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"Seismic fragility assessment of RC framed structures under varying ground motion duration"

by

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A THESIS REPORT

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IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE

DEGREE OF MASTER OF SCIENCE IN STRUCTURAL ENGINEERING

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……………………………….

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The undersigned certifies that he/she has read and, recommended to the Institute of Engineering for acceptance, a thesis entitled, “Seismic fragility assessment of the RC framed structures under varying ground motion duration” submitted by Seema KC (076/MSStE/018) in partial fulfillment of the requirement for the degree of Master of Science in Structural Engineering.

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ABSTRACT

Earthquakes are considered as the major unpredictable natural phenomenon often resulting in major disasters. Here in this study, the effect of the duration is isolated from other parameters related to ground motion like amplitude, and response spectral shape by assembling spectrally equivalent long and short duration pairs of records using Seismomatch and Seismosignal. The performance of buildings constructed in accordance with Nepal's National Building Codes of Practice in relation to seismic design is carried out. Incremental dynamic analysis is performed using SAP2000. From results, it shows that the lateral story displacements increase with the increase in duration. The collapse prevention state for lower story buildings is attained at higher values of Peak ground acceleration compared to higher story buildings with a decrement in value of PGA at collapse by 8.8%, 15.38%, 18.32% and 29% respectively for long-duration motions for 4 to 7 story building respectively. The fragility curve shows the increase in the probability of collapse by 40%, 30%, 45 % and 50% respectively for 4 to 7-story buildings with decreasing value of PGA. Also, there is an increase of collapse capacity ratio respectively for higher-story building when significant duration value increased from 2 to 4 times. So, it is concluded that longer-duration earthquakes have significant effects on seismic responses of the structure.

Index Terms: Long-duration Earthquakes, Fragility Curve, Collapse Capacity, Incremental Dynamic Analysis , Peak Ground Acceleration

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LIST OF ABBREVARATIONS

ATC	Applied Technology Council
ASCE	American Society of Civil Engineers
DM	Damage Measure
IM	Intensity Measure
DL	Dead Load
LL	Live Load
UDL	Uniformly Distributed Load
IDA	Incremental Dynamic Analysis
NBC	Nepal National Building Code
NLTHA	Non-Linear Time History Analysis
IS	Indian Standard
NDA	Non-Linear Dynamic Analysis
FEMA	Federal Emergency Management Agency
PEER	Pacific Earthquake Engineering Research Center
PBEE	Performance based earthquake engineering
SDOF	Single Degree of Freedom
MDOF	Multiple Degree of Freedom
I	Importance Factor
Z	Seismic Zoning Factor
EDP	Engineering Demand Parameter
AI	Arias Intensity
M	Mean
POA	Push Over Analysis
CDF	Cumulative Distribution Function

LIST OF SYMBOLS

μ	Mean
σ	Standard deviation
C	Basic seismic coefficient for fundamental translational period
C(T1)	Elastic Spectra
Cd	Design horizontal seismic force coefficient
Cd(T1)	Horizontal Base Shear Coefficient
Ch(T)	Spectral shape factor
Cs(T1)	Serviceability Elastic Site Spectra
Ω	Overstrength factor
Ωu	Overstrength factor for ultimate limit state
Ωs	Overstrength factor for serviceability limit state
A	peak spectral acceleration normalized by PGA
Rμ	Ductility factor
Sa/g	design acceleration coefficient for different soil types
t_{max}	Maximum time recorded
Ø	Standardized Normal Distribution
I_{eff}	Effective Moment of Inertia
I_{gross}	Gross Moment of Inertia
a(t)	Ground motion acceleration
W	Total seismic weight of the structure
ʎ	Live load participation factor

CHAPTER 1: INTRODUCTION

1.1 BACKGROUND

Earthquakes are considered as the major unavoidable and unpredictable natural phenomena which often results in major disasters. In any real earthquake, the time of shaking occurs in sequence with different duration and their effects also seem to be different. The longer duration may cause additional damage due to accumulation hampering the reoccupation and restoration.

Nepal being a seismic prone country, witness earthquake at regular interval of time. Many historical data and ongoing studies have also shown the fact. Also, Nepal has experienced many powerful and devastating events of earthquakes. They were of moment magnitude greater than or equal to 7.6 since 1255 causing serious loss of nonliving things and living lives. So, the structure to be designed in this region shall consider seismic risk explicitly.

The latest earthquake, Gorkha Earthquake 2015 with moment magnitude of 7.8 caused severe effect in different corners of the country together with major destruction in the capital city Kathmandu. Kathmandu Valley and adjoining areas area designated as a severe zone for earthquake with zone factor of 0.35 and soil type ‘D’ (which is a soft soil type) according to NBC 105:2019). Also Looking back, the region has been widely damaged by various earthquakes like 1408 earthquake -Bagmati zone (Mw 8), 1767 earthquake -Northern Bagmati zone (Mw 7.9), 1833 Kathmandu -Bihar earthquake (Mw 8),1988 Kathmandu Bihar earthquake (Mw 6.9).

Prior to the Gorkha Earthquake, the seismic coefficient and response spectrum method was the only method used in structural engineering. The Nepal building code, however, makes use of additional trustworthy and widely accepted methods in its application.

With an increasing number of high-rise buildings and also the occurrence of a large magnitude earthquakes worldwide, the need for vulnerability assessment has also increased. The amplitude, frequency content, and duration are widely recognized characteristics of the earthquake ground motions used for vulnerability assessment and are factors that affect the structural response. But the duration parameter is given second priority over the other parameters in general practice.

Recently, the interest on studying the effect of motion duration on collapse assessment of structure has been seen increasing with the field observation of large magnitude earthquakes occurred in 2010 Maule, Chile, Japan Earthquake 2011 which has caused large destruction in infrastructures. Recently many studies have been done to study the effect of the duration but the results are different in different considerations. Many papers are reviewed for the detail study on the aforementioned topic.

Performance-Based Earthquake Engineering (PBEE) is one of a emerging and rapidly growing idea for addressing the dynamic response of structure during earthquakes and is presented in many of the guidelines that were published like Vision 2000 (SEAOC, 1995), ATC-40 (ATC, 1996), FEMA-273 (FEMA, 1997), and SAC/FEMA-350 (FEMA, 2000). Among various methods used for PBEE, Incremental Dynamic Analysis (IDA) is a new structural analysis method that uses seismic demand to study the nonlinear seismic behavior of structures. IDA entails running a series of nonlinear dynamic analyses on multiple sets of scaled ground motion data.

A number of studies have been conducted, with a focus on fragility analysis and the development of fragility curves to estimate the likelihood of damage and seismic assessment of structures in post-disaster situations. The results and conclusion from these types of researches are useful in finding the possibility of restoring the infrastructure or their ability of re functioning after the earthquake has occurred. Also, it has been widely used for studying dynamic behavior of the structure at design phase also in order to extract the probability of damages rather than adopting expensive rehabilitation techniques later.

1.2 PROBLEM STATEMENT

Nepal is situated in seismically prone area with population exceeding 30 million. In its long standing history ,it has experiences numerous devastating earthquake in year 1255, 1408, 1505, 1833, 1934 and 2015 each with magnitude greater or equal to 7.6. Also earthquake of 2015 , Gorkha earthquake caused many serious casualties. Looking back to the earthquake occurrence the duration of the earthquakes varies from one another. The magnitude and distance of an earthquake are typically used as the primary parameters in selecting an accelerogram for time history analysis. The impact of earthquake ground motion duration on structural collapse capacity has been shown in recent studies. Additionally, they have suggested strategies for explicitly accounting for this period effect in structural design and assessment processes by modifying the design ground motion duration. But fewer studies are being conducted. According to Nepal's current seismic design regulations, a single design earthquake is used to generate the response spectrum or a single severe ground motion is used to analyze the structure's time history. The idea of employing ground motion duration for RC structure analysis has not yet been incorporated into any codes and is currently only utilized sparingly for design reasons. Since there are various effects that increasing duration can add, analysis based on long duration earthquake need to be done in order to be well known about the effects and disasters it brings.

1.3 OBJECTIVE OF THE STUDY

The major purpose of the study is to know the effect of the duration of ground motion vibration on the seismic performance of the structure. The specific objectives are enlisted as below:

- To compare the response of structure when subjected to earthquake of varying duration.
- To determine the collapse capacity of the structure for varying ground motion duration.

1.4 SCOPE OF THE STUDY

The study's primary focus is on the research and design of structures using the Kathmandu valley as a model. Based on the size of the ground motion and the peak ground acceleration value, the input data for the ground motion are chosen. The relevant duration value is used to categorize the chosen data. Peak ground acceleration value is used as the intensity measure in the incremental dynamic analysis, and the interstorey drift ratio% is used as the damage measure.

1.5 LIMITATION OF THE STUDY

Following are the limitations of the study:

- The selection of input data may be done on the basis of other various parameter such as: fault type (example: reverse fault type), rupture distance (near fault or far fault earthquake) and so on. In this study magnitude and PGA value is only taken

- The accuracy of the result present here also depends upon the type of the structure taken and the seismic hazard characteristics of the area. So, the similar study may be carried out considering the infill wall, staircases in structures or taking a real building and varying the seismic hazard characteristics like seismic zone factor, importance factor etc. Here in the study, general building with dead and live load and total load of infill wall is considered only.
- For better result interpretation, the intensity measure and damage measure parameters may be changed. In this work, PGA is used as the intensity metric and inter-story drift as the damage metric.

1.6 METHODOLOGY

This section describes the methodology carried out to obtain the objectives aforementioned through study of the existing literature reviews and codal provisions for the designing of the building required in the study.

First of all, the problem related to effect of varying duration on response of the structure is studied. The structure on which the study is to be carried out is selected and modelled using SAP2000. Then the selection of the ground motion is carried out as explained in chapter 5 which is then scaled to obtain spectrally matched similar response spectrum with different duration. These obtained data are then matched to the targeted response spectrum of the area considered. Here in our case Kathmandu city is taken as the area of the study so the seismic zone factor, importance factor is taken for this particular area from the related code of seismic design.

Nonlinear dynamic time history analysis is performed because the input represents the time history of various ground motions. To build the IDA curves, incremental dynamic analysis is used. Following analysis, the findings were interpreted using the necessary factors, such as displacement and interstorey drift. To calculate the collapse capacity of a structure, fragility analysis is performed using the theoretical formulation discussed below in chapter 3 and the IDA curve drawn. The conclusion is then reached.

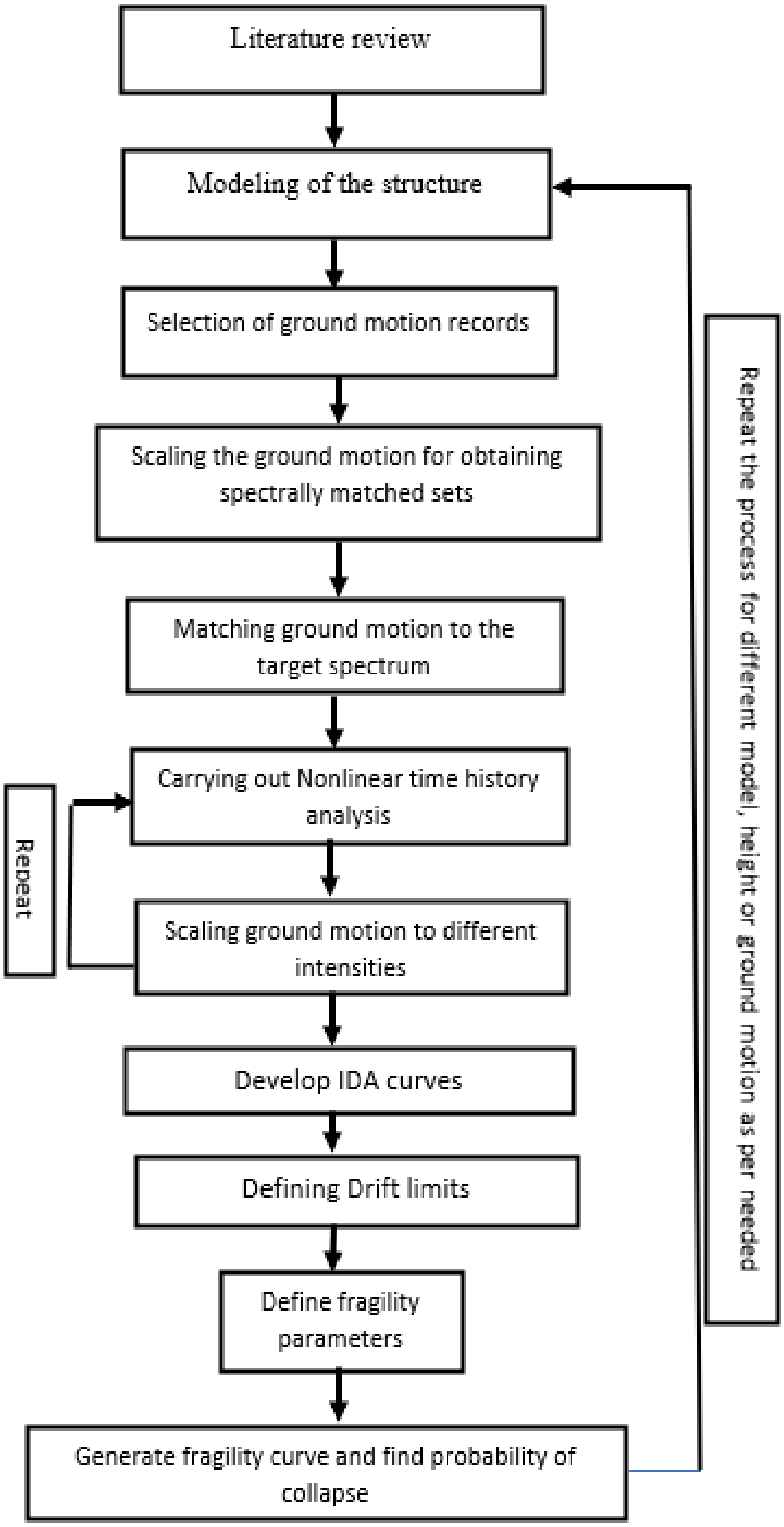


Figure 1- 1 Methodology of the study

1.7 ORGANISATION OF THESIS

The thesis work has been organized into different chapters like introductory section, methodology, literature review and so on.

- Chapter 1: Introduction

  It presents the brief introduction of the effects of the varying earthquake duration on seismic performance of the earthquakes, about performance-based design, problem statement, study objective and methodology.
- Chapter 2: Literature review

  It presents the literature review on the papers related to the topic like strong motion duration effect on the performance of the RC buildings, performance base design methods, incremental dynamic analysis, fragility analysis, collapse capacity, response spectrum various codes used.
- Chapter 3: Method of Analysis

  It presents the theoretical background of the study. The methods of the performance-based earthquake engineering. The process of carrying out the incremental dynamic analysis, fragility analysis, force displacement relation and various other theoretical formulations used in the study.
- Chapter 4: Case study of buildings

  It presents the case study of the buildings that are studied in this research. Fictious building of varying stories for the Kathmandu valley site are taken in this study and the brief discussion about the material selection, section selection and load applied are discussed.
- Chapter 5: Selection of Ground motion

  The selection of the ground motion data approach, a list of the seven selected pairs of data, scaling and matching methods, and spectrally matched data sets are all shown..
- Chapter 6: Result and discussion

  It discusses the findings of the analysis done throughout the course of the investigation.
- Chapter 7: Conclusion and Recommendation

  It presents the study's findings and conclusions.

CHAPTER 2: LITERATURE REVIEW

2.1 OVERVIEW

This chapter presents a brief summary of the literature and code provisions been used for this thesis work.

The nonlinear static analysis process is typically used to evaluate existing structures for seismic risk and to design new ones. However, nonlinear time history analysis is now more frequently employed to assess existing structures for seismic risk. Additionally, many rules and regulatory regulations mandate these kinds of analyses. In nonlinear time history dynamic analysis, the seismic requirements are established by analyzing the structures using various ground motion parameters. The ground motion histories may also be artificially created or synthetically generated. That is, one can use the ground motion received from other sources with comparable site conditions in the absence of actual earthquake records.

Numerous people have become interested in the research of the impact of ground motion duration on seismic performance and structure collapse as a result of the occurrence of big magnitude earthquakes as those in 2010 in Maule, Chile, and 2011 in Tohoku, Japan. The new long-duration strong motion records have also been made accessible, allowing for a better investigation of the influence of structural performance.

2.2 REVIEW ON PERFORMANCE BASED EARTHQUAKE ENGINEERING

The method that has been widely used to evaluate the seismic performance of the structures is performance-based earthquake engineering. This methods principal objective is to provide engineers with the resources they need to build structures that will respond predictably and dependably to earthquakes. It makes it possible to realistically account for occupancy, significant financial loss, and life risk while designing new structures or renovating old ones. The four sequential analysis procedures that make up this method are hazard analysis, structural analysis, damage analysis, and loss analysis.

(Deierlein, 2004) outlined the evaluation procedure, intensity measures, simulation technique for developing engineering demand parameters and damage measures, as well as the calculation of Decision variables, in order to undertake Performance Based Earthquake Engineering.

(Gunay, 2013) have provided a very straightforward explanation of the PEER PBEE framework to assist practicing engineers in comprehending the PBEE process. The hazard analysis, structural analysis, damage analysis, and loss analysis are the four sequential analyses that make up the PEER PBEE approach. The paper's main finding is that it establishes the significance of using PBEE as a design tool for typical structures such simple MRFs, moment resistant frames with unreinforced masonry infills, and various retrofitting techniques.

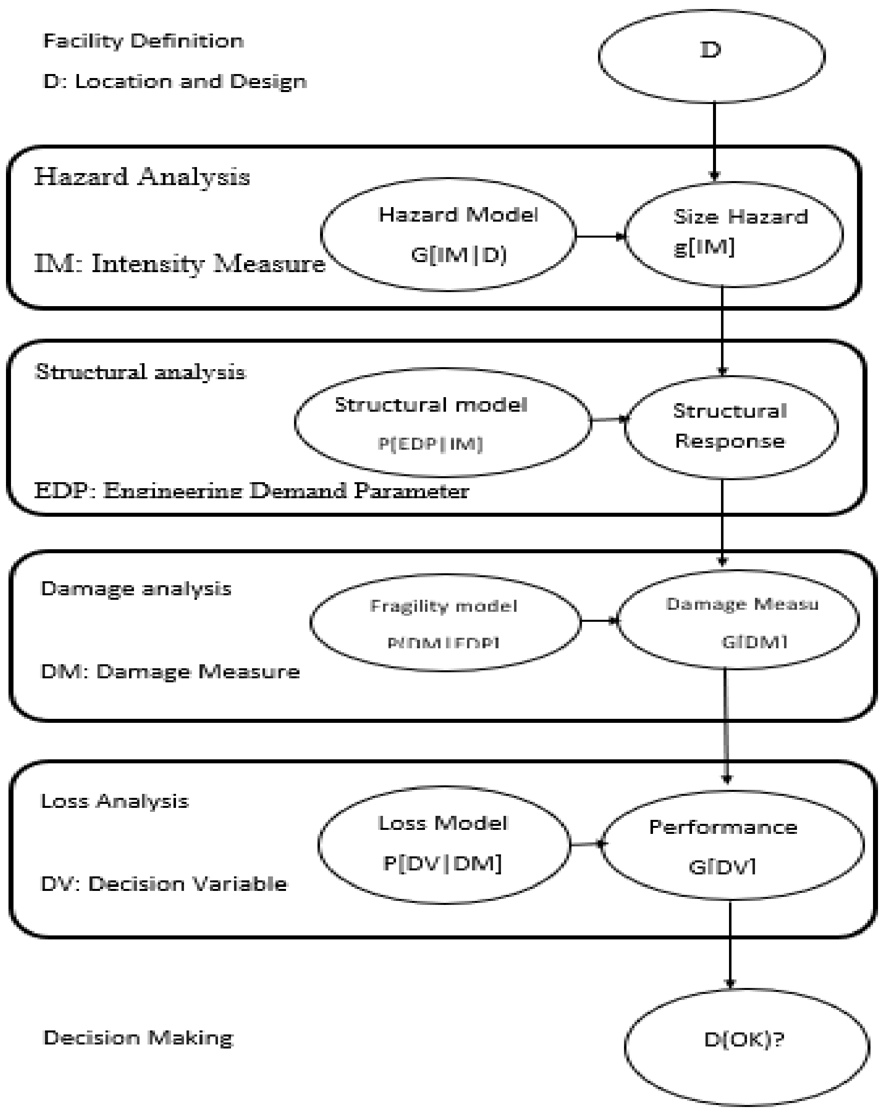


Figure 2- 1: Performance based Engineering Flowchart

Source: (Porter K. A., 2003)

2.3 REVIEW ON EFFECTS OF THE STRONG GROUND MOTION DURATION ON STRUCTURAL RESPONSE

(Manfredi G., 1997) revealed that the length of a strong -motion has a substantial impact on the structural reaction since it is assumed to be a primary factor in raising the frequency of earthquake cycles, which eventually impacts the structure's strength. Longer strong-motion duration, according to (Chai, 2005), enhances the design base shear. There is no relationship between the strong-motion duration and the structural reaction, according to other research (e.g., Bazzurro and Cornell 1992; Cornell 1997; Shome et al. 1998).

Some studies only using peak response measure like peak deformation , duration had no significant influence on structural response. According to **(Hancock j., 2006)**, Studies that used cumulative energy as a parameter came to the conclusion that damage and ground motion were positively correlated, but those that used maximal response as a parameter found no relationship between time and damage. Due to a lack of long duration strong motion records and cyclic and in-cycle strength duration, addressing the effects of duration has been difficult. Additionally difficult was the issue of separating the time from other significant ground motion factors. It is possible to modify and record the earthquake's spectral content. The ground motion pairs that were spectrally equivalent provided a strong argument for evaluating the ground motions for nonlinear dynamic analysis.

Recent structural degradation studies have demonstrated that persistent ground motion can cause severe deformations and reduce the capacity of the structure to collapse. According to Chandramohan's research, long-term ground recordings at high shaking intensities produce greater deformations. They came to the conclusion that when structures are exposed to long-lasting ground vibrations, the median collapse capability of structural systems reduces by 29% and 17%, respectively, by studying the 2D models of a 5-story steel frame and a single reinforced bridge pier.

(Bravo- Haro, 2018) examined 50 steel moment frames using spectrally equivalent pairings of short and long earthquake data in nonlinear dynamic analysis. They claimed that for structures exhibiting cyclic degradations, the impacts of duration are important. The collapse capacity typically reduced by 20% and for buildings with a high level of cyclic degradation, the collapse capacity can be reduced by up to 40%.

(Lopez, 2020) tested the low standard reinforced column's seismic performance experimentally corresponding to long-duration vibration. Compared to short duration spectrally equivalent vibration, they found that long-duration vibrations significantly

(Sarieddine, 2013)A paper on the effect of ground motion duration on various storey RC constructions was presented. The study discovered that for low to medium rise buildings, the value of displacement and inter-story drift increased when ground motion lasted longer than ground motion lasted shorter.

(Moniri, 2017) has published a study on the seismic response of RC buildings to close fault and far fault earthquakes, concluding that the effect of near fault earthquakes is more noticeable in terms of roof displacements.

The conclusions regarding the impact of the strong-motion duration on the reaction of structures shows significant effects on various parameter of seismic response as seen from the discussion above. The results are varying . The response parameters used to quantify the effects of strong-motion duration and the various definitions utilized to calculate the strong-motion duration are primarily responsible for the variations in findings between research. Overall, energy, the quantity of inelastic deformation cycles, base shear, and maximum inters-tory drift were the response characteristics used in the earlier investigations.

2.4 REVIEW ON INCREMENTAL DYNAMIC ANALYSIS

(Cornell, 2004) proposed a technique for performing incremental dynamic analysis. Since IDA requires a significant computational effort, he has regarded it as a complex analysis method. To demonstrate how to implement IDA, an in-depth analysis and study of a 9-story steel moment-resisting frame is conducted. The process for evaluating the data and using them within the context of performance-based earthquake engineering is then demonstrated.

(Vamvatsikos, 2002), developed the fundamental aspects of incremental dynamic analysis (IDA). Intensity measurements (IM), damage measures (DM), IDA curves, and the fundamentals of single record IDA and multi-record IDA have all been briefly covered in this work. It included the development of response-intensity curves and analysis for 20 various types of structures. These curves were thoroughly examined in order to provide effective methods for performing an incremental dynamic analysis. Additionally, it came to the conclusion that incremental dynamic assessments are a useful technique for addressing both the structure's global capacity and its seismic demand.

(Vamvatsikos D. a., 2005) performed Pushover analysis (POA), a nonlinear static analysis, and IDA, a nonlinear dynamic analysis, on single- and multiple-degree-of-freedom structures. Additionally, the relationship between IDA and traditional POA is defined. SPO2IDA, a new piece of software, was also presented. This enables it to directly estimate the findings of the IDA through analysis. The key conclusion of the paper is that IDA addresses both structure demand and capacity giving better results.

