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A methodology for development of seismic fragility curves for URBM buildings

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Abstract. This paper presents a simple methodology that integrates an improved storey shear modelling, Incremental Dynamic Analysis and Monte Carlo Simulation in order to carryout vulnerability analysis towards development of fragility curves for Unreinforced Brick Masonry buildings. The methodology is demonstrated by developing fragility curves of a single storey Unreinforced Brick Masonry building for which results of experiment under lateral load is available in the literature. In the study presented, both uncertainties in mechanical properties of masonry and uncertainties in the characteristics of earthquake ground motion are included. The research significance of the methodology proposed is that, it accommodates a new method of damage grade classification which is based on 'structural performance characteristics' instead of 'fixed limiting values'. The usefulness of such definition is discussed as against the existing practice.

Keywords: vulnerability analysis; fragility curve; capacity curve; capacity spectrum; improved storey shear modelling; incremental dynamic analysis; monte carlo simulation

1. Introduction

Unreinforced Brick Masonry (URBM) is widely used for low-rise buildings because of economy, ease in construction, eco-efficiency, durability and various other considerations (Balasubramanian *et al*. 2011). They form a major building stock especially in the developing and under developed countries. In India, about 45% of existing buildings are made with burnt bricks walls (BMTPC 2006). Most of these buildings are not designed to resist the earthquake forces. On the other hand, post earthquake damage surveys indicate that, these are one of the most vulnerable

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types of buildings when subjected to earthquakes (Dowrick 2003). Seismic vulnerability analysis of these buildings would be useful in assessing the extent of possible damage in the event of an earthquake (Priya *et al*. 2007). Thus, assessment of the seismic vulnerability of URBM buildings is of practical significance. Evaluation of the seismic vulnerability of URBM buildings is critical for proper earthquake disaster mitigation. Different procedures are available in the literature for carrying out vulnerability analysis of URBM buildings. The differences in the methods are primarily in (a) modelling and (b) analysis of URBM buildings.

In modelling of URBM buildings, many techniques have been used including (i) equivalent frame type models (Gambarotta and Lagomarsino 1997, Kappos *et al*. 2006, Chen *et al*. 2008, Mallardo *et al*. 2008, Pasticier *et al*. 2008, Rota *et al*. 2008), (ii) finite element analysis based models using 2D membrane elements (Mallardo *et al*. 2008), (iii) limit analysis based models (Lang 2002, Lang and Bachmann 2003, Priya *et al*. 2007, Pagini *et al*. 2008) and (4) storey shear mechanics based models (Park *et al*. 2009). While finite element models using 2D membrane elements are capable of producing accurate results, these require intensive computational effort. Hence, other simplified modelling techniques are preferred especially for vulnerability analysis which involves number of repetitions.

The limit analysis based model (Lang 2002, Lang and Bachmann 2003, Priya *et al*. 2007) is found to have a drawback of not taking into account the structural behavior of the spandrels. Such approximations could significantly affect the estimated structural performance of the walls whose spandrel is weaker than the piers (i.e., weak spandrel-strong pier condition). Similarly, in the limit analysis based model (Pagini *et al*. 2008), the shear capacity of the first floor is assumed (approximately) to represent the shear capacity of the entire building. In the equivalent frame type models, while the portion of the spandrel above openings alone is considered to be deformable, the portion of the spandrel above the piers is assumed to be rigid. The rigid portions thus, only ensure the compatibility between the piers and deformable portion of the spandrels. In real structures, the portions of spandrels above the piers are found to have finite stiffness and strength (Balasubramanian *et al*. 2011), and, hence the idea of considering them to be rigid, needs to be revisited. Moreover due to orthotropic behavior of brick masonry, the usage of common force-deformation relations for piers and the deformable portion of spandrels also need to be revisited. The storey shear mechanics based model (Park *et al*. 2009) could be a suitable choice, but considering the entire spandrel as a single element is proved to be incorrect. The improved storey shear model incorporates the refinement in the discretization scheme in order to overcome the aforementioned issue (Balasubramanian *et al*. 2011).

The commonly used methods of analysis of buildings include (i) non-linear static analysis based methods or capacity spectrum based methods (Lang 2002, Lang and Bachmann 2003, Kappos *et al*. 2006, Priya *et al*. 2007, Kaplan *et al*. 2008, Pagini *et al*. 2008) and, 2. non-linear time history analysis based methods or Incremental Dynamic Analysis (IDA) based methods (Singhal and Kiremidjian 1996, Dumova-Jovanoska 2004, Erberik 2008a, Erberik 2008b, Pasticier *et al*. 2008, Park *et al*. 2009). The capacity spectrum based method compares the capacity spectrum with a demand spectrum to obtain the performance point and the fragility curve is obtained as the Cumulative Distribution Function (CDF) of the corresponding spectral displacement. In IDA based method nonlinear dynamic analyses of a structure have to be carried out under several levels of severity by scaling ground motion record. The fragility for a damage grade is the area under the Probability Density Function (PDF) of the response of the structure beyond the level of response corresponding to the damage grade. Further, severity of ground motion (Intensity Measure, IM) is often characterized by different parameters such as spectral displacement, Peak Ground Acceleration (PGA), Peak Ground Velocity (PGV), EMS98 and Modified Mercalli Scale (MMS). Damage grades and performance indicators are often classified based on drift, crack width and volume loss ratio (Balasubramanian *et al*. 2008).

