EVALUATION OF DESIGN METHODOLOGIES FOR STRUCTURES INCORPORATING STEEL UNBONDED BRACES FOR ENERGY DISSIPATION

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SUMMARY

This paper outlines design studies and large-scale tests of tension/compression yielding braces (also called “unbonded braces”) in support of their first applications in the United States. The core steel in these braces provides stable energy dissipation by yielding under reversed axial loading, while the surrounding concrete-filled steel tube resists compression buckling. A slip surface or unbonding layer separates the steel core from the surrounding tube. The first section of the paper is focused on establishing the seismic demands on axial hysteretic elements in multi-story steel structures. The mathematical modeling employed reproduces the force-displacement behavior of unbonded braces, but the results can be generalized easily to buildings with other types of hysteretic damping ele-ments.

The second portion of the paper summarizes a series of tests on large-scale unbonded braces. Three braces, having yield forces of 270, 360, and 470 kips were subjected to a cyclic loading pat-tern consistent with that used widely for testing steel beam-column connections. Additional tests explored the behavior of the braces under a near-field loading history, a displacement time history derived from a seismic analysis of an idealized 5-story building, and a low-cycle fatigue test.

INTRODUCTION

This paper proposes a design procedure for hysteretic yielding braces based on the equivalent static force method currently prescribed for eccentric braced frames (EBFs) in the Uniform Building Code. Conceptually, a structure using a lateral force-resisting system of hysteretic dampers based on plastic deformation of steel should behave similar to an eccentric braced frame. In fact, a properly-designed damped frame may prove to be more economical than an EBF, even when designed to code-minimum forces. A three-story building previously used in the Phase 2 SAC Steel Project to evaluate design procedures for steel moment-resisting frames (SMRFs) is redesigned with unbonded braces, specially-detailed steel components which can provide stable hysteretic behavior in both ten-sion and compression without buckling. A series of nonlinear analyses is then undertaken to provide comparisons of the performance of the unbonded braced frame (UBF) with the SMRF.

Results from a series of large-scale tests of unbonded braces are also presented as evidence of the stable hysteretic behavior that can be achieved with these elements. Increasing-amplitude cyclic tests, low-cycle fatigue tests, and earthquake-loading tests were performed on three different specimens. A number of cycles of displacement at rel-atively large axial yield strains were sustained in the braces prior to failure, giving designers confidence that a lat-eral force-resisting system incorporating these elements will provide at least equivalent performance to an EBF.

The first application of unbonded braces in the United States is also described.

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2. BACKGROUND OF UNBONDED BRACES

While the studies reported in this paper can be generalized to any type of yielding steel damping element, the focus here is on a class of steel braces which dissipate energy through stable tension-compression yield cycles. A variety of these “unbonded braces” having various materials and geometries have been proposed and studied extensively over the last 10-15 years. A summary of much of the early development of unbonded braces which use a steel core inside a concrete-filled steel tube is provided in [Watanabe, et al., 1988]. Since the 1995 Kobe Earthquake, these elements have been used in numerous major structures in Japan [e.g., Reina, and Normile, 1997].

The basic principle in the construction of the most popular type of unbonded brace is to prevent Euler buckling of a central steel core by encasing it over its length in a steel tube filled with concrete or mortar (Fig. 1). The term

