Seismic Demands on Steel Braced Frame Buildings with Buckling-Restrained Braces

by

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Abstract

This paper highlights research being conducted to identify ground motion and structural characteristics that control the response of concentrically braced frames, and to identify improved design procedures and code provisions. The focus of this paper is on the seismic response of three and six story concentrically braced frames utilizing buckling-restrained braces. A brief discussion is provided regarding the mechanical properties of such braces and the benefit of their use. Results of detailed nonlinear dynamic analyses are then examined for specific cases as well as statistically for several suites of ground motions to characterize the effect on key response parameters of various structural configurations and proportions.

Introduction

Steel moment-resisting frames are susceptible to large lateral displacements during severe earthquake ground motions, and require special attention to limit damage to nonstructural elements as well as to avoid problems associated with P-Δ effects and brittle or ductile fracture of beam to column connections [FEMA, 2000]. As a consequence, engineers in the US have increasingly turned to concentrically braced steel frames as an economical means for resisting earthquake loads. However, damage to concentrically braced frames in past earthquakes, such as the 1985 Mexico [Osteraas, 1989], 1989 Loma Prieta [Kim, 1992], 1994 Northridge [Tremblay, 1995; Krawinkler, 1996], and 1995 Hyogo-ken Nanbu [AIJ/Kinki Branch Steel Committee, 1995; Hisatoku, 1995; Tremblay, 1996] earthquakes, raises concerns about the ultimate deformation capacity of this class of structure.

Individual braces often possess only limited ductility capacity under cyclic loading [Tang, 1989]. Brace hysteretic behavior is unsymmetric in tension and compression, and typically exhibits substantial strength deterioration when loaded monotonically in compression or cyclically. Because of this complex behavior, actual distributions of internal forces and deformations often differ substantially from those predicted using conventional design methods [see, for example, Jain, 1979 and Khatib, 1987]. Design simplifications and practical considerations often result in the braces selected for some stories being far stronger than required, while braces in other stories have capacities very close to design targets. This variation in story capacity, together with potential strength losses when some braces buckle prior to others, tend to concentrate earthquake damage a few "weak" stories. Such damage concentrations place even greater burdens on the limited ductility capacities of conventional braces and their connections. It has also been noted that lateral buckling of braces may cause substantial damage to adjacent nonstructural elements.

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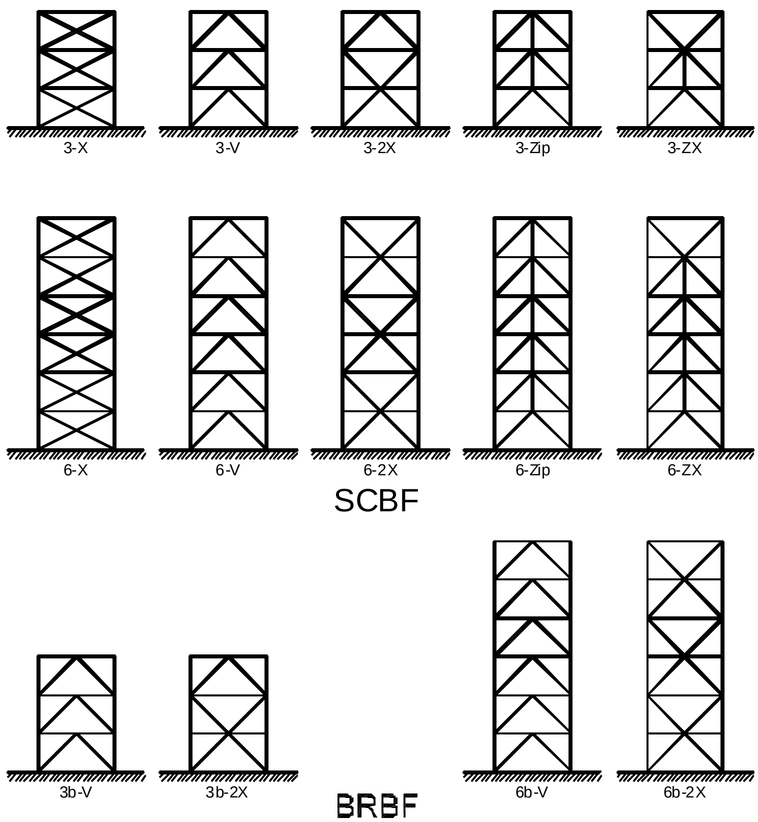
Prompted by these observations and concerns, seismic design requirements for braced frames have changed considerably during the 1990s, and the concept of special concentric braced frames has been introduced [AISC, 1997; ICBO, 1997]. Considerable research has also been initiated improve the performance of concentrically braced frames through the introduction of new structural configurations [see, for example, Khatib, 1987] or the use of special braces, including those utilizing composite action [Liu, 1987], metallic yielding [Watanabe, 1992; Kamura, 2000; and others], high performance materials [Ohi, 2001], friction and viscous damping [see, for example, Aiken, 1996]. During the past decade, there have also been parallel advances in research related to characterizing the seismic hazard at a site, simulating seismic response, and theories for characterizing seismic performance in probabilistic terms. As such, a review of the overall seismic performance characteristics of concentrically braced frames designed to current standards is timely.

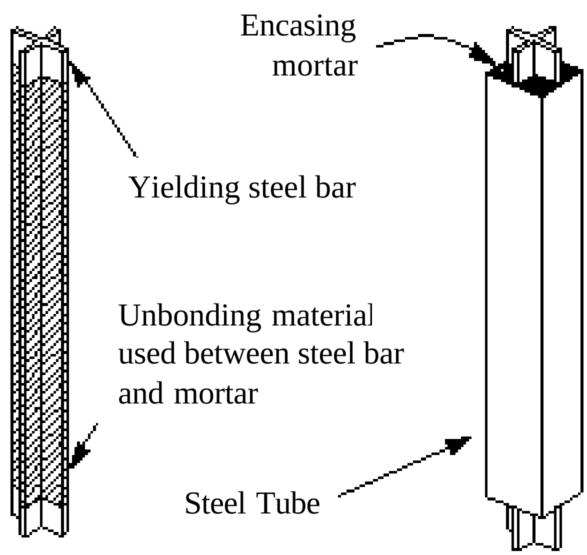


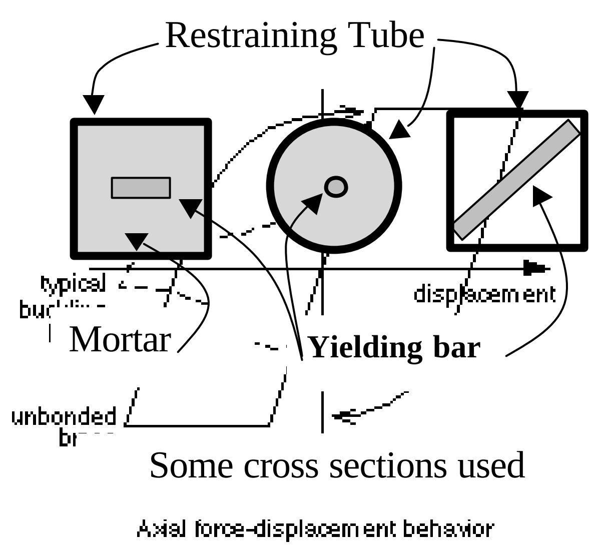
The goal of the overall project described in this paper is to investigate the system level performance of concentrically braced buildings subjected to seismic loads with the intention of understanding the structural and ground motion characteristics that control behavior, and to assess and, where necessary, propose improved design and analysis procedures. A series of nonlinear dynamic analyses has been carried out examining the behavior of concentrically braced frames having conventional braces, high performance hysteretic braces, and visco-elastic dampers. Some of the basic configurations being studied are shown in Figure 1. This paper highlights results obtained for frames utilizing buckling-restrained braces.

