A Probabilistic Performance-Based Approach for Seismic Design of Buckling-Restrained-Braced Frames (BRBF)

Hamed Ebrahimian[1], Mohammad-Reza Banan[2], Mahmoud-Reza Banan[3]

[1] *Graduate Student, Dept. of Civil Engineering, Shiraz University, Shiraz, Iran*
Email: hamedeb@yahoo.com

[2] *Associate Professor, Dept. of Civil Engineering, Shiraz University, Shiraz, Iran*
Email: banan@shirazu.ac.ir

[3] *Assistant Professor, Dept. of Civil Engineering, Shiraz University, Shiraz, Iran*
Email: bananm@shirazu.ac.ir

ABSTRACT:

In this paper the principles of a proposed method for probabilistic performance-based seismic design (PPSD) are presented. The probabilistic design approach is proposed for traditional performance-based seismic design method using fragility-design curves. The PPSD method can be used for seismic evaluation and design of any structure. The PPSD procedure and its capabilities are illustrated for designing of a 5-story BRB frame. The PPSD can provide a thorough representation of conservatism and economics of a design for different performance levels assigned to a range of design measures.

KEYWORDS:
Probabilistic performance-based seismic design, Buckling-restrained-braced frame (BRBF), Fragility curve, Damage index

1. INTRODUCTION

Current seismic design provisions are generally based on deterministic seismic hazards where design measures such as strength and displacement are limited to allowable values. The economic losses caused by Earthquakes in California during 1990s were too large compared to main-stream design expectations. This brightly shows the need for development of more advanced seismic provisions to assure life safety as well as controlling the economic and social losses of earthquake events (Fajfar, 1997). In 1995 SEAOC Vision 2000 introduced the performance-based seismic design in which the building damage state could be engineered against different levels of ground motions for selected building performance levels.

Due to uncertainties in material and structural properties and earthquake ground motions deterministic seismic design and evaluation of a structure is risky especially for important structures. Reliability-based design takes into account such design uncertainties, as used in LRFD method for designing steel structures (Jalayer et al., 2003). In 1994 the SAC research group organized to reevaluate existing code provisions and construction practice for welded steel frames, combined performance-based and reliability-based design and developed a probabilistic seismic evaluation method as shown in FEMA-350(2000). In this method (Cornell et al. 2002), the probability that the demand exceeds specified limit state during structure life time is expressed as:

$$P(D > LS) = \int P_{D>LS}(MI)h(MI)d_{MI} \quad (1.1)$$

where D is the structural seismic demand (such as damage index, force, and deformation), LS is limit state associated to a structural performance level, MI represents earthquake intensity during design life of structure and $h(MI)$ is the probability density function of MI. The fragility function shows the probability that $P_{D>LS}(MI)$ demand exceeds specified limit state during an earthquake event with intensity MI. The determined probability distribution for each demand and response could be used for seismic evaluation and design of a structure. A probabilistic approach can be used for performance-based reliability-based design where exceedance probabilities for seismic hazard and certain limit states are defined for all performance levels. Furthermore such an approach can become deterministic by setting code-based predefined exceedance probabilities which are not project-specific.

In the following section such probabilistic performance-based approach will be explained and then illustrated for a 5-story steel frame with buckling-restrained-braces (BRB).

2. PROBABILISTIC PERFORMANCE-BASED SEISMIC DESIGN (PPSD)

The proposed PPSD method is a simple routine for designing a structure for the preferred seismic performance objective with selected seismic hazard risk level. Performance objective is defined by setting seismic hazard risk and limit states for main response demands for performance levels such as collapse-prevention, life-safety, and immediate occupancy levels. In PPSD limit states for each performance level is set by limiting the probability of exceedance for certain demands. The seismic hazard risk is project-specific for each performance level. The design demands can represent strength, deformations, ductility, damage index, and repairability depending on the project and its specified codes. In order to design a structure, satisfying the limit state for each required design demand may lead to different design where envelope of such designs should be used to satisfy all limit states.

