FINITE ELEMENT PARAMETRIC STUDIES OF BUCKLING-RESTRAINED BRACED FRAME CONNECTIONS

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ABSTRACT :

Large-scale experimental studies of buckling-restrained braced frames (BRBFs) have shown that although they display good overall performance, they may have limitations due to connection failure modes that do not allow the braces to realize their full ductility capacity. These experimental results motivate further investigation of BRBF connection behavior and performance. In this study, nonlinear finite element models are used to study BRBF connections with emphasis placed on the beam-column-brace connection regions. The models focus on a one-story subassembly that is part of a four-story BRBF, which was tested previously. After the baseline finite element analysis results are verified with experimental data, parametric studies are conducted to explore variations in connection configuration. Results are discussed on global and local levels. The effects of the parametric connection variations are assessed and key issues that influence performance are identified.

KEYWORDS: braced frames, buckling-restrained braces, steel connections, seismic effects

1. INTRODUCTION AND MOTIVATION

1.1. Background

Buckling-restrained braced frames (BRBFs), which are concentrically braced frames (CBFs) with buckling-restrained braces (BRBs), provide significantly better seismic performance than conventional steel CBFs. The superior performance of BRBFs results from the robust cyclic performance exhibited by BRBs. Whereas conventional steel braces yield in tension but buckle in compression, leading to sudden strength and stiffness degradation, BRBs yield in tension and compression and develop significant energy dissipation capacity and ductility. These favorable attributes have prompted rapid implementation of BRBFs in the western United States in regions of high seismicity. Figure 1 illustrates a typical BRB configuration (Fahnestock et al., 2007a). Numerous isolated tests of BRBs have demonstrated the favorable cyclic characteristics described above and have supported the quick adoption of BRBFs into U.S. design provisions (AISC, 2005b; ASCE, 2005). Table 1 shows a sample of ductility demands, with maximum ductility demand μ_{max} and cumulative plastic ductility demand μ_c, imposed on BRB test specimens and Table 2 shows a summary of story drift θ_{story} and BRB ductility demands that were obtained from nonlinear dynamic analyses of prototype BRBFs subjected to suites of earthquake ground motions scaled to the maximum considered earthquake (MCE) hazard level. These results suggest that BRBs are capable of sustaining the demands that are expected under major seismic events, assuming that BRBs in frame systems will perform in the same way as the isolated BRBs.

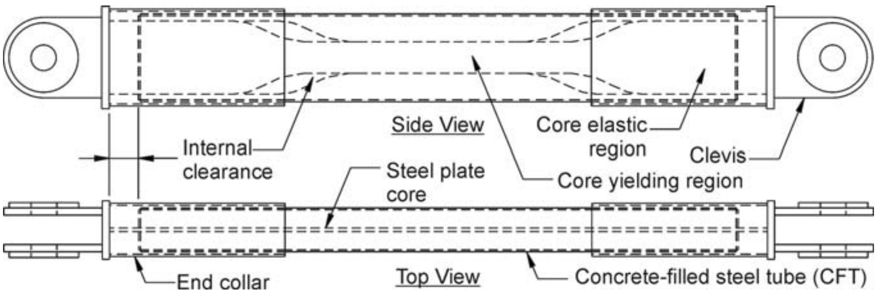

Figure 1 Typical BRB configuration (Fahnestock et al., 2007a)

Table 1 Experimental BRB ductility demands

Reference	BRB	μ_{max}	μ_c
Black et al. (2002)	99-1	20	324
	99-2	10	879
	99-3	10	279
	00-11	15	1045
	00-12	15	538
Merritt et al. (2003)	1	15	900
	2	15	600
	3	10	1600
	4	15	1100
	5	15	1300
	6	15	800
	7	10	1000
	8	10	1000

Table 2 Demands from nonlinear time-history analysis (MCE hazard level)

Reference	Response	θ_{story}	μ_{max}	μ_c
Sabelli (2001)	mean	0.045	17.4	139
Model: BRBF-6vb2	mean + one standard deviation	0.066	25.1	185
Fahnestock et al. (2007a)	mean	0.033	18.4	179
Model: BRBF-4	mean + one standard deviation	0.041	22.7	391