(Tehrani, 2013) performed IDA on four-span bridges. The report explains that because IDA is a intensive method that necessitates numerous nonlinear assessments, it is rarely implemented. However, by using the IDA algorithm, it is feasible to considerably cut down on computing time, making it simple to extract precious information for accessing seismic risk and performance.

(Asgarian, 2010) IDA was used to obtain a seismic performance evaluation of a steel moment resistant frame. Three types of moment-resisting frames are considered for the analysis: Special, Intermediate, and Ordinary Moment Frames, each with a different degree of ductility. Comparative analyses of the seismic performance of these three different structures are performed in this work. This study found that incremental dynamic analysis is an effective method for studying structural performance. The Incremental Dynamic Curve can be used to describe the yielding and collapse stages of structural behavior based on the number of time histories examined. The incremental dynamic analysis of the structure's response represents the building's actual response to the earthquake under consideration. Damage outcomes, monetary losses, and structural responses can be easily examined from IDA.

2.5 CONCEPT OF FRAGILITY ANALYSIS AND COLLAPSE CAPACITY

(Nazri, 2018) delivered a paper on two buildings, one with two stories and the other with six. He carried out IDA and created fragility curves using data from both near and far faults, and he came to the conclusion that fragility analysis could be done on a structure before or after an earthquake and that the results could be used to illustrate the probability that the structure would sustain damage at various performance levels.

(Biglari, 2020) presented that prior to the implementation of a risk reduction program, the vulnerability assessment of a structure's hazards and vulnerabilities is a crucial step, and the fragility curve is one of its main components. A risk assessment of 274 masonry buildings in Iran under the magnitude 7.3 Sarpol earthquake of 2017 was done. Along with building techniques and materials, the fragility curves depicted helped in predicting potential seismic damages in those masonry structures. By interpreting these curves, it was possible to identify which structures were most at danger and offer recommendations for prioritizing retrofitting efforts to lower seismic risk.

(Regan Chandramohan, 2016) used IDA to create the fragility curves using ground motion with spectrally similar short and long durations. The variance in ground motion durations on a steel structure is what caused the projected median collapse capacity to decline by 29%, according to a calculation of the median collapse capacities. Using the long duration set instead of the short duration set increases the predicted likelihood of collapse at the MCER level by nearly 7 times.

(Raghunandan, 2013) When a ground motion is scaled to the level at which structural collapse occurs, the inelastic spectral displacement measures the structure's collapse capability. A higher value of Sdi (displacement) at collapse for a given building period dy indicates that the structure can withstand stronger ground motions before collapsing. The results of structural analysis using the generalized linear model for all structures are fitted to a multivariate regression model to assess the impact of ground motion duration on a structure's potential to collapse.

(Shafei, Zareian, & Lignos, 2011) has defined structural system collapse capacity as the spectral acceleration value at which the structure becomes dynamically unstable as a result of component strength and stiffness deterioration and/or delta effects. The study also provided a framework for nonlinear static (pushover) analysis to estimate the collapse capacity of structural systems.

(Tehrani, 2013) developed fragility curves based on incremental dynamic analysis findings The fragility curves represent the conditional probability that a limit state will exceed it at a given intensity measure value. The spectral acceleration at any time T is assumed to be the IM in this case. The fragility curves displayed the mean annual rate of exceedance and the overall chance of failure in 50 years. These forecasts helped determine the mean annual failure rate.

(Gautam D. G., 2018) derived fragility functions for the Nepali Residential buildings considering different earthquake (1934 Bihar earthquake Mw 8.4, 1980 Chainpur earthquake Mw 6.5, 1988 Eastern Nepal Earthquake Mw 6.8 and so on and their damage data. Fragility function for mainly three different buildings classes i.e., reinforced concrete, stone masonry and brick masonry buildings. The fragility function is derived considering the site effects and comparison has been made with existing fragility functions for any discrepancies. It was found that stone and brick masonry structures were more vulnerable when exposed to strong ground motion. In terms of the PGA, RC buildings were prone to damage at PGA ranging from 0.15 to 0.8 g whereas the stone and brick masonry were found to be damaged at even 0.075g PGA in some locations.

(Gautam D. R., 2021) The fragility of RC buildings affected by the Gorkha earthquake in 2015 was demonstrated. Seismic fragility has been quantified at both the global and component levels in this paper. The paper demonstrated that Nepali RC buildings, even when built after the Nepal building code was implemented, pose a significant risk of damage during moderate to strong shaking. At maximum ground acceleration of 0.3-0.4 g, approximately half of the buildings may be damaged beyond immediate occupancy.

(Adhikari, 2022) Seismic vulnerability analysis of a low-rise RC-framed building with masonry infill, taking into account soil structure interaction. The paper compares different models of low-rise buildings in Kathmandu Valley with and without infill, as well as SSI effects on soft soil. The fragility function is derived using a nonlinear time history analysis that takes into account various damage states. Fragility functions generated for the limit states show that the soil structure interaction has a greater impact o lower damage states whereas the infill have a greater impact on the higher damage states increasing their exceedance probability at specified spectral acceleration levels. The results show that as compared to empirical fragility models the analytical fragility models appear to be much larger than those inferred from actual loss data. The finding has also highlighted the significance of modelling infill walls considering SSI.

(Pan Y. e., 2019) Took two sets of short and long duration earthquake and used for nonlinear dynamic analysis. When fragility curves and the median collapse capacity were displayed as part of incremental dynamic analysis, it was discovered that extended duration earthquakes had a drop in collapse capacity of 18%. The calculated median and average damage index for the long duration ground motion rose by 36% at the highest assessed earthquake intensity.

(Korkmaz, 2008) offered two new methodologies: traditional approach and Monte Carlo simulations (analytical approximations). Following the definition of these approaches, a chosen R/C structure was used to implement an application. When these approaches were compared, they produced findings that were nearly identical when assessing symmetric structures. More trustworthy fragility analysis was shown to be simulation-based. The following steps were used in this study:

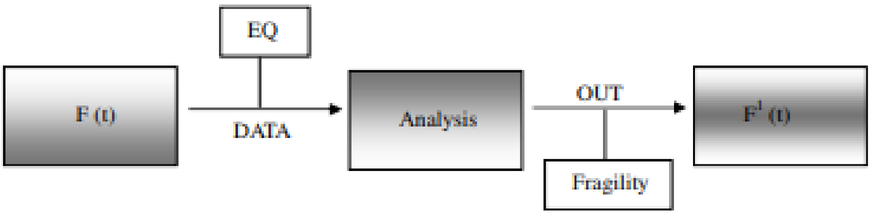


Figure 2- 2 Input output relationship in fragility analysis

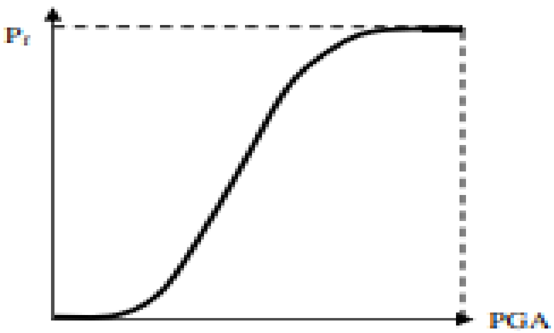


Figure 2- 3 Schematical of fragility curve

2.6 REVIEW OF CODES

For different countries the seismic design codes for building is different for Non-Linear Analysis. The seismic Zoning of Kathmandu valley according to Nepal building code is presented in this section.

2.6.1 Nepal National Building Code (NBC 105: 2020)

Nepal National Building code has established the following relation for the spectral factor and given the graph as given below. The spectral factor $C_h(T)$ for the relevant soil type can be obtained from the figure 2 or equation below:

$$\mathbf{Ch(T)} = \left\{ \begin{array}{c} \mathbf{1} + (\boldsymbol{\alpha} - \mathbf{1}) \times \frac{\boldsymbol{T}}{\boldsymbol{Ta}} \boldsymbol{if\ T < Ta} \\ \boldsymbol{\alpha\ if\ Ta \leq T \leq Tc} \\ \boldsymbol{\alpha} \left[\boldsymbol{K} + (\mathbf{1} - \boldsymbol{k})(\frac{\boldsymbol{Tc}}{\boldsymbol{T}})^2 \right] \frac{\boldsymbol{Tc}^2}{\boldsymbol{T}}\ \boldsymbol{if\ Tc \leq T \leq 6} \end{array} \right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Where, α= peak spectral acceleration normalized by PGA

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

K= the coefficient that controls the descending part of the spectrum

Table2- 1: Soil Parameter

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
A	2.5	2.5	2.5	2.25
K	1.8	1.8	1.8	0.8

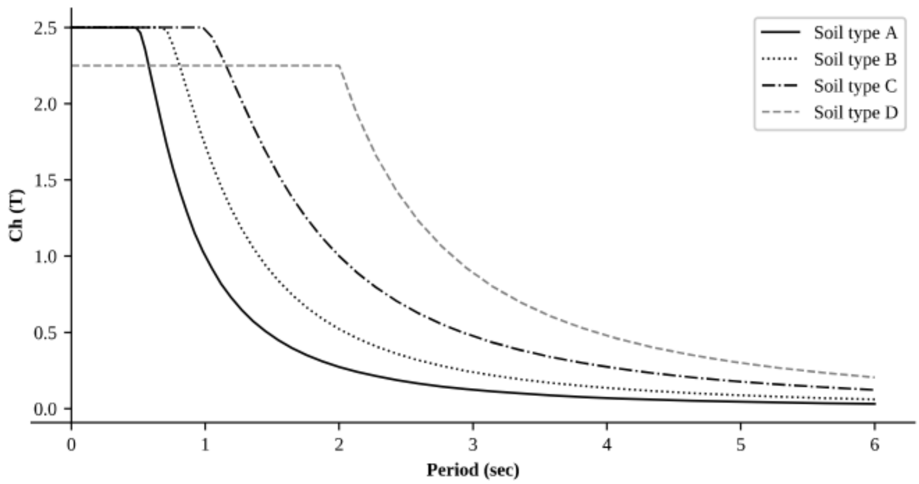


Figure 2- 4: Spectral shape factor $C_h(T)$ for equivalent Static Method

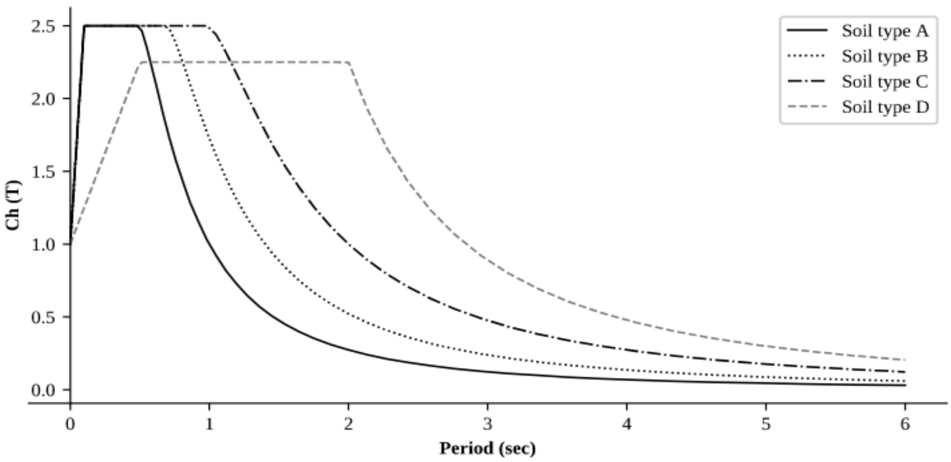


Figure 2- 5: Spectral Shape factor, $C_h(T)$ for Modal Response Spectrum Method, Nonlinear Time history analysis

The NBC105:2020 is an improvement over NBC 105:1994. It has introduced two different states which are ultimate limit state and serviceability limit state to calculate the horizontal base shear coefficient. The structural performance factor in NBC 105:1994 has been replaced with over strength and ductility factor in NBC 105:2020.

The elastic site spectra for horizontal loading is given as

$$C(T) = Ch(T).Z.I \quad \ldots\ldots\ldots\ldots (2)$$

Where, $C_h(T)$=Spectral Shape factor

Z=Seismic zone factor

I=Importance factor

The elastic site spectra for Serviceability Limit State is given as

$$C_s(T) = 0.2\ C(T) \quad \ldots\ldots\ldots\ldots (3)$$

Equivalent static method

The horizontal base shear coefficient is given by

$$C_d(T_1) = \frac{C(T1)}{R\mu * \Omega u} \quad \text{(for Ultimate Limit State)} \ldots\ldots\ldots\ldots (4)$$

$$C_d(T_1) = \frac{Cs(T1)}{\Omega s} \quad \text{(for Serviceability Limit State)} \ldots\ldots\ldots\ldots (5)$$

Where, $C(T1)$=Elastic site spectra

$Cs(T1)$=Elastic site spectra determined for serviceability limit state

$R\mu$ = Ductility factor

Ωu =Overstrength factor for ULS

Ωs =Overstrength factor for SLS

CHAPTER 3: METHOD OF ANALYSIS

3.1 NON-LINEAR ANALYSIS

When applied forces and displacements have a non-linear relationship, the analysis is said to be non-linear. During intense ground shaking, the buildings do not react as a linearly elastic system. Therefore, nonlinear analysis is required for more accurate global displacement forecast and realistic seismic demand prediction. The P-effects and huge displacements that result from the structure's changing shape are the source of the geometric nonlinearity that gives rise to the nonlinear effects. The next is material non-linearity, which happens when concrete and steel are stretched beyond their proportional limits and exhibit inelastic behavior, leading to cracking, crushing, yielding, and other problems.

A structure's nonlinear model is capable of accurately identifying structural damage and performance with regard to deformation demand-to-capacity ratios. A linear elastic model is less realistic and significant than a seismic simulation. Therefore, it is essential to provide this useful tool to the upcoming generation of structural engineers in order to help them comprehend the intricate inelastic structural behavior.

The Takeda Hysteresis Model, as described in Takeda, Sozen, and Nielsen, employs a deteriorating hysteretic loop (1970). This straightforward approach is more suitable for reinforced concrete than for metals and requires no additional parameters. Compared to the kinematic model, less energy is lost.

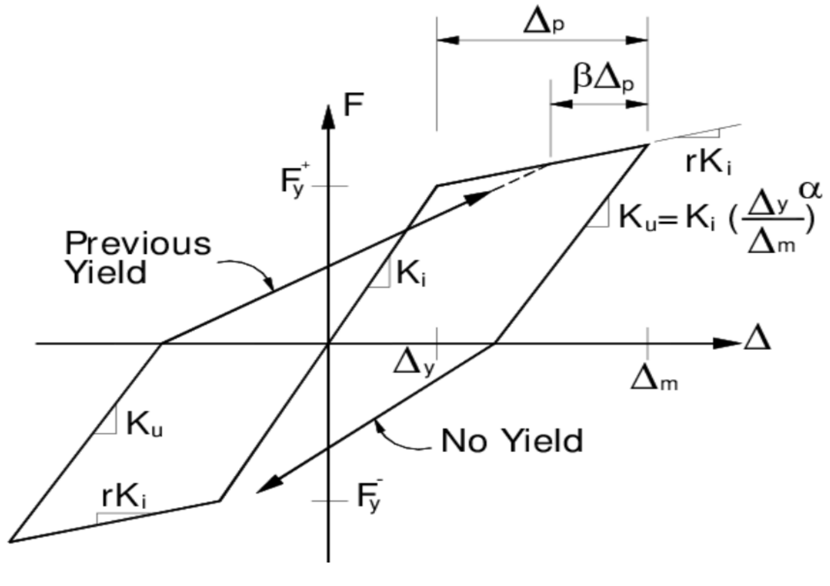


Figure 3- 1 Takeda Hysteresis curve

3.2 INCREMENTAL DYNAMIC ANALYSIS

Incremental dynamic analysis is one of the powerful methods of PBEE framework. It involves series of nonlinear dynamic analyses using number of scaled ground motion records. The results are useful in considering the seismic performance of structural system. The IDA offers result through which seismic demand and limit-state capacity prediction is possible. Many research works have been done to describe the process of IDA. Among them, the one which that gave the detailed and simplified knowledge about IDA is described below. (Vamvatsikos D. a., 2005) This study provided detail knowledge about the procedure for performing the IDA. As IDA is very complicated analysis since it involves number of ground motion data and number of nonlinear dynamic analysis, the fundamental concepts and suitable algorithms for performing IDA must be clear for obtaining better results. In the first step of IDA, a proper structural model needs to be modeled on suitable finite element software like SAP. Secondly the appropriate number of ground motion records need to be considered. And further need to be scaled to different intensity levels and the dynamic analysis is performed and results are extracted.

3.2.1 IDA Algorithm

Various algorithms were developed for IDA by various articles and journals. [40]gave a stepping algorithm. It showed the simple method of increasing the Intensity measure by a constant step from zero until the structure is collapsed. The results acquired after applying this algorithm are uniformly-spaced (in IM) grid of points on the curve. The Steps involved in IDA are:

Increase IM.

Scaling the record, Run analysis.

Extract the value of damage measure until collapse.

3.2.2 IDA in SAP 2000

SAP 2000, a finite element based structural program is broadly used for all types of linear or non-linear dynamic and static analysis. It allows quick and proper appliance of NLTH analysis procedures as mentioned in ATC-40 and FEMA-273. IDA involves multiple non-linear time history analysis of structural model under multiple ground motions which are scaled to several levels. The following steps are carried out to perform incremental dynamic analysis:

1.Three dimensional models of the proposed structure needs to be modeled. The Reinforced concrete building model is modeled by drawing the structural element considering the , joint restraints, geometry , material properties and loads over the members.

2.Preliminary, the linear seismic analysis of the structure is performed.

3. The Geometric non-linearity and Material non-linearity in the structures are shown in the form of p-delta effect and hinges respectively. Hinges may be auto hinges or user defined hinges as SAP transforms them to generated hinges.

4. The appropriate kind and quantity of ground motions data must be chosen. If not enough recorded ground motions are available, the necessary total number can be made up with suitable generated ground motions. Here, actual ground movements are utilized.

5. To match the intended spectrum, all of the chosen ground motions must be scaled with the proper scale factor. The scale factor is chosen so that it falls between the periods Tn and √RµxT1,, where T1 is the fundamental period of the structure's vibration, Tn is the period of the maximum vibration mode to guarantee 90% mass involvement, and R is the ULS ductility factor. (NBC)

6. The supports of the structural model are then subjected to the scaled ground motions.

7. After executing NLTH, the analysis is continued with different scaled factor for IDA until the structure meets target value or collapse.

3.3 FRAGILITY FUNCTION

A fragility function is a function that, given an amount of ground shaking, describes the likelihood of exceeding various limit states. These are crucial tools for creating the fragility curve and calculating the likelihood that the structure will collapse.

Some of the papers related to the fragility function presented below.

(Baker J. W., 2015) This paper used the statistical procedures for evaluating the fragility functions parameters using results obtained from performing nonlinear dynamic analysis of structure and those procedures were used to evaluate fragility functions by using various non-linear dynamic analyses. The paper has given lognormal cumulative distribution function as a fragility function as shown in equation:

$P(C\backslash IM=x) = \phi\left(\frac{\ln\left(\frac{x}{\theta}\right)}{\beta}\right)$………………………………………………………. (6)

where 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).

(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;

$P(x) = \emptyset\left(\frac{\ln x - \lambda}{\xi}\right)$ ……………………………………………………….. (7)

Where, (Ø) is the standardize normal distribution,

λ is the mean of ln x,

ξ is the standard deviation of ln x.

(**Porter K. R., 2007)**Fragility functions are probability distributions used to indicate the probability that a component or system will be damaged to a given or more severe damage state as function of a single predictive demand parameter such as story drift or floor acceleration. So, fragility function takes the form of log normal cumulative distribution functions, having median value Ɵ and logarithmic standard deviation, β. The mathematical form is given as:

$Fi(D) = \phi\left(\frac{\ln\left(\frac{D}{\theta i}\right)}{\beta i}\right)$……………………………………………………….. (8)

Where, Fi(D) is the conditional probability that the component will be damaged to damage state I or a more severe damage state as a function of demand parameter D

Φ denotes the standard normal cumulative distribution function

θi denotes the median value of the probability distribution,

βi denotes the logarithmic standard deviation.

For various limit states specified in ATC 40, pushover analysis utilizing the capacity spectrum method is used in this study to characterize a structure's performance. Using pushover analysis, a bilinear capacity spectrum is produced. Damage state thresholds for the four damage states—light, moderate, extensive, and complete—are specified on the bi-linear capacity spectrum (Krishna, 2017). The table below provides the median spectral displacement for the four-damage condition based on these thresholds.

Table 3- 1 Damage threshold spectral displacement

$Sd_1=0.7D_y$	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 and variability values are derived for the damage state under consideration. The following equation then defines the conditional likelihood of damage exceeding the damage state.

$$P(ds\backslash sd) = \phi[\frac{1}{\beta ds}\ln\left(\frac{Sd}{Sdds}\right)] \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(9)}$$

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

Sd=Given peak spectral displacement

β_{ds}= Variability of spectral displacement for damage state, ds

(Tehrani, 2013) Have also developed a fragility curve through the development of the Incremental Dynamic curves by analyzing a 4-span bridge. Here the data is assumed to be log normally distributed and thus is possible to develop a fragility curve at the point of collapse or any other limit states by the calculation of the median collapse capacity and the logarithmic standard deviation of the results obtained from IDA. The fragility function is given by the equation below.

$$P(failure|Sa = x) = \phi\left[\frac{ln(x) - ln(Sa^{c}_{50\%})}{\beta_{RTR}}\right] \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(10)}$$

Where, Φ= cumulative normal distribution function

$Sa^{c}_{50\%}$= Median capacity determined form IDA

β_{RTR}=Record to record variability

(Pan Y. e., 2019) conducted incremental dynamic analysis and quantified the collapse capacity rate of the building for long and short duration ground motion. The IDA outcomes were used to create the collapse fragility curves. The log normal distribution was used to build the empirical fragility curves. The equation below provides the total likelihood of collapse:

$$P = 1 - \emptyset\left[\frac{lnX-\mu}{\sigma}\right] \quad (11)$$

Where, X is ground **motion** intensity index and μ *and* σ are mean and standard deviation of lognormal spectral acceleration for long and short ground motions.

Formulation used in the study

The algorithm used for conducting Incremental Dynamic analysis is as given below;

- Increase IM.
- Scale the record, Run analysis.
- Extract the damage measure until collapse.

Following formulation is used in the study as presented in the paper of Vamvatsikos, Dimitrios, and C. Allin Cornell. For fragility analysis. Here the Peak ground acceleration is used as Intensity Measure and maximum interstorey drift percentage as damage measure. The particular formula used in the study is given below,

$$P(x) = \emptyset(\frac{\ln x-\mu}{\sigma}) \quad (12)$$

WhereØ =standardize normal distribution,

μ = mean of lnx

σ = standard deviation of lnx

Generation of fragility curve

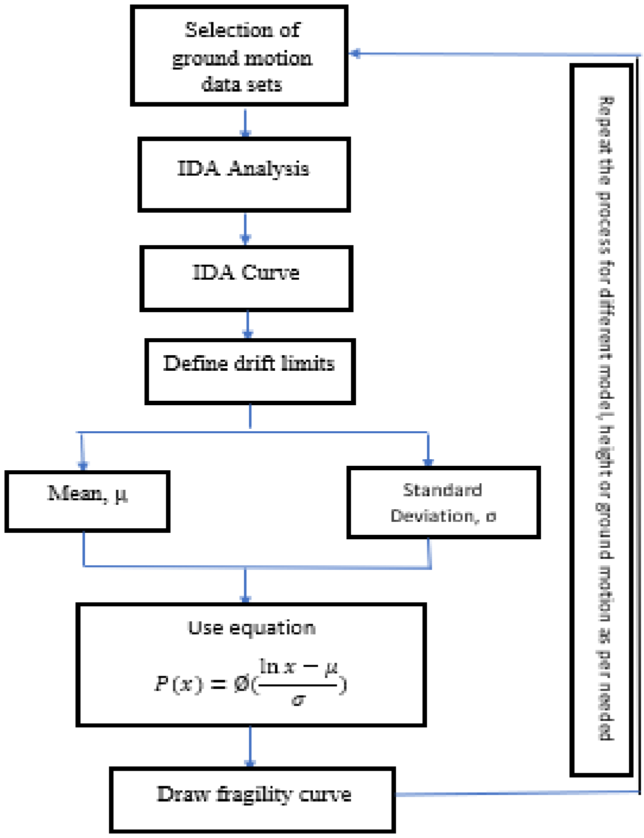


Figure 3- 2 Flow chart for developing the fragility curve

CHAPTER 4: CASE STUDY OF BUILDINGS

For describing the real geometry of the considered structures, the model is extended in the direction of all three axes.3D modeling of the structure of this research work is done using Finite Element Analysis software SAP2000 'Integrated software for structural analysis and design". Four different storied building are taken.

