INELASTIC SYSTEM BEHAVIOR OF BUCKLING-RESTRAINED BRACED FRAMES AT MODERATE AND LARGE DRIFTS

K. Palmer¹, C. Roeder², D. Lehman²

ABSTRACT

Buckling-restrained braced frames (BRBF) are one of the most popular lateral force resisting systems for use in high seismic areas in the U.S. Numerous component tests on isolated buckling-restrained braces (BRB) have demonstrated their well-defined, approximately symmetric and stable hysteretic behavior. Combined with their large component ductility, this makes them an ideal energy dissipating mechanism. However, this system is not without its share of problems. Nonlinear response history analyses have demonstrated that BRBF are susceptible to relatively large residual drifts. Additionally, recent large-scale frame experiments suggest that while the BRB may reach its expected strength and deformation capacity, the system deformation capacity may be limited by damage to other components or by unintended system response mechanisms. While AISC component and subassemblage tests may ensure adequate BRB behavior, they cannot ensure adequate system performance. This paper will review prior BRBF test results and discuss the system behavior at moderate and large drifts. This system behavior includes significant yielding, local buckling, twisting and fracture of the beams and columns, tearing of the gusset plate welds to framing members and out-of-plane movement of the BRB. An analytical study was performed which used high-resolution models validated with prior test results. The study is used to develop strategies to mitigate damage and improve system performance. The study reveals that damage to the beams and columns at a corner gusset plate connection is related to the ratio of the component web thickness to the gusset plate thickness, suggesting a modest design change that will significantly improve BRBF performance.

¹Senior Engineer, Simpson Gumpertz & Heger, San Francisco, CA 94111 (email: kdpalmer@sgh.com)
²Professor, Dept. of Civil Engineering, University of Washington, Seattle, WA 98195

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ABSTRACT

Buckling-restrained braced frames (BRBF) are one of the most popular lateral force resisting systems for use in high seismic areas in the U.S. Numerous component tests on isolated buckling-restrained braces (BRB) have demonstrated their well-defined, approximately symmetric and stable hysteretic behavior. Combined with their large component ductility, this makes them an ideal energy dissipating mechanism. However, this system is not without its share of problems. Nonlinear response history analyses have demonstrated that BRBF are susceptible to relatively large residual drifts. Additionally, recent large-scale frame experiments suggest that while the BRB may reach its expected strength and deformation capacity, the system deformation capacity may be limited by damage to other components or by unintended system response mechanisms. While AISC component and subassemblage tests may ensure adequate BRB behavior, they cannot ensure adequate system performance. This paper will review prior BRBF test results and discuss the system behavior at moderate and large drifts. This system behavior includes significant yielding, local buckling, twisting and fracture of the beams and columns, tearing of the gusset plate welds to framing members and out-of-plane movement of the BRB. An analytical study was performed which used high-resolution models validated with prior test results. The study is used to develop strategies to mitigate damage and improve system performance. The study reveals that damage to the beams and columns at a corner gusset plate connection is related to the ratio of the component web thickness to the gusset plate thickness, suggesting a modest design change that will significantly improve BRBF performance.

Introduction

Buckling-restrained braced frames (BRBFs) are concentrically braced frames (CBFs) that utilize buckling-restrained braces (BRBs) to provide stiffness, strength and energy dissipation during an earthquake. A BRB prevents the brace from buckling in compression providing nearly symmetric force-displacement behavior in tension and compression. This results in superior energy dissipation capacity for the BRB component relative to a conventional buckling brace.

The design and testing of BRBs in the U.S. is governed by the AISC Seismic Provisions [1]. BRBs must pass qualifying cyclic brace component and subassemblage tests as specified in Section K3 of the Seismic Provisions. The component test consists of cyclic uniaxial loading, but the subassemblage test also attempts to simulate connection rotational demands on the BRB caused by frame action. Recent experiments on BRBF systems suggest that the BRB may reach its

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²Professor, Dept. of Civil Engineering, University of Washington, Seattle, WA 98195

expected strength and deformation capacity, but the system deformation capacity may be limited by damage to other components or by unintended system response mechanisms [2]. As a consequence, the BRBF system performance is often more complex than suggested by current design methods.

**SUMMARY OF PRIOR RESEARCH RESULTS**

Numerous component tests [3, 4, 5] have been performed on BRBs in the U.S, and these tests demonstrate the ability of the BRB component to achieve ductility and cumulative inelastic deformation greater than demands expected during the Design Basis and the Maximum Considered Earthquakes.

