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# Seismic Fragility Analysis of Hollow Concrete Block Infilled Reinforced Concrete Buildings

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***Abstract* - Masonry infills are usually treated as non-structural elements in buildings, and their interaction with the bounding frame is often ignored in analysis and design of reinforced concrete structures. The main aim of this study is to develop a seismic fragility curves showing the probability of exceeding a damage limit state for a given structure type subjected to a seismic excitation. For the purpose of this study, three distinct buildings namely, seven-story, eleven-story and sixteen-story, with typical floor plan were proposed as the case study. Each building cases are explicitly modeled as a bare frame and HCB in-filled model with varying percentage of infill configurations. All building models under the case study were analyzed using Seismo-Struct software to assess seismic vulnerabilities. Non-linear dynamic time history and pushover analysis were employed to generate fragility curves. 30 generated artificial accelerograms were employed in the nonlinear dynamic time history analysis. Accordingly, for developing a fragility curve, nonlinear dynamic analyses of 30 building models for each case are conducted and the maximum roof displacement (ID) for each ground motion is recorded. Results of the study showed that bare frame has a highest probability of failure and building models with a larger percentage of infill configurations have lesser failure probability than slightly infilled building models. Basically these infills have significant contribution in arresting large lateral deflections and results in lower and most tolerable story displacements under excited earthquake motion and eventually reducing the structure's probability of failure at life safety and collapse prevention limit states.**

***Keywords:*** Bare frame, infilled model, fragility curve, limit state capacities.

## I. INTRODUCTION

HCBs are frequently used infill walls among the most commonly used masonry infills in Ethiopia. These infills participate in the lateral response of buildings and as a consequence alter the lateral stiffness of buildings. Hence, natural periods and modes of oscillation of the building are affected in the presence of masonry infills. The conventional design practice considers only the masses of the infill walls without an attempt to incorporate their lateral stiffness. As a result modeling the infill walls along with the frame elements is necessary to incorporate additional lateral stiffness offered by masonry infill walls.

Neglecting the significant interaction between the infill walls and building frames is the main reason why structural systems incorporating integrated infills panels react to strong earthquakes in a manner quite different from the expected one. There are many different techniques proposed in the literature for the simulation of the infilled frames, which can be basically divided in two groups, namely the micro models and the simplified macro-models. The micro-models considers a high level of discretization of the infill masonry panel, in which the panel is divided into numerous elements to take into account the local effects in detail, while the simplified macro-models are supported in simplifications with the objective of representing the global behavior of the infill panel with main structural elements.

Macro-modeling is used to present accurate and realistic response of infill walls and it uses equivalent diagonal struts to model the contribution of the infill walls to the response of the infilled frame. This method replaces the infill panel by two diagonal, compression-only struts. This approach is advantageous since the masonry is a very heterogeneous material and it is hard to predict the material properties of the constituent members accurately. For the nonlinear analysis of large and complex structures under severe loadings, as the induced by earthquakes, in many cases it is not suitable to adopt refined models. Thus, many authors have in the last decades proposed and used simplified nonlinear models for RC structures.

The focus of this research is develop seismic fragility curves and assess the performance of HCB Infilled RC buildings in terms of a series of discrete performance levels identified as Operational, Immediate Occupancy, Life Safety, and Collapse Prevention. Three distinct building model cases (i.e. G+6, G+10 and G+15) each as a bare frame and distinctly having defined percentages of infill configuration are proposed for numerical analysis purpose. Bare RC frame

 

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buildings are analyzed and designed on ETABS 2016.2.1 [1]. Analysis and design of the proposed building model cases followed the conventional design approach as prescribed on the new Ethiopian Buildings Code Standards [2], [3] and [4]. While numerical modeling and nonlinear time history analysis of designed building model cases with the proposed infill configurations are computationally done on Seismo-Struct [5] which is a fiber-based finite element software package capable of predicting the large displacement behavior of space frames.

