Steel Structures Research Update

System Behavior and Collapse Assessment of Braced Frames

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INTRODUCTION

Current and recently completed research on system behavior and collapse assessment of braced frames is presented. The research includes work on concentrically braced frames, buckling restrained braced frames, and dual systems with braced frames and secondary moment frames.

Concentrically braced frames (CBFs) are popular for seismic force resisting systems (SFRSs) because of their lateral stiffness and strength. Numerous research investigations into CBF component and system behavior (e.g., Astaneh-Asl et al., 1986; Tremblay 2001, 2002; Uriz and Mahin, 2004) have formed the basis for the AISC *Seismic Provisions* (AISC, 2010). Two studies, highlighted here, build upon contemporary research and further advance the field by examining the dynamic response of chevron CBFs and computational methods for quantifying the collapse capacity of CBFs. The principal investigators for these studies are Professor Dimitrios Lignos, McGill University, and Professor Taichiro Okazaki, Hokkaido University.

Buckling restrained braced frames (BRBFs) have also gained in popularity in recent years, prompting the need for increased knowledge about component and system behavior (Fahnestock et al., 2007a,b). Design guidance on BRBFs has been provided, and researchers have investigated component and system behavior both experimentally and computationally (e.g., Lopez and Sabelli, 2004; Hikino et al., 2013). Further investigation into the connections affecting boundary conditions of the BRBs and BRBF collapse capacity will provide a major step forward in understanding BRBF behavior and design. This research is led by Dr. Luis Ibarra, AISC Milek Fellow and faculty member at The University of Utah.

Rounding out the discussion is a study on a dual-system alternative for seismic design. The secondary frames serve to improve upon the system behavior of the braced frames. The principal investigator for this study is Professor Paul Richards, Brigham Young University.

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DYNAMIC RESPONSE OF CONCENTRICALLY BRACED FRAMES

Large-scale shake table tests of a chevron CBF were conducted to confirm expected component and system behavior. Much of the knowledge base on CBFs is from static loading tests. Specifically, the research program examined the cyclic brace behavior, effects of framing action and frame behavior (Okazaki et al., 2013). The experimental study also incorporated recent design developments for balanced design of gusset plates (Roeder et al., 2011a, 2011b), so as to avoid potential failure of the gusset plate welds as well as for the CBF to withstand large lateral deformations during ground motion shaking.

Test Specimen

Chevron CBFs were chosen for this study because of their architectural advantages and, hence, frequent use. However, the chevron configuration introduces additional considerations for the CBF beams due to the force unbalance from a loaded tension brace and a buckled compression brace. Plastic hinging in the beam may occur, preventing the tension brace from developing its full capacity; inelastic behavior of chevron CBFs with stronger beams is more stable (Tremblay and Robert, 2001). Furthermore, "a large-scale test by Uriz (2005) suggests that, even if the beam is designed for the force unbalance per the … AISC *Seismic Provisions*, the elastic deflection caused by the force unbalance can be large enough to prevent tensile yielding of the brace" (Okazaki et al., 2013).

The single-story, one-bay CBF test frame represented the bottom story of a three- to five-story steel frame building in Japan. The 70% scale, 13.6-ft (4.15-m) wide, 7.38-ft (2.25-m) tall frame was built with wide-flange beams, HSS columns and square HSS braces (Figure 1). Lateral bracing of the frame was provided at the X's shown in Figure 1. Beam-to-column connections were built as rigid. The column bases were rigidly connected to the shake table through base beams. The 0.177-in. (4.5-mm) gusset plates were designed following the balanced design procedures outlined by Lehman et al. (2008), incorporating an elliptical fold line to accommodate out-of-plane deformations of the brace (Figure 2). The specimen was subjected to six different target amplification levels, ranging from 10% to 70% of the Takatori EW motion (1995 Kobe earthquake). Additional details on the CBF specimen, the test procedure, the instrumentation and loading can be found in Okazaki et al. (2013).

Shake Table Test Results

The shake table tests provided valuable information about chevron CBF behavior. In particular, the final test, at 70% of the Takatori record, demonstrated the effects of unbalanced forces from the braces. Yielding of the beam, near the connections to the columns, began after the specimen obtained a story drift ratio of 0.01 radian. Buckling of the East brace occurred on the brace's first major contraction. After initially recovering its compressive strength, the brace steadily lost its compressive capacity before fracturing near mid-length on its third major extension. Fracture of the West brace was gradual; fracture initiated with failure of the East brace, and the brace saw three more extension cycles before fracturing completely (Figure 3b). Data revealed that the beam deflected down about 0.59 in. (15 mm) due to the force unbalance from the braces but did not develop a plastic hinge as predicted by static pushover analysis. This deformation of the beam affected the brace behavior, causing the braces to deform more in compression than in tension and to not develop their full tensile strength. The researchers also commented that the "rather limited fatigue life" may have been related to the width–thickness ratios because the braces did not qualify as seismically compact (Okazaki et al., 2013). Meanwhile, the gusset plate connections performed well, developing elliptical yield lines as the braces buckled and deformed out of plane (Figure 3a, Figure 3c). A detailed synthesis of results, including comparisons to numerical simulations, is provided in Okazaki et al. (2013).

