

Satisfying Inelastic Rotation Requirements for In-Plane Critical Axis Brace Buckling for High Seismic Design

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ABSTRACT

When a vertical brace buckles during a seismic event, its connections must be able to resist the available flexural strength of the brace about its critical buckling axis without fracture. This is achieved in most current practice by orienting the brace to buckle out-of-plane and introducing a hinge line in the gusset to permit large inelastic rotations with small out-of-plane flexure demand on the connections and the supporting members. In this paper, the authors introduce a connection configuration that allows the development of a hinge line, which will permit large inelastic rotations for in-plane brace buckling with small flexural demand on the connection and supporting members.

Keywords: bracing connections, gusset plate, buckling, seismic design, inelastic rotation.

INTRODUCTION

The design of vertical bracing connections for lateral force resisting systems in regions of high seismicity requires that the connections be designed to resist not only the expected axial tension and compression demands, but also the flexural force that will be induced in the connection when plastic hinges form at the brace ends and mid-length of the brace. In this paper, the discussion, equations and calculations are presented in the context of a Load and Resistance Factor Design (LRFD) design philosophy. The same concepts and calculations can be easily applied in an Allowable Strength Design (ASD) evaluation but are not presented in this paper. The materials used in all examples are ASTM A500 Gr. B for hollow structural sections (HSS), ASTM A992 Gr. 50 for W-shapes, ASTM A572 Gr. 50 for angles, and ASTM A572 Gr. 50 for plate material.

For special concentrically braced frames (SCBFs), Section F2.6c(3) of the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2010a) explicitly requires that the connections be designed for an amplified expected moment, as shown in Equation 1:

$$M_e = 1.1R_y F_y Z \quad (1)$$

where Z is the plastic section modulus about the critical buckling axis. When the brace is designed to buckle out-of-plane,

design for the moment of Equation 1 is extremely difficult. Rather than design for the moment of Equation 1, the *Seismic Provisions* allow an exception when a hinge line is introduced as shown in Figure 1. This is referred to in Section F2.6c(3)(b) as a connection with “sufficient rotation capacity to accommodate the required inelastic rotation ...” This allows designs using Equation 1, but with Z taken to be the plastic section modulus of the gusset plate at the hinge line (as shown in Figure 1). Thus, M_e is very small and is ignored, and large inelastic rotations can occur without connection failure. When referring to Figure 1, note that the “pull-off dimension” can be less than or greater than that shown, depending on the brace size, because the connection strength must be equal to or greater than the expected tensile strength of the brace. This potentially increases the required gusset-to-beam and/or gusset-to-column connection length. The pull-off dimension is also controlled by the Whitmore spread as can be seen in Figure 2.

The problem with the large Whitmore spread is that it usually results in very large gusset plates, as can be seen in Figure 2, which can compromise the intent of the *Seismic Provisions* by resulting in very short braces. Furthermore, depending on frame bay dimensions, a concomitant reduction in ductility occurs. Figure 3 shows how the performance of a brace can be compromised when bay dimensions or geometric requirements result in a “barn-door” gusset at both ends of a relatively short brace. Figure 4 depicts an example of a constructed barn-door gusset. Note the actual length of the brace relative to the theoretical work point-to-work point length.

The brace connection shown in Figure 2 exists in the 25-ft bay shown in Figure 3, with a center-to-center floor height of 12 ft 10¾ in. The work point dimension of the brace

