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Analytical fragility curves from capacity spectrum: A case study for reinforced concrete frame building with unreinforced masonry infill walls

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Abstract

This paper represents generation of analytical fragility curves of a six story Reinforced Concrete (RC) frame structure with Unreinforced Masonry (URM) infill walls from capacity spectrum. Over the years RC building with URM infill walls became most preferable and suitable construction practice inside urban city areas in Bangladesh. Proper modeling of URM walls prior design and lack of incorporation in earthquake resistant features became vital issue in case of vulnerability assessment of existing building. In general practice, buildings are designed without considering interaction effects between infill walls and RC frame. Current study aims to incorporate acceptable response of unreinforced masonry infill walls using masonry strut element model. The basic model for infill walls and reinforced concrete frame are chosen based on state-the-art literature review. Structural software package SeismoStruct v7.0 is used to develop structural model and perform structural analyses. Fragility curves were derived for four performance states using results obtained from pushover analyses.

Keywords: Capacity spectrum, fragility curves, reinforced concrete, unreinforced masonry.

1. Introduction

Bangladesh is one of developing countries in the world and rapid urbanization is on the peak of its progression whereas RC building with URM infill walls building became most popular structural construction type over last decade. The purpose of URM infill walls is to protect inside of a structure from outside environment and to allow separate internal spaces. In typical structural design practices, infill walls in a RC frame building are considered as nonstructural elements. Therefore interaction between infill walls and RC frames are ignored which may results inaccurate prediction of the lateral stiffness, strength, and ductility of the structure. In this study a six story RC frame with URM infill walls model building was considered which represents typical mid-rise building for present time inside Bangladesh. Analytical fragility curves were derived from capacity spectrum based method. During structural modeling of URM walls, it was given as prime responsibility to develop acceptable model which account interaction effect between RC frame and infill walls. Macro element strut model was chosen as an effective choice in case of modeling unreinforced brick masonry infill walls as found in previous literature (Mazumder 2015). SeismoStruct software package version 7.0 was used for the purpose of structural modeling and analyses of index building. Fragility curves were obtained for different performance states of pushover analyses results.

2. Structural modeling

2.1 Modeling of URM infill walls

In last decades significant numbers of research works were performed to characterize seismic behavior of masonry infill wall inside a RC panel. Experimental and analytical results of interaction between RC frame and URM infill walls are compared by several researchers (Decanni et al. 2004; Baran and Sevil 2010). Performance of URM infill walls inside RC frame varied during lateral loads applied on the URM infill remains in contact with RC frame under very low lateral loads and act as a composite system. With the increment of lateral loading, this type of structure starts behaving poorly. Most of the cases lateral stiffness increased initially for the URM infill model in compare to bare frame. Over the years equivalent diagonal strut model became preferable choice among scientific community due to its simplification in modeling of URM behaviors (Mazumder et al. 2015). Masonry infill generally contains high stiffness and strength which played an important role during lateral loading on main RC moment resisting frame structure. Material properties of masonry vary largely from place to place depending on local raw material sources and surroundings environment (Mazumder 2015). For simulation of actual performance of unreinforced masonry infill frame, different micro models were proposed in past literatures (Ellul et al. 2012). However, the diagonal strut model (Figure 1) is the most widely accepted by the researchers because of its simplified approach for bulk analysis, and advocated in many documents (CSA 2004, NZSEE 2006). In current study, simplified diagonal strut model was developed.