Performance-Based Assessment of Braced Frames

In the development of new guidelines for steel moment-resisting frames following the Northridge earthquake, the US Federal Emergency Management Agency adopted a performance-based seismic-resistant design approach [FEMA, 2000]. In this methodology, performance levels are stipulated in terms of the performance goal (the degree of damage) and the seismic hazard level (severity) for which the structure is expected to attain this goal. A key advance in the new FEMA methodology is that uncertainties and randomness in the seismic hazard, structural response, analytical procedures and modeling, and system and member level capacities are accounted for explicitly. Based on this reliability framework, the methodology quantifies the confidence that the structure will not exceed the targeted performance level. In the case of new construction, emphasis is placed on life safety and collapse prevention (though the method provides for voluntary consideration of other performance levels, such as continued occupancy). Based on calibrations to current design practices and expert opinion, FEMA has suggested that new steel moment frames be able to attain for a seismic hazard corresponding to a 2% probability of exceedence in a 50 year time period, at least a 90% confidence of avoiding behavior modes that would jeopardize global stability of the structure, and a 50% confidence of avoiding local collapse modes.

This performance-based evaluation framework permits comparison of the seismic performance of different types of structural system on a consistent basis, and the development of new design provisions that would provide uniform levels of reliability for different structural systems, design and analysis methods, seismic hazards, etc. To undertake such a study, the seismic demands need to be first quantified for various hazard levels and these are then compared with capacities. In this paper, some of the structural and ground motion characteristics affecting seismic demands of braced frames having buckling-restrained braces are examined. Future publications will address issues related to the capacity assessment of such frames and the performance-based design of concentrically braced frames in general.





Buckling-Restrained Braces

Since many of the potential performance difficulties with conventional concentrically braced frames rise from the difference between the tensile and compression capacity of the brace, and the degradation of brace capacity under compressive and cyclic loading, considerable research has been devoted to development of braces that exhibit more ideal elasto-plastic behavior. One means of achieving this is through metallic yielding, where buckling in compression is restrained by an external mechanism. A number of approaches to accomplish this have been suggested (see Fig. 2) including enclosing a ductile metal (usually steel) core (rectangular

Figure 2 Some schematic details used for buckling restrained braces [after Clark, 2000]

or cruciform plates, circular rods, etc.) in a continuous concrete filled tube, within a continuous steel tube, a tube with intermittent stiffening fins, and so on. The assembly is detailed so that the central yielding core can deform longitudinally independent from the mechanism that restrains lateral and local buckling. Through appropriate selection of the strength of the material, and the areas and lengths of the portions of the core that are expected to remain elastic and to yield, a wide range of brace stiffnesses and strengths can be attained. Since lateral and local buckling behavior modes are restrained, large inelastic capacities are attainable. Theoretically based methods have been developed to design the restraining media [Yoshida, 1999; Yoshida, 2000]. Provisions have been developed in draft form [SEAOC, 2001] for design, specification and testing of buckling-restrained braces to help insure braces meet performance expectations.

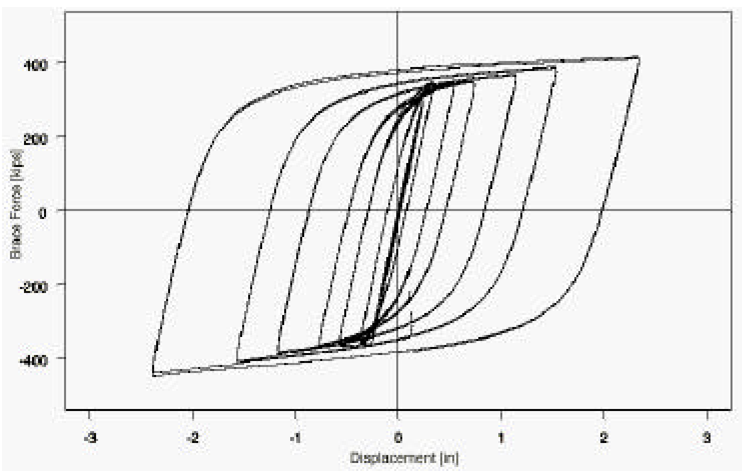


Fig. 3 Axial Force-Displacement Plot for Buckling Restrained Brace with Steel Core Unbonded from Mortar Filled Steel Tube [Clark, 2000]

The inelastic cyclic behavior of several types of buckling-restrained braces have been reported [see, for example, Watanabe, 1989; Clark, 2000, Iwata, 2000 and Kamura, 2000]. These tests typically (see Fig. 3) result in hysteretic loops having nearly ideal bilinear hysteretic shapes, with moderate kinematic and isotropic hardening evident. Interestingly, the difference between the tensile and compressive strength of steel results in greater strength of the buckling restrained braces in compression than in tension (differences up to 10% have been reported [Clark, 2000]). Finite element analysis studies have shown excellent agreement with test results [Saeki, 1998]. Low cycle fatigue (failure) characteristics have been shown to depend on a variety of factors, including the restraining mechanism used, material properties, local detailing, workmanship, loading conditions and history, etc. Inelastic deformation (ductility) capacities are generally quite large, with cumulative cyclic inelastic deformations often exceeding 300 times the initial yield deformation of the brace before failure.

Buckling-Restrained Braced Frames

An interesting design approach for buckling-restrained braced frames has been proposed [Wada, 1992] in which the basic structural framework is designed to remain elastic during seismic response, and all of the seismic damage (yielding) occurs within the braces. By making the framework flexible and elastic, and using capacity design approaches to proportion members of the braced bays, an effective and economical structure can be achieved. Several parametric studies have been carried out to identify optimal design parameters [Watanabe, 1996], and this approach to "damage-controlled structures" has been applied to several buildings in Japan [Wada, 1999].

Currently, design provisions in the US (e.g., ICBO, 1997 and FEMA, 1997] do not contain specific requirements for braced frames incorporating buckling-restrained braces. As such, most investigations in the US have focused on specific questions related to the applicability of the code provisions and design methods developed originally for special concentrically braced frames, and the value of the Response Modification factor, R, contained in building codes that is appropriate to account for the inelastic response and damping of buckling-restrained braced frames.

