



SEISMIC PERFORMANCE OF BUCKLING RESTRAINED BRACED FRAME SYSTEMS

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SUMMARY

A growing number of buildings in the U.S. are using buckling-restrained braces as the primary seismic lateral force-resisting system. At the University of California, Berkeley, buckling-restrained Unbonded BracesTM have already been used for both retrofit and new construction. Recently, as part of the validation of the design of buckling-restrained braced frames for a major new laboratory building at the campus, tests of large-scale braced frame subassemblies were performed. Three frame subassemblies, with Unbonded BracesTM in one chevron and two single-diagonal configurations, were subjected to design-level and beyond design-level cyclic lateral loading. The tests showed good behavior of the braces, and the results indicated a number of important considerations for the design of buckling-restrained braced frames and also of braced frames in general. The paper describes the laboratory building project, the design of the test subassemblies, and presents an overview of the findings from the test program and subsequent analyses.

INTRODUCTION

The 1994 Northridge and 1995 Hyogo-ken Nanbu earthquakes demonstrated the susceptibility of steel moment-resisting frames to various types of damage associated with large lateral displacements. To alleviate such problems, engineers are increasingly turning to concentrically braced frames as a practical and economical means of enhancing the lateral strength and stiffness of steel buildings. However, conventional bracing systems have performed poorly in several recent earthquakes (Sabelli *et al* [1]), and recent research findings suggests that such systems may not be as reliable as other common systems (e.g., Uriz *et al* [2]). Consequently, considerable activity is underway worldwide to improve the performance of concentrically braced steel frames through the development of new structural configurations (e.g., Khatib *et al* [3]), response mechanisms (Hucklebridge [4]), or bracing elements, including those utilizing composite action (Liu, [5]), metallic yielding (e.g., Watanabe [6]; Kamura, [7]), high performance materials (Ohi [8]), friction and viscous damping (e.g., Aiken, [9]). In particular, bracing elements having yielding cores confined within a matrix that restrains local and lateral buckling have found widespread

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application in the US, Japan, Taiwan and elsewhere (see Aiken [10], Brown [11]). These elements exhibit nearly ideal bilinear hysteretic characteristics, have large cumulative energy dissipation capacities, and employ conventional design and construction for their incorporation into a structural system.

Questions often arise with the introduction of new technologies, such as those being proposed for enhancing the performance of braced steel frames, regarding their impact on performance during future earthquakes, and the effectiveness of methods utilized in design, analysis and construction. For example, is an adequate margin of safety achieved (and how is this quantified), and what are the appropriate tradeoffs among stiffness, strength and toughness the desired performance (and how is this performance characterized)? The research reported in this paper is part of an on-going project with an overall goal of developing and validating a comprehensive performance-based design approach for steel braced frame structures, which incorporate conventional braces, buckling restrained (or other hysteretic) bracing elements or supplemental viscous damping devices. Initial efforts to characterize probabilistically the performance of braced frame systems are examined by Uriz and Mahin, and reported elsewhere (Uriz [2]). This paper focuses on initial experimental and analytical investigations of the behavior of buckling-restrained braced frames. A series of tests on three nearly full-scale buckling restrained braced frames is reported. These specimens are subjected to detailed 3D nonlinear finite element analyses as well as simpler analyses of 2D representations.

THE UC BERKELEY TEST PROGRAM

The Berkeley campus of the University of California has been one of the major early adopters of buckling-restrained braced frame technology in the US. Most of the Berkeley campus lies within approximately 2 km of the Hayward Fault, which is capable of producing events of M7 or even larger. During the next 30-50 years the fault is recognized to present the single largest threat of strong shaking in the San Francisco Bay Area. To mitigate the effects of a large near-by earthquake, buckling-restrained braces have been used already in four buildings on the Berkeley campus.

While uniaxial tests have demonstrated that individual buckling restrained braces have good ductility and hysteretic characteristics (Black *et al* [12], Ko *et al* [13]), the Berkeley campus Seismic Review Committee recommended that a subassembly test be performed as part of the design process for a building to replace Stanley Hall (a biotechnology complex located less than 100 m from the Hayward Fault). This recommendation was based on the growing use of buckling-restrained braces at UC Berkeley, the importance of the new Stanley Hall to campus, and a number of technical concerns. These technical concerns focused on the behavior of the braces under frame-induced axial and rotational deformations, the appropriateness of extrapolating brace performance expectations from uniaxial test results, and the behavior of connections under frame lateral deformations. While subassembly tests that induce axial as well as frame rotational deformations on the braces have already been performed in Japan (Fujimoto *et al* [15]; Konami *et al* [16]; Iwata *et al*. [17]), tests had not yet been performed using details similar to US practice.

