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Seismic Fragility Assessment of Rc Frame Structures Under Main Shock-Aftershock Sequences Using Incremental Dynamic Analysis

by

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ABSTRACT

Earthquakes are major unpredictable natural phenomenon which often results in major disasters. In any real earthquake, shaking occurs in sequence of foreshocks, main shock and aftershocks. These repeated earthquakes may occur several times with in even few hours or minutes leaving very limited time between occurrences of tremors. This may hamper the re-occupancy and restoration activities of structures in post disaster situations. When structures are subjected to repeated earthquakes, structural damages gets further accumulated which results in degradation in stiffness and strength characteristics of structural members. Generally, Aftershocks may have smaller magnitude than main shock but it may have higher peak ground acceleration, longer time duration than main shock. Hence, after a major earthquake, it is important to check whether damaged buildings can continue to be occupied, also keeping threat of aftershocks in consideration. This study is mainly focused on fragility assessment of RC framed structure designed according to the Nepal National Building codes of practice under single and repeated ground motions. Incremental dynamic analysis is performed using SAP2000. Results obtained in this study are evaluated in terms of lateral displacement, residual displacement, and maximum inter-story drift ratio for particular peak ground acceleration. Depending on the results it is concluded that repeated earthquakes have significant effects on seismic responses of structure. It is also found that seismic vulnerability considerably increases when structure is exposed to sequence earthquake.

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List of Symbols

ASCE	American Society of Civil Engineers
ATC	Applied Technology Council
DL	Dead load of the structure
DM	Damage Measure
E	Design earthquake load
FEMA	Federal Emergency Management Agency
fm	Compressive stress in masonry
IDA	Incremental Dynamic Analysis
IM	Intensity Measure
NBC	Nepal National Building Code
NDA	Non-linear Dynamic Analysis
NTHA	Non-linear Time History Analysis
IS	Indian Standard
K	Structural performance factor
LL	Live load
MDF	Multi Degree freedom
PEER	Pacific Earthquake Engineering Research Center
PBEE	Performance based earthquake engineering
POA	Push Over Analysis
SDOF	Single Degree of Freedom System
W	Total seismic weight of the structure
Z	Seismic zoning factor
μ	Mean
σ	Standard deviation
Ø	Standardized Normal Distribution Function

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CHAPTER 1: INTRODUCTION

1.1 Background of study

Earthquakes are major unavoidable and unpredictable natural phenomenon which often results in major disasters. In any real earthquake, shaking occurs in sequence. Sometimes earthquake has foreshocks which are small earthquakes followed by large major earthquakes. Largest earthquake is called Main shock which is generally followed by smaller magnitude earthquakes known as ‘aftershocks’. Aftershocks can occur randomly and may repeat multiple numbers of times after the main shocks. Looking at earthquakes occurred worldwide, Aftershocks may have smaller magnitude than main shocks but it may have higher peak ground acceleration, longer time duration than main shock. So, Aftershocks can cause addition to damages caused to structures and infrastructure after main shock hampering their reoccupation and restoration in post-disaster situation.

Nepal being situated in diffuse collisional boundary of two tectonic plates- Indo-Australian plate and Eurasian plate where Indian plate under thrusts Eurasian plate, gets continuously hit by strong earthquakes and exposed to serious seismic hazard. Nepal has experienced many powerful destructive historic earthquakes with moment magnitude greater than or equal to 7.6 since 1255 which lead to serious loss of lives and sizeable economic loss.

Kathmandu Valley and adjoining areas are designated as a severe zone with seismic zoning factor of 0.35 and categorized to soil type 'D' which is very soft soil sites (According to NBC 105:2019). Looking back, this region has been widely damaged during different historic earthquakes like 1408 earthquake- Bagamati Zone (Mw=8), 1767 earthquake- Northern Bagamati zone (Mw=7.9), 1833 Kathmandu- Bihar earthquake (Mw=8), 1988 Kathmandu -Bihar earthquake (Mw=6.9). Recently in 2015, an earthquake named Gorkha earthquake with moment magnitude 7.8 struck near Kathmandu city in central Nepal which devastated rural villages around the region and some of mostly densely populated parts of Kathmandu city. Two large main shocks with magnitude 6.6 and 6.7 shook the region within one day and next day of main shock with several dozen of smaller aftershocks during succeeding days which further added no of death count and damaged large no of structures. In summary, Nepal including Kathmandu lies in high seismic hazard zone and there is urgent need to access the non-linear behavior and perform fragility assessment of RC buildings in Kathmandu valley.

Performance-Based Earthquake Engineering (PBEE) is one of a rapidly growing idea to address dynamic response of structure during earthquakes and it is presented in all guidelines that were published: Vision 2000 (SEAOC, 1995), ATC-40 (ATC, 1996), FEMA-273 (FEMA, 1997), and SAC/FEMA-350 (FEMA, 2000). Among the different methods of PBEE, Incremental Dynamic Analysis (IDA) is an emerging structural analysis method that helps to study non-linear seismic behavior of structures by extracting seismic demand in detail. Carrying out IDA involves performing series of nonlinear dynamic analyses under a suite of multiply scaled ground motion records. IDA also helps to perform Probabilistic Seismic Hazard Analysis to calculate the mean annual frequencies of limit-state exccedance when it is combined with fragility analysis (Vamvatsikos, 2002).

Many research works has been carried out focusing on fragility analysis and development of fragility curves to estimate the probability of damage and seismic assessment of structure in post disaster situations. These research works has been utilized for restoring the function of infrastructures and estimating the probability of re-functioning of the structure after being damaged by earthquakes. Further, this tool has been widely used to study dynamic behavior of the structure at design phase itself to extract the probability of damages flaws rather than adopting expensive rehabilitation treatment.

1.2 Need of Research

Looking at the history of earthquake, repeated earthquakes may occur several times with in even few hours or minutes leaving very limited time between occurrences of tremors. This may hamper the re-occupancy and restoration activities of structures in post disaster situations. Many buildings which withstand the main shock have been collapsed or severely damaged in aftershocks. Even though aftershocks possess normally smaller magnitude, they may have higher ground motion intensity. Generally magnitudes of aftershocks are smaller than that of main shock but they may have higher peak ground acceleration and longer time duration than main shock. Hence, after a major earthquake, it is important to check whether damaged buildings can continue to be occupied, also keeping threat of aftershocks in consideration. The existing code provision of seismic design is using single design earthquake in form of response spectrum or using single sever ground motion for time history analysis of structure. To date, the concept of using main shock and aftershock event for the analysis of RC structures have been rarely used in design purposes and has not been used in codes. Since there are various effects that aftershocks can add, analysis based on repeated earthquakes is found to be necessary.

1.3 Objectives and Purpose of Study

The objective of the study is Seismic Fragility assessment of RC frame Structures under main shock-aftershock sequences using Incremental Dynamic especially in Kathmandu region .The general objectives of this study are listed below

1. To compare the response of the structure when subjected to Series of Main shock and Back to back Main-shock- Aftershock earthquake sequences.

2. To compare the Residual displacement at the end of Major shock and repeated earthquake.

3. To measure the Post main-shock Collapse capacity and Post Aftershock Collapse capacity of the structure.

1.4 Methodology

This section describes the methodology followed to obtain the above-mentioned objectives which are as follows:

1. Review of existing literatures and seismic design code provisions for designing buildings are studied.
2. Definition of problem related to effects of sequence earthquake in dynamic response of structures.
3. Selection of parameters to be considered and fictitious RC framed building for study.
4. Modeling considered structure in SAP2000.
5. Selection of earthquake ground motions for performing Non-linear dynamic analysis.
6. Matching selected earthquake ground motions with selected response target spectrum.

7. Perform Incremental dynamic analysis for to develop IDA curves.
8. Identification of drift limits states.
9. Identification of fragility curve parameters.
10. Generation of fragility curve.
11. Analysis of probability of damage in different damage states.
12. Analysis and interpretation of results.
13. Conclusion, discussion and recommendation are made based upon the results.

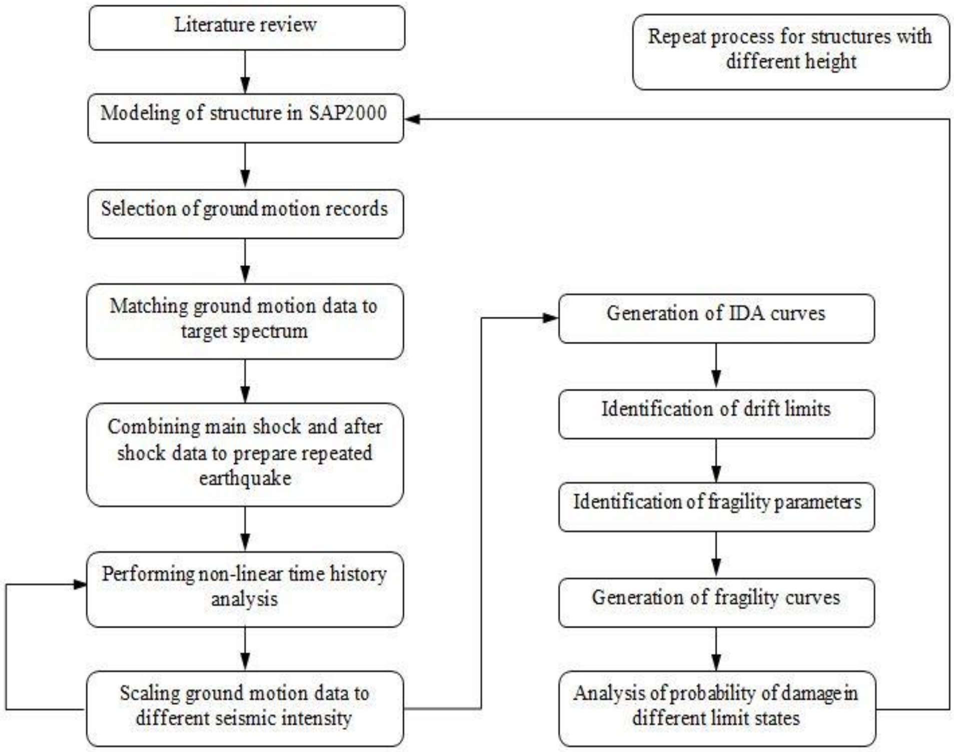


Figure 1.1: Methodology of study

1.5 Organization of Thesis

This thesis work has been organized in seven different chapters. This introductory chapter (CHAPTER 1) gives brief introduction about Sequence earthquake, Seismic vulnerability of Nepal including Kathmandu, PBEE and Fragility analysis. Need of study, Objectives of research work are defined along with methodology which is adopted to complete this thesis work.

- **Chapter 2:** presents the review of provisions in different codes for non-linear dynamic analysis, response target spectrum for Kathmandu valley region. This chapter also includes the literature review for PBEE, Effects of seismic sequence earthquake, Incremental dynamic analysis, Fragility analysis, Residual displacement of structure.

- **Chapter 3:** contains theoretical background used in this study. This chapter describes the methods of Performance based earthquake engineering and in particular it describes about procedures and important parameters for performing Incremental dynamic analysis. This also describes the theoretical formulation of fragility assessment of structures. This chapter further discuss about developing IDA curve and Fragility curve in detail.

- **Chapter 4:** contains the description of selected structure, structural modeling parameters, and details about the structural components.

- **Chapter 5:** includes details about selected earthquake ground motions. This chapter describes in brief about the selection, matching and scaling of ground motion data.

- **Chapter 6:** presents the results obtained from matching and scaling ground motion parameters, Non-linear dynamic analysis, Incremental dynamic analysis and fragility assessment of building models. This chapter evaluated and compared the dynamic responses and fragility assessments of structures for single earthquake ground motions and sequence earthquake ground motions for different types of models.

- **Chapter 7:** contains the conclusions drawn from the analysis and results from the study and some of the recommendations for future works.

CHAPTER 2: REVIEW OF CODE PROVISIONS AND LITERATURE

2.1 Overview

This chapter presents a brief summary of the literature and code provisions been studies for this thesis work. This chapter contains two parts. The first part deals with the review of literatures whereas the second part briefs about the code provisions.

2.2 Literature Review

Many research papers are reviewed to carry out the research work and to act as guidance for successful completion of the thesis work. Some of the references used in this study are discussed below:

2.2.1 Performance Based Earthquake Engineering

Gunay and Mosalam (2012) summarized the PEER PBEE framework in a very simple way to help practicing engineers to understand the PBEE methodology. The paper described about PEER PBEE methodology which consists of four successive analyses: hazard analysis, structural analysis, damage analysis, and loss analysis. The paper concluded that PEER PBEE methodology proves to be important design tool for designing conventional structural such as moment resisting frames with unreinforced masonry infill walls and also for the advanced and sustainable analysis and retrofit strategies such as base isolation, rocking foundations, and self-centering systems.

Deierlein (2004) developed the robust methodology for performing Performance Based Earthquake Engineering. The paper clearly described about process of assessment, defining the intensity measures, simulation process for establishing engineering demand parameters and damage measures and also defined the calculation of Decision variables. The paper further stated that robust methodology can be used as greatest potential for reducing losses in future earthquakes.

2.2.2 Incremental Dynamic Analysis

Dimitrios Vamvatsikos (2002) established and defined the basic principles of Incremental Dynamic Analysis (IDA). The paper provided the brief knowledge about Intensity measures (IM), Damage Measures (DM), IDA curves and principles of Single record IDA and Multi-record IDA. It carried out analysis for different types of structures and developed response-intensity curves. It thoroughly studied those curves and proposed efficient techniques to perform an incremental dynamic analysis. It further incremental dynamic analyses are valuable tool that simultaneously addresses tool that simultaneously address the seismic demand of the structure and their global capacities.

Cornell (2004) proposed a practical method to perform Incremental dynamic analysis as the complicated analysis method like IDA requires an intense computational effort. 9-story steel moment-resisting frame is analyzed and studied in detail to show how to apply it, how to interpret the results, and how to use these results within the framework of performance-based earthquake engineering.

Dimitrios and Cornell(2005) performed both Pushover analysis (POA) and IDA on both Single degree of freedom and multiple degrees of freedom structures and defined how IDA is associated with conventional POA. It also introduced new software, SPO2IDA that allows direct estimation of the summarized IDA results. The paper concluded that as IDA addresses both demand and capacity of structures and can be used as very important part of PBEE framework in future also.

2.2.3 Approach of Fragility Analysis

Gautham. A(2016) performed nonlinear static using Capacity Spectrum method on the structural models using SAP2000 and developed fragility curves to determine effects of infill wall in spectral displacement. The paper concluded that fragility curves can be effectively used to identify the effects of irregularities such as the soft story structures in functioning of structural systems.

Nazri and Saruddin (2015) performed Incremental Dynamic Analysis (IDA) using SAP2000 software considering seven ground motion records and compared results with the limit state to develop fragility curve. It concluded that Fragility curve is a very helpful method in determining the level of probable damage during earthquakes.

Nazri F. M (2018) performed fragility analysis for different story RC structures and Steel structures. The paper used POA and IDA for developing fragility curves. Both the Far-field and near- field ground motions data were considered for analysis. (Nazri F. M (2018) and Yassif & Motsafa (2011) concluded that the fragility analysis can be done to every individual buildings separately, be it pre-earthquake or post-earthquake conditions and presented the probability of damage of building in different performance level.

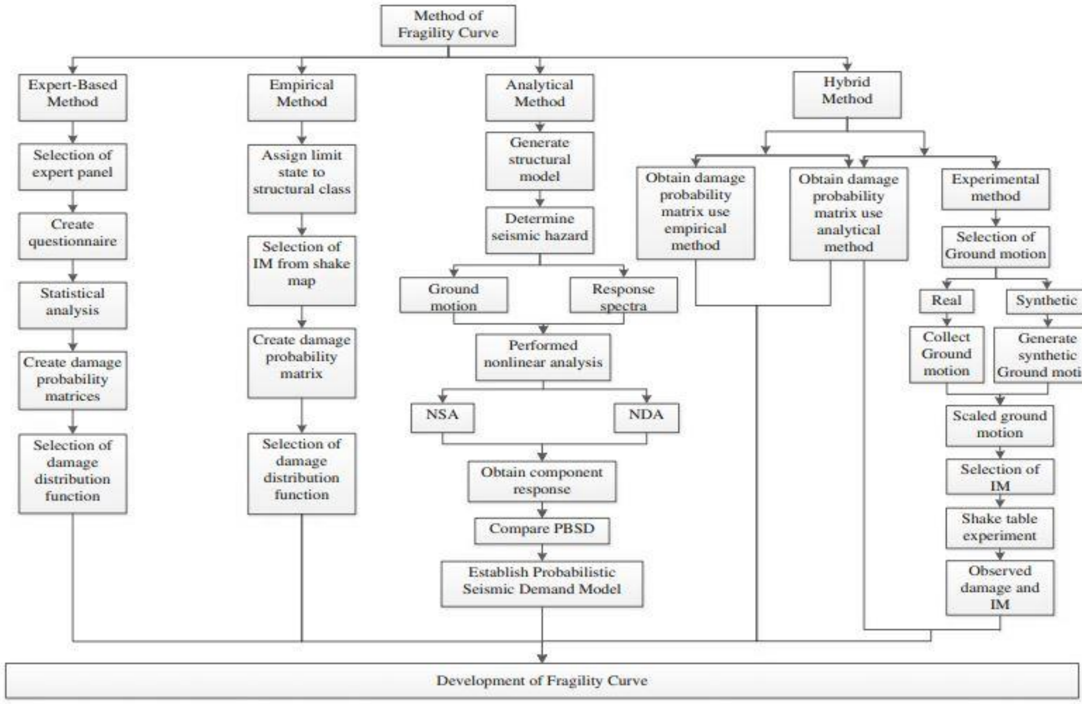


Figure2.1: Methods to develop fragility analysis

2.2.4 Effects of sequence earthquakes on dynamic performance of buildings

Jorge Ruiz (2012) analyzed that in the absence of the real aftershock earthquake data, randomized approach of generating earthquake data is useful because the artificial randomized aftershock ground motions should have smaller amplitude than the real main shock ground motion and also most of them should have shorter predominant period and concluded that the predominant period of the aftershock significantly influences the post-main shock response.

Hatzivassiliou and D.Hatzigeorgiou (2015) And Haider, Nizamani, and Yip (2020) concluded that the roof level displacements under repeated earthquakes are doubled or even more than during single earthquake. It also found that inter-storey drift ratio also increases for repeated earthquake. Also, structural members perform elastically under single ground motions while they behave in elastically during repeated earthquake, i.e., they are damaged.

Oggu and Gopikrishna (2020) carried out vulnerability assessment of three-dimensional RC building frames (with and without vertical irregularities) designed based on the Indian Standard codes of practice under bi-directional single and repeated ground motions. The results concluded in this paper are that repeated earthquakes considerably reduce the collapse capacity of RC buildings compared to that of the most severe single earthquake.

2.2.5 Residual Displacement

Kawashima, MacRae, Hoshikuma and Nagaya (2013) performed analysis of many single degree-of-freedom bilinear oscillators varying natural periods, damping ratios, ductility factors, bilinear factors, and input ground motions and described how to estimate the residual displacements of real single-degree-of-freedom structures. The paper developed residual displacement response spectrum and residual displacement response ratio spectrum using 63 ground motions to extract residual displacements of structures subjected to large ground motions.

Iyama (2012) discussed that how the probability distributions of the residual displacement is affected by ground motion and vibrational system characteristics when excited by earthquake ground motions. The paper presented the theory "Random walk" and used it to explain the probability distribution of the residual displacement in isotropic hardening bilinear SDOF systems. The paper aimed at developing the method to calculate the residual displacement of a building manifested during earthquake.

2.2.6 Collapse Capacity

Ibarra and Krawinkler (2004) developed a method to determine global collapse capacity of structures under earthquake ground motions. The paper used the Sa(T1)g as the intensity measure for the analysis, where Sa is spectral acceleration at T1, T1 is fundamental time period and g is acceleration due to gravity. The methodology used to determine the collapse capacity involved increasing the intensity measure until the curve in graph of intensity measure versus normalized maximum roof drift curve becomes flat. Therefore, the collapse capacity is of structure is equal to the relative intensity measure at which this curve becomes flat.

Shafei, Zareian and Lignos (2011) Defined collapse capacity of structural systems as the spectral acceleration value at which the structure becomes dynamically unstable due to component strength and stiffness deterioration and/or –Delta effects. The paper further presented a methodology for estimation of collapse capacity of structural systems using nonlinear static (pushover) analysis.

Yue Li, Ruiqiang Song and John W. Van De Lindt (2014) adopted various methodologies to evaluate the collapse fragility of the steel building. The paper determined the effects of main shocks on the structural collapse capacity and found that structures are exposed to high intensity main shocks the collapse capacity of structure reduces significantly. So that after being damaged by main shock the likeliness of structure to collapse increases even if it is further exposed to aftershocks.

Raghunandan, Liel and Luco (2014) studied the vulnerability of four modern ductile reinforced concrete (RC) framed buildings in California under the aftershocks using incremental dynamic analysis. After performing non-linear dynamic analysis, collapse and damage fragility curves were developed for both intact and damage structures. And the paper concluded that damage indicators related to the drift experienced by the damaged building are the most useful to predict the reduced aftershock collapse capacities for these ductile structures.

