A Comparison of Seismic Performance and Vulnerability of Buckling Restrained and Conventional Steel Braced Frames

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ABSTRACT :

A study is performed on the system level performance of Braced Steel Frames. Two types of braced frames are investigated: Conventional and Buckling restrained steel braced frames. A methodology to evaluate the structural response of the building in a probabilistic framework is used. The procedure to estimate the probability of exceedance of engineering demand parameters (EDP) conditioned on the ground motion intensity measure is applied on the structures. Emphasis is given to estimation of the probability of exceedance of peak Interstory Drift Ratios (IDR). The peak interstory drift ratio provides a way to estimate the damage to structural components and some of nonstructural components. For this purpose a three-story structure, followed the criteria developed by SAC for an analytical study of steel moment-frame buildings, designed with both bracing types is used. Response of the structures under earthquake excitation was defined performing nonlinear dynamic analysis using OpenSees Framework. Based on the dates from the nonlinear dynamic analysis of the structures, the seismic fragility of the structures is developed. The results are presented as families of conditional probability curves plotted against the ground motion intensity measure and compared for each certain limit state.

KEYWORDS: Non-Buckling and Concentrically Braced Frames, Performance-based earthquake engineering, Nonlinear Dynamic analysis, Fragility assessment, OpenSees framework

1. INTRODUCTION

The use of Buckling Restrained Braced Frames (BRBF) in lieu of Concentrically Braced Frames (CBF) is gaining popularity for new or rehabilitation projects in seismic prone zones. Compared to conventional braces, the BRB have the advantages of exhibiting a more stable hysteretic response. Since most of the studies only examined seismic demands on these braced frames, In this study, structural vulnerability and performance of Braced Frame structures is investigated by use of analytical fragility curves.

2. METHODOLOGY FOR DEFINITION OF FRAGILITY CURVES

The vulnerability of a certain structure is defined in the form of ground motion intensity versus damage relationship. These relationships yield the probability distribution of the occurrence of damage for a given earthquake intensity and are most frequently presented in the form of damage probability matrices and fragility curves.

A fragility curve describes the probability of reaching or exceeding a damage state at a specified ground motion level. Thus, a fragility curve for a particular damage state obtained by computing the conditional probabilities of reaching or exceeding that damage state at various levels of ground motion. A plot of the computed conditional probabilities versus the ground motion parameter describes the fragility curve for that damage state. The conditional probabilities are defined as follows:

$$P_{ik} = P(D \geq d_i | Y = y_k) \tag{2.1}$$

where P_{ik} is the probability of reaching or exceeding damage state d_i given that ground motion is y_k; D damage random variable defined on damage state vector $D = \{d_0, d_1, d_2, \dots, d_n\}$; and Y is ground motion random variable. Then the main point of defining the ground motion intensity versus damage relationship is to anticipate the structural response to excitation with a given intensity; whereas, there are two key solutions to be made: (i) selection of an indicator of ground motion Intensity Measure (IM) and (ii) selection of an indicator of response (i.e. damage measure (DM)).

2.1. Ground Motion IM

Beyond convention or convenience, the selection of an appropriate IM is driven by the "efficiency" and the "sufficiency" of the IM. The efficiency and sufficiency of an IM can depend not only on the type of ground motions considered (e.g., near-source versus ordinary), but also on the characteristics of the structure of interest. An efficient IM is defined as one that results in a relatively small variability of DM given IM (Shome & Cornell, 1999). A sufficient IM is defined as one that renders DM conditionally independent, given IM, of earthquake magnitude (M) and source-to-site distance (R) (Luco, 2002).

In the past, the Peak Ground Acceleration (PGA) of the earthquake was commonly used as an IM. However, this indicator reveals very little as opposed to other characteristics of the earthquake such as the amplitude and frequency content, the time duration of the stronger part of the earthquake, and the like. Attempts have been made by researchers to improve this situation by application of spectral acceleration (Singhal & Kiremidijan, 1997). Shome and Cornell (1998) have shown that for a low and moderate period steel frame, for which DM is maximum interstory drift, the 5% damped first-mode-period spectral acceleration $\left(S_a(T_1, 5\%)\right)$ will provide good estimates of the distribution of DM given IM.

2.2. Response Indicator, DM

In this investigation, the peak Interstory Drift Ratio was used as a parameter for expression of damage. The peak interstory drift ratio provides a way to estimate the damage to structural components and some of nonstructural components.

2.2.1 Definition of discrete damage states

Definition of damage state, which corresponds to pre-defined performance levels, influences extensively the ground motion intensity versus damage relationship; wherefore, it deserves to be paid particular attention. The damage state must be expressed in terms of response indicator.