COLLAPSE ASSESSMENT OF CONCENTRICALLY BRACED FRAMES

Karamanci and Lignos (2014) proposed a computational approach for collapse assessment of CBFs under seismic loading. The work was motivated by observations of CBF behavior, the resulting challenges in computationally capturing that behavior and challenges in quantifying the collapse capacity for performance-based engineering. Local story mechanisms are common to CBFs. Cyclic deterioration in stiffness and strength of CBFs aggravates P-delta effects and plays a significant role in sidesway instability. This cyclic behavior is affected by many geometric and material parameters; researchers have sought to represent this behavior through phenomenological, physical theory, fiber-based and detailed finite element models, as discussed in Karamanci and Lignos (2013). "The main challenge to reliably predict the dynamic behavior of CBFs near collapse is to accurately

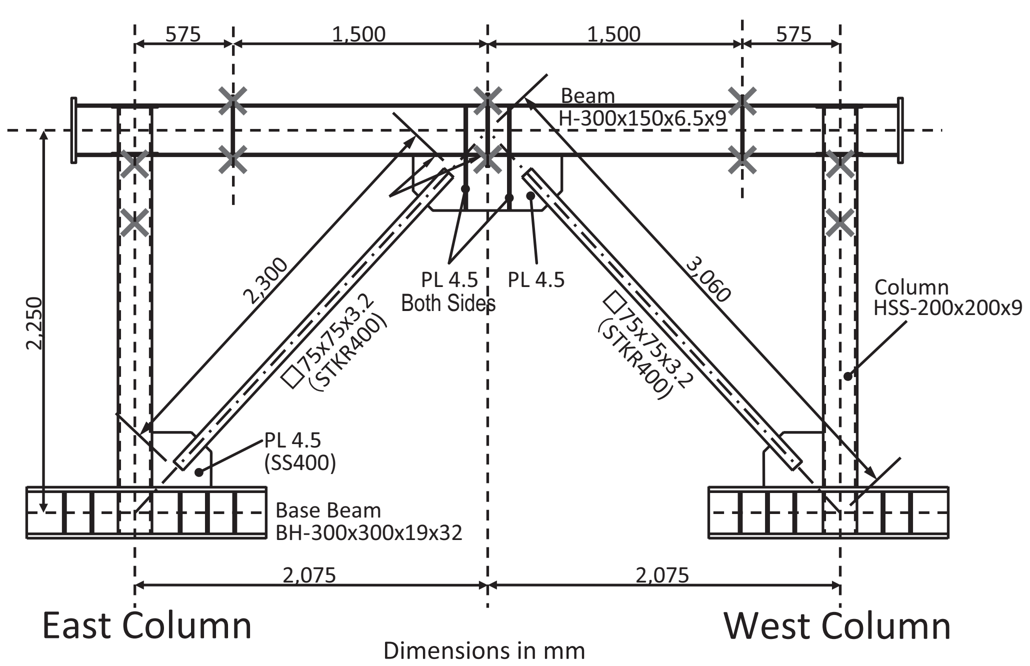


Fig. 1. CBF test specimen (based on Okazaki et al., 2013).

represent the input parameters of the steel brace simulation models that control global/local instabilities and ultimately fracture attributable to low-cycle-fatigue … Another challenge is to explicitly simulate the strength and stiffness deterioration of steel columns and gusset plate beam-to-column connections under cyclic loading" (Karamanci and Lignos, 2014). Viscous damping has been identified as another important parameter in computational simulations of collapse (e.g., Charney, 2008). Karamanci and Lignos developed a steel brace database and a computational approach that incorporates consideration of all of these parameters for collapse assessment of CBFs with various types of steel brace shapes.

Modeling of Braces, Connections and Columns

In this computational approach, the model by Uriz (2005) was used to simulate the cyclic behavior of the steel braces. This model captures axial force and second-order bending moment interaction and accounts for large displacements. Fibers represent the brace cross-section in this model, which

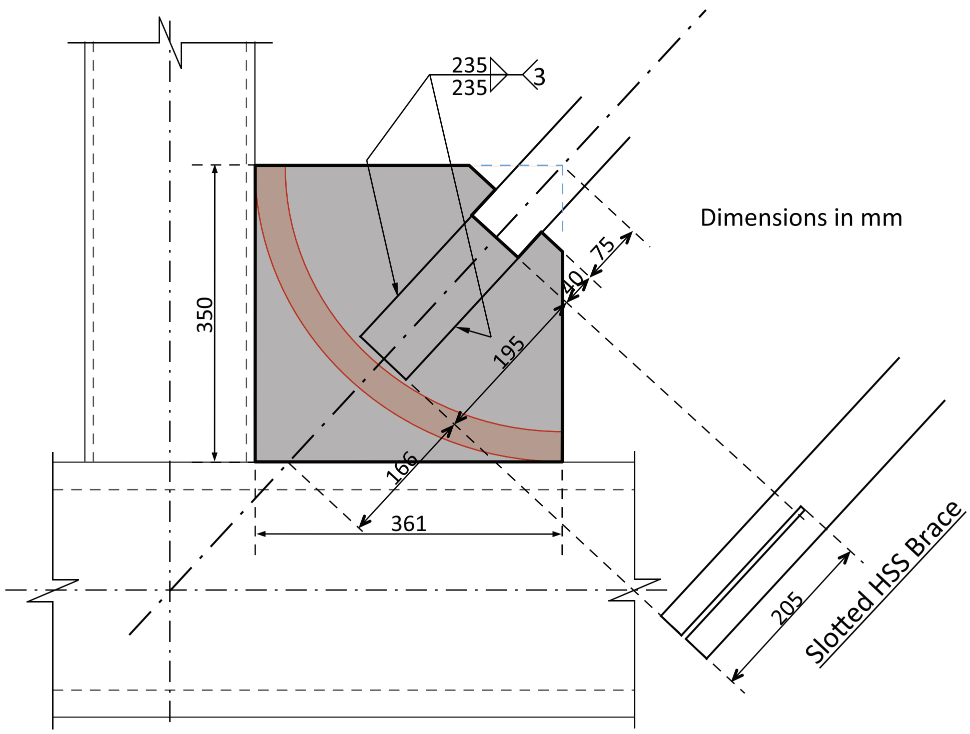
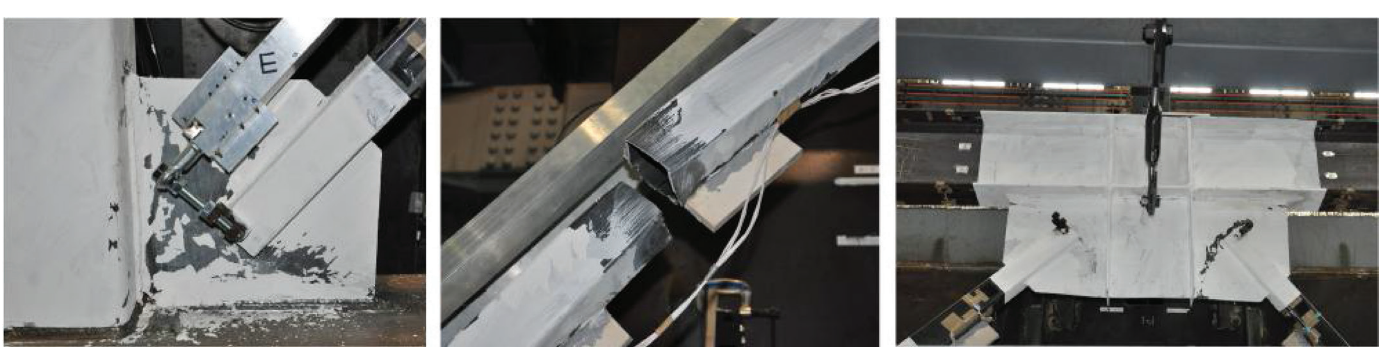


