VULNERABILITY ASSESSMENT OF MASONRY STRUCTURES IN SEISMICALLY PRONE REGIONS OF PAKISTAN

By

Muhammad Waleed Khan

(NUST-2016-MS STRUCTURAL ENGINEERING- 00000171778)

Thesis submitted in partial fulfillment of the requirements for the degree of

Master of Science

In

Structural Engineering

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School of Civil and Environmental Engineering (SCEE)

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THESIS ACCEPTANCE CERTIFICATE

It is certified that final copy of MS thesis written by **Mr. Muhammad Waleed Khan**, Registration No. **00000171778**, of **NUST INSTITUTE OF CIVIL ENGINEERING (NICE)** has been vetted by undersigned, found complete in all respects as per NUST Statutes/Regulations, is free of plagiarism, errors, and mistakes and is accepted as partial fulfillment for award of MS degree in Structural Engineering.

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DECLARATION

I certify that this research work titled "**Vulnerability Assessment of Masonry Structures in Seismically Prone Regions of Pakistan**" is my own work. The work has not been presented elsewhere for assessment. The material that has been used from other sources has been properly acknowledged/referenced.

Signature of Student

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DEDICATED

TO

MY LOVING PARENTS

FOR THEIR SUPPORT, LOVE, AND ENCOURAGEMENT

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ABSTRACT

The buildings can withstand earthquake more effectively if the structural performance of the existing buildings can be assessed accurately. This study aims at determining the effect of masonry as an infill on the vulnerability of reinforced concrete frame buildings by using probabilistic performance based assessment. Refined linear and nonlinear structural models were developed, from data collected through professional surveys, using PERFORM-3D platform. Nonlinear – static and dynamic - analyses were carried out, for fifteen natural ground motions, to examine the plastic behavior of the models. Subsequently, vulnerability was assessed using fragility relationships. The fragility parameters were determined by employing Maximum Likelihood Method (MLM). The results indicated decrease in the probability of exceedance of specific damage states of the structures with respect to seismic intensity for masonry infill frames. From fragility curves it is concluded that although the use of masonry as an infill temporarily enhances the capacity of Reinforced Concrete (RC) Frame buildings as the probability of exceedance for masonry infilled RC frames is significantly reduced due to the increase in overall stiffness of the structure but it restricts the ability of structure to deform which leads to localized failures.

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CHAPTER 1

INTRODUCTION

1.1. BACKGROUND

Earthquakes are generally catastrophic in nature and it challenges the strength of structures to withstand. From the advent of time man is in constant struggle to build structures which are capable of sustaining nature's forces of destruction like earthquake, windstorm etc. Structural performance of all structures, especially buildings, under extreme conditions is of paramount importance to civil engineer in general and structural engineer in particular. With the passage of time efforts have been made to determine the response of buildings to tremors. Building codes have also been updated from time to time in order to incorporate the advancements made in the field. New buildings can be built under latest provisions, provided by building codes, in order to make them sustainable against earthquakes but the performance state of existing buildings is of prime concern. It is important to determine whether or not they meet the required standards. Moreover, knowing the performance state of buildings is also important as it will serve the policy makers, engineers and the populace in decision making about safety, rehabilitation, construction practices and risk assessment of buildings. Therefore, seismic vulnerability assessment is required to quantify the "damage risk" of a structure or building in response to Ground motion of given intensity. Results from vulnerability assessment can be utilized for damage and loss evaluation, disaster response planning and retrofitting decision making of buildings. The damage risk can be quantified graphically and can be represented using fragility curves. These curves give us the probability of occurrence of a certain damage state or performance level as a function of Intensity measure.

1.2. PROBLEM STATEMENT

Since it is a known fact that earthquakes are generally disastrous in nature, therefore, can render severe damage in the form of human lives and infrastructure failure. These earthquakes cannot be predicted accurately. Globally, concerted efforts have been made by researchers to either predict or to counter the effects of earthquake on structures.

Pakistan lies on the cusp of distinctive global tectonic framework with 3 major tectonic plates existing in this part of the world. They are Eurasian, Indian and Arabian Tectonic Plates. Therefore, the presence of these tectonic plates indicate that there is every chance of seismic activity. Consequently, it is implicit to know about the existing state of the buildings in terms of their structural performance against possible seismic activity .In order to carryout necessary preparatory actions to avert casualties and fiscal losses in some due analogous event, such a proactive disaster preparedness approach is only possible when the necessary data available to analyze which can remain helpful for decision making purposes.

1.3. SIGNIFICANCE OF RESEARCH

Kashmir earthquake s in October 2005 was catastrophic. It was a wakeup call for policy makers and engineers in particular and force majeure in general. It was necessary to have realization about importance of having safe structures that should be resistant and flexible to bare such seismic activities. Due to poor construction practices and seismically undermined infrastructural facilities particularly school buildings, lives of people especially youth becomes vulnerable. During its life span a structure may encounter severe seismic loadings, therefore it is essential to have an adequate engineering expertise to estimate the diminishing structural capacity in order to evade whole structural failure or to adopt every necessary interventions (Zain et al. 2018). Approximately 87,000 causalities occurred during Kashmir earthquake (2005), among which almost 25 thousand were children going to schools. The number of affected people was about 3.5

million (Khazai et al. 2006). It was also reported by (Khazai et al. 2006) that nearly Two thirds of the educational institutions in the region met full destruction. However, besides the deaths and socioeconomic consequences, this event resulted in the concept of "Build Back Better" in Pakistan, and Pakistan's building code (BCP,2007) – (Seismic Provisions 2007) is arguably a physical outcome of it. UNDP, in 2015 appraised the applicability of BCP 2007 and by-laws. The UNDP in its report (Bonowitz 2015) stated that due to tight development programme, there was mass scale assimilation of American building code and prohibited the establishment of a code based on Pakistan's conditions.

In main metropolises and its suburbs, by-laws are implemented by developmental authorities. However, the by-laws of developing authorities are incomplete, inconsistent or even contradictory at times to the Building code of Pakistan and its provisions for earthquake design. Therefore, to ensure safety the onus lies on Design professional for sustainable design (Bonowitz 2015).

Bearing in mind the prevalent state of exercise of Building Code of Pakistan 2007 and its requirements in construction industry, the present study is intended to evaluate the seismic vulnerability of Reinforced Concrete school building with and without Masonry infill which have been designed and built in the Pakistan's high seismic zone in aftermath of 2005 Kashmir Earthquake.

1.4. OBJECTIVES

In view of above problem statement, the present research is concentrated on the following objectives:

i. To examine the effect of masonry infill on structural performance of RC frame school buildings in Pakistan using non-linear static and dynamic procedures.

ii. To assess the vulnerability of Reinforced Concrete frame school buildings with and without masonry infill walls, situated in high seismic zone of Pakistan

1.5. METHODOLOGY:

The methodology that was followed is presented as a follows:

1. Data was collected from field.
2. Development of an inventory containing information about the architectural and structural dimensions of the stone masonry school buildings along with the physical observations according to FEMA 154.
3. Development of analytical generic masonry school building(s) model(s).
4. Application of non-linear static and dynamic procedures to gauge the structural performance against seismic hazards of seismic zone 4 of Pakistan.
5. Vulnerability assessment of masonry school buildings, with and without masonry infill incorporation, by development of fragility curves.

1.6. THESIS OVERVIEW

Chapter 1 describes background, problem statement, research objectives, summary of methodology and thesis layout.

Chapter 2 includes literature review. It comprises of previous studies on

Chapter 3 explain research methodology. It describes the processes involved in

Chapter 4 includes results obtained by following research methodology mentioned in Chapter 3. A detailed discussion is done to explain the results.

Chapter 5 consists of derived conclusions about the topic and also features future recommendations.

CHAPTER 2

LITERATURE REVIEW

2.1. INTRODUCTION TO CHAPTER

In this chapter we will discuss about the literature about the seismic performance of reinforced-concrete masonry infilled buildings. We will discuss about the various parameters that are important in estimating the fragility of the structure. Fragility basically tells us about the probability of exceedance of a particular damage state against a specific seismic intensity measure for a given building stock of specific engineering demand parameter. Thus in this chapter a brief explanation about various parameters that influence the fragility behavior will be discussed. Definition of limit states, seismic intensity measures and engineering demand parameters and their use by various researchers will also be discussed. In final section of the chapter, literature about the use of masonry as an infill material will be discussed. Moreover, deliberation on the use of different techniques to evaluate seismic performance and fragility behavior over period of time will also be carried out. In the end, the importance of using performance based design to evaluate the vulnerability will be described by supplementing it with supporting literature.

2.2. LITERATURE REVIEW

This study is conducted to evaluate the effect of masonry infill on the overall vulnerability or fragility of RC Frame structures. Fragility assessments are done to gauge the seismic vulnerability of buildings or other structures when exposed to ground motions. The fragility curves, obtained through fragility assessment, are crucial for foreseeing the general scale of damage to the infrastructure and also are helpful to estimate the monetary misfortune related to such events.

These curves are also critical for retrofitting of damaged or old buildings and are primary source of information for Earthquake response units. Fragility curves graphically represents the probabilities of exceedance or occurence of certain damage level or limit state against the seismic intensity measures like spectral displacement (S_d), peak ground acceleration (PGA) or spectral acceleration (S_a) etc.

The fragility analysis is generally done to assess the seismic vulnerability of structures (Akkar et al. 2005; Cornell et al. 2002; Ellingwood et al. 2007; Lagaros 2008; Lang and Bachmann 2004; Mosalam et al. 1997; Ramamoorthy et al. 2006; Serdar Kirçil and Polat 2006; Seyedi et al. 2009). In fragility investigation of structures, the demands are log normally distributed (Cornell et al. 2002) this means that the relationship between the intensity measure and demand can be characterized by a two parameter model (Cornell et al. 2002; Ellingwood et al. 2007; Konstantinidis and Makris 2009; Ramamoorthy et al. 2006). In order to acquire best fit curve Regression analysis can be executed using either straight regression relation or bilinear regression relation (Ramamoorthy et al. 2006). The chance of exceeding various damage states for a selected Intensity Measure can be obtained only when the median and standard deviation are determined utilizing the Maximum likelihood Method (Baker and Eeri 2015). Some of the limit state conditions for structures are immediate occupancy (*IO*) , life safety (*LS*) and collapse prevention (*CP*) ("Applied Technology Council, ATC-40 (1996) Seismic Evaluation and Retrofit of Concrete Buildings, Vols. 1 and 2, California. " 2015).

2.2.1. DEFINITIONS

In this section a brief account is given about the parameters used to evaluate fragility. Parameters that influences the outcome of fragility curves like IM (intensity measures), the structures performance level or (Damage States) are concisely discussed .Moreover, summary of the procedures that has been used to carry out fragility analyses are also explained here:

2.2.1.1. SEISMIC INTENSITY MEASURE

Seismic Intensity Measure (IMs) is ground motion factor in terms of which the seismic activity is quantitatively measured (Vamvatsikos et al. 2004). It is one of the most significant parameter for fragility study. The significance of Intensity measure can be ascertained from the fact that this parameter is reflective in the final output of fragility assessment, the fragility curves, in which the probability of occurrence of a given damage state is plotted against any Intensity Measure. The Intensity measures are mainly classified as:

1. The empirical IMs
2. The instrumental IMs.

Empirical Intensity Measures

Empirical Intensity Measures, diverse macro-seismic intensity scales are employed mainly for qualitative assessments to measure damage to the building. Few examples of these scales are:

1. The Mercalli Cancani- Sieberg (MCS).
2. The Modified Mercalli Intensity Scale (MMI).
3. The European Macroseismic Scale (EMS-98).

These macro-seismic intensity scales have a variety of uses in the area of fragility analysis.

Instrumental Intensity Measures

In the instrumental intensity measures, accelerograms are used to measure the severity of the ground quaking. The commonly used intensity measures to evaluate seismic vulnerability are:

i. Spectral acceleration, S_a

ii. Peak ground acceleration, *PGA*

iii. Spectral displacement, S_d

iv. Peak ground velocity, *PGV*

2.2.1.2. PERFORMANCE LEVELS OF BUILDINGS

As per FEMA recommendations the performance of a structure or a building should be selected from four Structural Performance Levels Ranges (FEMA 356 2012).These performance levels of a building are listed in increasing order of damage as :

1. Immediate Occupancy,
2. Life Safety,
3. Limited Safety Range and
4. Collapse Prevention.

Immediate Occupancy (IO) Structural Performance Level

The immediate occupancy level of structural performance is characterized as the post-earthquake damage state in which there is minor or secondary damage have occurred and that structural components in the building are safe. Principally, the structure or building have not lost any strength i.e. the structure quality and firmness of the structure is restored. The damage is basically low and some minor secondary repairs may only be desirable at most. However, in most the cases these repairs are not required before reoccupying.

Damage Control Range

This is not exactly a performance level but a range between Immediate Occupancy performance level and Life Safety Structural Performance level.

Life Safety (LS) Structural Performance Level

The life safety structural performance level is one in which after seismic activity the structure or building is damaged to certain extent. However, it is important to mention here that the structure still has some capacity or strength to withstand against incomplete or complete failure. Also the main structural components are damaged extremely, yet no complete failure is brought about, either inside or outside, of the structure or building. Damage to the structure may happen during the seismic tremor but the extent or degree of damage is not life threatening. Repairs and maintenance is required before reoccupancy.

Limited Safety Structural Performance Range

This is not a performance level but a short range like damage control, the range is between the Life Safety Structural Performance Level and the Collapse Prevention Structural Performance Level.

Collapse Prevention (CP) Structural Performance Level

Collapse prevention performance level is that post earthquake damage state in which the structure or building is near to its collapse. The structure is damaged considerably and substantial stiffness and strength degradation has also occurred in the lateral force resisting mechanism. Moreover, large permanent deformations in lateral directions have happened. The axial load carrying capacity is also reduced significantly. Technically the structure can be said to be irreparable and is therefore not fit to reoccupy.

2.2.1.3. METHODS TO DERIVE SEISMIC FRAGILITY FUNCTIONS:

To calculate the fragility a number of methods and techniques have been proposed by the researchers. Broadly seismic vulnerability can be assessed through four different methods i.e. empirical, expert opinion based, analytical and hybrid.

(a) Empirical methods

The empirical fragility curves (Calvi et al. 2006) are developed using insights from the damage data observed or information gathered by post-earthquake surveys . In this method the observational information is processed in the most practical way in order to obtain fragility curves. However, it is pertinent to mention that the fragility curves achieved through this method requires concerted effort and are hard to obtain due to the chance of having inadequacies during data collection process as well as due to the mistakes, which may occur, during post-processing of the obtained on-site information. The main types of empirical methods are:

i. The Vulnerability Index Method
ii. The Damage Probability Matrices (DPM)
iii. The Continuous Vulnerability Functions

(b) Expert opinion-based methods

These methods to derive fragility functions or curves, as the name suggests, are based on the judgment of the experts on the information available to them. The experts are inquired to provide an educated guess or estimate the probability of damage for various types of structures and numerous levels of ground shaking from the past earthquake events. The limitations regarding the quantity and quality of structural damage does not affect the results. Though, as the results are based on the judgment of individual experience thus a degree of biasness is introduced. Moreover, the level of knowledge the expert has also influences the results of fragility curves.