## II. LITERATURE REVIEW

### 2.1 Fragility Curves and Probabilistic Seismic Demand Models

Seismic performance evaluation of Building structures is undergoing drastic changes from time to time by variety of reasons. However, the current trend of procedure for seismic performance evaluation of buildings structures requires identification of the seismic hazard, analysis of structural fragilities, and calculation of limit state probabilities. The structural fragility curves are said to be the key component while quantifying the seismic risk assessment. Fragility curves are usually defined as the probability of exceeding a specific limit state of building for a given level of ground motion intensity.

Ellingwood [6] highlighted the importance of the probabilistic analysis of building response in understanding the perspective of building behaviour. This paper outlined a relatively simple procedure for evaluating earthquake risk based on seismic fragility curve and seismic hazard curve. This study shows the importance of inherent randomness and modelling uncertainty in forecasting building performance through a building fragility of a steel frame.

Tantala and Deodatis [7] considered a 25 story of reinforced concrete moment resisting frame Building having three-bays. They have generated fragility curves for a wide range of ground motion intensities. Simulation was done by power spectrum probability and duration of earthquake by conducting 1000 simulation for each parameter. The nonlinear analysis is done by considering the P-Δ effects Cand by ignoring soil-structure interaction. They have considered the nonlinearity in material properties in model with nonlinear rotational springs a bilinear moment-curvature relationship by considering the stiffness degradation through hysteretic energy dissipation capacity over successive cycles of the hysteresis. The simulation for the durations of strong ground motions is done at 2, 7 and 12 seconds labels to observe the effects.

Ellingwood [8] developed fragility response for RC framed building structure due to the potential impact of earthquake in low-to moderate seismicity regions of the United States. Three-story and six-story framed buildings designed according to ACI 318 were considered. Opensees 2007 programme was used for modelling and fiber approach nonlinear uniaxial constitutive concrete and steel model were used to develop element section. Synthetic earthquake was generated, 10 ground motions were generated. Nonlinear static pushover analysis was performed for each structure to identify the structural behaviour, maximum inter-story drift was considered as demand variable and 5% damped spectral acceleration at fundamental period was adopted as ground motion intensity measure. The author concluded that gravity designed concrete frames may suffer severe damage or collapse with current design-basis ground motions.

Celik and Ellingwood [9] studied the effects of uncertainties in material, structural properties and modelling parameters for gravity load designed RC frames. It was found that damping, concrete strength, and joint cracking have the greatest impact on the response statistics. However, the uncertainty in ground motion dominated the overall uncertainty in structural response. The study concluded that fragility curves developed using median (or mean) values of structural parameters may be sufficient for earthquake damage and loss estimation in moderate seismic regions.

### 2.2 Models for the Infill Panels

In modelling of infill panels the problem relies on identifying a reliable and simple model which could represent the masonry infill. Many difficulties were due to the intrinsic characteristics of masonry. As it is a non-homogeneous and anisotropic material, it is difficult to find a generally valid constitutive law. Furthermore the masonry shows significant degradation of stiffness and strength under cyclic loading. The result showed that the ratio of the estimated equivalent strut width to the diagonal length of infill (w/dinf) are ranging between about 0.1 to 0.33 except the result calculated by using Stafford Smith and Carter [10] method equation which generate large value for the equivalent strut width.

**Table 1: Strut width and coefficient by various researchers [11]**

| S.No. | Researchers | Strut Width (m) | Coefficient ($w/d_{inf}$) |
|---|---|---|---|
| 1 | Holmes [1961] | 0.93 | 0.333 |
| 2 | Stafford Smith and Carter [1969] | 2.61 | 0.935 |
| 3 | Mainstone [1971] | 0.29 | 0.103 |
| 4 | Mainstone and Weeks [1974] and Mainstone [1974] | 0.27 | 0.097 |
| 5 | Liauw and Kwan [1984] | 0.56 | 0.201 |
| 6 | Paulay and Preistley [1992] | 0.7 | 0.250 |
| 7 | Durrani and Luo [1994] | 0.49 | 0.176 |
| 8 | Hendry [1998] | 0.68 | 0.244 |
| 9 | Al-Chaar [2002] | 0.27 | 0.097 |
| 10 | Papia *et al.* [2008] | 0.44 | 0.158 |

