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Design Procedures for Buildings Incorporating Hysteretic Damping Devices

Peter Clark, Ian Aiken
Seismic Isolation Engineering, Inc.
Oakland, California

Eric Ko
Ove Arup and Partners
San Francisco, California

Kazuhiko Kasai
Tokyo Institute of Technology
Yokohama, Japan

Isao Kimura
Nippon Steel Corporation
Tokyo, Japan

Abstract

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 elements.

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 pattern 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.

The final portion of the paper describes design studies in support of the first application of unbonded braces in the United States. Nonlinear pushover analyses of several different braced frame designs corresponding to an eccentric braced frame system, a concentric braced frame system, and an unbonded braced frame system are summarized.

Introduction

Engineers around the world are now considering the use of seismic energy dissipation devices in structures large and small. The primary benefit of dampers — reliable absorption of earthquake energy in elements separate from the primary structural frame — is well established, but designers continue to struggle to identify appropriate design techniques for the sizing and distribution of dampers in multi-story buildings. Establishing straightforward design approaches for hysteretic dampers is important for realizing the advantages of these devices, particularly their low cost, long-term reliability, and lack of dependence on mechanical components.

There has been considerable debate in the structural engineering community regarding the target performance level for structures with passive damping devices. Originally, dampers were envisioned as means of enhancing the performance of structures which already meet lateral force-resisting requirements, making supplemental damping suitable only for high-performance structures. Recently, it seems that for code-minimum performance levels there is a trend toward allowing reductions in the mandated lateral force-resisting system in proportion to the benefits derived from including dampers. Because structures with passive damping systems can provide predictable and stable behavior under seismic loading, it is appropriate that designers be given the freedom to use these systems to attain a range of performance levels, from collapse-prevention to immediate occupancy.

This paper proposes a design procedure for hysteretic dampers 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 tension and compression without buckling. A series of non-linear 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 next presented as evidence of the stable hysteretic behavior that can be achieved. A number of cycles of displacement at relatively large axial yield strains can be sustained in the braces prior to failure, giving designers confidence that a lateral force-resisting system incorporating these elements will provide at least equivalent performance to and EBF.

As more designers begin to investigate the potential benefits of using unbonded braces, the first applications are being introduced. A brief summary of the design studies in support of the implementation of unbonded braces in a university research laboratory closes the paper.

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, and since the 1995 Kobe Earthquake, these elements have been used in numerous major structures in Japan (e.g., Reina, and Normile, 1997). In fact, the concept of “damage tolerant structures” — in which the the primary structural system is designed to remain elastic while all energy dissipation occurs in specially-detailed components of the lateral force-resisting system — is gaining broad acceptance in Japan (Wada, et al., 1997). According to records from the Building Center of Japan for the year 1997, approximately two-thirds of all tall buildings (greater than 60 meters) approved for design that year incorporate some form of passive damping system, and the majority of these use hysteretic dampers (Building Center of Japan, 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 “unbonded brace” derives from the need to provide a slip

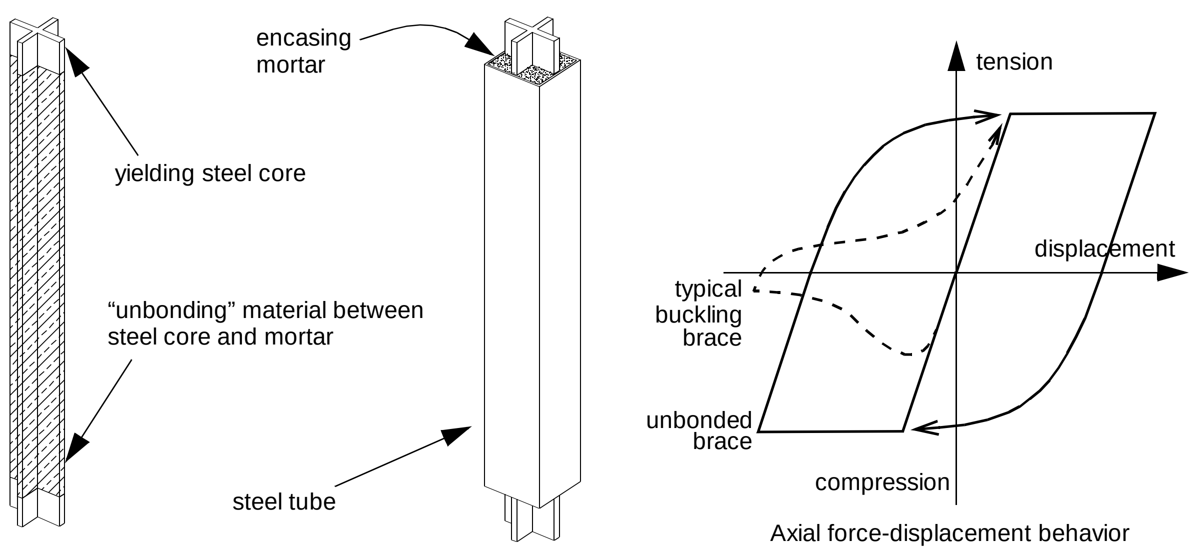

Figure 1. Schematic of Mechanism of Buckling-Resistant Unbonded Braces

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.

The stable hysteretic behavior of a properly detailed unbonded brace contrasts with the behavior of bracing elements in typical concentrically-braced frames (CBFs). While a number of studies over the past two decades have resulted in improved detailing requirements for brace elements in CBFs and special CBFs, the anticipated behavior mode in compression remains global buckling of the brace and a consequent loss of strength and stiffness. Connection detailing then becomes critical to the ability of the brace to develop its full tension capacity under reversed loading, and consideration must also be given to the unbalanced forces transferred to beams in frames with V-bracing or inverted V-bracing. While economical, such systems clearly have drawbacks in terms of seismic performance, and, as a consequence, equivalent static design forces for these systems tend to be quite large.

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. It is also noted that while the focus of this paper is on the design of new buildings, unbonded braces are clearly applicable for the upgrade of existing buildings such as non-ductile reinforced concrete frames where additional stiffness, strength, and energy dissipation may be beneficial.

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.

Building Example Considered in this Study

The structure investigated in this analytical study is based on a three-story special moment-resisting frame (SMRF) originally developed for a series of nonlinear time history analyses in Phase 2 of the FEMA/SAC Steel Project (SAC, 1999). The building is assumed to be located in Los Angeles (seismic zone 4) on UBC soil type S2, and was designed to meet the 1994 UBC provisions. Using the structural period given by Section 1628.2.2 (Method A) of the 1994 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 are sized to meet drift requirements and therefore have a much higher yield base shear. Figure 2 shows the geometry and member sizes of one of the moment-resisting frames in the North-South direction of the building that is used for the time history analyses. For this structure, grade beams were used at the foundation level to achieve full fixity of the column bases.

