SEISMIC RELIABILITY OF CONCENTRICALLY BRACED STEEL FRAMES

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ABSTRACT:

The purpose of this paper is to discuss the performance of concentrically braced steel frames that have been designed according to the current seismic provisions. To this end the performance of a six story concentrically braced frames utilizing conventional as well as buckling restrained braces are considered. A performance evaluation procedure based on nonlinear dynamic analysis and reliability method is used to estimate the probability of each structure exceeding the onset of collapse, as well as immediate occupancy performance levels in 50 years. The results of this study show that Special Concentric Braced Frames (SCBFs) designed according to current seismic provisions are not able to achieve the expected performance goals. On the other hand, Buckling Restrained Braced Frames (BRBFs) proved to be a more reliable alternative. However, there are concerns about performance of the latter frames against more frequent ground motions.

KEYWORDS: SCBF, BRBF, Seismic Performance, Incremental Dynamic Analysis

1. INTRODUCTION

The use of concentrically braced steel frames has become more popular in seismic regions of the world. Prior to publication of the 1994 Uniform Building Code, concentrically braced frames had been treated by building codes as elastic truss systems. Postelastic behavior was only considered in prescribing the reduced brace strength. This effectively increased the elastic force capacity of such systems, without addressing the postelastic modes of behavior. Damage to concentrically braced frames designed according to such requirements in past earthquakes, such as the 1985 Mexico, 1989 Loma Prieta, 1994 Northridge and 1995 Hyogo-ken Nanbu earthquakes, raised concerns about the performance of this class of structures. The problem was lake of ductile postelastic behavior. Because of these concerns, seismic design requirements for braced frames were changed considerably during the last decade and the concept of special concentric braced frames, with ductile design and detailing provisions, was introduced in the 1994 Uniform Building Code [1].

The category of special concentrically braced frames is intended to lead to a system in which braces can undergo several large cycles of buckling and yielding. Such systems are expected to be ductile and to perform well in earthquakes. But, recent analytical and experimental studies have shown that SCBFs are not able to perform according to the expectations. Conventional braces have unsymmetrical hysteretic behavior in tension and compression. They also exhibit substantial strength deterioration when loaded monotonically in compression or cyclically. Because of this, the actual distribution of internal forces and deformations often differ substantially from those predicted by conventional design methods. Design simplifications and practical considerations often result in the braces selected for some stories to be much stronger than required, while those in other stories having capacities very close to the design target. This variation in story capacity, together with potential strength losses, with some braces buckling before others, tends to concentrate the earthquake damage in a few weak stories. Such damage concentrations place even greater burdens on the limited ductility of conventional braces and their connections. At present, the low cycle fatigue issue is the most important concern of SCBFs. When braces buckle, either in-plane or out-of plane typically plastic hinges form that have great rotational demands and consequently undergo very large strain deformation histories. These strains can themselves cause local fracture due to low cycle fatigue. Local fracture of braces due to low cycle fatigue has been observed in some analytical and experimental studies [1, 2].

The disadvantage of the CBF system can be overcome if the brace is able to yield during both tension and compression without buckling. One means of achieving this, is through metallic yielding, where buckling in compression is restrained by an external mechanism. Hysteretic loops of Buckling Restrained Braces (BRBs) have nearly symmetric and ideal bilinear shapes, with moderate kinematic and isotropic hardening and a difference of about 10% between tensile and compressive strengths. Also inelastic deformation (ductility) capacities of BRBs are generally quite large, with cumulative cyclic inelastic deformations often exceeding 300 times the initial yield deformation of the brace before failure. Theses hysteretic qualities, while generally desirable, raise some concerns about the seismic behavior of buckling restrained braced frames. The lack of overstrength and low post-yield stiffness possibility could lead to a propensity to concentrate damage at one level. The behavior of BRBFs subjected to more frequent ground motions is also of concern because of the low overstrength the system possesses, as compared with the inherent overstrength of conventional braced frames or steel moment-resisting frames [1].

