ANALYTICAL AND EXPERIMENTAL STUDIES ON BUCKLING RESTRAINED BRACED COMPOSITE FRAMES

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SUMMARY

Analytical and experimental studies on the seismic behavior of buckling restrained braced frames (BRBFs) with concrete filled steel tubular (CFT) columns are being conducted at the ATLSS Center, Lehigh University. The objectives of these studies are to investigate the seismic performance of this type of frame, to evaluate existing design criteria, and to calibrate analytical models. The project is sponsored by the National Science Foundation in conjunction with the U.S.-Japan Cooperative Research Program on Composite and Hybrid Structures. A 4-story prototype building was designed with buckling restrained braced frames (BRBFs) as the lateral load resisting system. The columns in this frame are CFT members and the beams are structural steel sections. Design criteria were taken from the IBC 2000 and the AISC/SEAOC Recommended Design Provisions for BRBFs. A 1-bay prototype BRBF, representing one-quarter of the lateral load resisting system in one direction, was extracted from the prototype building for analysis. The analysis program DRAIN-2DX was used to model the prototype frame including material and geometric nonlinearities. The force-deformation relationship used in the buckling restrained brace model incorporates both isotropic and kinematic hardening. Nonlinear time history analyses were conducted using an ensemble of ground motions consisting of natural earthquake records and one artificial earthquake record. The earthquake records were scaled to two different seismic input levels: Design Basis Earthquake (DBE) and Maximum Considered Earthquake (MCE). A statistical summary of the analysis results was developed at both seismic input levels. Performance objectives were defined and used to evaluate the analysis results. The life safety (LS) performance level was the target level for the DBE and the collapse prevention (CP) performance level was the target level for the MCE. Acceptable BRBF behavior was observed at both seismic input levels. A large-scale BRBF experimental investigation is planned. This paper presents the results of the analytical study and briefly summarizes the upcoming experimental program.

Keywords: *structural systems, buildings, structural response, seismic codes and standards, experimental testing, performance-based design.*

INTRODUCTION

Because of the limited ductility and energy dissipation capacity of conventional concentrically braced frame (CBF) systems, significant research effort has gone towards developing new CBF systems with stable hysteretic behavior, significant ductility, and large energy dissipation capacity. One CBF system with improved seismic behavior is the buckling restrained braced frame (BRBF). A buckling restrained brace (BRB) has two basic components: (1) a steel core element that carries the entire brace axial load and (2) a confining or restraining exterior element that prevents the core from buckling in compression and allows it to yield in both tension and compression. Figure 1(a) illustrates a typical BRB configuration in which the steel core is confined within a concrete filled steel tube (CFT). The steel core is debonded from the confining concrete so that the core is able to carry the axial load independent of the CFT. The core is tapered in the center to create a specific region of contained yielding, as shown in Figure 1(b).

The BRB concept was first explored in Japan over 25 years ago, and in the last 15 years, BRBs have been used in nearly 200 Japanese buildings (Black et al. 2002). However, BRBs have been used in North America only

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quite recently. Tremblay et al. (1999) conducted an experimental and analytical research program on BRBFs in preparation for using the system to seismically retrofit a four-story building in Quebec City, Canada. The first application of BRBFs in the United States was in the Plant and Environmental Sciences Building on the campus of the University of California, Davis (Clark et al. 2000). As the interest in using BRBFs in the United States has quickly grown in the last several years, so has the need for an understanding of both member and system behavior, and for rational design guidelines that will lead to BRBFs with superior seismic performance. Sabelli (2001) conducted the most extensive analytical study on BRBFs to date, in which he subjected a series of BRBF designs to an ensemble of ground motions scaled to various seismic input levels and evaluated the results statistically. In addition, numerous experimental studies have been performed on isolated BRBs and BRB subassemblies in North America (e.g., Lopez et al. 2002, Merrit et al. 2003) and Asia (e.g., Watanabe et al. 1988, Tsai et al. 2003). However, investigations of large-scale multi-story frames are still needed so that system behavior, design, and performance can be better understood. The goal of the research program summarized in this paper is to address this knowledge gap through a combined analytical and experimental program.

BRBF DESIGN PROVISIONS

Because BRBFs are a relatively new structural system in the United States, provisions governing their design have not yet been incorporated into building codes. A joint task group formed by the American Institute of Steel Construction (AISC) and the Structural Engineers Association of California (SEAOC) has developed the current *Recommended Provisions for Buckling-Restrained Braced Frames* (AISC/SEAOC 2001). The intent of these provisions is that they will be incorporated into the next edition of the *AISC Seismic Provisions for Structural Steel Buildings* (AISC 2002) and that the proposed BRBF system design parameters will be incorporated into the next edition of the *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA 2000a). Until that time, the draft BRBF provisions are intended to be used in conjunction with the above mentioned design standards.

FRAME DESIGN

The intent of the analytical study presented below was to design a BRBF based on the draft provisions described above and to evaluate the performance of the BRBF design through time history analysis. The goal of this phase of research was not to develop a BRBF frame design with optimal performance, but to use a conventional design approach and then determine if the established performance objectives were met. The prototype structure for this study was a four-story office building assumed to be located in Los Angeles, CA. The building design was based on the International Building Code (IBC) 2000 (ICC 2000). The structure was assumed to be located on Site Class D, stiff soil, with an importance factor, I_E=1.0. The equivalent lateral force procedure was used and the design accelerations were determined using the deterministic limit for the maximum considered earthquake response spectrum. Design lateral loads were determined based on the actual fundamental period of the building instead of the approximate formula presented in the code. As established in the *Recommended Provisions for BRBFs*, the following system design factors were used: response modification coefficient, R=8; system overstrength factor, Ω_0=2; and deflection amplification factor, C_d=5.5.