(c) Analytical methods

Seismic Vulnerability can also be assessed by using Analytical methods. In analytical methods stepwise algorithm is used to estimate the seismic vulnerability and hazard associated with structures. The fragility curves in this type of vulnerability assessment are based on certain damage states. These methods are employed when no onsite data is available or there is need to know about the existing vulnerability of building or group of buildings to any impending hazard. Principally, simulations are carried out on structural models developed in high-end softwares under increasing earthquake intensity. However, these methods besides having accurate prediction capabilities have limitation of high computational effort. Thus in order to minimize the computational effort, analytical models are simplified to incorporate large number of analyses and consequently obtaining results with high accuracy and less uncertainties. Some of the analytical methods used to assess seismic vulnerability are mentioned below:

i. Lateral force analysis (linear)
ii. Modal response spectrum analysis (linear)
iii. Non-linear time history dynamic analysis (NTHA)
iv. Non-linear static (pushover) analysis (POA)

It is noteworthy to mention that studies on seismic fragility functions on Reinforced Concrete structures are predominantly based on Analytical methods especially in the last few decades. (Akkar et al. 2005; Dumova-Jovanoska 2000; Erberik and Elnashai 2004; Serdar Kirçil and Polat 2006) presented studies based on analytical methods. (Esra Mete Güneyisi and Gülay Altay 2008) derived analytical fragility curves for 12 storey RC Moment Resisting Frame in Turkey. That study concluded that computational effort can be minimized by performing vulnerability assessment in weaker direction of RC frames without compromising significantly on the accuracy of results. Therefore, in this particular study is conducted on the weaker direction only. The weaker direction can be found by carrying out pushover analysis.

(d) Hybrid methods

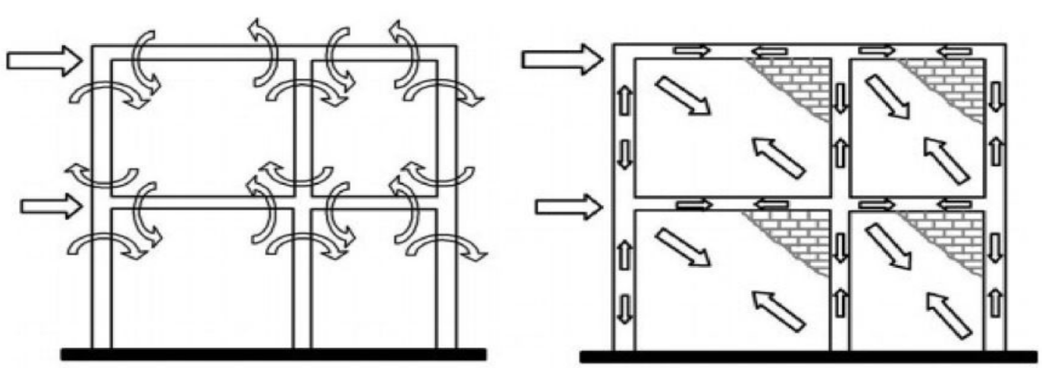
These methods are a natural consequence in the quest to achieve accurate results with minimum possible efforts. As the name suggests hybrid fragility curves are basically the combination of various methods for predicting damage and evaluating loss. The main objective or need for such techniques is to recompense the unavailability of observed information, the insufficiencies of the structural models and the biasness in expert opinion data. (Barbat et al. 2018; Kappos et al. 2006) conducted Researches are some of the few studies available on hybrid methods.

2.2.3. LITERATURE ON MASONRY INFILL

Most buildings, those generally ranging from medium to low rise RC frames, are not engineered to withstand major earthquakes (Bonowitz 2015). Masonry infills have been primarily been used for aesthetic reasons and deemed as non-structural elements during design phase (Lee and Woo 2002).The behavior of masonry infilled frames under lateral loads have been examined by several researchers. Reinforced concrete frames confines masonry infill walls on all lateral the sides and plays a major role in resisting the lateral load on structure. (Moghaddam and Dowling 1988) deduced that infill walls provide high lateral stiffness and have low deformability in lateral direction. Therefore, this behavior of infill RC frame structures changes the lateral-load transfer mechanism of the structure from a frame action to predominantly truss action (Murty and Jain 2000) as can be seen in Fig.2.1, in which decrease in the bending moments of the structure at the expense of increases axial forces due to presence of infill.

(Bertero and Brokken 1983) carried out experimental investigation to study the lateral stiffness of the infilled frames in comparison to bare frames. The study was carried out on 1/3 scale models for 11 storey building. Quasistatic cyclic and Monotonic load tests were applied on the lower 3.5 storey of the building. The study concluded that the effective inter-storey lateral stiffness was 5.3 to 11.7 times the lateral stiffness of the bare frame.

(Mehrabi et al. 1996) tested 12 half scale, single bay, single storey models with concrete block masonry as an infill. The models were designed as per code provisions. The main objective of the study was to determine the influence of relative strength and stiffness to control bare frame. After the application of lateral stiffness the results showed positive improvement significantly in comparison to bare frames. It was also showed that infill frames had better energy dissipation capabilities. Although these researches and many others showed that the lateral stiffness does increase in case of infill Reinforced Concrete frames through experimental setups yet it did not suffice for the heterogeneous and complex behavior of Masonry infill structures, therefore, a more elaborate technique was required to explain this complex behavior in more profound way. Thus, in order to understand the complex behavior of masonry infill researchers have now turned their focus to nonlinear procedures, both static and dynamic, depending upon the amount of accuracy and precision required. (Lodi and Mohammad 2012) suggested the use of nonlinear static procedure to understand the behavior of Masonry infill in RC structures. The study concluded that nonlinear analysis is a powerful tool to establish accurate results. The study also suggested that nonlinear static are relatively simple and results are easy to work through than its nonlinear dynamic counterpart. However, the later has more accurate results than former technique. In this research we will mainly use nonlinear dynamic analysis to obtain well conforming results and then assess the probability of exceedance of both RCF and MIRCF against seismic intensity measure. From Global response we can infer about the behavioral response and structural performance of the building when subjected to real time histories.



(a) Frame action (b) Truss action

Fig 2. 1 Transformation of lateral-load transfer mechanism due to presence of infill walls (Murty and Jain 2000).

2.2.4. LITERATURE ON MODELLING OF MASONRY INFILL

In order to model infill several methods have been developed over the years. Broadly they can be classified into two main categories namely, Micro-modelling and Macro-modelling. Micro-modelling focus on the detailed behavior of Individual infill while in Macro-modelling we study global response of the structure .In this research we are using Macro-modelling because one significant advantage of Macro-Modelling is its simplicity in computational efforts. The computational effort is largely reduced due to the application of "Equivalent Strut Model", which were first introduced by (Polyakov 1956) . He was the first to give the idea to replace the infill by employing Diagonal Compression struts. In this study we also have employed diagonal compression struts to represent the Masonry infill RC Frames. (Priestley and Paulay 2011) computed the width of strut using equation:

$$\boldsymbol{w = 0.25\, d_{inf}} \qquad \textbf{\textit{2. 1}}$$

Where d_{inf}=Diagonal Length of Infill

Here in current study the width 'a' of the strut is calculated as per FEMA provisions (FEMA 356 2012) by following equation:

$$a = 0.175\left(\lambda_1 h_{col}\right)^{-0.4} r_{inf} \qquad \textbf{\textit{2. 2}}$$

where:

$$\lambda_1 = \left[\frac{E_{me} t_{\text{inf}} \sin 2\theta}{4E_{fe} I_{col} h_{\text{inf}}}\right]^{\frac{1}{4}} \qquad \textbf{\textit{2. 3}}$$

h_{inf} = Height of infill panel, in.

I_{col} = Moment of inertia of Column, in^4.

E_{fe} = Expected modulus of elasticity of frame material, ksi.

E_{me} = Expected modulus of elasticity of infill material, ksi

t_{inf} = Thickness of infill panel and equivalent strut, in.

θ = Angle whose tangent is the infill height-to-length aspect ratio, radians

h_{col} = Column height between centerlines of beams, in

r_{inf} = Diagonal length of infill panel, in.

λ_1 = Coefficient used to determine equivalent width of infill strut

(Khan and Rawat 2016) conducted research on nonlinear seismic analysis of masonry infill RC building in India to check the applicability and efficiency of Eccentric bracings on the soft story level. The study concluded that the first storey with and without infill, being a preferred configuration locally, lacks stiffness and undergo large displacements during seismic action and thus become highly vulnerable .In order to counter such insecurities they proposed Strengthing technique which employs the use of Steel eccentric bracings placed at first storey, thus eventually improving the stiffness of first storey.

CHAPTER 3

METHODOLOGY

3.1. INTRODUCTION

This chapter deals with the development of fragility curves and describes the methodology that was followed to obtain these curves. The first section explains about collection of data for school buildings in the selected seismic region. It also discusses about the development of models for Reinforced Concrete Frame both with and without masonry infill. Detailed description about how the ground motion data was obtained and then application of this data on the developed models using Incremental Dynamic Analysis (IDA) is also covered comprehensively in this chapter. Results obtained from IDA on models are further processed to calculate the probability of exceedance. The fragility curves are then obtained from the probability against a particular seismic intensity using Maximum likelihood method for both structural configurations.

3.2. METHODOLOGY:

The methodology which was followed had two main components namely Data Pre-processing and Vulnerability Assessment. Each component carried out is linked and had its own methodology which is explained in the subsequent text. Fig 3.1 shows a flow sheet of steps which embodies the methodology process.

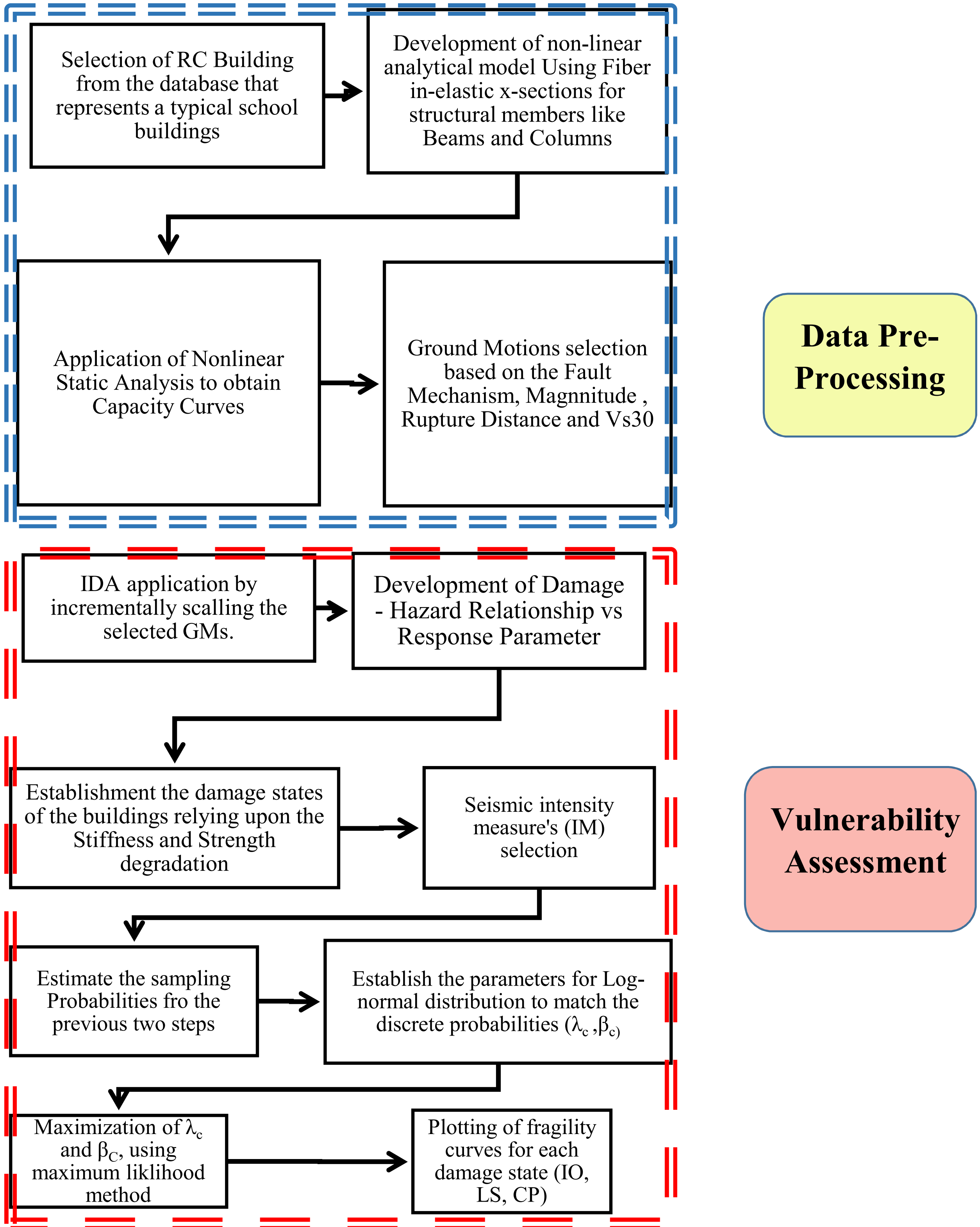


Fig 3. 1 Flow sheet of methodology process

3.2.1. DATA PRE-PROCESSING:

3.2.1.1. SELECTION OF BUILDING FROM EXISTING DATABASE:

In order to assess vulnerability a school building was selected from an existing study done by (Zain et al. 2019) . The goal of developing classifications in this study for school structures with changing structural configurations to evaluate their vulnerability, a total of 19 types of RC school buildings were recognized through on ground inspections and professional discussions (Zain et al. 2019). The catalogue developed after the identification of the building types, along with their structural aspects, are given in table 3.1**.** The name of the model is designated in Column 1 of table 3.1.It is aimed to represent the generic nonlinear model for that category of schools. It can be seen that the names begin from "BLR", and subsequently, the model number is mentioned i.e. BLR-1 all the way up to BLR-19, signifying all the 19 school building classifications with varying architectural and structural arrangements. The "BLR" is abbreviation of "Building-Low-Rise", as the buildings in the region are basically 1 to 3 stories, thus signifying the low-rise nature of buildings. The 2nd column presents the number of stories. The 3rd and 4th column in table 1 provides the amount of bays in x- and y- directions, respectively. The x- and y- directions represent the path along the elevation front face and the orthogonal-face direction respectively. It was observed that mostly two bays were in the y-direction that is one for the alley which primarily functioned as a corridor so that students could walk to get in and out of the classrooms, whereas the other one operated as classroom. The over-all height of the selected buildings are specified in 5th column. The 6th column shows the covered floor area whereas the 7th column relates with the area of openings in slabs, located just above the stair-cases. Covered floor area is for a first floor only.

Table 3.1: Features of the buildings from the developed database.