Several recent large-scale experimental studies of BRBFs have shown that although they display good overall performance, limitations exist due to undesirable connection failure modes. The standardized BRB qualification testing protocol (AISC, 2005b) attempts to replicate the demands that would be imposed on a BRB in a frame system, but it has become evident that realistic frame conditions lead to BRB demands that have not been fully represented in qualification tests. Results from large-scale experimental studies of BRBFs provide the best insight into system performance since they more realistically represent interaction between the various frame elements (e.g., BRBs, beams, columns and connections). Results from the four research programs summarized below, which studied BRBF system performance, motivate further investigation of BRBF connection behavior.

1.2. Relevant Previous Research

Aiken et al. (2002) conducted three cyclic tests on a 0.7-scale one-bay one-story BRBF with full-penetration welded beam-column connections and bolted brace-gusset connections, similar to the detail shown in Figure 2a. Column, beam and gusset plate yielding, gusset plate distortion and cracking at the column-gusset welds and column-beam bottom flange welds were observed at story drifts less then 0.02 radians. During the last test, weld fracture led to beam torsional rotation and BRB out-of-plane displacement at a story drift of 0.0263 radians and subsequently the strength degraded severely.

Tsai et al. (2003) completed two phases of testing on a full-scale 3-story 3-bay MRF-BRBF with bolted brace-gusset connections and bolted beam web splices in the BRBF. Gusset plate distortion was observed in Phase 1, which concluded when the BRBs failed due to out-of-plane instability at a story drift of 0.025 radians. Gusset plates were repaired, new BRBs installed and stiffeners added at the gusset plate free edges. Phase 2 tests demonstrated acceptable performance up to a story drift of 0.025 radians. In a later experimental program, a full-scale one-bay two-story BRBF with improved connection details performed reasonably up to a story drift of 0.022 radians, although cracking occurred in column-gusset welds (Tsai et al., 2006).

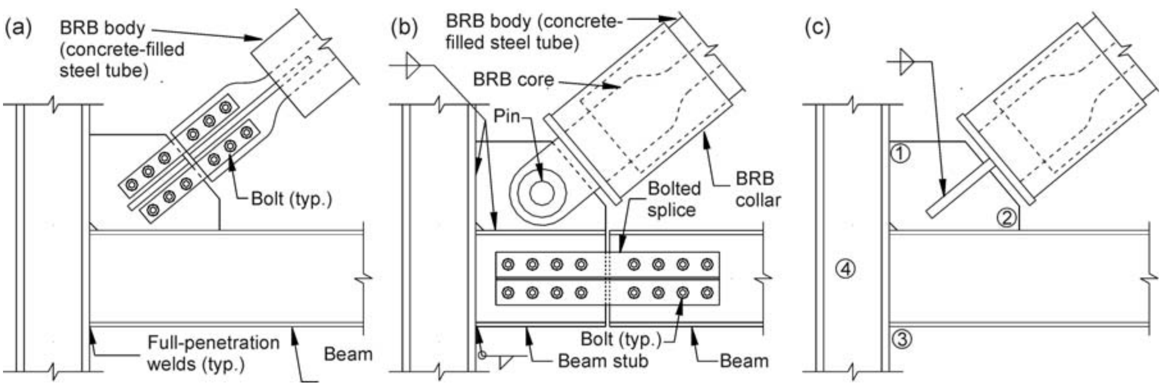

Figure 2 BRBF connection details

Christopulos (2005) tested five full-scale one-bay one-story BRBFs under cyclic displacement histories. Brace-gusset connections were bolted and beams were connected to columns with single-plate shear tabs. The tests investigated the effects of gusset plate geometry, type of bolted brace-gusset connection and orientation of the BRB core plate. Variations between test specimens had minimal influence on performance and four of the five BRBFs failed by BRB out-of-plane deformation at story drifts between 0.022 and 0.024 radians. BRB failure was typically preceded by yielding and buckling of the beams and columns adjacent to the gusset plates.

