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95 Prying Action for Slip-Critical Connections with Bolt Tension and Shear Interaction
William A. Thornton and Larry S. Muir

99 Satisfying Inelastic Rotation Requirements for In-Plane Critical Axis Brace Buckling for High Seismic Design
William A. Thornton and Patrick J. Fortney

109 AASHTO LRFD Provisions for the Seismic Design of Steel Plate Girder Bridges
Ahmad M. Itani, Eric V. Monzon and Michael A. Grubb

115 Technical Note
Effective Length Factors for Gusset Plates in Chevron Braced Frames
Bo Dowswell

119 Current Steel Structures Research No. 31
Reidar Bjorhovde

127 Errata

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Prying Action for Slip-Critical Connections with Bolt Tension and Shear Interaction

WILLIAM A. THORNTON and LARRY S. MUIR

ABSTRACT

Bolted connections subjected to both shear and tension must be checked for prying action and the interaction between tension, and shear must be considered. The 2010 AISC *Specification for Structural Steel Buildings* (AISC 360-10) presents interaction equations both for bearing connections and for slip-critical connections. This paper demonstrates two methods to account for tension and shear interaction when prying action must be considered in slip-critical connections. The prying action procedure outlined in the 14th edition *Steel Construction Manual* is assumed.

Keywords: bolt tension, bolt shear, prying action, slip-critical connections.

INTRODUCTION

When bolted connections subjected to both shear and tension must be checked for prying action, the interaction between tension and shear must be considered. The AISC *Specification for Structural Steel Buildings* (AISC, 2010) presents interaction equations for bearing connections and for slip-critical connections. However, little guidance for applying these equations to prying action analysis has been available. This paper will demonstrate how these interaction equations may be used in the prying action analysis presented in the 14th edition *Steel Construction Manual* (AISC, 2011) by comparing two methods. This paper is formulated in terms of Load and Resistance Factor Design (LRFD), but the principles are similar for Allowable Strength Design (ASD).

The 14th edition *Steel Construction Manual* outlines a design approach for prying action on pages 9-10 through 9-13. The quantity B is used to represent the available tension strength per bolt. When there is no applied shear,

$$B = \phi r_t \tag{1}$$

where ϕr_t, the available tensile strength of the bolt, kips, is given in Table 7-2 of the *Manual* or is calculated from *Specification* Section J3.1 and Table J3.2.

William A. Thornton, Corporate Consultant, Cives Engineering Corporation, Roswell, GA (corresponding author). E-mail: bthornton@cives.com

Larry S. Muir, Structural Steel Consultant, Atlanta, GA. E-mail: larrymuir@larrymuir.com

BEARING-TYPE CONNECTIONS

For bearing connections, the presence of shear reduces the available tensile strength of the bolt. From *Specification* Section J3.7, Equation J3-3a (for LRFD; repeated here as Equation 2) calculates F'_{nt}, the nominal tensile stress modified to include the effects of shear stress:

$$F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_{rv} \le F_{nt} \tag{2}$$

where

A_b = the area of the bolt, in.^{2}

F_{nt} = nominal bolt tensile stress from Table J3.2, ksi

F_{nv} = nominal bolt shear stress from Table J3.2, ksi

f_{rv} = required bolt shear shear stress, ksi

= V_u/A_b

Substituting terms in Equation 1 and expanding terms incorporates F'_{nt} (ksi) and the area of the bolt, A_b (in.), producing Equation 3:

$$B' = \phi r'_t = \phi F'_{nt} A_b \tag{3}$$

Note that $\phi r'_t$ represents the reduced available tensile strength of the bolt, kips.

SLIP-CRITICAL CONNECTIONS

For slip-critical connections, the situation is somewhat different. When a tension is applied that reduces the net clamping force, the factor k_{sc} given by *Specification* Equation J3-5a is applied to the available slip resistance per bolt. Rewriting

Equation J3-5a for k_{sc} in terms of required tension force per bolt (rather than total tension force) and applying the factor to ϕr_v produces Equation 4 for $\phi r'_v$, the shear strength per bolt reduced by the applied tension:

$$\phi r'_v = \phi r_v \left(1 - \frac{T_u}{T_e}\right) \text{ with } T_u \le \min\{\phi r_t, T_e\} \tag{4}$$

where

T_e = the expected mean pretension per bolt, kips

= $D_u T_b$

T_u = required tension force per bolt, kips

ϕr_v = available shear strength per bolt for slip-critical connections, kips (*Manual* Table 7-3)

T_b = minimum bolt pretension, kips (*Specification* Table J3.1)

D_u = calibration factor, usually 1.13 (*Specification* Section J3.8)

In Equation 4, the limit on T_u is necessary because T_e is less than ϕr_t for ASTM A325 bolts with diameters larger than 1 in. This anomaly occurs because the 2010 *Specification* uses a minimum specified tensile strength of 120 ksi for all ASTM A325 bolt diameters. However, the ASTM A325 standard uses 120 ksi for bolts up to and including 1-in. diameter and then uses 105 ksi for larger bolts. The pretension values (T_b) in *Specification* Table J3.1 are based on the ASTM values, while *Specification* Table J3.2 uses 120 ksi "across-the-board" for all ASTM A325 bolt diameters. The difference occurs only for ASTM A325 bolts; for ASTM A490 bolts, both Table J3.1 and Table J3.2 are based on the ASTM minimum tensile strength value of 150 ksi.

Note that in *Specification* Table J3.2, the values of F_{nt} are 75% of the bolt tensile strength F_u. Thus, for ASTM A325 bolts, F_{nt} = 0.75 × 120 ksi = 90 ksi and for ASTM A490 bolts, F_{nt} = 0.75 × 150 ksi = 113 ksi. The factor 0.75 is the ratio of the threaded area to the shank area.

For bearing connections, Equation 3 produces a reduced available tensile strength, $\phi r'_t$, due to the presence of shear, V_u. For slip-critical connections, Equation 4 produces a reduced shear strength, $\phi r'_v$, due to the presence of applied bolt tension, T_u.

Note that in Equation 4, T_u does not include the prying force, q. The reason for this is that the total faying surface compression force is not reduced by q. The bolt tension, T_u, is increased by q, but an equal and opposite q acts as an additional compression force on the faying surface. Thus, the slip-critical shear resistance—while reduced by the applied tension, T_u—is unaffected by the prying force, q.

The slip-critical interaction Equation 4 may be mathematically rearranged to produce Equation 5:

$$\phi r'_t = T_e \left(1 - \frac{V_u}{\phi r_v}\right) \le \min\{\phi r_t, T_e\} \text{ with } V_u \le \phi r_v \tag{5}$$

where V_u is the applied shear per bolt, kips, and all other terms are as previously defined. Similar to Equation 3, $B' = \phi r'_t$; thus, Equation 5 produces the value B' required for prying action calculations.

In spite of its mathematical relationship to Equation 4, Equation 5 does not accurately represent the physical behavior of slip-critical connections. The reason that *Specification* Equation J3-5a (Equation 4 of this paper) is written in terms of a reduced shear stress is as follows: while T_u affects slip-critical connection shear strength per bolt as shown in Equation 4, applied shear, V_u, does not affect the tensile strength of the bolt in quite the same manner, even though Equation 5 would indicate otherwise. The reason for this lies in the physical behavior of slip-critical connections. Connection shear V_u is carried by the faying surface through friction—rather than by the bolt shank as Equation 5 appears to indicate—until slip occurs. Thus, the bolt itself "sees" no shear until the connection slips, and its tensile strength is consequently unaffected until slip. Once slip occurs, bearing interaction Equation J3-3a from the *Specification* and Equations 2 and 3 of this paper must be used.

SOLUTION STRATEGIES

Method A. Use the mathematically inverted *Specification* Equation J3-5a, rearranged as Equation 5, to solve for B' for use in prying action calculations:

$$B' = \phi r'_t = T_e \left(1 - \frac{V_u}{\phi r_v}\right) \le \min\{\phi r_t, T_e\}$$

As previously discussed, this approach does not capture the physical behavior of the connection because it does not account for the pre-slip condition. However, Equation 5 provides a conservative solution. This approach has been previously presented the literature by Thornton (1985), Brockenbrough (2006) and Tamboli (2010).

Method B. Account for pre-slip and post-slip behavior as follows:

Step 1 Calculate the slip-critical shear strength as reduced by the applied tension, using Equation 4:

$$\phi r'_v = \phi r_v \left(1 - \frac{T_u}{T_e}\right) \text{ with } T_u \le \min\{\phi r_t, T_e\}$$

Step 2 If $\phi r'_v < V_u$, the slip-critical shear strength is insufficient and the connection fails.

Step 3 If $\phi r'_v \geq V_u$, the connection is in the "pre-slip" state. Use Equations 2 and 3 to calculate B' for the post-slip state (bearing):

$$B' = \phi r'_t = \phi F'_{nt} A_b$$

$$F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_{rv} \leq F_{nt}$$

Once the value B' is determined, the calculations follow the process outlined in the 14th edition *Manual* for both methods.

EXAMPLE

Given: ASTM A325 bolts, ⅞-in. diameter, slip-critical, Class A faying surface, oversize holes, threads included in the shear plane. $V_u = 5.56$ kips/bolt and $T_u = 22.0$ kips/bolt.

$D_u = 1.13$ from *Specification* Section J3.8

$T_b = 39$ kips from *Specification* Table J3.1

$T_e = D_u \times T_b = 1.13 \times 39 = 44.1$ kips

$\phi r_v = 11.2$ kips from *Manual* Table 7-3

$F_{nv} = 54$ ksi from *Specification* Table J3.2

$\phi r_t = 40.6$ kips from *Manual* Table 7-2

$F_{nt} = 90$ ksi from *Specification* Table J3.2

Solution A:

From Equation 5,

$$B' = \phi r'_t = T_e\left(1 - \frac{V_u}{\phi r_v}\right)$$

$$B' = 44.1 \text{ kips}\left(1 - \frac{5.56 \text{ kips}}{11.2 \text{ kips}}\right)$$

$$= 22.2 \text{ kips/bolt} \leq \phi r_t = 40.6 \text{ kips/bolt} \quad \textbf{OK}$$

Use $B' = 22.2$ kips/bolt.

Solution B:

Step 1 Check Equation 4:

$$\phi r'_v = \phi r_v\left(1 - \frac{T_u}{T_e}\right) \text{ with } T_u \leq \min\{\phi r_t, T_e\}$$

$$T_u \leq \min\{\phi r_t, T_e\}$$

$$\leq \min\{40.6 \text{ kips}, 44.1 \text{ kips}\} = 40.6 \text{ kips}$$

$$T_u = 22.0 \text{ kips} \leq 40.6 \text{ kips} \quad \textbf{OK}$$

$$\phi r'_v = 11.2 \text{ kips}\left(1 - \frac{22.0 \text{ kips}}{44.1 \text{ kips}}\right)$$

$$= 5.56 \text{ kips/bolt}$$

Step 2 $\phi r'_v = 5.56 \text{ kips} \geq V_u = 5.56$ kips **OK to proceed**

Step 3 Calculate F'_{nt} using Equation 3:

$$F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_{rv} \leq F_{nt}$$

$$= 1.3(90 \text{ ksi}) - \frac{90 \text{ ksi}}{(0.75)(54 \text{ ksi})}\left(\frac{5.56 \text{ kips/bolt}}{0.601 \text{ in.}^2}\right)$$

$$= 96.4 \text{ ksi}$$

$$F'_{nt} = 96.4 \text{ ksi} > F_{nt} = 90 \text{ ksi}$$

Use $F'_{nt} = 90$ ksi.

Calculate B' using Equation 2:

$$B' = \phi r'_t = \phi F'_{nt} A_b$$

$$= (90 \text{ ksi})(0.75)\left(0.601 \text{ in.}^2\right)$$

$$= 40.6 \text{ kips/bolt} \leq \phi r_t = 40.6 \text{ kips/bolt} \quad \textbf{OK}$$

Use $B' = 40.6$ kips/bolt.