The prototype building is square in plan, with six equal bays in each direction (Figure 2). The lateral load is resisted by four braced bays in each direction. A schematic elevation of a single braced bay is shown in Figure 3(a). This single braced bay is the prototype frame for the current study and it is assumed to resist one-quarter of the total building lateral load. As noted in Figure 3(a), the building has one level below grade. The lateral loads in the braced frames are assumed to be transferred to the foundation at grade. The columns are concrete filled steel tubes (CFTs) made up of A500 Grade B steel (317 MPa yield strength) and normal strength concrete (34.5 MPa compressive strength). The beams are A992 steel wide flange sections (345 MPa yield strength).

To determine the member design forces, a two-dimensional model of the prototype frame was constructed and an elastic analysis was performed. The brace design forces were determined based on the equivalent lateral forces acting alone while the design forces for the beams and columns were determined using the special load combinations containing the system overstrength factor. This approach, as outlined in the IBC 2000, was used in lieu of a rigorous capacity design procedure. Torsional effects due to the potential eccentricity of the seismic load were included in the design. Figure 3(a) shows the prototype frame member sizes. Connections were designed for the maximum brace forces that develop after strain hardening using the Uniform Force Method (AISC, 2001). Beam-column-brace connections are made using structural tees and double angles, as shown schematically in Figure 3(b). The fundamental period of the prototype building is 0.86 seconds.

ANALYTICAL MODEL

The computer program DRAIN-2DX (Prakash et al. 1993) was used to develop a model of the prototype frame. Fiber elements were used to model the CFT columns and wide flange beams. The segment of the CFT column at the base (ground level) was modeled using the procedure developed by Varma (2000) in order to accurately capture the behavior when a plastic hinge forms. The BRBs were modeled using inelastic truss elements. Rigid offsets were used at the ends of the beams and columns to account for the rigidity of the panel zones. Beam-column elements with large stiffness and strength were used to model the beam-column-brace connection gusset plates. The brace elements, with pinned ends, span between these elements modeling the connection regions. At beam-column-brace connection nodes, the beam end connections were modeled as fixed due to the high stiffness of the large gusset plate connections. At beam-column connection nodes, the beam end connections were modeled as pinned. The basement level of the frame was included in the model in order to accurately capture the foundation flexibility. Gravity loads for the prototype frame were applied to the beams and the columns in the frame. A leaning column was included to model the P-Δ effects due to the building gravity load. Floor masses were lumped at the nodes of the leaning column. Rayleigh damping was used with a viscous damping ratio of 2% assumed in the first and third modes.

A significant aspect of BRBs is their hardening behavior, which includes both isotropic and kinematic components. The inelastic truss element (ITE) in the current version of DRAIN-2DX only models kinematic hardening. To deal with this issue, the ITE in DRAIN-2DX was modified to incorporate isotropic hardening (Fahnestock et al. 2004). The isotropic hardening rule that was implemented includes two primary terms, one related to the maximum total brace deformation, and the other related to the cumulative plastic deformation of the brace. The basic form of the hardening rule is similar to one used by Ricles and Popov (1987) to model isotropic hardening of shear links in EBFs.

The ITE has a bilinear force-deformation relationship with a positive post-yield stiffness (kinematic hardening). This behavior is implemented in DRAIN-2DX by decomposing the ITE into elastic and elastic-plastic components. As a result, the isotropic hardening expression was written to modify the yield force of the elastic-plastic component. Separate isotropic hardening expressions were written for yielding in tension and compression. The following discussion is written in terms of the ITE being in tension (positive), but the principles also apply to the compression side of behavior.

The hardening model was defined so that the yield force is updated when the ITE begins to unload elastically, after yielding in a given direction. This means that the tension (positive) yield force is updated when the ITE is in compression and its incremental deformation changes from negative to positive. As a result, the expression for isotropic hardening of the positive yield force is controlled by the cumulative plastic deformation and the maximum negative deformation. The expression defining the new yield force due to isotropic hardening is:

$$P_{yp,IH} = P_{yp,max} - (P_{yp,max} - P_{yp,o})\left\{\gamma_p \exp\left(-\beta_p\left|\frac{\Delta^-_{max}}{\Delta^-_y}\right|\right) + \left[1-\gamma_p\right]\exp\left(-\alpha_p\left|\frac{\sum\Delta_{plastic}}{\Delta^-_y}\right|\right)\right\} \tag{1}$$

where:
$P_{yp,IH}$ = the new positive yield force of the ITE elastic-plastic component due to isotropic hardening,
$P_{yp,max}$ = the maximum positive yield force of the ITE elastic-plastic component after isotropic hardening,
$P_{yp,o}$ = the initial positive yield force of the ITE elastic-plastic component before isotropic hardening,
α_p = coefficient which controls the positive yield force hardening rate due to cumulative plastic deformation,
β_p = coefficient which controls the positive yield force hardening rate due to maximum deformation,
γ_p = weighting coefficient which controls the relative contributions of the two positive yield force hardening components,
Δ^-_y = negative yield deformation of the ITE elastic-plastic component,
Δ^-_{max} = maximum negative deformation of the ITE elastic-plastic component,
$\sum\Delta_{plastic}$ = cumulative plastic deformation of the ITE elastic-plastic component.

The strain hardening behavior of the model should be based on representative BRB tests. Thus, $P_{yp,o}$ and $P_{yp,max}$ are based on BRB test data, and the coefficients α_p, β_p, and γ_p are determined through regression analysis of BRB test data.