Redesign of SAC SMRF Model Building Incorporating Unbonded Braces

Design Assumptions

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. A rational UBF design method to obtain optimum performance has been developed previously (Kasai et al. 1998). However, in the present paper we will illustrate a method similar to those presented in the UBC for conventional steel structures. The method does not necessarily seek optimum performance. 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. 3). The required cross sectional area of the yielding portion of the brace, A'_{br}, is then calculated as

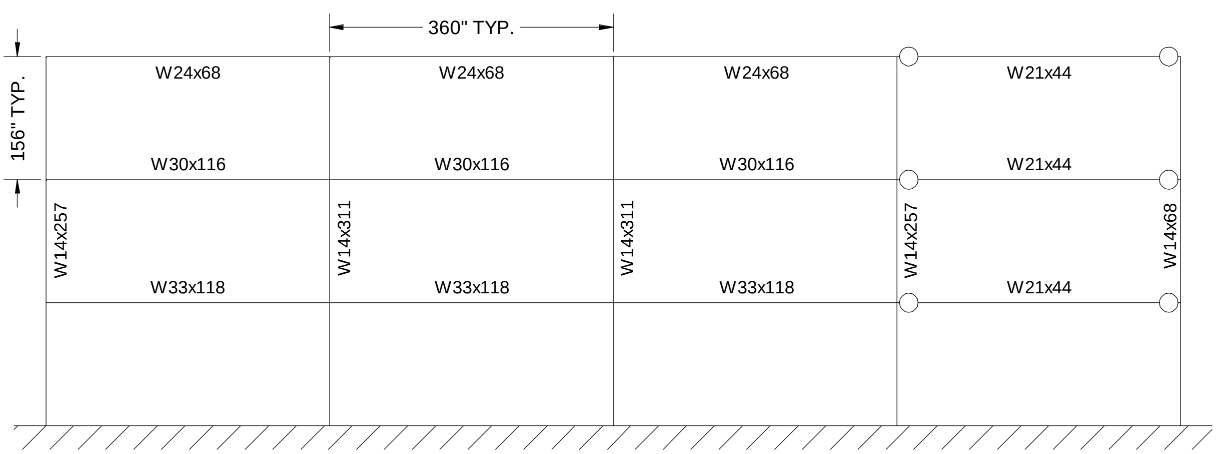

Figure 2. Three-Story Special Moment-Resisting Frame from FEMA/SAC Project

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

Table 1: Design Parameters for Three Structural Systems

$$A'_{br} = \frac{P_{y,\,br}}{\phi F_{y,\,br}} \qquad \text{(Eq. 1)}$$

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. (A method to explicitly consider the effect of strain-hardening is presented in Kasai and Goyal, 1993). However, unlike EBF design, the moments in the beams and colums of the UBF at its ultimate state are difficult to estimate before knowing the element sizes and deformations (which depend on the story drifts), although the axial forces are easy to obtain. Our preliminary design therefore employs some conservatism 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.

Design Results

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. 3. 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. Note that when compared with the SMRF (Fig. 2) 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.

The length of the yielding portion of the braces, L'_{br} =130 in., is obtained by subtracting the lengths of the splice, gusset, beam, and column regions (at both ends of the brace) from the center-to-center length, L_{br} = 238 in. Since the yielding portion has a substantially smaller cross sectional area A'_{br} as compared with the end connections, most of the elastic deformations take place therein, as well as all of the inelastic deformations. Considering this, the elastic axial stiffness of each unbonded brace is approximated by

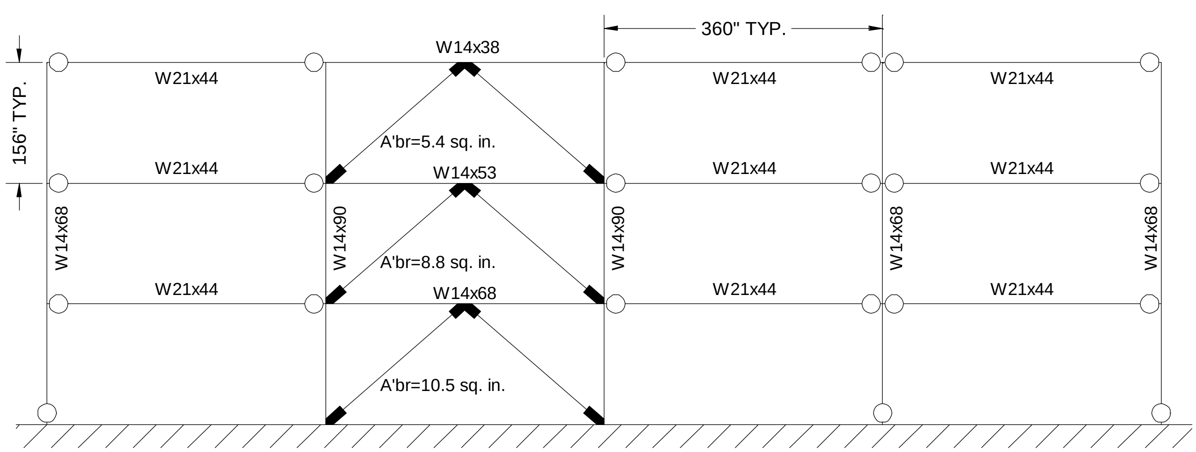

Figure 3. Three-Story Frame Redesigned with Unbonded Braces

$$K_{br} = \frac{EA'_{br}}{L'_{br}}. \quad \text{(Eq. 2)}$$

Also, the axial strain of the yielding portion is expressed as

$$\varepsilon_{br} = \frac{\delta_{br}}{L'_{br}}, \quad \text{(Eq. 3)}$$

where δ_{br} is the axial deformation of the unbonded brace. Note that 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 $\varepsilon_{br} = \pm 0.02$. 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} (Eq. 3) is acceptable. Such an approach will produce a higher brace stiffness K_{br} (Eq. 2) 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. This characteristic is very attractive from the viewpoint of giving designers flexibility in proportioning bracing elements, and it can be extended further by considering steels of a variety of strengths (lower or higher yield strengths) and controlling A'_{br} (see Eqs.1 and 2).

Results of Nonlinear Analyses

A brief series of nonlinear analyses were 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 the 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:

- 1.495 times the 1940 El Centro North-South ground motion
- 2.824 times the 1952 Taft East-West ground motion
- 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. 4. 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. 4), 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 5 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. 4 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.

