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A SIMPLIFIED METHOD FOR SEISMIC VULNERABILITY ASSESSMENT OF RC BUILDINGS WITH OPEN GROUND STOREY

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Abstract

RC buildings with open ground storey (OGS) are commonly constructed in many countries where the ground storey is left open (i.e., without infill walls) primarily for parking or business purposes. Such buildings have performed quite poorly even during moderate shakings in the past. The presence of infills only in the upper stories make them significantly stiffer as compared to the open ground storey. Thus, most of the lateral displacement of the building is concentrated in the ground storey itself with a very little relative lateral deformation of the upper stories. As a result, the OGS columns are not able to resist the high ductility demand imposed on them during earthquakes leading to severe damage or collapse of the entire building. This calls for a need to understand and qualify such building construction for seismic performance in earthquake-prone countries and classify them into groups based on its seismic vulnerability. As OGS frames are found to be most vulnerable building with any configurations, a parametric study was carried out with different number of bays and stories, and varying central opening sizes in infills of the frame. Seismic displacement demand of the frames is assessed based on ATC 40 performance assessment methodology. The results of seismic performance assessment were used to develop a simple numerical model for quick estimation of spectral displacement demand for OGS frames based on multiple variable linear regression for large building inventory including low to mid rise buildings. Further, the effectiveness of the developed regression model for spectral displacement demand estimation (and hence fragility assessment) was demonstrated using some examples.

Keywords: Reinforced Concrete buildings; Open Ground Storey; Nonlinear Analysis; Seismic Fragility, Vulnerability.

1. Introduction

Reinforced concrete (RC) frame buildings with unreinforced masonry infill walls are commonly constructed in many countries including India. Provision of infill only on upper floors makes the ground storey relatively more flexible and weaker than the upper storey, in which masonry infills are provided. Such buildings are commonly known as open ground storey (OGS) buildings. During earthquake shaking, most of the lateral displacement of the building is concentrated in the soft ground storey itself with a little relative lateral deformation of the upper stories. Generally columns of the OGS do not have adequate ductility and strength to resist the high ductility demand on it, and therefore, the building suffers severe damage or may even collapse during earthquakes (as shown in Fig. 1). Previous experiences from earthquakes all over the world also reveal the fact that such buildings performed quite poorly even during moderate shaking, and in some cases, complete collapse of the ground storey was observed [1]. Infills provide lateral stiffness to structures, but openings in it significantly reduce the lateral strength and stiffness of masonry infilled RC frames; this further alters the failure modes of such buildings.

In the current study, a detailed nonlinear seismic analysis of OGS frames based on ATC 40 [2] methodology is carried out in order to understand the behavior of such buildings with varying opening sizes in masonry infill walls (*Op*) of upper stories, total number of bays, and number of stories. Also, the effect of these parameters on the global seismic behavior of the buildings is studied under varying PGA (Peak Ground Acceleration) levels. The study was extended to fragility analysis that further assists in predicting seismic vulnerability. The primary objective of the present study is to develop a quick and easy method to assess the seismic vulnerability of low to mid rise masonry infilled RC buildings. It is further intended to bring awareness among various stakeholders that providing openings in walls of upper stories of OGS buildings may not improve the safety, and hence, may not reduce their overall seismic vulnerability.

Fig. 1 - Collapse of open ground storey (OGS) RC frame residential building with small to large openings at Bhuj during 2001 Bhuj earthquake [Source: *www.nicee.org*]

2. Methodology

2.1 Structural Modelling

An extensive parametric study considering a matrix of reinforced concrete (RC) frame building models with number of bay and storey varying from 1 to 6 (1B – 6B) and 2 to 6 (2S – 6S), respectively, is carried out in the current study. Infill walls were modeled as equivalent diagonal struts [3]. Influence of central openings in the masonry infill walls on lateral load behavior and seismic vulnerability of such frames was also studied. The models considered represent the most common practice of neglecting the strength and stiffness of masonry infills in analysis and design procedure. A typical bay considered in the study for bare frame, Fully Infilled (FI) frame, and OGS frame is shown in Fig. 2. The Structural Analysis Program SAP2000 [4] was used for carrying out nonlinear static analysis of the frames considering material nonlinearity.

The beam and column sectional details and location of plastic hinges considered in RC frame and infill are as shown in Fig. 3. Beam and columns were modeled as frame elements in which the base of columns were assumed to be fixed. The material properties used in the frame were assumed to be non-varying (deterministic) as shown in Table 1. In RC members, plastic hinges were assumed to form at a distance equal to one-half of the average plastic hinge length *lp* from the member ends and calculated using Eq. 1 [5]. In case of diagonal struts, the plastic hinges were assumed to have length of 3/4th of length of struts and the hinges were assumed to form at center of the struts.

$$lp = 0.08L + 0.022d_bf_y \tag{1}$$

where L=length of member in m; d_b=diameter of longitudinal steel in m; and f_y=yield strength of longitudinal steel in MPa.