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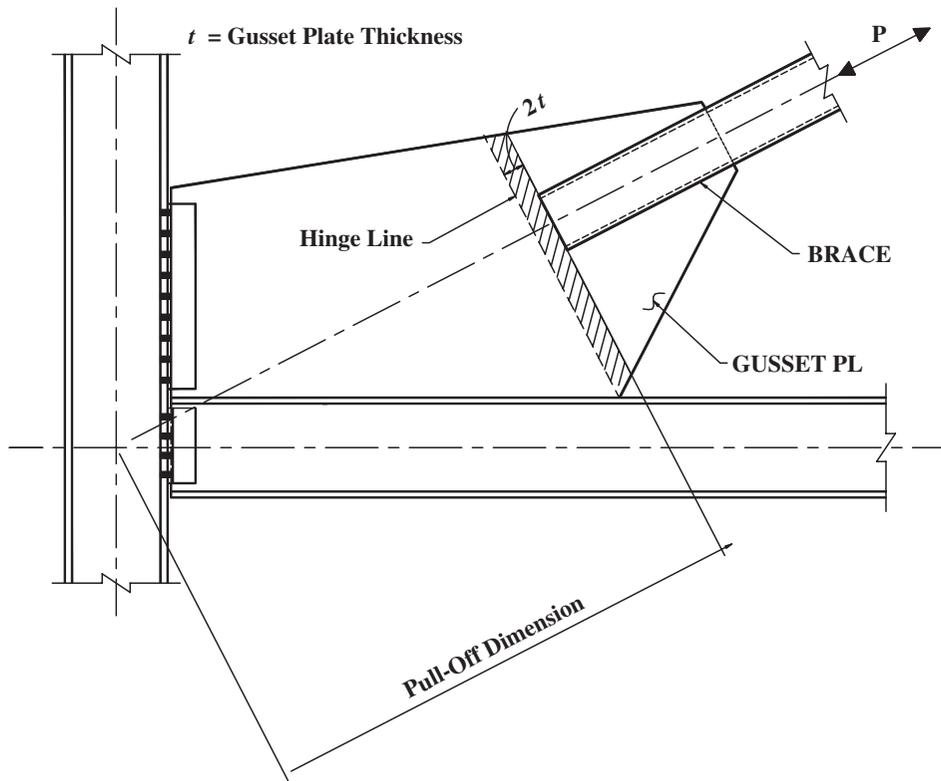


Fig. 1. Gusset hinge line.

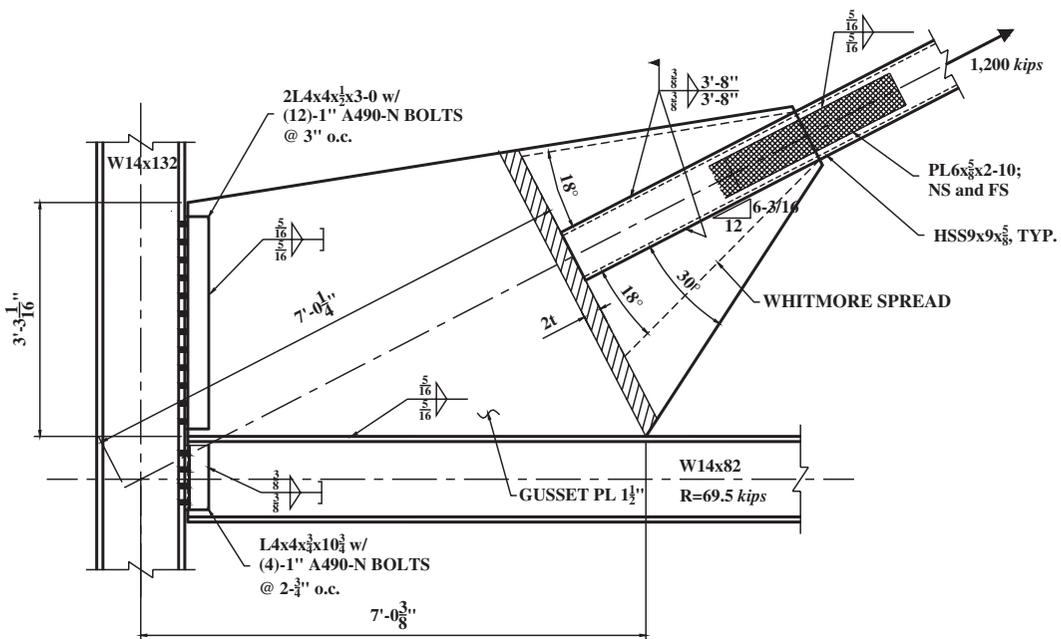


Fig. 2. Brace connection in an SCBF using an 18° Whitmore section.

measures approximately 28 ft. After satisfying the letter of the *Seismic Provisions*, the actual length of the brace between reinforcements is approximately 4 ft, which is appreciably shorter than the unbraced gusset length. Thus, while the letter of the *Seismic Provisions* is satisfied, the spirit certainly is not.