SUMMARY AND CONCLUSIONS

Method A gives $B' = 22.2$ kips/bolt while Method B gives $B' = 40.6$ kips/bolt. Method A is conservative but does not capture the pre-slip/post-slip phenomenon. Method B captures the pre-slip/post-slip phenomenon and is less conservative. The authors recommend the use of Method B because it is based on a mathematical model that more closely resembles the connection behavior.

Kulak et al. (1987) indicate that, at ultimate loads, the effects of prying will be the same regardless of whether the bolts are pretensioned or not. This observation further supports the use of Method B. Another observation by the authors of this paper suggests that Method B may also prove to be overly conservative in practice. This point is easiest to see if we transition to ASD and look at service loads. *Specification* Equation J3-1 calculates the tensile strength of bolts using $\Omega = 2.00$. Tension service loads, T_a, must satisfy the following:

$$T_a \leq \frac{R_n}{\Omega} = \frac{F_{nt} A_b}{\Omega} = \frac{0.75 F_u A_b}{2.00} = 0.375 F_u A_b$$

The specified pretension values from *Specification* Table J3.1 are approximately 70% of the tensile strength of the bolt: $T_b \approx 0.70F_uA_b$. Thus, at service loads the applied tension—limited to $0.375F_uA_b$—is less than the tensile strength of the bolt—$0.70F_uA_b$—and the bearing behavior reflected in Equation 3 will never occur. This is true for both the ASD and LRFD design approaches. Further study is necessary to establish how—or if—this reality should be incorporated into the design process.

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

WILLIAM A. THORNTON and PATRICK J. FORTNEY

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:

$$M_e = 1.1R_yF_yZ \tag{1}$$

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

William A. Thornton, Corporate Consultant, Cives Engineering Corporation, Roswell, GA (corresponding author). E-mail: bthornton@cives.com

Patrick J. Fortney, Manager and Chief Engineer, Cives Engineering Corporation, Roswell, GA. E-mail: pfortney@cives.com

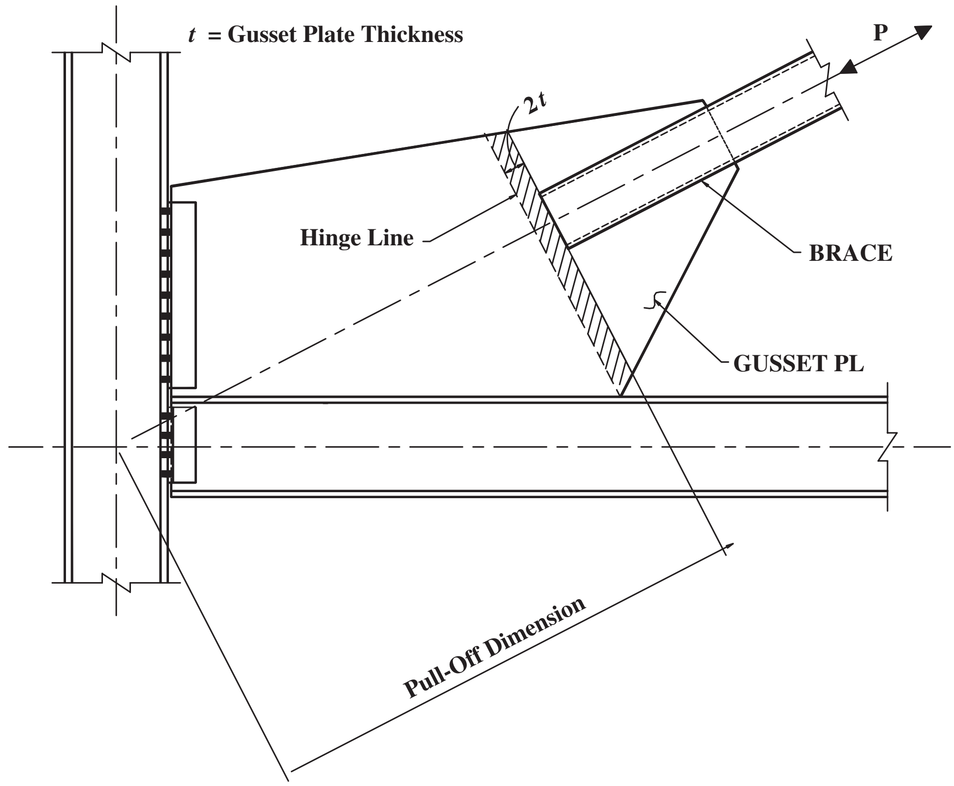

Fig. 1. Gusset hinge line.

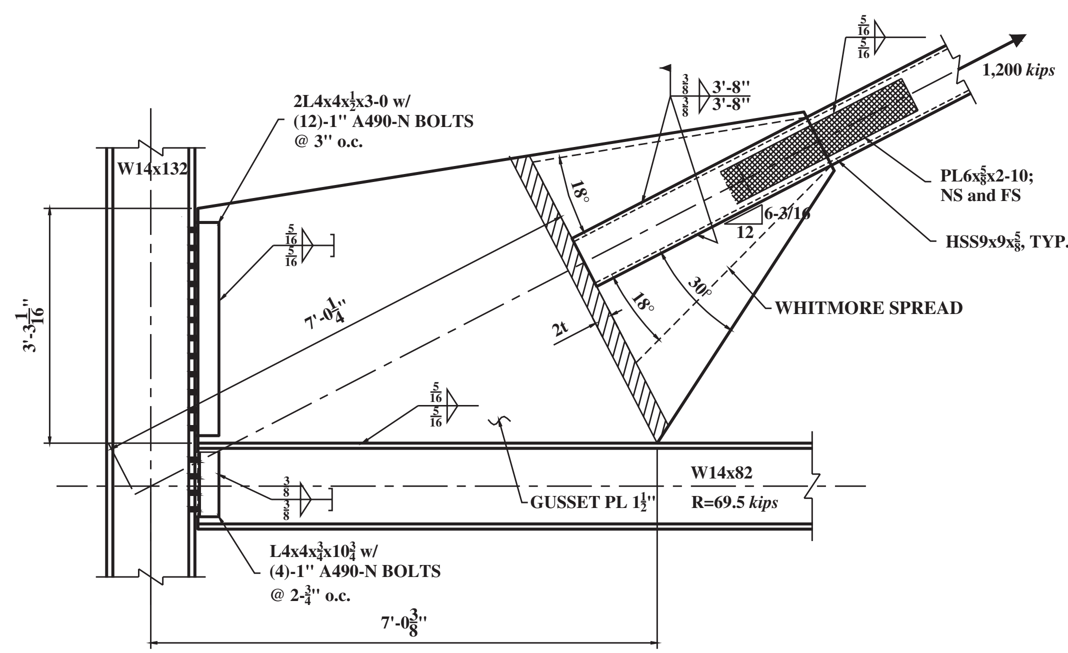

Fig. 2. Brace connection in an SCBF using an 18° Whitmore section.

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

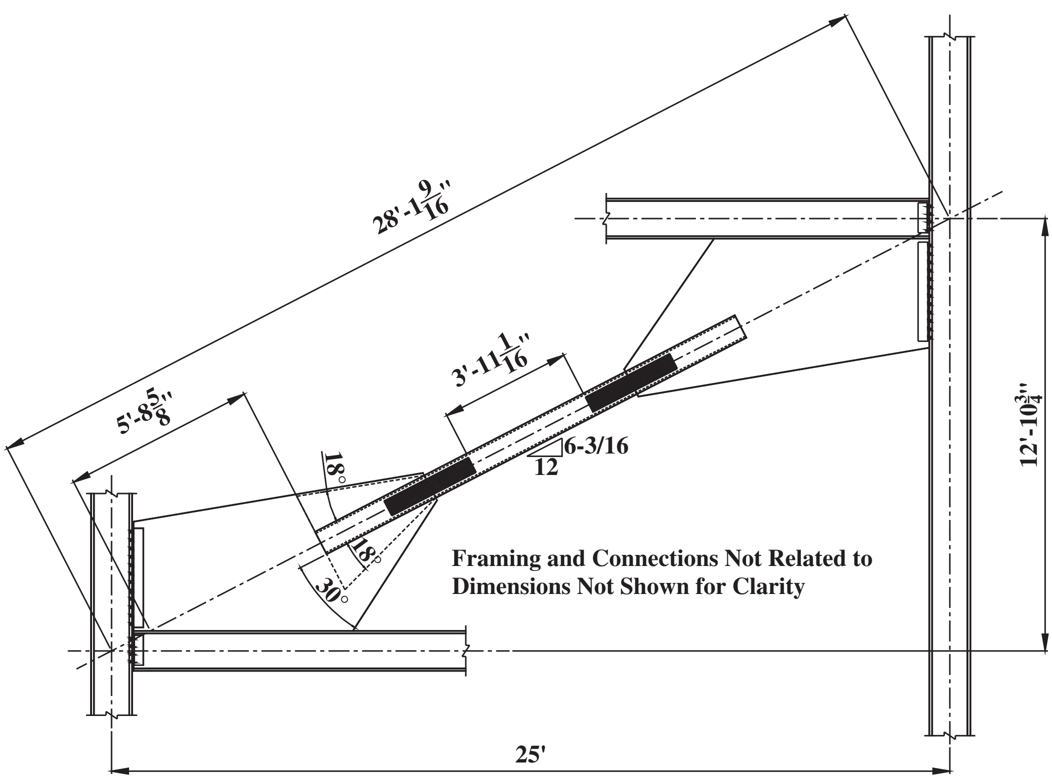

Fig. 3. Gusset and brace dimensions with an 18° (top) and 30° (bottom) Whitmore spread. Refer to Fig. 2 for details of brace connection.

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 1⁹⁄₁₆ 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.

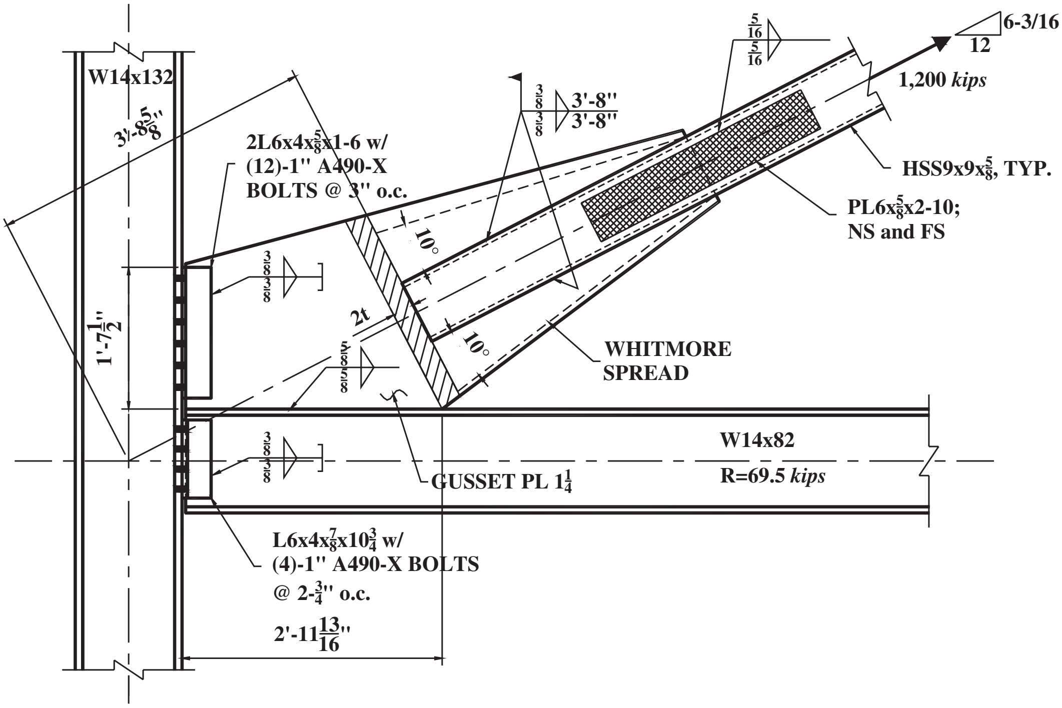

Fig. 5. Brace connection in an SCBF using a 10° Whitmore spread.