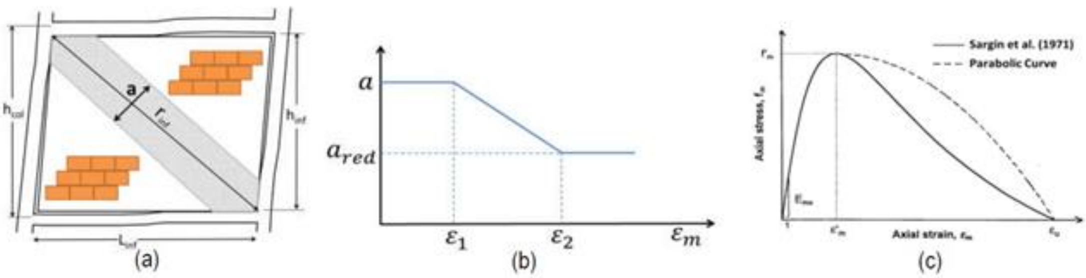

Fig.1. Diagonal strut for masonry infill panel (a) equivalent diagonal strut representation of an infill panel; (b) variation of the equivalent strut width as function of the axial strain; (c) envelope curve in compression (Source: SeismoStruct v7.0, 2015)

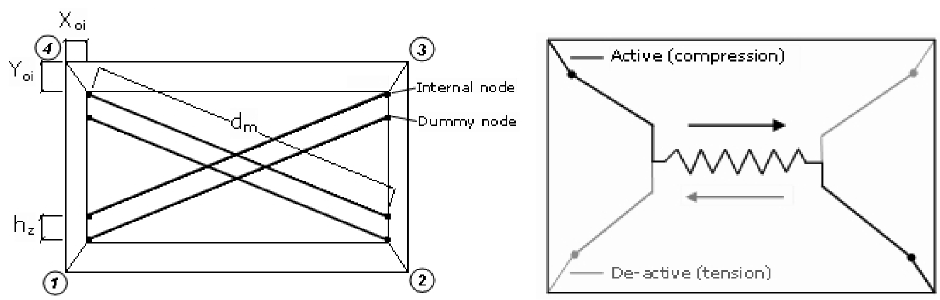

Fig. 2. Infill panel element: compression struts and shear strut (Crisafulli, 1997)

A four-node masonry panel element was used defined by (Crisafulli 1997) and later implemented in SeismoStruct (SeismoSoft v7.0, 2015) by Blandon (2005) for the modeling of the nonlinear response of infill walls in frame structure. Each panel is represented by six strut members where each diagonal direction features two parallel struts to carry axial loads across two opposite diagonal corners and a third one to carry the shear from the top to the bottom of the panel. These later strut only acts across the diagonal that is on compression, its activation depend on the deformation of the panel. The axial load struts use the masonry strut hysteresis model, while the shear strut uses a dedicated bilinear hysteresis rule (Crisafulli 1997, Blandon 2005). In this model, strut area (a) is defined as the product of the panel thickness and the equivalent width of the strut, which normally varies from 10 to 40 percentage of the diagonal length of an infill panel. This value is suggested in several research works performed based on experimental data and analytical results (D' Ayela and Meslem 2013). When the elastic limit of the infill panel is exceeded due to the cracking, the contact length between the frame and the infill decreases as the lateral and consequently the axial displacement increases, affecting thus the area of equivalent strut. To take into account this fact the width of the equivalent strut must be reduced. It is assumed that the area varies linearly as function of the axial strain (as shown in 1b), with the two strains between which this variation takes place being defined as input parameters of the masonry strut hysteresis model. For the equivalent contact length, dimensionless relative stiffness parameter (λ) is computed (Mazumder 2015). Stiffness and strength of an infill panel is calculated from width of equivalent strut using formula proposed by Mainstone and Weeks (Mainstone et al. 1970; Mainstone 1971).

$$a = 0.175(\lambda_I h_{col})^{-0.4} r_{\inf} \tag{1}$$

where, $$\lambda_I = \left[\frac{E_m t_{\inf} \sin 2\theta}{4 E_f I_{col} h_{\inf}}\right]^{\frac{1}{4}} \tag{2}$$

λ is the coefficient used to determine equivalent width of infill strut; h_{col} is column height between centerlines of beam; h_{inf} is height of infill panel; E_c is expected modulus of elasticity of frame material; E_m is expected modulus of elasticity of infill material; I_{col} is moment of inertia of column; r_{inf} is diagonal length of infill panel; t_{inf} is thickness of infill panel and equivalent strut; and θ is angle whose tangent is the infill height to length ratio.

