Structural Design and Experimental Verification of a Buckling-Restrained Braced Frame System

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Combining numerous science disciplines under one roof, the University of California at Berkeley's new Stanley Hall will serve as an archetype for scientific research, teaching, and engineering innovations. The new building (see Figure 1) will replace the existing Stanley Hall and house the Department of Bioengineering, part of the Department of Molecular and Cell Biology, and particular research veins of the departments of Chemistry and Physics. The 270,000 sq. ft facility will provide state-of-the-art laboratories and technology for the study of chemistry, microbiology, bioengineering and nanophysics at a cost of approximately $150 million. Construction is planned to be completed by January 2006.

Built into a sloping site with several setbacks, as shown in Figure 2, the building includes seven levels of steel frame construction above grade, three levels of concrete frame construction below grade, and a large skylit atrium on the entry axis. A steel-framed superstructure was selected for economy and to reduce the mass of the building, which will be located approximately 700 ft from the highly active Hayward fault. Along the east side, and portions of the north and south sides, the steel-framed superstructure is isolated from the sloping hillside by the use of a permanent soil-nailed earth retention system.

To satisfy the vibration criteria for bench microscopes, vibrational velocity induced by a moderate walker was limited to less than or equal to 2,000 µ-in. per second. Laboratory floors are comprised of 4½" normal weight concrete fill over 2" steel deck, supported by 24". W shapes spaced at 7' on center and spanning 28'-9". In the orthogonal direction, columns are spaced at 21'. The typical floor-to-floor height is 15'-6", except at Basement Level Two which has a floor-to-floor height of 18'-6".

SEISMIC PERFORMANCE OBJECTIVES

For seismic performance, the University of California at Berkeley requires verification of baseline performance of "Life Safety" for seismic shaking intensity with a 10 percent chance of being exceeded in 50 years (10 percent/50-
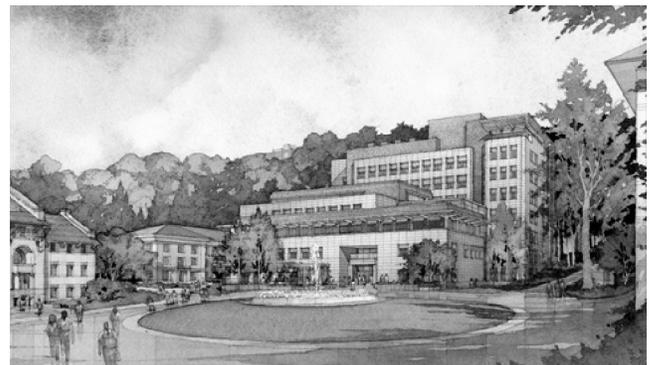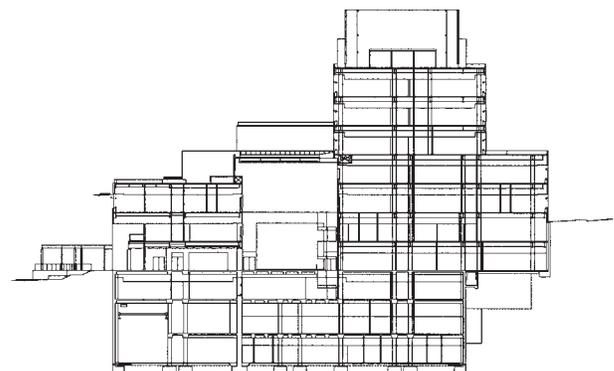

Fig. 1. View from Mining Circle.

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Fig. 2. Building Cross-Section.

year event) and "Collapse Prevention" for a 10 percent/100-year event. Beyond the baseline, the university desires enhanced performance which would allow the building to be reoccupied within, at most, a few weeks after a 10 percent/50-year event. This approach is commonly referred to as a performance goal of "Immediate Occupancy" in a 10 percent/50-year event. Design alternatives were evaluated to meet this objective, considering their cost efficiency in reducing potential damage as compared to the baseline design.

LATERAL SYSTEM SELECTION

The Buckling Restrained Braced Frame (BRBF) system was selected as the lateral system for this project because it combines a high degree of ductility (energy absorbing capability), and good initial lateral stiffness, with relative ease of repair after a major earthquake. Although a conventional Eccentrically Braced Frame (EBF) system could provide similar performance characteristics, post-earthquake repair of EBF systems, in most commonly used configurations, would be highly problematic. Properly configured concrete shear walls could also provide the desired performance although their post-earthquake repair is thought to be more disruptive. Further, the use of shear walls was considered to pose architectural constraints to future flexibility, and would create significant construction scheduling issues with a steel-framed superstructure.