The design approach most commonly used in the US for the design of buckling-restrained braced frames is similar to that used for special concentrically braced frame system [AISC, 1997]. It is generally anticipated that the behavior exhibited by buckling-restrained braces will overcome many of the perceived problems with concentrically braced frames. Buckling-restrained braces yield ductilely in both compression and tension. They are characterized by a full, stable, symmetric hysteretic loop with relatively low post-yield stiffness. As such, the redistribution of loads and deformations in braced frames with buckling-restrained braces should be far less than with conventional braces. The construction of the braces also permits the designer to stipulate a specific capacity; thus, story capacities can be much closer to the demands considered in design than possible with conventional braces, thereby mitigating the tendency to concentrate damage in weak stories. Since the braces do not buckle laterally, local damage to adjacent nonstructural elements should be reduced.

The qualities of buckling-restrained braces, while generally considered desirable, raise some questions as well. For instance, the low post-yield tangent stiffness of the braces might lead to the concentration of damage in one level even though brace capacities are relatively well balanced with demands over the height of the structure, thereby necessitating design provisions related to secondary lateral structural stiffness (to be provided by structural framing or some other mechanism) or a restriction on the relative over-strength permitted in adjacent stories. Similarly, the difference between the tensile and compressive capacities of the braces, while far less than that typically encountered in concentric braced frames, raises issues related to the design of the beams in chevron- (or V-) braced configurations. Similarly, the ability of the designer to closely specify brace strength has raised some concerns that the actual over-strength of such frames may become sufficiently low that significant yielding might occur under frequent ground motions under which continued occupancy might be expected (say, for excitations with a 50% chance of exceedence in 50 years).

To address these issues, several investigators have carried out inelastic dynamic analyses of hypothetical systems containing buckling-restrained braces [Ku, 1999; Huang, 2000; Clark, 2000]. Most of the design-oriented studies to date have focused on adaptations of a three-story frame model building developed to assess new design guidelines for steel moment frames [see MacRae, 1997]. To date, only a limited number of bracing configurations and ground motions have been considered in these studies. Detailed analyses have also been performed to examine the behavior of buckling-restrained braces within a braced frame system [Saeki, 1996]. In all of these studies, the seismic performance has been characterized as excellent and additional research was encouraged to examine more fully issues such as those identified above. Clearly, statistical information on system demands such as interstory drifts, and brace demands such as maximum and cumulative inelastic deformations, are needed to characterize performance and to develop test protocols for establishing brace qualification criteria and test protocols [SEAOC, 2001]. In this investigation, a series of model buildings were designed and their response to a large number of earthquake ground motions representing various seismic hazard levels were numerically simulated.

Model Buildings

To assess the performance of concentrically braced frames, a series of three- and six-story braced frame buildings were designed for a site in metropolitan Los Angeles. The buildings were designed according to the 1997 *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA 302/303) [FEMA, 1997]. This work is currently being extended to include nine and 20 story braced frame structures. The building configurations and non-seismic loading conditions were identical to those utilized in the development of the FEMA 350 guidelines for moment resisting frames, so that comparisons to moment frame behavior could be made [MacRae, 1999].

A variety of configurations of special concentrically braced frame configurations were considered, for both conventional and buckling-restrained braces (Fig. 1). Results are presented here only for systems with buckling-restrained braces oriented in a stacked chevron (inverted V-) pattern. In the design, buckling-restrained braces were envisioned as having an unbonded, yielding steel core within a mortar filled steel tube. However, nearly any unbonded-brace having equivalent properties may be assumed. A572 Gr.50 steel was used for all beams and columns.

The three-story building design follows the FEMA model building design criteria exactly [MacRae, 1999]. It has a typical 13-foot (4 m) story-height. Its nominal dimensions are 124 feet by 184 feet in plan; 30-foot by 30-foot bays are employed. Floors and roof have a 3-inch (76-mm) metal deck with normal-weight concrete topping. A small mechanical penthouse is provided. There are eight bays of bracing, four in each direction; the number of braced bays was set to prevent an increase in member design forces due to the Redundancy/Reliability factor, ρ, provided in the building code for systems with a limited number of lateral load resisting elements. The braced bays are located on the perimeter of the building, in non-adjacent bays. The columns are continuous for their full height. While these assumptions are not atypical, they serve to minimize system over-strength, so that conservative estimates of deformation demands can be obtained during the response simulations.

The six-story building design is adapted from the FEMA model building design criteria for the nine-story building. This height structure was added to the example studies, as it is a very common height for braced frame structures in the western US. The six story building has a typical 13-foot (4 m) story-height, but with an 18 feet (5.5 m) height at the first story.

Table 1 Frame Configurations Considered

Three Stories			Six Stories		
ID	R factor	Beams	ID	R factor	Beams
3vb	6	flexible	6vb	6	flexible
3vb2	8	flexible	6vb2	8	flexible
			6vb3	8	stiff

Its nominal plan dimensions are 154 feet by 154 feet (46.9m by 46.9 m); 30-foot by 30-foot (9.1 m by 9.1) bays are employed. Floors and roof have a 3-inch (76-mm) metal deck with normal-weight concrete topping. A small penthouse is located on the roof. Twelve bays of bracing are provided; six in each direction. Again, the number of braced bays was selected to prevent an increase in member design forces due to the ρ factor. These are located on the on the perimeter; bracing is located in non-adjacent bays. Both frame and non-frame columns are spliced mid-height at the fourth story.

In the design of the model buildings using FEMA 302/303, the equivalent static lateral force procedure was employed based on a response-spectrum corresponding to a hazard of 10% chance of exceedence in a fifty year period. A Response Modification Factor (R) of six was considered; a parallel design was also done using a Response Modification Factor of eight. A System Over-strength Factor (Ω_o) of 2 was used. Since code displacement criteria were not expected to control the design of these systems, and the Deflection Amplification Factor (C_d) remains to be defined for these systems, drifts under static design forces were calculated, but not used to limit the design. The buildings were designed consistent with Seismic Use Group I and Seismic Design Category D with an Importance Factor of 1.0. Site Class D (firm soil) was used for determining the response spectrum in conjunction with acceleration data obtained from seismic hazard maps prepared by the US Geological Survey. For the determination of design forces, the building period and the force distribution over the building height was determined using the approximate methods provided in the provisions (where period is based on building height, and lateral forces are distributed in proportion to elevation), rather than by employing a more realistic dynamic analysis.

Beams connecting to braces at their mid-span were designed for the maximum expected unbalance force from the braces. Based on earlier tests [Clark, 2000] the compression strength was assumed to be 10% larger than the strength in tension. In order to capture the greatest demands on the braces and beams, very flexible beams were used. An alternate six-story model design was also considered using stiffer beams designed to limit the vertical deflection under the maximum unbalance load.

Frame columns were designed using the Ω_o over-strength amplification factor applied to forces rather than computing the maximum forces that could be delivered to the frame system based on the actual capacity of the braces. Non-frame columns were designed for their tributary gravity loads only.