4.1 ASSUMPTIONS

- Soil-structure interaction is not taken into account and the foundation is presumed to be rigid.
- Only the mass of infill wall is considered and applied in corresponding beam as uniformly distributed load (UDL).
- Diaphragm is assumed to be rigid.
- Secondary effects like temperature, creep, shrinkage etc. are not considered to simplify the analysis procedure.

4.2 LIMITATIONS OF THE CASE STUDY

For focusing on the problem and bringing out a proper and concise result it is necessary to limit the field of study. So, the limitation of this study is:

- The input time history data is confined to seven in number.
- The height of the building is only varied but not the bay size or material used.
- The site of the study is limited to Kathmandu valley.
- The storey of building is varied from 4 to 7 only.

4.2 BUILDING NOMENCLATURE

In this study, building with equal bays are used and only the height of the buildings is varied. Buildings comprises of the 3 bays in each direction of 5 m each while the height of the building is taken as 3.2 m. However, the bays of the building are not differed throughout the study. Building of 4 to 7 stories linearly increasing their height is taken for the study purpose. The 2D and 3D images of the 4 structures taken along with the description of the material, section, loads, seismic weight applied are explained below.

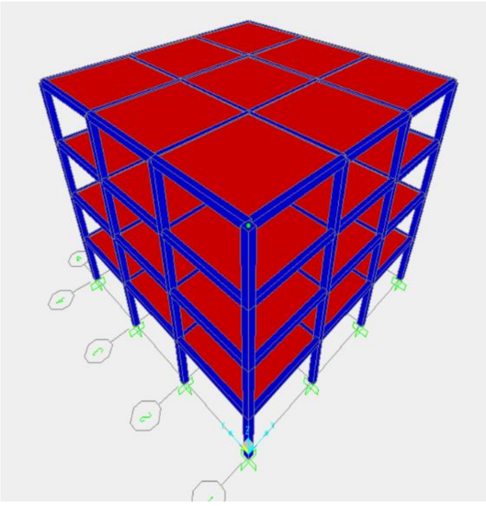


Figure 4- 1 3D view of 4story building

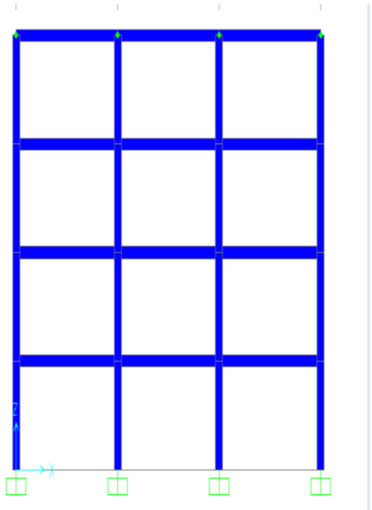


Figure 4- 2 2D view of 4 story building

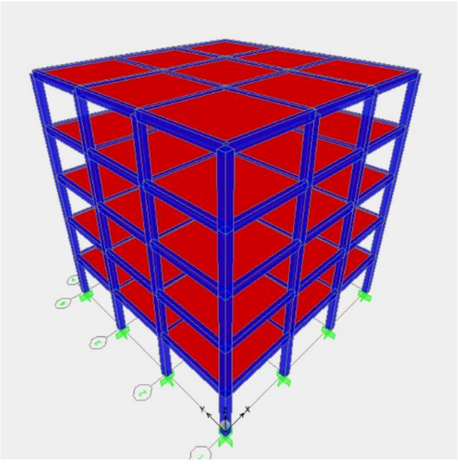


Figure 4- 3 3D view of 5 story building

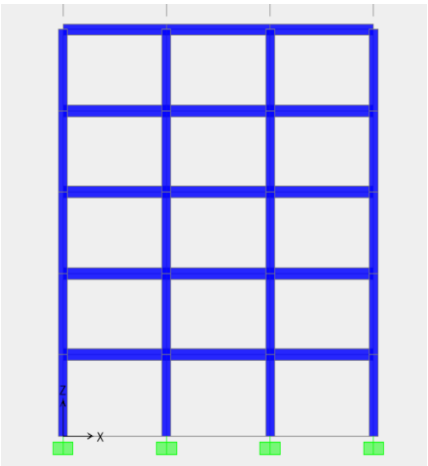


Figure 4- 4 2D view of 5 story building

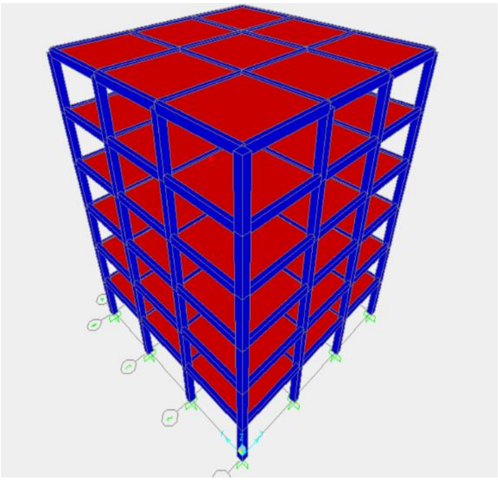


Figure 4- 5 3D view of 6 story building

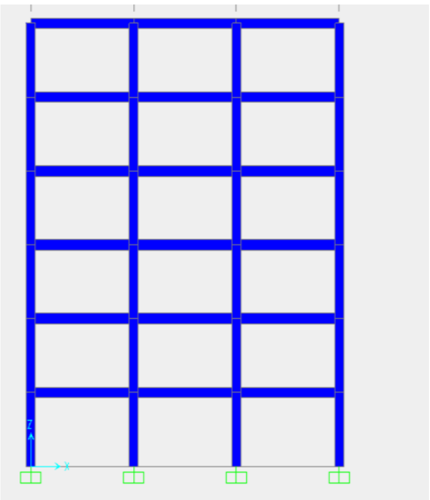


Figure 4- 6 2D view of 6 story building

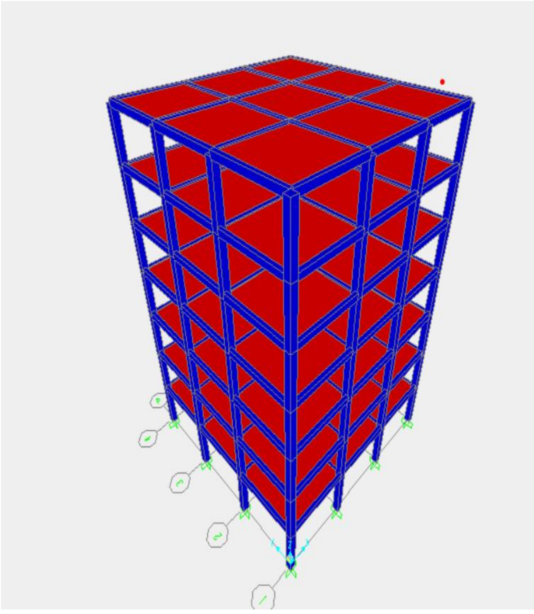


Figure 4- 7 3D view of 7 story building

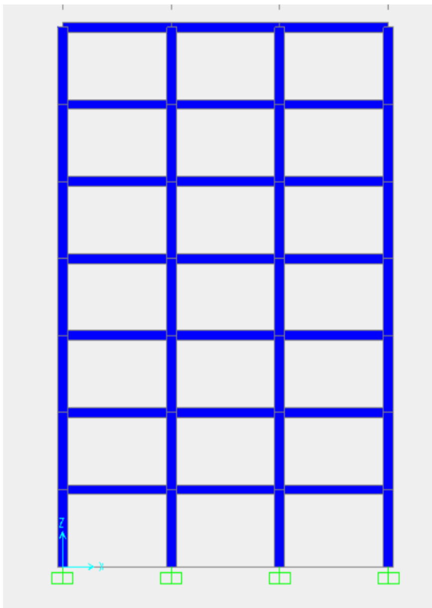


Figure 4- 8 2D view of 7 story building

4.3 MATERIAL PROPERTIES

The grade of Reinforcement is HYSD500 TMT with elastic modulus of 200000 MPA1 is used in the design. The unit weight and poison's ratio are taken to be 76900 N/m2 and 0.3 respectively. Similarly. the concrete grade used is M20 with an elastic modulus equal to 20000 MPa. The concrete weight per unit volume is assumed to be 23600 N/mm2 with poison's ratio of 0.2.

The sizes of beams and columns used are different for the different story building according to their need. With increase in the height of the building their sizes are increased.

Table 4- 1 Beam and column size for different buildings

Number of story	Beam size(mm)	Column size(mm)
4	250*350	350*350
5	250*400	400*400
6	250*400	400*400
7	250*450	450*450

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., for beams (M3 hinges) and columns (P-M2-M3 hinges) using default hinges in SAP2000 at their ends. Takeda hysteresis model is used to define degradation caused by cyclic loading.

4.4 LOADS

Following loads are considered for analysis and calculation.

- Dead load
- Dead load of 1.5 KN/m^2 on slab for floor finish.
- Live load 3KN/m^2
- Wall load with thickness of 230mm, assuming unit weight of brick wall as 19.2 KN/m^2.

W=1*19.2*0.32*3.2=14.13Kn/m^2

For the top floor parapet wall is kept.

4.4.1 Seismic Weight

The total seismic weight of the structure (W) shall be taken as the summation of dead loads and factored live loads given as:

W=DL + ʎ LL

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

DL is the total dead load of the structure including self-weight of the structures, slab loads, floor finish and wall loads.

LL is the live load and ʎ is the live load participation factor taken as 0.3.

4.4.2 Load Combination

For designing of the structure, the seismic loads effects and other effects are combined. The following load combination as given in NBC is used in this study,

1.2DL+1.5LL

DL+ʎ LL± E Where ʎ =0.3

CHAPTER 5: SELECTION OF GROUND MOTION DATA

Selection of ground motion data play vital role in non-linear time history analysis. All the data used in the study are real earthquakes and no any artificial ground motion is generated. In this study earthquake ground motion with moment magnitude greater than 5.5 and less than 8 and PGA value greater than 0.15g are considered and only the horizontal component of the earthquake data is taken into consideration. The earthquake data are taken from Pacific Earthquake Engineering Research (PEER) and Consortium of organizations for strong ground motion Observation System (COSMOS) databases.

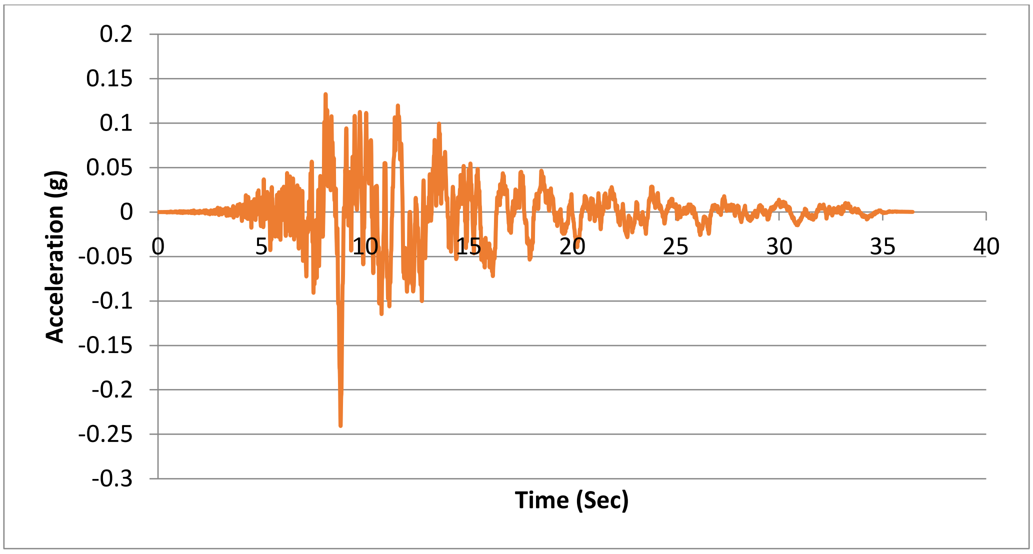
5.1 SELECTION OF GROUND MOTION DATA

The correct evaluation of the structures' response requires taking into account an adequate and sensible amount of ground motion data. If less than seven values or data are considered, the maximum values of the response quantities from the ground motions should be taken into account, according to NBC 105:2020 for seismic design. However, if there are more than 7 ground motion data , the average of those data points will be utilized to evaluate the respondents' responses. Therefore, for this study, seven sets of real earthquake ground motions were collected. The measured earthquake data have a moment magnitude of at least 5.5 but not more than 8, and a peak ground acceleration of at least 0.15g. Gautam D. R. and 2021 have demonstrated that the RC framed structures prone to damage at PGA of 0.15g to 0.8 g for Kathmandu valley.

There is no any appropriate definition and representation of the standard ground motion duration. We have different types of duration like significant duration, bracketed duration, uniform duration, effective duration. But with the literature review it is known that the most reliable one is significant duration. So, in this study the 5% to 75% significant duration is used as the significant duration metric. It is defined as the time required to develop the Arias intensity in range between 5 % to 75% of the total energy record as given in equation 1. The term a(t) in the equation corresponds to the ground motion acceleration, $\boldsymbol{t_{max}}$ is maximum time recorded.

$$\boldsymbol{AI} = \int_0^{t_{max}} a(t)^2\, d(t) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

Since the process includes scaling of the taken ground motion to the target spectrum various factors change resulting to ambiguous results. Thus, the duration metrics that don't vary with scaling like significant duration for analysis like IDA is used in this study. (Regan Chandramohan, 2016)



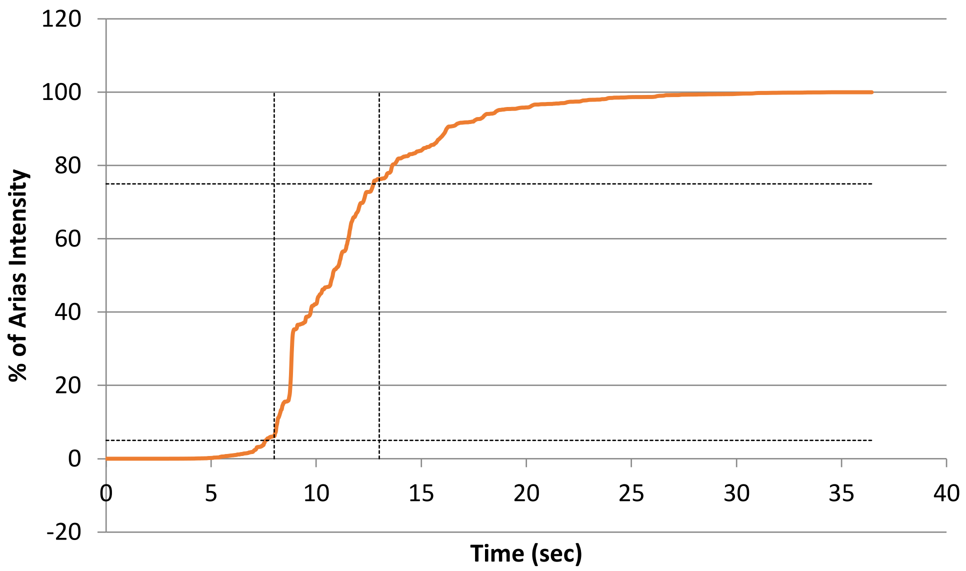


Figure 5- 1 Significant duration calculation of a ground motion

For the determination of the significant duration, the Seismo signal software version 2020 is used by loading the data obtained from the various sites like PEER and COSMOS. The Earthquake ground motion with significant duration greater than 25 seconds are considered as long duration earthquake and the one with significant duration less than 25 seconds are considered as short duration earthquake [Error! Reference source not found.].

When the large duration data are chosen, a set of companion short duration data set for each long duration data is taken with significant duration less than 25 seconds. According to Chandramohan et. al. the corresponding short duration data is chosen by following procedure. The target response spectrum of the long duration data was discretized at periods of 0.05s to 4s at interval of 0.05 seconds to obtain the spectral ordinates $L_1, L_2, L_3\ldots$ and so on with mean L. And similarly for short duration data spectral ordinates $S_1, S_2, S_3\ldots$so on with mean S. Then the spectral acceleration of the short duration record is scaled by K=L/S, such that the spectral ordinates of the scaled KS equal to L. This is summarized by table below.

Table 5- 1 Spectral Equivalent Approach method used by Chandramohan et al

Time	sa/g (LD)	sa/g (SD)	scaled sa/g (SD)
0	L1	S1	S1 x K
T1	L2	S2	S2 X K
T2	L3	S3	S3 X K
.	.	.	.
.	.	.	.
Tn	Ln	Sn	S4 x K
Lavg	Avg (L1,L2…Ln)		
Savg	Avg (S1,S2…Sn)		
K	Lavg/Savg		

As explained in the above table after calculating the significant duration and response spectra of the ground motions, the ground motions with similar response spectra are taken and scale in order to obtained spectrally equivalent data as shown in the figure below. A set of spectrally matched ground motion pairs is plotted as below in terms of response spectra and time series respectively in figure5-2 and figure5-3. The other seven pairs of the data are presented in table and their response spectra and time series is plotted in Annex.

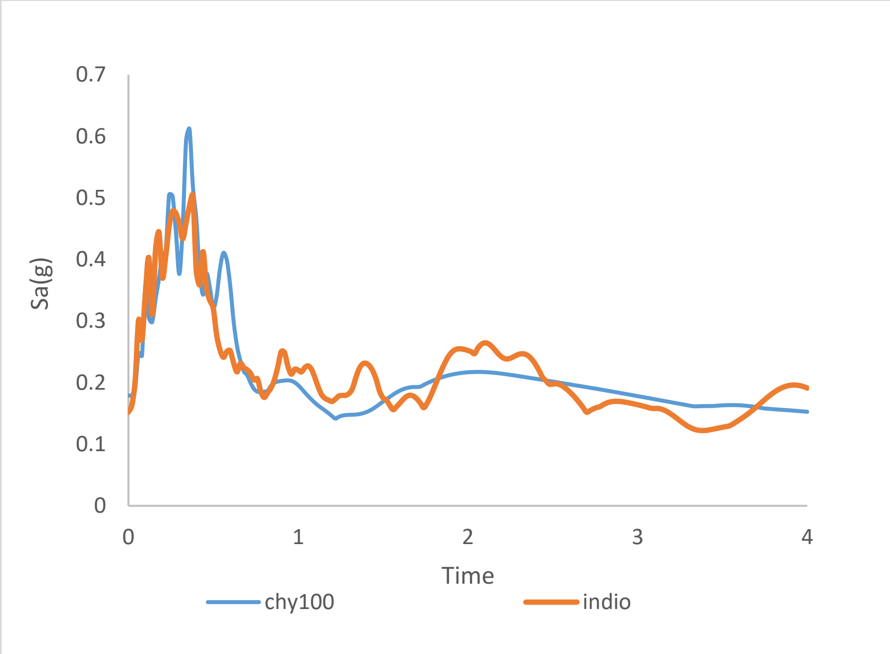
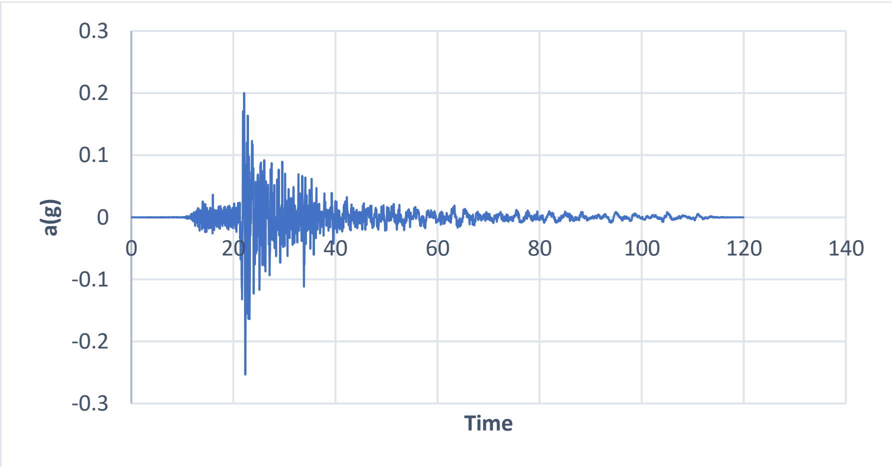


Figure 5- 2 Ground motion data with similar response spectra



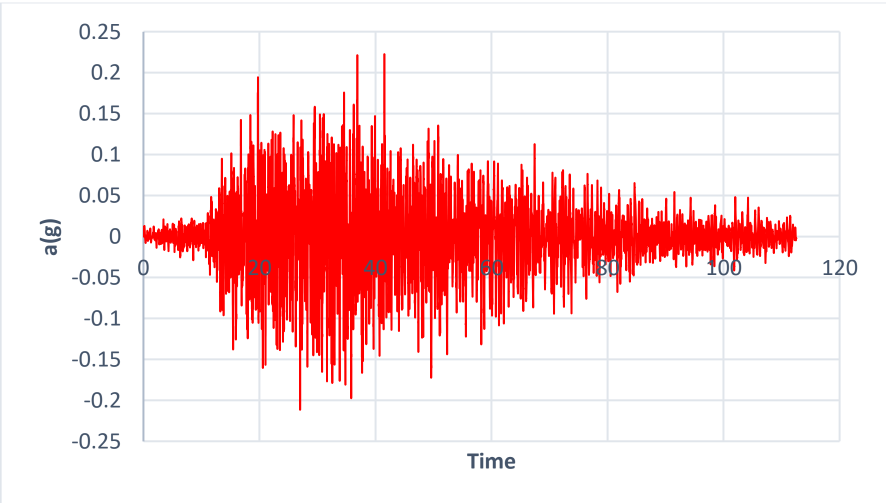


Figure 5- 3 Spectrally equivalent Ground motion data with different duration values.

The table below shows the 7 pairs of spectrally equivalent long and short duration ground motion data.

Table 5- 2 List of Earthquake ground motion

SN	Earthquake	Station name	Scale	Duration	Source
1	2010 EL Mayor Cucapah	Chihuahua	-	27	PEER
	2010 Drafield,New Zealand	DORC	2.97	16	PEER
2	2010 EL Mayor Cucapah	Ejido Satillo	-	33	PEER
	1999 Chi Chi Taiwan	TCU075	0.54	18	PEER
3	2010 EL Mayor Cucapah	Ejido Saltillo	-	33	PEER
	1999 Chi Chi Taiwan	TCU101	0.82	16	PEER
4	2010 EI Mayor Cucapah	tamaulipas	-	27	PEER
	1992 Landers	Amboy	1.55	17	PEER
5	2010 EI Mayor Cucapah	Chihuahua	-	24	PEER
	1999 Hectormine	Amboy	1.47	11	PEER
6	1992 Landers	Indio-Coachella Canal	-	25	COSMOS
	1999 Chi chi Taiwan	CHY100	1.62	12	COSMOS
7	1985 Valparaiso, Chile	Llolleo	-	28	COSMOS
	1994 Northridge-01	Sun Valley - Roscoe Blvd	1.82	6	COSMOS

The data here selected are from all round the world since the data form specific regions may not have spectrally matched set in the same region. Some of the data are from same place but either their direction is different or station is different.