The distribution and configuration of braced frames was carefully coordinated with the laboratory planning to minimize both the impact to future flexibility, and torsion on the building during a seismic event. This led to the use of both single diagonal and inverted-V braced frames. Examples of both are shown in Figures 4 and 5. In Special Concentric Braced Frames, due to requirements of the Uniform Building Code, configurations using single diagonal braces are often not feasible, and the use of stacked inverted-V braces increases the cost of the intersected beam and its end connections. In BRBFs, however, such additional requirements do not apply because overall Euler buckling of the brace is precluded. As a result, varied configurations such as those shown in Figures 4 and 5 are possible. The steel BRBFs above grade transfer lateral forces to concrete shear walls below grade, either directly or through the diaphragm at Level One.
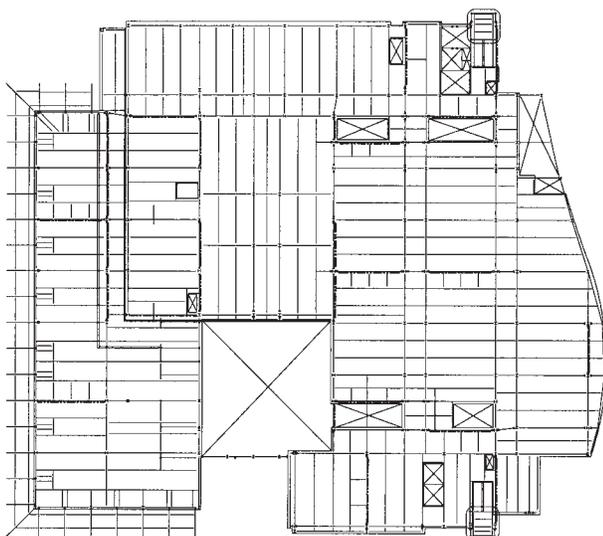

Fig. 4. Brace Configurations.

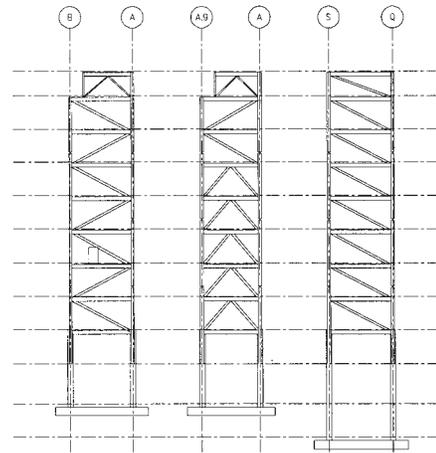

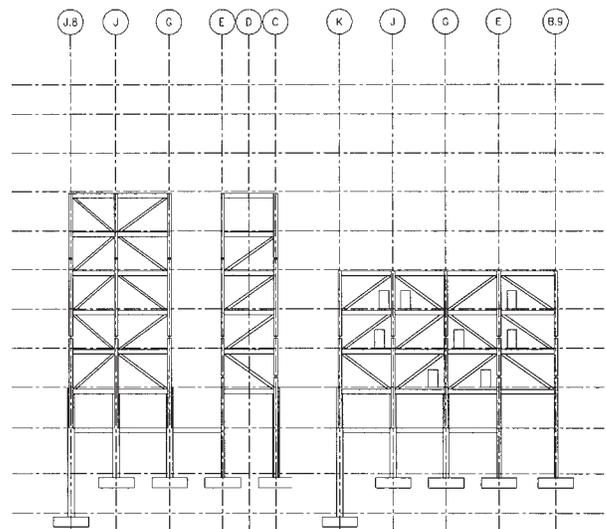Fig. 3. Framing Plan at 4th Floor.

Fig. 5. Brace Configurations.

LATERAL SYSTEM DESIGN

The BRBF system was designed using the following material specifications:

W sections:
ASTM A992 (F_y = 50 ksi)

Gusset Plates:
ASTM A572 Grade 50 (F_y = 50 ksi)

High Strength Bolts:
ASTM A490 slip-critical

Weld Electrodes:
E70XX (notch toughness: 20 ft-lb at −20 °F)

Buckling-Restrained Brace:
Steel Core: Japanese Industrial Standard G3136 SN400B with supplemental yield requirement: F_y = 43 ksi (+4 ksi, −0 ksi).

Steel Casing:
Japanese Industrial Standard G 3466 STKR 400 (F_y = 46 ksi)

The BRBF system was designed to resist the seismic forces prescribed by the 1997 *Uniform Building Code* (ICBO, 1997) for Seismic Zone 4, Soil Profile Type S_B, R = 8, Near Source Factors N_a = 1.5 and N_v = 2.0, and I = 1.0. The building period is approximately 1.4 seconds in the north-south direction and 1.2 seconds in the east-west direction. The design base shear is about 11.4 percent of the building weight. The redundancy factor, ρ, was set to 1.0, and the building interstory drift ratio was limited to 2 percent. The 1997 UBC spectrum was established as the baseline to meet code requirements.

