Limit States for Horizontal Shear at a Braced Frame Beam Flange

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ABSTRACT

The most common design methodologies for bracing connections are based on the lower bound theorem. If the lower bound theorem is the basis cited for a steel connection design, the theorem's requirements must be satisfied. While the ductility requirement is essential to the theorem, it is the implications of the equilibrium and material strength limit requirements that will be investigated here for a portion of a common load path in a braced frame connection. Two limit states will be outlined that precisely define and expand on a limit state for web horizontal yielding. The limit states are also applicable to other types of connections with similar loading and geometry. These limit states are rational approaches to ensure that adequate resistance is provided to critical portions of a commonly assumed load path for braced frame connections. The limits likely will not control many typical braced frame configurations, but several conditions in which they may govern have been outlined for consideration. Additionally, it has been shown that it may be too conservative to require that all horizontal force from the gusset plate be transferred by shear into the beam web within the length of the gusset connection, although there are defined limits to the local beam capacity that can be obtained in these connections.

Keywords: beam flange, horizontal shear, limit states.

The most common design methodologies for bracing connections are based on the lower bound theorem, based on the author's experience and observations. For example, the Uniform Force Method is recommended by AISC for bracing connections (AISC, 2005), and it is an application of the lower bound theorem. Also, the KISS method, a common alternate method, is also an application of the lower bound theorem. If the lower bound theorem is the basis cited for a steel connection design, the theorem's requirements must be satisfied. While the ductility requirement is essential to the theorem, it is the implications of the equilibrium and material strength limit requirements that will be investigated here for a portion of a common load path in a braced frame connection. In this portion of the load path, two limit states will be outlined that precisely define and expand on a limit state for web horizontal yielding described by Thornton (Tamboli, 1999). The limit states are also applicable to other types of connections with similar loading and geometry.

CONNECTION CONFIGURATION AND LOAD PATH

Consider the braced frame connection shown in Figure 1. Here a gusset plate is directly welded to the top flange of a beam, and that weld will be expected to transfer some combination of shear, axial load and moment.

Of the resisted loads, shear is frequently a dominant component because a large portion of the horizontal component of the brace force is typically required to be transferred at the gusset-to-beam weld. The forces to be transferred in the braced frame connection are shown in Figure 2(a), and the horizontal forces are isolated and labeled in Figure 2(b). Refer to Part 13 of the *Steel Construction Manual* (AISC, 2005) for forces not labeled.

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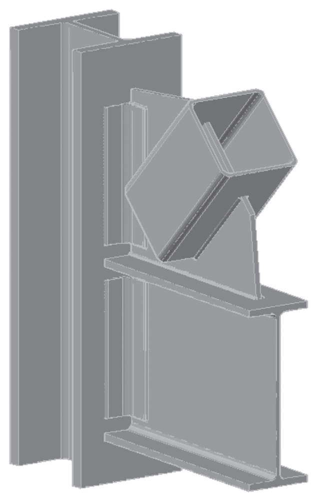

Fig. 1. Connection configuration for a braced frame.

It is expected for member design that the axial load from the beam end connection and shear from the gusset connection will distribute uniformly in the beam cross section at a distance away from the connection. The lower bound theorem requires an engineer to ensure this load path has adequate resistance by investigating equilibrium and material strength limit states between the gusset-to-beam weld and the beam section at a distance away from the connection. To make certain that the assumed load path has adequate resistance, two rational limit states for load transfer at critical sections of the beam are developed using the basic AISC equations for shear [2005 *Specification* Equation J4-3, $\phi R_n = (1.0)(0.60)F_y A_s$] and axial resistance [2005 *Specification* Equation J4-1, $\phi R_n = 0.9F_y A_s$] in a connection. Both limits should be checked in order to verify that the gusset-to-beam connection has adequate length for load transfer into the beam.