2.3 Review of codes

The seismic design codes of buildings of different countries have different provisions for Non-linear dynamic analysis. Seismic Zoning of Kathmandu region according to Indian code and Nepal building code is also presented in this section. Some of national and international codes that had been reviewed are listed below:

a. Nepal- NBC 105: 2020– Structural analysis and design

b. Nepal- NBC 105: 1994 – Spectral factor

c. Indian- IS 1893(Part I): Spectral factor

d. USA- ASCE 7: 2016- Non-linear dynamic analysis

e. Europe- BS EN 19981: 2004- Methods of analysis

2.3.1 Nepal National Building Code NBC 105: 2020

Nepal national building code established following relation and figure for Spectral factor. The spectral factor $C_h(T)$ for relevant soil type shall be obtained from following Figure 2.2 and Figure 2.3 or Equation 2.1 .

$$Ch(T) = \begin{cases} 1 + (\propto -1) \times \frac{T}{Ta} if\ T < Ta \\ \propto\ \ if\ Ta \le T \le Tc \\ \propto \left[K + (1-K)\left(\frac{Tc}{T}\right)^2\right]\left(\frac{Tc}{T}\right)^2\ if\ Tc \le T \le 6 \end{cases} \tag{2.1}$$

Where,

$\propto$ = peak spectral acceleration normalized by PGA

Ta and Tc= the lower and upper periods of the flat part of spectrum

K = Coefficient that controls the descending part of spectrum

Table 2.1: Soil parameters

Parameters/ soil type	Soil type A	Soil type B	Soil type C	Soil type D
Ta	0.1	0.1	0.1	0.5
Tc	0.5	0.7	1	2
$\propto$	2.5	2.5	2.5	2.25
K	1.8	1.8	1.8	0.8

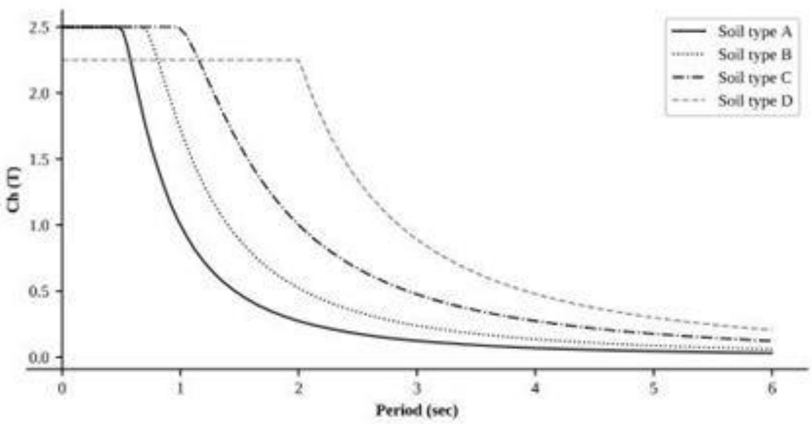


Figure2.2: Spectral shape factor, $C_h(T)$ for Equivalent Static Method

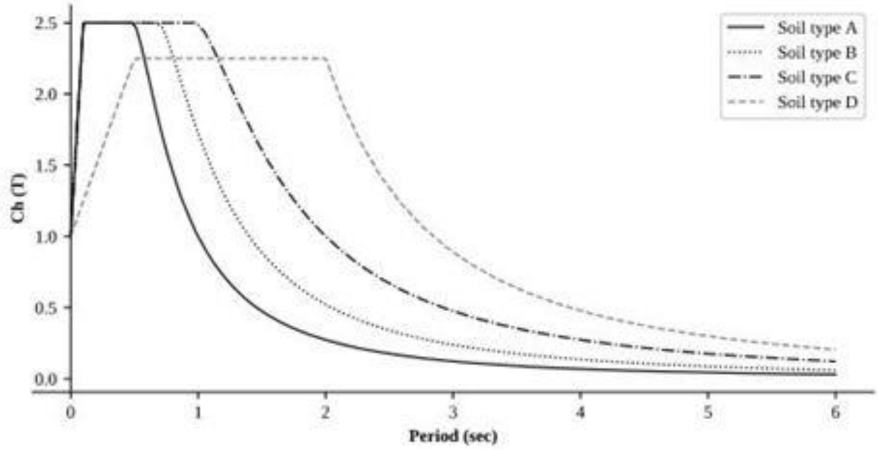


Figure2.3: Spectral Shape factor, $C_h(T)$ for Modal Response Spectrum Method, Non-Linear Time history analysis, Vertical loading parts and components

2.3.2 Indian Standard IS 1893 (Part 1): 2016

Design spectral acceleration for different soil type, normalized with peak ground acceleration, corresponding to natural period T of structure is given by IS 1893(part 1) 2016. When structure is not specified, it shall be taken as corresponding to 5% damping, given by expressions below:

a) For use in equivalent method

$$
{Sa}/{g} = \begin{cases} For\ Rockey\ or\ hard\ soil\ sites \begin{cases} 2.5 & 0 < T < 0.4s \\ \dfrac{1}{T} & 0.40s < T < 4s \\ 0.25 & T > 4s \end{cases} \\ For\ medium\ stiff\ soil\ sites \begin{cases} 2.5 & 0 < T < 0.55s \\ \dfrac{1.36}{T} & 0.55s < T < 4s \\ 0.34 & T > 4s \end{cases} \\ For\ soft\ soil\ sites \begin{cases} 2.5 & 0 < T < 0.67s \\ \dfrac{1.67}{T} & 0.67s < T < 4s \\ 0.42 & T > 4s \end{cases} \end{cases}
$$

b) For use in response spectrum method

$$
Sa/g = \begin{cases} For\ Rockey\ or\ hard\ soil\ sites \begin{cases} 1+15T & T<0.10s \\ 2.5 & 0.10<T<0.40s \\ \dfrac{1}{T} & 0.40s<T<4s \\ 0.25 & T>4s \end{cases} \\ For\ medium\ stiff\ soil\ sites \begin{cases} 1+15T & T<0.10s \\ 2.5 & 0<T<0.55s \\ \dfrac{1.36}{T} & 0.55s<T<4s \\ 0.34 & T>4s \end{cases} \\ For\ soft\ soil\ sites \begin{cases} 1+15T & T<0.10s \\ 2.5 & 0<T<0.67s \\ \dfrac{1.67}{T} & 0.67s<T<4s \\ 0.42 & T>4s \end{cases} \end{cases}
$$

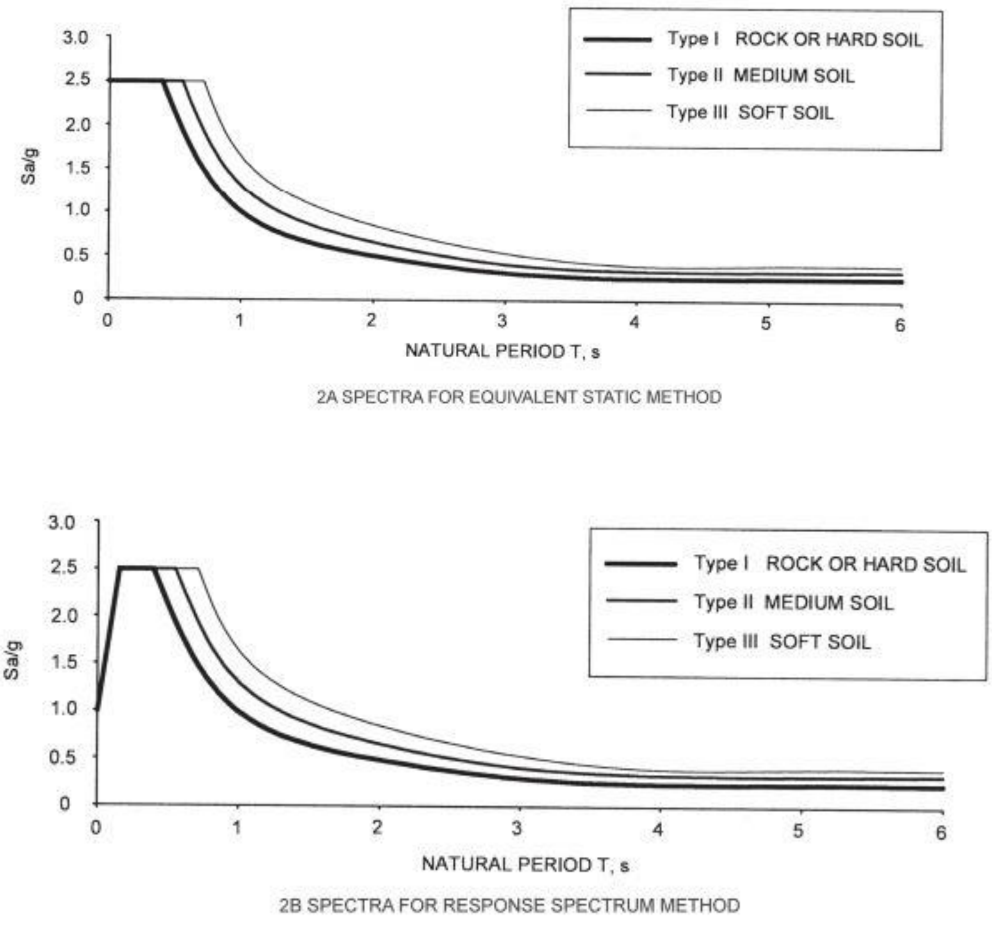


Figure2.4 Design acceleration coefficient (Sa/g) (Corresponding to 5% damping)

CHAPTER 3: THEORETICAL FORMULATION

3.1 Performance based earthquake engineering

A Performance Based Earthquake engineering (PBEE) is the methodology that has been widely used for understanding the seismic performance of structures. Pacific Earthquake Engineering Research Center (PEER) has developed the very useful method to perform PBEE focusing on the probabilistic calculation of structural performance measures. PEER PBEE methodology consists of four successive analyses steps: hazard analysis, structural analysis, damage analysis, and loss analysis.

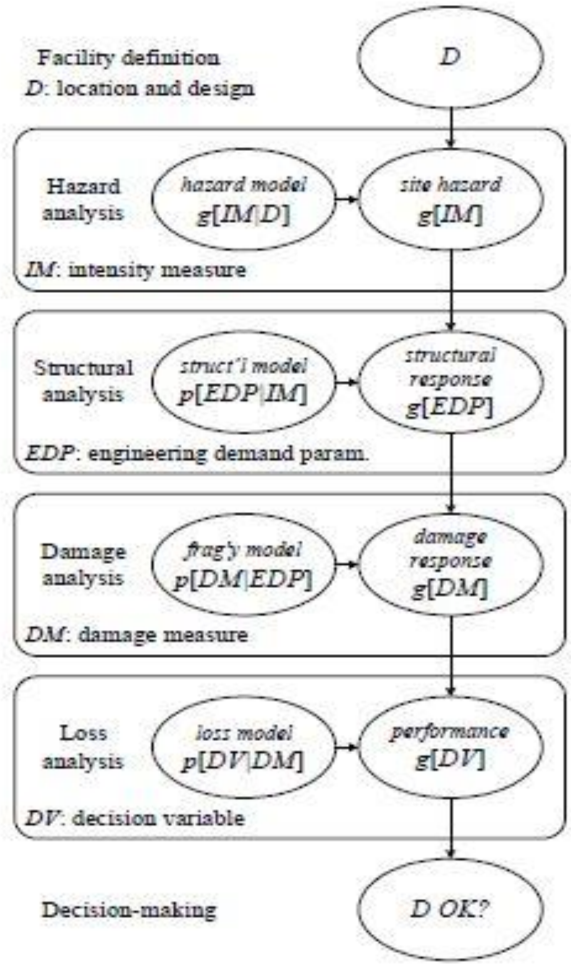


Figure 3.1: Flowchart of PEER PBEE framework

3.2 Incremental Dynamic Analysis

Incremental dynamic analysis is the powerful method of PBEE framework. IDA involves series of nonlinear dynamic analyses under a suite of multiple scaled ground motion records for addressing seismic performance of structural system. Also IDA offers detailed knowledge in seismic demand and limit-state capacity prediction capability. Many research works has been done to describe IDA but the one which gave detail knowledge about IDA is described below

(Dimitrios Vamvatsikos, 2002)

This study provided detail knowledge about the common frame of reference to perform the IDA. As the IDA is very complicated parametric analysis, the fundamental concepts and suitable algorithms for performing IDA must be clear. Carrying out IDA to determine the non-linear behavior of structure involves several computational steps. At first, a proper nonlinear structural model needs to be modeled on suitable finite element software then appropriate number of ground motion records need to be considered for performing non-linear dynamic analysis. Further, selected ground motion records need to be scaled to different intensity levels and the dynamic analysis is performed and results are extracted.

3.2.1 IDA Algorithm

Theoretical study of an IDA process is very simple but performing the actual analysis can be resource intensive. Various algorithms were developed for IDA by various journals. (Seung-Yul Yun, Hamburger, Cornell, & Foutch, 2002) Gave a simple stepping algorithm which involved the simple method of increasing the IM by a constant step from zero until the structure is collapsed. The results obtained after using this algorithm are uniformly-spaced (in IM) grid of points on the curve. The algorithm needs only a pre-defined step value and a rule to determine when to stop, i.e., when a run is collapsing.

Repeat

Increase IM by the step

Scale record, run analysis and extract DM (s)

Until collapse is reached

3.3 Fragility function

Fragility functions are very important tool used to develop the fragility curves and further to determine the probability of the structure to reach the collapse point or other seismic limit state. Fragility function is a function of some ground motion intensity measure (IM) which is determined from different approaches such as field observations of damage, static structural analyses, or judgment.

Many studies have been carried out to determine the Fragility Function, some of which are presented below:

(Bakera & M.EERI, 2015)

This paper used the statistical procedures to evaluate the fragility functions parameters using results obtained from performing nonlinear dynamic analysis of structure. Those procedures were used to evaluate fragility functions by using various non-linear dynamic analyses. This paper gave lognormal cumulative distribution function as a fragility function as shown in equation 3.1

$$P(C\backslash IM = x) = \Phi\left(\frac{\ln(x/\theta)}{\beta}\right) \qquad (3.1)$$

where $P(C\backslash IM = x)$ is the probability that a ground motion with IM = x will cause the structure to collapse, Φ( ) is the standard normal cumulative distribution function (CDF), is the median of the fragility function (the IM level with 50% probability of collapse) and is the standard deviation of lnIM (sometimes referred to as the dispersion of IM).

(Gautham. A, 2016)

In this study, performance of structural is characterized by performing pushover analysis for various limit states defined in ATC 40. Damage state thresholds are represented on four damage states; Slight, Moderate, Extensive and Complete defined based on the bi - linear capacity spectra. Based on those thresholds, the Median Spectral Displacements are calculated for all of four damage states as specified in Table (3.1).

Table 3.1: Damage threshold spectral displacements

$Sd_1 = 0.7D_y$	Slight
$Sd_2 = D_y$	Moderate
$Sd_3 = D_y + 0.25(D_u - D_y)$	Extensive
$Sd_4 = D_u$	Complete

The median spectral displacement is calculated for the considered damage state and variability values are determined. Further, developed the conditional probability of exceedance of damage corresponding to the damage state is defined by equation 3.2

$$P(ds \backslash sd) = \emptyset \left\lfloor \frac{1}{\beta_{ds}} ln \left(\frac{Sd}{Sd_{ds}} \right) \right\rfloor \qquad (3.2)$$

Where,

Sd_{ds} = Median value of spectral displacement at which the structure reaches threshold damage state, ds

Sd = Given peak spectral displacement.

β_{ds} = variability of spectral displacement for the damage state, ds.

(Nazri F. M., 2018)

In this study, Fragility curves are established for both near field and far field ground motion data for steel structure and RC framed structure. Fragility function used in this study is as shown in equation 3.3

$$P(x) = \emptyset \left(\frac{LnX - \lambda}{\xi} \right) \qquad (3.3)$$

Where,

($\emptyset$) is the standardize normal distribution,

λ is the mean of ln x,

ξ is the standard deviation of ln x.

3.4 Residual displacement:

Residual displacements are permanent drifts which are manifested in structure at the end of the analysis. These permanent drifts are developed due to the inelasticity of the structural members. Residual displacements recorded in post- earthquake condition can be used as the very important indexes to represent the probability of repairing of damaged building.

(Iyama, 2012), This study described use of the hypothesis of random walk to perform the theoretical probability distribution for distribution of residual displacements. The hypothesis of a random walk states that at each occurrence of yielding (walk) the plastic deformation is constant. The paper further described, the plastic deformation at i-th occurrence of yielding is represented as d_i. The residual displacement d_r and the cumulative plastic displacement d_t are represented as,

$$d_r = \sum_{i=1}^{n} d_i$$

$$d_t = \sum_{i=1}^{n} |d_i|$$

The normalized values of the plastic deformation at each occurrence of yielding, di , and the residual displacement, dr , are defined as,

$$\overline{d_\iota} = \frac{d_i}{d_t}$$

$$\overline{d_r} = \frac{d_i}{d_t}$$

The figure below shows a conceptual diagram of time histories of the residual displacement and the cumulative plastic displacement.

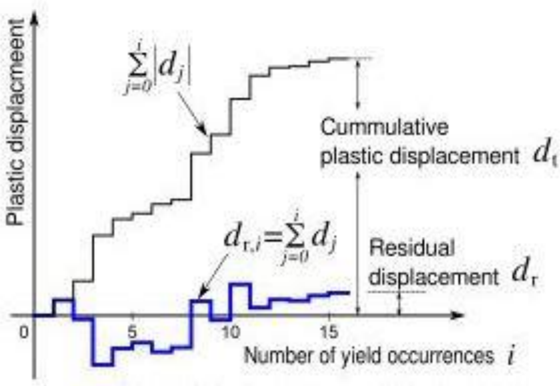


Figure3.2: Example of response analysis result

3.4.1 "Random walk" hypothesis with constant plastic displacement

In this approach, study is carried out by assuming that the behavior of the time history of the residual displacement is a "random walk", which means that the magnitude of every single plastic displacement is constant and its direction is completely random. Therefore, the normalized plastic deformation at the i-th occurrence of yielding, di , should be either $+1/n$ or $-1/n$, each with a 1/2 probability of occurring, where n represents the number of occurrences of yielding during an earthquake. This means that the probability distribution of the post-earthquake residual displacement, dr , is a binomial distribution with a standard deviation of $1/\sqrt{n}$.

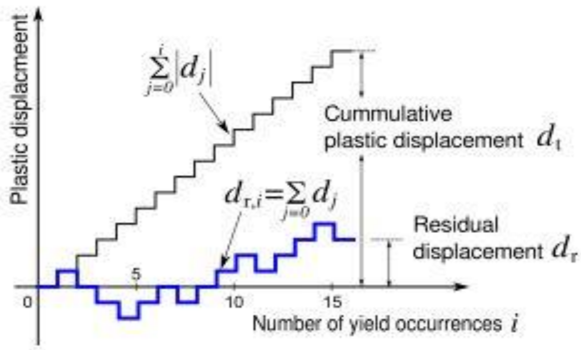


Figure3.3: Random walk hypothesis with constant plastic displacement

3.4.2 "Random walk" hypothesis with constant energy input

In this section, the plastic energy input is same at each occurrence of yielding instead of the magnitude of the plastic displacement. This assumption is clearly shown in fig below:

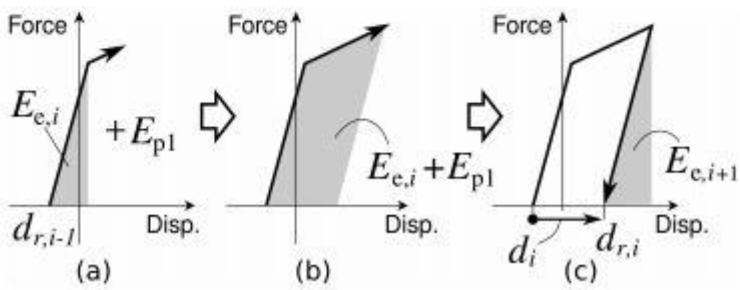


Figure3.4: Assumed behavior during a single plastic deformation

Using this hypothesis, extracting the exact closed-form formula for estimating the standard deviation of the residual displacement is very hard and complicated. However, by using Monte Carlo simulation, the probability distribution and its standard deviation can be calculated numerically using the procedure shown in Figure 3.8.

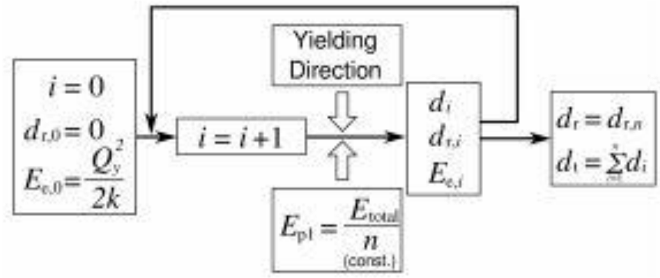


Figure3.5: Procedure to compute the residual displacement under the hypothesis of constant energy input

3.5 Collapse Capacity

The collapse capacity of structural systems is defined as the spectral acceleration value at which the structure becomes dynamically unstable due to component strength and stiffness deterioration and/or –Delta effects. It is very crucial to determine collapse capacity of structures after being hit by strong ground motions. This helps to determine life safety and re-occupancy of structures during seismic hazards. There are various analysis methods to determine collapse capacity of structures and some of them are described below.

3.5.1 Non-Linear static procedure:

Non-linear static procedure which is also known as push over analysis is one of the popular tool used to determine the probable damage and safety of structures during seismic hazard like earthquake. This method involves analyzing the structure considering the nonlinear force deformation behavior of its elements .The structure is subjected to monotonically increasing lateral force until the displacement of a control node exceeds a target displacement or the structure collapses and the force displacement relation is established. A global collapse of structure is supposed to be reached when the slope in the base shear-lateral displacement curve becomes negative due to P-Δ effects and reaches afterward a point of zero base shears.

3.5.2 Incremental dynamic analysis

Determining the collapse capacity by using incremental dynamic analysis, involves performing a series of nonlinear dynamic analyses. The intensity of the ground motion is incrementally increased to different intensity levels until the global collapse capacity of the structure is reached. The result of process is presented by plotting a measure of the ground motion intensity e.g., peak ground acceleration against a response parameter (demand measure) such as peak story drift ratio. The structure is supposed to reach its global collapse when the curve in that plot becomes flat which means that even a small increase in the ground motion intensity generates a large increase in the structural response.

3.6 Method of analysis

After studying all procedures and formulas given by many researches to perform IDA, fragility analysis, residual displacement and collapse capacity, this section demonstrates the analysis procedures and formulas used in this research work.

3.6.1 Method of analysis

Analysis of structures is carried out to determine and study the responses of structures. Various analysis methods are linear static analysis, nonlinear static analysis, linear dynamic analysis and nonlinear dynamic analysis. Among the different analysis methods, nonlinear static analysis is of much importance and discussed more in detail in following sections than others.

a) Linear static analysis: Linear static analysis which is also known as equivalent static method is the simplest of all which is based on provisions provided in code of practice. Design base shear is computed for the whole building and then distributed along the height of the building. The lateral forces at each floor levels thus obtained are distributed to individual lateral load resisting elements based on floor diaphragm action.

b) Linear dynamic analysis: Linear dynamic analysis which is also called Response spectrum analysis is the method in which structures are subjected to transient dynamic loading to measure response of structure. It considered contribution of each mode of vibration to evaluate maximum response based upon SRSS (Square Root of Sum of Squares) or CQC (Complete Quadratic Combination) method. This provides an insight into dynamic behavior by measuring pseudo spectral acceleration, displacements, velocity as a function of structural period for a given time history and level of damping.

c) Non-linear static analysis: This method is also called Push over analysis which holds relation between applied forces and displacements. It evaluates the expected performance of a structural system by estimating its strength and deformation demands in design earthquakes by means of a static inelastic analysis, and comparing these demands to available capacities. The analysis is carried out up to failure of the structure and therefore it helps to determine the collapse load and the ductility capacity of the structure. This can be used to obtain the performance point which can be used in retrofitting works as well.

d) Non-Linear dynamic analysis: In Non-Linear dynamic analysis or Non-linear time history analysis, structures are subjected to ground motion record that represents the expected earthquake at the base of structure. It is the most complex method of analysis and attempts to completely represent the seismic response of the structures. This method is the only universally appropriate method for verifying the performance of structure especially when the responses of structure are non-linear. Selection of appropriate ground motion records is very crucial to perform this analysis. Time history analysis is a computational analysis which determines the seismic response of a structure under dynamic loading of representative earthquake.