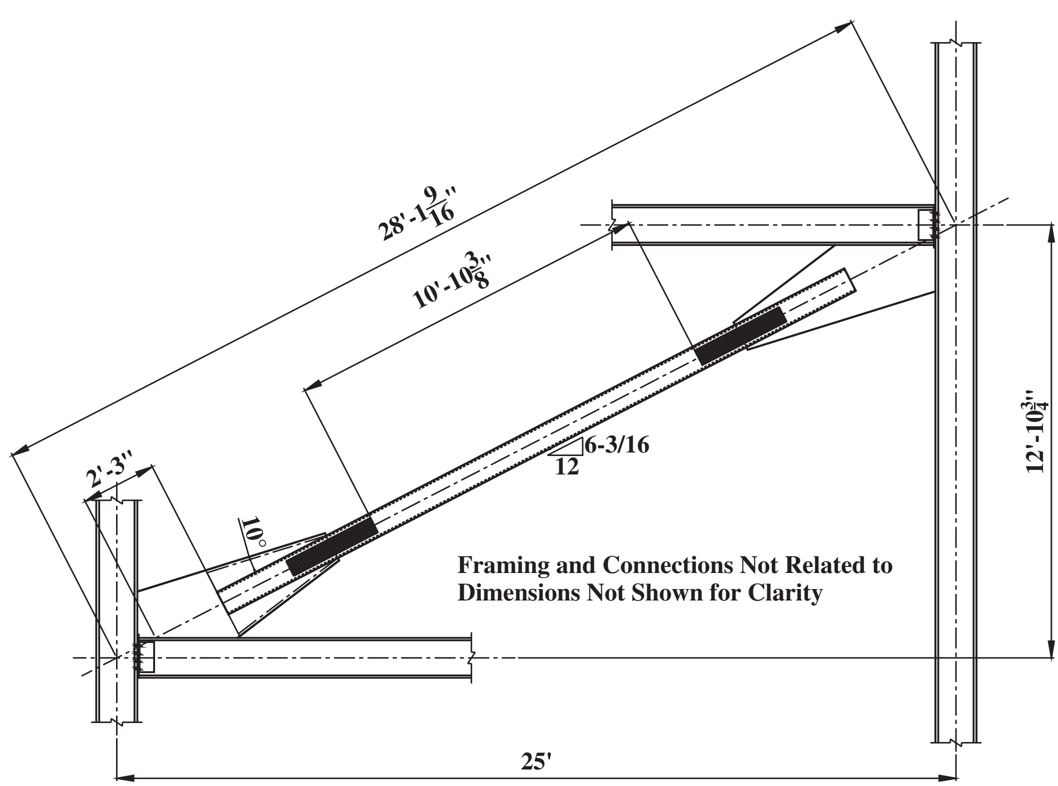

Fig. 6. Gusset and brace dimensions using a 10° Whitmore spread. Refer to Fig. 5 for details of the brace connection.

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 9×9×⅝ 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

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

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

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

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

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

and the flexural strength of the HSS 9×9×⅝ is

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

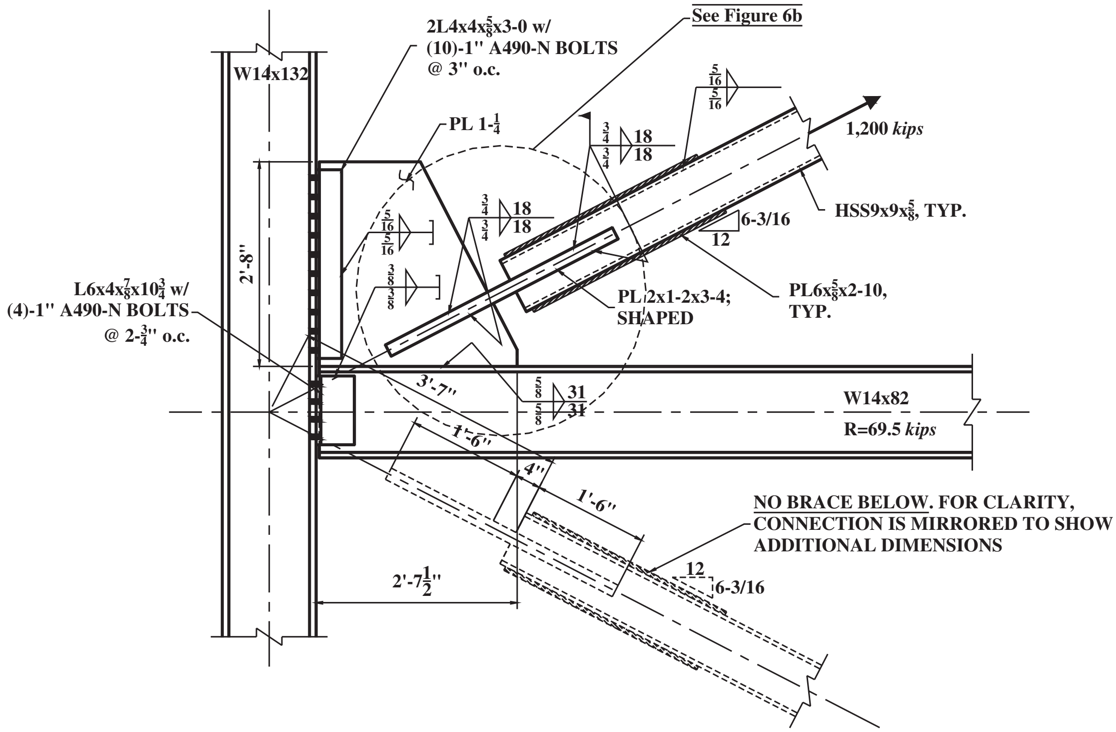

Fig. 7a. Illustration of in-plane buckling with the hinge line concept. See Fig. 7b for an enlarged view of the hinge plate.

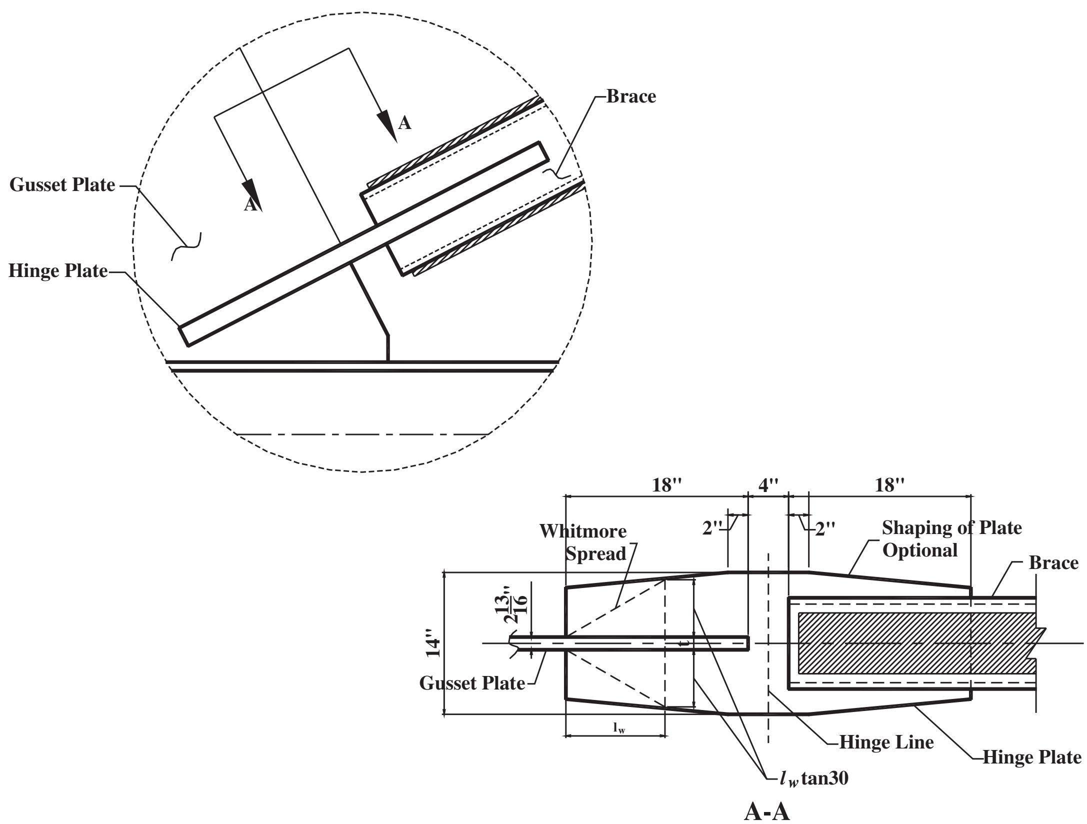

Fig. 7b. Enlarged view of the hinge plate shown in Fig. 7a.

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

$$D = \frac{F}{1.392l(\text{number of welds})}$$

$$= \frac{424 \text{ kips}}{(1.392)(18 \text{ in. per weld})(2 \text{ welds})}$$

$$= 8.44 \text{ sixteenths}$$

Because 8.44 sixteenths of an inch of weld is less than the provided ¾-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.

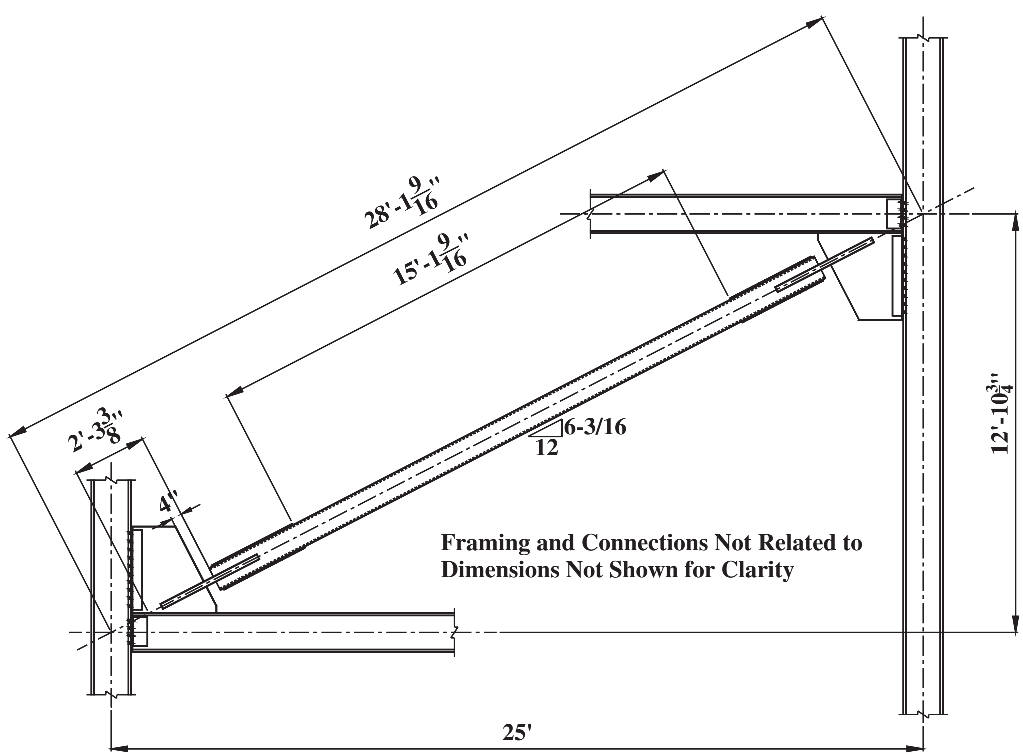

Fig. 8. Gusset and brace dimensions using the hinge plate concept. Refer to Fig. 7 for details of the brace connection.

