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APPLICATION OF BUCKLING-RESTRAINED BRACES IN THE UNITED STATES

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ABSTRACT

Buckling-Restrained Braces (BRBs) are well established in Japan, and have been used in more than 250 buildings. It is only in the last four years that they have seen use in the U.S. There are over 30 buildings in the United States underway or already completed that utilize buckling-restrained braces. Tests of large braces have been carried out in support of some projects as required by the regulatory agencies. These are summarized in the paper. The results of recent tests conducted on large capacity braces are described in more details. The paper examines the adequacy of nonlinear models built-in to widely used structural analysis computer programs by comparing analytical results with test results. The comparisons demonstrate that a bilinear force-deformation Wen model is adequate to represent the nonlinear hysteretic behavior of BRBs. To facilitate the use of BRBs a joint effort by professional and steel industry organizations has resulted in a set of recommended provisions for the design of buckling-restrained braced frames. The paper provides a summary of these provions.

1. INTRODUCTION

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

Unbonded Braces™ are one type of buckling-restrained brace, and consist of a yielding steel core confined by mortar within a steel tube. They have been quickly accepted in the U.S., with about 30 projects underway or already completed in the space of about three years.

Although buckling-restrained braced frames as a seismic lateral force-resisting system are not yet defined by any building code in the U.S., significant work has taken place in the development of code-type provisions. A two-year effort to develop design provisions for buckling restrained braced frames, undertaken by a joint Structural Engineers Association of California and American Institute of Steel Construction (SEAOC/AISC) Task Group, has paralleled the growing number of applications of buckling-restrained braces. The development work has produced a set of draft design provisions, called the *Recommended Buckling-Restrained Braced Frame Provisions* (or the *Recommended Provisions*) (Sabelli et al., 2003), which have been developed with the intent of ultimately being incorporated in a future edition of the *AISC Seismic Provisions for Structural Steel Buildings.* The *Recommended Provisions* are first to be published in an upcoming volume of the AISC Engineering Journal. In the interim prior to AISC adoption they have been approved for inclusion in the 2003 update of the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA, 2000). These provisions are not directly part of any building code but are recommendations to those writing and revising codes.

The *Recommended Provisions* recognize buckling-restrained braced frames as a special class of concentrically-braced frames (CBFs). Buckling-restrained braced frames have more ductility and are capable of more effective energy absorption than CBFs. The *Recommended Provisions* are based on the use of buckling-restrained brace designs that are qualified by testing, and are intended to ensure that braces are used only within their proven range of deformation capacity and that yield and fracture modes other than stable brace yielding are precluded at the maximum inelastic drifts corresponding to the design earthquake.

A key aspect of the design requirements for a buckling-restrained brace element is that the design be based on actual brace behavior and actual material properties. Specific requirements are defined related to the prevention of global and local buckling under the maximum design forces and deformations. It is required that the buckling-restraining mechanism limit local and overall buckling without restraining the yielding section from movement though a range of deformation corresponding to 1.5 times the Design Story Drift (which is defined by the governing code). It is recognized that in practice there will be some interaction between the buckling-restraining mechanism and the yielding section, but limits on the extent of interaction are specified to ensure acceptable behavior. Two factors are defined that relate to the actual post-yield force-deformation behavior of a buckling-restrained brace, and these are used in the design of connections and surrounding structural elements. The Compression Strength Adjustment Factor (β) recognizes that buckling-restrained braces typically exhibit higher apparent post-yield forces in compression than in tension, and the Tension Strength Adjustment Factor (ω) includes both the variation of expected material yield strength from nominal yield strength (i.e., specification minimum strength), and strain-hardening that occurs at the design deformation level. Both of these factors are to be determined from tests of actual braces. Connections are required to be designed to remain elastic for the actual maximum post-yield force of the brace and their design shall include consideration of local and overall buckling.

The *Recommended Provisions* are based on the use of buckling-restrained brace designs that are qualified by testing, which is intended to confirm acceptable brace behavior under the required design deformations. The rationale of the testing requirements contained in the *Provisions* is similar to the FEMA/SAC and AISC approach for the testing of steel moment-resisting frame connections, that is, tests must be conducted to confirm acceptable behavior but such tests need not be project-specific, rather prior testing of appropriately similar elements may be used to ‘qualify’ a brace design and concept.