Performance evaluation and design of civil facilities against earthquakes is a challenge to engineers because of the large uncertainty in the seismic demand and capacity of structures. In view of the large uncertainties in both demand and capacity, the performance of the structural systems can be described meaningfully only when these uncertainties are taken into consideration explicitly. In other words, evaluation of the performance needs to be described in terms of reliability of the structural system against various limit states over a given period of time. In this paper we discuss the performance of special concentrically and buckling restrained braced frames that have been designed according to the current seismic provisions. The fragility-hazard format of performance evaluation procedure is used to estimate the performance of both SCBFs and BRBFs. The objective here is to derive the probability of exceeding the collapse prevention and immediate occupancy limit states in 50 years assuming randomness is the only source of uncertainty in the hazard and capacity variables.

2. MODEL BUILDING

To assess the performance of concentrically braced frames, a six-story building was designed with both conventional and buckling restrained braces, with a stacked chevron (inverted V) pattern. This building has a typical 3.4 m story-height. Its nominal dimensions are 30 m by 30 m in plan with five 6 m bays. 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. The braced bays are located on the perimeter of the building, in non adjacent bays. Both frame and non-frame columns are spliced at mid-height of the fourth story. Figure 1.1 shows the plan of the model building.

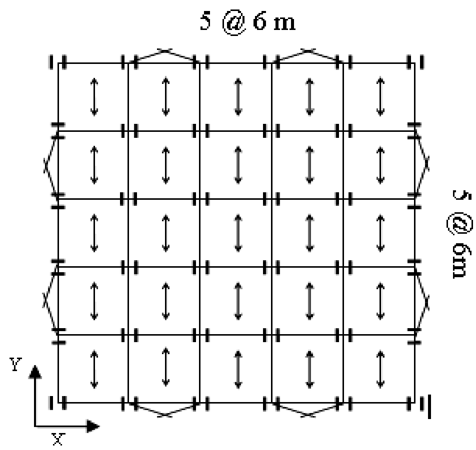


Figure 1.1 Plan of model building

The case study models were designed in accordance with the 2003 NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures (FEMA 450, [3]). The Load and Resistance Factor Design Specification for Structural Steel Buildings (ANSI/AISC 360-05, [4]) was also used. The equivalent static lateral force procedure used employed a response spectrum corresponding to a hazard having about 10% chance of exceedence in a fifty-year period. The Response Modification Factor, R, of 7 and 6 was used to design BRBF and SCBF systems. Unbonded Braces™ with JIS G3136 SN400B steel core were used for design of BRBF model. This steel corresponds to the Japanese standard and has a specified minimum yield stress of 38ksi. A572 Gr.50 steel was used for all beams and columns. Also, ASTM A500GradeB steel was used for conventional braces. The model building is assumed to be located near Holiday hotel site in Van Nuys, California. Holiday hotel is one of the PEER testbeds and there are extensive documents about the seismic hazard of this site [5]. The site condition is classified as NEHRP category S_D. The design spectral accelerations at short period (S_{DS}) and one second (S_{D1}) were 1.30g and 0.7g, respectively. For the determination of design forces, the building period and the force distribution over the building height were determined using the approximate methods provided in the NEHRP provisions rather than by employing the more realistic dynamic analysis. In the design of BRBF model, the yielding length of 70% of the brace length was assumed and the brace stiffness of 0.7 stiffness of yielding zone (consistent with current design practice). Based on earlier tests the compression strength was assumed to be 10% larger than the strength in tension [6]. In Both systems, frame beams and frame columns were designed using the maximum forces that could be delivered to the frame system based on the actual capacity of the braces. The member sizes determined for SCBF and BRBF models are shown in Table 1.1.