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## III. METHODOLOGY

### 3.1 Introduction

Based on the reviewed documents and well stated standards, methodologies for the research work were explicitly developed. Since performance evaluation of buildings mainly involve nonlinear analysis along with considering geometric nonlinearity, material nonlinearity and material inelasticity; basic nonlinear analysis which considers the above mentioned inputs were adopted. For this study, three building model cases having typical floor plans and functions for apartments (G+6, G+10, and G+15) with varying percentage of infill configuration were studied. All building models were studied as both bare frame and infilled (25%, 50%, 75% and 100%) building model cases.

### 3.2 Seismic Design of RC Buildings

The proposed building models were analyzed and designed based on the conventional method on ETABS 2016.2.1 software. Design of these bare frame building models followed the basic steps and approaches that are operational and practical in real world construction industry. Proposed building cases were to be situated in Addis Ababa whose seismic zone according to the new code is III, and lateral load analysis followed response spectrum approach during preliminary design of buildings. All the design outputs for structural members obtained from these step were detailed in the way that numerical modelling on fiber-based software was easily done. Five model cases namely Bare Frame Building Model, 25% Infilled Model, 50% Infilled Model, 75% Infilled Model, and 100% Infilled Model were simulated for each building types (G+6, G+10 and G+15). These building models were numerically modelled on finite element software called SeismoStruct which is a fiber-based finite element packages capable of predicting the large displacement behaviour of space frames under static or dynamic loading, considering both geometric nonlinearities and material inelasticity. Masonry infills were modelled using a double-strut cyclic nonlinear approach which was implemented in fiber-based finite element program (SeismoStruct). Performance evaluation were done both on local and global response states with roof displacements against well-defined seismic motions and fragility curves were finally developed.

### 3.3 Macro-Modelling of Infill Walls

Macro-modeling is used to present accurate and realistic response of infill walls and it uses equivalent diagonal struts to model the contribution of the infill walls to the response of the infilled frame. This method replaces the infill panel by two diagonal, compression-only struts. The adopted model assumes that the contribution of the masonry infill panel to the response of the infilled frame can be modeled by replacing the panel by a system of two diagonal masonry compression struts. The individual masonry struts are considered to be ineffective in tension.

Accordingly, infill panels are modeled by equivalent diagonal struts, which carry loads only in compression. The shear strut model, representing the infill panels shear capacity normal to the gravitational direction is implemented in an equivalent discrete shear-type model. In the proposed infill panel model, each masonry panel is structurally defined by considering four support strut-elements, with rigid behavior, and a central strut element, where the nonlinear hysteretic behavior is concentrated. The forces developed in the central element are purely of tensile or compressive nature. Besides it is possible to obtain mechanical properties of the infill walls from prism tests to model the equivalent struts, in this paper test machines used to determine the mechanical properties of the masonry prisms are not available that most prevalent values of compressive and shear strengths of HCB masonry prisms were browsed from relevant literatures and code conforming values are thus used as input data for numerical modeling of infilled RC frames on finite element software packages.