For the present study, regression analysis of BRB test results returned values of zero for both γ_p (defined above) and γ_n (the weighting factor which controls the relative contributions of the two negative yield force hardening components), indicating that the dominant factor in the isotropic hardening of the BRBs studied is cumulative plastic deformation. However, other BRBs may be more accurately modeled by using a combination of the two hardening components. It is also important to note that the cumulative plastic deformation and maximum deformation effects are not entirely uncoupled. As a result, other coefficient values may be calculated depending upon the cyclic loading history of the test that provides the data for the model calibration. However, the goal of the BRB hardening model calibration is not a precise replication of the hardening behavior for a specific cyclic test, but rather a good overall representation of the strength increase of BRBs that comes from isotropic hardening. Figure 4 illustrates the good overall agreement between BRB experimental data and analysis results from the calibrated DRAIN-2DX brace model. The trilinear force-deformation behavior in the BRB analytical model was obtained by using two ITEs in parallel.

GROUND MOTIONS

The natural ground motions used in the present study were selected from the Pacific Earthquake Engineering Center (PEER) database. As described by Rojas (2003), an initial selection process was performed to narrow the set of records to those of interest for time history analysis of steel frame buildings. From this set of 140 records, 11 records were chosen to be scaled to the Design Basis Earthquake (DBE) seismic input level. Table 1 lists characteristic information for the selected records. Records were chosen with a wide variety of spectral shapes (Figure 5) in order to evaluate building performance for a range of natural ground motions. In addition to the natural ground motions described above, one artificial ground motion was also used. This ground motion was generated by Garlock (2002) to be compatible with the IBC 2000 design response spectrum. The scaling procedure used in the present study was based on the recommendations of Sommerville et al. (1997), with slight modifications due to the fact that the recommended procedure includes two ground motion components while the present study only used one component. Scale factors for all earthquake records are listed in Table 1.

PERFORMANCE OBJECTIVES

A prerequisite to evaluating time history analysis results is establishing building performance objectives. Performance objectives relate the seismic input level to the expected seismic performance level. The two seismic input levels considered in design are the design basis earthquake (DBE) and the maximum considered earthquake (MCE). The commentary to the NEHRP *Recommended Provisions* (FEMA 2000b) describes four performance levels: operational, immediate occupancy, life safety, and near collapse. For this study of BRBFs, the life safety (LS) performance level was chosen as the target performance level for the DBE and the near collapse, or collapse prevention (CP), performance level was chosen as the target performance level for the MCE. However, the specific design criteria defined below for the prototype BRBF in terms of structural limit states and response quantities actually provide performance that exceeds the LS and CP performance levels as defined by NEHRP.

In order to evaluate the time history analysis results for the prototype BRBF, specific structural limit states and system response limits were defined for each target performance level. Table 2 lists the primary limit states and response limits that were used to evaluate the prototype BRBF performance. These limit states and response limits are based upon previous analytical studies of BRBFs and experimental results from frame component tests. The deterministic nature of these response limits and the merit of using them directly as design criteria are discussed at the end of this section.

The BRB limit states were established based on BRB tests. BRBs have demonstrated tremendous cumulative ductility capacity, with one test reaching a cumulative ductility of 1700. Even at this extreme cumulative ductility level, the BRB core did not fracture and the test was stopped in the interest of time (Merrit et al. 2003). Thus, the MCE response limit of 400 is quite conservative. The cumulative ductility response limits of 200 and 400 for the DBE and the MCE, respectively, were chosen because they significantly exceed previously reported BRB demands but are much less than expected BRB capacity. While the cumulative ductility capacity of BRBs is well established, the maximum ductility capacity is not as clearly defined. Almost all experimental programs on isolated BRBs have been conducted to maximum ductility demands of 15. Although experimental data beyond this ductility level is almost nonexistent, the excellent BRB performance that has been observed in past experimental programs coupled with knowledge about the mechanics of BRB behavior indicate that BRBs should be capable of sustaining maximum ductility levels much greater than 15. For this reason, the MCE limit on maximum ductility demand has been set at 25, a level that still needs to be verified experimentally. Maximum ductility demands of this magnitude were observed in a previous analytical study by Sabelli (2001) where he reported a maximum ductility demand of 23.2 in a 6-story BRBF subjected to a DBE level ground motion.

The CFT column response limits were determined by examining the results from CFT beam-column subassembly tests (Fujimoto et al. 1996). The tests indicate that CFT beam-columns with b/t ratios up to 35 and constant axial load levels of $0.4P_o$ (where P_o is the axial capacity of the column) can sustain cyclic loading up to 1% plastic rotation without strength degradation. In addition, when plastic rotations of 3% were observed in the tests, the beam-columns had not fractured, their axial load capacity had not degraded, and they were able to sustain at least 60% of the maximum moment that had been achieved earlier in the test. The drift limits were established by evaluating analysis results from Sabelli (2001). Design criteria for the drift limits were based on the mean drifts reported by Sabelli for the DBE and the MCE.

In performance-based design, a significant task is determining if the performance objectives have been met. The approach presented here compares maximum frame response quantities and a statistical evaluation of frame response quantities to deterministic limits. While this is a commonly used approach, for example see Garlock (2002) and Rojas (2003), and it provides a reasonable framework for response evaluation, it is not rigorous from a structural reliability perspective. A more rational approach would be to compare statistical evaluations of frame response from analytical studies – these are structural demands – with statistical evaluations of frame structural capacities. This type of reliability-based approach accounts for scatter and uncertainty in demand and capacity data and allows for global and local probabilities of failure to be determined. A reliability-based design framework similar to the load and resistance factor design approach (AISC 2001) could be established. Demand magnification and capacity reduction factors could be calculated statistically based on analysis results, test data, and target reliability indices. Conversely, statistical evaluations of analysis results and test data could be used to calculate the reliability index for a structure subjected to a defined level of seismic input. These reliability-based evaluations are a necessary component of a robust performance-based design approach and they will be investigated in more detail as the current BRBF research program progresses.

