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SEISMIC PERFORMANCE ASSESSMENT OF CONCENTRICALLY BRACED STEEL FRAMES

Patxi URIZ[1], Stephen A. MAHIN[2]

SUMMARY

This paper summarizes two aspects of an on-going investigation at UC Berkeley that focuses on the seismic behavior of concentrically braced steel frame structures. While the overall investigation includes systems that utilize conventional braces, buckling restrained braces and braces incorporating viscous damping devices, this paper focuses on evaluating the seismic performance of relatively conventional concentric braced frames and buckling restrained braced frames. In the first part of the paper, the same reliability framework as used to assess Special Moment Resisting Frame (SMRF) structures during the FEMA/SAC Steel Project was employed to assess the confidence with which Special Concentric Braced Frames (SCBF) and Buckling Restrained Braced Frames (BRBF) might achieve the seismic performance expected of new SMRF construction. In the second part, a test program to help improve modeling of SCBF systems is described, including the design of a nearly full-size, two-story SCBF test specimen.

INTRODUCTION

Prior to the 1994 Northridge earthquake, relatively few special concentrically braced steel frames were constructed in California. It has generally been believed within the structural engineering community that the seismic performance of concentric braced frames is inferior to that of moment-resisting frames. Extensive damage has been reported in concentrically braced frames following many recent earthquakes, including the 1985 Mexico (Osteraas, [15]), 1989 Loma Prieta (Kim, [11]), 1994 Northridge (Tremblay, [21]; Krawinkler, [12]), and 1995 Hyogo-ken Nanbu (AIJ/Kinki Branch Steel Committee, [3]; Hisatoku, [9]; Tremblay, [22]) events. Because of these observations, building codes stipulate comparatively low values for the response modification factor used in design to account for the inherent ductility of a system, and additional restrictions are imposed for braced frames located in regions of high seismic risk. However, with the introduction of more complex and stringent guidelines for the design and construction of SMRFs following the Northridge earthquake, a rapid increase in the use of special concentrically braced frames has occurred, especially for low- and mid-rise construction.

[1] Graduate Student,UC Berkeley. Email: patxi@ocf.berkeley.edu
[2] Byron L. and Elvira E. Nishkian Professor of Structural Engineering, UC Berkeley. Email: mahin@ce.berkeley.edu

Few analytical studies have examined the seismic performance of SCBF systems, and no full or nearly full-scale tests of these systems have been carried out in the past 20 years. Recent analytical studies of seismic demands on modern steel braced frames (Sabelli, [18]) have indicated that ground motions representing a 10% probability of exceedence produce on average interstory drifts in a six-story tall, chevron configured SCBF about 12% larger than those in a comparable frame containing buckling-restrained braces (BRBF). When the height of the structure is reduced to three stories, the SCBF develops drifts that average 180% greater than those in a comparable BRBF. This increase in demand is in large part associated with the deterioration and fracture of conventional bracing elements during inelastic cycling. For instance, 70% of the ground motions considered in these analyses caused at least one of the braces to fracture during the simulations off the 3-story building. Increasing the intensity of the ground motion, for example by considering a 2% in 50-year probability of exceedence, further widens the difference between the behavior exhibited by SCBFs and BRBFs. While such studies focusing on demands can be used to identify vulnerabilities of particular types of structures, and compare the response of different types of systems under the same excitations, they are not by themselves able to assess the likelihood of failure of the systems nor are they able to compare the reliabilities of different structural systems.

Following the Northridge earthquake, the FEMA/SAC Steel Project extended the application of reliability methods to assess the seismic performance of steel moment-resisting frame buildings. In this case, the deformation demands on a system were compared to estimates of the capacity of the system to withstand seismically induced displacements, globally and locally, in order to estimate the ability of the structure to prevent collapse (or achieve other performance goals, such as continued occupancy) for a stipulated seismic hazard level (FEMA, [7]).

The primary difference between this method and earlier load and resistance factor design approaches is the explicit consideration of behavior at the system level, rather than at the member level (Lee and Foutch, [13]). As such, the failure of one or more elements is not by itself considered to represent the failure of the overall structure, provided the system can redistribute the required lateral forces in a ductile manner and continue to support gravity loads. This performance-based earthquake engineering (PBEE) framework employed for SMRFs is sufficiently general so that it can be easily extended to other types of lateral-load resisting system.

Four preliminary case studies are considered herein to apply this PBEE methodology to SCBFs and BRBFs. The three- and six-story chevron conventionally braced steel frames(3V and 6V, respectively) along with three- and six-story chevron buckling restrained braced steel frames (3VB and 6VB, respectively) cited earlier are used as the basis of these studies. The studies examine the potential benefits and limitations of this PBEE methodology as applied to braced steel structures.

This paper also highlights some of the details of a test program underway to improve understanding of the behavior of typical SCBF systems, and to evaluate and refine the accuracy with which this behavior can be simulated numerically. These tests include nearly full-scale two-story SCBF specimens, experiments on individual braces, and frame connections including gusset plates. It is expected that these preliminary tests will suggest areas for further research.