Table1.1 Member properties for BRBF and SCBF models

Story	BRBF			SCBF		
	Brace Core	**Beam**	**Column**	**Brace**	**Beam**	**Column**
6	10 cm^2	W14×38	W14×132	HSS5×5×3/8	W27×94	W14×132
5	18 cm^2	W14×38	W14×132	HSS5×5×3/8	W27×94	W14×132
4	25 cm^2	W14×38	W14×132	HSS6×6×3/8	W30×108	W14×132
3	28 cm^2	W14×38	W14×211	HSS6×6×3/8	W30×108	W14×211
2	31 cm^2	W14×38	W14×211	HSS6×6×1/2	W30×132	W14×211
1	34 cm^2	W14×38	W14×211	HSS6×6×1/2	W30×132	W14×211

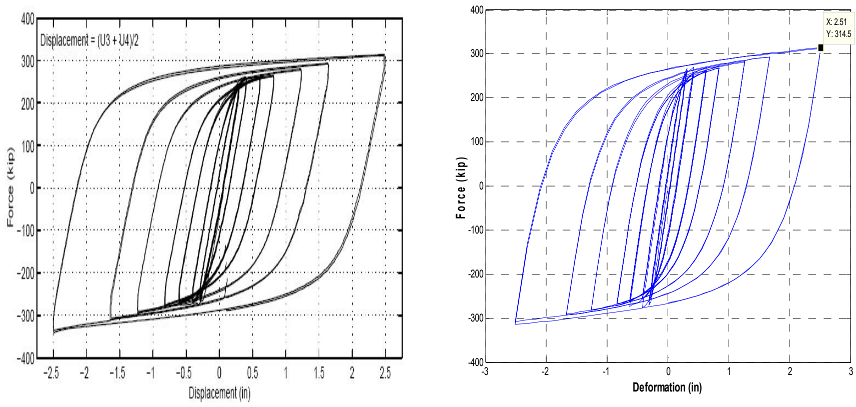
3. ANALYTICAL MODELS

Only a single braced bay in the X direction was modeled and analyzed for each system. Although the frames were not explicitly designed to be moment resisting, all beam to column connections with gusset plates attached were modeled as rigid connection. Columns were modeled as having a fixed base. The foundation was modeled as being rigid and foundation nonlinearities such as footing uplift were excluded from the analyses. Braces were modeled as pin-ended members. The contribution of the gravity load framing was approximated in the analysis using equivalent column member running the full height of the structure. This equivalent column was constrained to have the same lateral displacement as the braced bay. An effective viscous damping coefficient of 4.1% was assumed, according to the report by Goel and Chopra [7]. The floor level masses used in the analysis to account for horizontally acting inertia forces was taken as the total mass of 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 this mass.

The analytical model was created using the OpenSees (Open System for Earthquake Engineering Simulation) framework. OpenSees is an object-oriented and open-source software framework for simulating the seismic response of structural and geotechnical systems. It has been developed as the computational platform for research in performance-based earthquake engineering at the Pacific Earthquake Engineering Research (PEER)

Center. OpenSees is also the simulation component for the NEESit since 2004. OpenSees has advanced capabilities for modeling and analyzing the nonlinear response of systems using a wide range of material models, elements, and solution algorithms.

Buckling restrained braces were represented by truss element having steel02 (Giuffre-Menegotto-Pinto model with kinematic and isotropic strain hardening [8]) hysteretic model with properties selected to represent characteristics of BRBs observed in tests. Quite good agreement was obtained using this simple approach. For example, Figure 3.1(a) shows an analytically predicted hysteretic behavior of the test specimen of Figure 3.1(b).



(a) (b)

Figure 3.1 Simulation of hysteretic behavior of buckling restrained braces using Giuffre-Menegotto-Pinto steel material model (a) Test result [6] (b) OpenSees simulation result

For simulation of hysteretic behavior of conventional braces, the model proposed by Uriz was used [2]. This model consists of multi-element Nonlinear Beam-Column element with fiber discretization of the cross section and initial camber. Nonlinear Beam-Column element object is based on force formulation and considers the spread of plasticity along the element [8]. Figure 3.2 shows a schematic illustration of this proposed model.