Figures 6 and 7 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

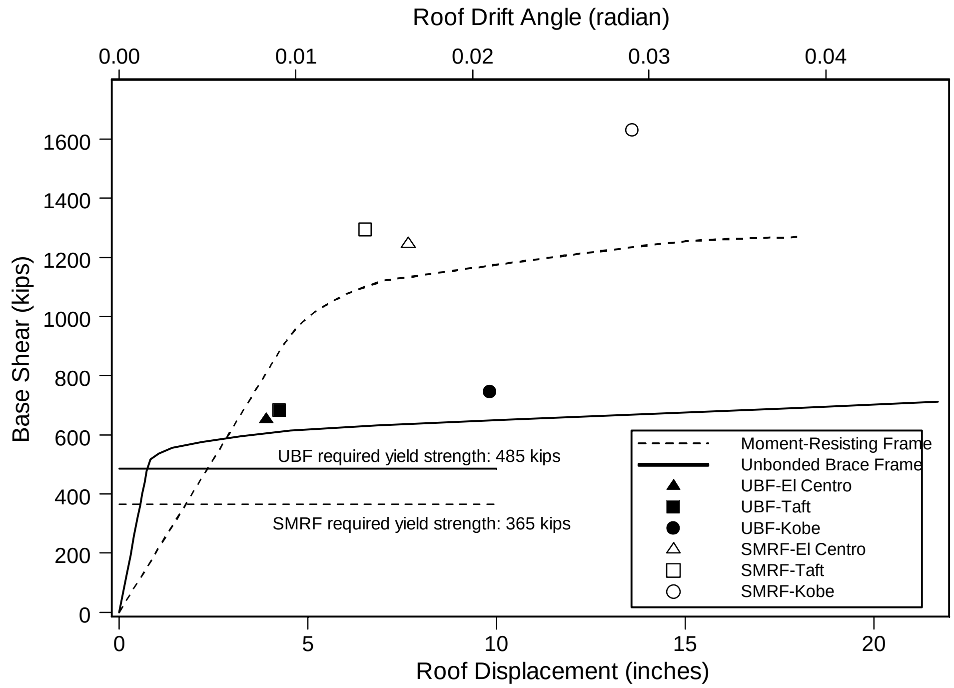

Figure 4. Results from Time History Analyses Superimposed on Pushover Curves

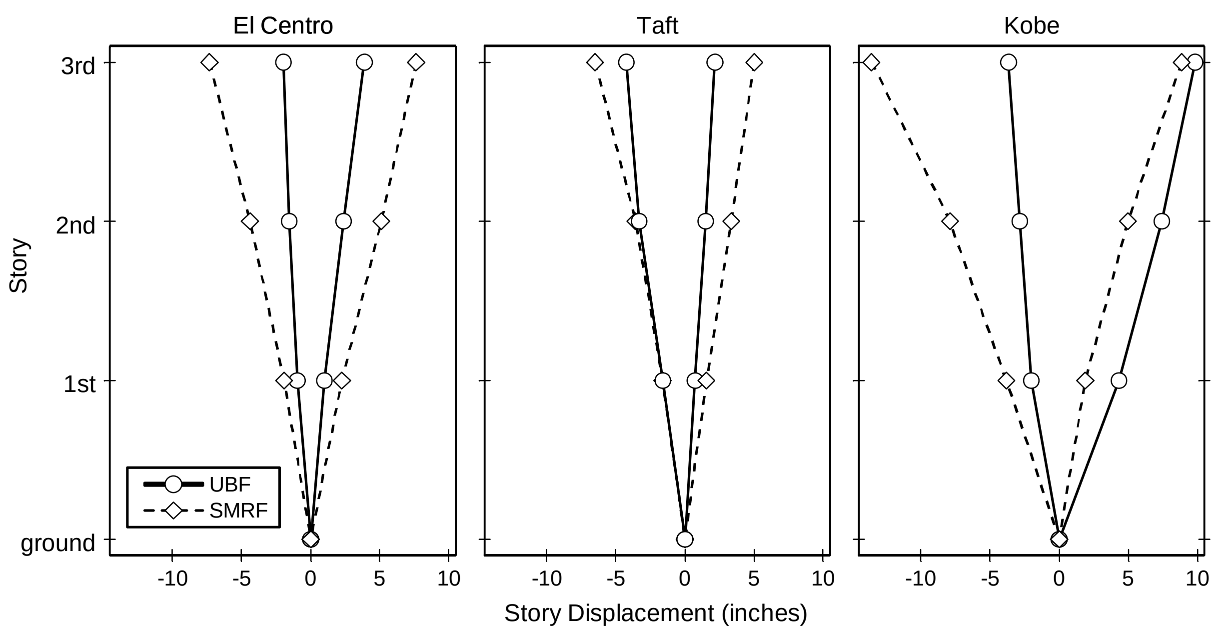

Figure 5. Story Drift Profiles for Unbonded Braced Frame and Moment-Resisting Frame