3.6.2 Force-Displacement Relationships

The force-displacement relationship of frame elements in non-linear static analysis is often represented by that of the plastic hinges assigned to desired location of frame elements. Since it is likely that yielding will occur at the ends of members which are subjected to lateral load, plastic hinges are assigned to those locations. FEMA-356 defines force-displacement criteria for hinges used in non-linear analysis which is shown in figure 3.3.

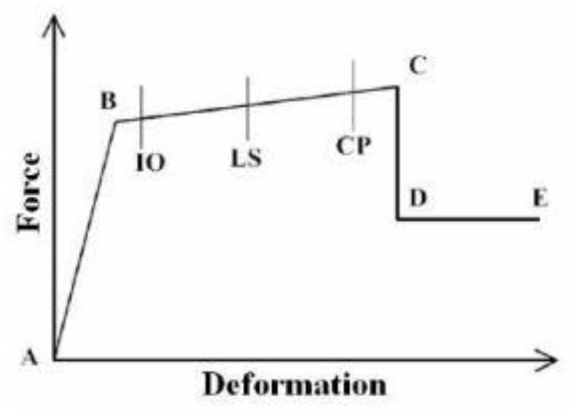


Figure3.6: Component of force displacement curve

3.6.3 Incremental Dynamic Analysis

Incremental Dynamic Analysis (IDA) is a parametric analysis method used to estimate structural performance in more detail under seismic loads. IDA is the computational analysis which involves performing multiple dynamic analyses of structural model under suit of ground motion records each scaled to several levels of seismic intensity. The scaling levels are selected accordingly to understand responses of the structure through the entire range of behavior, from elastic to inelastic and finally to global dynamic instability. To carry out the IDA, structure are subjected to one (or more) ground motion record(s), each scaled to multiple levels of intensity, which gives one (or more) curve(s) of response parameters versus intensity level.

Importance of IDA

- Gives the detail knowledge about intensity measure, response parameters and relationship between them.
- Determining the performance of structure when subjected to ground motions of different intensity.
- Estimating the collapse capacity of structure in post-disaster situation.

3.6.4 IDA in SAP2000

SAP 2000, a finite element based structural program is widely used for all types of analysis whether its linear static or non-linear dynamic that allows quick and better implementation of NLTH analysis procedures prescribed in ATC-40 and FEMA-273 documents. IDA involves multiple NLTH of structural model under multiple ground motions scaled to several levels. The following steps are carried out to perform incremental dynamic analysis:

1. At first three dimensional models of the structure needs to be modeled for carrying out the analysis. The RC building is modeled by drawing the structural element with the geometry, joint restraints, material properties and providing loads over the members.
2. The linear seismic analysis is performed for preliminary design.
3. Geometric non-linearity in the form of p-delta effect can be considered in the analysis. Material non-linearity of frame element is represented by hinges and

hinges can be assigned at discrete locations where mechanism is expected. Hinges can be auto hinges or user defined hinges and SAP converts them to generated hinges.

4. Appropriate ground motions; one or more (usually more) data shall be selected. Where the required number of recorded ground motions is not available, appropriate simulated ground motions shall be used to make up the total number required.
5. The selected ground motions needs to be scaled to match the target spectrum. The scale factor is computed so that it lies between periods Tn and √RμxT1, where T1 is the fundamental period of vibration of the structure, Tn is the period of the highest vibration mode to ensure 90% mass participation and Rμ is the ULS ductility factor.
6. The scaled ground motion/s is applied to the supports of the structural model.
7. After performing NLTH then the analysis is continued with different scaled factor for IDA until the target state of structure meets.

3.6.5 Fragility Analysis

Fragility analysis is one of Probabilistic approach which correlates demand and capacity of structure. It is the probabilistic characterization of the demands of structure with respect to certain seismic limit states. Fragility Curves emerging tool which are widely being used to evaluate damage and loss during earthquakes. It is used to predict if the damage caused during earthquakes meets or exceeds a certain performance level under a given set of ground motion parameters. Also, the fragility analysis can be carried out in both pre and post-earthquake situations. The results obtained from fragility analysis are presented in form of fragility curves. These curves are unique because every building has specific fragility analysis.

Fragility curves are used to determine the probability of reaching or exceeding a specific damage state under earthquake excitation. The general equation to develop fragility or conditional probability is expressed by equation 3.4.

$$Fragility = P\left\lfloor \frac{LS}{IM} = y \right\rfloor \qquad (3.4)$$

Where, LS is the limit state or damage state (DS), IM is the intensity measure (ground motion), and Y is the realized condition of ground motion IM.

The fragility curves are being used as the very important indicator to find out the probable damage during the earthquakes. These curves also help to perform the seismic risk assessment of structures. So, the fragility function can be used to reduce probability of damage, economic loss from damage and loss of life during earthquakes. Therefore, fragility curves can be used in both pre- and post-earthquake situations as an important decision-making tool.

3.7 Formulation used in this study

The following procedures and formulation are used in this study:

Incremental dynamic analysis is performed according to procedures presented in (Dimitrios Vamvatsikos, 2002). For this study, Peak ground acceleration is chosen for Intensity Measure (IM) and Inter-story drift ratio percentage is chosen for Damage Measure (DM) to develop IDA curve.

Fragility curve is developed according to (Nazri F. M., 2018) and fragility function is calculated using equation 3.5

$$P(x) = \emptyset \left(\frac{LnX - \lambda}{\xi}\right) \qquad (3.5)$$

Where,

(Ø) is the standardize normal distribution,

λ is the mean of ln x,

ξ is the standard deviation of ln x.

CHAPTER 4: CASE STUDY OF BUILDINGS

To describe the real geometry of structure, the model is extended in the direction of all three axes. 3D modeling of structure can be done by discretizing it as an assemblage of beam and column element interconnected at nodes. For this research work, Finite element analysis software SAP2000 v21.2 "Integrated software for structural analysis & design" is used for both linear elastic analysis and design and for non-linear dynamic analysis. Three types of buildings (low rise-4 storey, midrise-7 storey, high rise- 10 storey) are considered for this study.

4.1 Assumptions of study

To reduce complexity in analysis and computation different assumptions are made without much variation in result of modal and real structure. Assumptions made in this study are as follows.

a) Foundation: Foundation is assumed to be rigid i.e Soil structure interaction is not considered in this study.
b) Participating elements: Primary elements of structure like beam, column and slab are modeled. Effect of secondary and non-structural components like staircase, partition wall etc. are assumed to be negligible.
c) Infill wall: Only the mass of infill wall is considered and applied in corresponding beam as uniform distributed load. Stiffness of infill wall that might contribute to the stiffness of lateral load resisting system is not considered.
d) Diaphragm: The diaphragm is assumed to be rigid. Rigid diaphragm m forces constrained joints to move together as a rigid planar diaphragm.
e) Secondary effects such as temperature, creep, shrinkage etc. are not considered to simplify the analysis process.

4.2. Limitation of study

As the field of research is always vast, to limit the study and focus on specific problem, this study has following limitations:

a) Numbers of ground motions selected are limited to seven ground motions.
b) Structures are designed and analyzed for the Kathmandu region only.

c) Effect of variation in number of bays, bay length and storey height on seismic performance is not studied.

4.3 Building Nomenclature

In this study, buildings with equal bay in both directions are considered and only number of storey of structure is varied in different buildings. Building comprises three bays of 5 m each in both horizontal directions with a story height of 3.2 m. However variation in number of bay and size of bay is not considered. Low rise, Midrise and High rise buildings with corresponding number of story 4,7,10 are modeled. Typical elevation and 3D model of three different structures in shown in Figure 4.1 to Figure 4.6.

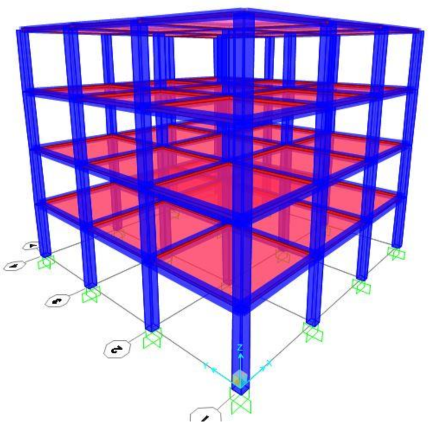


Figure4.1: 3D of low rise building

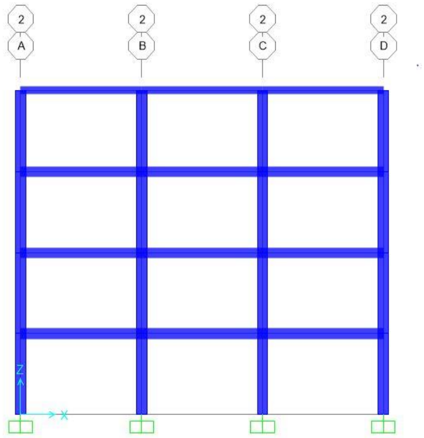


Figure4.2: 2D of low rise building

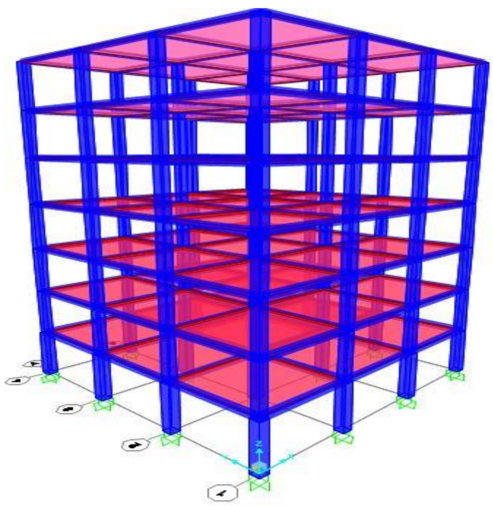


Figure4.3: 3D of Midrise building

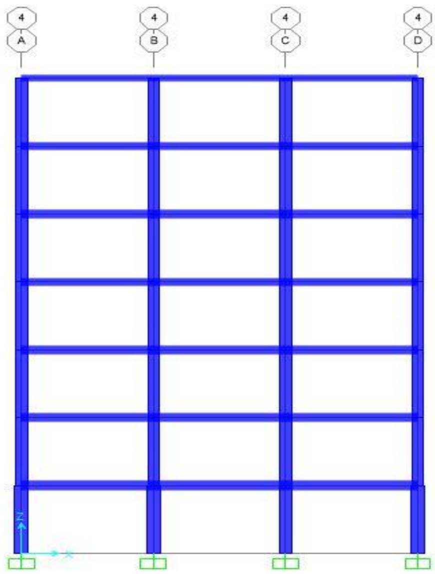


Figure4.4: 2D of Midrise building

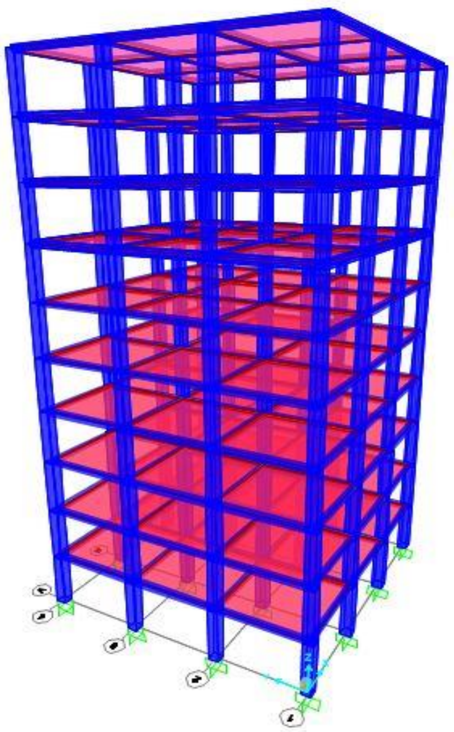


Figure4.5: 3D of High rise building

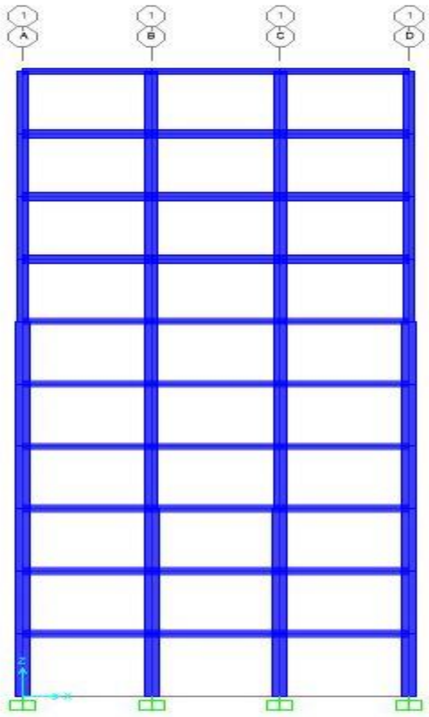


Figure4.6: 2D of High rise building

4.4 Materials and Section

The concrete grade used is M25 with an elastic modulus equal to 25000 MPa. The concrete weight per unit volume is assumed to be 25,000 N/mm2 with poisson's ratio of 0.2. Reinforcement grade HYSD415 TMT with elastic modulus of 200000 MPa is used in the design process. Its unit weight is taken to be 76900 N/m2 and poisson's ratio is fixed to be 0.3.

Column has square section and beams are designed as rectangular section and varied for different floor: For First Floor (250mm*450mm), For floors between first and last floor (250mm*400mm) and For last floor (250mm*300mm).Size of column varied for different buildings and details are given in table below:

Table 4.1: Column detail for low rise building

Type of Building	First Floor	Second and third Floor	Last Floor
Low rise building	450mm*450mm	450mm*450mm	450mm*450mm

Table 4.2: Column detail for midrise building

Type of Building	First Floor	Second to fourth Floor	Fifth to Seventh Floor
Mid rise building	508mm*508mm	508mm*508mm	508mm*508mm

Table 4.3: Column detail for high rise building

Type of Building	First Floor	Second to fourth Floor	Fifth to Seventh Floor
Highrise building	520mm*520mm	520mm*520mm	520mm*520mm

The reduced or effective moment of inertia (Ieff) of the sections for beams Ieff = 0.35Igross and for columns Ieff = 0.70Igross, where Igross is the gross moment of inertia are taken as stated in NBC: 105:2020. Slab is designed as area section of thickness of 127mm.

The non-linearity of beams and columns are represented by developing plastic hinges i.e., beams (M3 hinges) and columns (P-M2-M3 hinges) using default hinges in SAP2000 at their ends. The Takeda hysteresis model is used to define the degradation caused by cyclic loading.

4.5 Loads

Loads are action applied to the structure. Following loads are considered for analysis and calculation.

1. Dead load
2. Dead load of 3.75 Kn/m^2 on slab(inclusive of floor finish)
3. Lateral load
4. Live load of 3 Kn/m^2
5. Wall load as uniformly distributed load on beams. Assuming unit weight of brick wall as 19.2kN/m2 ,

Wall load for external wall of thickness 230mm

= 1*19.2*0.23*(3.2) =14.13 Kn/m (Deduction of Beam depth has been neglected for simplicity)

4.5.1 Seismic Weight

The total seismic weight of the structure, W, shall be taken as the sum of the dead loads and factored seismic live loads, i.e. $W = DL + \lambda\ LL$

Where, W is the total seismic weight of the structure,

DL is the total dead load of the structure which includes the self-weight of the structural elements, floor finish and wall loads

LL is the live load and λ is the live load participation factor. It is taken as 0.30 in this study

4.5.2 Load combination

For the design of structure, the seismic load effect is combined with other effects. The following load combination is adopted to design the structure.

$1.2DL + 1.5LL$

$DL + \lambda LL \pm E$

Where, $\lambda = 0.3$ is taken for this study.

CHAPTER 5: SELECTION OF GROUND MOTION DATA

Selection of ground motion data is very important step in IDA. All real earthquake data are chosen for study and no artificial data are generated for both main shock and after shock ground motions. In this study, mainshock and aftershock of earthquakes having moment magnitude greater than 5.5 and lesser than 8.0 are considered. The After Shock data for the Chosen earthquakes are taken for the same station as that of Main shock. Only the Horizontal components of earthquake data for both main shock and after shock are taken for the study. The Earthquake record data are downloaded from Pacific Earthquake Engineering Research (PEER) Center and Consortium of Organizations for Strong-Motion Observation Systems (COSMOS) database.

5.1 Selection of ground motion data

For correct evaluation of responses of structure to dynamic loading, it is very crucial to consider adequate number of ground motion data. According to NBC 105:2020 , If less than 7 numbers of ground motion records are considered for analysis, maximum values of the response quantities from these ground motions shall be used. If the number of ground motions used is more than 7, then average values of the considered number of ground motions shall be used for evaluation of response quantities. Also according to the ASCE/SEI-7, if at least seven ground motions are analyzed, the design values of engineering demand parameters (EDPs) are taken as the average of the EDPs determined from the analyses. So, Seven Real earthquake ground motions are taken for this study.

The table below shows seven main-shock and corresponding aftershock data. For this study, earthquake data of only one orthogonal direction is applied to structure. No artificial earthquakes have been generated for this study.

Table 5.1: List of earthquake ground motions

SN	Earthquake	Station	Date	Magnitude	Source	Denotation
1	Irpinia Italy	Auletta	11/23/1980	6.90	PEER	Irp-1
	Irpinia Italy	Auletta	11/23/1980	6.20	PEER	Irp-2
2	Northridge	Anaverde valley	01/17/1994	6.69	PEER	Nor-1
	Northridge	Anaverde	01/17/1994	6.05	PEER	Nor-2

		valley				
3	Gorkha	KATNP	25/04/2015	7.80	COSMOS	Gkh-1
	Gorkha	KATNP	12/05/2015	7.30	COSMOS	Gkh-2
4	India-Burma border	Berlongfer	08-06-1988	7.20	COSMOS	Inb-1
	India-Burma border	Berlongfer	01-09-1990	6.10	COSMOS	Inb-2
5	Friuli	Tolmezzo	05-06-1976	6.50	PEER	Fri-1
	Friuli	Tolmezzo	05-07-1976	5.20	PEER	Fri-2
6	Hollister	Hollister city hall	04-09-1961	5.60	PEER	Hol-1
	Hollister	Hollister city hall	04-09-1961	5.50	PEER	Hol-2
7	Livermore	APEEL 3E Hayward CSUH	01-24-1980	5.80	PEER	Liv-1
	Livermore	APEEL 3E Hayward CSUH	01-27-1980	5.42	PEER	Liv-2

5.2 Matching and Combining ground motion data

The selected ground motion records taken from data centers should be scaled to match certain target response spectrum of specified location to meet the specified level of seismic hazard as per site location. In this study the target spectrum is response spectrum provided in NBC: 105: 2020. Seismomatch software version 2020 is used for matching and scaling of above selected ground motion data. Selected ground motion records are scaled up or scaled down to match with the target response spectrum provided in NBC: 105:2020 for Kathmandu area using appropriate scale factor. The scaled factor used to match the target spectrum are calculated so that it lied between periods Tn and $\sqrt{R\mu}xT1$, where T1 is the fundamental period of vibration of the structure, Tn is the period of the highest vibration mode to ensure 90% mass participation and Rμ is the ULS ductility factor as stated in NBC:105: 2020.

Description of seismic hazard of location as per NBC: 105: 2020

Seismic Zone factor = 0.35

Soil type= Very soft soil

Importance class of building = 'I'

Structural importance factor = 1

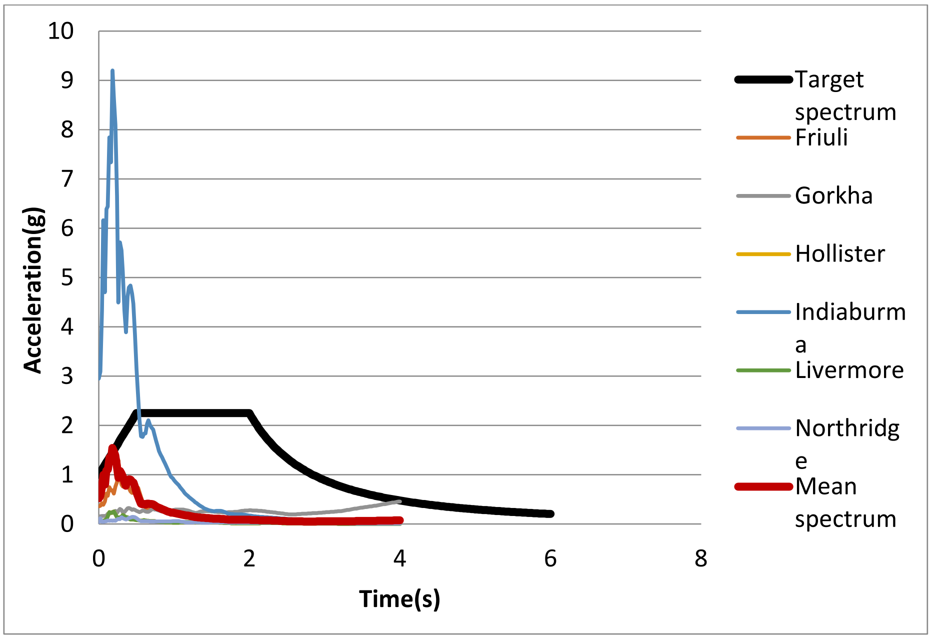


Figure5.1: Unmatched response spectrum of Main shock ground motions

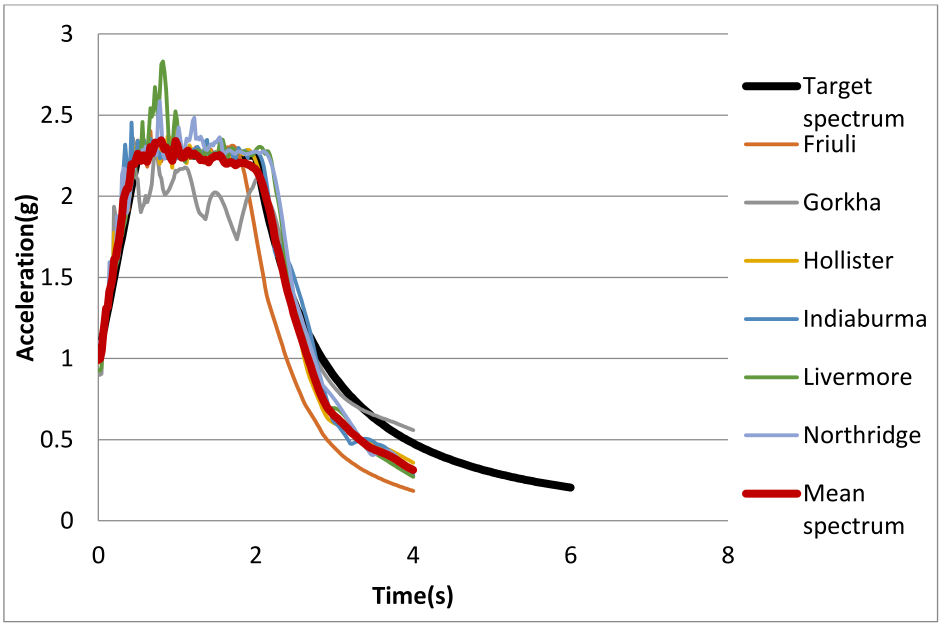


Figure5.2 Matched response spectrum of combined ground motions

The matched data of main-shock and aftershock are combined together by keeping a time interval of 100 seconds in between two ground motions. This gap has zero acceleration ordinates and is absolutely enough to cease the moving of any structure due to damping (Liolios, 2010). Thus sequential earthquake is formed which is used for the nonlinear time history analysis. The accelerograms of seven sequential earthquakes generated (i.e. Irp, Nor, Gkh, Hol, Inb , Fri, Liv) from their corresponding single accelerograms (Irp1 and Irp2, Nor1 and Nor2, Gkh1 and Gkh2, Hol1 and Hol2, Inb1 and Inb2, Fri1 and Fri2, Liv1 and Liv2) are shown in Figure5.3 to Figure 5.9.

