Abstract

A growing number of buildings in the U.S. are using buckling-restrained braces as the primary seismic lateral force-resisting system, and the University of California has been one of the major early adopters of this new technology.

At the University of California, Berkeley, buckling-restrained Unbonded Braces 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 Braces 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 for 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.

Keywords: buckling-restrained brace, unbonded brace, braced frame, subassembly testing, steel buildings, yielding, ductility

1. Introduction

The lessons learned with regard to steel moment-resisting frames in the Northridge earthquake of 1994 and the Kobe earthquake of 1995 resulted in a renewed willingness on the part of engineers to consider braced frame lateral systems. This willingness, coupled with the fact that the performance of buckling-restrained braces overcomes the recognized shortcomings of conventional concentric bracing systems, has been a contributing factor to the acceptance and rapid adoption of buckling-restrained braces in the U.S. Buckling-restrained braced frames are regarded as being capable of providing similar, or perhaps even better, performance than eccentrically-braced frames, and with the added benefit of removing ductility and energy dissipation demands from the primary gravity load-supporting frame of the structure and confining it to structural elements — the buckling-restrained braces — specifically-designed and better suited to those tasks.

Unbonded Braces, one type of buckling-restrained brace that consists of a yielding steel core confined by mortar within a steel tube, have been quickly accepted in the U.S., with more than 20 projects underway or already completed in the space of about three years (Aiken and Kimura, 2001, Brown et al., 2001;). The University of California, and in particular the Berkeley campus, has been one of the early adopters of buckling-restrained braced frame technology.

The University of California, Berkeley, campus is located in the East Bay of the San Francisco Bay Area. Most of
the campus lies within approximately one mile of the Hayward Fault, which is capable of producing events of M7 or even larger. In the next 30-50 years the fault is recognized to present the single largest threat of strong shaking in the San Francisco Bay Area. Because of the proximity of the fault to the campus, and with a large inventory of old and seismically vulnerable buildings, several years ago the University of California, Berkeley, embarked on a major building seismic upgrade and replacement program.

Amongst a large number of buildings that have already been upgraded or replaced on the UC Berkeley campus, two — one retrofit and one new construction — have already utilized buckling-restrained Unbonded Braces. Stanley Hall, a five-story reinforced-concrete laboratory building constructed at the beginning of the 1950s, was rated as “poor” in seismic vulnerability assessments, and will soon be replaced by a new state-of-the-art laboratory facility with enhanced seismic resistance. Buckling-restrained braces have been selected as the seismic later force-resisting system for the new building.

In Japan, testing of Unbonded Braces and other types of buckling-restrained braces has generally involved testing in a subassembly configuration to induce axial as well as frame rotational deformations on the braces (Fujimoto et al., 1988; Konami et al., 1999; Hasegawa et al., 1999; Iwata et al., 2000). In the U.S., however, testing of Unbonded Braces, the only type of buckling-restrained brace so far utilized, has involved only uniaxial testing of braces (Black et al., 2002).

Good ductility and hysteretic characteristics of Unbonded Braces had already been demonstrated in uniaxial tests of braces performed for a number of different projects (Black et al., 2002, Ko et al., 2002). Considering both the growing use of the braces in projects at UC Berkeley, and specifically also the significance of the new Stanley Hall project, it was decided that a subassembly testing program to confirm the behavior of Unbonded Braces under frame loading conditions was appropriate. In particular, interest focused on the behavior of the braces under frame-induced axial and rotational deformations, the appropriateness of assuming brace performance as determined from uniaxial tests for frame conditions, and also the behavior of connections under frame lateral deformations.

Although buckling-restrained braced frames as a seismic lateral force-resisting system are not yet defined by any building code in the U.S., significant work has taken place in the development of code-type provisions. Analytical studies, as well as designs developed for specific projects, have improved the understanding of this type of braced framing system and facilitated the development of design provisions (Sabelli et al., 2001a, 2001b). In parallel with the growth of use of the system, a joint Structural Engineers Association of California and American Institute of Steel Construction (SEAOC/AISC) Task Group undertook a two-year effort to develop design provisions for buckling-restrained braced frames (Sabelli and Aiken, 2003). The development work has produced a set of draft design provisions, called the Recommended Buckling-Restrained Braced Frame Provisions (hereafter referred to as the Recommended Provisions). The Recommended Provisions have been released by SEAOC (SEAOC, 2001), and are in the process of being incorporated into the 2003 edition of the NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (FEMA, 2001). It is also expected that similar provisions will ultimately be incorporated into a future edition of the AISC Seismic Provisions for Structural Steel Buildings (AISC, 1997).