At the time of the design of the new Stanley Hall, buckling-restrained braced frames had not yet been incorporated as a seismic lateral force-resisting system in any building code in the U.S. A number of nonlinear dynamic analyses had been undertaken by various investigators to help understand the seismic response of this type of braced framing system and evaluate possible design provisions (Sabelli *et al* [1] and [18]). In addition, a joint Structural Engineers Association of California and American Institute of Steel Construction (SEAOC/AISC) Task Group was in the process of completing a two-year effort to develop design provisions for buckling-restrained braced frames (Sabelli and Aiken [19]). These *Recommended Buckling-Restrained Braced Frame Provisions* (SEAOC [20]), hereafter referred to as the

Recommended Provisions, were utilized along with the AISC *Seismic Provisions for Structural Steel Buildings* (AISC [21]) in the design of the structure (Lopez *et al* [22]). The *Recommended Provisions* do not mandate project-specific testing. Prior testing of appropriately similar elements could be used to ‘qualify’ a brace design or concept. The *Recommended Provisions* stipulate, however, that for each brace design at least one qualifying subassembly test be performed that imposes both axial and rotational demands on the buckling-restrained braces. The lack of such a test for a frame subassembly with Unbonded Braces subjected to lateral deformations consistent with U.S. design criteria was another factor that, combined with those already discussed above, contributed to the need for the subassembly test for the Stanley Hall project.

STANLEY HALL REPLACEMENT BUILDING

Project Description

The new Stanley Hall on the University of California, Berkeley, campus is a 290,000 sq. ft. laboratory building for several departments working on bioengineering and biotechnology. It replaces an existing five-story, reinforced concrete laboratory building constructed at the beginning of the 1950s. The building consists of seven levels of steel framing over a three-level concrete basement, with a seismic lateral force-resisting system of buckling-restrained braces in the upper levels and concrete shear walls at the basement levels (Lopez *et al.* [22]).

Seismic Performance Goals

The University of California, Berkeley established a baseline design performance requirement of Life Safety in a design event with a probability of exceedence of 10 percent in 50 years, and Collapse Prevention in a larger 10 percent in 100-year event. It was desired to achieve an enhanced Immediate Occupancy performance level for this project, if this was economically justifiable, which aimed at resuming occupancy and use of the structure within a period of a few weeks after a 10 percent in 50-year event.

Buckling-restrained braces were chosen as the seismic lateral force-resisting system for the building because of their large ductility, energy dissipation capability, and also for the ease of repair after a major earthquake, a factor that was regarded as less problematic than for any other type of steel framing or bracing system (Lopez [22]).

Building Design

The new Stanley Hall structure was designed to meet the requirements of the 1997 Uniform Building Code (ICBO, 24). A more detailed description of the design has been given by Lopez *et al* [23], and is summarized here. Seismic forces were defined based on the following factors: Seismic Zone 4, Soil Profile Type SB, $R=8$, $I=1.0$, and Near Source Factors $N_a = 1.5$ and $N_v = 2.0$. A value of 1.0 was used for the Redundancy Factor, ρ . The design based shear was approximately $0.114W$, and the building story drift ratio was limited to 2.0 percent. Nonlinear static (pushover) analysis was used to check the design. The design of the buckling-restrained braces was based on the *Recommended Provisions*. Expected material stresses were used to define brace strengths, and the design of connections, columns and collector elements followed a capacity-based procedure (Lopez [22]).

TESTING PROGRAM

A series of three, nearly full-scale subassemblage tests were carried out to address three main questions (Lopez [22]):

- How do brace end rotations, induced by frame lateral deformations, affect overall brace behavior?

- Is the hysteretic behavior of an Unbonded Brace in a frame subassembly the same as that for a brace tested uniaxially?
- How do the brace to frame gusset plate connections perform under the expected building drifts?

The subassembly tests were performed in the Department of Civil and Environmental Engineering Structures Laboratory in Davis Hall, at the University of California, Berkeley. The following section describes the main features of the three test set-ups, the buckling-restrained brace designs, and the loading protocols used. A more complete description may be found in Uriz and Mahin [25].

Test Set-Ups

The testing program investigated the behavior of a one-story, single-bay, beam-column frame with Unbonded Braces. Three different subassemblies in two different configurations were tested, all utilizing the same beam-column components but each with different Buckling-restrained Brace designs. The three configurations are referred to as Test Nos. 1, 2 and 3 (Figs. 1, 2 and 3). Test No. 1 consisted of a chevron or inverted-V brace configuration, and Test Nos. 2 and 3 were both single-diagonal brace configurations. Lateral load was applied to the test frame by a 6,700 kN actuator attached to a “hat brace” arrangement. This upper level of bracing was intended to achieve a better overall representation of frame and connection characteristics in the subassemblies.