The fragility function, $P_{D>LS}(MI)$ in Eqn. 1.1, is typically determined through Monte Carlo simulations for a given structure. There are predefined fragility curves used in scenario-based hazard studies which are not accurate for a specific structure. According to the probability distribution function of each considered design variable, a sample set of random values are generated. Using the generated values, a sample set of models are constructed. By analyzing each model, the discrete sample set of each design demand is determined and is used to determine the probability distribution and fragility function. The accuracy of the simulation is increased by increasing the number of generated models. Since such simulations are computationally intensive; the use of fragility functions in seismic evaluation and design is more often limited to important projects.

For PPSD the fragility curves should be developed for different design demands for different design levels. A design level can be simply defined by the design seismic base shear or can be complicated and defined by multi-variable design space for example defined by seismic and non-seismic base shears and structure and substructure ductilities. A set of fragility curves developed for different design levels (DL) are called fragility-design functions from this point forward and shown schematically in Fig. 1(a). For a design earthquake level (*MI*) specified by the seismic hazard risk and a desired reliability level defined by probability of exceedance, the design level (*DL*) of the structure is determined, as shown in Fig. 1(a).

Furthermore, fragility-design functions can be used to determine the probability of exceedance (reliability level) for an existing structure. Using the fragility curves for different limit states for a given design measure, one can generate for a given earthquake level (*MI*) a curve representing the probability of exceedance versus the limit state value (LS). By determination the design capacity for the given design measure of an existing structure, as shown in Figure 1(b), one can estimate the probability exceedance of the determined design capacity for the required earthquake intensity (*MI*). The determined exceedance probability can be compared with the required acceptable risk to decide if retrofitting is required for the given design measure.

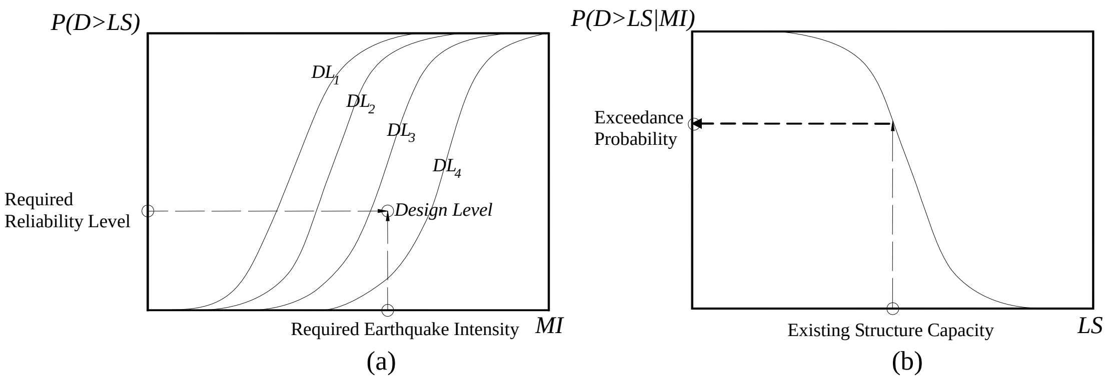


Fig. 1. Probabilistic performance-based seismic modeling
(a) Design process, (b) Evaluation process