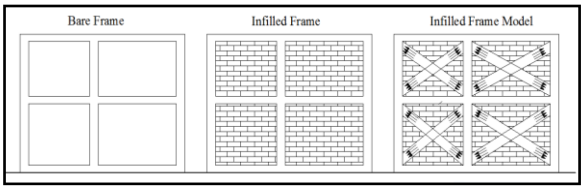


**Figure 1: Structural layout of bare frame, infilled frame and infill frame models**

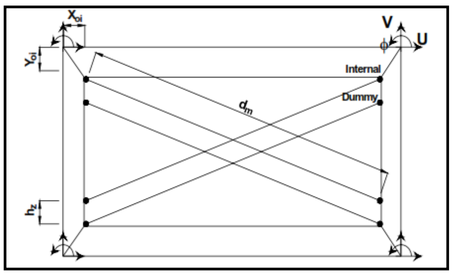


**Figure 2: Equivalent diagonal strut model**

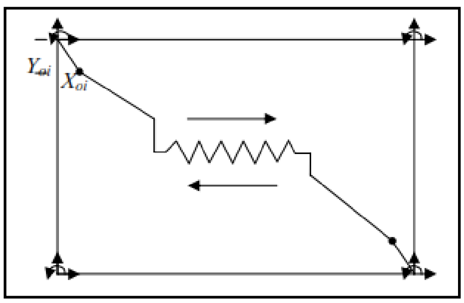


**Figure 3: Equivalent shear spring model**

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The proposed building model cases with various infill configurations are thus numerically modelled on SeismoStruct 2016. This computer program is analytical software works on principles of finite element package for structural analysis, capable of predicting the large displacement behavior of space frames under static or dynamic loadings, taking in to account both geometric nonlinearities and material inelasticity. The software has inbuilt nonlinear and hysteretic material properties for concrete, steel, infills and other engineering materials. Five (5) infill configuration models are proposed for each designed buildings model cases to use in numerical modelingand assessment of seismic performances. All the proposed building models having infill panels are introduced with 20cm thick HCB as external wall and 15cm thick as internal walls. Also the effect of openings due to windows and doors has been considered through stiffness reduction factor. Static pushover and nonlinear time-history analysis are performed after complete numerical model of buildings in their three dimensional state.

### 3.4 Seismic Fragility Curve

In this paper, pushover and nonlinear dynamic time history analyses are performed on SeismoStruct 2016 software to evaluate the seismic performance of the case study buildings. To predict the response of the selected structures during an earthquake, 30 artificial accelerograms using SeismoArtif 2016 are generated, scaled, and matched with Ethiopian response spectrum and loaded on all building model cases for nonlinear dynamic time history analysis.

Fragility analysis is ideal for showing the probability of structural damage due to earthquakes as a function of ground motion intensity indices. It is essential for seismic risk assessment and performance-based earthquake engineering. Seismic performance evaluations using nonlinear time history analysis for reinforced concrete moment-resisting frames incorporate the classical concept of fragility curves. Fragility curves are developed based on the fragility concept and it provides conditional probability of exceeding a certain limit state at each seismic performance state for a given seismic intensity level. A fragility curve can be typically generated with the use of a mathematical function related to seismic capacity and demand of the structure, accounting for their uncertainties. Accordingly, seismic fragility curves corresponding to individual performance levels are developed on the basis of nonlinear time history analyses for the building model case studies.

The fragility function represents the probability of exceedance of a selected Demand Parameter (EDP) for a selected structural limit state (LS) for a specific ground motion intensity measure (IM). Fragility curves are cumulative probability distributions that indicate the probability that a component or system will be damaged to a given damage state or a more severe one, as a function of a particular demand. The seismic fragility, FR(x) can be expressed in closed form using the following equation as per Cornell et. al. [12]; and a fragility curve is obtained for different limit states using this equation.

$$P(D \geq C/IM) = 1 - \emptyset\left\{\frac{\ln\frac{S_C}{S_D}}{\sqrt{\beta_{D/IM}^2 + \beta_C^2 + \beta_M^2}}\right\}$$

Where:- 'D' is the drift demand, 'C' is the drift capacity at chosen limit state, SC and SD are the chosen limit state and the median of the demand (LS) respectively, βd/IM, βC and βM are dispersions in the intensity measure, capacities and modeling respectively.

All building models under the case study were analyzed using Seismo-Struct software to assess seismic vulnerabilities. Non-linear dynamic time history and pushover analysis was employed to generate fragility curves. 30 generated artificial accelerograms were employed in the nonlinear dynamic time history analysis. Accordingly, for developing a fragility curve, nonlinear dynamic analyses of 30 building models for each case are conducted and the maximum roof displacement (ID) for each ground motion is recorded. Thus the maximum roof displacements obtained in each time history analysis were combined with the results of pushover analysis so as to develop seismic fragility curves at defined performance levels. The parameters of the power law model are found out by regression analysis for each frame to develop PSDM model. The fragility curves depict probabilities of exceedance for different damage states, and used for seismic performance evaluation of building models under the study.