In general, for carrying out vulnerability analysis, the following points are considered:

a. Unreinforced brick masonry buildings in general have a high initial stiffness and hence the contribution of the fundamental mode is significant compared to higher modes towards their dynamic response. Nevertheless, in estimating the structural performance during an event of earthquake, non-linear time history analysis is more rational than non-linear static analysis provided: (i) the accelerogram(s) used for the dynamic analysis and (ii) the hysteretic model are properly selected.

b. Many countries, including India, are developed/developing PGA contour maps for regional seismic hazard specification (Balaji Rao *et al*. 2011, NDMA 2011). Thus, PGA can be the relevant choice for the ground motion intensity parameter.

c. Similarly, the drift can be a good choice of damage measure because, it is a measure based on structural performance considerations, and also, a good representative of the overall behaviour/ damage unlike crack width or volume loss ratio (Balasubramanian *et al*. 2008). Moreover it is a convenient measure that can be obtained from dynamic analysis.

d. It is to be noted that the buildings of different plan geometry can undergo damage to a particular level of severity at different ranges of drifts; hence it is more rational to define damage grades that are specific to the typology. While doing so structural performance characteristics may be taken into account.

A new methodology is proposed by incorporating the improvements in ‘modelling’ (i.e., refinement in the discretization scheme as explained earlier) and points of consideration regarding ‘vulnerability analysis’ as mentioned above. The methodology is demonstrated through an illustrative study and the results are discussed in subsequent sections.

2. Proposed methodology

The proposed methodology integrates, (1) improved storey shear modelling (Balasubramanian *et al*. 2010), (2) Incremental Dynamic Analysis (Vamvatsikos and Cornell 2004) and, (3) Monte Carlo Simulation (MCS) for handling uncertainties. The step-by-step procedure is as follows (also presented in Fig. 1);

1. Consider PGA as the Intensity Measure (IM) and the drift as the Damage Measure (DM).
2. Consider a ‘suite’ of earthquake records and perform ‘multi-record IDA’ in order to take into account the uncertainty with respect to characteristics of earthquake ground motions.
3. From the geometry of the URBM building to be studied, identify the in-plane walls.
4. Consider possible uncertainties in mechanical properties of material used.
5. Generate capacity curve using the improved storey shear modelling technique (Balasubramanian *et al*. 2010) for each realization. It is to be noted that the improved storey shear modelling is suitable for URBM building with rigid diaphragm only. For the above, consider the following:

a. Only the in-plane walls resist the lateral forces and the out-of-plane walls remain intact with the in-plane walls.

b. Discretize the in-plane walls into an assemblage of masonry piers.
c. Estimate the shear capacity of piers in accordance with Eq. (1) (ENV 1996-1-1:1995).

$$\text{shear capacity}, V_u = \min\begin{cases} (f_{v0} + \mu\sigma)Dt \\ \text{or} \\ \dfrac{f_t Dt}{b}\sqrt{1+\dfrac{\sigma}{f_t}} \\ \text{or} \\ \dfrac{\sigma Dt}{2\alpha_v}\left(1-\dfrac{\sigma}{\kappa f_M}\right) \end{cases} \tag{1}$$

where, f_M is the compressive strength of masonry; f_t is the tensile strength of masonry; f_{v0} is the shear strength of masonry under zero compressive load; D is the length of the pier; t is the thickness of the pier; μ is the bed-joint sliding friction; σ is the compression stress on the pier; α_v is the shear ratio; κ is the coefficient which takes into account the vertical stress distribution at the compressed toe $(= 0.85)$; b is the constant that accounts for the distribution of shear stress at the centre of the wall $(= 1 \le q \le 1.5)$; q is the geometric aspect ratio $(= H/D)$; H is the height of the pier.

d. Estimate the initial stiffness of piers using the improved storey shear modelling technique, Eq. (2) (Balasubramanian *et al*. 2011).