3. DEVELOPMENT OF FRAGILITY-DESIGN FUNCTIONS FOR BRB FRAMES

In order to illustrate the proposed PPSD method for BRB frames, the fragility curves determined for a 5-story BRB frame are considered (Ebrahimian, et al., 2007). The geometrical properties of the studied frame are shown in Figure 3.1. The designed sections of the 5-story frame are shown in Figure 3.2 for the design level associated to code required base shear. The number of bays (*nb*) is 5. The added bays outside the braced bay are to assure that each frame bears maximum applicable code-based overturning moment. At beam-column-brace joints, the beam-column connections are assumed to be moment resisting because of rigidity caused by corner gusset plates. The frame is loaded according to Iran building code with seismic design based on IBC2000 and steel design according to AISC/LRFD (2005). The buckling-restrained braces (BRB) are designed according to AISC/SEAOC (2001). The seismic design parameters using IBC2000 are: Response modification factor $(R) = 8.0$, System over-strength factor $(\Omega_0) = 2.0$, Deflection amplification factor $(C_d) = 5.5$, Site class = *D*, and Five-percent structural damping design spectral response acceleration at short periods $(S_{DS}) = 0.769g$ and at 1-second period $(S_{D1}) = 0.55g$ which are selected to represent Shiraz seismic zone in Iran. Each frame is designed for three different strength levels of 80%, 100%, and 130% of the code required design base shear.

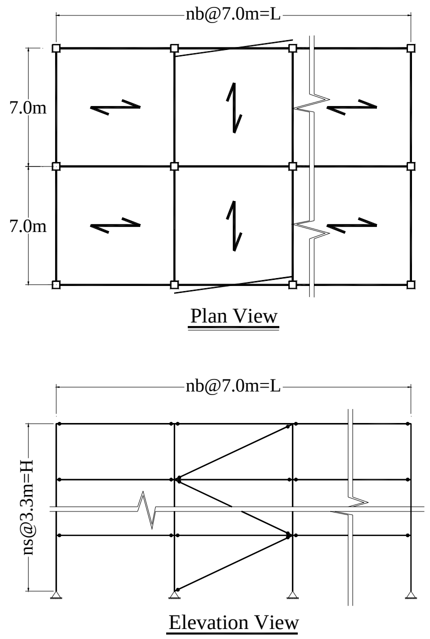


Figure 3.1. Geometrical properties of the Investigated BRBF

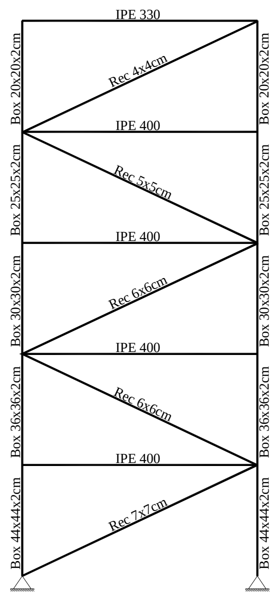


Figure 3.2. Designed sections of the 5-story BRBF at code-design level

A set of 11 earthquake records are used to consider the stochastic nature of ground motion time histories. The details of each record are shown in Table 1. These records are collected from PEER strong ground motion database and are scaled for peak ground acceleration (PGA) values in the range of 0.25g-1.6g. Nonlinear time history analyses are carried out by the Inelastic Damage Analysis of Reinforced Concrete program (IDARC 6.1). The force–displacement properties of beams and column (moment-curvature relations for beams and axial load-moment-curvature relations for columns) are characterized based on FEMA-356(2000) recommended curves. Axial force-deformation relation for BRBs is defined based on Bouc-Wen model. A bilinear hysteretic model is chosen for beams and columns plastic hinges and a smooth hysteretic model is used for BRBs. The damping ratio is set at 5% for the whole frame due to effects of non-structural elements. The P-Δ effects and rigid floor diaphragm assumptions are included in the modeling. About 330 nonlinear time history analyses are performed for the three type of frame in three strength level at different intensity levels of the 11 ground motions. The resulting damages in primary members (beams and columns), secondary members (braces) and in whole structure are evaluated using a damage index representing the change in displacement ductility ratio and hysteretic ductility ratio. Damage indices for members and the whole structure are based on Park-Ang damage Index (*DI*) (Park and Ang, 1985) as a sum of displacement ductility and hysteretic ductility ratios and is determined for member level and structure level as follows:

i^{th} member Damage Index: $$DI_i = \frac{x^i_{\max}}{x_{y,mon}} + \beta \frac{E_{hi}}{F_y x_{y,mon}} \tag{3.1}$$