## IV. NUMERICAL MODELING AND ANALYSIS

Non-linear dynamic time history analysis is considered as the most advanced and comprehensive analytical method for evaluating the seismic response and performance of multi-degree-of-freedom building structures subjected to seismic excitation.

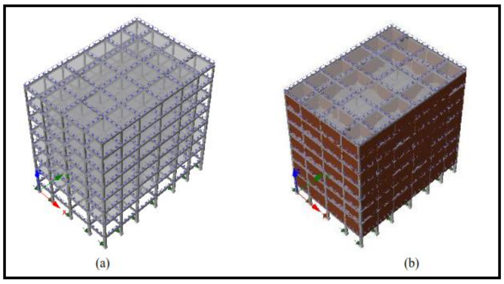


**Figure 4: (a) 3D simulated G+6 bare frame building model, (b) 3D simulated G+6 infilled frame building model**

 

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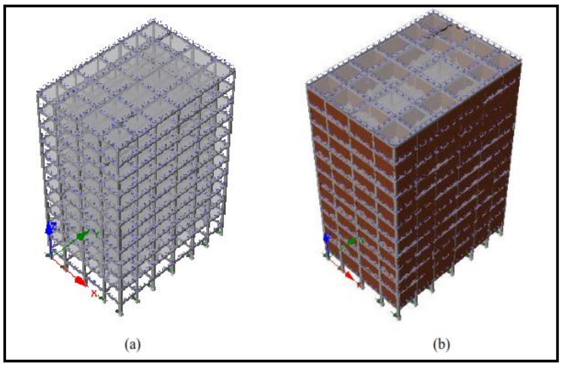


**Figure 5: (a) 3D simulated G+10 bare frame building model, (b) 3D simulated G+10 infilled frame building model**

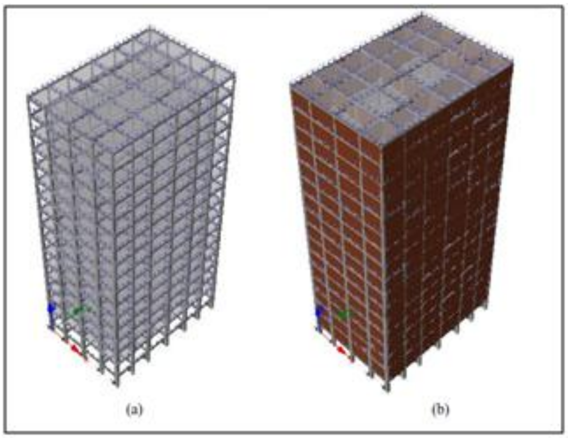


**Figure 6: (a) 3D simulated G+15 bare frame building model, (b) 3D simulated G+15 infilled frame building model**

**Figure 7: Computer laboratory for running nonlinear dynamic time history analysis**

## V. RESULTS AND DISCUSSION

Fragility curves for all building model cases has been generated for three performance levels namely Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) based on the administered maximum roof displacements obtained from nonlinear dynamic time history analysis. Basically the curve showed variations of exceedance probability of the roof displacement with the PGA. For implicit discussion and quantitative investigation of the infill effects in advance comparisons are made here under based on a PGA value 0.65g which is maximum considerable earthquake in Ethiopia as presented in the seismic hazard curve.