The pull-off dimension in a vertical brace connection is typically based on the common assumption of a 30° Whitmore spread, which is the maximum Whitmore spread as discussed in the 14th edition *Steel Construction Manual* (AISC, 2011). Most of the work observed by the authors in real jobs uses this assumption. The connection shown Figure 2 has an 18° Whitmore spread. However, the gusset at the bottom side has been tapered at 30°, which allows a single-pass 5/16-in. fillet weld to be used at the gusset-to-beam connection. The 18° spread at the top side was found to be sufficient to produce a 5/16-in. weld at the clip angles-to-gusset connection. Imagine the size of the gusset if a 30° spread was taken at both the top and bottom sides. Also, the connection shown in Figure 2 is shown in context of the frame shown in Figure 3 to illustrate the effect of the geometry on the brace length.

More recently, the idea of a narrower Whitmore spread such as 20° or even 10° has been suggested. This narrower

spread will reduce the pull-off dimension needed for the brace but may necessitate heavier gusset-to-column and gusset-to-beam connections—and, possibly, a thicker gusset where yield limit states control. Figure 5 shows the same connection as shown in Figure 2, but with a 10° Whitmore spread, and Figure 6 shows its effect on the brace length in context of the frame. As can be seen by comparing the brace lengths shown, reducing the spread from 18° to 10°—along with reducing the taper on the bottom side of the gusset from 30° to 10°—increases the brace length from 3 ft 11 in. to 10 ft 10 in. Note that in the connection shown in Figure 5, the fillet weld of the gusset to the beam flange has increased from 5/16 in. to 5/8 in., and the fillet weld at the gusset to the clip angles on the gusset-to-column connection has increased from 5/16 in. to 3/8 in.

A “NEW” CONCEPT

Rather than have the brace buckle out-of-plane in order to reduce M_e of Equation 1 to an insignificant value by applying the hinge line concept shown in Figure 1, this same hinge line concept can be applied to in-plane brace buckling. Figures 7a and 7b illustrate the concept proposed here. Note that the 4-in. dimension between the gusset edge and the

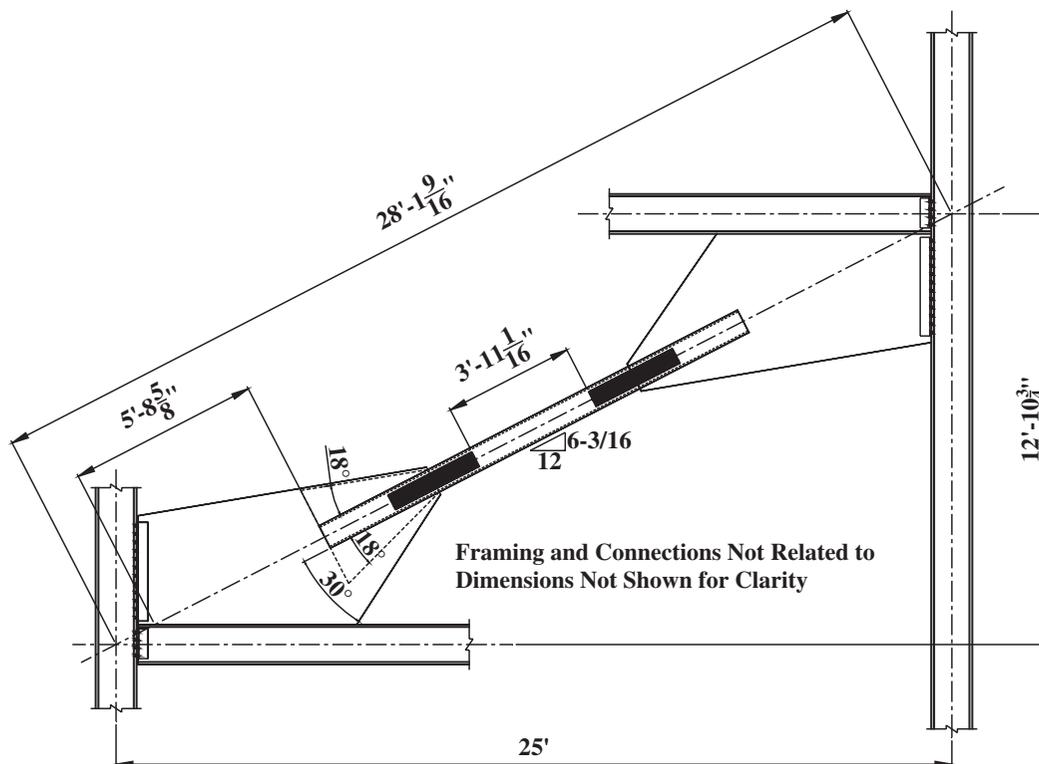


Fig. 3. Gusset and brace dimensions with an 18° (top) and 30° (bottom) Whitmore spread. Refer to Fig. 2 for details of brace connection.