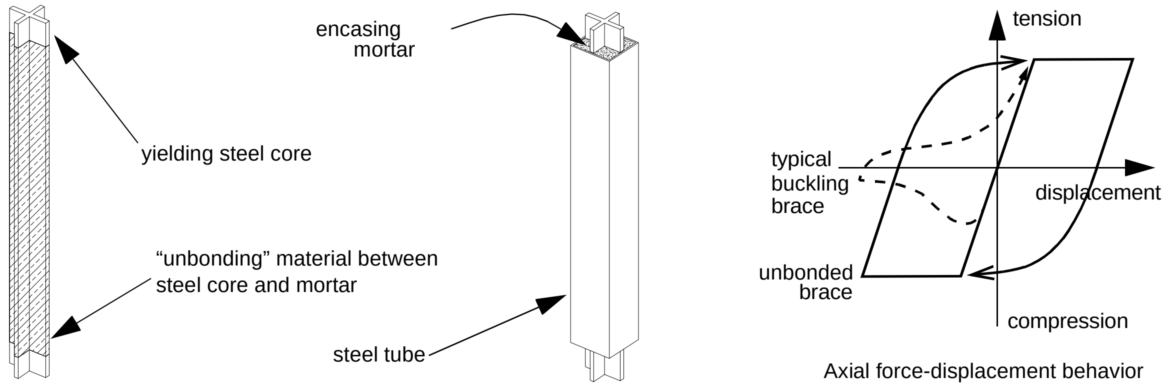


Figure 1: Schematic of Mechanism of Buckling-Resistant Unbonded Braces

“unbonded brace” derives from the need to provide a slip surface or unbonding layer between the steel core and the surrounding concrete, so that axial loads are taken only by the steel core. This materials and geometry in this slip layer must be carefully designed and constructed to allow relative movement between the steel element and the concrete due to shearing and Poisson’s effect, while simultaneously inhibiting local buckling of the steel as it yields in compression. The concrete and steel tube encasement provides sufficient flexural strength and stiffness to prevent global buckling of the brace, allowing the core to undergo fully-reversed axial yield cycles without loss of stiffness or strength. The concrete and steel tube also help to resist local buckling.

For structures designed in accordance with the life-safety philosophy of most building codes, this paper treats a frame having unbonded braces as essentially equivalent in seismic performance to an EBF. Such as system should thus require no additional design effort beyond an equivalent static analysis to determine the design brace forces. However, a useful result of the unbonded brace construction is the ability to independently control strength, stiffness, and yield displacement or ductility by varying the cross-sectional area of the steel core, the yield strength of the steel, and the length of the core which is allowed to yield. This provides designers with the opportunity to accurately tailor the force-displacement relationship of their lateral force-resisting elements according to the needs of the application, making unbonded braces useful in the context of design for performance levels other than those mandated by the code, such as for critical facilities, retrofit of existing structures, etc.

3. DESIGN PROCEDURES FOR BRACED FRAMES

The equivalent static lateral-force provisions in the 1994 Uniform Building Code [UBC, 1994] are used as the basis for the design of the unbonded braced frame (UBF) studied in this paper. Before going into the details of the design, it is useful to briefly contrast the provisions for special concentrically-braced frames (SCBFs) and EBFs. For a given building site, the design forces implied by the code requirements for SCBFs are typically 1.5 times those for EBFs, reflecting the increased likelihood for stiffness and strength deterioration of buckling braces under cyclic loading (SCBFs have a force reduction factor, R_w, of 9, compared with the R_w of 10 which is used for EBFs), as well as the presumably shorter elastic period of a concentrically braced frame (C_t is 0.020 for SCBFs, compared with 0.030 for EBFs).

It is proposed here that frames designed to incorporate unbonded braces under equivalent static lateral-force provisions consistent with the UBC should use forces compatible with those used for EBFs, rather than those used for SCBFs. This is justified because the unbonded braces do not exhibit buckling and the stiffness and strength deterioration which inevitably accompanies buckling. Their stable hysteretic behavior more closely resembles the behavior of a shear link in an EBF. Further, the unbonded braces do not need to be designed using the compression stress-reduction factor, B (ϕ_c in the AISC provisions), that takes into account the global buckling stability of the brace element. This means that the unbonded braces will have smaller steel cross-sectional areas, and therefore the structure will have a longer elastic period, comparable to that of an EBF.

4. BUILDING EXAMPLE CONSIDERED IN THIS STUDY — SAC MODEL BUILDING

The structure investigated in this analytical study is based on a three-story special moment-resisting frame (SMRF) originally developed in Phase 2 of the FEMA/SAC Steel Project [SAC, 1999]. The building is designed for Los Angeles (seismic zone 4) on UBC soil type S2, to meet the 1994 UBC provisions. Using the structural period given by Section 1628.2.2 (Method A) of the UBC, Table 1 presents the code-mandated design forces for each of three different types of structural systems. It is clear that the SMRF has an advantage in terms of design base shear, but in fact, the frames in the SMRF examined in this study were sized to meet drift requirements and therefore have a much higher yield base shear. For this structure, grade beams were used at the foundation level to achieve full fixity of the column bases. Complete details of this structure are provided in [SAC, 1999].