Fig. 2. Gusset plate with elliptical fold line (based on Okazaki et al., 2013).



(a) (b) (c)

Fig. 3. (a) Gusset plate yielding; (b) steel brace fracture; (c) mid-beam gusset plate yielding.

utilizes the force formulation by Spacone et al. (1996). The use of fibers, following a modified rain-flow counting rule, can simulate gradual or complete fracture through the cross-section. An offset, or initial camber, in the two-element brace component triggers flexural buckling. In this modeling approach, slender cross-sections may behave in a manner that does not correspond well to the plane-sections-remain-plane assumption; therefore, Karamanci and Lignos (2014) recommend detailed finite element approaches (e.g., Fell et al., 2009; Huang and Mahin, 2010) for slender sections. Meanwhile, the models of the gusset plates capture out-of-plane flexural strength and stiffness, based on Hsiao et al. (2012).

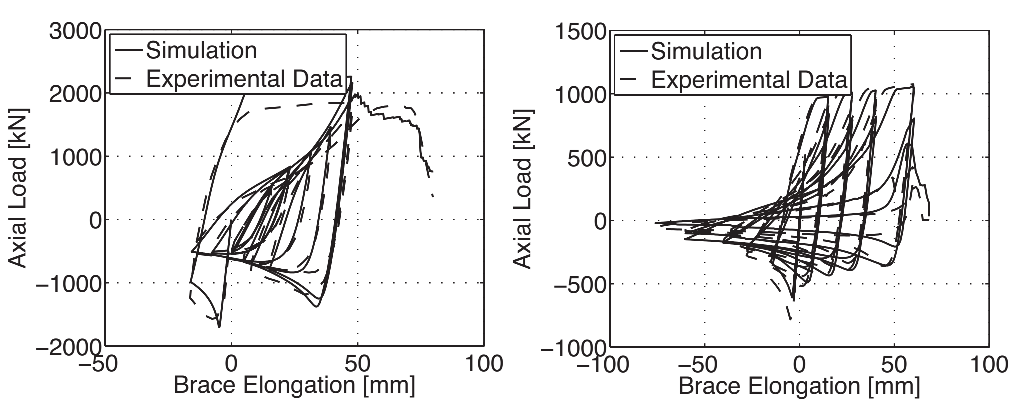
An important product from the research was a database of steel braces for use in modeling post-buckling behavior. Databases had existed (e.g., Hsaio et al., 2013; Tremblay, 2002; Lee and Bruneau, 2005), but the work by Karamanci and Lignos (2014) brought together more data and for more types of cross-sections that are commonly used in practice. Information for 317 steel braces was collected; the inventory included 158 rectangular or square HSS braces, 55 round HSS braces, 65 W-shape braces, 37 angle braces and 2 stitched angle braces. The data pulled from 24 experimental programs was for steel braces that failed due to low-cycle fatigue following local buckling. Model input parameters—in particular, those that control the fatigue life of the brace—were related to the brace properties. Measured geometric and material properties, when available, were used in the calibration of the fiber-based model. Expected material properties, following AISC 341-10 (AISC, 2010), were used as needed. Parameters such as initial camber (to trigger global buckling), number of elements or segments for the brace model, and number of fibers to represent the cross-section were optimized. Detailed information on the calibration procedure can be found in Karamanci and Lignos (2013, 2014); the steel brace database, with searchable interface, will be provided at http://dimitrios-lignos.research.mcgill.ca/databases/. The result is excellent correspondence between simulation results and experimental data, as shown in Figure 4 for HSS brace tests by Uriz and Mahin (2008) and Fell et al. (2009).

Modeling recommendations for inelastic buckling and fracture of steel HSS, round HSS and wide-flange braces are summarized in Karamanci and Lignos (2014). Meanwhile, a number of observations of HSS, W-shape and angle brace behavior can be made. Karamanci and Lignos (2014) comment that "fracture life of the three steel brace shapes is primarily affected by the individual local slenderness parameters of the cross sections. However, as the kL/r slenderness increases, global (elastic) buckling of a steel brace is triggered (prebuckling behavior)." Fracture potential also increases with higher yield stress and kL/r values. However, the effect of yield stress is not as pronounced for slender steel braces. Severe local buckling and "crimping" at the corners of rectangular and square HSS braces increase their susceptibility to fracture.