Three-dimensional spectral analyses, including P-Δ effects, were performed to capture the influence of the dynamic characteristics, in other words, torsional effects of the upper levels of the building, in the distribution of story forces. The spectral analyses included the above-grade steel superstructure, as well as the below-grade concrete walls, with the base of the dynamic oscillator set at exterior grade level. Beams and braces were modeled with pins at their ends to conservatively determine the brace sizes required to meet code strength and drift criteria. To reduce the amount of time to perform each analysis, the number of degrees of freedom was simplified by assuming a rigid diaphragm at each floor level. Modal combinations were considered using the CQC method, and orthogonal column forces were combined using the SRSS method. Brace, beam, and column sizes were determined on an element-by-element process based on the governing forces. After several iterations, a final design that met both code strength and drift requirements was chosen.

Once a code-compliant design was developed, it was necessary to verify that such a design met the university's

seismic performance objectives. Nonlinear static (pushover) analyses were performed. As the beam-to-column and brace-to-gusset-plate connections had been finalized, pushover models incorporated both beam-column fixity and brace-end rigidity created by the gusset plates.

A "backbone curve" bounding the strain hardening characteristics of the braces was generated from experimental data of previous uniaxial tests (Seismic Isolation Engineering, 2001) and was used to define the nonlinear axial hinge properties assigned to the braces. A bilinear axial hinge element with kinematic strain hardening as shown on Figure 6 was defined for each brace size in the building. Each element had its own unique yield strength in kips, which was identical both in tension and compression. To replicate observed experimental behavior, the post-yield stiffness, as a measure of the initial stiffness, was defined as 3.4 percent in tension, and 4.0 percent in compression. To accurately capture the strain hardening in the braces, different hinge elements were assigned to braces with different yield lengths even if the brace size was the same. The maximum brace strain allowed in the hinge element was the maximum strain observed experimentally.

Nonlinear axial-flexural hinge elements were assigned to frame beam and columns at the locations where hinges were expected to form. The nonlinear hinge properties were based on the recommendations of FEMA 356 (FEMA, 2000b). Pushover analyses performed prior to the three tests used beam and column elements that did not incorporate nonlinear shear hinges. After conducting the tests and learning that shear-controlled behavior can occur in braced frames, hand calculations on the most shear-sensitive beam and column configurations were performed. Flexure-controlled behavior was confirmed on the worst configurations. Therefore, pushover analyses already performed were deemed to adequately represent expected behavior and were not rerun.
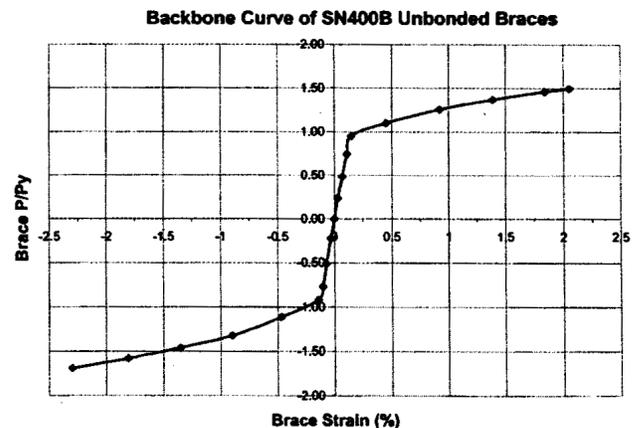

Fig. 6. Backbone Curve/Axial Hinge Element.

The capacity curves from the pushover analyses were superimposed with the demand curves, as shown in Figure 7. The 1997 UBC spectrum (10 percent/50-years) was used as a baseline for the demand curves. Initial story forces were based on the baseline spectrum. Story forces based on constant acceleration were also applied to check for weak stories. Other spectra used were the University of California site-specific spectra for the 10 percent/50-year, and 10 percent/100-year events. Each spectrum was reduced for hysteretic damping based on Miranda's R-μ-T relationship (Miranda and Bertero, 1994). Target displacements for the 10 percent/50-year and 10 percent/100-year events were defined at the intersection of the demand-capacity curves.

The BRBF system design was based on the American Institute of Steel Construction/Structural Engineers Association of California (AISC/SEAOC) committee's *Recommended Provisions for Buckling-Restrained Braced Frames* dated October 2001 (*Recommended Provisions*) and followed a two-step approach. First, brace sizes were determined from elastic spectral analysis results. Second, brace overstrengths were determined using pushover analyses results. After performing pushover analyses to the appropriate target displacements, brace forces were extracted. Brace overstrength factors, defined as extracted brace forces divided by initial yield forces, were calculated to range from 1.5 to 1.75. Brace yield lengths were estimated in a manner similar to that shown in Figure 8. The initial analysis assumptions were that, for a single-diagonal frame, the yield length was two-thirds of the work point length, and for an inverted-V configuration, it was half the work point length. Upon completion of the design, a check was made of the yield length assumptions. After determining

reasonable brace and connection demands, connection dimensions were estimated, and estimates of yield lengths were obtained from the brace manufacturer. A comparison against the yield lengths obtained from the manufacturer indicated that the initial assumptions were within acceptable ranges of deviation.