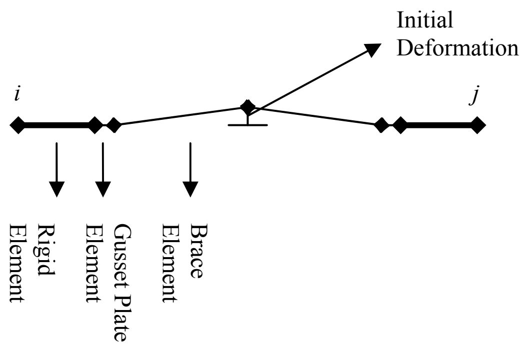
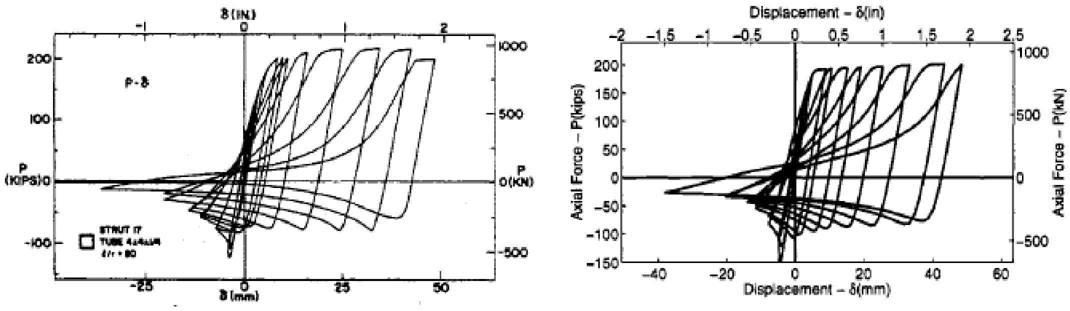


Figure 3.2 schematic illustration of the model proposed by Uriz for simulation of hysteresis of conventional braces

The proposed model can realistically represent the buckling strength, the post-buckling behavior, the tensile strength, the out-of-plane deformations and overall hysteretic behavior of braces with compact cross sections. Figure 3.3 shows the hysteretic behavior of an HSS 4×4×1/4 hollow strut and corresponding OpenSees simulation. In this study the brace members in SCBF model were subdivided into 6 Nonlinear Beam-Column elements and the initial camber of $1/500^{th}$ of brace lengths were specified at brace mide-span.



(a) Experimental (b) Analytical

Figure 3.3 Simulation of hysteretic behavior of conventional braces using model proposed by Uriz [2].

Beams and columns were modeled using Nonlinear Beam-Column elements with fiber discretization of the cross section. As a simplification, the equivalent column was given an area and moment of inertia equal to 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 X axes of the building.

As stated previously, the effect of low cycle fatigue is an essential ingredient in modeling of steel braces. In this study the effect of low cycle fatigue were modeled using OpenSees Fatigue material with calibrated parameters for conventional and buckling restrained braces, beams and columns. The fatigue material uses a modified rainflow cycle counting algorithm to accumulate damage in a material using Miner's Rule. Element stress/strain relationships become zero when fatigue life is exhausted [2, 8]. It should be noted that the median value of existing random variables such as material properties, viscous damping coefficient, etc were used in the analytical models. Figure 3.4 schematically illustrates the two dimensional special concentric and buckling restrained models analyzed in this paper.

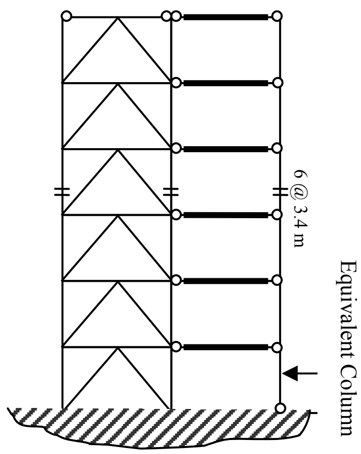


Figure 3.4 schematically illustration of analytical models used in this paper

4. SEISMIC HAZARD AND EARTHQUKE RECORDS

It has been demonstrated by Shome and Cornell that, for short- and moderate-period structures, the spectral acceleration at a period approximately equal to the fundamental mode of the structure is a suitable (sufficient and efficient) and scalable Intensity Measure (IM) [9]. Therefore, in this study, a 5%-damped spectral acceleration was used at the fundamental period of models (T_1 = 0.89sec and 0.6 sec for BRBF and SCBF models, respectively) as intensity measure parameter. For example, Figure 4.1 shows the median spectral acceleration hazard curve of the site for T=0.89sec. It is defined as the median annual frequency that the intensity of future ground motion events are greater than or equal to a specific value x and is denoted by $H_{S_a}(x)$.