3. TESTING OF UNBONDED BRACES™

To date tests have been done on Unbonded Braces™ in support of nine U.S. projects. The properties of the braces tested in four of these projects are summarized in Table 1. The first tests were conducted at University of California Berkeley during the spring of 1999. The tests were conducted in support of the University of California at Davis Plant and Environmental Sciences Building (Aiken et al., 2000). This was the first project in the U.S. to use buckling-restrained braces. Uniaxial component tests were performed on three different brace sizes. The braces had capacities ranging from 1,217 kN to 2,155 kN. The second set of tests were of representative braces designed for a new Kaiser hospital constructed in the San Francisco Bay Area (Ko et al., 2002). Two identical braces with a capacity of 2,033 kN were tested. The testing program for both series consisted of two phases. First, each brace was subjected to a standard loading protocol consisting of a sequence of cyclic tests with increasing displacement amplitudes. Following these tests each brace was subjected to a seismic loading or a large-deformation, low-cycle fatigue test consisting of fully reversed displacement cycles. Details of these tests can be found in (Black et al., 2002).

The third series of tests summarized in Table 1 was carried out in support of a new laboratory building at the University of California, Berkeley, Campus. The building consists of seven levels of steel framing over a three-level concrete basement, with a seismic lateral force-resisting system of buckling-restrained braces in the upper levels and concrete shear walls at the basement levels. BRBFs were chosen as the seismic lateral force-resisting system for the building because of their large ductility, energy dissipation capability, and also for the ease of repair after a major earthquake, a factor that was regarded as less problematic than for any other type of steel framing or bracing system (Lopez, 2002). These tests differed from the first two series in that the braces were tested in a subassembly to confirm the behavior of Unbonded Braces™ under frame loading conditions was acceptable. In particular, interest focused on the behavior of the braces under frame-induced axial and rotational deformations, the appropriateness of assuming brace performance for frame conditions as determined from uniaxial component tests, and also the behavior of connections under frame lateral deformations. The subassembly tests demonstrated good behavior of the braces. Their hysteretic and elongational behavior appeared not to be influenced by the combined axial and flexural demands associated with loading in a frame configuration (Aiken et al., 2002).

The fourth series of tests summarized in Table 1 consisted of full-scale sub-assembly tests in support of the Kaiser Santa Clara Medical Center Phase II project located in the San Francisco Bay Area. The tests were performed at the Building Research Institute (BRI) Large Size Structural Laboratory in Tsukuba, Japan. The four braces tested comprised two “short” braces, designated B1 and A1, with an overall length of 4,221 mm, and two “long” braces, designated B2 and A2, with an overall length of 7,552 mm. The type B braces (B1 and B2) had a yield force of 3,485 kN, and the type A braces had a yield force of 5,174 kN. Figure 1 shows the overall setup that was used to test the short and long braces. Figure 2 shows the sub-assembly test set-up with a long brace installed. Three hydraulic actuators were used to load the top of the column sub-assembly. A total of three load cells, one included in-line with each actuator, provided direct measurement of the force applied to the brace specimen. The loading protocols for the four specimens are given in Table 2 and Figure 3. Figure 4 shows the axial force-axial displacement response of specimen B1. All specimens tested exhibited extremely stable hysteretic behavior over the entire range of displacement amplitudes, without any degradation in the measured properties.