Table 1: Design Parameters for Three Structural Systems

Code Design Requirements	Special Moment-Resisting Frame	Special Concentrically Braced Frame	Eccentric Braced Frame
R_w	12	9	10
C_t	0.035	0.020	0.030
V_{base}	0.075W	0.145W	0.099W

5. REDESIGN OF SAC MODEL BUILDING INCORPORATING UNBONDED BRACES

This section describes the details of the three-story building when redesigned according to the equivalent static lateral-force provisions for EBFs, but configured with unbonded braces. The goal is to use the same general assumptions as were used for the SMRF and then compare the configuration and performance of the UBF with that of the SMRF. The advantages offered by designing the UBF to the forces prescribed for the EBF as compared to those for the SCBF are significant, as described above and shown in Table 1. Because drift demands can be met in a braced system more easily, there are further advantages over the moment frame, as there is no need to increase member sizes to control drift.

Since the 1994 UBC seismic load is based on working stresses, it is increased by a factor of 1.5 times to estimate the required yield lateral strength of the frame (per Div. I and VIII of Chapter 22, UBC). The required yield strengths of the unbonded braces, $P_{y,\,br}$, are obtained by conservatively ignoring the moment resistance provided by the beams and columns, thereby allowing a statically determinant truss analysis to be used for the design of the braced bay (Fig. 2). The required cross sectional area of the yielding portion of the brace, A'_{br}, is then calculated as

$$A'_{br} = \frac{P_{y,\,br}}{\phi F_{y,\,br}} \tag{1}$$

where $F_{y,\,br}$ = yield strength of the brace steel material, and
ϕ =0.9 is the strength reduction factor.

For ultimate state design of elements around the link in an EBF, beams and columns typically are sized to support the axial forces and moments generated by 1.25 times the yield strength of the link. Our preliminary design employs some conservatism in this regard by considering amplified axial forces in the braces due to strain hardening at their ultimate state to indirectly account for the unknown seismic moment. The beams and columns are then designed as beam-column elements to remain elastic even at the ultimate state. Note that for the UBF, there is no force imbalance at the connection of the braces to the midpoint of the beam, because the tension and compression forces in the two braces in each bay are essentially equal.

Based on the assumptions outlined above, the beams, columns, and cross-sectional areas of the yielding portion of the braces, A'_{br}, selected for the 3-story UBF are illustrated in Fig. 2. For the unbonded brace, Japanese steel SS400 (F_y = 2.4 tonf/cm^2 = 34 ksi) is used, and for the beams and columns A572 Grade 50 (F_y = 50ksi) is used. When compared with the SMRF using similar steel (F_y is about 50 ksi), the UBF requires much smaller steel sections. In fact, the total weight of the steel (including unbonded braces) in the UBF is only 0.51 times that of the SMRF. There are also substantially fewer rigid connections used in the UBF, so it would be expected to be less expensive than the SMRF. However, because all of the lateral force-resisting elements are concentrated in a single braced bay, for this particular building on UBC soil type S2, it is likely that pile foundations would be required to resist uplift under seismic input.