Limit State 1: Web Shear Yielding and Axial Yielding of the Top Flange and k-Area (see Figure 3)

Much of the horizontal load transferred from the gusset to the beam must also pass through the top edge of the beam web. However, some load will remain in the top flange and k-area based on an assumed uniform stress distribution in the beam cross section away from the connection. The load that must transfer to the beam web in excess of the web shear capacity along the connection length must initially be transmitted as axial load in the top flange and k-area of the beam. Then, away from the connection along the beam length, the excess load is expected to migrate by shear from the top flange and k-area into the beam web in order to attain a uniform stress distribution. In this event, the top flange and k-area may be compared to a collector beam, dragging load into the beam web away from connection. If the axial capacity of the top flange and k-area is less than needed for the required axial load, the gusset-to-beam connection must be lengthened or the beam reinforced. This limit state is intended to preclude a failure that may be analogous to block shear rupture: there is a portion of the steel member that may fail with shear along one edge and axial load on its perpendicular edge. A comparable failure mode was indicated by Epstein and D'Aiuto (2002) in their Figure 2(c), reproduced here as Figure 4.

Limit State 1 for web shear yielding and axial yielding of the top flange and k-area is developed in the following manner (refer to Figure 5 for variable definitions):

1. Determine the maximum possible resistance that the top flange and k-area, A_{f+k}, can provide by axial yielding.
2. Determine the minimum connection length, L_c, needed to provide the balance of the required force resistance by beam web shear.

$$\phi R_{n,w+f+k} \geq H_{ub} \tag{1}$$

$$\phi V_{n,w} + \phi P_{n,f+k} \geq H_{ub}$$

$$(1.0)(0.6)F_y L_c t_w + 0.9F_y A_{f+k} \geq H_{ub} \tag{2}$$

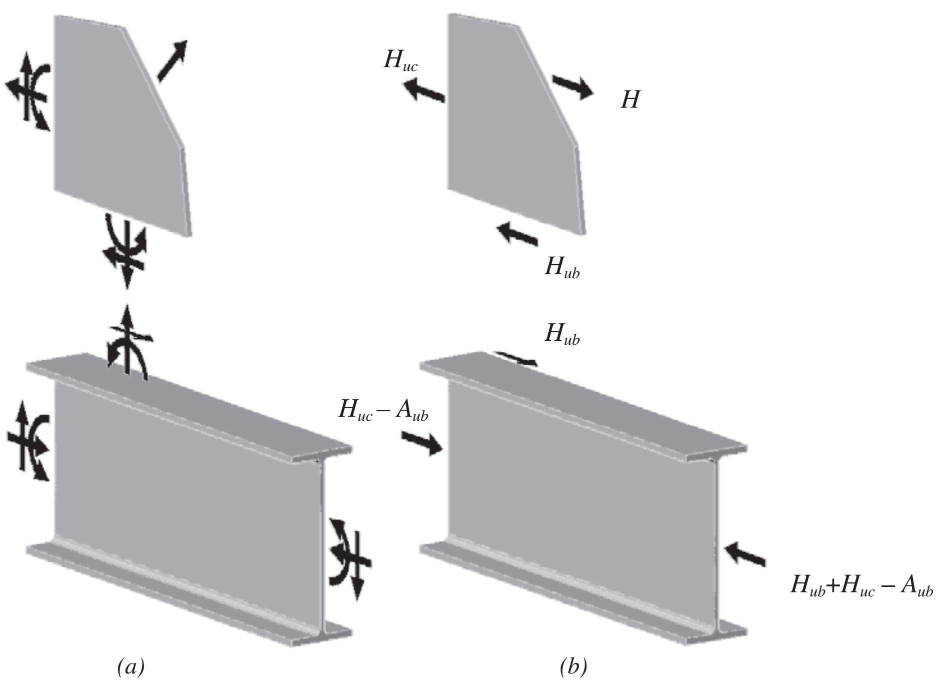

Fig. 2. Free-body diagrams for forces on (a) gusset plate and beam and (b) only horizontal forces on gusset plate and beam.

where

$$A_{f+k} = 0.5\left[A_g - t_w\left(d - 2k_{des}\right)\right]$$

Minimum connection length format:

$H_{ub} - 0.9F_y A_{f+k} \le 0$:

$$L_{c,\min,w} = 0 \qquad (3)$$

$H_{ub} - 0.9F_y A_{f+k} > 0$:

$$L_{c,\min,w} \ge \frac{H_{ub} - 0.9F_y A_{f+k}}{0.6F_y t_w} \qquad (4)$$

The use of Equation 3 would indicate that the connection length is not governed by horizontal shear of the beam web because the top flange and k-area has sufficient capacity to drag the required force into the web beyond the connection length.