Two different levels of performance are considered in this investigation: Immediate Occupancy (IO), Structural Damage (SD) and Collapse Prevention (CP). These levels are related to appropriate response measures in the buildings. Although FEMA-273 contains recommended performance levels for several construction technologies, most of the suggested limits are based on professional judgment and are believed to apply mainly to building construction in high-seismic zones. Only steel moment-resisting frames governed by seismic design requirements have been studied in detail in the recent SAC Project (Yun et al., 2002; Lee & Foutch, 2002).

Immediate Occupancy (IO): The Immediate Occupancy (IO) level is the state at which the building is safe to be occupied immediately following the earthquake with little or no repair. This requires that the structure remains essentially in the elastic range during the earthquake and that nonstructural components of the building are not damaged significantly. The IO performance limit given in FEMA-273/356 for steel frames, 0.5% IDR for braced-frames was used as the limit for this performance level.

Structural Damage (SD): In FEMA 273/356 (FEMA, 1997/2000), the intermediate damage state is identified as Life Safety. Relating this performance level to a response measure determined from structural analyses has proved to be problematic, as there is no obvious way of quantifying the ISDA or floor acceleration that threatens life safety; these limits clearly would depend on details of building construction. While the life safety limit would generally be expected to fall between IO and CP limits, it also depends on the building occupancy. For instance, floor accelerations only slightly above the IO level may well be dangerous for some occupants due to the unwanted movement of heavy objects and some non-structural components. As defined in FEMA-273 and FEMA-302, the life safety level is a performance state in which "significant" damage has been sustained, although some margin remains against either partial or total building collapse. SD is associated with a deformation of 1.5% IDR in this study.

Collapse Prevention (CP): The Collapse Prevention (CP) level is the point at which the structure can no longer support its own weight due to large P- Δ effects. This is accompanied by large interstory drifts and is manifested by non-convergence of the analytical model. The recommended limit for CP performance limit is 5% for CP in FEMA-273/356. Corresponding limits for steel moment frames which are not governed by seismic considerations, steel braced frames and other types of construction have yet to be determined. The CP performance limit, 2% IDR for braced frames, appears to be acceptable for ordinary concentrically braced frames (Kinali, 2007).

2.3. Selection of a set of representative earthquakes

An extraordinary important step in each methodology for definition of fragility curves is the selection of a representative set of earthquakes. Many studies have been performed on the number of response history analyses (RHA) required to provide sufficient accuracy in the estimation of seismic demands. Previous studies have shown that for mid-rise buildings, 10 to 20 records are usually enough (Shome & Cornell, 1999). Consequently, we have used a set of 20 ground motion records were those developed for use in the FEMA project on steel moment-resisting frames, SAC Steel Project (Somerville, 1997). These suits 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 specified 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 suits corresponding to downtown Los Angeles, California, were selected for seismic hazard levels corresponding to a 2%, 10% and 50%probability of exceedance in a 50 year period.

3. Definition of fragility curves

Cornell, et.al (2002) have shown that the variation of the median of the peak IDR with changes in the linear elastic spectral ordinate at the fundamental period of the structure is assumed to have the following form:

$$exp\left[\mu_{\ln(IDR)|S_a}\right] = a(S_a)^b \tag{3.1}$$

Where a and b are constants that control the slope and degree of nonlinearity can be determined by a regression analysis of Nonlinear Time Histories Analysis (NTHA) responses to ensembles of earthquake ground motion.IDR is distributed lognormally about Eqn. (3.1), with the standard deviation of $\sigma_{\ln(IDR)|S_a}$ (Shome & Cornell, 1999).

The probability of occurrence of the peak IDR conditioned to the occurrence of a given ground motion IM is assumed to be lognormally distributed as follows:

$$P(IDR|S_a) = \Phi\left(\frac{\ln(IDR) - \mu_{\ln(IDR)|S_a}}{\sigma_{\ln(IDR)|S_a}}\right) \quad (3.2)$$

where Φ is the cumulative normal distribution function and $\mu_{\ln(IDR)|S_a}$ is the mean of the natural log of the peak IDR, occurring at any story of the structure for a given spectral ordinate, and $\sigma_{\ln(IDR)|S_a}$ is a measure of dispersion computed as the standard deviation of the natural log of IDR.
The dispersion of the interstory drift is assumed constant with changes in the ground motion IM. Two recommendations have been proposed for the evaluation of this dispersion parameter. Luco and Cornell (1997) used a global measure of dispersion corresponding to the dispersion computed over a range of spectral ordinates. In another study, Luco and Cornell (1998) state that a dispersion corresponding to any given intensity level of interest can be used. In both cases the dispersion used in Eqn. 3.2 is assumed constant. In this study a global measure of dispersion corresponding to the dispersion computed over a range of spectral ordinates was used.

