SEISMIC RELIABILITY ASSESSMENT OF TYPICAL SOFT-STOREY RC BUILDING IN MANIPUR REGION

Monalisa Priyadarshini, Robin Davis P, Haran Pragalath D C and Pradip Sarkar

Abstract—Significantly low stiffness and strength in any storeys compared to adjacent storeys is commonly referred to as soft-storied buildings. When car parking space is provided in the ground storey like in an open ground storey building (OGS), the building invariably becomes a soft-ground storey. These types of buildings are found to be the most affected in an earthquake as seen from the past Indian earthquakes. The ground storey columns of this type of buildings are the weakest element that may experience failure due large inter-storey drifts. For such ground storey columns, magnification factors (MF) are suggested by the design codes. The present study is focus on the seismic reliability of typical OGS building configurations in Manipur region (Ukhraul), which is one of the most vulnerable regions in India. Reliability indices for each building are estimated by combining fragility curves with the available hazard curve of the Manipur region. Building frames with different heights (6, 8 &10 stories) and MFs considered for the design. Fragility curves are developed for each type of buildings by conducting nonlinear dynamic analysis. Thirty natural time history data are selected, and modified to match with Indian response spectrum. Uncertainties in concrete and steel are included. Conclusions are drawn based on the reliability indices obtained.

Index Terms—Open ground storey, Maginification Factor, Fragility, Reliability, Peak Ground Acceleration, Performance levels and Hazard curve

1 Introduction

Car parking space for residential apartments in urban areas is a matter of increasing concern. The ground storey of the buildings is generally used for parking facilities. In such cases the ground storey will be free of any infill walls, this type of buildings are called open ground storey (OGS). The stiffness and strength of the ground storey is significantly low compared to adjacent storeys. Open ground storey buildings are considered as extreme soft-storey buildings. These types of buildings are found to be more vulnerable during the past earthquakes. [1] , Clause 7.10.3(a) states: "The columns and beams of the soft-storey are to be designed for 2.5 times the storey shears and moments calculated under seismic loads of bare frame". The prescribed magnification factor (MF) of 2.5, applicable for all OGS framed buildings. The proposed MF is reported as empirical [2]. The code proposal has also met with resistance in design and construction practice due to cost implications and congestion of heavy reinforcement in the ground storey columns. As the magnification factor increases the ground storey of the building becomes stronger, which may improve the performance of the building. The present study focus on the probability of failure and corresponding reliabilities of the building designed considering existing codal procedures. The reliability of the designed buildings is found out by combining a fragility curve and the hazard curve for a selected region in India. The building is designed for seismic zone V and the hazard curve of Manipur Region is considered for the reliability evaluation.

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2 Methodology

2.1 Assessment of Seismic Reliability

A methodology for the assessment of seismic risk of building structures is presented [3]. 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 [4] for Manipur 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 storey. 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 at 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, $G_A(a)$, that specifies values of acceleration are exceeded. This relationship implies that A is described by following equation,

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

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 MANIPUR REGION IN INDIA

Hazard curve of Manipur region, which is one of the most vulnerable earthquake prone areas, is developed by [4] Seismic hazard curves in terms of PGA for a 2500 year return period (2% exceedance probability in 50 years) for all nine district headquarters of Manipur (Senapati, Tamenglong, Churachandpur, Chandel, Imphal east Imphal west, Ukhrul, Bishnupur and Thoubal) obtained from the probablitistic seismic hazard analysis (PSHA) are shown in *Fig. 1*. Out of these hazard curves, the curve corresponding to Ukhrul region is found to be the more severe and it is selected in the reliability estimation. The PGA considered for the evaluation of probability of failure and reliability index is 1.6g as shown in *Fig.1*