Based on the loads generated in the braces at the appropriate target displacements, load capacities of beams, columns, and connections were checked. Brace connections were primarily designed by the Uniform Force Method and utilized $1\frac{1}{8}$-in.-diameter or $1\frac{1}{4}$-in.-diameter ASTM A490 slip-critical bolts. Superstructure-to-substructure connections were also designed for the maximum load developed in the braces. Collector design followed a code prescriptive approach utilizing a Seismic Force Amplification Factor, Ω_o, of 2.8. Based on the deformation demands at the appropriate target displacements, deformation capacities of beams, columns, and connections were checked. All seismic performance objectives established by the owner were met.

REASONS FOR TESTING

The BRBF system is not yet specifically incorporated into building codes in the United States. A joint AISC/SEAOC committee has prepared *Recommended Provisions* (Sabelli, 2004) that require designs to be based on the results of qualifying tests. It is recommended that the qualifying tests consist of, at a minimum, one uniaxial brace test and one subassemblage test. Both tests have to be successful cyclic tests. The test results need not be project-specific, and may include those reported in research literature and those performed for other projects. For the tests to satisfy the *Recom-*
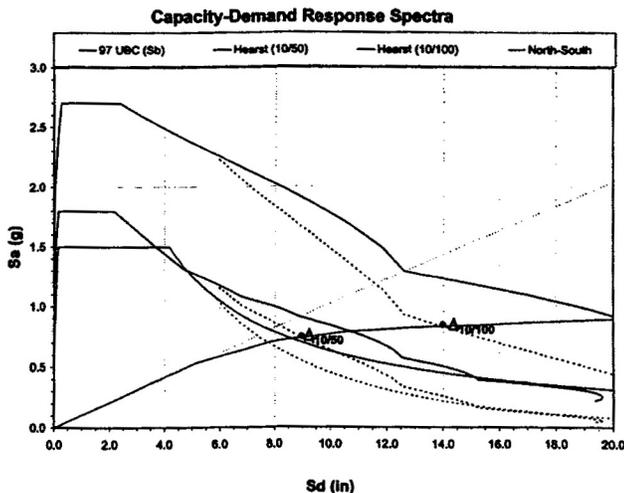

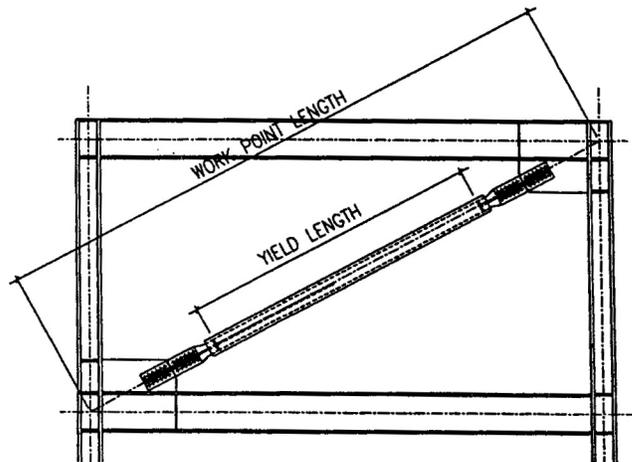Fig. 7. Demand-Capacity Response Spectra.

Fig. 8. Brace Yield Length.

mended Provisions, they must reasonably match project conditions, and they must be accepted by the agency having jurisdiction over the project. For projects at the University of California at Berkeley, the tests needed to satisfy the independent Project Structural Peer Reviewer, the university's Seismic Consultant, and the university's Seismic Review Committee.

It was determined that, while sufficient uniaxial test results existed to satisfy the brace test requirements for Nippon Steel Corporation's Unbonded Braces, at least one project-specific subassemblage test should be performed for each configuration of the braces, in other words, the single diagonal configuration and the inverted-V configuration, to comply with the *Recommended Provisions*.

The purpose of the testing was to answer several specific questions:

1. How will kinematic end rotations affect the brace performance?

2. Does the design methodology used to proportion frame beams, frame columns, and connections, produce adequate results?

3. Will the hysteretic behavior of the brace tested uniaxially be duplicated once within a frame?

4. How will brace connections perform at the expected building drifts?

DESCRIPTION OF TESTING

The *Recommended Provisions* require that the braces be subjected to concurrent axial and flexural strains and recognize that there are several ways of achieving that objective. The simplest way of effecting concurrent axial and flexural strains on the brace is to hold one brace end stationary while subjecting the other end to concurrent displacements in both the longitudinal and transverse axes of the brace. Two buckling restrained brace manufacturers, in their project-independent validation tests, have used such an approach. This was one alternative considered for Stanley Hall.

It was also considered that, if a subassemblage, reflecting the conditions of the actual building as closely as practical, were built and tested, much more could be learned about the performance of the system. Realizing the uniqueness of the opportunity, it was decided to place representative braces within a test frame and subject the frame to increasing levels of interstory drift.