Braces were designed for the force calculated based on the computed equivalent static base shear. Brace sizes were set to within two percent of the computed required cross-sectional area (based on a nominal yield stress of 36 ksi (248 MPa) for the yielding core); no strength-reduction factor, ϕ, was used. The brace stiffness was calculated assuming a yielding length of 70% of the brace length and cross-sectional area of the non-yielding zone of three to six times that of the yielding zone; this is consistent with current design practice. Analytical results are discussed in this paper for the five model building configurations listed in Table 1. Sizes of members determined for model 6vb2 are shown in Table 2.

Analytical Modeling Assumptions

Only a single braced bay was modeled and analyzed for each frame configuration. Although the frames were not explicitly designed to be moment resisting, all beam to column connections with gusset plates attached (i.e., all connections except those at the roof) were modeled as being fixed. Possible contributions of the floor slabs to the beam stiffness and strength were ignored. Beams were assumed inextensible in the analyses. Columns were modeled as having a fixed base. The foundation was modeled as being rigid; footing up-lift was not permitted. Braces were modeled as pin-ended members.

Table 2 Member Properties for Model 6vb2

Story	Buckling-Restrained Braces		Beams	Columns
	Tension Capacity (Kips)	Axial stiffness (kip/in.)		
6	173	888	W14x48	W14x132
5	288	1432		
4	317	1566		
3	349	1707		W14x211
2	389	1886		
1	511	1907		

The floor level masses used in the analysis to account for horizontally acting inertia forces was taken as the total mass of the each floor divided by the number of braced bays used in the building in each principal direction. Global P-Δ effects were considered based on his mass. Since only horizontal ground excitations were considered, local tributary masses were not distributed along the floors. An effective viscous damping coefficient of 5% was assumed, according to common practice for code designed steel structures.

The analytical model included a single additional column member running the full height of the structure. This column was intended to approximate the contributions of the gravity load framing to the lateral stiffness of the structure. While this column provides little overall resistance to lateral loads, it is expected to help redistribute loads across a story when localized yielding occurs in that story. Since the connections of a beam to a column in the gravity-only load resisting system were assumed pinned, only the properties of the columns need be included to model the lateral stiffness of the gravity system. In the analysis, the equivalent column was constrained to have the same lateral displacement as the braced bent. As a simplification, the equivalent column was given a moment of inertia and moment capacity equal to the sum of the corresponding values for all of the columns in the gravity-only frames divided by the number of braced frames oriented along the principal axes of the building being analyzed.

The analyses were carried out using the nonlinear dynamic analysis computer program SNAP-2DX [Rai, 1996]. The buckling-restrained braces were modeled using element type 1 (a simple truss element with ideal bilinear hysteretic behavior, exhibiting no stiffness or strength degradation). In order to clarify the potential consequences of the nearly elasto-perfectly plastic hysteretic characteristics of the buckling-restrained braces considered on the formation of weak story mechanisms, the secondary post-yield stiffness of braces was set to zero. To maximize demands on the braces and beams, the tension capacity was calculated from the cross-sectional area assuming no material over-strength; the compression capacity was set to 110% of the tension capacity. Beams and columns were modeled using element type 2 (a beam-column element allowing axial load-bending moment interaction, but with no stiffness or strength degradation).

Earthquake Ground Motions

The models were analyzed using the suites of ground motions developed previously by Somerville and others for use in the FEMA project on steel moment-resisting frames [Somerville, 1997; MacRae, 2000]. These suites consist of 20 horizontal ground acceleration records (two components for each of ten physical sites) adjusted so that their mean response spectrum matches the 1997 NEHRP design spectrum (as modified from soil type of S_B-S_C to soil type S_D and having a hazard specified by the 1997 USGS maps for downtown Los Angeles). For this study, the earthquake suites corresponding to downtown Los Angeles, California, were selected for seismic hazard levels corresponding to a 50%, 10% and 2% probability of exccedence in a 50 year period. These acceleration time histories were derived from historical recordings or from simulations of physical fault rupture processes. The two horizontal components of the original records were initially resolved into fault-normal and fault-parallel orientations. The records were adjusted in the frequency domain to have characteristics appropriate for NEHRP S_D soil sites. The records were further amplitude scaled so that the average spectra for the two horizontal components matched the target spectrum. The individual components were lastly rotated 45 degrees away from the fault-normal / fault-parallel orientations to avoid excessive near-fault directivity effects biasing individual analyses. A separate study of near-fault effects on braced frames is underway.

Case Study Example

In this section, the response results for a six story braced frame model, designed with an R factor of eight (Model 6vb2) are described in detail for the specific case of one of the records in the 10% in 50 year hazard suite. The record in question is designated LA20, and was derived from a near-fault site during a moderate magnitude event, the 1986 North Palm Springs earthquake. For the 10% in 50 year probability of exceedence, this record has been amplitude scaled to 0.98g. For this severe record, the peak roof displacement computed is 11.93 in. (300 mm), corresponding to an average maximum interstory drift of only about 1.2 %. The maximum interstory drift ratio that occurs at any level during the earthquake is 2.3%, suggesting that some concentration of damage occur within one or more stories. The permanent displacement offset that can be seen in the roof level displacement time history (Fig. 4) suggests that considerable inelastic action does occur during this earthquake. As will be elaborated on in the next section, peak roof displacements ranged from 5.48 in to 16.6 in

(140 mm to 422 mm) for the records considered in this suite; averaging 9.74 in (247 mm). Thus, the response to this record is well above average.

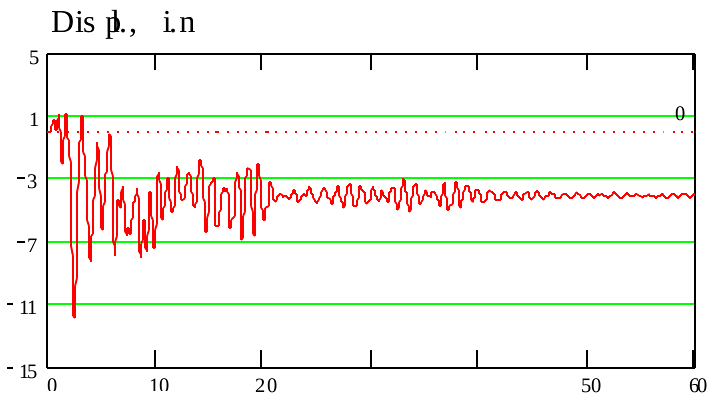
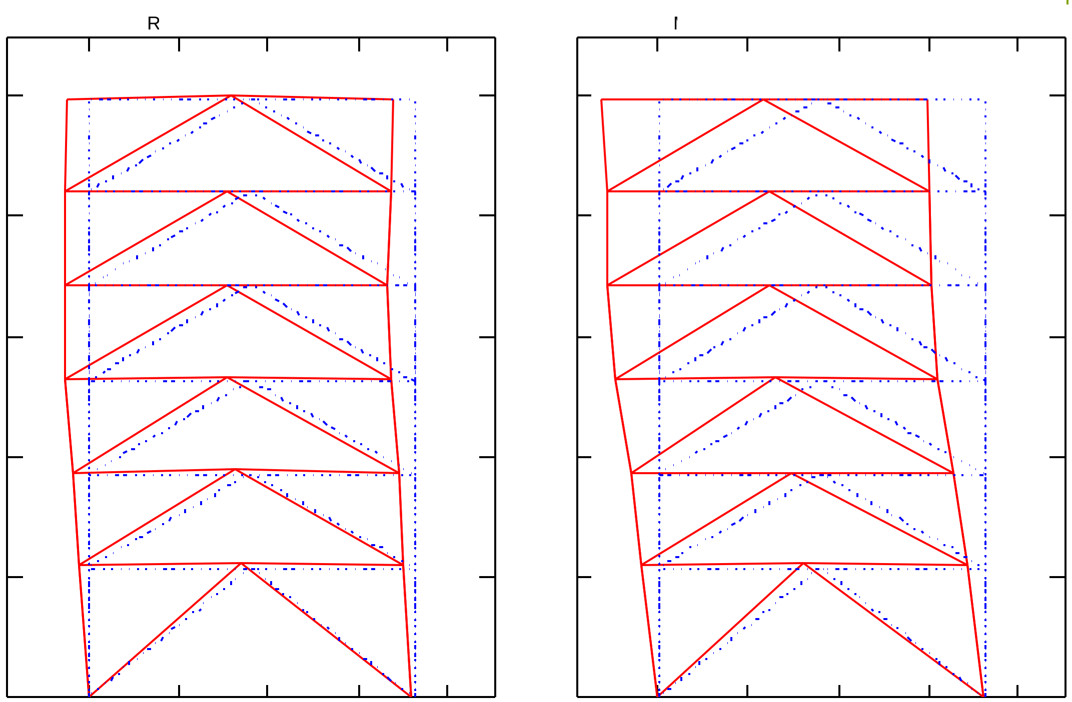


