment distribution for beam evaluations.

Note that both methods give the same maximum beam shear in this example. However, it should be noted that when braces frame to both the top and bottom of the beam, and the Δ parameter is non-zero, it is possible that the resultant load method will produce a slightly larger maximum beam shear. The resultant load method will be conservative when comparing required beam shear strength to available beam shear strength. However, when the required beam shear strength exceeds the available beam shear strength, the distributed load method should be used to determine required web doubler thickness as well as the extent of the required beam web

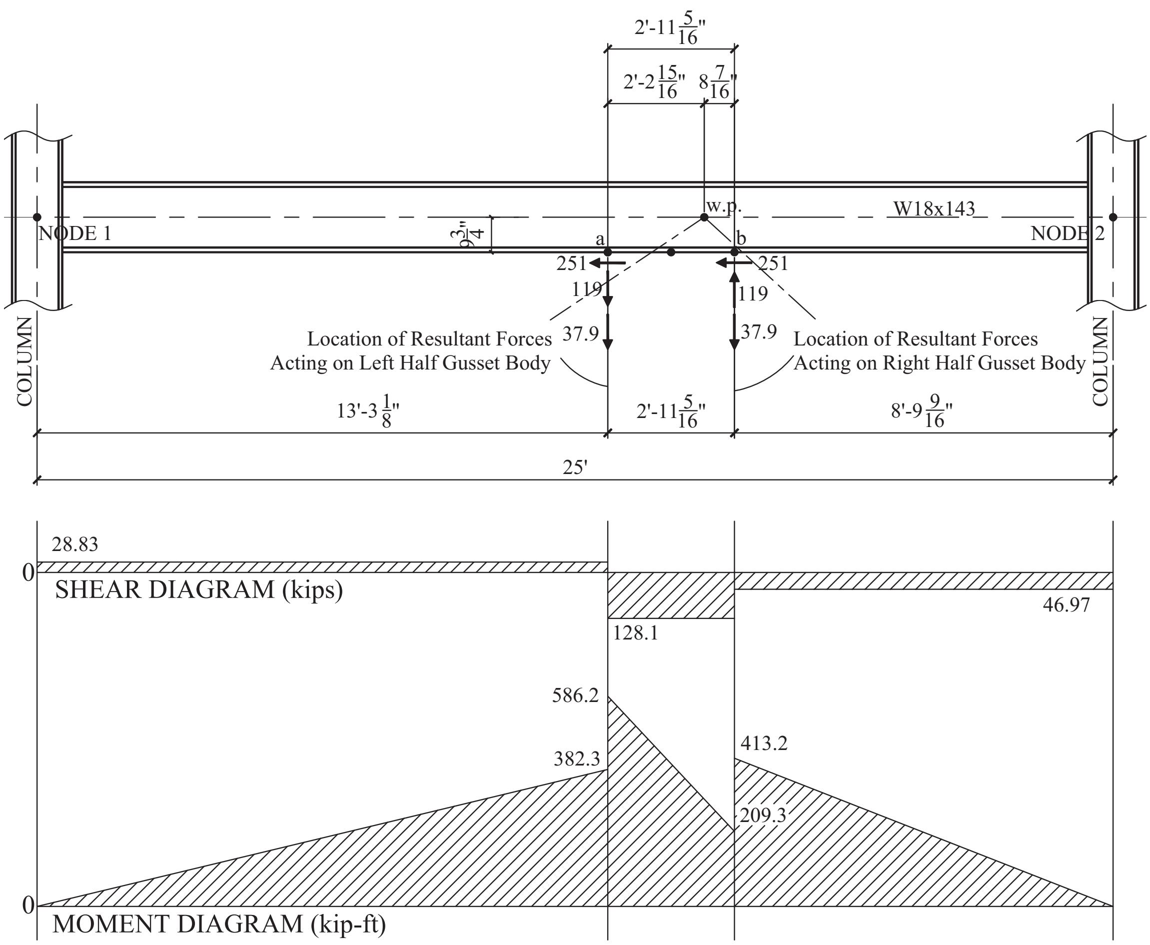


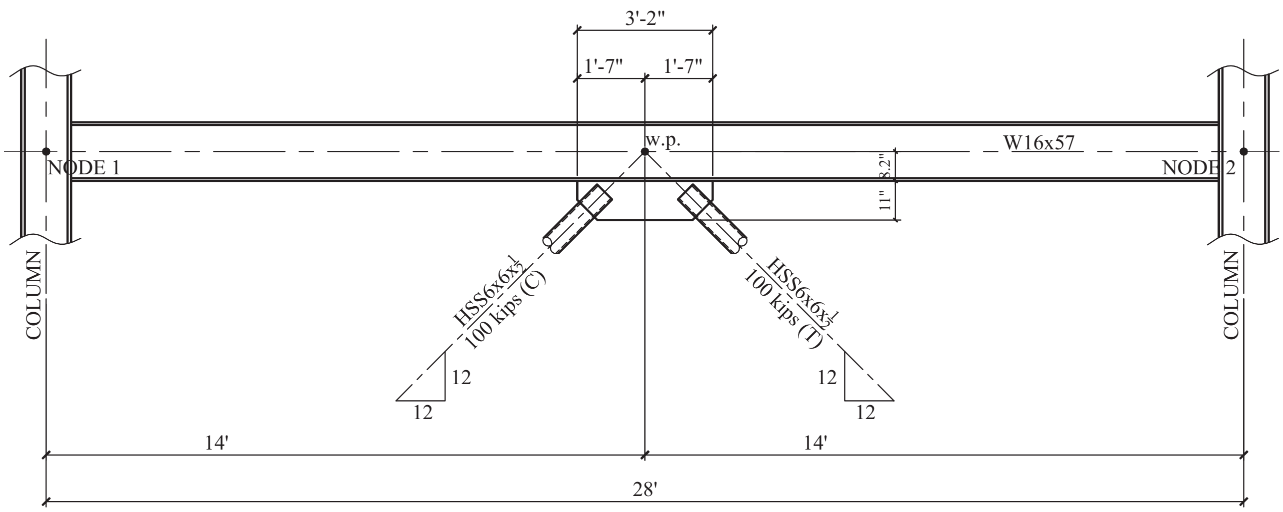
Fig. 12. Beam shear and moment with resultant forces acting at interface.

doubler plate. An example of how to detail a beam web doubler plate is provided in Part 4 of Example 3 presented later in this paper.

4. BEAM SHEAR AND MOMENT DISTRIBUTION

As discussed previously, the shear and moment imparted to the frame beam by the brace forces is typically evaluated as if the connection joint is isolated from the frame. Is this a valid approach, or does the span of the beam and the location of the work point along the span of the beam need to be considered when evaluating the brace load effects on the frame beam? If the algebraic sum of the vertical components of the brace forces is zero, shear and moment imparted to the beam is independent of the span of the beam and the location of the work point. If the algebraic sum of the vertical components of the brace forces is non-zero, the shear and moment imparted to the beam is dependent on the span of the beam and the location of the work point.

Figures 13a and 13b show a W16×57 spanning 28 ft. The brace geometry and forces are such that the algebraic sum of the vertical components of the brace forces is zero. Figure 13a has the work point located at mid-span of the beam, while Figure 13b has the work point located 6 ft (for simplicity, the brace bevels are assumed unchanged to illustrate a point) from the right support. Figures 14 and 15 show the beam shear and moment diagrams for the two work point locations, respectively. Referring to Figures 14 and 15, it can be observed that the beam shear and moment imparted to the beam is contained within the connection region. Beam



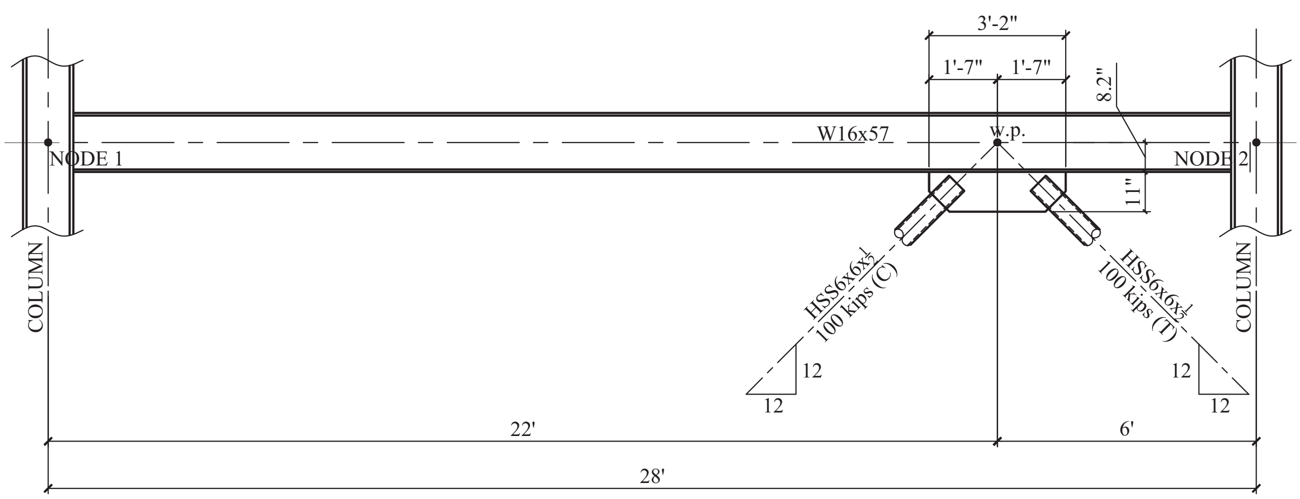


Fig. 13. $\Sigma V_i = 0$ at different locations along beam span: (a) connection located at mid-span; (b) connection located 6 ft from right support.

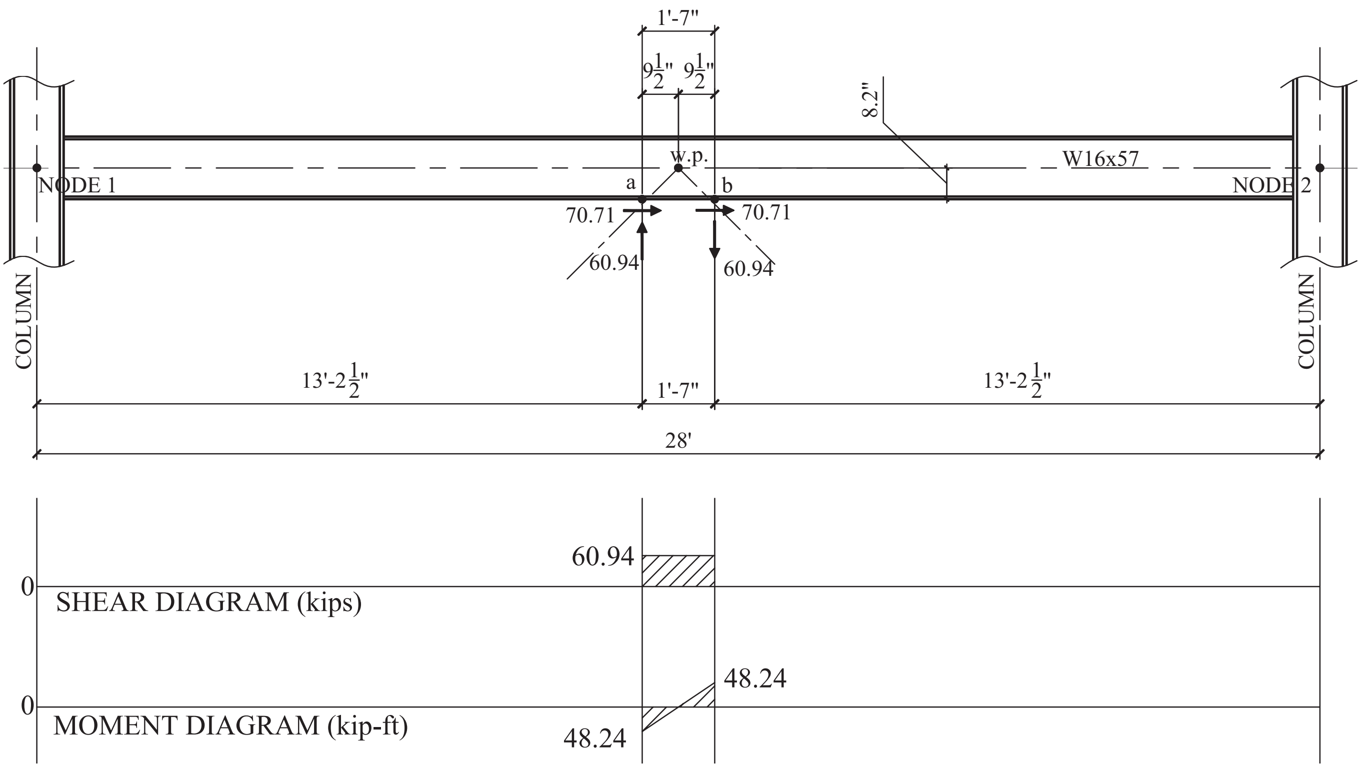


Fig. 14. $\Sigma V_i = 0$*; beam shear and moment with work point at mid-span.*

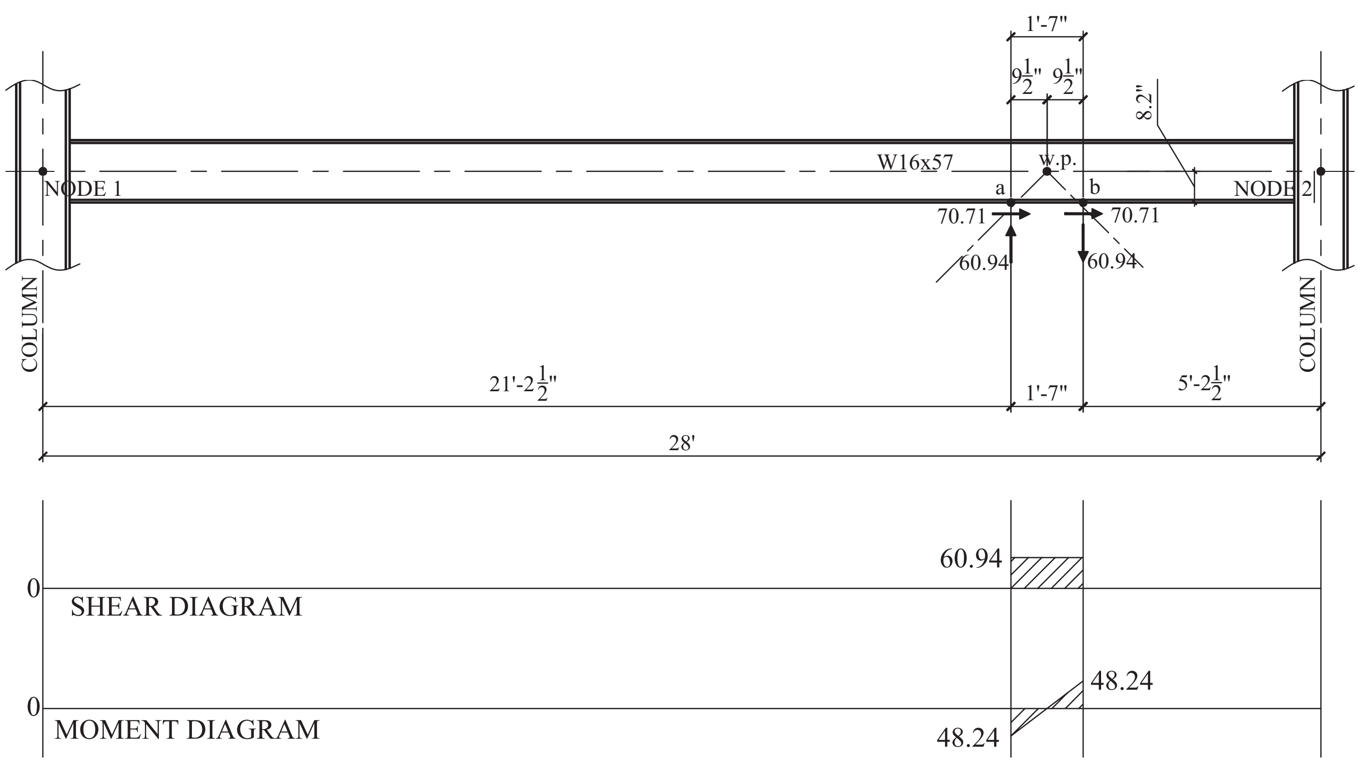


Fig.15. $\Sigma V_i = 0$*; beam shear and moment with work point off mid-span.*

shear and moment outside of the connection region is zero. If the connection was evaluated as if the joint was isolated from the frame, the beam shear and moment would be the same as that shown in Figures 14 and 15. Thus, the shear and moment imparted to the beam by the brace forces is independent of the beam span and the location of the work point along the beam span. Therefore, when the algebraic sum of the vertical components of the brace forces is zero, it is sufficient to evaluate the beam as if the connection is isolated from the frame. Historically, this is probably the reason that this type of connection has been designed in isolation.