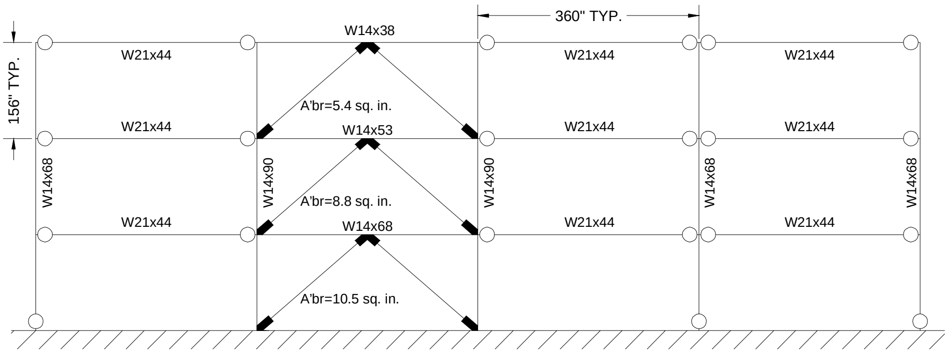


Figure 2: Three-Story Frame Redesigned with Unbonded Braces

The length of the yielding portion of the braces, L'_{br}, is assumed to be 130 inches. When the story drift angle in the present UBF configuration is 0.02 radian, δ_{br} = 2.36 in., and ε_{br} = 2.36/130=0.018. This strain demand is relatively low when compared with results from recent experimental studies in Japan that indicate that the unbonded brace can take more than 20 fully reversed cycles of ε_{br}= ±0.02. The low-cycle fatigue behavior of the braces designed for this structure should be more than adequate for the demands predicted from the analyses.

An alternative design approach for sizing the unbonded braces could be to use smaller L'_{br} as long as the increase of ε_{br} is acceptable. Such an approach will produce a higher brace stiffness K_{br} and better drift control. This demonstrates how variations in L'_{br} can be investigated to control the stiffness of the unbonded brace independent of its strength. Similarly, steels of various yield strengths can be considered, giving designers the opportunity to modify the strength of a braced frame while keeping its stiffness constant.

6. RESULTS OF NONLINEAR ANALYSES

A brief series of nonlinear analyses was performed on computer models of both the UBF and SMRF frames. The strain-hardening modulus is set to 0.5% of the elastic modulus for the unbonded braces. The beams are modeled to have trilinear characteristics; the moment is assumed to reach 1.3 times the plastic moment at a plastic rotation of 0.02 radian, and remain almost constant beyond that point. Rayleigh damping is used to create a damping ratio of 0.02 at both the first mode period and a higher mode period (0.2 times the first mode period). Gravity load effects are also considered.

The earthquake ground motions used for the time history analyses are three records which are commonly specified for design and evaluation of buildings in Japan. The records are: a) 1.495 times the 1940 El Centro North-South ground motion; b) 2.824 times the 1952 Taft East-West ground motion; and c) 1.0 times the 1995 Kobe (JMA) North-South ground motion.

Static pushover analyses were first conducted on the two framing systems, and the results of these are shown in Fig. 3. The UBF has a smaller yield strength but a larger stiffness than the SMRF. Note that the SMRF has significant overstrength relative to the code-required yield strength (Fig. 3), since its design was governed by stiffness and drift control rather than strength. As discussed in Kasai et al. (1998), better drift control is achieved with a smaller vibration period, and better acceleration (and base shear) response is achieved with a smaller lateral yield strength. This is especially true when the system ductility demand is less than about 10. Based on these results, although the UBF has much less steel, it may be expected to show better seismic performance.