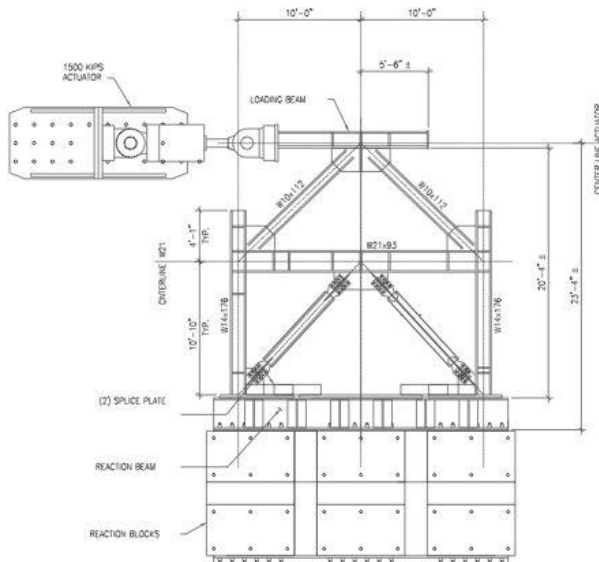


Fig. 1 Test Set-Up for Test No. 1

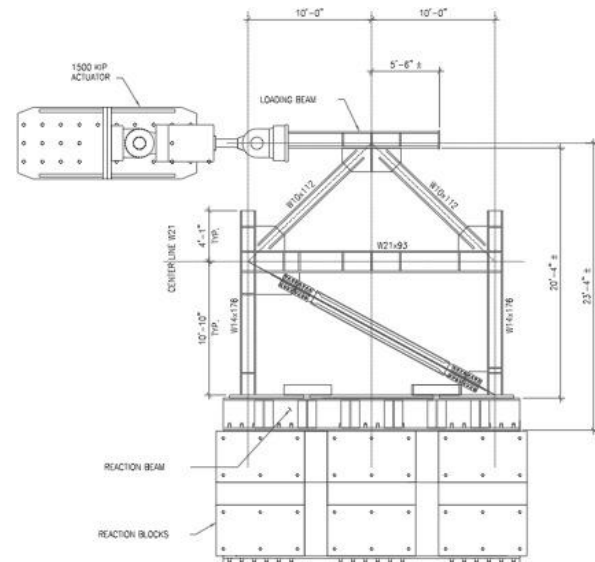


Fig. 2 Test Set-Up for Test Nos. 2 and 3

Capacity limitations of the testing equipment, as well as constraints on the overall size of the test frame specimen, dictated that the test subassemblies were slightly less than full-size when compared to frames in the actual building. In terms of geometry, the test subassemblies were approximately 70 percent of the actual building bay width and story height. The test specimens had a bay width of 6.1 m, a story height of 3.3 m, and comprised grade A572/50 W14x176 columns and a W21x93 beam. The upper level “hat bracing” members were designed to remain elastic for all expected loading conditions. The beam-column connections were designed to be moment resisting, generally satisfying the FEMA requirements for a WUF-W pre-qualified detail (FEMA [26]). The same basic beam-column frame subassembly was re-used for all three tests, varying only in the details for connecting the bracing system. The test frame was shop fabricated, then assembled in the laboratory in the upright position, and laid on its side after welding. The braces were installed after the frame was in the horizontal position. Notch-tough filler metal was used for all welds in the test frames.

The slip-critical bolted connections between the braces and the gusset plates used 32-mm diameter A490 bolts in standard size holes, direct tension indicator washers and Class A faying surfaces. The connections were designed for expected brace forces, including strain-hardening and compression over-strength contributions. From results for similar size braces tested in a previous testing program (Black *et al* [12]), over-strength factors of 1.65 for the braces in Test No. 1 and 1.50 for braces in Test Nos. 2 and 3 were used in the design of the connections.

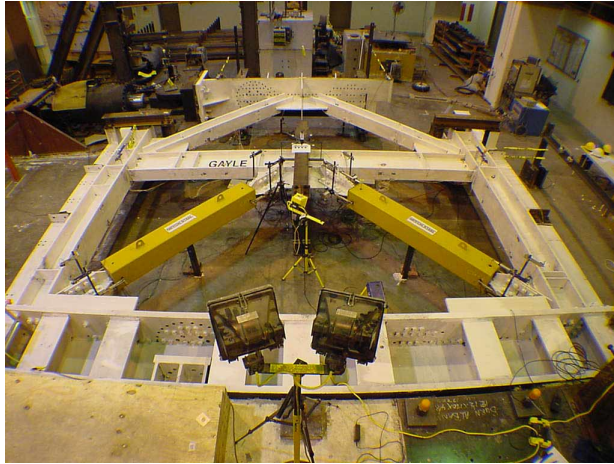


Fig. 3 Photograph of Test Specimen No. 1

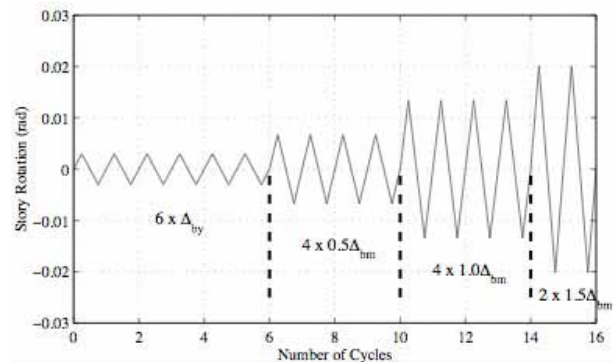


Fig. 4 Loading Protocol – Test No. 1

The gusset plate configurations were different for each of the three tests, and so after each test the old gusset plates were removed by gas cutting and new gusset plates were welded in place. Usual practice would use fillet welds for the gusset plate attachment to the beam and column flanges, however, since the test frame was in the horizontal position, single bevel full penetration welds were used, and the backing bar was not removed after welding.

