Seismic Reliability Assessment of RC Frame in a High Seismic Zone of India

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ABSTRACT

Uncertainties are common everywhere in day-today life. Deterministic approach will not be suitable always, in such cases Probabilistic approach yields better results. Characterizing the probabilistic nature of structural parameters can be done through the use of '*Fragility Curves*'. A fragility analysis assesses the probability that the seismic demand placed on the structure exceeds the capacity conditioned on a chosen Intensity Measure (*IM*), representative of the seismic loading. Demand (*D*) and capacity (*C*) are assumed to follow a lognormal distribution, and the probability of exceeding a specific damage state for a particular component can be estimated with the standard normal cumulative distribution function. In the present Study, Darjeeling region hazard Curves is chosen which is one of the most vulnerable zones in India. Hazard curve is the annual frequency of motion intensity at or above a given intensity level. It is expressed in complimentary cumulative distribution function (*CCDF*). Reliability assessment of RC structures is carried out by combining Fragility curve and Hazard curve. It is found that a building designed as per Indian code failed to achieve the Target Reliability. Although the present study ignored the contribution of infill walls, the building failed to achieve the target as the expected *PGA* corresponding to the target reliability is much higher that the design *PGA*.

Keywords– Peak Ground Acceleration, Intensity measure and Engineering Demand parameter

1.0 INTRODUCTION

The seismic performance assessment requires probabilistic approaches rather than deterministic approaches due to the uncertainties involved. The major uncertainties are in the material properties of concrete and steel, time history data, building geometries etc. The seismic performance of the buildings depends on the these uncertainties. A reliability analysis considering the possible uncertainties of the designed buildings can give a better insight on the performance of the buildings. In order to find our reliability, a fragility analysis is to conduct and combined with the hazard curve of the selected region. In this study a hazard curve of Darjeeling region, which is a high seismic zone in India.

1.1 Objective and scope:

This study examined the seismic reliability of reinforced concrete buildings situated in Darjeeling region in India. The scope of the present study is limited to consideration of uncertainties in material properties and Earthquakes. Latin hyper cube sampling is employed for the sampling of the strength of concrete and steel.

2.0 METHODOLOGY

2.1 Assessment of Seismic Reliability

A methodology for the assessment of seismic risk of building structures is presented [2]. This assessment involves three parts. First part is the identification of the seismic hazard, P [A = a], described by the annual probabilities of specific levels of earthquake motion. In this study, hazard curve developed by [3] for Darjeeling region is considered. Second part is the analysis of global response of the structural system. The response analysis of the structure is carried out by conducting a nonlinear time history analysis for different earthquakes, and the response is expressed in terms of maximum inter- storey drift at any store. Third part is the calculation of limit state probabilities of attaining a series of (increasingly severe) limit states, LS_i, through the expression:

$$P[LS_i] = \sum_a P[LS_i \mid A = a] P[A = a] \tag{1}$$

A point that estimate of the limit state probability for state i can be obtained by convolving the fragility $F_R(x)$ with the derivative of the seismic hazard curve, $G_A(x)$, thus removing the conditioning on acceleration as per Eq. (1).

$$P[LS_i] = \int F_R(x) \frac{dG_A}{dx} dx \tag{2}$$

The parameters of the fragility-hazard interface must be dimensionally consistent for the probability estimate to be meaningful. The reliability index for corresponding probability of failure can be found by the following standard Equation as shown below.

$$\beta = -\phi^{-1}(pf) \tag{3}$$

Φ^{-1} is the inverse standard normal distribution.

2.2 Seismic Hazard Analysis

The seismic hazard at a building site is displayed through a complimentary cumulative distribution function (CCDF). The hazard function is the annual frequency of motion intensity at or above a given level, x, to the intensity. Elementary seismic hazard analysis shows that at moderate to large values of ground acceleration, there is a logarithmic linear relation between annual maximum earthquake ground or spectral acceleration, and the probability, $GA(a)$, that specifies values of acceleration are exceeded. This relationship implies that A is described by the following equation,

$$G_A(x) = 1 - \exp[-(x/u)^{-k}] \tag{4}$$

u and k are parameters of the distribution. Parameter k defines the slope of the hazard curve which, in turn, is related to the coefficient of variation (COV) in annual maximum peak acceleration.