Fahnestock et al. (2007b) tested a 0.6-scale four-story BRBF using hybrid pseudo-dynamic earthquake simulations and extensive quasi-static cyclic loading. The brace-gusset connections were pinned and bolted web splices were used to connect the beams to beam stubs, as shown in Figure 2b. During the earthquake simulations, the test frame sustained story drifts up to 0.048 radians with minimal damage and no stiffness or strength degradation. Following the earthquake simulations, the bolted beam splices were removed from one level of the test frame, making the beam continuous. The test frame was then subjected to quasi-static cyclic displacement histories that imposed story drifts up to 0.05 radians. The connections and BRBs did not exhibit undesirable failure modes during the testing program. Testing concluded when the core yielding regions of five BRBs fractured. In contrast to the tests summarized above where undesirable connection and BRB failure modes were typically observed at story drifts between 0.02 and 0.025 radians, this testing program demonstrated that BRBFs can withstand large story drifts and maintain load-carrying capacity.

1.3. Research Motivation and Plan

The first three experimental programs described above (Aiken et al., 2002; Tsai et al., 2003 and 2006; Christopulos, 2005) exhibited similar performance but contrast starkly with the performance exhibited in the final experimental program described above (Fahnestock et al., 2007b). The large difference in story drift capacity, 0.025 radians compared to 0.05 radians, indicates that connection details have a major impact on global system performance. Further research is necessary to explore the critical connection parameters that affect system performance and to determine reliable approaches for designing connections that do not lead to undesirable failure modes but that allow the significant energy dissipation capacity and ductility of BRBs to be fully exploited. In view of this need, the present research aims to explore seismic behavior and performance of BRBF connections through finite element (FE) analysis parametric studies. A three-dimensional FE model of the four-story BRBF tested by Fahnestock et al. (2007b) was created using Abaqus (ABAQUS, 2006). The model was verified with test results and was used to study various connection configurations.

2. FINITE ELEMENT MODEL DEVELOPMENT AND VERIFICATION

2.1. Baseline Model

The third story of the four-story BRBF from Fahnestock et al. (2007b) was chosen for detailed study. As shown in Figure 3, this subassembly was modeled with three- and four-node shell elements. The remainder of the BRBF was modeled using frame elements with lumped plastic hinges for beams and columns and nonlinear springs for BRBs. For the BRBs in the detailed region of the model, the restraining concrete-filled hollow structural section (HSS) was not modeled explicitly. Instead, beam elements with negligible axial stiffness and a flexural stiffness equal to the HSS were tied to the centerline of the steel core to replicate the effect of the restraining mechanism. At all welded interfaces, full attachment was modeled. For the bolted beam splices, bolts and bolt holes were not modeled, but the corresponding nodes in the connected parts were slaved to represent force transfer through bolt bearing.

Material nonlinearities were incorporated through the von Mises material model with associated flow rule and combined nonlinear isotropic and kinematic strain hardening. Steel stress-strain properties were based on measured material properties (Fahnestock et al., 2007b) and hardening parameters were calibrated to match representative cyclic steel stress-strain data (Kaufmann and Pense, 1999). For the BRBs in the detailed subassembly region, a yield stress of 38 ksi with the calibrated hardening parameters was used instead of the actual yield stress of 46 ksi in order to better match stress-strain behavior at larger strains, as shown in Figure 4.