Limit State 2: Shear Yielding of Flanges (see Figure 6)

If the horizontal load to be transferred to the beam exceeds the web shear capacity over the gusset-to-beam connection length, the balance of the load must be resisted by axial load in the top flange and k-area of the beam. To achieve the required load distribution in the top flange and k-area, some axial load may need to distribute beyond the toes of the fillets

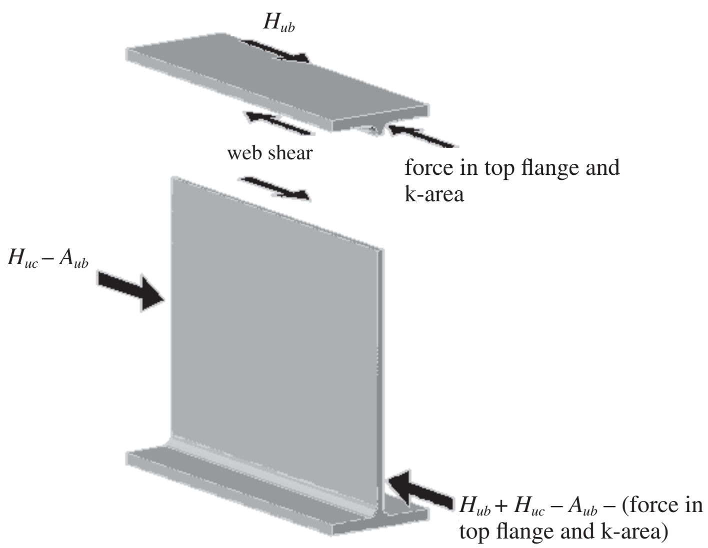

Fig. 3. Horizontal forces with section cut at beam web.

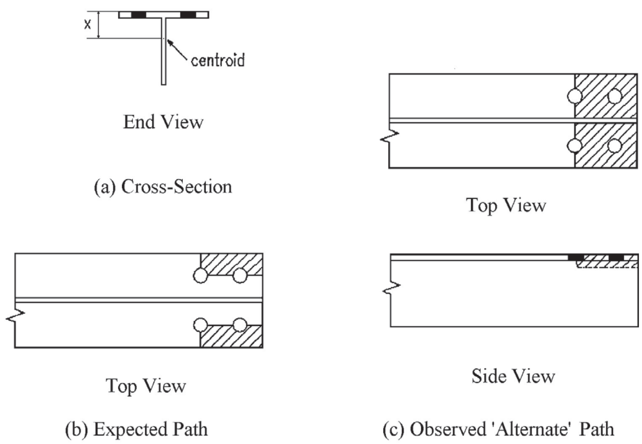

Fig. 4. Block shear paths in structural tees (Epstein and D'Aiuto, 2002).

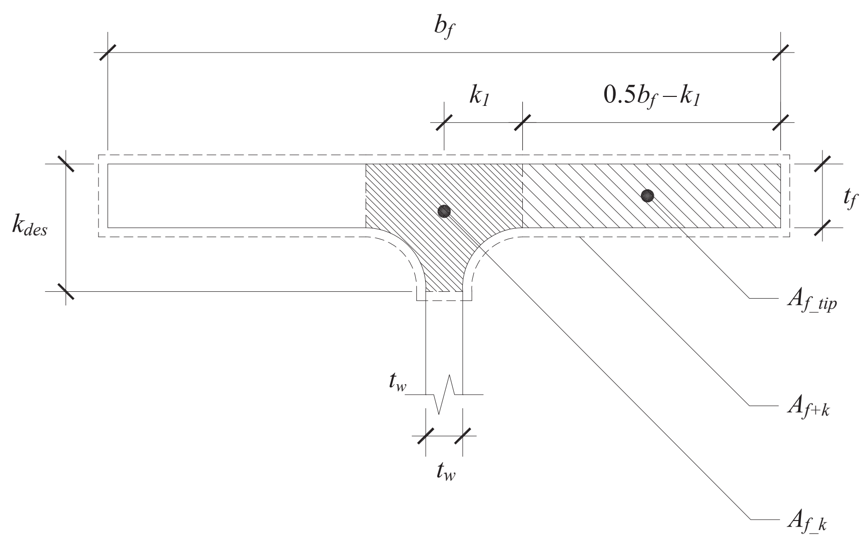

Fig. 5. Top flange cross-section variables for Limit States 1 and 2.