$$\begin{aligned} &\text{intitial stiffness, } K = \frac{Et}{(pq^3 + 3q)} \\ &\text{constant}, p = \frac{\begin{pmatrix} q^4 + 4q^3r + 4q^3s + 3q^2r^2 + 3q^2s^2 + \\ 14q^2rs + 12qr^2s + 12qrs^2 + 12r^2s^2 \end{pmatrix}}{q^4 + q^3r + q^3s + 2q^2rs} \end{aligned} \tag{2}$$

where, E is the elastic modulus of masonry; t is the thickness of the pier; p is the constant depending on geometric aspect ratio of the pier and boundary condition at top and bottom; q is the geometric aspect ratio of the pier; r and s are the geometric aspect ratios of the piers that are below and above respectively (also, ‘0’ for fixed end and ‘1’ for free end).

e. Estimate the deformability limits, yield and ultimate displacement of the piers in accordance with Eq. (3) (Tomaževič 1999).

$$\begin{aligned} &\text{yield displacement}, \delta_e = \frac{V_u}{K} \\ &\text{ultimate displacement}, \delta_u = 1.5\delta_e \end{aligned} \tag{3}$$

f. Consider all the three different modes of failure, namely, diagonal shear failure, sliding shear failure and flexure/rocking failure, for the piers adjacent to the openings. On the other hand, consider only diagonal shear failure and sliding shear failure as possible mechanisms for the piers within spandrel and sill portion (because of shear interaction between adjacent piers in horizontal direction as explained in Balasubramanian *et al*. 2011).

g. Obtain the capacity curve of the building by assembling the capacity curves of the piers in accordance with the load sharing path.

6. Define suitable damage grades against which the performance of the building needs to be

studied.

7. Carry out the IDA by considering the hysteretic behaviour of masonry as shown in Fig. 2 (Park *et al*. 2009).

8. Determine the number of samples within each damage grade for a given intensity level and hence compute values of fragility.

To demonstrate the methodology, an illustrative study has been carried out. In this study, the URBM building tested under displacement controlled lateral cyclic loading (Murty *et al*. 2004), presented in Fig. 3 is considered. This particular building is chosen since relevant results of

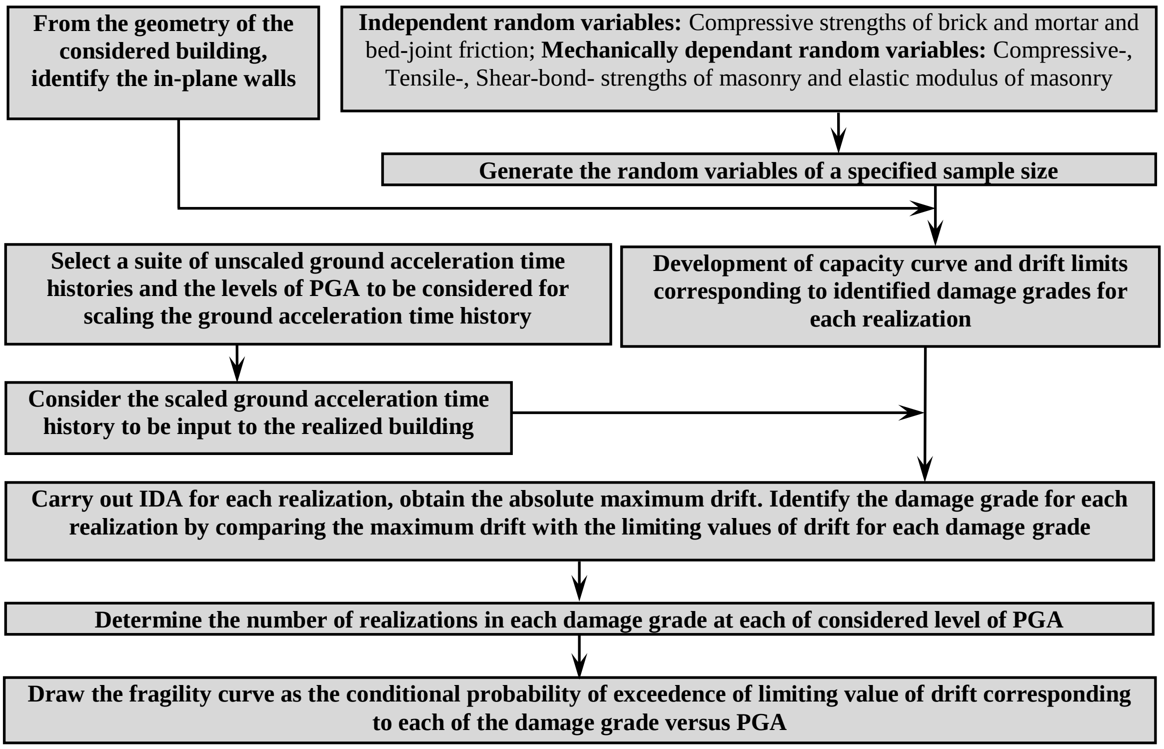


Fig. 1 Schematic sketch showing step by step procedure for developing fragility curves for URBM buildings