Table 1. Summary of Unbonded Brace™ Tests for Four U.S. Projects

Series	Specimen	Type	Area mm^2 (in^2)	Yield Length mm (in)	Steel Grade and Yield Stress MPa (ksi)	Yield Force kN (kips)	References
1	99-1	(-)	2,907 (4.5)	3,090 (121.7)	JIS SM490A 418.5 (60.7)	1,217 (273.2)	Aiken et al. (2000) Black et al. (2002)
	99-2	(-)	3,876 (6.0)	2,990 (117.7)		1,622 (364.2)	
	99-3	(+)	5,149 (8.0)	3,450 (135.8)		2,155 (485.6)	
2	00-11	(+)	7,125 (11.04)	3,410 (134.3)	JIS SN400B 285.4 (41.1)	2,033 (453.7)	Ko et al. (2002) Black et al. (2002)
	00-12	(+)	7,125 (11.04)	3,410 (134.3)		2,033 (453.7)	
kN3	CP-1A	(-)	4,084 (6.33)	2,365 (93.1)	JIS SN400B 280 (40.9)	1,786 (259)	Aiken et al. (2002) Uriz et al. (2003)
	CP-1B	(-)	4,084 (6.33)	2,365 (93.1)		1,786 (259)	
	CP-2	(-)	4,084 (6.33)	4,008 (157.8)		1,786 (259)	
	CP-3	(+)	7,542 (11.69)	3,409 (134.2)		3,296 (478)	
4	A-1	(+)	18,080	2,747	JIS SN400B 286-299 (41.5-43.4)	5174 (1163)	SIE (2003)
	A-2	(+)	18,080	6,018		5174 (1163)	
	B-1	(+)	11,650	2,907		3485 (784)	
	B-2	(+)	11,650	6,178		3485 (784)	