5.2 COMBINING AND MATCHING GROUND MOTION DATA

The selected ground motion should be scaled to certain target spectrum of the specified location to meet the specified level of seismic hazard as per site location. Here the target spectrum is response provided in NBC 105:2020. Seismomatch software is used for scaling and matching of above selected ground motion data. The scaled factor used to match the target spectrum calculated so that it lies between periods Tn and $\sqrt{R\mu} \times T1$,

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

Seismic Zone factor =0.35

Importance factor=’I’

Soil Type= Very Soft Soil

Structural importance factor =1

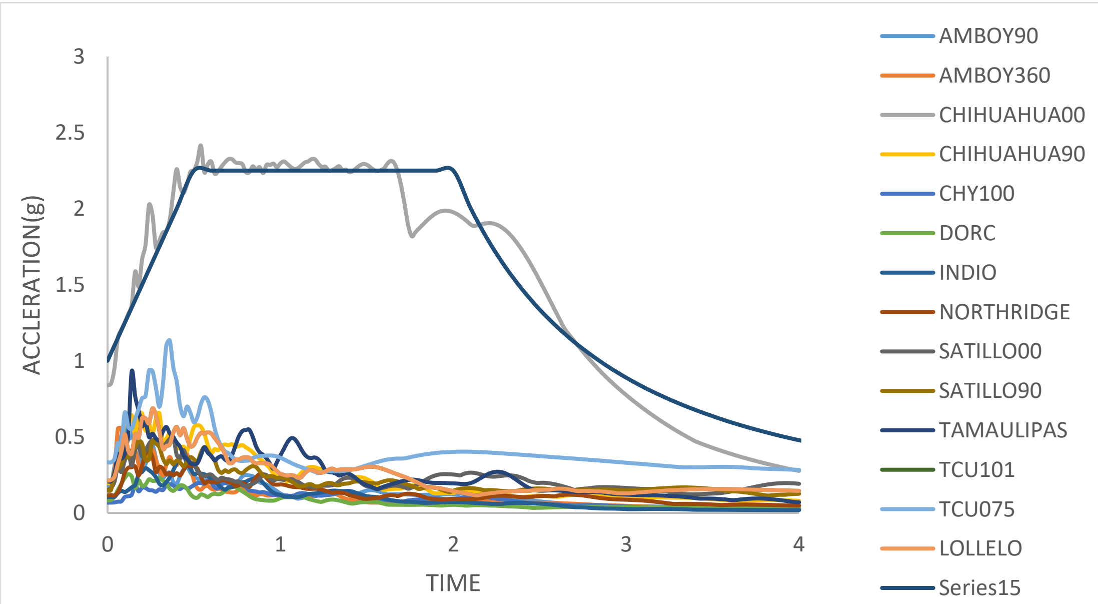


Figure 5- 4 Unmatched response spectrum

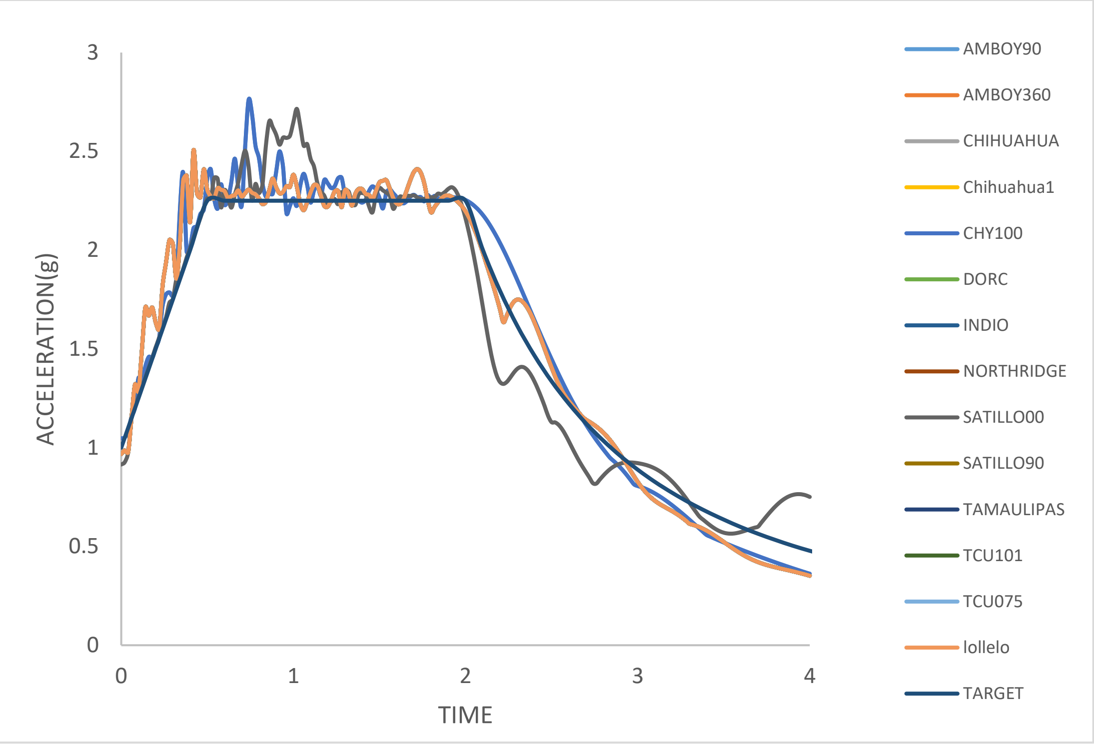
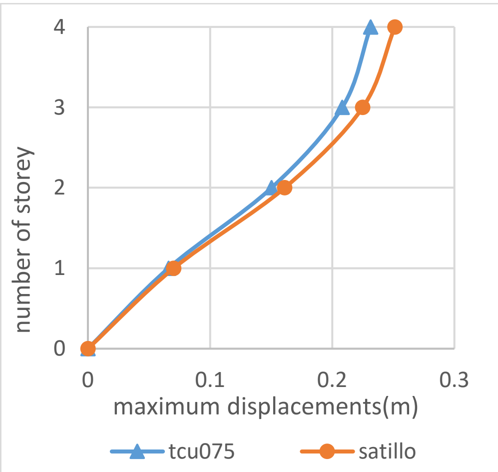


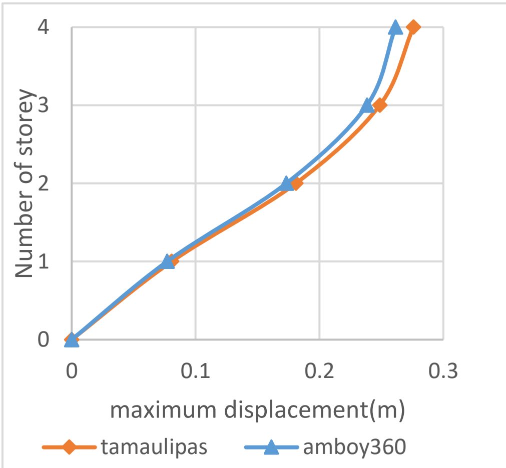
Figure 5- 5 Matched response spectrum

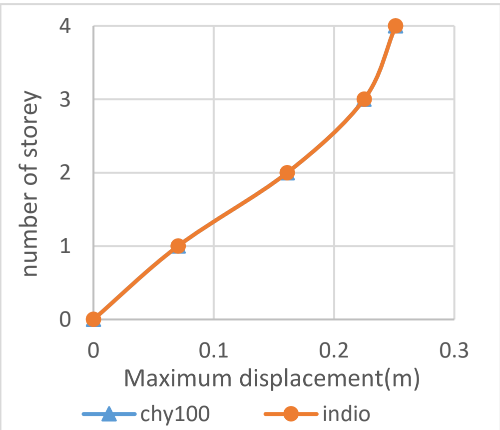
CHAPTER 6: RESULTS AND DISCUSSION

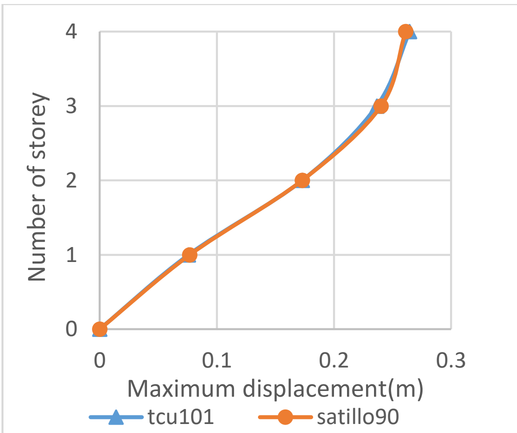
6.1 MAXIMUM STORY DISPLACEMENT

When structures are subjected to lateral loads like earthquake and wind loads, lateral displacement becomes important. Lateral displacement is dependent on the height and slenderness of the structure since taller buildings become more susceptible to lateral stresses because they are more flexible. The top level experiences significantly greater lateral stresses than the bottom storey, which causes the building to exhibit cantilever behavior. One of the study's parameters is the lateral story displacements, which are collected from [36]. The lateral story displacements for all the models used in the tale are computed, and the displacements and narrative graphs are presented as shown below.









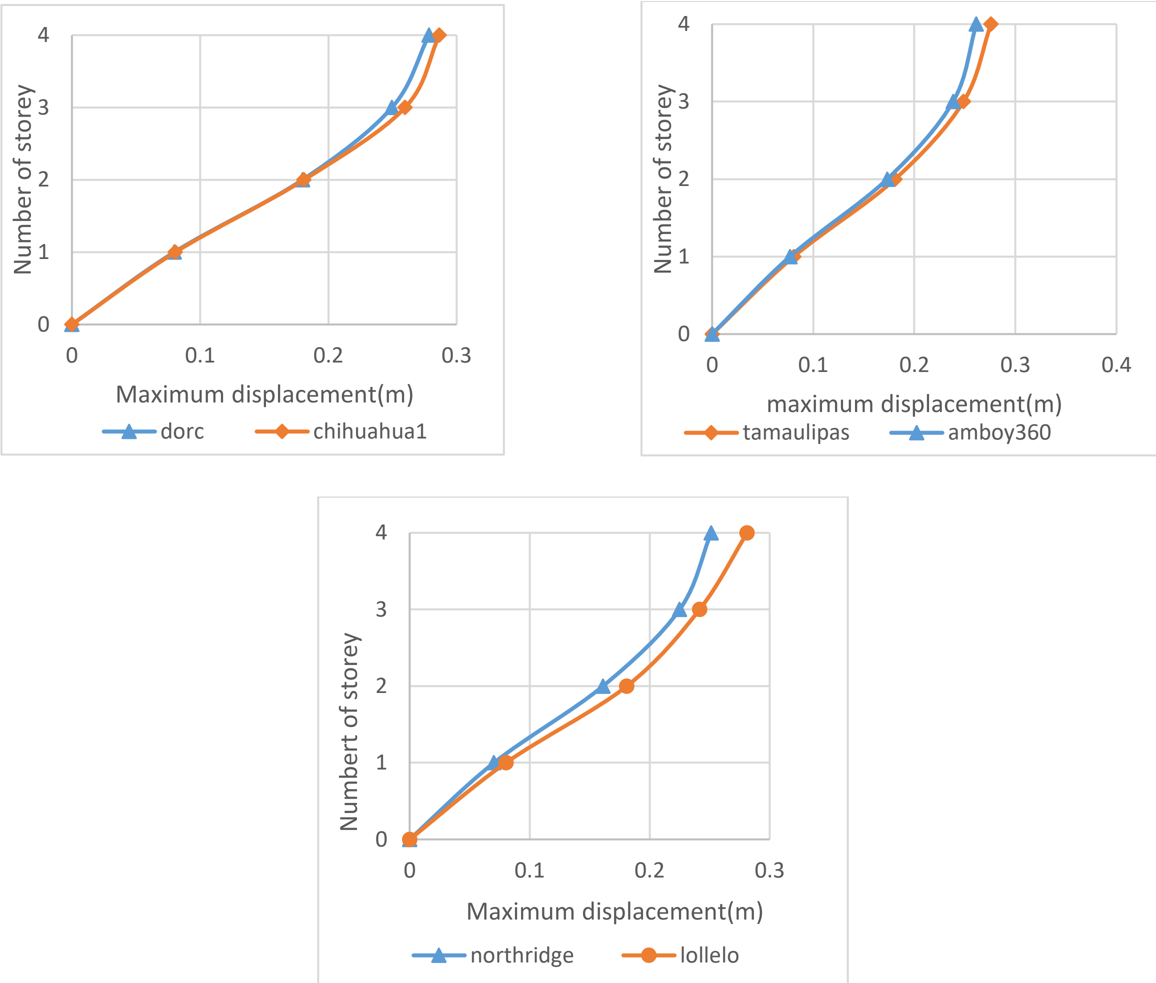
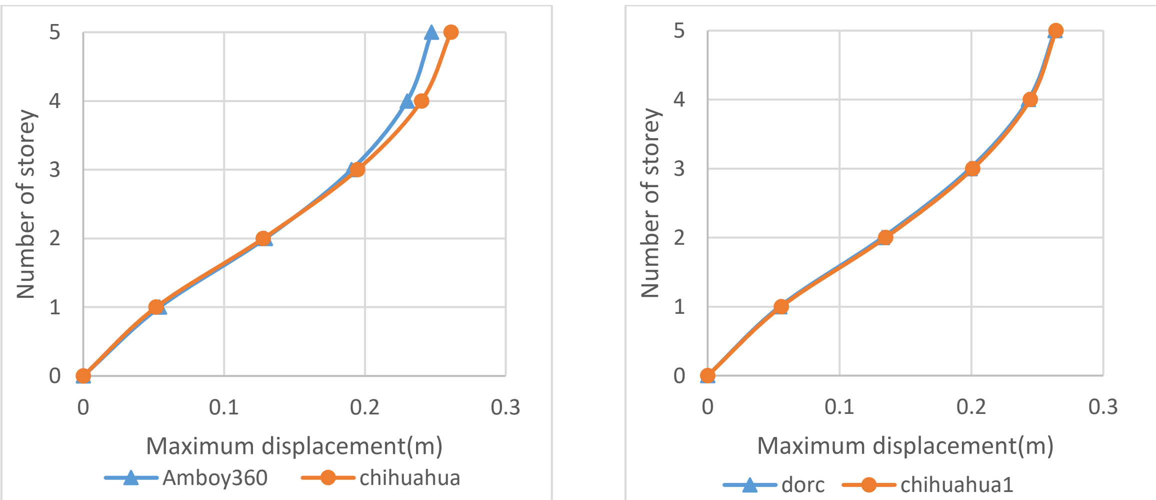


Figure 6- 1 Lateral story displacement of the 4-storey building for short and long duration motion



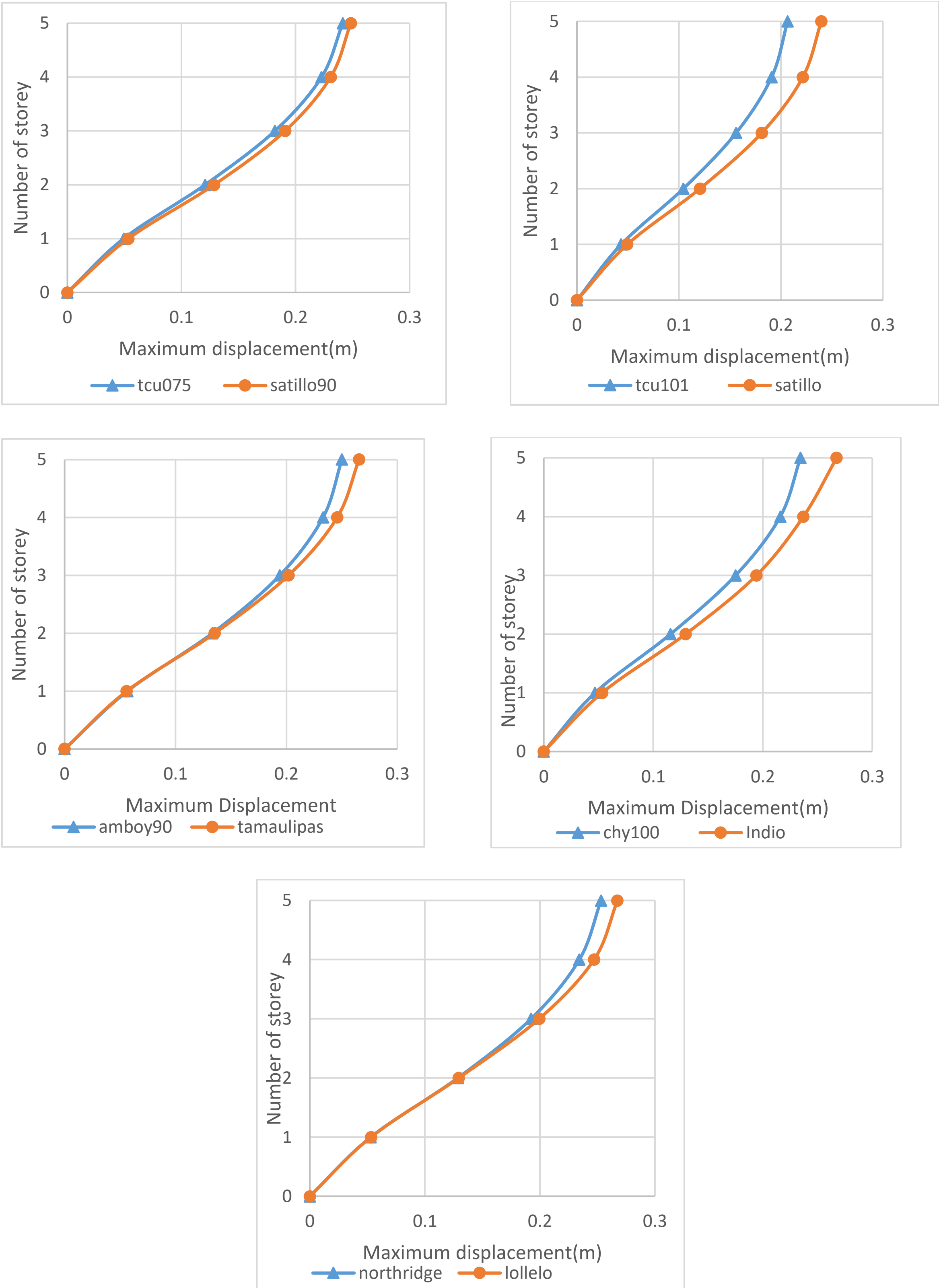
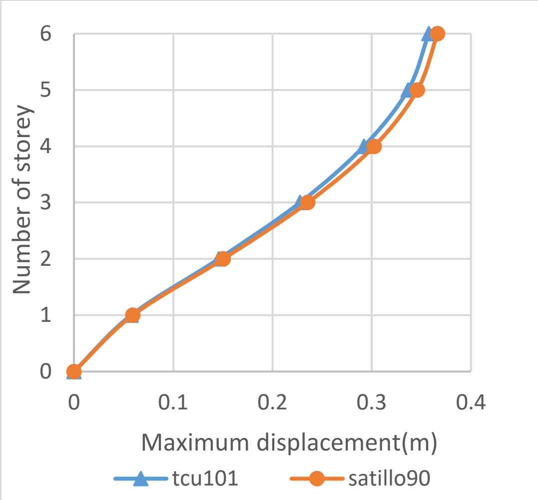
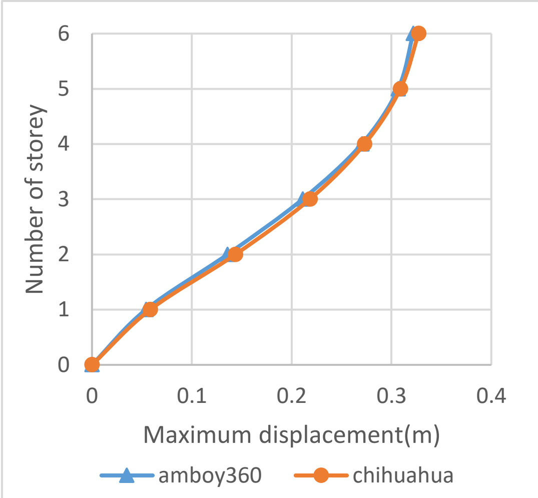
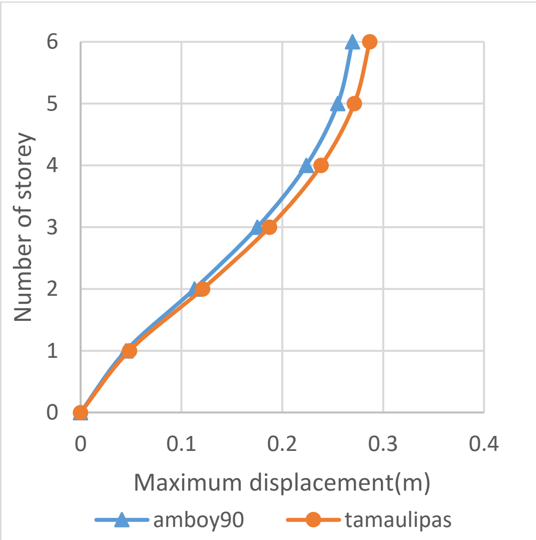
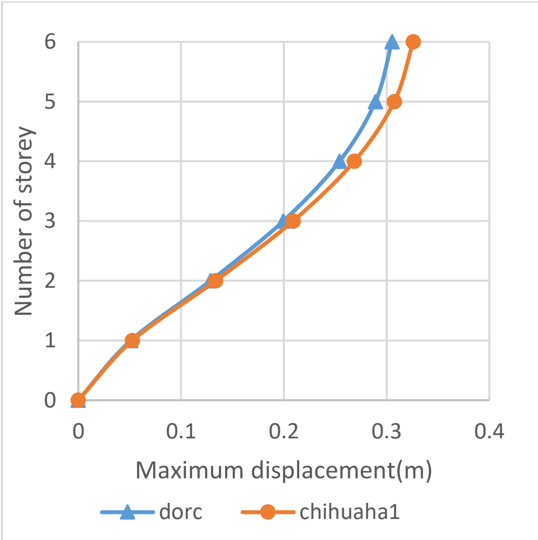


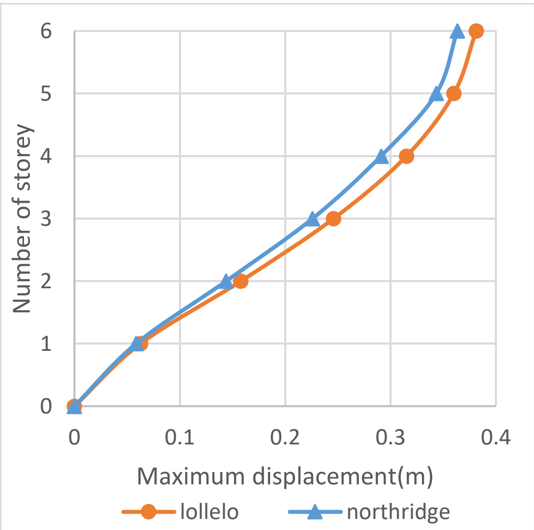
Figure 6- 2 Lateral story displacement of the 5-storey building for short and long duration motion

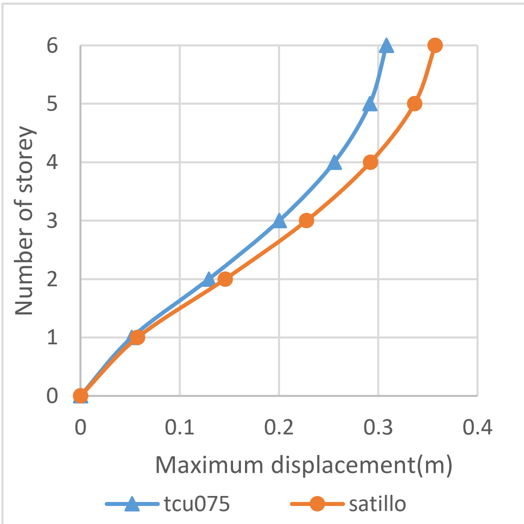

Number of storey
Maximum displacement(m)
tcu101
satillo90


Number of storey
Maximum displacement(m)
amboy360
chihuahua


Number of storey
Maximum displacement(m)
amboy90
tamaulipas


Number of storey
Maximum displacement(m)
dorc
chihuaha1


Number of storey
Maximum displacement(m)
lollelo
northridge


Number of storey
Maximum displacement(m)
tcu075
satillo

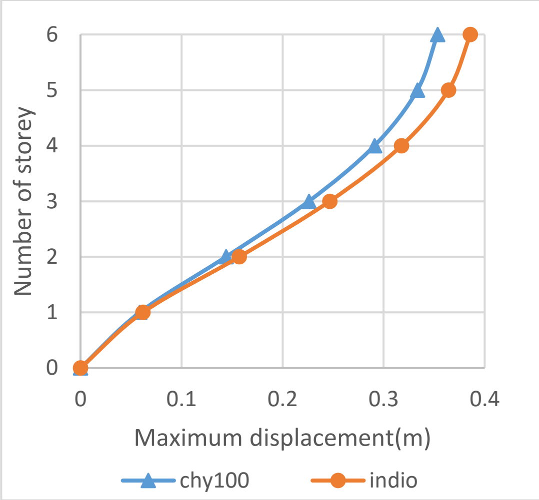
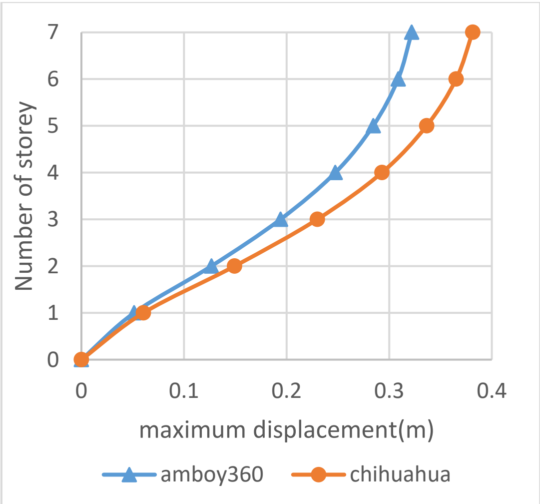
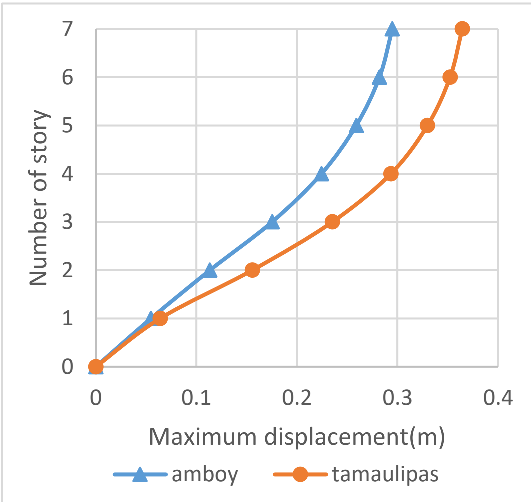
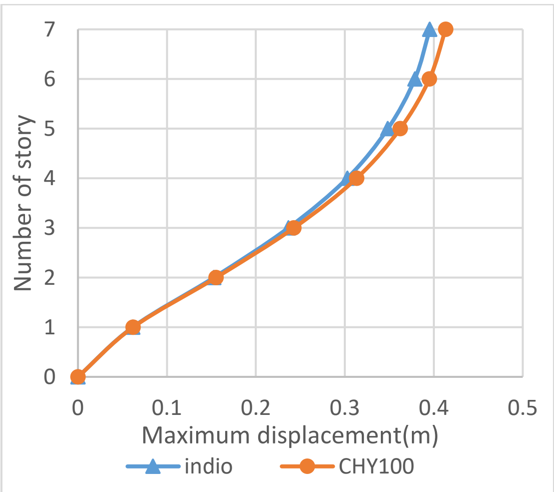
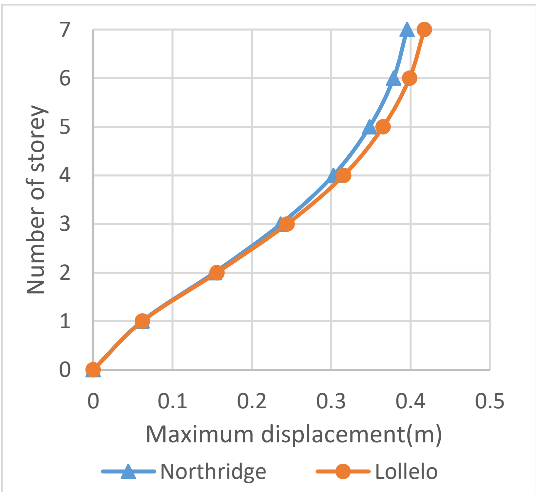