Whole Structure Damage Index: $$DI_{total} = \sum_{i=1}^{\#of\ members} \lambda_i DI_i, \quad \lambda_i = \frac{E_{hi}}{\sum_{i=1}^{\#of\ members} E_{hi}} \tag{3.2}$$

where x^i_{max} is the i^{th} member maximum dynamic displacement demand, x_y is the member monotonic yield displacement, F_y is the member yield force, E_{hi} is the i^{th} member maximum hysteretic energy, and β is an adjusting parameter (set to be 0.15 for this study). The member damage index *DI* is a sum of displacement ductility and hysteretic ductility ratios.

Table 1. Selected ground motions

No.	Earthquake	Date	Magnitude (Ms)	Station Location	Component	PGA(g)	Source
1	Superstint	11/24/87	6.6	01335 El Centro Imp. Co. Cent.	000	0.358	PEER
2	Northridge	01/17/94	6.7	90057 Canyon Country-W Lost Cany	000	0.41	PEER
3	Loma Prieta	10/18/89	7.1	47380 Gilroy Array#2	090	0.322	PEER
4	Chi-Chi, Taiwan	9/20/99	7.6	TCU122	N	0.261	PEER
5	Loma Prieta	10/18/89	7.1	47380 Gilroy Array#3	090	0.367	PEER
6	Northridge	1/17/94	6.7	90053 Canoga Park-Topanga Can	196	0.42	PEER
7	Chi-Chi, Taiwan	9/20/99	7.6	CHY101	W	0.353	PEER
8	Superstitn	11/24/87	6.6	01335 El Centro Imp. Co. Cent.	090	0.258	PEER
9	Northridge	01/17/94	6.7	90053 Canoga Park-Topanga Can	106	0.356	PEER
10	Imperial Valley	10/15/79	6.9	5115 El Centro Array #2	140	0.315	PEER
11	Imperial Valley	10/15/79	6.9	5058 El Centro Array #2	230	0.38	PEER

For each ground motion, nonlinear time history analysis is done by incrementally increasing the motion PGA. Damage indices for members and structure are determined. Repeating the analyses for all selected 11 ground motions, a set of 11 values are estimated for each damage index for a specific PGA. Based on Monte Carlo simulation, each set of 11 values represents the probability distribution of the specified damage index which is treated as a random variable. A standard Normal distribution function is fitted to each set using mean and standard deviation of each set. The probability of exceedance for each earthquake PGA is calculated as follows:

$$P(DI > DI_i) = \int_{-\infty}^{DI_i} \frac{e^{-\frac{(DI-\mu_{DI})^2}{2\sigma_{DI}^2}}}{\sigma_{DI}\sqrt{2\pi}} d(DI)$$

(3.3)

Where DI_i is the required damage level, σ_{DI} is the damage index standard deviation as a function of PGA, μ_{DI} is the damage index mean as a function of PGA, and $P(DI > DI_i)$ is the probability of exceedance DI_i or fragility as a function of PGA.

By changing the structure design level the fragility-design curves are produced for design levels of 80%, 100%, and 130% of the required code design base shear which are shown in Figures 3.1 through 3.3. These curves show the primary members' maximum damage indices and the whole structure mean damage index plotted for different limit states defined by Damage Indices (DI) of 0.2, 0.4, 0.6, and 0.8 and the maximum story relative drift plotted for limit states defined by story drift of 10, 15, 25, and 30mm. The shown graphs are limited for 5S frame and the other results are omitted for abbreviation. It can be observed that the investigated BRBF has low risk of exceeding significant damage index values at reasonably high PGA values indicating highly ductile performance of BRBFs.