2.2 Modeling of RC frame

Reinforced Concrete material properties were chosen in this study for typical Bangladeshi construction practice context.

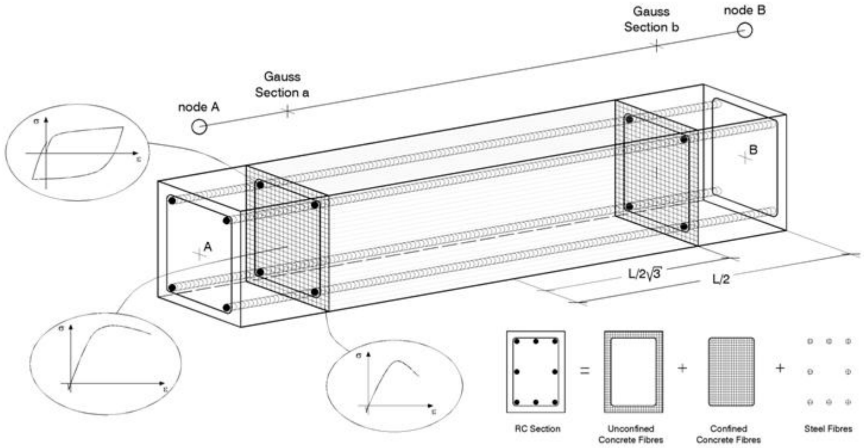

Fig. 3. Reinforced Concrete Section (Source: SeismoSoft v7.0, 2015)

Uniaxial concrete constitute law incorporated following model defined by (Mader et al. 1988) whereas cyclic rules of confined and unconfined concrete was used as (Martinez-Rueda 1997) and (Elnashai 1993) model. Reinforced Concrete members basically consist of three types. These are unconfined concrete (corresponding to the cover), confined concrete (corresponding to the core concrete) and reinforcing steel. These reinforced concrete components are detailed with reinforcement rebar for both main and transverse direction. Transverse reinforcements provide both shear and confinement strength for the concrete core. Fiber section behavior where each fiber is associated with a uniaxial stress-strain relationship in terms of sectional stress-strain state of beam-column elements are subdivided (Mazumder 2015). The discritization of a typical reinforced concrete cross-section shows in the Figure 3.

3. Building description

The selected building prototype is a six storey masonry building located in Chittagong city corporation area. This building was a representative structure for RC frame with URM infill wall type in this region. The building has a trapezoidal identical plan having 10 ft story height in each floor. The plan sketch and dimensions are given in Figure 4b. Material properties are taken as typical values used in Bangladesh (Table 1 and Table 2). Concrete compressive strength and steel rebar strength are taken as variable parameters for structural analyses. Compressive strength values range taken from 2500 psi to 4500 psi with a variable of 500 psi and yield strength of steel range taken from 40ksi to 60ksi with a variable of 5ksi.

Table 1
Material properties

Parameter	Value
Compressive strength of concrete (f_c)	3000 psi
Tensile strength of steel (f_y)	60000 psi
Unit weight of brick masonry	120 lb/ft^3
Compressive strength of infill (f_w)	145 psi

Table 2
Structural Details

Parameter	Value
Shorter Length (L1)	44 ft
Larger Length (L2)	57.67 ft
Width (W)	30 ft
Floor to Floor Height (H)	10 ft
Thickness of infill walls (t_w)	6.3 in
Column 1	12" x 12"
Column 2	15" x 12"
Beam 1	14" x 10"
Beam 2	16.5" x 10"
Beam 3	16.5" x 10"

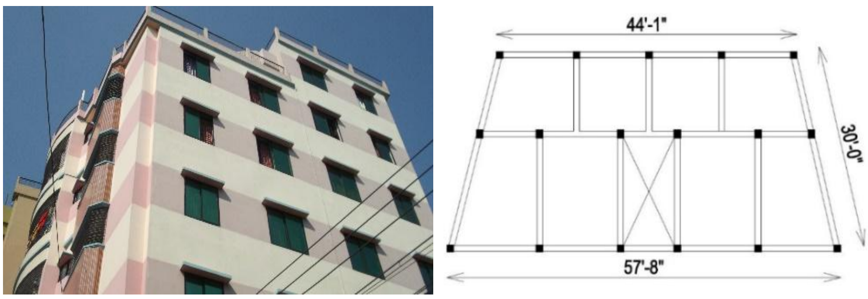

Fig. 4. (a) Prospective view of the index building; (b) typical floor plan.