The frame subassemblies were extensively instrumented. Instrumentation included displacement transducers for global frame displacements and brace displacements, and strain gauges to allow subsequent determination of frame forces and brace forces. Strain gauges were also included on the core yielding sections of the Buckling-Restrained Braces. More than 175 channels of information were recorded in each of the three tests.

Buckling-Restrained (Unbonded) Brace Designs

Three different buckling-restrained brace designs were investigated in the test program. All braces were of the Unbonded Brace™ type manufactured by Nippon Steel. The area of the flat bar yielding core plate was 4080 mm² for both Test Nos. 1 and 2, and the cruciform core of Test No. 3 had an area of 7540 mm². Japanese Industrial Standard (JIS) grade SN400B steel was used for the core plate for all braces, with a mill certificate yield stress of 280 MPa. The yield forces were approximately 1150 kN for the braces in Test Nos. 1 and 2, and 2130 kN kips for Test No. 3. To evaluate the possible effect of core plate orientation on brace behavior, the two flat-bar core plate braces of Test No. 1 were installed with one core plate oriented horizontally and the other vertically. The flat-bar core plate brace of Test No. 2 was oriented vertically.

Loading Protocols

The loading histories used for all three tests basically followed the requirements of the *Recommended Provisions*. These tests were intended to subject the brace(s) to a maximum deformation at least equal to

the design deformation, and to a cumulative plastic ductility demand of at least 140. For the test program, maximum drift ratios considered were approximately equal to those determined from a nonlinear static analysis of the building corresponding to 10 percent in 100-year demands. Figure 4 shows the loading history for Test No. 1. The general characteristics of the loading histories for Test Nos. 2 and 3 were similar, varying only in terms of the maximum frame lateral displacement. For Test No. 1, the design level cycles (Δ_{bm}) were carried out at an interstory drift of 1.34%, and beyond design level cycles were imposed corresponding to a drift of 2.01%. For Tests Nos. 2 and 3, these interstory drift values were 1.72% and 2.6%, respectively.

TEST RESULTS

A comprehensive description of the test results may be found in Uriz and Mahin [25]. Highlights of the test results are provided below. Since it was not desirable to include a load cell in series with the braces in the test set-ups, frame strain gauge information was used to estimate the brace forces. Strain gauges on the columns and beams, located at sections expected to remain elastic, were used along with considerations of mechanics and equilibrium to derive member forces. However, in the larger deformation cycles the column webs yielded in shear, and the assumption of elastic behavior at instrumented sections was no longer valid. As such, brace forces shown below are accurate only during low amplitude cycles.

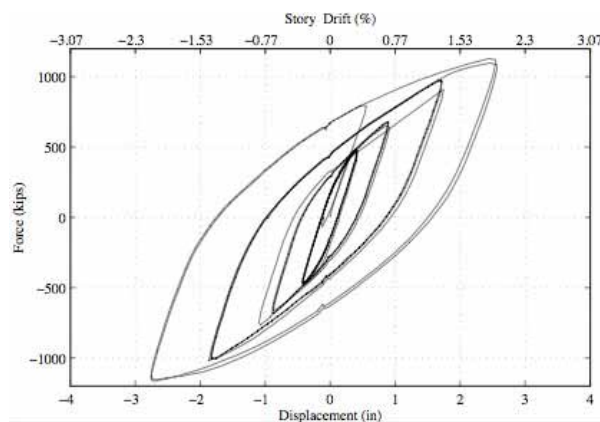


Fig. 5 Lateral Force-Story Lateral Drift, Test 1

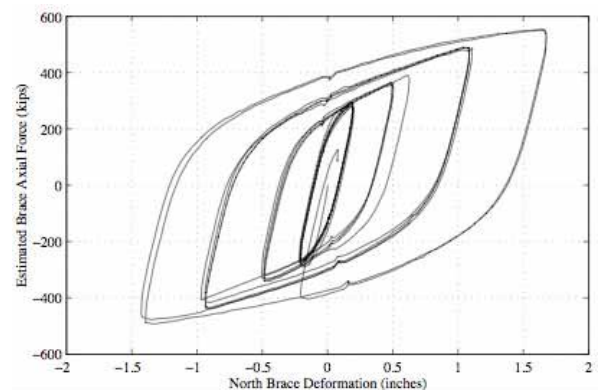


Fig. 6 Brace Hysteretic Behavior, Test No. 1

Test No. 1

The frame subassembly showed very good behavior overall through the entire sequence of cycles in this test (Fig. 5). The Unbonded Braces exhibited stable and repeatable hysteretic behavior (Fig. 6), through a brace ductility of approximately 15. At the time was stopped, the total cumulative plastic ductility sustained by the braces was approximately 326, or more than twice that required by the *Recommended Provisions*. The frame peak forces are extremely consistent in opposing directions as well as from cycle to cycle, implying similarly consistent behavior of the two braces. Yielding of the frame became apparent at the 0.67% drift cycles, in the columns near the top of the column base stiffeners, and in the column stiffeners themselves. During the 2% drift amplitude cycles, yielding was seen in the beam column connections, in the column webs over their entire height, and in the bottom gusset plates (Figures 7 - 9). No slip occurred in any of the brace bolted connections throughout the entire test. No yielding was observed in the top gusset plate at the beam mid-span at any stage of the test, and very little up-down movement at the center of the beam was noted (associated with differential forces in the braces (Figure 10).