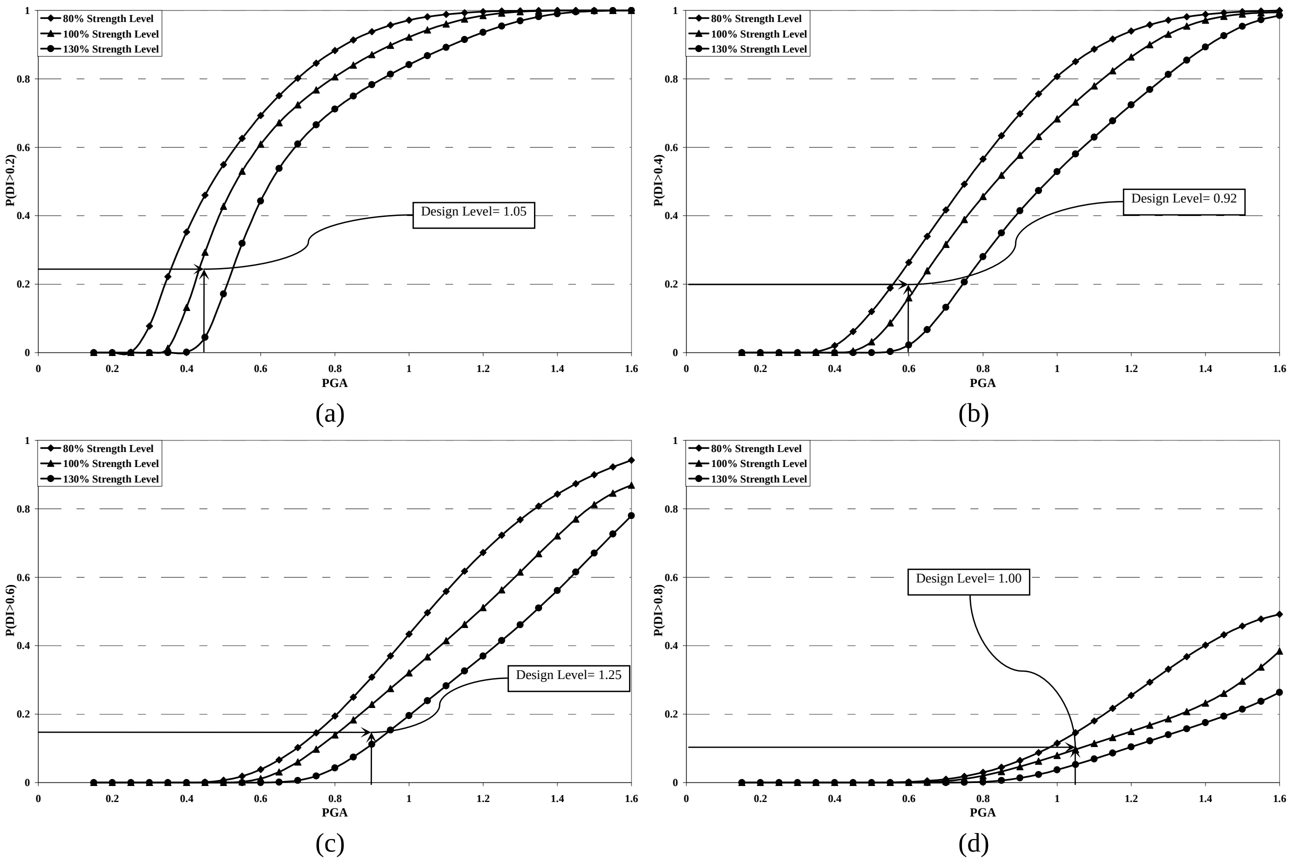