PERFORMANCE-BASED EVALUATION OF SCBFS

A recent special edition of *Earthquake Spectra* contains many published articles describing the performance-based evaluation methodology used in this paper (Mahin et. al., [14]; Hamburger et. al., [8]; Saunders, [20]; Hooper, [10]). Consequently, specific details of the reliability framework will not be discussed herein.

Quantification of the confidence that a performance goal can be achieved for a particular structure and seismic hazard level includes several inter-related steps: site-specific hazard assessment, structural demand estimation, and structural capacity evaluation. In the procedure implemented herein, median peak interstory drift demands are computed for an ensemble of earthquake records representative of the seismic hazard level of interest, dynamic interstory drift capacities are estimated, and various types of aleatoric and epistemic uncertainties are characterized. Nonlinear dynamic analyses were used in this study to estimate seismic demands and capacities. From the data obtained, a confidence parameter, λ, can be determined from Equation 1:

$$\lambda = \frac{\gamma \gamma_a D}{\phi_U \phi_R C} \tag{1}$$

This confidence parameter is associated with the probability of a specific performance being met, given a specific hazard level (Lee and Foutch, [13]). The following sections step through the assumptions used to calculate each of the coefficients in Equation 1 for the four structures mentioned above. Because of the lack of supporting tests and data, several significant assumptions need to be made. As such, the values of confidence computed are only approximate, but the computations illustrate the process and help identify areas requiring further refinement.

Site Specific Probabilistic Hazard Assessment

In the SMRF procedures developed for the FEMA/SAC project, primary emphasis is placed on the ability of the structure to achieve the collapse prevention performance goal. Because of (a) the high consequences of violating this goal, (b) the realistic methods used to predict response (with few built in conservatisms), and (c) the numerous assumptions made, a high confidence of achieving this goal was stipulated for a severe seismic hazard level (2% probability of exceedence in 50 years). A 90% confidence level was recommended for global collapse modes, while a 50% confidence was cited for more localized failure modes. These confidence levels were found to be consistent with the behavior of ductile SMRF structures designed according to the 1997 NEHRP provisions (Lee and Foutch, 2000).

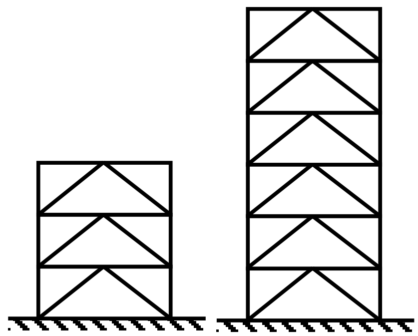
For the analytical studies presented in this paper, seismic hazards were assumed to be the same as those used in the FEMA/SAC studies for a representative location on firm soil in Los Angeles, California. The underlying hazard estimates were based the 1997 NEHRP (FEMA 273) provisions. Twenty ground motions from the FEMA/SAC database were used in the analyses, corresponding to the 2% in 50-year probability of exceedence. For more information regarding development of the ground motions, the reader is referred to Somerville (1997).

Structural Demand Assessment

Interstory drift ratios and axial load demands, generally speaking, are the primary damage intensity parameters used for characterizing the behavior of SMRF structures. Prediction of interstory drifts in SMRF structures is relatively insensitive to modeling assumptions and analytical procedures, and computed drifts can be readily related to expected damage in and around connection regions. Interstory drift has also been adopted in this study to characterize the global dynamic response of SCBFs. However, additional studies are needed to assess the validity of this assumption (for example, it may be necessary to monitor local damage in connections, gusset plates, braces, etc., and more rigorously check for buckling or fracture of the column elements). For our studies, the peak interstory drift predicted in any story is used to characterize damage for a specific ground motion. Computation of median values and standard deviations over a suite of ground motions is based on an assumed lognormal probability distribution (see Hamburger et. al. [8]).

Basic dimensions, weights and loads were adopted from the FEMA/SAC model buildings. More information for the specific analytical assumptions used in this paper is provided by Sabelli [19].

The case study buildings were designed using the 1997 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (FEMA, [6]) and the Load and Resistance Factor Design Specification for Structural Steel Buildings (AISC, [1]). The equivalent static lateral force procedure used employed a response spectrum corresponding to a hazard having about a 10% chance of exceedence in a fifty-year period.



(a) (b)

FIGURE 1. (a) Three Story (3V and 3VB) and (b) Six Story (6V and 6VB) Configurations

Figure 1 schematically illustrates the two dimensional special concentric and buckling restrained frames analyzed in this paper. The three-story and six-story chevron frames are representative of "typical" proportions for a building found in California. More detailed information about the model and design assumptions can be found in Sabelli ([18]). Important key features of the structures are listed in Table 1.