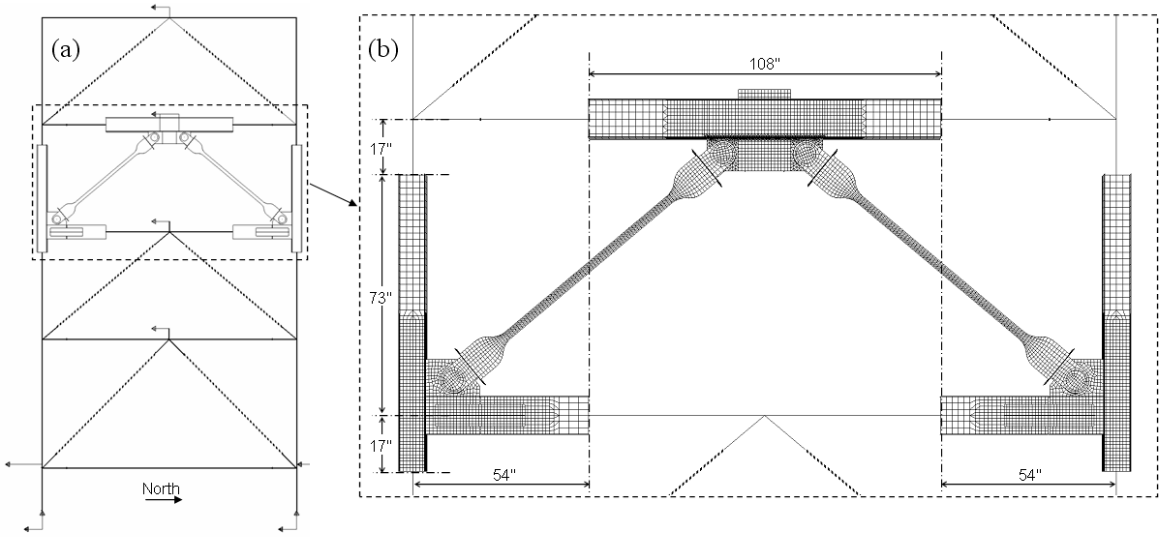

Figure 3 FE model – (a) whole model (shown with boundary condition locations) and (b) detailed region

2.2. Verification

To verify the accuracy of the BRBF model, it was subjected to boundary conditions that were recorded during experimental earthquake simulations (Fahnestock et al., 2007b) at the locations shown in Figure 3a. Comparisons between the FE analysis and the experimental results appear in Figures 5 and 6. Brace behavior matched reasonably well although the FE model underestimated compressive response at larger deformations. This is due to confinement of the core and the frictional forces that develop between the core and the restraining concrete when the brace experiences large compressive deformation. The confinement effect was not included in the BRB core material model and as a result, negative story shear was also under-predicted by the model. Apart from this, story behavior agreed sufficiently well with the experimental results and the model was deemed adequate for parametric studies.