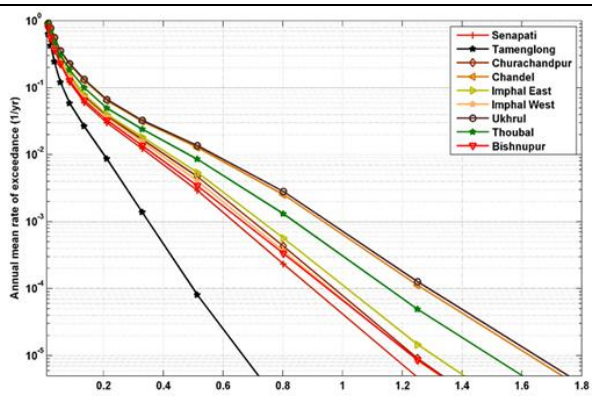

Fig 1. Seismic hazard curves at the district headquarters of Manipur at rock level [4]

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),"

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

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

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

where, "$IM_m = (\exp(\ln S_c - \ln a))/b$", *a* and *b* are the regression coefficients of the probabilistic Seismic Demand Model (PSDM) and the dispersion component, β_{comp} is given as,

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

The dispersion in capacity, β_c is dependent on the building type and construction quality. For β_c, [6] 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 [7] that the estimate of the median engineering demand parameter (*EDP*) can be represented by a power law model as given in "(8)".

$$EDP = a(IM)^b \quad (8)$$

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 [8]. However, [6] draft recommends a suite of 11 pairs of ground motions for a reliable estimate of the response quantities. A set of thirty Far-Field natural Ground Motions are collected from [9]. These are converted to match with Indian spectrum [10] using a program, WavGen developed by [21]. Figure 2 shows the Response spectrum for converted ground motions along with Indian spectrum.

4.2 Building Design

The buildings frames considered for numerical analysis in the present study are located in Indian seismic zone V with medium soil conditions. These frames are designed as an Ordinary moment resisting frames, seismic loads are estimated as per [10] and the design of the RC elements are carried out as per [12] standards. The characteristic strength of concrete and steel were taken as 25MPa and 415MPa. The buildings are assumed to be symmetric in plan. Typical bay width and column height in this study are selected as 3m and 3.2m respectively for all the frames. The different building configurations are chosen from 6 storeys to 10 storeys by keeping the number of bays as six for all the frames. The building configurations of different frames are shown in *Fig. 2*.

4.3 Sampling

Material properties of concrete, steel and masonry 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 [13] program. The mean and covariance values for concrete and steel are taken [14] and that for masonry is taken from [2].

4.4 Modelling, Analysis and Performance Levels

30 models are considered for each case, which is modelled in [15] for nonlinear analysis. Concrete is modelled as per [16] and reinforcements using a bilinear steel model with kinematic Strain hardening. Infilled masonry walls are modelled according to [17], which take into account of the stiffness and strength degradations in each cycle, which is implemented in SeismoStruct. Hilber-Hughes Taylor series scheme is adopted for the time step analysis and skyline technique is used for matrix storage. 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 [18].

5 Fragility Curves For Building Frames

Fragility curves for building frames BF, FF, OGS-1, OGS1.5, OGS2 and OGS-2.5 for three performance levels namely, IO, LS and CP are generated. The variation of exceedance probability of the inter-storey drift with the PGA for the storeys ranging from 6 to 10 storeys for the performance level of Immediate Occupancy (IO) obtained is shown in Figure 3.

The bare frame (BF) is found to be more vulnerable than the FF and OGS frame for the performance levels considered. The exceedance probability of inter-storey drift of OGS buildings designed by magnification factors 1.5, 2 and 2.5 are less than that of FF in all the cases. The magnification factor 2.5 is likely to increase the performance than actually needed by decreasing the inter-storey drift. The same behaviour is observed in the case of eight and six storied frames. These fragility curves are combined with the harzard curve to estimate the joint probability of failure using the "(2)," as discussed.

6 Assessment Using The Reliability Indices

The Target Reliability Indices in accordance with [18] 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 [18]. The assessment of performance of each building is carried out by comparing the reliability indices obtained for each building with corrsponding target reliability indices corresponding to moderate level of relative cost measure. The target reliability indices are taken as 2.5, 3.0 and 4.0 respectively for the performance levels IO, LS and CP.