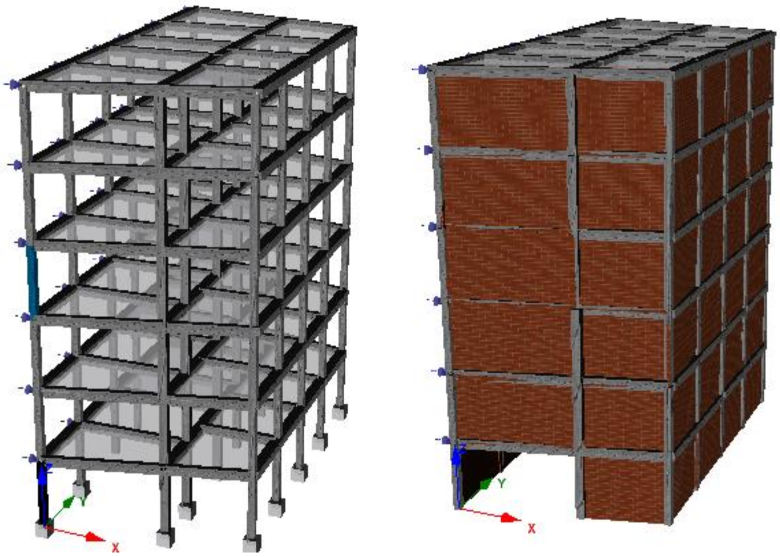

Fig. 5. (a) Bare frame model; (b) Frame with Infill walls model.

4. Capacity spectrum and fragility curves

Expected damage states defined from results obtained from pushover analyses. Inverse triangular displacement pattern (similar to dynamic first mode shape of a structure) was applied during pushover analyses. Simplified method (Risk-UE 2004) was used in this study to obtain damage states thresholds as shown in Table 3. Seismic fragility function of structure defines the probability of physical damage as a function of ground motion intensity parameter.

For a given type of building and damage state, these curves define the probability of equaling or exceeding a considered damage state for a given seismic action. The damage is quantified by the fragility curves which can be obtained from the following equation:

$$P[ds|\mathrm{Sd}] = \Phi\left[\frac{1}{\beta_{ds}}\ln\left(\frac{s_d}{s_{d,ds}}\right)\right] \tag{3}$$

Where Φ is the cumulative lognormal distribution, d is the expected damage, S_d is the spectral displacement and $S_{d,dsi}$ and β_{dsi} are the median values and standard deviations of the corresponding normal distributions. For simplicity we call $S_{d,dsi}$ as μ_i and β_{dsi} as β_i.$\mu_i$is also called damage state threshold, and the probability of exceedance of the damage state $d_{si}$for $S_d= \mu_i$is equal to 0.5. The following simplified assumptions allow obtaining fragility curves from the bilinear form of the capacity spectrum. Two assumptions were used as per Risk-UE project (Risk-UE 2004; Milutinovic and Trendafiloski 2003; Lagomarsino and Giovinazzi 2006) and used to estimate fragility curves in many studies.

i) μ_i are related to the yielding and ultimate capacity points as in Table 3.

ii) the expected seismic damage follows a binomial probability distribution. The first assumption is based on expert opinion and relates the expected damage to the stiffness degradation of the structure; the second one is based on the damage observed in past earthquakes (Grünthal 1998). Once capacity curve of the structure is calculated, it is useful to transform it into capacity spectrum by means of the procedure proposed in the (ATC-40 1996). The capacity spectrum is represented in spectral acceleration-spectral displacement coordinates (s_a-s_d) and is often used in its simplified bilinear form, defined by the yielding point (D_y, A_y) and the ultimate capacity point (D_u, A_u), as it can be seen in Figure 6.