Building ID	No. of Stories	No. of Bays in x direction	No. of Bays in y direction	Total Building Height (ft.)	Typical Floor Area (ft.^{2})	Area of Slab opening (ft.^{2})
BLR-1	1	6	3	12.5	3537	-
BLR-2	1	3	3	10	1092	-
BLR-3	1	4	2	12.5	1540	-
BLR-4	1	4	2	12.5	1578	-
BLR-5	1	5	2	12	2121	-
BLR-6	1	6	3	12	3530	-
BLR-7	1	6	3	10	3300	-
BLR-8	1	3	2	12.5	870	-
BLR-9	2	4	2	25	1563	100
BLR-10	2	4	2	25	1351	100
BLR-11	2	11	2	20	3675	232
BLR-12	2	2	2	20	715	73
BLR-13	2	5	2	22	2074	175
BLR-14	**2**	**8**	**3**	**24**	**2733**	**390**
BLR-15	2	3	2	20	624	61
BLR-16	2	5	2	20	2400	83
BLR-17	3	4	2	36	4200	267
BLR-18	3	5	2	36	2252	165
BLR-19	3	12	2	30	4065	230

For the purpose of illustration, a double-story structure, BLR-14, is selected. The building groups with 2 number of stories contained class rooms and staff rooms mainly; however it was established that out of all classes, BLR-14 houses one of the most complex configuration of the schools from the considered seismic zone. The main purpose was to present a rational approach that can be implemented by the researchers, and particularly, by the working engineers to evaluate the associated structural vulnerability, and to provide a case study for demonstrating the procedure so that stakeholders may work to establish proactive response towards any further impending disasters. Thus, this study makes it possible to decide any need for intervention to enhance seismic performance of buildings (schools) in Pakistan's high seismic zone.

The data collected in the first component needed to be processed in order to obtain results from which important conclusions could be drawn. Pre-processing involved development of nonlinear models of selected building configuration from the database. Moreover, Equivalent Diagonal compressive strut modelling for representation of masonry infill is also part of this component. In order to obtain capacity curves pushover analysis is carried out. Ground motion data selection for the application of Incremental Dynamic Analysis (IDA) is also done in this component. Finally, after the application of IDA, Damage-Hazard relationship was obtained by plotting engineering demand parameter against seismic intensity parameter. In the text below, Pre-Processing is explained in detail.

3.2.1.2. CONFIGURATION OF SELECTED BUILDING:

For the purpose of illustration, a two-story school structure is selected from Muzaffarabad, a region located in high seismic zone of Pakistan. This two story structure is specially considered as it represents the most complex configuration among the schools in the region. The school building selected have a two-story structure that contains of 8 bays in the x-direction, and 2 bays in the y-direction. The covered floor area is about 2273 sq.ft with slab openings of 390 sq.ft. It comprises of varying sizes of columns and beams. The plan of the selected building considered in the present study is shown in Figure 1.

In Figure 3.5, Floor beams are represented by FB, while floor columns are denoted by prefix "C". The actual buildings are built on comparatively stiff soil, however, during professional discussions it was found that all the buildings have been designed according to the Soft Soil type, S_d soil as per Building Code of Pakistan. The key structural arrangement of the buildings comprise of moment resisting RC frames in two directions that is the x and y directions. Cross-sectional measurements of structural members i.e. various Floor Beams and Floor Columns are shown in Table 3.2. Quantity of reinforcement, obtained through on ground examinations and expert consultations, are also specified in the same table.

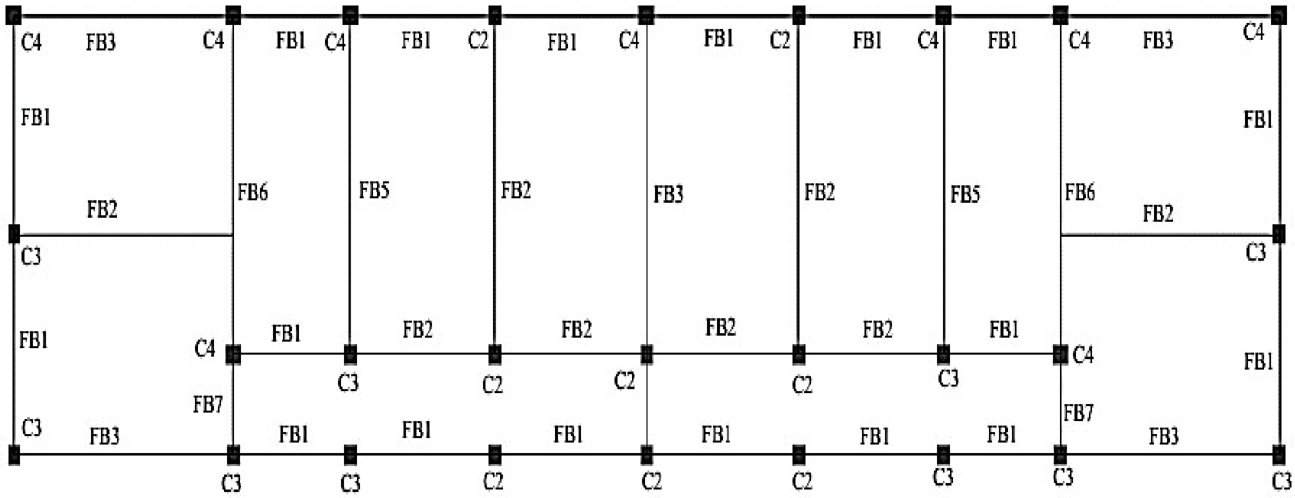


Fig 3. 2 Plan view showing roof beams and columns of considered building

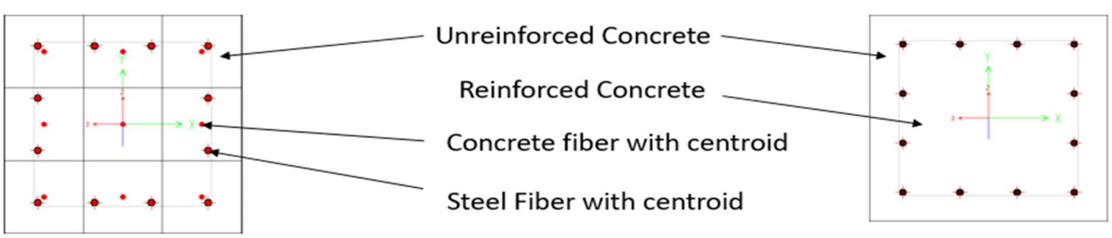
3.2.1.3. NONLINEAR STRUCTURAL MODELLING

A non-linear 3D model for selected building was developed using Perform 3D v 7.0. This software has excellent capabilities to evaluate various characteristics of performance of the structure, extending from hinge rotations in structural members to material strain in them. Therefore in order to represent the realistic nature or response, it was obligatory to take properties of material, into consideration, being used in the structure. Therefore a mean value for 28 days strength of concrete was taken from (Masood Rafi and Muhammad Murtaza Nasir 2014). Consequently, as per the recommendations given by ASCE 41 ("Seism. Eval. Retrofit Exist. Build." 2017); expected strength was used in the analysis. In order to model the reinforcing bars it was deduced from most of the on ground surveys that grade 40 rebar were employed in the building process, therefore, the present study incorporates the properties of grade 40 steel bars in its modelling as recommended by ASCE 41.

Table 3.2: Specifications of Beam and column cross-sections & reinforcement.

Structural Member (s)	Label	Width (inches)	Height (inches)	Top Steel	Bottom Steel	Mid Steel
Beams	FB-1	12	18	3_#6	3_#6	2_#6
	FB-2	12	18	3_#6	3_#6	-
	FB-3	12	18	3_#6	3_#6	2_#6
	FB-4	12	18	4_#6	3_#6	2_#6
	FB-5	12	24	3_#6	4_#6	2_#6
	FB-6	12	24	3_#6	3_#6	2_#6
	FB-7	12	24	5_#6	3_#6	2_#6
Columns	C-2	12	12	2_#6 &1_#4 (mid)	2_#6 &1_#4 (mid)	2_#4
	C-3	12	12	3_#6	3_#6	2_#6
	C-4	12	15	3_#6	3_#6	2_#6

Slabs were modelled as linear elements. Layer-by-layer nonlinear in-elastic fiber sections modelling was carried out for concrete and reinforcing rebar for entire Cross-sections. Force-deformation relationship of beams and columns is converted into stress-strain relationship of construction materials by means of fiber modelling. By using fibers, an suitable disunion may be introduced for unconfined and confined concrete fibers and therefore, composite section can easily be taken into account (Xuewei et al. 2011). Figure 3.6 shows the typical sections of the beam & column fibers (Xuewei et al. 2011).



(a) **(b)**

Fig 3. 3 Representation of *(a)* ***Fibers for beams and columns*** *(b)* ***Simple beams and columns***

In present study, the Confined Mander Model was used to represent concrete. Complete confinement effect was taken into account and strength loss was also accounted for in the analysis as per the details presented in Table 3.2. The model adopted to represent the confined mander model in PERFORM 3D is shown in Figure 3.4.

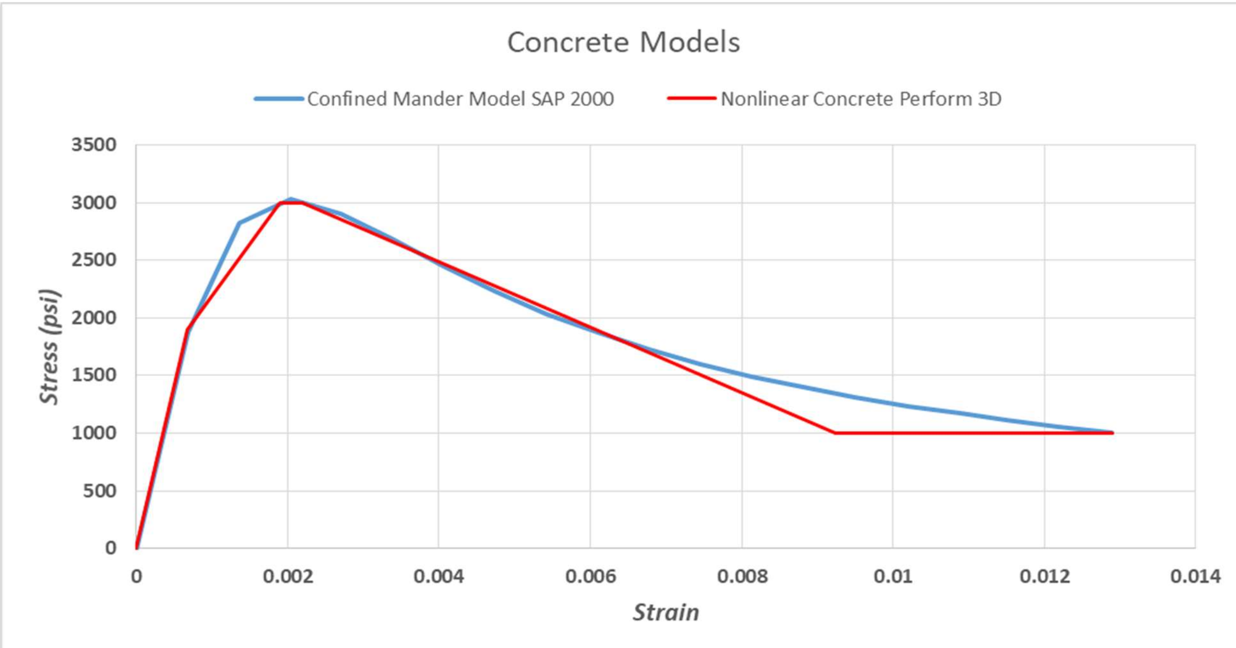


Fig 3. 4 Confined Mander model matched by non-linear concrete model in PERFORM-3D.

In order to represent reinforcements, buckling and non-buckling response of the reinforcements can be represented PERFORM-3D. For current study, non-buckling model of steel was employed as ductility design is primarily dependent on assertion that steel reinforcement can't be fragile or buckle abruptly. The representative model employed in PERFORM-3D is shown in Figure 3.5.Anticipated material's strength used in the study was based on the provisions given by ASCE 41. All the live loads, material strengths, and infill-partitioning loads, which were employed in the analysis in line with the expert consultations conducted during the on ground surveys, are presented in Table 3.3.

Table 3.3: Loading Values and Material Strengths used during modelling

Materials			**Loads**		
Characteristics	**Strength**	**Units**	**Name**	**Value**	**Units**
Concrete's Compressive Strength *(fc')*	3,000	Psi	Live-Load	55	Psf
Reinforcement's yielding stress *(fy)*	40,000	Psi	Infill-Partition	0.3	k/ft

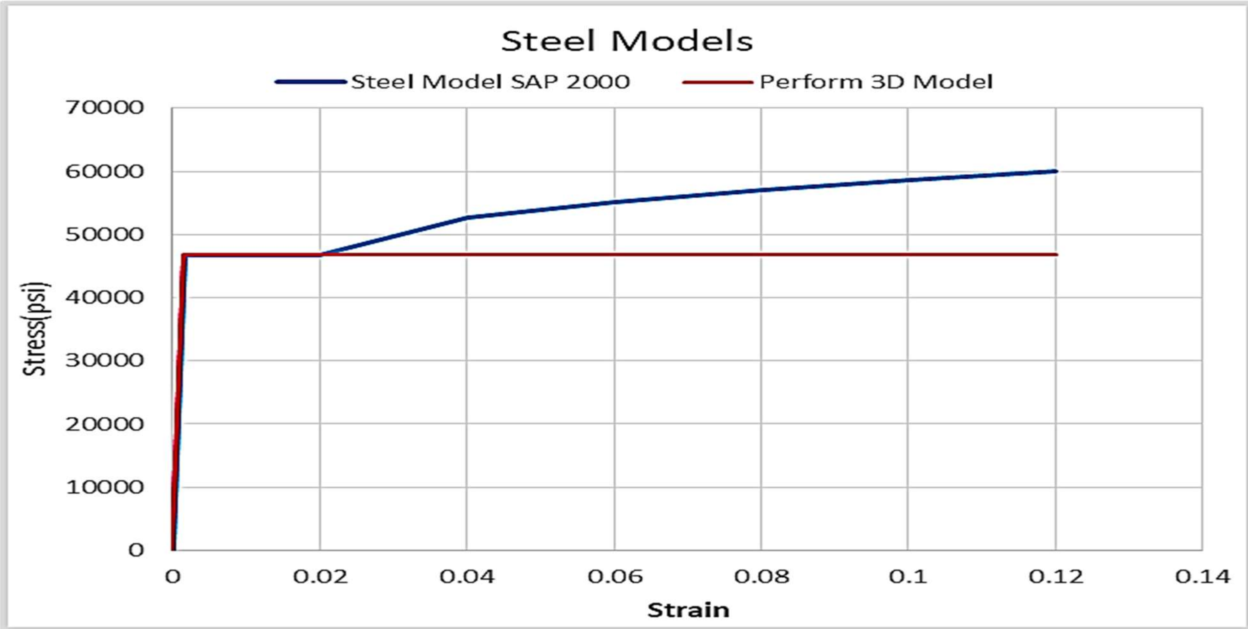


Fig 3. 5 Representative reinforcement model in PERFORM-3D

3.2.1.4. EQUIVALENT DIAGONAL COMPRESSIVE STRUT MODELLING FOR MIRCF:

As per recommendations given by (FEMA 356 2012), masonry infill can be non-linearly modelled as Equivalent-Diagonal compressive strut. The stiffness of the diagonal strut is dependent on its width ‘a’, which represented by equation given below. The recommendations also suggested that the modulus of elasticity and thickness of the strut should be identical as that masonry infill panel

$$a = 0.175 \left(\lambda_1 h_{col}\right)^{-0.4} r_{inf} \qquad \textit{Eq. 3. 1}$$

where:

$$\lambda_1 = \left[\frac{E_{me} t_{\text{inf}} \sin 2\theta}{4 E_{fe} I_{col} h_{\text{inf}}}\right]^{\frac{1}{4}} \qquad \textit{Eq. 3. 2}$$

h_{inf} = Height of infill panel, in.