TIME HISTORY ANALYSIS

An ensemble of 12 ground motions was used for time history analysis of the prototype BRBF. All 12 records were used for the DBE input level analysis while 6 records were used for the MCE input level analysis. Since the ground motions were selected to represent a wide range of earthquake events, the results are used to show the expected BRBF performance in a statistical sense. In addition, specific maximum response quantities are highlighted to illustrate maximum demands for both seismic input levels.

Table 3 provides a statistical summary of the prototype BRBF response to the DBE and MCE ground motion ensembles in terms of median and 85th percentile quantities. Comparison with the design criteria (Table 2) shows that the 85th percentile values for all response quantities, except for story drift at the DBE level and roof drift at both seismic input levels, are less than the design objective values. The 85th percentile response quantities that are above the design objectives are only slightly so. The IBC 2000 states that when seven or more time history analyses are performed, the average response parameters may be used for design. Based on this guideline, evaluation of response at the DBE level (the design level referred to in the IBC 2000) may be done using the median values. All of the median DBE level response quantities meet the design objectives with a considerable reserve margin.

Tables 4 and 5 list maximum response quantities for the DBE and MCE ground motion ensembles, respectively. Maximum values for each response quantity are noted with bold typeface. The maximum roof drift was 0.018 for the DBE level ground motions and 0.031 for the MCE level ground motions. The maximum base plastic rotations for DBE and MCE ground motion ensembles were 0.0088 and 0.0280, respectively. These rotation levels meet the design criteria established in the previous section.

Figure 6 shows story drift and maximum brace ductility demand envelopes for the DBE and MCE level time history analyses. Because of the kinematics of a BRBF, the maximum brace ductility demand envelope is similar in shape to the story drift envelope. The tendency for deformation to concentrate in the lower stories is noticeable, although no pronounced soft story behavior was observed even when the frame was subjected to MCE level ground motions. The brace properties may be adjusted in an effort to "tune" the structure and achieve a more uniform distribution of deformation and ductility demands. However, there is significant uncertainty in this process since the structural demands depend upon: (1) structural properties, (2) ground motion characteristics, and (3) the interaction of (1) and (2). While further research in this area is warranted, the basic observation from the time history analyses summarized here is that the equivalent lateral force design approach produces a reasonable design with acceptable overall BRBF performance.

The most significant response quantities from the time history analyses are the BRB ductility demands. The two types of ductility demand are cumulative and maximum. The 85th percentile cumulative ductility demands meet the design objectives with significant reserve margin. The maximum cumulative brace ductility demands observed for the DBE and MCE levels were 99 and 171, respectively. It is interesting to note that the artificial ground motion used in this study was the most demanding in terms of cumulative ductility demand, despite the fact that it was not the most demanding for any of the other major response quantities. This fact indicates that the artificial record used in the present study may be useful for determining upper bounds on BRB cumulative ductility demands. However, cumulative ductility demand should rarely, if ever, be the controlling factor in the design of a BRB because of the large cumulative ductility capacity that this type of brace has.

While the braces in the prototype BRBF have significant reserve cumulative ductility capacity, the maximum ductility demands exceed the limits that have been explored in tests. As indicated in Tables 4 and 5, the maximum ductility demand for the DBE level was 15.8 and the maximum ductility demand for the MCE level was 25.6. These maximum values exceed the design criteria established in the previous section, although the 85th percentile maximum brace ductility demands (Table 3) meet the design criteria. The maximum ductility demand under the MCE level exceeds the capacity that has been experimentally verified, highlighting the need for further experimental programs studying this issue.

In addition to comparing the prototype frame performance to the design objectives, another important issue is examination of the design parameters suggested in the *Recommended Provisions for BRBFs*. Based on the favorable performance of the prototype BRBF as described in this paper, the values of the response modification coefficient and the system overstrength factor, R=8 and Ω_0=2, seem appropriate. However, the recommended value for the deflection amplification factor, C_d=5.5, appears open to further consideration. Based on the results of the present study, this value of C_d seems to be low. Figure 7 shows the envelope floor displacement values for the DBE ensemble of time history analyses along with the amplified elastic displacements using C_d=5.5. As illustrated, the amplified elastic displacements are significantly below the median floor displacement envelope. When the displacement amplification factor is set equal to the response modification coefficient (C_d=R=8), the prediction of inelastic displacements is much more realistic. It is recognized that the probable intent of the writers of the *Recommended Provisions for BRBFs* was to propose system parameters similar to other ductile earthquake resistant building systems, such as MRFs or EBFs. Proposing a C_d value for BRBFs exceeding those currently used for other systems with similar performance could penalize the BRBF system in comparison to the other systems. However, codified values of C_d for conventional earthquake resistant building systems often do not accurately predict inelastic displacements either, pointing to the fact that use of the C_d factor needs to be addressed in a more general forum. This issue is mentioned here to highlight the need for further discussion.