The material specifications for the test frame were identical to those specified for the actual building project. ASTM A992 W sections used are shown in Figure 9. Coupon tests of the W sections were not taken. Mill test

reports indicate that their mechanical properties are $F_y = 50$ ksi and $F_u = 73$ ksi. Unbonded braces were fabricated of Japanese Industrial Standard G3136 SN400B plate stock with the following mechanical properties: $F_y = 40.9$ ksi and $F_u = 62.2$ ksi from mill test reports and $F_y = 41.2$ ksi and $F_u = 61.4$ from coupon tests. Coupon tests of the gusset plate material were not taken. The plate's mechanical properties from a mill test report are $F_y = 52.5$ ksi and $F_u = 70$ ksi.

A total of three tests were planned. To reduce the cost of the testing program, one test frame was designed to be used in all three tests as shown in Figure 9. The test frame was erected in a vertical position, as in normal construction, and then rotated to a horizontal position to have the gusset plates field welded and the W10 braces and unbonded braces attached. The tests were conducted with the frame in a horizontal position about 2'-6" above the laboratory floor. On Figures 9, 10, and 11, North and South are shown as gridline bubbles. The first test was designed to emulate the behavior of braces in an inverted-V configuration. See Figure 10. The second and third tests were designed to emulate the behavior of braces in a single diagonal configuration. See Figure 11. A total of four different braces were utilized in the three tests. Test 1 utilized two braces with initial yield strengths of 261 kips, core areas of 6.33 in.^{2}, and yield lengths of 76 in. Test 2 utilized one brace with an initial yield strength of 261 kips, core area of 6.33 in.^{2}, and a yield length of 144 in. Test 3 utilized one brace with an initial yield strength of 482 kips, core area of 11.69 in.^{2}, and a yield length of 136 in.
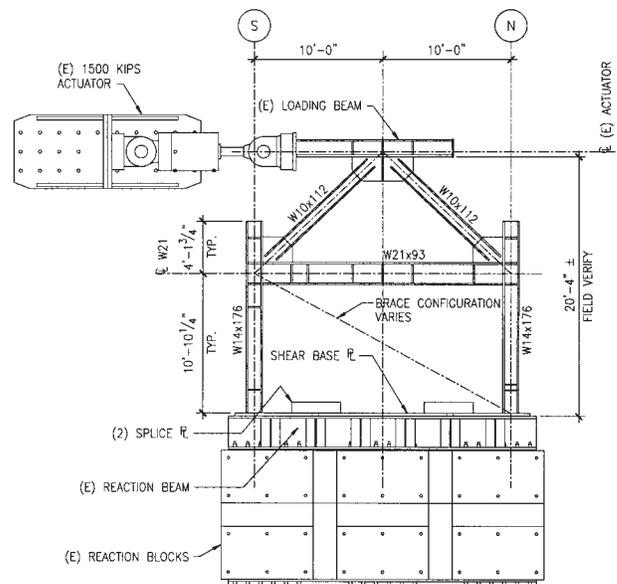

Fig. 9. Test Frame Setup.

The tests were conducted at the University of California at Berkeley. To reduce costs, the test setup for a previous testing program on steel plate shear walls was used. To accommodate the existing test setup, the test frame was scaled down to approximately 70 percent of the story height and bay width of the actual building. The test frame was designed as described below:

- Its configuration was chosen so that it would accurately model brace angles and boundary conditions in the actual building.

- The axial design strength of beam and columns exceeds the predicted required strength at the maximum proposed interstory drift of Test 3.

- The design strength of the W10 braces and connections exceeds the predicted required strength at the maximum proposed interstory drift of Test 3.

- The deformation capacity of its members and beam-to-column connections exceeds the predicted required deformation demands at the maximum proposed interstory drift of all tests.

- Nonlinear static analyses were performed to check load and deformation demands versus capacities.