Now consider that the tension brace force shown in Figure 13 is increased from 100 kips to 300 kips while the compression brace force remains at 100 kips, as shown in Figures 16a and 16b. The algebraic sum of the vertical components of the brace forces is now non-zero. The summation of the vertical components of the brace forces is 141.2 kips. Figures 16a and 16b show the geometry and forces for these cases. Figures 17 and 18 show the beam shear and moment diagrams for the case where the work point is at mid-span and when the work point is 6 ft from the right support. As can be seen in Figures 17 and 18, the location of the work

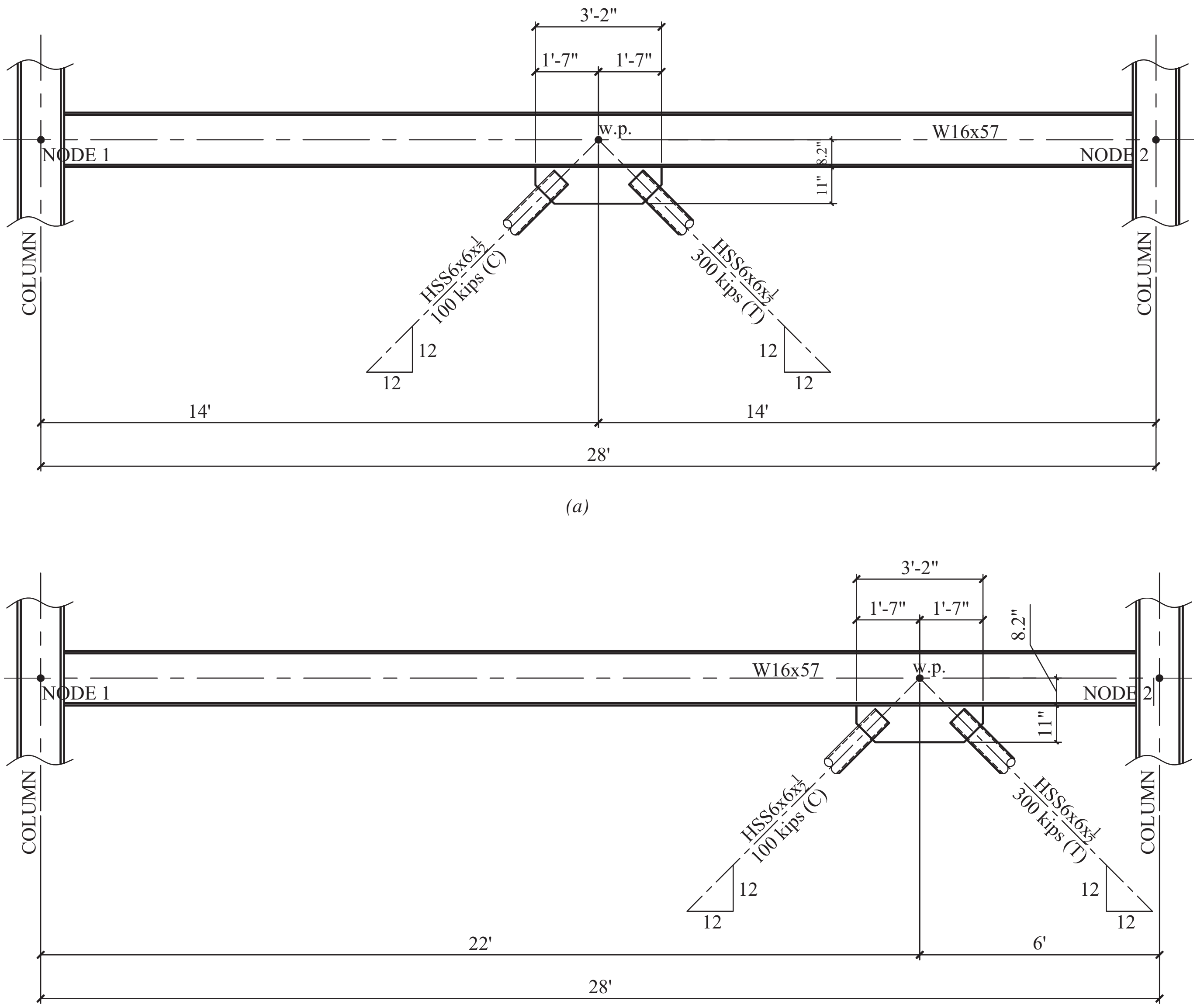


Fig. 16. $\Sigma V_i \neq 0$ (141.2 kips) at different locations along beam span:
(a) connection located at mid-span; (b) connection located 6 ft from right support.

point has an effect on how the beam shear and moment are distributed along the length of the beam and has an effect on the maximum beam shears and moments.

Referring to the beam shear diagram in Figure 17, it can be seen that the beam shear within the connection region is equal to the couple of the moment, $M_{a\text{-}a}$ (in this case, 122.1 kips), and the shear outside of the connection region is equal to one-half of the algebraic sum of the vertical components of the brace forces (in this case, 70.6 kips). This may lead one to conclude that this is always the case when there is an unbalanced vertical force and the work point is at mid-span of the beam. However, this is not always the case; there is one other parameter that must be satisfied. The centroid of the gusset interface must also be vertically aligned with the work point. In other words, the parameter Δ must be zero. For the beam shear within the connection region to be equal to the couple, and the shear outside of the connection region to be equal to one-half of the unbalanced force, the work point must be located at mid-span of the beam and the parameter Δ must be equal to zero. This will also be true if Δ is non-zero but the centroid of the gusset happens to be vertically aligned with the mid-span work point. Simply put, this is only true when the resultant normal forces acting on the interface are symmetric about the mid-span work point.

Comparing the beam shear distributions shown in Figures 17 and 18, it can be concluded that the beam shear and moment distribution, as well as the maximum shear and moment, are dependent on the location of work point along the span of the beam. If the beam shear and moment are evaluated as if the connection is isolated from the frame, the beam shear and moment would be determined to be 122.1 kips and 96.7 kip-ft, respectively, regardless of the beam span and the location of the work point along the span of the beam (refer to previous discussion). If the beam shear and moment are evaluated considering the frame, with the work point at mid-span, the maximum beam shear and moment is 122.1 kips and 1,030 k-ft, respectively. The beam shears are the same because the resultant normal forces are symmetric about the work point. However, the beam moment is 1,030/96.7 = 10.6 times larger when the frame is considered. When the frame is considered and the work point is not at mid-span (6 ft from the right support in this case), the maximum beam shear and moment is 111.0 kips and 674.4 k-ft (see Figure 18). The maximum beam shear is overestimated by 122.1/111.0 = 1.10 times and the moment is underestimated by 674.4/96.7 = 7.97 times when the frame is considered. Furthermore, as can be seen in Figure 18, the maximum beam shear occurs outside of the connection region. This would not be noticed if the beam shear is evaluated as if the connection is isolated from the frame—a problematic issue if one was to evaluate the need for web doubler plates and the location where such reinforcement would be required.

The effect of the location of the work point has been illustrated in Figures 17 and 18. The span of the beam also has an effect on the beam shear and moment distribution. Although not illustrated in this discussion, one can deduce that a change in beam span has an effect on the beam end

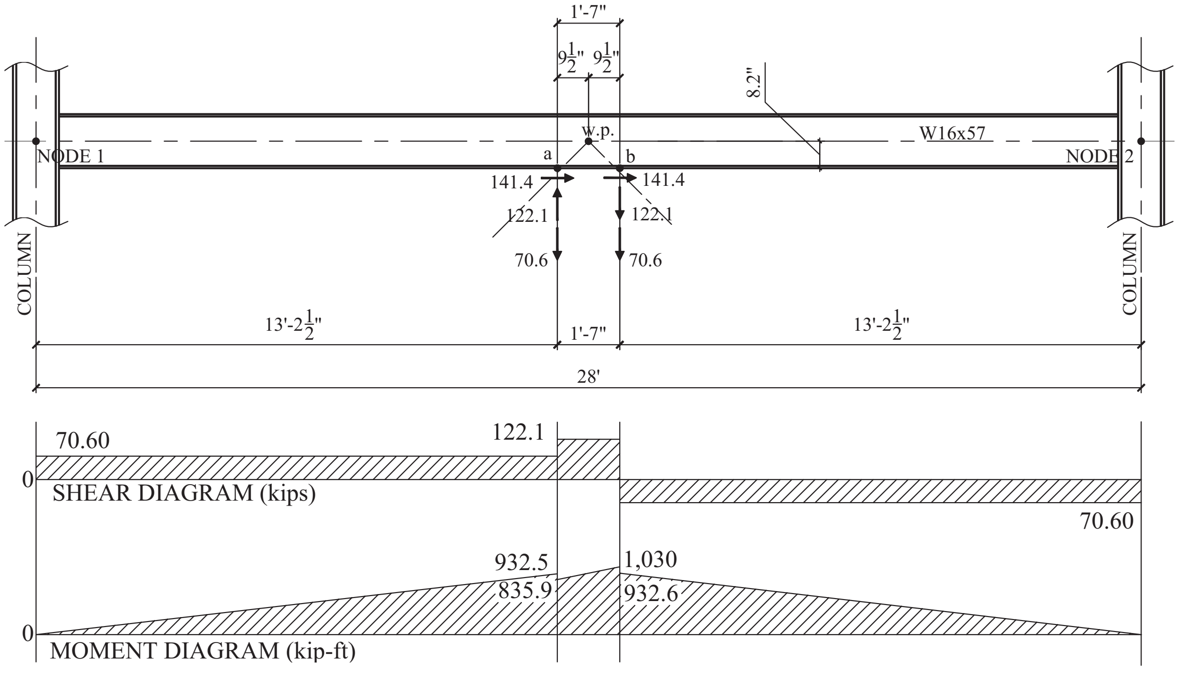


Fig. 17. $\Sigma V_i \neq 0$ *(141.2 kips) beam shear and moment with mid-span work point.*

reactions. Given that the beam shear and moment distributions are a function of the beam end reactions, the span of the beam also affects the beam shear and moment distribution as well as the maximum beam shears and moments.

5. THE CHEVRON EFFECT

When evaluating the brace forces in a chevron braced frame subjected to lateral loads, the analysis will reveal that one brace is in tension while the other is in compression. For static equilibrium, the vertical components of the brace forces will sum algebraically to zero. However, it is sometimes necessary, or required, to perform some type of mechanism analysis where in such a case, the algebraic sum of the vertical components of the brace forces will be non-zero. One example of a mechanism analysis is that required in a seismic braced frame where the brace in tension is assumed to reach the expected tensile strength of the brace, while the brace in compression is assumed to reach its buckling strength, or even a post-buckling strength. The impact of the brace forces on the frame beam needs to be evaluated in either case.

The following example problem illustrates the chevron effect, and emphasizes the importance of accounting for the span of the beam as well as the location of the work point along the span of the beam.

Example 2: The Chevron Effect with Mechanism Analysis

For the chevron bracing configuration shown in Figure 19:

1. Determine the force distribution in the connection for the brace forces given in Table 1. For this analysis only the forces acting on section *a-a* need to be determined using Equations 1 through 4. For an actual gusset design, the forces acting on section *b-b* are also required, but not necessary, for this example problem.
2. Determine the beam shear and moment distribution along the span of the beam for each load case based on the forces and moments acting on section *a-a* determined in Section 1 of this paper.
3. Compare the maximum beam shears and moments determined as if the connection was isolated from the frame to those values obtained from the beam shear and moment diagrams.

Assume that $(KL)_x$ and $(KL)_y$ for both braces is 22 ft. Note that this length accounts for the pull-off dimensions at both ends of each brace.

Typically, both directions of lateral load would be considered. For this example, only the three load cases shown in Table 1 will be considered.

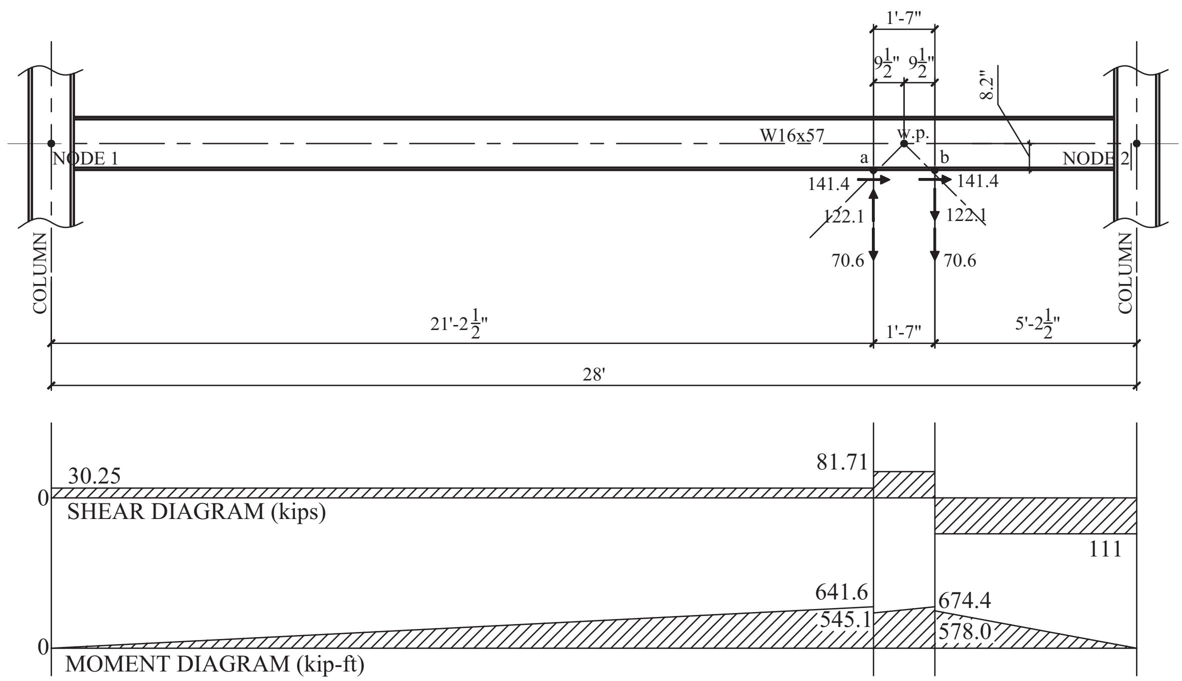


Fig. 18. $\Sigma V_i \neq 0$ *(141.2 kips) beam shear and moment with work point off mid-span.*

Table 1. Load Cases for Example 2		
Load Case	**P1 (kips)**	**P2 (kips)**
1	+449	–540
2	$+R_yF_yA_g = +1{,}205$	$Pb = -1.14F_{cre}A_g = -778$
3	$+R_yF_yA_g = +1{,}205$	$(0.3)P_b = -233$

Sign convention: (+) indicates tension; (–) indicates compression.

It is worth noting here that load cases 2 and 3 are representative of the mechanistic analysis required by the AISC *Seismic Provisions.*

Example 2: Solution

The variables for Example 2 are shown below.

$$L_g = 47.75 \text{ in.},\ L_1 = 20.50 \text{ in.},\ L_2 = 27.25 \text{ in.}$$
$$e_b = 10.70 \text{ in.},\ h = 9.125 \text{ in.}$$
$$\Delta = \frac{1}{2}(20.50 \text{ in.} - 27.25 \text{ in.}) = -3.375 \text{ in.}$$

Load Case 1

$$H_1 = -269.40 \text{ kips}$$
$$H_2 = -403.25 \text{ kips}$$
$$V_1 = -359.18 \text{ kips}$$
$$V_2 = 359.18 \text{ kips}$$

Figure 20 shows the geometry and brace forces for Load Case 1. From the data given in Figure 20, the forces acting on section *a-a* can be determined.