Table 1. Key Structural Features

Governing Code	1994 Uniform Building Code; 1997 NEHRP Recommended Seismic Provisions
Building Height	
3V & 3VB	39 ft / 13 ft per floor typ.
6V & 6VB	83 ft / 13 ft per floor typ.
R value	6
Fundamental Period	
3V & 3VB	~ 0.5 sec
6V & 6VB	~ 0.9 sec
Soil Site Characterization	NEHRP, Class D stiff soil

The analytical model was created using the SNAP-2DX analytical platform (Rai et. al., [17]). SNAP-2DX is a structural, nonlinear analysis program capable of dynamic analysis for two-dimensional models. This program has a library of nonlinear elements including a uniaxial truss element that models phenomenological buckling and low-cycle fatigue failure of brace members. These multi-linear, hysteretic models have proven to accurately capture the nonlinear response of buckling members (Black et. al., [4]). The contribution of the gravity load only framing was approximated in the analyses, but all connections in the braced frame were assumed to be ideally pinned.

Seismic demands were characterized by extracting the maximum interstory drift from the analysis output for the twenty 2% in 50 year events (SAC ground motions LA21 – LA40). The variability (uncertainty) of response for this hazard level is represented by the standard deviation of the natural logarithms of the drift demands (β_{DR}). β_{DR} is used in conjunction with parameters characterizing the variability of ground motion intensity at the site for the stipulated seismic hazard to compute a demand variability factor, γ. Table 2 and Figure 2 summarize the demands and demand factors that were calculated. Figure 2 illustrates the scatter in drift demands for the two models.

Table 2. Demand Results and Parameters

	Median Drift Demand (in/in)	Median Plus 1 Standard Deviation (in/in)	β_{DR}	γ
3V	0.0609	0.118	0.661	1.93
3VB	0.0386	0.059	0.554	1.58
6V	0.0510	0.088	0.614	1.76
6VB	0.0309	0.046	0.413	1.29

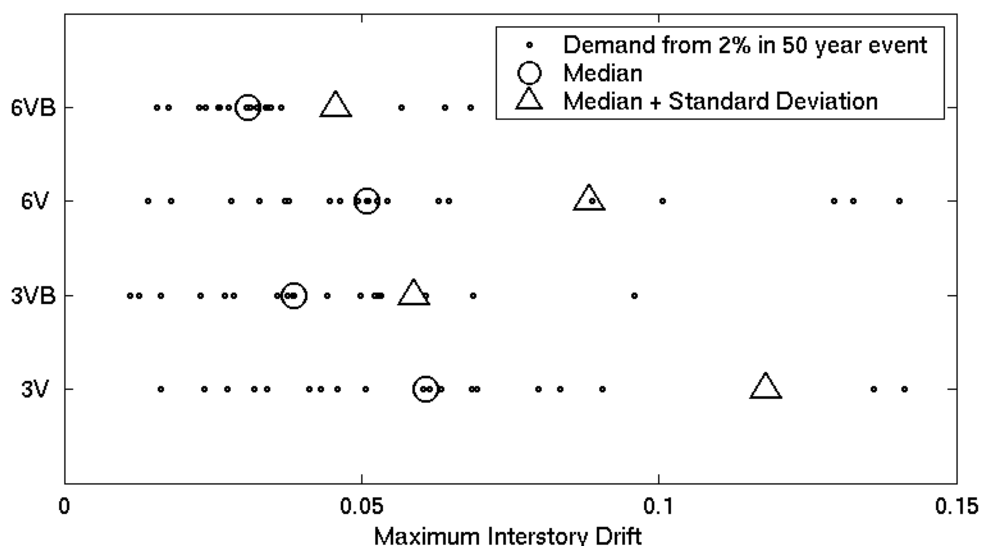


Figure 2. 2% in 50 Year Drift Demands for 3V, 3VB, 6V and 6VB Structure.

Uncertainty also comes in the form of epistemic errors associated with the analytical models and procedures used. For example, Figures 3 and 4 illustrate the difference between the analytical and experimental hysteretic loops (Black et. al., [4]) for a tubular truss member. Representative test results are shown in Figure 3 for a tubular brace with a slenderness ratio of 80. The hysteretic loops in Figure 4a are obtained from a SNAP-2D analysis in a brace having similar slenderness parameters. While the general post buckling character of the analytical model are similar to those observed in tests, it is clear that significant simplifications have been introduced. Other epistemic uncertainties can arise from the inability to model accurately other structural components (e.g., gusset plates, connections, columns, base plates, etc.), define viscous damping, characterize the effects of nonstructural components, or account for biases introduced by different analytical procedures. These uncertainties give rise to an analytical uncertainty factor, γ_a. This source of uncertainty can be reduced by using more refined and accurate analytical models. If simpler elastic analysis models are used, it is expected that γ_a would increase significantly.

Future research is planned to better characterize γ_a and to reduce analytically related uncertainty using more intricate fiber models. For example, Figure 4b shows an analytically predicted hysteresis of the test specimen of Figure 3. This element is based on an efficient large-displacement buckling formulation incorporated in the OpenSees computational framework currently being developed at UC Berkeley (Fenves, [5]). This element is based on a fiber representation of sections so that bi-axial bending as well as axial load and buckling effects can be simulated. Material models have been incorporated that allow fracture due to low-cycle fatigue to be simulated. Models such as these may permit uncertainties associated with analysis methods to be reduced.