DISPLACEMENT HISTORY

Concurrent axial and flexural strains were produced on the braces by subjecting the test frame surrounding the braces to increasing levels of interstory drift. Test 1 was subjected to a maximum interstory drift ratio of 2 percent. Tests 2 and 3 were subjected to a maximum interstory drift ratio of 2.5 percent. Following the requirements of the *Recommended Provisions,* the maximum interstory drift ratio to be applied in all tests need not exceed 1.3 percent. A 1.3 percent interstory drift ratio corresponds to the maximum demand of a 10 percent/50-year event determined by pushover analyses. However, given the unique opportunity to perform a frame test, it was decided to impose on the frame the maximum interstory drift ratios allowed by the 1997 UBC; namely, 2 percent and 2.5 percent. An interstory drift ratio of 2 percent for Test 1 was predicted to induce in the braces a ductility demand of about 19, and a cumulative plastic ductility demand in excess of 400. Given that two other tests had to be performed, and that the braces were subjected to ductility demands in excess of the *Recommended Provisions* requirements, the 2 percent limit chosen for Test 1, to protect the integrity of the test frame, seemed appropriate. For Tests 2 and 3 the 2.5 percent maximum interstory drift ratio was predicted to induce in the braces ductility demands in excess of 12 and cumulative plastic ductility demands in excess of 200.
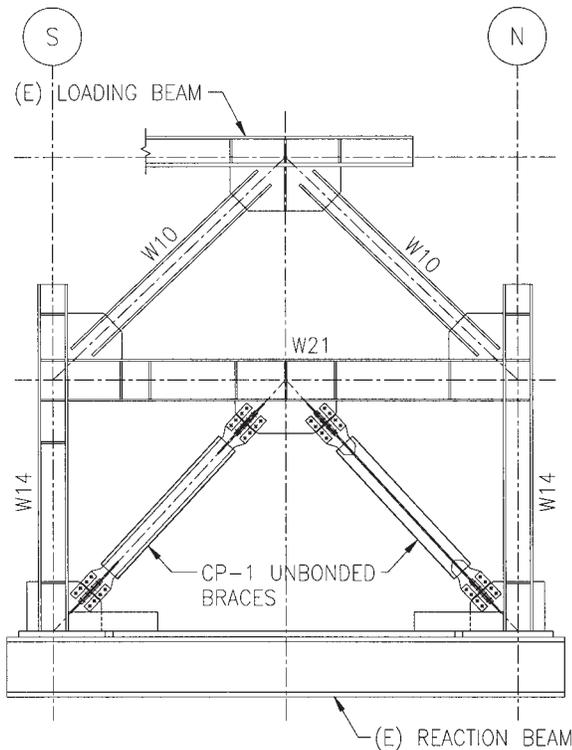

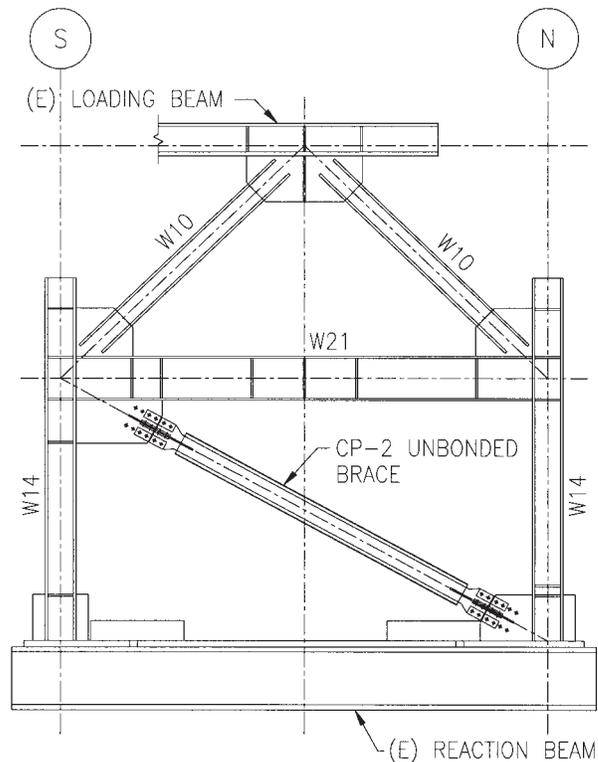*Fig. 10. Partial Elevation. Setup for Test 1.*

Fig. 11. Partial Elevation. Setup for Tests 2 & 3.

Theta (θ) is defined as the interstory drift ratio of the first story of the test frame. This definition of θ is similar to that found on Figure 2-2 of FEMA 350, (FEMA, 2000a). Tests were conducted by controlling the level of interstory drift imposed on the first story of the test frame. The value of significant interstory yield, θ_y, refers to the interstory drift required to cause yielding of the buckling restrained brace. The value of θ_y varied with brace yield length and was calculated based on the fabrication drawings submitted by the brace manufacturer.

Loads were applied to the loading beam to produce the following interstory drifts (average values of the 3 tests):

6 cycles of loading at θ_y
4 cycles of loading at $2.5\theta_y$
4 cycles of loading at $5\theta_y$
2 cycles of loading at $7.5\theta_y$

Because of the different brace configurations, Test #1 utilized a different displacement protocol from that used in Tests 2 and 3.

SUMMARY OF EXPERIMENTAL RESULTS

During Test 1 neither the braces nor the test frame were loaded to their ultimate strengths because of the need to perform the other two tests. The unbonded braces and the test frame performed adequately up to the maximum interstory drift. The manufactured steel core projections were adequate to accommodate the imposed brace deformations. The behavior of the frame beam closely resembled the behavior predicted analytically. Starting with the first six cycles of loading, yielding of some frame elements was observed. Columns exhibited some flexural yielding and noticeable distributed shear yielding. The frame beam exhibited some flexural yielding at the bottom of the north beam-to-column connection.