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°) 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°), 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,

$$\begin{aligned} w_h \le t + 2l_w(\tan 30°) &= 1.25 \text{ in.} + (2)(18 \text{ in.})(\tan 30°) \\ &= 22.0 \text{ in.} \end{aligned}$$

Therefore, because $w_h = 14$ in. ≤ 22 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

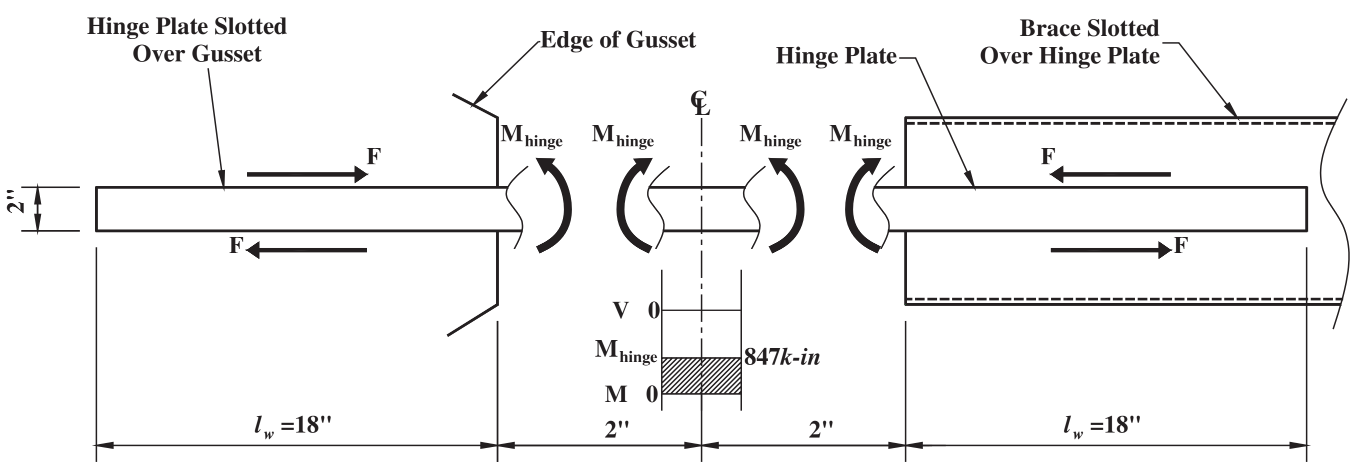

Fig. 9. M_{hinge} in hinge plate. Interface forces at the hinge plate-to-brace and hinge plate-to-gusset-connections.

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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AASHTO LRFD Provisions for the Seismic Design of Steel Plate Girder Bridges

AHMAD M. ITANI, ERIC V. MONZON and MICHAEL A. GRUBB

ABSTRACT

Recent earthquakes have exposed the vulnerability of steel plate girder superstructures to seismic forces. Damage has occurred in cross frames and their connections, shear connectors, and steel plate girders. These earthquakes have revealed the shortcomings of U.S. bridge design specifications for these types of bridges. Section 6 of the AASHTO LRFD specifications does not have any seismic design provisions for steel plate girder bridges. Recently, these specifications have adopted seismic design provisions that are proposed by the authors for steel superstructures to overcome this shortcoming. The adopted specifications are the result of analytical and experimental investigations by various researchers and work published by many seismic provisions and guide specifications. This paper summarizes the new seismic design provisions and outlines the background behind them.

Keywords: bridges, seismic design, AASHTO provisions, plate girders, cross frames, shear connectors.

INTRODUCTION

The AASHTO Load and Resistance Factor Design (LRFD) bridge design specifications (AASHTO, 2010) do not require the explicit design of concrete or steel bridge superstructures for earthquake loads. It is implicitly assumed that the superstructure that is designed for dead and live loads will have sufficient strength, by default, to resist earthquake loads. This assumption appears to be justified for structural concrete box girder superstructures, which are heavier and stiffer than steel plate girder superstructures. However, during the recent earthquakes of Northridge, Kobe and Nisqually, several components of steel plate girder superstructures experienced inelastic response and premature failure (Itani et al., 2010). This showed that these superstructure components were in the seismic load path and were subjected to seismic forces for which they were not designed. Therefore, improvement in the seismic performance of steel bridges is warranted, along with design provisions for steel superstructures. Better insight is required regarding the seismic load path, as well as the resistance of individual components and assembled systems.

Steel plate girder bridges have generally suffered minor to moderate damage in past earthquakes compared with the significant damage suffered by structural concrete structures. However, these earthquakes have identified critical components in the superstructure and substructure that should be designed and detailed to resist seismic demand. The common thread among these earthquakes is that the components of steel plate girder superstructures are vulnerable during seismic events and need to be designed and detailed to resist the seismic forces without premature failure and fracture. Failure in the superstructure components will interrupt the seismic load and will alter the overall seismic performance of such bridges.

The 1992 Petrolia earthquakes in northern California (Caltrans, 1992) exposed the importance of the support cross frames and the shear connectors in steel plate girder superstructures in transferring the seismic forces. The South Fork Eel River Bridge, a curved steel plate girder bridge, suffered considerable damage, including buckling and fracture of end cross frames and their connections and damage to the reinforced concrete deck at support locations. This earthquake highlighted the significance of shear connectors in transferring the lateral forces that are generated by the mass of the deck. The Northridge (Astaneh-Asl et al., 1994) and Kobe earthquakes (Bruneau et al., 1996) showed similar damage to support cross frames and their connections in addition to the damage of the steel plate girders at bents and abutment locations.

These earthquakes confirmed the vulnerability of steel-girder bridges during seismic events. New areas of concern that emerged included:

- Lack of understanding of the seismic load paths in steel-girder bridges.

Ahmad M. Itani, Ph.D., S.E., Professor, Department of Civil and Environmental Engineering, University of Nevada, Reno, NV (corresponding author). E-mail: Itani@unr.edu

Eric V. Monzon, Ph.D., Graduate Student Research Assistant, Department of Civil and Environmental Engineering, University of Nevada, Reno, NV. E-mail: emonzon@unr.edu

Michael A. Grubb, P.E., Structural Engineer, Bridge Software Development International Ltd. (BSDI), Coopersburg, PA. E-mail: mgrubb@zoominternet.net

- Damage to steel superstructure components such as girders, shear connectors, end cross frames, bearing stiffeners, bearings and anchor bolts.
- Failure of steel substructures.

BEHAVIOR OF STEEL PLATE GIRDER BRIDGES UNDER LATERAL LOADING

Earthquake loading in the transverse direction causes transverse bending of the superstructure, resulting in transverse reactions at the abutments and bents. Consequently, the loads are distributed from the middle of each span to the supports. Because the reinforced concrete deck and concrete traffic barriers in a steel plate girder bridge typically account for around 80% of the bridge, the majority of the inertia loads are generated in the deck slab. The bearing supports are at the bottom flange of the girders; thus, the inertia loads need to be distributed down through the superstructure components. Numerical analyses have shown that these loads are largely distributed through the deck to the ends of each span. The seismic forces are distributed vertically through the abutment and bent cross frames (Itani, 1995; Itani and Rimal, 1996; Zahrai and Bruneau, 1999a). These forces are then transmitted to the bearings and shear keys at support locations. Because the primary function of the bearings is to allow the bridge to expand and contract longitudinally due to temperature variation, the bearings are usually restrained from translation in the transverse direction. Thus, the transverse shear forces in the bearings are transferred into the abutments and bents.

To ensure favorable transverse seismic load path, adequate composite action should be provided between the girders and the deck for transverse earthquake loading. Analytical investigation by Carden et al. (2002) showed the importance of having shear connectors along the entire length of the bridge. If shear connectors were not used over the negative moment regions, the entire transverse load path will be altered. Consequently, the intermediate cross frames between support locations will be subjected to significant seismic forces. Therefore, it is recommended in seismic zones that shear connectors be placed on the girders over the entire length of the bridge and over the top chord of support cross frames to ensure that the seismic forces will be transferred to the substructure. Experimental investigation (Bahrami et al., 2010) showed that attaching the top chord of the support cross frames to the reinforced concrete deck facilitated the transfer of the earthquake loads directly from the deck into the cross frames. The results of this experimental investigation showed that the shear connectors at support locations are subjected to tension forces in addition to lateral shear. This tension force can be substantial and may cause the failure of the connectors, thus interrupting the seismic load path.

End cross frames or diaphragms—elements placed transversely between the plate girders at the supports—have been identified analytically (Itani and Rimal, 1996; Astaneh-Asl and Donikian, 1995; Zahrai and Bruneau, 1998; Dicleli and Bruneau, 1995a, 1995b) and experimentally (Zahrai and Bruneau, 1999a, 1999b; Carden et al., 2005; Bahrami et al., 2010) as critical components in the transverse seismic load path. These members are designed and detailed as secondary members for straight steel bridges but become primary members at support locations responsible for transferring the seismic forces from the deck to the bearings. Any failure in these members will interrupt the seismic load path and alter the overall seismic response of the bridge.

SEISMIC DESIGN SPECIFICATIONS FOR STEEL BRIDGES

The 1971 San Fernando earthquake demonstrated the vulnerability of structural concrete box girder bridges to seismic forces (California Department of Public Works, 1971). Several bridges of the aforementioned type suffered complete collapse. Recognizing the urgent need for new design provisions, the California Department of Transportation (Caltrans) began to develop new criteria for the seismic design of bridges. The structural system of concrete box girders with monolithic connection between the superstructure and the substructure dictated that the inelasticity should occur in the column. Thus, the concept of “weak substructure–strong superstructure” emerged in the seismic design of highway bridges. No attention was given to steel bridges due to the fact that only one steel plate girder bridge (San Fernando Road Overhead) was damaged as a result of the bearing failure and short seat width (California Department of Public Works, 1971). Subsequently, the seismic design guidelines for bridges did not present information on the seismic design of steel plate girder bridges. This shortage of information continued in the 5th edition of the AASHTO LRFD specifications (AASHTO, 2010). Prior to May 2011, the AASHTO LRFD specifications had no provisions for the seismic design of steel bridges.

In an effort to remedy this lack of information, the AASHTO guide specification (AASHTO, 2009) offered provisions for the seismic design of steel bridges. However, the guide specifications lacked the depth and the breadth for the seismic design of steel plate girder bridge components. Furthermore, the basic design methodology of the guide specification is displacement-based, but for steel bridges the design methodology is force-based. This discontinuity in the guide specification forces bridge engineers to use two distinct specifications for the seismic design of steel bridges.

NEW AASHTO LRFD SEISMIC DESIGN PROVISIONS FOR STEEL PLATE GIRDER BRIDGES

An effort was undertaken to synthesize the available experimental research, analytical research and seismic guidelines to establish seismic provisions that could be adopted to be part of the AASHTO LRFD Section 6 (Itani et al., 2010). These new provisions are based on the recent work published by Itani et al. (2010), NCHRP (2002, 2006), ATC/MCEER (2003), Caltrans (2001), AASHTO (2009), and AISC (2005). These provisions are limited to the seismic design and details of steel-girder bridge superstructure components.

The new provisions for seismic design are presented under Article 6.16 of the AASHTO LRFD specifications. The overarching requirements for all seismic zones is the importance of seismic load path, minimum support length and capacity design to ensure that connections stay elastic where any expected inelasticity is limited to the members. An overview of the new provisions and background behind them are presented in the rest of this section.

General

The provisions require a clear seismic load path to be established within the superstructure to transmit the inertia forces to the substructure based on the stiffness characteristics of the concrete deck, cross frames or diaphragms and bearings. The flow of the seismic forces is accommodated through all affected components and connections of the steel superstructure within the prescribed load path, including, but not limited to, the longitudinal girders, cross-frames or diaphragms, steel-to-steel connections, deck-to-steel interface, bearings and anchor bolts.