EXPERIMENTAL PROGRAM

The frame which will be tested in the ATLSS Laboratory at Lehigh University is a 3/5 scale model of the prototype BRBF. The test frame is four stories tall with a modified basement level. The experimental investigation will execute a series of pseudo-static tests on the test frame using a simulated seismic loading that consists of imposed lateral displacement histories at the first through fourth floor levels. The floor level displacement histories will be selected from the nonlinear time history analysis results. The series of pseudo-static tests for the frame will generate structural response ranging from the serviceability limit state to the collapse limit state, resulting in test data on the stiffness, strength, ductility, and collapse mechanism of the structure. The lateral stiffness and dynamic characteristics of the test frame will be determined before and after the pseudo-static tests. The results from the experimental investigation will be used to verify analytical studies and to calibrate the analytical model of the BRBF. The experimental program will generate new information about the behavior and performance of multi-story BRBFs. In addition, the maximum ductility capacity of BRBs will be explored as their behavior and performance will be monitored during the frame tests.

SUMMARY AND CONCLUSIONS

While the BRBF described in this paper was designed to meet life safety and collapse prevention performance levels at the DBE and MCE seismic input levels, respectively, the performance of the frame exceeded these performance levels. Although the performance of the prototype BRBF under DBE level ground motions cannot be categorized as immediate occupancy performance, it is appreciably better than the life safety performance level. Roof and story drifts were kept to acceptable levels and residual drift of the frame was minimized, indicating that repair of the frame after a DBE level earthquake should be feasible and economical. The BRBs possess the required ductility capacity to withstand a DBE level earthquake and to remain in service post-earthquake. Similarly, the performance of the prototype BRBF when subjected to MCE level ground motions was much better than the targeted collapse prevention performance level and approached the life safety performance level.

One of the most significant issues related to ensuring good performance of BRBFs under earthquake loading is fully understanding the limits of their ductility capacity. While BRBs have been shown to have excellent cumulative ductility capacity, enough to withstand multiple severe earthquakes, further research is required to expand the knowledge base regarding the maximum ductility capacity of BRBs. Time history analysis results for MCE level ground motions indicate maximum brace ductility demands that are higher than the BRB ductility capacities that have been explored in experimental studies. While the behavior of BRBs that has been demonstrated experimentally seems to indicate that much higher maximum ductility demands can be accommodated without core fracture, this assumption should be explored through further testing. Previous analytical and experimental studies on BRBFs have demonstrated their excellent potential for use in earthquake resistant design of structures. Extended knowledge of BRB behavior, obtained through the experimental program that is part of the research project described in this paper and through other experimental programs, will more robustly qualify the system for use in regions of high seismicity.

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Table 1 – Ground Motion Data

Earthquake	Year	Station and Component	M	R (km)	PGA (g)	Scale Factors	
						DBE	MCE
Chi-Chi	1999	CHY036, 270	7.1	20.4	0.294	1.04	1.55
Duzce	1999	Bolu, 000	7.1	17.6	0.728	1.08	1.62
Kocaeli	1999	Iznik, 090	7.4	31.8	0.136	2.39	-
Loma Prieta	1989	Gilroy Array #3, 090	7	14.4	0.367	1.50	-
Loma Prieta	1989	Hollister-Diff Array, 165	7	25.8	0.269	1.63	-
Loma Prieta	1989	Hollister-South&Pine, 000	7	28.8	0.371	0.95	1.42
Loma Prieta	1989	Sunnyvale Colton Avenue, 360	7	28.8	0.209	1.69	-
Northridge	1994	Canyon Country-W Lost Canyon, 000	6.7	13.3	0.410	1.63	-
Northridge	1994	Canoga Park-Topanga Canyon, 196	6.7	34.2	0.420	1.28	1.92
Northridge	1994	Northridge-Saticoy, 180	6.7	13.3	0.477	0.89	1.33
Superstition Hills	1987	Westmoreland Fire Station, 180	6.6	13.3	0.211	1.72	-
Artificial	-	-	-	-	0.399	1.00	1.50

Table 2 – BRBF Response Limits

Element	Limit State/Response Quantity	DBE Limit	MCE Limit	Comments
Brace	Core Yielding	OK	OK	
	Core Fracture	No	No	Evaluated from cumulative ductility demand
	Maximum Ductility Demand	15	25	
	Cumulative Ductility Demand	200	400	
WF Beam	Yielding	No	OK	
CFT Column	Local Buckling	No	OK	Permitted @ column base only
	Steel Yielding	OK	OK	Permitted @ column base only
	Plastic Base Rotation	0.01	0.03	
Drift	Maximum Roof Drift	0.015	0.03	
	Maximum Interstory Drift	0.02	0.04	

Table 3 – Statistical Evaluation of Maximum Response Quantities

Response Quantity	Seismic Input Level	Median	85th Percentile
Roof Drift	DBE	0.0135	0.0168
	MCE	0.0243	0.0310
Residual Roof Drift	DBE	0.0021	0.0079
	MCE	0.0092	0.0155
Story Drift	DBE	0.0167	0.0214
	MCE	0.0294	0.0395
Residual Story Drift	DBE	0.0036	0.0104
	MCE	0.0116	0.0185
Column Base Plastic Rotation	DBE	0.0012	0.0031
	MCE	0.0103	0.0239
Cumulative Brace Ductility Demand	DBE	66	80
	MCE	114	151
Maximum Brace Ductility Demand	DBE	10.3	12.9
	MCE	17.4	23.3