Uriz [6] tested three large-scale partial two-story, one-bay, planar BRBFs and Christopolus [7] tested five, full-scale, single-story, one-bay planar BRBFs. The BRB ductility values achieved during these tests ranged between 14 and 22, which are comparable to those achieved in BRB component tests. Additionally, the cumulative ductility values achieved in these BRBF experiments exceeded the minimum value of 200 required by the Provisions for BRB component tests. However, significant column and/or beam yielding and local buckling and tearing of flanges, welds, and gusset plates occurred in these tests such as illustrated in Figure 1a. Severe local yielding occurred at drifts between 1.4 and 1.7%. Less severe beam and column local buckling and yielding occurred in the specimens with tapered, more compact gusset plates. In many cases, out-of-plane rotation of and plastic hinge formation in the BRB core plate outside of the restrainer occurred at story drift ratios less than 2.5% as illustrated in Figure 3b. This plastic hinge formation was likely a consequence of a combination of frame yielding and deformation and the stiffness of the gusset plate and unrestrained BRB core.

A large-scale, two-story, one-bay by one-bay, 3-dimensional BRBF was tested by Palmer [8]. The BRBs were placed in two orthogonal bays, with a single BRB in each story (Figure 1a); the remaining bays were designed and detailed as gravity frames. The floor system consisted of intermediate beams and composite slab on metal deck on the first level to simulate the strength and stiffness of the floor system, and a reinforced concrete slab on the second level to also transfer loads to the frame from the actuators. A bi-directional, cyclically increasing displacement history was applied to the top floor of the system. In the braced bays, the beam flanges and webs were attached to the columns with CJP welds. The gusset plates were attached to the beams and columns with fillet welds sized according to the Uniform Force Method (with interface moments included) and Seismic Provisions [1, 9]. The beams and columns were checked to ensure that they passed the web crippling and web yielding checks. The dimensions and member sizes of the south frame are shown in Figure 1b.

Significant BRBF damage concentrated adjacent to the beam-column-gusset plate connections was observed, and very little damage was observed at joints without gusset plates. Tearing of the gusset plate-column interface welds occurred when the brace was in compression and these tears propagated the total length of the weld in three locations by story drifts ranging between 2.3 to 2.9%. The BRBs were still performing well after these tears and no other negative behaviors were observed due to these tears. Local flange and web buckling occurred at the base of two braced frame columns and in the beams adjacent to the gusset plate corner connections at approximately 2.5% story drift. Extensive beam flange and web tearing and fracture occurred at approximately 3.5% story drift. Column flange and web tearing also occurred in one location and initiated in the column K-region at the CJP weld connecting the braced frame beam bottom flange.
to the column. The second story BRBs fractured at 4.2% and 3.6% story drift in Frames 1 (in foreground) and B (on right), respectively.

This brief summary of prior research shows that the excellent component behavior measured for BRBs does not necessarily translate to equivalent system behavior. BRBF system performance often fell short of the expected performance based on the component results. Experiments on BRBF systems can be used to evaluate mechanisms to improve the system response, but experiments are too costly for evaluation of all critical parameters. This following sections describe the use of robust, high-resolution numerical simulation for this purpose.

![Figure 1. Damage Noted in Prior BRBF Test Results](image1.jpg)

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![Figure 2. BRBF used for finite element simulations (Palmer 2012): (a) experimental frame; (b) frame elevation.](image2.jpg)

Figure 2. BRBF used for finite element simulations (Palmer 2012): (a) experimental frame; (b) frame elevation.

**Finite Element Modeling**

The ABAQUS analysis platform [10] was used to accurately simulate the global and local
responses of the BRBF test specimen shown in Figure 4 and summarized above. Variations on that basic model were then made to study selected parameters. The ABAQUS model simulated only one planar braced frame as highlighted in the foreground in Figure 2a and shown schematically in Figure 2b.

Beams, columns, slabs and connections were modeled using 3 and 4-node quadrilateral shell elements as shown in Figure 3a. The BRBs were modeled using nonlinear truss elements and was calibrated to measured BRB behavior and provides accurate simulation of the local and global performance of the BRBF system. A rigid, control node was used to apply the displacement history of the frame. The concrete slabs were simulated with shell elements, which were linked to the shell elements of the steel beam to develop composite action. The cyclic displacement history for the simulations matched that applied during the test. Geometric nonlinearities were included in the analyses using the updated Lagrangian approach to capture local (and global if it occurs) buckling of the beams, columns and gusset plates. Steel material nonlinearities were simulated using the von Mises yield criterion with an associative flow rule and a combined, nonlinear kinematic and isotropic hardening law.

A mesh refinement study indicated that a mesh size of 1 inch (25.4 mm) was needed in the connection regions. A 2 inch (50.8 mm) mesh size was used in regions away from the connection regions. Fracture is not simulated directly by the ABAQUS computer program but equivalent plastic strain was tracked at the expected locations of fracture based on the methodology proposed by Yoo [11].

Figure 3. (a) BRBF finite element model with element types and boundary conditions (see Figure 2b for component sizes and frame dimensions); (b) global base shear-story drift response.