The infill walls were modeled as single equivalent diagonal struts, taking care of the reduction in strut width for increase in openings. The width of compression strut was considered as one-fourth of diagonal length of infill [5]. With increase in openings, the width of the strut decreases, and the reduction factor for consideration of opening in infill was evaluated as in Eq. 2 [6].

$$\left.\begin{aligned} Reduced\ strut\ width\ &= \alpha_s \times \left(original\ strut\ width\right) \\ \alpha_s &= 1 - r_0^{\,0.675} \end{aligned}\right\} \tag{2}$$

Nonlinearity in members was defined as lumped plasticity model at the critical locations of the members. Nonlinearity in the flexural members were defined using FEMA 356 [7] flexural hinge near the ends of member, and for the equivalent strut, using stress strain properties for masonry under compression [3].

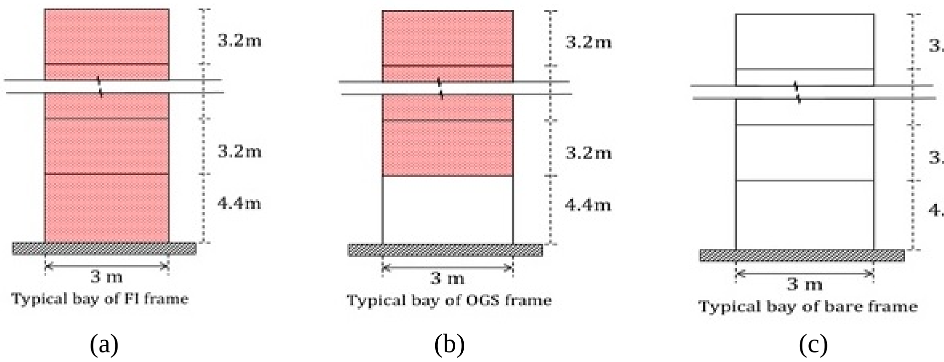


Fig. 2 – Typical elevation details of different building frames – (a) Fully Infill, (b) Open Ground Storey, (c) Bare considered in the study.

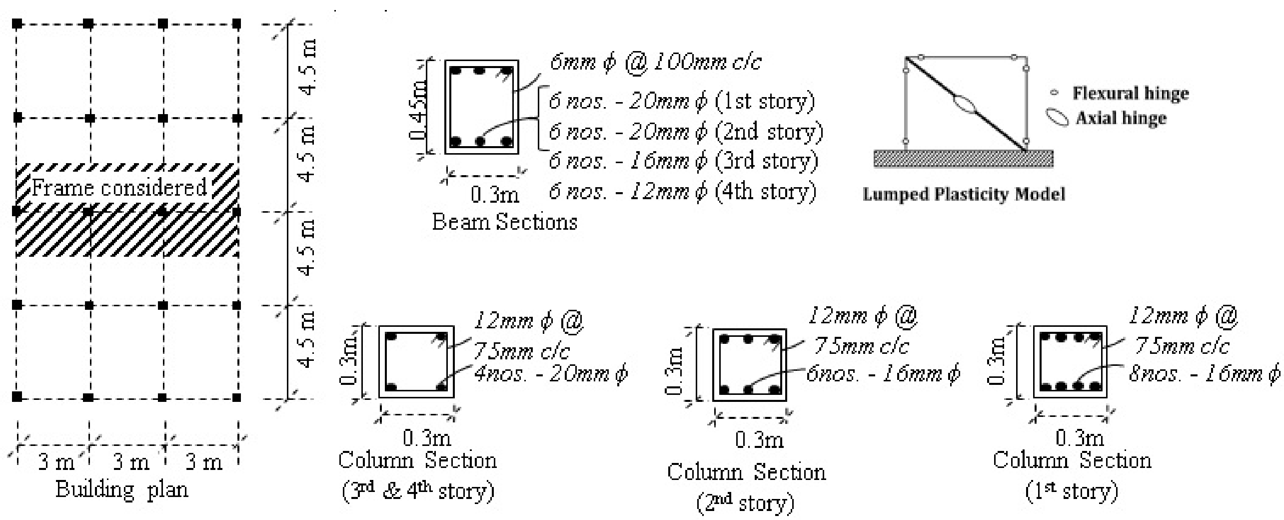


Fig. 3 – Building plan, sectional details and plastic hinge locations.

Table 1 – Material properties considered in building frame model

Material	Strength (MPa)	Elastic Modulus (MPa)	Poisson's Ratio	Weight Density (kN/m^3)
Concrete	25 (Compressive Strength of Concrete Cubes)	25000	0.2	25
Rebar	450 (Expected Yield Stress of Reinforcing Bars)	200000	0.3	77
Masonry	4.1 (Compressive Strength of Masonry Prisms)	2255	0.15	18

2.2 Nonlinear Static Performance Assessment

In this study, nonlinear static (pushover) analysis of typical RC frames, both regular (Fully Infilled) and irregularly (pilotis or open ground storey) infilled frame were carried out in SAP 2000 [4]. For a 3 bay 4 storey (OGS and FI) RC frame, central openings in the infill walls were varied from 0 to 90 percent of the infilled wall area covered within adjacent columns and beams (hereafter referred to as openings). Following nonlinear static capacity estimation of the frames, seismic performance assessment was done. Detailed nonlinear analyses (static or dynamic) are required to be carried out for assessing seismic capacity, and hence, performance of structures for a given hazard (demand) on the structure. The pushover technique provides useful information on the overall characteristics of the structural system and allows tracing of the sequence of yielding and failure of the members. In general, a sequence of inelastic static analysis is performed on the structural model by applying a predefined load pattern (here, 1st mode), and distributed along the building height. The lateral forces are monotonically increased until roof displacement reaches a desired target displacement.