The Recommended Provisions are based on the use of buckling-restrained brace designs that are qualified by testing, and are intended to ensure that braces are used only within their proven range of deformation capacity and that yield and fracture modes other than stable brace yielding are precluded at the maximum inelastic drifts corresponding to the design earthquake. The Recommended Provisions do not mandate project-specific testing, rather prior testing of appropriately similar elements may be used to ‘qualify’ a brace design or concept. The Recommended Provisions require 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.

2. Stanley Hall Replacement Building

2.1 Project Description

The new Stanley Hall on the University of California, Berkeley, campus is a 290,000 sq. ft. laboratory building for the Department of Bioengineering that is to replace 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., 2002).

2.2 Seismic Performance Goals

The University of California, Berkeley, has an established baseline design performance requirement of Life Safety in a design event with a probability of exceedance 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, which aims to achieve 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, 2002).
The new Stanley Hall was designed to meet the requirements of the 1997 Uniform Building Code (ICBO, 1997). A more detailed description of the design has been given by Lopez et al, 2002, 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 Na = 1.5 and Nv = 2.0. A value of 1.0 was used for the Redundancy Factor, r. The design based shear was approximately 0.114W, and the building story drift 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, 2001).

3. Testing Program

A number of reasons that have already been outlined led to the decision to perform braced frame subassembly tests to confirm the design for the project. The subassembly tests were intended to address three main questions (Lopez, 2002):

- 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 Unbonded Brace designs, and the loading protocols used. A more complete description of the test set-ups maybe found in Uriz and Mahin, 2003.

3.1 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 Unbonded Brace designs. The three configurations are referred to as Test No. 1, Test No. 2 and Test No. 3 (Figures 1 and 2). 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 1,500-kip 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.

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 20 ft., a story height of 10 ft. 10-1/2 inches, and comprised grade A572/50 W14 x 176 columns and a W21 x 93 beam. The upper level “hat bracing” comprised W10 x 112 members designed to remain elastic for all expected loading conditions. The beam-column connections were designed to be moment resisting, satisfying the FEMA requirements for a WUF-W pre-qualified detail (with the exception of the weld access hole requirements), FEMA 2000. The same basic beam-column frame subassembly was used for all three tests, varying only in the details of 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 assembly. 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 1-1/4 inch diameter A490 bolts in standard size holes, direct tension indicator (DTI) washers and Class A faying surfaces. The connections were designed for expected brace forces, including strain-hardening and compression overstrength contributions. From results for similar size braces tested in a previous testing program for a hospital project (Black et al., 2002), overstrength factors of 1.65 for braces CP-1A/B in Test No. 1 and 1.50 for braces CP-2 and CP-3 in Test Nos. 2 and 3 were used in the design of the connections.

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

3.2 Unbonded Brace Designs

Three different Unbonded Brace designs were investigated in the test program. The area of the brace yielding core plate was 6.33 in.² for both Test Nos. 1 and 2, and 11.69 in.² for Test No. 3 (see Table 1). Japanese Industrial Standard (JIS) grade SN400B steel was used for the core plate for all braces, with a mill certificate yield stress of 40.9 ksi. The yield forces were approximately 259 kips for braces CP-1A/B and CP-2, and 478 kips for brace CP-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. The Unbonded Brace of Test No. 3 had a cruciform core plate cross-section, so orientation was not important for this brace. The core plate orientations for each of the brace specimens is indicated in Table 1.

### 3.3 Loading Protocols

The loading histories used for all three tests basically followed the requirements defined in the *Recommended Provisions*. The testing requirements of the *Recommended Provisions* embody two main goals: to subject the tested brace to a maximum deformation at least equal to the design deformation, and to subject the tested brace to a cumulative plastic ductility demand equal to or greater than 140. The *Recommended Provisions* define a maximum displacement of 1.5 times the design story drift as determined from equivalent static analysis, but the design story drift need not be amplified if nonlinear analysis methods are used (which was the case for the Stanley Hall design). For the test program, however, maximum drift ratios defined were approximately equal to those determined from the building design analyses for the 10 percent in 100-year demands. Figure 4 shows the entire 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. The loading protocols for all three tests are summarized in Table 2.

### 4. Test Results

The following sections provide summaries of the main results and observations for each of the three tests. A more comprehensive description of the test results may be found in Uriz and Mahin, 2003.

Since it was not feasible to include a loadcell in series with the braces in any of the test set-ups, frame strain gauge information was used to determine the brace forces. Strain gauges on the columns, located at sections expected to remain elastic, were used to calculate column moments, frame shears and then brace forces based on geometry. However, in the larger deformation cycles the column webs yielded in shear,
and the elastic plane sections assumption for determining column forces was no longer exact. Therefore, the brace forces shown in the following sections, while accurate at the lower deformation levels, are only estimates of the actual brace forces in the larger deformation cycles.

