An approach to derive seismic fragility curves

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Abstract Seismic risk assessment is needed for the evaluation of disaster management, emergency preparedness and prior retrofitting. Based on feasible collapse mechanisms and calculation of their associated seismic parameter Peak Ground Displacement, the failure analysis of the structure is identified. Further, the outputs are manipulated to produce a measure vulnerability obtained from failure analysis. In regions of moderate and high seismicity, Zone – III Fragility analysis is used. An investigation is made on low rise Unreinforced masonry (URM) structure located in Zone – III, particularly in Chennai. To obtain capacity curve and bi-linear capacity curve the structure is modelled as Equivalent Frame Model (EFM), and non-linear static pushover analysis (NPOA). The structure damages, under various level of seismic excitation was developed by an analytical seismic fragility curve. For prioritizing retrofit, pre-earthquake planning, and loss estimation tools such as cost-benefit analysis this curve is utilized. The damage states considered for determining probability of exceeding a specified damage state involved in fragility curve generation are slight, moderate, extreme and collapse.

Key words: vulnerability, retrofitting, PGD, fragility curves, seismic assessment, URM.

1. Introduction
Those structures which are built from individual units laid in and bound together by mortar are said to be masonry structures. Without the intervention of skilled labours till twentieth century masonry structures were constructed in Chennai. Poor performance in masonry structures results in failure of structure during earthquake masonry structures are well known for economical construction and simplicity.

Fragility curve refers to the probability of reaching or exceeding a given damage state defined as a function of earthquake loading. Alternative forms of construction such as confined masonry or reinforced masonry, considered less vulnerable, have been developed. Empirical, judgmental, analytical and hybrid approaches are used to obtain fragility curve [11].

1.1. Need of the study
The concepts of evaluating seismic safety of URM structures are the primary part of this Project. The research on seismic behaviour of URM structures and developing fragility curves for URM buildings, subject to seismic loading are included in this study. The fragility curves can be directly used to perform a thorough risk analysis for historical structures.
1.2. Objective
The objective of this investigation is:

- To analyse current strength characteristics of URM structure.
- To describe different damage or limit state by varying seismic parameter.
- To introduce a procedure for developing fragility curves for typical URM structure in Chennai and to present the generated results.

1.3 Scope
- To determine strength characteristics, soil conditions and influence of age on seismic response by nonlinear static pushover analysis.
- To evaluate seismic performance of the structure by displacement coefficient method.
- To develop fragility curve by lognormal distribution method for historic masonry structures in Chennai.

2. Validation
Research papers by Ruiz Garcia, et.al., (2010) [17] and S.L.Dimova and K.Hirata (2000) [18] were validated.
SAP software was used for the Equivalent-frame modelling and SAP2000 for Non-linear static pushover analysis. The comparison with the results of the journal was done by generating fragility curves and adopting HAZUS [20] method. The same method applied on many other journals showed similar results up to 98.9%, hence this method will be effective.

3. Derivation process
The derivation process of analytical fragility curves for a particular building or building typology comprises several components [2]. They are as follows:

- Building Inventory,
- Damage/limit state model,
- Building capacity model,
- Seismic hazard analysis,
- Seismic demand model, and
- Fragility generation.

The methodology worked out to develop analytical fragility curves is shown in figure 1.

![Figure 1. Derivation process – seismic fragility curves](image-url)
4. Building inventory
Residential building in Old Washermenpet has been chosen and was constructed during the year 1920 and has successfully served its purpose for almost 93 long years. The building is a masonry load bearing type structure. Brick masonry was plastered with clay and egg albumen mortar. Renovation work was carried out during the year 1962 and 2000. The building is double storied, 8.8 m long and 9.2 m wide building. The foundation is stepped spread footing type, constructed along the length of walls at ground floor resting on Type –II soil as per IS 1893:2012. The ground floor and first floor are nearly 2.74 m tall. The structure chosen for analysis & it plan details is represented in figure 2.

5. Seismic hazard analysis
In this investigation we focus on the seismic hazard analysis of Chennai, where the structure is located with respect to the seismic hazard map of India. It could be inferred that there is a chance of occurrence of moderate earthquake to occur in Chennai. Also, a study on the damages occurred to the structure due to past earthquakes is done [1, 3, 4, 10]. The damages occurred to the structure chosen due to past earthquakes is represented in table 1. The hairline cracks formed on the exterior walls of

![Building under study](image-url)

Figure 2. Building under study
the building chosen for study due to the earthquake occurred during December 2004 is shown in figure 3.