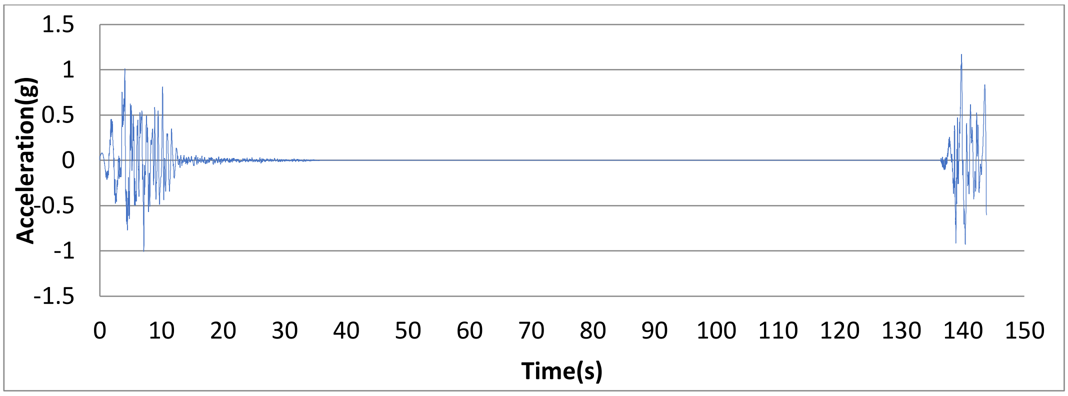


Figure5.3: Sequential earthquake (Friuli)

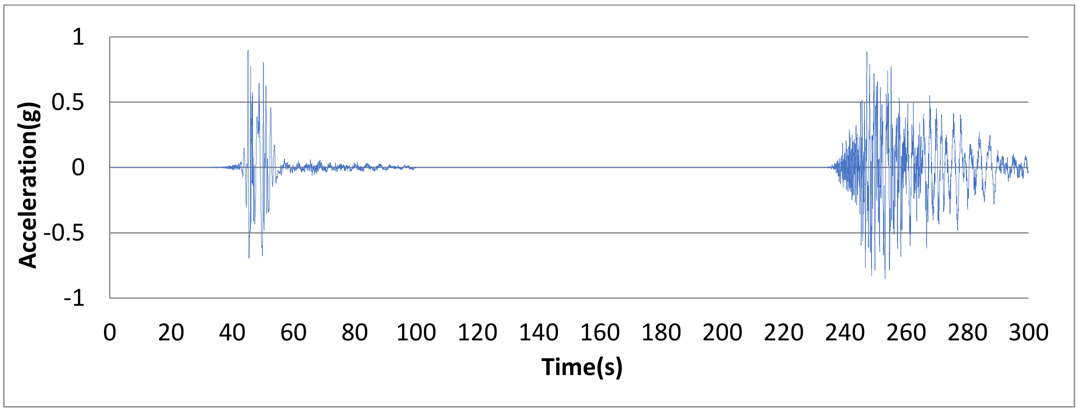


Figure5.4: Sequential earthquake (Gorkha)

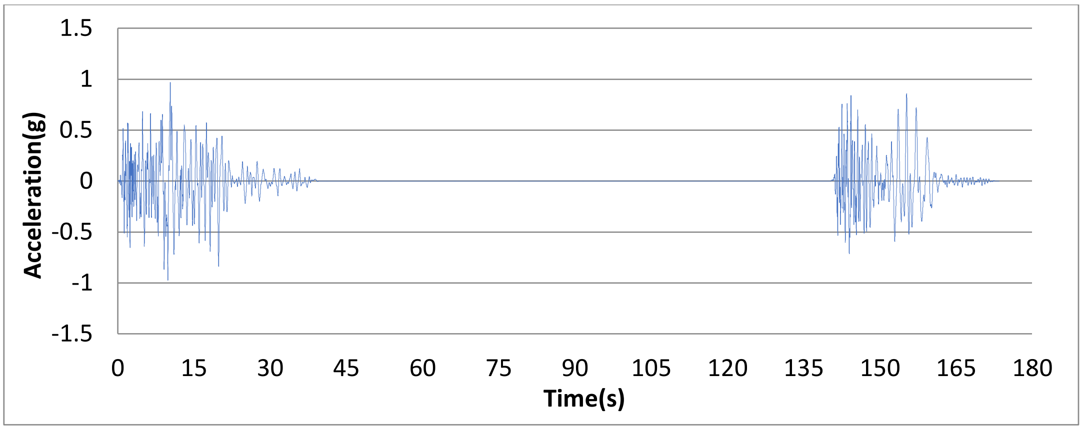


Figure5.5: Sequential earthquake (Hollister)

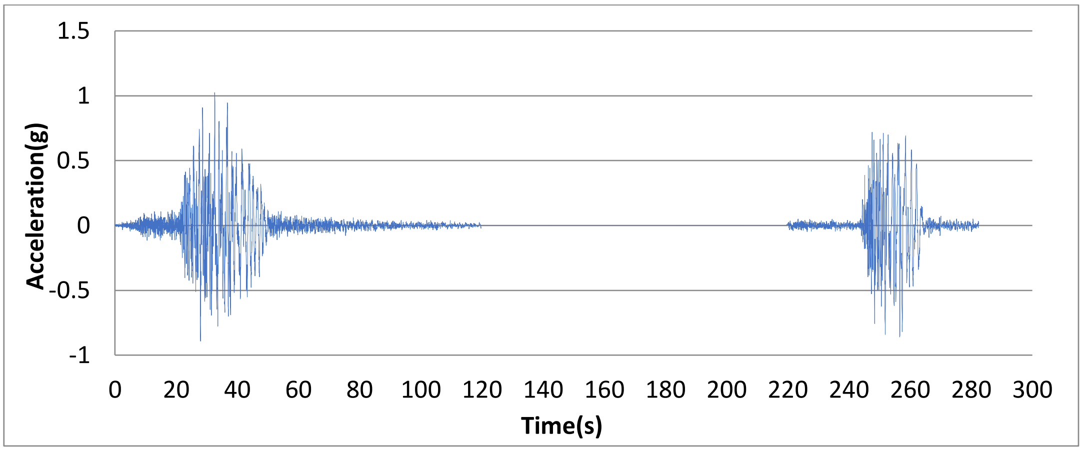


Figure5.6: Sequential earthquake (Indiaburma)

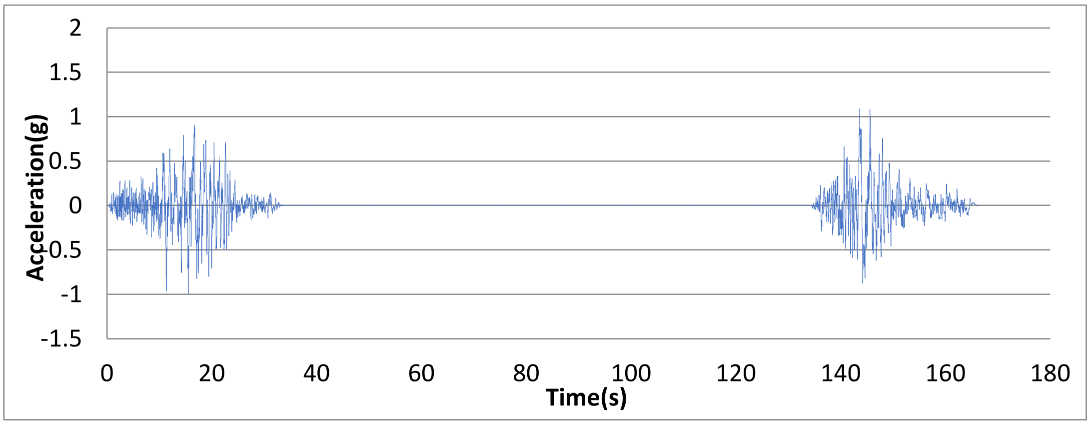


Figure5.7: Sequential earthquake (Irpinia)

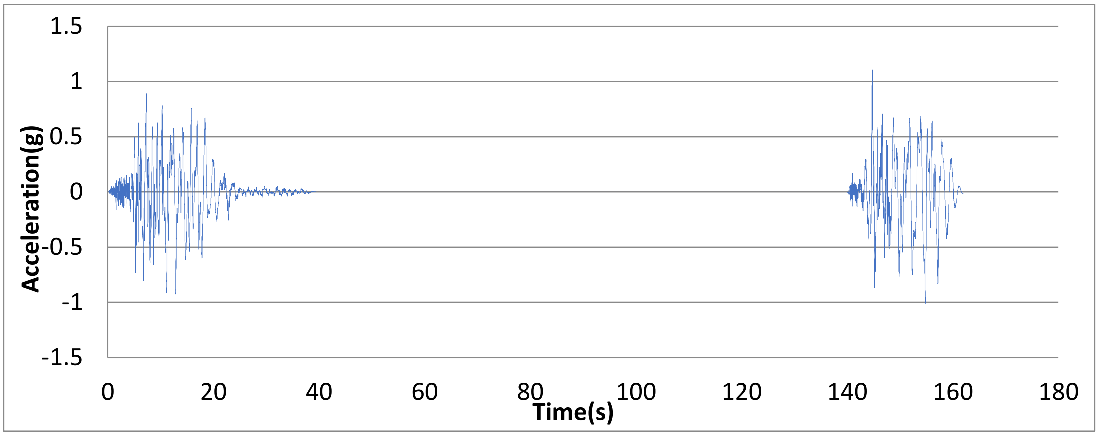


Figure5.8: Sequential earthquake (Livermore)

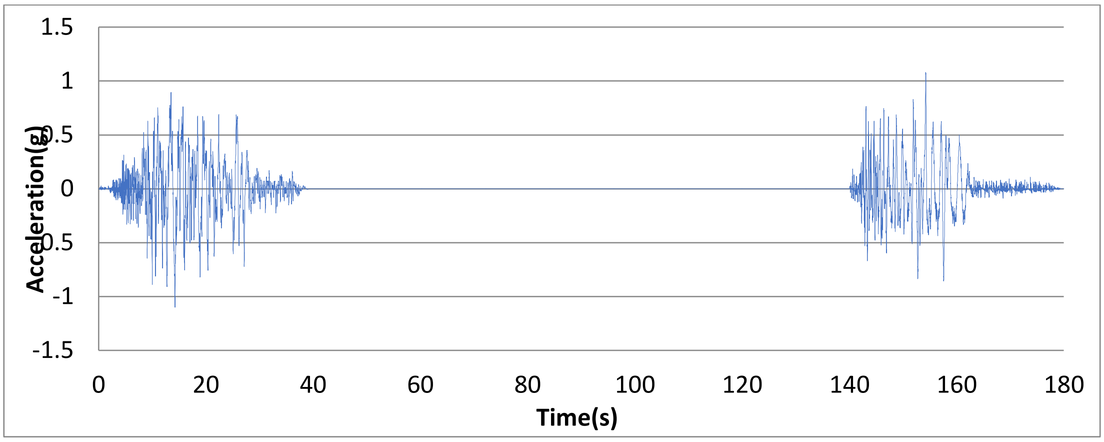


Figure5.9: Sequential earthquake (Northridge)

CHAPTER 6: RESULTS AND DISCUSSIONS

Moment-resisting RC building frame (OMRF) consisting (Low rise- 4 story, Midrise- 7 story, High rise- 10 story) situated in Kathmandu City are considered in this study. The structure is designed by equivalent static method and all members are checked to see if they are capable of resisting the applied loads. Non-linearity of beams and columns were modeled with lumped plasticity at their ends. The Takeda hysteresis model is used to define degradation under cyclic loading in the SAP2000 software, as shown in Figure 6.1.

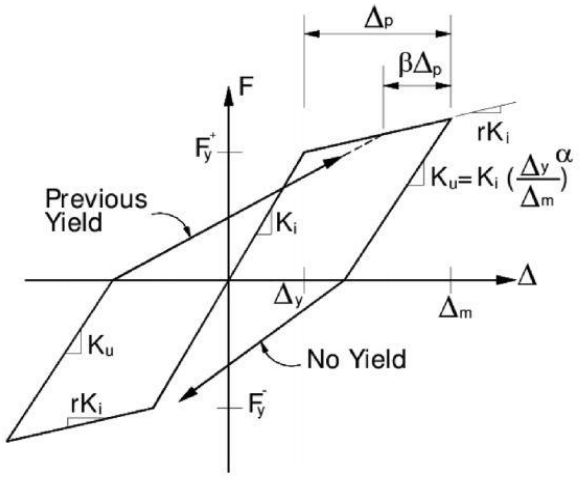


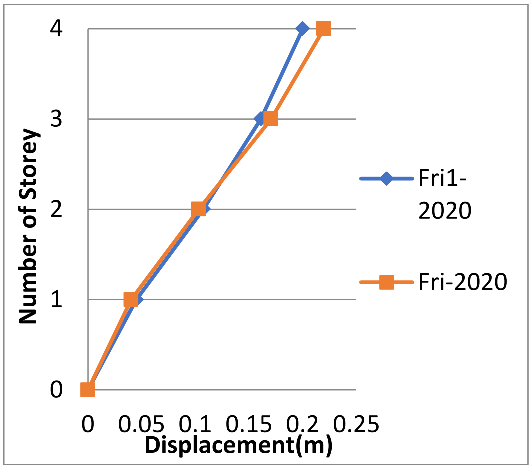
Figure6.1: Takeda hysteresis model

For non-linear dynamic analysis, a nonlinear gravity case is defined as the initial case which includes the total dead load plus 30% live load. The Time history load case is applied at the end of non-linear gravity case. Further, in this present study, the Peak Ground Acceleration (PGA) is chosen as the IM, and the maximum inter-story drift ratio (IDR) is chosen as the EDP. For time integration method, Newmark-beta method is used and for considering geometric non-linearity effects in the models, P-delata effects are also taken into account. In order to carryout IDA, each earthquake ground motion data has been scaled down or scaled up to targeted designed peak ground acceleration. For this study, ground motion data are scaled to target PGA at the interval of 0.05g until the collapse state is reached. In the IDA approach, the engineering demand parameter (EDP), i.e., IDR is monitored as a 3% threshold was adopted to designate the collapse state of the structure as stated in (Xue, Wu, Chen, & Chen, 2007).

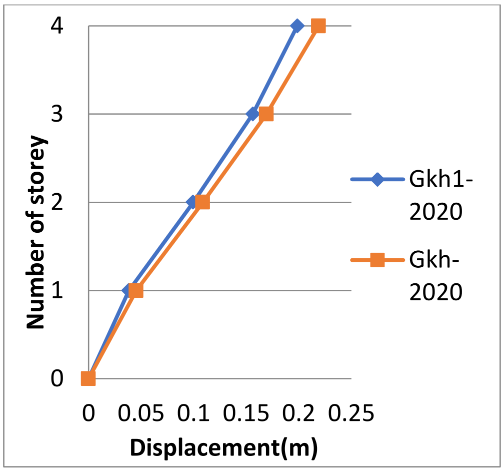
The seismic behavior of the building is monitored and expressed in terms of following parameters:

6.1 Maximum Lateral Storey Displacement

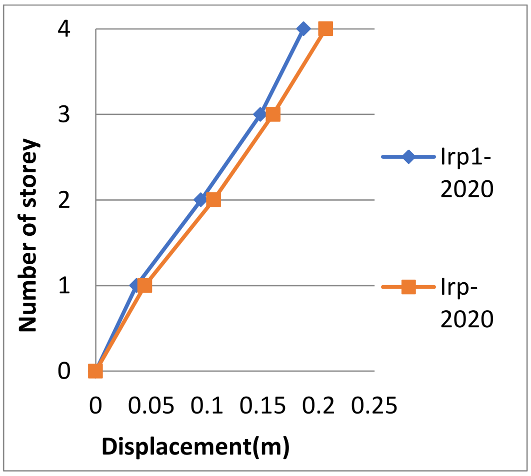
Studying drift limits are the most common method to study the response of structures. So, maximum lateral storey displacement is chosen as one of the parameter to present the results of this study. In order to study the effect of sequence earthquakes, the maximum lateral story displacements are computed for all three structural models under all single and combined earthquake data. The comparison of results at a particular intensity measure is shown in Figure 6.2, Figure 6.3 and Figure 6.4.



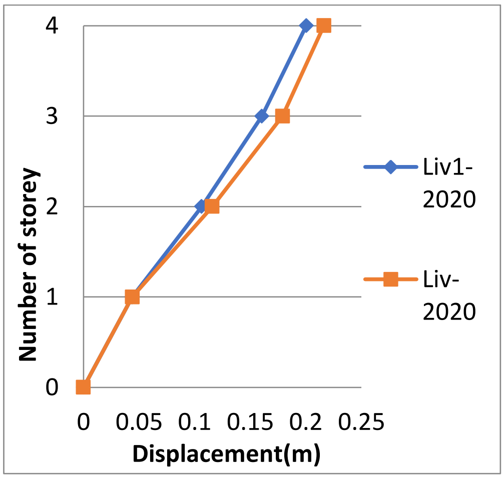
(a)



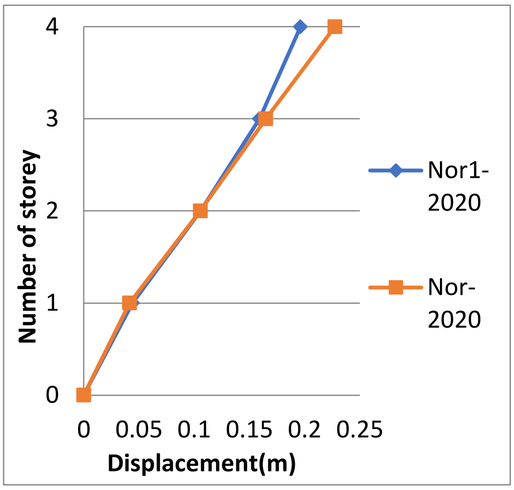
(b)



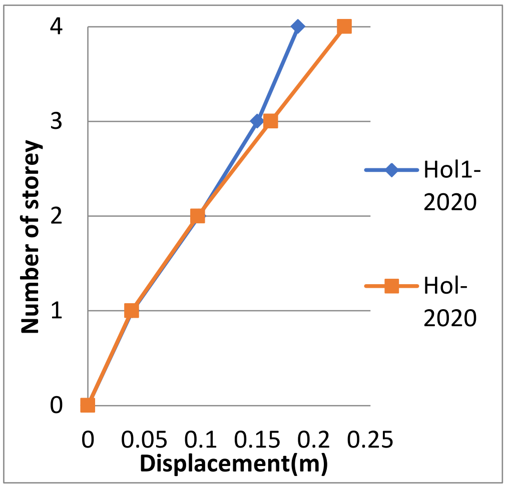
(c)



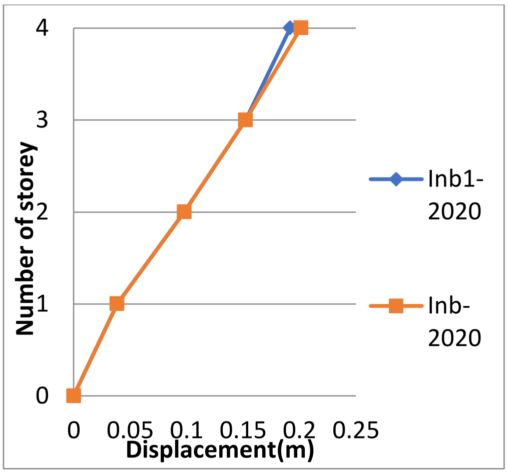
(d)



(e)

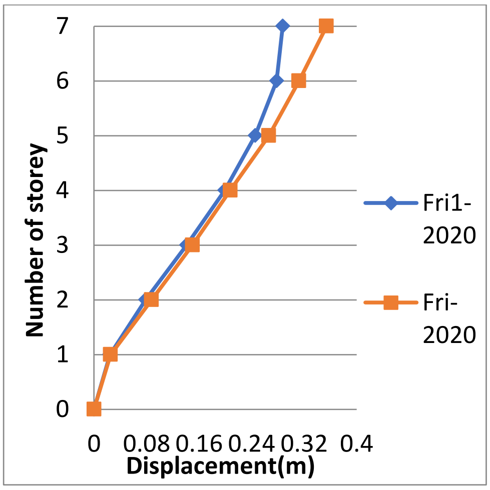


(f)

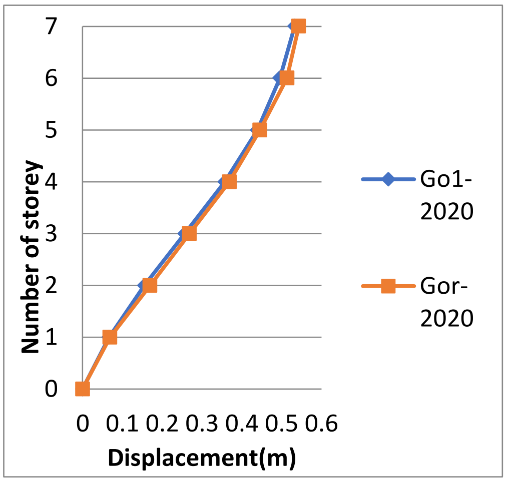


(g)