The probability of failure of each frame and corresponding reliability indices are presented in the Table1. For example, the values of reliability indices for a 6S6B bare frame are 1.61, 2.10 and 2.71 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.

Among all frames, bare frames are found to be more vulnerable due to higher values of failure probability. The stiffness and strength of infill walls are neglected in the bare frame analysis and the force demands in the bare frame is high and hence they are more vulnerable. In reality the infill walls will contribute stiffness and strength to the building, which increases the performance of the building. For example, the reliability of bare frame for immediate occupancy performance level is about 1.61 where as, for fully infilled frame it is 3.80. This shows that fully infilled frame perform better than a bare frame.

From Table 1, it can be seen that Bare frames (BF) are not able to meet the target reliability suggested [18], in all the performances levels where as the full infilled frames (FF) meets the target reliability in all performances levels.

The infill walls are ignored at anlaysis and design stage, in the current design methodology. In reality, the infill walls which is ignored and provided at the time of construction, contribute to some stiffness and strength to the global performance of the buildings (eg. Fully infilled frames).

However, for an Open ground storey building the same design methodology may not guaranty the required performance. But in the present study OGS1 marginally reaches the Target Reliability in all the performance levels, which may not be always possible. This implies that more research is required in this direction. For OGS 2.5 Reliability Indices are found to be twice that of target reliability, which indicates that the factor MF may be more conservative. For optimum design of an OGS building, particularly for the design magnification factor, the target reliability can be a considered as a basis.