Fig 4 Roof displacement time history for Model 6vb2 to the LA20 record

An examination of the displaced shape of the building when the maximum roof response occurs (Fig. 5) suggests a relatively uniform distribution of interstory drift over the height, with higher than average drifts in the lower three stories, and lower than average values in the upper three stories. Similarly, the residual displacements retained in the structure are nearly uniform over the full height (Fig. 5). It should be noted that the maximum residual displacement remaining at the roof level at the end of the earthquake is 4.95 in. (124 mm), corresponding to a average permanent drift ratio of about 0.5%. The peak residual drift in any story is slightly less than 1% for this earthquake.



a. Permanent Displacements b. Maximum Displacements

Fig 5. Displaced shapes of Model 6vb2 for LA20

The severity of the inelastic response can be better visualized from Fig. 6, which plots for each story in the braced bay the relation between story shear and interstory drift. As can be seen in this figure, there is substantial yielding. However it is significant to note that this is nearly uniformly distributed over the height of the structure, and in spite of the relatively small post-yield stiffness of the structure resulting from the modeling assumptions, that there is little tendency to concentrated damage at weaker stories that yield substantially more than other stories. Another parameter used in this study to assess the tendency to concentrate damage in a floor, and to place significant flexural demands on the columns and beams, is the column rotation angle, defined herein as the difference in drift ratios for adjacent stories for a floor level. This change in drift represents the need for the column to bend or kink at the floor level. For this building, the maximum value of the column drift ratio was 1.6%, suggesting that at some time during the response, significant local bending action is demanded of the frame. However, at the instant of maximum displacements, such large differences in interstory drift do not occur.

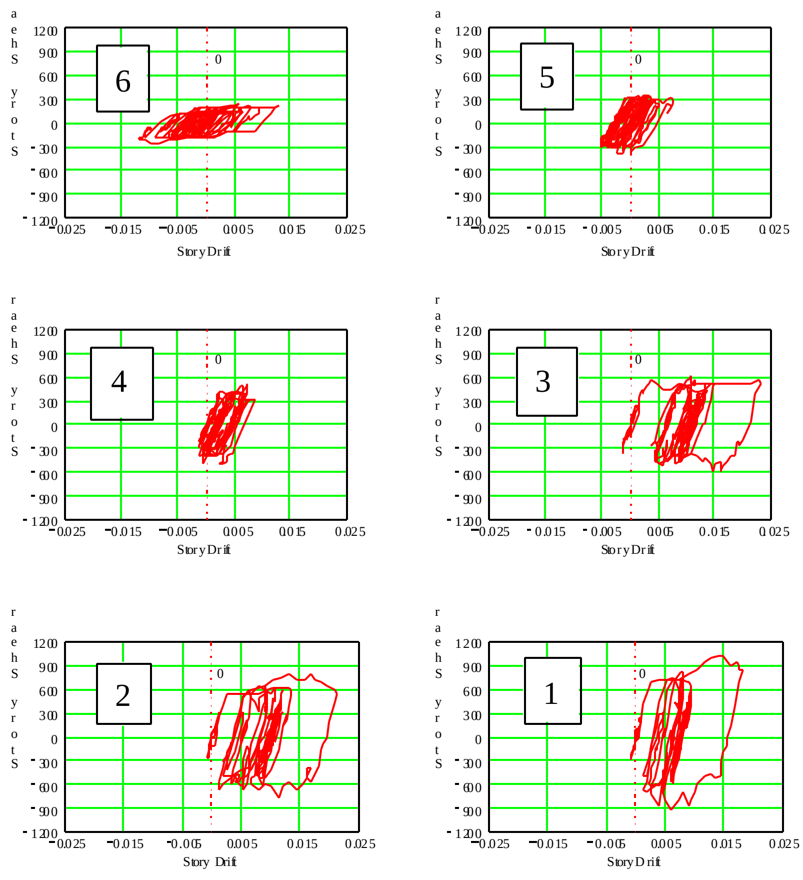


Fig. 6 Story level relations between story shear and interstory drift – Model 6vb2 subjected to Record LA20

The maximum brace ductility demand (computed as change in brace length divided by the yield displacement in tension) is 15.3 in extension and 9.4 in contraction. This difference is associated in large part with the difference in the tensile and compressive capacities of the braces. The worst cumulative ductility (taken as the plastic deformations occurring in a brace summed over all cycles throughout the entire response history, in either tension or compression, divided by the tensile yield displacement of the brace plus unity) demanded for any brace in the frame is 127. Test data suggest that such demands are well within the capacity of many types of buckling-restrained braces.

It is apparent from Fig. 4 that there is some vertical movement (displacement) at the center of the beam. Because the buckling-restrained braced considered are slightly stronger in compression than in tension, the tendency is for the flexible beam considered in this analysis to displace upward as the tensile brace will yield before the compression brace. For this earthquake, the center of the beam deflects upward 1.08 in. (27 mm) (and only 0.01 in. (0.3 mm) downward). While this is a small displacement over the 30 ft (9 m) span of the beam, nearly 90% of the peak value remains after the earthquake, and it represents a large fraction (about 2/3) of the worst interstory drift developed at any level. It should be noted that the braces were intentionally modeled to maximize this behavior. Inclusion of more realistic beam and brace properties would be expected to reduce this vertical movement. Results of a companion study [Ku, 1999] demonstrates the effectiveness of double story X-bracing configurations (see Fig. 1) in reducing this movement.