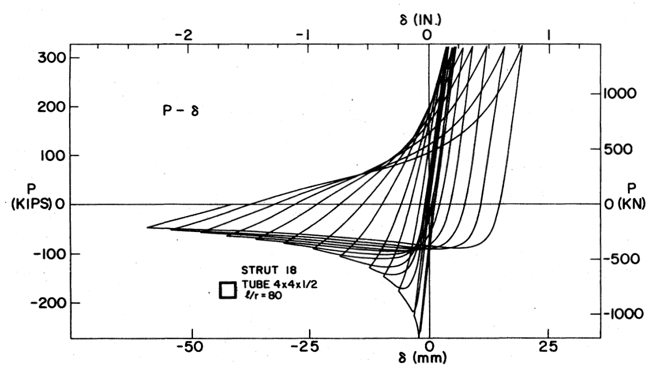
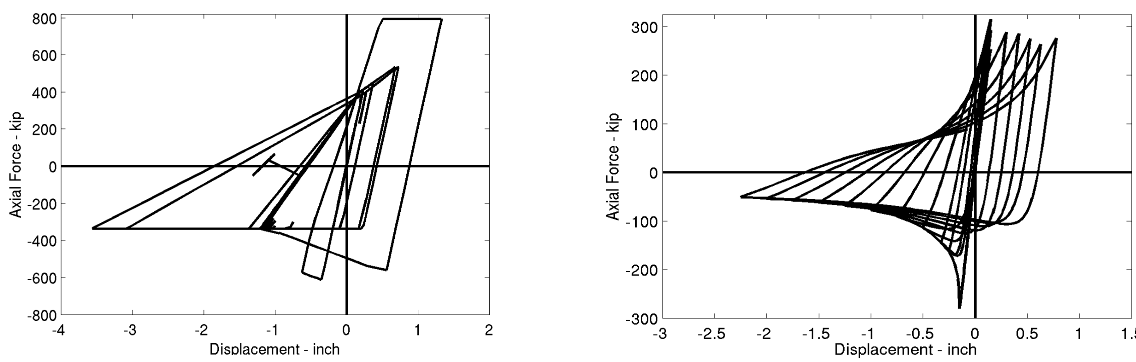


Figure 3. Experimental Test Results of Tubular Strut



(a) (b)

Figure 4. (a) Phenomenological Numerical Hysteresis (SNAP-2DX). (b) Tubular Strut Modeling in of Pin-Ended Truss Member in OpenSees

Table 3 lists the analysis uncertainty parameters that were used for the collapse prevention evaluations reported herein. Due to the preliminary nature of these investigations, these values are based on nonlinear analysis of SMRF structures. As such, these parameters are only approximate when applied to SCBF models, and additional testing and analysis is needed to refine these values.

Table 3. Analysis Uncertainty Parameters

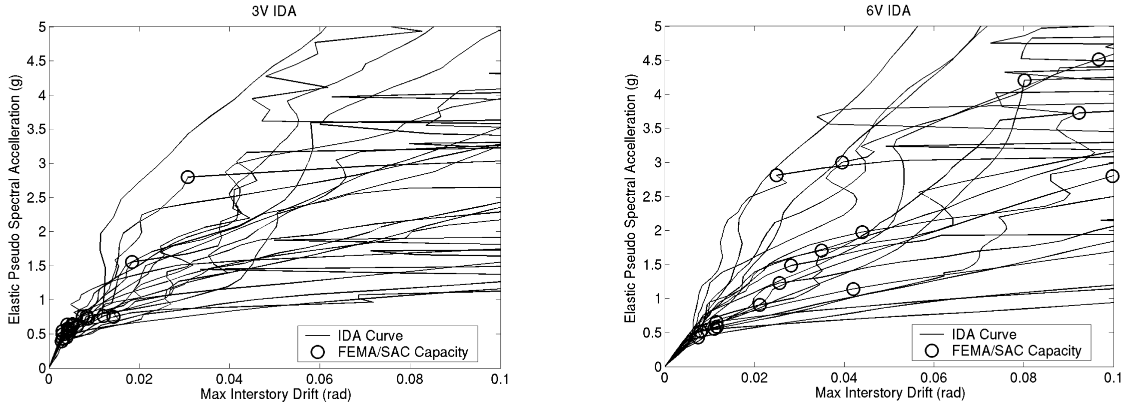
	β_{DU}	γ_a
3V	0.15	1.03
3VB	0.15	1.03
6V	0.20	1.06
6VB	0.20	1.06

Capacity Assessment

To estimate the seismic capacity of a SMRF, the FEMA/SAC guidelines suggest using an Incremental Dynamic Analysis (IDA) procedure. Default values are provided based on application of this method to various model SMRF structures. These default values are not appropriate for braced frame structures, so direct computation is necessary. The IDA method involves carrying out several nonlinear dynamic analyses, in which the intensity of the ground motion accelerograms considered are incrementally increased until a limit state "failure" is observed (Lee and Foutch, [13]). In the FEMA/SAC methodology, failure is defined when the rate of increase of peak interstory drift with increasing ground motion intensity exceeds five times that associated with an elastic system (or at a prescribed maximum interstory drift ratio beyond which the reliability of the analysis is considered doubtful (e.g., 10%)). Other criteria have been suggested by other investigators (Vamvatsikos and Cornell, [23]). To be consistent with procedures used for assessing SMRF, the criteria suggested in FEMA 351 (FEMA, [7]) are used herein.