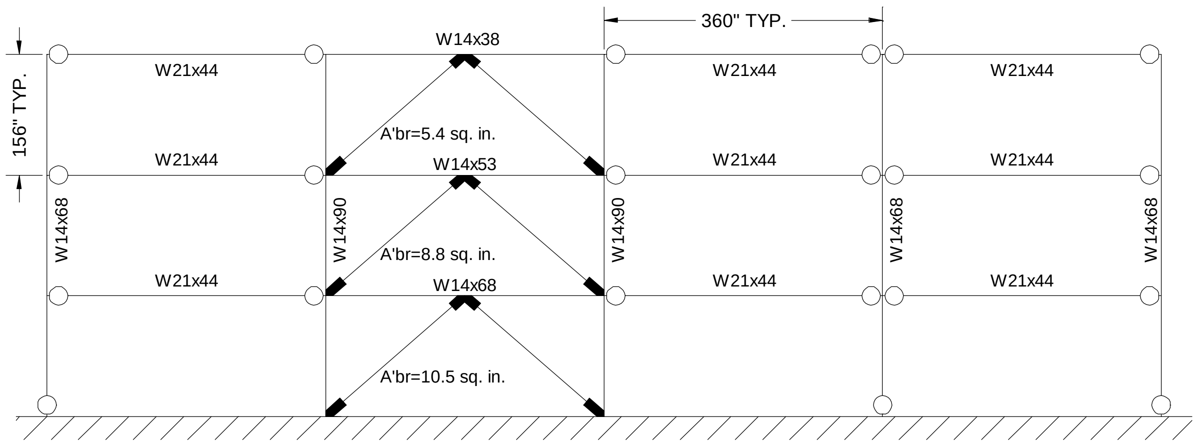

Figure 6. Three-Story Frame Redesigned with Unbonded Braces

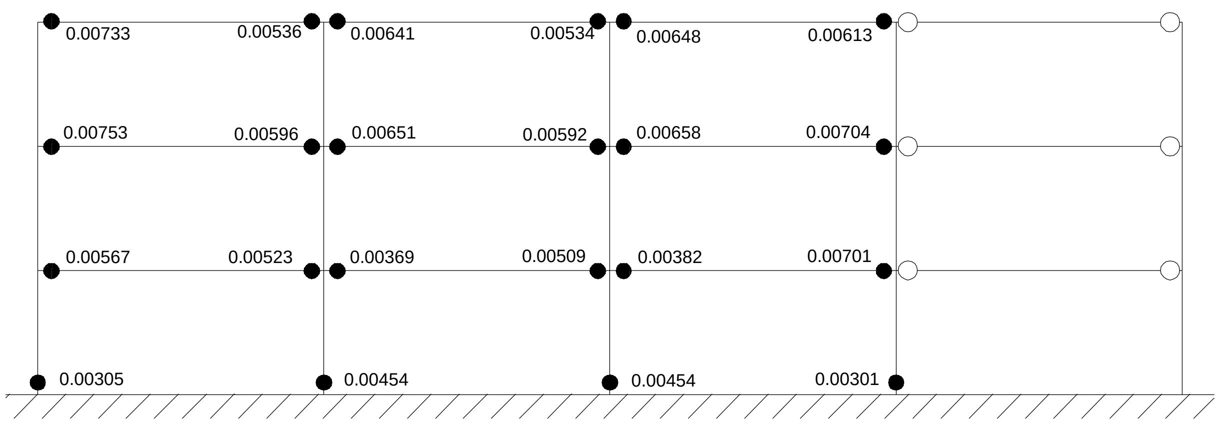

Figure 7. Peak Plastic Rotation Demands in Moment-Resisting Frame Under 1.495 Times El Centro

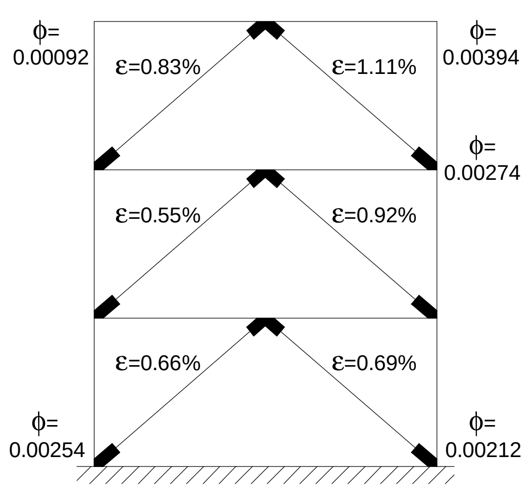

Figure 8. Peak Plastic Rotation Demands and Brace Strains in Unbonded Brace Frame Under 1.495 Times El Centro

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.

Results from Large-Scale Tests

Introduction

Three large-scale unbonded braces were tested recently in the Structures Laboratory of the Department of Civil & Environmental Engineering at the University of California at Berkeley. The test program was initiated to support the design of the new UC Davis Plant & Environmental Sciences Replacement Facility (see next section). 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.

Test Specimens and Test Program

All of the specimens 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. The three specimens, denoted T-1, T-2, and T-3, had core areas of 4.5 in^2, 6.0 in^2, and 8.0 in^2, respectively, and yield forces of approximately 270 kips, 360 kips, and 470 kips, respectively. Specimens T-1 and T-2 had a rectangular yielding core section, and the core specimen 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, which is equivalent to the A913 steel recently introduced into the United States market. Coupon tests of the steel indicated an average yield stress of 60.7 ksi, an average ultimate stress of 78.2 ksi, and an average ultimate elongation of 28 percent.

The test program was as follows:

- Specimen T-1:
  - SAC Basic Loading History
  - SAC Near-Field Loading History
- Specimen T-2:
  - SAC Basic Loading History
  - Low-cycle fatigue test
- Specimen T-3:
  - SAC Basic Loading History
  - Earthquake displacement records

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; for the purposes of these tests, interstory drift in one of the braced frame from the UC Davis building was converted to an equivalent strain in the full-length brace, then this strain time history was applied to the test brace. The target interstory drift was approximately 3 percent, consistent with the largest drift computed for the UC Davis building under the MCE ground motions. The corresponding brace strain was approximately 2 percent, varying slightly between the different test specimens because each had a slightly different yielding length. Figure 9 shows the SAC Basic Loading History.

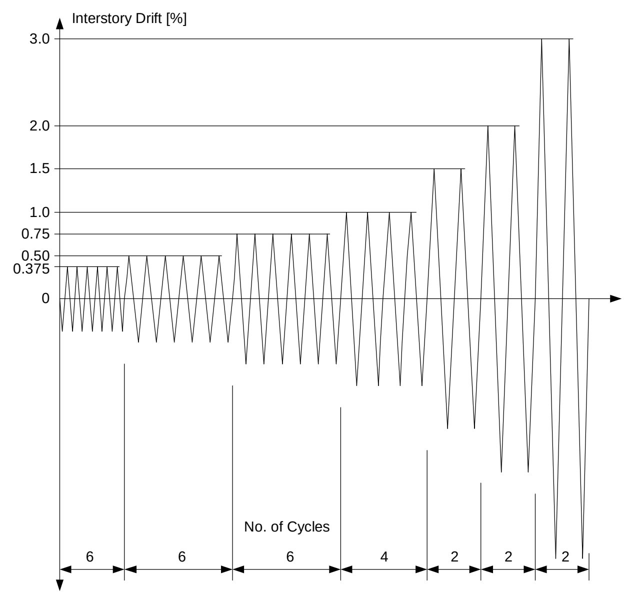

Figure 9. SAC Basic Loading History

The SAC Near-Field Loading History, shown in Fig. 10, was developed to represent the type of biased response that might be anticipated in a structure subjected to a near-field velocity pulse. The maximum displacement in this loading history corresponds to an interstory drift of 6 percent. The corresponding brace strain was 4 percent.