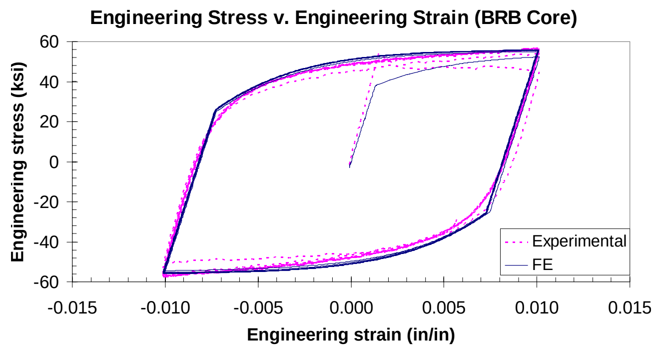

Figure 4 Steel stress-strain properties

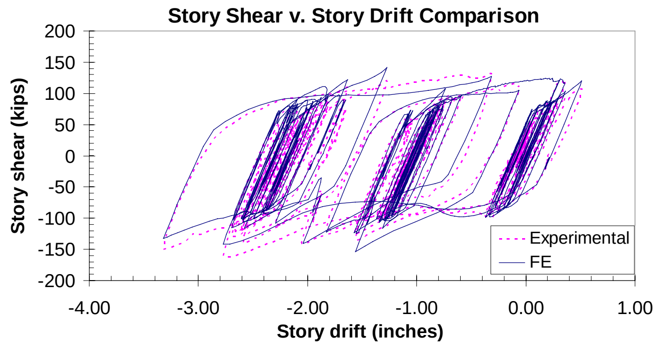

Figure 5 Story shear-drift model verification

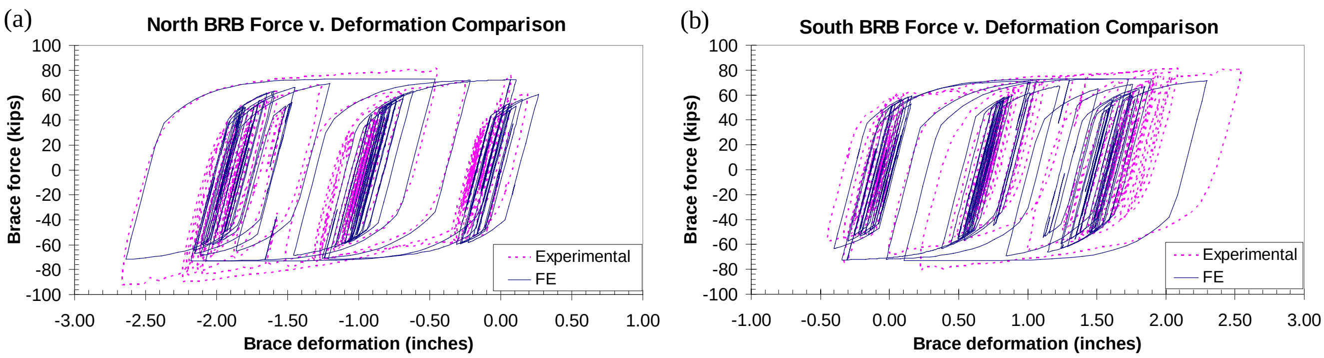

Figure 6 Brace axial force-deformation model verification at (a) north brace and (b) south brace

2.3. Analysis Matrix for Parametric Studies

The baseline model described above was used to study behavior and performance for variations in the beam-column-brace connection configuration. Variations were made to the brace end configuration (bolted, pinned or welded, as shown schematically in Figures 2a, 2b and 2c, respectively), beam end condition (continuous, shown in Figures 2a and 2c, or spliced, shown in Figure 2b) and gusset plate thickness (1 inch or ½ inch). Thus, twelve different cases were considered. For the bolted BRB cases, stiffeners were added to the gusset plates to accommodate the typical cruciform connection configuration. Since the bolts were assumed to be slip critical, the plates connecting the BRB core to the stiffened gusset were tied directly. For a BRB with welded connections, the brace core is welded to an end plate like the BRB with pinned connections, but instead of knife plates and a pin, two plates oriented perpendicular to the brace core are welded to the end plate and fillet welded to the gusset plate. Like other welded interfaces in the model, these welded interfaces were modeled as fully attached.

3. FINITE ELEMENT PARAMETRIC STUDIES

Each frame model was loaded with one symmetric cycle to maximum story drifts of 0.04 radians to simulate the typical story drift expected under MCE-level seismic input. The results presented below focus on in-plane response and explore demand distribution within the connection region to identify critical locations and parameters affecting performance. Global force and deformation quantities as well as local measures of von Mises stress and equivalent plastic strain are explored with emphasis on the beam-column-gusset connection at the compression BRB in story 3.

Figures 7a and 7b present story shear-drift and BRB axial force-deformation, respectively, for the different connection configurations. Configurations with continuous beams exhibited increased story shear strength when compared to the configurations with spliced beams due to the large moment transfer through the beam-column connections for the continuous beams. The moment transferred through the beam splices was 15% of the moment transferred to the connection regions in the continuous beam cases. Brace behavior changed minimally when the beam end conditions were changed. For the different brace end connection cases (bolted, pinned and welded), there was little variation, although a minor increase in elastic stiffness was observed in the bolted case over the pinned and welded BRBs due to the more rigid end regions in the bolted BRBs. Little dissimilarity was observed in global behavior when the gusset plate thicknesses was varied.