Materials

Previous earthquakes, analyses and experimental investigations have shown that cross frames at support locations transfer the inertia forces from the superstructure to the substructure. Therefore, the connections of the adjoining cross-frame members must be protected during seismic events. This is achieved by utilizing a capacity-design methodology in which the cross-frame connections are designed based on the expected nominal resistance of the adjoining members. In the capacity-design methodology, all the components surrounding the nonlinear element are designed based on the maximum expected nominal resistance of that element. The capacity-design methodology requires a realistic estimate of the expected nominal resistance of the designated yielded members. To this end, the expected yield strength of various steel materials has been established through a survey of mill test reports, and ratios of the expected to nominal yield strength, R_y, have been provided by AISC (2005) and are adopted herein. The expected resistance of the designated member is therefore to be determined based on the expected yield strength, R_yF_y.

Design Requirements for Zone 1

For steel-girder bridges located in Seismic Zone 1, defined as specified in AASHTO (2010), no consideration of seismic forces is required for the design of the superstructure components—except that the design of the connections of the concrete deck to the girder at all support cross-frame or diaphragm locations, the connections of all support cross-frame or diaphragm members, and the connections of the superstructure to the substructure shall satisfy the minimum requirements specified in specifications.

Design Requirements for Seismic Zones 2, 3 or 4

The seismic performance criterion for steel plate girder bridges is to be classified into one of the following two response strategies:

- Type 1: Design an elastic superstructure with a ductile substructure.
- Type 2: Design an elastic superstructure and substructure with a fusing mechanism at the interface between them.

Type 1 represents the conventional seismic design response strategy in which the superstructure stays in elastic range while the inelasticity is limited to the substructure. The provision of an alternative fusing mechanism, Type 2, between the interface of the superstructure and substructure by shearing off the anchor bolts is also an adequate seismic strategy in the new provisions. However, it is important to mention here that caution must be taken to provide adequate seat width and to stiffen the girder web at support locations. It is anticipated that large deformations will occur in the superstructure at support locations during a seismic event when this strategy is employed.

The reinforced concrete deck and shear connectors are to be designed and detailed for the seismic forces. Support cross-frame members in either category are considered primary members for seismic design. Structural analysis for seismic loads will consider the relative stiffness of the concrete deck, girders, support cross-frames or diaphragms and the substructure.

Reinforced Concrete Deck

In general, reinforced concrete decks on steel-girder bridges with adequate stud connectors have sufficient rigidity in their horizontal plane that their response approaches rigid-body motion. Therefore, the deck can provide a horizontal diaphragm action to transfer seismic forces to support cross frames or diaphragms. The seismic forces are collected at the support cross frames or diaphragms and transferred to the substructure through the bearings and anchor bolts. Thus, the support cross frames or diaphragms must be designed for the resulting seismic forces. The lateral loading of the intermediate cross frames in between the support locations for straight bridges is minimal in this case, consisting primarily of the local tributary inertia forces from the girders. Adequate stud connectors are required to ensure the necessary diaphragm action; previous earthquake reconnaissance showed that, for some bridges in California in which the shear connectors at support locations were damaged during a seismic event, the deck in fact slid on the top of the steel girders (Roberts, 1992; Carden et al., 2005).

During a seismic event, inertia forces generated by the mass of the deck must be transferred to the support cross frames or diaphragms. The seismic forces are transferred through longitudinal and transverse shear forces and axial forces. The transverse seismic shear force on the deck, F_{px}, within the span under consideration shall be determined as:

$$F_{px} = \frac{W_{px}}{W} F \tag{1}$$

where

F = total of the transverse base shears, as applicable, at the supports in the span under consideration, kips

W = total weight of the deck and steel girders within the span under consideration, kips

W_{px} = weight of the deck plus one-half the weight of the steel girders in the span under consideration, kips

In cases where the deck can be idealized as a rigid horizontal diaphragm, F_{px} is distributed to the supports based on their relative stiffness. In cases where the deck must be idealized as a flexible horizontal diaphragm, F_{px} is distributed to the supports based on their respective tributary areas. Decks idealized as rigid diaphragms need only be designed for shear. Decks idealized as flexible diaphragms must be designed for both shear and bending because maximum in-plane deflections of the deck under lateral loads in this case are more than twice the average of the lateral deflections at adjacent support locations. Concrete decks may be designed for shear and bending moments based on strut and tie models.

In cases where the deck cannot provide horizontal diaphragm action, the engineer should consider providing lateral bracing to serve as a horizontal diaphragm to transfer the seismic forces.

Shear Connectors

Stud shear connectors play a significant role in transferring the seismic forces from the deck to the support cross frames or diaphragms. These seismic forces are transferred to the substructure at support locations. Thus, the shear connectors at support locations are subjected to the largest seismic forces unless reinforced concrete diaphragms connected integrally with the bridge deck are used. Failure of these shear connectors will cause the deck to slip on the top flange of the girder, thus altering the seismic load path (Caltrans, 2001; Carden et al., 2005, Bahrami et al., 2010).

The shear center of composite steel-girder superstructures is located above the deck (Zahrai and Bruneau, 1998; and Bahrami et al. 2010). Therefore, during a seismic event the superstructure will be subjected to torsional moments along the longitudinal axis of the bridge that produce axial forces on the shear connectors in addition to the longitudinal and transverse shears. Lateral deformations during a seismic event produce double curvature in the top chord of the cross frame, creating axial forces in the shear connectors on that member that must be considered. Experimental and analytical investigations (Carden et al., 2002; Bahrami et al., 2010) showed that the seismic demand on shear connectors that are placed only on the girders at support locations may cause significant damage to the connectors and the deck.

Appendix D of the ACI specification (2008) provides equations for anchorage to concrete of pre- and post-installed anchors subject to shear and axial forces. However, these equations are not used herein for the design of shear connectors on slab-on-steel-girder bridges subject to combined shear and axial forces. Mouras et al. (2008) investigated the behavior of shear connectors placed on a steel girder under static and dynamic axial loads. The effects of haunches in reinforced concrete decks, stud length, the number of studs and the arrangement of the studs in the transverse and longitudinal directions of the bridge were investigated. Based on this investigation, several modifications were recommended to the ACI Appendix D equations that are reflected in the equations in the AASTHO specifications. These modifications ensure a ductile response of the shear connectors that is beneficial in seismic applications. The modifications are as follows:

- Provision for adequate embedment of the shear connectors to engage the reinforcement in the deck slab.
- Use of an effective haunch height instead of the effective height given in the ACI Appendix D equations.
- Consideration of a group modification factor for longitudinal and transverse spacing. This factor accounts for the overlapping of the cones when studs are closely spaced.

The shear connectors on the girders assumed effective at the support under consideration shall be taken as those spaced no further than $9t_w$ on each side of the outer projecting element of the bearing stiffeners at that support. The diameter of the shear connectors within this region shall not be greater than 2.5 times the thickness of the top chord of the cross frame or top flange of the diaphragm. This requirement is new for the AASHTO specifications because shear connectors may be placed over cross-frame top chords.

At support locations, shear connectors on the girders and/or on the support cross frames or diaphragms, as necessary, are designed to resist the combination of shear and axial forces corresponding to the transverse seismic shear force, F_{p}. Experimental investigation by Bahrami et al. (2010) showed that the modified ACI equations for the shear and axial resistance and their interaction can be used to satisfactorily determine the resistance of stud shear connectors under the combined loading effects.

Elastic Superstructure

To achieve an elastic superstructure, the various components of the support cross frames or the support diaphragms, as applicable, must be designed to remain elastic under the forces that are generated during the design earthquake. The superstructure and its components should be capacity protected based on the material expected strength and overstrength of the ductile element. No other special seismic requirements are specified for these members in this case. The elastic superstructure can have steel cross frames of various configurations, steel diaphragms or reinforced concrete diaphragms. The Tennessee Department of Transportation and Caltrans have, as an alternative, used reinforced concrete diaphragms over bent locations. The details of these diaphragms and others are discussed in Bahrami et al. (2010) and Itani and Reno (1995).

CONCLUSIONS

The AASHTO LRFD specifications for the seismic design of steel bridges are relatively limited compared to those for concrete bridges. This is partly because the AASHTO specifications assume that all bridge superstructures have sufficient in-plane strength by default and remain elastic during the design earthquake. Thus, no special provisions are required for their seismic design, apart from requiring that a continuous load path be identified and designed for strength. While this may be a satisfactory approach for concrete superstructures—concrete box girder superstructures in particular—it is not necessarily the case for steel plate girder superstructures. Steel-girder superstructures may be vulnerable to collapse during seismic events if they are not designed and detailed properly to resist the seismic motions. Recent moderate earthquakes around the world have shown that a continuous seismic load path should be clearly defined, analyzed and designed to transmit the superstructure inertia forces to the substructure in order to prevent significant damage to the steel superstructure. Seismic design specifications summarized herein were recently included in Section 6 of the AASHTO LRFD specifications in a new Article 6.16. The new provisions are based on recent work published by NCHRP, ATC/MCEER, AASHTO [such as the *Guide Specifications for Seismic Design* (AASHTO, 2009)], Caltrans, and AISC. The new provisions are for the seismic design of steel plate-girder bridge superstructures located in Seismic Zones 2, 3 and 4. Bridges in Seismic Zones 3 or 4 are to be classified into one of the following two categories for seismic design: Type 1, an elastic superstructure with a ductile substructure; or Type 2, an elastic superstructure and substructure with a fusing mechanism at their interface. Bridges in Seismic Zone 2 may be classified into one of these two categories at the owner's discretion. Provisions for the seismic design of the superstructure components including the concrete deck, stud shear connectors, and support cross frames were summarized in this paper.

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TECHNICAL NOTE

Effective Length Factors for Gusset Plates in Chevron Braced Frames

BO DOWSWELL

In a previous paper by the author of this technical note (Dowswell, 2006), effective length factors were proposed for stability calculations of five different gusset plate configurations shown in Figure 1. These effective length factors will be referenced in the next edition of the AISC *Seismic Design Manual*.

The value of k for chevron braces (Figure 1e) has been discussed at several meetings of the AISC Committee on Manuals and Textbooks. Those discussions indicated that the value of $k = 0.75$ as recommended in Dowswell (2006) may be slightly conservative. In light of the common use of $k = 0.65$ for chevron gusset plates, the available test data were reevaluated to determine if the value of $k = 0.75$ as proposed in the original paper could be reduced.

The nominal values for $k = 0.75$ for chevron gussets from Dowswell (2006) are shown in Table 1 (Table 6 from Dowswell, 2006). Calculations for nonstiffened chevron gusset plates were made using $k = 0.65$ with the 13 specimens from Table 1. The nominal load, P_{calc}, was calculated with the actual yield strength reported for the tests and does not include a safety factor or reduction factor. The results for all specimens are shown in Table 2. An example calculation for specimen 1 is provided at the end of this technical note.

Using an effective length factor of 0.65, the mean ratio of experimental load to calculated capacity, P_{exp}/P_{calc} is 1.17 and the standard deviation is 0.19. Therefore, the use of $k =$ 0.65 for chevron gusset plates is safe and is proposed here for design use. Table 3 is a revised version of Table 7 of the original paper, updated to reflect this recommendation.

When a vertical stiffener is welded at the center of the gusset plate—as is common in seismic design—the behavior is similar to that of corner gusset plates (Tsai et al., 2004). In that case, the effective lengths proposed by Dowswell (2006) for compact, noncompact and extended corner gusset plates (Figures 1a, 1b and 1c) can be used.

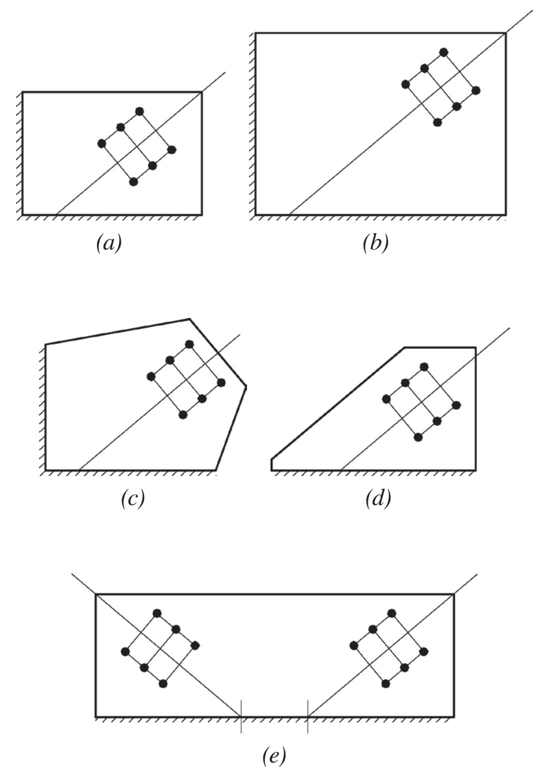

Fig. 1. Gusset plate configurations: (a) compact corner; (b) noncompact corner; (c) extended corner; (d) single brace; (e) chevron.

