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Fragility Function Generation for Masonry Residential Building for a Pilot Site in Hetauda

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Abstract
Fragility function is an essential component for seismic risk assessment and its determination plays a vital role in predicting the earthquake damage and preparation for post-earthquake scenario. Although many researches have been done in the field of fragility function in Nepal post 2015 Gorkha earthquake, it has been concentrated in Kathmandu valley. This paper is focused on developing fragility function for building typologies for masonry residential buildings of a site in Hetauda which can be considered as a basis for further work across the country. Non-linear static pushover analysis has been done for evaluating the capacity of selected structure. It is observed that for unreinforced masonry structure, the probability of exceedance of life safety limit state increases considerably as the number of story increases. It is quite obvious result considering the fact that unreinforced masonry structures are prone to earthquake hazard.

Keywords
Non-linear pushover analysis, Masonry Residential Building, Capacity Spectrum Method, Fragility Curve

1. Introduction

Nepal being seismically active region with active fault movement due to subduction of Indian plate below Eurasian plate at 2cm per year results in cyclic release of accumulated stress [1, 2, 3]. In the past century, Nepal have been struck with devastating earthquake one of which is the 2015 Gorkha Earthquake (Mw 7.8) which resulted in huge casualties and economic loss. Predicting the earthquake damage and preparation for post-earthquake damage scenario fragility functions is a vital component for seismic risk assessment. Fragility function gives the probability of exceedance of a particular limit state of a structure with respect to intensity measure (IMs). [4] derived empirical fragility function for residential buildings of Nepal considering the damage in the past earthquakes. [5] provided fragility functions for low rise reinforced concrete frame structures with brick masonry as infill in high seismic zone with alluvial deposits. The 2015 Gorkha earthquake resulted more than 8790 fatalities and 22300 injuries along with 1,047, 261 housing damage [6]. The major issue with building construction in Nepal is the lack of implementation of codal provisions and minimum guidelines provided for safer construction. Therefore, the need for fragility assessment is more significant. Hetauda is selected for the investigation of building typologies considering the fact that it is easily accessible from Kathmandu valley and being the capital of Bagmati province with semi-urban settlement

2. Selection of building typology

For the selection of dominant building typology, visual assessment with the help of site visit and field measurement was conducted. A total of 560 household's data was observed and a number building measurement was done for the dominant building typology. Apart from visual inspection, verbal conversion with the engineer at Hetauda municipality was done for obtaining the data regarding the present construction practice and the type and extent of structural design practice. As per observation and conversations made it was found that majority of the building constructions were non-engineered structures. Post 2015 Gorkha earthquake, the trend of construction has shifted from non-engineered construction to pre-engineered and engineered housing construction. The building typology for the surveyed area and story-wise distribution of masonry structure is presented in Figure 1.

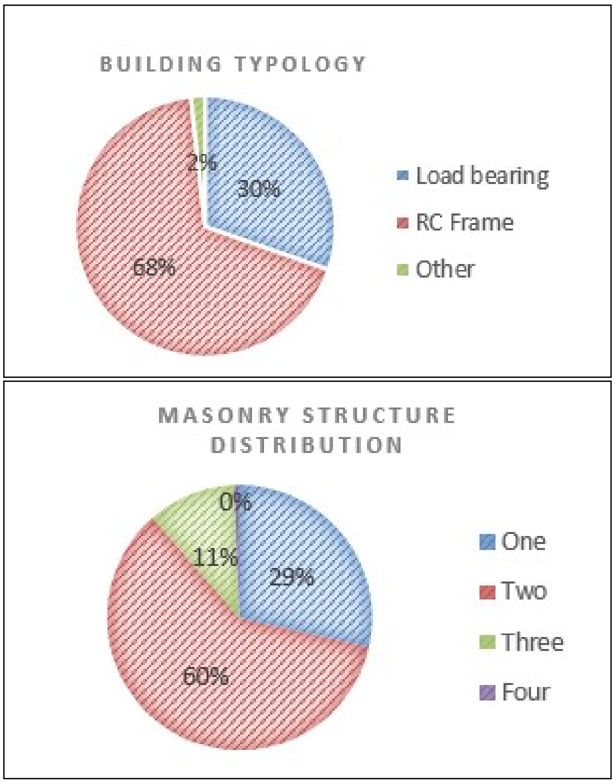

Figure 1: Building typology for surveyed area and distribution of masonry structure based on number of stories