The common practice is to use simple nonlinear static analysis procedures or PO analysis for performance evaluation of structure using Capacity Spectrum Method (CSM) [2]. CSM is one of the commonly used procedures to evaluate nonlinear seismic performance of complex structures, originally developed by Freeman [8]. It requires construction of damped elastic response spectrum (or demand spectrum) and capacity spectrum for a SDOF system (Fig. 4a) in acceleration displacement response spectrum (ADRS) format as shown in Fig. 4b and Fig. 4c. In this study, elastic response spectrum according to provisions in IS 1893 [9] for different levels of PGA were considered as demand spectrum (though one may consider inelastic ductility based response spectra for the same). The elastic response spectra are effectively reduced to a damping that takes account of nonlinear effects based on the area covered under one perfect hysteresis loop [2]. Finally, intersection of the capacity spectrum with the reduced demand spectrum is defined as the performance point (PP) of the structure for the given demand. In this study, to represent worst case scenario, Type "C" building Category [2] for which the shape of hysteresis curve is pinched ($\kappa = 0.33$) was only considered. The maximum allowable effective damping (βeff) is limited to 20% for Type C building category. For Type A or B, higher values of effective damping are allowed due to which performance, and hence, seismic fragility may differ.

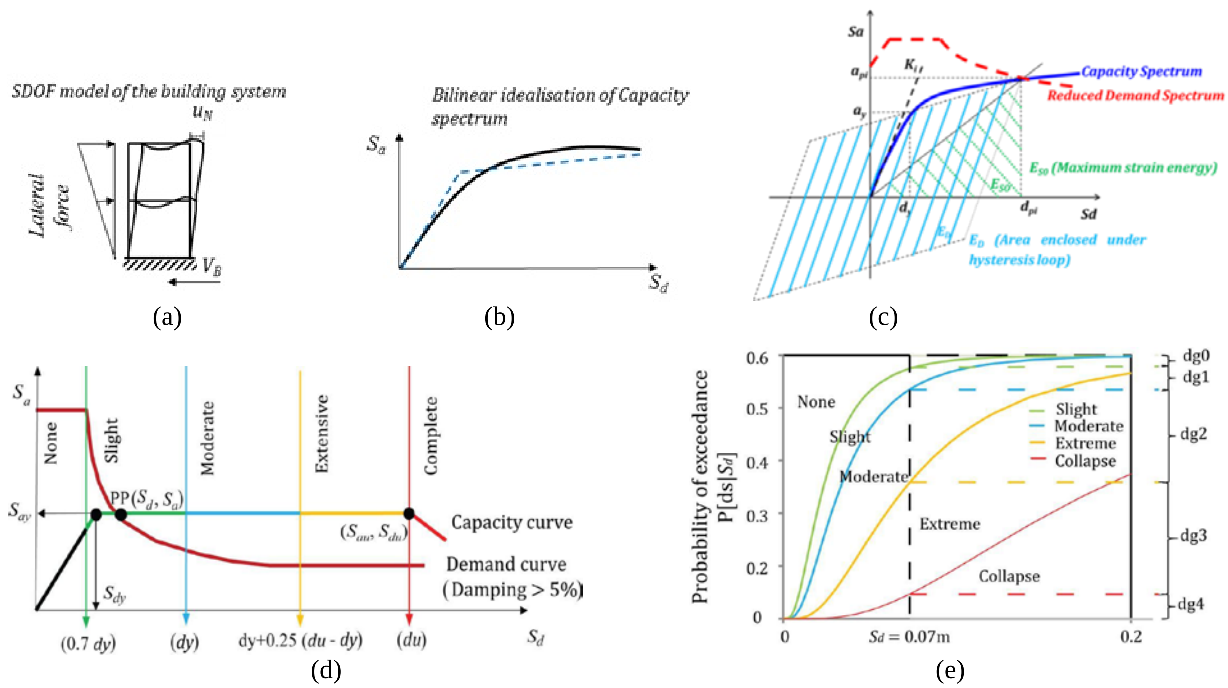


Fig. 4 – (a) – (c) Seismic performance assessment using ATC 40 methodology; (d) discrete damage states identified on a capacity curve; (e) fragility curves for damage states in (d).

2.3 Seismic Damage and Fragility Asssessment

Seismic damage assessment is required to estimate the probability of occurrence of a given damage state or a certain performance level to a particular building class under earthquake. In other words, it is the numerical interpretation of the probable damage to a building due to a given hazard. In order to assess damage of a given degree, damage states are specified that categorize the degree of damage to the members of the frame into some discrete points. Several damage indices are mentioned in literature for the measure of damage after an earthquake. These damage indices are expressed in terms of either strength parameters or parameters related to ductility and hysteretic energy dissipation. The damage levels or the performance levels can be associated with the pushover curves. Such relationships are available in literature where the damage states are basically expressed in terms of two parameters of capacity curves, i.e., yield displacement (d_y) and ultimate displacement (d_u) as shown in Fig 4(d). Each vertical line in Fig. 4(d) demarcates the damage states in the PO curve based on yield and ultimate points. These are known as the damage state thresholds [10] and are estimated as the median value of the considered damage state.