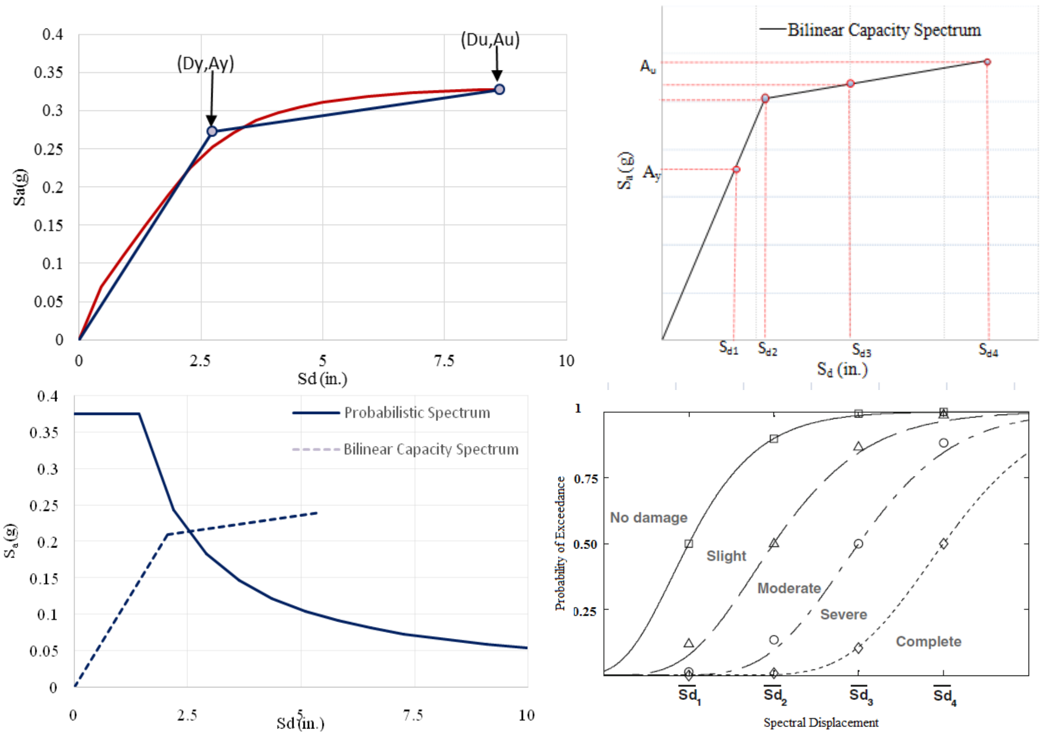

Fig. 6. a) An example of capacity curve and its simplified bilinear form; b) damage state definition; c) capacity spectrum performance and d) fragility curves for damage states.

Table 3
Damage state thresholds according to the capacity spectrum

Damage State	Damage state thresholds
Slight (S_{d1})	$\mu_1 = 0.7D_y$
Moderate (S_{d2})	$\mu_2 = Dy$
Severe (S_{d3})	$\mu_3 = D_y + 0.25\ (D_u - D_y)$
Collapse (S_{d4})	$\mu_4 = D_u$

5. Results and analyses

Results of RC building with URM infill and bare frame models are compared in order to understand URM infill walls contribution in the structural integrity. Equivalent static seismic loads calculated following Bangladesh National Building Code (BNBC 1993) guideline.