Bo Dowswell, P.E., Ph.D., Principal, SDS Resources, LLC, Birmingham, AL. E-mail: bo@sdsresources.com

Table 1. Details and Calculated Capacity of Chevron Brace Gusset Plates* $k = 0.75$								
Specimen	t (in.)	L_w (in.)	l_1 (in.)	F_y (ksi)	E (ksi)	P_{calc} (k)	P_{exp} (k)	$\frac{P_{exp}}{P_{calc}}$
Reference: Chakrabarti and Richard (1990)								
1	0.472	14.8	9.8	43.3	29000	252	286	1.14
2	0.315	14.8	6.4	40	29000	158	222	1.41
3	0.315	14.8	6.4	43.2	29000	169	264	1.56
4	0.315	14.8	9.8	72.3	29000	168.7	292	1.73
5	0.315	21.6	11.2	44.7	29000	174.1	175	1.01
6	0.394	14.8	9.6	36.8	29000	173	191	1.11
7	0.512	14.8	8.8	46.7	29000	309	429	1.39
8	0.394	14.8	6.0	82.9	29000	400	477	1.19
1-FE	0.472	14.8	9.8	43.3	29000	252	274	1.09
2-FE	0.315	14.8	6.4	40	29000	158	201	1.27
5-FE	0.315	21.6	11.2	44.7	29000	174.1	228	1.31
8-FE	0.394	14.8	6.0	82.9	29000	400	431	1.08
Reference: Astaneh (1992)								
3	0.25	4.96	4.0	36.0	29000	40.8	42.4	1.04

*Table 6 from Dowswell (2006).

Table 2. Details and Calculated Capacity of Chevron Brace Gusset Plates $k = 0.65$								
Specimen	t (in.)	L_w (in.)	l_1 (in.)	F_y (ksi)	E (ksi)	P_{calc} (k)	P_{exp} (k)	$\frac{P_{exp}}{P_{calc}}$
Reference: Chakrabarti and Richard (1990)								
1	0.472	14.8	9.8	43.3	29000	263	286	1.09
2	0.315	14.8	6.4	40	29000	165	222	1.35
3	0.315	14.8	6.4	43.2	29000	176	264	1.50
4	0.315	14.8	9.8	72.3	29000	200	292	1.46
5	0.315	21.6	11.2	44.7	29000	200	175	0.875
6	0.394	14.8	9.6	36.8	29000	182	191	1.05
7	0.512	14.8	8.8	46.7	29000	319	429	1.34
8	0.394	14.8	6.0	82.9	29000	419	477	1.14
1-FE	0.472	14.8	9.8	43.3	29000	263	274	1.04
2-FE	0.315	14.8	6.4	40	29000	165	201	1.22
5-FE	0.315	21.6	11.2	44.7	29000	200	228	1.14
8-FE	0.394	14.8	6.0	82.9	29000	419	431	1.03
Reference: Astaneh (1992)								
3	0.25	4.96	4.0	36.0	29000	41.7	42.4	1.02

Table 3. Summary of Proposed Effective Length Factors[a]			
Gusset Configuration	Effective Length Factor	Buckling Length	$\dfrac{P_{exp}}{P_{calc}}$
Compact corner	—[b]	—[b]	1.36
Noncompact corner	1.0	l_{avg}	3.08
Extended corner	0.6	l_1	1.45
Single brace	0.7	l_1	1.45
Chevron	0.65	l_1	1.17

[a] Table 7 from Dowswell (2006) with revisions.
[b] Yielding is the applicable limit state for compact corner gusset plates; therefore, the effective length factor and the buckling length are not applicable.

EXAMPLE CALCULATION FOR TABLE 2 (SPECIMEN 1)

$$r = \frac{t}{\sqrt{12}} = \frac{0.472 \text{ in.}}{\sqrt{12}} = 0.136 \text{ in.}$$

$$\frac{kl_1}{r} = \frac{(0.65)(9.8 \text{ in.})}{0.136 \text{ in.}} = 46.8$$

$$F_E = \frac{\pi^2 E}{\left(\frac{kl_1}{r}\right)^2} = \frac{\pi^2 (29{,}000 \text{ ksi})}{(46.8)^2} = 131 \text{ ksi}$$

$$F_{cr} = F_y\left(0.658^{F_y/F_e}\right) = (43.3 \text{ ksi})\left[0.658^{(43.3 \text{ ksi}/131 \text{ ksi})}\right] = 37.7 \text{ ksi}$$

$$P_{calc} = F_{cr}A_g = F_{cr}L_w t = (37.7 \text{ ksi})(14.8 \text{ in.})(0.472 \text{ in.}) = 263 \text{ kips}$$

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Current Steel Structures Research
No. 31

REIDAR BJORHOVDE

INTRODUCTION

This issue of "Current Steel Structures Research" focuses on a selection of research projects at one university in Europe and one in Hong Kong. The descriptions will not include all of the current projects at the schools. Instead, selected studies provide a representative picture of research under way and demonstrate the importance of these schools to the home countries and, indeed, to the efforts of industry and the profession worldwide.

The universities and many of their researchers are very well known in the world of steel construction: the Technical University of Lisbon in Lisbon, Portugal, and the University of Hong Kong in Hong Kong, China. The studies presented reflect elements of the projects as well as other major, long-time activities. All of the projects are multiyear efforts, emphasizing the need for careful planning and implementation of research needs and applications, including the education of graduate students and advanced researchers. As is also the case for important studies in the United States, the outcomes of the projects focus on industry needs and implementation in design standards.

The lead researchers have been active for many years, as evidenced by their leading roles in research and development of their respective countries. They have also been frequent participants in the work in other countries and regions: large numbers of English-language technical papers and conference presentations have been published, contributing to a collection of studies that continue to offer solutions to complex problems for designers as well as fabricators and erectors. Many of the projects also complement current work in the United States and elsewhere.

References are provided throughout the paper, whenever such are available in the public domain. However, much of the work is still in progress, and in some cases reports or publications have not yet been prepared for public dissemination.

Reidar Bjorhovde, Dr.-Ing., Ph.D., P.E., Research Editor of the *Engineering Journal.* Tucson, AZ. E-mail: rbj@bjorhovde.com

SOME CURRENT RESEARCH WORK AT THE TECHNICAL UNIVERSITY OF LISBON IN LISBON, PORTUGAL

Established by the university in 1995, the Institute of Structural Engineering, Territory and Construction (ISETC) is a broad-based research unit that aims to develop novel concepts in scientific and technological research and to transfer those findings to society through education, standards and practical utilization by designers. The institute is housed in the Department of Civil Engineering, Architecture and Geosciences and is financed partly by the Portuguese National Research Council (through noncompetitive funding), partly by competitive funds allocated to specific projects by various governmental agencies and partly by funds coming from consulting work performed by institute staff. The institute is entirely autonomous for administrative and financial services, and it incorporates a library and five laboratories. For structural engineering, the most important units are those focusing on computational mechanics, structures and strength of materials, and construction. The staff of the institute currently numbers 101 faculty members and post-doctoral fellows, with 83 doctoral and master degree students.

Research on steel structures is carried out almost exclusively within two groups: (1) Mechanics, Modeling and Analysis of Structures and (2) Structural Design and Geotechnical Engineering. The directors of these groups are Professor Dinar Camotim and Professor Luis Calado, respectively. A number of the current projects are presented in the following.

Analysis of Thin-Walled Steel Structures Using Generalized Beam Theory (GBT): Professor Dinar Camotim is the project director and chief researcher, and Drs. Nuno Silvestre, Rodrigo Gonçalves, Cilmar Basaglia and Rui Bebiano are the researchers. The project is sponsored by the Portuguese National Research Council.

Generalized beam theory (GBT) is a one-dimensional beam modeling approach that incorporates folded-plate concepts. Developed by Professor Camotim, it is an alternative and very promising approach to shell finite element theory and finite strip analysis. It has proven to be particularly suitable for members and structures incorporating the complex shapes and systems typically associated with cold-formed construction (Camotim et al., 2010a, 2010b). Due to the unique modal features, GBT-based analyses exhibit high computational efficiency along with accurate and elegant solutions for structural behavior and performance.

The objectives of the current project are:

1. Complete the development, numerical implementation and application of GBT formulations currently in progress.

2. Develop, implement, validate and apply novel GBT formulations to solve structural problems not yet tackled by this approach.

3. Develop design applications based on GBT results.

4. Promote and disseminate the use of GBT in the technical community, including practical use in design offices.

Recent achievements of the research team are the development and numerical implementation of GBT solutions for the following subject areas:

1. Performing local, distortional and global buckling and post-buckling analyses of thin-walled steel frames.

2. Accounting for load application effects (distance from the shear centre) in thin-walled members, including localized buckling.

3. Analyzing the buckling behavior associated with the occurrence of web crippling (or patch loading) for various loading configurations.

At this time, it is also important to mention the development of Version 1.0 of the user-friendly GBT-based software GBTUL (an acronym for Generalized Beam Theory at the Technical University of Lisbon), which can perform buckling and vibration analyses of open-section thin-walled steel members. The software is available for free and can be downloaded from www.civil.ist.utl.pt/gbt.

Figure 1 is an example of what has been done using the GBT approach in comparison with results obtained with the well-known software ANSYS. The figure shows the results for a two-span I-beam under top and bottom loading. Figure 1a shows the ANSYS and GBT pre-buckling transverse normal stress distributions, Figure 1b shows the buckling mode given by ANSYS and Figure 1c gives the results of the GBT analyses. The correlations are clearly very good, especially in view of the complexity of the subject.

Distortional Mode Interaction in Cold-Formed Steel Columns: This project is a joint effort among the Technical University of Lisbon; the Federal University of Rio de Janeiro, Brazil; and the University of Hong Kong, China. Professor Dinar Camotim is the project director, with researchers Pedro Dinis and Nuno Silvestre from the Technical University of Lisbon; Eduardo Batista, Federal University of Rio de Janeiro; and Ben Young from the University of Hong Kong. The project is sponsored by the Portuguese National Research Council.

Cold-formed steel columns are often highly susceptible to local (L), distortional (D) and global (G) buckling and exhibit similar L, D and/or G buckling stresses for some commonly used geometries. This implies that their post-buckling behavior, ultimate strength and failure modes are influenced by effects that involve some or all of these buckling modes. The objectives of the project are the following:

1. Provide in-depth understanding of the mechanics of the interaction phenomena.

2. Through physical tests, determine the individual failure modes and assess their effect on the column response.

3. Use numerical and experimental ultimate strength data to develop and validate design approaches that are based on the direct strength method (DSM) (Silvestre et al., 2012).

Recent results have provided in-depth understanding of the response of simply supported and fixed-lipped channel columns affected by L-D, D-G and L-D-G buckling (Dinis and Camotim, 2011). There is clear experimental evidence regarding the occurrence of L-D and L-D-G interaction and of the subsequent reduction of the ultimate strength. Further, efficient DSM design approaches are now being developed for lipped-channel columns affected by L-D-G and D-G interaction (Silvestre et al., 2009; Silvestre et al., 2012). Figure 2 illustrates the interaction between local and distortional buckling, as demonstrated by tests at the University of Hong Kong.