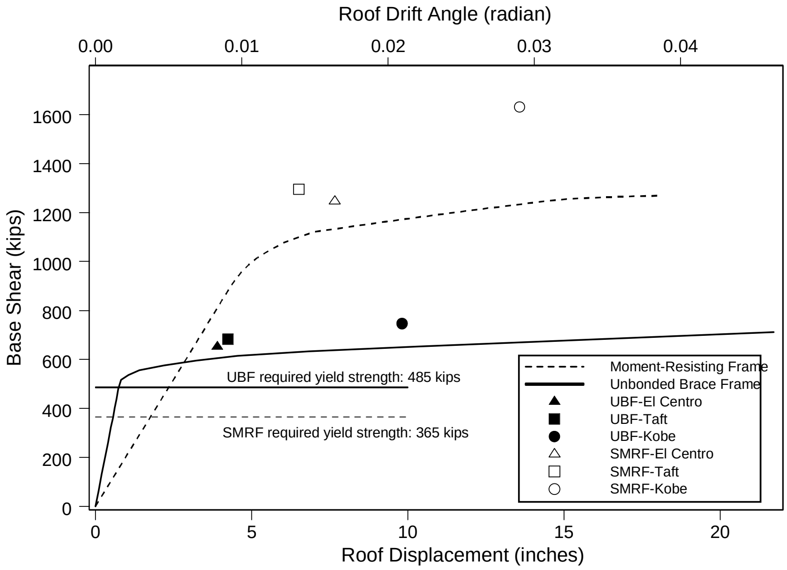


Figure 3: Results from Time History Analyses Superimposed on Pushover Curves

Figure 4 shows the displacement envelopes of the UBF and SMRF under 1.495 times El Centro (PGA=0.521g), 2.824 times Taft (0.506g), and 1.0 times JMA Kobe (0.83g) earthquakes, respectively. The absolute maximum roof drifts of the UBF are 0.51, 0.65, and 0.72 times those of the SMRF, due to these three earthquakes. Fig. 3 also shows that the UBF base shear under the three earthquakes are about 0.5 times that of the SMRF, which implies that the accelerations developed in the UBF are smaller than those in the SMRF. Note also the large discrepancy between the base shear in the static pushover analyses and those determined from the Kobe earthquake analyses for the SMRF.

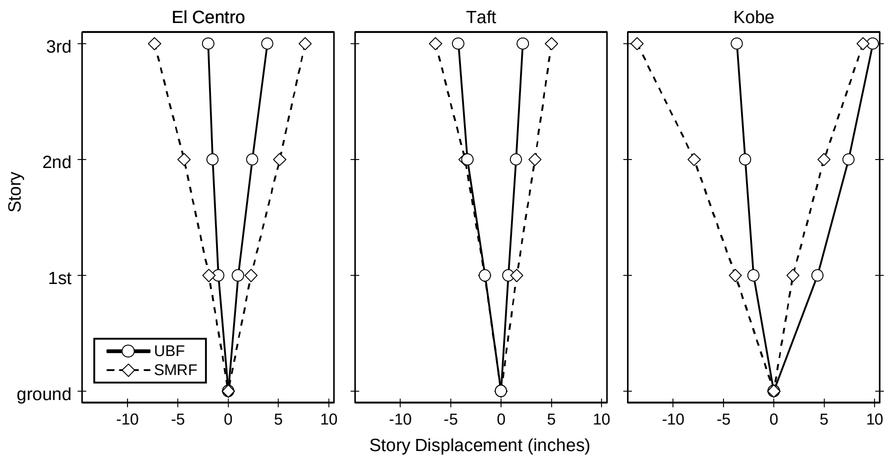


Figure 4: Story Drift Profiles for Unbonded Braced Frame and Moment-Resisting Frame

Figures 5 and 6 show plastic rotation demands in the beams and columns (UBF and SMRF) as well as axial strains of the unbonded brace (UBF only) under the 1.495 times El Centro earthquake. The beams and columns of the UBF are almost elastic, indicating better frame damage control than in the SMRF. The largest unbonded brace strain is only 1.1%. Although not shown, under the Kobe earthquake the UBF plastic rotations are less than half those in SMRF which developed plastic rotations of 2 to 3 % radian. Increasing the beam and column sizes at the braced bay of the UBF (which will cause only a slight increase in cost) could make this frame virtually damage free, even against the Kobe record.

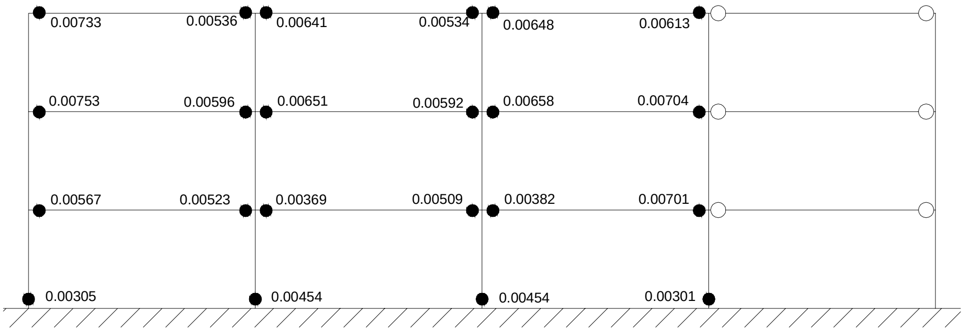


Figure 5: Peak Plastic Rotation Demands in Moment-Resisting Frame Under 1.495 Times El Centro