Forces acting on section* a-a*:

$$\begin{aligned} H_{a-a} &= -(H_1 + H_2) \\ &= -(-269.40 - 403.25) \\ &= 672.65 \text{ kips} \end{aligned}$$

$$\begin{aligned} V_{a-a} &= -(V_1 + V_2) \\ &= -(-359.18 + 359.18) \\ &= 0 \text{ kips} \end{aligned}$$

$$\begin{aligned} M_{a-a} &= (V_1 + V_2)\Delta - (H_1 + H_2)e_b \\ &= (-359.18 + 359.18)(-3.375) \\ &\quad -(-269.40 - 403.25)(10.70) \\ &= 7{,}195.57 \text{ kip-in.} \end{aligned}$$

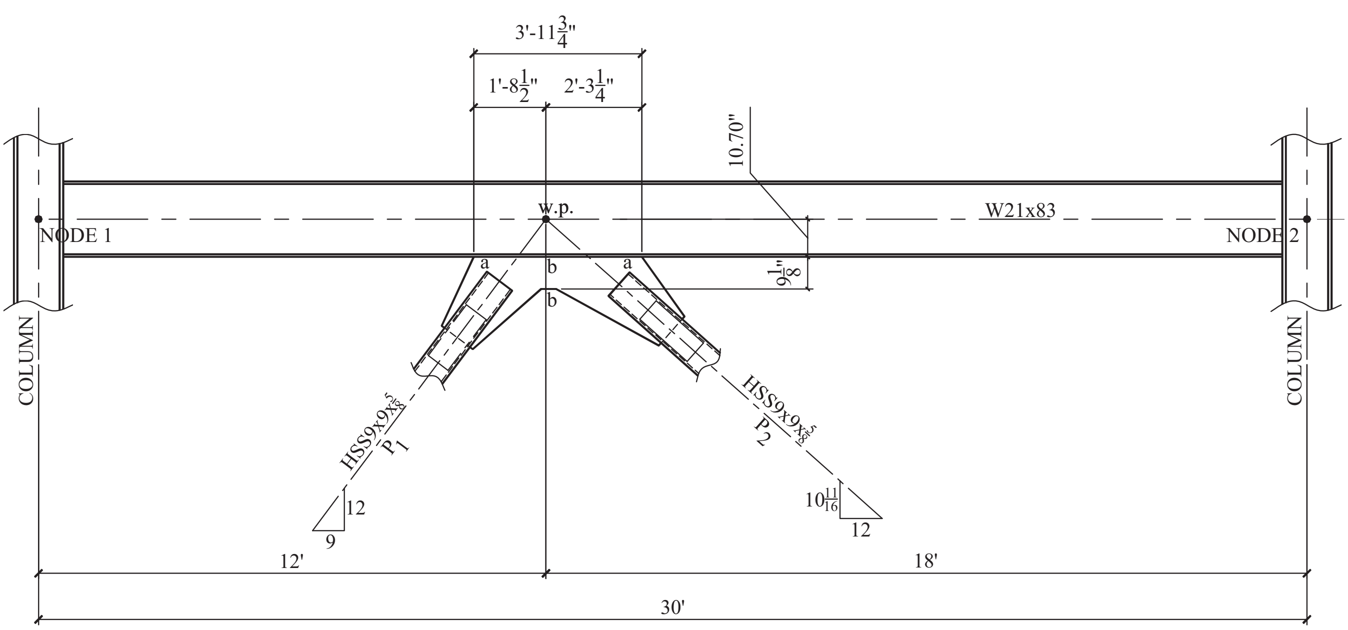


Fig. 19. Connection geometry and dimensions for Example 2.

The couple of $M_{a\text{-}a}$ is:

$$N_{eq} = \frac{(2)(7,195.57)}{47.75} = 301.4 \text{ kips}$$

Figure 21 shows the resulting interface forces and beam shear and moment distributions. As can be seen in Figure 21, the maximum beam shear and moment are:

$$V_{u,\max} = 301.4 \text{ kips}$$
$$M_{u,\max} = 299.8 \text{ kip-ft}$$

Load Case 2

Figure 22 shows the geometry and brace forces for load case 2. From the data given in Figure 22, the forces acting on section *a-a* can be determined.

$$H_1 = -723.00 \text{ kips}$$
$$H_2 = -580.89 \text{ kips}$$
$$V_1 = -964.00 \text{ kips}$$
$$V_2 = 517.44 \text{ kips}$$

Forces acting on section* a-a*:

$$\begin{aligned} H_{a-a} &= -(H_1 + H_2) = -(-723.00 - 580.89) \\ &= 1,303.98 \text{ kips} \end{aligned}$$

$$\begin{aligned} V_{a-a} &= -(V_1 + V_2) = -(-964.00 + 517.44) \\ &= 446.56 \text{ kips} \end{aligned}$$

$$\begin{aligned} M_{a-a} &= (V_1 + V_2)\Delta - (H_1 + H_2)e_b \\ &= (-964.00 + 517.44)(-3.375) \\ &\quad -(-723.00 - 580.98)(10.70) \\ &= 15,459.77 \text{ kip-in.} \end{aligned}$$

The couple of $M_{a\text{-}a}$ is:

$$N_{eq} = \frac{(2)(15,459.77)}{47.75} = 647.53 \text{ kips}$$

Figure 23 shows the resulting interface forces and beam shear and moment distributions. As can be seen in Figure 23, the maximum beam shear and moment are:

$$V_{u,\max} = 602.9 \text{ kips}$$
$$M_{u,\max} = 3,605 \text{ kip-ft}$$

Load Case 3

Figure 24 shows the geometry and brace forces for load case 3. From the data given in Figure 24, the forces acting on section *a-a* can be determined.

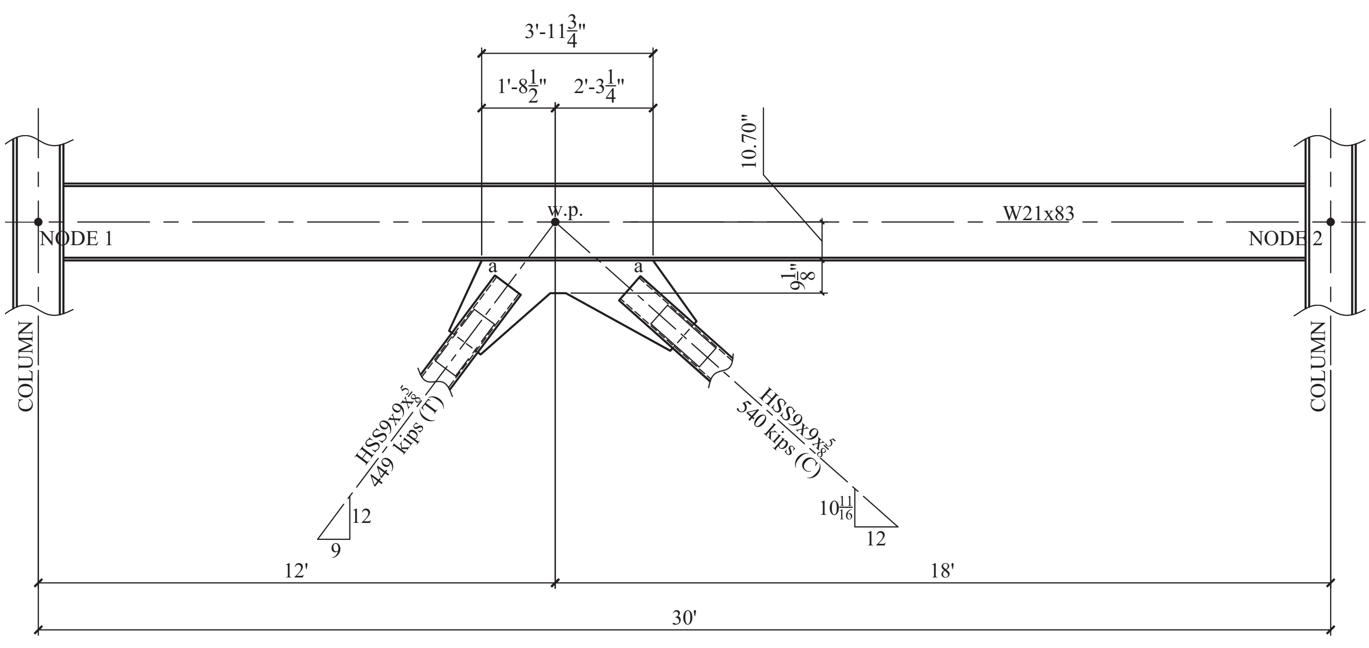


Fig. 20. Geometry and brace forces for Example 2, load case 1.

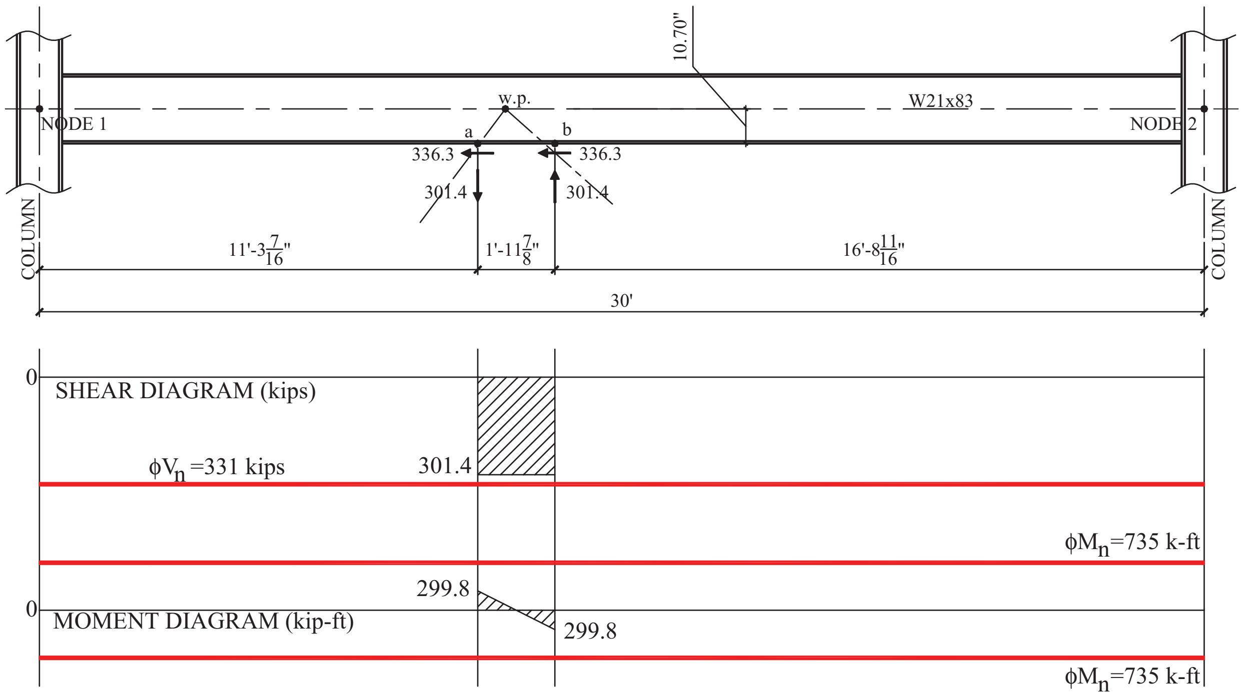


Fig. 21. Beam shear and moment distribution for load case 1.

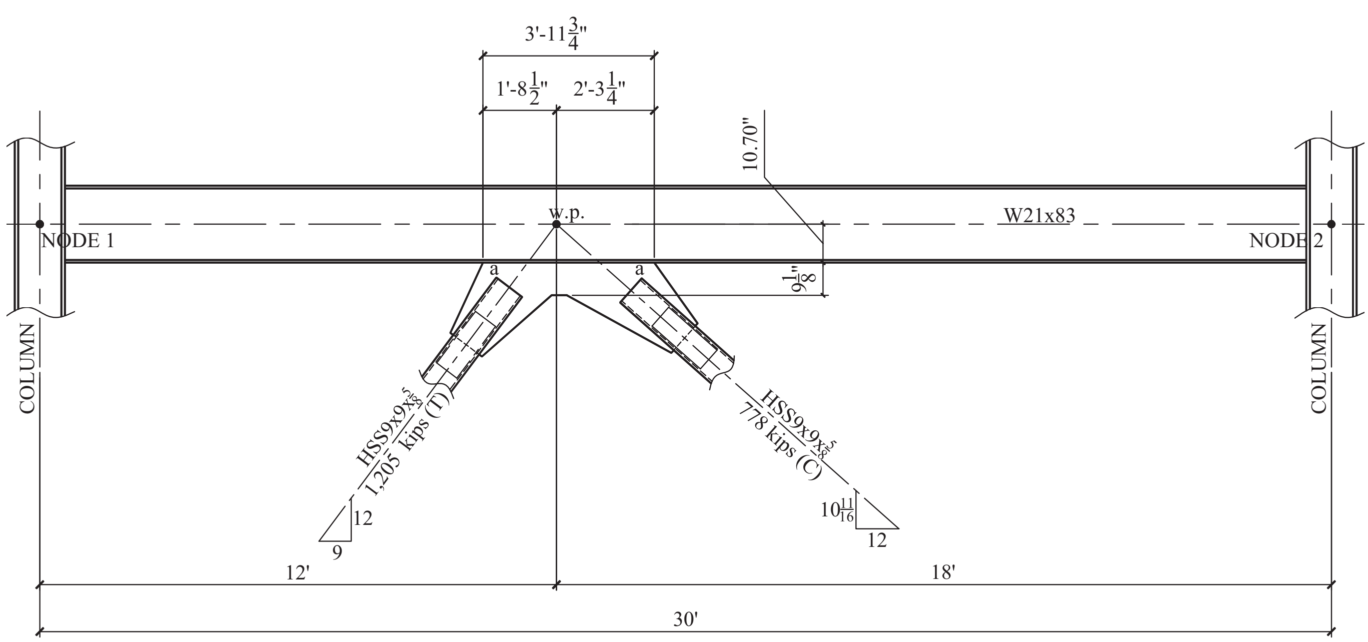


Fig. 22. Geometry and brace forces for Example 2, load case 2.

$H_1 = -723.00$ kips
$H_2 = -174.00$ kips
$V_1 = -964.00$ kips
$V_2 = 154.97$ kips

Forces acting on section* a-a*:

$$
\begin{aligned}
H_{a-a} &= -(H_1 + H_2) = -(-723.00 - 174.00) \\
&= 897.00 \text{ kips} \\
V_{a-a} &= -(V_1 + V_2) = -(-964.00 + 154.97) \\
&= 809.03 \text{ kips} \\
M_{a-a} &= (V_1 + V_2)\Delta - (H_1 + H_2)e_b \\
&= (-964.00 + 154.97)(-3.375) \\
&\quad -(-723.00 - 174.00)(10.70) \\
&= 12{,}328.35 \text{ kip-in.}
\end{aligned}
$$

The couple of $M_{a\text{-}a}$ is:

$$N_{eq} = \frac{(2)(12{,}328.35)}{47.75} = 516.37 \text{ kips}$$

Figure 25 shows the resulting interface forces and beam shear and moment distributions. As can be seen in Figure 25, the maximum beam shear and moment are:

$V_u = 485.6$ kips
$M_u = 5{,}881$ kip-ft

A summary of Example 2 results are shown in Table 2. Upon reviewing these results, three primary observations can be made.