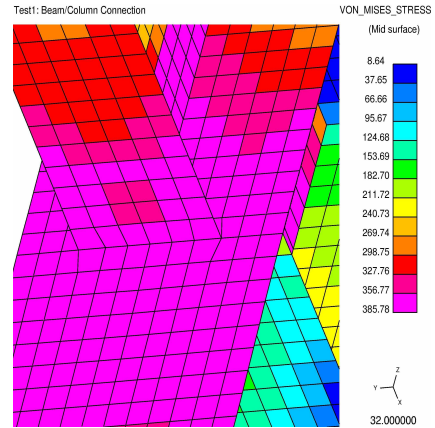
The different core plate orientations for the two braces appeared to have no influence on their hysteretic behavior. The rotations that occurred at the ends of the braces also appeared not to have any negative influence on the behavior of the braces.

Test No. 2

At the conclusion of Test No. 1, the braces and gusset plates were removed, new gusset plates were welded in place, and a new single diagonal brace was installed.

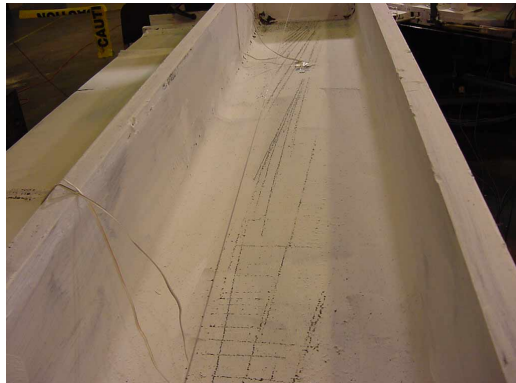


(a) Test Results

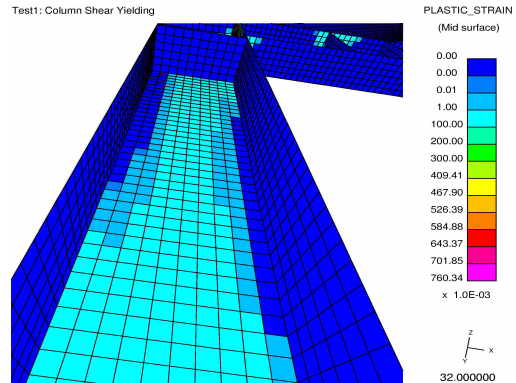


(b) Finite Element Results (Field *et al* [27])

Fig. 7 Underside of Beam - Column Connection, 2% Story Drift Cycles, Test No. 1



(a) Test Results

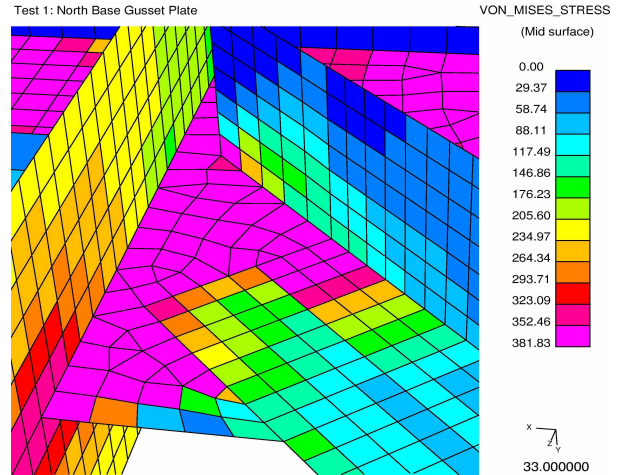


(b) Finite Element Results (Field *et al* [27])

Fig. 8 Column Web Shear Yielding, 2% Story Drift Cycles, Test No. 1



(a) Test Results



(b) Finite Element Results (Field *et al* [27])

Fig. 9 Column Base Gusset Plate Yielding, 2% Story Drift, Test No. 1

Throughout the entire loading history, the frame subassembly showed good behavior (Figure 11). The Unbonded Brace exhibited stable and repeatable hysteretic behavior (Figure 13), up to a maximum ductility of approximately 14 in cycles to 2.6% interstory drift. The total CPD sustained by the braces was approximately 299. The frame peak forces are extremely consistent, implying nearly symmetric tension-compression behavior of the Unbonded Brace. No slip occurred in the brace bolted connections at any stage in this test. Initial yielding in the frame was observed in the top gusset plate and in both column base stiffeners at the 0.87% drift amplitude cycles. In the 1.72% drift cycles, yield lines in the column webs became apparent, starting at the column bases and spreading up the entire height of the columns.