It follows that to compute the probability of reaching or exceeding a specific IDR is:

$$P(IDR|S_a) = 1 - \Phi\left(\frac{\ln(IDR/a(S_a)^b)}{\sigma_{\ln(IDR)|S_a}}\right) \quad (3.3)$$

4. MODEL BUILDINGS

Basic dimensions, weights and loads were adopted from FEMA/SAC model buildings. A three story braced frame building were designed for a site in metropolitan Los Angeles. The Building was designed according to the 1997 NEHRP Recommended Provisions for Seismic Regulation for New Buildings and other structures for both CBF and BRBF (FEMA 302/303). The building configuration and non-seismic loading conditions were identical to those utilized in the development of FEMA 350 guidelines for moment resisting. More detailed information about the model and design assumptions can be found in Sabelli (2000). Sizes of members determined for models 3v and 3vb are shown in table 4.1 and 4.2.

Table 4.1 Member properties for model 3v

Story	Brace HSS	Beams	Columns
3	6x6x3/8	W18x46	W12x96
2	8x8x1/2	W27x84	
1	8x8x1/2	W30x90	

Table 4.2 Member properties for model 3vb

Story	Buckling restrained Braces		Beams	Columns
	Tension Capacity (Kips)	Axial Stiffness (Kin/in)		
3	117	588	W14x48	W12x96
2	196	943		
1	243	1088		

5. ALYTICAL MODELING ASSUMPTIONS

Only a single braced bay was modeled and analyzed. Although the frames were not explicitly designed to be moment resisting, all beam to column connections with gusset plates attached 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 analysis. Columns were modeled as having a fixed base. The foundation was modeled as rigid; footing up-lift was not permitted. The floor level masses used in the analysis to account for horizontally acting inertia forces was taken as the total mass of the floor divided by the number of braced bays used in the building in each principal direction. Global $P-\Delta$ effects were considered based on this mass. 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 contribution 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 distribute loads across a story if 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 column were included to model the lateral stiffness of gravity system. In the analysis, the equivalent column was constrained to have the same lateral displacement as the braced bent.

The analyses were carried out using the nonlinear dynamic analysis computer program OpenSees software framework. OpenSees is a software framework with the library of materials, elements and analysis powerful tool for numerical simulation of nonlinear systems. Fiber representation of sections provides ability to simulate bi-axial bending as well as axial effects.

6. Fragility curves

The seismic demands were assessed using NTHAs on model 3v. Analysis of the response from each ensembles using Eqn. 3.1 yields:

$$IDR = 0.01007.S_a^{1.614} \;, \beta_{ln\,IDR|S_a} = 1.051 \quad \text{[2\%/50 year GMs]} \tag{6.1}$$

$$IDR = 0.00972.S_a^{1.634} \;, \beta_{ln\,IDR|S_a} = 1.110 \quad \text{[10\%/50 year GMs]} \tag{6.2}$$

$$IDR = 0.00712.S_a^{0.997} \;, \beta_{ln\,IDR|S_a} = 1.163 \quad \text{[50\%/50 year GMs]} \tag{6.3}$$

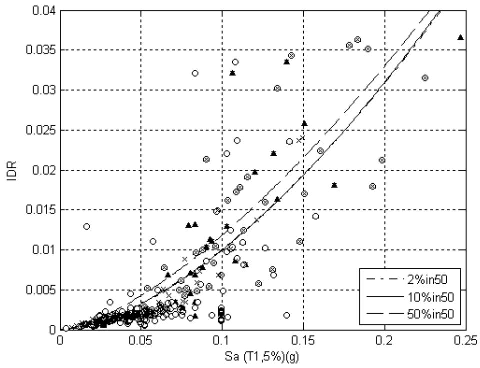


Figure 1 NTHA results for model 3v using each ensembles

The median relationships are similar. The study by Shome and Cornell (1998) also indicated that the relation between deformation and S_a was relatively insensitive to the ensemble selected, provided that accelerograms were selected from events of similar magnitude and distance and no near-field records were included. Similar median relationships will result in similar seismic fragilities are computed using the fragility defined in Eqn. 3.3. For example the seismic fragility of model 3v for CP performance level is presented in Fig. 2. Three different demand relations i.e., Eqn. (6.1), (6.2) and (6.3) were utilized in order to demonstrated the lack of sensitivity of the seismic fragilities to the choice of ground motion ensemble selected to generate these relationships.