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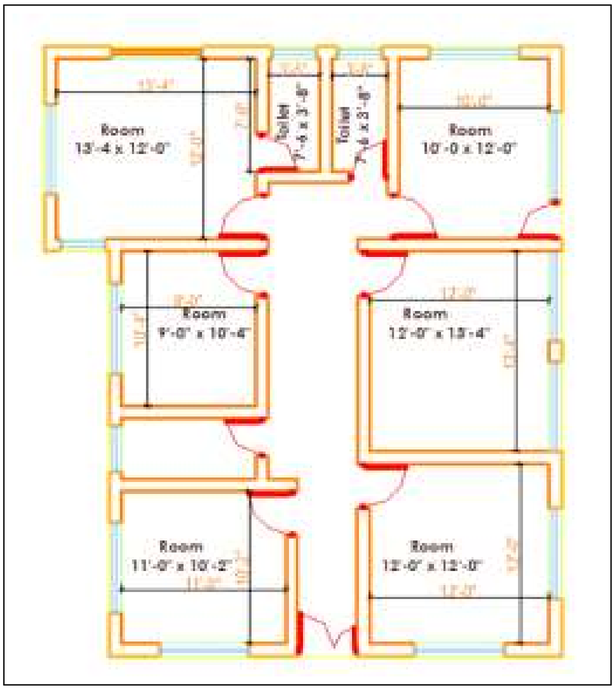

Figure 2: Plan of unreinforced masonry structure

The material properties used for structural analysis are listed in table 1.

For the analysis of masonry structure, a dominant masonry structure typology is selected. The structural plan for the selected unreinforced masonry structure is shown in Figure 2.

3. Structural Analysis

Structural analysis was performed in DIANA FEA [7] interface, a FEM based tool. Different type of material model is available in DIANA. The thickness of wall was found to be 350mm for the ground floor and for upper stories it was 230mm.Rigid floor diaphragm has been considered for the analysis purpose. Here, for this analysis purpose, total strain crack model is used. It follows a smeared cracking approach, which considers cracking as a distributed effect with directionality and cracked material is simulated as a continuous medium with anisotropic characteristics [7].Non-linear static pushover analysis has been done to obtain the capacity of the structure through pushover curve. The structure has been pushed upto 1% of the total story height in the direction of dominant first mode of vibration. The strucural model used in DIANA is shown in Figure 3

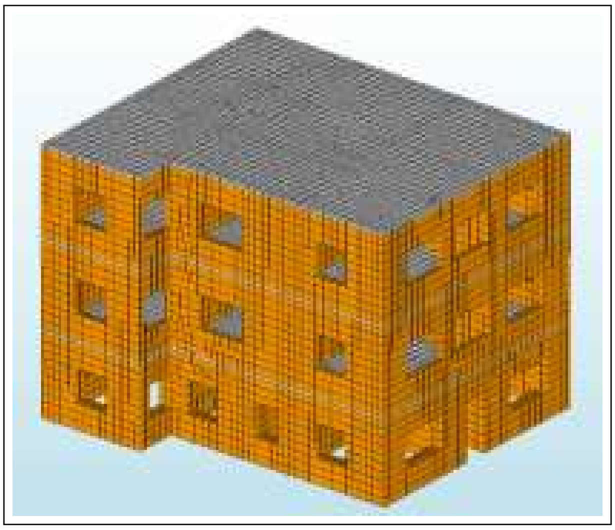

Figure 3: Structural model for 3 storied Unreinforced masonry

The material properties used for structural analysis are listed in Table 1.

Table 1: Material properties used for analysis

Sn.	Description	Value	Remarks
1	Brick Strength	5 MPa	Minimum Strength adopted
2	Mortar	1:6 Cement: Sand	General Construction practice
3	Strength of Mortar	3 MPa	IS1905 clause 3.2.1
3	Basic comp. stress of masonry	0.44 MPa	As per IS1905:1987, Table 8
3	Compressive Strength (f_m)	1.76 MPa	IS1905 Annex B-2.1
4	Elasticity of wall (E=550*f_m)	968.00 MPa	IS1893:2016 Part -1, Clause 7.9.2.1 [Adopted for analysis]
5	f_m= 0.433*$f_b^{0.64}$*$f_{mo}^{0.36}$	1.80 MPa	IS1893:2016 Part -1, Clause 7.9.2.1
6	Elasticity of wall (E=550*f_m)	990.73 MPa	IS1893:2016 Part -1, Clause 7.9.2.1
7	Poisson's ratio	0.15	

Capacity spectrum method was used for obtaining the performance point for different intensity measures (IMs). Methodology proposed by /citeOtani2002 have been utilized for the capacity spectrum method. The pushover curve is converted to capacity curve Sa vs Sd represented by single degree of freedom system (SDOF) system.

$$S_A(T) = \frac{V_B}{M} \tag{1}$$

Where, V_B is base shear and M is the effective modal mass

$$M = \frac{L_n^2}{M_n} = \frac{(\sum m_i(\delta_{i1}))^2}{\sum m_i(\delta_{i1})^2} \tag{2}$$

and,

$$S_D(T) = \frac{D_R}{\beta} = \frac{(\delta_{i1})^T[m](\delta_{i1})}{((\delta_{i1})^T[m](1)} \tag{3}$$

Where, displacement vector of first mode is given by:$(\delta)_1 = (\phi)_1\beta_1 S_D(T)$ [m] is the lumped floor mass matrix.