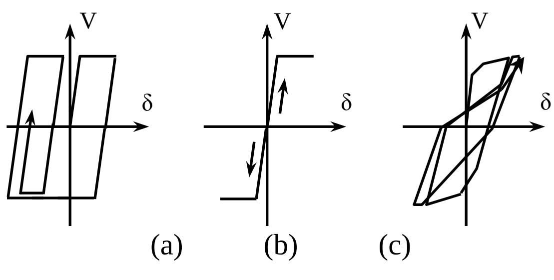
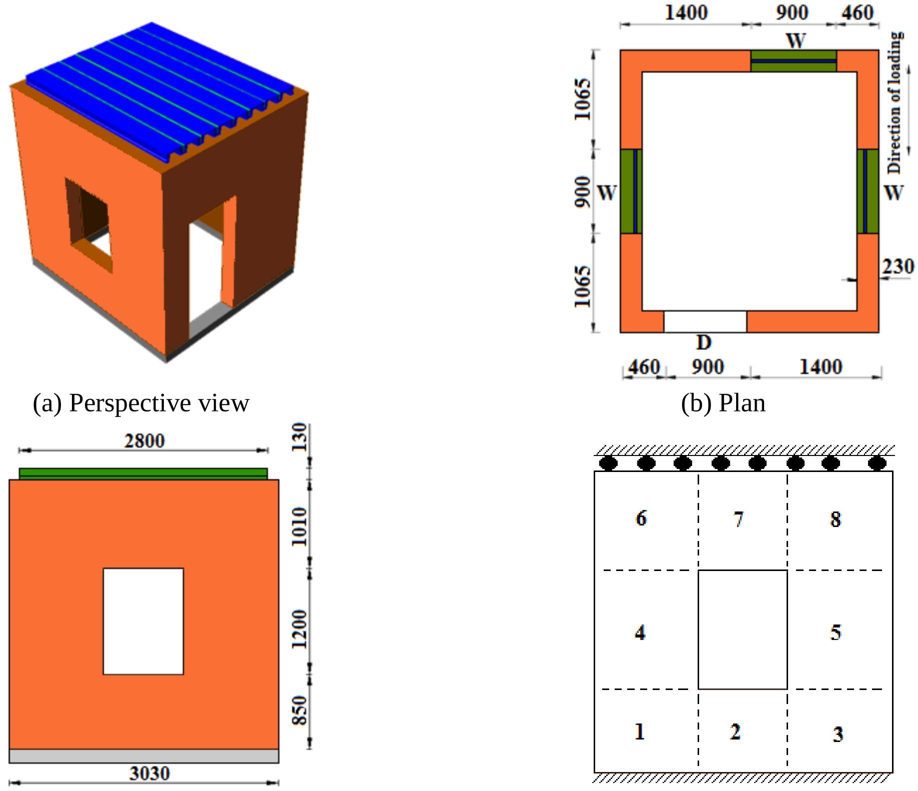


Fig. 2 Idealization of hysteretic behaviour of masonry piers corresponding to each failure mode: (a) Sliding (b) Flexural/Rocking and (c) Diagonal tension (in line with Park *et al*. 2009)



(a) Perspective view (b) Plan

(c) In-plane wall, geometry and discretized model

Fig. 3 Brick masonry building considered in the study (All dimensions are in ‘mm’)

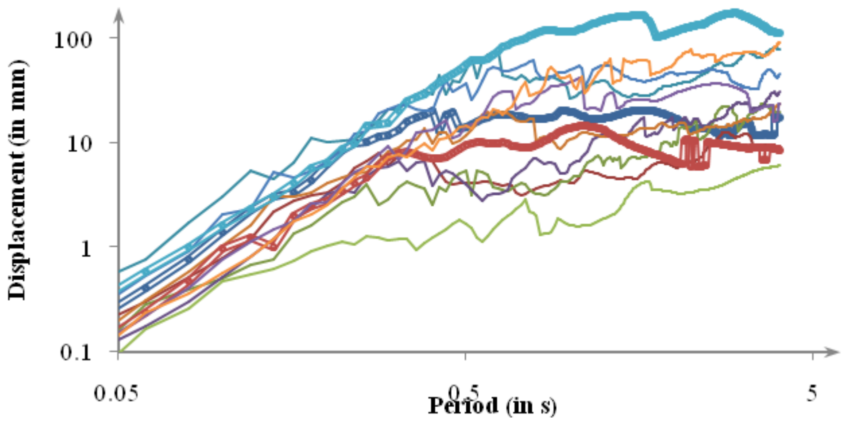


Fig. 4 Inelastic response spectra for the earthquake time histories considered in the study in log-log scale.[Note: In plotting the response spectra, a constant ductility of 1.85 (which is the ratio between the mean value of drift limits corresponding to DG3 and DG2, see Section 3 for more details) has been used. The near-field earthquakes are indicated using “bold, double lines” and the far-field earthquakes are indicated using “thin, single lines”]