Four damage states have been considered in the present study, viz, Slight (S), Moderate (M), Extreme (E), and Collapse (C). For defining the damage state thresholds, yield displacement is defined as the initiation of formation of first plastic hinge in any ground storey columns, and the ultimate displacement is defined as the displacement where 50 percent or more of the assigned hinges in the ground storey columns fail in the OGS frames and additionally failure of all ground storey infills is also considered in the case of FI frames. Given damage state thresholds and the PP of the structure for different levels of PGA, the seismic fragility can be obtained using Eq. 3 as given in HAZUS [11] and plotted as fragility curves (Fig. 4e). Difference between consecutive damage states are shown as different damage grades (dg) in Fig. 4(e) at an S_d of 0.07m.

$$\mathrm{P}[\mathrm{ds} \mid S_d] = \Phi\left[\frac{1}{\beta_{\mathrm{dsi}}}\ln\left(\frac{S_d}{\overline{\mathrm{S}}_{\mathrm{d,ds}}}\right)\right] \tag{3}$$

Eq. 3 represents the conditional probability P[ds|S_d] of being in, or exceeding, a particular damage state (ds), given the spectral displacement at the performance point, S_d, where Φ is the standard normal cumulative distribution function and β_{dsi} is the normalized standard deviation of the natural logarithm of the displacement threshold ($\overline{\mathrm{S}}_{\mathrm{d,ds}}$) indicating uncertainties in capacity curve properties, damage levels, model errors and ground shaking. For this β_{dsi}, FEMA P-58 [12] suggested a default value of 0.6. On the other hand, HAZUS [11] suggested values of 0.75, 0.70, and 0.65 for buildings designed to old, moderate, and modern codes, respectively. Considering the building frames analyzed to be moderately designed, and also from an understanding that there is minor change in fragility with increase in uncertainty after a value of about 0.7 [13], a value of 0.7 was adopted for each damage state for carrying out fragility analyses in the current study. It should be realized that shape of the vulnerability curves depends on the structural type because of the variations in their rate of accumulation of damage with increasing ground motion.

3. Results

3.1 Nonlinear Static Performance Assessment

Results of pushover analysis demonstrate resistance of the building in terms of base shear force versus roof displacement, commonly referred to as the capacity curve of the building. Fig. 5 (a and b) represents nonlinear force displacement relationship (pushover curves) for the OGS and FI frames, respectively, with different openings varying from 0% to 90% along with the pushover curves (PO) of bare frame. The change in lateral load capacity of OGS frames is merely affected by the change in opening size in infills. It is obviously observed from the PO curves that capacity of FI frames is much higher in comparison to OGS frames. Further, there appears to be a good balance between the infill wall capacity and the RC frame capacity of the FI frame when an opening of about 60% or more is provided in the infilled wall area. This ensures a gradual fall in the post peak lateral load carrying capacity compared to the infilled frames with less than 60% opening in which the post peak fall is sudden and drastic.

The PPs are obtained for each of the frames considered in the study for different design level of PGA, each representing a displacement demand on the structure for a given PGA. Table 2 shows the yield (S_{dy}) and ultimate (S_{du}) displacement demands obtained using ATC 40 [2] performance assessment methodology for OGS and FI frames with 50% and 60% openings in infill for Zone IV (PGA = 0.24g) and Zone V (PGA = 0.36g) for the two higher seismic zones defined in the Indian Seismic Code [9]. It is clear from the comparative table that the displacement demand on OGS frame (with any opening size) is more compared to that on the FI frame for a given PGA. A comparison of ductility demand (μ_D) for varying levels of PGA was also done in order to verify the above fact.

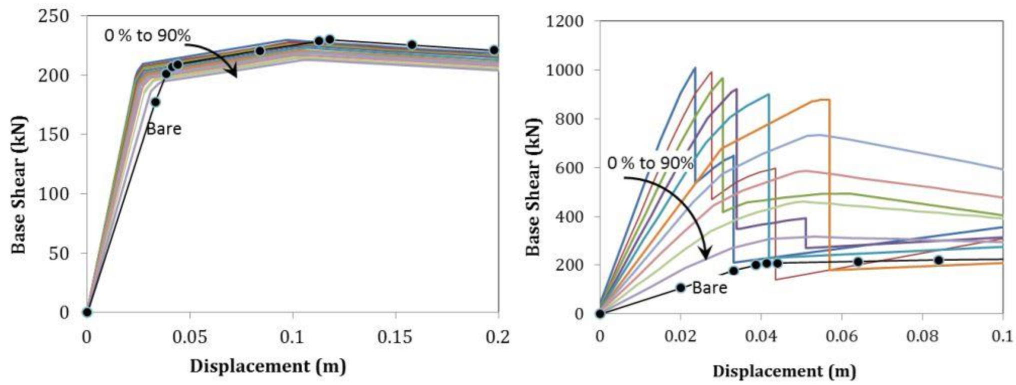


Fig. 5 - Pushover curves obtained for bare frame along with (a) OGS frames and (b) FI frames with different opening sizes (in % of the total wall area).

Table 2 - Assessment of ductility demand (μ_D) on OGS and FI frame for a given hazard.