1. The maximum beam shear occurs within the connection region in load cases 1 and 2 and outside of the

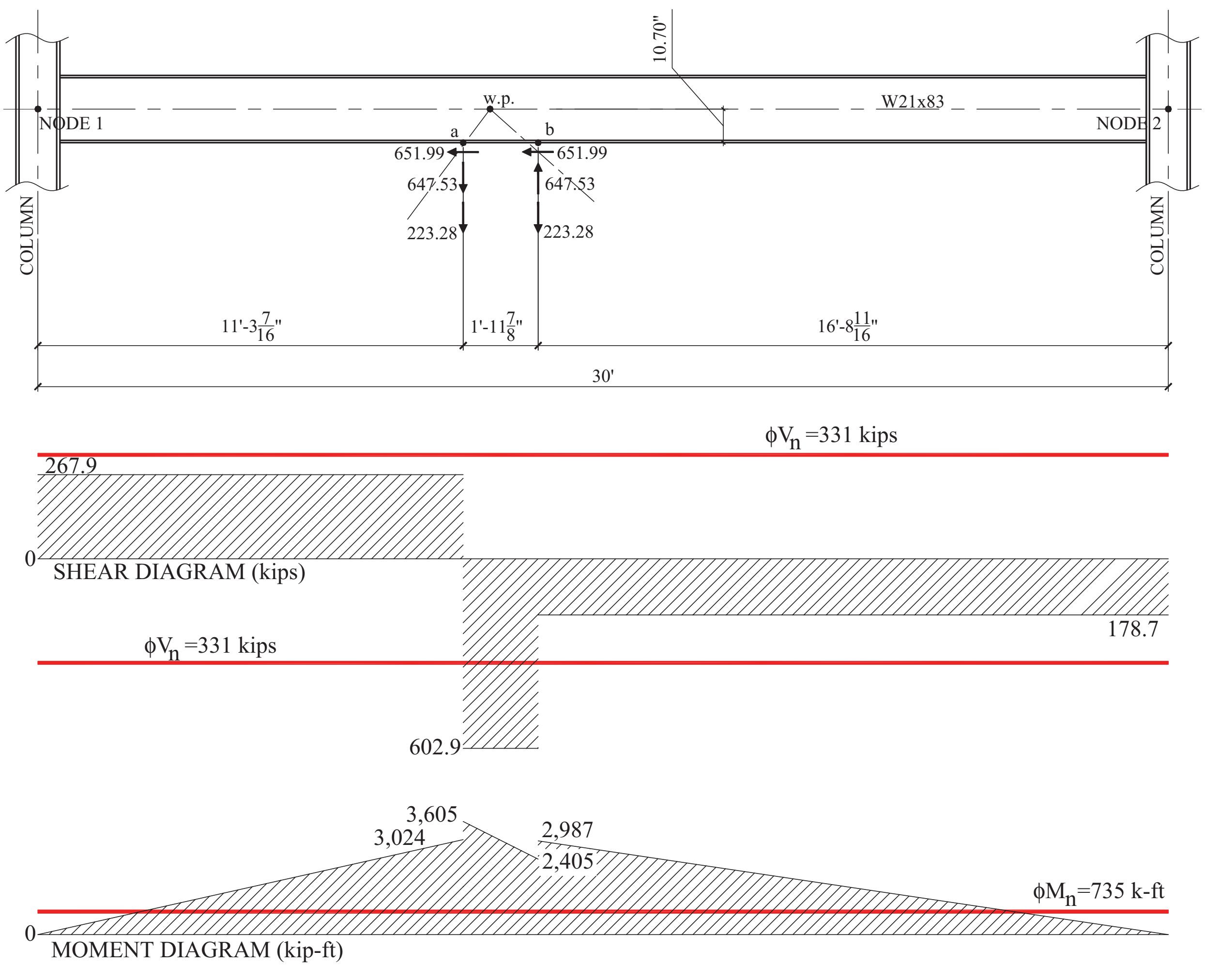


Fig. 23. Beam shear and moment distribution for load case 2.

Table 2. Beam Shears and Moments: Summary of Example 2 Results											
			Connection Isolated from Frame				Considering Frame Beam Span and Work Point Location				V_u,max [Within (W) or outside (O) connection region]
Load Case	ϕV_n (kips)	ϕM_n (k-ft)	$V_{u,\max}$ (kips)	$M_{u,\max}$ (k-ft)	$\frac{V_{u,\max}}{\phi V_n}$	$\frac{M_{u,\max}}{\phi M_n}$	$V_{u,\max}$ (kips)	$M_{u,\max}$ (k-ft)	$\frac{V_{u,\max}}{\phi V_n}$	$\frac{M_{u,\max}}{\phi M_n}$	
1	331	735	301.4	299.8	0.911	0.408	301.4	299.8	0.911	0.408	W
2	331	735	647.5	644.1	1.96	0.876	602.9	3,605	1.82	4.90	W
3	331	735	516.4	513.7	1.56	0.699	485.6	5,881	1.47	8.00	O

connection region in load case 3. Thus, if the span of the beam and the location of the work point are not considered when evaluating the beam shear, the maximum beam shear would not be captured for load case 3.

2. The algebraic sum of the vertical components of the brace forces is zero for load case 1. For load cases 2 and 3, the algebraic sums of the vertical components of the brace forces are −446.6 kips and −809.0 kips, respectively. Thus, the vertical components of the brace forces for load cases 2 and 3 are unbalanced. For load case 2, if the maximum beam shear is determined assuming the connection is isolated from the frame, the maximum beam shear is determined to be 647.5 kips. When the span of the beam and the location of the work point are considered for load case 2, the maximum beam shear is determined to be 602.9 kips, an overestimation of approximately 7.3%. For load case 3, if the maximum beam shear is determined assuming the connection is isolated from the frame, the maximum beam shear is determined to be 516.4 kips. When the span of the beam and the location of the work point are considered for load case 3, the maximum beam shear is determined to be 485.6 kips (and is located outside of the connection region), an overestimation of approximately 6.3%. For this example, the maximum beam shear is overestimated by 6.3% to 7.3%. Under different geometry and loading, it's quite possible to significantly underestimate or overestimate the maximum beam shear when the connection is evaluated as if it is isolated from the frame.

3. The beam moment is significantly underestimated when the connection is evaluated as if it is isolated from the beam when the vertical components of the

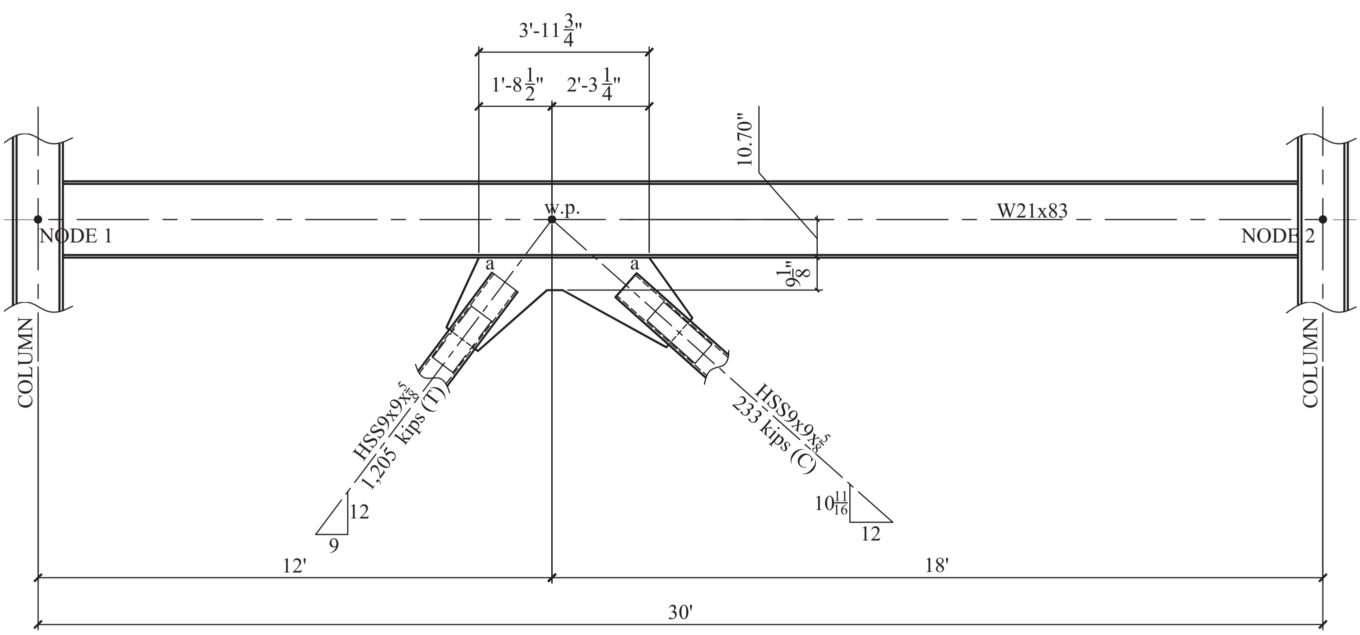


Fig. 24. Geometry and brace forces for Example 2, load case 3.

brace forces are unbalanced. Referring to Table 2, the ratios of available flexural strength to required flexural strength for load cases 2 and 3 are 0.20 and 0.12, respectively. Thus, the actual beam moment demands for load cases 2 and 3 are 4.90 and 8.00 times larger, respectively, than what would be determined if the span of the beam and location of the work point is not considered.

It is important to reiterate that the beam shears and moments calculated for this example are based on the brace forces only. Load effects from other types of loads (e.g., dead, live, etc.) must be superimposed to get the total shear and moment demands on the beam. In almost all cases, the additional loads will increase the maximum beam shear and moments beyond those imparted to the beam by the brace forces alone.

6. FINAL BEAM SIZE SELECTION

As demonstrated in the previous discussions, it is important to include brace force effects when making final beam size selections. To account for the brace force effects, the geometry of the connection must be known in order to calculate the gusset-to-beam interface forces. Typically, the connection geometry is not known at the time final beam size selection is made and, therefore, can be problematic. This is especially problematic when connection design is delegated to a contractor that is not the engineer-of-record for the design of the structure. To address this issue, the authors recommend a rule of thumb for accounting for the brace force effects.

To approximate the brace force effects on the beam, assume that the length of the gusset is approximately one-sixth of the beam span, and assume that the depth of the beam, d_b, in inches, is 75% of the span of the beam in feet.

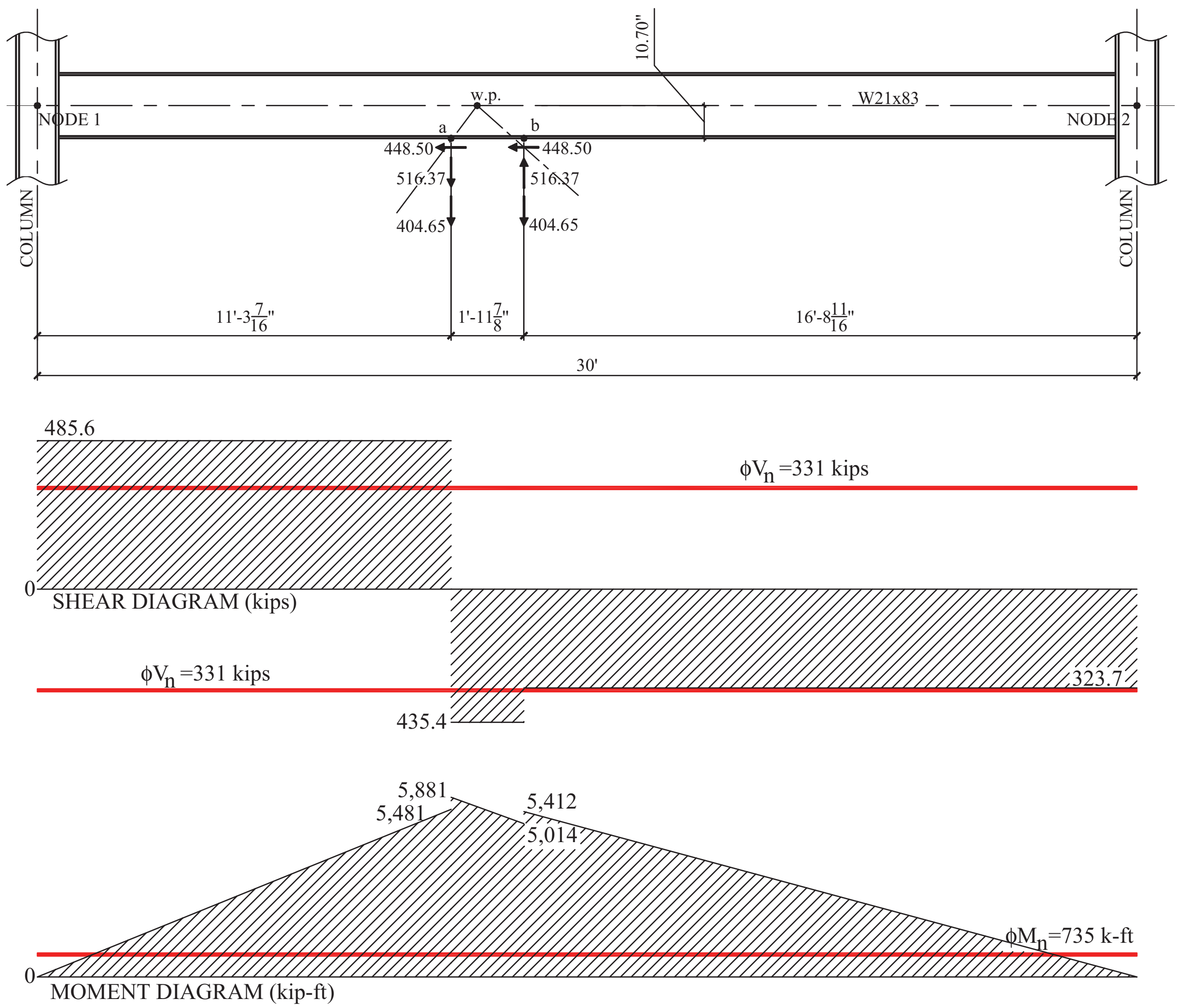


Fig. 25. Beam shear and moment distribution for load case 3.

Thus, the approximate value for e_b can be taken as one-half of the approximated beam depth. These approximations for a trial beam size are given in Equations 15 and 16. These approximations are ratios averaged from 20 different chevron brace connections taken from real connections designed by the authors over several years. The 20 different chevron connections were taken from a mix of different types of projects with varying types of braces, bevels, and brace forces.

$$L_{g,app} = \frac{L}{6} \tag{15}$$

$$e_{b,app}(\text{in.}) = 0.375(\text{span of the beam, ft}) \tag{16}$$

With the length of the gusset and e_b approximated, the moment acting at the gusset-to-beam interface can be conservatively estimated using Equation 17. Equation 17 contains the term with the horizontal components of the brace forces given in Equation 4. The couple of the moment acting on the gusset-to-beam interface can be estimated by dividing Equation 17 by the approximated gusset length. Equation 18 is the simplified expression for the approximated couple. The couples are placed at the centroids of the two half gusset bodies (i.e, at $L_{g,app}/4$ in from each gusset edge; $L_g/2$ apart). The direction of the couple should be considered to act in each direction to capture the "worst case" effect when combining the brace force effects with other types of loads (e.g., dead, live, wind load, etc.).

$$M_{a-a,app} = (H_1 + H_2)e_{b,app} \tag{17}$$

$$\begin{aligned} N_{eq,app} &= \pm\frac{2M_{a-a,app}}{L_{g,app}} \qquad (18) \\ &= \pm\frac{(2)(6)(H_1+H_2)e_{b,app}}{L} \\ &= \pm\frac{12(H_1+H_2)(0.375L)}{(12\text{ in./ft})L} \\ N_{eq,app} &= \pm 0.375(H_1+H_2) \end{aligned}$$

Referring to Equation 4, the moment at the gusset-to-beam interface, $M_{a\text{-}a}$, has two terms; the first term is a function of Δ and the second term is a function of e_b. The proposed method presented here for approximating the moment at the gusset-to-beam interface does not consider any potential vertical misalignment of the work point with the centroid of the gusset interface. That is, the first term of Equation 4, $(V_1+V_2)\Delta$, is not accounted for in the approximation. Upon close examination of Equation 4, it can be seen that the two terms may be the sum of the two terms or the difference of the two terms. Each term has the possibility of being positive or negative. When the signs of each parameter are such that the moment is the difference between the two terms, the approximated moment will be overestimated. When the signs of each parameter are such that the two terms are additive, the approximated moment will be underestimated. This is not a significant concern. Generally, the Δ term is a relatively small percentage of the total moment acting at the gusset interface. Additionally, the approximated gusset length given in Equation 15 will generally underestimate the actual gusset length resulting in a relatively larger couple. Thus, the rule of thumb presented here will provide a reasonably conservative estimate to be used for beam size selection.

7. DESIGN EXAMPLE

Example 3: Accounting for Brace Forces When Sizing Beam

Gravity Loads
D = 118 psf (includes all self-weight and all other superimposed dead loads)
L = 50 psf (non-reducible)
The tributary width of the frame beam is 28 ft.

Lateral Loads
The load effects on the braces from a wind load analysis are given in Figure 26. The brace forces given are LRFD loads and are used with load case 6 shown below.

Load Combinations
Evaluate only load combinations 2 and 4 from ASCE 7-10 (ASCE, 2010), as given below.
Load case 2: $1.2D + 1.6L$
Load case 4: $1.2D + 0.5L + 1.0W$ (note that L is less than 100 psf)

Deflection Limits for Frame Beam (gravity)
D+L: L/240
L: L/360
When checking deflection, assume that the clear span of the beam is from column centerline to column centerline.

Problem Statement
A partial elevation of the braced frame is shown in Figure 26. As can be seen in the figure, the geometry and brace forces are given. The brace forces shown are load effects from a wind analysis.