Table 1. Earthquakes that affected Chennai in past 20 years (1994 – 2014).

| Month & year of occurrence | Epicentre | Magnitude | Damage occurred to the building chosen |
|----------------------------|-----------|-----------|--------------------------------------|
| September 2001             | Chennai   | <3        | Slight tremor was observed by persons living in the building |
| December 2004              | 250km SSE of Banda Aceh, Sumatra | 9.1       | Formation of hairline cracks on exterior wall |
| April 2012                 | Indian Ocean | 8.6 and 8.2 | Propagation of hairline crack formed during 2004 earthquake |

Figure 3. Hairline cracks formed on exterior wall (arrow mark)

6. Modelling of the structure

The structure is modelled as an Equivalent Frame Model (EFM) to estimate the building capacity and seismic demand of the structure. The building capacity is determined from a plot of base shear versus roof displacement, whereas seismic demand is determined from a plot of spectral acceleration versus spectral displacement; both the plots are obtained from Non-linear static pushover analysis (NPOA) [5,6]. The mechanical properties of masonry units used during early 20th century were considered while modelling the structure. The properties are tabulated in table 2.

Table 2. Properties of masonry units considered for analysis

| Properties considered | Values used |
|-----------------------|-------------|
| Weight per unit volume| 18.4 kN/m³  |
| Mass per unit volume  | 1.8763 kN/m³ |
| Modulus of Elasticity (E) | 1.6 x 10⁶ kN/m² |
| Poisson’s Ratio (µ)   | 0.23        |
| Shear Modulus (G)     | 650406.5 kN/m² |
| Co-efficient of thermal expansion | 5.5 x 10⁻⁶ |
| Material geometry     | Anisotropic for Earthquake loading condition. |
|                       | Orthotropic for Normal loading condition. |

EFM of masonry walls has been used successfully in many previous studies for the assessment of seismic behaviour of masonry buildings in Bhuj, India. In this method, multi-storey masonry walls are modelled as equivalent frames made of vertical (pier) and horizontal (spandrels) elements with rigid intersecting joint elements. The equivalent frame model adopted in this study is shown in figure 4(a)
for a typical masonry wall. The effective height of the pier is calculated according to the criteria of Dolce [1989] [7] and can be expressed as:

\[ H_{eff} = h' + \left\{ \frac{1}{3h'} \times D \times (H - h') \right\} \]  

(1)

where, \( h' \), \( D \), \( H \) for the given pier can be obtained as shown in figure 4 (b). The nonlinear behaviour of piers and spandrels is modelled by inserting plastic hinges at pre-defined locations in the frame elements, to overcome the limitation of in-ability of EFM to automatically simulate the effect of varying axial stresses on piers. The effective height of piers on left (L) and right (R) side of the opening in facade wall found as per formula 1 is tabulated in table 3.

**Table 3.** Effective height of piers as per Dolce expression

| Notation of Opening | Height of Opening (m) | Height of Wall (m) | Pier Dimensions (m) | Effective Height of Pier, \( H_{eff} \) (m) |
|---------------------|----------------------|-------------------|---------------------|---------------------------------|
|                     | \( h' \)             | \( D \)           | L | R | L | R | L | R |
| D1                  | 1.83                 | 2.743             | 2.41         | 2.23 | 2.01 | 1.39 | 2.50 | 2.34 |
| W1                  | 1.07                 | 2.743             | 1.47         | 1.45 | 1.39 | 1.32 | 1.87 | 1.84 |
| W2                  | 1.07                 | 2.743             | 1.45         | 1.47 | 1.32 | 1.39 | 1.84 | 1.87 |
| W3                  | 1.07                 | 2.743             | 1.47         | 1.65 | 1.39 | 2.01 | 1.87 | 2.09 |
| W4                  | 1.07                 | 2.743             | 1.47         | 1.47 | 1.39 | 1.39 | 1.87 | 1.87 |
| W                   | 0.31                 | 2.743             | 0.57         | 0.69 | 0.91 | 0.91 | 1.72 | 1.99 |
| V                   | 1.22                 | 2.743             | 1.60         | 1.48 | 1.32 | 0.91 | 1.91 | 1.74 |

**Figure 4.** (a) Equivalent frame modelling of a typical multi-story masonry wall with door and window openings, and (b) Definition of effective height of piers as per Dolce [1989]

6.1. Nonlinear static pushover analysis

In this investigation we focus on the nonlinear static pushover analysis (NPOA) procedure in the FEMA – 273 and ATC 40. The floor displacements, story drifts, joint rotations, plastic hinge rotations, etc., computed at the target displacement represent the earthquake induced demands on the structure [8, 12].