**Model Validation**

The model was validated by comparing global response, observed damage modes, and local deformation measures. Figure 3b shows the total base shear as a function of the average story
drift of the top of the frame. There is excellent agreement between the measured experimental results and the simulation. Fracture was not simulated and, therefore, the loss in strength resulting from BRB fracture is not captured in the FE model. Figures 4a and 4b show the excellent agreement between the test and the simulations of the local flange buckling and out-of-plane web deformation at the second floor beam-column-gusset joint.

**Parametric Study**

The verified model was expanded to investigate salient parameters that affect the BRBF response and to evaluate potential improvements to BRBF design. Approximately 50 different models were analyzed to study the effects of nine parameters, of which only the following are discussed in this paper:

1. Beam reinforcement at the joint, including web doubler plates and flange stiffeners (Figure 5a).
2. Column size and reinforcement at the joint, including web doubler and continuity plates (Figure 5a).
3. Gusset plate thickness

Comparisons and evaluations were based upon global and local force-displacement behaviors, computed von Mises stress distributions and deformed shapes in critical regions, and local strains along the gusset plate interfaces with the beams and columns. These performance indices were evaluated at story drift levels of 2 and 3.5%, since these drift levels approximate the Design Basis and the Maximum Considered Earthquake demands, respectively [12].

![Figure 4. Reference model validation: (a) observed beam yielding and local buckling; (b) simulated beam yielding and local buckling.](image)

**Reinforcement of Beam and Column Webs**

While some columns were larger than required in the 3-dimensional test specimen, significant damage was noted in nearly all beams and columns adjacent to gusset plates in test frames. This raises logical questions as to how the specimen would have performed if the minimum permissible column were employed, and how damage can be reduced in the beam and column locations without excessively increasing beam and column sizes. Frames with columns designed with demand-capacity-ratios close to unity were analyzed to assess this impact.
Figure 5b plots the envelope of the cyclic moment-story drift response, where the moment is measured at the top of the column of the second story for 4 different models. The analysis shows that the reference model (test specimen) provided somewhat larger base shear resistance than the model with weaker columns because of the larger column shear resistance. Deterioration in resistance occurred with both the reference model and the reduced column model due to the inelastic damage in the beam and column which including local buckling, but the deterioration and damage was significantly greater with the smaller column. The addition of web reinforcement (a doubler plate to the column web to create a column web thickness of 75% of the gusset plate thickness) eliminated the deterioration of resistance and reduced column damage for both the reference column and the smaller column specimen. While the addition of column web reinforcement reduced damage to the column, it invariably increased damage to the beam unless comparable measures were taken for the beam web.

Figure 5. (a) Finite element model details: beam and column reinforcement; (b) Moment-story drift backbone at top of second story column.

Comparison of analytical results in Figure 6 shows the effect of local damage to the beam. Figure 11a shows the extensive damage to the beam and lesser damage to the column for the reference specimen at 3.5% story drift for one connection in the frame. It must be emphasized that the ABAQUS model does not directly include cracking, tearing or fracture, and prior discussion has demonstrated extensive cracking and fractures in beams, columns, and gussets at deformations well below this level. Figure 6c shows the increased damage to the column web and reduced damage to the beam if the lighter column is employed. Figure 6b shows the reduced damage to the beam web if the web of the reference model is reinforced as noted earlier, while comparison of Figure 6c and 6d shows the reduced damage and stress levels in the column when the web of the smaller column is reinforced. Finally, Figure 6e shows the reduced damage to both the beam and the column webs are reinforced with the model with the lighter column section. These comparisons show that the addition of web reinforcement to create a total effective thickness of 75% of the gusset plate thickness eliminated all damage to the beam and the columns, and significantly changed and reduced local stress demands in the beam and column webs.

Extensive beam and column damage was observed in the experiment, and the comparisons in Figs. 5b and 6 suggest that the relative thickness of the gusset plate to beam and column web
thickness is a contributing cause of this damage. In the experiments, the relatively thicker gusset plates sustained minimal yielding in contrast to the extensive yielding in the relatively thin beam and column webs. This is logical when considering that the webs of the W16×50 beam, W12×72 column and W12×106 column were 0.38, 0.43, and 0.61 inches (9.7, 11, and 15.5 mm) thick, respectively, while the gusset plates were 1 inch (25.4 mm) thick. The stress in the gusset plate has a direct path into the beam and column web, and so a BRB that requires a thick gusset plate should also require a relatively thick beam and column web. This situation is aggravated with BRBs as compared to buckling braced frames, because brace buckling reduces stress demands on the gusset although increasing deformation demands on the gusset.