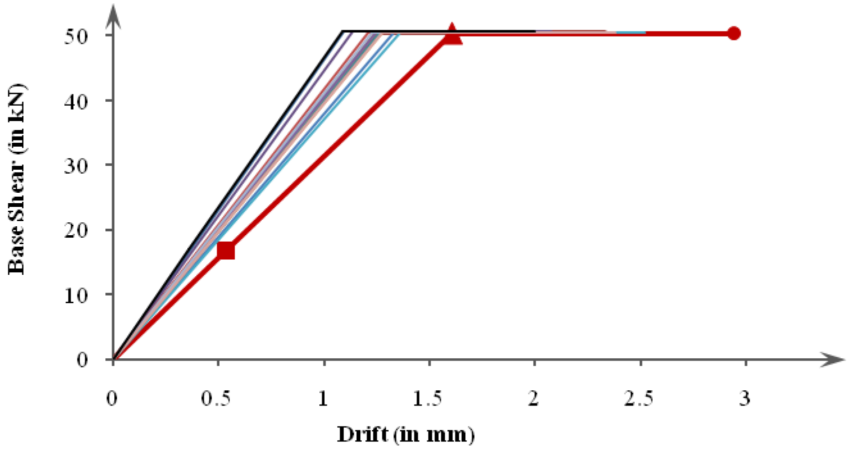


Fig. 5 Capacity curves for a set of 15 realizations, different damage grades are indicated on one of the realizations ('■': DG1; '▲': DG2; '●': DG3)

Table 1 Earthquake time histories considered in the study

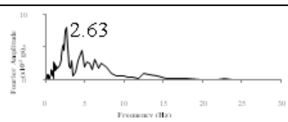
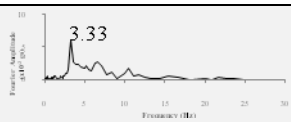
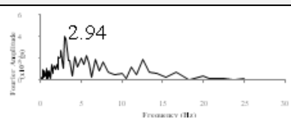
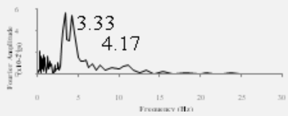
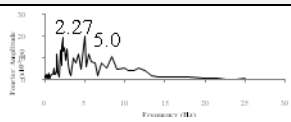
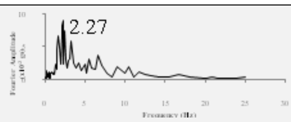
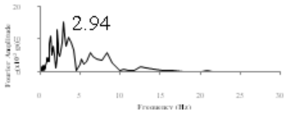
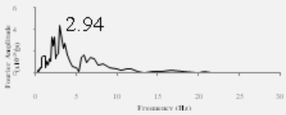
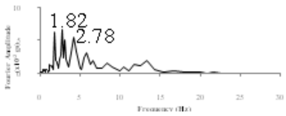
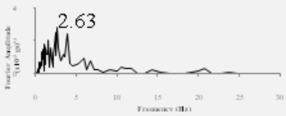
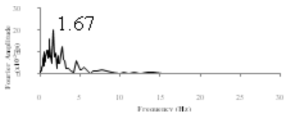
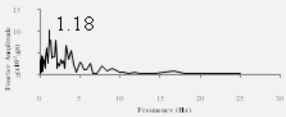
Sl. No.	Earthquake, Date/ Recording station direction]	R (in km) [NF/FF]	PGA (in g)	Fourier spectra (indicating frequencies corresponding to peak amplitudes in Hz)
1	Dharmsala Earthquake, 26/04/1986 Shahpur (32.21° N, 76.19° E) [T, 165°]	9.98 [NF]	0.248	
2	North-East India Earthquake, 10/09/1986 Saitsama (25.72° N, 92.39° E) [T, 175°]	44.79 [FF]	0.139	
3	India-Burma Border Earthquake, 18/05/1987 Diphu (25.92° N, 93.44° E) [L, 90°]	105.03 [FF]	0.086	
4	India-Bangladesh Border Earthquake, 06/02/1988 Nongkhlaw (25.69° N, 91.64° E) [T, 170°]	116.27 [FF]	0.114	
5	India-Burma Border Earthquake, 06/08/1988 Diphu (25.92° N, 93.44° E) [T, 180°]	189.94 [FF]	0.337	
6	India-Burma Border Earthquake, 10/01/1990 Berlongfer (25.77° N, 93.25° E) [L, 256°]	230.01 [FF]	0.145	