edge of the brace is equivalent to $2t$. The advantages of this configuration in terms of the spirit of the *Seismic Provisions* are immediately obvious, as can be seen in Figure 8. The brace length is as large as it probably can be, at 15 ft $1\frac{1}{16}$ in. Thus, the ductility of the frame is maximized. Additional advantages include the following:

1. Compact and more economical gusset plates.
2. Less distress to the gusset-to-beam and gusset-to-column connections due to out-of-plane gusset buckling. With the hinge line concept of Figure 1, there will tend to be some gusset out-of-plane distortion, which will place extra demands on the gusset-to-beam and gusset-to-column connections. Because of this, some designers (e.g., Yoo et al., 2008) have recommended complete-joint-penetration (CJP) gusset-to-beam and gusset-to-column welds. Although the authors do not

agree with this recommendation, this in-plane buckling concept eliminates the concern.

3. In-plane brace buckling will be much less destructive to the building façade and interior partitions relative to out-of-plane brace buckling.
4. Erection versatility. Referring to Figure 1, where the brace is field-welded directly to the gusset plate, the slot in the HSS must usually be made longer than the connection length to allow for erection clearance—sometimes as much as 6 in. longer. This in turn will require any reinforcing plates and welds to also be made longer. The hinge plate of Figure 7 can be shop-welded to either the gusset plate or to the brace. When the hinge plate is shop-welded to the gusset (as shown in Figure 7a), the erection tolerance required is significantly less than what would be required for the brace



Fig. 4. An example of the barn-door gusset effect.

connection shown in Figure 1; a slot only ½ in. longer than the required connection length may be sufficient. When the hinge plate is shop-welded to the brace, the slot requires similar considerations to those discussed for out-of-plane conditions.

There has been previous research investigating the behavior of the hinge plate connection. Tremblay et al. (2008) performed some tests consisting of brace component tests with various types of connections—one being a brace connection similar to that shown in Figure 7. Although Tremblay et al. report that the in-plane buckling arrangement has very similar hysteretic response to that of the typical out-of-plane arrangement, no discussion regarding the behavior of the connection itself is presented.

Additionally, through a telephone conversation with Charles Roeder, professor of structural engineering and mechanics at the University of Washington (Roeder, 2012), the authors learned that a researcher in Taiwan has recently completed some physical tests of braced frames that evaluated the behavior of a frame with brace connections similar to that shown in Figure 7. The preliminary results suggest that the performance of the frame using this type of connection is very similar to that of the typical out-of-plane arrangement. While a formal report is expected in 2013, preliminary results suggest a $3t$ dimension for the hinge rather than the $2t$ dimension recommended currently in the *Seismic Provisions* for braces oriented to buckle out-of-plane. It should be noted that this research evaluated the performance of this type of connection providing a $2t$ dimension. Roeder noted that due to erection issues, one connection had a dimension of approximately $1t$. This connection apparently performed adequately despite the reduced separation between the end of the brace and the gusset edge.

Theory

Figure 7 shows a design satisfying the requirements of the *Seismic Provisions* (AISC, 2010a) and the *Specification for Structural Steel Buildings* (AISC, 2010b). The hinge plate is 2 ft × 1 ft 2 in. × 3 ft 4 in., shaped as shown in Section A-A of Figure 7b. The hinge plate width dimension, w_h , of 1 ft 2 in. is chosen to be no wider than the W14×132 column flange or the W14×82 beam flange width, whichever is smaller. This is essentially an arbitrary limit but is chosen to ensure that the hinge plate lies within the envelope formed by the beam and column. Geometrically, the hinge plate must be wider than the HSS 9×9×5/8 brace member to allow the hinge plate-to-brace connection to be made. Section F2.3 of the *Seismic Provisions* gives the expected tensile strength of the brace as

$$T_u = R_y F_y A_g = (1.4)(46 \text{ ksi})(18.7 \text{ in}^2) = 1,200 \text{ kips}$$

Applying *Specification* Section J4.1, the required hinge plate thickness is

$$t_h = \frac{T_u}{\phi F_y w_h} = \frac{1,200 \text{ kips}}{(0.90)(50 \text{ ksi})(14 \text{ in.})} = 1.90 \text{ in.}$$