$$H_{S_a}(x) = H[S_a \geq x] \tag{4.1}$$

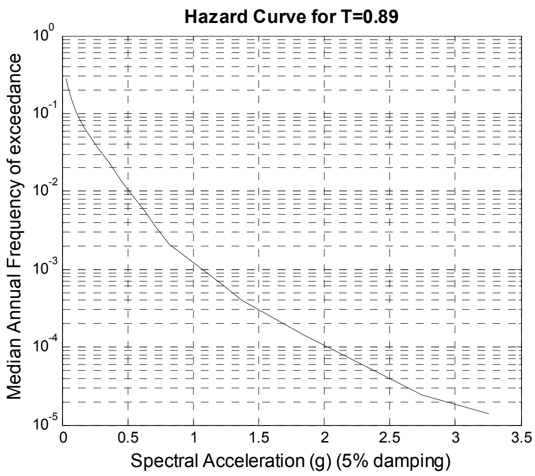


Figure 4.1 Hazard curve for T=0.89sec

Shome and Cornell have also shown that for mid-rise buildings, ten to twenty records are usually enough to provide sufficient accuracy in the estimation of seismic demands, assuming a relatively efficient IM, like $Sa(T_1,5\%)$, is used [9]. Consequently, a set of ten ground motion records were selected as listed in Table 4.1. Ground motions have been carefully selected from the Pacific Earthquake Engineering Research (PEER) Center strong motion database, where all ground motions have been processed with the same procedure. All ground motions were recorded in free-field sites in various earthquakes in California that can be classified as site class D, and bearing no marks of directivity.

Table 4.1 Detailed characteristics of earthquake ground motions used in this study

Record ID	Event	Year	Magnitude	Station	R	PGA (g)
G1	Imperial Valley	1979	6.5	El Centro Array #12	18.2	0.116
G2	Morgan Hill	1984	6.2	Gilory Array #7	14	0.113
G3	Whittier Narrows	1987	6	Compton-Castlegate St.	16.9	0.332
G4	Loma Prieta	1989	6.9	Gilory Array #4	16.1	0.212
G5	Northridge	1994	6.7	LA-Fletcher Dr.	29.5	0.24
G6	San Fernando	1971	6.6	LA-Hollywood Stor Lot	21.2	0.174
G7	Superstition Hills	1987	6.7	Westmoreland Fire Station	13.3	0.172
G8	Loma Prieta	1989	6.9	Palo Alto-SLAC Lab.	36.3	0.194
G9	North Palm Springs	1986	6	Palm Springs Airport	9.6	0.694
G10	livermore	1980	5.8	San Roman-Eastman Kodak	17.6	0.076

5. SEISMIC CAPACITY ASSEMENT

The capacity variable (limit for acceptable structural performance) is an important ingredient of Performance Based Earthquake Engineering (PBEE). Engineers have used inelastic static pushover analyses in the past to estimate this capacity. Vamvatsikos and Cornell extended the concept of pushover analysis to dynamic response in the form of Incremental Dynamic Analysis (IDA) [10]. IDA is a powerful method for assessing the structural capacity. 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.