I_{col} = Moment of inertia of Column, in^4.

E_{fe} = Expected modulus of elasticity of frame material, ksi.

E_{me} = Expected modulus of elasticity of infill material, ksi

t_{inf} = Thickness of infill panel and equivalent strut, in.

θ = Angle whose tangent is the infill height-to-length aspect ratio, radians

h_{col} = Column height between centerlines of beams, in

r_{inf} = Diagonal length of infill panel, in.

λ_1 = Coefficient used to determine equivalent width of infill strut

After calculating the factor 'λ_1', we find values of strut width 'a' depending upon the direction of bays along x-axis and y-axis. Minimum value strut width is selected. Stiffness of strut corresponding to strut width 'a' is calculated from the following equation. The resulting stiffness is then assigned to all struts. The model used to represent the strut is shown in Figure 3.6.

$$k = \frac{a\left(E_{me}\,t_{inf} + E_c\,t_c\right)}{L_{diag}} \qquad \textbf{\textit{Eq. 3. 3}}$$

Where, a = Strut width

E_{me} = Elastic modulus of infill

E_c = Elastic modulus of Concrete infill

t_{inf} = Thickness of infill

t_c = Thickness of concrete

L_{diag} = Length of Diagonal Strut

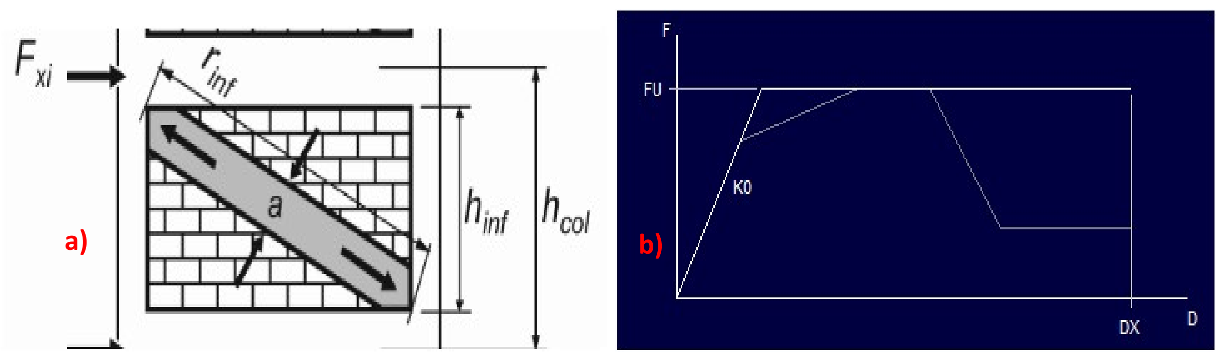


Fig 3. 6 (a) EDC Strut representing infill masonry. (b) Material Model representing EDCS

3.2.1.5. NONLINEAR ANALYSIS

Non-linear analyses can be divided into two main types, namely non-linear dynamic analysis and non-linear static analysis. The current study will mainly employ Nonlinear dynamic analysis, however, non-linear static analysis, also famously known as pushover analysis, is also employed in current work to obtain Capacity curve. The capacity curve, popularly known as force deformation curve, helped us to figure out the quantitative measure of various limit states used in current study.

Uncertainty treatment:

Uncertainties, being part and parcel of Non-linear Dynamic analysis, play crucial role in influencing the response of the structure. There are two main types of uncertainties i.e. aleatory and epistemic uncertainties (Kennedy 1999) . The uncertainty related with the innate inconsistency in the nature of tremor, as consequence of the natural ground motions, is an case of aleatory uncertainty. Therefore, the inherent randomness and inconsistency of ground motions introduce these uncertainties.

The uncertainties rather characterizing the misgivings associated to insufficiency of existing knowledge are termed as epistemic uncertainties. Usually, the small sample size of knowledge about the materials' strengths, during the vulnerability assessments, can be categorized under such uncertainty. However, (Zain 2017) has presented that disparity in material characteristics does not bring any major inconsistency in terms of structural response and only variance in the time histories essentially be deliberated as the major factor in stimulating the structure's dynamic response, as variation in each time history may induce critical changes in response of any building.

In this research application of 15 ground motions was used to compensate for wide-ranging disparity present in parameters of ground motions. As disparities in the material strengths might not stimulate any major change in structural response, therefore this disparity is not accounted for in the current study. The specifics regarding the ground motions selection process and scaling of these time histories are eloborated in next section.

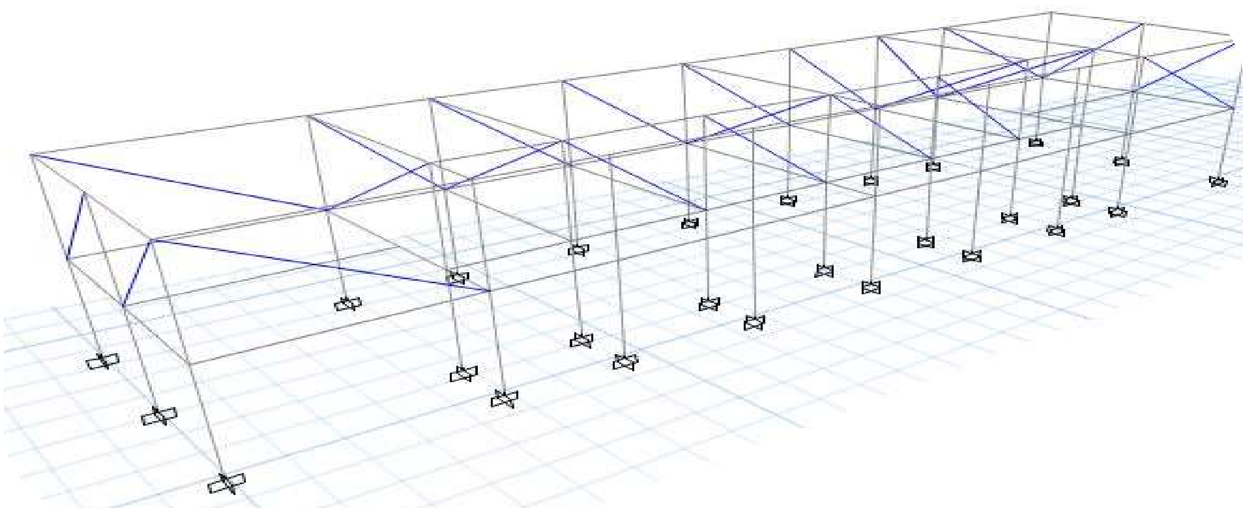


Fig 3. 7 Building Model with Equivalent Diagonal Compressive struts

3.2.1.6. SELECTION OF GROUND MOTIONS:

Path attenuation is one of the major cause of uncertain behavior in seismic demand of the vulnerability assessment process. Path attenuation, features of the instigating source, and the on-ground circumstances, all prove to be pivotal in inducing the uncertainty. A extensive range of seismic energy levels are used for selection of the ground motions in order to assess or catch the correct response. In current study, ground motion or time history records are meticulously selected by considering the magnitude, type of fault, source to site distance, and the shear wave velocity V_s, in the top 30 meters or 100 feet of the soil layers. The most significant fault in the area under study, whose movement resulted in 2005 Kashmir earthquake, is characteristically reverse-oblique, and as apparent in the 2005 Kashmir earthquake, it releases the energy in the form of large magnitude earthquakes. Thus, this research only considers the large magnitude earthquakes i.e. 6.50 up until 8.0, originated from the faults of similar nature.

The epicenter of 2005 Kashmir earthquake was approximately 12 miles away from the center of the district. In the present research, the distance between sources to site has been kept between 0 to 19 miles range to incorporate the effects of path attenuation, so as to take into account any possibility of varied site-specific soil layers in near and far sites. For shear wave velocity V_s in the top 100 feet or 30 meters of the soil, the values have been taken between 575 to 1,150 ft/sec (175 to 350 m/sec) relating to the soil type S_D as per BCP-2007.

The time histories that are selected are presented in the table 3.4, it includes the names of time histories, their magnitudes, shear wave velocities, rupture site distance and related PGA along with some other minor details. The aleatory uncertainty is effectively minimized in the current study by employing numerous records, with numerous magnitudes, PGA, and other characteristics.

3.2.2. VULNERABILITY ASSESSMENT:

3.2.2.1. INCREMENTAL DYNAMIC ANALYSIS (IDA) & RANDOM EFFECTS FROM GROUND EXCITATION:

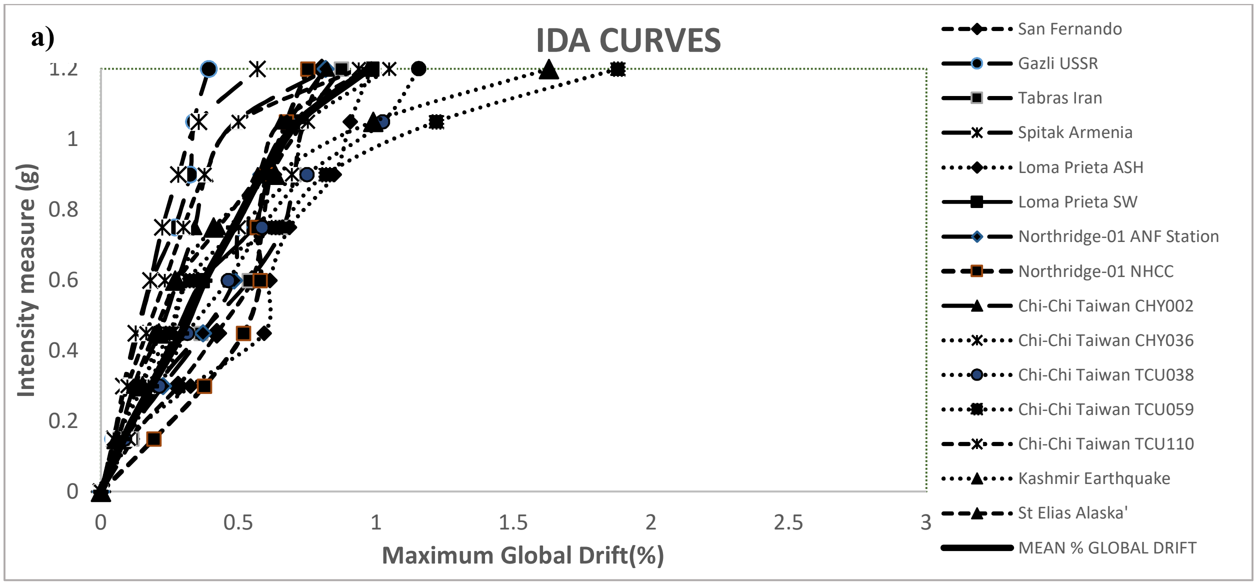
Incremental-Dynamic Analysis or IDA functions as exceptionally consistent technique to determine the behavioral-performance of structures against ground motions (Soleimani et al. 2018). Several researchers i.e. (Fereshtehnejad et al. 2016; Kostinakis and Athanatopoulou 2016; Zarfam and Mofid 2011) implemented this procedure for assessment of non-linear dynamic responses of various structures and although of it requires cumbersome computational effort, Incremental Dynamic Analysis has been demonstrated as an impressive instrumentation technique for vulnerability and seismic risk assessments. In IDA, the time histories are scaled in such a way that the accelerations increments at specific proposed intervals.

Table 3.4: Selected natural ground motions (Source: PEER Strong ground motion database)

Sr. No.	Earthquake Name	Year	Station Name	Mag.	Mechanism	PGA (g)	Rrup (miles)	Vs30 (ft./sec)
1	San Fernando	1971	LA - Hollywood Stor FF	6.61	Reverse	0.225	14.15	1038.25
2	Gazli USSR	1976	Karakyr	6.8	Reverse	0.864	3.40	851.67
3	Tabas Iran	1978	Boshrooyeh	7.35	Reverse	0.106	17.89	1064.86
4	Spitak Armenia	1988	Gukasian	6.77	Reverse Oblique	0.20	14.91	1127.07
5	Loma Prieta	1989	Agnews State Hospital	6.93	Reverse Oblique	0.1695	15.27	786.38
6	Loma Prieta	1989	Saratoga - W Valley Coll.	6.93	Reverse Oblique	0.331	5.78	1141.4
7	Northridge-01	1994	Arleta - Nordhoff Fire Station	6.69	Reverse	0.345	5.38	976.74
8	Northridge-01	1994	N Hollywood - Coldwater Can	6.69	Reverse	0.309	7.77	1071.1
9	Chi-Chi Taiwan	1999	CHY002	7.62	Reverse Oblique	0.137	15.51	771.42
10	Chi-Chi Taiwan	1999	CHY036	7.62	Reverse Oblique	0.273	9.97	764.90
11	Chi-Chi Taiwan	1999	TCU038	7.62	Reverse Oblique	0.1448	15.80	977.23
12	Chi-Chi Taiwan	1999	TCU059	7.62	Reverse Oblique	0.165	10.63	894.59
13	Chi-Chi Taiwan	1999	TCU110	7.62	Reverse Oblique	0.1918	7.19	697.90
14	Kashmir Earthquake	1999	ABD-Abbottabad	7.60	Reverse Oblique	0.2517	16.16	731.76
15	St Elias Alaska	1979	Icy Bay	7.54	Reverse	0.1759	16.44	1005.15

The present study employs the Incremental Dynamic Analysis for incorporating numerous levels of seismic intensity to forecast the response of structure against earthquakes, extending between 0.15g to 1.2g with 0.15g as the increment in acceleration for each successive iteration. Figure 3.8 demonstrates the results of IDA by plotting between Percentage Global Drift and the incrementally scaled PGA from 0.15g to 1.2g**.** It can be seen form the Figure 3.8 that inherent behavior of different time histories produce different results although having the same acceleration intensity at a particular position. The results are also produced and presented in the same figure by scaling the S_a - spectral acceleration - at the fundamental or primary time period of the building for the identical intensity levels.

It is clearly obvious from the results that characteristic properties of time histories show considerable sensitivity to the seismic requirement which are principally attributed to variation in their energy distribution in terms of frequency content, which primarily effects the response of the structure.