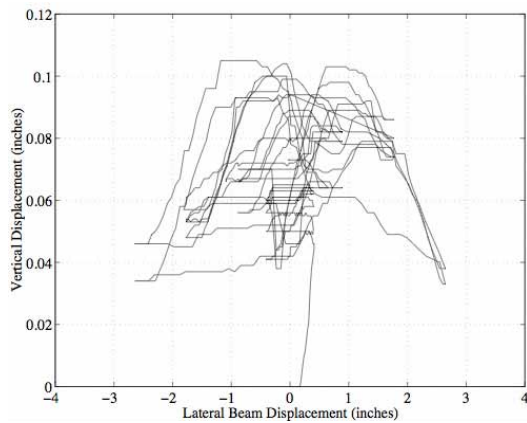


Fig. 10 Beam Mid-Span Vertical Displacement, Test No. 1

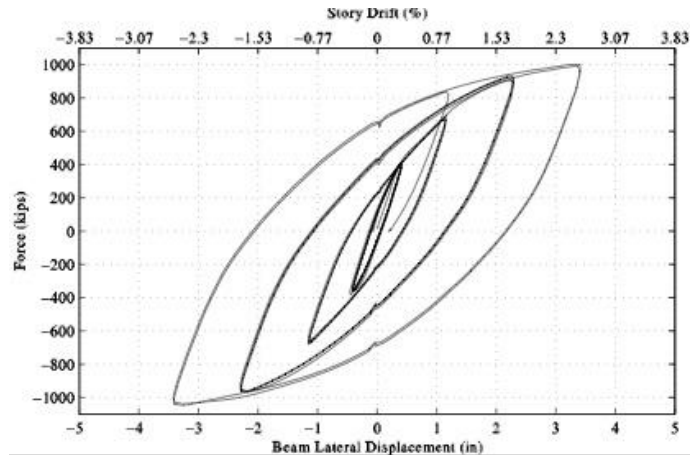


Fig. 11 Lateral Force-Story Lateral Drift, Test No. 2

In the last two cycles to 0.87% drift, weld cracks began forming at the bottom free edge of the top gusset plate, adjacent to the column. During the subsequent 2.6% drift cycles, the cracks in the top gusset plate continued to propagate. When the brace was in tension, noticeable buckling in the top gusset plate occurred, due to the frame action ‘pinching’ of the gusset between the beam and the column (Fig. 12). A nonlinear finite element analysis of the frame demonstrates the tendency for such buckling to occur [Field *et al* [26]]

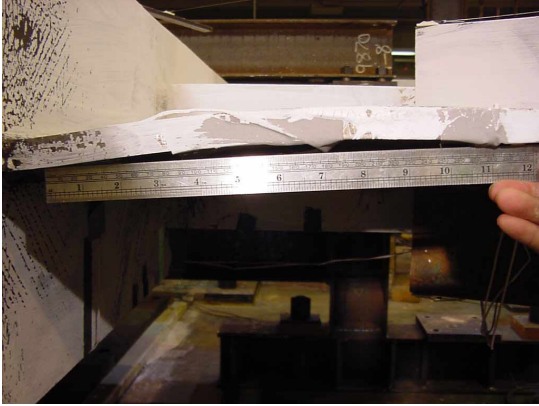


Fig. 12 Top Gusset Plate Buckling, Test No. 2

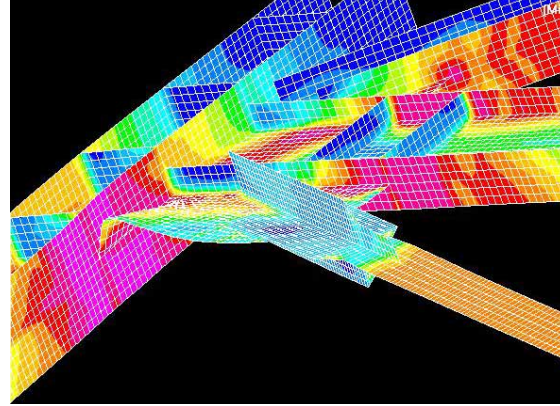


Fig. 13 Finite Element Analysis of Frame No. 2 Showing Buckling of Top Gusset Plate Due to Frame Pinching Action (Field *et al* [27])

As observed in Test No. 1, rotations did not appear to have any negative influence on the brace behavior. With the brace in a single-diagonal configuration, the maximum rotation at the ends of the brace was about half of the story rotation.

Test No. 3

At the conclusion of Test No. 2, the brace and gusset plates were again removed, new gusset plates were welded in place, and a new brace having a cruciform core was installed. In response to the gusset plate buckling and weld cracking that occurred in Test No. 2, a small rectangular stiffener plate was added to the bottom edge of the top gusset plate at the face of the column.

In the cycles to 1.72% drift, several damage conditions developed that indicated this deformation to be a limit state for the supporting beam-column frame. As was seen in Test Nos. 1 and 2, column web shear yielding developed progressively throughout the test. Yielding at the bases of the columns and also in the column base stiffeners also occurred, as previously. The beam-to-column connection at the column away from the unbonded brace in the lower level (Fig. 3) developed a transverse crack adjacent to the bottom CJP flange weld across the entire width of the flange (Figure 14). This crack continued to extend and widen during the remainder of the test. Several small cracks initiated at this level at the base of the columns as well. Although the peak rotations imposed on these connections were relatively small during these tests, this was the third loading sequence imposed on the connections in this specimen, suggesting that low cycle fatigue may be a consideration in assessing performance in long duration events.