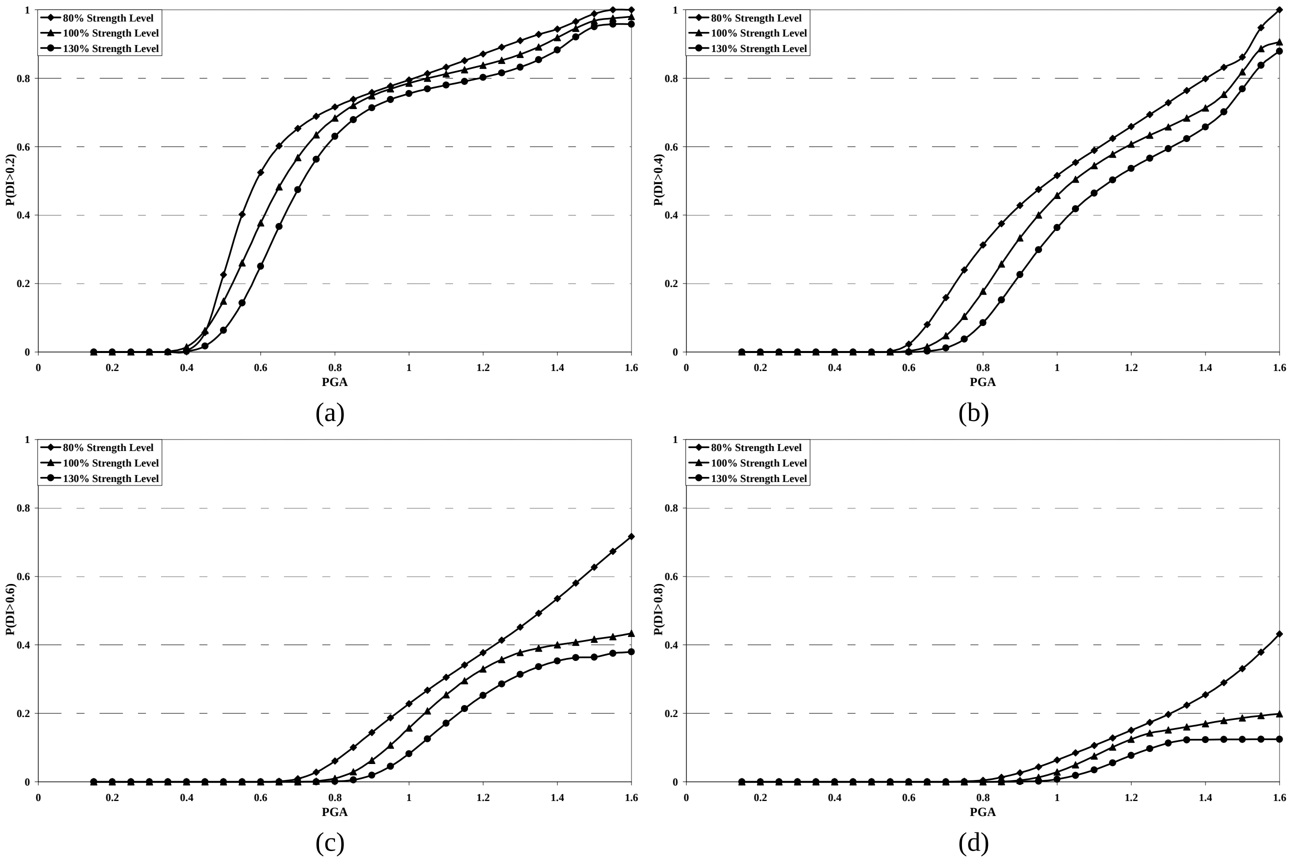