Table 1 Continued

7	Uttarkashi Earthquake, 10/10/1991 Uttarkashi (30.73° N, 78.45° E) [T, 345°]	32.52 [FF]	0.309	
8	Chamba Earthquake, 24/03/1995 Chamba (32.55° N, 76.12° E) [L, 0°]	8.2 [NF]	0.146	
9	India-Burma Border Earthquake, 06/05/1995 Diphu (25.92° N, 93.44° E) [T, 180°]	215.91 [FF]	0.102	
10	India-Bangladesh Border Earthquake, 10/05/1997 Jellalpur (25.00° N, 92.46° E) [T, 2°]	24.45 [FF]	0.138	
11	Chamoli Earthquake, 29/03/1999 Gopeshwar (30.40° N, 79.33° N) [T, 20°]	8.70 [NF]	0.359	
12	Kachchh Earthquake, 26/01/2001 Ahmedabad (23.03° N, 72.63° E) [L, 78°]	238 [FF]	0.106	

Note: R: Epicentral distance; T: Transverse direction; L: Longitudinal direction as referred in the Atlas of Indian Strong Motion Records (Shrikande 2001); True bearings are as obtained from internet (Cosmos) NF: Near-field earthquakes; FF: Far-field earthquakes.

Table 2 Mechanical properties of materials considered in the study

Sl. No.	Parameter	Statistical properties / Description
1	Compressive strength of brick, f_b	Mean: 24.2 MPa (Murty *et al*. 2004); COV: 0.15; Lognormal (Assumed); Statistically independent random variable.
2	Compressive strength of mortar, f_m	Mean: 5.9 MPa for portion below lintel and 2.1 MPa for portion above lintel (Murty *et al*. 2004); COV: 0.15; Lognormal (Assumed); Statistically independent random variable.
3	Bed-joint friction coefficient, μ	Mean: 0.4 (ENV 1996-1-1:1995); COV: 0.10; Lognormal (Assumed); Statistically independent random variable.
4	Compressive strength of masonry, f_M	$f_M = 0.63 f_b^{0.49} f_m^{0.32}$ (Kaushik *et al*. 2007) with a modelling error 'e' which is an independent random variable assumed to have a normal distribution with COV: 0.1; Mechanically dependent random variable.
5	Elastic Modulus of masonry, E_M	$E_M = 250 f_M$ (Kaushik *et al*. 2007); Mechanically dependent random variable.

Table 2 Contined

6	Shear bond strength of masonry, f_{v0}	$f_{v0} = 0.0484 f_M^2 - 0.1651 f_M + 0.2841$(obtained from Fig. 9 of Sarangapani *et al*. 2005); Mechanically dependent random variable.
7	Tensile strength of masonry, f_t	$f_t = 0.0346 f_M^2 - 0.0991 f_M + 0.1546$(obtained from Fig. 9 of Sarangapani *et al*. 2005, corresponding to flexural bond strength); Mechanically dependent random variable.

experiments conducted are available for the purpose of comparison. The chosen URBM building conforms to IS:1905-1987(Reaffirmed 2002) with respect to vertical load resistance and IS:13828-1993 with respect to size and position of openings.

In order to take into account the uncertainties in the characteristics of earthquake ground motion, earthquake time histories corresponding to 12 earthquakes for which digital data is available in the Atlas of Indian Strong Motion Records (Shrikande 2001) are considered (Table 1). The suite of earthquake time histories considered in this study contains both near-field and far-field earthquake data put together, since: (1) the available data is scanty and, (2) both types of earthquakes are possible events to be considered while carrying out vulnerability analysis of buildings. Even though limited number of earthquake time histories are considered in this study, it is to be noted that the unscaled PGA, frequency corresponding to peak amplitude in Fourier spectra and spectral displacement at any given period (Fig. 4) show a considerable scatter, which is desirable. Also, from Fig. 4, it is noted that the response spectrum corresponding to the near-field and far-field earthquakes are not separated significantly; they form a uniformly dispersed band.

In order to handle uncertainty in the mechanical properties of masonry some of the properties are considered as statistically independent random variables and remaining as mechanically dependent random variables. The properties like compressive strengths of brick and mortar, and bed-joint friction coefficient are considered as statistically independent random variables. The remaining properties like compressive strength, shear strength, tensile strength, modulus of elasticity, and bed-joint friction coefficient of masonry are assumed to be mechanically dependent random variables (Table 2). In order to have sufficient sample, Monte Carlo simulation of 100,000 cycles have been carried out. Also, in this study, for the purpose of demonstration, the building is idealized as a single degree of freedom system with an estimated mass of 5175 kg and a damping ratio of 10%. Non-linear dynamic analysis program given in the literature (Paz 1997) has been used which accounts the hysteretic model, similar to that shown in Fig. 2(a).