Figure 6- 3 Lateral story displacement of the 6-story building for short and long duration motion









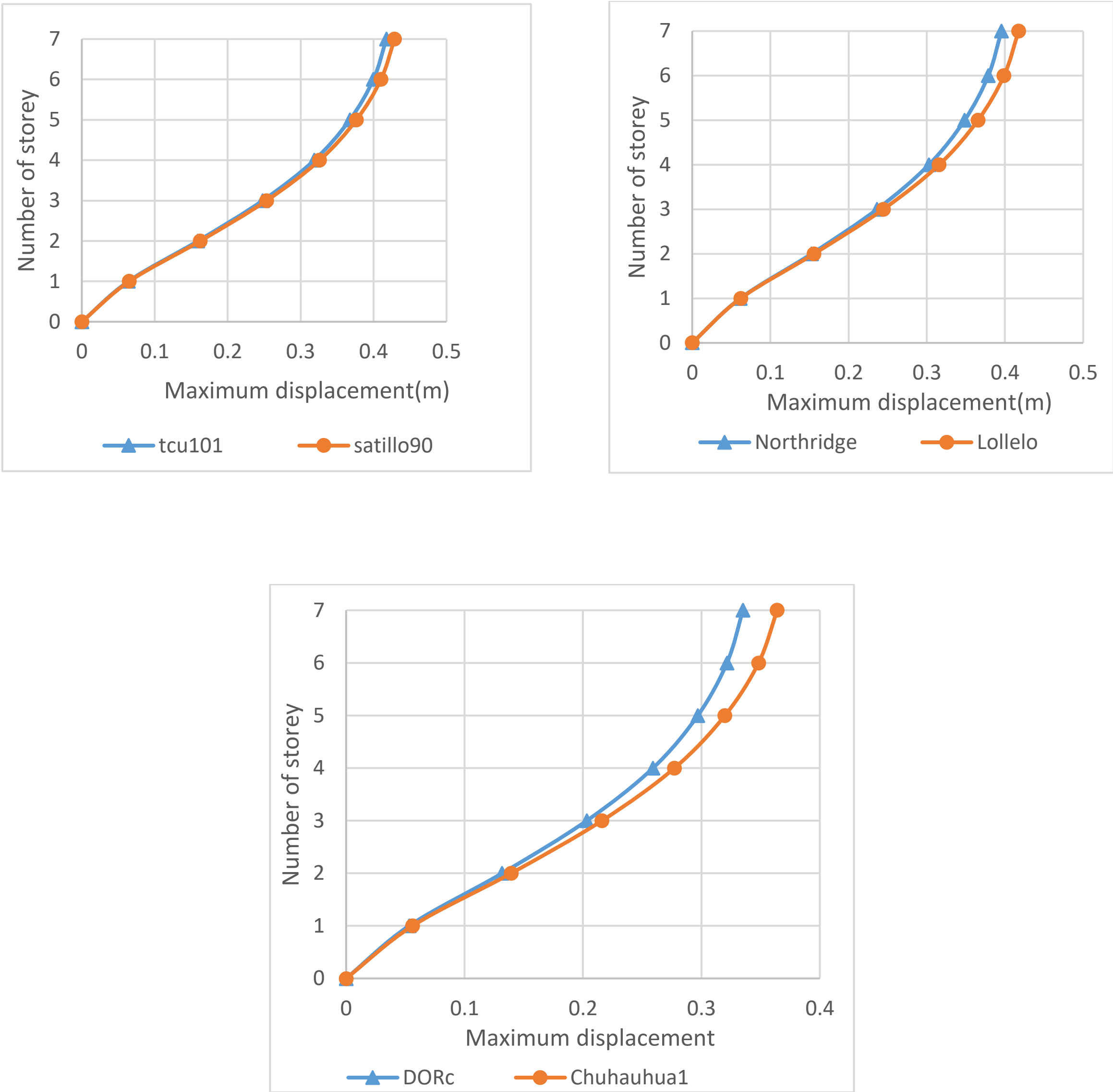
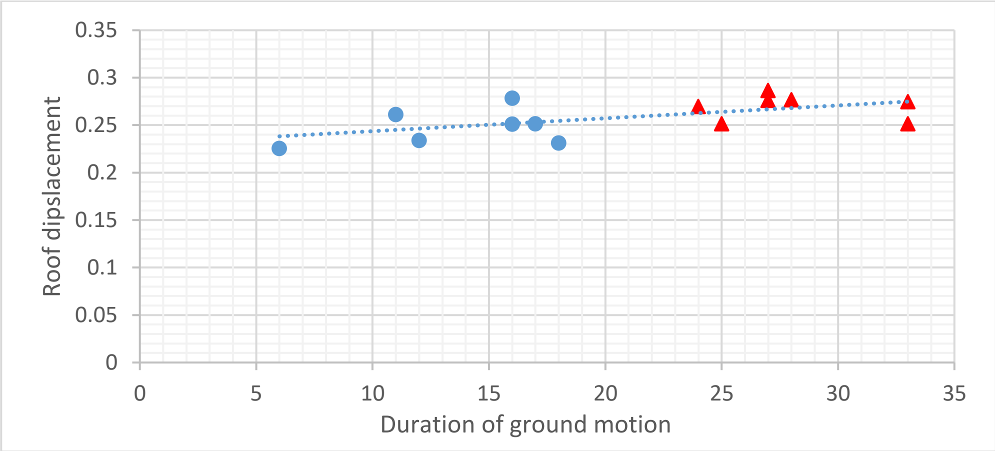


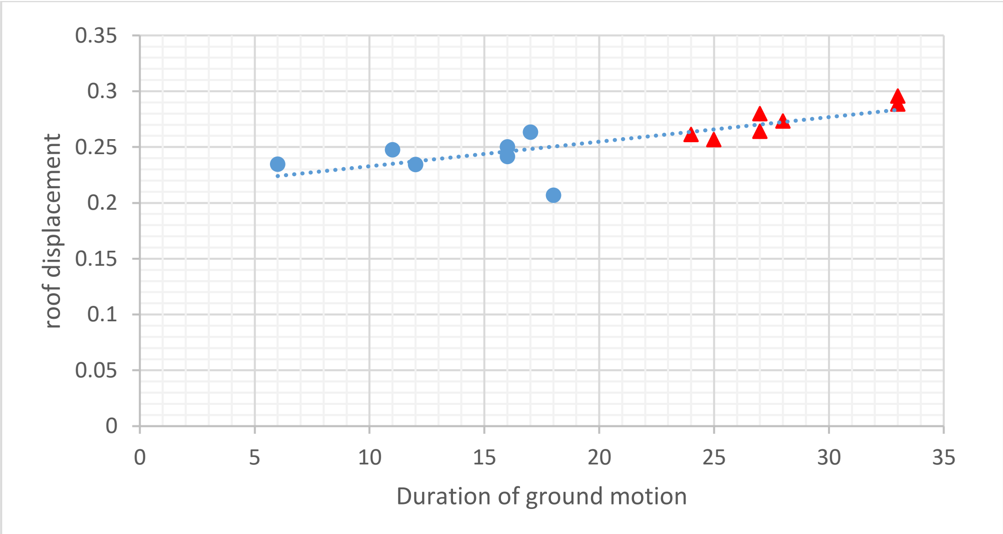
Figure 6- 4 Lateral story displacement of the 7-storey building for short and long duration motion

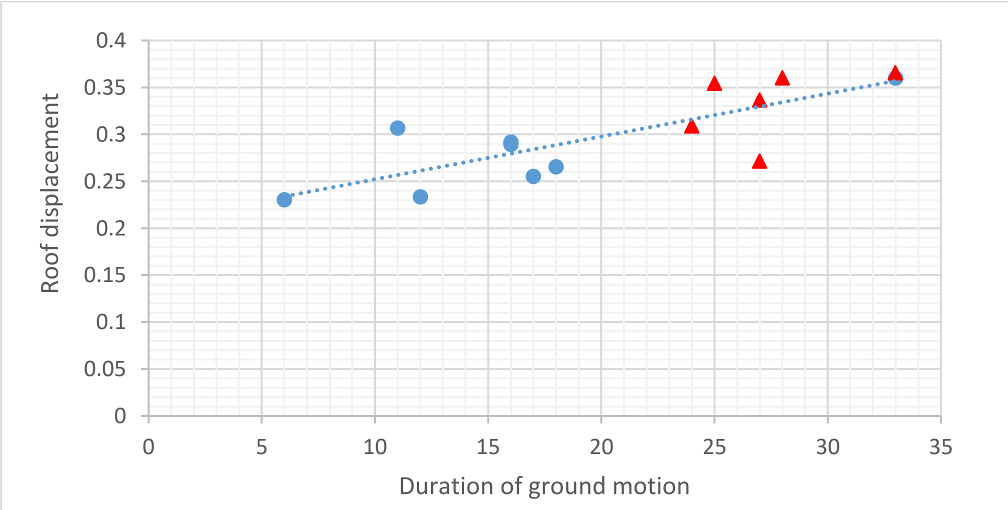
All of the aforementioned graph lines in Figures 6-1 to 6-4 illustrate the increase in displacement when comparing data on longer-duration ground motion to data on shorter-duration ground motion. It might be because a longer duration earthquake causes the building to lose strength and alter its stiffness characteristics. Additionally, the aforementioned graphs demonstrate that structures with fewer stories perform better when a longer-duration earthquake occurs. The performance level declines as a building's height, or number of stories, increases since a 7-story building has a higher value of displacement than a 6-story building, and similarly, a 6-story building has a higher value of displacement than a 5-story one, and so on.

6.2 SIGNIFICANT DURATION VERSUS DISPLACEMENT

The significant duration of the taken ground motion versus the top roof displacement value is plotted.







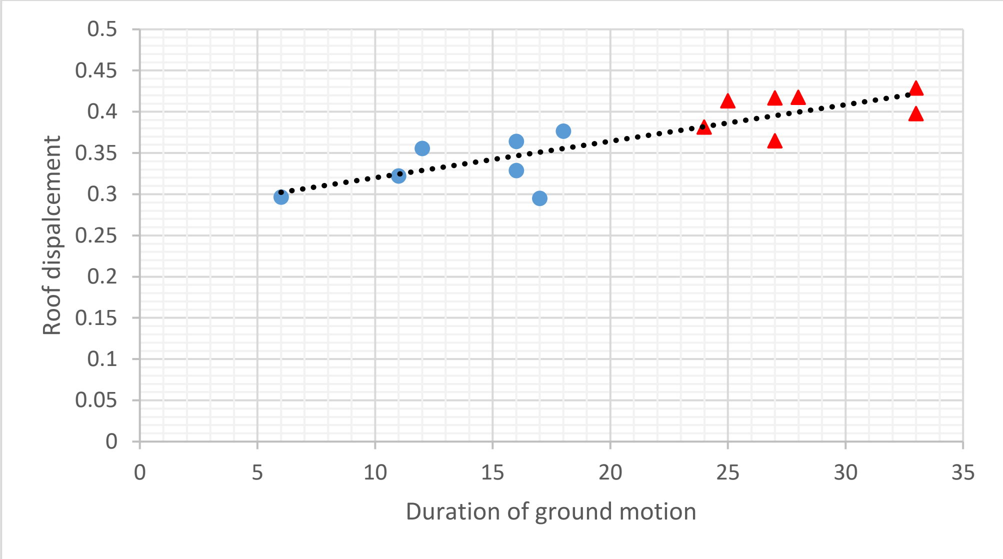


Figure 6- 5 Significant duration vs roof displacement graph for 4 ,5 6, and 7 story building respectively

Following simulations of the 14 ground motion inputs (seven pairs of spectrally matched short and long duration ones), the regression line demonstrates incremental order, indicating that the value of the displacement increases as the duration of the ground motion increases. This demonstrates that a structure's vulnerability has risen. Additionally, the increase is greater for a 7-story building than a 4 story building. As a result, the results suggest that while evaluating seismic performance, the duration of the ground motion should be taken into account.

6.3 MAXIMUM INTERSTORY DRIFT RATIO AND IDA CURVE

Maximum Inter-story Drift Ratio is selected as the Engineering Demand Parameter and Peak Ground Acceleration is used as the Intensity Measure, among other factors. The results of multiple simulations of non-linear time history analysis performed on four structure designs under seven pairs of varying-duration seismic ground motions led to the development of the IDA curves presented here. As seen in the picture, these curves are drawn between the PGA and the maximum inter-story drift ratios (IDR).

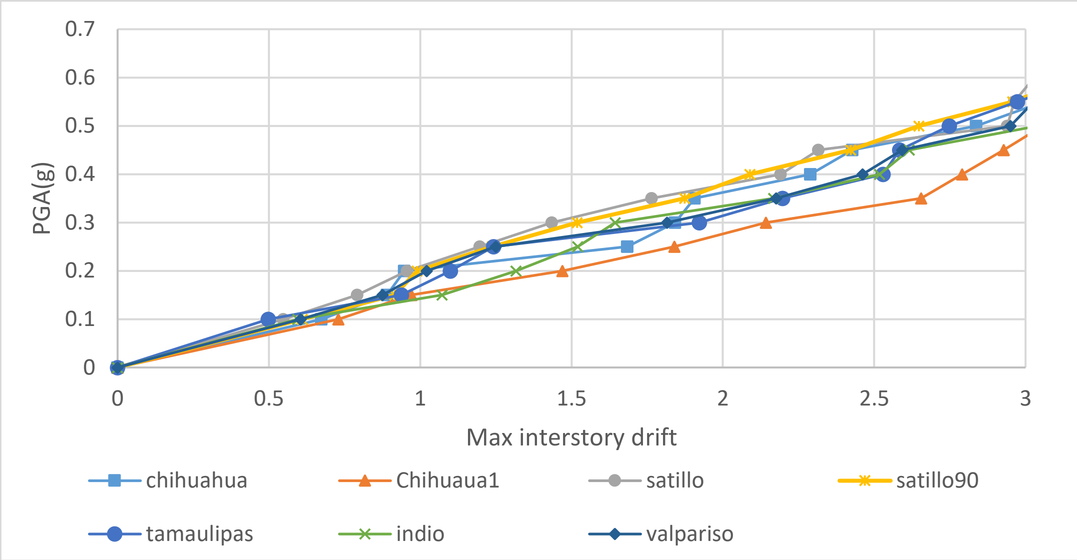


Figure 6- 6 IDA curve for ground motion with long duration on 4 story building

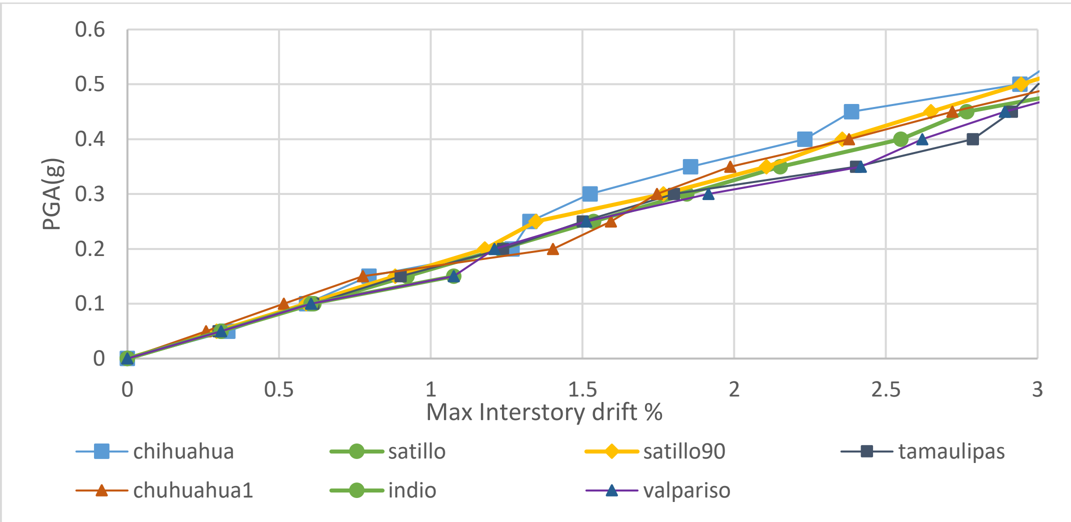


Figure 6- 7 IDA curve for ground motion with long duration on 5 story building

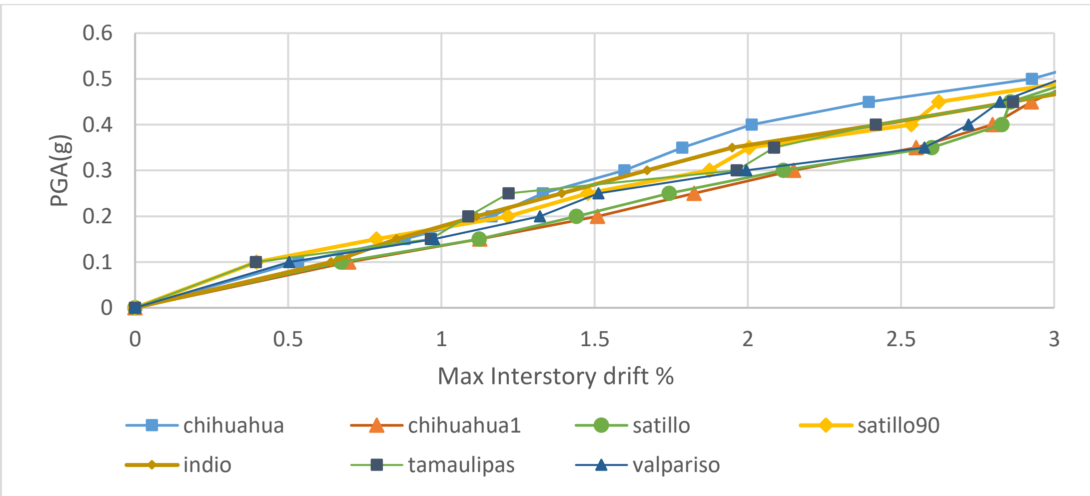


Figure 6- 8 IDA curve for ground motion with long duration on 6 story building

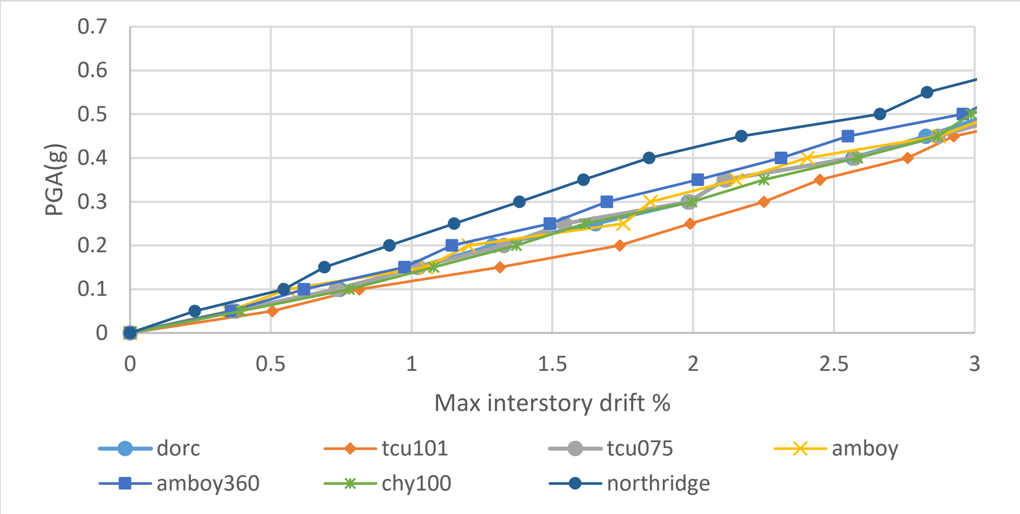


Figure 6- 9 IDA curve for ground motion with long duration on 7 story building

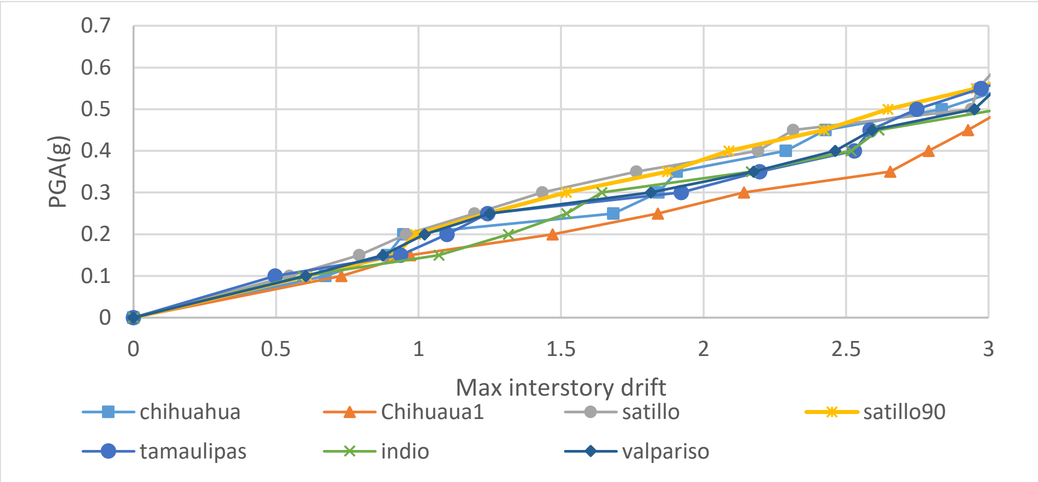


Figure 6- 10 IDA curve for ground motion with short duration of 4 story building

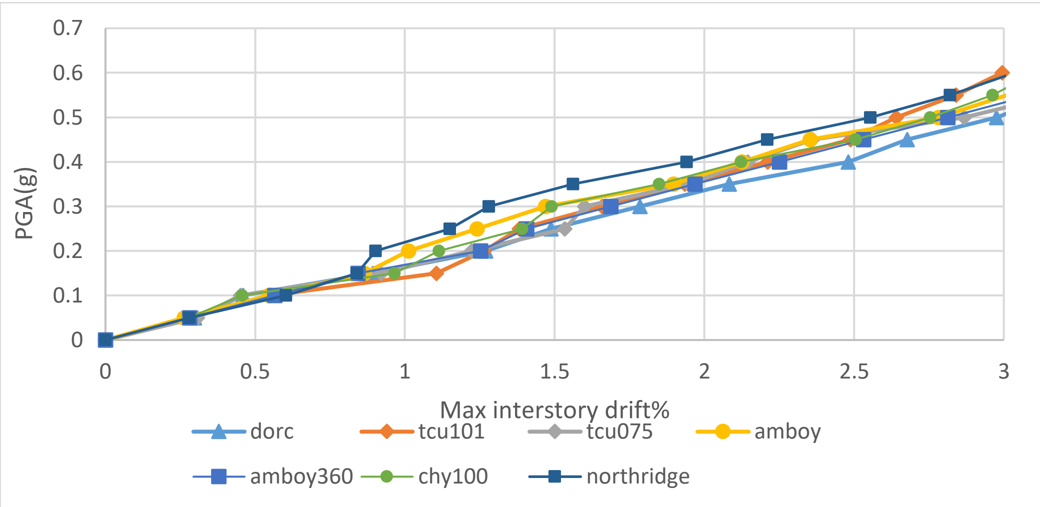


Figure 6- 11 IDA curve for ground motion of short duration for 5 story building

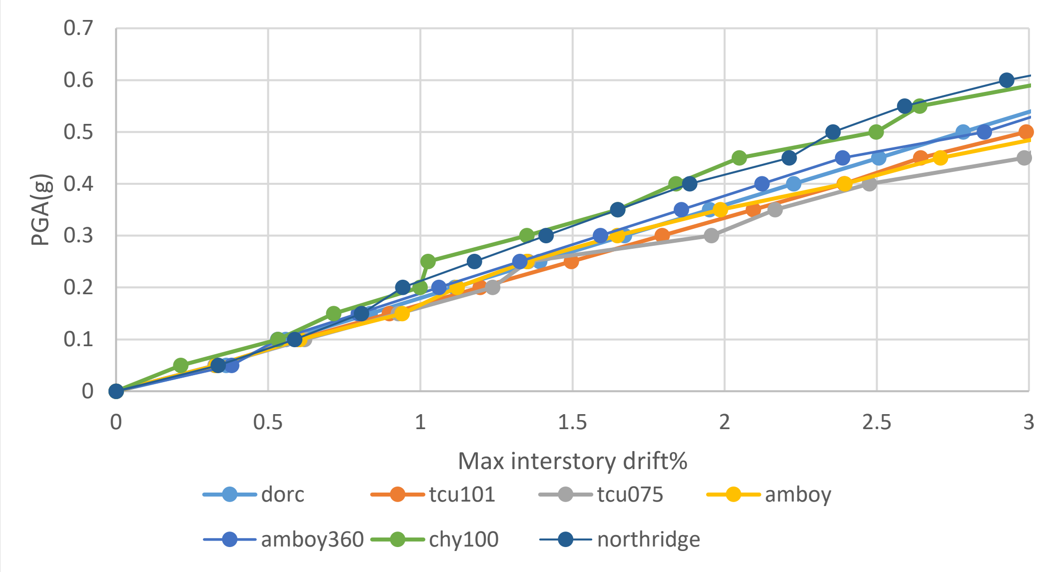


Figure 6- 12 IDA curve for ground motion with short duration for 6 story building