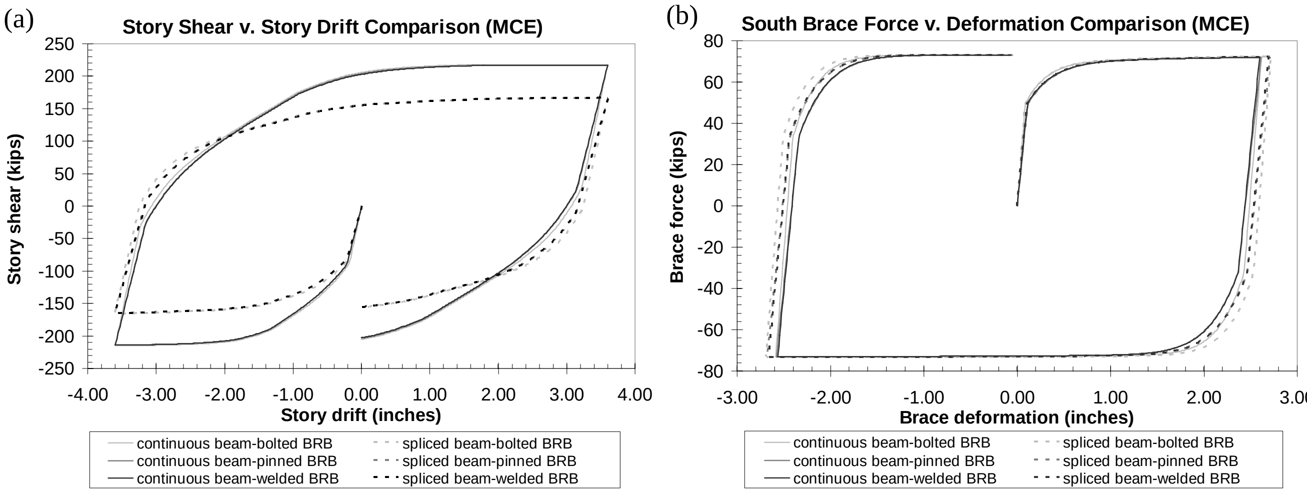

Figure 7 (a) Story shear-drift and (b) brace axial force-deformation (south brace) for 1-inch gusset

Locally, beam end condition and gusset plate thickness influenced plastic strain and stress demands most while brace end configuration had little effect on these response quantities. Contour plots of the von Mises stress for both beam end conditions with a 1-inch gusset plate, shown in Figures 8a and 8b, qualitatively illustrate the role that the splices had on limiting connection region demands. A contour plot for the ½-inch gusset plate appears in Figure 8c for the continuous beam case. Comparison of Figures 8b and 8c shows the distributed yielding that occurred in the thinner gusset plate in contrast to the thicker gusset plate's diagonal region of concentrated stress. Although the ½-inch gusset plate had more evenly distributed stresses, these stresses were generally higher than those in the 1-inch gusset plate.

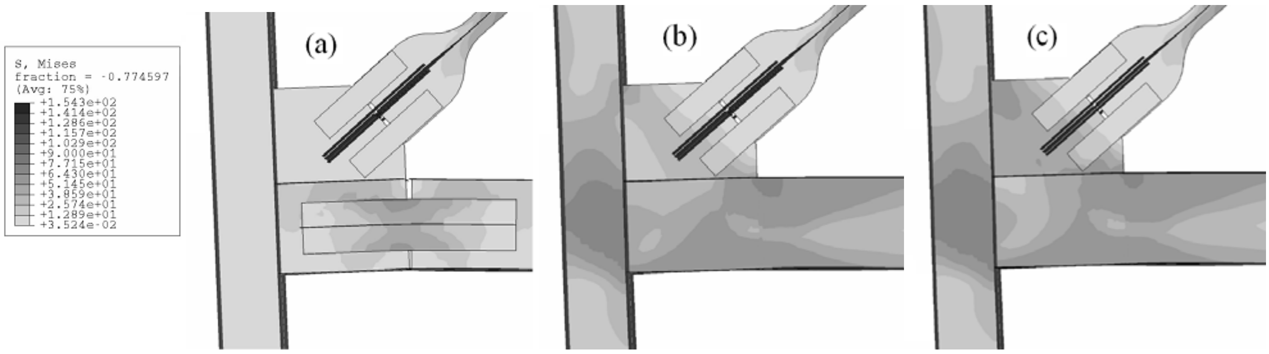

Figure 8 von Mises stress contours – (a) and (b) with 1-inch gusset and (c) with ½-inch gusset