During Test 2 neither the brace nor the test frame was loaded to its ultimate strength. Both the unbonded brace and the test frame performed adequately up to the maximum interstory drift. As in Test 1, the behavior of the brace was both dependable and acceptable. The steel core projection showed no deleterious effects from combined flexural and axial strains. The columns exhibited some flexural and noticeable distributed shear yielding. The gusset plates yielded noticeably at the column-to-gusset connections. Minor beam flexural yielding was observed near the edge of the beam-to-gusset connection. When the brace was in tension, at an interstory drift ratio of about 2.5 percent, the beam-to-column angle was reduced to less than 90°. This closing-in of the beam-column joint caused the free edge of the gusset plates to buckle elastically. (See Figure 12.) Buckling of the free edge of the gusset plates was observed to contribute to crack initiation at the gusset-to-column welds. With continued loading, cracks propagated from welds to gusset plates and damaged the plates.

Prior to commencing Test 3, the damaged gusset plates were removed from the frame and new gusset plates were installed. The new, replacement gusset plate was ¾ in. thick, as was the one for Test 2. To reduce stresses at the gusset-to-column welds, small transverse stiffener plates were added. (See Figure 13.)

During Test 3 the test frame was subjected to increasing cyclic displacements to a maximum of 2.25 percent interstory drift. Initially, it was planned to test to 2.5 percent interstory drift. However, the limit state of cumulative dam-

Fig. 12. Buckling of Free Edge of Gusset Plate.

Fig. 13. Gusset Plate Replacement for Test 3.

age of the frame beam was reached by 2.25 percent inter-story drift during the 15th cycle. It is important to note that the same test frame had previously been used for two cyclic tests and that yielding of frame elements had occurred in both of those tests. It is also important to note that the bottom flange of the beam had withstood significant heat input from welding and removal of one gusset plate, Test 2, and welding of another, Test 3. Therefore, it is reasonable to expect less than perfect frame behavior because of cumulative damage effects. The unbonded brace performed adequately up to the maximum interstory drift. The frame performed adequately during the first 14 cycles of loading and experienced bottom flange fracture of the beam and loss of strength during cycles 15 and 16. (See Figure 14.) After beam fracture, a crack initiated at the gusset-to-beam weld because of combined axial and flexural strains. This weld crack started where the beam bottom flange had fractured, and lengthened to about 4 in. by the end of the last 2 cycles.

After completing Tests 2 and 3, gusset plate design calculations were reviewed. It was found that no check for buckling of the free edge of the gusset plate had been made. A search in the 1997 UBC (ICBO, 1997), AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 1997) and the 2nd Ed. *Load and Resistance Factor Design Manual of Steel Construction*, Volumes I and II (AISC, 1998), did not produce any design requirements for free edges of gusset plates. Further literature search indicated that Astaneh-Asl (1998) had proposed a criterion to prevent cyclic buckling of the free edge of gusset plates that was never incorporated into any codes. A review of Astaneh's proposed criterion showed that the ¾ in., 50 ksi gusset plates used in Tests 2

and 3 did not meet the criterion. There are several possible explanations for the free edge of the gusset not buckling during Test 3. One is that the addition of the stiffener (Figure 13) significantly shortens the effective length of the free edge. Another is that the addition of the stiffener plate prevents the beam-to-column angle from closing much beyond 90°. A third is that the test frame was subjected to only one full cycle at about 2.25 percent interstory drift ratio.

A more detailed explanation of all three tests, including hysteretic responses and pictures of each test, is found in López, Gwie, Saunders, and Lauck (2002).

DESIGN VALIDATION

The testing program was successful in validating the design for the project-specific requirements. Specifically, the following was confirmed:

- Concurrent axial and flexural strains do not affect the energy-dissipating characteristic of the unbonded braces designed for the actual building.

- The design methodology used produced designs that perform adequately when subjected to large cyclic displacements.

- The overstrength in the braces could be predicted analytically by using a "backbone curve." The reaction values obtained experimentally closely matched those predicted analytically.

- The nonlinear behavior of the braces could be captured analytically by a bilinear axial hinge element.

- Slip critical bolts, sized using service loads at forces corresponding to the overstrength of the braces, did not slip.

- No low-cycle-fatigue failure of the braces occurred.

- Use of notch-tough weld electrodes to connect the gusset plates was effective in limiting crack propagation, once a crack in the weldment occurred, and therefore was critical to the overall performance of the system.

ADDITIONAL LESSONS LEARNED

Equally important as the validation of the design methodology for the project is the new knowledge acquired through the performance of the three tests. It is worth noting that the items described below correspond to design practices common to all steel braced frame systems. Therefore, regardless of the choice of braced frame system, it is important to consider the possible design implications of the experimental observations.

Fig. 14. Bottom Flange Beam Fracture.

- Frame elements may not necessarily be axial/flexure-controlled. Gusset plate sizes and their effect on shortening the clear dimensions of beams and columns and modifying their behavior from axial/flexure-controlled to shear-controlled should be recognized by the design engineer.