Therefore, a 2-in.-thick ASTM A572 Gr. 50 plate is chosen. From Equation 8-2a from the 14th edition *Steel Construction Manual* (AISC, 2011), the welds of the hinge plate to the HSS and the gusset plate, with 18 in. of length at both locations, are

$$\begin{aligned} D &= \frac{T_u}{1.392l(\text{number of welds})} \\ &= \frac{1,200 \text{ kips}}{(1.392)(18 \text{ in. per weld})(4 \text{ welds})} \\ &= 11.9 \text{ sixteenths or } \frac{3}{4} \text{ in.} \end{aligned}$$

as shown in the Figure 7a. Also note that in Figure 7b, the minimum $2t$ requirement discussed in the *Seismic Provisions* commentary to Section F2.6c is shown as $2t_h = 4$ in. (the clear length of the hinge plate between the end of the brace and the edge of the gusset plate).

The remaining checks for gusset thickness; HSS shear at the welds; and the gusset-to-beam, gusset-to-column and beam-to-column connections will not be discussed further here. Instead, checks on the hinge plate to ensure satisfactory performance will be addressed.

Hinge Plate Development

The expected hinge capacity in flexure is

$$M_{hinge} = 1.1R_y F_y Z_h$$

where Z_h is the plastic section modulus of the hinge plate and is calculated as

$$Z_h = \frac{t_h^2 w_h}{4} = \frac{(2 \text{ in.})^2 (14 \text{ in.})}{4} = 14.0 \text{ in.}^3$$

The expected flexural strength of the hinge plate at the hinge line is calculated as

$$\begin{aligned} M_{hinge} &= 1.1R_y F_y Z_h \\ &= (1.1)(1.1)(50 \text{ ksi})(14.0 \text{ in.}^3) = 847 \text{ kip-in.} \end{aligned}$$

and the flexural strength of the HSS 9×9×5/8 is

$$\begin{aligned} M_{HSS} &= 1.1R_y F_y Z \\ &= (1.1)(1.4)(46 \text{ ksi})(58.1 \text{ in.}^3) = 4,120 \text{ kip-in.} \end{aligned}$$

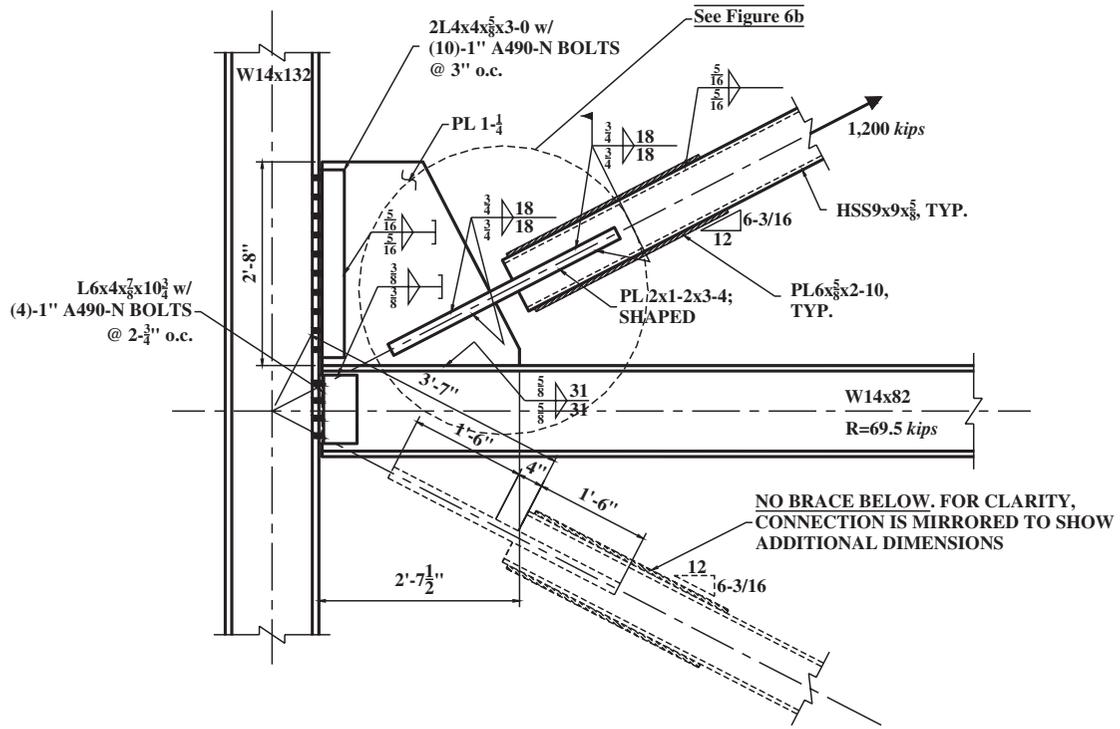


Fig. 7a. Illustration of in-plane buckling with the hinge line concept. See Fig. 7b for an enlarged view of the hinge plate.