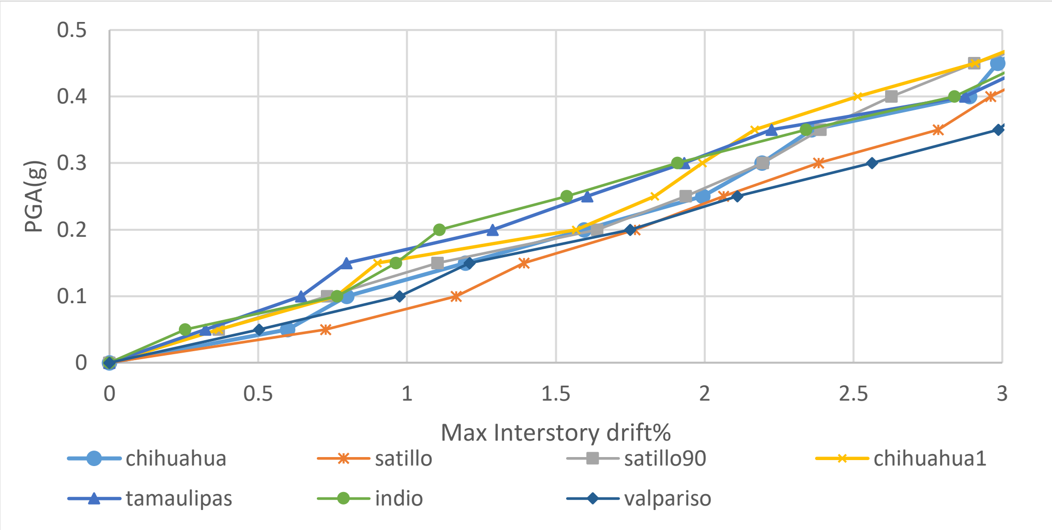


Figure 6- 13 IDA curve for ground motion with short duration for 7 storey building

From the above IDA curve, it shows that under long duration earthquake the ground motion data attains Collapse prevention state at around 0.5g to 0.38g. Similarly, for 5,6 and 7 story it is around (0.48-0.35g), (0.45-0.33g)and (0.47-0.33g) respectively. And under short duration it is (0.5-0.4g), (0.48-0.39g),(0.48-0.37g)and (0.4-0.3g) respectively for 4 to 7 story building. Thus, it shows that with the increase of the number of the story in the building the value of the maximum interstorey drift percentage increases rapidly at the lower value of PGA which indicates that the structure is approaching towards the collapse state at higher rate.

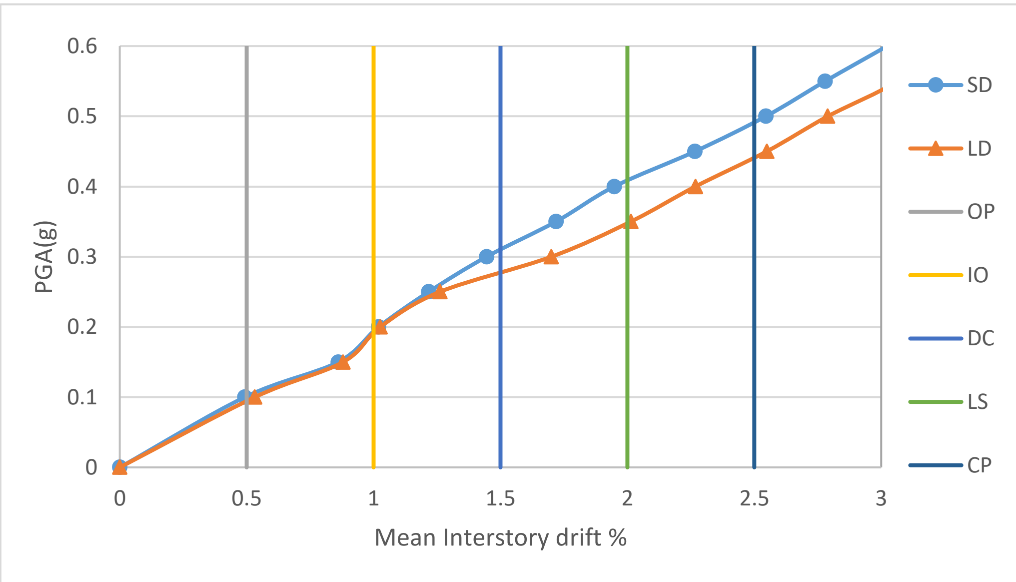
6.4 MEAN IDA CURVE

From the IDA curve plotted above for 4,5,6 and 7 story building respectively, a mean IDA curve is plotted as below. The mean IDA curve provides the distinct curves for the long and short duration earthquake making it easy to see the impact at various Intensity measure. These IDA curves are further used for extracting the fragility curves.

For generating the fragility curve, it is necessary to define the drift limits. Here the drift limit is defined as in table below. The performance limits for the study are defined through performance-based seismic design as operational phase (OP), Immediate occupancy (IO), Damage control (DC), life safety (LS), and collapse prevention (CP) as per FEMA 356.

Table 6- 1 Drift limits

Limit state	Drift %
Operational Safety (OP)	0.5
Immediate Occupancy(IO)	1
Damage Control(DC)	1.5
Life Safety(LS)	2
Collapse Prevention(CP)	2.5



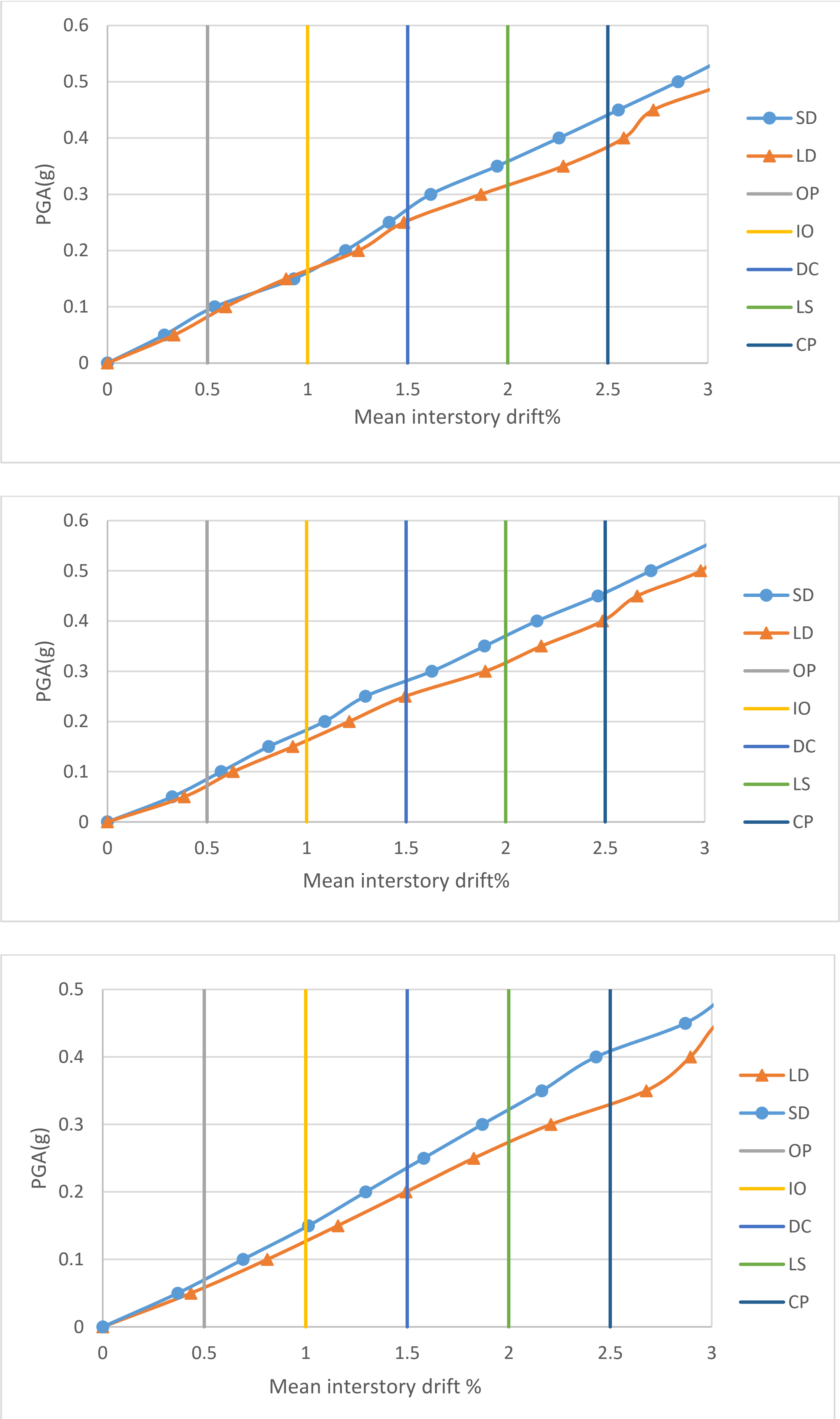


Figure 6- 14 mean IDA curve for 4,5,6 and 7 story RC buildings respectively

As seen from the graph above, the higher story building tends to have higher value of inter story drift ratio at lower value of PGA than that of the lower story building. This means the higher story buildings tend to reach collapse state (3% of IDA in our study) at lower intensity measure i.e., PGA as compared to the lower story building.

And from graph, for 4 story building when subjected to the longer duration ground motion reached collapse state at0.4g while when it was subjected to short duration ground motion reached collapse state at 0.49 g. Similarly for 5, 6 and 7 story building the collapse state is attained at 0.39g, 0.38g and 0.34g respectively for longer duration ground motion and 0.47g, 0.45g and 0.39g for short duration ground motion respectively. This shows that the buildings reach collapse state (3% of EDP in this study) at lower lave of PGA (Intensity measure). So, this justifies that when the buildings are subjected to longer duration motion, it loses its capability of resisting the seismic forces sooner.

Also, from the graphs below it shows that the effect of duration increases at lower intensity measure in case of higher story building as compared to small story building. For 6 and 7 story building we can see difference in short and long duration ground motion form Immediate occupancy level, while for 4 and 5 story building at IO level the effect of short and long ground motion is same so the mean curve seems to be coinciding each other.

Table 6- 2 : value of PGA at collapse for long and short duration earthquake

Number of stories	Value of PGA at collapse state(g)		
	Short duration	Long duration	%decrease in value at collapse
4	0.49	0.45	8.8
5	0.45	0.39	15.38
6	0.45	0.38	18.42
7	0.4	0.31	29

6.4 RESULTS FROM FRAGILITY ANALYSIS

The fragility parameters mean and standard deviation are assessed from the generated IDA curves in accordance with ATC 40 recommendations for the collapse limit state. Then the fragility curve for all 4 building for different limit states are plotted as below. In each of the figure the fragility curves for long and short duration earthquakes are compared with each other. Here IO1 , OP1, DC1, LS1 and CP1 represents the drift limits for long duration ground motions and OP,IO,DC,LS and CP represents the drift limits for short ground motion.

Table 6- 3 Mean and standard deviation value for 4 story building

Earth quake	IO		OP		DC		LS		CP	
	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ
Short	0.10 78	0.01 3641	0.204 2	0.011 3417	0.30 05	0.014 0812	0.396 8	0.019 8749	0.493 1	0.02 6811
Long	0.08 1235	0.01 266	0.177 3676	0.012 379	0.26 6775	0.012 5674	0.356 1874	0.013 207	0.445 5972	0.01 423

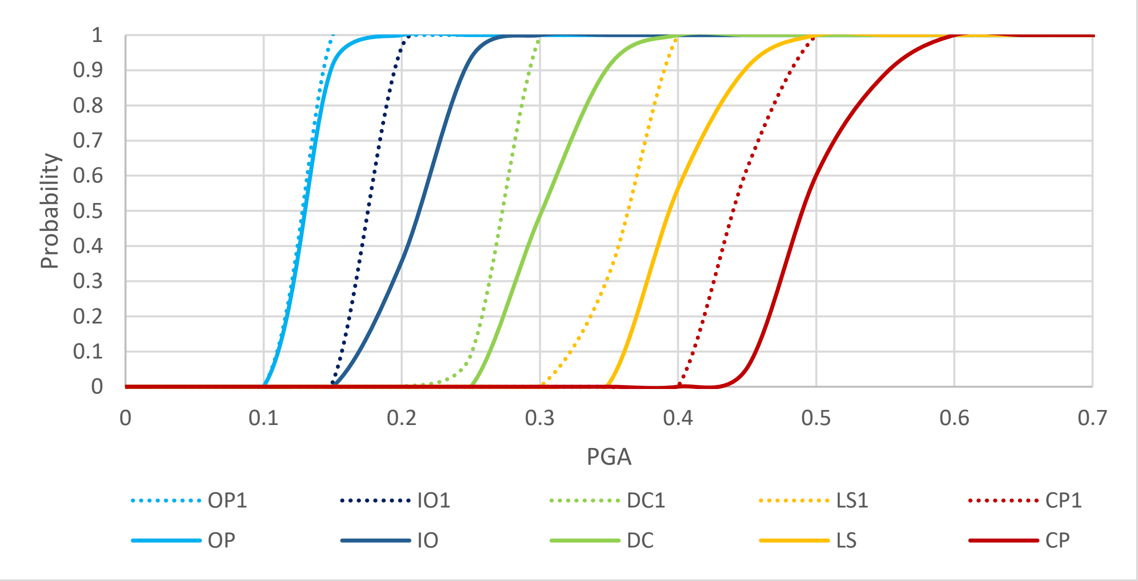


Figure 6- 15 Comparison of fragility curve for long and short ground motion for 4 story building

The table 6-3 shows the mean and standard deviation value as calculated per ATC 40 guidelines. The calculated value are then used to generate the fragility curves as in figure 6-15. As seen in figure 6-15, at 0.2g, the OP and IO level for 4 story building has probability of 100% under both long and short duration earthquake. At LS, DC and CP level, the probability is 0%. At 0.5g, the probability of exceeding and reaching the CP level is 100% for long duration earthquake while is only 60% for short duration earthquake. Thus, under the longer duration earthquake the probability of collapse decreased by 40%.

Table 6- 4 Mean and standard deviation value for 5-story building

Earthquake	IO		OP		DC		LS		CP	
	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ
Short	0.090256	0.0120959	0.182113	0.0143635	0.273966	0.0176068	0.3658219	0.0213863	0.4576772	0.0254645
Long	0.07671	0.0083104	0.162689	0.009804	0.248646	0.01147	0.33461	0.01385	0.42057	0.016101

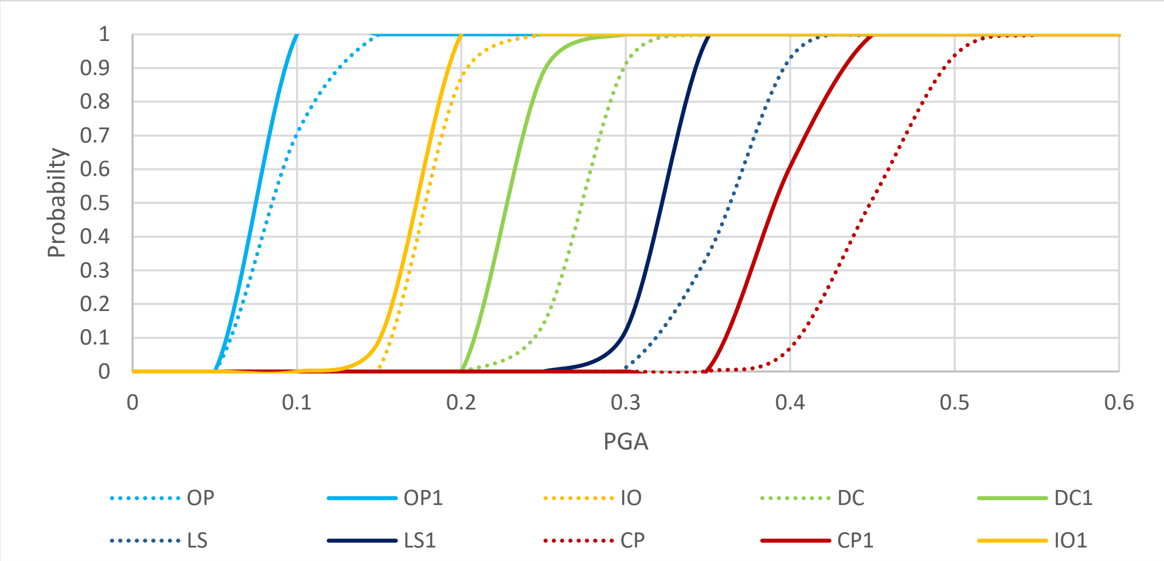


Figure 6- 16 Comparison of fragility curve for long and short ground motion for 5 story building

The table 6-4 shows the mean and standard deviation value as calculated per ATC 40 guidelines. From figure 6-16, for 5 story building, at 0.2g for OP level the probability is 100%. For CP level it is 100% for long duration and 90% for short duration earthquake. At 0.45g, the probability is 100% for long duration while only 70% for short duration earthquake. Thus, under the longer duration earthquake the probability of collapse decreased by 30%.

Table 6- 5 Mean and standard deviation value for 6 story building

Earthquake	IO		OP		DC		LS		CP	
	μ	σ	μ	σ	μ	σ	μ	σ	μ	σ
Short	0.0951	0.02987	0.1832	0.0184826	0.2712	0.023386	0.3593	0.0286659	0.4473	0.0341482

Long	0.07 148	0.01 9473	0.15 8189	0.014 539	0.244 8917	0.02 1343	0.33 1594	0.025 819	0.41 8297	0.031 2903

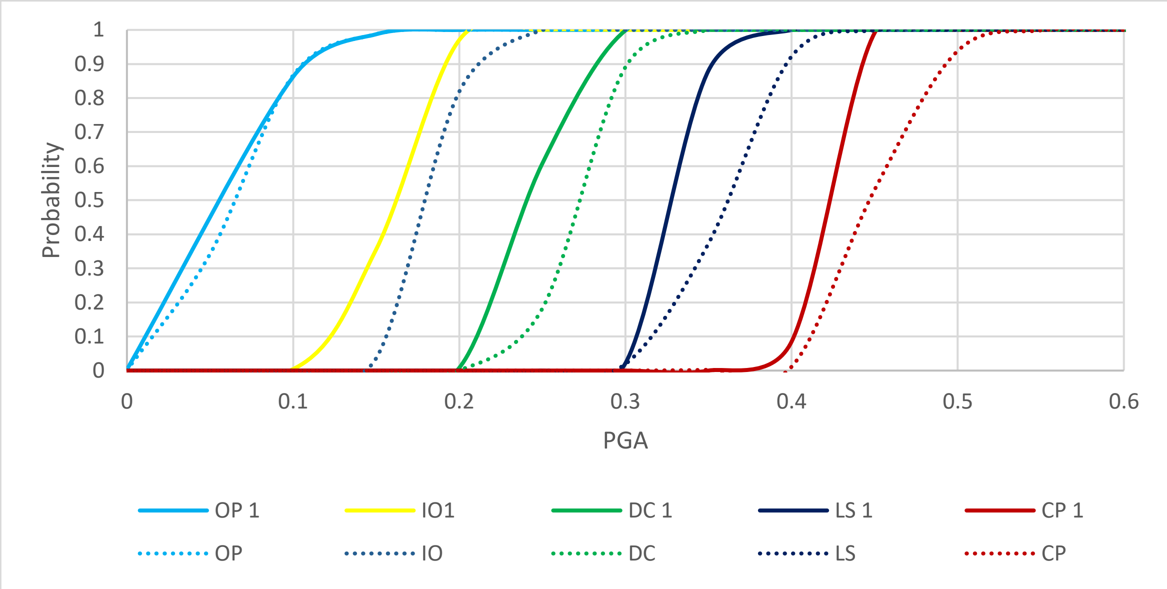


Figure 6- 17 Comparison of fragility curve for long and short ground motion for 6 story building

Similarly, from figure6-17, at 0.2g the OP level for 5 story building has probability of 100% while at IO level the probability is 100% for long duration earthquake while 80% for short one. At 0.44g the probability of reaching and exceeding the CP level is 100 for long duration earthquake while it is 55% for short ones. Thus, under the longer duration earthquake the probability of collapse decreased by 45%

Table 6- 6 Mean and standard deviation value for 6 story building

Earthquake	IO		OP		DC		LS		CP	
	μ	σ	μ	σ	μ	Σ	μ	σ	μ	σ
Short	0.07 48	0.016 54	0.15 94	0.020 50	0.2 441	0.025 19	0.3 287	0.030 28	0.41 34	0.035 60
Long	0.06 575	0.019 402	0.13 278	0.020 6248	0.2 098	0.022 6879	0.2 868	0.025 3873	0.36 387	0.028 5429

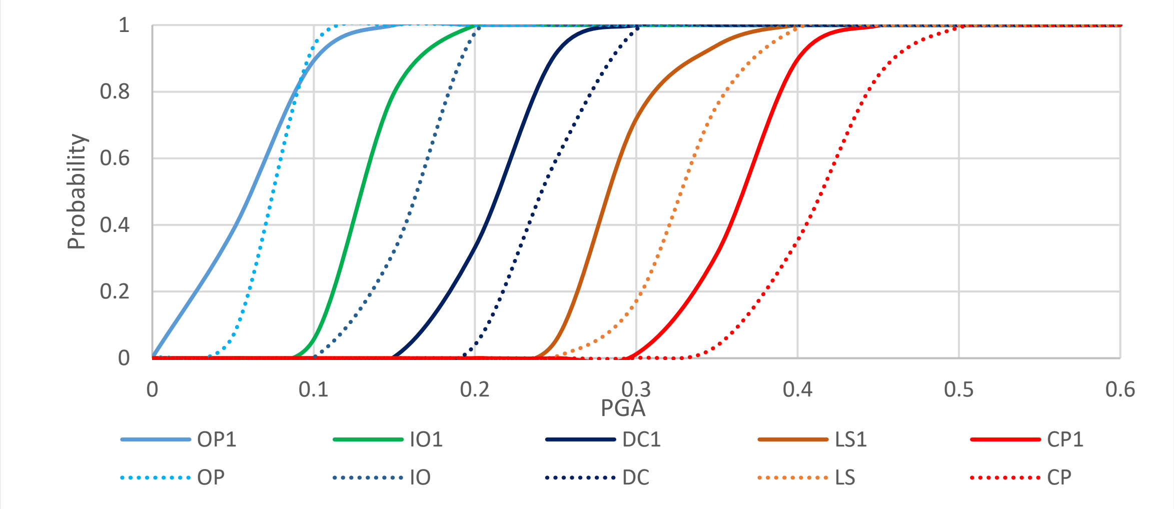


Figure 6- 18 Comparison of fragility curve for long and short ground motion for 7 story building

From figure 6-18, for the 7-story building OP level reaches 100% at 0.1g and CP at 0.2g. At 0.44g, the probability of CP level reaches 100% for long duration earthquake and 50% for short duration earthquake. Thus, under the longer duration earthquake the probability of collapse decreased by 50%.