on the flange. The axial load must distribute to this area over the length of the gusset-to-beam connection, and so the shear capacity of the flange must be capable of transferring this load. If the flange shear capacity is less than required, the gusset-to-beam connection must either be lengthened or the beam reinforced.

Limit State 2 for the shear yielding of flanges is developed in the following manner (refer to Figure 5 for variable definitions):

1. Determine the minimum axial force required to be resisted in the beam flange tips, A_{f_tip}, at the end of the connection length. This is horizontal force in the beam, less the maximum possible resistance by beam web shear over the connection length and axial yielding in the top flange k-area, A_{f_k}.
2. Determine the minimum connection length needed to transmit the required axial load to the beam flange tips by beam flange shear.

$$\phi V_{n,f\_tips} \geq P_{u,f\_tips,\min} \tag{5}$$

$$2(1.0)(0.6)F_y L_c t_f \geq P_{u,f+k,\min} - P_{u,f\_k,\max}$$

$$2(1.0)(0.6)F_y L_c t_f \geq P_{u,f+k,\min} - 0.9F_y A_{f\_k} \tag{6}$$

where

$$P_{u,f+k,\min} = H_{ub} - (1.0)(0.6)F_y L_c t_w$$

$$A_{f\_k} = A_{f+k} - 2A_{f\_tip}$$

$$A_{f\_tip} = t_f\left(0.5b_f - k_1\right)$$

Minimum connection length format:

$H_{ub} - 0.9F_y A_{f\_k} \leq 0$:

$$L_{c,\min,f} = 0 \tag{7}$$

$H_{ub} - 0.9F_y A_{f\_k} > 0$:

$$L_{c,\min,f} \geq \frac{H_{ub} - 0.9F_y A_{f\_k}}{0.6F_y\left(2t_f + t_w\right)} \tag{8}$$

The use of Equation 7 would indicate that the connection length is not governed by horizontal shear of the beam flange because the top flange k-area alone has sufficient axial capacity to drag the required force into the web beyond the connection length.

ADDITIONAL COMMENTS

Net Section Capacity

In both of the limit states previously described, the effect of net section on the shear and tension areas should be incorporated into design calculations. That is, the gross area minus the area of the bolt holes, copes and blocking of flange tips should be reflected in the input dimensions. It is also suggested to use the resistance factor and material limit for shear and tension rupture—0.75 and ultimate stress, respectively (AISC, 2005)—when material is removed from the failure planes. When beam material is removed from the failure planes and the brace in Figure 1 is in compression, Limit State 1 appears to address a failure very similar to the block shear failure observed by Epstein and D'Aiuto (2002).

Are These Limit States Significant?

If the assumed load path requires horizontal shear transfer at a beam flange, the flange and web must have adequate material strength to resist the loads that satisfy equilibrium. The limit states described here are intended to address conceivable failure modes on this load path. Therefore, they are suggested to be considered and applied to braced frame and similarly configured connections. Typically sized connection configurations for braced frames will not often have these limit states govern unless material is removed from the top flange or web of the beam in the vicinity of the connection. Furthermore, it may be excessively conservative to require that the length of the gusset connection be such that all the horizontal force from the gusset plate is transferred to the beam web over that length because the substantial axial capacity of the top flange and k-area is neglected.

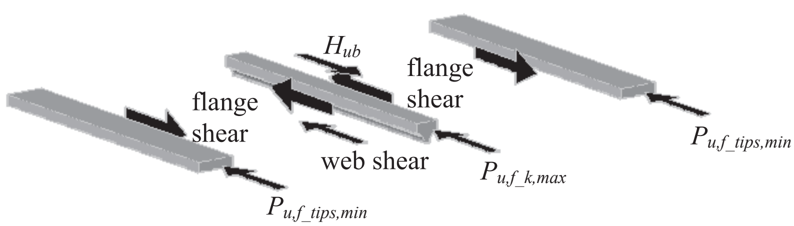

Fig. 6. Horizontal forces with section cuts at beam flange.