Figures 9a and 9b plot equivalent plastic strain and von Mises stress, respectively, for four critical connection regions. These regions are labeled 1 to 4 in Figure 2c. Figure 9 is for the configurations with bolted BRB connections, but similar trends were observed for pinned and welded BRB connections. Most notably, equivalent plastic strain was zero for all regions in the spliced beam, but for the continuous beam case, plastic strain developed throughout the connection region due to the increased demands created by the larger moment transfer. The highest concentration of plastic strain was at Region 2, due to the frame action that opens and closes the connection region. This location is expected to be a potential point of fracture initiation, as observed in the Aiken et al. (2002) experimental program. A similar but somewhat less pronounced plastic strain concentration occurred at Region 1. At Region 2 the maximum plastic strain was in the beam, but at Region 1 the maximum plastic strain occurred in the gusset. In each case, the maximum plastic strain developed in the more flexible component, and it is observed that when the connecting elements have closely proportioned stiffnesses, strain concentrations are less likely to occur. Like plastic strain, von Mises stress was much higher for the continuous beam case than for the spliced beam case.

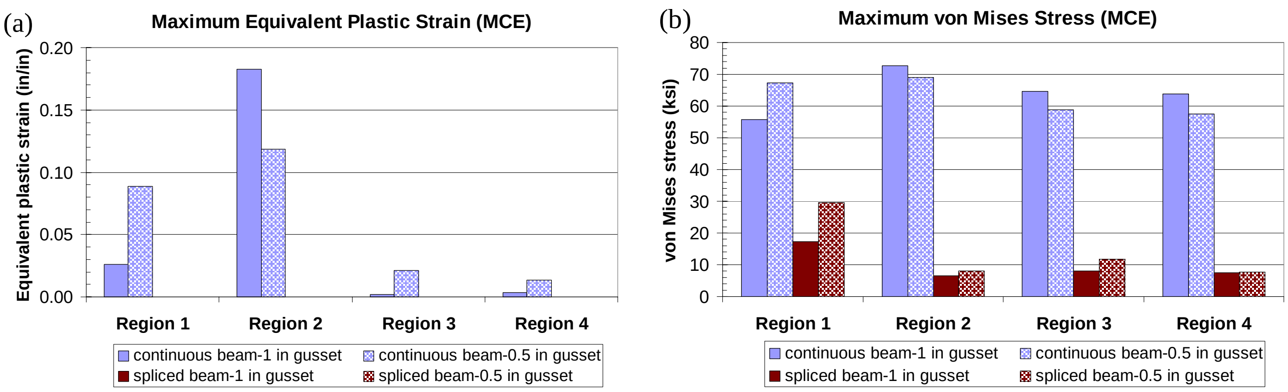

Figure 9 Maximum (a) equivalent plastic strain and (b) von Mises stress for bolted BRB

To further explore the distribution of demands within the connection regions, von Mises stress profiles were examined along the gusset plate interfaces. As shown in Figure 10, which plots von Mises stress profiles along the vertical gusset plate interface, the stress distribution in the gusset plate for the continuous beam case had a large gradient while for the spliced beam case, the distribution was more even. This implies that there was minimal moment on the connection interface in the spliced beam case, which agrees with the Uniform Force Method (UFM) design approach that seeks to produce bracing connections without moment (AISC, 2005a).

Approximate predicted stress distributions, one set based on brace force only and another set that also includes a distortional force as proposed by Thornton and Muir (2008), were calculated for comparison with the stress distributions from FE analysis. On the connection interfaces, the resultant forces associated with brace and distortional forces were decomposed into components normal and transverse to the interface being considered. For the brace force components, uniform normal and shear stress distributions were assumed to equilibrate the resultant forces. For the distortional force transverse component, a uniform shear stress distribution was assumed, and various normal stress distributions were evaluated for the normal force component. It was determined that a stress distribution with parabolic variation (nonzero at both ends) was the most representative of both shape and magnitude of the stress distributions from FE analysis. The calculated stress distributions also appear in Figure 10 with the FE analysis results and highlight the error introduced by neglecting the effect of distortional forces in the continuous beam case.