During the first excursion at the 2.6% amplitude cycles, but while the drift was still less than 1.72%, the bottom flange of the beam at the outside edge of the gusset plate developed a crack across the entire width of the flange and about 2 inches into the web. This fracture resulted in a loss of torsional stability in the beam-gusset region at the top end of the brace (Figure 15). Because of the beam fractures, only one cycle at the 1.72% drift amplitude was completed, followed by two additional cycles at 0.87% drift. The crack continued to propagate into the web, and the bottom beam flange displaced transversely to a great extent.

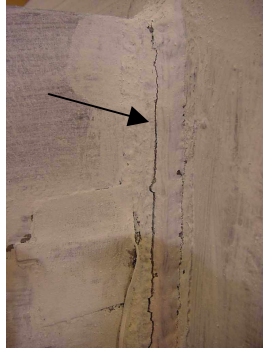


Fig. 14 Fracture in HAZ in Beam-Column Connection, Test No. 3



Fig. 15 Fracture in Beam at End of Gusset Plate, 2.6% Drift, Test No. 3

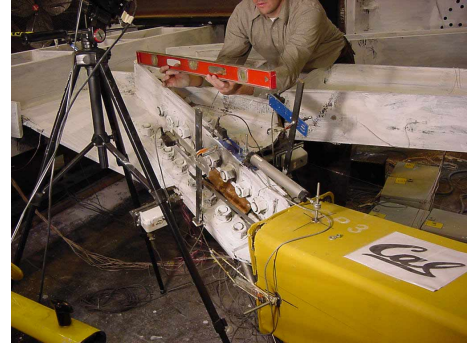


Fig. 16 Lateral Buckling of Brace due to Fracture of Bottom Flange and Web of Beam, 2.6% Drift, Test No. 3

In spite of the developing several small connection region fractures, the frame showed good overall hysteretic behavior up to the completion of the 0.87% cycles (Fig. 17). The Unbonded Braces showed good behavior up to the point of the beam flange fracture, and beyond that point accommodated several cycles of very large plastic rotations without any sign of local fracture. Because of the lateral displacements of the beam flange and gusset plate, the subsequent hysteretic behavior of the brace assembly was similar to that of a conventional brace (Fig. 18). The maximum brace strain demand in the test was measured to be 1.89 percent, and up to the point of beam flange fracture, the brace sustained a CPD of 219. Subsequent to the beam fracture, very large local inelastic rotations and axial deformations occurred near the ends of the buckling restrained brace. No slip occurred in the brace bolted connections at any stage in the test.

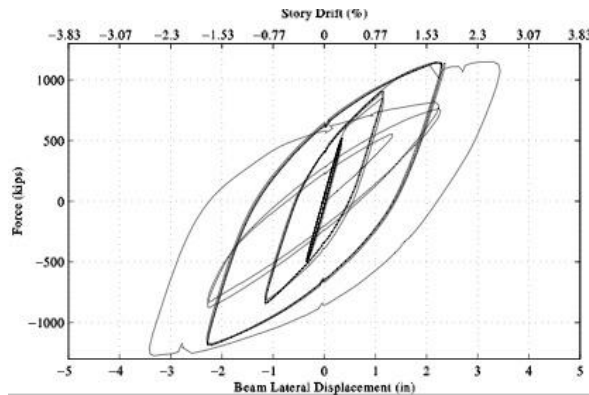


Fig. 17 Lateral Load-Story Drift, Test No. 3

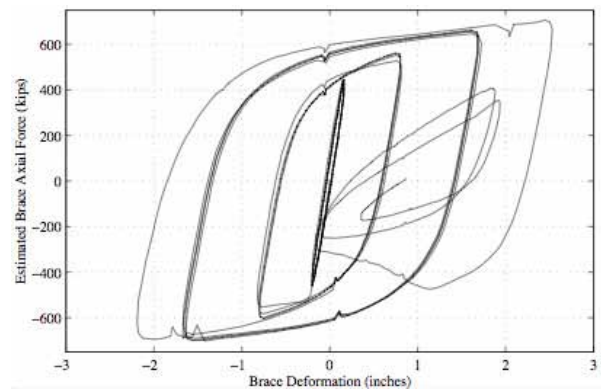


Fig. 18 Brace Hysteretic Behavior, Test No. 3

ANALYTICAL ASSESSMENT

Prior to testing, nonlinear models of the test frame subassemblies were developed using OpenSees (PEER [28]), in order to estimate the test set-up reaction forces. The model consisted of line elements with rigid end offsets at the gusset plates and stiffeners. Lumped plasticity frame elements with fiber cross-section accounted for moment-axial interaction. An axially uncoupled nonlinear shear spring was used to capture column shear deformations. A comparison of pushover analysis results with the experimental results for Test No. 1 is shown in Figure 19. It can be seen that the model incorporating column shear deformations better approximates the test results, but that the model without shear deformations still reasonably estimates the maximum frame force and deformation. Similarly good agreement was obtained for the other two test specimens.