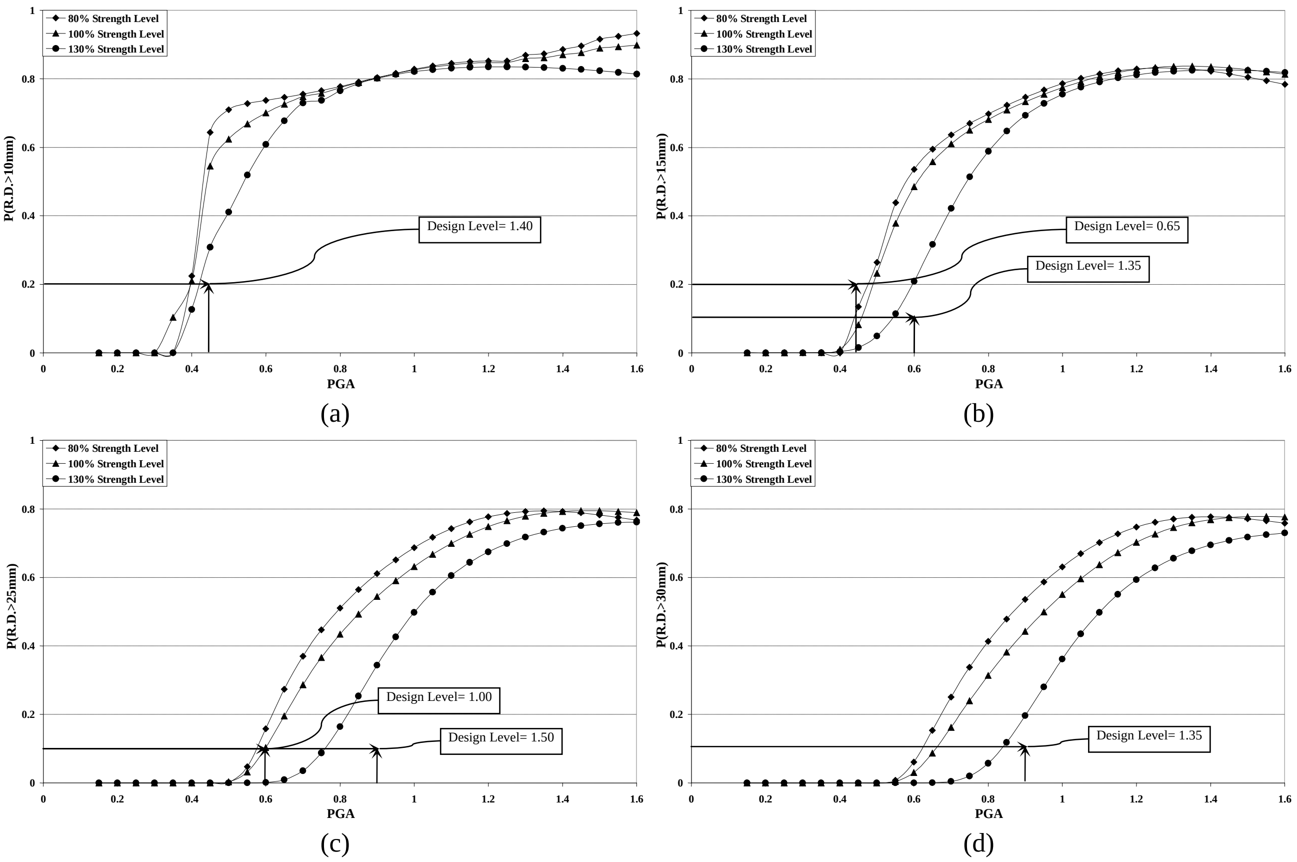