The bilinear capacity curve, a plot of spectral acceleration versus spectral displacement is determined by NPOA of structure subject to spectral acceleration gradually increasing until a target value of spectral displacement is reached as per ATC 40 procedure. The capacity curve and bilinear capacity curve of the building analyzed are obtained and represented in figure 6 and 7. The EFM for the building analyzed and the deformed shape after performing NPOA is represented in figure 5.

The steps involved in carrying out NPOA on the EFM of structure studied using SAP2000 is given below:

- Set default unit as ‘kN, m, C’
- Create grid lines.
- Define material specification – i.e. material properties.
- Define dimensions of lintel and frame
- Define supports
- Assign lintel and piers according to equivalent frame model procedure.
- Design Lintels and piers as RCC frame.
- Define DL & LL
- Define Response Spectrum as per IS 1893:2012 (this codal provision is included in SAP2000).
  - Select Zone-III and Soil type as Type-II (medium soil).
  - I = 1.0
  - R = 1.5 (for URM).
  - Sa/g = 2.50 for (T = 0.17s).
  - Z = 0.16.
  - Damping ratio = 5%
- Define Load case \( \Rightarrow \) DL + LL + EL
- Define nodes
- Define hinges – assign hinge using auto hinge property
- Assign loads.
- Define pushover load case with unit displacement along XX direction at node no 48 (node present at top extreme end)
- Run Analysis
- Display result

**Figure 5. Building modelling**

Pushover analysis of EFM of the structure studied resulted in generation of capacity curve for the structure as per FEMA – 273, shown in figure 6. The structure experienced a yield base shear of 2650.389kN and yield displacement of 45mm (0.045m), after which the structure collapses. The results of building capacity model are presented in table 4.

**Table 4. Analysis result of building capacity model**

| Base Shear (kN) | Roof Displacement (m) |
|-----------------|-----------------------|
| 0               | 0                     |
| 2335.1542       | 8.010 \times 10^{-4}  |
| 2434.5762       | 1.602 \times 10^{-3}  |
| 2645.3220       | 0.0320                |
| 2650.3899       | 0.045                 |
NPOA of EFM of the structure studied resulted in generation of capacity curve for the structure as per ATC 40, bilinear capacity curve shown in figure 7. The structure experiences a maximum spectral acceleration of 1.75 (1/g) at yield spectral displacement of 45mm (0.045m. The results of seismic demand model are presented in table 5.

Table 5. Analysis result of seismic demand model

| Spectral Displacement $S_d$ (m) | Spectral Acceleration $S_a$ (1/g) |
|---------------------------------|----------------------------------|
| 0                               | 0                                |
| $8.010 \times 10^{-4}$          | 1.518                            |
| $1.602 \times 10^{-3}$          | 1.586                            |
| 0.0320                          | 1.700                            |
| **0.0450**                      | **1.750**                        |

Figure 6. Capacity curve

Figure 7. Bi-linear capacity curve
6.2. Damage state model
Comparing the building capacity curve, bilinear capacity curve and standard response spectrum data; the damage states defined for the structure chosen are as follows:

- Slight,
- Extreme,
- Moderate, and
- Collapse.

As per HAZUS [20], these damage states are characterized in structure chosen by the occurrence of damages. The deformed shapes of EFM corresponding to the damage states considered are represented in figure 8 to figure 10.

Figure 8. EFM and Deformed shape for slight damage state

Figure 9. EFM and Deformed shape for moderate damage state

Figure 10. EFM and Deformed shape for extreme damage and collapse state
Based on the damages observed in the EFM analysed using NPOA method, the formation of damages are represented considering both building capacity and seismic demand of the structure. The damage state data of the structure studied is given in table 6, and the damage state model is represented in figure 11.

Table 6. Analysis result of damage state model

| Base Shear (kN) | Roof Displacement (m) | Spectral Acceleration $S_a (1/g)$ | Damage State Observed |
|----------------|-----------------------|----------------------------------|-----------------------|
| 0              | 0                     | 0                                | -                     |
| 2335.1542      | 8.010 x 10^{-4}       | 1.518                            | Slight               |
| 2434.5762      | 1.602 x 10^{-3}       | 1.586                            | Moderate             |
| 2645.3220      | 0.0320                | 1.700                            | Extreme              |
| 2650.3899      | 0.0450                | 1.750                            | Collapse             |