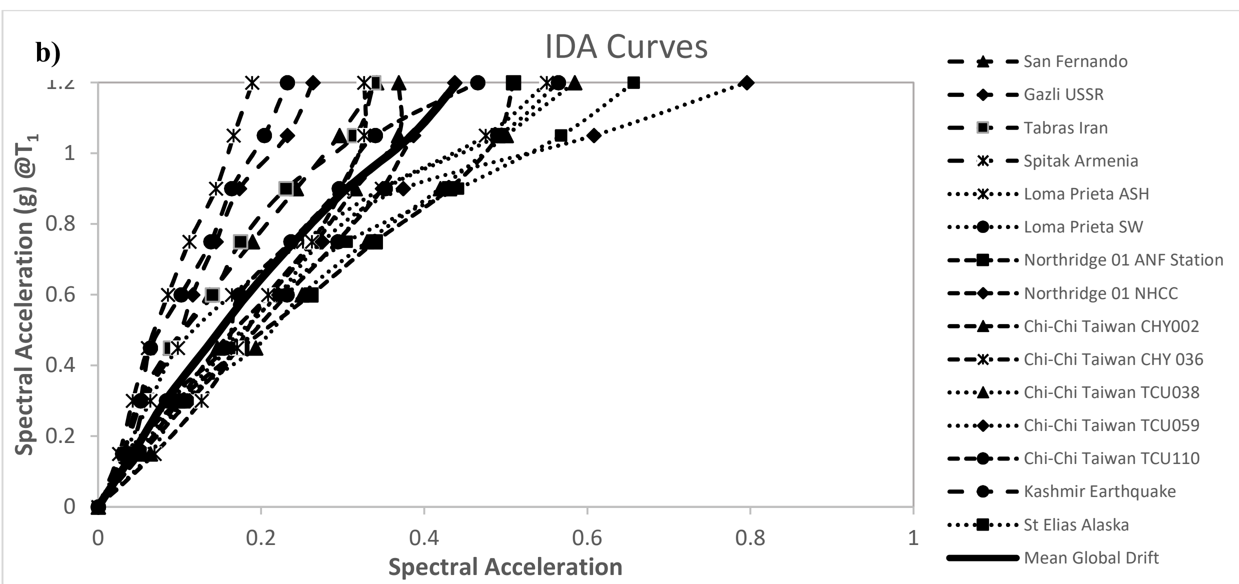


Fig 3. 8 IDA plots for** a) **Max Global Drift versus PGA (g) b) ***Max Global Drift versus S_a (g) scaled @ T_1***

3.2.2.2. HAZARD – DAMAGE RELATIONSHIP

Fundamentally, procedure of risk assessment in response to a hazard generally consists of two main sub-divisions, i.e. the hazard and the vulnerability of structure. For earthquakes, former part is principally related with seismologists, who can offer a representative assessment of the impending seismic hazard that may occur in a particular geographical area; whereas the later part, the vulnerability, is typically tackled by the experts and the engineers of relevant field, who carry out the design the structural amenities for variety of reasons. The present research is concerned with the vulnerability portion of risk assessment, and evaluates the damageability of the selected classification of school buildings by creating the "hazard versus damage" relationships, and successively, by generating the basic fragility relationships. "Hazard versus damage" relationships illustrates performance of the structure against earthquake tremors and demonstrate the dynamic response for all selected ground motions. Figure 3.9 shows a direct relation-ship between the seismic intensity measure and the structural damage. The damage to the structure is computed in terms of the percentage global response, and intensities are taken incrementally in PGA, as well as in terms of S_a at T_1, the fundamental time period of the structure.

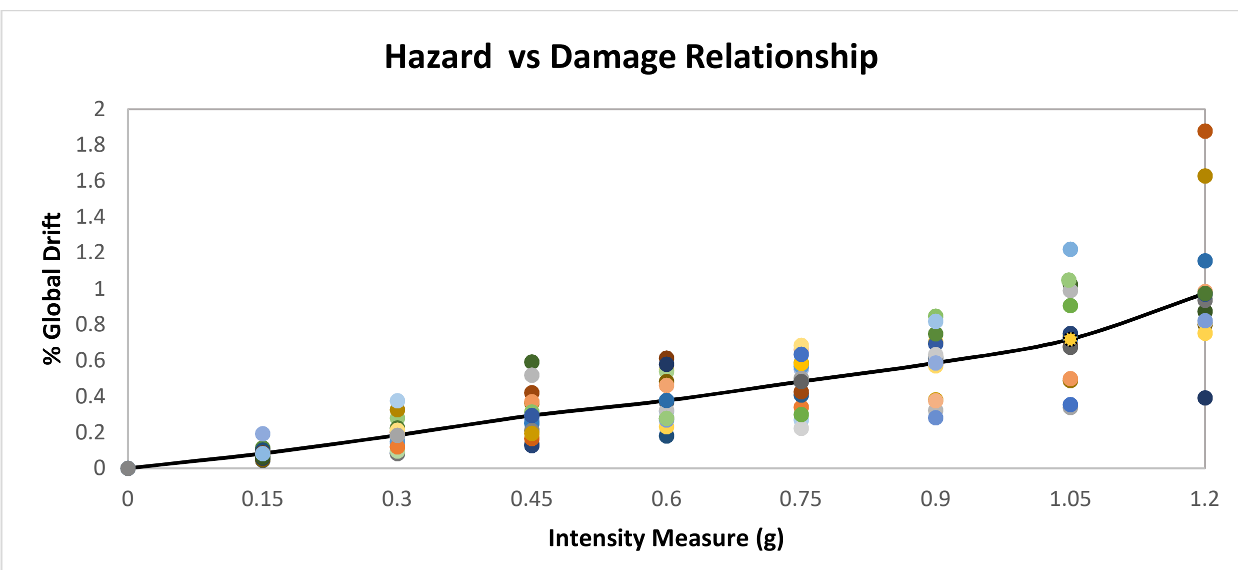


Fig 3. 9 Hazard versus Damage Relationships: Intensity Measure v/s percentage Global Drift

Every single point in hazard-damage relationships signify the global response of structure against each ground motion at each intensity. The extreme points, may also be called outliers, depicts the maximum and minimum percentage global drift of structure. The response is independent of the earthquake as the outliers are not of the same earthquake for each intensity. This behavior give important information about response of building to tremors. The response can be understood in a sense that building's global response stimulates to a particular intensity earthquake. The stimulation of this response is associated to the time period of the building. When the time period of any earthquake is close or equal to the fundamental time period of the structure or its multiples, the global response amplifies. The solid line in Figure 3.9 depicts the mean response of the structure, assessed at each intensity level, which shows that generally with the increase in intensity the global drift increases.

Vulnerability assessment involves development of fragility curves using the pre-processed data. Fragility curves, defined by equation (4), characterizes a conditional probability of occurrence, also synonymously termed as probability of exceedance, for particular limit or damage states corresponding to a specific seismic intensity measured using PGA, S_a, or any other seismic intensity indicator.

$$\boldsymbol{P_{fragility} = P[LS|IM = x]} \qquad \textbf{\textit{Eq. 3. 4}}$$

Eq 3.4 describes the probability of achieving a damage state at a specific Intensity Measure (IM), is equals to "x". The relationships for fragility of various limit states should be capable to predict the vulnerability of the structure against a given range of time histories. In order to achieve a likely probabilistic estimation large disparity in seismic demands needs to be taken into account so that sufficient variation in the structure's dynamic response is achieved. The approximation of vulnerability requires the establishment of particular damage states, indicators of damage, and seismic intensity measures. The forthcoming articles discourses the fundamental steps to generate fragility curves. Subsequently, the fragility curves are explicated for the considered classification.

3.2.2.3.. DAMAGE INDICATOR & THE ESTABLISHMENT OF LIMIT SATES

With the aim to derive analytical fragility curves characterization of particular damage states both qualitatively and quantitatively is required. In order to achieve this purpose, researchers have to use a specific limit/damage parameter or an indicator generally identified as *Engineering Demand Parameter (EDP)*, or a new damage measuring parameter that can characterize the damage to the structure in a more coherent way to compare it with that of earthquake loading may also be recommended. When the damage-indicator is selected, various damage states, also identified as the limit states for a given structure may be then defined (Zain et al. 2019).

The physical characterization of damages is represented by damage states which utilizes damage indices. Damage indices proposes onset limits for various limit states by depending on the factors of Structural performance. These or factors of structural performance can be Energy dissipation proficiency, strength, ductility etc. of the structure. Damage indices are used for relating the amassed or accumulated damage with particular damage states. Numerous dissimilar damage indices may exist for the same structure. For example, (Crowley et al. 2004) considered damage or limit states on the basis of strain levels in the steel and concrete. (Esra Mete Güneyisi and Gülay Altay 2008) deliberated the inter-story drift ratio in order to represent the response parameter corresponding to the four damage states, taken from the HAZUZ. (Casotto 2013) considered the member flexural strength to describe the first limit state i.e. the point at which steel reinforcement yields in columns, and 3% inter-story drift for collapse in flexure as second damage state and successively, correlated his second damage state with the support loss of the beam. (Chaulagain et al. 2016; Gautam et al. 2018) used 3 damage states ranging from slight damage to full collapse and employed empirical fragility relation-ships for structures in Nepal. They subsequently, linked their results with FEMA356 (Applied Technology Council 1997) classifications for building damage levels. Since, the current is study principally related with the development of fragility curves for a particular building class, it will not be representative enough to describe damage states based on the member's behavior or localized strains. Therefore, in present study, the percentage global drift is taken as the limit state indicator to co-relate it with the damage to the structure.

Qualitatively 3 main damage states were defined in this study, which are as under

a. LS1 as Serviceability damage state.

b. LS2 as Damage Control damage state.

c. LS3 as Collapse Prevention damage state.

Serviceability limit state or LS1 corresponds to the 1st yield of the main reinforcement in the building (Zain et al. 2019). Normally , LS1 or the first limit states are usually taken based upon the first hinge formation in the structure but as the current study involves nonlinear fiber modelling of the structure therefore, LS1 is taken based on yielding strain of structural element, found anywhere in the structure (FEMA 356 2012). Damage control limit state or LS2 is dependent on deformation and strength. It is established as 75% of LS3. Collapse Prevention or LS3, is conventionally governed by deformation (FEMA 356 2012). (Erberik 2008) defined the Collapse Prevention limit state as lower of the values for 75% of ultimate structural deformation or for which there is more than 20% drop in strength relative to maximum strength value.

Values calculated for the limit states in current study was not taken from the previous studies on analytical fragility as there exists no contemporary study for buildings in high seismic zone (Zone-IV) of Pakistan. In this particular study, the values calculated are given to all the limit state by employing the pushover curve of the building under consideration. The capacity or load deformation curve had been obtained through nonlinear static analysis. Table 3.5 depicts the values of each limit state in terms of percentage global drift. The capacity curve shown in Figure 3.10 also depicts the established limit state.

Table 3.5: Damage state values for selected building class i.e. BLR-14

Building/Model Name	Damage/Limit State		
	Serviceability (LS1)	Damage Control (LS2)	Collapse Prevention (LS3)
BLR-14	0.351	0.6285	0.838

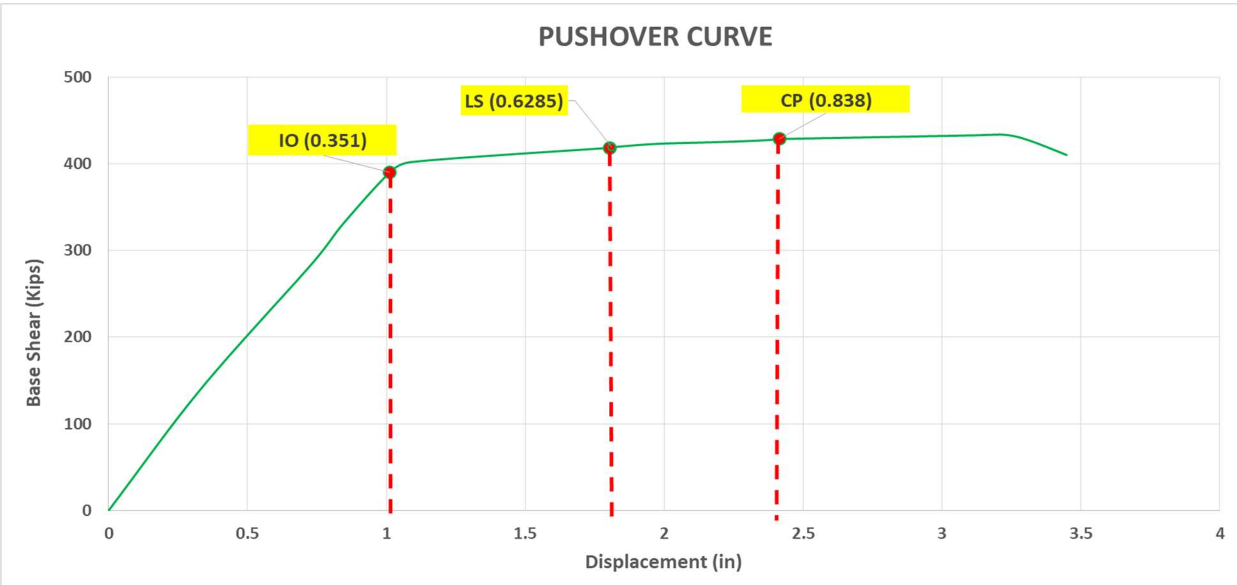


Fig 3. 10 Pushover Curve showing Limit States (IO, LS and CP)

3.2.2.4. SELECTION OF SEISMIC INTENSITY MEASURE

In order to depict the structure's dynamic response by using damage indicator, Seismic intensity measure (IM) is required to correlate the ground motion with the structural damage. PGA is one of the most vastly used Intensity measure by the professionals for fragility derivations. However, it is generally comprehended that Spectral Acceleration (S_a) is a better option as an IM to relate with the damage of the structure as it truly exemplifies the influence of ground trembling on the structure itself. (Jiang et al. 2015; Pang and Wu 2018) employed PGA for their researches. Besides the PGA, (Bojórquez and Lozoya 2009; Omine et al. 2008) used Peak Ground Velocity (PGV) as an Intensity Measure. (Bojórquez and Lozoya 2009) also employed Spectral Acceleration (S_a) at the fundamental period of structure as an Intensity Measure. (Choudhury and B Kaushik n.d.; Frankie et al. 2012) used S_d , Spectral Displacement, as Intensity measure for research on open ground story Reinforced-Concrete frames, with openings in walls in order to carry their vulnerability assessment. Some vector valued intensity measures have also been suggested by (Baker and Cornell 2005, 2006; Kohrangi et al. 2016; Tothong and Allin Cornell 2007)**.** (Pejovic and Jankovic 2015) examined variety of ground motion's Intensity Measures that are usually employed to examine the dynamic response and inferred that despite being the most commonly used Intensity Measure, PGA showed the highest diffusion in results when compared with PGV, S_a (T1), S_v (T1), and S_d (T1). Their research concluded that Spectral Response Velocity S_v (T1) as an effective intensity measure for the Reinforced Concrete frames, while also asserting, that the other two spectral-response parameters, i.e. S_a @ T_1 and S_d @ T_1 were reasonably suitable as they explained the behavior by integrating the dynamic characteristics of structures under consideration. This research employs both Spectral-acceleration at the fundamental time period (S_a @ T1) of the structure and PGA to be used as an intensity measures. As specified by the prior literature, PGA is one of the most frequently employed intensity measure because it is relatively understandable for common people with no explicit engineering education, while Sa @ T1 includes the characteristics of structure during the implementation of analysis.