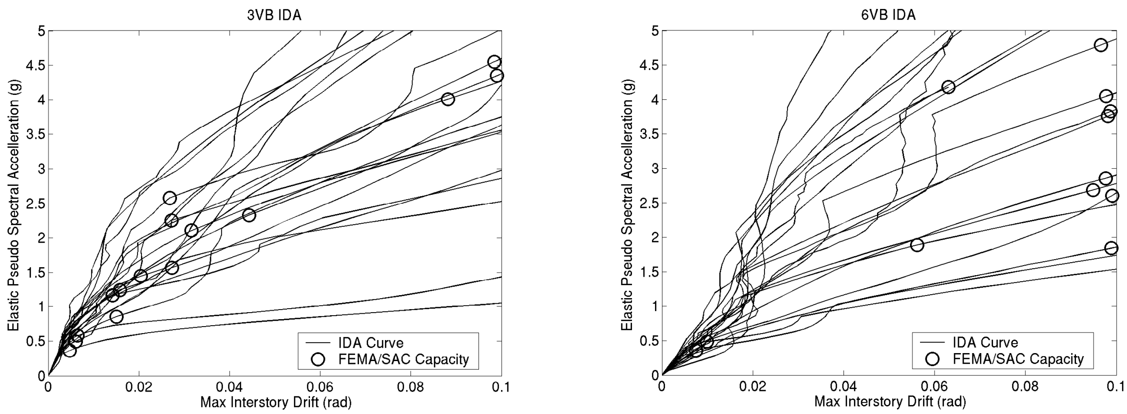
For this study, a total of 6400 nonlinear incremental dynamic analyses were computed using the UC Berkeley Millennium cluster, requiring approximately 20 hours to complete. Figure 5 and Figure 6 plot the results of the incremental dynamic analyses. In these plots, the peak inter-story drift obtained at any level from an inelastic analysis is plotted for a specific ground motion as a function of the intensity of the ground motion. Here the measure used to quantify the ground motion intensity is the pseudo-spectral acceleration of the scaled ground motion at the first mode period of the structure. The initial slope of each of the curves is defines as Se, and once the slope between one increment and the next is less than 0.2Se, then 'incipient collapse' is flagged. Each of the curves on these plots corresponds to one of the twenty FEMA/SAC ground motions from the 2% probability of exceedence in 50-year database. The circled points on the curves correspond to the ground motion intensity (and interstory drift) where the rate of increase in drift exceeds the criteria stated in the FEMA/SAC guidelines. Because a nonlinear dynamic analysis of the system is carried out in performing these analyses, and member yielding and failure are accounted for, these capacities are not based on the initial yielding or even failure of a single element, but rather on the situation where the rate of increase of lateral displacement of the overall system becomes excessive. This high rate is taken to be an indication of the onset of failure. As can be seen, the seismic capacity predicted in this way is different for each ground motion.

It should be noted that the analyses conducted included geometric nonlinearities, potential flexural yielding of beams and columns, and buckling and low-cycle fatigue related fracture of braces. However, beam to column and brace to frame connections were assumed infinitely ductile, and buckling or tensile failures in columns were disregarded. As such, several important modes of possible failure were not accounted for in these studies.



(a) (b)

Figure 5. Incremental Dynamic Analysis for (a) 3V and (b) 6V



(a) (b)

Figure 6. Incremental Dynamic Analysis for (a) 3VB and (b) 6VB

Figure 7 contains a plot of all of the analytically computed seismic capacities expressed as interstory drift for the three and six story buildings. The dispersion of the results obtained for different ground motions can clearly be seen in this figure. Table 4 lists the median capacities and logarithmic standard deviations of the seismic capacities. It is clear that the 3V frame has a relatively small seismic "capacity" based on this method (likely due to the larger number of cycles and higher drift demands imposed on this stiffer structure (see Figure 2 for comparison)). In fact, the predicted median capacity of the 3V frame is substantially smaller than the median demand. This does not necessarily signify that collapse or failure always occurs, due to the probabilistic scatter of demands and capacities. Consequently, a statistical interpretation of the demands and capacities is needed to assess the likelihood that the collapse prevention performance goal is violated. The analytical results of the BRBF frames are also shown in Table 4 and show substantial improvement in predicted median capacities.

Before proceeding to the assessment of the vulnerability of the system, it is useful to note that there are several aspects of this problem for SCBFs that differ significantly from that associated with SMRFs and BRBFs. Due to the sensitivity of interstory drift to brace fracture, the IDA curves obtained as outlined above are more complex than normally associated with most ductile moment-resisting frames. This is in part due to the large change in dynamic and mechanical properties that occur as a result of the deterioration and fracture of the braces, but also because braced frames are often significantly stiffer than moment frame structures and as such may not be in the so-called "displacement preserved" range of inelastic dynamic behavior. This fact has been previously noticed for low rise BRBFs where the peak drift demands are often larger than predicted by elastic analysis methods (Sabelli, [18]).