1. Calculate the design gravity load on the beam for load cases 2 and 4 given previously.
2. Make a beam selection neglecting the load effects of the brace forces acting at the gusset-to-beam interfaces:
   a. Provide a beam size that satisfies the strength, deflection and drift requirements given above. Do not include the effect of the brace force distributions

acting at the gusset interfaces, and assume the beam spans from column to column (i.e., the braces are not present to carry gravity load effects)

b. Calculate the brace force distributions at section *a-a* for the braces above and below the frame beam.

c. Draw the beam shear and moment diagrams that include both the LRFD gravity loads and forces acting on the beam imparted by the brace force distributions.

d. Compare the required beam shears and moments to the available beam shears and moments obtained in parts b and c.

3. Make a beam selection including the load effects of the brace forces acting at the gusset-to-beam interfaces:

   a. Determine a trial beam size that satisfies the strength, deflection and drift requirements given above. Include the effect of the brace force distributions using the rule of thumb recommended previously.

   b. Using the trial beam size selected in part 3a, calculate the brace force distributions at section *a-a* for the braces above and below the frame beam.

   c. Draw the beam shear and moment diagrams that include both the LRFD gravity loads and forces acting on the beam imparted by the brace force distributions.

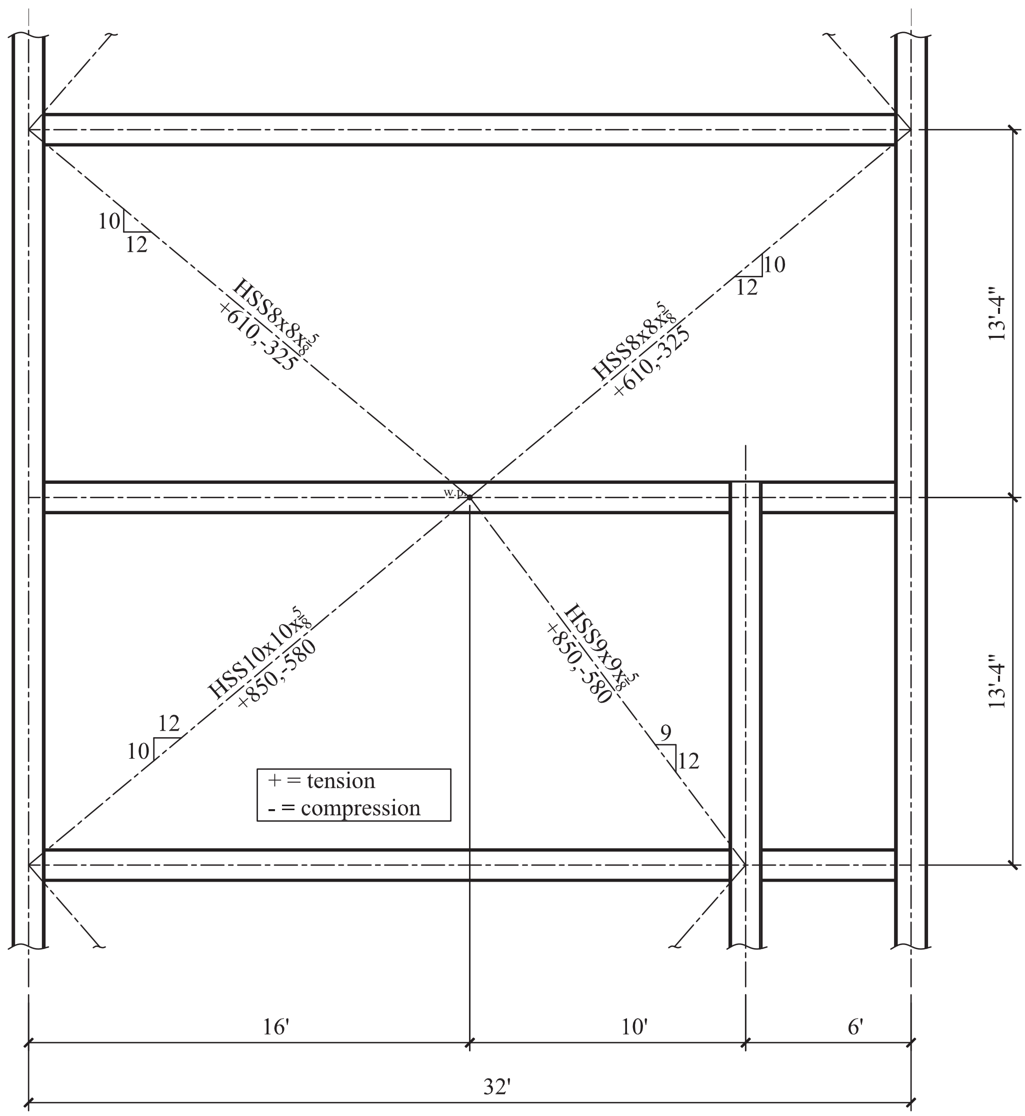


Fig. 26. Partial frame elevation for Example 3. Tributary width of frame beam is 28 ft.

d. Compare the required beam shear and moment to the available beam shear and moment

4. Assume that a connection designer is faced with a design scenario such as that shown later in Figure 30 where the required beam shear and moment exceed the available beam shear and moment. Calculate the required web doubler thickness and length of the web doubler for the beam and loading shown in Figure 30 (i.e., the beam used in part 2 of this problem). Evaluate the web doubler for each of the following two load distribution conditions:

   a. Using the resultant forces method.

   b. Using the distributed forces method.

Note that the available beam moment is also exceeded in this scenario which should be addressed in some manner. However, this issue is not covered here.

Use the brace force load case shown in Figure 27. In the following solution, the authors have established gusset plate geometry based on the trial e_b values established for each part of the problem.

Example 3: Solution

Part 1. *Calculate design beam gravity load.*

It is given in the problem statement that the tributary width of the frame is 28 ft. The design gravity loads for load cases 2 and 4 are:

Load case 2

$$w_D = \frac{(118 \text{ psf})(28 \text{ ft})}{1{,}000 \text{ lb/kip}} = 3.3 \text{ k/ft}$$

$$w_L = \frac{(50 \text{ psf})(28 \text{ ft})}{1{,}000 \text{ lb/kip}} = 1.4 \text{ k/ft}$$

$$w_{u,2} = 1.2D + 1.6L = (1.2)(3.3) + (1.6)(1.4)$$
$$= 6.20 \text{ k/ft}$$

Load case 4

$$w_D = \frac{(118 \text{ psf})(28 \text{ ft})}{1{,}000 \text{ lb/kip}} = 3.3 \text{ k/ft}$$

$$w_L = \frac{(50 \text{ psf})(28 \text{ ft})}{1{,}000 \text{ lb/kip}} = 1.4 \text{ k/ft}$$

$$= 1.2D + 0.5L = (1.2)(3.3) + (0.5)(1.4)$$
$$w_{u,4} = 4.66 \text{ k/ft}$$

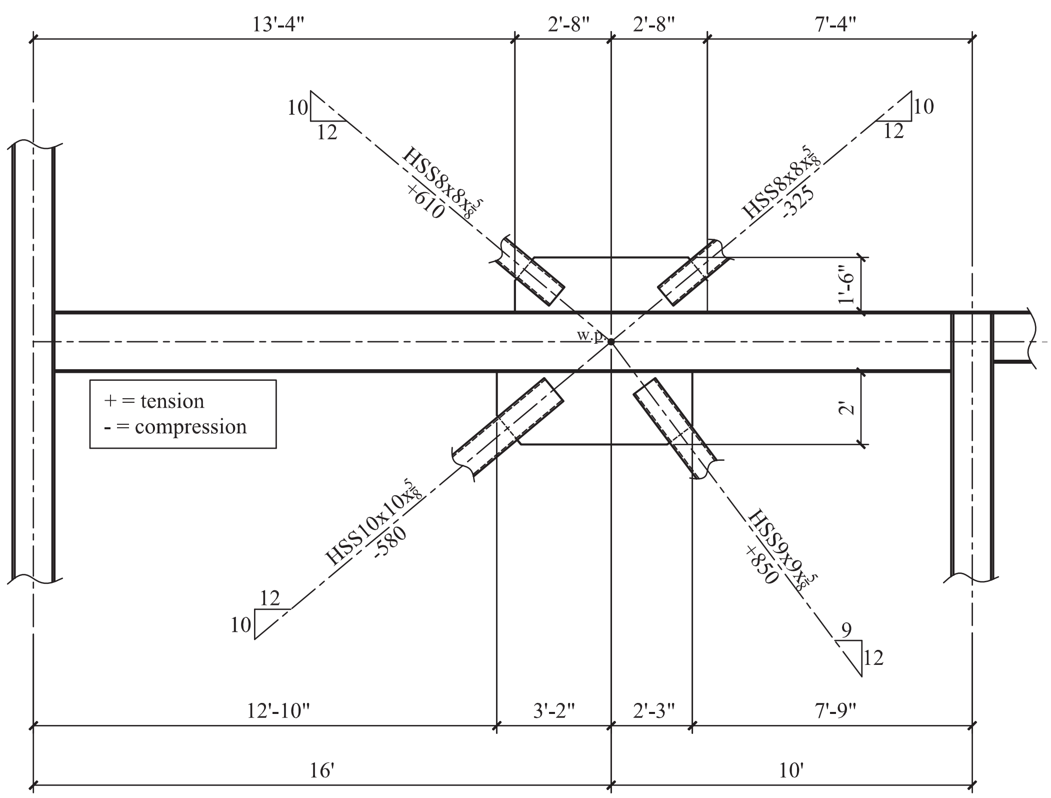


Fig. 27. Connection geometry and brace forces for Example 3.

Part 2a. *Size beam for load determined in part 1; include deflection check. Do not include brace forces.*

The required beam shears and moments are:

$$V_{u,2}=\frac{w_{u,2}L}{2}=\frac{(6.20\text{ k/ft})(26\text{ ft})}{2}$$
$$=80.6\text{ kips}$$

$$M_{u,2}=\frac{w_{u,2}L^2}{8}=\frac{(6.20\text{ k/ft})(26\text{ ft})^2}{8}$$
$$=524\text{ k-ft}$$

$$V_{u,4}=\frac{w_{u,2}L}{2}=\frac{(4.66\text{ k/ft})(26\text{ ft})}{2}$$
$$=60.6\text{ kips}$$

$$M_{u,4}=\frac{w_{u,2}L^2}{8}=\frac{(4.66\text{ k/ft})(26\text{ ft})^2}{8}$$
$$=394\text{ k-ft}$$

Load case 2 governs the design for strength. The plastic section modulus, Z, required to resist $M_{u,2}$ is:

$$Z_{req}\geq\frac{(524\text{ k-ft})(12\text{ in./ft})}{(0.9)(50\text{ ksi})}=140\text{ in.}^3$$

The moment of inertia required for the deflection limits is:

$$\delta_i=\frac{5w_iL^4}{384EI_i}\rightarrow I_i\geq\frac{5w_iL^4}{384E\delta_i}$$

$$I_{D+L}\geq\frac{(5)\left(\dfrac{3.3+1.4}{12}\right)[(26)(12)]^4}{(384)(29{,}000)\left(\dfrac{(26)(12)}{240}\right)}=1{,}282\text{ in.}^4$$

$$I_L\geq\frac{(5)\left(\dfrac{1.4}{12}\right)[(26)(12)]^4}{(384)(29{,}000)\left(\dfrac{(26)(12)}{360}\right)}=573\text{ in.}^4$$

Therefore, $I > 1{,}282$ in.^{4}

Thus, for a trial beam size, select a beam that satisfies the following requirements.

$$V_u=80.6\text{ kips}$$
$$M_u=524\text{ k-ft}$$
$$Z_{req}\geq 140\text{ in.}^3$$
$$I\geq 1{,}282\text{ in.}^4$$

Try a W21×83 beam.

$\phi M_n=735\text{ k-ft}>M_u=524\text{ k-ft}$ **o.k.**

$\phi V_n=331\text{ k}>V_u=80.6\text{ k}$ **o.k.**

$I=1{,}830\text{ in.}^4>I=1{,}282\text{ in.}^4$ **o.k.**

Part 2b. *Calculate brace force distributions on sections* a-a *at top and bottom of beam.*

With the size of the beam known, the force distributions at the gusset interfaces can be calculated using the procedure presented previously in this paper. Figure 28 shows the beam shear and moment diagrams for gravity load. Note that load case 4 is the load combination with wind. Thus, the diagrams shown in Figure 28 are based on load case 4 (i.e., 1.2D + 0.5L).

Part 2c. *Draw beam shear and moment diagrams for braces force determined on part 2b.*

Figure 27 shows the geometry and brace forces. From the data given in the figure, the forces acting on sections *a-a* at the top and bottom of the beam can be determined.

Note that the forces acting at the top gusset-to-beam interface are calculated using the analysis procedure and sign convention presented in Section 1, Figure 3, of this paper, assuming that the free body diagrams shown in Figure 3 are rotated 180 degrees about an axis perpendicular to the work point.

Section* a-a*—top of beam:

$$H_1=249.67\text{ kips}$$
$$H_2=468.61\text{ kips}$$
$$V_1=208.06\text{ kips}$$
$$V_2=-390.51\text{ kips}$$

Forces acting on section* a-a*:

$$H_{a-a}=-(H_1+H_2)$$
$$=-(249.67+468.61)$$
$$=-718.29\text{ kips}$$
$$V_{a-a}=-(V_1+V_2)=-(208.06-390.51)$$
$$=182.45\text{ kips}$$

$$\Delta=\frac{1}{2}(L_1-L_2)=0.5(32.0-32.0)$$
$$=0$$
$$e_b=10.70\text{ in.}$$
$$M_{a-a}=(V_1+V_2)\Delta-(H_1+H_2)e_b$$
$$=(208.06-390.51)(0)$$
$$-(249.67+468.61)(10.70)$$
$$=-7{,}685.67\text{ kip-in.}$$

The couple of $M_{a\text{-}a}$ is:

$$N_{eq}=\pm\frac{(2)(7{,}182.67)}{64.0}=240.18\text{ kips}$$

Section* a-a *– bottom of beam:

$$H_1 = 445.57 \text{ kips}$$
$$H_2 = 510.00 \text{ kips}$$
$$V_1 = 371.31 \text{ kips}$$
$$V_2 = -680.00 \text{ kips}$$

Forces acting on section* a-a*:

$$\begin{aligned} H_{a-a} &= -(H_1 + H_2) = -(445.57 + 510.00) \\ &= -955.57 \text{ kips} \\ V_{a-a} &= -(V_1 + V_2) = -(371.31 - 680.00) \\ &= 308.69 \text{ kips} \\ \Delta &= \frac{1}{2}(L_1 - L_2) = 0.5(38.0 - 27.0) \\ & 5.50 \text{ in.} \\ e_b &= 10.70 \text{ in.} \\ M_{a-a} &= (V_1 + V_2)\Delta - (H_1 + H_2)e_b \\ &= (371.31 - 680.00)(5.50) \\ &\quad -(445.57 + 510.00)(10.70) \\ &= -11{,}922.39 \text{ kip-in.} \end{aligned}$$

The couple of $M_{a\text{-}a}$ is:

$$N_{eq} = \pm\frac{(2)(11{,}922.39)}{65.0} = 366.84 \text{ kips}$$

Figure 29 shows the force distribution acting at the gusset-to-beam interfaces as determined in the preceding calculations, along with the beam shear and moment diagrams generated by the brace forces acting on the beam. Figure 30 shows the beam shear and moment diagrams for all of the load effects given in load case 4 (the combination of the load effects shown in Figures 28 and 29).

Part 2d. *Compare beam shear and moment diagrams generated for parts 2b and 2c.*

Referring to Figure 28, if the beam is evaluated for gravity load effects only, the beam has sufficient shear and moment strength. This should be no surprise considering that the beam size was selected based on required gravity load and deflection considerations. However, the brace forces do have an impact on the forces imparted to the beam.