Figure6.2: Comparison of maximum displacement for main shock and combined shock for low rise structure



(a)



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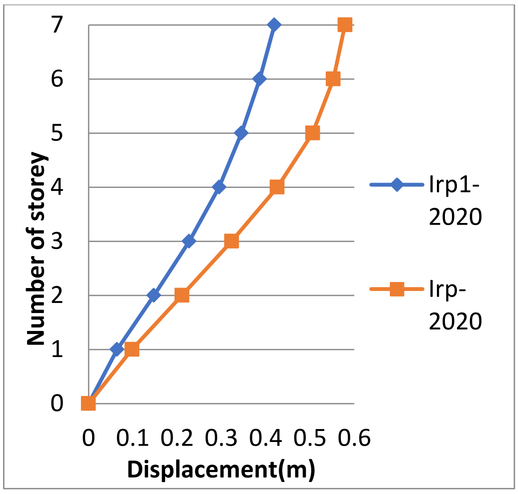
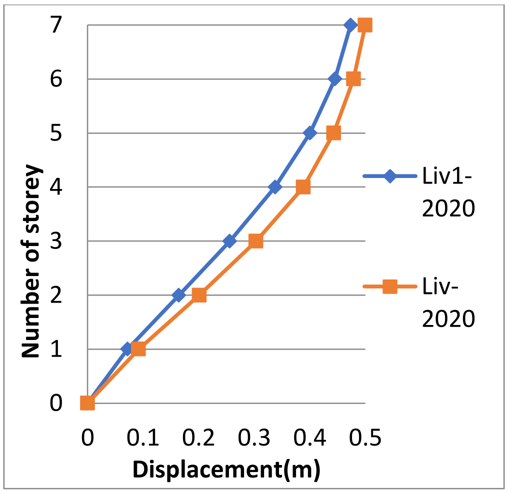
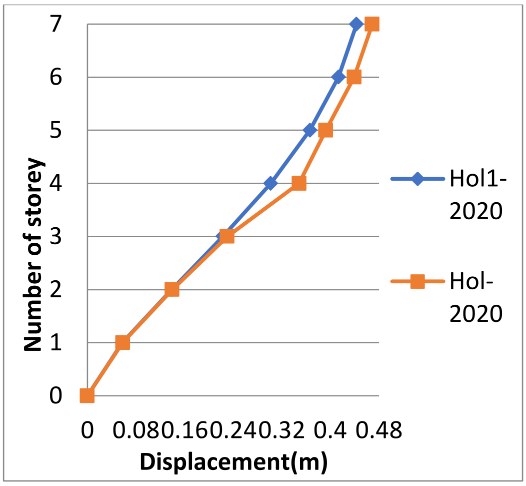


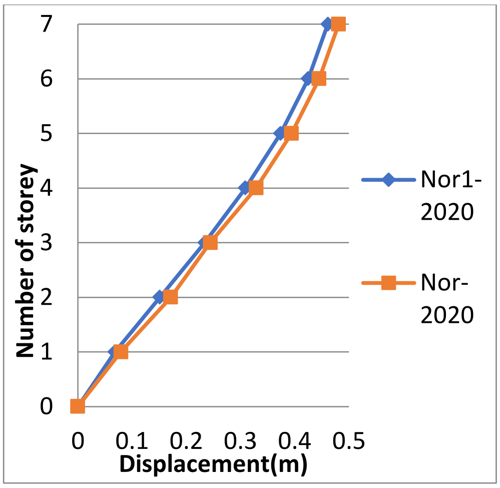
(c)



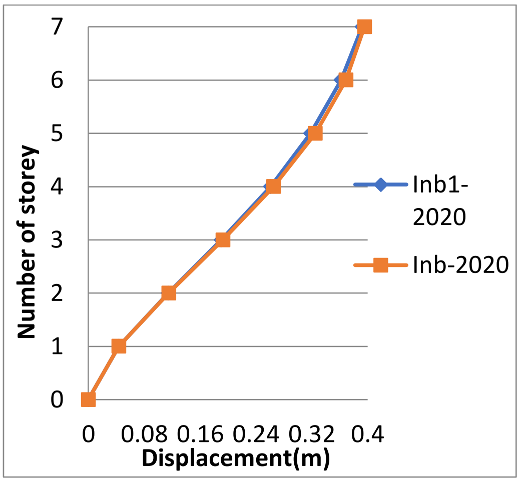
(d)



(e)

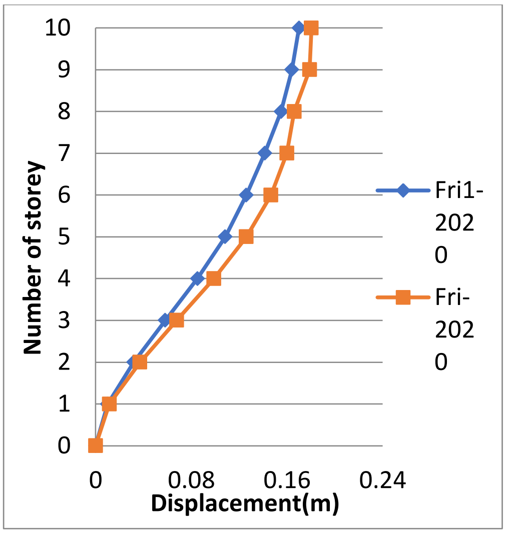


(f)

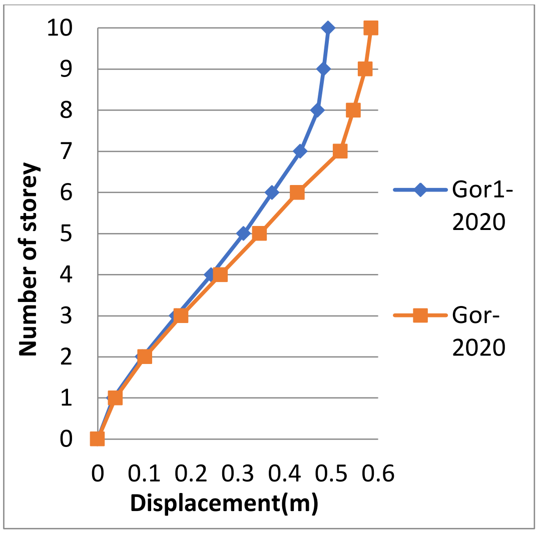


(g)

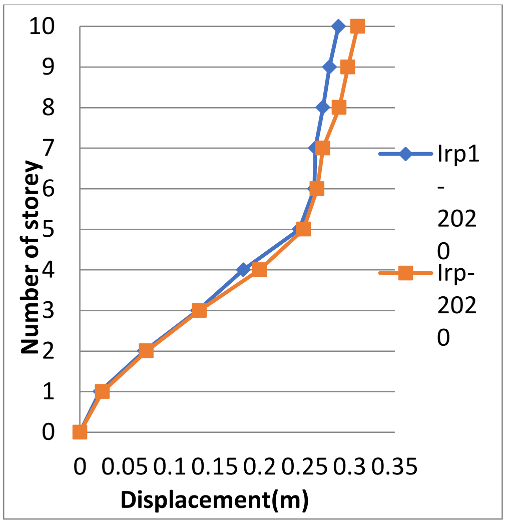
Figure6.3: Comparison of maximum displacement for main shock and combined shock for midrise structure


Number of storey
Displacement(m)
0
0.08
0.16
0.24
Fri1-2020
Fri-2020

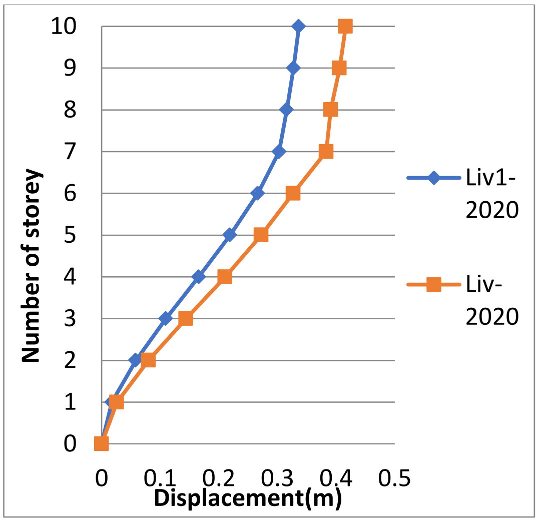
(a)


Number of storey
Displacement(m)
0 0.1 0.2 0.3 0.4 0.5 0.6
Gor1-2020
Gor-2020

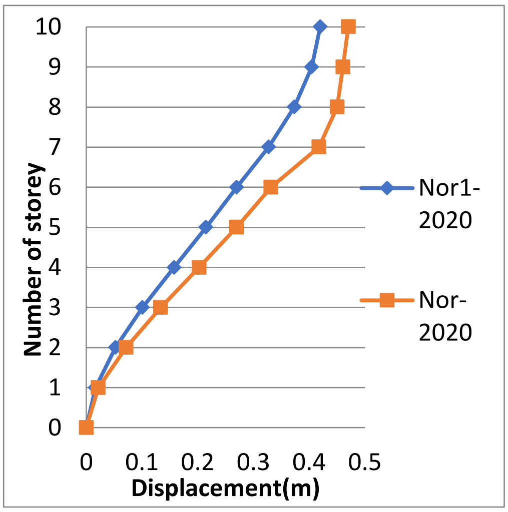
(b)


Number of storey
Displacement(m)
0 0.05 0.1 0.15 0.2 0.25 0.3 0.35
Irp1-2020
Irp-2020

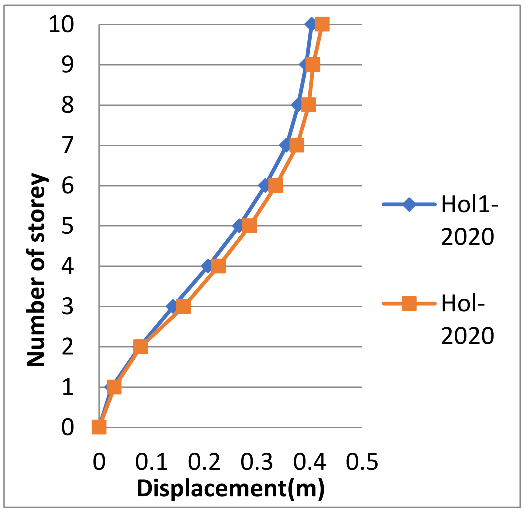
(c)


Number of storey
Displacement(m)
0 0.1 0.2 0.3 0.4 0.5
Liv1-2020
Liv-2020

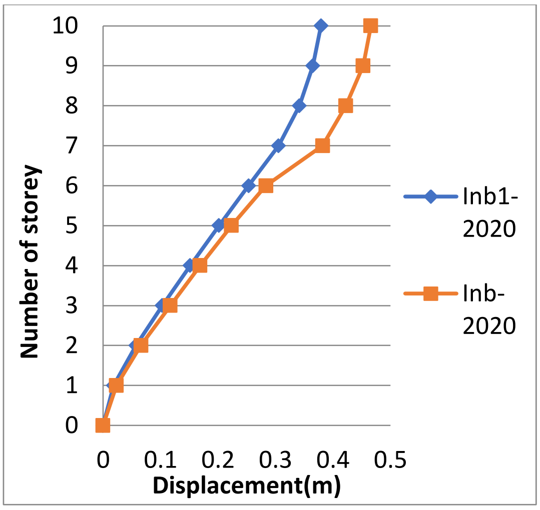
(d)


Number of storey
Displacement(m)
0 0.1 0.2 0.3 0.4 0.5
Nor1-2020
Nor-2020

(e)


Number of storey
Displacement(m)
0 0.1 0.2 0.3 0.4 0.5
Hol1-2020
Hol-2020

(f)



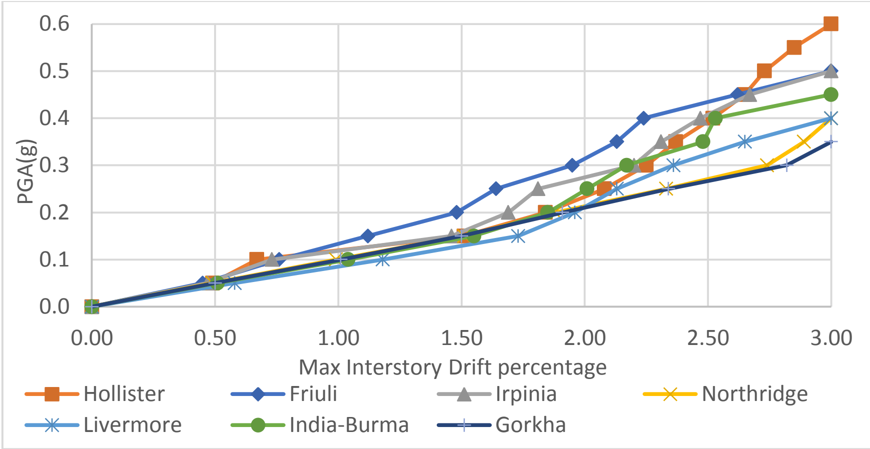
(g)

Figure6.4: Comparison of maximum displacement for main shock and combined shock for high rise structure

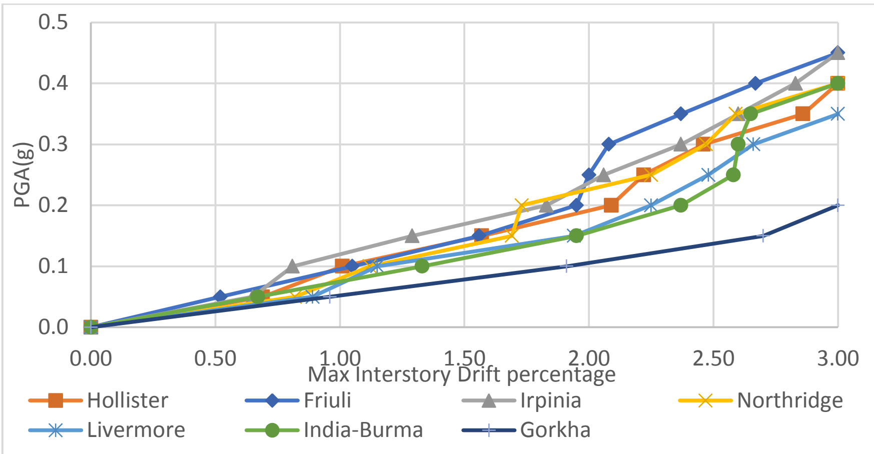
From the Figures6.2, Figure6.3 and Figure6.4, we can clearly observe that Lateral displacement of building increases significantly in case of sequence earthquakes. The average increase in roof displacements in the case of seismic sequences was found to be of the order of 11.58%, 18% and 16% for the low rise, midrise and high rise configurations respectively. The increment in roof displacement might be because the building faces reduction in strength, changes in stiffness characteristics when it faces the major shock. And when it is further exposed sequence earthquakes, building experiences more damage than during major earthquake. Also, the low rise structure seems to perform better during sequence earthquake than midrise and high rise structures. However, as seen in results the mean roof displacements in high rise structure is less than that of mid-rise structure which may be because of the dispersion available in ground motions with respect to the period and ductility of structures.

6.2 Maximum Inter-story Drift Ratio (IDR)

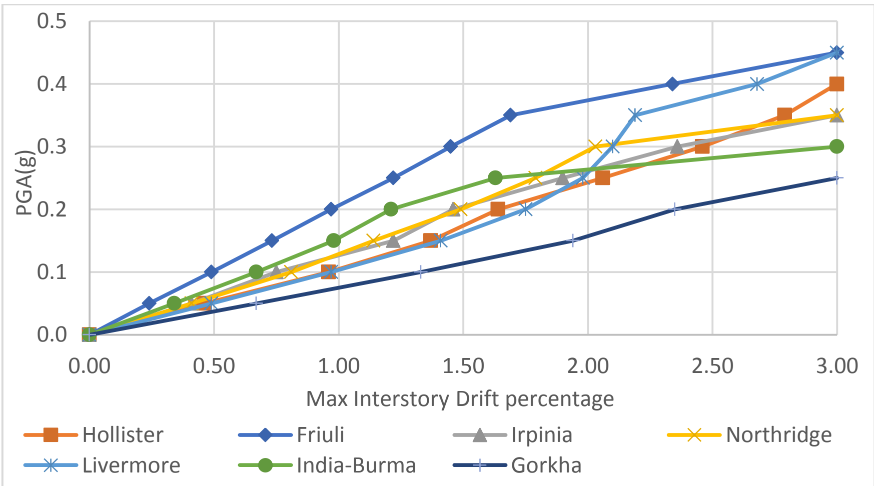
Among different parameters, Maximum inter-story drift ratio is chosen for the Engineering demand parameter for this study. So IDR is used as EDP to develop IDA curves and later fragility curves. The IDA curves developed here are the outcome of around 310 simulations of non-linear time history analysis performed on three different types of structural configurations under 7 single earthquake ground motions and 7 seismic sequences. These curves are plotted between PGA and maximum inter-story drift ratios (IDR) as shown in Figures 6.5 and Figure 6.6.



(a)

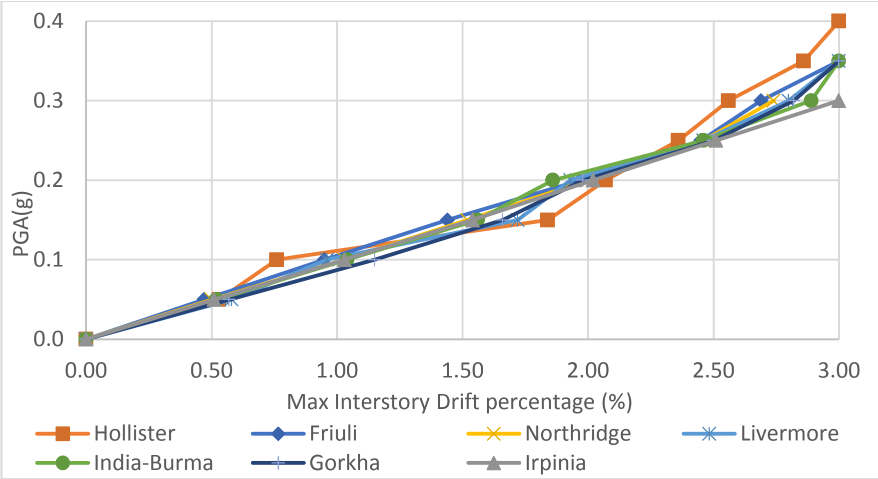


(b)

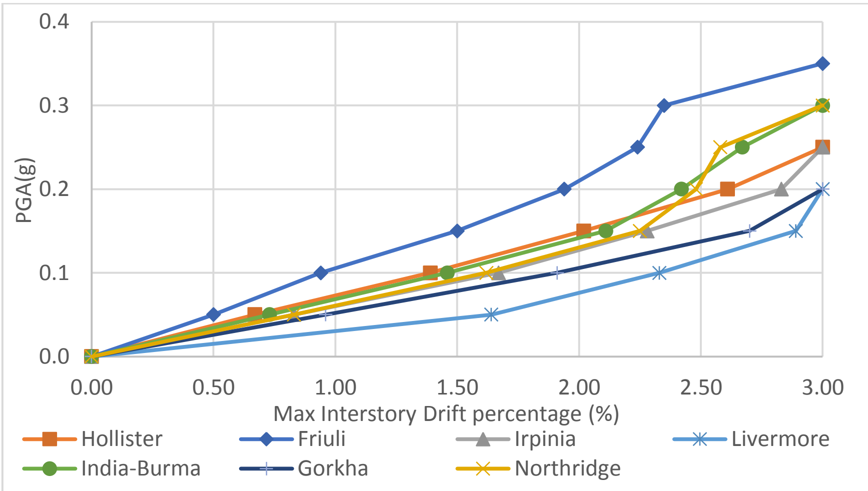


(c)

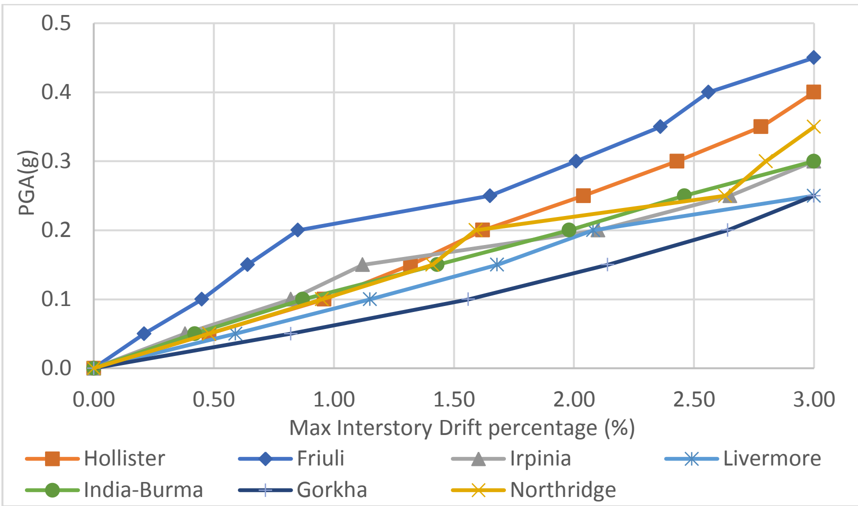
Figure6.5: IDA curve for Main shock: (a) Low rise (b) Midrise (c) High rise



(a)



(b)



(c)

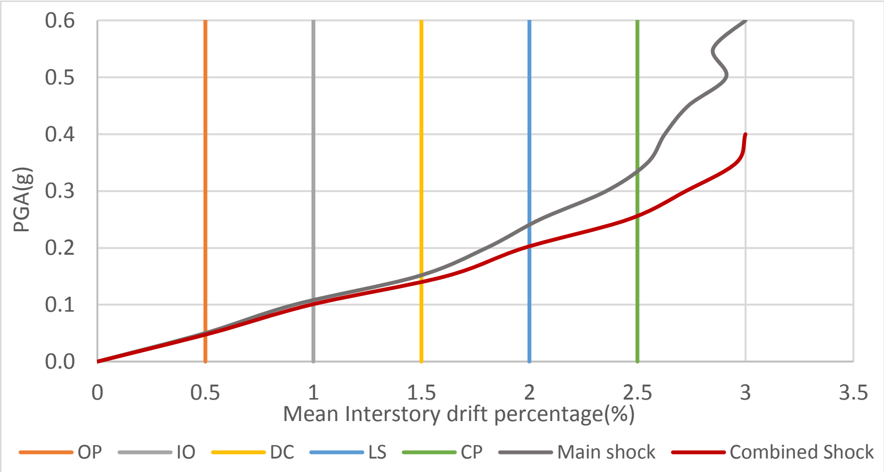
Figure 6.6: IDA curve for Sequence earthquake: (a) Low rise (b) Midrise (c) High rise

6.2.1: Mean IDA Curve

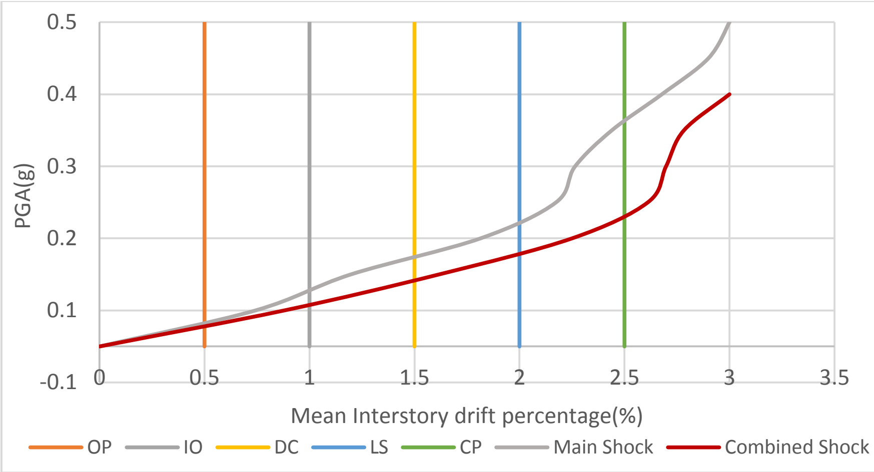
After developing IDA curves, mean IDA curve is generated by taking mean of all the IDA curves. So, Figure6.7 is developed by taking mean of IDA curves in Figure 6.5 and Figure 6.6 respectively. Mean IDA curve is further used to extract fragility parameters to perform fragility analysis of structures.