Figure 11. Damage state model

7. Fragility curve generation

As discussed earlier, fragility curve is defined as the probability of exceeding a given damage state defined as a function of earthquake loading [9, 13]. In this study, the function of earthquake loading considered is PGD. The Cumulative Distribution Function (CDF) defining probability of exceeding a specified damage state obtained as per HAZUS and Zentner, et. al., [14] is as follows:

$$
P[ds|S_d] = \Phi \left[ \frac{1}{\beta_{ds}} \ln \left( \frac{S_d}{S_{d,ds}} \right) \right] = \int_0^{S_d} \frac{1}{x\beta_{ds}(2\pi)} e^{-\frac{1}{2}\left( \frac{\log(S_d/S_{d,ds})}{\beta_{ds}} \right)^2} dx$$  

where, $S_{d,ds}$ is the median value of spectral displacement; $\beta_{ds}$ is the standard deviation of natural logarithm of spectral displacement for damage state and $\Phi$ standard normal cumulative distribution function [15]. Since the demand spectrum for equivalent damping is dependent on building capacity, a convolution process is required to find the total variability $\beta_{ds}$ as shown below:

$$
\beta_{ds} = \sqrt{\text{CONV} \left[ \beta_c, \beta_D \right]^2 + \left( \beta_{td,ds} \right)^2}$$  

where, $\text{CONV} [\beta_c, \beta_D]$ is a complex numerical process and difficult to perform as the demand and capacity are correlated.

HAZUS classifies URM structures as Un–Reinforced Masonry Medium rise buildings (URMM) and Un–Reinforced Masonry Low rise buildings (URML), depending on the height of the structure. The structural classification based on height of the structure as mentioned in HAZUS ® – MH 2.1 technical manual [16, 18] is given in table 7. Since the height of structure chosen for analysis is 7.925 m, it is considered as URMM.
To avoid this difficulty, sets of pre-calculated valued of damage-state beta values have been compiled and provided in tabular form in HAZUS. The pre-calculated $\beta_{ds}$ values for URMM structure of height 15.855m, as given in HAZUS are reproduced in table 8.

Table 8. Median and $\beta_{ds}$ for URMM of height 15.855m as per HAZUS

| Damage States | Median ($S_{d,ds}$) (in inches) | Median ($S_{d,ds}$) (in mm) | Beta ($\beta_{ds}$) |
|---------------|---------------------------------|----------------------------|--------------------|
| Slight        | 0.50                            | 12.7                       | 0.99               |
| Medium        | 1.01                            | 25.7                       | 0.97               |
| Extreme       | 2.52                            | 64.08                      | 0.90               |
| Collapse      | 5.88                            | 149.352                    | 0.88               |

The median and $\beta_{ds}$ values for the structure chosen for analysis are calculated from the standard values given table 9 and considering the damage state model represented in figure 11, and these values for structure chosen is represented in table 9.

Table 9. Median and $\beta_{ds}$ for URM structure analysed

| Damage States | Median ($S_{d,ds}$) (in mm) | Beta ($\beta_{ds}$) |
|---------------|----------------------------|--------------------|
| Slight        | 0.801                      | 0.062              |
| Medium        | 1.602                      | 0.060              |
| Extreme       | 32                         | 0.45               |
| Collapse      | 44.9                       | 0.265              |

Considering the building capacity, seismic demand and damage states; the probability of exceeding a particular damage state is obtained using the formula 2.

8. Results and discussion

Fragility analysis is performed on the structure modelled as per HAZUS ® - MH 2.1 for pre-code URMM structure, since the structure studied was constructed during early 20th century, exceeding a specified damage state is obtained using the formula discussed earlier. The fragility curves corresponding to four damage states i.e. slight, moderate, extreme and collapse damage states are given in figure 12 respectively. The combined fragility curve is represented in figure 13.
Figure 12. Fragility curves (c) extreme & (d) collapse damage state

The fragility curves were derived using simplified log – normal distribution based procedure as per HAZUS considering four damage states. From figure 13, it is clear that slight and moderate damages occur at a very short interval after the occurrence of seismic action. The probability of occurrence of extreme damage is 80%, whereas the probability of structure to collapse is 60%.

Figure 13. Fragility curves – combining all four damage states

9. Conclusion
A methodology for derivation of analytical fragility curves for URM buildings constructed during early 20th century, based on EFM and NPOA of the entire structure was discussed in detail. URM building investigated in this paper is a representative for structures constructed during early 20th century present in seismic zone – III (Chennai).

Probability of exceeding a specified damage state for slight, moderate, extreme and collapse damage states are 100%, 100%, 80% and 60% respectively. Formation of hairline cracks on walls, large cracks around door and window openings, movement of lintels, cracks at base of parapets were observed at earlier stage of seismic excitation. With increased seismic intensity, the structure fails/collapses due to out – of – plane failure of the wall [19]. The structure’s damage probability is high at slight and moderate seismic intensity. The structure needs to be retrofitted for slight and moderate damages that are expected to occur.

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