3.2.2.5. FRAGILITY ASSESSMENT OF THE SCHOOL BUILDINGS

The fragility curves are extensively used to achieve a detailed understanding of the structural-performance of the structure against earthquake loading. After obtaining the structural response data, already established limit state definitions are applied for assessing the performance for each damage state. Only if the calculated value is bigger than a damage state numeric value, an occurrence will be counted in the sample's tally count, encompassing of all the time histories scales. In this manner, for each Intensity measure i.e PGA and S_a direct sampling probabilities over the 15 selected ground motions on all incrementing scales were obtained in the form of points.

It is a frequently acknowledged postulation that a log-normal cumulative distribution function may be fittingly employed for the manifestation of conditional-probabilities for deriving a fragility curve, (Dang et al. 2017; Shinozuka et al. 2002a; b; Zain et al. 2019). From standpoint of literature, regression analysis of fragility relationships was done using lognormal distribution, as is mentioned in equation 3.5;

$$\boldsymbol{P(LS|IM) = \phi\left(\frac{lnIM - ln\lambda_c}{\beta_c}\right)} \qquad \textbf{\textit{Eq. 3. 5}}$$

where $P(LS|IM)$ defines the probability of occurrence or exceeding from a specific LS, for a given Intensity Measure value. 'Φ ( )' denotes CDF or standard normal cumulative distribution function. The governing parameters for this log-normal distribution function is "median-point" (λ_c) and standard deviation (β_c). It is essential to estimate the optimized values of these parameters in order to get a best approximation curve of CDF against the already estimated probabilities. (Baker and Eeri 2015; Dang et al. 2017; Shinozuka et al. 2002a; b; Zain et al. 2019) stated that Maximum Likelihood Method (MLM) as an appropriate technique to attain most likely values of the 2 factors. In this particular study, Maximum Likelihood Method is used to calculate the median-point and the standard deviation of the obtained fragility curve. The likelihood function for more than one Intensity measure levels, as deliberated in the current research, can be expressed as follows in equation 3.6;

$$\boldsymbol{Likelihood = \prod_{j=1}^{m} \binom{n_j}{z_j} \phi\left(\frac{ln(IM_i/\lambda_c)}{\beta_c}\right)^{z_j} \left(1 - \phi\left(\frac{ln(IM_i/\lambda_c)}{\beta_c}\right)\right)^{n_j - z_j}} \qquad \textbf{\textit{Eq. 3. 6}}$$

where 'm' is the number of Intensity measure levels, and 'Π' represents a product of every level. It is presumed that event of reaching a specific damage state from each time history is not dependent on the events from other ground motions. In equation 3.6, 'z_j' denotes the number of attaining certain limit state out of 'n_j' ground motions with particular value of IM = IM_i. By maximizing the likelihood function we can achieve the fragility curves parameters by equation 3.7.

$$\{\lambda'_c, \beta'_c\} = arg\,max_{\lambda_c,\beta_c} \sum_{j=1}^{m} \left\{ ln \binom{n_j}{z_j} + z_j \, ln\phi \left(\frac{ln(IM_i/\lambda_c)}{\beta_c} \right) + (n_j - z_j) ln \left(1 - \phi \left(\frac{ln(IM_i/\lambda_c)}{\beta_c} \right) \right) \right\} \qquad \textit{Eq. 3. 7}$$

Table 3.6: Log-normal distribution parameters for fragility calculation against each IM i.e. PGA and S_a @ T1 for selected typology.

Building Typology	Fragility Parameter	Serviceability (LS1)		Damage Control (LS2)		Collapse Prevention (LS3)	
		PGA	**Sa @ T1**	**PGA**	**Sa @ T1**	**PGA**	**Sa @ T1**
RCF	λ_c	0.5865	0.8275	0.8862	1.1052	1.1535	1.2394
	β_c	0.4073	0.2410	0.3352	0.2200	0.1568	0.2119
MIRCF	λ_c	0.7568	0.9920	1.1370	1.2010	1.2194	1.3060
	β_c	0.4556	0.3210	0.3232	0.2247	0.1867	0.2340

The lognormal distribution parameters for fragility obtained from Eq 3.6 and 3.7 are tabulated in Table 3.6. The plots and results of fragility curves for vulnerability assessment are given in next chapter where a detailed explanation on fragility curves for both RCF and MIRCF will also be done.

CHAPTER 4

RESULTS & DISCUSSION

4.1. INTRODUCTION TO CHAPTER

In this chapter results obtained by following methodology mentioned in previous chapter will be presented. Application of IDA on nonlinear models, subsequent formation of Fragility curves for both types of structural configurations and their performance comparison with regards to statistical probability will be discussed in this chapter. The crux of the discussion will be delegated for next chapter in which conclusions drawn from these results will be elaborated.

4.2. RESULTS FROM METHODOLOGY

To carry out comparative analysis for fragility between MIRCF and RCF school frame building in high seismic zone of Pakistan, fragility curves were obtained through methodology mentioned in previous chapter. Nonlinear models for both MIRCF and RCF were developed using PERFORM 3D. The model for MIRCF was similar in all aspect to RCF except that the masonry infill, which had to be incorporated in MIRCF, was modelled as concrete compressive strut as per recommendations given in FEMA 356 (Chapter 7). In this chapter main results from each component of methodology will be presented.

4.2.1. RESULTS FROM IDA

IDA Curves

IDA curves developed by applying selected ground motions on nonlinear building models. The building models were developed using PERFORM-3D software. The software package produced results in tabulated form. Thus, it had to be exported to MS EXCEL to prepare graphical manifestation of tabulated results. The IDA results for engineering demand parameter (percentage maximum global drift) versus seismic intensity measures i.e PGA and S_a at Fundamental time period are shown in figures below.

Hazard- Damage Relationships

Hazard-Damage relationships were also developed in pre-processing component by plotting every maximum percentage global drift value against seismic intensity measure for all selected 15 ground motions. Figures below show Hazard-Damage relationships which describes increase in Hazard and Damage as the Seismic intensity increases. The hazard or damage is quantified in terms of Percentage Global Drift.

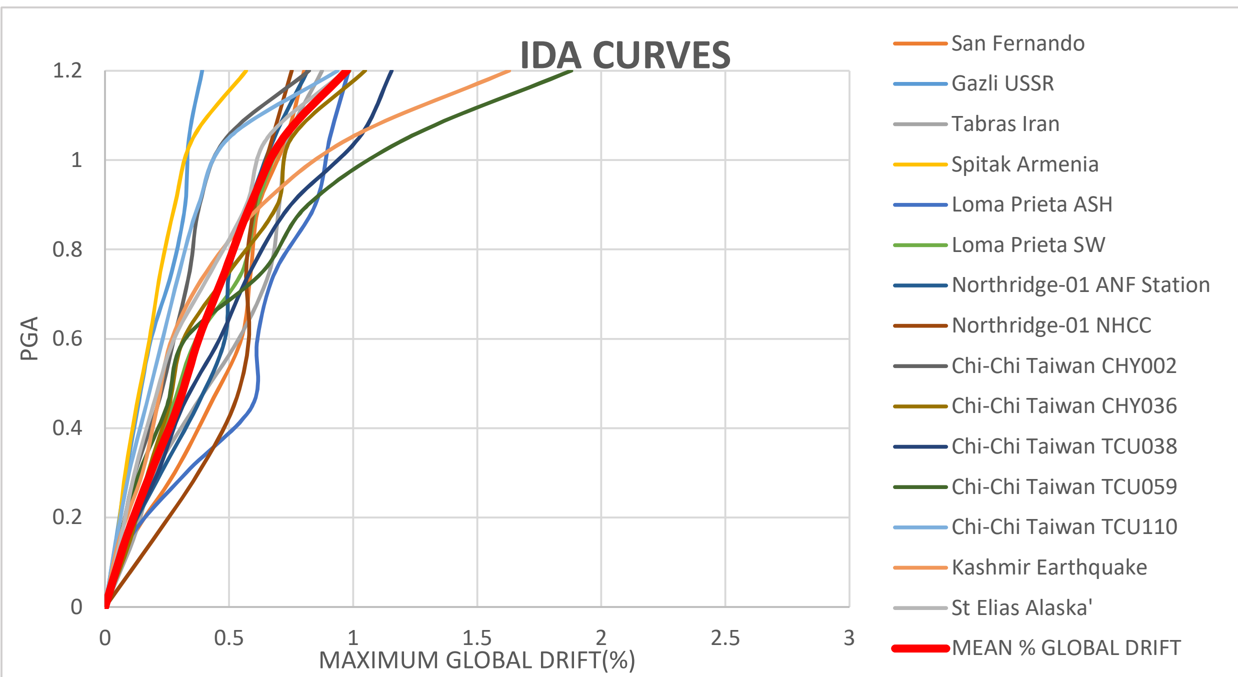


Fig 4. 1. IDA Curves for Max Global Drift (%) vs Peak Ground Acceleration

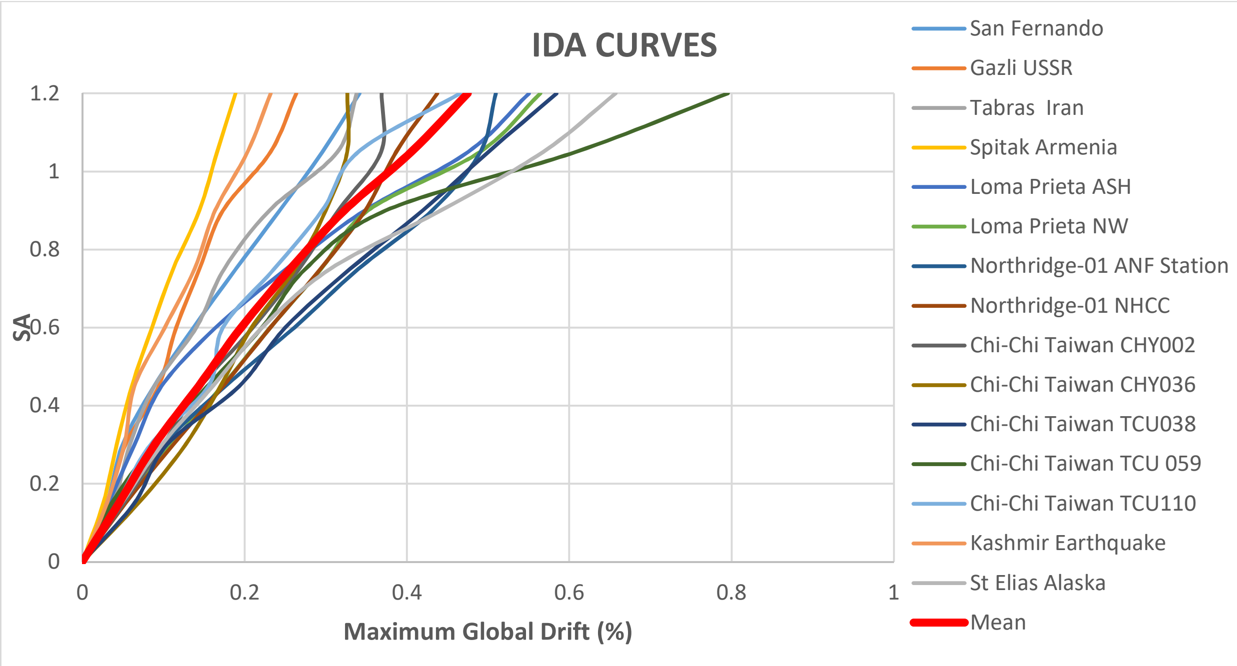


Fig 4. 2. IDA Curves for Max Global Drift(%) vs Spectral Acceleration

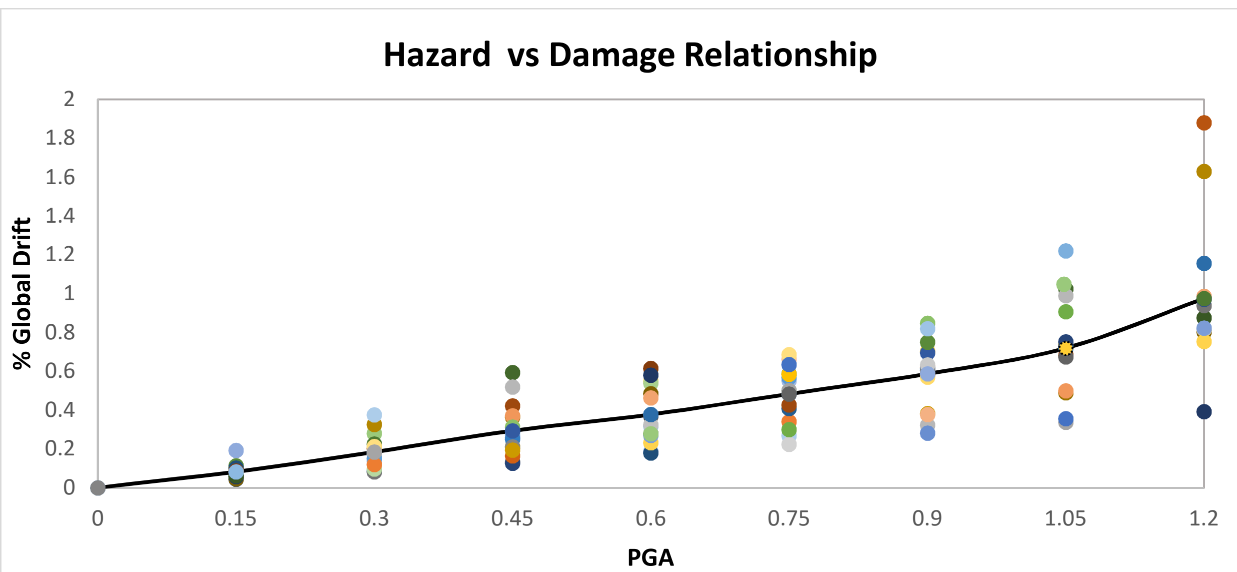


Fig 4. 3: Hazard versus Damage Relationships: PGA v/s Global Drift in Percentage

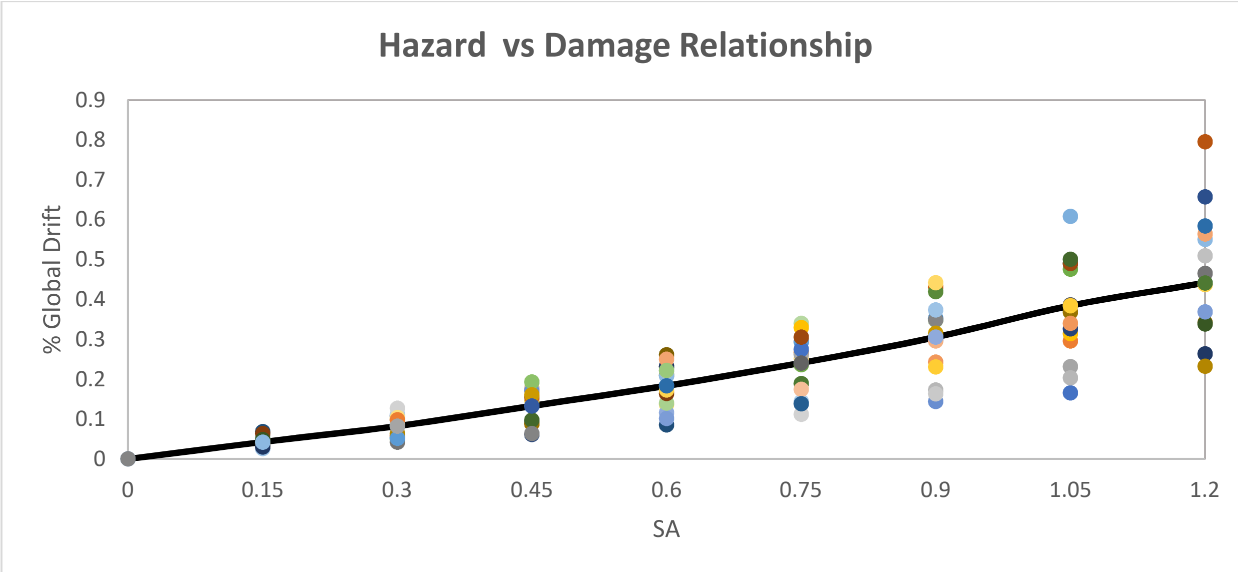


Fig 4. 4: Hazard versus Damage Relationships: Spectral Acceleration v/s Global Drift in Percentage