Energy-Absorbing Fuses for Seismic-Resistant Steel Frames: This project is a joint effort among the Technical University of Lisbon; the Polytechnic University of Milano, Italy; the Technical University (RWTH) of Aachen, Germany; the National Technical University of Athens, Greece; and the steel company Sidenor SA from Greece. Professor Luis Calado, Lisbon, is the project director, with researchers Carlo Castiglioni from Milano, Ioannis Vayas from Athens, Bruno Hoffmeister from Aachen and D. Kalteziotis from Sidenor. The project is sponsored by the Research Fund for Coal and Steel of the European Union.

The project focuses on the development of structural systems that are simple to repair. Two innovative systems of seismic-resistant steel frames with energy-absorbing (dissipative) fuses were developed, as follows:

1. The FUSEIS1 type consists of two closely spaced strong columns, rigidly connected to multiple beams, with the beams connecting columns. Alternatively, the beams are interrupted and connected by short pins. An example is shown by the test setup in Figure 3, where the frame was

tested at the Polytechnic University of Milano (Castiglioni et al., 2011).

2. The FUSEIS2 type consists of seismic fuses for steel and composite moment resisting frames. The fuse is developed by plates bolted or welded to the web and the flanges of the beam. A sample application is shown in Figure 4, where the frame and its fuse elements were tested at the National University of Athens (Castiglioni et al., 2012).

One of the advantages of these systems is that any damage will only take place within the fuses, and these are easily replaced following a strong earthquake. Specifically, the FUSEIS2 systems dissipate energy through inelastic deformations and combine ductility and architectural transparency with structural stiffness. The project has also provided a design guide for practical applications of the systems, including design examples.

Steel-Concrete Composite Truss Bridges: The project is a joint effort between the Technical University of Lisbon

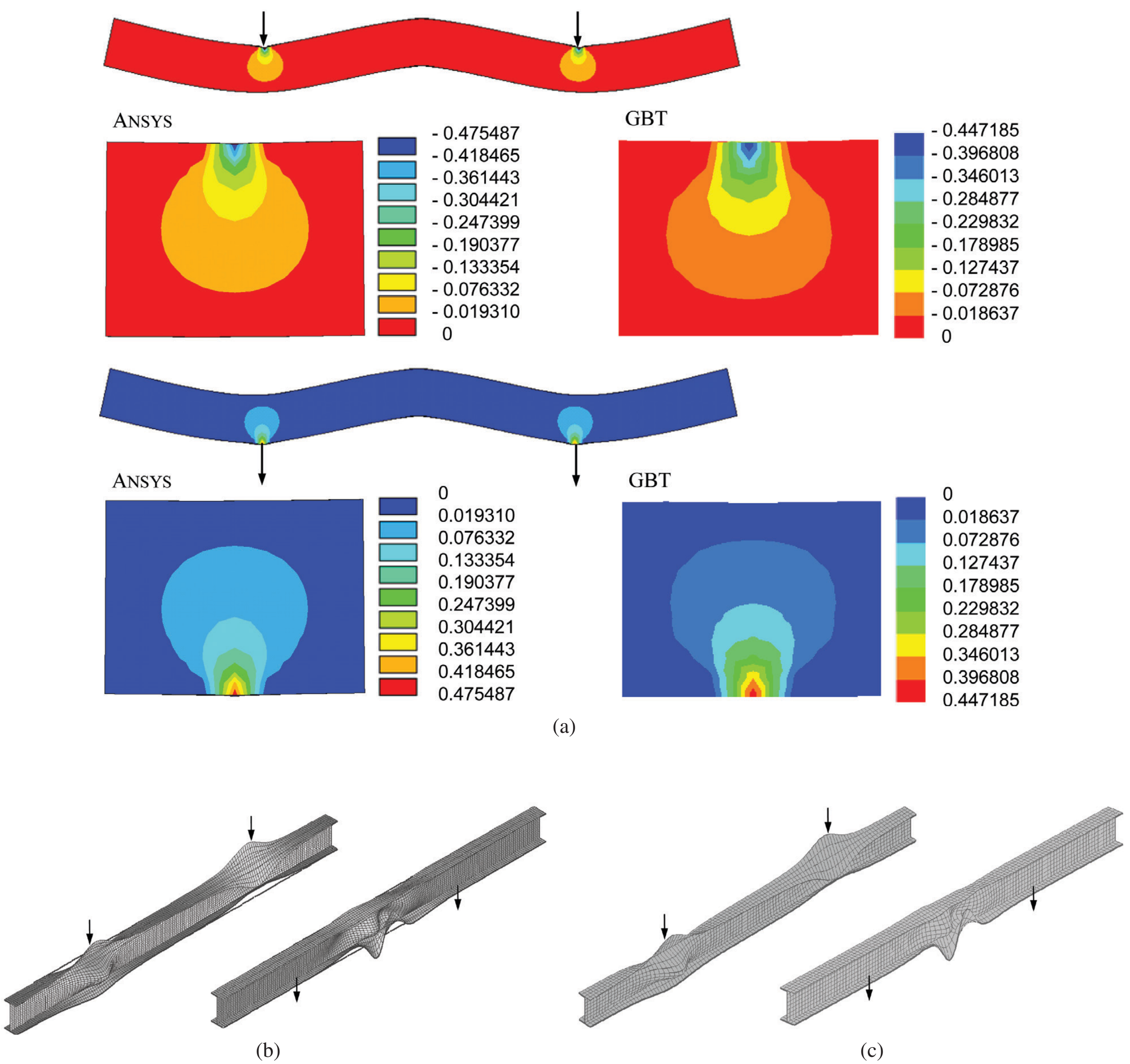
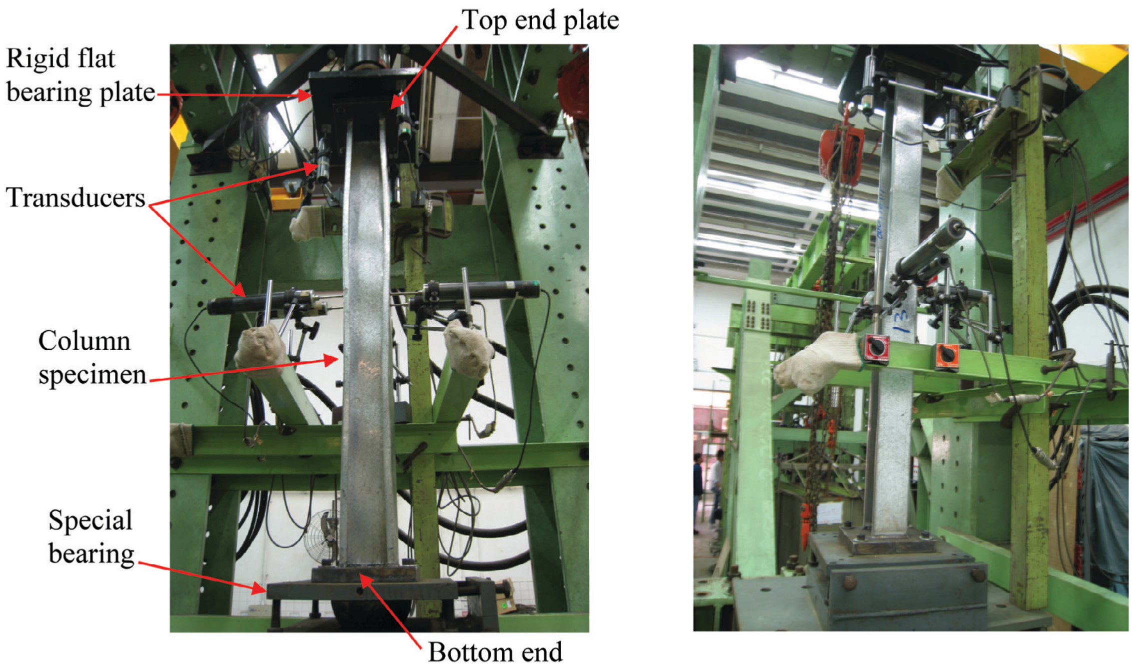


Fig. 1. Two-span I-beam under top and bottom loading: (a) ANSYS and GBT pre-buckling transverse normal stresses; the failure modes according to the results of (b) ANSYS and (c) GBT (Drawings courtesy of Professor Dinar Camotim).



(a) (b)

Fig. 2. Tests of lipped channel columns showing local and distortional buckling interaction (a) front view of test setup; (b) side view of test setup (photos courtesy of Professor Ben Young).

Fig. 3. Full-scale test frame with FUSEIS1 devices and failure mode (photo courtesy of Professor Carlo Castiglioni).

and the GRID Consulting Engineers Company in Lisbon. The project researchers are Professors António Reis (director) and José Oliveira Pedro. The project has been supported by grants from the Portuguese National Research Council and from the research partner, the GRID company.

The aim of this project is to examine the behavior and structural performance of three types of composite truss bridge decks—namely, wide decks supporting overhangs in curved bridges, three-dimensional triangular decks, and semi-through decks as are now commonly used in railroad bridges. In the curved bridge decks, main and secondary Warren-type trusses are combined to form an equivalent box section. Horizontal tubular bracing is placed between the lower chords, thus achieving box-type behavior while being subjected to torsion. The chords and diagonals are made of welded rectangular hollow sections (RHS) and the lateral trusses are built with circular hollow sections (CHS). To avoid intermediate internal diaphragms—for the sake of construction simplicity and aesthetics—a key issue is the differential displacement between the main trusses under eccentric loading. The secondary bending moments at the transverse cross beams, connecting the upper and lower chords, were evaluated by means of shell finite element analysis. Figure 5 shows a typical curved bridge deck with main and secondary Warren-type trusses forming an equivalent box cross-section.

Composite truss bridge decks with triangular cross-sections constitute one of the most interesting developments over the past several years (Reis, 2008). Examples are shown in Figure 6. Several major issues have been addressed:

1. The shear failure of the lower chord in the vicinity of the internal supports, where high girder shear forces must be balanced by diagonal axial forces.

2. The negative bending moment redistribution near the supports at the ultimate limit state.

3. The torsional stiffness of the bridge deck under eccentric traffic loading.

Two tests were carried out on ⅕-scale bridge models subjected to two-point loading. The tests aimed at simulating the bending moment between the support section and the mid span cross-sections.

So-called semi-through trusses are currently adopted for railroad bridges, where vertical clearances are often the main deck constraint. An example of such a bridge is shown in Figure 7. Low slenderness values are currently required for the chord to reduce its size as well as to satisfy the serviceability limit states for deformability and vibration in these bridges. This is particularly important for very high speed railroads, where the deck maximum vertical accelerations are restricted, to ensure reasonable comfort levels (Reis and Pedro, 2011).

Steel-Concrete Composite Cable-Stayed Bridges: This project is being conducted at the Technical University of Lisbon. It has been funded by the Portuguese National Research Council. The researchers are Professors José Oliveira Pedro (director) and António Reis.

Cable-stayed structures provide elegant and efficient

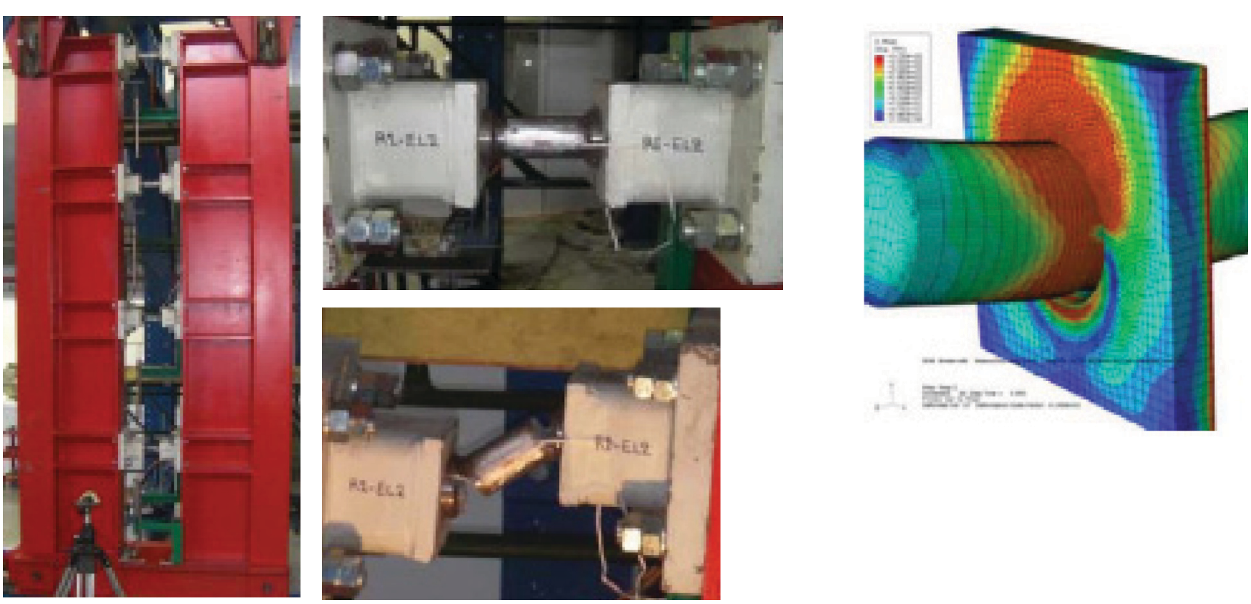


Fig. 4. Test setup details for a composite frame using the FUSEIS2 device (photos courtesy of Professor Ioannis Vayas).