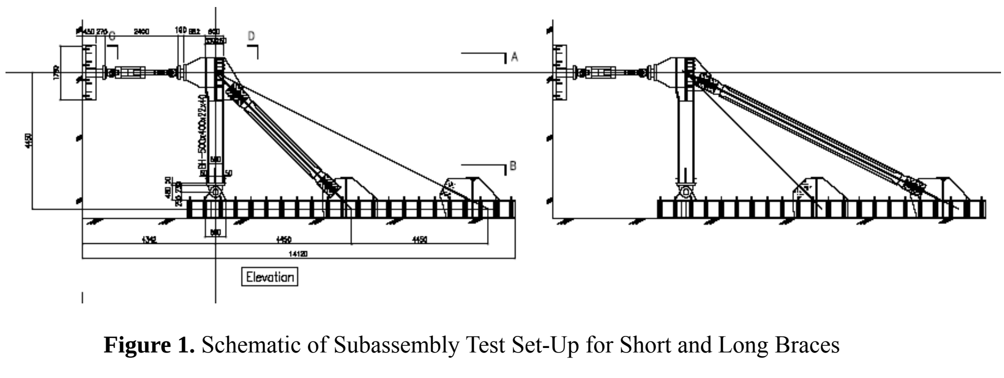

Figure 1. Schematic of Subassembly Test Set-Up for Short and Long Braces

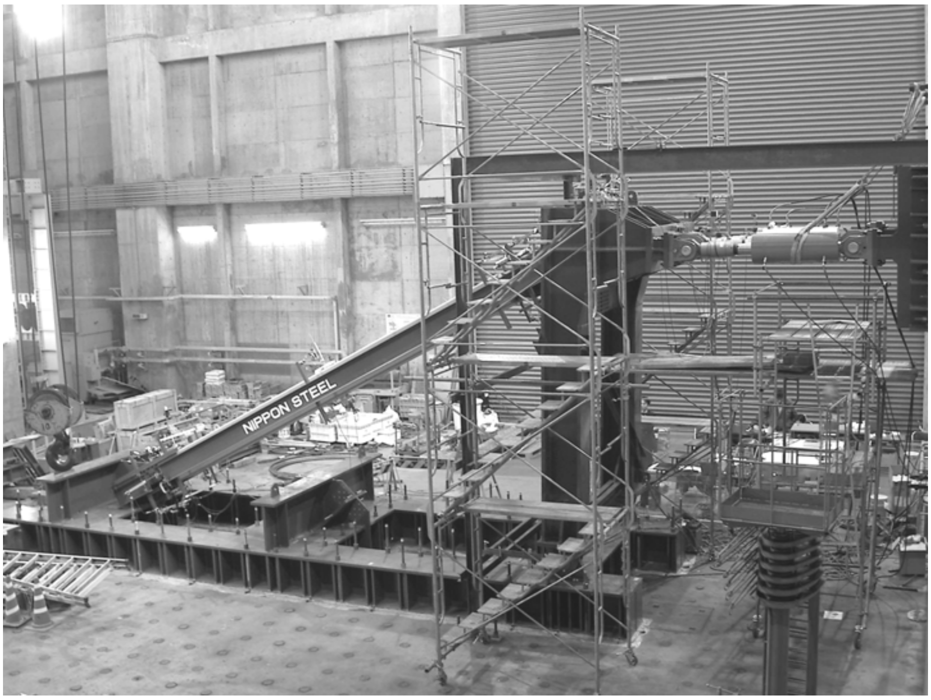

Figure 2. General View of Subassembly Test Set-Up for Long Braces

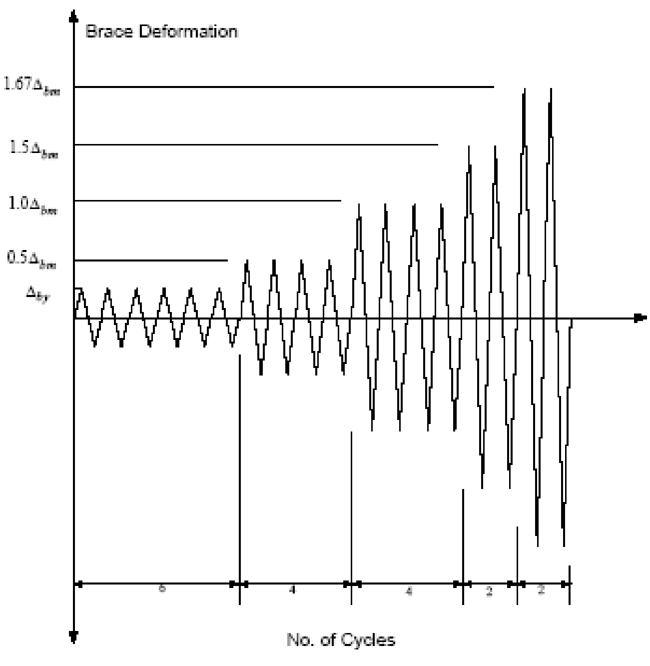

Figure 3. Brace Loading History for Test Series No. 4

Table 2. Summary of Deformation Demands for Test Series No. 4

Load Step	$\pm \Delta_{by}$		$\pm 0.5\Delta_{bm}$		$\pm 1.0\Delta_{bm}$		$\pm 1.5\ \Delta_{bm}$		$\pm 1.67\Delta_{bm}$	
No. of Cycles	6		4		4		2		2	
	Displ. mm	Strain (%)	Displ. mm	Strain (%)	Displ. mm	Strain (%)	Displ. mm	Strain (%)	Displ. mm	Strain (%)
A1	4.6	0.16	24.4	0.88	48.5	1.77	74.9	2.73	81.3	2.96
A2	9.1	0.15	30.7	0.5	60.7	0.98	90.4	1.47	99.6	1.63
B1	4.6	0.16	24.1	0.83	48.3	1.66	72.1	2.48	79.5	2.73
B2	9.4	0.15	31.0	0.50	61.5	1.00	91.2	1.48	101.3	1.64