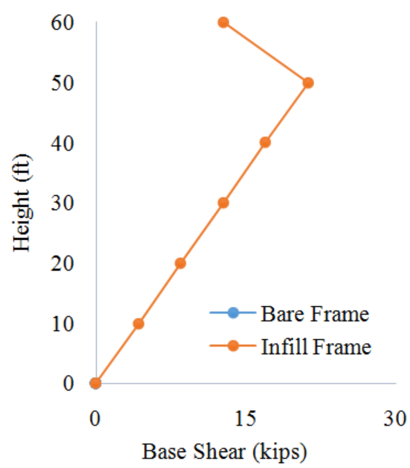

Fig. 7. Base shear distribution

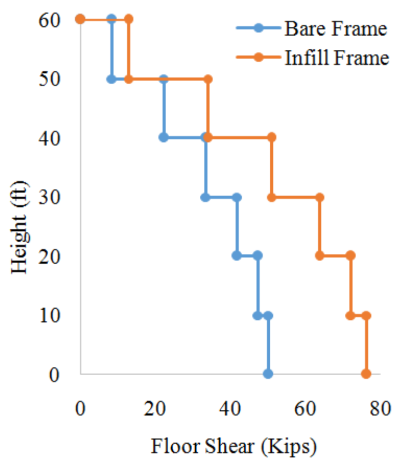

Fig. 8. Story shear distribution

Figure 7 and Figure 8 represent comparison of seismic base shear and floor share distributions for both model, respectively. Base shear for bare frame structure was 50.14 kips whereas base shear of structure incorporated with URM infill walls was 76.49 kips.

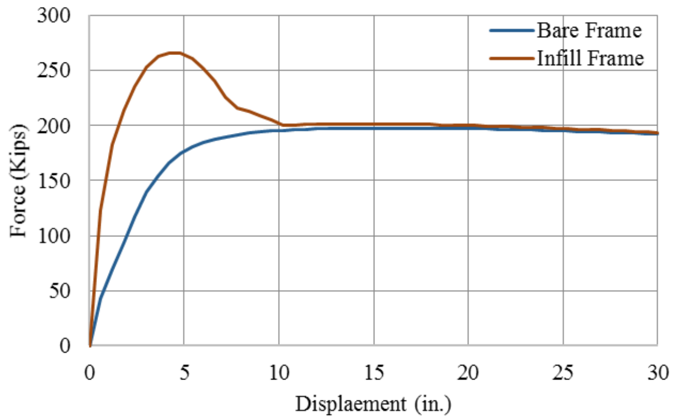

Fig. 9. Comparison in pushover curves

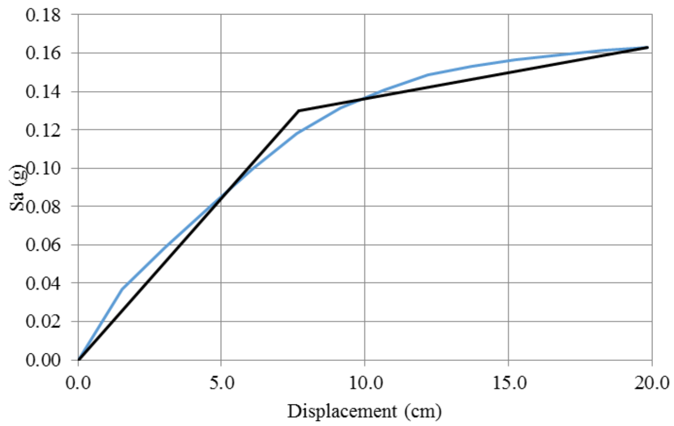

Fig. 10. Capacity curve of bare frame model

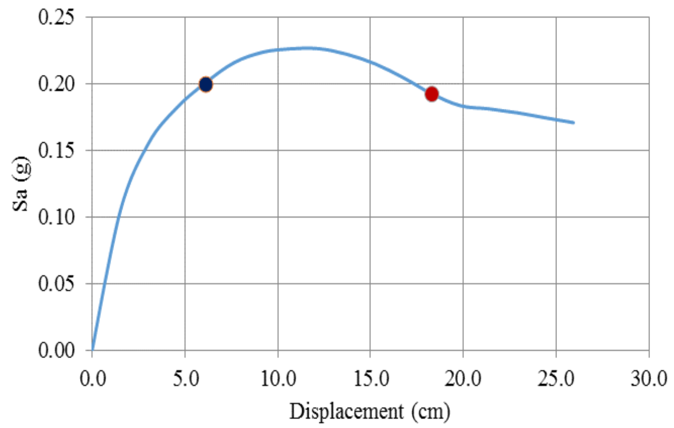

Fig. 11. Capacity curve for Infill model.