Figure 29 shows the beam shear and moment diagrams for the brace force effects only. Note in the force distributions acting at the gusset-to-beam interfaces that the couples, N_{eq}, of the moments, $M_{a\text{-}a}$, acting at the top and bottom of the beam act in the same direction. However, the normal forces acting at the top and bottom of the beam due to the unbalanced vertical components of the brace forces, $\frac{1}{2}V_{a\text{-}a}$, act in the opposite direction. This is always true for the typical case where one line of braces is in tension while the other line of braces is in compression. Thus, the moments acting at the interfaces of a two-story X-braced frame accumulate,

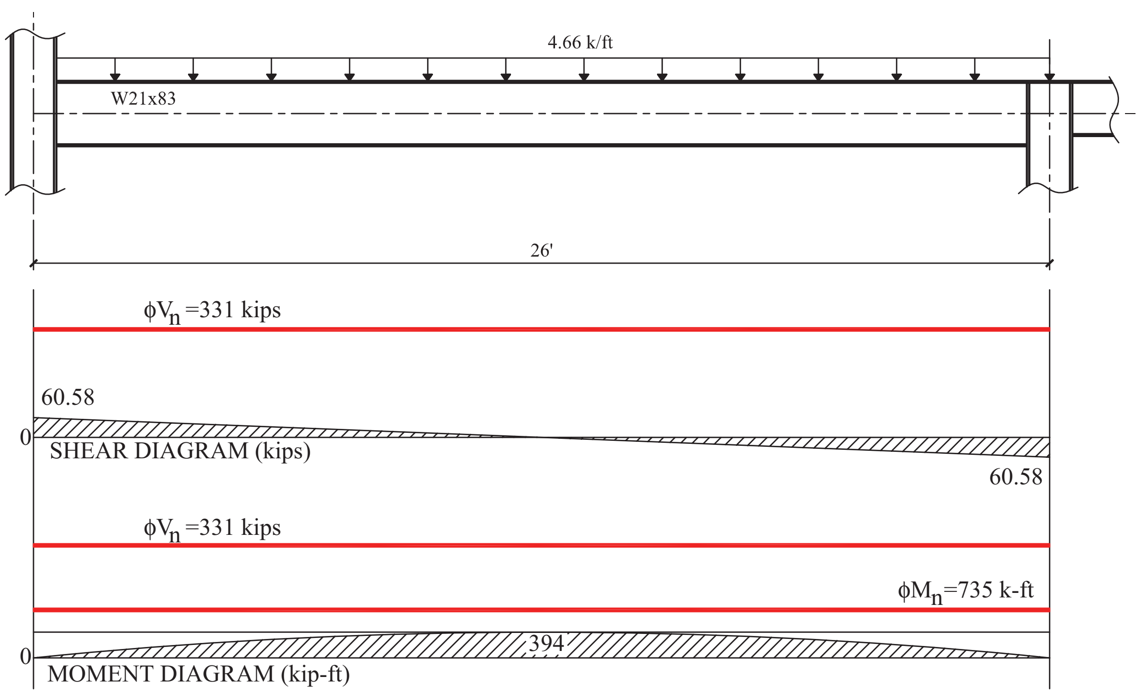


Fig. 28. Beam shear and moment for gravity: load case 4 for part 2 of Example 3.

while the unbalanced vertical component force at the top and bottom subtract in regard to the impact on the beam.

Referring to the shear and moment diagrams in Figure 29, considering the brace forces only, the required beam shear and moment strength is 592 kips and 1,389 k-ft, respectively. The available beam shear and moment strength is 331 kips and 735 k-ft, respectively. The beam is undersized for the brace forces alone with required strength to available strength ratios for shear and moment equal to 592/331 = 1.79 and 1,389/735 = 1.89, respectively. These ratios will be more severe when considering the gravity load effects in combination with the brace forces.

It is also worth noting that if the beam is evaluated for shear and moment as if the joint is isolated from the frame, the maximum beam shear and moment due to the brace force distribution would be determined to be 607 kips and 712 kip-ft, respectively (see Figure 31). This analysis gives a conservative value for the required shear and significantly underestimates the moment demand on the beam. This is another example of why the impact of chevron bracing on the frame beam should not be evaluated as if the joint is isolated from the frame when an unbalanced vertical force is present. The isolated approach is further complicated when the Δ parameter is non-zero as with this example problem. Referring to Figure 31, note the $\Sigma V\Delta$ moment shown in the loading diagram; this couple needs to be included in order to close the moment diagram. However, the $\Sigma V\Delta$ moment must be neglected in order to satisfy static equilibrium within the isolated free body diagram; this is a further complication associated with incorrectly evaluating the beam as if the joint is isolated from the frame.

Referring to Figure 30, it can be seen that when all load effects from load case 4 are considered, and the span of the beam and location of the work point are considered, the required beam shear and moment is 585 kips and 1,748 kip-ft, respectively. The W21×83 beam has available shear and

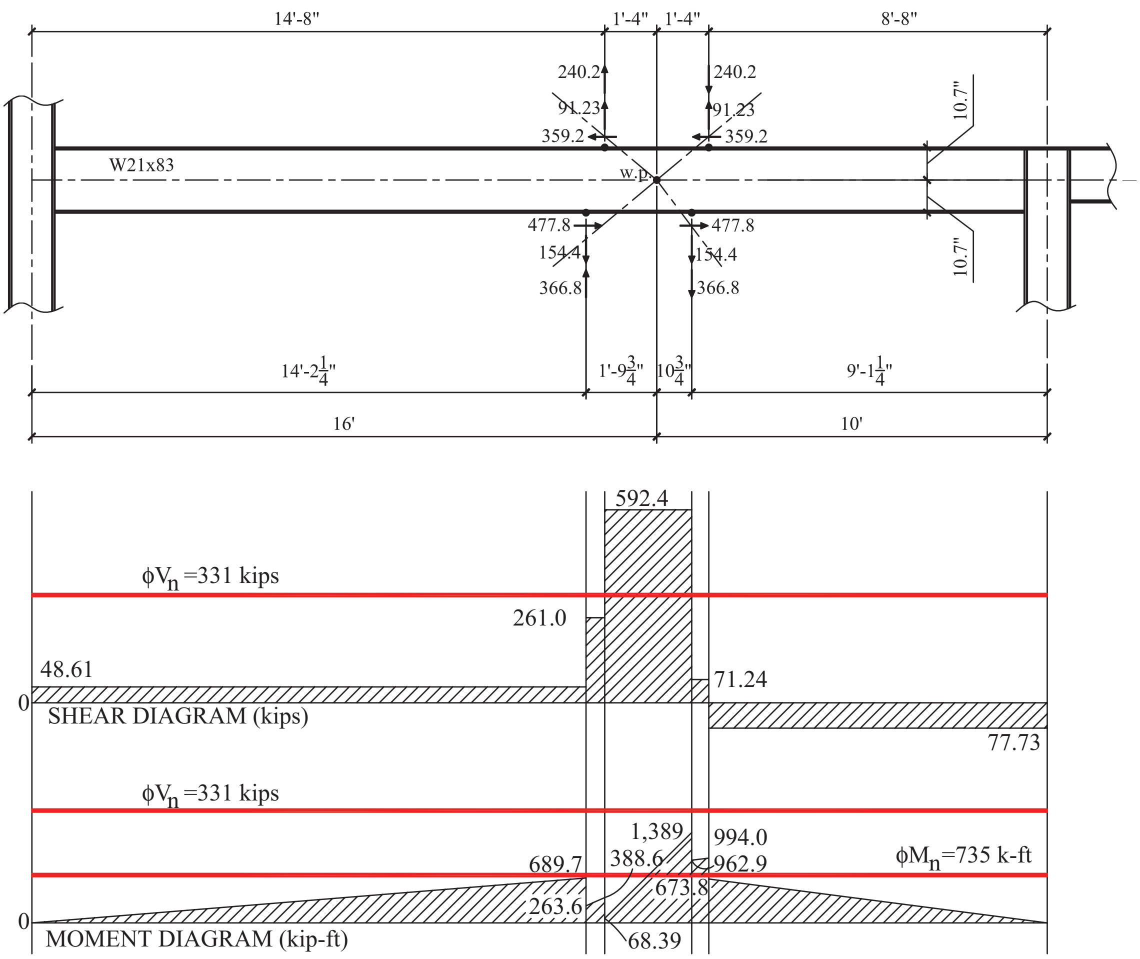


Fig. 29. Forces and moments acting at gusset-beam interfaces and beam shear and moment from brace forces: load case 4 for part 2 of Example 3.

moment strength equal to 331 kips and 735 k-ft, respectively. Thus, the beam is inadequate and has required strength to available strength ratios for shear and bending equal to 585/331 = 1.77 and 1,748/745 = 2.35, respectively. At the connection design stage, modifications to the beam would be required to increase both the shear and moment strength of the beam. Web doubler plates would be required to increase the beam shear strength. Cover plates, or some other manner of reinforcement would be required to increase the beam moment strength.

Based on the observations made here in part 2d, it is evident that the effect of the brace forces should be included when sizing the frame beam. Furthermore, the span of the beam and the location of the work point along the span of the beam should be considered when evaluating the frame beam.

Part 3a. *Select a trial beam size based on strength and deflection; include effects of brace forces using the rule of thumb presented previously.*

To obtain a trial beam size, the length of the gusset (L_g) and one-half the depth of the beam (e_b) are approximated using Equations 15 and 16. It was previously calculated that the gravity design load using load case 4 is 4.66 k-ft, and the beam shear and moment distribution is given in Figure 28. To determine the beam shear and moment distribution resulting from the brace forces, the moments and normal forces acting on the gusset-to-beam interfaces need to be approximated.

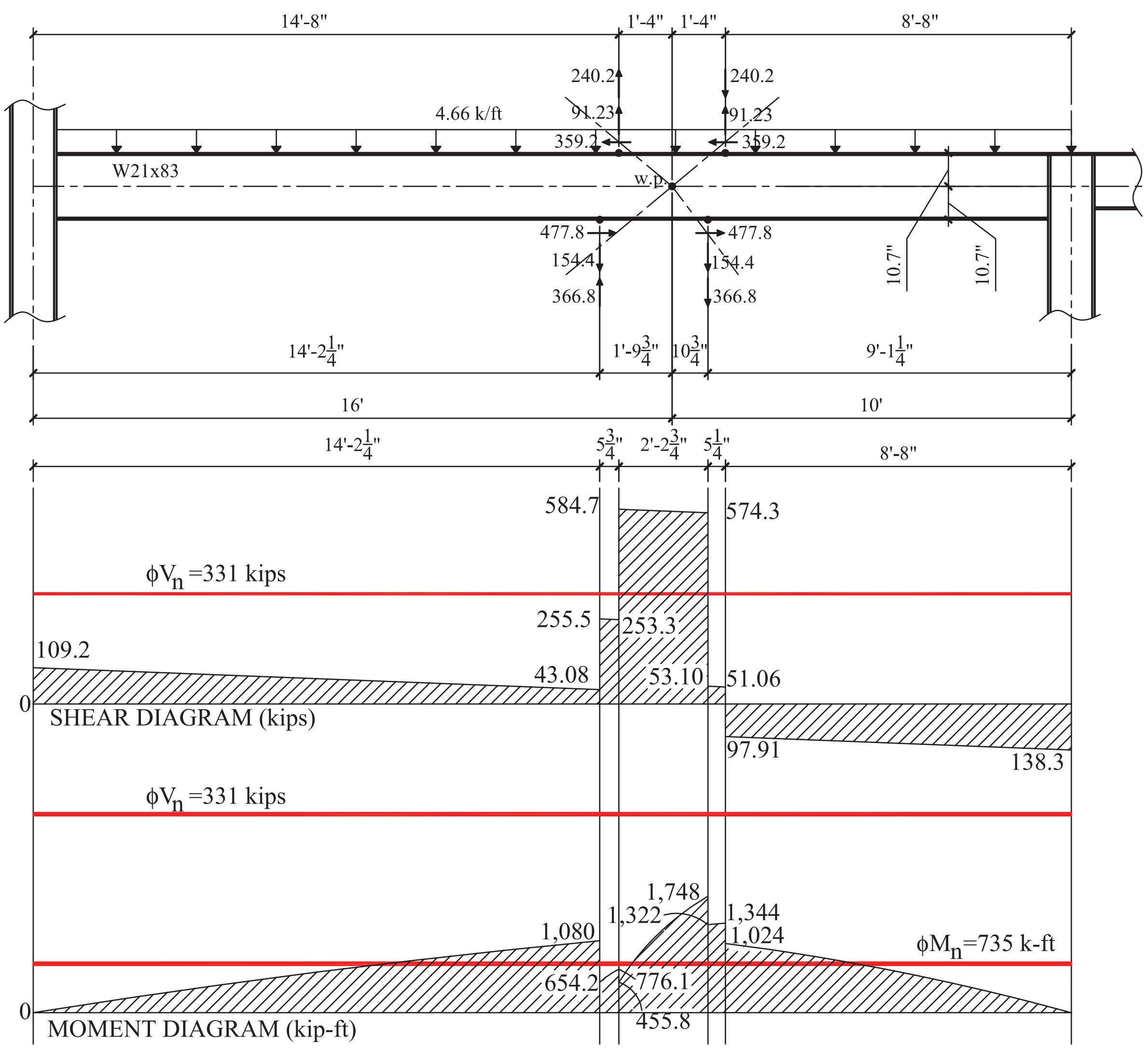


Fig. 30. Forces and moments acting at gusset-beam interfaces and beam shear and moment from gravity load plus brace forces: load case 4 for part 2 of Example 3.

Section* a-a—*top of beam:

$$H_1 = 249.67 \text{ kips}$$
$$H_2 = 468.61 \text{ kips}$$
$$V_1 = 208.06 \text{ kips}$$
$$V_2 = -390.51 \text{ kips}$$

The length of the gusset is approximated as:

$$L_{g,app} = \frac{L}{6} = \frac{(26 \text{ ft})(12 \text{ in./ft})}{6}$$
$$= 52.0 \text{ in.}$$

One-half of the beam depth, e_b, is approximated as:

$$e_{b,app} = (0.375)(26 \text{ ft}) = 9.75 \text{ in.}$$

The approximated moment acting on section *a-a* at the top of the beam is:

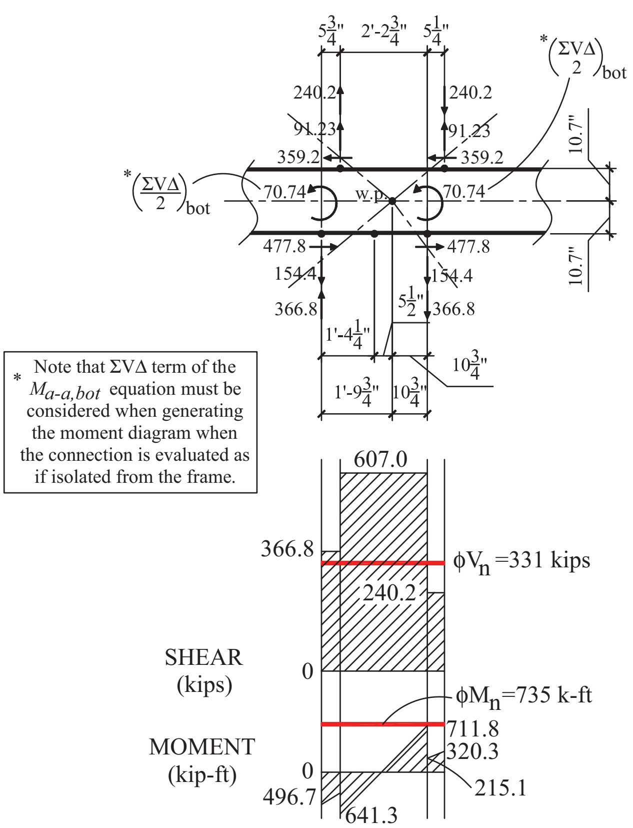


Fig. 31. Beam shear and bending if connection is evaluated as if isolated from the frame. Effect of brace forces only; part 2 of Example 3.

$$M_{a-a,app} = (H_1 + H_2)e_{b,app}$$
$$= (249.67 + 468.61)(9.75)$$
$$= 7{,}003.23 \text{ k-in.}$$
$$N_{eq,app} = \pm 0.375(249.67 + 468.61)$$
$$= \pm 269.36 \text{ kips}$$

The horizontal force acting on section *a-a* is:

$$H_{a-a} = -(249.67 + 468.61)$$
$$= -718.29 \text{ kips}$$

For each half gusset body, the horizontal force is $-718.29/2 = -359.15$ kips.

The normal force acting on section *a-a* from the unbalanced vertical force is:

$$V_{a-a} = -(V_1 + V_2) = -(208.06 - 390.51)$$
$$= 182.45 \text{ kips}$$

For each half gusset body, the normal force is $182.45/2 = 91.23$ kips.