In this study three limit states were defined for Collapse Prevention (CP) performance level. Capacity point defined as 1) the last point that the rate of increase of Maximum Interstory Drift Ratio (MIDR) with increasing ground motion intensity exceeds five times that associated with an elastic system or 2) the last point with 10% maximum interstory drift ratio, beyond which the reliability of the analysis is considered doubtful or 3) the last point that compressive Demand Capacity Ratio (DCR) of one of the columns reaches one. With appearance of each of these limit states the structure is no longer capable of satisfying collapse prevention performance level. Such a procedure has been used in the FEMA/SAC studies [11]. For the present study, a total of 1200 nonlinear time history dynamic analyses were computed. Figure 5.1 and Figure 5.2 plot the results of the incremental dynamic analyses. Each of the curves on these plots corresponds to one of the 10 ground motions listed in table 4.1. The circled points on the curves correspond to the capacity points of collapse prevention performance level. As can be seen, the seismic capacity predicted in this way is different for each ground motion.

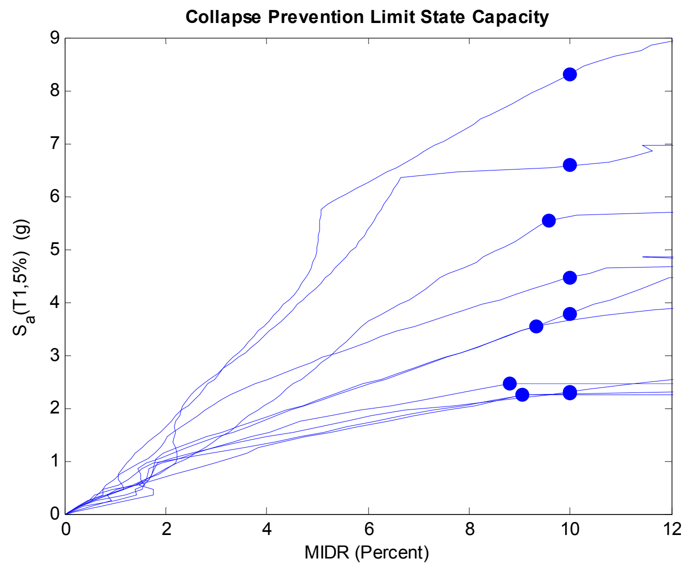


Figure 5.1 IDA curves and collapse prevention capacity points for BRBF model

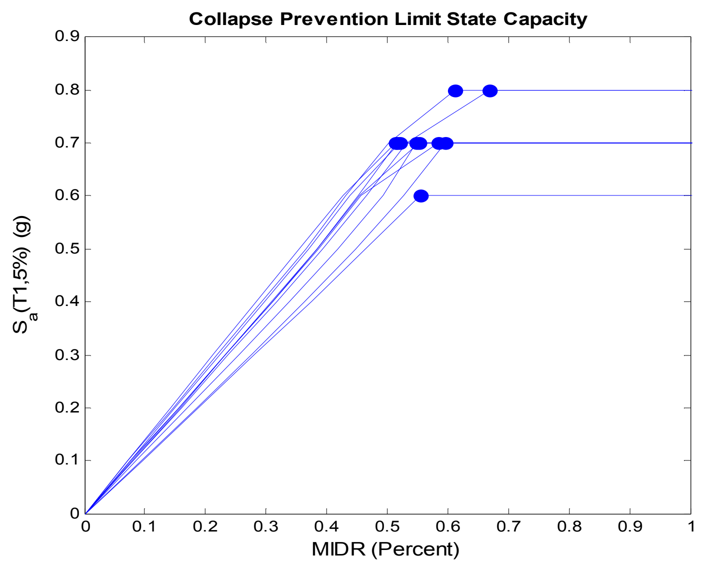


Figure 5.2 IDA curves and collapse prevention capacity points for SCBF model

As stated previously, the behavior of BRBFs subjected to more frequent ground motions is of most concern in the seismic performance of BRBFs. Figure 5.3 shows Immediate Occupancy (IO) capacity of the BRBF model. Capacity points were calculated assuming a maximum interstory drift ratio of 1% (approximately twice as elastic limit state) for limit state of immediate occupancy performance level.