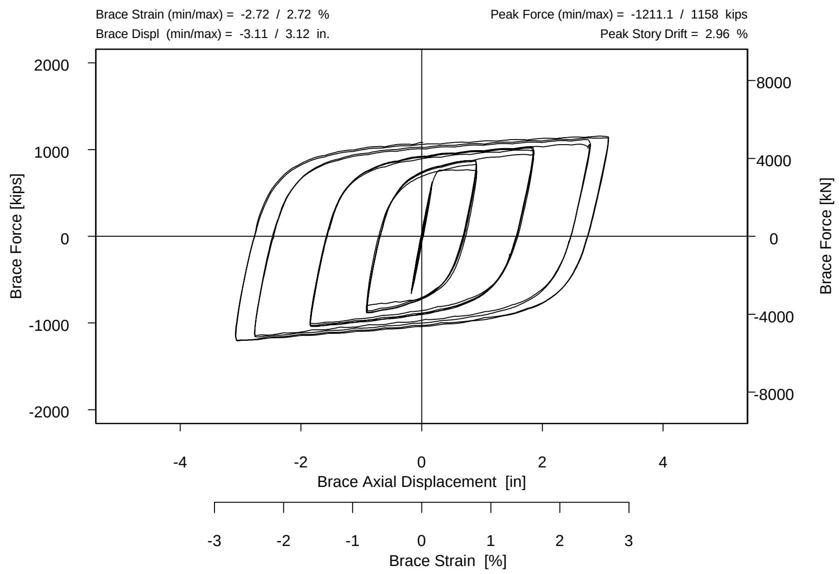

Figure 4. Axial Force-Displacement Relationship for Brace B1, Tests 1 - 5

3. MODELING OF UNBONDED BRACES™

The majority of Unbonded Brace™ designs in the U.S. have been developed using static design methods prescribed by the Uniform Building Code for conventional concentrically braced frames but recognizing and taking advantage of the superior characteristics of buckling-restrained braces. The equivalent lateral force is applied to the structure and structural capacities and drifts are calculated. The lateral force is based on the code prescribed elastic force reduced by a Response Modification Coefficient (R). An R factor of 7, the same as eccentrically braced frames is often used.

Nonlinear dynamic time history analysis of frame systems incorporating BRBFs is straight forward. Most structural analysis computer programs with nonlinear capabilities include a bilinear element suitable for modeling Unbonded Braces™. A comprehensive analytical study of a range of building designs incorporating BRBFs, and subjected to a large suite of earthquakes, was completed (Sabelli, 2001). The research investigated ground motion and structural characteristics that influence the response of concentrically-braced frame buildings, and a major component of the study was an investigation of buckling-restrained braced frames. The analyses of the model buildings were performed using the nonlinear dynamic analysis computer program SNAP-2DX (Rai et al., 1996). The buckling-restrained braces were modeled using a simple truss element with ideal bilinear hysteretic behavior, exhibiting no stiffness or strength degradation.

Commonly used commercial computer programs such as SAP2000 and ETABS (Computers and Structures, 2002) also incorporate special elements, which can be used for performing both static nonlinear push-over analyses as well as dynamic time history analysis. In order to investigate the adequacy of the model in representing the behavior of buckling restrained braces, a simple model is developed in SAP2000 and the results are compared with test results described in section 2. The brace is modeled with two *Link* elements in series. The Link element is a one-dimensional spring placed between two nodes in the structure. The first Link element represents the nonlinear properties of the yielding portion of the brace. This element is based on the hysteretic behavior proposed by Wen (1976). The nonlinear force deformation curve is defined by:

$$f = (\text{ratio})kd+(1-\text{ratio})F_y z$$

where k is the elastic stiffness, ratio is the specified ratio of post-yield stiffness to elastic stiffness, F_y is the yield force and z is an internal hysteretic variable. This variable has a range of $|z| \leq 1$, with the yield surface represented by $|z| = 1$. The initial value of *z* is zero, and it evolves according to the differential equation:

$$\dot{z} = \begin{cases} \dot{d}\left(1-|z|^{\exp}\right) if\ \dot{d}z > 0 \\ \dot{d}\ otherwise \end{cases}$$

where exp is greater than or equal to 1. The second Link element is an elastic spring and represents the non-yielding end connection regions of the brace.

Brace A1 is selected from the four tested braces to be modeled, see Table 1. All properties used in the analysis are based on information available prior to testing. The brace has a yielding cross-sectional area (A_y) of 28 in^2 (180.6 cm^2), yield length (L_y) of 108.1 in (2.747 m). The elastic modulus k is calculated from $A_y L_y/E$, where E is the elastic modulus and is equal to 29,000 ksi (2x10^5 MPa). The elastic stiffness is equal to 7517 kip/in (1.32x10^6 kN/m). Mill certificates of the core plate steel indicated that yield stress was 41.5 ksi (286 MPa) giving a yield force of 1163 kips (5174 kN). The non-yielding ends of the brace have a cross-sectional area of 59.4 in^2 and a length of 58.1 in (1.476 m) giving an elastic stiffness of 29,629 kips/in (5.19 kN/m). This value was assigned to the elastic Link element. The same displacement time history as that imposed in the testing (Figure 5) was imposed on one of the brace nodes. The node representing the other end of the brace was fixed.