Fig. 3.1. Fragility-design curves for primary members' maximum damage index



(a) (b) (c) (d)

Fig. 3.2. Fragility-design curves for whole structure mean damage index



(a) (b) (c) (d)

Fig. 3.3. Fragility-design curves for maximum story drift (R.D.)

4. PPSD CASE STUDY

In this case study, the design of a 5-sory simple steel frame with BRBs is considered to illustrate PPSD method. The project-based desired performance objective is determined by seismic hazard levels and probability of exceedance for limit sates defined in Table 2. Based on seismic hazard study for the project site the ground PGA values for all seismic hazard levels are determined and shown in Table 3. Furthermore the quantitative performance levels are defined by damage indices for whole structure and primary members and also story drifts and are shown in Table 3.

Table 2. Performance objective and seismic hazard levels

Performance Level	Seismic Hazard Level in 50 Years (%)	Structural Performance Level	Acceptable Risk	Nonstructural Performance Level	Acceptable Risk
1	50	Immediate Serviceability	75%	Immediate Occupancy	90%
2	10	Immediate Occupancy	80%	Not Defined	90%
3	5	Life Safety	85%	Life Safety	95%
4	2	Collapse Prevention	90%	-	-

Table 3. Quantitative performance and seismic hazard levels

Performance Level	PGA	Primary Members Damage Index	Acceptable Risk	Maximum Story Drift (mm)	Acceptable Risk
1	0.45g	0.2	25%	15	10%
2	0.6g	0.4	20%	20	10%
3	0.9g	0.5	15%	30	5%
4	1.05g	0.8	10%	-	-

Using the estimated fragility-design curves for the 5-story BRB frame, for the determined quantitative performance levels, the design levels of the structure are determined as a multiplier of the IBC2000 required design base shear. Interpolation between fragility-design curves should be used to determine design levels for a given quantitative performance level at a given earthquake PGA. The summary of estimated design levels is shown in Table 4.

Taking into account the required reliability levels, it is observed that the structure designed at the code required base shear cannot performed to the performance levels required for damage indices of primary members and story drifts. The required design level to meet all required performance levels is 1.36 which means the structure must be designed for a base shear of 1.36 times the IBC2000 code required base shear.

Table 4. Required design levels

Performance Objective ID	Primary Members Damage Index	Maximum Story Drift
1	1.05	0.92
2	0.92	1.18
3	1.25	1.36
4	1.00	-

5. SUMMARY AND CONCLUSION

In this paper the principles of a proposed method for probabilistic performance-based seismic design (PPSD) are presented. The probabilistic design approach is proposed for traditional performance-based seismic design method using fragility-design curves. The PPSD method can be used for seismic evaluation and design of any structure. The PPSD procedure and its capabilities are illustrated for designing of a 5-story BRB frame. The PPSD can provide a thorough representation of conservatism and economics of a design for different performance levels assigned to a range of design measures.

REFERENCES

1. Fajfar, P., Krawinkler, H. (eds. 1997), Seismic Design Methodologies for the Next Generation of Codes, Balkema Publishers, Rotterdam.

2. Jalayer, F., Cornell, C.A. (2003). A Technical Framework for Probability-Based Demand and Capacity Factor Design (DCFD) Seismic Formats, *PEER Report No. 2003/08*, Pacific Earthquake Engineering Center.

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

4. FEMA-350 (2000), Recommended Seismic Design Criteria for New Steel Moment frame Buildings, Developed by SAC Joint Venture for Federal Emergency Management Agency, Washington, D.C.

5. Ebrahimian H., Banan, M. R., and Banan, M. R. (2007). Probabilistic-Based Seismic Modeling of Buckling-Restrained-Braced Frames (BRBF). *Proceedings of the Fifth International Conference on Seismology and Earthquake Engineering (SEE5)*, Tehran, Iran. May.

6. Ebrahimian H., Banan, M. R., and Banan, M. R. (2007). Comparison of fragility curves for steel frames braced with buckling-resisting braces and x-braces. Proceedings of Third National Congress in Civil Engineering, Tabriz University, Tabriz, Iran, May.

7. AISC-LRFD (2005), Load and Resistance Factor Design Specification for Steel Buildings, American Institute of Steel Construction Inc., Chicago, Illinois.

8. AISC/SEAOC (2001), Recommended Provisions for Buckling-Restrained Braced Frame, Structural Engineers Association of Northern California, Seismology and Structural Standards Committee.

9. IBC (2000), International Building Code, International Code Council, ICBO, Whittier, California.

10. PEER Strong Ground Motion Database, Available at http://peer.berkeley.edu/smcat.

11. FEMA-356 (2000), Perstandard and Commentary for Seismic Rehabilitation of Buildings, Federal Emergency Management Agency, Washington, D.C.

12. Park, Y.J., Ang, A.H.-S. (1985). Mechanistic Seismic Damage Model for R/C, *ASCE Journal of Structural Eng.* **111**, 722-739.