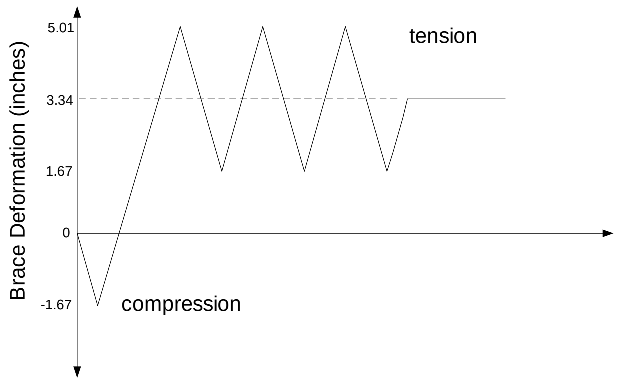

Figure 10. SAC Near-Field Loading History

The earthquake displacement time histories were derived from a simplifed analysis of a 5-story building subjected to the 1940 El Centro NS record and the 1994 Sylmar NS record. An equivalent initial period and yield base shear coefficient were selected appropriate to a 5-story braced steel frame, and then a nonlinear time history analysis was performed on the equivalent single degree of freedom system. The resulting displacement was then multiplied by 3/2 and divided by the total assumed building height to obtain a mean interstory drift time history. Finally, this was converted to an equivalent brace strain in the test specimen.

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.

Test Results

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.

Figures 11 through 13 show the force-displacement relationship measured during the SAC Basic Load History test for specimens T-1 through T-3, respectively. It is clear that each of the braces sustained the Basic Loading History with very little change in properties. Also, the yield force values observed in the tests are very similar to those predicted based on coupon testing, providing confidence in the reliability of coupon testing for predicting brace behavior. It should be noted that there was some bolt slip during the tests of specimen T-3, when the force developed in the brace was greater than 600 kips. The bolt slip is evident in the sudden drops in load in the hysteresis loop for T-3 (Fig. 13).

It can be seen that in each of the tests, the brace force in compression is slightly higher than that in tension, perhaps caused in part by variations in the cross-sectional area and therefore the true stress on the central steel core as it yields in tension and compression within the concrete-filled tube. The difference between the peak tension load and the peak compression load ranged between 7.3 and 9.5 percent for the three specimens. A simple calculation shows that the difference between engineering stress and true stress at a strain of 2 percent does not entirely account for the observed difference between compression and tension stress, so there are likely other mechanisms at work, such as compression of spacing materials inside the tube, or constraint on the steel core when it goes into compression inside of the tube.

The force-displacement relationship for the SAC Near-Field Loading History test of specimen T-1 is shown in Fig. 14. It can be seen that the specimen exhibited stable behavior even when cycled about an offset displacement of 3.34 inche, and for a maximum tension displacement of 4.84 inches, approximately two times the maximum design displacement for the brace.

The force-displacement relationship for the low-cycle fatigue test of specimen T-2 is provided in Fig. 15. The brace exhibited extremely stable cycle behavior with veirtually 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 cycles. 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 earthquake displacement test corresponding to the 1994 Sylmar record is shown in Fig. 16. Again, the specimen exhibited very predictable hysteretic behavior with no strength or stiffness degradation. Although the lower-amplitude response from the El Centro test is not shown here, the brace force-displacement was similar to that observed in the small-amplitude cycles during the Sylmar test. The results from these earthquake tests will be used to calibrate analytical models for unbonded braces for future studies.

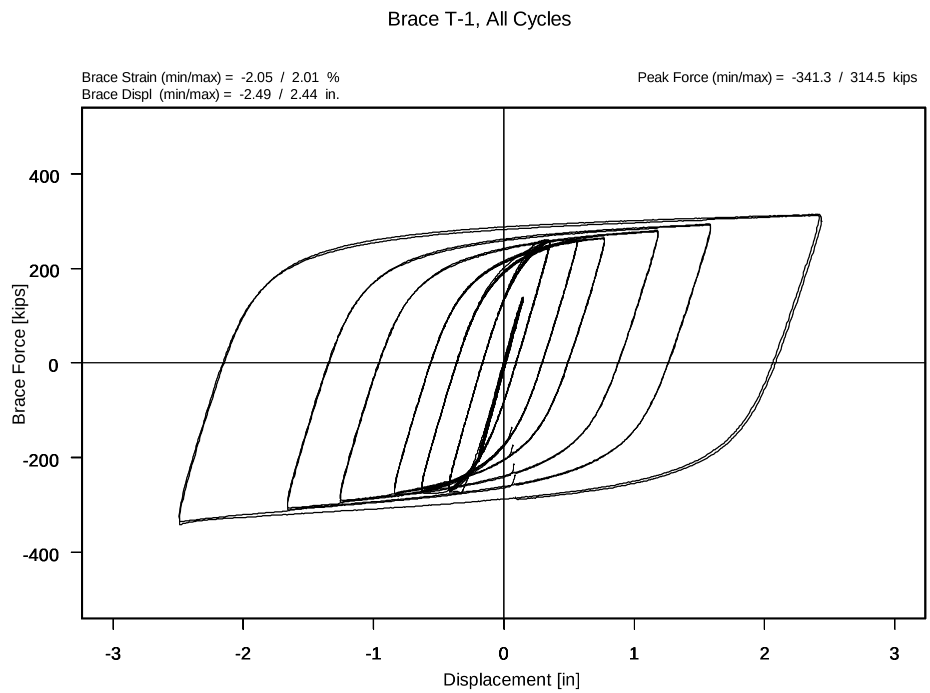

Figure 11. Hysteretic Behavior of Brace Specimen T-1 in Basic Loading History Test

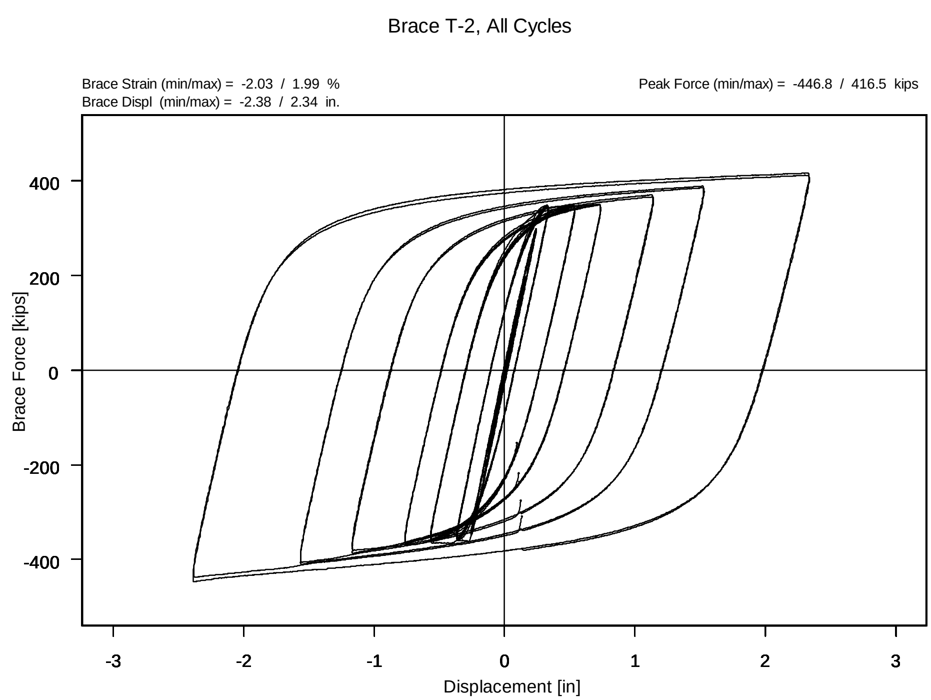

Figure 12. Hysteretic Behavior of Brace Specimen T-2 in Basic Loading History Test