Similarly, the input response spectrum curves in SA – T, resulting from input parameters of certain earthquake intensity, is converted to Acceleration-Displacement Response Spectrum (ADRS) i.e., SA – SD plot by the relation:

$$S_D(T) = (\frac{T}{2\pi})^2 S_D(T) \tag{4}$$

Response spectra for Hard soil as provided by National Nepal Building code NBC105:2020 [8] is selected for the study.A single performance point is obtained for one PGA level of the particular earthquake spectrum.A similar procedure is followed to obtain performance points for different PGA levels of different sources of earthquake response spectra.

3.1 Fragility analysis

The capacity spectrum method provides a set of performance point for each intensity measures we desire to obtain. These are further used for the development of fragility curves which is a plot of probability of exceedance of a particular damage state as a function of particular IMs. Methodology provided by [9] has been adopted for generation of fragility curves. Probability that the structure exceeds the limit-damage state for given Ground Motion Intensity (GMI) is given by the formula:

$$P(LS_i/GMI) = 1 - \phi(\frac{\lambda_{cl}^i - \lambda_{D/GMI}}{\beta_{D/GMI}}) \tag{5}$$

Where the mean and standard deviation parameters are defined by:

$$\lambda_{D/GMI} = \ln(a_1) + a_2\ln(GMI) \tag{6}$$

$$\beta_{D/GMI} = \sqrt{\frac{\sum_{K=1}^{n}[\ln(GMI_k) - \lambda_{GMI}(GMI_k)]^2}{n-2}} \tag{7}$$

Linear regression analysis is used to obtain the constants a1 and a2 for which a logarithmic plot of storey drift and ground motion intensity (GMI) parameters in terms of PGA.

The target performance for different damage state of the structure [10] from the structural capacity viewpoint for unreinforced load bearing structure are: Immediate Occupancy (IO) = 1/750 (i.e., 0.13%) Life safety (LS) = 1/500 (i.e., 0.2%) Collapse Prevention (CP) = 1/250 (i.e., 0.4%)

4. Results and Discussion

Capacity curves obtained from the non-linear static pushover analysis is shown in Figure 4.

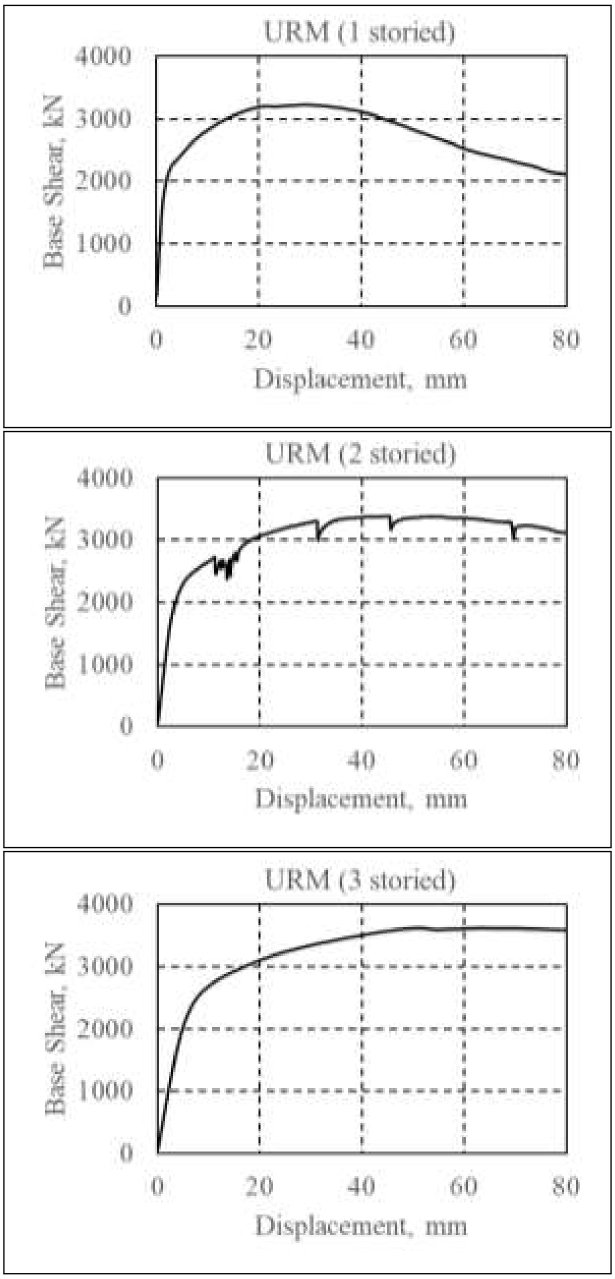

Figure 4: Capacity Curve for Unreinforced masonry for one-, two- and three-storied structure

It is observed that the for a horizontal displacement of 80mm, the equivalent horizontal acceleration for one, two and three storied unreinforced masonry is about 1.38, 0.91, 0.71. The corresponding maximum base shear is 3221kN, 3390kN and 3319kN respectively. Capacity spectrum method have been used for the generation of capacity at different intensity measures. Fragility curves are than plotted using MATLAB program. The fragility curves plotted are shown in Figure 5.