Moreover, it is noted that the IDA analyses can identify situations where the rate of increase interstory drifts with increasing ground motion intensity becomes quite large, but then decreases drastically before suddenly increasing again. This is an artifact of the methodology, as previously noted by Vamvatsikos and Cornell [23] and others for single-degree-of-freedom structures. In these cases, a pulse in a waveform may cause the structure to yield substantially in one direction. When a larger intensity is imposed, an earlier pulse in the record may cause the structure to yield in the opposite direction. This yielding may shield the structure from the effect of the later pulse that was critical for a lower intensity record. For instances where braces buckle or there is negative post-yield stiffness, this phenomena is exaggerated.

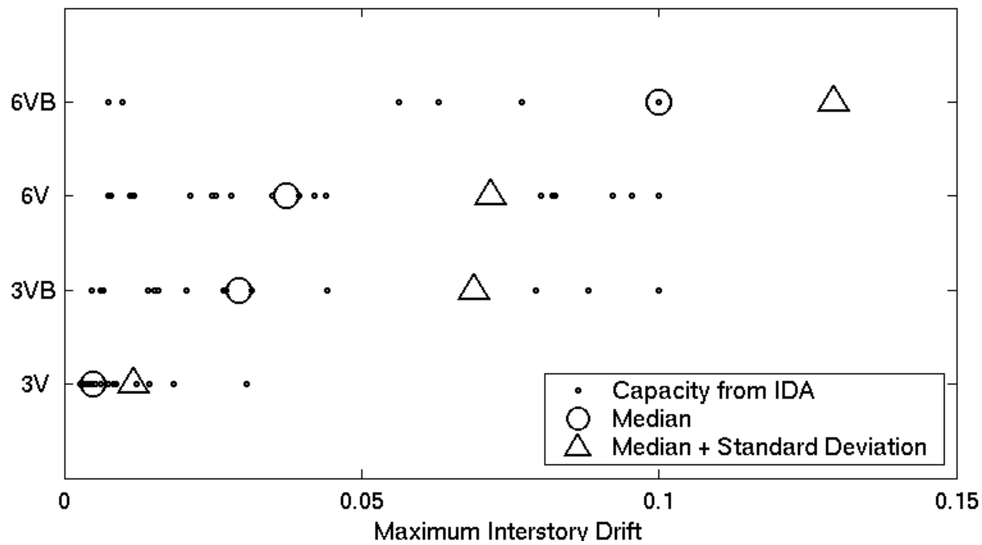


Figure 7. Incremental Dynamic Analysis Capacity Results for special concentric and buckling restrained braced frames

Table 4 summarizes the median capacities along with the standard deviation term $\beta_{CR.}$ It also includes a statistically based capacity reduction factor ϕ_R derived accounting for the variability in the computed capacity and statistics related to the aleatory randomness in ground motion hazard. The last column of this table shows the typical ϕ_R used with SMRF frames that is included in FEMA 351, clearly this value is much different than those found in these studies.

Similarly, using slightly different definitions of failure may lead to substantially different estimates of capacity and its associated reliability. Typically the value of the rate of increase of inelastic drifts to elastic drifts of 0.2 Se, as described above, is used to define the location on the IDA curve that defines capacity. For this reason a small study was performed to assess the sensitivity of this definition applied to BRBFs and SCBFs. Table 5 lists median capacity values for the same IDA data for varying values of the drift capacity slope: 0.17 Se, 0.15 Se, 0.13 Se and 0.1 Se. Table 6 shows the reduction factor calculated from these different definitions of capacity

Several other sources of uncertainty need to be accounted for in this PBEE methodology. For example, Table 7 includes assumed β_{CU} and ϕ_U values associated with inherent randomness in capacity estimates (taken from the FEMA/SAC guidelines). Because very few full-scale or nearly full-scale tests of modern SCBF component or subbassemblages have been compared to analytical predictions, actual uncertainty parameters are most certainly different from those assumed.

Table 4. Capacity Results and Randomness Parameters

	Median Drift Capacity (in/in)	Median Plus Capacity + Standard Deviation (in/in)	β_{CR}	ϕ_R	FEMA 351 ϕ_R
3V	0.0049	0.012	0.659	0.521	0.9
3VB	0.0296	0.069	1.045	0.194	0.9
6V	0.0372	0.072	0.897	0.299	0.85
6VB	0.1000	0.129	0.752	0.428	0.85

Table 5. Median Capacities For Varying Definitions of Capacity

	0.17 Se	0.15 Se	0.13 Se	0.10 Se
3V	0.0090	0.0123	0.0176	0.0263
3VB	0.0611	0.1000	0.1000	0.1000
6V	0.0567	0.0774	0.0853	0.0853
6VB	0.1000	0.1000	0.1000	0.1000