### 5.1 G+6 Building Model Cases

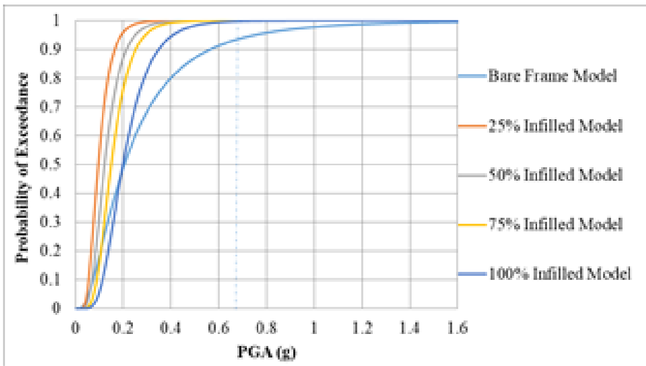


**Figure 8: Fragility Curves at Immediate Occupancy (IO)**

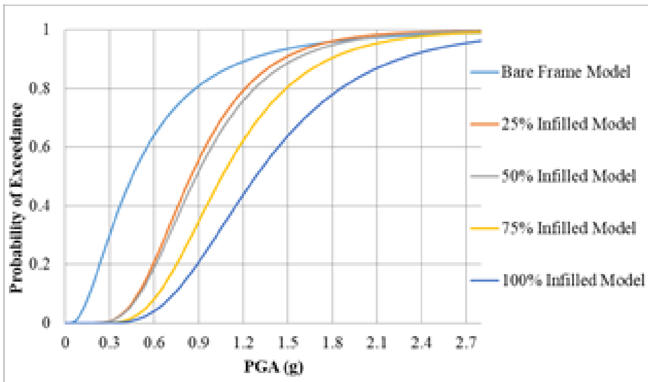


**Figure 9: Fragility Curves at Life Safety (LS)**

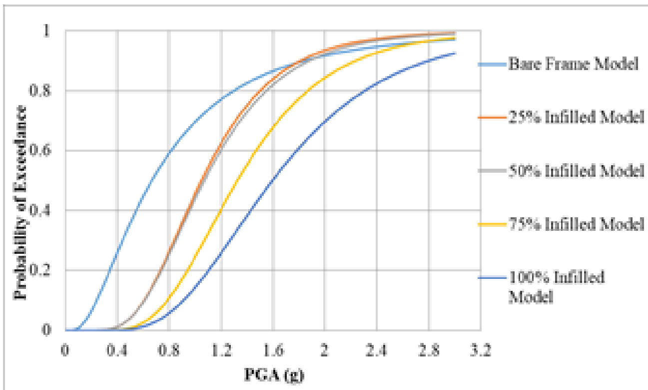


**Figure 10: Fragility Curves at Collapse Prevention (CP)**

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### 5.2 G+10 Building Model Cases