Table 4 – Maximum Response Quantities for DBE Level Time History Analyses

	Response Quantity						
Ground Motion	Roof Drift	Residual Roof Drift	Story Drift	Residual Story Drift	Column Base Plastic Rotation	Cumulative Brace Ductility Demand	Max. Brace Ductility Demand
Chi-Chi	0.009	0.0008	0.011	0.0020	0.0004	52	9.0
Duzce	0.012	0.0034	0.016	0.0039	0.0006	66	10.0
Kocali	0.014	0.0019	0.019	0.0043	0.0017	78	11.4
Loma Prieta, Gilroy	**0.018**	**0.0100**	**0.026**	**0.0134**	**0.0088**	39	**15.8**
Loma Prieta, Hollister DA	0.013	0.0015	0.015	0.0017	0.0007	43	9.4
Loma Prieta, Hollister S&P	0.014	0.0018	0.017	0.0023	0.0010	53	10.7
Loma Prieta, Sunnyvale	0.011	0.0031	0.016	0.0036	0.0012	41	9.8
Northridge, Canyon	**0.018**	0.0074	0.021	0.0103	0.0023	72	12.5
Northridge, Canoga Park	0.013	0.0024	0.018	0.0036	0.0016	83	11.0
Northridge, Saticoy	0.016	0.0011	0.023	0.0013	0.0045	77	13.6
Superstition Hills	0.013	0.0008	0.016	0.0013	0.0011	67	9.9
Artificial	0.014	0.0090	0.016	0.0108	0.0009	**99**	9.9

Table 5 – Maximum Response Quantities for MCE Level Time History Analyses

	Response Quantity						
Ground Motion	Roof Drift	Residual Roof Drift	Story Drift	Residual Story Drift	Column Base Plastic Rotation	Cumulative Brace Ductility Demand	Max. Brace Ductility Demand
Chi-Chi	0.013	0.0000	0.018	0.0017	0.0016	89	14.8
Duzce	0.017	0.0062	0.023	0.0075	0.0053	98	14.0
Loma Prieta, Hollister S&P	0.028	0.0099	0.034	0.0124	0.0154	87	20.0
Northridge, Canoga Park	**0.031**	**0.0175**	**0.044**	**0.0218**	**0.0280**	145	**25.6**
Northridge, Saticoy	**0.031**	0.0086	0.038	0.0109	0.0226	130	22.5
Artificial	0.021	0.0149	0.025	0.0174	0.0051	**171**	14.5

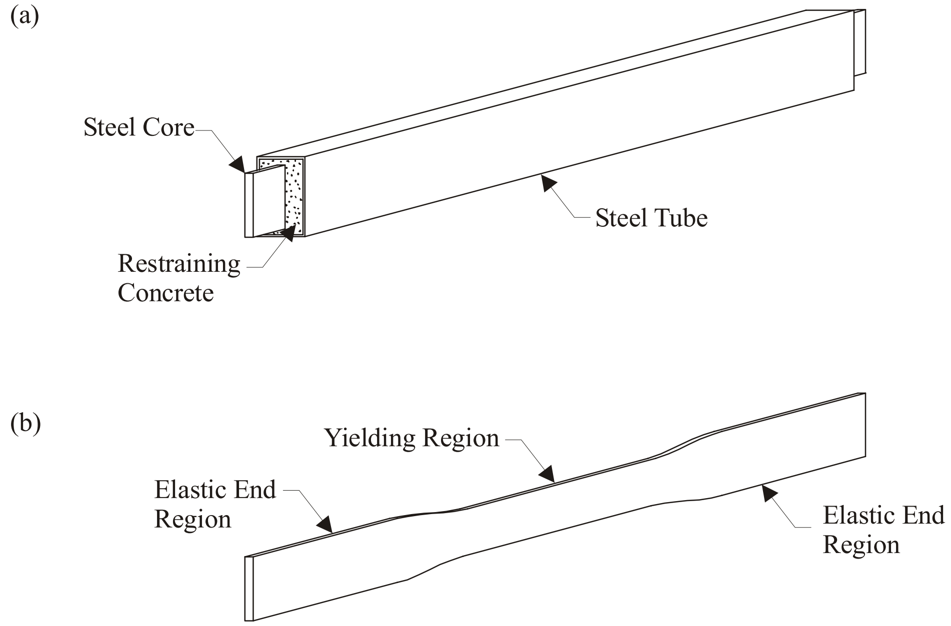

Figure 1 – Schematic of a Buckling Restrained Brace: (a) Complete Brace; (b) Steel Core Detail