CONCLUSIONS

The three buckling-restrained braced frame subassembly tests demonstrated the good performance of this seismic lateral system, and validated the selection and design of buckling-restrained braces for the new Stanley Hall building. The testing program represents the first large-scale subassembly investigation of the behavior of buckling-restrained braced frames in the U.S. The use of U.S. design details and deformation demands in the test program has resulted in much useful information, focused on the context of U.S. design practice. The Unbonded Braces tested performed well and provided a ductile braced frame with a large energy dissipation capacity. Various issues discussed below, related to both the general frame design and also the detailed design of gusset plate components, have been identified and warrant future study. Additional studies related to dynamic response of buckling restrained braced frames are described elsewhere by Sabelli, Chang and Mahin, [1] and Uriz and Mahin [2].

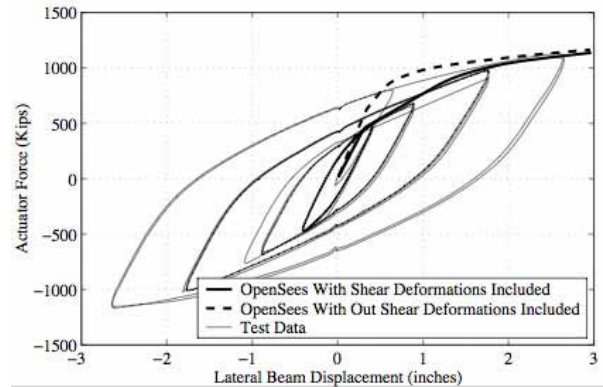


Fig. 19 Comparison of Static Pushover Analysis Results with Test Results, Test No. 1

Unbonded Braces

The Unbonded Braces performed very well in all three tests. Their hysteretic and elongation behavior appeared not to be influenced by the combined axial and flexural demands associated with loading in a frame configuration. In Test No. 1, there was no apparent difference in behavior between the two chevron configuration braces due to their different core plate orientations. The braces behaved well in both the chevron and single-diagonal configurations, and in the chevron configuration induced only small unbalanced force demands on the mid-span of the beam. As a result, the vertical displacement at the mid-span of the beam was small, and tended to stabilize following the first yield excursion. In general, it was found that the actual brace strength, including strain-hardening and compression over-strength contributions, could be accurately established from previous uniaxial test data, as well as from detailed finite element analytical assessments based on measured material properties.

Frame Subassemblies

The test frame subassemblies performed well at all drift levels for Test Nos. 1 and 2. In Test No. 3, a crack developed in the beam-column connection during the design story drift cycles. In the first cycle beyond the design level drift, the beam bottom flange fractured at the outside edge of the brace gusset plate connection. The beam flange fracture resulted in a loss of torsional stability and subsequent buckling of the end of the Unbonded Brace and the gusset plate connection region. The beam flange was highly stressed in tension at this time, and the material in the flange at the tip of the gusset plate is likely to be highly constrained. The failure of the beam at the brace gusset may also be associated with the fact that gusset plates in this location were removed and replaced twice in the course of the test program. The effects of heat from cutting and re-welding, combined with the substantial inelastic demand that occurred throughout the entire test program may have contributed to the beam flange failure in Test No. 3. Additional research on such gusset plate to beam and column connections is warranted.

Additional conclusions that can be drawn from the behavior of the buckling-restrained braced frame subassemblies include:

- The presence of the brace connection gusset plates may lead to rigid frame action in the braced bay, whether explicitly intended by the designer or not, and therefore the contribution of the gusset plates to the overall frame rigidity should be carefully considered.
- The FEMA pre-qualified WUF-W moment connections permitted the frame to complete Test Nos. 1 and 2 and part of Test No. 3 without failure.
- The use of notch-tough filler metal for all welds in the frames appeared to be beneficial in limiting the ductile propagation of weld fractures that occurred in various locations.

As identified by Lopez, 2002, a number of broader observations can be made on the importance of gusset plate design in the behavior of any type of braced frame:

- The effect of gusset plate size on shortening beam and column clear dimensions, and to shift behavior from flexural to shear modes needs to be recognized. Especially important in these tests is the occurrence of widespread shear yielding of the column webs.
- Kinematic deformations imposed on gusset plates as a result of frame lateral deformations are not well understood and require further study.
- For large frame drifts, force-based design methods for gusset plates may not result in acceptable designs. Further, the large range of variables in the design of gusset plates necessitates that a reasonable calculations-based methods and not simply relying upon test results.

ACKNOWLEDGEMENTS

Numerous significant contributions made the subassembly-testing program a success. The new Stanley Hall was designed by Rutherford & Chekene, which also designed the test subassemblies; Gayle Manufacturing Company fabricated the test frames, which were erected by California Erectors; Nippon Steel Corporation provided the Unbonded Brace test specimens; and inspection of the fabrication of the test frames was performed by URS/Signet Testing Laboratories. The Stanley Hall Replacement Project is being undertaken by the University of California, Berkeley, Capital Projects, which has benefited from the advice of their seismic consultant, Comartin-Reis. Finally, the testing program would not have been possible without the Structures Laboratory staff at the University of California, Berkeley.

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