Frames	**0% opening**			**50% opening**			**60% opening**		
PGA = 0.36*g*									
	S_{dy}	S_{du}	μ_D	S_{dy}	S_{du}	μ_D	S_{dy}	S_{du}	μ_D
FI	0.013	0.012	0.96	0.018	0.018	1	0.019	0.021	1.08
OGS	0.021	0.059	2.81	0.021	0.051	2.43	0.021	0.05	2.38
PGA = 0.24*g*									
	S_{dy}	S_{du}	μ_D	S_{dy}	S_{du}	μ_D	S_{dy}	S_{du}	μ_D
FI	0.013	0.008	0.64	0.018	0.012	0.67	0.019	0.014	0.72
OGS	0.021	0.035	1.67	0.021	0.031	1.48	0.021	0.031	1.46

3.2 Seismic Fragility Curves

Fig. 6 shows the fragility curves for 3 bay 4 storey OGS frames (left) and FI frames (right) with different opening sizes (in %) in the infill wall with spectral displacement (S_d) as the intensity measure. In both the cases, though the seismic fragility decreases with increase in opening size, it was observed that seismic fragility of OGS frames always remained high even after providing an opening of more than 50 %. Whereas, in case of FI frames, when an opening of about 50% or more is provided in the infill walls, its fragility tends to coincide with that of bare frame. Again, considering PGA as the intensity measure, though the seismic fragility of OGS frames was found to decrease with increase in opening size in the infill walls (Fig. 7), the decrease was only marginal compared to the decrease observed in the fully infilled frames. Only for higher damage states (i.e., Extreme and Collapse), fragility of OGS becomes comparable to that of bare frame and that too only when a very large opening of about 50% or more is provided. Whereas, for FI frames with any opening size, seismic fragility always remained significantly low as compared to bare frame. Thus, OGS frames are less sensitive to openings in the infill walls as compared to FI frames with the same opening sizes. Due to this fact, further fragility analysis is carried out for 0% (i.e., no opening) and 50% openings in infill only.

Fig. 8 illustrates the effect of different variable parameters, on the spectral displacement demand (S_d) for the entire domain of OGS buildings, i.e., 1 bay (1B) to 6 bay (6B), each having number of stories varying from two to six (2S – 6S) and with 0% and 50% opening (0% Op and 50% Op) in infills for a PGA range of 0.02g to 1g. Fig. 8(a) shows the variation of S_d with respect to number of bays (*Nb*) for the entire OGS buildings and PGA range. It was observed that the variation in S_d is not much sensitive to number of bay variation. In contrast, the S_d of OGS frames are more affected by variation in the number of storeys (*Ns*) as observed from Fig. 8(b).

Fig. 8(c) shows the effect of PGA levels on the S_d for the entire building domain considered. Clearly, the variation in S_d is more for higher PGA. In order to numerically interpret the effect of different parameters considered and find the relationship between variables, multiple variable linear regression analysis is carried out. In general, multiple regression procedures will estimate a linear equation of the form:

$$\mathrm{Y} = K + b1*\ln(\mathrm{X1}) + b2*\ln(\mathrm{X2}) + \ldots + bn*\ln(\mathrm{Xn}) + \sigma\varepsilon \tag{4}$$