Table 6 .1: Defining limit states

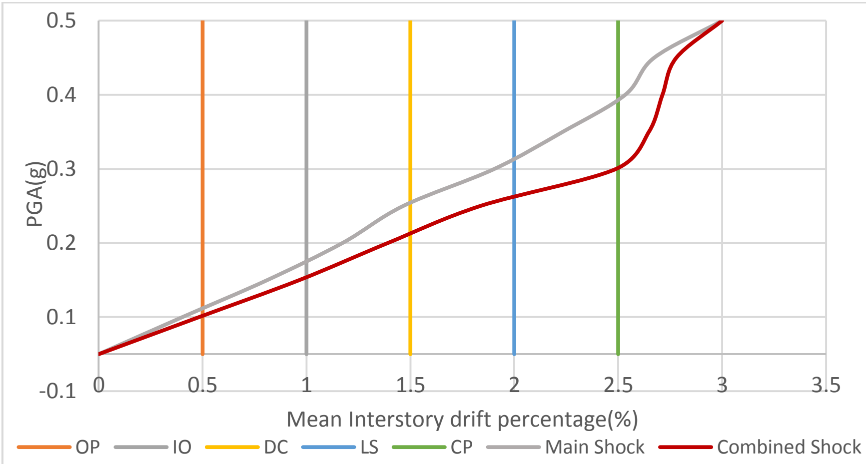
Limit state	Drift %
Operational phase(OP)	0.5
Immediate occupancy (IO)	1.0
Damage control (DC)	1.5
Life safety (LS)	2.0
Collapse prevention (CP)	2.5



(a)



(b)



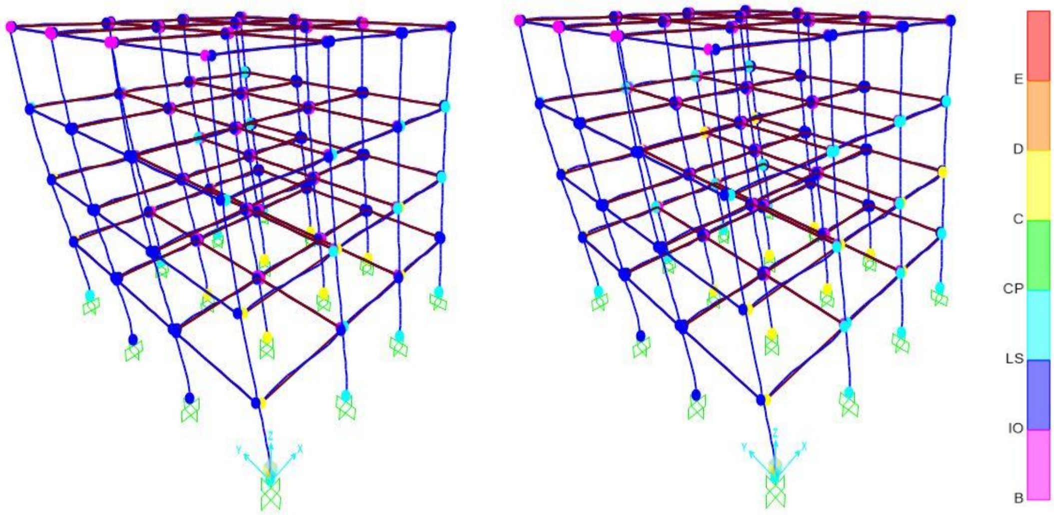
(c)

Figure6.7: Comparison of Mean IDA curve of Main shock and Sequence earthquake: (a) Low rise (b) Midrise (c) High rise

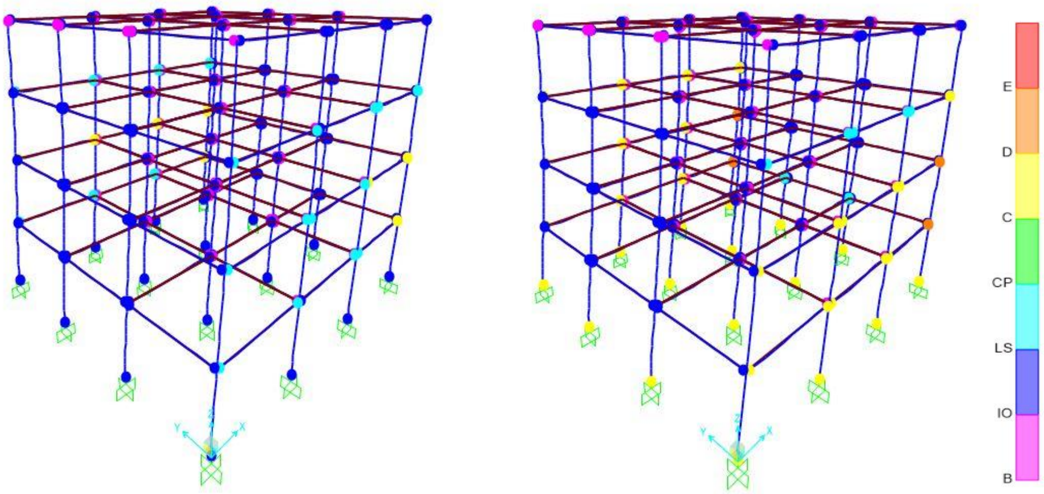
As seen in Figure6.7, it can be observed that when low rise, midrise and high rise structures are subjected to sequence earthquakes, they reached collapse point at much lower PGA values i.e,0.25g, 0.23 and 0.3g respectively whereas while subjecting to only single earthquake, they collapsed at 0.33g,0.36gand 0.38g. After studying these graphs from Figure6.10, it is evident that buildings reaches collapse limit state (3% IDR in our study) at lower IM, i.e., at lower PGA value under repeated earthquake force compared to that of individual earthquake forces. This justifies that when the building is exposed to further aftershocks after being hit by major shocks within small time interval, building further lose its capability to bear the seismic force gradually and reaches the collapse point more sooner.

6.3 Plastic Hinge Pattern

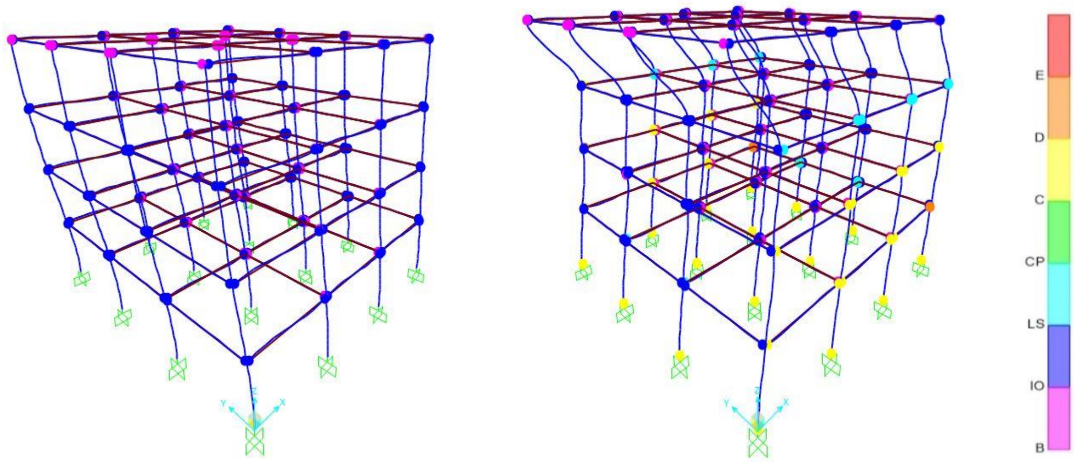
The plastic hinge formation in an RC building is an indicator of structural damage induced when subjected to seismic events. The severity of plastic hinge pattern for the structural models under seismic forces (Fri1 and Fri, Gkh1 and Gkh , Hol1 and Hol, Inb1 and Inb) for low rise, midrise and high rise structures at just before reaching collapse state is expressed in Figure6.8, Figure6.9 and Figure6.10 respectively. The legends in Figures. 6.8 to 6.10, describes different damage states of plastic hinges (i.e., IO: immediate occupancy, LS: life safety, CP: collapse prevention) with appropriate labels and colors, as per FEMA 356.



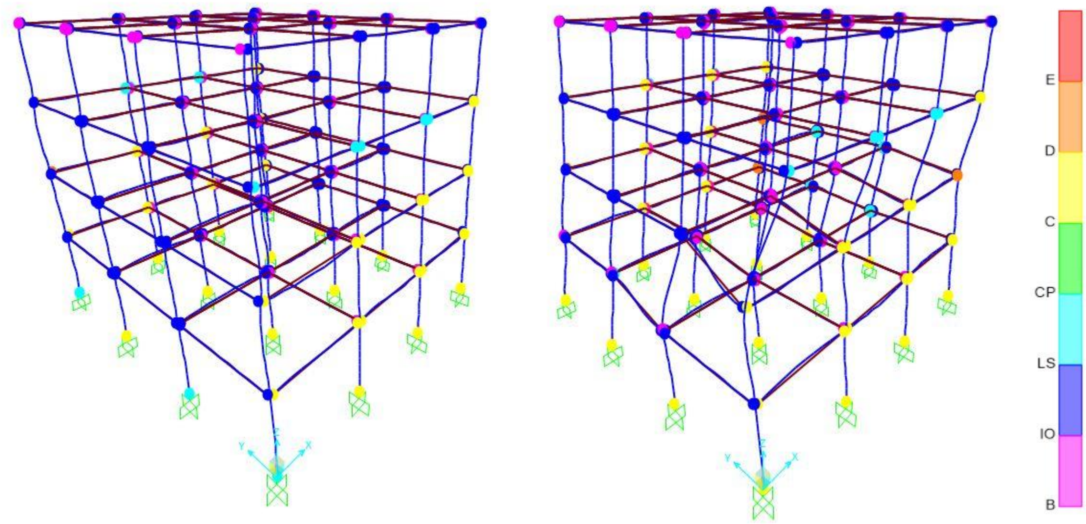
(a) Main shock earthquake (Fri)   (b) Sequence earthquake (Fri)



(c) Main shock earthquake (Gkh)   (d) Sequence earthquake (Gkh)

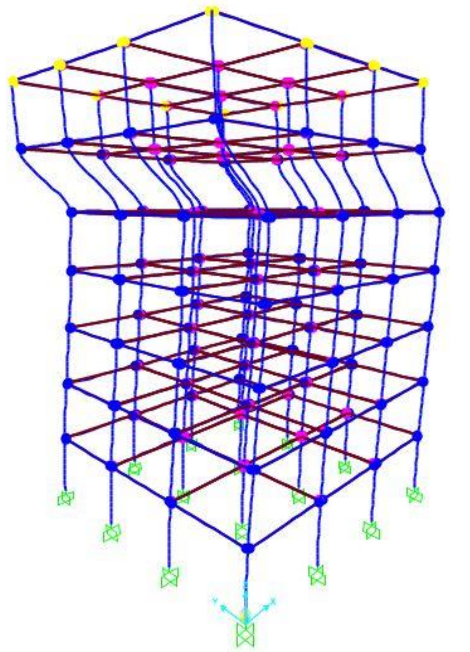


(e) Main shock earthquake (Hol) (f) Sequence earthquake (Hol)

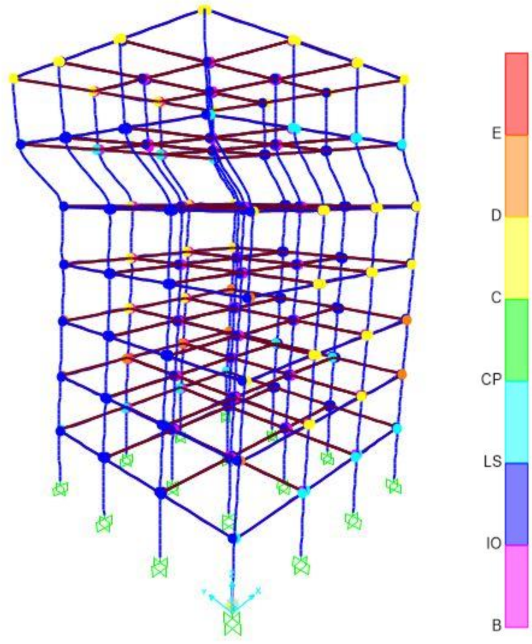


(g) Main shock earthquake (Inb) (h) Sequence earthquake (Inb)

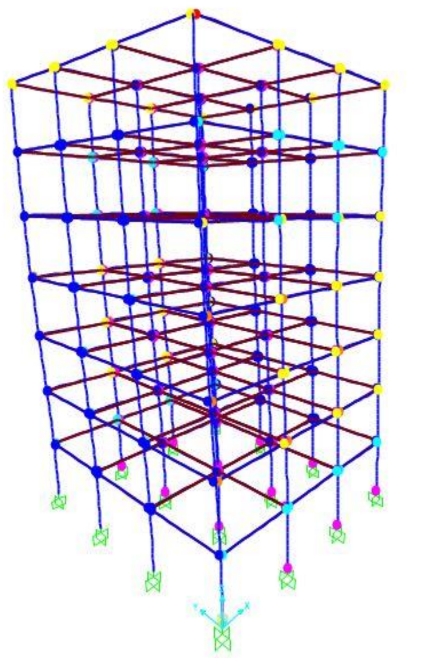
Figure6.8: Comparison of Hinges pattern for different earthquakes for low rise building



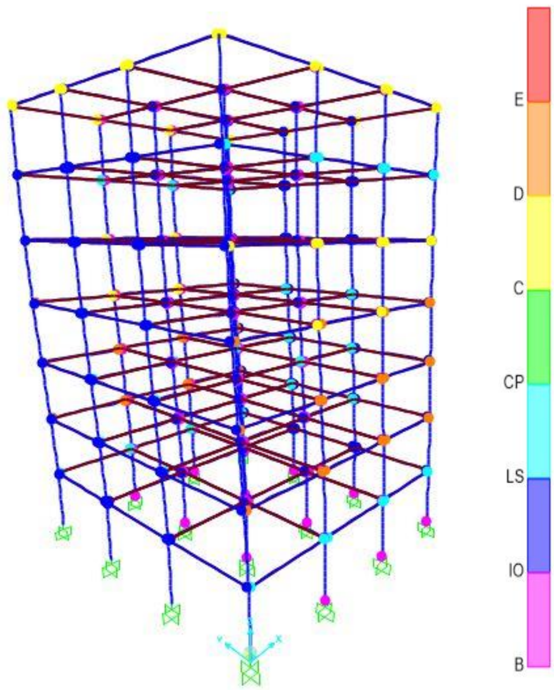
(a) Main shock earthquake (Fri)



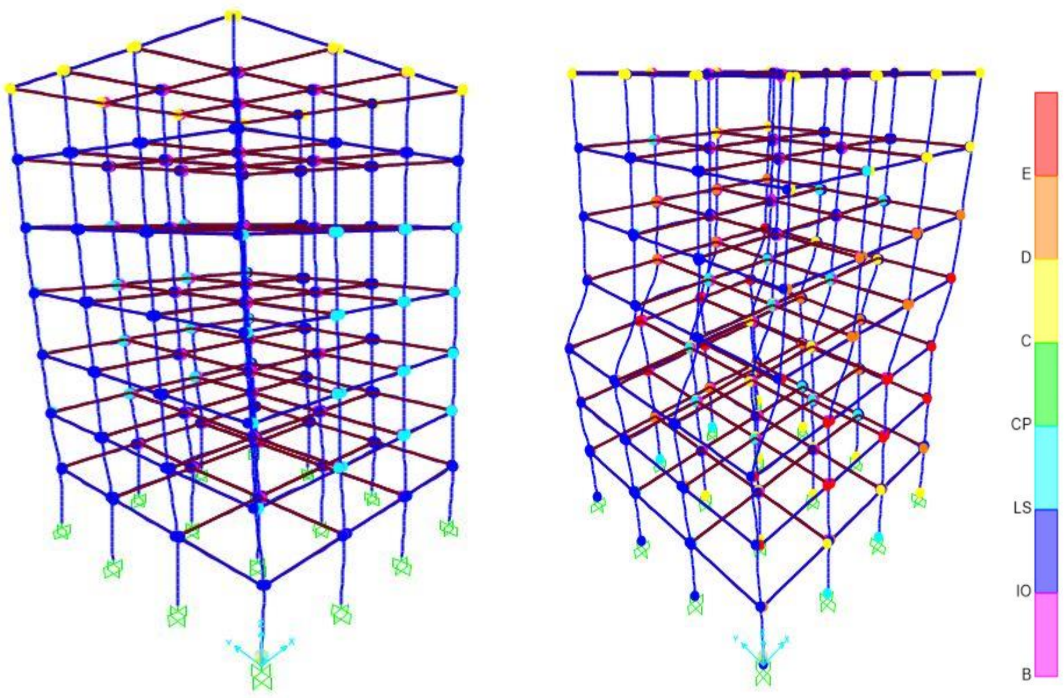
(b) Sequence earthquake (Fri)



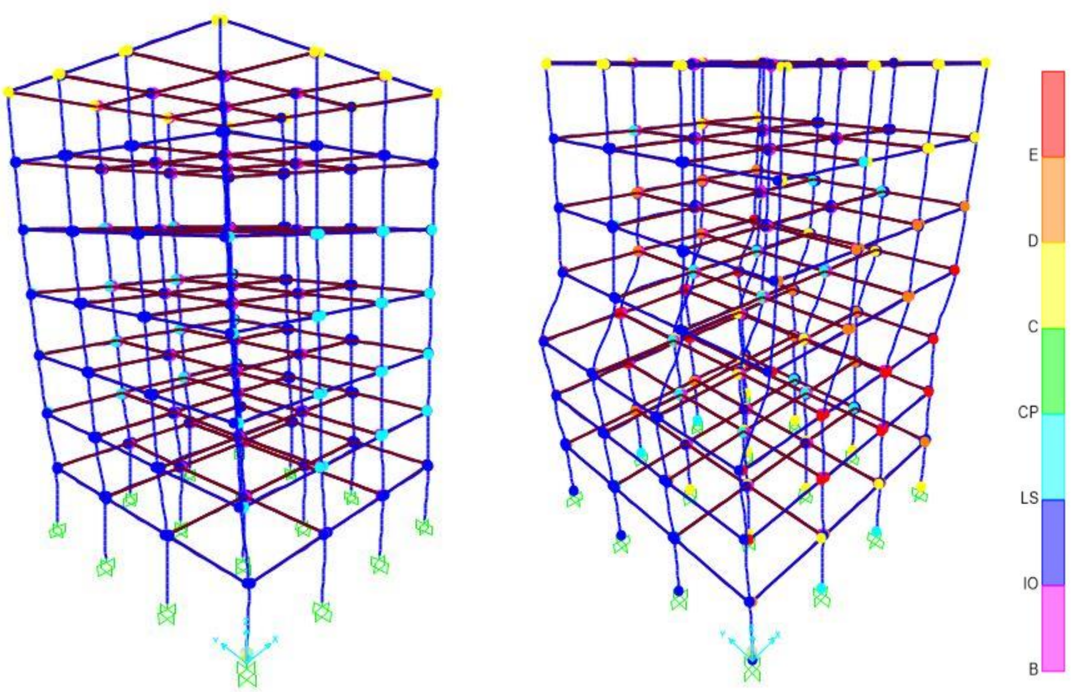
(c) Main shock earthquake (Gkh)



(d) Sequence earthquake (Gkh)