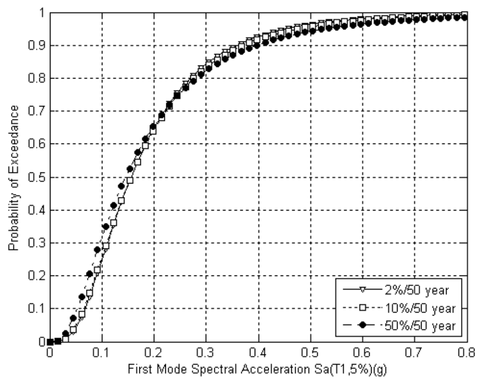
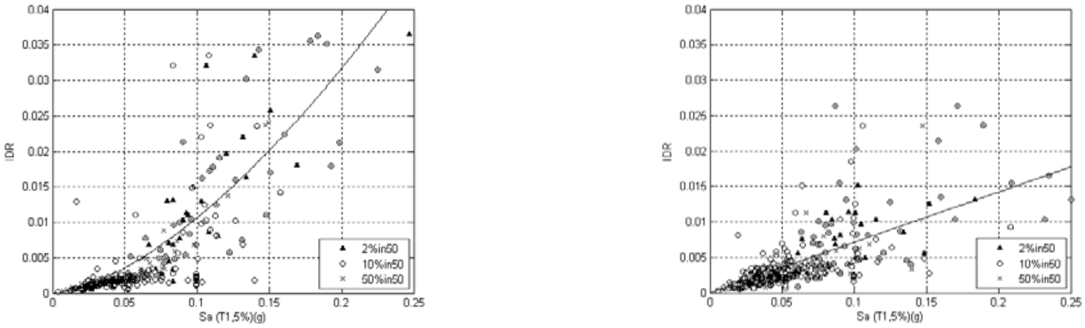


Figure 2 Seismic fragility results for model 3v for CP performance level using the relations in Eqn. (6.1), (6.2) and (6.3)

The results of the NTHAs for both models are presented in Fig. 3 using all ensembles. Analysis of the response from all ensembles using Eqn. 3.1 yields:



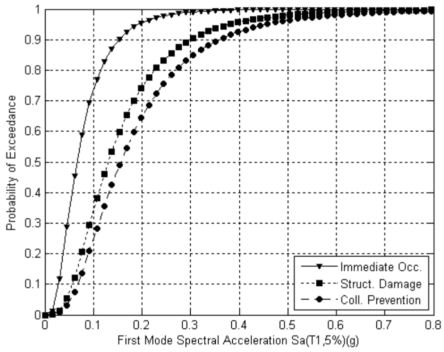
3v model 3vb model

Figure 3 NTHA results for models using all ensembles

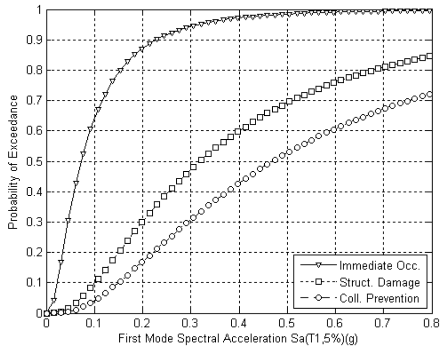
$$IDR = 0.01064.S_a^{1.575} \quad ,\beta_{ln\,IDR|S_a} = 1.122 \quad \text{[3v Model]} \tag{6.4}$$

$$IDR = 0.00712.S_a^{0.997} \quad ,\beta_{ln\,IDR|S_a} = 0.811 \quad \text{[3vb Model]} \tag{6.5}$$

The seismic fragilities of both model for three performance level (IO, SD and CP) are presented in Fig. 4 and Fig. 5, respectively for 3v and 3vb models. Demand relations (6.4) and (6.5) were used to compute the median capacity. A constant global measure of dispersion corresponding to the dispersion computed over results of seismic demand analysis.