The resulting force-deformation loops are compared with the test results in Figure 6. The dashed curve represents the calculated results using the brace properties described above. A post-yield stiffness ratio equal to 0.0325 and an exponent equal to one was used. It can be seen that the calculated results are close to the test results for the first three-displacement levels. For the large displacement loops, the effective stiffness is close to the test results but the calculated energy dissipation area is smaller. The reason for this difference is that the Wen model does not capture strain hardening effects which increase the brace yield force as the displacements are increased. The second dashed curve is calculated using the same model but assuming a yield stress that is 25 percent higher that the initial yield stress. This results in a very good match between the analytical and tested results at higher strain levels. It should be noted that in the analysis of real buildings it is recommended that a model based on the actual yield stress be used since the brace before an earthquake would not have undergone any strain hardening due to yielding. Furthermore, such an approach would produce more conservative results.

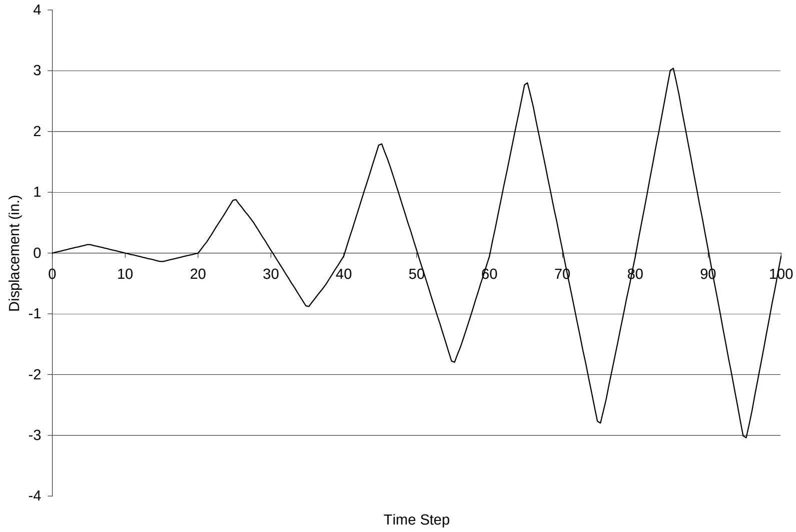

Figure 5. Displacement Loading Function used in SAP2000 Analysis

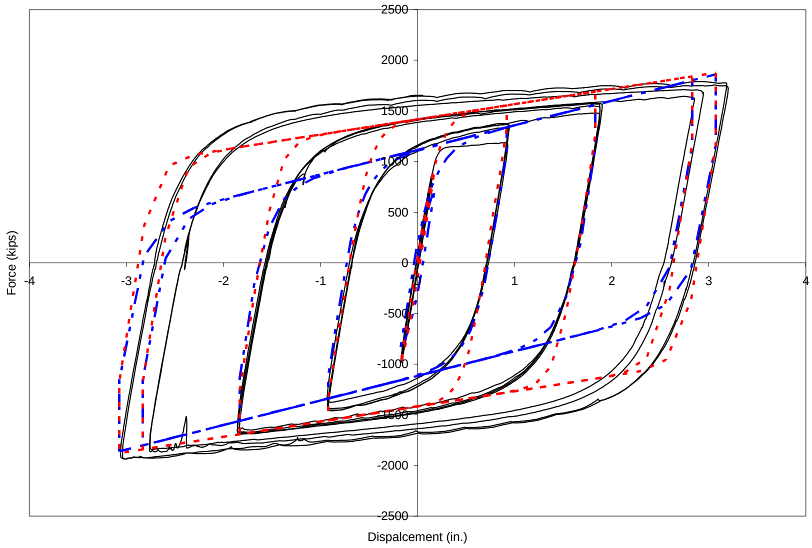

Figure 6. Comparison of Brace A1 Test Hysteresis Loops with SAP2000 Results

A similar model was used to analyze Brace 99-3 of Test Series 1 (Table 1). This brace was subjected to a cyclic loading history similar to that described above. After the cyclic loading test, the brace was subjected to an interstory displacement history calculated for the nonlinear response of a five-story building to the 1994 Sylmar N-S and the 1940 El Centro

N-S ground motions. Figure 7 shows the Sylmar interstory displacement time history that was also applied to the SAP2000 model.