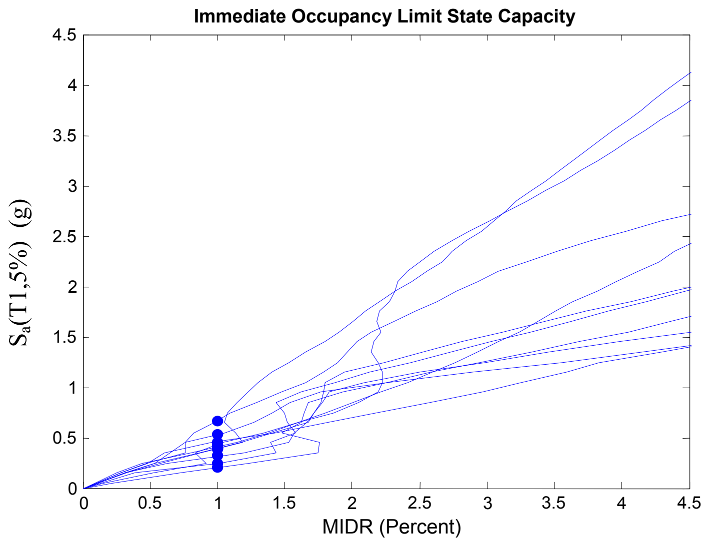


Figure 5.3 IDA curves and immediate occupancy capacity points for BRBF model

Figures 5.1 and 5.2 show that the BRBF model is able to undergo large inelastic deformations (often more than 10%) whereas, the SCBF model is not, due to fracture of conventional braces after buckling. In fact, predicted collapse prevention capacity of SCBF model is substantially smaller than that of the BRBF model.

6. SEISMIC PERFOMANCE ASSEMENT

The median annual frequency of exceeding a limit state can be calculated from Eqn. 6.1, where the first term under integral sign is the limit state fragility function.

$$\lambda_{LS} = \int F_{LS}(x).\left|d\widehat{H}_{S_a}(x)\right| \tag{6.1}$$

In addition, assuming a Poisson process, the probability of the limit sate in t years can be obtained by Eqn.6.2.

$$P_{LS,t} = 1 - \exp(-\lambda_{LS}.t) \tag{6.2}$$

The limit state Fragility curve can be defined as the probability that $S_{a,C}$ (spectral acceleration limit state capacity) is less than or equal to x (Eqn. 6.3).

$$F_{LS}(x) = P[S_{a,C} \leq x] \tag{6.3}$$

If it is assumed that the probability distribution of the spectral acceleration capacity, $S_{a,C}$, is lognormal with median, $\lambda_{Sa,C}$, and standard deviation of the natural logarithm, $\zeta_{Sa,C}$, fragility can be expressed in terms of the standardized Gaussian distribution function (Eqn.6.4).

$$F_{LS}(x) = \phi\left(\frac{\ln(x) - \ln(\lambda_{S_{a,C}})}{\zeta_{S_{a,C}}}\right) \tag{6.4}$$

The lognormal distribution is a logical selection for several reasons: 1) most of the individual capacity data has a skewed distribution with a longer tail for upper values, 2) capacity values are always positive and, 3) previous studies have associated the distribution of spectral acceleration and the response of a nonlinear structure (in terms of EDP) to lognormal distribution [9]. Also, in order to verify whether a lognormal probability distribution can be assumed, a Kolmogorov-Smirnov goodness-of-fit test was conducted in this study [12]. This test substantiated that the lognormal probability distribution assumption is reasonable for spectral acceleration limit state capacity with 95% confidence level. Table 6.1 includes lognormal distribution parameters of limit states capacity and the preliminary probability of limit states in 50 years for SCBF and BRBF models.

