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SEISMIC FRAGILITY OF OPEN GROUND STORY BUILDINGS IN INDIA

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ABSTRACT

'Open ground storey' (OGS) framed buildings are very common in urban areas in countries like India. In design practice, the influence of the infill stiffness in the upper stories of the building is usually ignored, and unless the ground storey columns are specially designed for enhanced bending moments and shear forces, OGS framed buildings is seismically vulnerable due to vertical irregularity. In this study, the seismic fragility of existing OGS buildings in India are evaluated and compared with the corresponding fragility curves for building frames without infill (bare frames) and with full infill in all storeys. The inter-storey drift at the ground storey is treated as the demand variable using a power law model, considering a soft-storey failure mechanism. A regression analysis is performed to estimate the parameters of the demand model, from the peak responses estimated from nonlinear dynamic analyses of OGS buildings using an ensemble of 30 ground motions that represent the seismicity of the region. The probability distributions of the capacities are assumed to be lognormal. The OGS frames are found to be significantly more fragile compared to the fully infilled frames at all limit states, and in general, the fragility increases with increase in number of storeys, but decreases when a large number of bays are involved.

Introduction

Open ground storey (OGS) buildings are commonly provided in India since they provide much-needed parking space in an urban environment. These buildings are observed to be the most vulnerable type of vertically irregular buildings (Jain et. al 2001). Many of the buildings that failed in recent earthquakes in India were of this type. Collapse of these buildings is predominantly due to formation of soft-storey mechanism, combined with P-Δ effect, in the ground storey columns. The inter-storey drift is relatively very large in the ground storey compared to the storeys above, resulting in large curvatures and bending moments of the ground storey columns. This leads to concentration of damage in the ground storey columns, due to plastic hinge formation at the top and bottom. In conventional design practice OGS buildings are

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usually analysed as bare frames (modelled beams and columns as frame elements) by neglecting the stiffness of infill walls. The current Indian design code, IS 1893 (2002), permits such simplified analysis, but requires the designer to design the ground storey columns for bending moments and shear forces, enhanced by a factor as high as 2.5. Realistic behaviour of OGS framed buildings, however, can be captured only by modelling the stiffness and strength of the infill walls in the upper storeys in the analysis (Davis 2009).

Literature Review

Kampanya et. al (2001) investigated the buildings with soft storey designed using 1997 NEHRP Recommended Provisions. Soft first storey creates the vertical irregularity, which leads to the soft-storey mechanism resulting in higher inter-storey drifts in first and second storeys compared to the above floors. Also, it is reported that the non-structural walls increase the soft-storey effect on the ground floor. Nagae et. al (2006) computed the annual frequency of maximum inter-storey drift ratios exceeding a specific value. The shapes of the curves of PGA and IDR_{max} are found to be significantly influenced by the type of the failure mechanism. Lagaros (2008) studied the effectiveness of the fragility curves in assessing the performance of RC buildings with soft storey designed to prescriptive code provisions.

Example Frames

The OGS building frames considered for numerical analysis in the present study are located in Indian seismic zone IV with medium soil conditions. The design peak ground acceleration (PGA) of this zone is specified as 0.12*g*. These frames are designed as per prevailing practice in India, ignoring the soft-storey effect. Seismic loads are estimated as per IS 1893 (2002) and the design of the RC elements are carried out as per IS 456 (2000) standards. The characteristic strength of concrete and steel were taken as 25MPa and 415MPa. The buildings are assumed to be symmetric in plan, and hence a single plane frame may be considered to be representative of the building along one direction. Typical bay width and column height in this study are selected as 3m and 3.2m respectively, as observed from the study of typical existing OGS residential buildings. The bare frame (4s5b-bare), OGS frames (4s5b, 4s7b, 4s9b, 8s5b and 10s5b) and a building frame with infill walls extended to the ground storey (4s5b-full) considered for the present study are shown in Fig. 1.