3. Hazard Curve of Darjeeling Region in India

The hazard curve of Darjeeling region, which is one of the most vulnerable earthquake prone areas, is developed by [3] Seismic hazard curves in terms of PGA for a 2500 year return period (2% exceedance probability in 50 years and 10% exceedance probability in 50 years) for Darjeeling obtained from the probabilistic seismic hazard analysis (PSHA) are shown in Fig. 1. The PGA considered for the evaluation of the probability of failure and reliability index is upto 0.83g for 2% in 50 years of exceedance probability and 1.6g for 10% in 50 years of exceedance probabiliyy as shown in Fig .1.

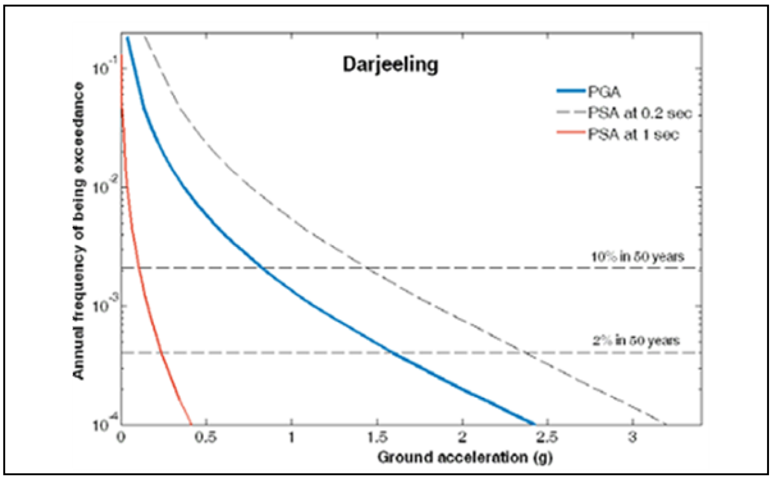


Fig 1 Seismic hazard curves of the Darjeeling region [4]

4. Development of Fragility Curves

A fragility function represents the probability of exceedance of the selected Engineering Demand Parameter (EDP) for a selected structural limit state (DS) for a specific ground motion intensity measure (IM). These curves are cumulative probability distributions that indicate the probability that a component/system will be damaged to a given damage state or a more severe one, as a function of a particular demand. Fragility curves damaged to a given damage state or a more severe one, as a function of a particular demand. Fragility curve can be obtained for each damage state and can be expressed in closed form as using “(5),”

$$P(C-D\leq 0\mid IM)=\phi\left(\frac{\ln\dfrac{s_d}{s_c}}{\sqrt{\beta_{d|IM}^{2}+\beta_c^{2}}}\right) \tag{5}$$

where, C is the drift capacity, D is the drift demand, S_d is the median of the demand and Scis the median of the chosen damage state (DS). $\beta_{d/IM}$ and β_c are dispersed in the intensity measure and capacities respectively. Equation “(5)” can be rewritten as “(6)” for component fragilities [5] as,

$$P(DS \mid IM) = \phi\left(\frac{\ln IM - \ln IM_m}{\beta_{comp}}\right) \tag{6}$$

where, "IMm =(exp (ln Sc –ln a))/b" , a and b are the regression coefficients of the probabilistic Seismic Demand Model (PSDM) and the dispersion component, β_{comp} is given as,

$$\beta_{comp} = \sqrt{\frac{\beta_{d|IM}^2 + \beta_c^2}{b}} \tag{7}$$

The dispersion in capacity, β_c is dependent on the building type and construction quality. For β_c, [5] 50% draft suggests 0.10, 0.25 and 0.40 depending on the quality of construction. In this study, dispersion in capacity has been assumed as 0.25. It has been suggested by [6] that the estimate of the median engineering demand parameter (*EDP*) can be represented by a power law model as given in "(8)".

$$EDP = a(IM)^b \tag{8}$$

In this study, inter-storey drift (δ) at the first floor level (ground storey drift) is taken as the engineering damage parameter (*EDP*) and peak ground acceleration (*PGA*) as the intensity measure (*IM*).