4.1 Test No. 1

The frame subassembly showed good behavior overall through the entire sequence of cycles in this test (Figure 5). The Unbonded Braces exhibited stable and repeatable hysteretic behavior (Figure 6), up to a maximum strain of 2.16 percent (ductility of approximately 15) in the $1.5\Delta_{bm}$ cycles. The total cumulative plastic ductility (CPD) sustained by the braces was approximately 326, or more than two times that required by the Recommended Provisions. Maximum brace response quantities are given in Table 3, while Table 4 gives maximum and minimum frame lateral forces. The frame peak forces are extremely consistent, implying similarly consistent behavior of the two braces. Yielding of the frame became apparent at the $0.5\Delta_{bm}$ amplitude, in the columns near the top of the column base stiffeners, and in the column stiffeners themselves. At the $1.5\Delta_{bm}$ amplitude 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 the mid-beam exhibited very little up-down movement as a result of differential brace forces (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. Figure 11 shows the rotations that occurred at the ends of the North brace throughout the test, as a function of story drift. The rotations at the bottom of the brace were approximately equal to the story rotation, while the rotations at the top of the brace were approximately half of the story rotation, reflecting the effect of beam flexibility on the top gusset plate connection region.
4.2 Test No. 2

At the conclusion of Test No. 1 the braces and gusset plates were removed, new gusset plates were welded in place in the South top and North bottom frame locations, and brace CP-2 was installed.

The frame subassembly showed good behavior overall through the entire sequence of cycles in this test (Figure 12). The Unbonded Brace exhibited stable and repeatable hysteretic behavior (Figure 13), up to a maximum strain of 1.91 percent (ductility of approximately 14) in the $1.5D_{bm}$ cycles. The total CPD sustained by the braces was approximately 299, or more than two times the requirement of the Recommended Provisions. Maximum brace response quantities are given in Table 3, while Table 4 gives maximum and minimum frame lateral forces. 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 the test.

Initial yielding in the frame was observed in the top gusset plate and in both column base stiffeners at the $0.5D_{bm}$ amplitude. In the $1.0D_{bm}$ cycles yield lines in the column webs became apparent. Yielding starting at the column bases and spread up the height of the columns. In the last two $1.0D_{bm}$ cycles weld cracks began forming at the bottom free edge of the top gusset plate, adjacent to the South column.

During the $1.5D_{bm}$ cycles the top gusset plate cracks continued to propagate. When the brace was in tension (lateral
frame deformation to the South), there was noticeable buckling in the top gusset plate, due to the frame action ‘pinching’ of the gusset between the beam and the column (Figure 14).

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. The maximum vertical displacement of the beam mid-span was approximately 0.17 inch.

At the conclusion of Test No. 2 the brace and gusset plates were removed, new gusset plates were welded in place in the South top and North bottom frame locations, and brace CP-3 was installed. In response to the gusset plate buckling and weld cracking that occurred in Test No. 2, a small stiffener plate was added to the bottom edge of the top gusset plate at the face of the South column, to protect the weld in this location.

In the 1.0 $\Delta_{bm}$ cycles of Test No. 3 several damage conditions developed that indicated this deformation to be a limit state for the frame. The beam column connection at the North column developed a transverse crack adjacent to the bottom flange weld across the entire width of the flange (Figure 15). This crack continued to develop for the remainder of the test. In the first excursion at the 1.5 $\Delta_{bm}$ amplitude, at approximately 2.2 inches story drift, 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 16). Because of the beam fractures, only one cycle at the 1.5 $\Delta_{bm}$ amplitude was completed, followed by two additional cycles at 1.0 $\Delta_{bm}$.

In spite of the developing beam-column connection fracture, the frame showed good overall hysteretic behavior up to the completion of the 1.0 $\Delta_{bm}$ cycles (Figure 17). The Unbonded Braces showed good behavior up to the point of the beam flange fracture, and beyond that point accommodated the very large rotations that developed, but with subsequent hysteretic behavior similar to a conventional brace (Figure 18). The maximum brace strain demand in the test was 1.89 percent, and up to the point of beam flange fracture, the brace sustained a CPD of 219. Maximum brace response quantities are given in Table 3, while Table 4 gives maximum and minimum frame lateral forces. No slip occurred in the brace bolted connections at any stage in the test.

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.

### 4.4 Analytical Assessment

Prior to testing, nonlinear models of the test frame subassemblies were developed using OpenSees (PEER), 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 are 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.

5. 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.

5.1 Unbonded Braces

The Unbonded Braces performed very well in all three tests. Their hysteretic and elongational 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
5.2 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 North beam-column connection in the design story drift (1.0/D_{bm}) cycles. In the first 1.5/D_{bm} cycle, 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 failure of the beam at the brace gusset has been tentatively 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 are believed to be major contributors to the beam flange failure in Test No. 3.

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 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.
- 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.

6. Acknowledgements

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7. References