More importantly, in the present study, the damage grades are defined based on the considerations like immediate occupancy, necessity of minor repair and necessity of major repair. Accordingly, three damage grades, namely, DG1 corresponding to 33% of yield point of the building, DG2 corresponding to yield point of the building, DG3 corresponding to the collapse of the building (Fig. 5) are considered. Then, for each realization, the drift limits corresponding to the considered damage grades are to be obtained. It is to be noted that this definition of damage grade classification is at variance with that of FEMA-356 recommendation which is based on fixed limiting values (i.e., drift as a fraction of the height of the building). By using fixed value of drift for classifying damage grades, the seismic vulnerabilities of any two buildings of equal height but, with different plan geometry, cannot be differentiated and hence, the proposed definition is based on the capacity curve itself is more rational.

3. Results and discussion

For the one roomed, single storey URBM building of size 3.03m x 2.76m x 3.19m, damage states and the earthquake ground motion, considered in the study, the results are presented in the Figs. 6-8. The following observations are made from the results of the study:

1. The mean value of initial stiffness as realized in the study is 41.34 kN/mm which is comparable to 45.81 kN/mm, obtained from the experiment (Murty *et al*. 2008). Similarly, mean value of base shear corresponding to damage grades DG2 (and hence for DG3, see Fig. 5) is 50.52 kN which is comparable to 43.7 kN, obtained from the experiment.
2. From Fig. 8, initiation of DG1 is observed around 0.024g and the fragility against DG1 approached unity around 0.51g.
3. From Fig. 8, initiation of DG2 is observed around 0.024g and the fragility against DG2 approached unity around 1.04g.
4. From Fig. 8, initiation of DG3 is observed around 0.051g and the fragility against DG3 approached unity around 1.15g
5. From Fig. 7, it is noted that the differences in the mean values of the limiting drift ratios, obtained using the capacity curves, are respectively 0.027% and 0.035% between DG2 and DG1, and, between DG3 and DG2.
6. Using the proposed methodology the fragility curves are drawn for the three damage grades. For the hysteretic behaviour considered, once the building enters the plastic regime, it is expected that the response oscillates about the permanent displacement/set leading to large displacements. Thus, once the building reaches DG2 approaching DG3 becomes imminent. This is also evident from the fragility curves shown in Fig. 8. IDA Curves of typical 3 realizations out of 1,00,000 simulation cycles considered in the study (Fig. 6), show the non-monotonic relation between the PGA (IM) and maximum Drift (DM) as pointed out in literature (Vamvatsikos and Cornell, 2004). Nevertheless, the overall trend is that DM is increasing with IM. This observation could result in localized kinks in the fragility curves obtained from IDAs. To generate fragility curves of engineering significance the fragility curves so obtained are smoothened as shown in Fig. 8.
7. The frequency distribution of initial stiffness, maximum base shear and drift corresponding to the different damage grades are presented in Fig. 7. The initial stiffness, mechanically dependent on compressive strength of masonry showed a COV of about 0.13. On the other hand, maximum base shear showed an insignificant value of COV (0.0015). This is because, for the building considered in this illustrative study, rocking of piers adjacent to the window opening was observed to be the governing mode of failure which, as expected, is not significantly influenced by the compressive strength of masonry.
8. From Fig. 7(c), two significant observations can be made. Firstly, the mean drift ratio limits corresponding to the identified damage grades are lower when compared to the drift ratio limits specified in FEMA-356 (for structural performance levels of immediate occupancy: 0.3%, life safety: 0.6% and collapse prevention: 1.0%). This can be attributed to the configuration of the building considered, which imparts high stiffness to the building. Secondly, the values of drift ratio limits for each of the three damage grades considered in the study show significant scatter (COV of approximately 0.13). The upper tail of less severe damage grades are extending beyond the lower tail of more severe damage grades in this case. Considering these two observations, the perception of defining damage grades based on single limiting value for each damage grade

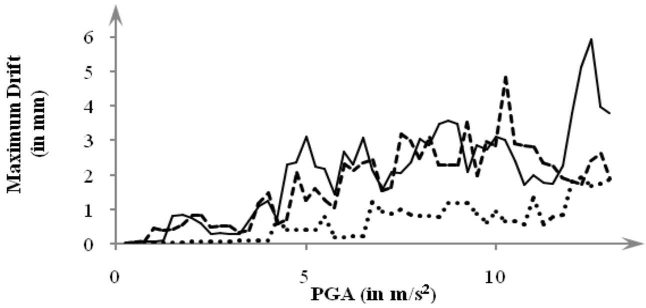
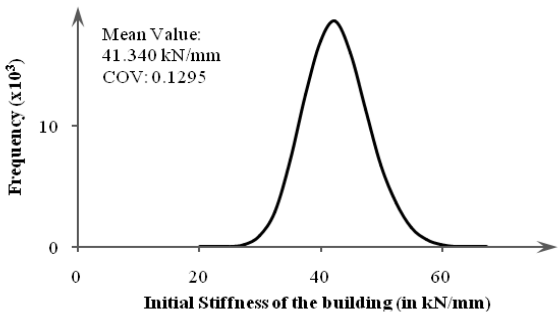
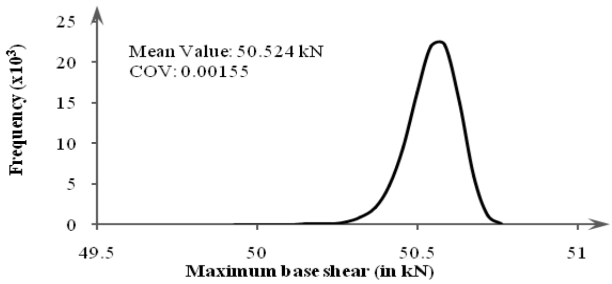