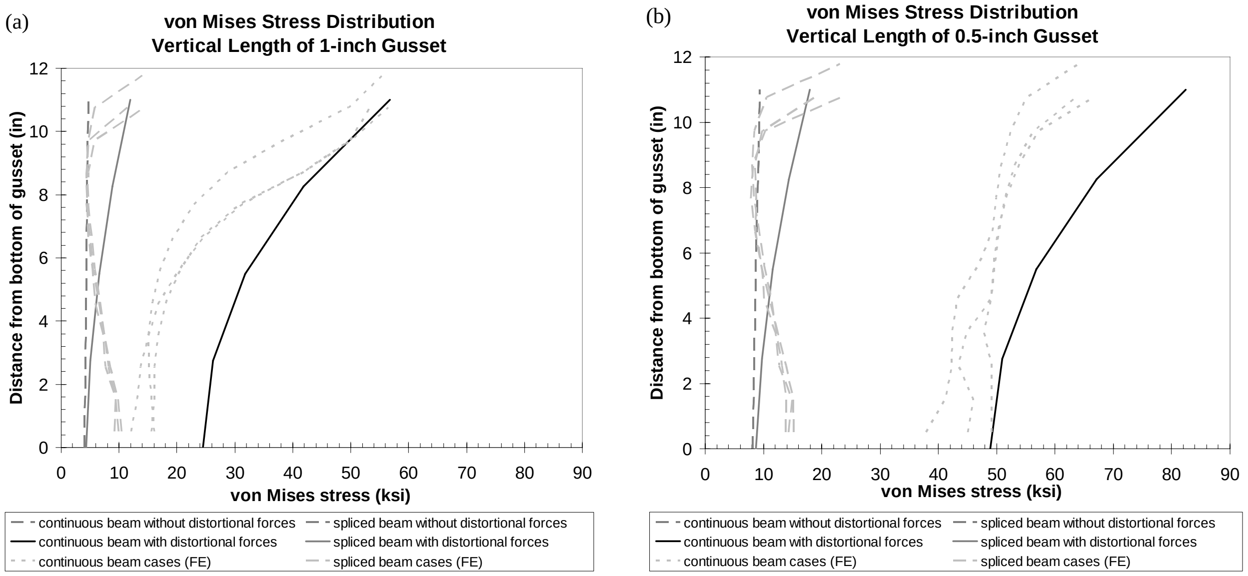

Figure 10 Stress distributions from FE analysis and predictions– (a) 1-inch gusset and (b) ½-inch gusset

4. SUMMARY AND CONCLUSIONS

The results presented in this paper provide general insight into parameters that influence behavior and performance of beam-column-brace connections in BRBFs. From this, the following conclusions can be drawn:

- Brace behavior showed minimal variation in the twelve configurations considered. If undesirable connection-related failure modes are precluded, it is expected that the BRBs are capable of achieving performance similar to that observed in isolated BRB tests.
- Story shear-drift behavior showed minimal variation for different BRB end connections and gusset plate thicknesses but showed significant variation due to the beam end condition (continuous or spliced). Thus, for the continuous beam case, there are essentially two lateral systems in parallel, the BRBF and a de facto moment frame.
- BRB connection type had little effect on the stress and plastic strain demands in the connection region, but stress and plastic strain magnitudes and patterns were appreciably different for the continuous and spliced beam cases. The bolted beam splice was found to significantly reduce demands on the connection region.
- For cases with thinner gusset plates, gusset plate stresses were generally larger and more evenly distributed, whereas a thicker gusset plate created a diagonal zone of concentrated stress. Extensive yielding in the gusset plates is judged not to be ideal since it makes the plate more vulnerable to out-of-plane deformation and undesirable failure modes in the connection region.
- While a thicker gusset plate will reduce stress and distributed yielding within the gusset, a gusset that is too thick as compared to the connected members will create higher localized plastic strain demands at the

connection interfaces. Thus, gusset plates should be sufficiently thick to prevent large distributed stresses, but the stiffness of the gusset should be proportioned to match the connected members to mitigate large strain concentrations at the interfaces.

- Minimal stress gradients along the gusset-beam and gusset-column interfaces for the spliced beam case indicated minimal moment at those interfaces, which was consistent with the Uniform Force Method in the AISC *Steel Construction Manual* (2005) that aims to design braced connections that have no moment.
- Approximate calculated stress distributions at the gusset plate interfaces due to the brace force only and due to the combined effects of the brace and distortional forces showed that neglecting the effects of the distortional force introduced significant error for the continuous beam configuration.

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