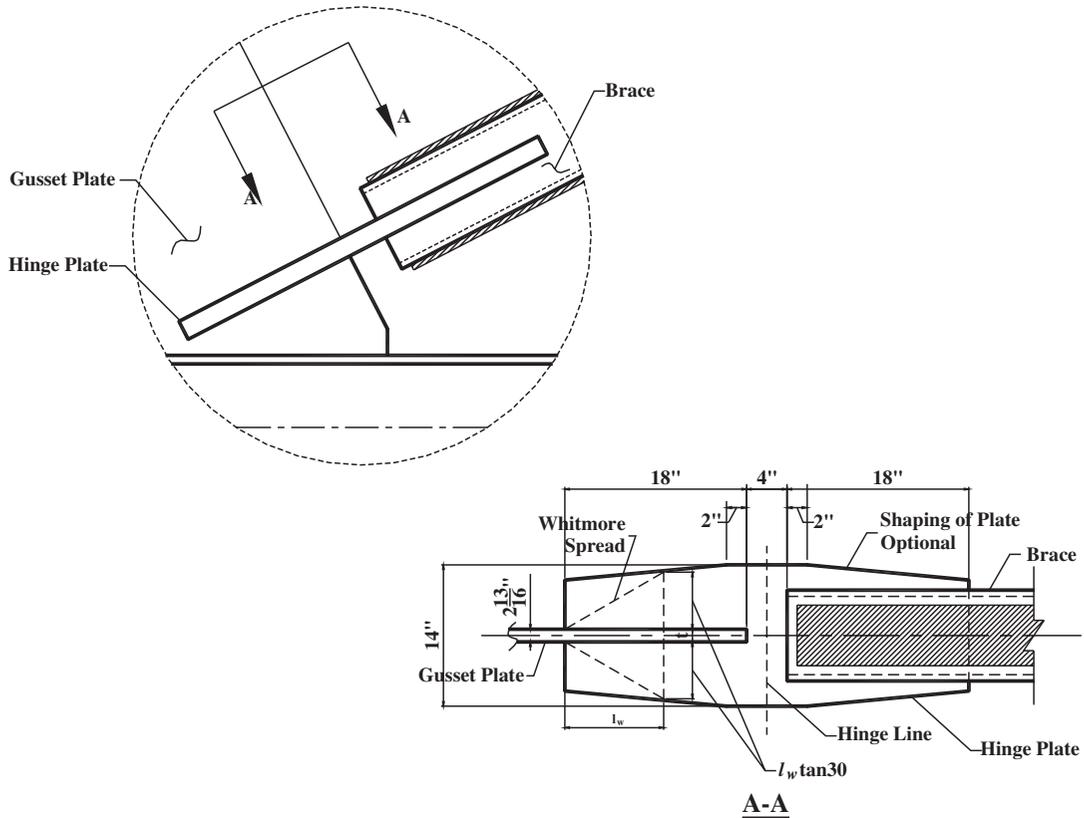


Fig. 7b. Enlarged view of the hinge plate shown in Fig. 7a.

Because 847 kip-in. is much less than 4,120 kip-in., the hinge will perform as a fuse, similar to the conventional method shown in Figure 1.

Development of Hinge Moment in Hinge Plate Welds

The region of the hinge plate between the gusset plate and the end of the HSS brace is in pure moment at the instant the brace buckles and M_{hinge} is generated. Figure 9 shows the mechanics.

The moment M_{hinge} can be equilibrated by two equal and opposite forces F in the hinge plate-to-gusset and hinge plate-to-brace welds. Note that this same effect occurs in the usual out-of-plane buckling of the configuration shown in Figure 1, but nowhere have the authors seen an explicit consideration of this. For example, the mechanics shown in Figure 9 can be applied to the brace-to-gusset connections shown in Figures 1 and 2.