The dead load of the slab (3 m x 3 m panel), including floor finishes, is taken as 2.5 kN/m^2 and live load as 3 kN/m^2. The unit weight of brick masonry infill is taken as 18 kN/m^3. The design base shear (V_B) is calculated as per IS 1893 (2002).

$$V_B = \left(\frac{ZI}{2R} \frac{S_a}{g} \right) W \tag{1}$$

where seismic zone factor, Z = 0.24, Importance factor I = 1.0, Response reduction factor R = 3.0. The design details for ground storey column for each frame are given in Table 1.

Table 1 Design details for the example frames

Frame designation	Ground storey column section	% Reinforcement provided
4storey 5bay (4s5b)	300 x 300	2.79
4storey 7bay (4s7b)	300 x 300	2.79
4storey 9bay 4s9b	300 x 300	2.79
8storey 5bay 8s5b	400 x 400	2.45
10storey 5bay (10s5b)	400 x 400	3.68

Modelling

OGS buildings are modelled in the program Seismostruct (2007) for nonlinear time history analysis. Seismostruct uses fiber based spread plasticity elements for frame elements. Mander's confinement model is used for concrete. For reinforcement steel, Menegotto and Pinto stress-strain relationship with Filippou's isotropic hardening rule is used. The brick masonry walls are modelled using the Crisafulli's model. Rayleigh damping model is used with 3% damping in the first mode and 5% damping in the third mode.

Fragility Curves

It is imperative to resort to the analytical fragility curves, with the scarcity of post-earthquake reconnaissance data available for the reliable estimate of the vulnerability. The 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*). Fragility 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. A fragility curve can be obtained for each damage state. The fragility can be expressed in closed form using Eq. 2,

$$P\left(C - D \leq 0 \middle| IM\right) = \Phi\left(\frac{\ln {S_d}/{S_c}}{\sqrt{\beta_{d|IM}^2 + \beta_c^2}}\right) \qquad (2)$$

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*). $\beta_{\mathrm{d/IM}}$ and β_{c} are dispersion in the intensity measure and capacities respectively. Eq. 2 can be rewritten as Eq. 3 for component fragilities (Nielson, 2005) as,

$$P(DS|IM) = \Phi\left(\frac{\ln IM - \ln IM_m}{\beta_{comp}}\right) \qquad (3)$$

where $IM_m = \exp\left(\frac{\ln S_a - \ln a}{b}\right)$, a and b are the regression coefficients of the Probabilistic Seismic Demand Model **(**PSDM) and the dispersion component, β_{comp} is given as,

$$\beta_{comp} = \frac{\sqrt{\beta_{d|IM}^2 + \beta_c^2}}{b} \quad (4)$$

Probabilistic Seismic Demand Model

It has been suggested by Cornell et. al (2002) that the estimate of the median engineering demand parameter (*EDP*) can be represented by a power law model as given in Eq. 5.

$$\widehat{EDP} = a(IM)^b \quad (5)$$

In this present 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*).

The number of ground motions required for an unbiased estimate of the structural response is 3 or 7 as per ASCE 7-05. However, ATC 58 50% draft recommends a suite of 11 pairs of ground motions for a reliable estimate of the response quantities. ASCE/SEI 41 (2005) suggests 30 recorded ground motions to meet the spectral matching criteria for NPP infrastructures.

A set of thirty IS 1893 (2002) spectrum compatible ground motions are generated using WavGen (Mukherjee and Gupta, 2002) from the thirty natural time histories. The response spectra of the transformed ground motions along with the target design spectrum specified in IS 1893 (2002) are shown in the Fig. 2.

To consider the uncertainty in the material properties, the characteristic strength of concrete, f_{ck} and the yield strength of the steel, f_y are taken as the random variable. The statistical details (Table 2) of the parameters, f_{ck} and f_y have been taken from Ranganathan (1999). A set of thirty statistically equivalent analytical models are generated using the Latin Hypercube Sampling (LHS) scheme.