Table 6.1 Preliminary results of performance assessment of BRBF and SCBF models

Performance Assessment Results	BRBF Model		SCBF Model
	CP Limit State	IO Limit State	CP Limit State
$\lambda_{Sa,C}$	1.2496	-0.9749	-0.3454
$\zeta_{Sa,C}$	0.6884	0.4178	0.0804
λ_{LS}	0.000220	0.024136	0.012300
$P_{LS,50}$	1.09%	70.08%	45.85%

7. ASSESSMENT OF THE EFFECT OF EARTHQUAKE ATTACK DIRECTION

Median annual frequencies, Table 6.1, were calculated assuming that the earthquake direction is parallel to the X direction of the model building. However, the earthquake attack direction is a random variable; therefore, results of Table 6.1 are overestimates. The purpose of this section is to evaluate the effect of earthquake attack direction randomness on limit state probability of the models studied.

The models are perfectly symmetric and have no columns which form a part of two or more intersecting braced frame elements. So, the models can be assumed to behave in the X and Y directions, independently. In addition, supposing the same capacity in the X and Y directions of the model building, the median annual limit state frequencies considering earthquake attack direction randomness can be obtained by Eqn. 7.1; Where A is earthquake attack direction (Figure 7.1).

$$\lambda_{LS} = 4 . \int_0^{\pi/4} (\lambda_{LS} | \mathrm{A} = \alpha) . f_A(\alpha) . d\alpha \tag{7.1}$$

Conditional median annual limit state frequency given the earthquake attack direction of α, can be calculated by Eqn. 7.2.

$$\begin{aligned} &\textit{for}\ \ 0 \le \alpha < \pi/4 \\ &(\lambda_{LS}|\mathrm{A} = \alpha) = \int F_{LS}(x) . \left| d(\widehat{H}_{S_a}(x)|\mathrm{A} = \alpha) \right| = \\ &\int F_{LS}(x) . \left| d(\widehat{H}_{S_a}(x)|\mathrm{A} = 0) \right| . \cos\alpha = (\lambda_{LS}|\mathrm{A} = 0) . \cos\alpha \end{aligned} \tag{7.2}$$

Supposing uniform distribution for the earthquake attack direction, the probability density function of the earthquake direction, $f_A(\alpha)$, is equal to $1/\pi$. Therefore, Eqn. 7.1 can be replaced by Eqn. 7.3.

$$\lambda_{LS} = 0.900(\lambda_{LS}|A=0) \tag{7.3}$$

Eqn. 7.3 depicts that the randomness in earthquake attack direction can reduce the median annual limit state frequency about 10%. The proposed method can be developed for each structure with independent behavior in the principal directions. The results of performance assessment of BRBF and SCBF models, considering earthquake attack direction randomness are listed in Table 7.1.

Table 7.1 Results of performance assessment with considering earthquake attack direction randomness

Performance Assessment Results	**BRBF Model**		**SCBF Model**
	CP Limit State	**IO Limit State**	**CP Limit State**
λ_{LS}	0.000198	0.021722	0.011070
$P_{LS,50}$	0.98%	66.25%	42.51%

This table shows that the BRBF system is more reliable than the SCBF system in collapse prevention performance level. However, there are concerns about performance of this system in more frequent ground motions, such as those with a 50% chance of exceedence in 50 years.

8. CONCLUSIONS

Recent performance based seismic provisions specify that the performance objective for seismic use group I buildings should be less than 2% of the collapse prevention probability and 50% of the immediate occupancy probability in 50 years. The results of this study show that the collapse prevention probability of SCBF model is about 43%, a value much higher than the 2% recommended. The main reason is fracture of buckling braces due to low cycle fatigue. By comparison, the collapse prevention probability of BRBF model is less than 2%. Therefore, the BRBF model can more readily satisfy the collapse prevention criteria. On the other hand the results indicate that the immediate occupancy probability of BRBF model is 66%. This is of course greater than the expected value for such performance level. This validates the existing concerns about the performance of BRBFs when subjected to more frequent ground motions. The reason of the latter problem is the low overstrength at the system, as compared with the inherent overstrength of conventional braced frames, or steel moment-resisting frames.

A number of assumptions and approximations where introduced in this study for applying the reliability framework to the assessment of the concentrically braced frames. There are also a number of issues, such as characterization of the epistemic uncertainties and more realistic modeling of the beam-column gusset plate connections behavior, which could enhance the results of this study.

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