From Figure 9,

$$F = \frac{M_{hinge}}{t_h} = \frac{847 \text{ kip-in.}}{2 \text{ in.}} = 424 \text{ kips}$$

The weld size required to carry this force is

$$\begin{aligned} D &= \frac{F}{1.392l(\text{number of welds})} \\ &= \frac{424 \text{ kips}}{(1.392)(18 \text{ in. per weld})(2 \text{ welds})} \\ &= 8.44 \text{ sixteenths} \end{aligned}$$

Because 8.44 sixteenths of an inch of weld is less than the provided 3/4-in. fillet welds, the connection for F is satisfactory. It should be noted that the *Seismic Provisions* permit the direct axial demand and the flexural demand to be considered independent of each other. That is, the effects do not need to be added. Thus, a brace connection is satisfactory given that the two checks, performed independent of each other, are satisfied. This is because when the brace buckles, the post-buckling compression demand is about 30% of the expected axial demand for which the welds were originally sized.

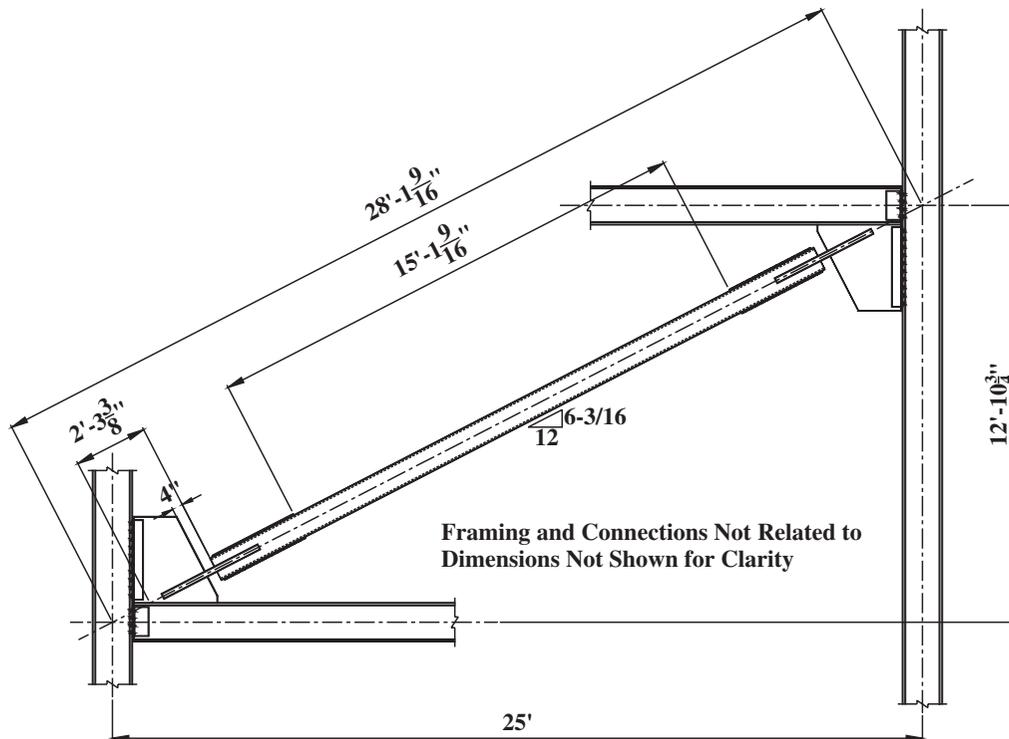


Fig. 8. Gusset and brace dimensions using the hinge plate concept. Refer to Fig. 7 for details of the brace connection.

Distribution of the Hinge Moment Across the Hinge Plate Width

Unlike the hinge line moment distribution in the usual out-of-plane buckling case (see Figure 1), for the in-plane buckling case the moment appears to be induced on the gusset plate side over the thickness of the gusset plate. In reality, however, the hinge plate moment is not induced over only the gusset thickness. Just as the axial force is assumed to spread over a gusset width with a maximum Whitmore width of $2l_w(\tan 30^\circ)$ plus the HSS width B , so also are the axial and flexural demands on the hinge plate side. The maximum spread is $t + 2l_w(\tan 30^\circ)$, where t is the gusset plate thickness (refer to Figure 7b). The force F (shown in Figure 9) can be considered to be an axial force over half of the hinge plate thickness. This is consistent with the theoretical Z_h , which produces tension over half the plate thickness and compression over the other half. For the design shown Figure 7a,

$$w_h \leq t + 2l_w(\tan 30^\circ) = 1.25 \text{ in.} + (2)(18 \text{ in.})(\tan 30^\circ) = 22.0 \text{ in.}$$

Therefore, because $w_h = 14 \text{ in.} \leq 22 \text{ in.}$, the 14-in. hinge plate is satisfactory.