Although perhaps uncommon, there are conditions that may cause these limit states to govern. Several of these conditions are listed here:

1. High horizontal shear at the gusset-to-beam connection:
   - The braced frame beam is near yielding due to the brace force.
   - The brace connection is to column web rather than column flange.
   - There is no brace connection to the column.
   - The brace angle to the horizontal is small.
   - The KISS method is used.
2. Short gusset-to-beam connection:
   - The ratio of the gusset plate yield strength to the beam yield strength is greater than 36 to 50 ksi.
   - The gusset plate is very thick.
   - There is an architectural or spatial constraint.
3. Braced frame beams that are not rolled wide flange sections:
   - The braced frame beam is a hollow structural section (HSS).
   - The braced frame beam is a built-up section.
4. Material removed from the braced frame beam:
   - The top flange of the braced frame beam is coped.
   - The top flange tips are coped to fit beam between column flanges.
   - The top flange tips are blocked to allow a double-angle connection that is continuous across the gusset plate and beam web.
   - There are bolt holes within the gusset connection length in the beam top flange or web.

DESIGN EXAMPLE

AISC *Manual*, 13th Ed., Example II.C-2 (see Figure 7)

Given

W18×106 beam:

A_g = 31.1 in.^{2}
d = 18.7 in.
b_f = 11.2 in.
t_f = 0.94 in.
t_w = 0.59 in.
k_{des} = 1.34 in.
k_1 = 1.125 in.
F_y = 50 ksi
L_c = 42 in.
H_{ub} = 355 kips

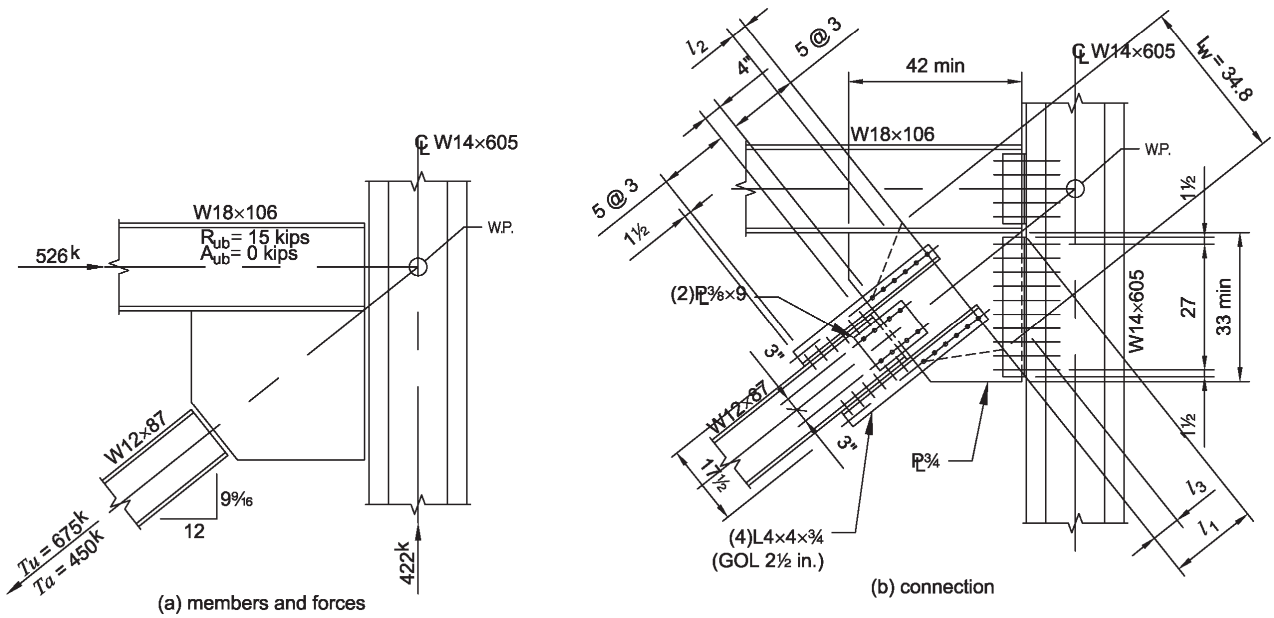

Fig. 7. AISC Manual, 13th Ed., Example II.C-2.