Table 2: 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

Nonlinear time history analyses of all the thirty statistically equivalent analytical models have been performed to obtain a set of thirty inter-storey drifts (δ) for the corresponding PGAs. The parameters '*a*' and '*b*' of the Eq. 5 are determined for the set of thirty values using regression analysis. The demand models for each frame is obtained using linear regression analysis. Typical generated models are as shown in Fig. 3.

Capacity Model

Limit states define the capacity of the structure to withstand different levels of damage. It can be represented qualitatively (HAZUS) or quantitatively (FEMA 356). In this study, the inter-storey drift limits suggested by Ghobarah (2004) has been taken as the median values of the capacity, S_c. The median inter-storey drifts for light repairable damage (IO), moderate repairable damage (LS) and near collapse (CP) for moment resisting frames (MRF) with infill walls and without infill walls (bare frame) are listed in Table 3.

The dispersion in capacity, β_c is dependent on the building type and construction quality. Wen et. al (2004) have suggested a value of 0.3 for β_c whereas ATC 58 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. The details of the capacity model considered in this study are listed in Table 3.

Table 3: Damage limits and dispersion associated with various structural performance levels

Limit states designation	**Performance level**	**Median Inter-storey Drifts Sc for MRF with infill walls, (%)**	**Median Inter-storey Drifts S_c for MRF without infill, (%)**	**Dispersion, β_c**
IO	Light repairable damage	0.2	0.4	0.25
LS	Moderate repairable damage	0.4	0.1	0.25
CP	Near collapse	0.8	3	0.25

After developing probabilistic seismic demand models and the capacity limit state models, fragility curves are generated using Eq. 3 for each of performance levels for each frames (4s5b-bare, 4s5b, 4s7b, 4s9b, 8s5b and 10s5b, 4s5b-full) and are shown in Figs. 4, 5 and 6.

Discussion and Conclusions

Comparison of fragility curves for OGS and bare frame

Fig. 4 shows comparison between fragility curves for a typical 4 storeyed OGS frame (4s5b), bare frame (4s5b-bare) and a full infilled frame (4s5b-full). It can be seen that at all the limit states, for any given PGA, OGS frames are found to be significantly more fragile compared to the fully infilled frames, as expected. OGS frames perform better compared to the bare frames in IO and LS limit states, which perhaps suggest that the inter-storey drift may not be the proper demand parameter that reflects the damage levels in bare frames.

The fragility curves generated using the methodology adopted in the present study is sensitive to the quantitative limits prescribed for the capacity limit states considered for IO, LS and CP. Further study is required to arrive at a generalised conclusion.

Fragility curves - influence of number of bays

Fig. 5 shows that in the case of 4-storey OGS buildings, the ones with 9 bays are found to be less fragile compared to those with 5 bays and 7 bays. In general, as the number of bays increases significantly, the probability of failure becomes less, for any specified PGA. This can be associated with the increased redundancy and stiffness in the frames with increase in number of bays. However, this trend is not pronounced and can even be marginally reversed, when the number of bays is not high, depending on the fundamental time period of the building. For example, the 4-storey OGS building with 5 bays (4s5b), having a fundamental time period, T = 0.439s, is seen to perform marginally better than the one with 7bays (4s7b), which has a marginally higher period, T = 0.453s. This trend can be seen in all the damage limit states.

Fragility curves - influence of number of storeys

Fig. 6 shows the comparison of fragility curves for 8 storeyed and 10 storeyed building frames. 8s5b building frame is found to perform better at IO and LS damage levels, compared to 10s5b, whereas in CP limit state, there is a marginal reversal in the trend. This can be attributed to the relative increased strength of the ground storey columns of 10s5b building frame in the inelastic regime compared to those of 8s5b. In general, it is seen that the fragility gets increased with increase in number of storeys at IO and LS damage levels, but not necessarily at the CP limit state.