Statistical Evaluation of Seismic Demands on Buckling Restrained Braced Frames

Because there is considerable variation in response from record to record, and the records considered in these analyses were not selected to represent a particular type of earthquake, but rather a range of earthquake events that might occur at the building site over a long period of time, it is important to examine the results in a statistical sense as well as on a case by case basis. For instance, Fig. 7 shows the peak interstory drifts obtained at any floor level for Model 6vb2 for each of the records in the suite of ground motions corresponding to a 10% in 50 year probability of exceedence. The bar on the far right corresponds to Record LA20 discussed in the previous section. The mean drift ratio is 1.6% (and the mean plus one standard deviation value is 2.2%). The extent to which the bars fall below the horizontal axis represents the peak column rotation angle, defined previously. The median and median plus one standard deviation values for this parameter are 1.0% and 1.4%, respectively. Record 9,which gives the largest interstory drift, is derived from 1992 Landers (Yermo) record scaled to about 54% g.

It is significant to note that the mean value of the maximum interstory drift computed for a comparably designed conventional special concentric braced frame [Sabelli, 2001] is slightly larger at 1.8 % (with a mean plus one standard deviation value of 2.5%). However, it is especially important to recognized that even for these 10% in 50 year events, the low-cycle fatigue model used to control brace behavior and failure in the analysis [Tang, 1989] predicts a low-cycle fatigue related fracture of at least one brace in the frame for 6 of the 20 records considered. While FEMA [MacRae, 1997] did not consider a 6 story moment frame, the mean values of maximum interstory drift were 1.26% for the three story frame and 2.0% for the 9 story frames with ductile connections. Thus, the behavior of the frames with the buckling restrained braces is comparable and often better than that associated with conventional concentric braced frames and moment frames.

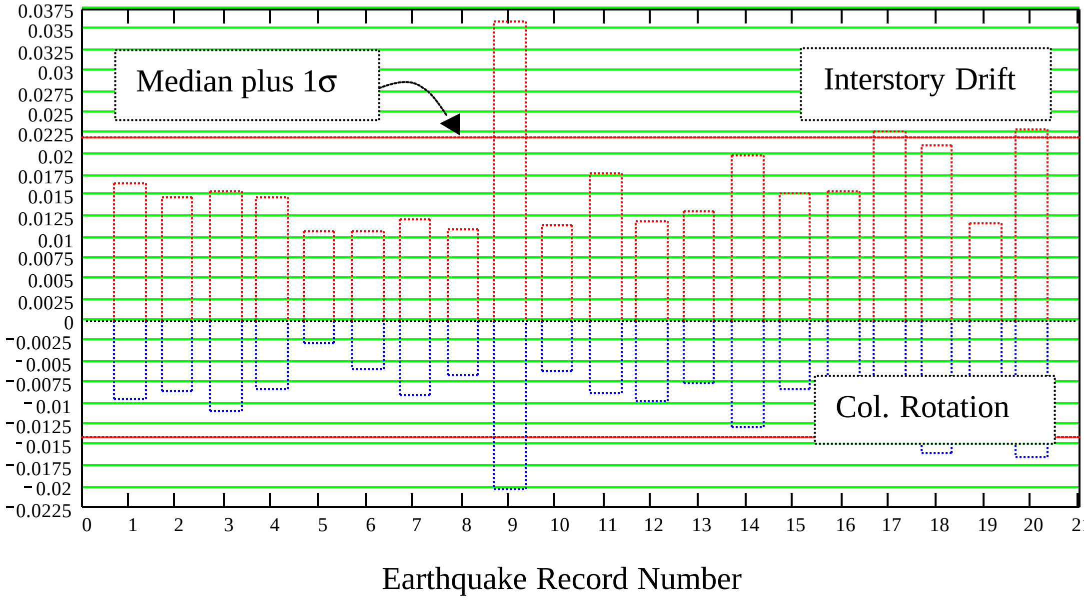


Fig. 7 Peak interstory drift and column rotation for each record in the 10% in 50 year probability of exceedence suite of ground motions

To examine the effect of various design parameters for the different seismic hazards, attention will focus herein on mean responses of key response parameters. For example, for the six story braced frame, the envelop of the mean peak lateral floor displacements for the 10% in 50 year events is shown in Fig. 8 for the cases where the beams are flexible and the R value used in design is either 6 or 8. It can be seen that the envelope of peak lateral displacements increases with increasing R. Because this structure has a relatively short period (about 0.55 sec.) and many of the ground motions considered are near-fault records containing relatively long duration velocity pulses, the assumption that the peak displacements of inelastically responding structures can be predicted by an analysis of an elastic model may not be correct.

None the less, Table 3 shows that the computed average worst interstory drift occurring within the frame do not change for R of 6 or 8. This table presents the mean of the largest interstory drift occurring at any level in the structure. It is also clear from Fig. 8 that, as was the case for specific case of the LA20 record, the peak drift demands are appreciably higher (by a factor of about two) for this building in the lower three stories than in the upper levels. This is more clearly seen in Figure 9. Figures 8 and 9 also indicate that the residual displacements present in the building following these events is on average about 40 to 60% of the maximum displacement attained. While this percentage may appear to be a large, it is consistent with that computed for ductile rigid moment frame structures [MacRae, 1997].

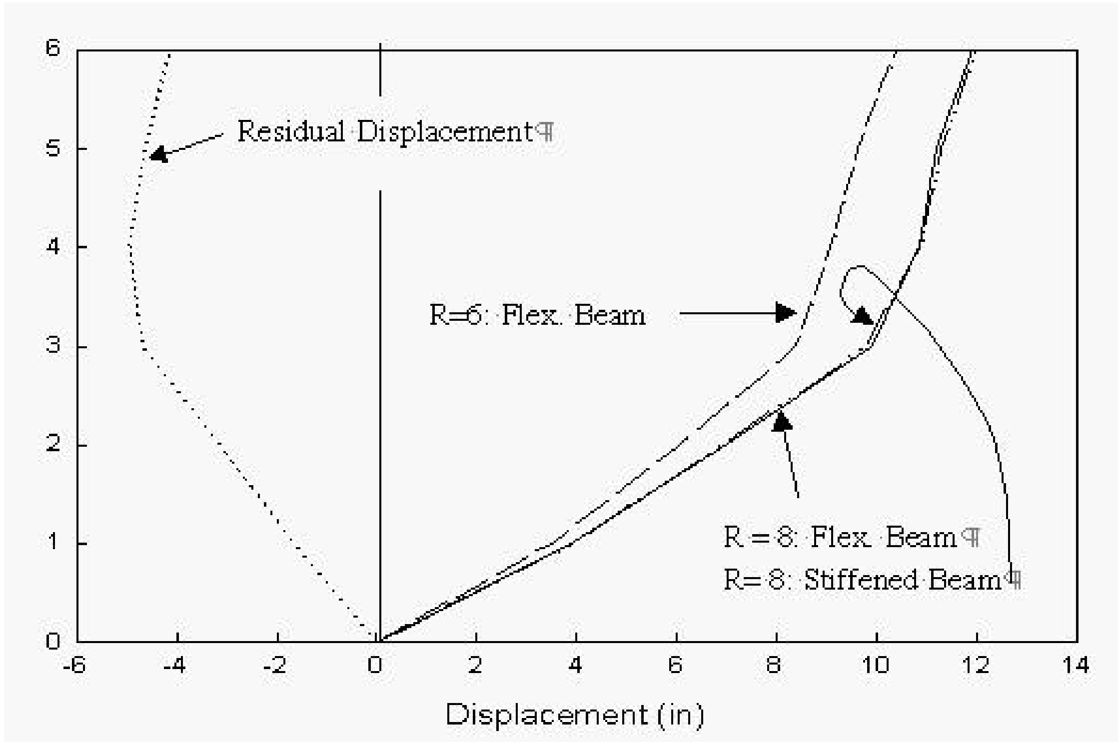


Fig. 8 Envelopes of median peak lateral displacements for six story frame to 10 in 50 year events