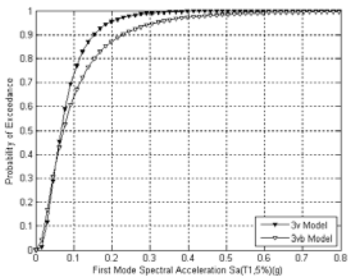
3v model



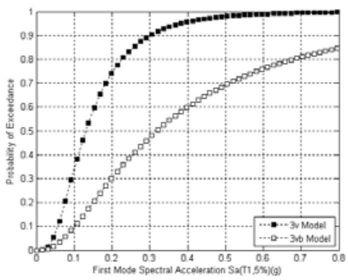
3vb model

Figure 4 Seismic fragility results for models

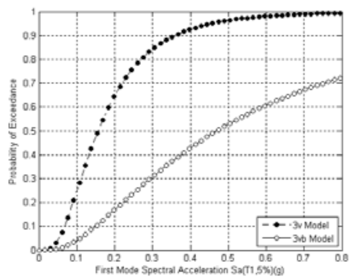
Comparison of seismic fragilities for models for three performance level is presented in Fig. 5.



IO performance level



SD performance level



CP performance level

Figure 5 Comparison of seismic fragility
results for models for three performance level

7. CONCLUSION

Building fragility assessment is an essential ingredient of many methodologies which developed to evaluate the probability of occurrence of a decision variable such as loss estimation. In this paper, steel braced frames with two different lateral load carrying system, conventional and buckling restrained braces were analyzed using ensembles of ground motions. Seismic fragilities are computed using the defined fragility and assessed for three performance levels for both models. Seismic demand relationships for these frames do not appear to depend on ground motion ensemble selected for purpose of analysis. It is shown that similar relationships will result in similar seismic fragilities. Comparison of seismic fragilities for different performance levels of models suggest that conventional braced frames are more vulnerable than BRBFs and is more likely to reach or exceedance of a performance level for a given ground motion intensity measure.

REFRENCES

Cornell C.A., Jalayer, F., Hamburger, R., Foutch, D.A. (2002). Probabilistic Basis for 2000 SAC Federal Emergency Management Agency Steel Moment Frame Guidelines. *Journal of Structural Engineering, ASCE* **128:4,** 526-533.

FEMA 273/356. (1997/2000). NEHRP Guidelines for the Seismic Rehabilitation of Buildings. Prepared by the Applied Technology Council for the Building Seismic Safety Council, Published by the Federal Emergency Management Agency, Washington, DC.

Kinali, K. (2007). Seismic fragility assessment of steel frames in the central and eastern United States. Dissertation for the degree of doctor of philosophy, School of Civil Engineering and Environmental Engineering, Georgia Institute of Technology.

Lee, K., Foutch, D.A. (2002). Seismic performance evaluation of pre-Northridge steel frame buildings with brittle connections. *Journal of Structural Engineering* **128:4**, 546-555.

Luco, N. (2002). Probabilistic seismic demand analysis, SMRF connection fractures, and near-source effects. Dissertation for the degree of doctor of philosophy, Dept. of Civil Engineering and Environmental Engineering, Univ. of Stanford university.

Luco, N. and Cornel, C.A. (1997). Numerical example of the proposed SAC procedure for assessing the annual exceedance probabilities of specified drift demands and of drift capacity. Internal SAC report.

Luco, N. and Cornel, C.A. (1998). Effects of random connection fractures on demands and reliability for a 3-story pre-Northridge SMRF structure. 6th U.S. national conference on earthquake engineering, Seattle, Washington.

Mazzoni, S., McKenna, F., Scott, M.H., Fenves, G.L., et al. (2007). OpenSees, Open System for Earthquake Engineering Simulation. University of California.

Sabelli, R. (2000). Research on improving the design and analysis of earthquake resistant steel braced frames. The 2000 NEHRP professional Fellowship Report, EERI.

Shome N., Cornell C.A. (1999). Probabilistic seismic demand analysis of nonlinear structures. Reliability of Marine Structures Program Report No. RMS-35, Department of Civil and Environmental Engineering, Stanford University, California.

Shome, N., Cornell, C.A. (1998). Earthquakes, records and nonlinear responses. *Earthquake Spectra* **14:3,** 469-500.

Singhal, A., Kiremidijan, A.S. (1997). A method for earthquake motion-damage relationships with application to reinforced concrete frames. Technical report NCEER-97-0008, State university of New York at Buffalo.

Somerville, P., et al. (1997). Development of ground motion time histories for phase 2, SAC background document SAC/BD-97/04, SAC joint venture, Sacramento, CA.

Song, J., Ellingwood, B.R. (1999). Probabilistic Modeling of Steel Moment Frames with Welded Connections. *Engineering Journal*, AISC **36:3,** 129-137.

Yun, S.Y., Hamburger, R.O., Cornell, C.A., Foutch, D.A. (2002). Seismic performance evaluation for steel moment frames. *Journal of Structural Engineering* **128:4,** 534-545.