Area of the flange and k-area:

$$A_{f+k} = 0.5\left[A_g - t_w\left(d - 2k_{des}\right)\right]$$

$$A_{f+k} = 0.5\left\{31.1\text{ in.}^2 - (0.59\text{ in.})\left[18.7\text{ in.} - 2(1.34\text{ in.})\right]\right\}$$

$$A_{f+k} = 10.8\text{ in.}^2$$

Area of the flange tip:

$$A_{f\_tip} = t_f\left(0.5b_f - k_1\right)$$

$$A_{f\_tip} = (0.94\text{ in.})\left[0.5(11.2\text{ in.}) - 1.125\text{ in.}\right]$$

$$A_{f\_tip} = 4.21\text{ in.}^2$$

Area of the k-area:

$$A_{f\_k} = A_{f+k} - 2A_{f\_tip}$$

$$A_{f\_k} = 10.8\text{ in.}^2 - 2\left(4.21\text{ in.}^2\right)$$

$$A_{f\_k} = 2.38\text{ in.}^2$$

Limit State 1:

$$\phi R_{n,w+f+k} \geq H_{ub}$$

$$\phi V_{n,w} + \phi P_{n,f+k} \geq H_{ub}$$

$$(1.0)(0.6)F_y L_c t_w + 0.9F_y A_{f+k} \geq H_{ub}$$

$$(1.0)(0.6)(50\text{ ksi})(42\text{ in.})(0.59\text{ in.}) + 0.9(50\text{ ksi})\left(10.8\text{ in.}^2\right) \geq 355\text{ kips}$$

$$1230\text{ kips} \geq 355\text{ kips} \qquad \textbf{o.k.}$$

Minimum axial load in the flange and k-area:

$$P_{u,f+k,\min} = H_{ub} - (1.0)(0.6)F_y L_c t_w$$

$$P_{u,f+k,\min} = 355\text{ kips} - (1.0)(0.6)(50\text{ ksi})(42\text{ in.})(0.59\text{ in.})$$

$$P_{u,f+k,\min} = -388\text{ kips} \rightarrow P_{u,f+k,\min} = 0\text{ kips}$$

Minimum axial load required to be in the flange tips:

$$P_{u,f+k,\min} = 0\text{ kips} \rightarrow P_{u,f\_tips,\min} = 0\text{ kips}$$

Limit State 2:

$$\phi V_{n,f\_tips} \geq P_{u,f\_tips,\min}$$

$$2(1.0)(0.6)F_y L_c t_f \geq P_{u,f\_tips,\min}$$

$$2(1.0)(0.6)(50\text{ ksi})(42\text{ in.})(0.94\text{ in.}) \geq 0\text{ kips}$$

$$2369\text{ kips} \geq 0\text{ kips} \qquad \textbf{o.k.}$$

Both Limit State 1 (capacity 1,230 kips) and Limit State 2 (capacity 2,369 kips) are satisfied because the resistance of each significantly exceeds the required horizontal force of 355 kips. The same conclusion may be reached by verifying that the design connection length, L_c, exceeds both minimum connection lengths $L_{c,\min,w}$ and $L_{c,\min,f}$. Those calculations are performed next.

Minimum connection length based on maximum axial capacity of flange and k-area (Limit State 1):

$$L_c \geq L_{c,\min,w}$$

$$L_c \geq \frac{H_{ub} - 0.9F_y A_{f+k}}{0.6F_y t_w}$$

$$42\text{ in.} \geq \frac{355\text{ kips} - 0.9(50\text{ ksi})\left(10.8\text{ in.}^2\right)}{0.6(50\text{ ksi})(0.59\text{ in.})}$$

$$42\text{ in.} \geq -7.40\text{ in.} \rightarrow L_{c,\min,w} = 0\text{ in.} \qquad \textbf{o.k.}$$