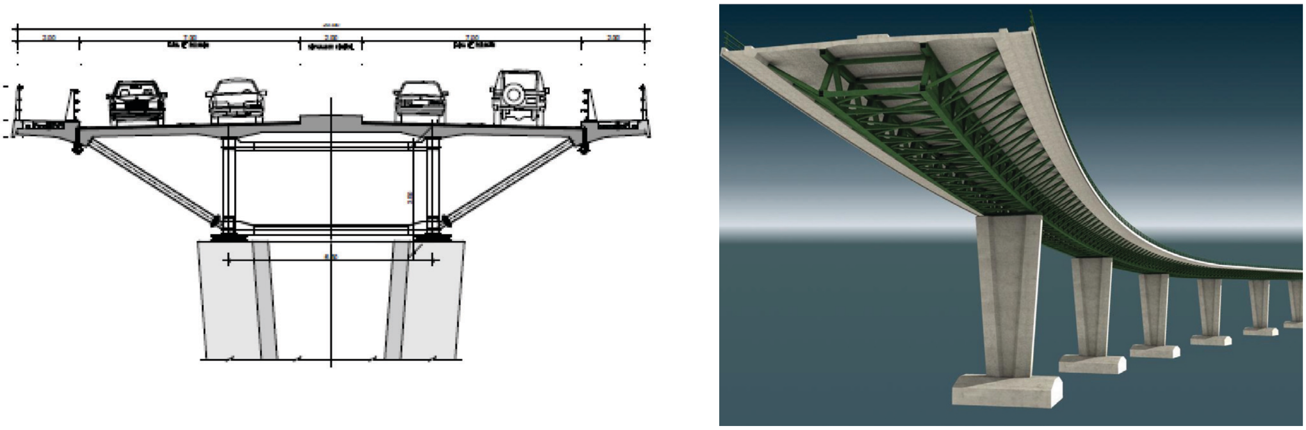
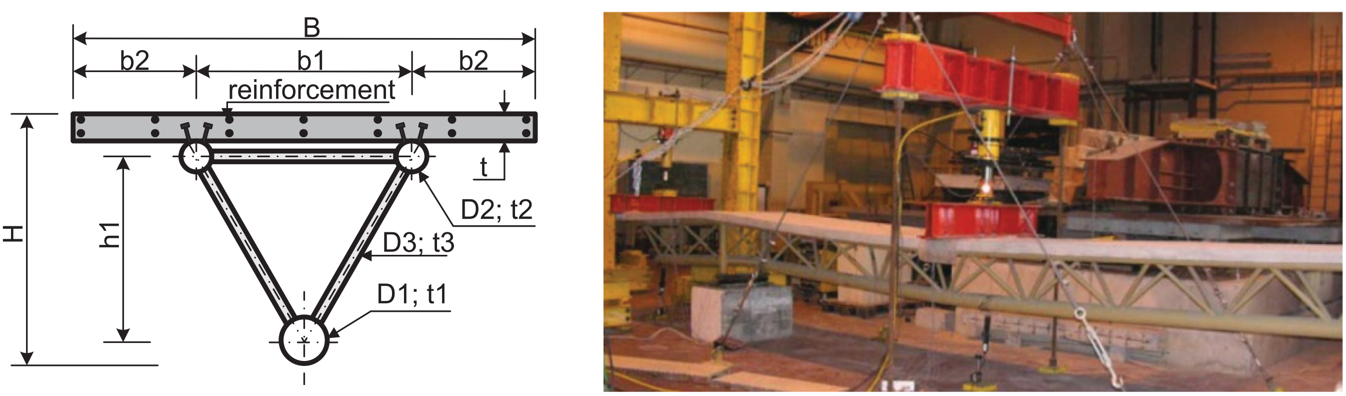


Fig. 5. Curved composite bridge deck cross-section and bridge (images courtesy of Professor António Reis).



(a) (b)

Fig. 6. Triangular bridge deck model tests at the Technical University of Lisbon: (a) bridge deck cross-section; (b) testing of ⅕-scale bridge model (drawing and photo courtesy of Professor António Reis).

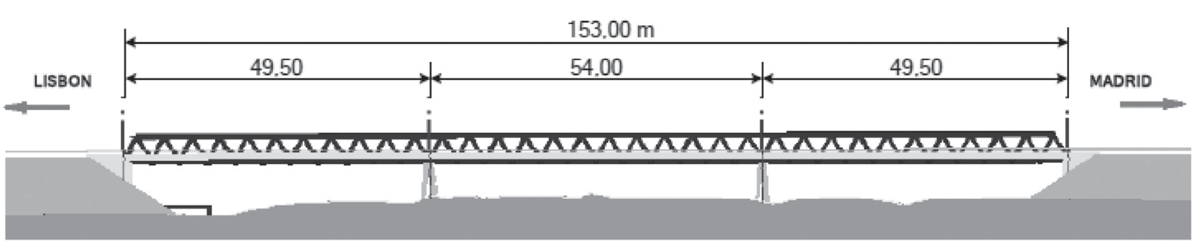


Fig. 7. Semi-through truss bridge for the railroad line between Lisbon and Madrid (drawing courtesy of Professor António Reis).

solutions for long-span bridges. For spans up to 2,000 to 2,500 feet, steel-concrete composite girder decks are the most economical and competitive solutions, as confirmed by a variety of bridges built in the past 20 years. Contemporary designs typically use very thin and light decks; such solutions are adopted to reduce the areas exposed to wind and to provide significant savings for the construction of the deck, the cables, the piers and the foundations. Very flexible decks can only be used by continuous-stay cable systems with multiple stays. As a result, deck slenderness of long-span, cable-stayed bridges has been increasing steadily, with slenderness ratios now ranging from about 100 to approximately 300 (Pedro and Reis, 2008).

Bridge safety may be evaluated by identifying the critical cross-sections of the deck and determining the ultimate load and the collapse mechanism. Failure occurs when the ultimate strain is attained at one or more deck elements. A nonlinear analysis modeling the construction stages is required to ensure the correct distribution of dead loads. Service live loads with different patterns are then applied, and increasing load scenarios are evaluated until failure is reached (Pedro and Reis, 2010). Further, the structural response of the bridge during its lifespan is assessed by taking into account the concrete time-dependent effects (shrinkage and creep). Finally, the global and local stability of the composite deck under the high compressive forces that are induced by the staying scheme may also be relevant (Pedro and Reis, 2011).

The aim of this project is to investigate specific issues related to the service and ultimate response of composite cable-stayed bridge decks, with particular emphasis on the global stability of the deck under high applied load levels. It is anticipated that major developments and proposals will be provided toward state-of-the-art criteria for bridge analysis and design in accordance with Eurocodes 3 and 4 (ECS, 2004, 2005).

SOME CURRENT RESEARCH WORK AT THE UNIVERSITY OF HONG KONG

Bolted and Screwed Connections under Ambient and Elevated Temperatures: The project has been funded by STALA Tube Company, Finland; BlueScope Lysaght (Singapore) Pte., Ltd.; and the University of Hong Kong. Professor Ben Young is the director of this project; the staff researchers have been Shu Yan and S. M. Zahurul Islam.

A very large number of single-shear-bolted, double-shear-bolted and single-shear-screwed connections of thin sheet steels at ambient and elevated temperatures have been tested. The experiments were performed by steady-state and transient-state testing methods, as illustrated by the test set-up in Figure 8.

For the analysis of the connections, finite element models were developed and verified by the test results, and

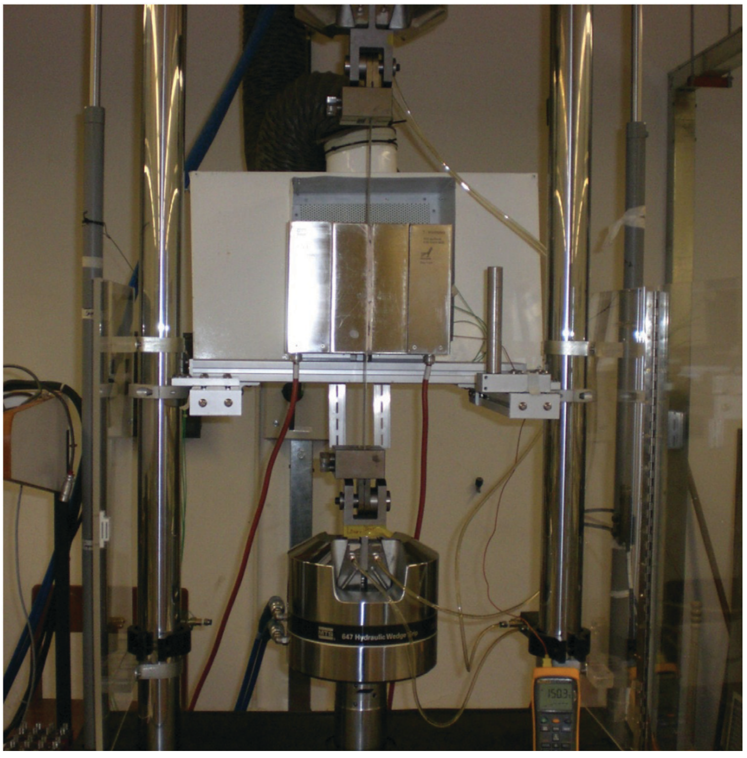


Fig. 8. Test setup for bolted and screwed connections (photo courtesy of Professor Ben Young).

parametric evaluations were performed using the verified finite element models. Bearing factors were determined for the limit states of the connections, including the limit states of tilting and bearing for the screwed connections (Yan and Young, 2011a, 2011b). Considering the effects of elevated temperatures, design equations have been proposed on the basis of the data. The predicted bearing strengths of bolted and screwed connections at elevated temperatures using the proposed equations have been found to be more accurate than those based on the current design rules (Yan and Young, 2012).

Additional testing has been performed for cold-formed stainless steel tubular shapes subjected to web crippling. The findings will be reported in forthcoming publications.

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ERRATA

Fillet Weld Design for Rectangular HSS Connections

Paper by JEFFREY A. PACKER and MIN SUN
(1st Quarter, 2011)

On p. 32, the line of text prefacing Equation 2b should read: "When $\theta \geq 60°$:"

On p. 33, the line of text prefacing Equation 4b should read: "When $\theta \geq 60°$:"

On p. 39, the last sentence in the left column should read: "The predicted strength of each welded joint, without consideration of the $(1.00 + 0.50 \sin^{1.5} \theta)$ effect…"

The captions of Figures 5 and 6 should be altered to read: "…without inclusion of the $(1.00 + 0.50 \sin^{1.5} \theta)$ term…"

The headings of Tables 6 and 7 should be altered to end with: "…without Inclusion of the $(1.00 + 0.50 \sin^{1.5} \theta)$ Term"

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