4.1 Ground Motion Data

The number of ground motions required for an unbiased estimate of the structural response is 3 or 7 as per [7]. However, [6] draft recommends a suite of 11 pairs of ground motions for a reliable estimate of the response quantities. A pair of twenty two Far-Field natural ground motions are collected from [8]. These are converted to match with Indian spectrum [9] using a program, WavGen developed by [10]. Fig. 2 shows the Response spectrum for converting ground motions along with Indian spectrum.

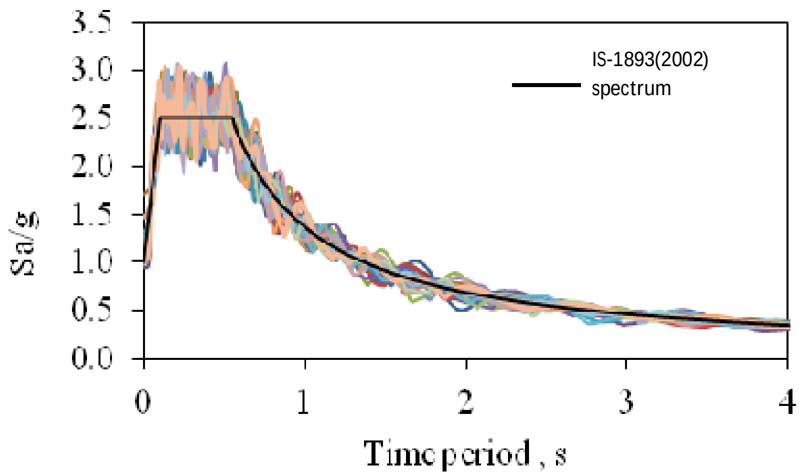


Fig 2. Response Spectra for 44 converted ground motions along with IS 1893 (2002) design spectrum

4.2 Building Design

The building frames considered for numerical analysis in the present study are located in the Indian seismic zone V with medium soil conditions. These frames are designed as an Ordinary moment resisting frames, seismic loads are estimated as per [9] and the design of the RC elements is carried out as per [11] standards. The characteristic strength of concrete and steel were taken as 25MPa and 415MPa. The buildings are assumed to be symmetric in the plan. Typical bay width and column height in this study are selected as 5m and 3.2m respectively for all the frames. The building configuration is chosen for the present study is 4 storeys 4 bays.

4.3 Sampling

Material properties of concrete and steel used in the construction are random in nature. To incorporate the uncertainties in concrete, steel and masonry strength, a Latin Hypercube sampling scheme is adopted using [12] program. The mean and covariance values for concrete and steel are taken [13] and shown in table 1.

Table 1: Details of random variables used in LHS scheme

	Variable	Mean	COV (%)	Distribution	Remarks
Concrete	f_{ck} (MPa)	30.28	21.0	Normal	Uncorrelated
Steel	f_y (Mpa)	468.90	10.0	Normal	Uncorrelated

4.4 Modelling, Analysis and Performance Levels

The 44 models are considered for each case, which is modelled in Opeensees for nonlinear analysis. Concrete is modelled as per [15] and reinforcements using a bilinear steel model with kinematic Strain hardening. Newmark's Beta Method is adopted for the time step analysis. Three performance levels, Immediate Occupancy (IO), Life safety (LS) and collapse Prevention (CP) are considered in the present study. The inter-storey drift (Sc) corresponding to these performance levels has been taken as 1%, 2% and 4% respectively as per [16].