Figure 8 also suggests that stiffening the beam to limit vertical displacements (Model 6vb3) has little effect on the peak lateral displacement of the building. However, as seen in Fig. 9, the permanent vertical displacements of the flexible beam case can be on average a small (about 20) but significant percentage of the total interstory drift.

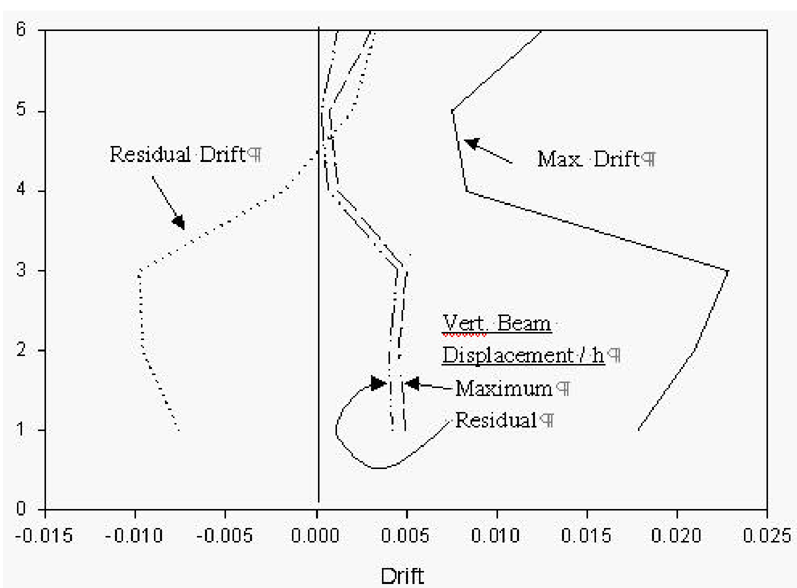


Fig. 9 Average Interstory Drifts (and Vertical Beam Displacements) for Model 6vb2 for the Records Corresponding to the 10% in 50 year Hazard

It is also interesting to examine the change in mean envelope of peak lateral displacements as the seismic hazard is changed. Figure 10 presents the mean maximum displacements and interstory drift ratios corresponding to the ground motions with 50%, 10% and 2% probability of exceedence in a 50 year period. For the 50% probability events, the displaced shape envelope has a shape characteristic of flexural behavior. That is, larger interstory drifts occur at the top than at the bottom. However, as the severity of the earthquakes increase, the pattern of deformations changes significantly, with more and more of the drift concentrating in the lower three levels. This distribution appears to be associated with the distribution of lateral loads used in the original design. While the brace sizes in these frames have been carefully tailored to match the demands computed with the FEMA 302 equivalent lateral forces, this distribution may not adequately reflect the actual dynamic force distribution that occurs in the building. This was noted by Khatib [Khatib, 1989] who showed for similar braced frame configurations that a more uniform distribution of interstory drifts will develop when the design forces are based on a rational dynamic modal analysis procedures.

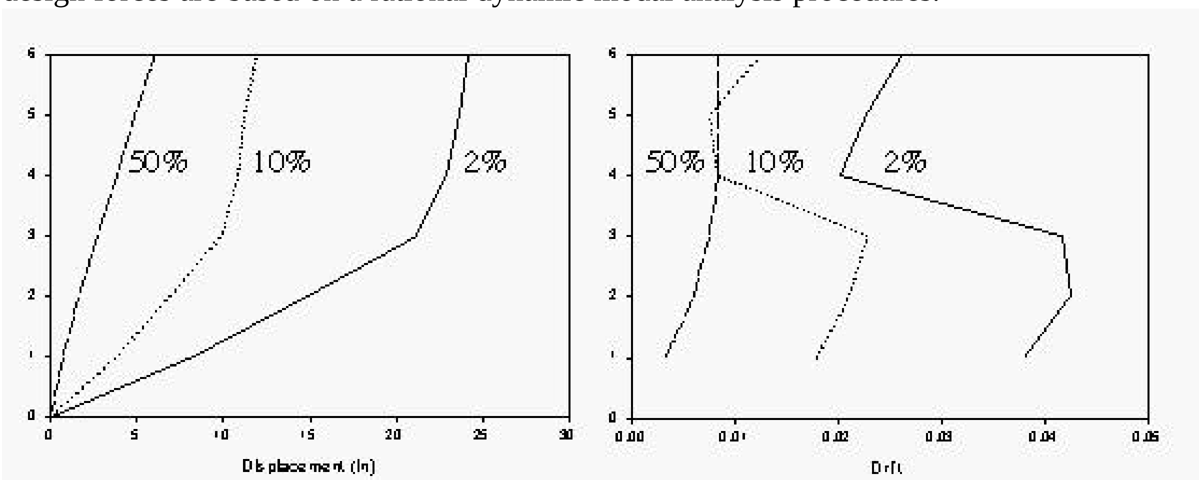
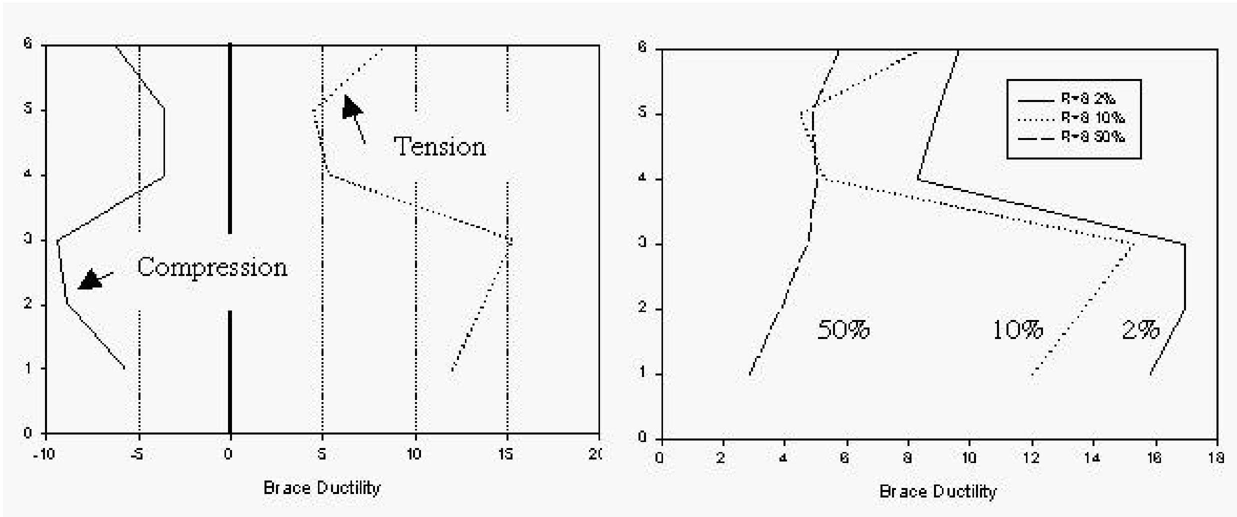