As mentioned earlier, all the remaining checks for the HSS, the gusset, the gusset-to-beam connection, the gusset-to-column connection, and the beam-to-column connection are performed in the usual fashion and will not be presented in this paper.

CONCLUSIONS

A new in-plane buckling vertical brace connection for an SCBF design is presented. The configuration is justified on the basis of structural mechanics and limited structural

testing, and is very similar in concept to the usual out-of-plane hinge idea. This concept has several advantages over the traditional vertical brace connection, resulting in a more compact, economical connection. The concept has the added benefit of minimizing—if not eliminating—the barn-door effect illustrated in Figure 4.

SYMBOLS

A_g	Gross cross-sectional area of brace, in. ²
B	Width of face of brace for which Whitmore length is considered, in.
D	Number of sixteenths-of-an-inch in fillet weld size
F	Force at hinge plate–gusset plate interface due to M_{hinge} , kips
F_y	Specified minimum yield stress of the type of material to be used, ksi
M_e	Required flexural strength of brace connection, kip-in.
M_{hinge}	Expected plastic moment strength of hinge plate, including strain hardening, kip-in.
M_{HSS}	Expected plastic moment strength of brace about critical buckling axis, including strain hardening, kip-in.
P	Representative axial load in brace, kips
R_y	Ratio of the expected yield stress to the specified minimum yield stress, F_y
T_u	LRFD factored axial tension force in brace, kips

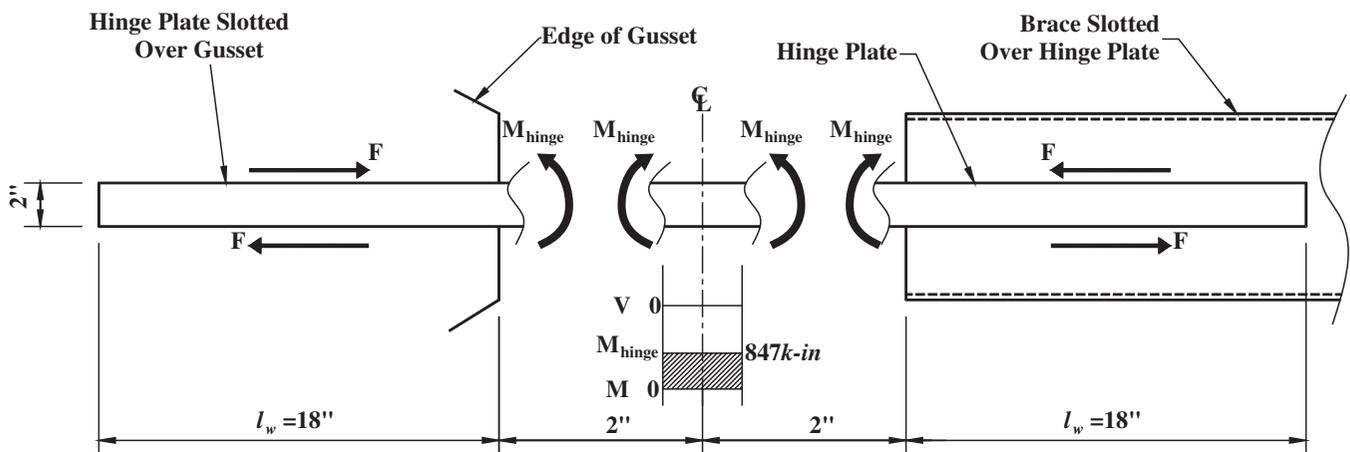


Fig. 9. M_{hinge} in hinge plate. Interface forces at the hinge plate-to-brace and hinge plate-to-gusset-connections.

Z	Plastic section modulus of member being considered, in. ³
l_w	Length of weld, in.
t	Gusset plate thickness, in.
t_h	Hinge plate thickness, in.
w_h	Width of hinge plate, in.

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