Displacement based static non-linear pushover analyses were performed by applying control displacement at the top of the frame. Comparative performance from pushover analysis of the index building (for mean material strength consideration) is shown in Figure 9. It was observed that initial stiffness in the infill frame is significantly higher than bare frame model. It reveals that lateral stiffness was increased due to the contribution of masonry infill walls.

However, after first yield occurred capacity of infill frame dropped and first crush in confined concrete was observed earlier in infill model in compare to bare frame model. Both yielding (moderate damage state) and ultimate capacity (collapse damage state) point were identified considering first yield and first crush in confined concrete member respectively. Then other two performance points are obtained using definition stated in Table 3. Bi-linear capacity curve was derived for the bare frame model in order to obtain yielding point. Figure 10 and Figure 11 show capacity curves obtained from bare frame and frame with infill wall model, respectively. Fragility curves are developed for different damage states using Equation 3. Figure 11 and Figure 12 represent fragility curves that obtained for bare frame and frame with infill wall model respectively.

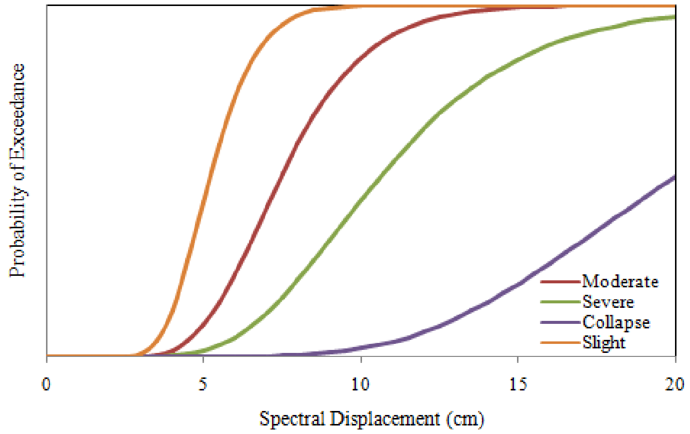

Fig. 12. Fragility Curves of Bare Frame Model

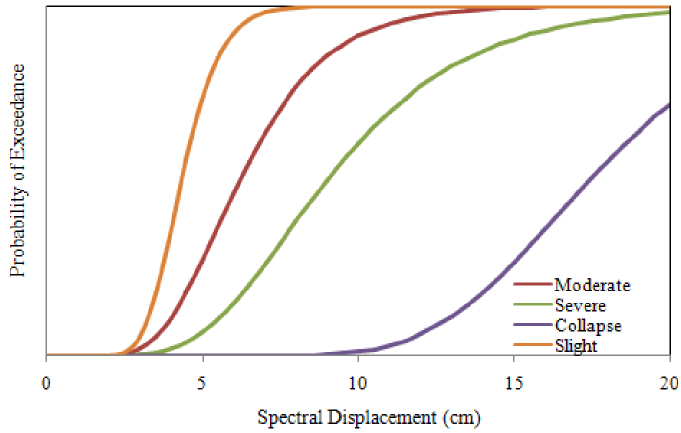

Fig. 13. Fragility Curves of RC Frame with Infill walls model.

6. Conclusion

The index building considered in this study was a story ordinary moment resisting RC frame with URM infill walls in Chittagong. Overall objective of this study was to derive fragility curves based on nonlinear static pushover analysis. The results from the analysis reveal the effect of masonry infill integration. It has been found that infill walls contribute for increasing lateral strength of the building. Initial stiffness of the structures increased significantly whereas ductile behaviour reduces in compare to bare frame model. With the increment of lateral loading, once first crush obtained inside masonry, lateral strength start decreasing. After a certain stage, frame structure with infill model started act as bare frame (Figure 9). Only a limited number of analyses were performed to obtain standard deviation parameter for each limit states.

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