4.2.2. RESULTS FROM VULNERABILITY ASSESSMENT

Vulnerability Assessment involved development or organization of IDA results into fragility curves. This process involved calculation of discrete probability of a structure past a specific limit state and then by means of log normal distribution to obtain fragility curves. This can be achieved by using Conditional Formatting in MS Excel to evaluate the probability of exceedance for a structure against specific damage state for each time history. After obtaining the structural response, established limit state definitions are applied for computing the performance of each damage state. The sampling probabilities calculated for various limit states are shown as follow:

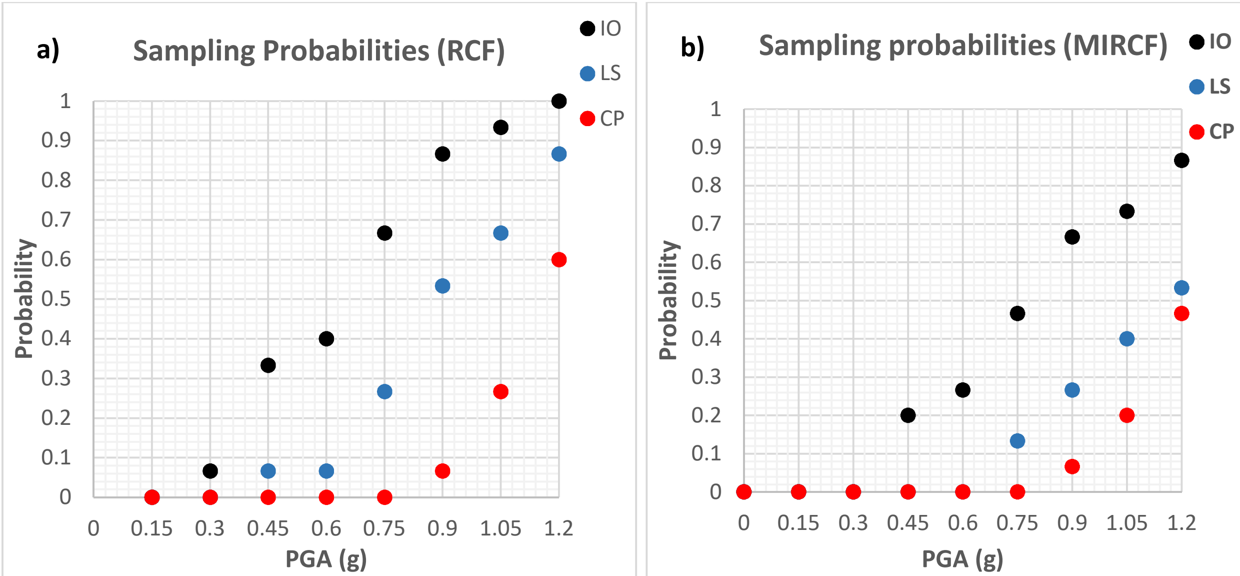


Fig 4. 5: Sampling Probabilities for a) RCF and b) MIRCF against Peakground acceleration as an intensity measure.

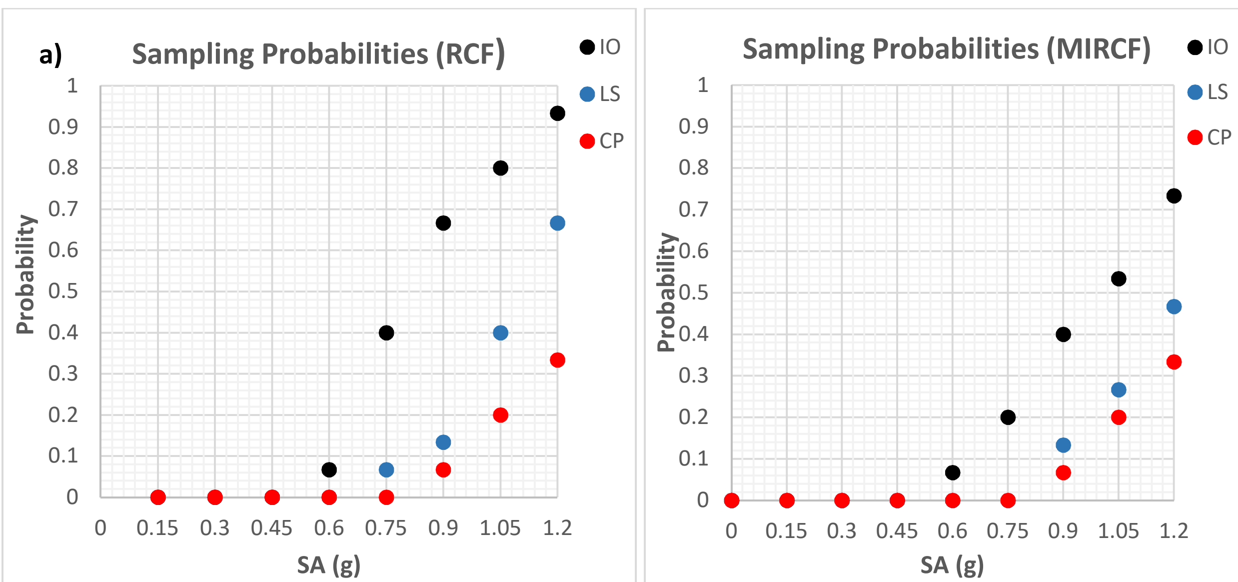


Fig 4. 6: Sampling Probabilities for a) RCF and b) MIRCF against Spectral acceleration as an intensity measure.

4.2.3. THE FRAGILITY CURVES

Given below are the fragility curves for MIRCF and RCF .Probability of all three limit states are shown against given Intensity measure. Both S_a and PGA are used as intensity measures. In the figure the plotted points show the direct probability of exceedance against particular Intensity measure. The solid lines represent Log normal distribution function for each limit state. Log normal distribution function is employed to obtain a continuous curve in order to understand fragility response in a better way. Figure 4.7 shows the established fragility curves for the selected class of RC structures in high seismic-zone of Pakistan. Discrete probabilities, acquired from the IDA, are also plotted in the figure along with log-normally regressed fragility curves.

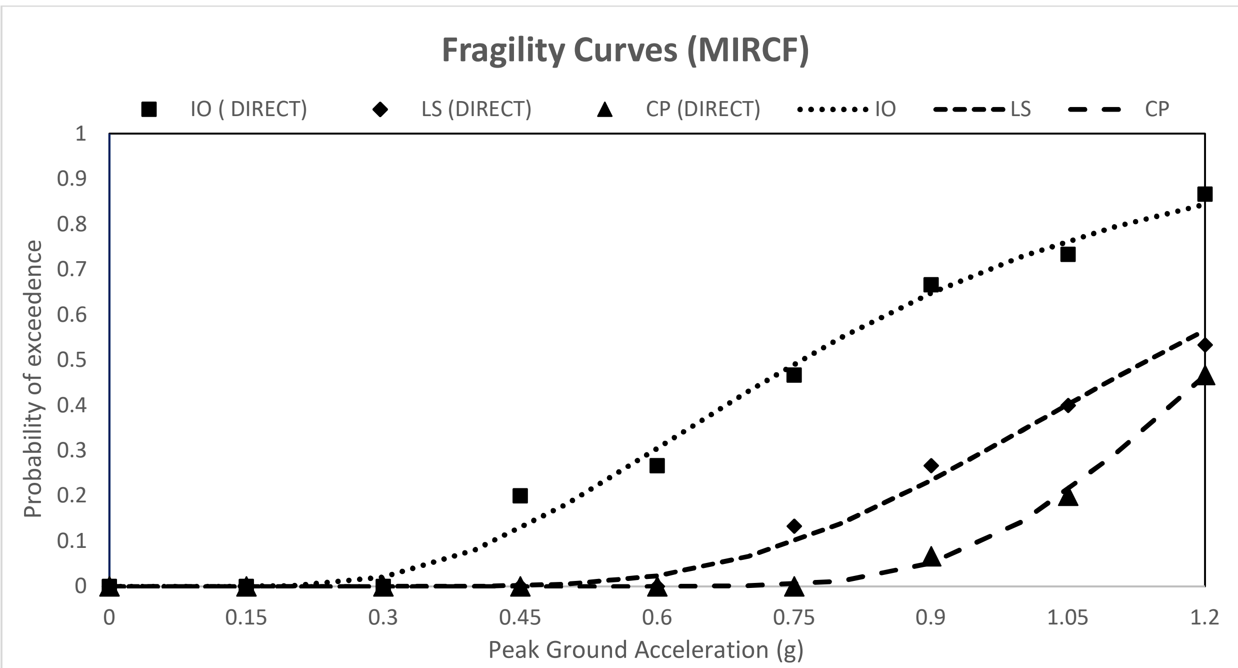


Fig 4. 7: Fragility curves showing probability of exceedence of a MIRCF against PGA of varying intensities. Dots showing the direct probability while the curve shows the log-normal distribution.

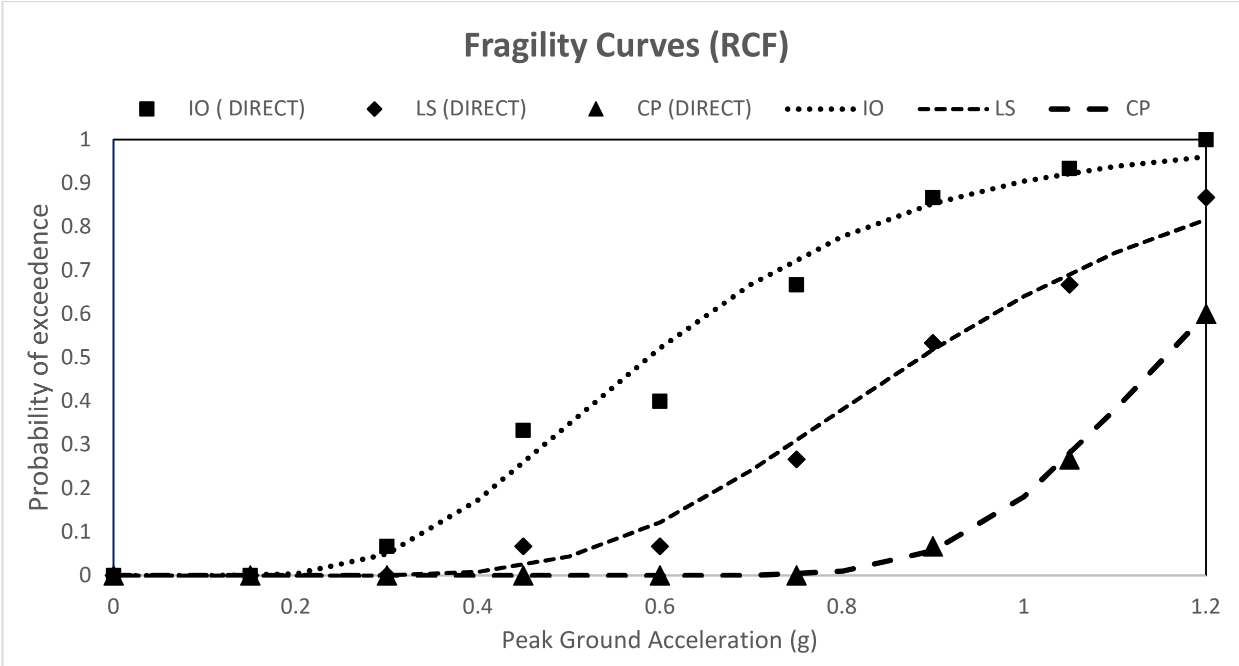


***Fig 4. 8: Fragility curves showing probability of exceedence of a RCF against PGA of varying intensities. Dots showing the direct probability while the curve shows the log-normal distribution*.**

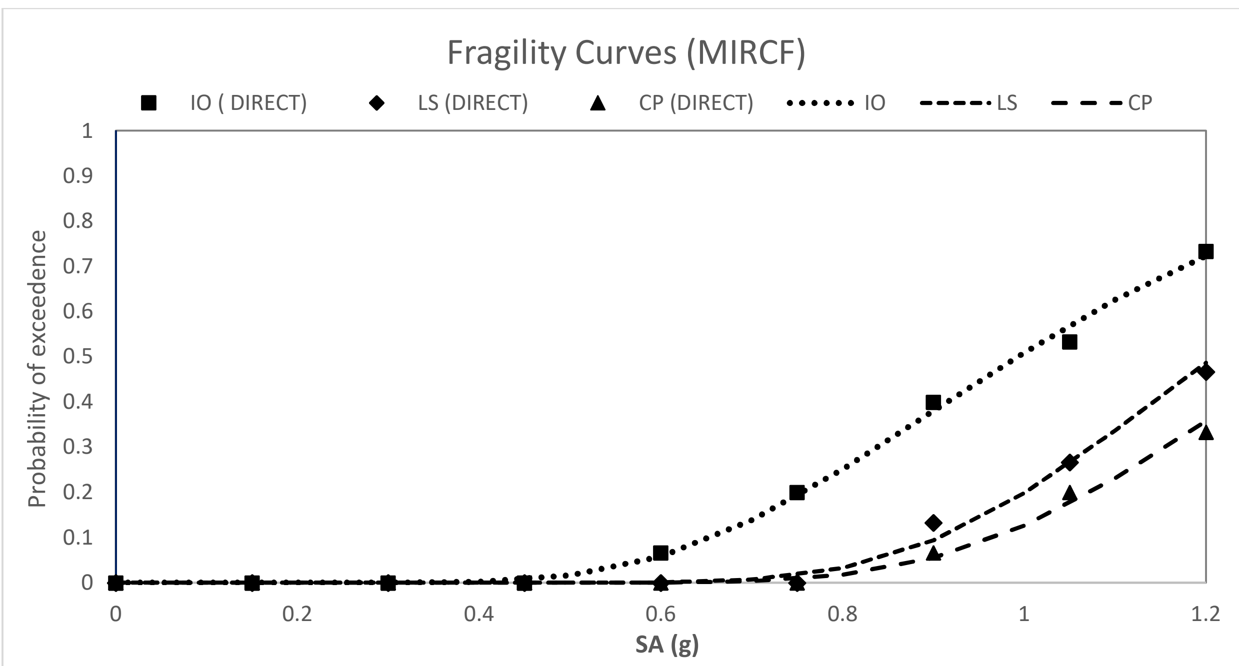


Fig 4. 9: Fragility curves showing probability of exceedence of a MIRCF against SA of varying intensities. Dots showing the direct probability while the curve shows the log-normal distribution.