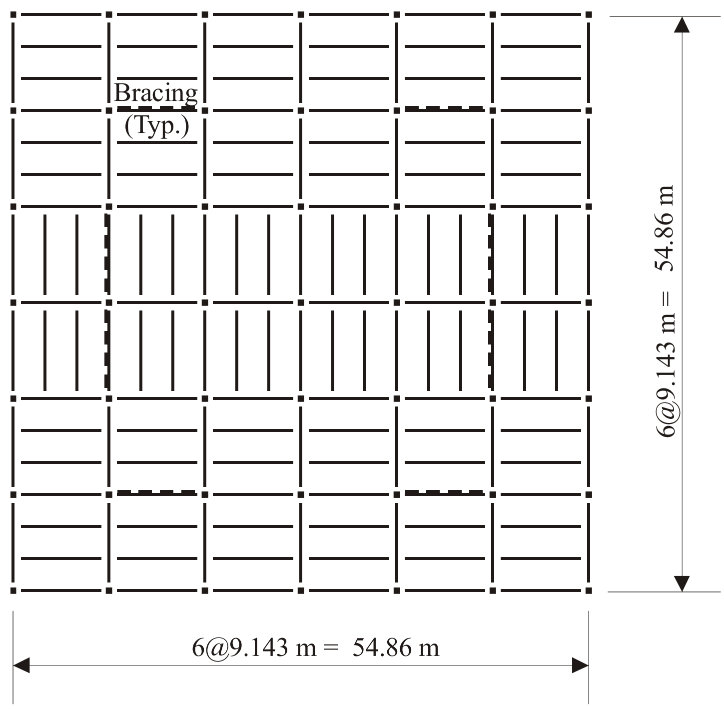

Figure 2 – Plan of Prototype Building

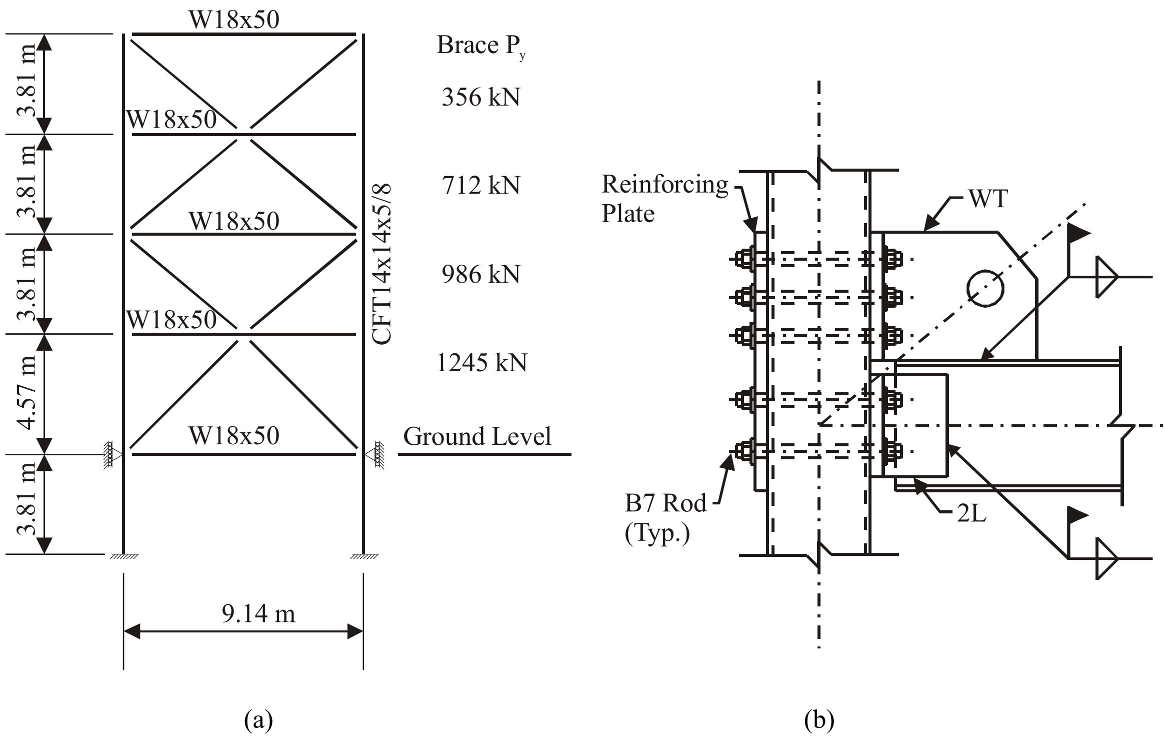

Figure 3 – Prototype Frame: (a) Elevation; (b) Schematic Connection Detail

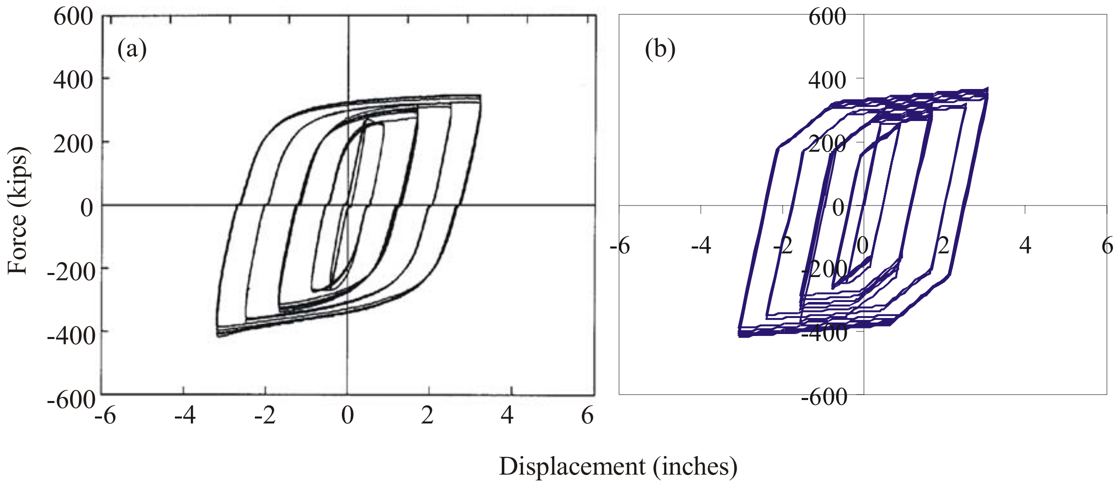

Figure 4 – Buckling Restrained Brace Cyclic Behavior: (a) Experimental Data for Star Seismic BRB (adapted from Merrit et al. 2003); (b) DRAIN-2DX BRB Analytical Model