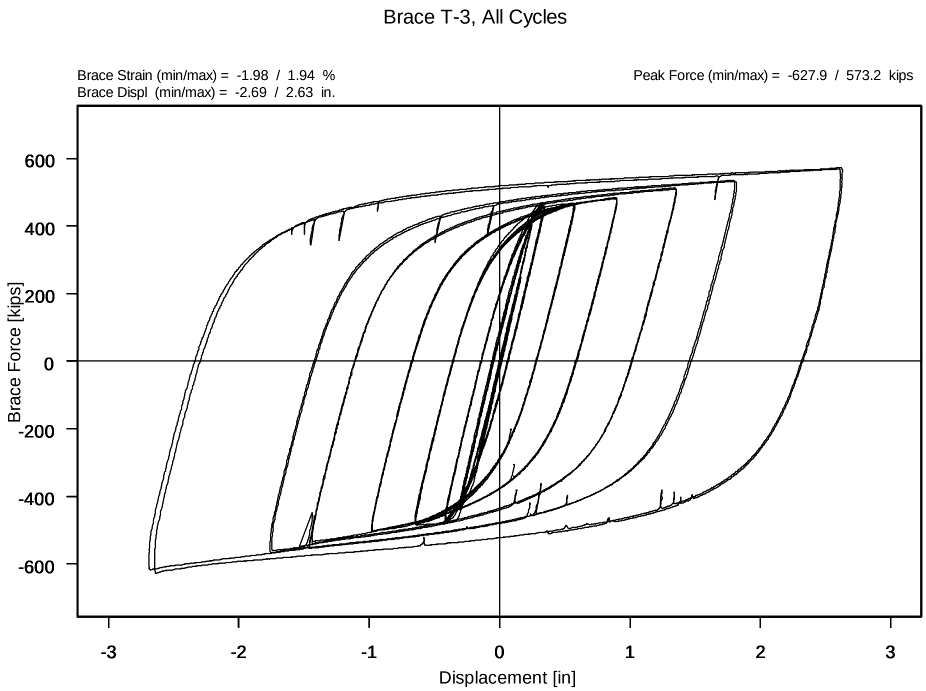

Figure 13. Hysteretic Behavior of Brace Specimen T-3 in Basic Loading History Test

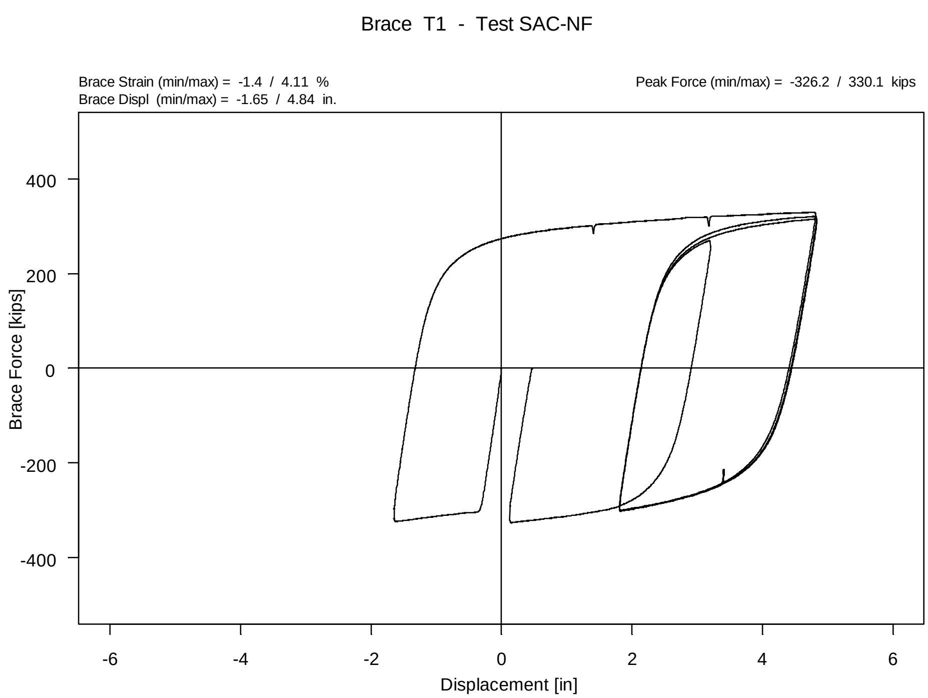

Figure 14. Hysteretic Behavior of Brace Specimen T-1 in SAC Near-Field Loading Test

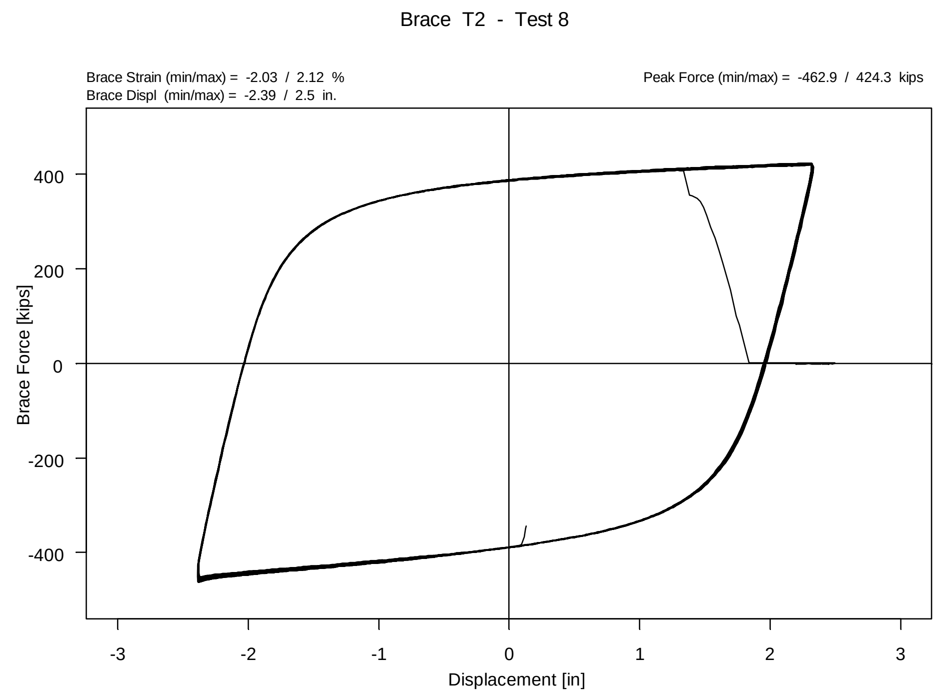

Figure 15. Hysteretic Behavior of Brace Specimen T-2 During Low-Cycle Fatigue Test (15 complete cycles shown in the figure prior to failure)