The computational approach for collapse assessment of CBFs also includes modeling of the cyclic deterioration of the steel columns and the beam-to-column connections at the gusset plates. Model development for the connections was based on work by Stoakes and Fahnestock (2011) and Liu and Astaneh-Asl (2000, 2004). Model development for the cyclic deterioration of the steel columns was based on work by Nam and Kasai (2011), Scott and Ryan (2013), and others. The contribution of the connections and columns is on the order of 40% of story shear forces once braces have fractured (Karamanci and Lignos, 2014).

Sidesway Collapse and Viscous Damping

Karamanci and Lignos (2014) define dynamic, or sidesway, collapse of a CBF as "the point in which an individual story (or a series of stories) displaces sufficiently so that the



(a) Data from Uriz and Mahin (2008)

(b) Data from Fell et al. (2009)

Fig. 4. Comparisons of simulation results to experimental data [Simulation results adapted from Karamanci and Lignos (2014)].

second order P-delta effects accelerated by component deterioration fully offset the first order story shear resistance of the CBF. At this point, the CBF loses its gravity load resistance.” This definition highlights the need for incorporating cyclic deterioration of the various frame components in the computational approach. The definition also correlates well to shake table collapse tests of steel moment resisting frames and CBF frames (e.g., Suita et al., 2008; Lignos et al., 2011; Okazaki et al., 2013), as well as quasi-static tests of CBF frames (e.g., Uriz, 2005). Base shear and story drift ratio responses (Figure 5) illustrate sidesway collapse of a two-story CBF modeled using the computational approach described previously. The story drift histories indicate a first story collapse mechanism following brace fracture and the onset of column strength deterioration.

The role of viscous damping was investigated through collapse assessment of a 12-story steel CBF building. Details of the building, modeling and evaluation can be found in Karamanci and Lignos (2014). The researchers found that use of initial stiffness of CBF braces for formation of the Rayleigh damping matrix can significantly overestimate collapse capacity. Buckling and fracture of the braces cause large stiffness changes in the numerical model, in turn creating large artificial damping forces. For more realistic CBF behavior, the researchers recommended that tangent stiffness values be used for modeling of components that behave nonlinearly, excluding their geometric stiffness component to avoid a negative stiffness component of effective damping.

Summary and Future Work

The modeling approach captures important characteristics of the steel CBF components and behavior. Cyclic strength and stiffness deterioration, fracture, P-delta effects and viscous damping are addressed through this comprehensive, computational approach. A valuable new database for modeling of post-buckling behavior of steel braces has been created, and a rigorous definition of sidesway collapse has been developed. The work has been validated against results from quasi-static cyclic and shake table tests. However, the researchers call for further validation against shake table collapse experiments (Karamanci and Lignos, 2014). The proposed approach is currently being utilized to (1) quantify the effect of gravity-framing on the collapse resistance of archetype steel frame buildings with CBFs, (2) quantify the earthquake-induced economic losses in steel frame buildings with CBFs (Hwang et al., 2015), and (3) develop recommendations for the expected post-buckling resistance loads for the seismic design of bracing connections and other members (Cerri et al., 2015).

BEHAVIOR AND COLLAPSE CAPACITY OF BUCKLING RESTRAINED BRACED FRAMES

Similar research on system behavior and collapse capacity, but of buckling restrained braced frames (BRBFs), is in progress. The 2013 Milek Fellowship research, “Effects of BRB Boundary Conditions on Seismic Resilience of BRBFs,” is considering effects of gusset plates and beam-column-gusset connections. BRBFs meet serviceability and collapse capacity objectives under the design basis earthquake (DBE), but some performance issues require more study. For example, Ariyaratana and Fahnestock (2011) indicated that BRBFs may exhibit relatively large residual interstory drifts at DBE seismic level, whereas Sabelli et al. (2003) reported residual