FIGURES

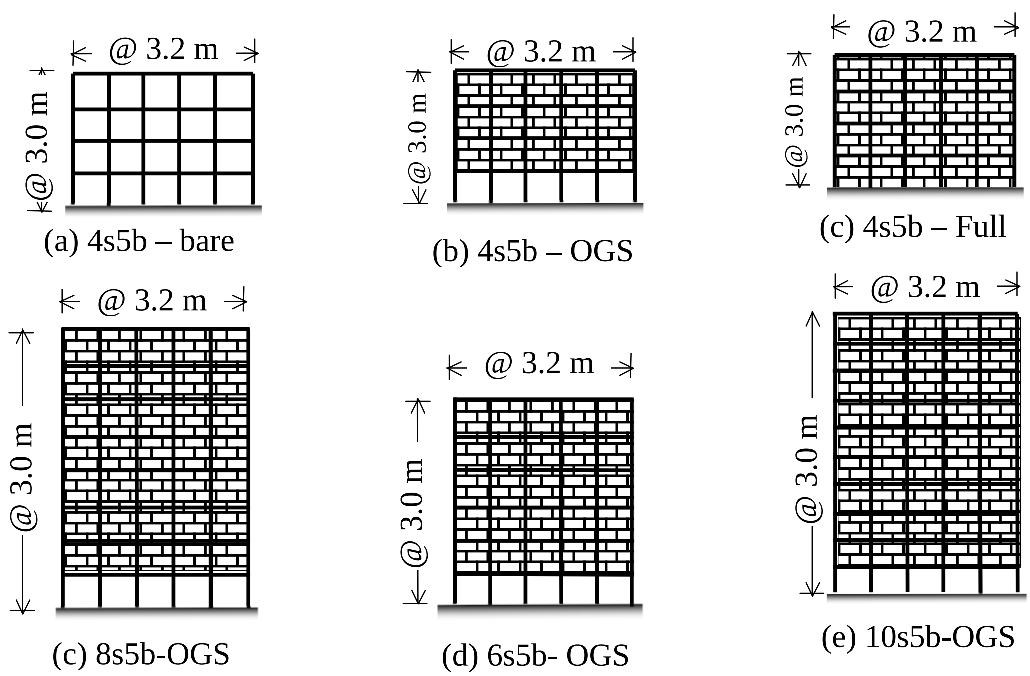


Figure 1. Details of bare frame and OGS frames considered

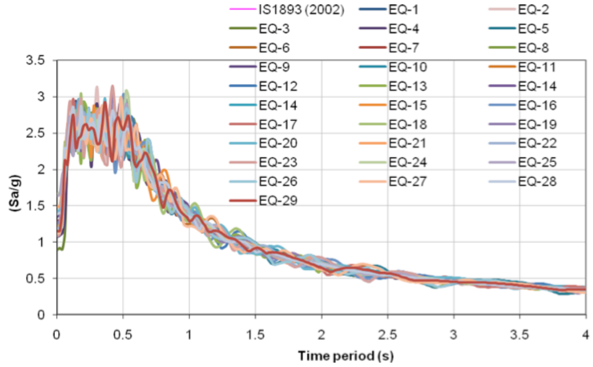
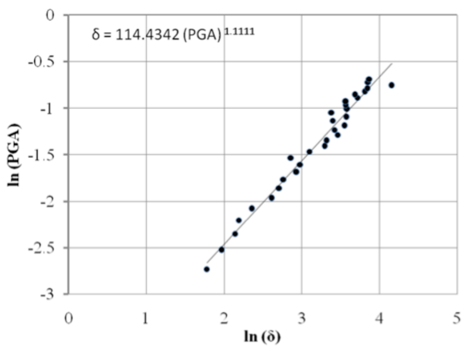
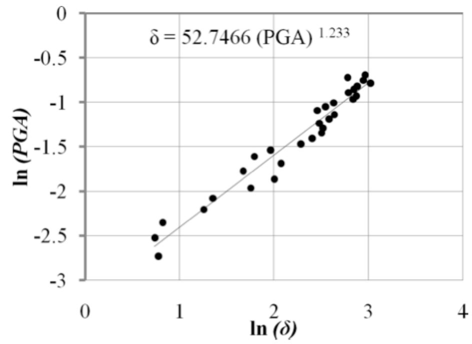