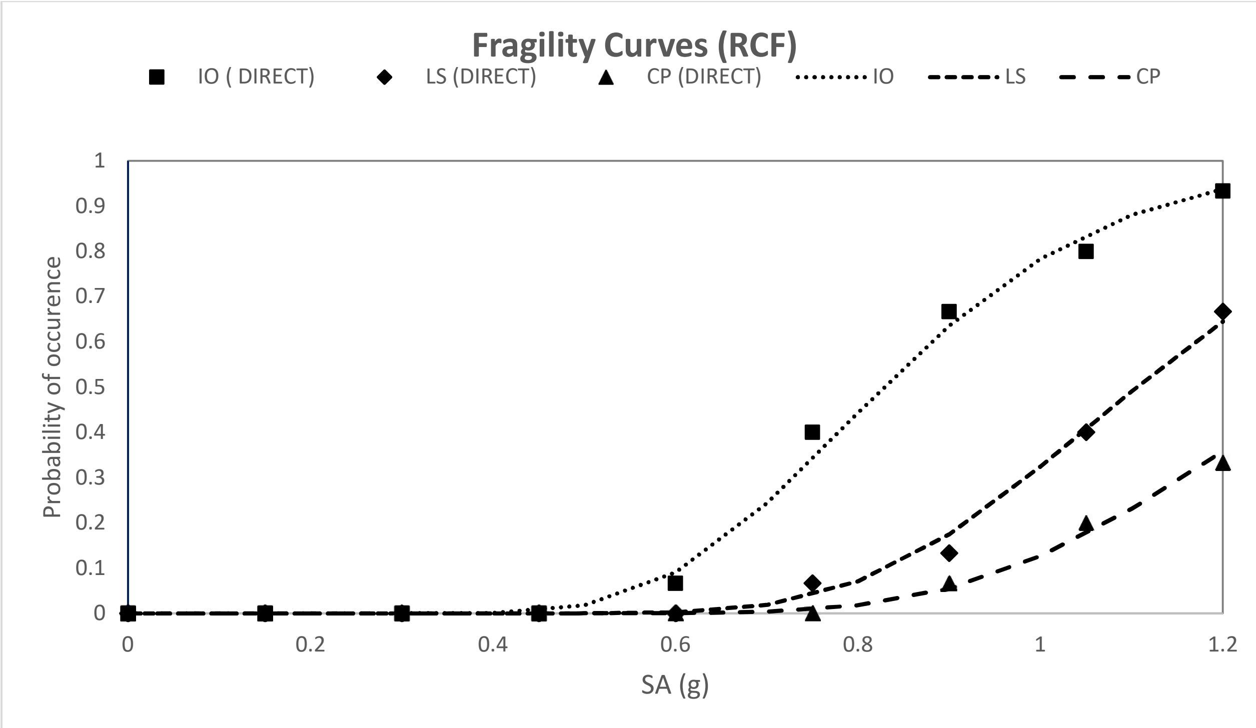


Fig 4. 10: Fragility curves showing probability of exceedence of a RCF against PGA of varying intensities. Dots showing the direct probability while the curve shows the log-normal distribution.

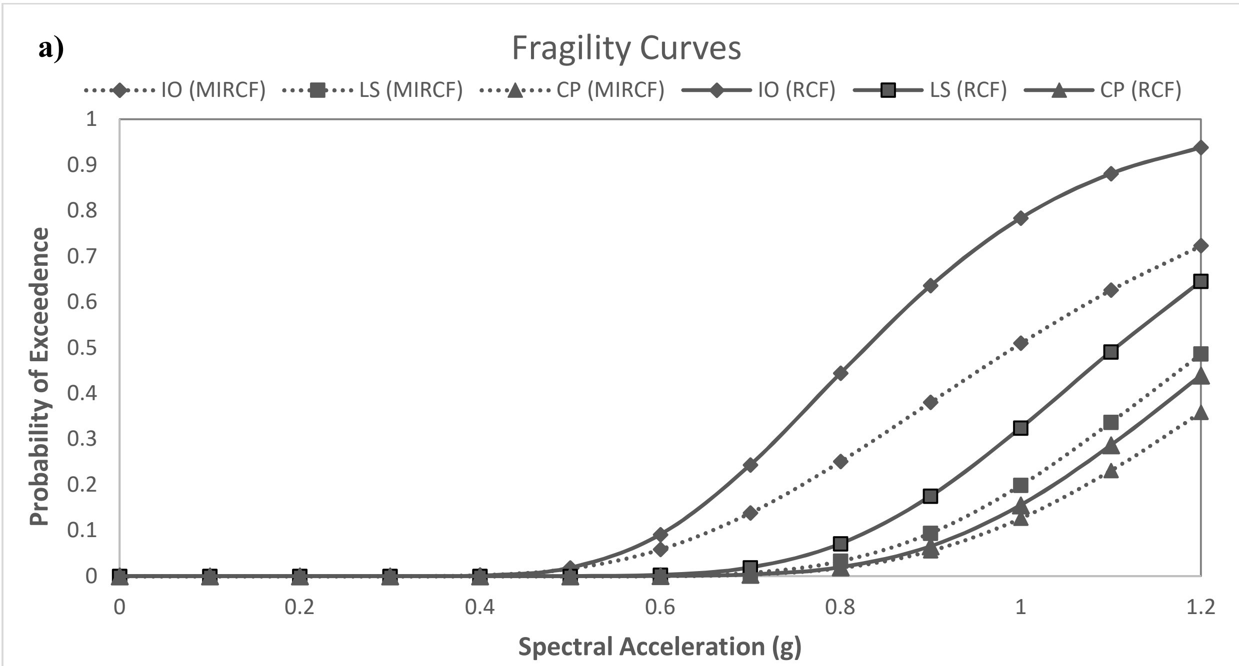
4.3. DISCUSSION

Observing the fragility curves it is evident that the probability of exceedance for each limit state in MIRCF is lower than that of RCF against given intensity measures i.e. both PGA and Spectral Acceleration (S_a). This behavioral response can be characterized to the increased stiffness of the whole structure when masonry infill is introduced in the building. This behavior can be better understood in terms of energy dissipation of the building when under tremor or ground motion. In case of RCF, energy imparted to the building is dissipated only to the frame of the structure. However, in case of MIRCF, the masonry infill panels also absorbs some of the dissipated energy initially. This may lead to localized failures although the overall stiffness of the whole structure is improved but the individual stiffness of the infill panel is low relative to surrounding frame. However, it is significant to notice that this variation of increase in stiffness minimizes with increase in the intensity of ground motion. This response is due to the fact that at higher intensity ground motions the lateral stiffness of the masonry infill is compromised in brittle manner and the mass of infill panels, which helped in increasing the overall stiffness in lower intensities, will not contribute to stiffness.

It can be seen from the above figures that when PGA is used as an Intensity Measure, higher probabilities were achieved for each of the limit state for the proposed building typology, but, relatively low conditional probabilities for exceeding limit states were observed for incrementally scaled S_a. This vulnerability behavior is mainly characterized due to the incorporation of structural parameters i.e. stiffness and deformation capacity during the vulnerability assessment process when S_a was used as an Intensity Measure. Moreover, as also indicated by the literature, the current study also states that S_a at fundamental time period portrays a better existing condition of vulnerability in comparison with the PGA. Table 6 offers a statistical manifestation of obtained fragility curves at an intensity of 1.2g.

For limit state of immediate occupancy (IO) the decrease in probability of exceedance is about 22.91% when Spectral Acceleration is used as Intensity measure and 12.12% when Peak ground Acceleration is used as an intensity measure. Similarly, percentage decrease in probability of exceedance for other limit states of Life Safety (LS) and Collapse Prevention (CP) are 24.58% and 18.38% respectively when Spectral acceleration $\mathbf{S_a}$ is used as intensity measure and 30 .69% and 22.32% respectively when Peak ground acceleration is used as intensity measure. Table 5 offers the simulated values for λ_c and β_c for every damage state against each Intensity Measure for the evaluation of fragility curves.

From the above discussion it is concluded that although there is appreciable increase in the stiffness of the overall building in MIRCF i.e its ability to resist deformation is increased and thus the overall behavior is different from that of RCF and apparently it seems that MIRCF must perform better than RCF but that is not the case as due to increased stiffness, the deformation for which the building is designed to achieve against a particular dynamic force is limited thus inducing localized failure in the infill panels which forms the main cause of collateral damage.



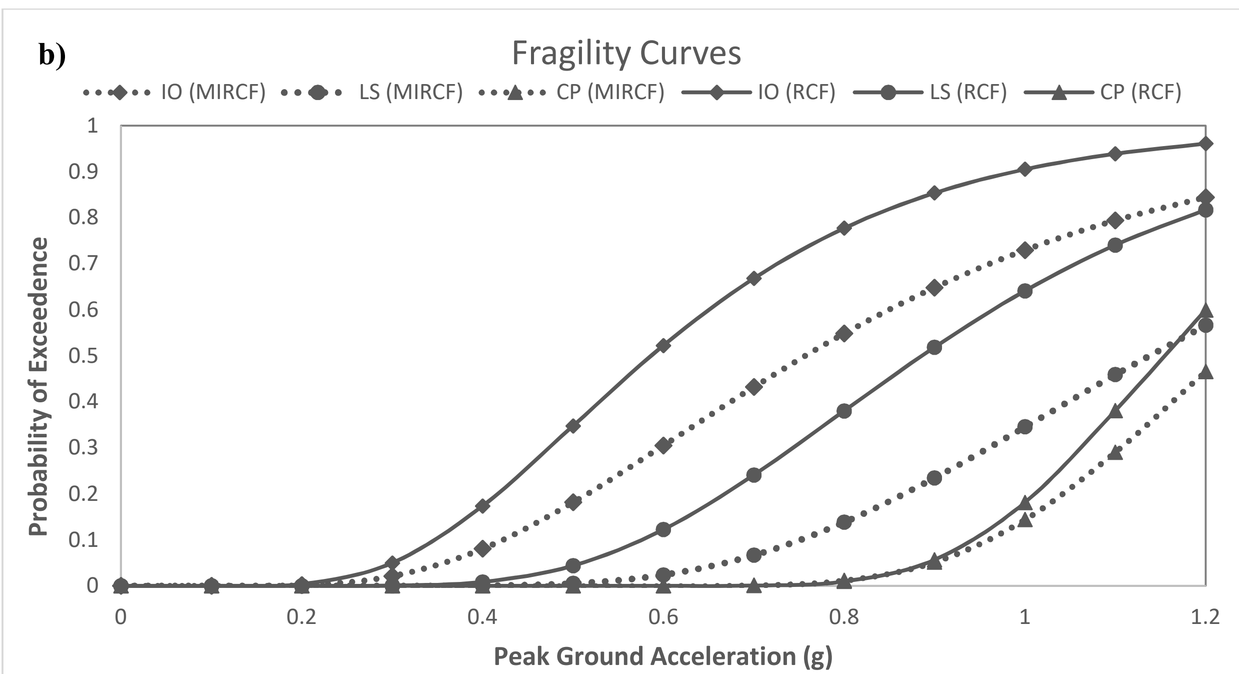


***Fig 4. 11: Fragility relationships for both Intensity Measures: a) Comparison of Fragility curves for* Seismic fragility for S_a @ T_1 (b) Comparison of Fragility curves for Seismic fragility for PGA**

Table 4. 1: Percentage decrease in Probability of exceedance for MIRCF against RCF

Intensity Measure	Limit State	Building Classification	Intensity(g)	Probability of exceedance	% Decrease
SA	IO	RCF	1.2	0.9384	22.91
		MIRCF	1.2	0.7234	
	LS	RCF	1.2	0.6457	24.58
		MIRCF	1.2	0.4870	
	CP	RCF	1.2	0.4394	18.38
		MIRCF	1.2	0.3586	
PGA	IO	RCF	1.2	0.9606	12.12
		MIRCF	1.2	0.8441	
	LS	RCF	1.2	0.8170	30.69
		MIRCF	1.2	0.5663	
	CP	RCF	1.2	0.5994	22.32
		MIRCF	1.2	0.4656	

CHAPTER 5

CONCLUSIONS & RECOMMENDATIONS

5.1. INTRODUCTION TO CHAPTER

In final chapter of this study we will discuss about the conclusions derived from the whole process of development of fragility curves for Simple and Masonry infilled RC frame building. We will discuss about how the performance of RC Frame structure is influenced by the introduction of Masonry infill and how it implicates the performance against seismic forces. At the end of the chapter possible recommendation about future work in performance based probabilistic seismic study will be discussed.

5.2. CONCLUSIONS

1. In current study, fragility curves are developed for both RCF and MIRCF low rise structures. It was concluded from the results that MIRCF performs well than its RCF counterpart due to the temporarily improved stiffness characterized by the provision of masonry infill panels. The stiffness is said to be temporarily improved because at higher intensities the stiffness effect imparted by masonry becomes negligible.
2. The most telling information that can be inferred from the fragility curves is that the overall probability of exceedance was reduced appreciably when Masonry infill is employed in the structural system. The percentage reductions in probability deliberates the importance of masonry infill. However, it should be emphasized that when the intensities are increased beyond 1.2g, the effect on stiffness due to the introduction of masonry infill panels in the reinforced concrete frame system and variation in the percentage decrease in probability of exceedance is negligible. This behavior can be credited to the fact that at higher intensities

the masonry panels do not provide increased stiffness as the bond between frame members and the panels fail instantly at high intensities and can have negative impact.

3. The time period of the structure or building is also reduced due to the provision of infill panels. The reduction in time period however does not guarantee improved performance since earthquake ground motions with low time periods can still have impact on buildings or structures having lower natural time periods.

5.3. RECOMMENDATIONS

At the end this case study recommends

1. Further research on vulnerability of same building stock with varying engineering demand parameter.
2. Integration of current study with seismic hazard assessment to obtain a comprehensive risk assessment plan (Hazard maps) of the region against such hazard.
3. Study soil-structure interaction on the vulnerability of same or other building stock.
4. Study pounding effect of the nearby close buildings on the vulnerability.
5. Integration of Fire vulnerability assessment in the present study to develop an enhanced and multi-response fragility curves.
6. Incorporation of higher modes and check if they significantly influences the fragility of current building stock.

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