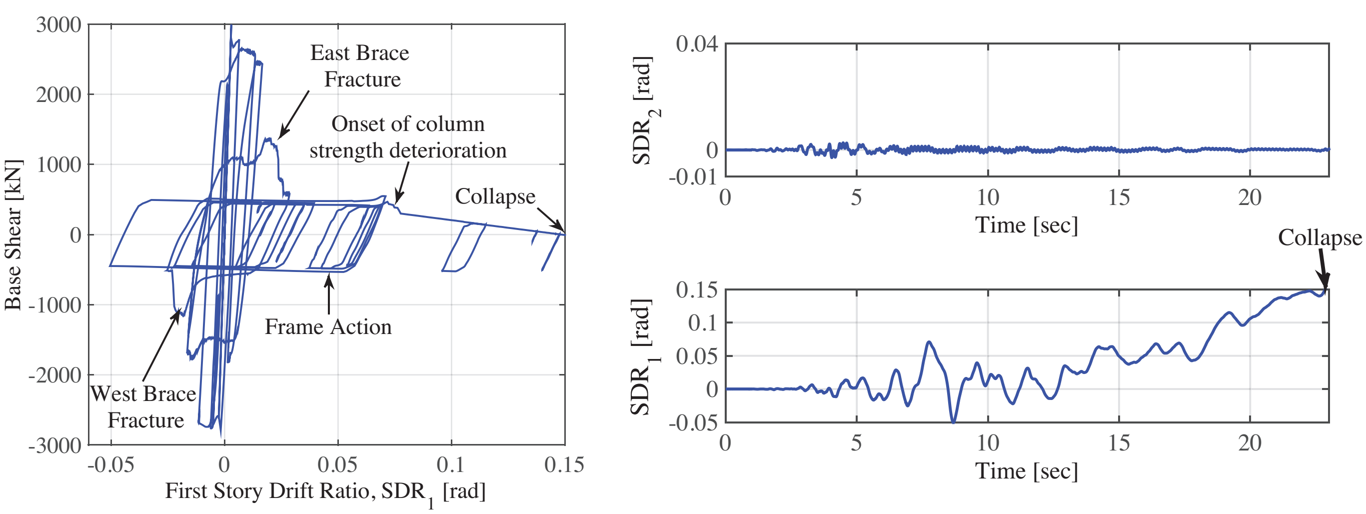


Fig. 5. Base shear–drift ratio and story drift histories for a two-story CBF [simulation results adapted from Karamanci and Lignos (2014)].

story drifts that were on average 40 to 60% of the maximum drifts. In addition, BRB qualification tests show that most braces present stable hysteretic loops under axial loading up to drifts of 2.5 to 3.5%. At these drift levels, the BRB may fail due to core plate fracture, post-buckling behavior, local case buckling and brace plate failure, among other factors. However, these tests usually include overdesigned gusset plates that guarantee failure will take place in the BRB, preventing failure modes on gusset plates and beam-column-gusset connections. Therefore, this fellowship research includes experimental tests and numerical simulations to evaluate the effect of BRB boundary conditions on the collapse capacity of BRBFs. Some simulation results will be presented, and the planned experimental tests will be introduced.

BRB Boundary Conditions

Some studies have investigated effects of BRB boundary conditions (BCs) on BRBF behavior. Tsai and Hsiao (2008) designed single gusset plates adopting Whitmore's approach, where the gusset plate buckling strength, P_{cr}, was required to be greater than the BRB maximum compressive capacity force, P_{maxC}:

$$P_{cr} = \frac{\pi^2 E}{(kL_c/r)^2} b_e t \geq P_{maxC}$$

L_c is the critical unbraced length, and k is the effective length factor and was taken as 0.65. Tsai and Hsiao found that the gusset plates buckled out of plane at a brace force significantly lower than the calculated gusset buckling strength, P_{cr}. They then designed gusset stiffeners to address the

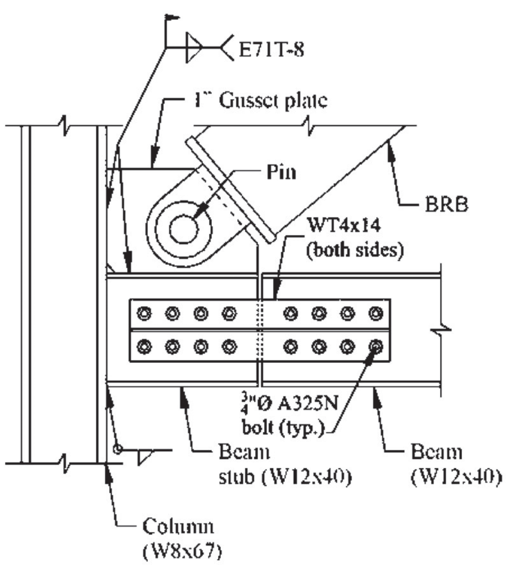


Fig. 6. Hinged beam at gusset plate connection [AISC, 2010; from Fahnestock et al., (2006)].

buckling failure. Similar conclusions were reached by Chou et al. (2012) when evaluating BRBFs with single and dual gusset connections. Also, Takeuchi et al. (2014) performed cyclic experiments on six BRBs set to an out-of-plane story drift of 1/100 using different core-confinement clearance, encasement, embedment ratios and, in some cases, free-edge stiffeners on gusset plates. Finite element modeling (FEM) work performed by Sheng et al. (2002) and Naghipour et al. (2013) showed that a gusset under compression will displace minutely in the out-of-plane dimension elastically until the gusset buckles. Once buckling occurs, the gusset will continue to carry some axial load but with large out-of-plane displacements.

Regarding beam-column-gusset connections, Fahnestock et al. (2007a) and Prinz (2007) performed numerical simulations of BRBFs using hinged, or spliced, beams (i.e., non–moment resisting connection, as shown in Figure 6) and continuous beams. They showed that beam splices reduce stresses in the beam-column-gusset connection region by more than 50%. The experimental results also indicate that the potential for gusset plate failure may be reduced if non–moment resisting connections are considered (Fahnestock et al., 2007b). However, use of these connections may increase residual story drifts (Wigle and Fahnestock, 2010), which is one of the concerns in terms of BRBF seismic resilience. Such behaviors are being investigated through the Milek Fellowship research.

Numerical Study

The numerical investigation has addressed effects of hinged beams and gusset plate out-of-plane behavior on collapse capacity of BRBFs. The prototype structure used in the numerical study is a four-story, single diagonal braced bay structure modified from the AISC *Seismic Manual* (2012) BRBF example (Figure 7). The BRBF has a bay width of 19.33 ft, a 14-ft-tall first story and 12.5-ft-tall upper stories. The structure's beams, columns and gussets are designed with a 50-ksi yield strength, and the BRB core cross-sectional area (A_{sc}) are sized based on a yield stress (F_y) of 42 ksi. The brace overstrength factors for tension (ω) and compression (β) are assumed as 1.36 and 1.1, respectively.