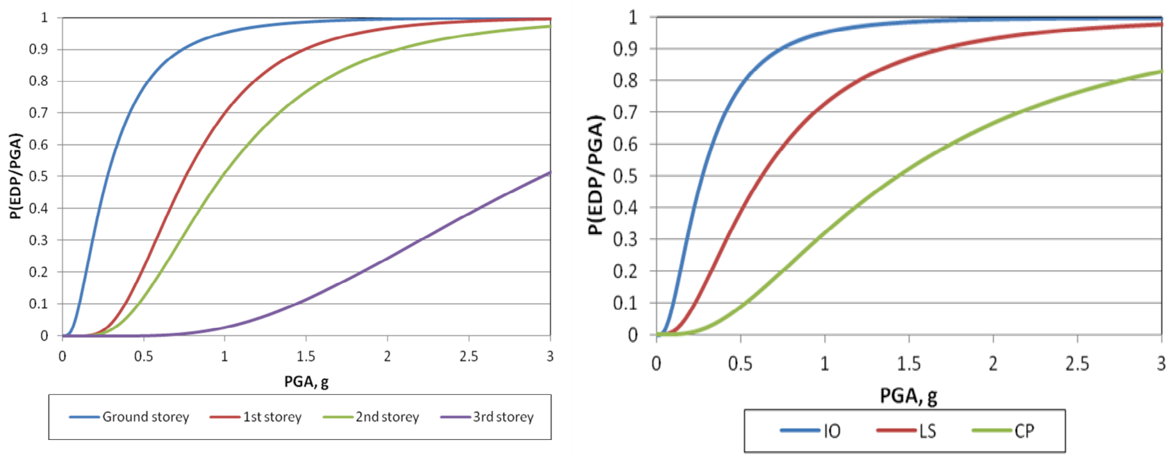
5. RESULTS AND DISCUSSIONS

5.1 Fragility Curves For Building Frames

Fragility curves for 4S4B building frames for three performance levels, namely, IO, LS and CP are generated. The variation of the exceedance probability of the inter-storey drift with the PGA for the 4S4B for the performance level of Immediate Occupancy (IO) obtained is shown in Fig. It is found that Ground storey is more vulnerable compare to other storey inter-storey drift. Fig.3b. Shows the variation of exceedance probability of Ground Interstorey drift with PGA for different performance levels.

5.2 Assessment Using The Reliability Indices

The Target Reliability Indices in accordance with [17] is used in the present study. The target reliability requirement for each performance level (consequences of failure) for each relative cost of measures is shown [17]. The assessment of performance of each building is carried out by comparing the reliability indices obtained for building with corresponding target reliability indices corresponding to moderate level of relative cost measure. The target reliability indices are taken as 2.3, 3.1 and 3.8 respectively, for the performance levels IO, LS and CP.



a) Fragility Curve for 4S4B IO performance level for different storey levels

b) Fragility Curve for 4S4B performance levels for Ground Storey

Fig 3. Fragility Curves for 4S4B frame

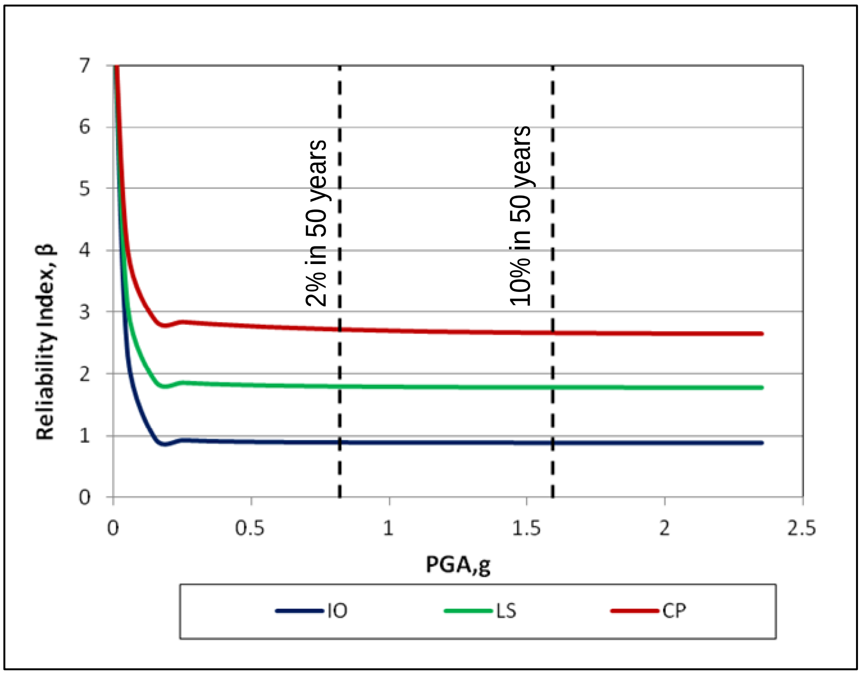


Fig 4. Reliability Curves for 4S4B frame

Table 2: Probability of Failure and Seismic Reliability of the frames for each Limit states