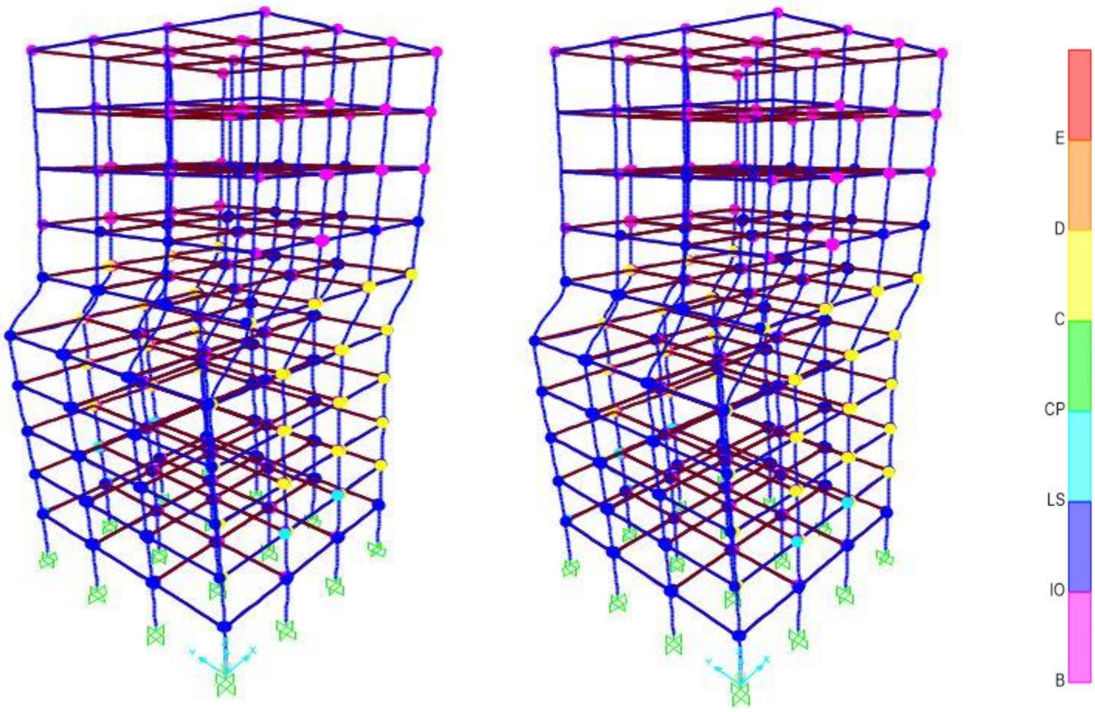


(e) Main shock earthquake (Hol) (f) Sequence earthquake (Hol)



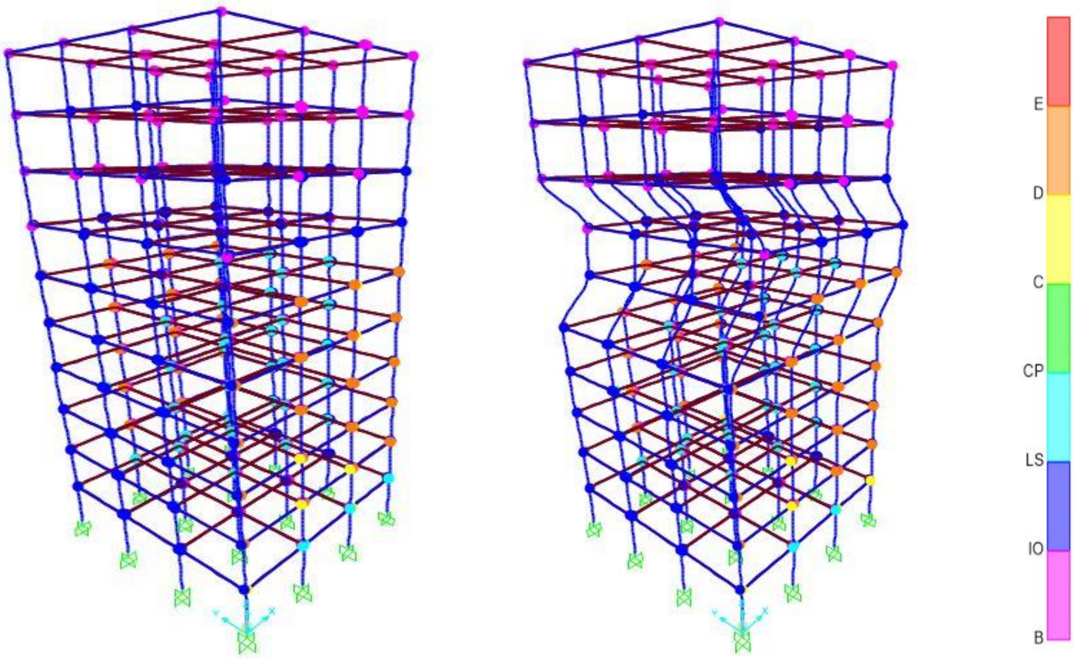
(g) Main shock earthquake (Inb) (h) Sequence earthquake (Inb)

Figure6.9: Comparison of Hinges pattern for different earthquakes for Midrise building



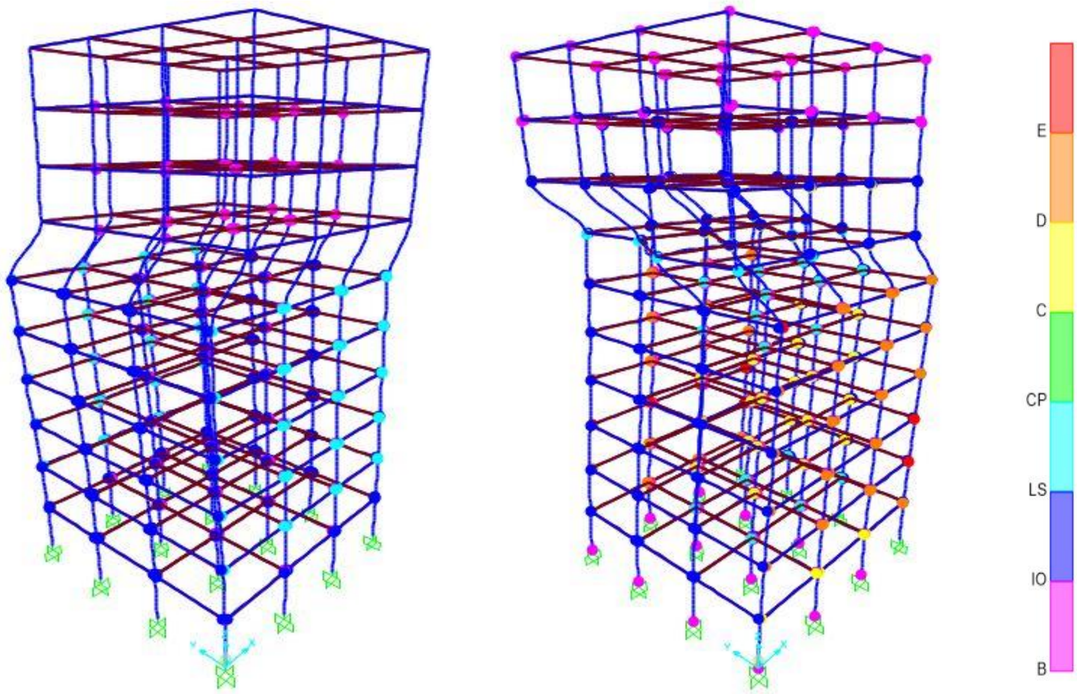
(a) Main shock earthquake (Fri)

(b) Sequence earthquake (Fri)

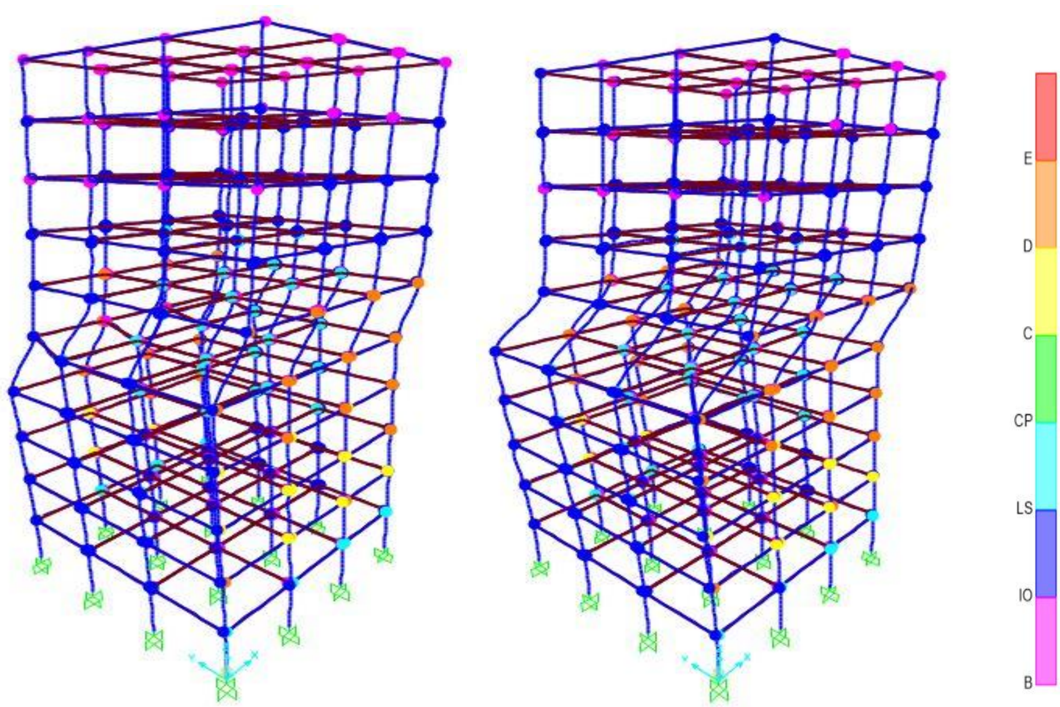


(c) Main shock earthquake (Gkh)

(d) Sequence earthquake (Gkh)



(e) Main shock earthquake (Hol) (f) Sequence earthquake (Hol)



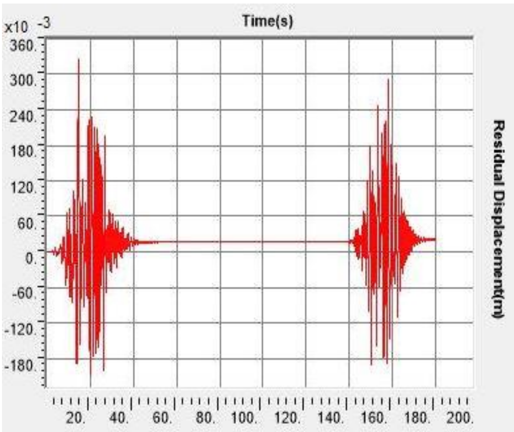
(g) Main shock earthquake (Inb) (h) Sequence earthquake (Inb)

Figure6.10: Comparison of Hinges pattern for different earthquakes for High rise building

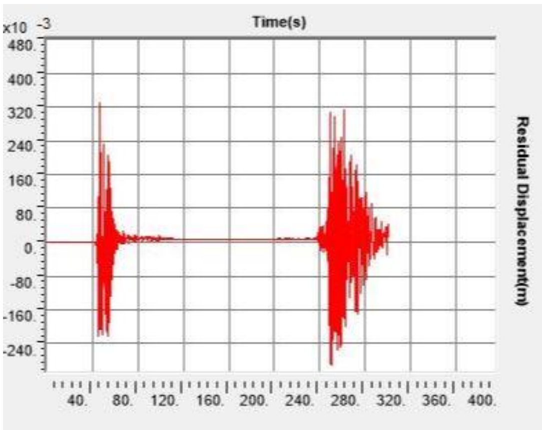
In all building configurations, it can be observed that structural components reached the more Sevier plastic state when subjected to repeated earthquakes than that in case of individual earthquakes. For example, taking the case for Gorkha earthquake for all three types of buildings (from Figure 6.8: (d), Figure 6.9: (d) and Figure 6.10: (d)), more numbers of structural members reached the collapse state when structure is subjected to sequence earthquake than the single earthquake. This might be because of decrease in strength and reduction in stiffness properties of structural components while building is exposed to aftershocks after major shock.

6.4 Residual Displacement

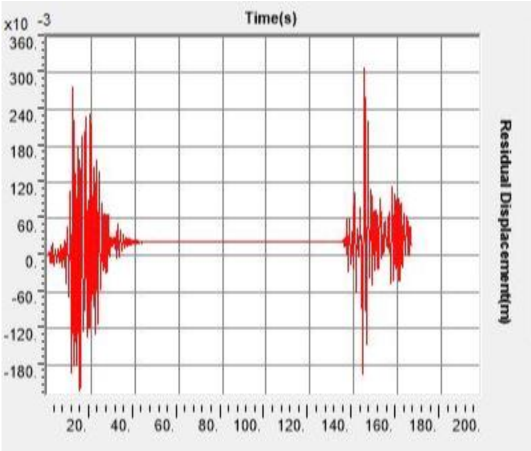
Residual displacements are manifested in the structural response when the structure remains in the plastic state after the Main shock and is an indicator of permanent damage caused to the structure. After first earthquake if the structure is exposed to repeated events within a short duration, this damage get more accumulated. As there is short interval of the time span between the sequential events the rehabilitation measures become impractical which further leads to a substantial increase in structural damages. Plot of residual displacement for seismic sequence events (Nor, Gkh, Irp, Inb) are developed for the case of NBC: 105:2020 only and are presented in Figure 6.11, Figure 6.12 and Figure 6.13.



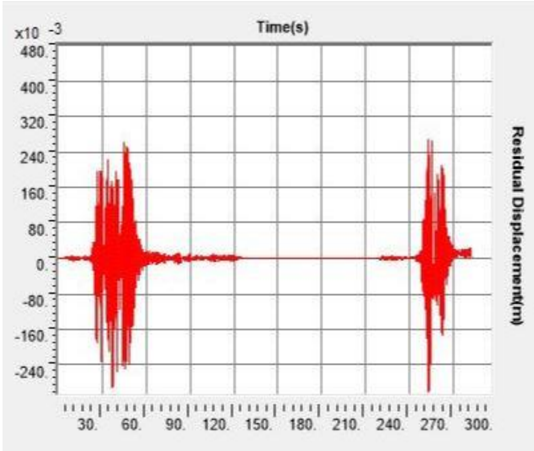
(a) Sequence earthquake (Nor)



(b) Sequence earthquake (Gkh)

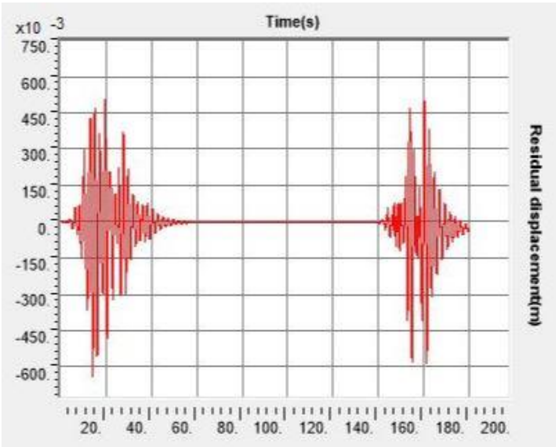


(c) Sequence earthquake (Irp)

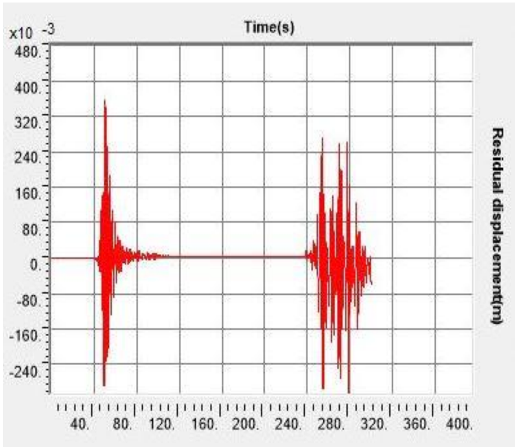


(d) Sequence earthquake (Inb)

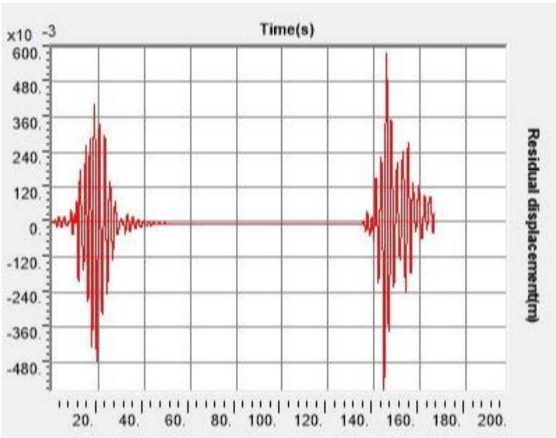
Figure6.11: Residual Displacement after different sequence earthquakes for low rise building



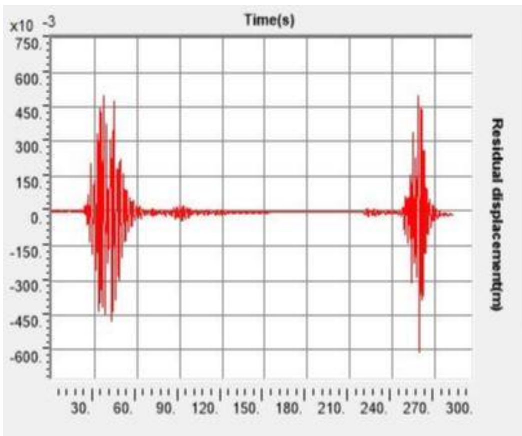
(a) Sequence earthquake (Nor)



(b) Sequence earthquake (Gkh)

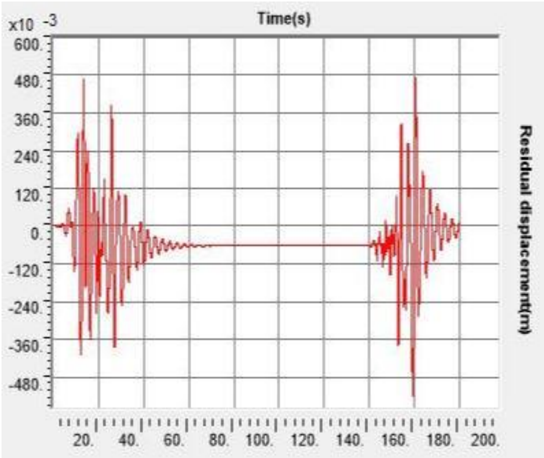


(c) Sequence earthquake (Irp)

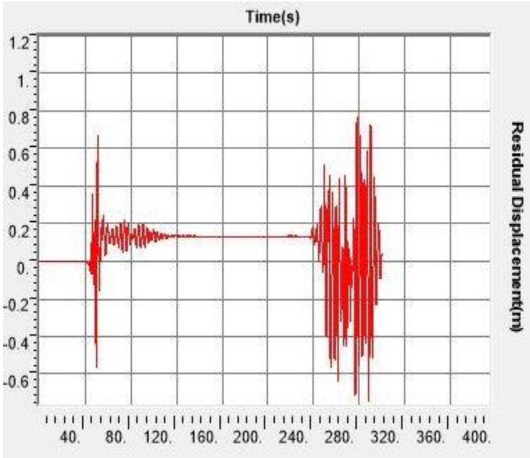


(d) Sequence earthquake (Inb)

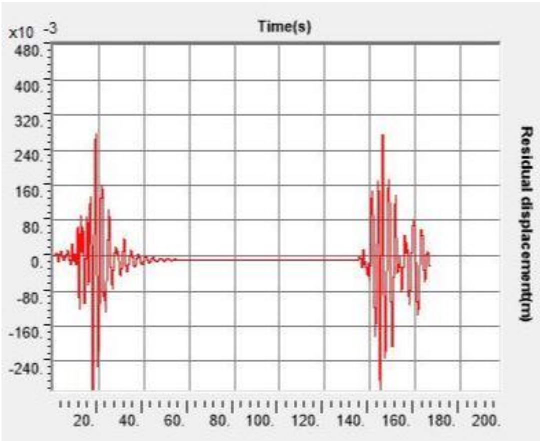
Figure6.12: Residual Displacement after different sequence earthquakes for midrise building



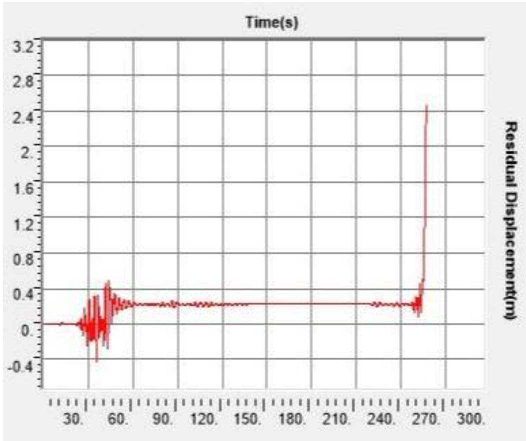
(a) Sequence earthquake (Nor)



(b) Sequence earthquake (Gkh)



(c) Sequence earthquake (Irp)



(d) Sequence earthquake (Inb)

Figure6.13: Residual Displacement after different sequence earthquakes for midrise building

As seen in above figures, residual displacement resulted after first earthquake is further accumulated and increased when structure is continuously hit by aftershocks. As seen in Figure 6.11: (b), when building is subjected to single main shock of Gorkha earthquake, the residual displacement at the end of quake is approximately 0.015m and it is clearly seen that after experiencing further shock this displacement is accumulated and increased to 0.08m at the end of sequence earthquake. Similarly, in Figure 6.12: (c), the residual displacement manifested in midrise structure after the single major shock of Irpinia earthquake is -0.010m (displaced in opposite direction). This displacement is further accumulated to 0.04m at the end of sequence earthquake. Also, the structure experienced higher displacement when it is subjected to another shock after main shock. The further increment in accumulation of residual displacement evident the weakness of building as the accumulation of plastic deformation is not the good sign for structure.

6.5 Results from Fragility Analysis

From the generated IDA curves, the fragility parameters (viz., the mean ‘μ’ and standard deviation ‘σ’ values) are evaluated as per ATC 40 guidelines for collapse limit state. These parameters are evaluated from mean IDA curves presented in Figure 6.7 and are listed in Table 6.2 , Table 6.3 and Table 6.4.