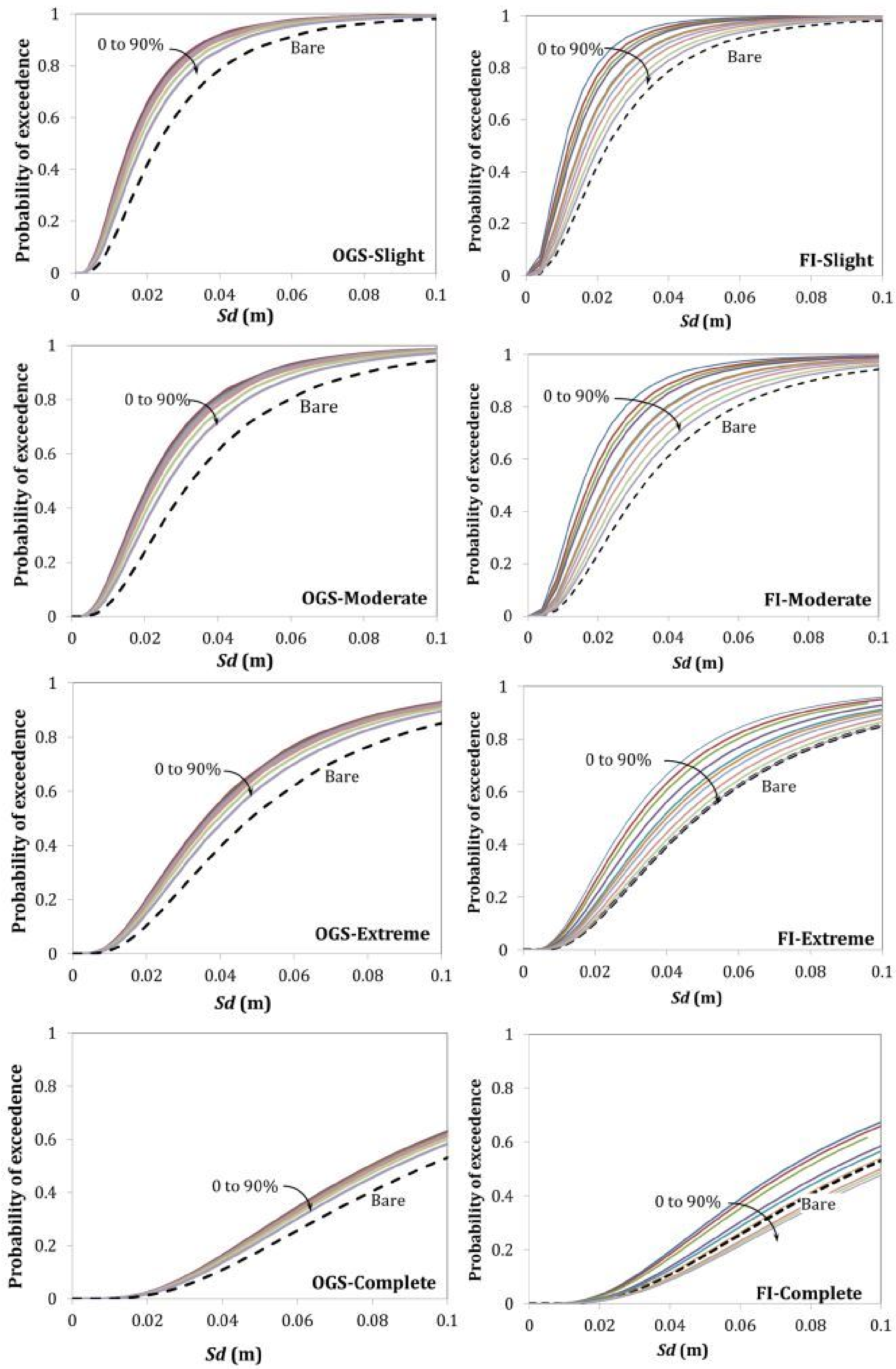


Fig. 6. Comparison of fragility obtained for OGS (left) and FI frames (right) with varying opening size in the infills of all stories with respect to S_d as intensity measure.

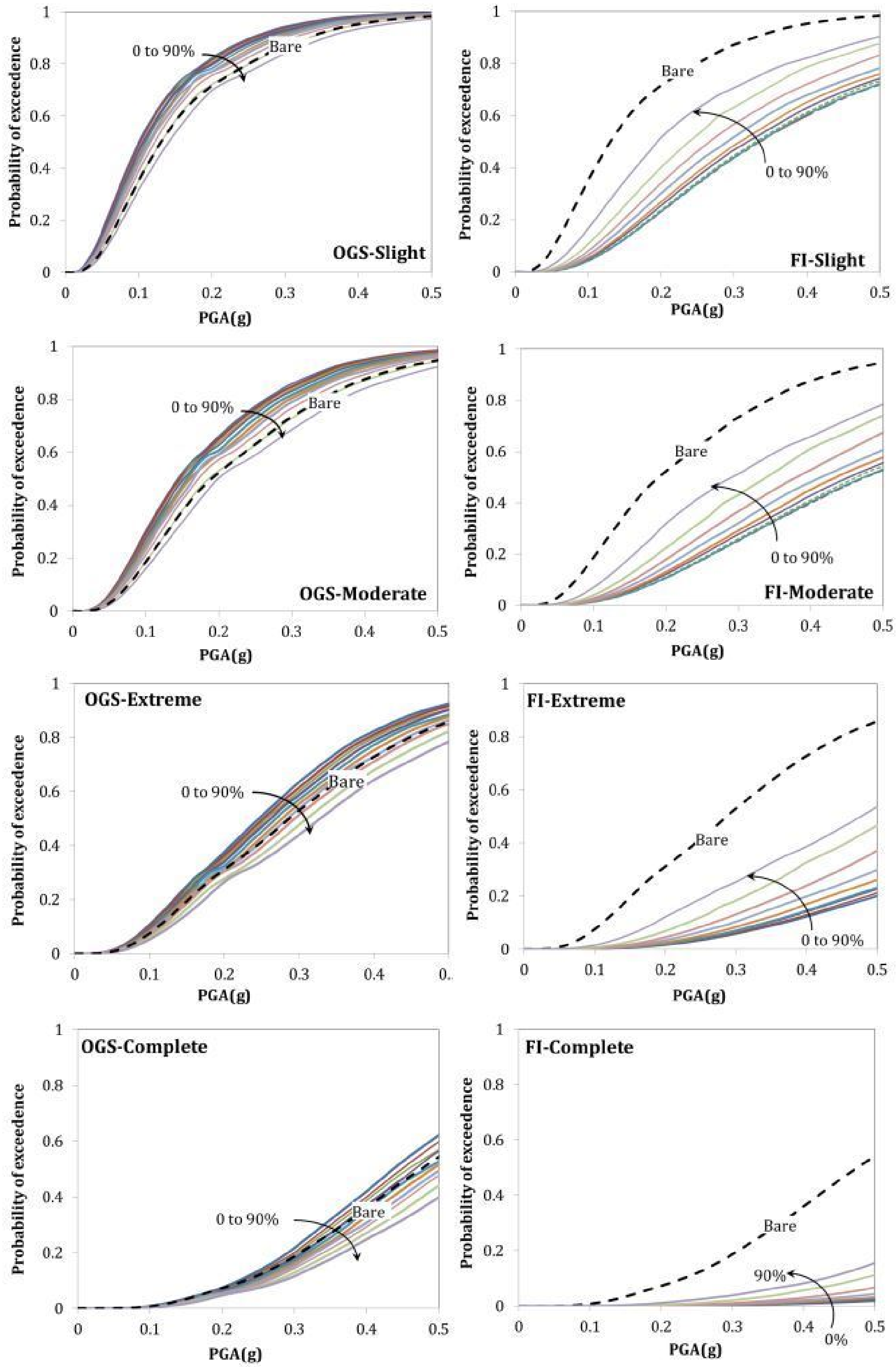


Fig. 7. Comparison of seismic fragility obtained for OGS (left) and FI frames (right) with varying opening size in the infills of all stories with respect to PGA as intensity measure.