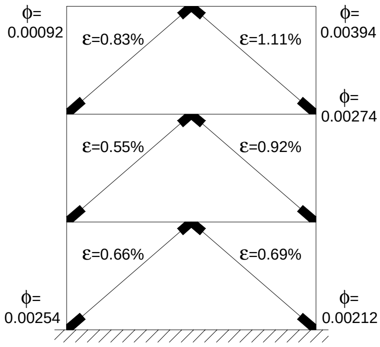


Figure 6: Peak Plastic Rotation Demands and Brace Strains in Unbonded Brace Frame Under 1.495 Times El Centro

7. LARGE-SCALE TESTS OF UNBONDED BRACES

Three large-scale unbonded braces were tested in the Structures Laboratory of the Department of Civil & Environmental Engineering at the University of California at Berkeley. The main purpose of the tests was to demonstrate the behavior of full-size braces under a increasing-amplitude cyclic loading history, derived from the protocol used in the Phase 2 SAC Steel Project. (This load pattern is referred to here as the SAC Basic Loading History). The behavior of the braces under other types of loading (SAC near-field protocol, simulated earthquake loading, and low-cycle fatigue) was also investigated.

The three specimens, denoted T-1, T-2, and T-3, were the same overall length, approximately 14.75 ft., but each had an axial load-carrying steel core plate with a different cross-sectional area (4.5 in^2, 6.0 in^2, and 8.0 in^2, respectively, with corresponding yield forces of approximately 270 kips, 360 kips, and 470 kips, respectively.) Specimens T-1 and T-2 had a rectangular yielding core section. The core of specimen T-3 was a cruciform (+) cross-section. The core plate and end connection splice plates were manufactured from JIS (Japanese Industrial Standard) grade SM490A steel, equivalent to ASTM A913.

The test program is shown in Table 2. The “SAC” loading histories were derived from the standard protocol specified for steel moment connections in the SAC Steel Project test programs. This loading protocol is expressed in terms of interstory drift but was converted to an equivalent brace strain for these tests. The target interstory drift of 3 percent thereby converted to a brace strain of 2 percent. The SAC Near-Field Loading History was used to reproduce the biased response anticipated in a structure subjected to a near-field velocity pulse. The peak displacement in this loading history corresponds to an interstory drift of 6 percent or a brace strain of 4 percent. The low-cycle fatigue test consisted of 18 tension-compression cycles at the MCE interstory drift of 3 percent, corresponding to a brace strain of approximately 2 percent. This test was initiated after the SAC Basic Loading History was completed, so the specimen had already experienced 2 cycles at the MCE drift.