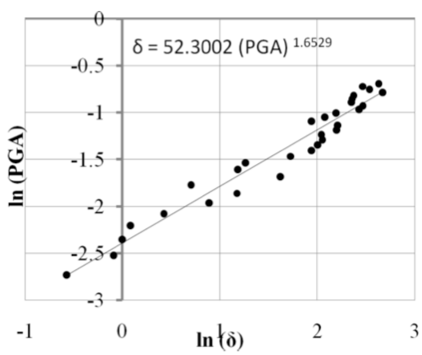
Figure 2. Response spectra generated from spectrum compatible ground motions



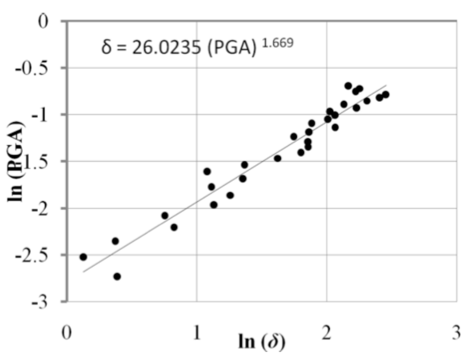
(a) 4s5b



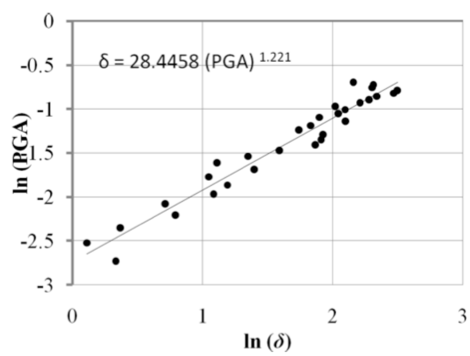
(b) 4s5b-bare



(c) 4s5b-full

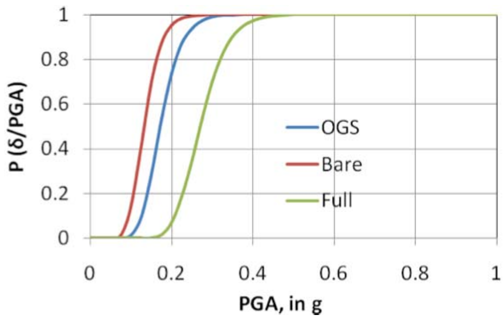


(d) 8s5b

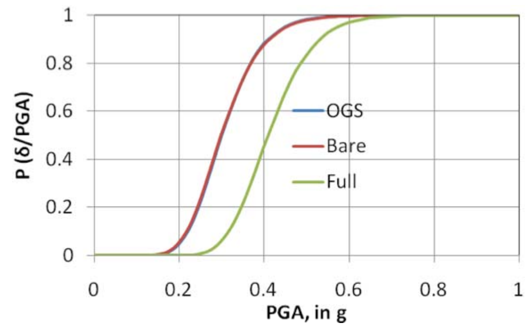


(e) 10s5b

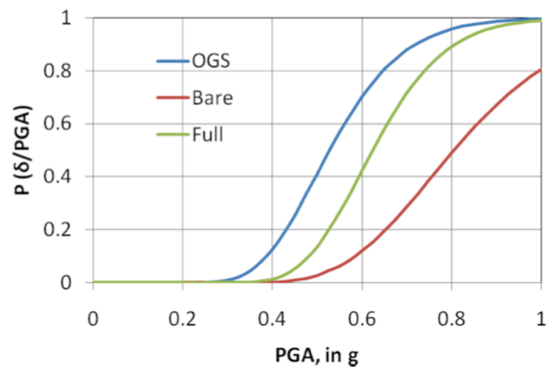
Figure 3. Demand models for frames considered