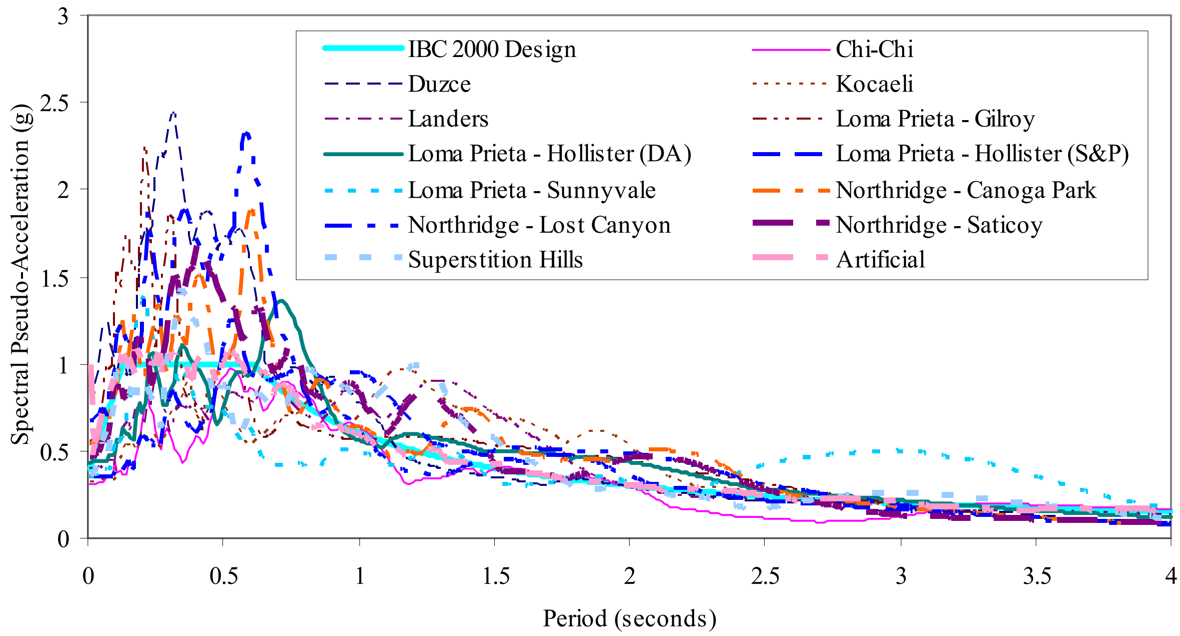

Figure 5 – Pseudo-Acceleration Response Spectra for Ground Motions Used in Analyses (Scaled to DBE)

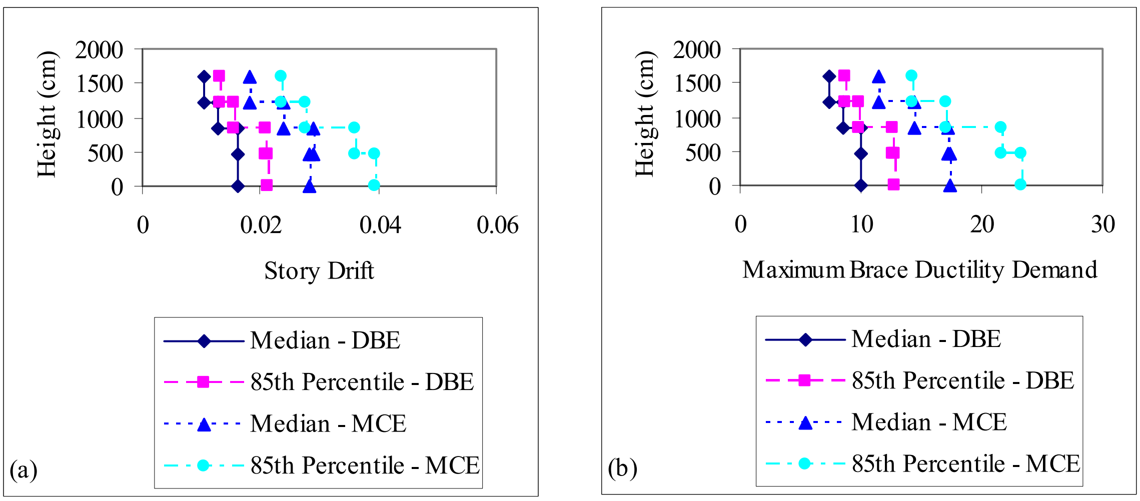

Figure 6 – Time History Response Envelopes Plotted Over the Frame Height: (a) Story Drift; (b) Maximum Brace Ductility Demand

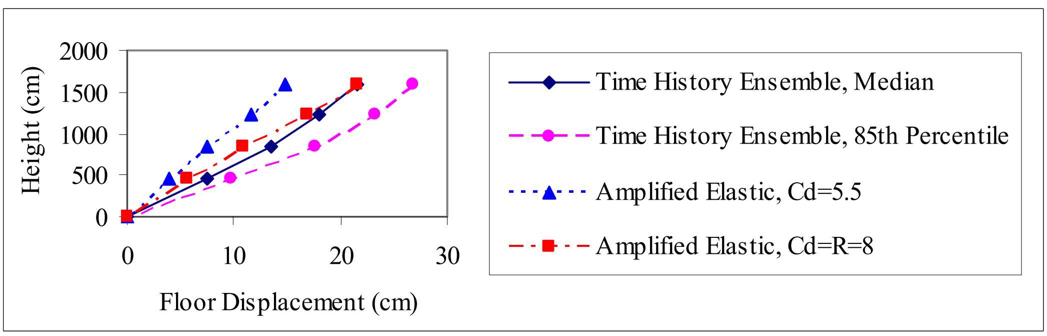

Figure 7 – Envelope of Absolute Maximum Floor Displacements for DBE Time History Analysis Ensemble