Here, *K* is a constant and the independent variables (X1 to Xn) are *Nb*, *Ns*, *Op* and PGA. The coefficients (*b1* to *bn*) are estimated and used to interpret the dependent variable (Y) i.e., the spectral displacement demand (S_d) for varying PGA levels. The regression analysis is done separately for different ranges of PGA (as shown in Eq. 5 and Eq. 6), based on the scatter of S_d obtained with respect to PGA (Fig. 8c). $\sigma\varepsilon$ is the model error in logarithmic form, expressed separately for each equation as standard error of estimate (SEE). The coefficient of determination (R-squared) also shows how well the observed outcomes are replicated by the estimated model.

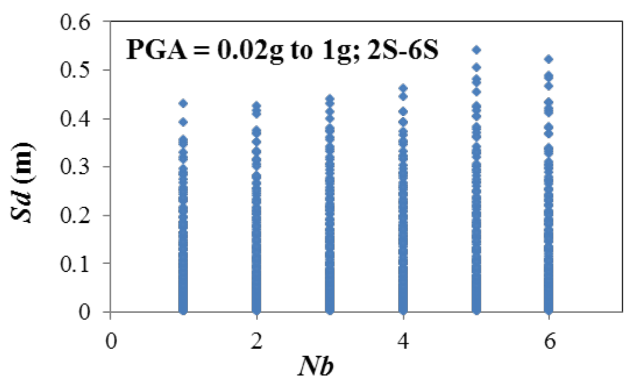
For PGA > 0.02 to >= 0.3g:

$$S_d = 0.04 \times N_b^{0.1} \times N_s^{0.77} \times (1.5 - Op)^{0.1} \times PGA^{1} \text{ (R-squared: 0.994; SEE: 0.001 m)} \quad (5)$$

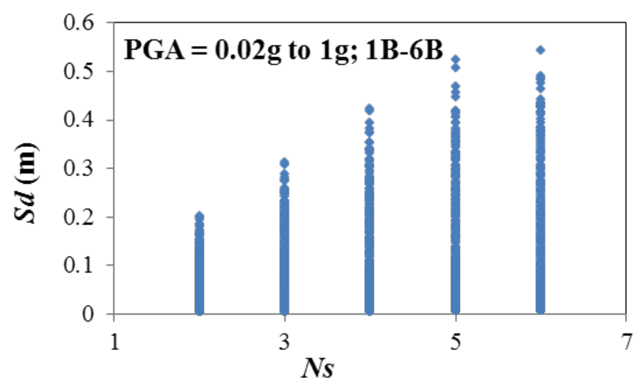
For PGA > 0.3 to >= 0.65g:

$$S_d = 0.058 \times N_b^{0.17} \times N_s^{0.94} \times (1.5 - Op)^{0.42} \times PGA^{1.64} \text{ (R-squared: 0.993; SEE: 0.004 m)} \quad (6)$$

The efficiency of the derived model for prediction of spectral displacement demand (S_d) is further tested by using the predictive equations to estimate the spectral displacement values for similar building frames analyzed in the past and reported in the literature. S_d for different PGA levels is obtained using ATC 40 methodology for a 3 bay 4 storied OGS frame studied in the past [3]. The values of S_d for the same frame are also obtained using the proposed regression model for OGS frames. A comparison of the analytical and predicted results of S_d shows that the developed predictive equations estimate the S_d values reasonably well (Fig. 8d). Such predictive equations can be of a great help in determining seismic response without a need for performing nonlinear analysis. This further aids in rapid assessment of seismic fragility and, hence, vulnerability of structures in a large scale as mentioned in Section 2.3. The prediction of S_d using proposed equation, however, needs to be validated with more analytical results.



(a)



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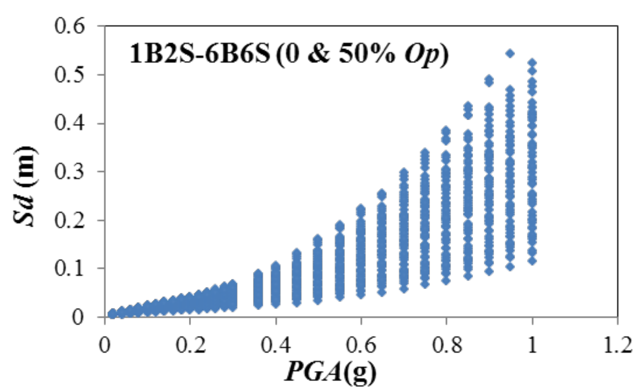
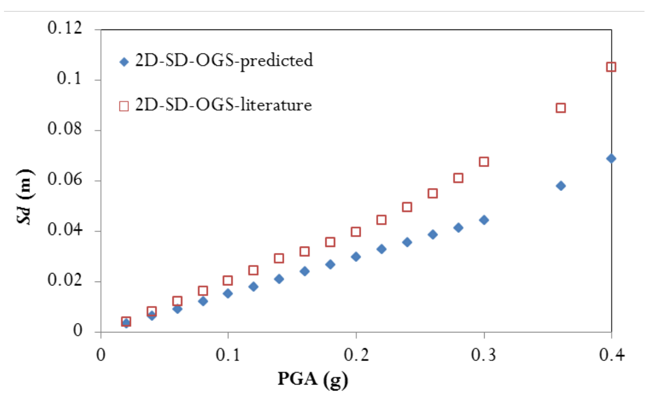


(c)



(d)

Fig. 8 –Parametric variation of spectral displacement demand (S_d) with respect to (a) number of bays, (b) number of storeys, (c) PGA, and (d) Comparison of S_d estimated using the predictive equation developed in the current study with that reported in the literature.