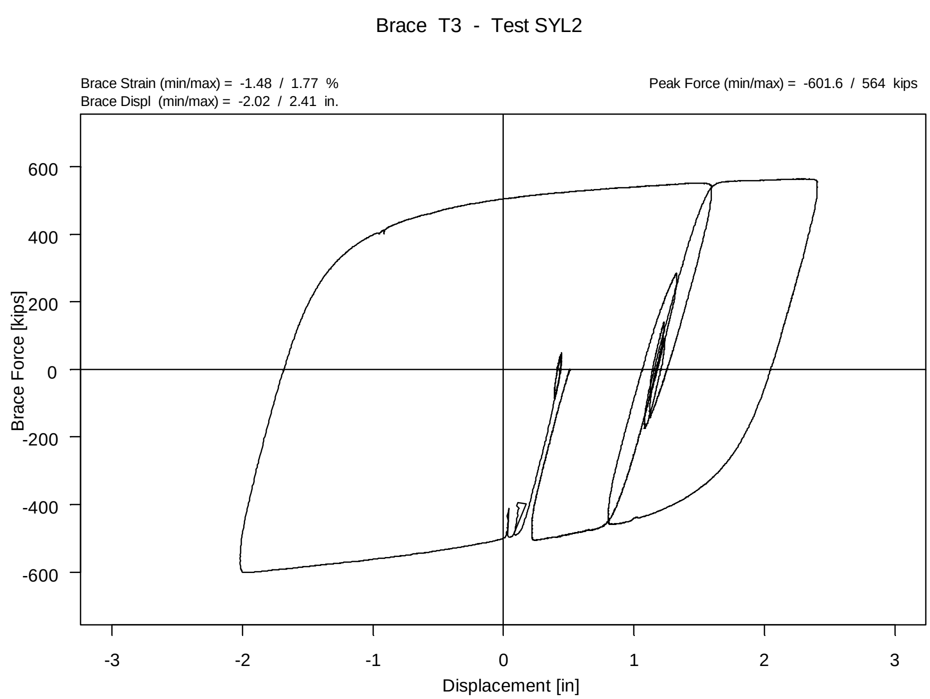

Figure 16. Hysteretic Behavior of Brace Specimen T-3 During Sylmar Earthquake Test

Conclusions

The results for all three test specimens indicated very good agreement with the elastic stiffness and yield force predicted based on coupon testing. The SAC Basic Loading History tests showed the unbonded braces capable of stable cyclic hysteretic behavior over the entire range of displacement amplitudes. Finally, the behavior of the braces in the additional tests indicated their resistance to fracture, even after severe loading, and their stable, predictable force-displacement characteristics, even under non-cyclic transient loadings such as earthquakes.

First Application in the United States: UC Davis Plant & Environmental Sciences

Introduction

The Plant & Environmental Sciences Replacement Facility is a three-story laboratory project located at 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 (Fig. 17). A seismic joint divides the building into two separate "L-shaped" structures.

A lateral system using Eccentrically Braced Frame (EBF) was selected 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.

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.

Lateral Systems Comparison – Pushover Analyses

The Eccentrically Braced Frame (EBF) system was selected as the base seismic system for the project. In order to justify the inclusion of the Unbonded Braced Frame (UBF) as an alternate, nonlinear static pushover analyses were conducted to compare the performance of EBF, UBF and CBF (Concentrically Braced Frame) structural systems. A typical bay at the perimeter of the East Wing was picked for this exercise. (Note that this bay was not designed for the same level of lateral force in each of the three systems, because the number and distribution of braced bays was slightly different for each system.)

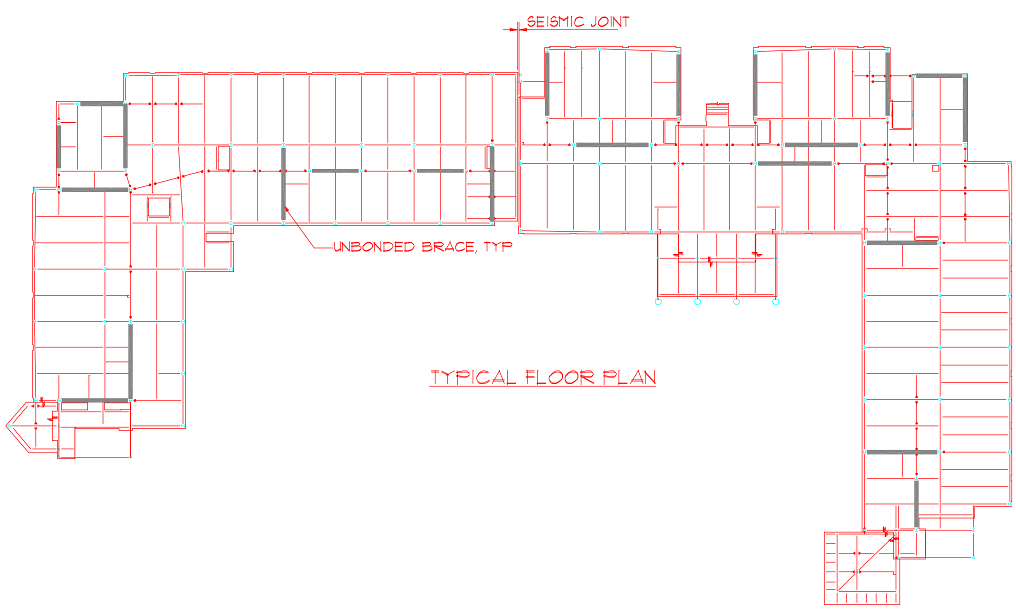

Figure 17. Plan of UC Davis Plant & Environmental Sciences Replacement Facility

For the purposes of evaluating the behavior of the different braced framing systems, the “performance point” is defined as the intersection between the demand spectrum and capacity spectrum of a particular building. A response spectrum is determined for the site considering near source factors, seismic zone, and performance objectives. This spectrum typically assumes the structure has 5% damping and will behave elastically during an earthquake. Buildings are expected to have inelastic response during a design earthquake event, so the spectrum is subsequently be scaled to account for this. As earthquake intensity increases, the building responds with increasing inelastic behavior. The effect is to then increase the damping and the effective period, leading to a response that is typically smaller than for a building with less damping or a shorter period. The response spectrum is scaled to the design spectrum based on the level of damping achieved when the building capacity matches the demand placed on it. Since the demand will depend on this damping level, the solution of this performance point is iterative. The performance point represents the maximum structural displacement expected for the demand earthquake ground motion. Note that this may not coincide with the maximum force level since it may occur after yielding and a drop in load capacity.