- Load-dependent design of gusset plates by the Uniform Force Method produces connections with adequate strength. However, the Uniform Force Method does not address buckling of the free edge of the gusset when the beam-to-column angle is less than 90°. Gusset plate buckling due to kinematic deformations of a beam-column joint was first noticed by Richard (1986) in analytical work featuring monotonically applied loads. Neither Richard's (1986) nor Astaneh-Asl's (1998) findings are currently incorporated into any design provisions. The structural engineering community does not know enough about gusset plate buckling caused by kinematic joint rotations to formulate design recommendations. Richard's observations were based on monotonically-applied loads that caused the free edge to buckle when the brace was in tension. Richard's work is the closest in relevance to the phenomenon witnessed in Test 2. However, analytical work, based on cyclic loads, would be even more relevant. Astaneh's proposed criterion is based on his experimental work that applied cyclic loads to a beam-brace joint such as that at midspan on Figure 10. Compliance with Astaneh's criterion would ensure that the gusset plate compressive capacity is reached before the free edge buckles. However, the limit state that requires understanding is characterized by gusset plate buckling under tensile loads, not compressive. Other researchers, for example, Brown (1988) and Yam and Cheng (1993), have studied buckling of gusset plates under compressive loads, not tensile loads. Published literature does not adequately address the behavior of a gusset plate when subjected to cyclic deformation within a beam-column joint. Further study is required to fully understand this behavior and translate the findings into design requirements. In the meantime, design engineers are encouraged to acknowledge such behavior in the detailing of brace connections.

- It was found that the strength of single-bevel, complete-joint-penetration welds between gusset plates and beam or column flanges is sensitive to buckling of the free edge of gusset plates. When the beam-to-column angle is reduced from its unloaded condition and the free edge buckles, the root of the weldment is subjected to both tensile strains from tension in the brace, and flexural strains due to deformation of the free edge. These flexural strains are perpendicular to the longitudinal axis of

the weld and their tensile value can be highest at the root of the weldment. The addition of direct tensile, and buckling-induced tensile strains, creates required strengths in excess of the design strength of the root and a weld crack is initiated. To avoid crack formation, it is recommended that if weld strength is an issue, double-sided fillets or double-sided partial joint penetration welds be specified.

CONCLUSIONS

For the conditions tested, it was confirmed that the cyclic behavior of the unbonded braces was not affected by kinematic end rotations, and the design methodology employed produced adequate results. Current design approaches for all types of concentrically braced frames can benefit from the lessons learned in these tests.

REFERENCES

American Institute of Steel Construction, Inc. (AISC) (1997), *Seismic Provisions for Structural Steel Buildings,* AISC, Chicago.

AISC (1998), *Load and Resistance Factor Design Manual of Steel Construction*, Volumes I and II, 2nd Ed., AISC, Chicago.

Astaneh-Asl, A. (1998), "Seismic Behavior and Design of Gusset Plates," *Steel TIPS*, Structural Steel Educational Council, Moraga, CA.

Brown, V.L. (1988), "Stability of Gusseted Connections in Steel Structures," Ph.D. Dissertation, Department of Civil Engineering, University of Delaware, Newark, DE.

Federal Emergency Management Agency (FEMA) (2000a), *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*, FEMA 350, Washington, D.C.

Federal Emergency Management Agency (FEMA) (2000b), *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*, FEMA 356, Washington, D.C.

International Conference of Building Officials (ICBO) (1997), *Uniform Building Code*, ICBO, Whittier, CA

López, W.A., Gwie, D.S., Saunders, C.M., and Lauck, T.W. (2002), "Lessons Learned From Large-Scale Tests of Unbonded Braced Frame Subassemblages," *Proceedings of the 71st Annual Convention*, pp. 171-183, Structural Engineers Association of California, Sacramento.

Miranda, E. and Bertero, V.V. (1994), "Evaluation of Strength Reduction Factors for Earthquake Resistant Design," *Earthquake Spectra*, Vol. 10, No. 2, pp. 357-379, Earthquake Engineering Research Institute, Oakland, CA.

Richard, R.M. (1986), "Analysis of Large Bracing Connection Designs for Heavy Construction," *National Steel Construction Conference Proceedings,* pp. 31.1-31.24, AISC, Chicago.

Sabelli, R. (2004), "Recommended Provisions for Buckling Restrained Braced Frames," *Engineering Journal*, pp. 155-175, 4th Quarther, AISC, Chicago.

Seismic Isolation Engineering for Nippon Steel Corporation (2001), "Cyclic Tests of Nippon Steel Corporation Unbonded Braces," A report to Ove Arup & Partners California, Ltd. and Office of Statewide Health Planning and Development.

Yam, M.C.H, and Cheng, J.J.R. (1993), *Experimental Investigation of the Compressive Behavior of Gusset Plate Connections*, Structural Engineering Report No. 194, University of Alberta Department of Civil and Environmental Engineering, Alberta, Canada.