(a) Limit state IO

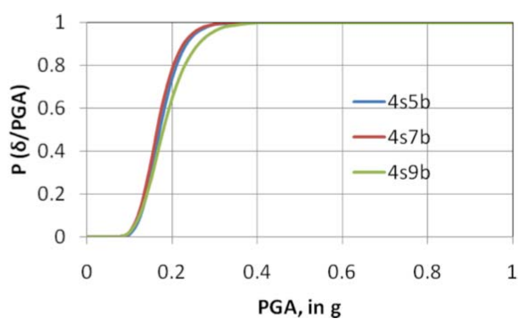


(b) Limit state LS

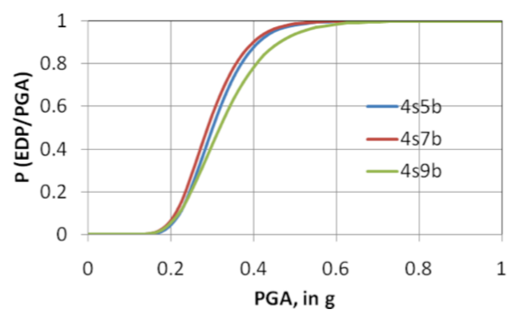


(c) Limit state CP

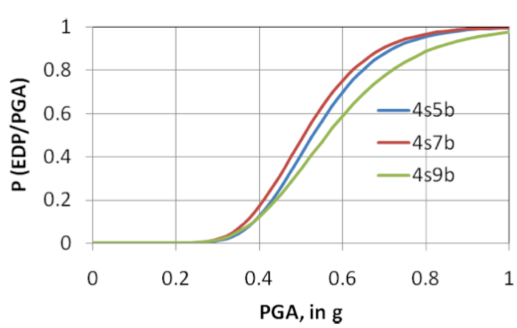
Figure 4. Fragility curves for 4s5b frames for different limit states



(a) Limit state IO

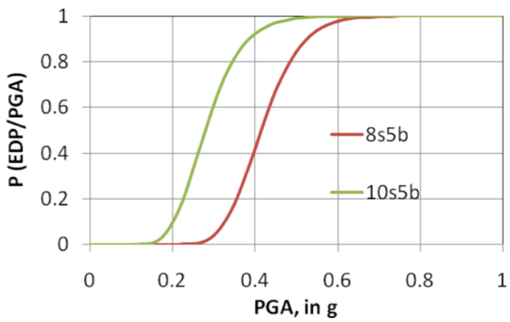


(b) Limit state LS

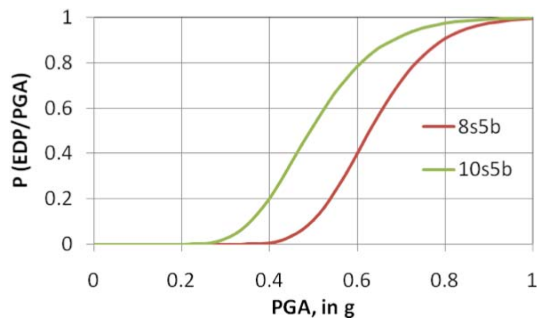


(c) Limit state CP

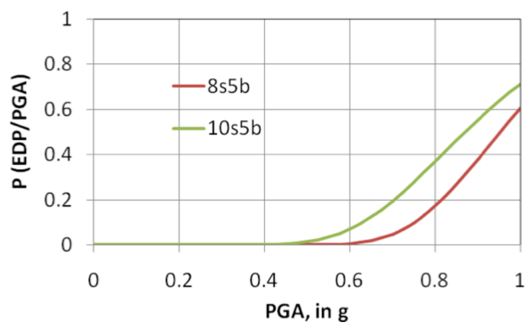
Figure 5. Fragility curves for 4storeyed OGS frames - influence of number of bays



(a) Limit state IO



(b) Limit state LS



(c) Limit state CP

Figure 6. Fragility curves for 5b OGS frames - influence of number of storeys

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