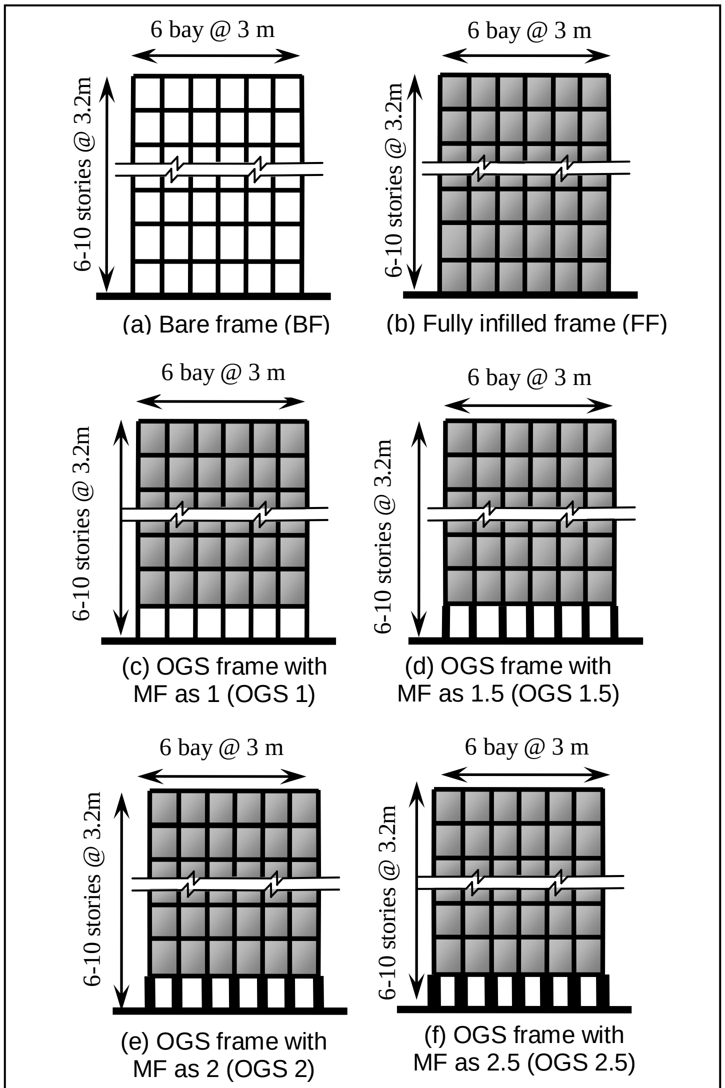

Fig. 2. Elevation of building frames considered

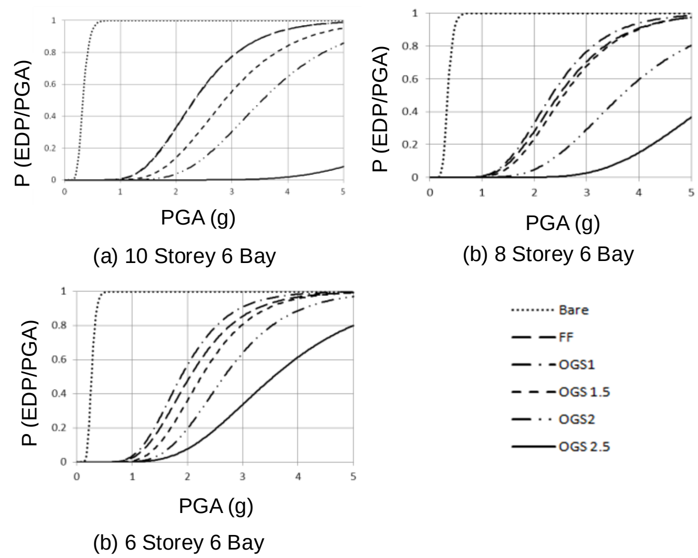

Fig. 3 Comparison of Fragility curves for buildings with various cases for different heights for IO

TABLE 1

PROBABILITY OF FAILURE AND SEISMIC RELIABILITY OF THE FRAMES FOR EACH LIMIT STATES

		6S6B		8S6B		10S6B	
		Pf	β	Pf	β	Pf	β
Bare	IO	0.0529	1.61	0.0333	1.83	0.0378	1.77
	LS	0.0177	2.10	0.0086	2.38	0.0089	2.36
	CP	0.0033	2.71	6.06E-04	3.23	5.31E-04	3.27
FF	IO	7.15E-05	3.80	1.95E-05	4.11	2.11E-05	4.09
	LS	5.52E-06	4.87	6.25E-08	5.28	6.01E-08	5.29
	CP	2.35E-11	6.58	2.34E-11	6.58	2.34E-11	6.58
OGS 1	IO	1.01E-04	3.71	2.33E-05	4.07	1.92E-05	4.11
	LS	3.36E-07	4.96	2.74E-08	5.43	1.83E-08	5.50
	CP	7.46E-11	6.40	9.09E-13	7.04	7.30E-13	7.07
OGS 1.5	IO	2.14E-05	4.09	9.84E-06	4.26	2.90E-06	4.53
	LS	4.55E-08	5.3	1.58E-08	5.53	1.86E-09	5.89
	CP	7.18E-12	6.75	1.96E-12	6.94	6.04E-14	7.41
OGS 2	IO	5.63E-06	4.39	7.32E-07	4.81	2.48E-07	5.02
	LS	1.58E-08	5.53	6.28E-10	6.07	1.07E-10	6.35
	CP	5.47E-12	6.79	5.00E-14	7.44	2.85E-15	7.81
OGS 2.5	IO	2.67E-06	4.55	8.94E-10	6.01	3.28E-12	6.86
	LS	1.81E-08	5.50	3.52E-14	7.48	8.77E-18	8.50
	CP	3.18E-11	6.53	5.32E-20	9.08	5.13E-25	10.2

7 CONCLUSION

The present study is focussed on the seismic reliability of typical OGS building configurations in Manipur region (Ukhraul). Exceedance probabilities (fragility curve) of inter-storey drift of each performance levels for all the frames are developed. The selected Hazard curve is combined with the fragility curve to find the joint probability of failure and corresponding reliability. It is seen that Bare frames are the most vulnerable, and these type of frames with no infill walls, failed to meet the Target reliability index. Open ground storey buildings designed with magnification factors 1.5, 2.0 and 2.5 performed well by meeting the corresponding target reliability index with some margin. The result of the present study shows that Open ground Storey building designed for Magnification factor of 1.0 reaches the Target Reliability Index marginally. In order to arrive at a generalized conclusion on the behavior of OGS buildings with Magnification 1.0, further research work is required in this direction.

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