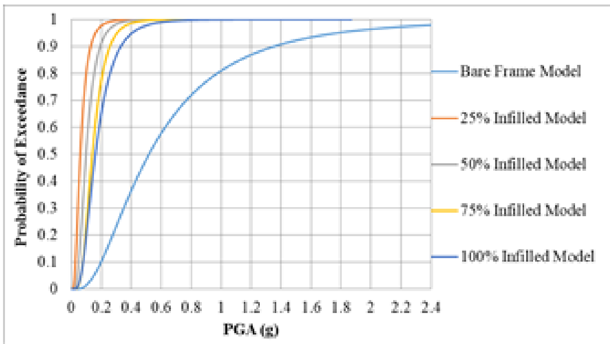


**Figure 11: Fragility Curves at Immediate Occupancy (IO)**

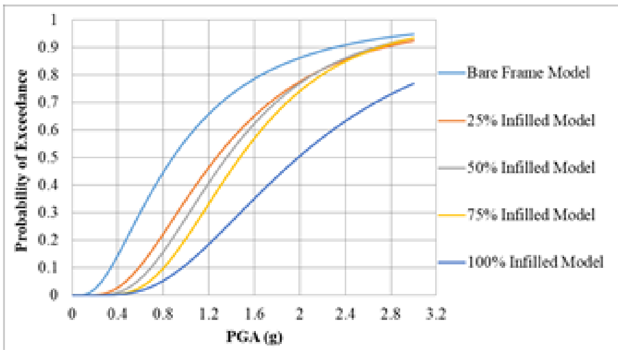


**Figure 12: Fragility Curves at Life Safety (LS)**

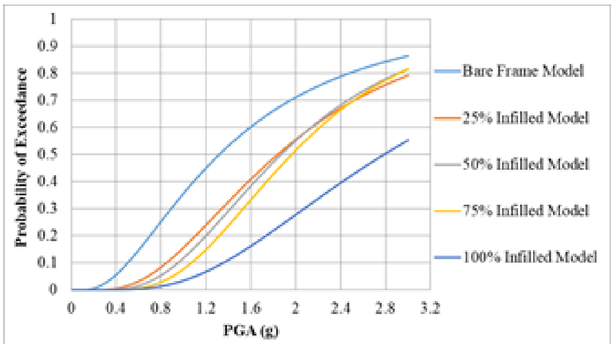


**Figure 13: Fragility Curves at Collapse Prevention (CP)**

### 5.3 G+15 Building Model Cases

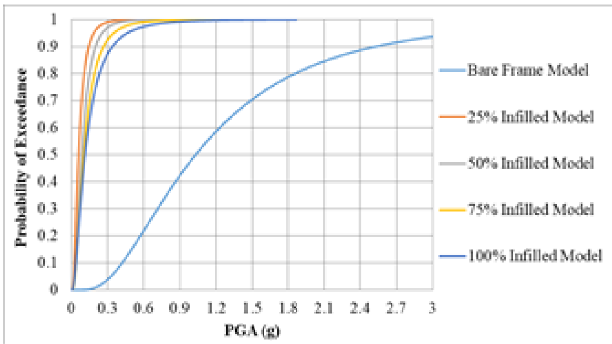


**Figure 14: Fragility Curves at Immediate Occupancy (IO)**

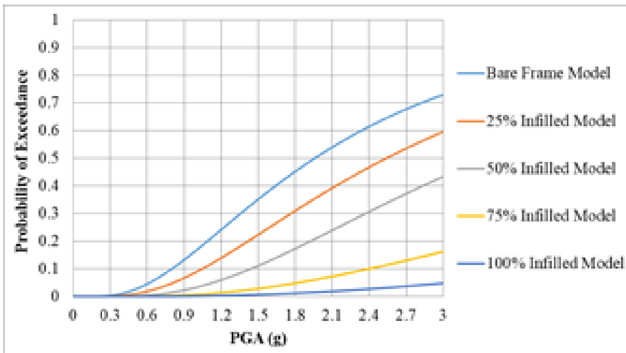


**Figure 15: Fragility Curves at Life Safety (LS)**

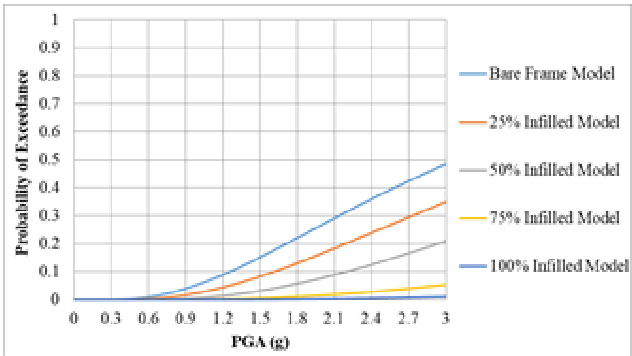


**Figure 16: Fragility Curves at Collapse Prevention (CP)**

Fragility curves for G+6, G+10 and G+15 building model cases respectively at three performance levels (i.e. Immediate Occupancy, Life Safety, and Collapse Prevention) were generated. Fragility curves are basically developed based on the maximum roof displacements due to simulated ground motions in nonlinear dynamic time history analysis for all building model cases. Accordingly, for all case study buildings bare frames perform very well in immediate occupancy performance levels as infill cracks are classified under serviceability limit state. Basically immediate occupancy is related to this limit state where minor structural cracks and significant nonstructural cracks occur.