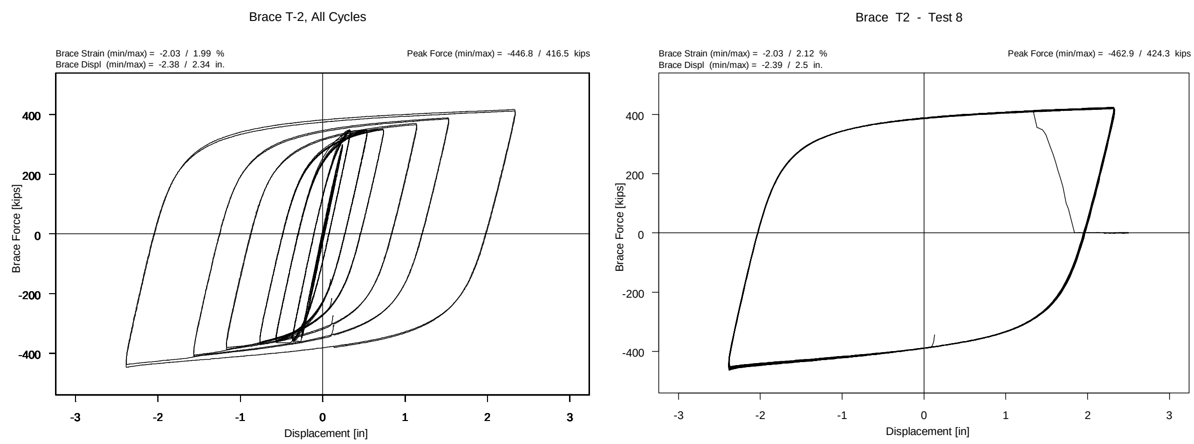
All of the specimens exhibited stable hysteretic behavior during the cyclic and earthquake loading tests, and only specimen T-2 failed, as a result of 17 cycles at 2 percent axial strain in the low-cycle fatigue test. Figure 7.a shows the force-displacement relationship measured during the SAC Basic Load History test for specimen T-2. Similar behavior was observed in the other specimens during this test, indicating that each of the braces sustained the Basic Loading History with very little change in properties, and the yield properties were easily predicted based

Table 2: Test Program for Unbonded Braces

Specimen ID		
T-1	T-2	T-3
SAC basic loading history	SAC basic loading history	SAC basic loading history
SAC near-field loading history	Low-cycle fatigue test	Earthquake displacement test

on coupon test results. The brace force in compression is slightly higher than that in tension; The difference between the peak tension load and the peak compression load ranged between 7.3 and 9.5 percent for the three specimens.

The force-displacement relationship for the low-cycle fatigue test of specimen T-2 is provided in Fig. 7.b. The brace exhibited extremely stable cycle behavior with virtually no degradation of strength or stiffness for all of the loading cycles up to failure, with a fracture failure of the core plate occuring inside the confining tube in the second half of the 15th cycle. These 15 cycles, combined with the two cycles at 2 percent brace strain in the Basic Loading History, give a total of 17-1/2 cycles to failure at a brace cyclic strain of 2 percent. The force-displacement relationship for the SAC Near-Field Loading History test of specimen T-1 is shown in Fig. 8. It can be seen that the specimen exhibited stable behavior even when cycled about an offset displacement of 3.34 inches, and for a maximum tension displacement of 4.84 inches, approximately two times the maximum design displacement for the brace. Further information regarding the test program and results is available in [Clark, et al., 1999].



a. Basic Loading History Test

b. Low-Cycle Fatigue Tests (15 cycles prior to failure)

Figure 7: Hysteretic Behavior of Brace Specimen T-2

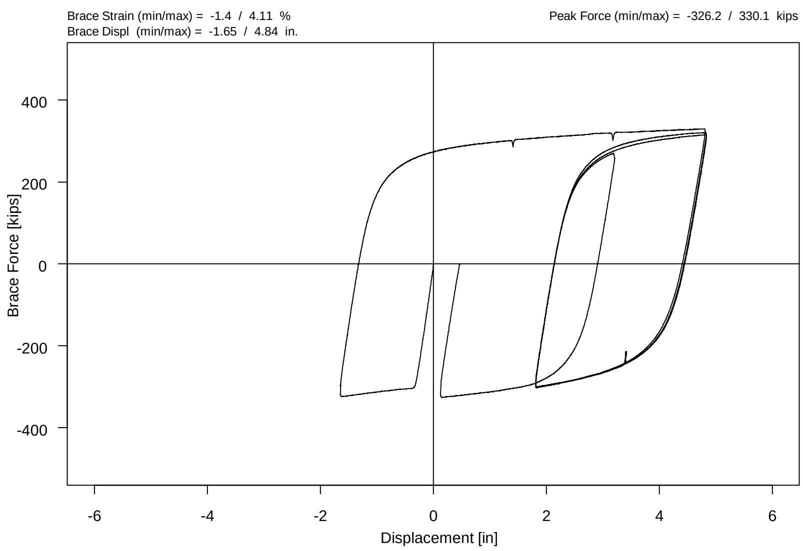


Figure 8: Hysteretic Behavior of Brace Specimen T-1 in SAC Near-Field Loading Test