There is. increase in the probability of structure reaching the severe seismic state at lower values of PGA during the occurrence of long duration earthquake. The probability of collapse decreases under longer duration earthquake as compared to shorter duration earthquake which also justifies that the occurrence of the long duration earthquake reduces the strength of the structure to resist the damage considerably thus introducing the necessity of its consideration during seismic analysis.

6.4.1 Shift of fragility curve

For each performance limit the shift of the fragility curve is plotted differently to have idea of change in collapse capacity at each performance limit with increases. The shift of the fragility curve for all the performance limits OP, IO, DC, LS and CP for particular building are shown in the figure .

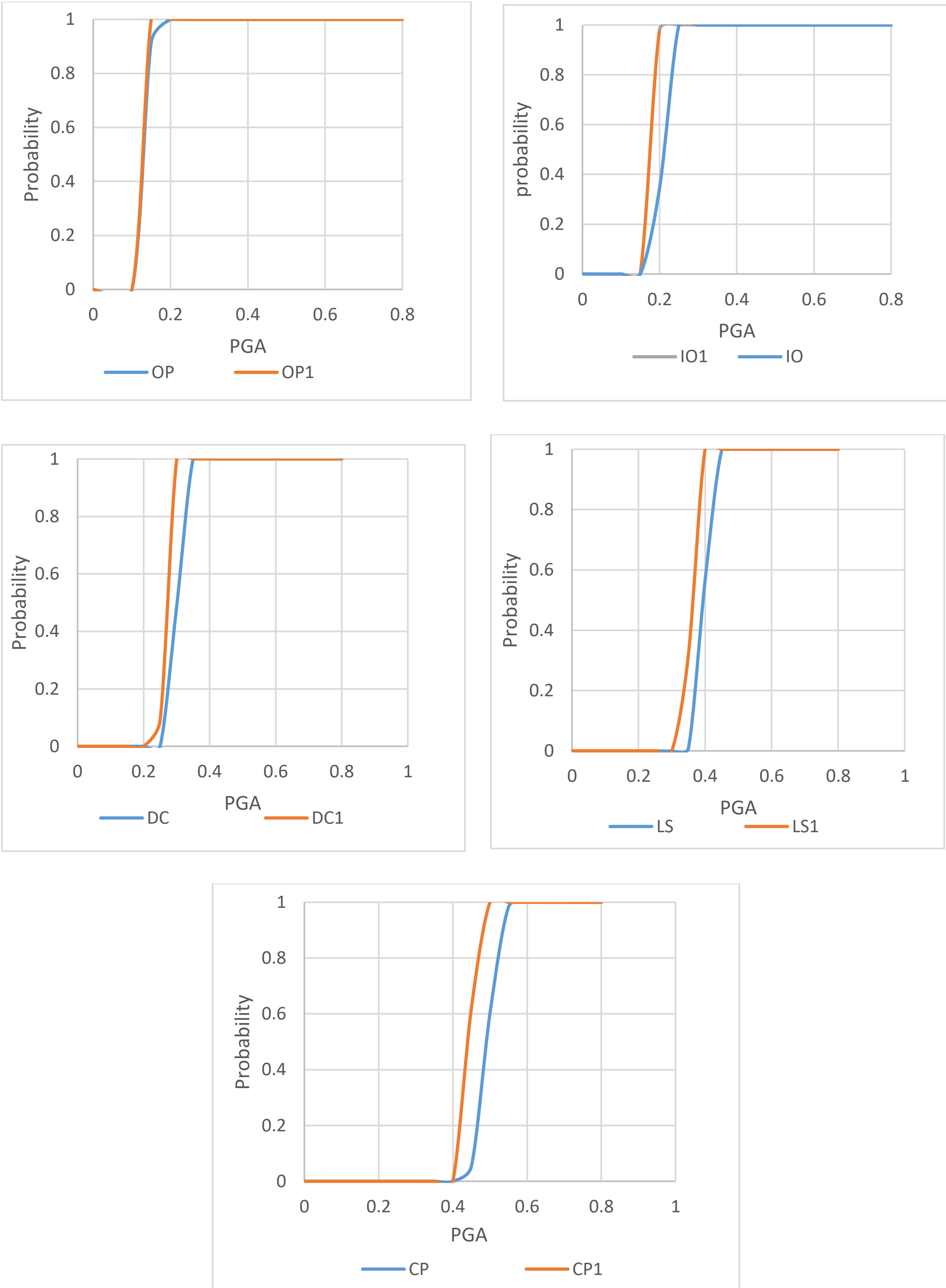
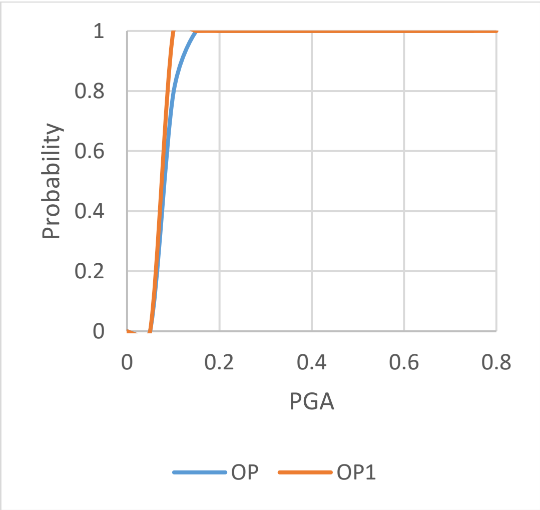
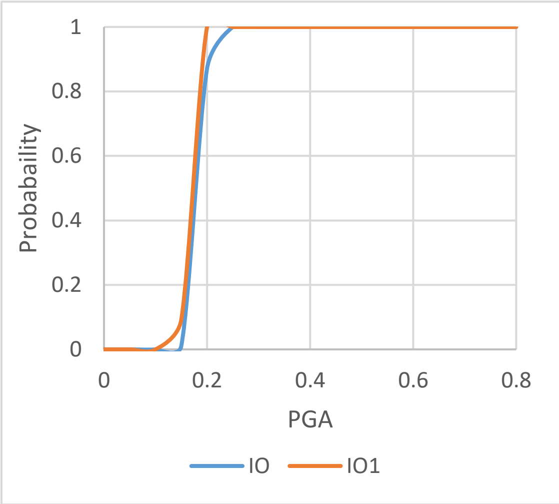
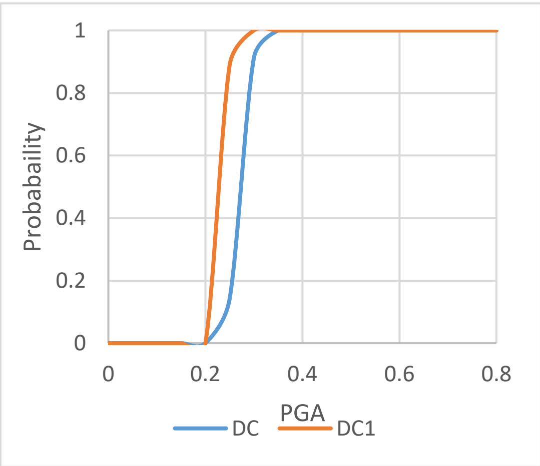


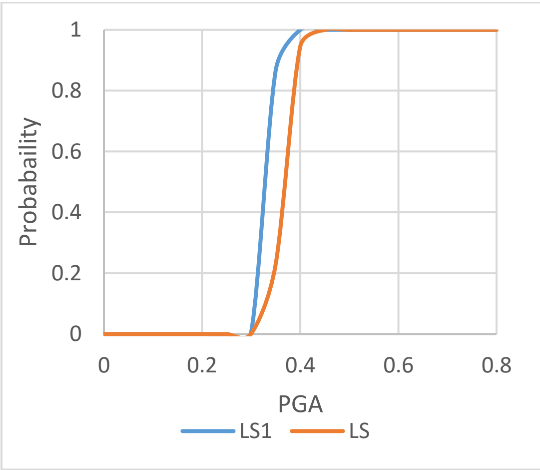
Figure 6- 19 Shift of the fragility curve for short and long duration ground motion for 4 story building

Here OP1, CP1 and so on shows the curve of long duration ground motion. It can be observed that from figure 6-19, the 4-story building reached the initial probability of collapse at 0.43g when subjected to long duration earthquake but achieved the same state at 0.4g in case of short duration earthquake. Similar to this, a structure has a 100% chance of collapsing during a long-duration earthquake at 0.5g, compared to 0.55g during a short duration earthquake.









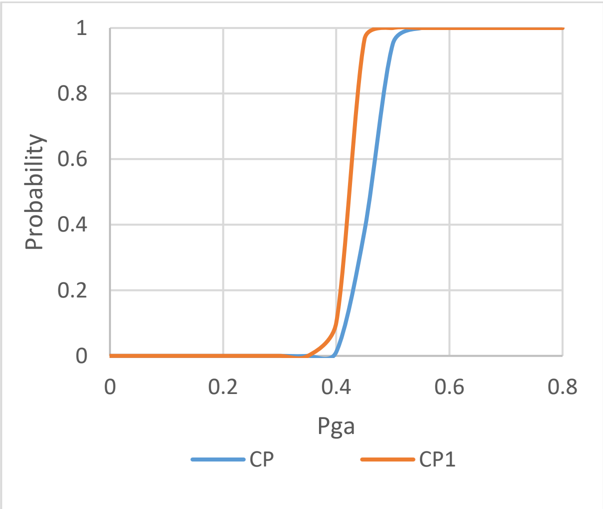


Figure 6- 20 Shift of the fragility curve for short and long duration ground motion for 5 story building

It can be observed that from figure 6-20, the 5-story building reached the initial probability of collapse at 0.35g when subjected to long duration earthquake but achieved the same state at 0.4g in case of short duration earthquakeSimilar to this, a structure has a 100% chance of collapsing during a long-duration earthquake at 0.45g, compared to 0.52g during a short-duration earthquake.

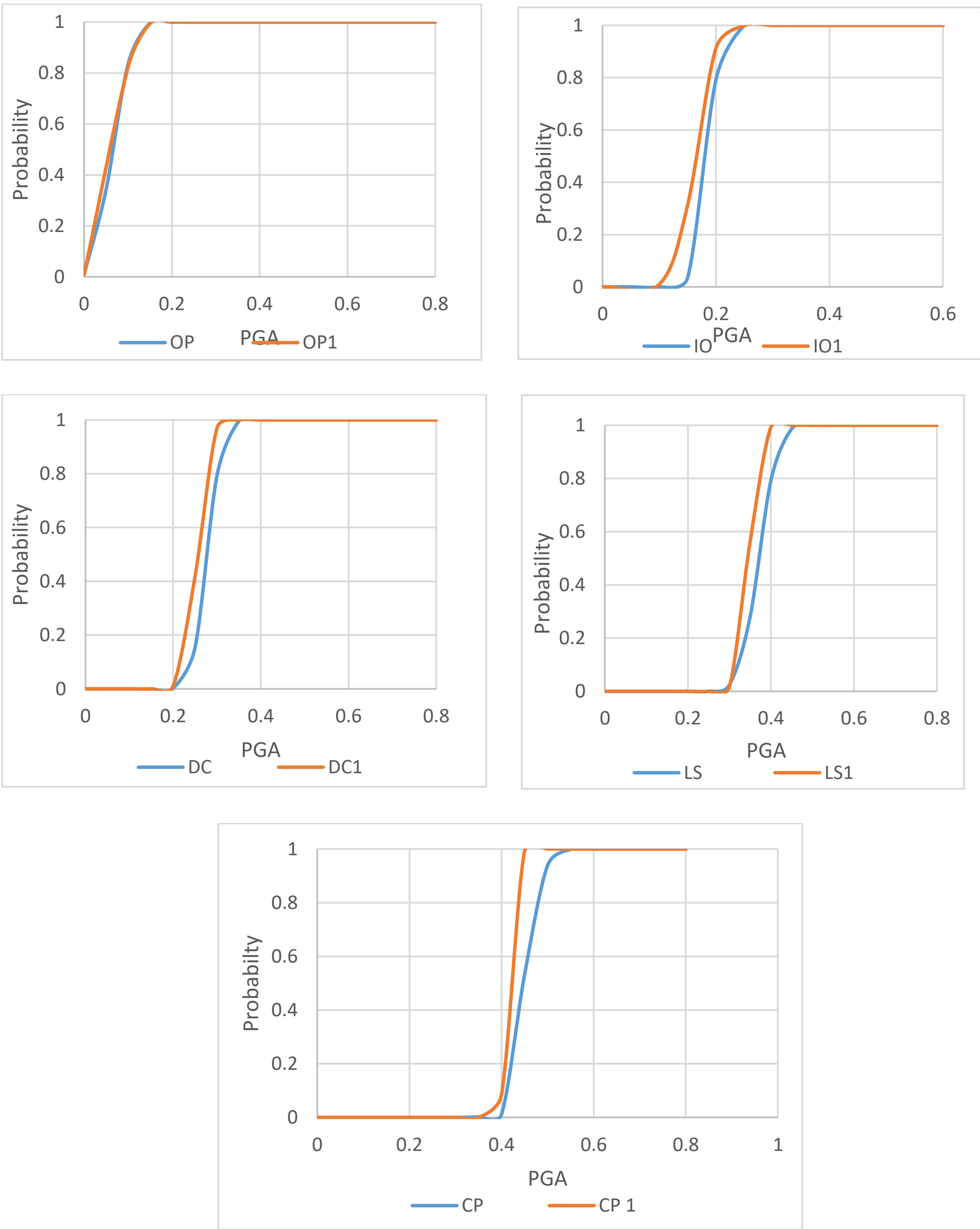


Figure 6- 21 Shift of fragility curve for long and short ground motion for 6 story building

It can be observed that from figure 6-21, the 6-story building reached the initial probability of collapse at 0.38g when subjected to long duration earthquake but achieved the same state at 0.4g in case of short duration earthquakeSimilar to this, a structure has a 100% chance of collapsing during a long-duration earthquake at 0.42g, compared to 0.5g during a short-duration earthquake.

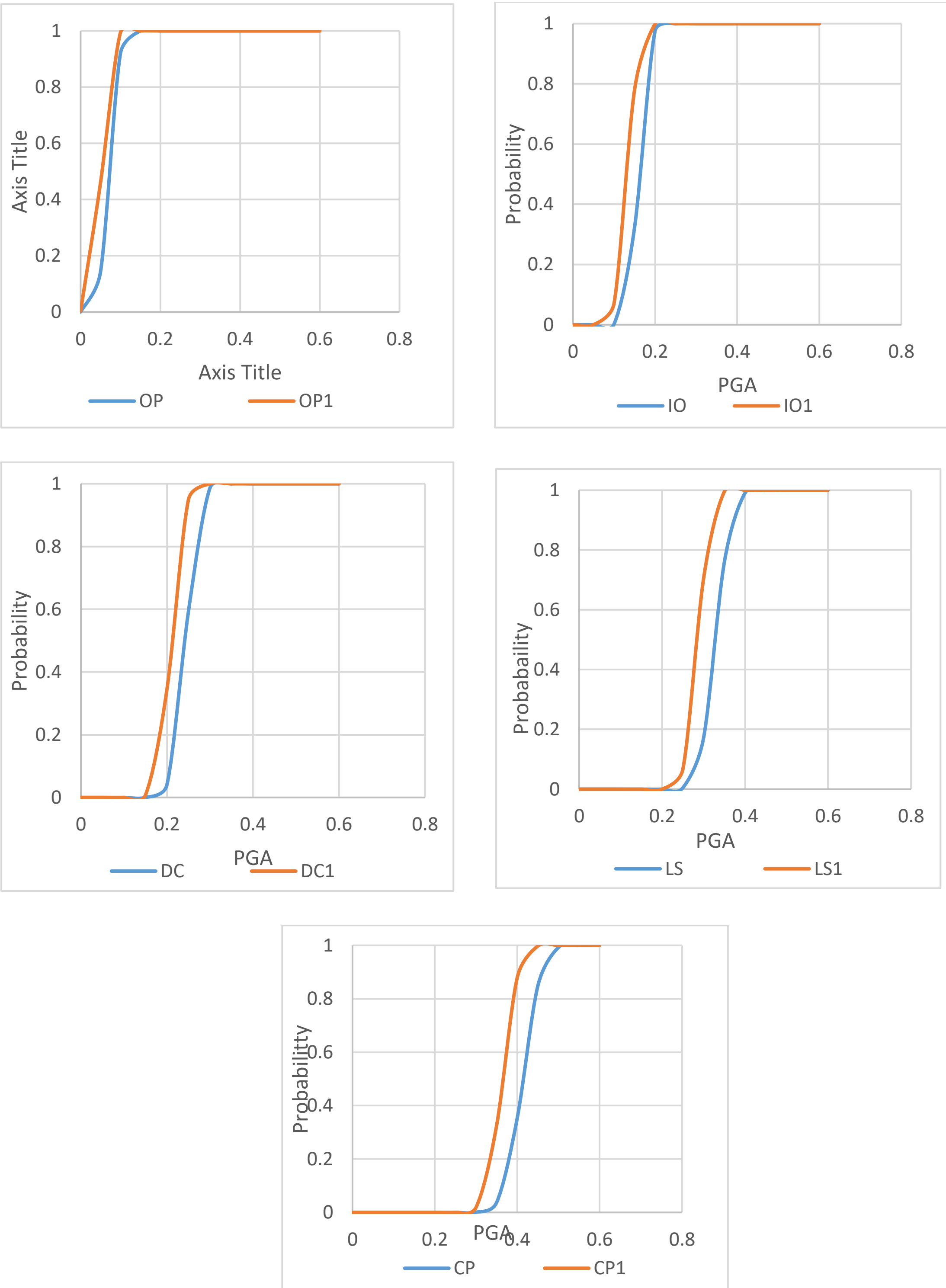


Figure 6- 22 Shift of fragility curve for long and short ground motion for 7 story building

It can be observed that from figure 6-22, the 7-story building reached the initial probability of collapse at 0.3g when subjected to long duration earthquake but achieved the same state at 0.35g in case of short duration earthquake. Similar to this, a structure has a 100% chance of collapsing during a long-duration earthquake at 0.42g, compared to 048g during a short-duration earthquake.

The shift of the curve of probability of collapse indicates the reduction in collapse capacity of structure with the occurrence of the longer duration earthquake. The cause for reduction may be due to the decrease in strength and stiffness characteristics of the structural members with increase in duration of the occurrence of earthquake.

6.5 DURATION VERSUS COLLAPSE CAPACITY RATIO

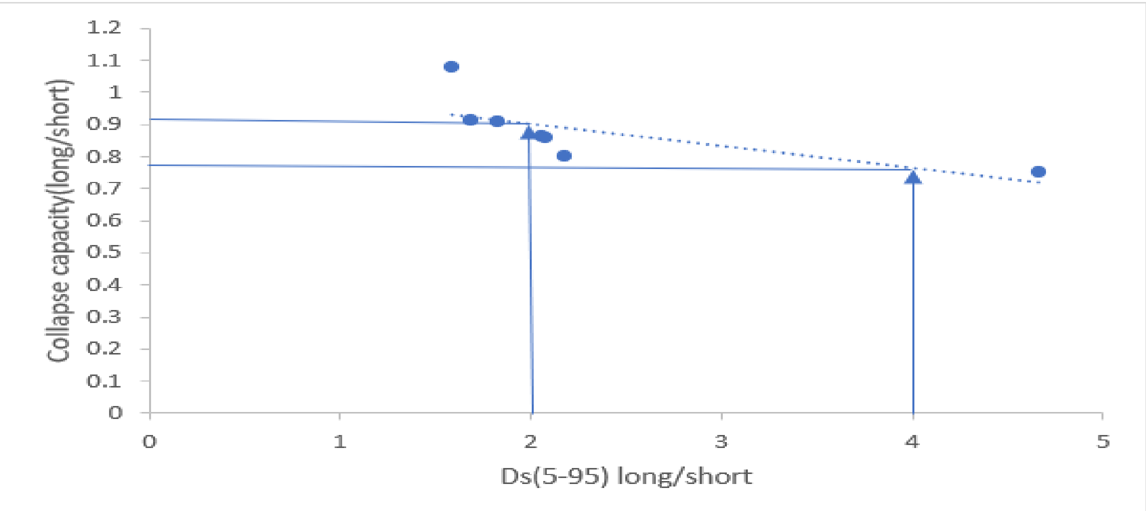


Figure 6- 23 Graph showing Ratio of significant duration versus ratio of collapse capacity for spectrally equivalent pairs for 4 story building

As can be seen in the graph above, for all equivalent record pairs, it is confirmed that, within spectrally equivalent record pairs, the longer the duration of ground motion relative to other record pairs, the lower the collapse capacity it forecasts. The value that has been underlined above demonstrates that, for a 4-story building, a ground motion that is twice as long as another predicts an average collapse capacity of 8% lower and one that is four times longer an average collapse capacity that is 22% lower.

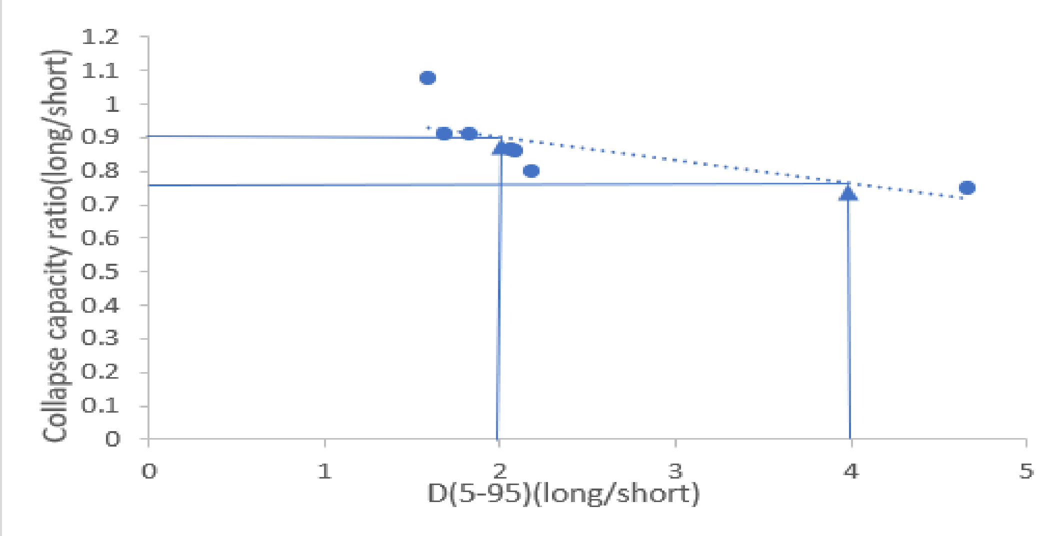


Figure 6- 24 Graph showing Ratio of significant duration versus ratio of collapse capacity for spectrally equivalent pairs for 5 story building

The value that has been highlighted above demonstrates that, for a five-story building, a ground motion with a duration that is twice that of another predicts an average collapse capacity that is 10% lower and one that is four times greater predicts an average collapse capacity that is 25% lower.

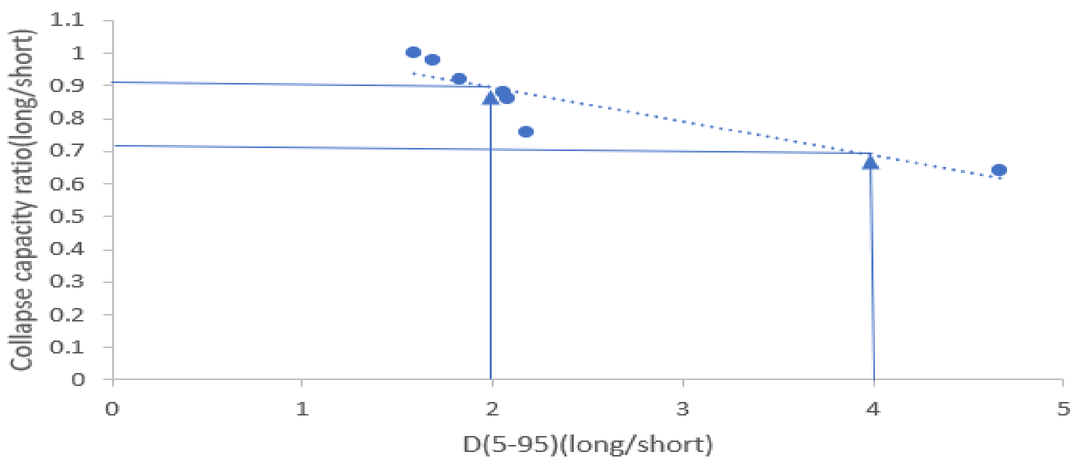


Figure 6- 25 Graph showing Ratio of significant duration versus ratio of collapse capacity for spectrally equivalent pairs for 6 story building

The value that has been highlighted above demonstrates that, for a six-story building, a ground motion with a duration that is twice that of another predicts an average collapse capacity that is 10% lower and one that is four times greater predicts an average collapse capacity that is 28% lower.