4. Conclusion

An extensive parametric study was carried out for understanding the seismic performance and fragility of OGS frames. On comparison of the behavior of OGS frames with FI frames, it was observed that the ductility demand imposed on the structure for any level of PGA always remain high for OGS frames with any opening size in the masonry infill walls. Comparison of OGS frames with varying opening size in upper storey infill with bare frame for collapse fragility shows that OGS frames even with 90 percent openings in the wall remain more vulnerable (i.e., higher probability of exceedence of a given damage state) than the bare frames. Interestingly, on the contrary, the lateral load behavior of FI frames was found to be better than even the bare frame when openings in the infill wall are more than 60%. The collapse fragility of FI frames was less than that of bare frame when an opening of about 60% or more is provided when spectral displacement is taken as the intensity measure. With respect to PGA as intensity measure, the performance of FI frame with any opening size is always better (i.e., having a lesser probability of exceedence of a damage state) than both OGS frame and bare frame. Based on the parametric study, a multiple variable regression equation is proposed for estimation of spectral displacement of OGS buildings. Such predictive equations can be of a great help in determining seismic response without a need for performing nonlinear analysis. This further aids in rapid assessment of seismic fragility and hence, vulnerability of structures in a larger scale.

Although the strength of Open Ground Storey (OGS) frames with different openings are comparable with the strength of bare frames, it was observed in the current study that the fragility of OGS frames is always more compared to bare frames with spectral displacement as the intensity measure. It is a common perception among stakeholders that increasing the opening size in infills of upper storey of OGS frames reduces its seismic vulnerability, but this was found to be incorrect from the results obtained in the current study. It was observed from the fragility curves that OGS frames even with more than 50 – 60% openings in infills shows higher probability of exceedence of a given damage state (in terms of displacement) as compared to bare frames, and hence are more vulnerable. The current study concludes that OGS frames with any reasonable opening size are highly vulnerable demanding that such construction practice must be stopped, especially in highly seismic prone areas.

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6. References

[1] Kaushik HB, Jain SK (2007): Impact of great December 26, 2004 Sumatra earthquake and tsunami on structures in Port Blair. *Journal of Performance Construction Facility*, ASCE, 21 (2), 128-142.

[2] ATC (1996): ATC 40, Seismic Evaluation and Retrofit of Concrete Buildings. Applied Technology Council, Redwood City, California, USA.

[3] Kaushik HB, Rai DC, Jain SK (2009): Effectiveness of Some Strengthening Options for Masonry-Infilled RC Frames with Open First Storey. *Journal of Structural Engineering*, ASCE, 135 (8), 925–937.

[4] CSI (2011): Structural Analysis Program (SAP2000) – Advanced, static and dynamic finite element analysis of structures. Computers and Structures, Inc., Berkeley, USA.

[5] Paulay T, Priestley MJN (1992): *Seismic design of reinforced concrete and masonry buildings*. Wiley, New York.

[6] Surendran S (2012): Lateral Load Behaviour of Masonry Infilled RC Frames with Central Opening. *Master of Technology Project Report*, Department of Civil Engineering, Indian Institute of Technology Guwahati, Guwahati, Assam, India.

[7] FEMA 356 (2000): Prestandard and commentary for the seismic rehabilitation of buildings. Building Seismic Safety Council (US), & Applied Technology Council.

[8] Freeman SA (1998): The Capacity Spectrum Method as a Tool for Seismic Design. *11th European conference on earthquake engineering*, 6–11.

[9] BIS (2002): IS 1893 (Part 1): 2002, Criteria for Earthquake Resistant Design of Structures: Part 1 General Provisions and Buildings, 5th Revision, Bureau of Indian Standards, New Delhi, India.

[10] Barbat AH, Pujades LG, Lantada N (2008): Seismic damage evaluation in urban areas using the capacity spectrum method: Application to Barcelona. *Soil Dynamics and Earthquake Engineering*, 28, 851–865.

[11] HAZUS (2003): Multi-hazard Loss Estimation Methodology, Earthquake Model, HAZUS-MH MR4 *Technical Manual*. National Institute of Building Sciences and Federal Emergency Management Agency (NIBS and FEMA). Washington, DC.

[12] FEMA-P-58-1 (2012): Seismic Performance Assessment of Buildings, Volume 1 – Methodology. Federal Emergency Management Agency, Washington, D.C., USA.

[13] Kaushik HB, Choudhury T (2015): Vulnerability Analysis of Buildings for Seismic Risk Assessment: A Review". *The Bridge and Structural Engineer*, The journal of the Indian National Group (ING) of International Association for Bridge and Structural Engineering (IABSE), 45 (1).