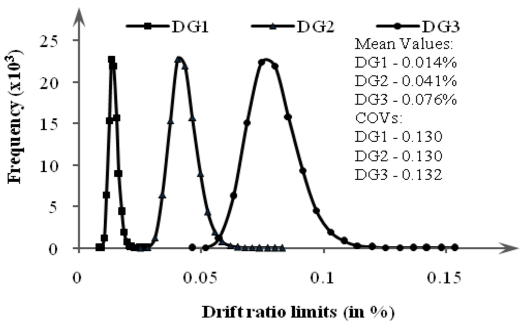
Fig. 6 Maximum drift versus PGA for typical three realizations



(a)



(b)



(c)

Fig. 7 Observed frequency distribution of the output random variables: (a) Initial Stiffness of the Building, (b) Maximum base shear, and, (c) Drift ratio limits corresponding to DG1, DG2 and DG3

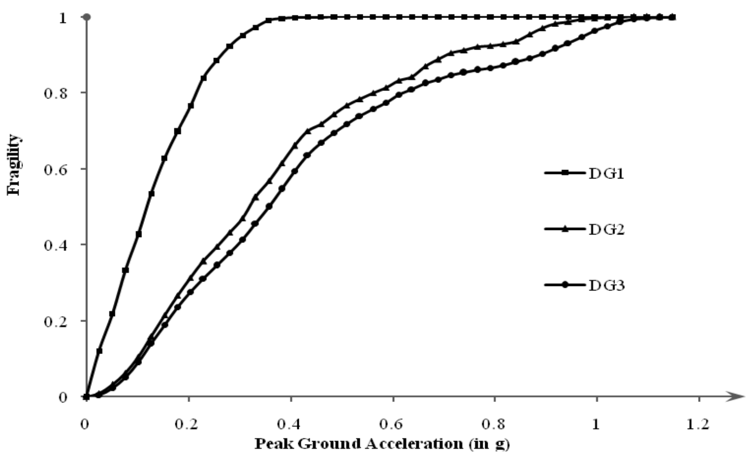


Fig. 8 Smoothened fragility curves against the damage grades for the considered building

to be revisited. Rather, it is proposed that the damage grades can better be defined based on the structural performance characteristics (as has been done in this study).

In this study, uncertainties in characteristics of earthquakes is assumed to be accounted for by considering time histories corresponding to 12 different earthquakes available in the Atlas of Indian Strong Motion Records (Shrikande 2001). Nevertheless, in order to 'properly' account for the uncertainties in the characteristics of earthquake ground motion, there is a need to identify "region specific suites" of earthquake ground motion records for different seismogenic regions of India, specifically to be used for brick masonry buildings.

In order to use these fragility curves for engineering decision making (viz; performance based design), it is vital to fix the acceptable probabilities of failure for different damage grades. The estimated conditional probability of failure corresponding to the seismic demand for a given region and damage grade is to be compared with the acceptable value. If the computed value is greater than the acceptable value, then there is a need to improve the seismic performance of the building for the particular damage grade (Balasubramanian *et al*. 2010a). Thus, these fragility curves will be helpful for identifying the measures to be adopted to improve the in-plane shear capacity of the URBM piers, and hence, the seismic resistance of the buildings.

4. Conclusions

This paper presents a simple methodology for carrying out vulnerability analysis towards development of fragility curves for URBM buildings. The proposed methodology integrates improved storey shear modelling, IDA and MCS. The improved storey shear modelling is suitable for buildings with rigid diaphragm only. The proposed methodology can handle the uncertainties with respect to mechanical properties of materials and also, uncertainties with respect to characteristics of earthquake ground motion. The fragility curves will be helpful for identifying indicating necessity the effective measures to be adopted to improve the seismic resistance of the buildings. It is also important to note from this study that, it is better to grade the damage levels based on 'structural performance characteristics' than 'fixed limiting values'.

Applicability of the improved storey shear modelling for buildings with flexible diaphragm needs to be studied. Similarly, there is a need to identify "region specific suites" of earthquake ground motion records for different seismogenic regions of India, specifically to be used for brick masonry buildings. Studies towards these are being continued at CSIR-SERC.

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