Table 6. Capacity Reduction Factors For Varying Definitions of Capacity

	ϕ_R (0.17 Se)	ϕ_R (0.15 Se)	ϕ_R (0.13 Se)	ϕ_R (0.10 Se)	FEMA 351 ϕ_R
3V	0.44	0.43	0.41	0.41	0.9
3VB	0.45	0.50	0.52	0.60	0.9
6V	0.47	0.48	0.46	0.60	0.85
6VB	0.70	0.73	0.76	0.93	0.85

Table 7. Physical Uncertainty Parameters

	β_{CU}	ϕ_U
3V	0.15	0.97
3VB	0.15	0.97
6V	0.20	0.94
6VB	0.20	0.94

Confidence Assessment

Now that all of the parameters for Equation 1 are determined, the next step is to evaluate the confidence parameter, λ, for these structures. Once that is determined, the standard Gaussian variate can then be computed by using Equation 2:

$$K_X = \frac{k\beta_{UT}}{2b} - \frac{\ln(\lambda)}{b\beta_{UT}} \tag{2}$$

where k is taken as 3.0 for the Pacific Northwest, California and Alaska, and b is assumed to be 1.0. β_{UT} is taken from Table 3-12 in FEMA-351, and accounts for various sources of uncertainty. The results of these calculations are shown in Table 8. Table 8 also includes the calculated confidence level of achieving the system level collapse prevention performance goal given ground motions consistent with a hazard level with a 2% in 50-year probability of exceedence. This calculation assumes that the capacity reduction factor (ϕ_R), is 0.9 and 0.85 for the three and six story structures, respectively. Based on this approach, the 3V structure has a particularly low confidence level, i.e., less than 1%. As noted previously, the median demand predicted for this structure is much larger than the median capacity predicted using the IDA approach. A 90% confidence recommended in the assessment of new SMRF structures, as such these values shown below are quite low. Table 9 illustrates the sensitivity to change in the confidence as calculated using the different definitions of capacity defined above.

Table 8. Confidence Parameters

	λ	β_{UT}	K_X	Confidence Level (%)
3V	28.	0.35	-9.0	<1
3VB	2.5	0.40	-1.7	4
6V	3.1	0.35	-2.7	<1
6VB	0.53	0.40	2.2	99

Table 9. Confidence Levels (%) For Varying Definitions of Capacity

	0.17 Se	0.15 Se	0.13 Se	0.10 Se	FEMA 351
3V	<1	<1	<1	<1	<1
3VB	5.3	45	49	63	4
6V	<1	<1	<1	9	<1
6VB	96	97	97	99	99

It is clear that there are a number of assumptions and approximations introduced in applying the FEMA/SAC reliability framework to the assessment of SCBF, and BRBF structures. However, the low confidence levels computed using methods consistent with those used to develop design criteria for SMRF structures, suggest that earlier concerns regarding concentric braced frames may be warranted, and that much more careful assessment of the seismic vulnerability of SCBF structures is needed. To do this, improved analytical models are needed to estimate demands, and further work is needed to identify realistic values of seismic capacities of SCBFs. In these studies, only global response parameters related to dynamic response have been investigated, and additional studies related other failure modes are required (i.e., column buckling or fracture, and local failure). To have confidence in results predicted using these methods, testing of individual braces, connections and realistic subassemblies or structures is also needed.

FULL-SCALE TESTING PROGRAM

Full-scale specimens, representative of typical SCBF construction practices in California, have been designed and detailed. The first of these, plan to be tested during the summer of 2004. Figure 8 shows a photograph of the first specimen to be tested. The two-story, one-bay frame was designed to have a similar capacity to a companion specimen having buckling-restrained brace elements oriented in a chevron pattern. Two other buckling-restrained braced frame specimens with single diagonal braces were tested in 2002. The conventional SCBF specimens being tested were designed using modern construction practices, and drawings have been reviewed by several practicing structural engineers.

The specimen is representative of a two-story building designed to 1997 AISC seismic provisions. The distance between the column centerlines is 20 feet (6 m), and the floor heights are nominally 10 feet (3 m) tall. The beams are considered pin-ended with a welded shear tab connecting the beam to the column. A single lateral load will be applied at the top of the structure by a 1.5 million pound (6.7 MN) capacity actuator. Out-of-plane restraints are provided for the columns and at the centerline of the beams representing likely field conditions.

These tests will help illuminate the performance characteristics of SCBF buildings. Several variations on detailing and proportions will be considered for future tests, as will OCBF detailing.