From the fragility curve all building model cases with varying infill configuration performed well than bare frames at life safety performance level as the roof displacements of infilled models are arrested somehow in best manner. The probability of exceedance of 100% infilled building models are lesser than 75% infilled building models. This inference applies to all building cases as the % of infills configurations decreases. And also the probability of exceedance of 100% infilled G+10 building model is less than 100% infilled G+6 building model case. Similarly the probability of exceedance of 100% infilled G+15 building model is less than 100% infilled G+6 building model case.

 

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**Table 2: Probability of failure of all building model cases at PGA = 0.65g**

| Building Model Types | Probability of Failure for each Performance Level at PGA = 0.65g | | |
|---|---|---|---|
| | Immediate Occupancy (IO), % | Life Safety (LS), % | Collapse Prevention (CP), % |
| G+6 Building Model | | | |
| Bare Frame Model | 92.7 | 67.8 | 48.7 |
| 25% Infilled Model | 99.9 | 26.8 | 13.1 |
| 50% Infilled Model | 99.9 | 24.3 | 12.7 |
| 75% Infilled Model | 99.9 | 11.7 | 2.2 |
| 100% Infilled Model | 99.6 | 6.0 | 2.0 |
| G+10 Building Model | | | |
| Bare Frame Model | 61.9 | 34.0 | 17.2 |
| 25% Infilled Model | 99.9 | 13.4 | 4.2 |
| 50% Infilled Model | 99.9 | 8.1 | 2.2 |
| 75% Infilled Model | 99.9 | 4.1 | 0.9 |
| 100% Infilled Model | 99.3 | 2.3 | 0.4 |
| G+15 Building Model | | | |
| Bare Frame Model | 25.4 | 5.70 | 1.2 |
| 25% Infilled Model | 99.9 | 2.40 | 0.4 |
| 50% Infilled Model | 99.9 | 0.60 | 0.08 |
| 75% Infilled Model | 99.3 | 0.10 | 0.01 |
| 100% Infilled Model | 97.9 | 0.01 | 0.01 |

It was noted that the probability of failure for both bare frame and infilled models decreases as the number of story increases. From the table above the probability of failure of G+ 6, G+10, and G+15 bare frame building models is 67.8%, 34.0% and 5.70% respectively at life safety performance level confirming the lesser probability of failure for high rise buildings. G+6 bare frame building model has 48.7% failure probability at collapse prevention limit state and as in the case in life safety limit state the probability of failure decreases with introduction of infills into the model. Basically these infills have significant contribution in arresting large lateral deflections and results in lower and most tolerable story displacements under excited earthquake motion and eventually reducing the structure's probability of failure. Similarly at collapse prevention limit state the probability of failure decreases as the number of story increases. As shown in the above table at collapse prevention limit state, the probability of failure of G+ 6, G+10, and G+15 bare frame building models is 48.7%, 17.2% and 1.2% respectively.

## VI. CONCLUSION

Evaluation of the case study buildings are based on performance-based seismic assessment approaches which includes a specific intent to achieve defined performance objectives in future earthquakes. Performance objectives relate to expectations regarding the amount of damage a building may experience in response to earthquake shaking, and the consequences of that damage on overall end users of the building and equipment attached thereto. Fragility curves for all building model cases has been generated for three performance levels namely Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) based on the administered maximum roof displacements obtained from nonlinear dynamic time history analysis. Basically the curve showed variations of exceedance probability of the roof displacement with the PGA.

- The results from the fragility curve showed that probability of exceedance at immediate occupancy (IO) for 100% infilled building models cases are less that 75% and ever lesser infilled building model cases. And it reveals that larger percentages of infills panels perform well at immediate occupancy limit states. Moreover, referring to the figure, it has been noted that the probability of exceedance gets smaller as the number of story increases.
- At life safety and collapse prevention performance level, it was noted that bare frame has a highest probability of failure and building models with a larger percentage of infill configuration have lesser failure probability than slightly infilled building models. Basically these infills have significant contribution in arresting large lateral deflections and results in lower and most tolerable story displacements under excited earthquake motion and eventually reducing the structure's probability of failure. And also it was noted that the probability of failure for both bare frame and infilled models decreases as the number of story increases.

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