Fig. 10 Envelopes of mean peak lateral displacements and interstory drifts for Model 6vb2 when subjected to ground motions representing different seismic hazards.

Table 3 lists similar information for three story building. Interestingly, the shorter building develops essentially the same peak interstory drifts as the taller building, and again, these drifts do not change significantly when the value of R used in design changes from 6 to 8. On the other hand, studies of concentrically braced steel frames [Sabelli, 2001] suggest that mean peak interstory drifts may reach nearly 4% for the 10% in 50 year event (and with braces fracturing for 14 out of the 20 records considered].

Brace ductilities for the buckling-restrained braced frames generally vary in the same manner as interstory drift. This can be seen by comparing Figs. 10 and 11. Also, since the interstory drifts for the 3 and 6 story frames are similar, the mean ductility demands for these two height structures are similar (Table 3). As noted previously, the degree to which the braces elongate is generally greater than to degree to which they shorten.

The most interesting difference between the responses for the different frame configurations relates to the cumulative ductilities developed by the buckling-restrained braces. As seen in Table 3, the peak cumulative demands on the three story frame are on the order of 43% of those of the six story frames even though the maximum ductility demand is nearly identical. The three story structure has a shorter period and likely to experience more cycles of yielding. For the six story frame, changing the R factor from 6 o 8 has virtually no effect on the maximum or cumulative ductility demands. However, providing a stiff beam (with R = 8) reduces both maximum and cumulative ductility demands by a considerable amount.



a. Brace ductilities for 10% in 50 year hazard   b. Ductilities for different hazard levels

Fig. 11 Envelopes of mean brace ductility for Model 6vb2

The cumulative ductility demands vary for the six story frame (with R = 8) from a mean peak value of 45 for the 50% in 50 year hazard level earthquake motions, to more than 185 for the mean plus one standard deviation case for the 2% in 50 year excitations. For the 50% in 50 year seismic events, the peak interstory drifts are less about 1% and the level of yielding in the braces is likely well within a range consistent with continued occupancy. For the more severe records, the level of yielding is conservatively within the range generally attained in tests of buckling-restrained braces.

Concluding Remarks

An extensive analytical investigation of the seismic response of concentrically braced steel frames has been undertaken. Results have identified a number of important parameters associated with the ground motion intensity and characteristics as well was with the structural configuration, proportioning and modeling that have important impacts on computed response. Results presented in this paper have focused on applications of buckling restrained bracing members. Results from this phase of the overall study indicate:

- Buckling-restrained braces provide an effective means for overcoming many of the potential problems associated with special concentric braced frames. To accentuate potential difficulties with this system, numerical modeling and design assumptions were intentionally selected to maximize predicted brace demands and the formation of weak stories.
- None the less, the predicted behavior is quite good, with significant benefits relative to conventional braced frames and moment resisting frames. For the cases studied to date, response is not sensitive to R factors selected in the range of 6 and 8.
- Response appears to be sensitive to proportioning suggesting that further improvements in response may be obtained by better estimation of a structure's dynamic properties.

Additional studies are recommended, as follows:

- This paper only examines seismic demands. As indicated in the paper, a thorough characterization of the capacities of the elements of the braced frame as well as of the overall braced frame system needs to be carried out to fully quantify the confidence that braced frames can attain specified performance levels. Work is underway by the authors adapting the methodology presented in FEMA 350 for moment resisting frames and extending the current work to 9 and 20 story structures.
- Experimental research is needed to assess the behavior of braced frames incorporating buckling-restrained braces. Braces are likely to develop significant bending and shear forces in actual applications, and the effect of this on brace and frame behavior is unclear. New analytical models to simulate this behavior may be required.
- New approaches to design of structures better able to resist damage, such as those recently recommended in Japan for use with buckling restrain braces should be studied [e.g., Wada, 1992]. Similarly, other approaches for improving seismic behavior of concentrically braced frames through innovative configurations, improved frame proportions and use of other types of bracing elements such as visco-elastic dampers should be investigated.

Table 3 Summary of Some Response Parameters

Model Properties			Maximum Response Quantities (Mean and Mean+1σ of worst-case story or worst-case brace for suite of ground motions)						
			%					Brace Ductility	
Model	R	Hazard (% in 50 years)	Elastic Drift under Design Loads	Max. Drift	Max. Drift/ Elastic Drift	Residual Drift	Column Rotation	Max. Brace Ductility	Cum. Brace Ductility
3vb 3- story	6	10%	0.20	1.5 (2.2)	7.8 (11.5)	0.6 (1.1)	0.9 (1.1)	10.6 (15.3)	38 (59)
3vb2 3- story	8	10%	0.19	1.4 (2.1)	7.6 (10.9)	0.5 (1.0)	0.8 (1.1)	9.7 (13.6)	39 (63)
6vb 6- story	6	10%	0.28	1.6 (1.9)	5.7 (7.0)	0.6 (1.0)	1.0 (1.3)	10.7 (12.8)	88 (132)
6vb2 6- story	8	10%	0.24	1.6 (2.2)	6.7 (9.1)	0.7 (1.1)	1.0 (1.4)	10.7 (14.5)	83 (135)
6vb2 6- story	8	50%	0.24	1.0 (1.2)	4.0 (5.0)	0.4 (0.5)	0.6 (0.9)	6.6 (8.2)	45 (71)
6vb2 6- story	8	2%	0.24	4.5 (6.6)	18.4 (27.0)	2.2 (3.2)	3.0 (4.6)	17.4 (25.1)	139 (185)
6vb3 Stiff Beam	8	10%	0.24	1.5 (2.1)	6.0 (8.5)	0.6 (1.0)	0.9 (1.3)	8.9 (12.9)	56 (92)

Acknowledgements

The authors would like to acknowledge the financial support provided by the FEMA/EERI Professional Fellowship that enabled the first author to conduct the research described in this paper. The Nishkian Professorship at the University of California at Berkley provided additional support. The assistance and comments by the AISC/SEAOC Task Committee on Buckling-Restrained Braced Frames is greatly appreciated. The findings and conclusions presented in this paper are, however, those of the authors alone.

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