In case of NBC: 105: 2020

Table 6.2: For low rise building

Type of earthquake	OP		IO		DC		LS		CP	
	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ
Main shock	-3.168	0.009	-2.136	0.016	-1.639	0.028	-1.309	0.04	-1.061	0.053
Combined Shock	-3.103	0.003	-2.295	0.003	-1.854	0.005	-1.549	0.008	-1.316	0.011

Table 6.3: For midrise building

Type of earthquake	OP		IO		DC		LS		CP	
	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ
Main shock	-3.507	0.007	-2.379	0.019	-1.862	0.033	-1.553	0.043	-1.27	0.061
Combined Shock	-3.583	0.011	-2.656	0.019	-2.183	0.029	-1.864	0.038	-1.622	0.048

Table 6.4: For High building

Type of earthquake	OP		IO		DC		LS		CP	
	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ
Main shock	-2.705	0.0246	-2.043	0.031	-1.648	0.04	-1.366	0.0508	-1.146	0.062
Combined Shock	-2.856	0.0215	-2.205	0.0298	-1.814	0.039	-1.533	0.049	-1.315	0.059

These parameters are further used to compute the probability of exceedance using a spreadsheet program as per Eq. (6.5.1):

$$P(D \backslash PGA) = \emptyset\left(\frac{Ln(PGA)-\mu}{\sigma}\right) \qquad (6.5.1)$$

6.5.1 Generation of Fragility curve

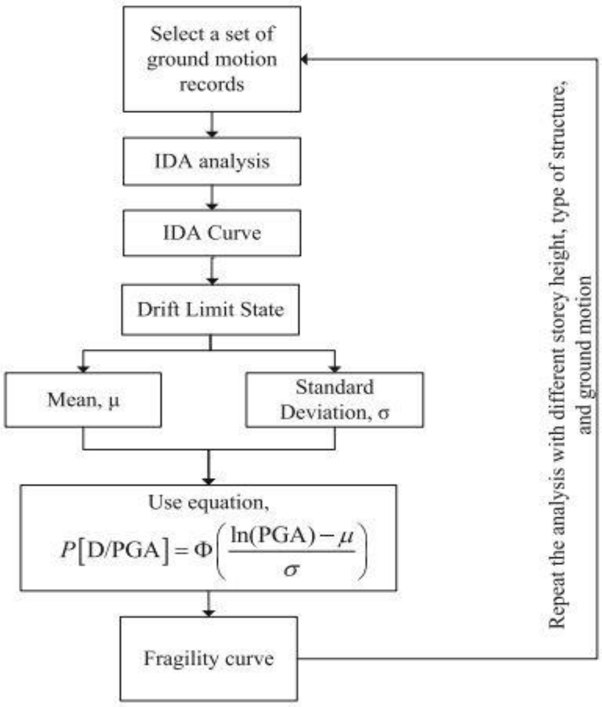
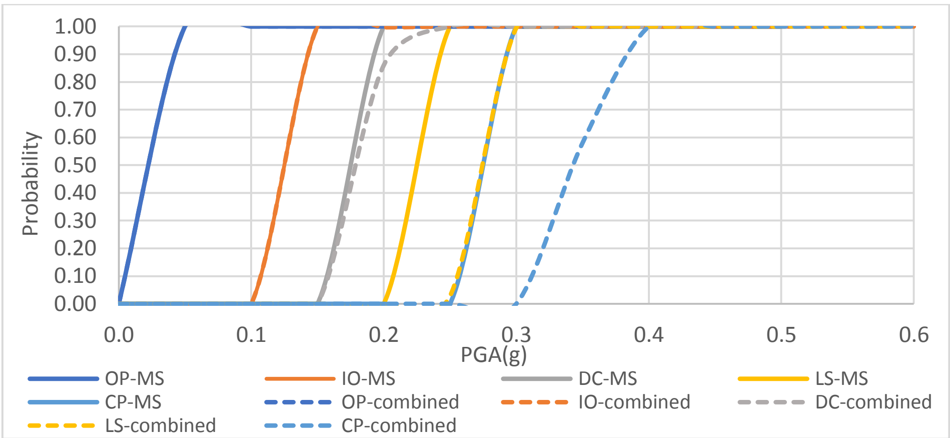
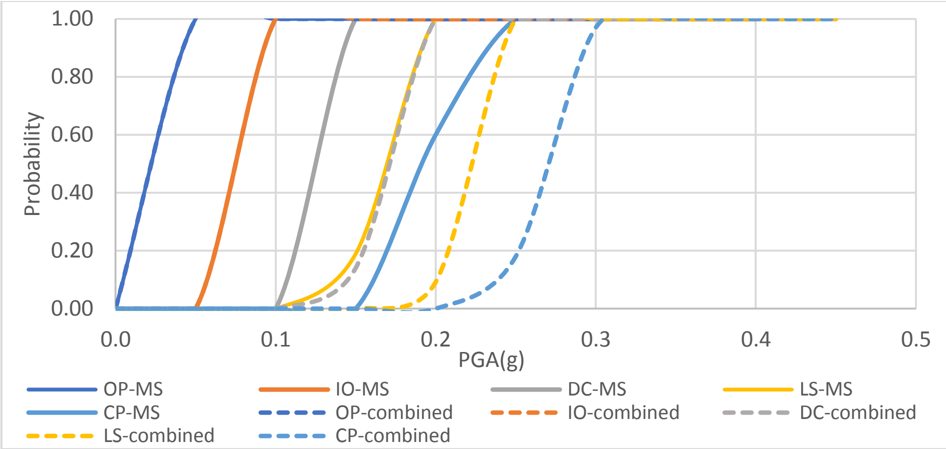


Figure6.14: Steps to develop fragility curve

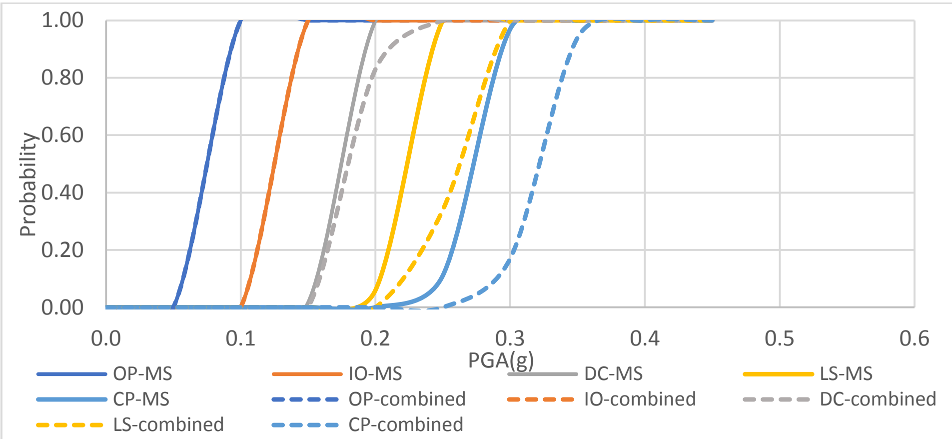
6.5.2: Development of fragility curve



(a)



(b)



(c)

Figure6.15: Comparison of Fragility curves for single and combined ground motions: (a) Low rise (b) Midrise (c) High rise

As seen in Figure 6.15 : (a), when the PGA is 0.2g, the OP level and IO level for low rise structure has probability of 100% under both single and sequence earthquake. At the CP level, the probability is 0% for both earthquake cases. However, when the PGA is 0.3g, the probability of reaching and exceeding CP level is 100% under combined earthquake where as it is 0% under single earthquake. Also from Figure 6.15: (b), when the PGA is 0.1g, both OP level and IO level for midrise structure has probability of 100% and for CP level is 0% under both single and sequence earthquake case. But at 0.2g PGA, the probability for CP level reached approximately 60% under sequence earthquake whereas probability was still 0% under single earthquake. Similarly, For high rise structure, from Figure 6.15: (c), it is clear that OP level reaches probability of 100% at 0.1g PGA and the probability of CP level is 0%

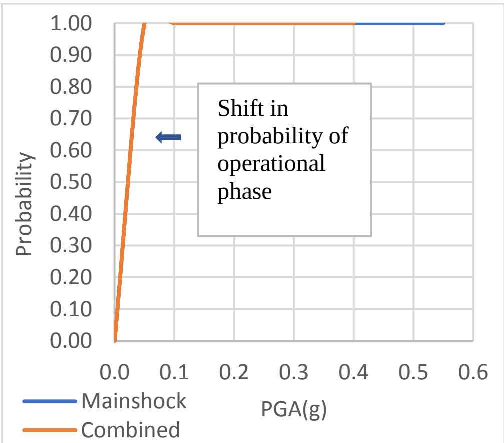
under both earthquake cases. Further, at 0.3g, the probability of reaching CP level is approximately 15% under single earthquake and 90% under sequence earthquake.

The increase in probability of structure to reach sever seismic state at lower PGA during sequence earthquake again justifies that the occurrence of aftershocks considerably reduces the strength of structure to resist the damage and collapse.

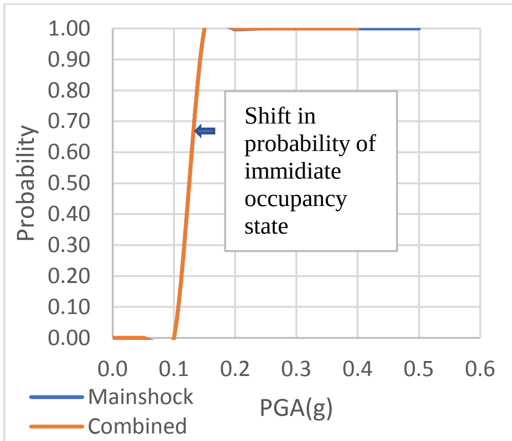
6.6 Collapse Capacity

Determining collapse capacity of structure is very crucial for determining seismic capacity of structure. For this study, collapse capacity is estimated by developing seismic fragility curves. Fragility curves for different limit states are studied separately from Figures 6.16 to Figure 6.18.

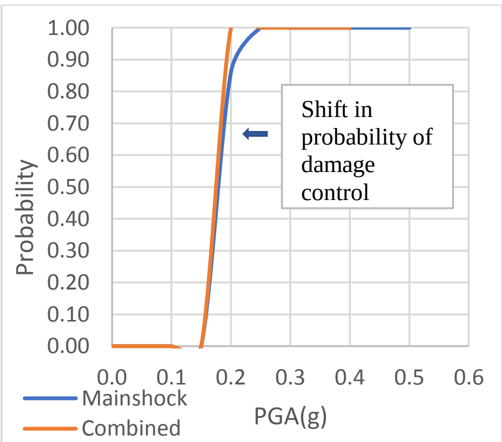
6.6.1Comparison of fragility curves for different limit states



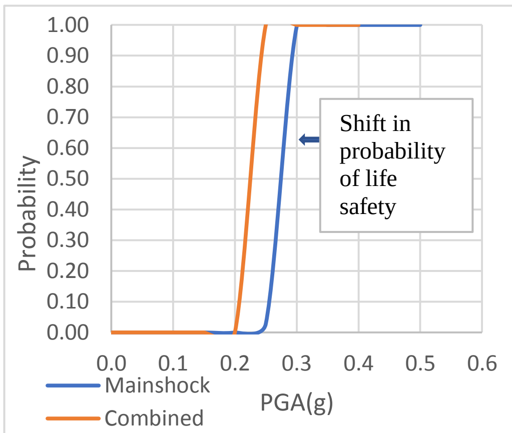
(a)



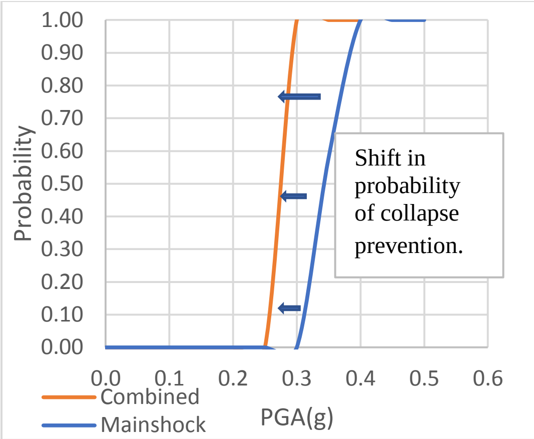
(b)



(c)

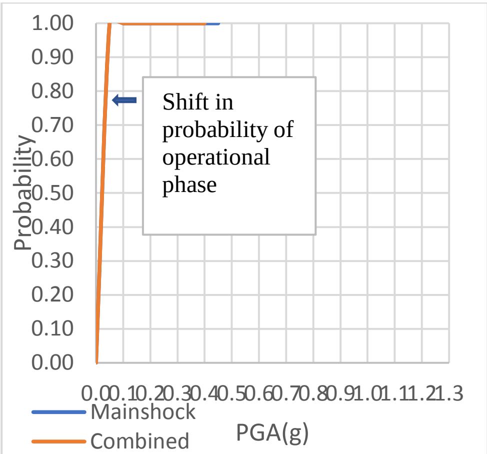


(d)

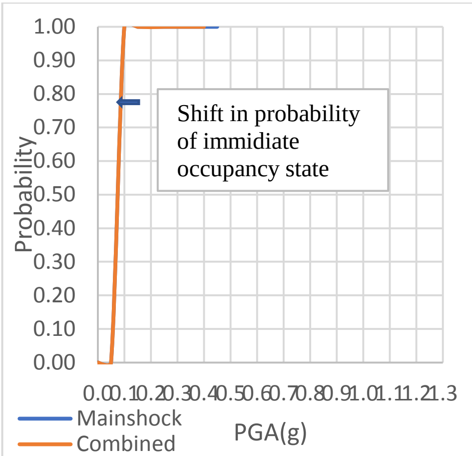


(e)

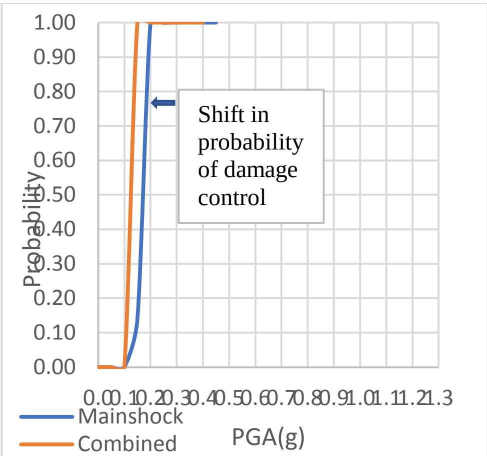
Figure6.16: Comparison of fragility curves for low rise structure for different limit state: (a) OP (b) IO (c) DC (d) LS (e) CP



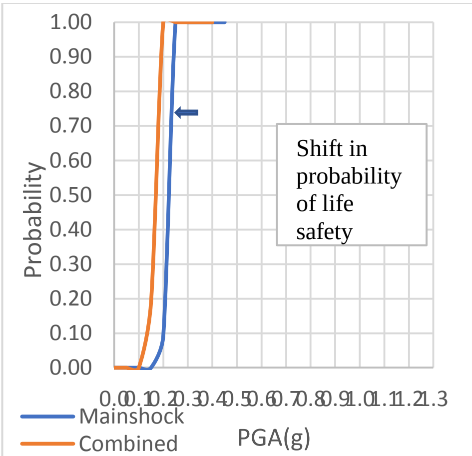
(a)



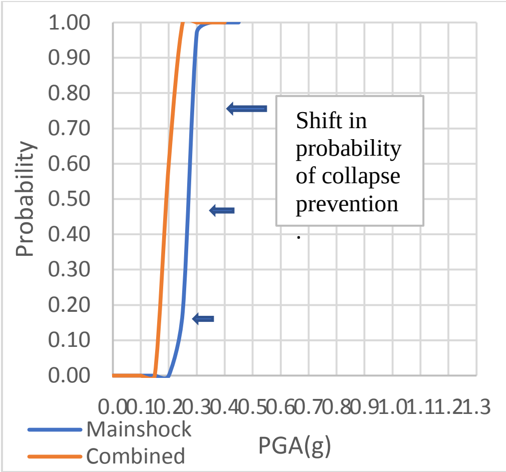
(b)



(c)

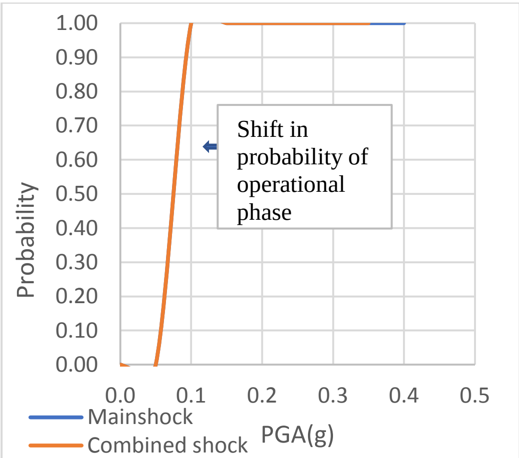


(d)

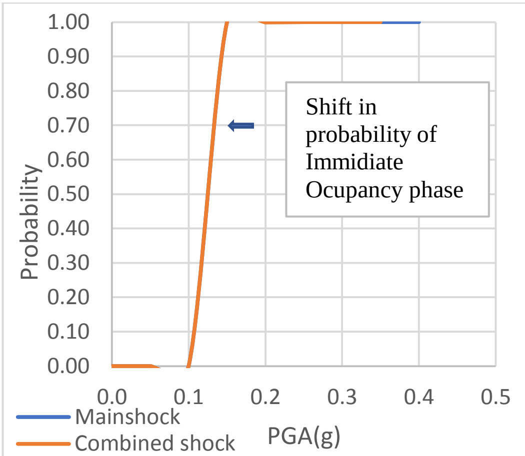


(e)

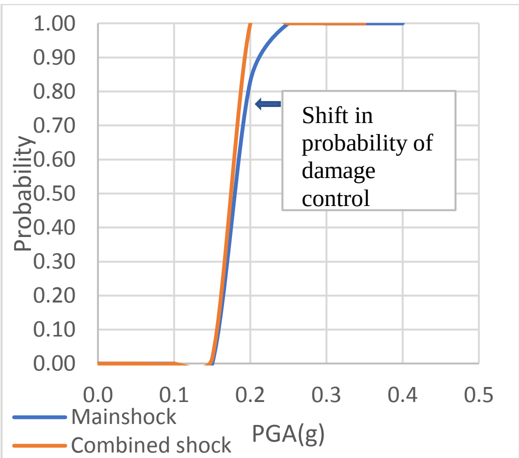
Figure6.17: Comparison of fragility curves for midrise structure for different limit state: (a) OP (b) IO (c) DC (d) LS (e) CP



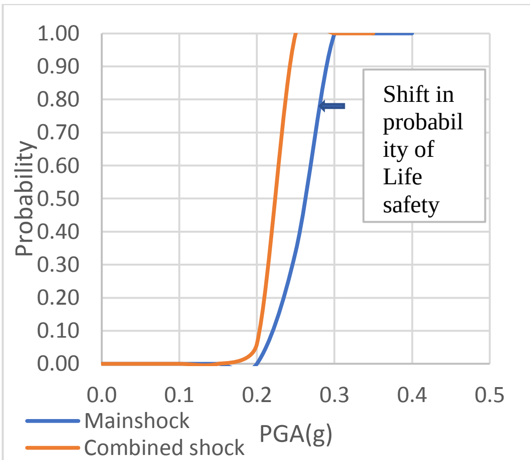
(a)



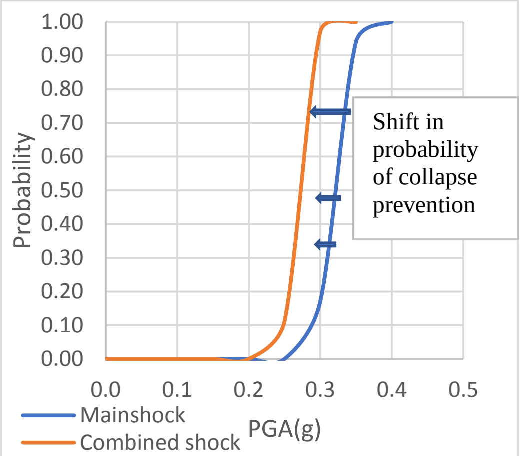
(b)



(c)



(d)



(e)

Figure6.18: Comparison of fragility curves for high rise structure for different limit state: (a) OP (b) IO (c) DC (d) LS (e) CP

It can be observed from figure 6.16 (e) that low rise building reached initial probability of collapse at 0.3g when subjected to main shock only but this state is achieved at 0.25g PGA in case of sequence earthquake .Also, 100 percentage probability of collapse of structure due to main shock occurs at 0.4 PGA and in case of sequence earthquake 100 percent collapse condition achieved at 0.3g PGA.

For midrise buildings, figure 6.17 (e) presents that when structure is subjected to single main shock only, it meets initial collapse state at 0.2g PGA and when it is subjected to sequence earthquake, initial collapse starts at 0.15g PGA. Further, structure meets 100 percent collapse probability at 0.35 PGA during individual earthquake but in case of sequence earthquake, the case is achieved at 0.25 PGA.

Similarly, It can be observed from fig 6.18 (e) that high rise structure meets initial collapse state at 0.25 PGA during single earthquake but when it experiences main shock- after shock sequence it reaches initial collapse state at 0.2 PGA. And the 100 percentage collapse of structure occurs at 0.4 PGA and 0.32 PGA for single shock and sequence earthquake respectively.

The shift in probability of collapse prevention curves signifies the reduction in collapse capacity of structure when the structure get shook by aftershocks. The reduction in collapse capacity may be due to degradation of stiffness and strength characteristics of structural members during repeated earthquake.

CHAPTER 7: CONCLUSIONS AND RECOMMENDATIONS

7.1: Conclusion

This study was mainly focused on seismic fragility assessment of structures with different number of storey in Kathmandu area of Nepal conformed to NBC 105:2020 under single earthquake and sequence earthquakes. In this investigation, IDA is performed for both single and repeated ground motions to investigate structural performance in terms of residual displacement with respect to repeated ground motions, inter-storey drift ratio (IDR) with respect to PGA and the probability of collapse in terms of PGA. Some of the major conclusions drawn from this sturdy are listed below:

- Maximum lateral roof displacement increases significantly when buildings are subjected to continuous repeated earthquake than during single major earthquake. This means that performance of structure is poor during the repeated earthquake as it experiences more damages when exposed to repeated earthquakes.
- Residual displacement accumulated in structure after the major shock found increased when the structure got hit by repeated earthquakes. This accumulation of residual displacement shows the vulnerability of structures while experiencing sequence earthquake. So analyzing structures considering repeated earthquake is found to be necessary.
- Fragility of structure is found to be more while experiencing repeated earthquake as the buildings tends to reach more severe seismic level at lower PGA during repeated earthquake than during single earthquake only.
- Also the influence of repeated earthquakes is found to be significant in collapse capacity of structure. Probability of collapse of structure became high in lower PGA in case of repeated earthquake than in single earthquake. This shows the considerable reduction in the capacity of the buildings while facing a second or subsequent earthquake after getting damaged by the first one.

Hence, These all conclusions pronounces the weakness of most of the existing and new buildings designed as per the seismic provisions considering only one isolated earthquake force during design phase. Hence, this study accentuates the necessity of considering repeated earthquake forces to analyze and design the structure to make it seismic resilient.

7.2: Recommendations for future work

1. This study considers only one aftershock after main shock. The study can be carried by considering more aftershocks for same earthquake.
2. The study is focused on the structure of Kathmandu zone only. So the analysis and study can be done for other seismic zone of Nepal.
3. For this study, PGA is used as intensity measures. So the study can be carried out by choosing other intensity measures like Spectral accelerations.
4. SAP 2000 software is used for performing IDA in this study. While other advanced finite element software can be used for performing complicated analysis like IDA
5. The study can be done by considering effect of infill wall to reflect the realistic performance of structure.
6. Variation in material type is not considered in this study. So same study can be done by taking material property as a variable.

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