Table 2 summarizes the structural framing used in each of the braced frame systems considered. All of the members are assumed to be Grade 50 except the tube sections are assumed to have a yield stress of 46 ksi. A total of six masses of 0.9 kips/(386.4 in/sec^2) each are placed on column line locations on each floor.

Results of the pushover analyses and performance points for each system (denoted by ●) are shown in Fig. 18.

The unbonded brace system resists the most force (base shear) because it one of a fewer number of braced frames required in the building as compared with the other systems. It exhibits the smoothest response in the pushover analysis, with the CBF system demonstrating successive member failures in the incremental elastic analysis. The EBF system reaches the performance point in a smooth manner similar to the UBF, but for increased demand its strength drops dramatically when the link capacity is exceeded. The CBF system has several braces lose their capacity before the performance point is reached. While there is no collapse at this point, substantial damage would occur to this system at this design earthquake level.

Table 3 summarizes the differences in the structural systems at their individual performance points. Each system will satisfy the given performance objective with the quantities given in the Table.

The unbonded brace structural system has the lowest roof displacement and the highest base shear to weight ratio. This means that it has displaced the least and has the highest reserve capacity after the design earthquake has occurred. The effective period, Teff, will generally be largest for the system that has the most damage. Notice that this quantity is smallest for the unbonded brace system. In addition, the unbonded braces are only strained one-fifth the amount of the concentric braces. The link beam in the EBF system has an even larger strain. While the strain in the link beams of the EBF system seem large relative to the strains in the braces of the CBF system, the EBF behavior is better than the CBF behavior. The EBF scheme achieves the performance level at lower roof displacement and higher base shear. The larger strain can be attributed to the fact that the energy dissipation in the EBF system is more concentrated than in the CBF system.

		CBF	EBF	UBF
Columns	Story 3	W12x136	W12x152	W12x136
	Story 2	W12x136	W12x152	W12x136
	Story 1	W12x136	W12x152	W12x136
Beams	Roof	W16x67	W14x53	W16x67
	Floor 3	W16x67	W14x53	W16x67
	Floor 2	W16x77	W14x53	W16x77
Braces	Story 3	TS7x7x1/4	W10x60	5 sq. in.
	Story 2	TS7x7x3/8	W10x60	7 sq. in.
	Story 1	TS7x7x1/2	W10x60	8.5 sq. in.

Table 2: Structural Framing for Three Braced Frame Systems Considered

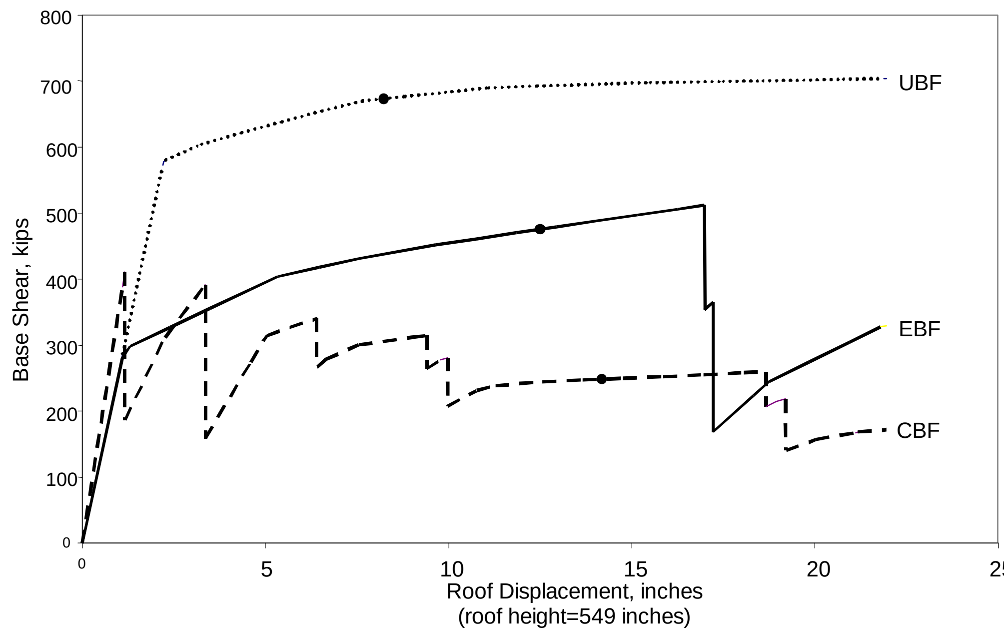

Figure 18. Results from Pushover Analyses of Three Different Braced Frame Systems

Performance Point Information			
System	CBF	EBF	UBF
βeff, Effective damping[a]	34.0%	23.7%	24.2%
Teff, seconds	3.19	2.01	1.29
Roof Displacement, in	14.2	12.5	8.2
Brace axial strain	0.0416	---	0.0090
Link shear strain	---	0.2390	---
V/W, Base shear/weight	0.118	0.227	0.321
Performance Level Achieved per FEMA-273			
System	CBF	EBF	UBF
Performance Level	Collapse Prevention	Collapse Prevention	Life Safety

a) The effective damping reported is primarily based on hysteretic behavior. Only 2% critical damping is assumed for linear elastic behavior

Table 3: Summary of Performance Measures for Three Different Braced Frame Systems

The performance point information can be used to determine what performance level is achieved based on the FEMA-273 provisions. The CBF and EBF systems provide collapse prevention, the lowest required level of performance. However, the UBF system achieves a higher performance level of life safety. Each of these systems could be modified to obtain higher performance levels, but overall, the UBF system achieves a higher performance level with better behavior than the other systems. Consequently, it was approved to be included as an alternate seismic system for this project.

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. The frame design investigated was compared with that of a steel moment-resisting frame previously studied in the FEMA/SAC project. 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 provision 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 (including the 9-story steel moment frame used in the FEMA/SAC project) and to develop design procedures to achieve higher performance levels and optimal frame designs 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.

Finally, a series of design studies in support of the first application of unbonded braces in the United States was described.

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