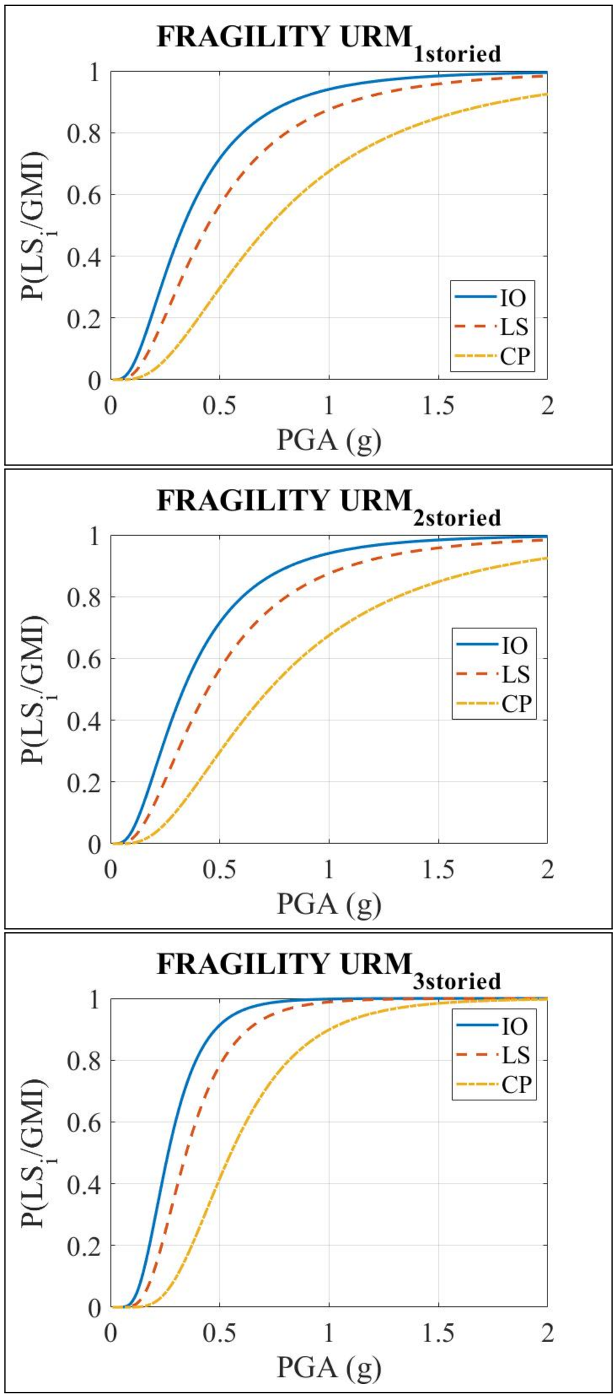

Figure 5: Fragility curve for unreinforced masonry for one, two and three storied structure

It is observed that for a particular PGA of 0.4g, which correspond to seismic zone factor of 0.4g for Hetauda municipality for 475 years return period with 10% probability as per NBC105:2020, the probability of exceedance of limit states for Immediate occupancy, life safety and collapse prevention is 60%, 43.9% and 19.7% for one storied structure. Whereas the same values for three storied structure are 81.22%, 62.00% and 24.6%. It is evident from the fragility curves that, the fragility of unreinforced masonry structure significantly increases with increase in number of stories for all the damage limit states

5. Conclusion

Dominant building typology for different structure was selected from the site visit. On-site measurement was taken for the selected unreinforced masonry building.The unreinforced masonry building of buddha chowk Hetauda has been analyzed using non-linear pushover analysis in FEM based tool DIANA FEA. The selected unreinforced masonry building is analyzed for up-to three story. Following conclusion are made from the fragility curved generated:

Probability of exceedance of life safety limit state for one story unreinforced masonry structure is comparatively lower as compared to three storied structure for 0.4g PGA the probability of exceedance of life safety limit state for single story is 43.9% which is about 18% less as comapred to three storied unreinforced masonry.

More than half of the total unreinforced masonry structures higher than two story in the surveyed area are vulnerable to damage for earthquake of intensity 0.4g PGA for 475 years return period with 10% probability as per NBC105:2020.

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