The prototype BRBF seismic performance was evaluated for two gusset plate designs with different average buckling lengths. These lengths represent the range of extremes of possible unbraced lengths while still qualifying as "compact," according to Dowswell (2006). The first gusset plate design (Gusset-1) has an average unbraced length of about 5.0 in. (Figure 8a) and meets Dowswell's (2006) and Thornton's (1984) design criteria. Dowswell checks a gusset's geometry for stability based upon a collection of experimental results, while Thornton checks a gusset's stability using column-buckling design. For the numerical model, Gusset-1 is given a strength that is equal to the AISC (2012) prescribed

$P_{cr} = 1.1\beta\omega A_{sc}F_y$. For the future physical experimentation, Gusset-1 will have an average unbraced length of about 5.0 in. (Figure 8a) to reach a strength that similarly just meets code requirements. In Gusset-1, the elliptical fold zone is the elliptical line that crosses virtually through the last bolt closest to the beam, giving the zone a width roughly equal to the gusset's thickness (t_g). The second gusset plate design (Gusset-2) (Figure 8b) will have a longer average unbraced length that meets Dowswell's criterion, but it does not have the required buckling strength when designed per the Thornton method with an effective length factor of 0.65. Gusset-2, as modeled, is now susceptible to premature buckling. For the future physical experimentation, Gusset-2 will have an average unbraced length of about 12.6 in. (Figure 8b). Gusset-2 has an elliptical fold zone width of approximately $16t_g$.

Two numerical models have been developed in PERFORM 3D (CSI, 2011) and OpenSees (McKenna, 2014) software. The model in OpenSees will be used to explicitly consider out-of-plane displacement of the gusset plates, but it is not in the scope of this research update. The PERFORM 3D model will be used to validate the results of the most complex OpenSees model and includes an element with a trilinear backbone curve that accounts for the isotropic and kinematic behavior observed in BRB hysteretic responses. Also, the model can capture the plastic rotational deformations of beam-column elements. For the numerical analyses, the BRBs were assigned an ultimate strain capacity of 4%. Experimental tests commonly report BRB ultimate strain levels between 2.5 and 3.5% (Xu, 2015; CoreBrace, 2015). The use of a larger maximum strain level implies that the BRBs would withstand larger ground motions, increasing the likelihood of gusset plate buckling failure.

For the gusset plate, the tensile stress-strain response was assumed to be elastic-perfectly-plastic with an ultimate allowable tensile strain of 0.2. For compressive loading, the gusset buckling capacity was approximated in PERFORM 3D with inelastic buckling elements, representing the gusset's unbraced length and located at each end of the BRB. The buckling is controlled by the member's axial deformation. Strain limitations for the PERFORM 3D model were

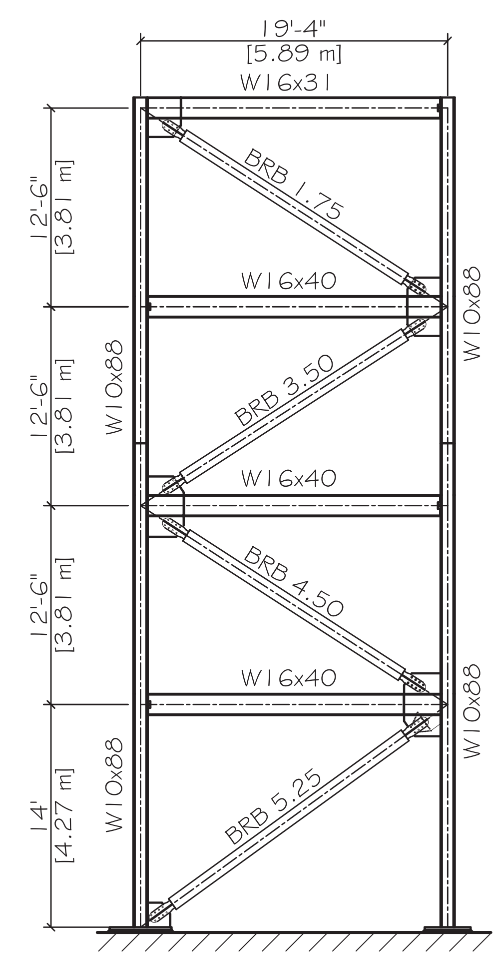
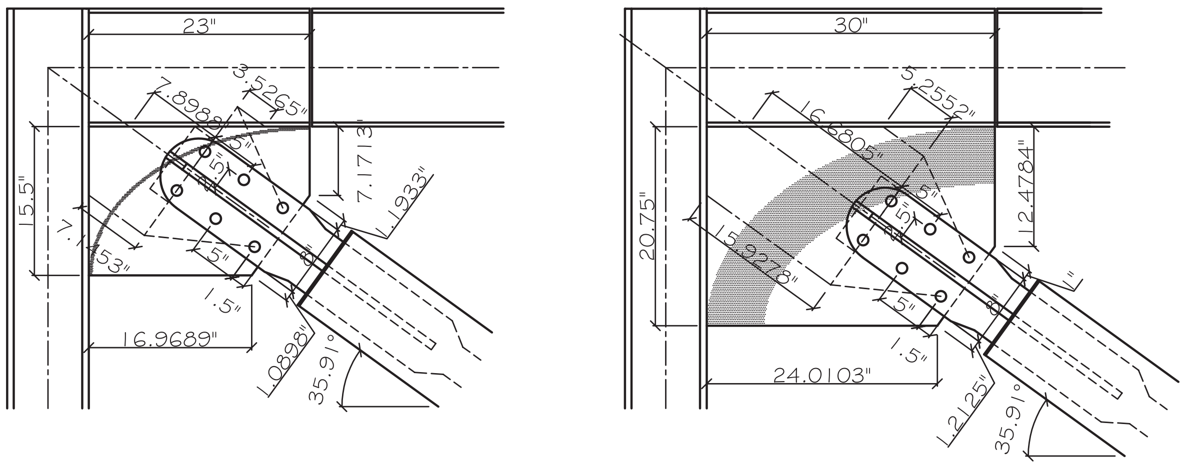
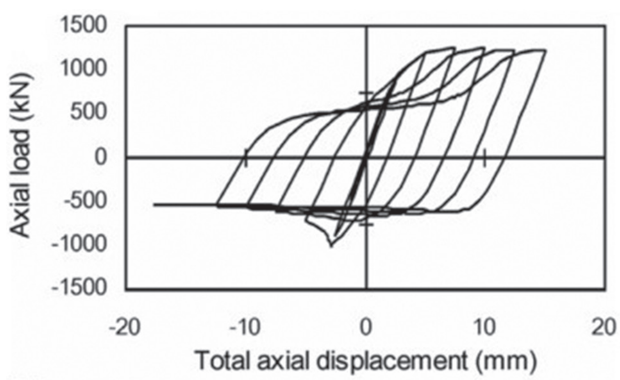