Section* a-a—*bottom of beam:

$$H_1 = 445.57 \text{ kips}$$
$$H_2 = 510.00 \text{ kips}$$
$$V_1 = 371.31 \text{ kips}$$
$$V_2 = -680.00 \text{ kips}$$

The length of the gusset is approximated as:

$$L_{g,app} = \frac{L}{6} = \frac{(26 \text{ ft})(12 \text{ in./ft})}{6} = 52.0 \text{ in.}$$

One-half of the beam depth, e_b, is approximated as:

$$e_{b,app} = (0.375)(26 \text{ ft}) = 9.750 \text{ in.}$$

The approximated moment acting on section *a-a* at the bottom of the beam is:

$$M_{a-a,app} = (H_1 + H_2)e_{b,app}$$
$$= (445.57 + 510.00)(9.75)$$
$$= 9{,}316.81 \text{ k-in.}$$
$$N_{eq,app} = \pm 0.375(445.57 + 510.00)$$
$$= \pm 358.34 \text{ kips}$$

The horizontal force acting on section *a-a* is:

$$H_{a-a} = -(445.57 + 510.00)$$
$$= -955.57 \text{ kips}$$

For each half gusset body, the horizontal force is $-955.57/2 = -477.79$ kips.

The normal force acting on section *a-a* from the unbalanced vertical force is:

$$V_{a-a} = -(V_1 + V_2) = -(371.31 - 680.00)$$
$$= 308.69 \text{ kips}$$

For each half gusset body, the normal force is $308.69/2 = 154.35$ kips.

With the forces acting on the top and bottom interfaces approximated, the loading diagram used to obtain a trial beam size can be generated. Figure 32 shows the loading diagram and the resulting beam shear and moment diagrams based on load case 4 loads. Note that the approximation does not take into account any potential Δ values that would be present in the final connection design. Therefore, the resultant forces acting on the gusset-to-beam interfaces are symmetrically placed about the work point location at a distance equal to $L_g/4$ to either side of the work point (i.e., the resultant forces are separated by a total distance of $L_g/2$).

Referring to required beam shears and moments shown in the diagrams in Figure 32, and considering the minimum moment of inertia given in the problem statement, the following are the design parameters for the beam selection.

$V_{u,max} = 604.2$ kips
$M_{u,max} = 1{,}729$ k-ft
$I_{min} = 1{,}282$ in.^{4}
Target beam depth, $d = (9.75 \text{ in.})(2) = 19.5$ in.

For this solution, two possible beam sizes will satisfy the design parameters. The properties of the two beams are shown in Table 3. The W21×201 is the lighter of the two beams, but the W18×211 has a depth closer to the approximated depth of 19.5 in. Given that the W18 is only a few pounds heavier than the W21, but has a depth closer to the approximated depth, the W18×211 is selected as the trial beam size. Note that selecting the W21×201 is an acceptable choice, if that is the preference of the designer.

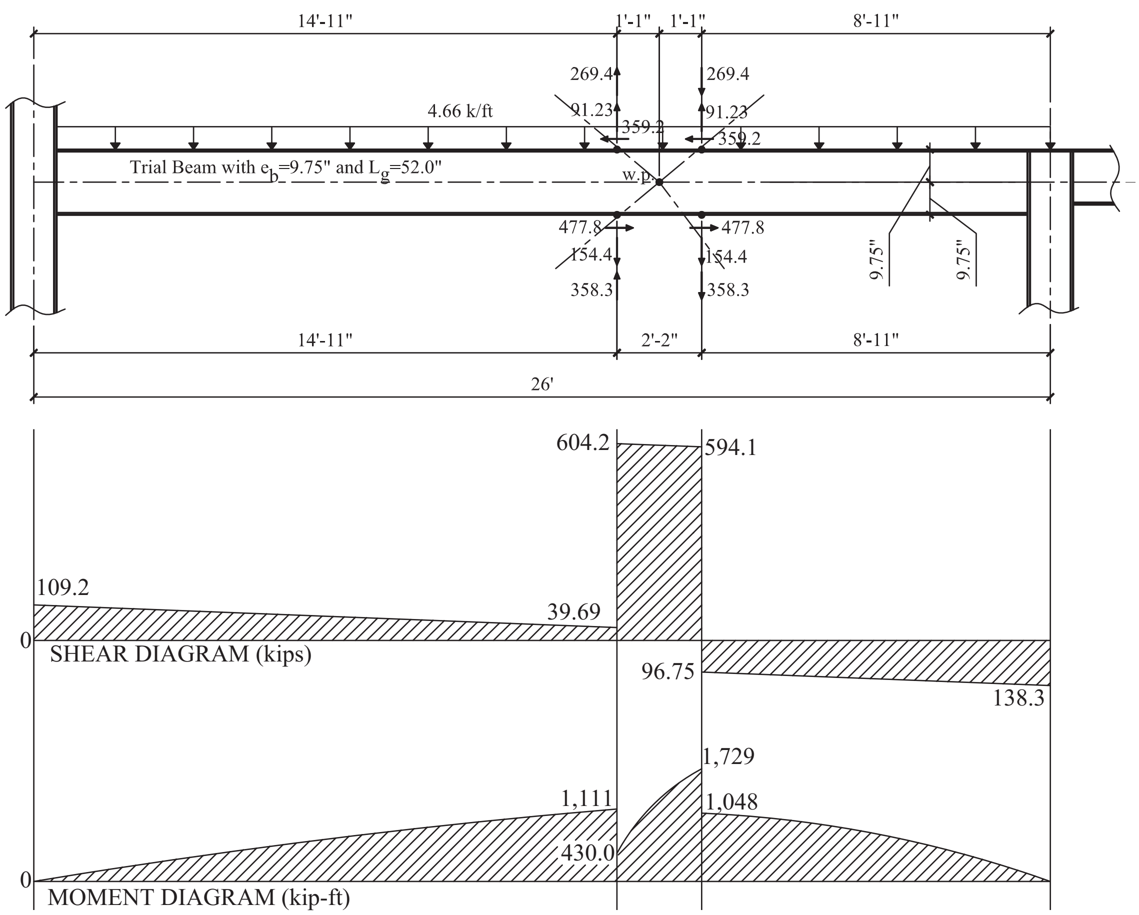


Fig. 32. Beam shear and bending for trial beam size: $L_g = 52.0$ *in.,* $e_b = 9.75$ *in.; part 3a of Example 3.*

Table 3. Possible Trial Beam Sizes for Part 3a of Example 3.				
Beam Size	ϕV_n (kips)	ϕM_n (k-ft)	I (in.^{4})	d (in.)
W21×201	629	1,990	5,310	23.0
W18×211	657	1,840	4,330	20.7

Try a W18×211.

Parts 3b and 3c. *Using the trial size selected in part 3a, calculate the brace connection force distribution on section* a-a. *Generate the beam shear and moment diagrams.*

A W18×211 has been selected for the frame beam. Knowing the loading, geometry and beam size, a gusseted brace connection can be designed. Figure 33 shows an elevation of the frame beam with the gusset geometry. Based on the geometry given in Figure 33, the brace force distributions at the gusset-to-beam interfaces can be calculated.

Section* a-a*—top of beam:

$$H_1 = 249.67 \text{ kips}$$
$$H_2 = 468.61 \text{ kips}$$
$$V_1 = 208.06 \text{ kips}$$
$$V_2 = -390.51 \text{ kips}$$

Forces acting on section* a-a*:

$$\begin{aligned} H_{a-a} &= -(H_1 + H_2) \\ &= -(249.67 + 468.61) \\ &= -718.29 \text{ kips} \end{aligned}$$

$$\begin{aligned} V_{a-a} &= -(V_1 + V_2) \\ &= -(208.06 - 390.51) \\ &= 182.45 \text{ kips} \end{aligned}$$

$$\begin{aligned} \Delta &= \frac{1}{2}(L_1 - L_2) = 0.5(33.0 - 33.0) \\ &= 0 \end{aligned}$$

$$e_b = 10.35 \text{ in.}$$

$$\begin{aligned} M_{a-a} &= (V_1 + V_2)\Delta - (H_1 + H_2)e_b \\ &= (208.06 - 390.51)(0) \\ &\quad -(249.67 + 468.61)(10.35) \\ &= -7{,}434.27 \text{ kip-in.} \end{aligned}$$

The couple of $M_{a\text{-}a}$ is:

$$N_{eq} = \pm\frac{(2)(7{,}434.27)}{66.0} = 225.28 \text{ kips}$$

Section* a-a*—bottom of beam:

$$H_1 = 445.57 \text{ kips}$$
$$H_2 = 510.00 \text{ kips}$$
$$V_1 = 371.31 \text{ kips}$$
$$V_2 = -680.00 \text{ kips}$$

Forces acting on section* a-a*:

$$\begin{aligned} H_{a-a} &= -(H_1 + H_2) = -(445.57 + 510.00) \\ &= -955.57 \text{ kips} \end{aligned}$$

$$\begin{aligned} V_{a-a} &= -(V_1 + V_2) = -(371.31 - 680.00) \\ &= 308.69 \text{ kips} \end{aligned}$$

$$\Delta = \frac{1}{2}(L_1 - L_2) = 0.5(38.5 - 27.5)$$
$$\Delta = 5.50 \text{ in.}$$
$$e_b = 10.35 \text{ in.}$$

$$\begin{aligned} M_{a-a} &= (V_1 + V_2)\Delta - (H_1 + H_2)e_b \\ &= (371.31 - 680.00)(5.50) \\ &\quad -(445.57 + 510.00)(10.35) \\ &= -11{,}587.94 \text{ kip-in.} \end{aligned}$$

The couple of $M_{a\text{-}a}$ is:

$$N_{eq} = \pm\frac{(2)(11{,}587.94)}{66.0} = 351.15 \text{ kips}$$

The force distributions at the gusset-to-beam interfaces are shown in Figure 34 along with the beam shear and moment diagrams resulting from load case 4.

Part 3d. *Compare required to available beam shear and moment strengths.*

The W18×211 is adequate for the required beam shear and bending. When the beam was selected based on gravity load effects only, the beam was found to be woefully inadequate for shear and moment. When the brace force distribution is considered in combination with the gravity load effects, a satisfactory beam is selected eliminating any need for web doubler plates, cover plates, or any other type of reinforcement. The method presented (rule of thumb) for approximating the moment at the gusset interface provides an adequate method for accounting for the effects of the brace forces during the beam selection process.

Parts 4a and 4b. *Determine the required beam web doubler for the shear distribution shown in Figure 30. Check assuming (a) resultant force distribution on section* a-a*, and (b) distributed uniform force distribution on section* a-a

Figure 30 shows the beam shear diagram using the resultant force method. Referring to Figure 30, the maximum shear is 584.7 kips and is constant over the region from 14 ft 8 in. to 16 ft 10¾ in. from the left support. The available shear strength of the beam is 331 kips. Therefore, a web doubler plate is required. The required web doubler thickness is:

$$\phi V_n = (1.0)(0.6)(50)(21.4)(0.515 + t_d) \geq 584.7$$
$$= 330.63 + 642 t_d \geq 584.7$$
$$t_d \geq \frac{584.7 - 330.63}{642} = 0.396 \text{ in.}$$

Use a ½ in. web doubler plate.

The web doubler plate must be within the region of the beam where the shear is 584.7 kips. Since the shear is constant over this region, the web doubler plate must be extended beyond this region a distance sufficient to get the load into the web doubler plate (see Figure 35). The shear required to be carried by the web doubler is 584.7 − 331 = 253.7 kips. Therefore, the shear on the horizontal edges of the web doubler plate is:

$$V_h = \frac{(253.7 \text{ kips})(26.75 \text{ in.})}{19.73 \text{ in.}} = 344.0 \text{ in.}$$

The length required to transfer the load into the doubler plate (i.e., develop the doubler plate) is the length, x (see Figure 35), of the ½-in.-thick web doubler plate required to develop the web doubler plate for a shear equal to one-half of the shear force acting on the horizontal edge of the web doubler plate and is calculated as shown below.

$$x = \frac{\left(\dfrac{344.0 \text{ in.}}{2}\right)}{(1.0)(0.60)(50 \text{ ksi})(0.5 \text{ in.})} = 11.46 \text{ in.}$$

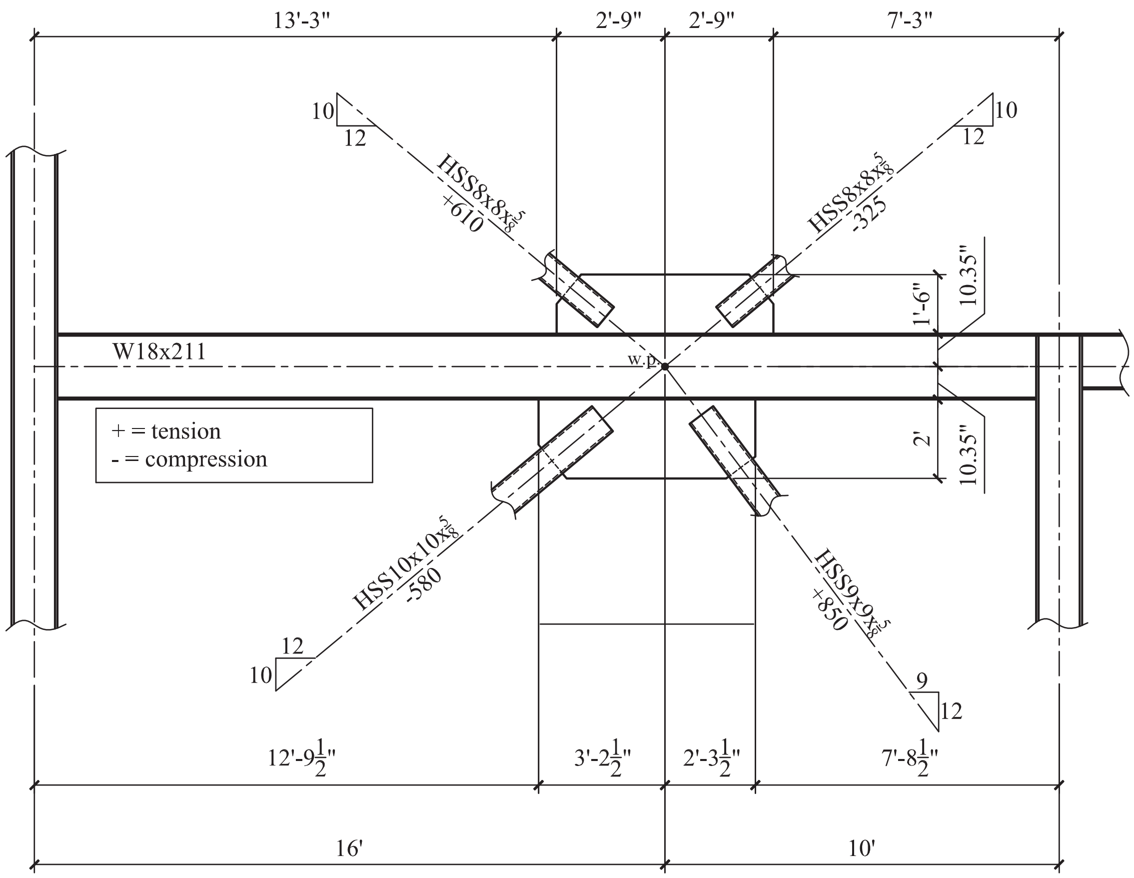


Fig. 33. Brace and gusset geometry for trial W18×211 *frame beam; part 3b of Example 3.*

The web doubler plate requires a development length of 11.5 in. to each side of the region of the beam where the beam's available shear strength is required (see Figure 35).

Figure 36 shows the beam shear diagram when the brace force distribution acting on the gusset-to-beam interfaces are uniformly distributed. Note that the maximum beam shear is 523.9 kips, compared to a maximum shear of 584.7 kips when the resultant force method is used (see Figure 30). As discussed previously, the difference in the maximum shear is due to the non-zero Δ parameter associated with the geometry of the bottom flange gusset. As can be seen in Figure 36, the beam's available shear strength is exceeded over a 2 ft 0-¾ in. portion of the beam starting at 14 ft 6 13⁄16 in. from the left support. A web doubler plate is required in this region. The required web doubler plate thickness is:

$$\phi V_n = (1.0)(0.6)(50)(21.4)(0.515 + t_d) \geq 523.9$$
$$= 330.63 + 642 t_d \geq 523.9$$
$$t_d \geq \frac{523.9 - 330.63}{642} = 0.301 \text{ in.}$$

Use a ⅜-in. web doubler plate.