Building Frame	Performance Level	Probability of Failure	Reliability Index	Target Reliability[17]
4S4B	IO	0.18943	0.88	2.3
	LS	0.03754	1.78	3.1
	CP	0.00402	2.65	3.8

The probability of failure of each frame and corresponding reliability indices are presented in the Table.2. The values of reliability indices for a 4S4B frame are 0.88, 1.78 and 2.65 for IO, LS and CP respectively. It can be seen that as the level of limit states increases from IO to CP, the probability of failure reduces which increases the reliability index for all the frames. This is due to fact that the exceedance probability decreases as the level of limits state increases.

It is seen that frame considered in the present study is not meeting the Target Reliability. It is because this building is designed for 0.36g and the stiffness and strength of infill walls are ignored in the analysis and the force demands in the frames is high and hence they are more vulnerable. But in reality, the infill walls will contribute stiffness and strength to the building, which may increase the performance of the building. To get generalized conclusion infill walls characteristics also should include in analysis to estimate the realistic Reliability index.

REFERENCES

1. IS 1893 (2002) PART 1, “Indian standard Criteria for Earthquake Resistance Design of Structures,” Bureau of Indian standards; New Delhi, 2002.
2. Bruce R. Ellingwood, “Earthquake risk assessment of building structures,” Reliability engineering and system safety, vol. 74, pp. 251-262, 2001.
3. Nath, S. K., Adhikari, M. D., Maiti, S. K. and Devaraj, N. (2013). “Seismic Hazard and Risk Microzonation of Darjeeling-Sikkim Himalaya”, Natural Hazards (Under review).
4. B.G. Nielson, “Analytical fragility curves for highway bridges in moderate seismic zones.” Ph.D. Thesis, Georgia Institute of Technology, 2005.
5. ATC 58 50% Draft, 2009. “Guidelines for Seismic Performance Assessment of Buildings”, Applied Technology council, Redwood City, CA. 2002.
6. C. Allin Cornell, Jalayer Fatemeh, O. Hamburger Ronald and A. Foutch Douglas,” The probabilistic basis for the 2000 SAC/FEMA steel moment frame guidelines.”Journal of Structural Engineering vol. 128 no.4, pp.526-533, 2002.
7. ASCE 7 (2005). “Minimum Design Loads for Buildings and Other Structures. American Society of Civil Engineers.” USA. 2005.
8. C.B. Haselton and G.G. Deierlein, “Assessing seismic collapse safety of modern reinforced concrete frame buildings,” BlumeEarthquake Engineering Research CentreTechnical report no. 156, stanford university, pp. 313, 2007.

9. IS 1893 (2002) PART 1, “Indian standard Criteria for Earthquake ResistanceDesign of Structures,” Bureau of Indian standards; New Delhi, 2002.
10. S. Mukherjee and V.K. Gupta, “Wavelet-based generation of spectrum compatibtime-histories,” Soil dynamics and Earthquake engineering vol.22, pp. 799-804, 2002.
11. IS 456 (2000), “Indian Standard for Plain and Reinforced Concrete”Code of Practice, Bureau of Indian Standards, New Delhi. 2000.
12. MatLab (2009),” MatLab - Programming software for all kind of problems” [online]. < http:// www.mathworks.com/ > (July 20, 2012).
13. R. Ranganathan,” Structural reliability analysis and design.”Jaico Publishing House, Mumbai, 1999.
14. Opeensees (2013), Open system for earthquake engineering simulation –“A Program for Statican Dynamic Nonlinear Analysis of Structures” [online]. < http://opensees.berkeley.edu/ >
15. J.B. Mander, M.J.N. Priestley and R. Park, “Theoretical stress-strain model for confined concrete.”Journal of Structural Engineering. Vol. 114, pp. 1804-1826, 1988.
16. FEMA 356 (2000). “Pre-standard and Commentary for the Seismic Rehabilitation of Buildings”American Society of Civil Engineers. USA. 2000.
17. ES.ISO.2394 (2012).”General Principles on Reliability of Structures”. Ethopiean Standards.