Fig. 7. Full-scale prototype structure.

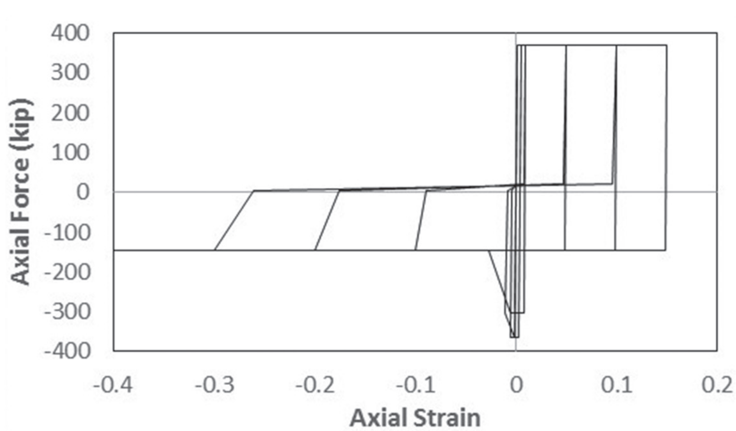


(a) (b)

Fig. 8. (a) Gusset-1 configuration, (b) Gusset-2 configuration.



(a)



(b)

Fig. 9. (a) Cyclic response of a gusset plate (Walbridge et al., 2005); (b) numerical simulation.

estimated using results primarily from Walbridge et al. (2005) and Sheng et al. (2002). By curve fitting the Sheng et al. data, the buckling strain was estimated to be 0.015 mm/mm, and the ultimate strain limit was assumed as 0.50 mm/mm; a best case scenario that allows for the level of gusset nonlinearity observed in the testing of Walbridge et al. Figure 9 shows the hysteretic response of one of the gusset plates evaluated by Walbridge et al. under quasi-static loading, along with the response obtained from a time-history analysis in PERFORM 3D.

The seismic performance to collapse of the prototype BRBF was computed using incremental dynamic analyses (IDAs) for both types of gussets. The frames were also evaluated assuming splices at the end of the gusset that created beam-hinges capable of releasing stresses from the gusset plates. To perform IDAs, a set of 13 records was selected from FEMA P-695 (2009) far-field ground motions. The spectral accelerations were scaled in incremental steps of 0.2 g, until collapse occurred. The collapse capacity was computed within a tolerance of 0.02 g. Figure 10 shows the median IDAs for the evaluated four-story BRBFs. The frames with Gusset-1 exhibited a larger collapse capacity because, in most cases, failure was caused by BRB ultimate drift capacity. BRBFs with Gusset-2 exhibited gusset plate buckling failure in about half of the simulations, resulting in a loss of collapse capacity of more than 20%. The inclusion of splices to create beam-hinges did not have a large effect for drifts of 2% or less. The collapse fragility curves presented in Figure 10 corroborate the small effect of beam splices on the frame performance.