Referring back to Figure 35, recall that the web doubler plate is required to be developed in order to get the load out of the beam and into the web doubler plate. Because the beam shear shown in Figure 30 is approximately constant over the region where the beam's available strength is exceeded, the web doubler plate needs to be developed outside of this region. However, when the distributed force method is used, the web doubler plate does not need to be developed outside

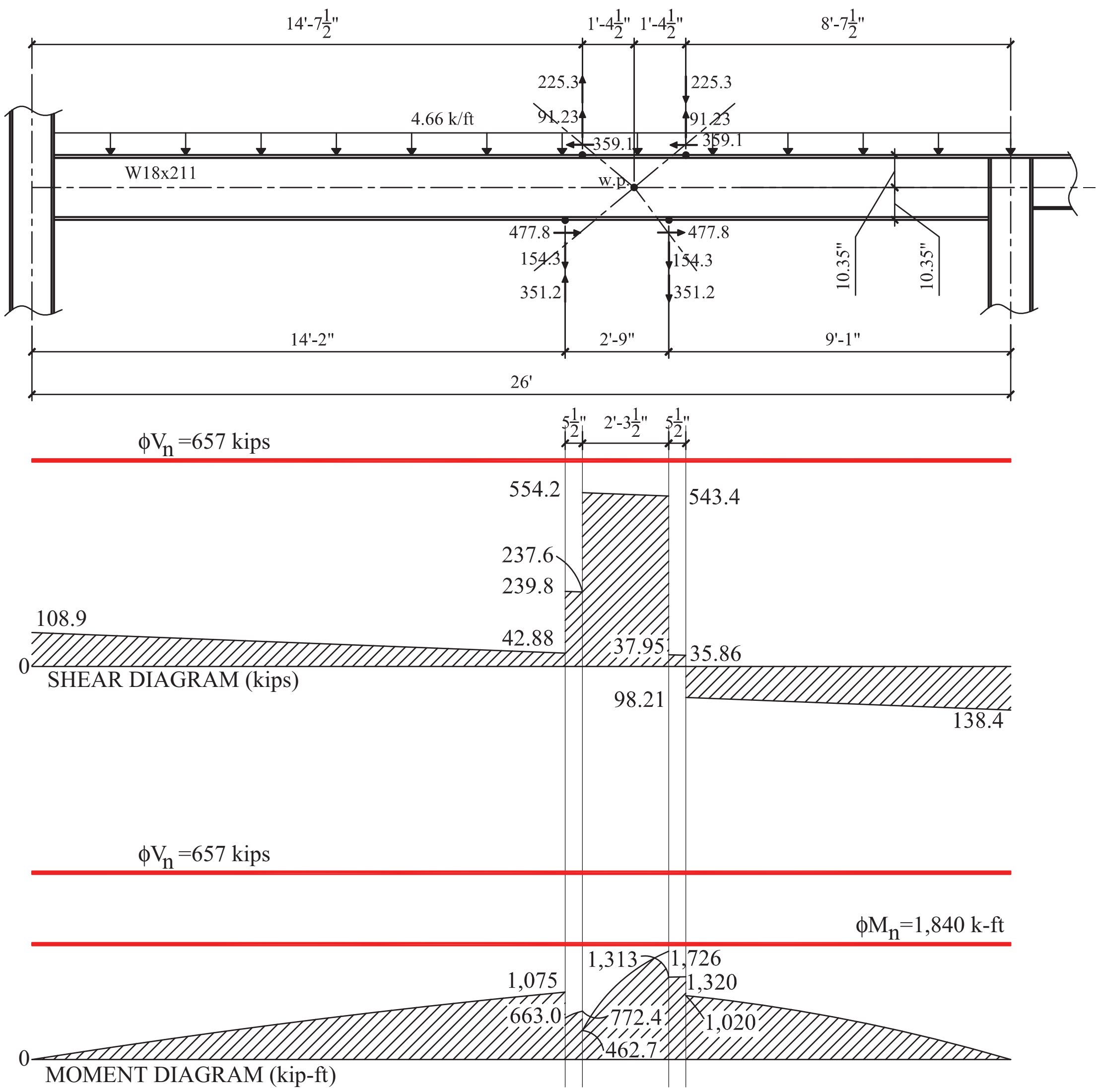


Fig. 34. Force distribution and beam shear and moment diagrams with W18×211 *frame beam; part 3c of Example 3.*

of the region where the beam's available shear strength is exceeded.

For this example, when the resultant load method is used, a ½-in. × 49.75-in. web doubler plate is required. When the distributed load method is used, a ⅜-in. × 24.75-in. web doubler plate is used. Using the distributed load method will always result in a more economical web doubler plate, relative to the resultant load method. Considering that the distributed load method is a more accurate analysis method, it is recommended that the distributed load method be used when evaluating the need for web doubler plates. It's worth noting that if the effects of the brace forces on the frame beam are appropriately considered, an appropriate beam size will be selected (as in part 3 of this example), making the discussion of web doubler plates moot.

SUMMARY

1. A method for generating an admissible force distribution in a chevron gusset connection has been presented. The analysis procedure uses the gusset-to-beam interface as a control section. The analysis procedure identifies both a horizontal critical section (gusset-to-beam interface) as well as a vertical critical section (section *b-b*). A set of equations for calculating the forces and moments acting on the two critical sections is provided.
2. Today's standard procedure during the connection design process used in chevron brace connection design is to evaluate the brace force effects on the beam as if the joint is isolated from the frame. When braced frame geometry and loading is such that the summation of the vertical components of the brace forces is zero (a balanced force),

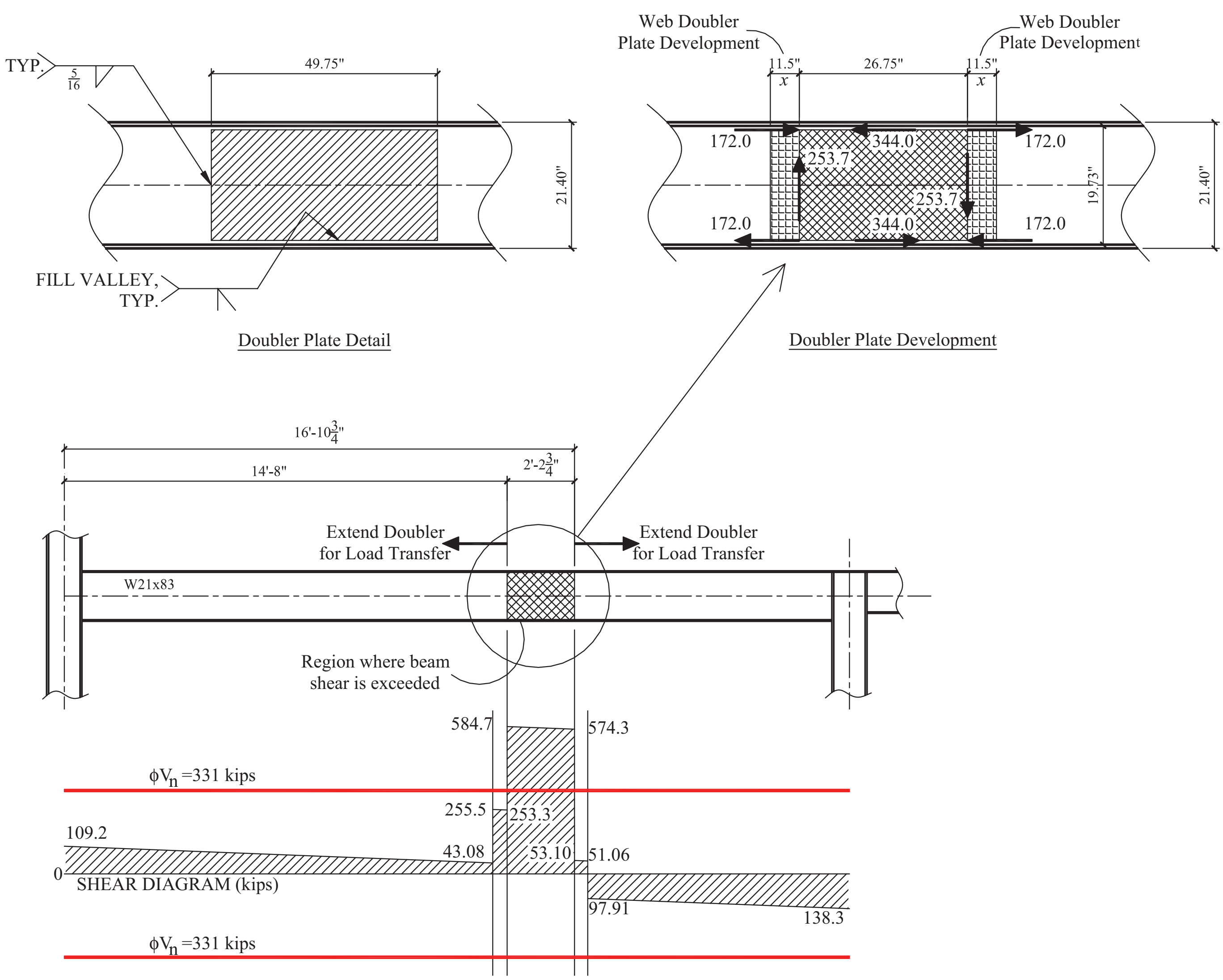


Fig. 35. Web doubler plate detail using the resultant load method.

this is an acceptable practice, regardless of the span of the beam or the location of the work point along the span of the beam.

3. When the braced frame geometry and loading is such that the summation of the vertical components of the brace forces is non-zero (an unbalanced force), the chevron effect must be evaluated. Because of the chevron effect, it is not adequate to evaluate the brace force effects on the beam as if the joint is isolated from the frame. The span of the beam as well as the location of the work point along the length of the beam must be considered.
   a. The maximum beam shear and moment can be over- or underestimated if the joint is evaluated as if it is isolated from the frame.
   b. Maximum beam shear and moment may be located within or outside of the connection region. Thus, the beam should not be evaluated as if the connection is isolated from the frame.
4. The effect of the brace forces on the beam should be considered during the process of making final member size selection. At this stage of design, information regarding connection geometry may not be known. An approximate method for estimating the brace force distribution at the gusset-to-beam interface has been presented. An example problem was provided demonstrating the application of the method during the member design process.
5. Two methods for distributing the section *a-a* forces were presented; the resultant method and the distributed method. The resultant method is a simplified method recommended to be used during the beam size selection process. The distributed method should be used for (a) the design of the gusset and the gusset-to-beam weld and (b) evaluating the required web doubler thickness as well as the portion of the beam where a web doubler should be provided.
6. If the effect of chevron brace forces is evaluated properly during the beam size selection process, the need for, and costs associated with, beam web doubler plates can be eliminated.

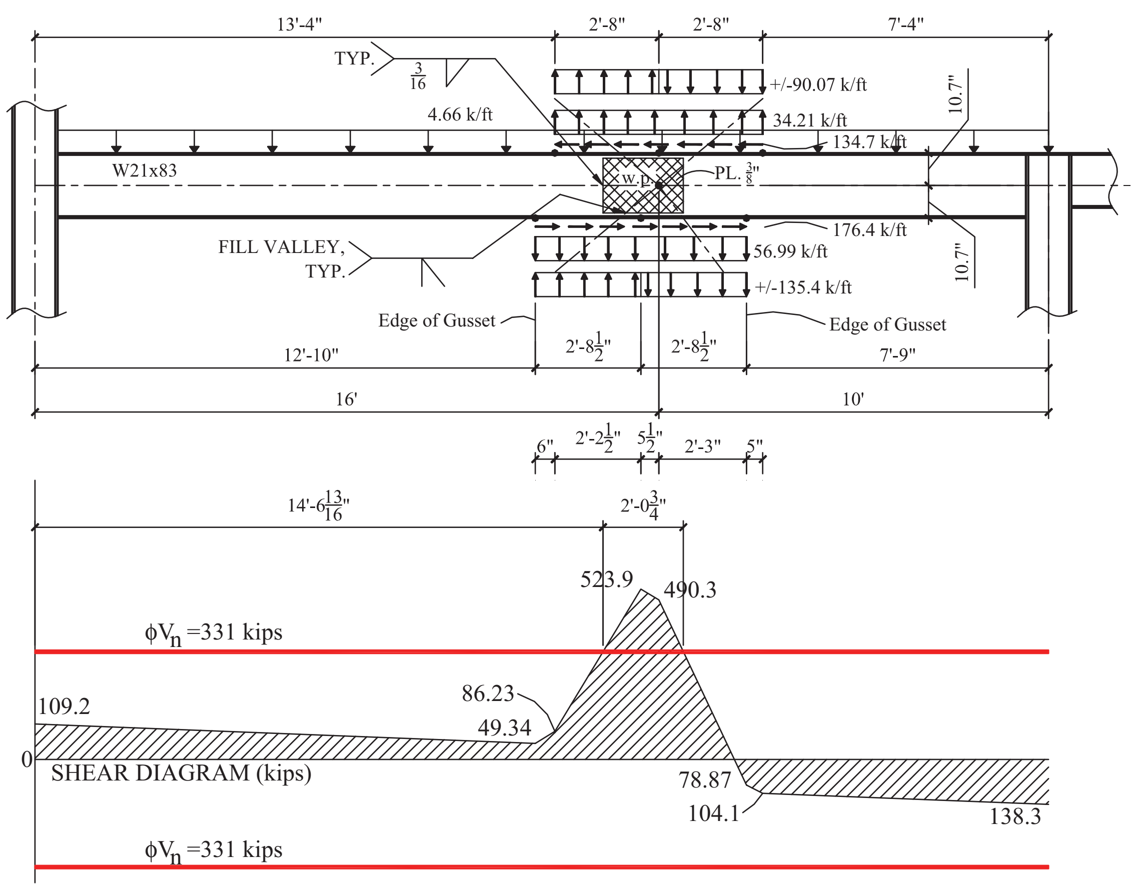


Fig. 36. Web doubler plate detail using the distributed load method.

SYMBOLS

A_g The gross cross-sectional area of a framing member

D Service level dead load (gravity)

F_{cre} Critical stress calculated from *Specification* Chapter E

F_y Nominal specified yield strength

H_1 The horizontal component of force in brace 1

H_2 The horizontal component of force in brace 2

$H_{a\text{-}a}$ The horizontal (shear) force acting at the gusset-to-beam interface

H_{bi} The horizontal (normal) force acting on the critical vertical section of the gusset

I_x Moment of inertia about bending axis

L_1 The horizontal distance from the left edge of the gusset to the work point

L_2 The horizontal distance from the right edge of the gusset to the work point

L Service level live load (gravity)

L Span of frame beam

L_g The contact length of the gusset-to-beam interface

$L_{g,app}$ Approximation of length of gusset, L_g

$M_{a\text{-}a}$ The moment acting at the gusset-to-beam interface

$M_{a\text{-}a,app}$ Approximation of moment, $M_{a\text{-}a}$

M_{bi} The moment acting on the critical vertical section of the gusset

M_{beam} The moment in the frame beam

M_n The nominal available flexural strength

M_u The required (design) moment strength

$M_{u,max}$ The maximum required (design) flexural strength

N_{eq} The couple of the moment, $M_{a\text{-}a}$

$N_{eq,app}$ Approximation of the couple of the moment, $M_{a\text{-}a}$

P_1 The axial force in brace 1

P_2 The axial force in brace 2

P_b The buckling strength of brace in compression, $1.14F_{cre}A_g$

R_y The ratio of expected yield stress to the specified minimum yield stress, F_y

V_1 The vertical component of the force in brace 1

V_2 The vertical component of the force in brace 2

$V_{a\text{-}a}$ The vertical (normal) force acting at the gusset-to-beam interface

V_{beam} The shear in the frame beam

V_{bi} The vertical (shear) force acting on the critical vertical section of the gusset

V_n The nominal available shear strength

V_u The required (design) shear strength

$V_{u,max}$ The maximum required (design) shear strength

W Service level wind load

Z The plastic section modulus

d depth of frame beam

e_b The perpendicular distance from the gusset interface to the gravity axis of the frame beam

$e_{b,app}$ Approximation of length of half-depth of the frame beam

h The vertical dimension of the gusset

n_{eq} The couple of the moment, $M_{a\text{-}a}$, per unit length of gusset

w_D LRFD dead load (gravity)

w_L LRFD live load (gravity)

w.p. The brace work point

w_u LRFD (design) uniform gravity load

Δ The horizontal misalignment between the work point and the centroid of the gusset-to-beam interface

ΣH_i The summation of horizontal brace force components

ΣV_i The summation of vertical brace force components

δ Beam deflection

ϕ LRFD strength reduction factor

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