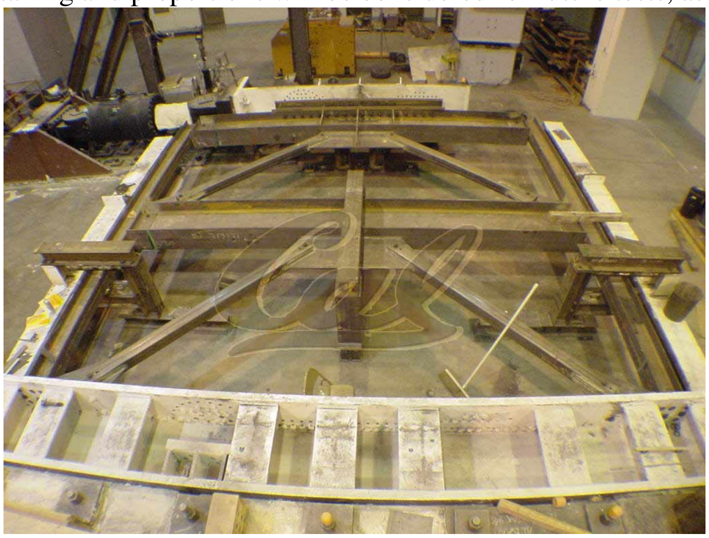


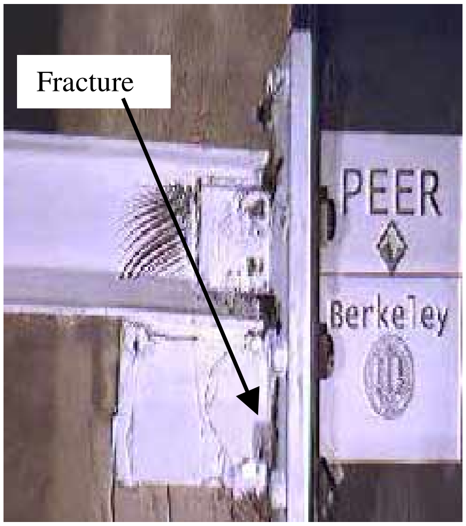
FIGURE 8. SCBF Test Specimen in Davis Hall

In addition to these tests, specimens are also being investigated to quantify better the low-cycle fatigue characteristics of tubular steel bracing elements. The low-cycle fatigue failure of the braces had a large impact on the computed performance of the model structures discussed earlier. Eight identical brace specimens, having the same effective length and other properties as those used in the SCBF subassemblage test, will be tested under different loading histories. In addition, several approaches to improving fatigue life will be assessed (e.g., filling with concrete). These tests will be used in conjunction with improved numerical models to help reduce uncertainties in predicted response associated with modeling assumptions.

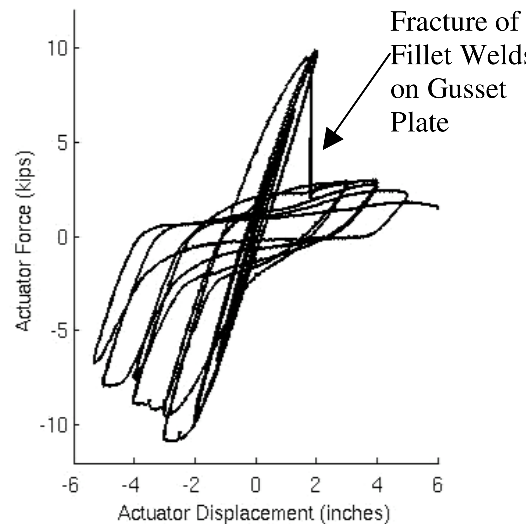
In addition, several tests are planned of "simple" beam to column connections with attached gusset plates. In addition to the complex mechanical behavior that may be exhibited by these connections, these studies are intended to examine the tendency of brittle fractures to initiate at the tip of gusset plates when plastic hinges form at these locations. This failure mode was observed in one of the previous BRBF tests, and frequently in SMRF connection tests employing cover plates. A reduced-scale pilot test has already been carried out at UC Berkeley, confirming the fracture susceptibility of these connections. Figure 9 shows a force-displacement hysteresis and a photo from these tests.

CONCLUSIONS

Performance-based earthquake evaluation methodologies like those implemented in FEMA 351 and elsewhere provide a consistent means for assessing the potential seismic performance of various forms of construction currently in use or proposed for use. As shown herein, application of these methods to SCBFs and BRBFs introduces various practical and theoretical problems that require further study. Nonetheless, the results presented suggest that low-rise SCBFs designed according to modern code provisions may be far more vulnerable to earthquake effects than generally accepted for SMRF structures. For instance, the confidence that a three story SCBF designed according to the 1997 NEHRP provisions is able to achieve the collapse prevention performance goal was less than 10% for all definitions capacity and a seismic hazard corresponding to a 2% probability of exceedence in 50 years. A similarly designed six-story BRBF was demonstrated to be much more reliable (greater than 60% confidence with all definitions of capacity). While these numbers must be viewed with considerable skepticism at this time due to the approximations introduced, the inelastic dynamic analyses showed that these structures were particularly sensitive to formation of weak stories due to the severe deterioration and even fracture of the braces, even at the 10% in 50-year hazard level. As such, considerable additional research, such as the improved analytical modeling and physical testing described herein along with the development of improved design details is highly recommended.



(a)



(b)

Figure 9. Simple Beam to Column Connection, (a) Photo and (b) Hysteresis

It is the goal of the authors to work towards a unified, performance-based evaluation approach for characterizing and improving the performance of steel braced frames incorporating conventional bracing, buckling restrained braces, friction and hysteretic devices, and viscous dampers. Clearly, achievement of this goal requires the collaborative participation of many professional engineers, researchers, fabricators, erectors, regulatory officials, and other experts.

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