8. FIRST APPLICATION IN THE UNITED STATES: UC DAVIS PLANT & ENVIRONMENTAL SCIENCES BUILDING

The Plant & Environmental Sciences Replacement Facility (P&ESRF) is a three-story laboratory building on the University of California Davis campus. It is a steel building with composite metal deck construction and has a total floor area of 125,000 square feet. The overall building is roughly "C" shaped in plan. A seismic joint divides the building into two separate "L-shaped" structures.

A lateral system using EBFs was selected initially over a steel moment frame system based on the willingness of the architect to incorporate braces and a cost-benefit study comparing the two systems. The braced bays were strategically located after careful coordination with the architect so that maximum program flexibility of laboratory spaces (brace free laboratory suites) could be achieved. With the EBF as a base system, the inclusion of the UBF as an alternate was justified after a series of nonlinear static pushover analyses were conducted to compare the performance of EBF, UBF and SCBF structural systems. To help optimize braced frame locations and to limit the rotational response of the structure, ETABS models of the east and west buildings were developed. The relative stiffness of the braced frames was first adjusted to get the center of rigidity to closely correspond to the center of mass. Dynamic analyses were then performed to capture the dynamic characteristics of the structures. As expected, the braces at the perimeter had to be increased in sizes to balance the rotational stiffnesses of the buildings. Construction on this building is underway and is expected to be complete in 2000.

9. CONCLUSIONS

This paper summarized a number of recent activities related to the implementation of unbonded braces within United States seismic design practices. First, a study was summarized which is intended to evaluate the suitability of using code-consistent equivalent static force procedures to design frames incorporating buckling-resistant unbonded braces.. Because the SMRF frame size was controlled by drift requirements, the frame exhibited a significant overstrength compared with the minimum yield base shear. The UBF, designed according to provisions for EBFs, did not suffer from this limitation and therefore had a much lower yield base shear and significantly less steel in the lateral load-resisting system. The results of a brief series of nonlinear time history analyses also showed that the UBF performed better than the SMRF in terms of interstory drift and base shear. This preliminary study is currently being extended to investigate taller structures and to develop design procedures for higher performance levels within the context of U.S. building codes. Parallel to the ongoing design studies, large-scale tests of unbonded braces have been carried out to demonstrate the stable hysteretic behavior that can be achieved with these elements. Three braces were subjected to a wide range of tests and showed predictable behavior and substantial overstrength in terms of both displacement and energy dissipation capacity.

10. ACKNOWLEDGEMENTS

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11. REFERENCES

Clark, P., Aiken, I., Kasai, K., Ko, E., and Kimura, I. (1999), "Design Procedures for Buildings Incorporating Hysteretic Damping Devices," *Proceedings of 1999 Annual Convention of the Structural Engineers Association of California*, Santa Barbara, California.

Kasai, K., Fu, Y., and Watanabe, A. (1998), "Passive Control Systems for Seismic Damage Mitigation," *Journal of Structural Engineering*, American Society of Civil Engineers, Vol. 124, No. 5, May, pp. 501-512.

Reina, P, and Normile, D (1997), "Fully Braced for Seismic Survival," *Engineering News Record*, July 21, 1997, pp. 34-36.

SAC (1999), "Model Building Designs for Three U.S. Cities," *SAC Background Document*, in preparation.

Uniform Building Code, 1994, International Conference of Building Officials, Whittier, CA.

Watanabe, A., Hitomi, Y., Saeki, E., Wada, A., and Fujimoto, M. (1988), "Properties of Brace Encased in Buckling-Restraining Concrete and Steel Tube," *Proceedings of Ninth World Conference on Earthquake Engineering*, Tokyo-Kyoto, Japan, Paper No. 6-7-4, Vol. IV, pp. 719-724.