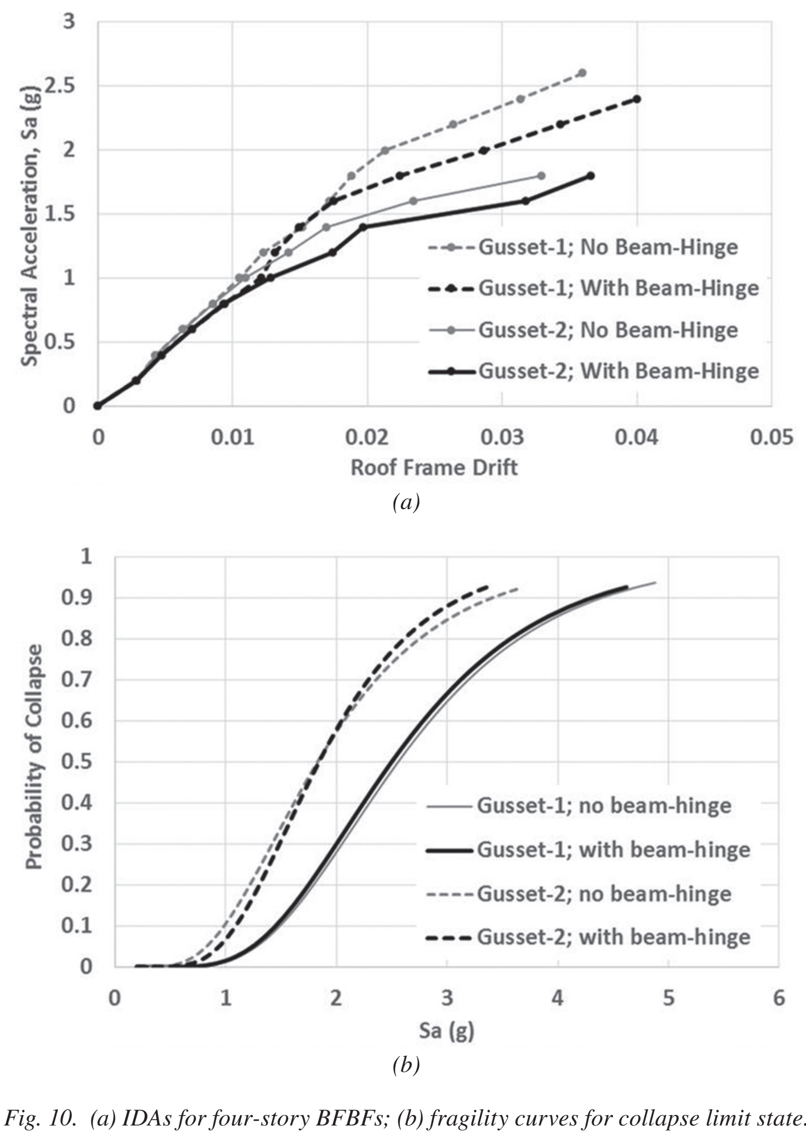


Fig. 10. (a) IDAs for four-story BFBFs; (b) fragility curves for collapse limit state.

Planned Experimental Tests and Expected Outcomes

For the experimental tests, a 75% scale, one-story portal frame will be built. The loading protocol will be based on the AISC *Seismic Provisions* (2010) for cyclic testing of BRBs. Alternative loading protocols are permitted by AISC and will be considered in the numerical simulation. Figure 11 shows the prototype that it is expected to be tested in the next phase of the project. It is anticipated that the frames will be first tested with beam splices until plate yielding starts, at which point the beam splice will be bolted at the flanges, creating a continuous beam. Then, the frames will be retested until significant frame or connection damage is detected. This approach will provide information about the BRBFs' maximum displacements, gusset stresses and buckling capacity, as well as the effect of BRB boundary conditions on frame collapse capacity. The experimental tests will also be used to calibrate the numerical models; a parametric study, based on time-history analyses, will be performed.

DUAL SYSTEMS

Recent, complementary research demonstrates how the performance of braced frames can be improved upon in dual systems through the use of secondary moment frames. Secondary moment frames can prevent concentrated story drifts and reduce residual drifts in braced frames (e.g., Pettinga et al., 2007). Furthermore, designing the frames for positive stiffness throughout a seismic event is beneficial for mitigation of residual drift. This positive stiffness can be achieved through high-yield-drift (HYD) moment frames with elastic response up to 4% drift. A possible HYD moment frame uses a theoretically pinned connection at one end of the beam and a rigid connection at the other. Another possible HYD configuration may use this one-end, rigidly connected beam at alternate stories within the frame.

Miller and Richards (2015) investigated and provided recommendations for dual systems with HYD moment frames; they used buckling-restrained braced frames for their study but commented on the ability to use HYD frames for dual systems with other types of braced frames. They conducted non-linear time-history analyses of 4- to 20-story dual systems, with both HYD frames and with conventional, secondary moment frames. Design basis earthquake (DBE) and maximum considered earthquake (MCE) demands were considered. The researchers concluded that the HYD configuration with the one-end, rigidly connected beams at alternate stories had the highest connection efficiency of the HYD frames studied, with half of the rigid connections required for a comparable conventional frame. Maximum and residual drift levels in the dual-HYD systems were lower than those for the conventional system. Additional details of the study and synthesis of results are provided in Miller and Richards (2015).

SUMMARY

Several unique but related studies on seismic behavior and collapse capacity of braced frames have been presented. Some recently completed work on the dynamic response of chevron CBFs has been highlighted, along with

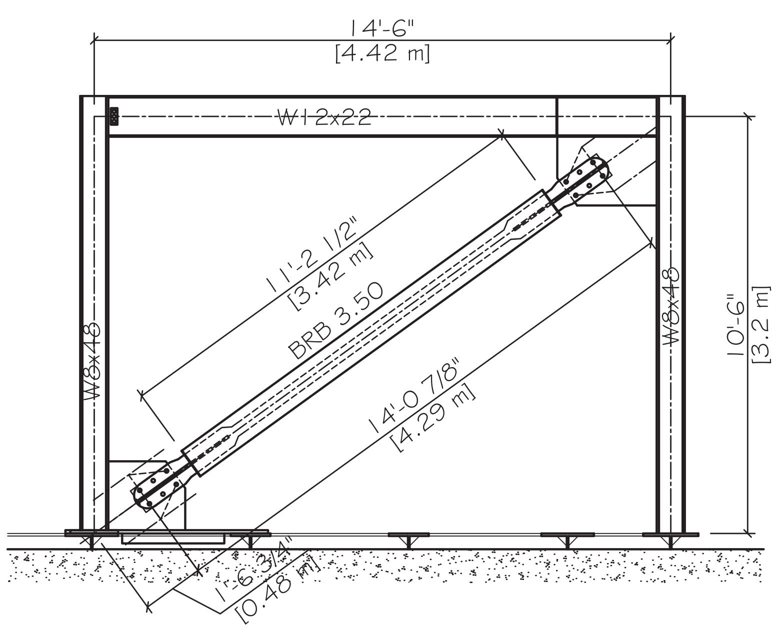


Fig. 11. Portal frame to test gusset plate out-of-plane behavior.

investigations into computational collapse assessment of CBFs. Similar themes, including the significance of out-of-plane gusset plate behavior, are found in research on effects of BRB boundary conditions on the seismic behavior of BRBFs. Meanwhile, a slightly different approach to seismic resilience is found in an investigation of high-yield-drift moment frames in dual systems with braced frames.

ACKNOWLEDGMENTS

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