Minimum connection length based on maximum shear capacity of web and maximum axial capacity of the k-area (Limit State 2):

$$L_c \geq L_{c,\min,f}$$

$$L_c \geq \frac{H_{ub} - 0.9F_y A_{f\_k}}{0.6F_y\left(2t_f + t_w\right)}$$

$$42\text{ in.} \geq \frac{355\text{ kips} - 0.9(50\text{ ksi})\left(2.38\text{ in.}^2\right)}{0.6(50\text{ ksi})\left[2(0.94\text{ in.}) + 0.59\text{ in.}\right]}$$

$$42\text{ in.} \geq 3.35\text{ in.} \qquad \textbf{o.k.}$$

The design connection length of 42 in. is much more than both $L_{c,\min,w}$ (0 in.) and $L_{c,\min,f}$ (3.35 in.), and so the connection as detailed is sufficiently long. The calculated minimum connection length $L_{c,\min,w}$ is negative, indicating that the axial capacity of the flange and k-area alone exceeds the applied horizontal force. As a result, the flange and k-area may drag shear into the web over the necessary beam length as determined by the web shear capacity.

Additionally, if the length of the gusset to beam connection were limited by requiring that all horizontal force from the brace must transfer to the beam web over the connection length, the resulting connection length would be:

$$L_c = \frac{H_{ub}}{(1.0)(0.6)F_y t_w}$$

$$L_c = \frac{355\text{ kips}}{(1.0)(0.6)(50\text{ ksi})(0.59\text{ in.})}$$

$$L_c = 20.1\text{ in}$$

This connection length, when only considering beam web shear, is six times the length required when the axial capacity of the top flange and k-area is considered.

CONCLUSION

The two limit states suggested here are rational approaches to ensure that adequate resistance is provided to critical portions of a commonly assumed load path for braced frame connections. The applicability of the two limit states is derived from the requirements of the lower bound theorem on which common connection design methodology is based. The limits likely will not control many typical braced frame configurations, but several conditions in which they may govern have been outlined for consideration. Additionally, it has been shown that it may be too conservative to require that all horizontal force from the gusset plate be transferred by shear into the beam web within the length of the gusset connection, but also there are defined limits to the local beam capacity that can be obtained in these connections.

NOTATION

A_g = gross cross-section area of a wide flange beam

A_{f+k} = area of the flange and k-area of a wide flange beam

A_{f_k} = k-area of a wide flange beam

A_{f_tip} = cross-section area of the flange tip beyond the toe of the fillet on the flange

A_{ub} = required transfer force from the adjacent bay

d = depth of a wide flange beam

t_w = web thickness of a wide flange beam

t_f = flange thickness of a wide flange beam

b_f = flange width of a wide flange beam

k_{des} = design fillet dimension along the web of the wide flange beam

k_1 = fillet dimension along the flange of the wide flange beam

L_c = length of the gusset-to-beam connection

$L_{c,\min,w}$ = minimum length of the gusset-to-beam connection required for web shear transfer without reinforcement

$L_{c,\min,f}$ = minimum length of the gusset-to-beam connection required for flange shear transfer without reinforcement

F_y = beam yield stress

H = horizontal component of the required brace axial force

H_{ub} = horizontal force transferred from the gusset plate to the beam

H_{uc} = horizontal force transferred from the gusset plate to the column

$P_{u,f\_tips,\min}$ = minimum axial force on the cross-section area of the flange tip beyond the toe of the fillet on the flange of a wide flange beam

$P_{u,f\_k,\max}$ = maximum axial force on the k-area of a wide flange beam

$P_{u,f+k,\min}$ = minimum axial force on the area of the flange and k-area of a wide flange beam

$\phi P_{n,f+k}$ = design axial strength of the flange and k-area of a wide flange beam

$\phi R_{n,w+f+k}$ = design strength for Limit State 1: design shear strength of the web along the length of the gusset-to-beam connection and axial strength of the flange and k-area

$\phi V_{n,f\_tips}$ = design strength for Limit State 2: design shear strength of the flange tips along the length of the gusset-to-beam connection

$\phi V_{n,w}$ = design shear strength of the web along the length of the gusset-to-beam connection

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