![Simulation results stress contours and deformed shape at 3.5% story drift](image)

Figure 6. Simulation results stress contours and deformed shape at 3.5% story drift (typical for deformed shapes)

**Reducing Gusset Plate Thickness**

Four modifications to the reference frame were made with 0.5, 0.625, 0.75, and 1.25 inch (12.7,
15.9 and 19.1 and 38.1mm) gusset plate thickness. The 0.5 and 0.625 inch (12.7 and 15.9 mm) thick gusset plates did not satisfy the standard buckling expression but were included in the analysis for comparison. Thinner gusset plates reduced the damage and stress levels in the beam and column webs but there was a corresponding increase in damage to the thinner gusset plates. Edge stiffeners were added to thinner gusset plates along the long edge to prevent extreme deformation of the gusset and these stiffeners reduced the damage to the gusset and correspondingly increased the damage to the beam and column so that it approximated the damage of the thicker gusset plates. With this evaluation, it is clear that thinner gusset plates are unlikely to be effective in reducing unwanted damage in BRBs.

**DESIGN AND DETAILING RECOMMENDATIONS**

Based on the experimental work and simulations, the following recommendations are made for the corner connection region of BRBF systems to mitigate damage at moderate to large drifts:

**Beam**

1. Beam web reinforcement should be placed at the corner gusset plate locations and extend at least to the larger of 0.75 \(d_b\) and 12 inches (300 mm) beyond the gusset plate edge, as shown in Figure 7. This reinforcement should be placed as close to the column face as possible and will be limited by the beam web connection plates or angles. The web reinforcement should increase the total web thickness to 75% of the gusset plate thickness. This ratio may be reduced given further experimental verification.

2. Backing bars should be removed at the beam bottom flange CJP connection to the column, as they are required in special moment frames per the Provisions (AISC 2010), unless there is a gusset plate connection to the column and bottom flange of the beam.

**Column**

3. Column web reinforcement should be installed in the panel zone and within the gusset region and extend at least to the larger of 0.75 \(d_c\) and beyond the gusset plate edge as shown in Figure 14. The web reinforcement should increase the total web thickness to 75% of the gusset plate thickness.

4. Continuity plates should be provided in the column at the beam flanges according to the Provisions (AISC 2010) when the beam flanges are connected to the column and expected to behave as a partially or fully-restrained connection (Figure 7).

**Gusset Plate**

5. The welds connecting the gusset plate to the beams and columns should be complete joint penetration welds (CJP) or fillet welds with a strength equal to the yield capacity of the gusset plate. The expected yield strength of the plate, \(R_yF_y\), should be used in this calculation.
Most BRB studies have focused on the BRB and neglected its interaction with the adjacent components. Recent tests indicate that unwanted damage modes are sustained by BRBFs including local buckling of the beams and tearing of the interface weld. Using high-resolution modeling techniques, a validated model was developed and used to perform a parametric study. The primary objectives of the study were to quantify the effects of various parameters on the demands and behaviors of the joint region and develop verified design and detailing recommendations to improve the performance of these systems.

The study resulted in the following conclusions:
1. Reducing the gusset plate thickness reduced the demands in the beam and column at the gusset interfaces. However, buckling of the gusset plate limits how thin these can be and is not an effective way of reducing component demands in the connection region of BRBFs.
2. Analyses shows that designing the weld for the strength of the plate significantly reduces the local strain demand in the connection and is expected to reduce crack initiation and prevent weld tearing and fracture until much larger story drifts. Therefore, weld connecting the gusset plate to the beam and column should be sized to meet the strength of the gusset plate to mitigate weld tearing.
3. Stresses in the beam and column webs may be reduced by increasing the thickness of the beam and column web by the addition of web reinforcement, decreasing the thickness of the gusset plate, or by changing the connection configuration.
4. A smaller column reduced the demands in the beam, including local web and flange deformation at all drift levels. However, the demand and deformation in the column was increased. Adding column web reinforcement within the gusset region mitigated these demands and deformations and while this caused a slight increase in beam demands, the resulting beam demands were still considerably less than those seen in the reference model and had negligible impact on the behavior and performance of the beam. In lieu of adding doubler plates, it may be more economical to increase the column or beam shape to a size that has an appropriate web thickness but may be overdesigned for flexure and axial load.
5. The demands and local deformation in the beams and columns within the connection region were shown to be inversely proportional to the ratio of the beam web thickness to the thickness...
of the gusset plate. In other words, for a given beam or column size, a thick gusset plate will increase the demands on these elements relative to a thinner gusset plate. These demands are also dependent on other factors such as the relative beam and column size, and the mechanism used to accomplish the target thickness ratio (e.g. thin gusset plate versus adding a web reinforcement). Given that there is a limit on how thin a gusset plate can be due to potential buckling, adding beam web reinforcement is a more appropriate solution to mitigate damage. Adding beam web reinforcement such that the web to gusset plate thickness ratio was 0.75 is recommended.

References