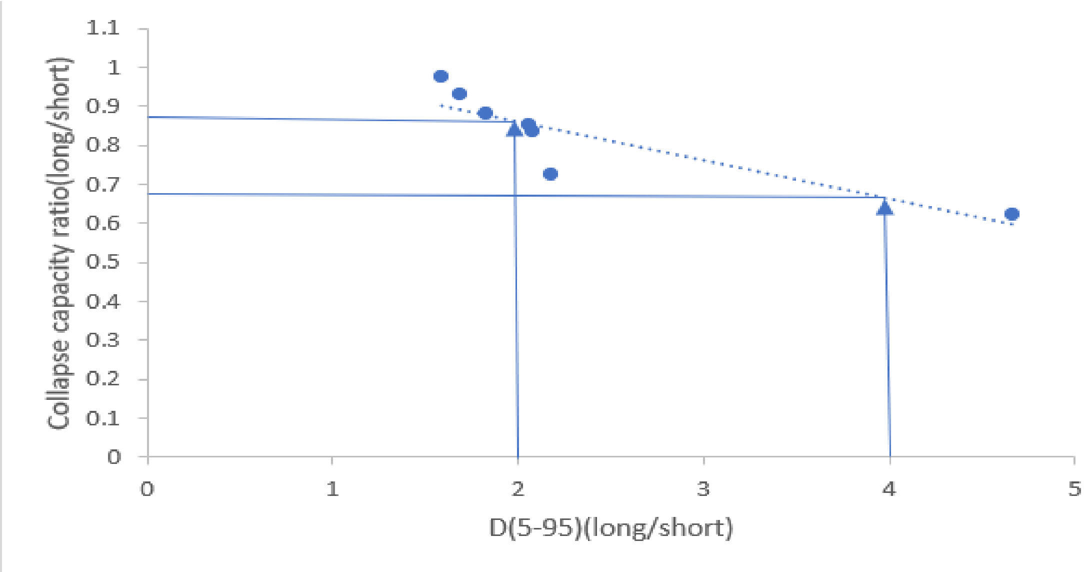


Figure 6- 26 Graph showing Ratio of significant duration versus ratio of collapse capacity for spectrally equivalent pairs for 7 story building

The value that has been highlighted above demonstrates that, for a seven-story building, a ground motion with a duration that is twice that of another predicts an average collapse capacity that is 15% lower and one that is four times greater predicts an average collapse capacity that is 34% lower.

As shown in the graph above, for all the equivalent record pairs confirms that within the spectrally equivalent record pairs, on average the longer the duration of ground motion with respect to other the lower the collapse capacity it predicts.

CHAPTER 7: CONCLUSION AND RECOMMENDATION

The above study was carried out for the linearly increasing building of 4 to 7 story in Kathmandu valley conformed to Nepal building code under varying ground motion duration. Incremental dynamic analysis is performed for both long and short duration ground motions taken in the study for all the 4 models taken. And results are interpreted with respect to the displacements, interstorey drift with respect to PGA, mean IDA curve, Collapse capacity ratio vs significant duration ratio. Fragility curves are drawn to study the collapse capacities and the following conclusions are drawn.

- Maximum roof displacement increases noticeably as ground motion duration increases, and the increase is more pronounced as building height increases. This explains the structure's poor performance by adding to its lifespan and increasing the likelihood of further deterioration.
- In context of the fragility of the structure, it is found to more for long duration earthquake since with occurrence of long duration earthquake the structure tends to reach severe damage state at lower values of the intensity measure i.e PGA in this study.
- Similarly, the effect of duration of the earthquake seems to be significant for collapse capacity of the structure. The probability of the collapse seems to be higher for lower values of PGA when subjected to long duration earthquake than short duration earthquake which shows the considerable reduction in collapse capacity of the structure when subjected to shaking for longer period of time.
- Last but not least, the graphs plotted between the ratio of significant duration for long and short ground motion vs collapse capacity ratio reveal a considerable decline in collapse capacity with increasing duration of ground motion. And as the height or number of stories rises, the decline becomes more pronounced.

Thus, these all conclusions highlights that the designed building with current seismic provisions are not enough to make structure seismically resilient since they consider just single isolated earthquake during deign. And this study draws attention on considering the duration parameter while analyzing building to make them seismically resilient.

It is recommended to consider the limitations aforementioned for a better application such as consideration of real building instead of fictious one accounting the infill wall , staircase and other loads as well. The consideration of the duration parameter shall be done along with every other parameter in analysis and design. With this consideration, the buildings constructed can be considered seismically resilie

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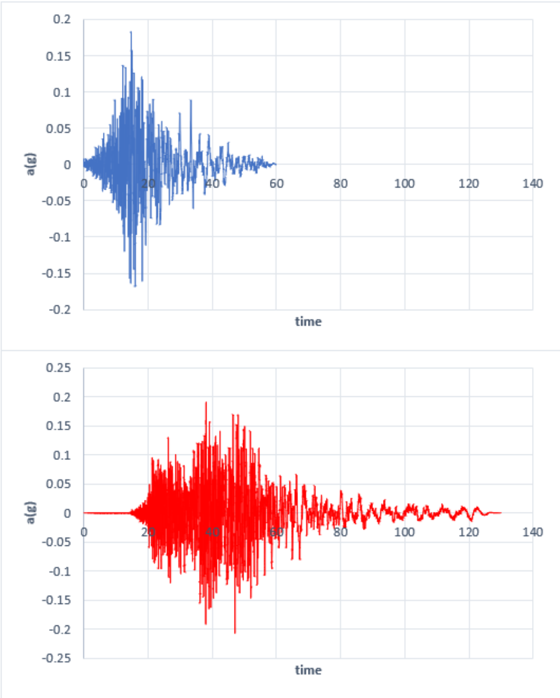
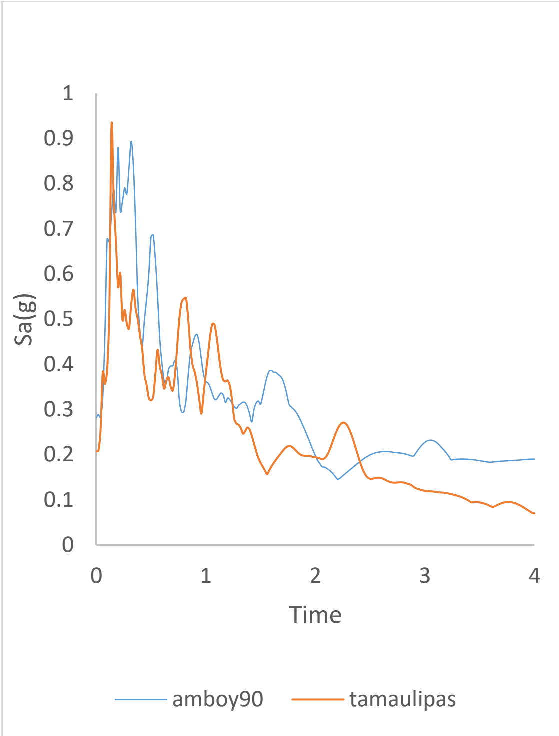
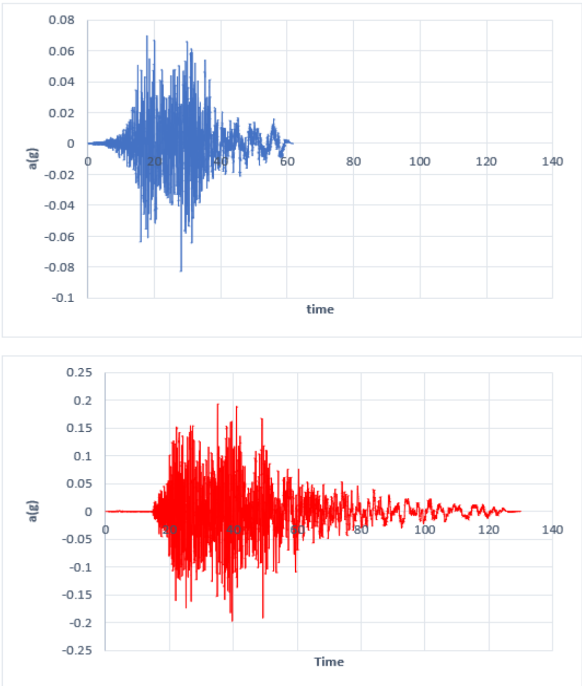
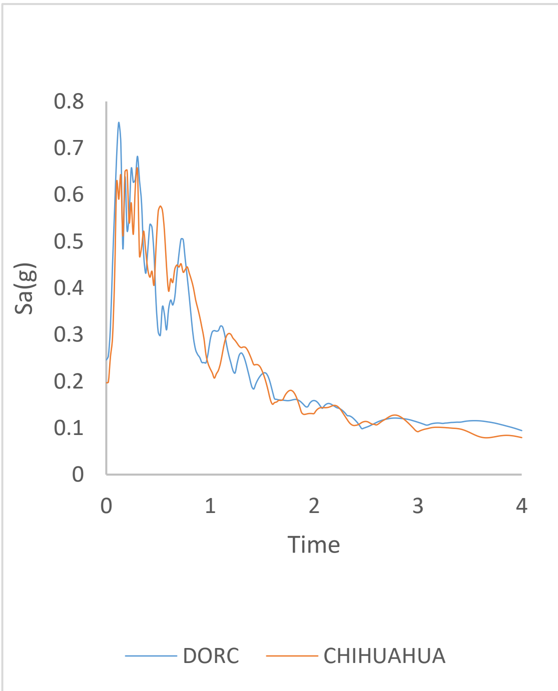
ANNEX

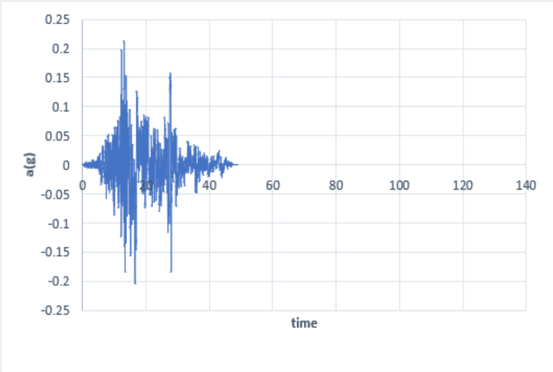
1. Spectrally equivalent pairs of the ground motion data







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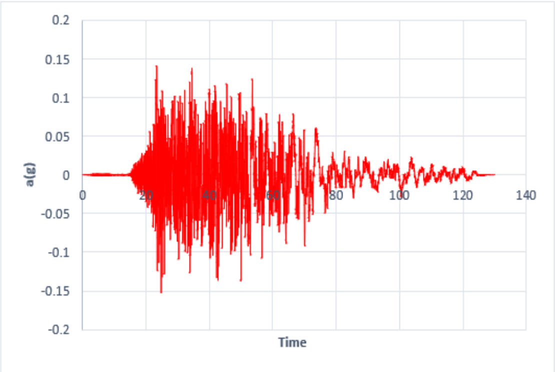
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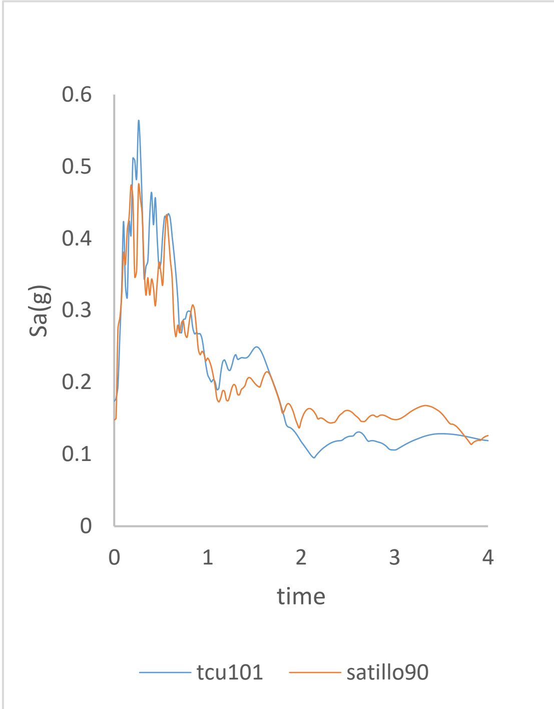
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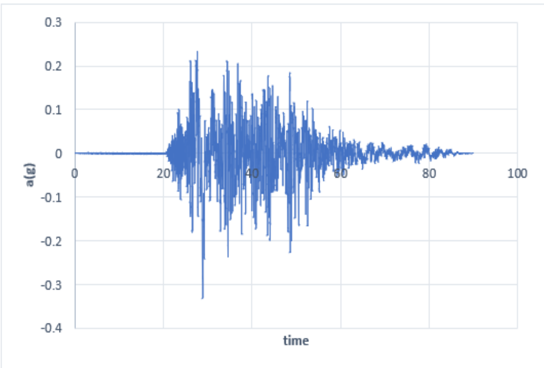
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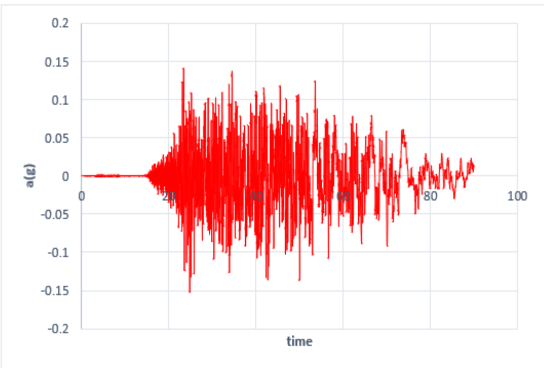
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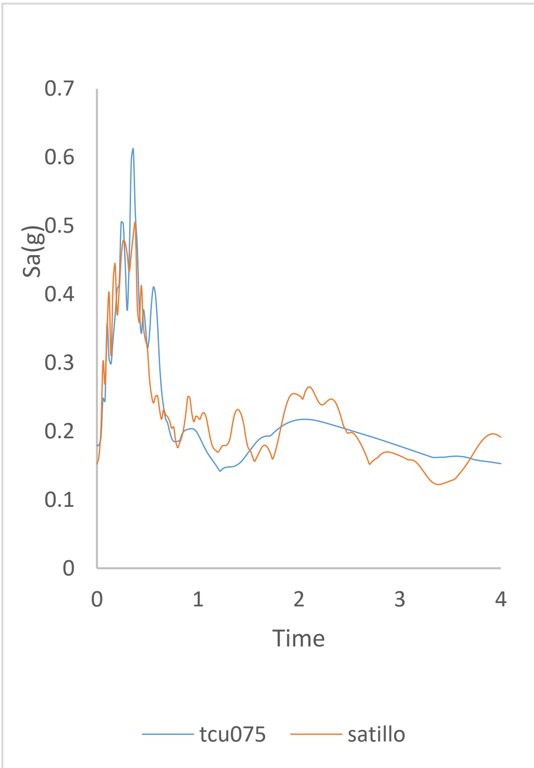
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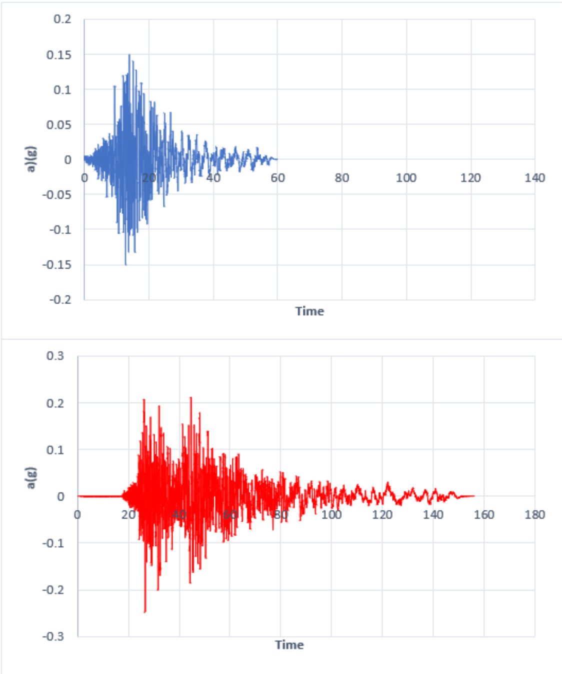
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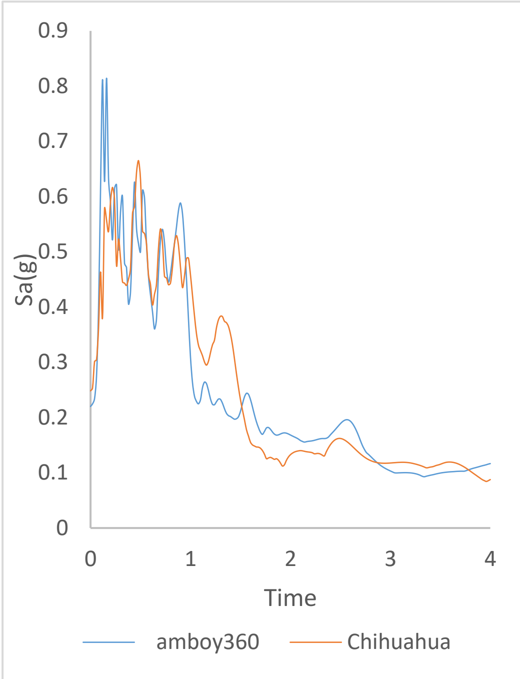
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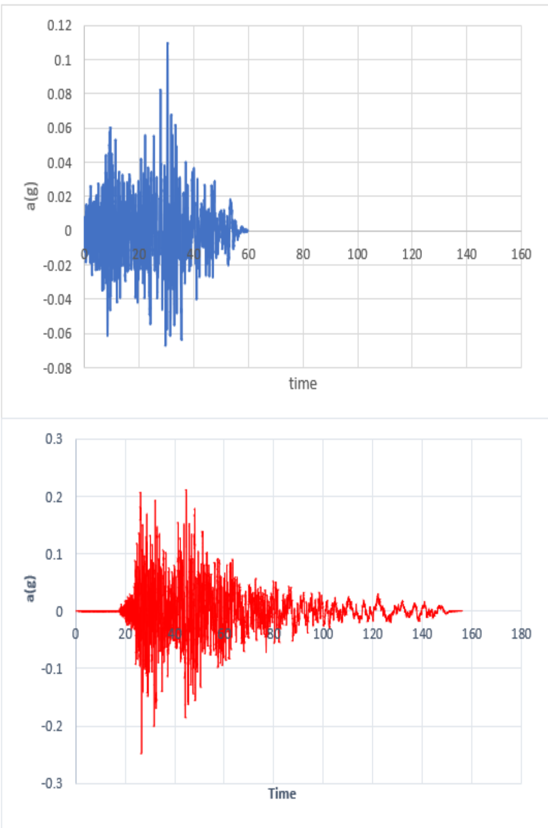
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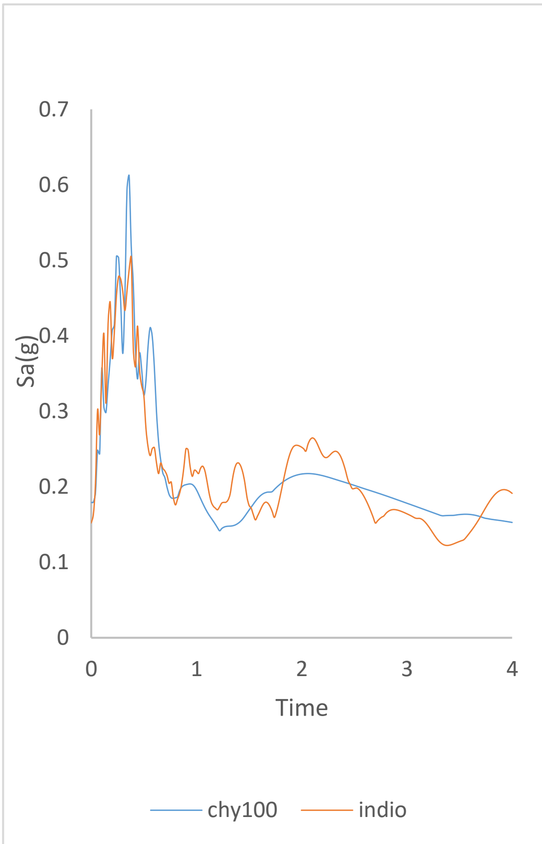
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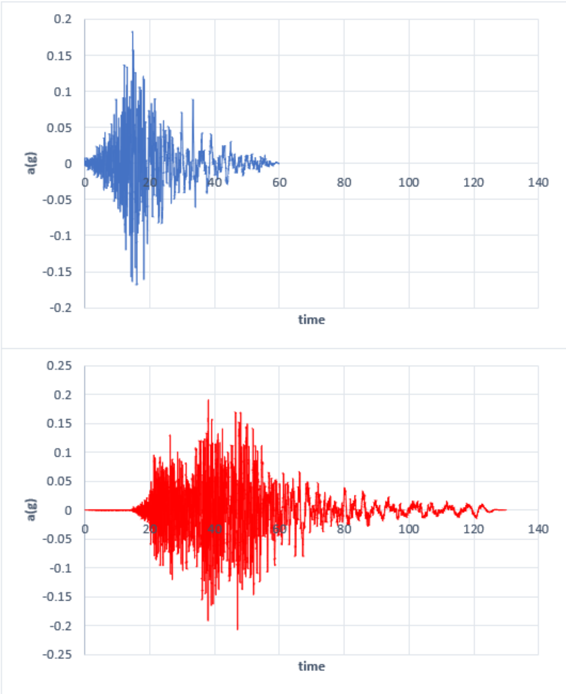

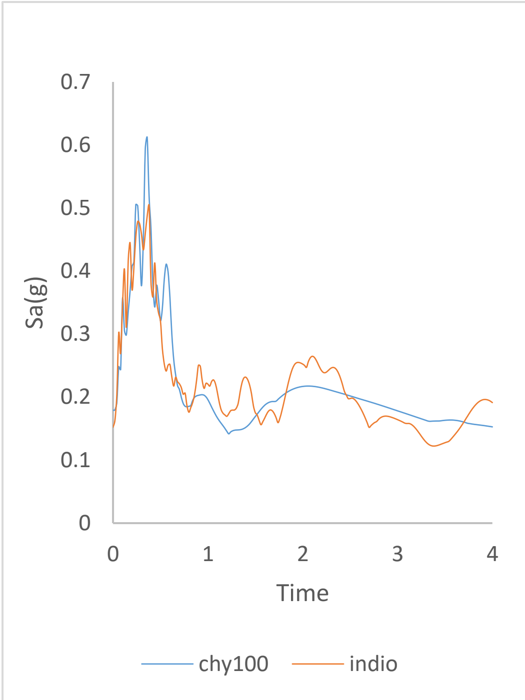

2. Source of the data with their file name from where they are taken.

SN	Earthquake	Station name	File name	Magnitude, PGA
1	2010 EL Mayor Cucapah	Chihuahua	RSN5823_SIERRA.MEX_CHI090.AT2	7.6, 0.18
	2010 Drafield,New Zealand	DORC	RSN6896_DARFIELD_DORCN20W.AT2	7,0.2
2	2010 EL Mayor Cucapah	Ejido Satillo	RSN5831_SIERRA.MEX_SAL000.AT2	7.2 , 0.19
	1999 Chi Chi Taiwan	TCU075	RSN1510_CHICHI_TCU075-E.AT2	5.9 ,0.28
3	2010 EL Mayor Cucapah	Ejido Saltillo	RSN5831_SIERRA.MEX_SAL090.AT2	7.3 , 0.25
	1999 Chi Chi Taiwan	TCU101	RSN1510_CHICHI_TCU101-E.AT2	6, 0.22
4	2010 EI Mayor Cucapah	tamaulipas	RSN5832_SIERRA.MEX_TAM000.AT2	7.2, 0.211
	1992 Landers	Amboy	RSN1762_HECTOR_ABY090.AT2	7.28 , 0.5

5	2010 EI Mayor Cucapah	Chihuahua	RSN5823_SIERRA.MEX_CHI000.AT2	7.6, 0.18
	1999 Hectormine	Amboy	RSN1762_HECTOR_ABY360.AT2	7.28, 0.7
6	1992 Landers	Indio-Coachella Canal	RSN862_LANDERS_IND090.AT2	7.28,0.41
	1999 Chi chi Taiwan	CHY100	RSN1243_CHICHI_CHY100-W.AT2	7.62,0.29
7	1985 Valparaiso, Chile	Llolleo	LLOLLEO10.th	7.8,0.22
	1994 Northridge-01	Sun Valley - Roscoe Blvd	NORTHR/RO3000.AT2	6.05,0.28