The resulting force-deformation loops are compared with the test results in Figure 8. It can be seen that the computed response compares well with the test results confirming again that the Wen bi-linear model adequately captures the behavior of Unbonded Braces™.

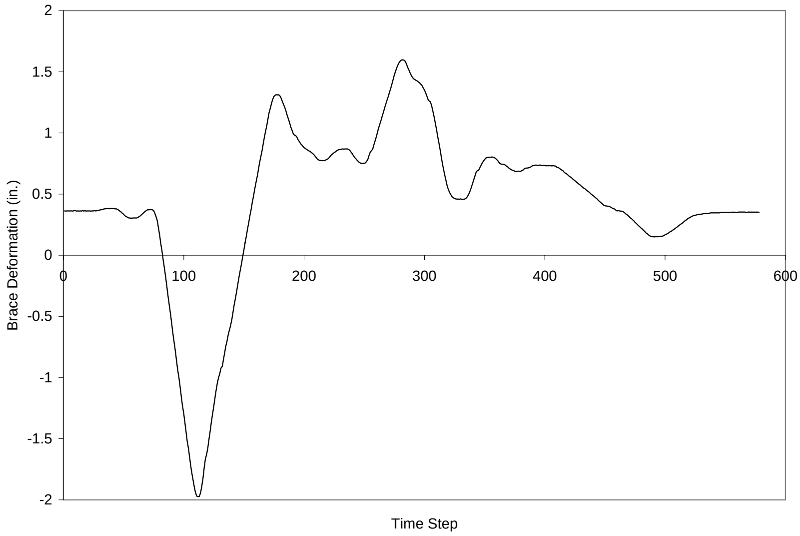

Figure 7. 1994 Sylmar N-S Interstory Displacement History Applied to Brace 99-3

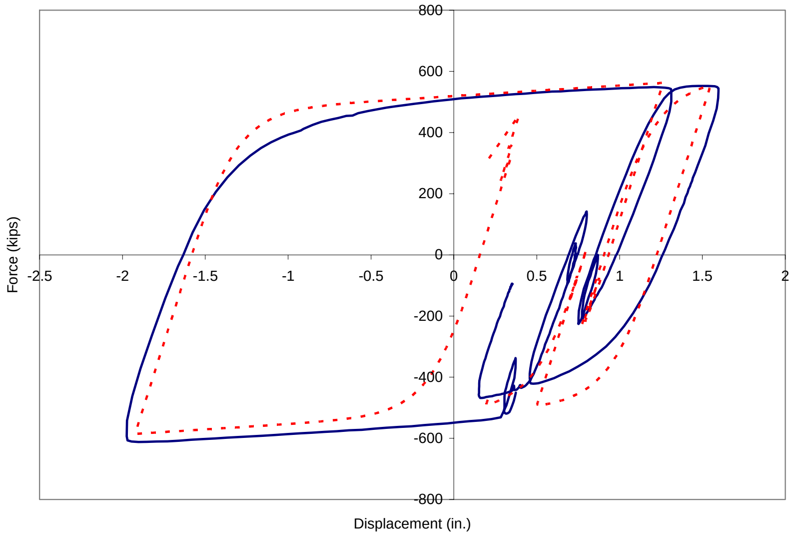

Figure 8. Comparison of Test Hysteretic Behavior with SAP2000 Results, 1994 Sylmar N-S Interstory Displacement History

3. CONCLUSIONS

Buckling-restrained braces have become an acceptable alternative in the U.S. for braced-frame seismic lateral systems, and have already been used in about thirty buildings for both new and retrofit construction. To date nine series of large-scale tests of Unbonded Braces^{TM} have been performed for U.S. projects consisting of uniaxial as well as subassembly tests. These test programs have validated the excellent hysteretic properties of the Unbonded Brace^{TM}. Comparison of analytical results using a Wen bi-linear model with test results showed that available analytical tools in commonly used structural analysis programs satisfactorily represent the Unbonded Brace^{TM} nonlinear behavior and are suitable for design purposes. Furthermore brace analytical models can be completely defined by knowing the brace geometry and yield properties of the core steel material.

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